FLEWELLING & MOODY
architects for education

BID CLARIFICATION NO. 1

June 22, 2023

The following changes and/or clarifications shall be made to the drawings and specifications and all other conditions shall remain the same. All changes and/or clarifications provided shall be included in the scope of contracted work. Pursuant to PR13-01 all work that requires approval by the DSA shall not begin until said approvals are obtained.

A. PROJECT REPORT

1. **Add** Geotechnical and Geohazards Report prepared by Earth Systems dated June 14, 2022, "Bid Clarification No.1", as attached hereto (89 pages total).

B. PROJECT MANUAL

1. **Add** Spec. Section 06 64 00, Fiberglass Reinforced Protective Wall Covering, "Bid Clarification No.1", as attached hereto (6 pages total).

2. **Add** SKUSD – AV equipment matrix of responsibility (by District IT Dept.), "Bid Clarification No.1", as attached hereto (1 page total). Contractor shall provide all equipment as listed as their responsibility and shall also install all equipment identified as Owner provided contractor installed.

C. **Revise** the following sheets, and/or drawings, as indicated either by the areas shown "clouded" on the attached sheets of the same designation and labeled "Bid Clarification No.1, 06-20-23" or as specifically noted otherwise:

1. L1.01 – Landscape Layout Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

2. L1.02 – Base Bid Details, "Bid Clarification No. 1", with revision clouds as attached hereto.

3. L1.03 – Bid Alternate No. 1 & 2 Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

4. L1.04 – Cal Green Shade Calculation Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

5. L3.01 – Planting Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

6. L3.02 – Planting Legends, Details & Notes, "Bid Clarification No. 1", with revision clouds as attached hereto.

7. C3.01 – Grading and Paving Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

8. C3.02 – Grading and Paving Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
9. C3.03 – Site Sections, “Bid Clarification No. 1”, with revision clouds as attached hereto.
10. C4.02 – Wet Utility Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
11. C5.04 – Sump Pump Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
12. A1.01 – Partial Demolition Site Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
13. A1.02 – Partial Demolition Site Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
14. A5.03 – Interior Elevations, “Bid Clarification No. 1”, with revision clouds as attached hereto.
15. A9.02 – Interior Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
16. S0.03 – General Notes, “Bid Clarification No. 1”, with revision clouds as attached hereto.
17. S0.20 – Typical Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
18. S0.40 – Typical Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
19. S1.00 – Foundation Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
20. S1.10 – Roof Framing Plan, “Bid Clarification No. 1”, with revision clouds as attached hereto.
21. S2.00 – Foundation Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
22. S3.00 – Framing Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
23. S3.10 – Framing Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
24. S3.20 – Framing Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
25. E101 – Electrical Notes, Symbol List & Details, “Bid Clarification No. 1”, with revision clouds as attached hereto.
26. E102 – Single Line Diagram, “Bid Clarification No. 1”, with revision clouds as attached hereto.
27. E103 – Panel Schedules, “Bid Clarification No. 1”, with revision clouds as attached hereto.
28. E104 – Lighting Control Wiring Diagram, “Bid Clarification No. 1”, with revision clouds as attached hereto.

29. E201 – Electrical Site Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

30. FS-1.0 – Equipment Plan, "Bid Clarification No. 1", with revision clouds as attached hereto.

D. **Add** the following sheets to the set of project contract documents. Provide all labor and materials necessary for the scope of work indicated therein:

   a. BD-01 – Concrete Band Detail (8.5"x11") will replace Det. 5/A1.06 only from DSA approved Sheet A1.06.

   b. A2.00A – Wall Types Floor Plan (30"x42")

E. **PRE-BID RFI**

All responses to submitted RFI's shall be incorporated into the contract documents and shall apply to all bidders:

See responses to RFI Nos. 01 through 21, 24 through 33, 36 through 39, 41 through 43, 46 through 52, 54 through 57 and 59 from Medallion Contracting, Inc. as attached hereto.

See responses to RFI Nos. 01 and 02 from Stevens Construction, Inc. as attached hereto.

Matt Buchanan
Flewelling & Moody, Inc.

Southern Kern Unified School District
P.O. Drawer CC
Rosamond, California 93560

Geotechnical Engineering and Geohazards Report
Southern Kern Unified School District
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

June 14, 2022

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File No.: 300234-002
Doc. No.: 22-06-705

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June 14, 2022

File No.: 301430-002
Doc. No.: 22-06-705

Southern Kern Unified School District
P.O. Drawer CC
Rosamond, California 93560

Attention: Ms. Barbara Gaines

Subject: **Geotechnical Engineering and Geohazards Report**

Project: **Southern Kern Unified School District**
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems Pacific (Earth Systems) presents this geotechnical and geohazards report for the new planned cafeteria and associated outdoor eating area at the existing Rosamond High School campus located at 2925 Rosamond Boulevard in Rosamond, Kern County, California. This report presents our findings and recommendations for site grading and foundation design. From a geotechnical engineering standpoint, the site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. This report should stand as a whole, and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our proposal (PAL-22-3-003) dated March 9, 2022. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. The Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. The Limitations (Section 6) of this report should be reviewed as they are vital to the understanding of this report.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
Earth Systems Pacific

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Geotechnical Engineering and Geohazards Report
Southern Kern Unified School District
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

Section 1
INTRODUCTION

1.1 Project and Site Description

This Geotechnical Engineering and Geohazards Report has been prepared for the proposed Rosamond High School Cafeteria Project to be constructed at the existing Rosamond High School campus located at 2925 Rosamond Boulevard, in Rosamond, Kern County, California. The proposed improvements will include a new cafeteria building with a footprint of approximately 7,246 square feet. The new building will be supported on concrete conventional spread foundations. A service yard and covered outdoor eating area are also planned, as well as other improvements necessary for site development. The approximate location of the campus is shown on Plate 1, Site Vicinity Map. The locations of the proposed improvements are shown on Plate 2, Exploration Location Map and Site Geology. From Google Earth, the site elevations range from approximately 2,347 feet and 2,415 feet (MSL) and the site is located at approximate site coordinates of 34.8661°N, 118.1831°W.

Currently, the proposed building site is generally flat and level. The proposed improvement areas are currently vacant of existing improvements except for an existing chain link fence. The ground surface is unpaved in the proposed improvement areas.

The project will be designed and constructed under the jurisdiction of the Division of the State Architect (DSA), and the 2019 California Building Code (CBC). We understand that the new building will be single-story, of steel and/or stud-frame construction. The building will be founded on conventional continuous and spread footings. For the purposes of this report, we have assumed maximum continuous wall loads of 3 kips per linear foot (klf) and maximum isolated column loads of 100 kips. The buildings will be served by the existing campus utility systems. A CMU wall trash enclosure and pedestrian PCC flatwork are also be included as part of the project. No preliminary design loading was provided by the structural engineer. If actual structural loading exceeds these assumed values, the given recommendations should be re-evaluated.

Grading has been assumed to consist of cuts and fills on the order of 5 feet or less from existing grade. New slopes are not proposed, and existing slopes are minimal as the improvement sites are relatively flat. Public roadways (Rosamond Boulevard and 30th Street West) are located south and west, respectively, and vacant land and single-family residences are present to the north and east, respectively, of the campus.

1.2 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that these proposed school site improvements will be developed under the regulation of the Division of the State Architect (DSA) and the current California Building Code (2019).

The conclusions and recommendations included in this report are based upon the data collected for this commission. The scope of services performed for this study included:

Task 1 - Literature and Photograph Reviews

We reviewed select geologic and geotechnical literature pertaining to the project, including various hazard, fault, and geologic maps prepared by the California Geological Survey (CGS), the U.S. Geological Survey (USGS), Kern County and other governmental agencies as they relate to the project area. Select historical aerial photographs were reviewed using the Google Earth Pro website and Historical Aerials website. Select information contained in these reports was utilized in the preparation of this report.

Task 2 – Utility Clearance, USA Dig Alert

Each of our proposed field exploration locations was located and marked in the field and cleared with known utility lines as identified by Underground Service Alert (USA), “Dig Alert”. Our exploration locations were located in the field by a hand-held GPS or sighting from landmarks identified in the field (i.e., trees or existing fences).

Task 3 – Field Exploration

We evaluated the general subsurface conditions at the site by drilling two small-diameter hollow-stem auger borings to depths of approximately 26½ feet to 51½ feet below the existing ground surface (bgs). The CGS Note 48 for school facilities requires a minimum of one exploration point within the near vicinity of the building footprint for every 5,000 square feet of first floor area, with a minimum of two borings per structure. Should the proposed structure be moved from the presently proposed and discussed locations, the current boring locations may not satisfy the required criteria and additional borings may be required. The field exploration also included a site reconnaissance of the project area and immediate surroundings. Plate 2 shows the surface geology and the approximate location of each boring.

Task 4 - Laboratory Testing

Laboratory tests were performed on selected samples to evaluate the physical characteristics of the materials encountered during our field exploration. Laboratory testing included moisture content, dry unit weight, maximum dry density/optimum moisture content, sieve analysis, consolidation/collapse potential, direct shear, and Expansion Index. The testing was performed in general accordance with American Society for Testing and Materials (ASTM) or appropriate test procedures. Selected samples were also tested for a preliminary screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates and water-soluble chlorides). Earth

Systems does not practice corrosion engineering; however, these test results may be used by a qualified engineer in designing an appropriate corrosion plan for the project.

Task 5 – Analysis and Report

Earth Systems analyzed the field data obtained, performed engineering analyses, and provided recommended design parameters for earthwork and foundations for the structures as described within. Our report includes:

- A description of the proposed project including a site plan showing the approximate boring locations;
- A description of the surface and subsurface site conditions including groundwater conditions, as encountered in our field exploration;
- A description of the site geologic setting and possible associated geology-related hazards, including liquefaction, subsidence, and seismic settlement analysis;
- A site-specific geologic map;
- A discussion of regional geology and site seismicity, including a regional geology map;
- A description of local and regional active faults, their distances from the site, their potential for future earthquakes;
- A discussion of other geologic hazards such as ground shaking, landslides, flooding, and tsunamis;
- A discussion of site conditions, including the geotechnical suitability of the site for the general type of construction proposed;
- A “General Procedure” and “Site Specific” seismic analysis including recommendations for geotechnical seismic design coefficients and soil profile type in accordance with the 2019 California Building Code;
- Recommendations for imported fill for use in compacted fills;
- Recommendations for foundation design including parameters for shallow foundations, and subgrade preparation;
- Anticipated total and differential settlements for the recommended foundation system;
- Preliminary recommendations for the mitigation of seismic induced settlement including liquefaction;
- Recommendations for lateral load resistance (earth pressures and drainage);
- Recommendations for site preparation, earthwork, and fill compaction specifications;
- Discussion of anticipated excavation conditions;
- Recommendations for underground utility trench backfill;
- Recommendations for stability of temporary trench excavations;
- Recommendations for slabs-on-grade;
- Recommendations for collapsible or expansive soils (if applicable);
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration;

- An appendix, which includes a summary of the field exploration (computer generated boring logs) and laboratory testing program (computer generated plots).

Not Contained in This Report: Although available through Earth Systems, the current geotechnical scope of our services does not include:

- A Phase 1 Environmental Site Assessment.
- An investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

The subsurface exploration program included advancing two exploratory borings to depths on the order of 26½ to 51½ feet bgs. The borings were drilled using a Mobile B-61 truck-mounted drill rig provided by GP Drilling of Apple Valley, California and equipped with 8-inch hollow-stem augers. Borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 2, Exploration Location Map. The locations shown are approximate, established by a hand-held GPS and relative landmark locations.

An engineer from Earth Systems maintained a log of the subsurface conditions encountered and obtained samples for visual observation, classification and laboratory testing. Subsurface conditions encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System [USCS] and ASTM D 2487 and 2488 (current edition). Our typical sampling interval within the borings was approximately every 2½ or 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered onsite. Samples were obtained within the test borings using a California type ring sampler (ASTM D 3550 with those similar to ASTM D 1586). The ring sampler has an approximate 3-inch outside diameter and 2.4-inch inside diameter. Further sampling and collection of disturbed soil samples was accomplished using the Standard Penetration Test (SPT) sampler in accordance with ASTM D 1586. The SPT sampler is a split barrel sampler with a 1-3/8 inch inside diameter. The SPT and ring sampler was mounted on a drill rod and driven using a rig-mounted 140-pound automatic hammer falling for a height of 30 inches. The number of blows necessary to drive the samplers within the borings was recorded.

Design parameters provided by Earth Systems in this report have considered an estimated 72% hammer efficiency based on data provided by the drilling subcontractor. The N-values (blow count) using the California type sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

Bulk samples were also obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. The depth to groundwater, if any, was measured in the boreholes. Following drilling, sampling, and logging, the borings were backfilled with the cuttings and tamped upon completion. Our field exploration was provided under the direction of a State of California Registered Geotechnical Engineer from our firm.

The logs of the borings represent our interpretation of the soils observed in the field and the results of laboratory testing performed on the samples obtained; the logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradual. In reviewing the logs and legend, the reader should recognize the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics observed during drilling. These include, but are not limited to cementation, variations in soil moisture, presence of groundwater, and other factors.

The boring logs present field blow counts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts on the logs are uncorrected (i.e., not corrected for overburden, type of sampler etc.). Consequently, the user must correct the blow counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or another appropriate test procedure. Selected samples were also tested for a screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project.

Our testing program consisted of the following:

- Density and Moisture Content of select samples of the site soils (ASTM D 2937 & 2216).
- Maximum Dry Density/Optimum Moisture Content tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by sieve analysis procedures (ASTM D 6913).
- Washed Sieve Analyses were performed on select samples of the site soils to determine the percent soil fraction passing the No. 200 sieve (ASTM D 1140).
- Direct Shear testing was performed on a representative sample remolded to 90 percent relative compaction (ASTM D 3080).
- Consolidation and Collapse Potential to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).
- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture contents of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Screening Level Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (APHA 2320-B), and Electrical Resistivity/Conductivity (ASTM G 187) to evaluate the potential for adverse effects of the soil on concrete and steel.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration and literature review indicate that site geologic materials consist generally of Quaternary Alluvial (Qa) deposits. The alluvial soils consisted of silty sands, poorly graded sands, and silty clays with sand (SM, SP, SP-SM, and CL-ML per the Unified Soil Classification System) with a medium dense to very dense consistency for the coarse-grained soils and medium stiff to very stiff consistency for the fine-grained soils. In general, the upper five feet of soils were damp. Detailed descriptions of the observed fill and alluvial soils are provided on the boring logs in Appendix A. Due to the flat site, geologic cross sections are not provided.

3.2 Groundwater

Groundwater was not encountered within the upper 51½ feet of the soil profile. Historically shallowest groundwater levels were not mapped by the California Geological Survey in the site vicinity; however, historically shallowest groundwater levels are mapped by CGS approximately 3 miles to the south of the site at depths ranging from of 100 to 150 feet below ground surface.

Nearby state monitored wells were researched for their most recent and historic well readings. The following is a summary of our findings for the two wells closest to the campus.

- Well No. 348672N1181846W001 was located in the northwest portion of the campus. In that State Well, depth to groundwater was measured at approximately 83 feet below the ground surface in 1951.
- Well No. 348644N1181846W001 was located in the southwest portion of the campus. In that State Well, depth to groundwater varied between approximately 99 feet to 160 feet below the ground surface between 1951 and 1967, indicating an increased depth to water with time. The most recent depth to groundwater was measured at approximately 160 feet below the ground surface in 1967.

Fluctuations in groundwater levels may occur due to variations in rainfall, regional climate, and in response to landscape irrigation. However, groundwater in the vicinity of the Rosamond High School campus is expected to be more than 50 feet below the existing ground surface. Perched water conditions were not observed, despite the site being surrounded by nearby residential housing and associated irrigation for decades.

3.3 Collapse and Consolidation Potential

Collapsible soil deposits generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Soils susceptible to collapse include loess, weakly cemented sands and silts where the cementing agent is soluble (e.g., soluble gypsum, halite), valley alluvial deposits within semi-arid to arid climate, and certain granite residual soils above the groundwater table. In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydro-consolidation) may occur when the soils are

lubricated or the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose configuration from deposition.

The degree of collapse of a soil can be defined by the Collapse Potential (CP) value, which is expressed as a percent of collapse of the sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.1, the severity of collapse potential can be evaluated by the following Table 1, Collapse Potential Values.

Table 1
Collapse Potential Values

Collapse Potential Value	Severity of Problem
0-1%	No Problem
1-5%	Moderate Problem
5-10%	Trouble
10-20%	Severe Trouble
> 20%	Very Severe Trouble

Table 1 can be combined with other factors such as the probability of ground wetting to occur on-site and the extent or depth of potential collapsible soil zone to evaluate the potential hazard by collapsible soil at a specific site. A hazard ranking system associated with collapsible soil as developed by Hunt (1984) is presented in Table 2, Collapsible Soil Hazard Ranking System.

Table 2
Collapsible Soil Hazard Ranking System

Degree of Hazard	Definition of Hazard
No Hazard	No hazard exists where the potential collapse magnitudes are non-existent under any condition of ground wetting.
Low Hazard	Low hazards exist where the potential collapse magnitudes are small and tolerable, or the probability of significant ground wetting is low.
Moderate Hazard	Moderate hazards exist where the potential collapse magnitudes are undesirable, or the probability of substantial ground wetting is low, or the occurrence of the collapsible unit is limited.
High Hazard	High hazard exists where potential collapse magnitudes are undesirably high and the probability of occurrence is high.

The results of collapse potential tests performed on five (5) selected single ring samples from depths ranging from 2½ to 10 feet below the ground surface in native soils indicated a collapse potential range of 0.8 to 1.6 percent. The goal of the collapse testing was to identify soils and densities where the potential for collapse decreased to accepted levels. This accepted level is defined as where on-site soils had collapse potential less than 2% or the estimated relative compaction is greater or equal to 85%, which is the typical standard of care based on the above Table 1 or where soil collapse becomes a concern for structural soils (2%) (County of Los Angeles, 2013). Plotting and analysis of the results of the five tests from whole stack ring samples indicates that collapse potential at the site appears to be directly related to in-place density (relative

compaction). Typically, it is assumed in engineering practice that at least 1% of total tested collapse is due to sample disturbance. This is demonstrated via Table 1.

Based on the field and laboratory testing performed, Earth Systems provides key items of interest that supports Earth Systems recommendations regarding collapse potential at this site:

1. Soils are generally granular in nature; however, cementation was observed in soils from 6.5 to 14 feet below ground surface.
2. Voiding was not observed other than minor occurrences.
3. Soil collapse at the site appears to be related to in-place density (relative compaction).
4. In-place densities as measured for a whole soil sample stack (6 rings) generally indicated higher densities, such that collapsible zones may be limited to thinner soil layers with corresponding lower density (i.e., it is considered low potential for a whole soil wetted zone to collapse of the same magnitude through the whole zone).

For some deposits without cementation, studies suggest some sites with densities above 103 pcf are “not likely to collapse” and N60 values > 10 do not fit into the category of “Likely Collapsible” (Lommler, C.J. and Bandini). In addition, soils with greater than 85 percent relative compaction are compact, and it is accepted that they are not likely to settle, especially after initial inundation. Eighty-five percent of the maximum density for the silty sand type soils is 116 pcf.

Based on the above criteria and our field and laboratory findings, we estimate there is “Low” to “Moderate” collapse potential from soil layers between 2½ and 10 ft bgs at the locations tested. Without collapse mitigation efforts, the collapse potential is estimated to be approximately over 1½ inches. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential is on the order of approximately ½ inch (1 inch in hardscape areas with the lesser overexcavation).

3.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. Based on our laboratory testing and experience with the project, the expansion potential of the on-site soils tested is generally “very low” as defined by ASTM D 4829 and the 2019 California Building Code.

Testing and/or observation of the subgrade soils during grading within the building pad and at the footing grade should be performed to further evaluate the expansion potential and confirm or modify the recommendations presented herein.

3.5 Corrosivity

One sample of the near-surface soils within the site was tested for potential corrosion of concrete and ferrous metals. A representative soil sample collected between 0 and 5 feet bgs was tested in general accordance with the ASTM Standard Test Methods to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

Water-soluble sulfates in soil can react adversely with concrete. ACI 318 provides the relationship between corrosivity to concrete and sulfate concentration, presented in the table below:

Table 3

Water-Soluble Sulfate in Soil (ppm)	Corrosivity to Concrete
0-1,000	Negligible
1,000 – 2,000	Moderate
2,000 – 20,000	Severe
Over 20,000	Very Severe

In general, the lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to ferrous structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (ASTM, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown in Table 4.

Table 4

Soil Resistivity (Ohm-cm)	Corrosivity to Ferrous Metals
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Test results show a pH value of 7.3, chloride content of 13 ppm, sulfate content of 83 ppm and minimum resistivity of 2,080 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being "Severely" corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI) 318, Section 4.3. The results of all chemical testing have been provided in Appendix B. The above values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

3.6 Geologic Setting

Regional Geology: The project site is located at the western portion of the Mojave Desert Geomorphic Province (see Plates 3 and 4), a broad interior region of isolated mountain ranges separated by desert plains and basins. The Western Mojave Desert is a wedge-shaped structural block bounded on the northwest by the Garlock fault and along the southwest by the San Andreas rift zone. The Sierra Nevada and Tehachapi Mountains are north of the Garlock fault. The Transverse Ranges and coastal basins are southwest of the San Andreas fault. Regional features of the western Mojave Desert include the alluvial-filled Antelope Valley, San Gabriel and San Bernardino Mountains, the Rosamond Hills, Rosamond Lake, and scattered buttes.

The Rosamond High School campus is located near the Rosamond Hills in the Antelope Valley. The site is underlain by a deep section of Quaternary sediments. Tertiary volcanic and sedimentary rocks and pre-Tertiary igneous rocks are exposed in the hills to the northwest and north.

The Garlock fault that forms the northwest boundary of the Antelope Valley is the closest active fault to the campus that has significant earthquake potential. The nearest reach of the fault is approximately 16.6 miles to the northwest of the campus.

No known active or potentially active faults trend through the campus. However, it should be noted that Jennings and Bryant (2010) show a fault identified as the Willow Springs fault located approximately 4.5 miles west of the campus and projecting in a trend slightly south of the site. It is characterized as a Quaternary fault (displacement within the last 1.6 million years).

In addition, a "pre-Quaternary fault", i.e., an inactive fault that is concealed by alluvium, trends along the north side of Rosamond Avenue and close to the campus. This fault has been identified on other geologic maps as the Rosamond fault.

3.7 Geologic Hazards

Geologic hazards that may affect the region include seismic shaking, surface fault rupture, soil liquefaction, landsliding and rock fall, flooding, expansive soils, ground subsidence, and erosion. A discussion follows on the specific hazards to this site.

3.7.1 Seismic Shaking

The primary seismic hazard to the site is strong ground shaking from earthquakes along regional faults such as the San Andreas fault, the Garlock fault, the Sierra Nevada fault zone, the Eastern California shear zone, the White Wolf fault zone, and the many faults within the Los Angeles basin and Inland Empire. The Garlock fault is located approximately 16.6 miles northwest of the site. The Mojave segment of the San Andreas fault is located approximately 17.3 miles southwest of the campus. About 40 other active faults or seismic zones lie within 52 miles of the project site (see Table A-1).

The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Approximately 35 magnitude 5.5 or greater earthquakes have occurred within 60 miles of the site since 1812. Plate 7, Earthquake Epicenter Map, depicts epicenters of significant seismic events greater than magnitude 5.5 that have occurred in southern California between 1812 and 1999. Magnitudes that are above 6 and prior to accurate instrumental measurements (after 1933) are based on moment magnitudes (M_W). Magnitudes that are below 6 or earthquakes prior to 1933 are based on local magnitudes (M_L).

Many of the major historic earthquakes felt in the vicinity of the Rosamond area have originated from faults located outside the area. These include the 1812 Wrightwood, 1857 Fort Tejon, 1872 Owens Valley, 1933 Long Beach, 1952 Arvin-Tehachapi, 1971 San Fernando, 1987 Whittier Narrows, 1992 Landers, 1994 Northridge, and 1999 Hector Mine earthquakes. Of recent note, the July 6, 2019 7.1 Mw Ridgecrest earthquake was about 73 miles northeast of Rosamond.

Seven historic seismic events (5.9 M or greater) have significantly affected the Antelope Valley in the last 70 years. They include:

- Fort Tejon (San Andreas) Earthquake – On January 9, 1857, a magnitude 8.0Mw earthquake occurred with an epicenter near Parkfield, California. The earthquake was felt throughout much of California with the greatest damage in Fort Tejon, California.
- Owens Valley (Sierra Nevada) – A magnitude 7.6Mw earthquake occurred on March 26, 1872, in Lone Pine, CA. This event caused massive structural damage, 56 injuries, and 27 deaths.
- Arvin-Tehachapi (White Wolf) – On July 21, 1952, a magnitude 7.5Mw earthquake occurred on the White Wolf fault near Wheeler Ridge. The town of Tehachapi suffered the greatest damage and loss of life.
- San Fernando – On February 9, 1971, a magnitude 6.6Mw earthquake occurred in the foothills of the San Gabriel Mountains in Southern California. The earthquake resulted in extensive damage to structures in parts of San Fernando and the Santa Clarita Valley. Uplift and other effects affected private homes and business. The epicenter of the earthquake was located near Soledad Junction. Strong motion accelerographs recorded

ground accelerations as high as 1.25g at Pacoima Dam near the epicenter of the earthquake. Some structures designed in accordance with the Building Code in affect at the time were extensively damaged.

- Northridge – The 1994 Northridge earthquake (6.7Mw) and related aftershocks significantly shook the Palmdale area. This event was produced by a buried thrust fault that underlies portions of the San Fernando Valley and the Santa Susana Mountains. No actual fault rupture associated with the main thrust faulting occurred at the surface. Primary fault rupture terminated approximately 3 to 4.3 miles (5 to 7 km) below the ground surface.
- Landers and Big Bear Earthquakes – Early on June 28, 1992, a magnitude 7.3Mw earthquake occurred near Landers, the largest seismic event in Southern California in 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.4Mw earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Antelope Valley area.
- Hector Mine Earthquake – On October 16, 1999, a 7.1Mw earthquake occurred on the Lavic Lake and Bullion Mountain faults north of Twentynine Palms. While this event was widely felt, no significant structural damage has been reported in the Antelope Valley.
- Ridgecrest Earthquake – The last major earthquake to affect the Lancaster area was the July 2019 Ridgecrest earthquake. The epicenter of this 7.1(Mw) earthquake was approximately 80 miles north-northeast of Lancaster. Surface rupture occurred near segments of the Little Lake fault zone near Ridgecrest. Geologic evaluation of this earthquake is underway.

One measure of ground shaking is intensity. The Modified Mercalli Intensity Scale of ground shaking ranges from I to XII with XII indicating the maximum possible intensity of ground movement. Structural damage begins to occur when the intensity exceeds a value of VI. It is estimated that the intensity of earthquake response was at least IX on the Modified Mercalli Scale during the Fort Tejon earthquake of 1957. Based on an AGU Geophysical Research Paper by Katherine M. Scharer of the USGS Earthquake Research Center, and Doug Yule of California State University Northridge (2020), historic and paleoseismic research data supports a magnitude 7.9 even for the southern San Andreas. Larger rupture events are not supported in the paleoseismic and historic records. As such we have utilized a magnitude 7.9 for our analyses.

It is assumed that the 2019 CBC and ASCE 7-16 guidelines will apply for the seismic design parameters used in design. The 2019 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The “general procedure” (i.e., probabilistic) seismic design parameters presented below were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the SEAOC/OSHPD website for ASCE 7-16 for the site coordinates (34.8661° North Latitude and 118.1831° West Longitude, Soil Site Class D (for stiff soils), for Occupancy (Risk) Category III (which includes public school structures). (A listing of the calculated 2019 CBC and ASCE 7-16 Seismic Parameters is presented below and again in Appendix A.)

Seismic Design Criteria: This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Andreas and Garlock fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2019 edition of the California Building Code and ASCE 7-16 using the seismic coefficients given in the table below. The site is not within Alquist-Priolo or other hazard zones. A site-specific response spectrum is provided per CGS guidelines. General Procedure and site-specific seismic parameters are presented below considering a Site Class D (results in Appendix A). Our site-specific analysis considered the proximity of the San Andreas and Garlock faults and the results are listed in the Seismic Hazards section of this report. See Appendix A for tables.

Earthquake Performance Statement: Depending upon the extent of structural design of structures, exterior flatwork, walls, utilities, roadways, and other similar site improvements, some damage due to seismic events will occur. Note that all of southern California in general is in earthquake country. Site developments in southern California are typically not designed to mitigate anticipated seismic events without some damage. In fact, the Building Code is intended to provide Life-Safety performance, not complete damage-free design. In other words, some damage from earthquakes in the form of structural damage, settlement, cracking, and disruption of utilities can occur. It is not the current standard of care for site designers to fully mitigate all anticipated earthquake induced hazards.

Table 5
Summary of Seismic Parameters – 2019 CBC "General Procedure"

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	III
Seismic Design Category	D
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	1.097 g
Spectral Response Acceleration at 1 sec. – S_1	0.435 g
Site Coefficient – F_a	1.06
Site Coefficient – F_v	1.87
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.164 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	0.811 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	0.776 g
One Second Spectral Response – S_{D1}	0.541 g
Site Modified Peak Ground Acceleration - PGA_M	0.543 g
Values appropriate for a 2% probability of exceedance in 50 years	

The seismic factor S_1 is greater than 0.2 g and the Site Class is "D" which is based on an average blow count of 39. If the structural engineer determines that the exemptions of Section 11.4.8 of ASCE 7-16 do not apply, a site-specific (i.e., deterministic) ground motion hazard analysis is required. The site-specific study takes into account soil amplification effects. The United States Geological Survey (USGS, 2009) has undertaken a probabilistic earthquake analysis that covers the continental United States. A reasonable site-specific spectral response curve may be developed from USGS Unified Hazard Tool web page, which adjusts for site-specific ground factors. The interactive webpage appears to be a precise calculation based on site coordinates. For the purposes of this study, the Dynamic: Conterminous U.S. 2014 (Update) (Version 4.20) values have been chosen for use in the analysis. The seismic shear velocity (Vs) used in the analysis for Site Class D was 270 meters per second.

NGA West2 2014 attenuation relationships were used in the analyses. These attenuations included those of Kumai, Abrahamson, and Silva (2013), Boore, Stewart, Seyhand, and Atkinson (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014).

Table 6
Summary of Seismic Parameters – 2019 CBC "Site-Specific Procedure"

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	III
Seismic Design Category	D
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	1.097 g
Spectral Response Acceleration at 1 sec. – S_1	0.435 g
Site Coefficient – F_a	1.06
Site Coefficient – F_v	2.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.350 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.105 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	0.900 g
One Second Spectral Response – S_{D1}	0.737 g
Site Modified Peak Ground Acceleration - PGA_M	0.550 g
Values appropriate for a 2% probability of exceedance in 50 years	

The Fault Parameters table in Appendix A lists the significant "active" and "potentially active" faults within an approximate 52-mile radius of the project site. The distance between the project site and the nearest portion of each fault is shown as well as the respective estimated maximum earthquake magnitudes.

Surface Fault Rupture: Surficial displacement along a fault trace is known as fault rupture. Fault rupture typically occurs along previously existing fault traces. No faults considered to be "active" or "potentially active" were observed to be located on or trending into the campus during the field study, during reviews of the referenced geologic literature, or during review of the aerial photographs taken of the site.

It should be noted that the Rosamond fault trends nearly east-west and near the campus (Jennings and Bryant, 2010, and Ken County Safety Element of the General Plan, 2009), but this fault was classified by the Jennings and Bryant work as a pre-Quaternary fault, which is also known as an inactive fault. As a result, it is the opinion of this firm that the potential for fault rupture on this site is low.

Landsliding and Rock Fall: The campus is nearly level. As a result, it appears that the hazards posed by landsliding and rock fall are nil.

3.7.2 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, lateral spreading, seiches, and tsunamis.

Soil Liquefaction: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. Our exploration did not reveal a shallow groundwater table or a perched water table (despite decades of irrigation). Our research of nearby water well data indicates the groundwater table is deeper than 50 feet below the ground surface. Based on the above, the potential hazard posed by liquefaction at the site is considered "low".

Lateral Spreading: Lateral spreading (lateral migration) is caused when a loose and saturated soil layer is liquefied by a nearby earthquake of sufficient magnitude and a sloping ground, free face conditions, or sufficient building/embankment stresses (Yazdi, J.S. and Moss, R.E. S., 2016) exist near the site. Currently, these factors are not present at this site. The exploration and laboratory work revealed that the soils within 50 feet of the surface are not liquefiable. Therefore, lateral spreading at this site has a low potential.

Dry Seismic Settlement: The amount of dry seismic settlement is dependent on relative density of the soil, ground motion, and earthquake duration. In accordance with current CGS policy (Earth Systems discussion with Jennifer Thornburg, CGS May 2014), we used a site peak ground acceleration of ⅔ PGA_M (PGA_M = 0.55) and an earthquake magnitude of 7.9 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the SEAOC online application (https://seismicmaps.org/) and adjusted per the site-specific evaluation. The reasoning for using ⅔ PGA_M has stemmed from the limitations of the Pradel (modified Tokimatsu and Seed) method of calculating dry seismic settlement. This issue manifested during the code change from ASCE 7-05 to ASCE 7-10 (2013) and has been continued into the ASCE 7-16 document, where it was no longer codified to evaluate liquefaction and seismic settlement as a design value ($S_{DS/2.5}$) but to evaluate it at full PGA. PGA_M equated to a maximum considered value and the design spectral response, S_{DS}. Is equal to ⅔ S_{MS}, where SMS

is the Site Class adjusted maximum considered earthquake value. When using the full PGA_M in Pradel's method, (Pradel's method is the Standard of Care methodology of calculating dry sand settlement), the potential settlements increased dramatically resulting in unreasonable amounts of assumed settlement (stress/strain curves defaulted to 10 percent vertical strain as high acceleration) which did not correlate with field observations of earthquake events. Given these unreasonably high potential settlements and after much discussion with agencies and CGS, the PGA used for dry seismic analysis was returned to a "design" value in practice. This is on the basis that since dry seismic shaking will typically not cause bearing failure (like liquefaction will) but is only settlement related deformation. Earth Systems is in concurrence with this methodology and thus the method utilized for our analysis and that ⅔ PGA_M may be used as an appropriate acceleration in the analysis of seismic settlement of unsaturated soils.

Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils for the depths explored was calculated for our borings at the site and estimated to be less than ½-inch (Boring B-1). Seismic settlement is based on post grading recommendations provided in Section 5.1. Due to the general uniformity of the soils encountered and the relative minor difference in settlement potential between the borings evaluated, seismic settlement is expected to occur on an areal basis and as such per Special Publication 117A (CGS, 2008), the differential settlement is estimated to be approximately ¼-inch considering the grading as recommended in Section 5.1.

Ground Subsidence: Subsidence is the gradual settling or sudden sinking of the Earth's surface due to subsurface movement of earth materials. It can result in the loss of aquifer storage, increased flooding, cracks and fissures at the land surface, damage to man-made structures, and associated economic costs. According to the Scientific Investigations Report 2014-5166 "Groundwater-Flow and Land-Subsidence Model of Antelope Valley, California" (Siade et al. 2014) the project site is not within an area of concentrated land subsidence (Plate 6). Therefore, the potential for subsidence settlement from groundwater removal is considered low. Changes in pumping regimes can affect localized groundwater depths, related cones of depression, and associated subsidence such that the prediction of where fissures might occur in the future is difficult. In the project area, groundwater depths remain fairly deep and we consider the current subsidence potential low. However, in the event of future nearby aggressive groundwater pumping and utilizations, the occurrence of deep subsidence cannot be ruled out. Changes in regional groundwater pumping could result in areal subsidence. The risk of areal subsidence in the future is more a function of whether groundwater recharge continues and/or over-drafting stopes, than geologic processes, and therefore the future risk cannot be predicted or quantified from a geotechnical perspective.

Slope Instability: The surface soils were found to be mostly silty sand. Correlations using Standard Penetration Testing and lab testing indicates the soils have an internal friction angle of 33 degrees. Any proposed 2(H):1(V) or steeper slopes have a high potential for surficial failures, the factor of safety being less than 1.0 when the slopes are saturated to a depth of 4 feet.

Flooding: According to the Flood Emergency Management Agency (FEMA) website, the project site lies within a zone defined by FEMA as Zone "X", which is defined as an area of 0.2% annual chance flood. Thus, the hazard posed by storm-induced flooding appears to be low.

The site is far inland, so the hazard from tsunamis is non-existent.

According to the Kern County Safety Element of the General Plan (2009), the campus is not within any inundation zones for reservoirs or seiche hazard zones. As a result, it appears that the hazards posed by dam failure and seiche are low.

Section 4
CONCLUSIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from the field and laboratory testing, geotechnical analysis, and our experience with similar projects in the general area.

Based on our field exploration, laboratory testing, and geotechnical analyses conducted for this study, it is our professional opinion that the site is suitable, from a geotechnical standpoint, for construction of the improvements and buildings as proposed, provided the recommendations presented in this report are incorporated into project design and construction.

The recommendations presented in this report may change pending a review of final grading plans and foundation plans. Recommendations presented in this report should not be extrapolated to other areas or be used for other projects (beyond those expressly identified within) without our prior review and comment.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional southern California faults. A major earthquake originating on the nearby segments of the San Andreas or Garlock fault zones and other southern California faults would be the critical seismic events that may affect the site within the design life of the campus. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.

- The underlying geologic condition for seismic design is Site Class D. The site is about 16.6 miles from the Garlock fault and about 17.3 miles from the San Andreas fault, which are "active" faults of significance. A qualified professional should design any permanent structure constructed on the site. The ***minimum*** seismic design should comply with the 2019 edition of the California Building Code.

- This site will have structures on Site Class "D" and the seismic coefficient S_1 (0.435) is greater than 0.2. A site-specific response spectrum is provided.

- The site is not within a currently designated Alquist-Priolo Earthquake Fault Zone or liquefaction zone. No known faults or obvious lineaments cross the site. Therefore, the potential for surface fault rupture at the site is considered very low.

- The potential for ground subsidence and liquefaction settlement hazards are considered to be very low to low for this project. The site is not within an area of documented areal subsidence due to groundwater. Dry seismic settlement potential is low.

- Other geologic hazards, including flooding, and landslides, pose low or negligible potential hazards to the proposed project.

- Based on current conditions, groundwater is not anticipated to be encountered during construction, however moist soils may be encountered.

- Much of the existing on-site fill and alluvial soils are very low in expansion potential and suitable for location under structures or hardscape after remedial grading. Building structure recommendations provided within are based upon using a very low to low in expansion potential fill material for the building pad.

- Hydro-consolidation is a potential hazard in this site. Adherence to the grading and structural recommendations in this report should reduce potential settlement problems from seismic forces, increased wetting of the soil profile, and the weight of the intended structures.

- Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1H:1V from the projection upward from the bottom of any foundation, whichever is greater.

- Laboratory testing of one sample showed potentially “Severely” corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further information. Site soils should be reviewed by an engineer competent in corrosion evaluation.

- In our professional opinion, structures can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on “very low” expansion category soils.

- Due to the project complexity, this report and its recommendations are contingent on a structural and grading plan review by Earth Systems to confirm or modify applicability.

Section 5
RECOMMENDATIONS

5.1 Site Development and Grading

A representative of Earth Systems should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing or decreasing the depth of recompaction and over-excavation. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction and is required by the 2019 California Building Code. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District.

Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 17A of the 2019 California Building Code. California Building Code requires full-time observation by the geotechnical consultant during site grading (fill placement). Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Additionally, the California Building Code requires the testing agency to be employed by the project owner or representative (i.e., architect) to avoid a conflict of interest if employed by the contractor. Unless noted otherwise, grading should be performed in general accordance with Appendix J of the 2019 CBC.

Clearing and Grubbing: At the start of site grading, existing vegetation, trees (including the entire root ball), large roots, overly wet and/or soft soil, undocumented fill, pavements, foundations, construction debris, septic tanks, leach fields, deleterious material, trash, and abandoned underground utilities should be removed from the proposed building areas. Organic growth should be stripped off the surface and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below.

Buried utilities may be located in the vicinity of the planned structures and within other areas of the project site. All buried structures which are removed should have the resultant excavation backfilled with soil compacted as engineered fill described herein or with a minimum 2-sack cement-sand slurry (minimum of 100 psi compressive strength) approved by the project geotechnical engineer. Abandoned utilities should be removed entirely, or pressure-filled with concrete or grout and be capped. Abandoned buried utilities structures, or foundations should not extend under building limits.

After stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist (or their representative). Before the placement of fill or after cut, the existing surface soils within the building pads and improvement areas should be over-excavated as follows:

Structure Building Pad Preparation: Due to the estimated settlements from collapse soils analysis and assumed structural loading on the soil profile, the existing soils within the building pad and foundation (including covered eating area) will need over-excavation for use as a soil mitigation measure. We have provided recommendations in this report for soil densification (mat of compacted fill) which is one of the accepted methods of reducing localized differential settlement provided in SP117A (2008, page 57) and *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California*, Martin and Lew, 1999.

The over-excavation should extend a minimum of 4 feet below existing or finished grade, or 2 feet below the bottom of foundations, whichever is deeper. The over-excavation should extend for at least 5 feet beyond the outer edge of the building pad or the depth of overexcavation, whichever is greater in length, and include all exterior footings or slabs and include any overhead canopy/walkway areas, where possible. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. A sample of the excavation bottom should be evaluated for maximum density (ASTM D 1557). Deeper over-excavation may be recommended if the over-excavation bottom densities are not met. Once the bottom subgrade is attained and approved, the surface should be scarified an additional 6 inches, moisture conditioned to 3% above the optimum moisture content, and recompacted to a minimum of 90% compaction relative to ASTM D 1557.

After the approved subgrade has been scarified, moisture conditioned, and compacted as required above, engineered fill should be placed in 8-inch loose lifts at 3% above the optimum moisture content and compacted to at least 90% relative compaction (ASTM D 15557) prior to the placement of subsequent lifts until the remaining pad elevation is achieved.

Utilities and lifeline services provided from outside the structure could still suffer disruption unless mitigation measures are employed that would account for the soil deformations that could occur between the structure and the supporting soils.

Auxiliary Structures Subgrade Preparation: Auxiliary structures such as fence or retaining walls, etc. should have the foundation areas overexcavated similar to Building Pad Preparation section noted described above. The overexcavation should extend laterally a distance of 3 feet from the edge of footings. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. The exposed soils should then be moisture conditioned to 3% above the optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557). Moisture conditioned engineered fill may then be placed to finished subgrade in suitable, compacted lifts. Compaction should be verified by testing.

Masonry fence walls should include construction joints every 10 feet to minimize the potential for distress cracks.

Subgrade Preparation: In areas to receive fill, but not supporting structures or hardscape, the subgrade should be scarified; moisture conditioned and compacted to at least 90% relative

compaction (ASTM D 1557) for a depth of 1 foot below existing grade, or finished subgrade, whichever is deeper. Compaction should be verified by testing.

Hardscape Area Preparation (Including non-structure building areas which have increased settlement sensitivity): In hardscape areas, the subgrade should be over-excavated a minimum depth of 1 foot below existing grade or finish grade (whichever is deeper) and to a lateral extent of 2 feet beyond the outside edge. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. The excavation bottom should be scarified 6 inches, moisture conditioned to 3% over the optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction to finish grade prior to the placement of subsequence lifts, with the upper 1 foot compacted to at least 95% relative compaction in parking drive and track areas. Compaction should be verified by testing. Re-leveling may be required if soil collapse occurs in hardscape areas due to nearby irrigation. Irrigation is recommended to be minimal adjacent to hardscape.

Slope Construction: Please see Section 5.5 for detailed slope preparation recommendations.

All over-excavations should extend to a depth where the project geologist, engineer or his/her representative has deemed the exposed soils as being suitable for receiving compacted fill. The materials exposed at the bottom of excavations should be observed by a geotechnical engineer or geologist from our office prior to the placement of any compacted fill soils to verify that a competent bottom has been attained. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation.

Engineered Fill Soils: The existing fill and native soils when processed appropriately are considered to be suitable for use as engineered fill. Engineered fill should be generally free from expansive soil (Expansive defined as Expansion Index (EI) greater than 20), vegetation, trash, large roots, overly wet and/or soft soil, clods larger than 3 inches, construction debris, oversized rock (greater than 6 inches) and other deleterious material as determined by the geotechnical engineer or his representative. Deleterious materials should be hauled offsite. Engineered fill soils should have a “very low” Expansion Index.

Engineered fill (and any import) should be placed in maximum 8-inch lifts (loose) and compacted to at least 90 percent relative compaction (ASTM D 1557)(unless noted otherwise) at 3% above the optimum moisture content prior to placement of a subsequent loose lift. Within pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction (ASTM D 1557). Compaction should be verified by testing. Rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material, with the exception of playfield areas, where criteria necessitating a smaller oversize allowance may apply. Typically, in play field areas, the maximum oversize allowed is 1 inch.

Imported fill soils should be “very low” expansion potential granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35 percent passing the No. 200 sieve (unless otherwise approved by the geotechnical engineer). The geotechnical engineer should evaluate the import fill soils before hauling to the site. However,

because of the potential variations within the borrow source, import soil will not be prequalified by Earth Systems.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of testing may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). Alternative methods may include methods outlined in ASTM D 1556 (Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method), or correlated hand probing to verify firm conditions.

All soils should be moisture conditioned prior to application of compactive effort and prior to foundation, slab-on-grade and pavement placement. Moisture conditioning of soils refers to adjusting the soil moisture to 3% above the optimum moisture content. If the soils are overly moist so that instability occurs, or if the minimum recommended compaction cannot be readily achieved, it may be necessary to aerate to dry the soil to 3% above the optimum moisture content or use other means to address soft soils (as approved by the geotechnical engineer prior to use).

Shrinkage and Bulking: The shrinkage factor for earthwork for the alluvial materials is expected to range from 5 to 10 percent for the upper excavated or scarified site soils. This estimate is based on compactive effort to achieve a weighted average relative compaction of about 93 percent.

Shrinkage is highly dependent on and may vary with contractor methods for compaction. Losses from site clearing, oversize rock removal, and removal of existing site improvements, as well as the addition of excavated soil (footings, piers, etc.) may significantly affect earthwork quantity calculations and should be considered.

Dust Control: The proposed site lies within an area of moderate potential for wind erosion. The site soils have a fine-grained component of their composition. As such, exposed soil surfaces may be subject to disturbed fine particulate matter (PM_{10}) which can create airborne dust if the soil surface or roadways are not maintained. During construction, watering the soil surface can reduce airborne dust. Alternatively, a dust control palliative may be spray applied to the soil surface to act as a tackifier to contain loose soil particles. Palliatives must be reapplied periodically as they weather and degrade. Further guidance for dust palliatives can be found in reviewing the United States Department of Agriculture Publication *Dust Palliative Selection and Application Guide*, Document No. 9977-1207-SDTDC. The recommended soil input parameters are Plasticity Index <8, and fines content between 10 and 25 percent, for typical onsite fills.

5.2 Excavations and Shoring

Excavations should be made in accordance with Cal/OSHA requirements. Using the Cal/OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site-specific soil type per Cal/OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the contractors *Competent Person* (as defined by OSHA) during grading and trenching operations.

Our site exploration and knowledge of the general area indicates there is a moderate potential for caving and sloughing of site excavations (over excavation areas, utilities, footings, etc.) due to dry and also overly moist/wet conditions. Where excavations in soils over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal/vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation. Excavations should be protected from water flow over the exposed surface and saturation.

Excavations which parallel structures, pavements, or other flatwork, should be planned so that they do not extend into a plane having a downward slope of 1:1 (horizontal: vertical) from the bottom edge of the footings, pavements, or flatwork. Shoring or other excavation techniques may be required where these recommendations cannot be satisfied due to space limitations or foundation layout. Where overexcavation will be performed adjacent to existing structures, ABC slot cutting techniques may be used as pre-approved by the project geotechnical engineer.

Shoring: Shoring may be required where soil conditions, space, or other restrictions do not allow a sloped excavation or slot cutting is not an option. A braced or cantilevered shoring system may be used. Trench boxes should not be placed below or within the pipe zone elevation as their removal may loosen compacted backfill. Positive trench shoring may be required (jacks and plates).

A temporary cantilevered shoring system should be designed to resist an active earth pressure equivalent to a fluid weighing as shown in the table below. Braced or restrained excavations above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure as presented in the table below.

Table 7
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
40	60

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. Fifty percent of an aerial surcharge placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 240 psf for auto or delivery truck (2 axle) traffic kept at least 3 feet from the back of the wall. Retaining walls with closer traffic or heavier traffic loads should be designed for a 450 psf surcharge load. Retaining walls should be designed with a minimum factor of safety of 1.5.

The wall pressures above the groundwater do not include hydrostatic pressures; it is assumed that drainage will be provided. If drainage is not provided, shoring extending below the groundwater level should be evaluated on a case-by-case basis.

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 400 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,000 psf.

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the exploration logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. Shoring should be sealed to prevent the piping of soil material and potential soil loss conditions which can cause settlement. The method of excavation and support is ultimately left to the contractor.

5.3 Utility Trenches and Backfill

Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.). Utility trench backfill within private property should be placed in conformance with the provisions of this report. Backfill operations should be observed and tested to monitor compliance with these recommendations.

Trench Width and Vertical Loads on Pipelines: Vertical loads to the pipeline are highly dependent upon the geometry of the trench. In general, the narrower the trench is at the top of the pipe/conduit with respect to the diameter of the conduit, the less vertical load is applied to the conduit. This is because as the trench backfill and bedding compress or consolidate over time, the weight of the soil mass is partially offset by the frictional resistance along the trench sidewalls. In addition, the type of bedding supporting the pipeline affects the bearing strength of the conduit. This is accounted by a load factor that is multiplied to the design strength of the conduit. The pipe manufacturer recommendations for trench installation and maximum width should be followed to reduce the potential for overloading the pipe due to excess backfill load.

Pipe Subgrade and Bedding: Pipeline subgrade should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative for a depth of 6 inches below any bedding. Bedding material shall consist of sand with 100 percent passing a No. 4 sieve and less than 5 percent fines (passing a No. 200 sieve), and a sand equivalent of 30 or more or as approved by the project inspector and geotechnical engineer.

Pipe-Zone, Trench–Zone, Trench Backfill and Compaction: Backfill of utilities should be placed in conformance with the requirements of the specifications. Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.).

Pipe-zone backfill material (the pipe area from the bedding to 12 inches above the top of pipe) may consist of sandy native soils screened to a ¾" maximum particle size or import sand (as described above for bedding) as dictated by the pipe designer or manufacturer. The pipe zone backfill material should be placed in maximum 8-inch lifts (loose) and compacted near its optimum moisture content prior to the placement of subsequent lifts. Pipe zone backfill should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative. Compaction should be assured in the pipe haunches.

The native soil is suitable for use as trench zone and street zone (and manholes) backfill (from the top of pipe zone up to finished grade), provided it is free of significant organic or deleterious matter and oversize materials. This backfill shall contain no particles larger than 3 inches in greatest dimension. The final backfill material should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) at 3% above the optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) when below pavement. Compaction or firmness should be verified by testing.

Backfill materials should be brought up at substantially the same rate on both sides of the pipe or conduit. Reduction of the lift thickness may be necessary to achieve the above recommended compaction. Care should be taken to not overstress the piping during compaction operations. Mechanical compaction is recommended; ponding or jetting is not recommended.

Alternatively, if the utility cannot accommodate the increased stress, or if compaction is difficult, we recommend the pipe be encased by at least 1 foot of 2-sack cement-sand slurry (at least 1 foot as measured from the top of pipe). Backfill operations should be observed and tested to monitor compliance with these recommendations.

In general, coarse-grained sand and/or gap graded gravel (i.e., ¾-inch rock or pea-gravel, etc.) should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel. Gravel should be separated from backfill with a filter fabric such as Mirafi 140N or equivalent as approved by the soils engineer. Water seepage or soil migration will cause settlement of the overlying soils.

Compaction should be verified by testing. Backfill operations should be observed and tested to monitor compliance with these recommendations. Trench backfill compacted per these requirements can be expected to settle 0.1 to 0.3 percent of the trench depth. This can cause an elevation difference between backfilled trenches and the surrounding soil or pavement. Increased relative compaction can reduce the settlement if the potentials presented are not acceptable. The geotechnical engineer should be consulted on a case-by-case basis to provide further recommendations to reduce the settlement potential.

STRUCTURES

In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on "very low" expansion category soils.

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 to 18 inches (below lowest adjacent grade) should be maintained and considers a bearing soil with a "very low" expansion potential. Lowest adjacent grade is the lowest grade within 5 feet laterally of the footing edge. A representative of Earth Systems should observe foundation excavations to verify compaction (minimum 90% per ASTM D 1557) before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. *All footing excavations should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. The bottom of all foundations should be tested to confirm compaction effort and moisture contents as stated in Section 5.1 of this report are met.* The moisture contents should be at least the indicated moisture content 24 hours prior to and immediately prior to placing concrete for a depth of at least 12 inches below the foundation subgrade. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Minimum Slope Setback for Foundations: Earth Systems recommends a minimum setback distance of 5 feet. The 2019 California Building Code provides setback distances for foundations along slopes. Setback distances are measured differently for foundations located above the slope and those located below the slope. For foundations located at the top of the slope, the measurement is taken horizontally from the outside face of the foundation footing to the face of the slope. For foundations located below the slope, the horizontal distance is measured from the face of the structure foundation to the toe of the slope. For pools and slopes steeper than 1H:1V (horizontal to vertical), please contact Earth System for these setbacks with submittal of detailed information using plan form.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous foundations, 1-foot minimum width and 3-foot maximum width embedded a minimum of 12 inches below finished grade:

  1,500 psf for dead plus design live loads

- Pad foundations, 2 x 2-foot minimum and 4 x 4-foot maximum in plan embedded a minimum of 18 inches below finished grade:

  2,000 psf for dead plus design live loads

- Foundations at Grade, 12-inch minimum width, 600 psf for Dead & Live loads. Bearing pressure is based on a soil having a minimum Soil Friction Angle (Phi) of 33 degrees with a dry unit weight 123 pcf or greater.

As the allowable bearing pressure is settlement governed, one-third (⅓) increase in the allowable bearing pressure may be used when calculating resistance to wind or seismic loads.

If the actual structural loads exceed the assumed values stated in Section 1.1 (100 kips for isolated pad footings and 3 kips/linear foot for continuous wall footings), the geotechnical engineer must reevaluate the allowable bearing values as the allowable bearing was controlled

by the allowable total differential settlement from dry seismic, collapse, and static loads. Underground utilities should be designed for an anticipated settlement within the building areas.

The spacing between any large spread footings should be evaluated by the geotechnical engineer during the plan review stage to confirm or modify the settlement estimates and bearing capacity due to large footings and the influences from adjacent footings. A preliminary analysis suggests spacing the footings (adjacent edge to adjacent edge) a lateral distance from one another of the width of the largest footing from any adjacent footing, such that influence effects are minor. A 1:1 projection downward from the bottom edge of footings may be considered in the evaluation of foundation influence zone.

Maximum foundation sizes given above are based on settlement due to Dead + Live loads. Transient loads such as earthquake or wind loads are not subject to the stated size limitations; however, the allowable bearing pressure (including ⅓ increase) should be followed considering the relevant foundation sizes given above.

An average modulus of subgrade reaction, k, of 150 pounds per cubic inch (pci) can be used to design lightly loaded footings, beams, pavement, and slabs founded upon compacted fill. Other foundations such as mat slabs, will require the use of differing modulus of subgrade reaction values than used for lightly loaded slabs. Please contact Earth Systems for k values used for mat foundations.

The table below is based upon the above presented allowable, short term, and ultimate bearing pressures. Short Term allowable bearing may use the values presented below (based on Allowable Stress Design) or be based on Code mandated structural reductions, whichever is less. Ultimate bearing capacities consider a factor of safety of 3 (ASD design) to control settlement and bearing failure considering deep groundwater (4,500 to 6,000 psf ultimate) and a safety factor of 2.25 on transient loads (2,000 to 2,667 psf). Ultimate bearing to soil failure depends on foundation size and could be greater than 6,000 psf. The restrictions of Section 1605A.1.1 apply to the cited bearing values for Allowable Stress Design (ASD).

Table 8

	Allowable Bearing Capacity (psf) (FS = 3)	**Short Term (Wind/Seismic) (FS = 2.25)**	**Ultimate Bearing Capacity (FS = 1)**
Continuous Foundations	1,500	2,000	4,500
Isolated Pad Foundations	2,000	2,667	6,000

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, settlement, or collapse soil conditions defined herein.

Stepped foundations should be designed in accordance with the 2019 CBC. CBC 2019 and ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. Seismic Design Category for compressive strength determination is 'D'. Due to the negligible sulfates in the site soils, normal cements may be and should be proportioned in accordance with ACI recommendations considering the time of year for placement. Hot weather proportions should be used during high ambient heat days during placement and curing.

Expected Settlement: Estimated total static settlement should be less than ½ inch, based on footings founded on compacted soils as recommended. Differential static settlement between similarly loaded members should be less than ¼ inch.

As such, considering static settlement applied over a typical foundation distance of 30 feet, we recommend the structural engineer design for the typical angular distortion of 1:480 (¾ inch in 30 feet). Settlement will not result in the complete loss of soil support but will be manifested as a tilting of the structure over the applied distance.

We estimate that up to ½ of the estimated collapse settlement could occur (¼ inch of differential collapse settlement with mitigation).

Estimated total static differential (¼ inch), seismic differential (¼ inch), and collapse differential (¼ inch) settlement should be less than ¾ inch, based on footings founded on compacted soils as recommended.

As such, considering static, seismic, and collapse settlement we recommend the structural engineer design for an angular distortion of 1:360. Settlement will not result in the complete loss of soil support but will be manifested as a tilting of the structure over the applied distance. The estimated differential settlement does not exceed the Differential Threshold of ASCE7-16 Table 12.13-3 for Risk Category III, "Other single-story structures".

Settlement calculations are presented in Appendix A and collapse results are provided in Section 3.3. The actual settlement of large spread footings should be evaluated by the geotechnical engineer during the plan review stage based on the actual column loads to confirm or modify the settlement estimates presented.

Minor Structures Deep Foundations: Although no specific elements were identified by the architect, for miscellaneous structural components such as light poles, small shade structures, gate posts, temporary retaining walls, and flag poles, may be supported on cast-in-place piles, or direct embed in drilled holes filled with concrete, and the design be based on parameters presented in the subsequent sections of this report. Construction employing poles or posts may utilize design methods presented in Section 1806A and 1807A of the CBC for Silty Sand (SM) material class and utilize the strength parameters of Table 1806A.2. For designs utilizing allowable frictional resistance, Earth Systems recommends the use of Section 1810A.3.3.1.4 and 5 of the CBC. For piles with an axial and/or lateral load, these design methods apply for piles spaced at least 3 and 6 pile diameters, respectively, center to center as graded in accordance with Section 5.1. Piles spaced closer than these limits could have soil strength reduction and should be evaluated on a case-by-case basis by geotechnical engineer. Piles should be designed for frictional resistance only.

For piers founded in areas with native soil at the surface, an additional 1.5 feet should be added to the calculated pile embedment due to the potential effects of long-term surficial disturbance and erosion. Additionally, where piers are constructed adjacent to the tops of slopes, there should be a minimum distance between the top of the slope and the closest edge of the pier of H/3, where 'H' is the height of the slope, otherwise a lateral resistance reduction must be applied. For piers founded closer than a distance H/3 to the crest or within the slope area itself, the calculated lateral resistance of the soil should be reduced by 30 percent. The above

recommendations have considered slopes no steeper than 2H:1V (horizontal to vertical). Piles within H distance from a slope or within a slope (where H is the slope Height) slopes will require additional analysis by the geotechnical engineer and may change the recommendations presented.

Drilled piers should have a minimum clearance of 3 inches between the embedded post and the soil side wall to allow for adequate placement and flow of concrete. Drill holes may end up oversize. Casing or other means may be required in a drilled hole. Any "slough" or loose soils at the bottom of the shaft must be removed or tamped prior to setting rebar cages and placing concrete. Extreme care must be exercised to carefully position reinforcing steel cages and place concrete without disturbing the sidewalls of the drilled shafts. We recommend centralizers be used to positively locate rebar cages within the pier shaft. It is recommended that pier excavations that have not received concrete, not be left open and concrete should be placed immediately. Caving is a high concern.

Normally, drilled pier excavations should be made without the use of water. If necessary, water may be used to facilitate removal of cuttings unless it aggravates caving problems. Added water that may accumulate at the bottom of the hole should be removed from the drilled hole prior to placing the concrete. Sidewalls which have softened from the addition of water should be cleaned of the soft/loose material. Each excavation should be completed in a continuous operation and the concrete should be placed without undue delay. The contractor should use appropriate means to clean the bottom of the excavation so that no loose material is present at the base of the pier. We do not recommend over drilling beyond specified pier tip elevations to eliminate the need for bottom cleaning in order to account for slough or loose materials at the excavation bottom. To reduce the potential for caving and sidewall sloughing which may contaminate concrete during placement, and segregation, concrete should be placed by tremie methods and not directly chute-dumped into the hole.

Where casing is used with drilled holes and cannot be withdrawn, the skin friction capacity is theoretically reduced, as are passive resistance and stiffness. The amount of reduction is subject to assessment by the geotechnical consultant. The use of casing with drilled holes should be approved prior to use by the geotechnical engineer.

If casing is required, it should be withdrawn as the concrete is being placed, maintaining a 3-foot minimum head of concrete within the casing. This is to prevent reduction in the diameter of the drilled shaft due to earth pressure on the fresh concrete and to prevent extraneous material from falling in from the sides and mixing with the concrete. Concrete placement should continue in this manner until suitable concrete extends to the top of the excavation or forms. The upper eight feet of the pier should be consolidated by vibratory means.

Pier capacity is greatly dependent on the soil conditions at the location of the pier and upon contractor means and methods of placement. It is recommended that drilling operations and concrete placement be performed in the continuous presence of the geotechnical consultant or his representative to confirm that suitable materials for pier support are penetrated, that the dimensions of the installed piers meet the design dimensions, and that the installation has been performed as specified by the 2019 California Building Code. Observation during drilling is required by the 2019 California Building Code on a full-time basis by the geotechnical engineer or their representative. If subsurface conditions noted during drilled pier installation are

significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

Prior to the placement of steel, and again prior to and during the placement of concrete, the excavation must be examined by the geotechnical consultant before proceeding with construction. The contractor should provide all aid and assistance required by the geotechnical and geologic consultants for field monitoring of the drilled pier operations.

Piers are accepted or rejected based on visual observation and testing during construction. The contractor should not allow nor cause any of this work to be permanently enclosed or covered up until it has been observed, tested, and accepted by the geotechnical engineer and all legally constituted authorities having jurisdiction.

5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Compacted fill slopes protected against erosion (per approved methods such as significant planting, facing, or erosion blankets, etc.) may be constructed at 2H:1V (horizontal to vertical) or flatter inclinations. Unprotected slopes with exposed native soils or compacted fill at the surface should be expected to require repair after heavy nuisance or storm runoff occurs due to significant erosion. Slope recommendations may change pending a more in-depth geotechnical evaluation once design plans are developed.

Compacted fill should be placed at 3% above the optimum moisture content and compacted to a minimum 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures. The exposed face of any cut or fill slope (upper 12 inches) should have a minimum relative compaction of 90 percent, as measured in relation to ASTM D 1557 test procedures and be compacted at 3% above the optimum moisture content. Due to the erodible site soils, slope faces should be protected with facing or densely spaced vegetation to reduce the erosion potential.

Surficial Slope Failures: All slopes will be exposed to weathering, resulting in decomposition of surficial earth materials, thus potentially reducing shear strength properties of the surficial soils. In addition, these slopes become increasingly susceptible to rodent burrowing. As these slopes deteriorate, they can be expected to become susceptible to surficial instability such as soil slumps, erosion, soil creep, and debris flows. Development areas immediately adjacent to ascending or descending slopes should address future surficial sloughing of soil material and erosion. Such measures may include debris fences, slope facing, catchment areas or walls, diversion ditches or berms, soil planting, velocity reducers or other techniques to contain soil material away from developed areas and reduce erosion. Additionally, foundations should be set back at least 5 feet from the face of slope (to daylight) or as per the 2019 CBC, whichever is greater.

Operation and maintenance inspections should be done after a significant rainfall event and on a time-based criterion (annually or less) to evaluate distress such as erosion, slope condition, rodent infestation burrows, etc. Inspections should be recorded, and photographs taken to document current conditions. The repair procedure should outline a plan for fixing and maintaining surficial slope failures, erosional areas, gullies, animal burrows, etc. Repair methods

could consist of excavating and infilling with compacted soil erosional features, track walking the slope faces with heavy equipment, as determined by the type and size of repair. These repairs should be performed in a prompt manner after their occurrence. Slope inclinations should be maintained, and a maintenance program should include identifying areas where slopes begin to steepen. Where future maintenance is not possible, slopes should be faced to reduce the erosion and degradation potential.

Slope faces are highly erodible even if compacted and will gradually erode and move down slope presenting maintenance issues and debris deposited in drainage devices and flatwork areas. The minimum material necessary to support landscaping should be specified by the landscape consultant (typically less than 6 inches).

5.6 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with Section 5.1 of this report. The moisture content below slabs should be at least 3% above the optimum moisture content 24 hours prior to and immediately prior to placing concrete for a depth 12 inches. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Vapor Retarder: In areas of moisture-sensitive floor coverings, coatings, adhesives, underlayment, goods or equipment stored in direct contact with the top of the slab, bare slabs, humidity-controlled environments, or climate-controlled cooled environments, an appropriate vapor retarder that maintains a permeance of 0.01 perms or less after ASTM E1745's mandatory conditioning tests should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, a vapor retarder (Stego wrap 15-mil thickness or equal) should underlie the floor slabs. If a Class A vapor retarder (ASTM E 1745) is specified, the retarder can be placed directly on non-expansive soil and be covered with a minimum 2 inches of clean sand.

Clean sand is defined as well or poorly graded sand (ASTM D 2488) of which less than 5 percent passes the No. 200 sieve and all the material passes a No. 4 sieve. The site soils do not fulfill the criteria to be considered clean sand. Alternatively, the slab designer may consider the use of other vapor retarder systems that are recommended by the American Concrete Institute.

Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, the successful sealing of the membrane around utility lines, and sealing the membrane at perimeter terminations and of all penetrations. Capillary breaks, if any, beneath slabs should consist of a minimum of at least four inches of permeable base material (Caltrans Class 2 Permeable Base) with the following specified gradation.

Table 9
Percent Passing Sieve Size

Sieve Size	Percent Passing
1 inch	100
¾ Inch	90-100
3/8 Inch	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Where vapor retarders are placed directly on a gravel capillary break, they should be a minimum of 15-mil thickness.

Where the 2-inch sand layer overlying the vapor retarder is omitted, and concrete is placed directly on the vapor retarder "plastic", proper curing techniques are essential to minimizing the potential of slab edge curl and shrinkage cracking. The edges of slabs can curl upward because of differential shrinkage when the top of the slab dries to lower moisture content than the bottom of the slab. Curling and cracking are caused by the difference in drying shrinkage between the top and bottom of the slab. Curling and cracking can be exacerbated by hot weather, or dry condition concrete placement, even with proper curing techniques.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. A design engineer should be retained to provide building specific systems to handle subgrade moisture to ensure compliance with SB800 with regards to moisture and moisture vapor.

Slab Thickness and Reinforcement: Slabs should be a minimum of 4 inches in actual thickness and be reinforced with # 3 bars at 18 inches on center both ways. Slabs in contact with earth should use closer joints to control cracking or be thickened to allow adequate earth to rebar clearance. Reinforcing bars should extend at least 40 bar diameters into the footings and slabs. Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with this report. If slabs are structural, they should be designed for the specific settlement conditions presented within.

Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and the Expansion Index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 150 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat slab design will require differing modulus values. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength.

If heavily loaded flatwork is proposed (forklift drive areas, heavy racking, etc.), the actual thickness should be designed by the structural engineer utilizing techniques of the American Concrete Institute (ACI) and may be greater than 4 inches in thickness. Concrete floor slabs may either be monolithically placed with the foundations or doweled (No. 4 bar embedded at least 40 bar diameters) after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or concrete inspector should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab. The minimum concrete rebar cover should be as per the project architect or structural engineer.

Slab-On-Grade Control Joints: Control joints should be provided in all regular concrete slabs-on-grade at a maximum spacing of 26 to 36 times the slab thickness (12 feet maximum on-center each way, 4 to 6 feet for sidewalks) as recommended by American Concrete Institute [ACI] guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented shrinkage cracks. Control joints in the slabs should be tooled at the time of the concrete placement or saw cut (¼ of slab depth) as soon as practical but not more than 8 hours from concrete placement.

Construction (cold) joints should consist of thickened butt joints with ¾-inch dowels at 18 inches on center embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks but may not prevent them from occurring.

Curing and Quality Control: The contractor should take precautions to reduce the potential of curling and cracking of slabs in this arid desert region using proper batching, placement, and curing methods. Curing is highly affected by temperature, wind, and humidity.

Quality control procedures should be used, including trial batch mix designs, batch plant inspection, and on-site special inspection and testing. Curing should be in accordance with ACI recommendations contained in ACI 211, 304, 305, 308, 309, and 318. Additionally, the concrete should be vibrated during placement. Concrete should be wet cured for at least 7 days with burlap or plastic and not allowed to dry out to minimize surface cracking.

5.7 Retaining Walls and Lateral Earth Pressures

Walls which are restrained at the top such as retaining wall returns, below-grade walls and walls tied to floor slabs should be designed with “at rest” earth pressures. Retaining walls, free to tilt (0.1% of wall height) at the top, may be designed for “active” earth pressures.

The following list presents lateral earth pressures for use in wall design. The values are given as equivalent fluid pressures ***without*** surcharge loads or hydrostatic pressure. Clay soils are not suitable for wall backfill as they are not free draining. Native sand material may be used for backfill or free draining material imported as wall backfill. For native or import free draining material, active and restrained walls equivalent fluid pressures are as follows:

- Conventional cantilever retaining walls may be backfilled with compacted on-site soils verified by the contractor to be “very low” in expansion potential. Provided the wall is backfilled at a 1:1 projection upward from the heels of the wall footings with granular

backfill, an active pressure of 40 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 50 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 2H:1V (horizontal to vertical). For the restrained level backfill condition, a pressure of 62 pcf of equivalent fluid weight should be used.

- In addition to the active or at rest soil pressure, the proposed wall structures should be designed (where not excepted) to include forces from dynamic (seismic) earth pressure. Dynamic pressures are additive to active and at-rest earth pressure, follow their pressure distribution, and should be considered as 0 pcf for flexible walls, and 2 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.55 g, Friction Soil Angle of 33°, and a maximum total density of 136 pcf.
- Retaining wall foundations should be placed upon compacted fill described in Section 5.1.
- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Design should be in accordance with the 2019 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum and should have a volume of 1 cubic foot per foot of length. Backfill immediately behind the retaining structure should be a free-draining granular material. Waterproofing should be according to the designer's specifications. Water should not be allowed to pond or infiltrate near the top of the wall. To accomplish this, the final backfill grade should divert water away from retaining walls.
- Compaction on the retained side of the wall within a horizontal distance equal to one wall height (to a maximum of 6 feet) should be performed by hand-operated or other lightweight compaction equipment (90% compaction relative to ASTM D 1557 at 3% above the optimum moisture content). This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment or dislodging modular block type walls.
- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a minimum uniform surcharge load equivalent of 240 psf for auto and 450 psf for truck traffic kept back at least 3 feet from the wall back edge. Retaining walls should be designed with a minimum factor of safety of 1.5.

<u>Frictional and Lateral Coefficients</u>:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying

soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.35 (Factor of Safety = 1.2) may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.30 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil.

- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf)(inherent Factor of Safety = 1.5). The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.
- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 350 pcf. The maximum passive resistance should not exceed 2,000 psf.
- The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are over widened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.

5.8 Surface and Subsurface Site Drainage and Maintenance

Positive drainage should be maintained away from the structures and undersides of raised floor structures (5 percent for 10 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts in conjunction with a 1 to 2% hardscape grade can be considered as a means to convey water away from foundations if increased fall is not provided. Drainage should be maintained for paved areas. Water should not pond on or near paved areas or foundations. Ponded water can saturate subgrade soils and lead to pavement failure. The following recommendations are provided in regard to site drainage and structure performance:

- Water control and conveyance is a critical aspect of project design. It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Landscaping grades should be lowered and sloped such that water drains to appropriate collection and disposal areas. All runoff water should be controlled, collected, and drained into proper drain outlets. Control methods may include curbing, ribbon gutters, 'V' ditches, or other suitable containment and redirection devices.
- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over slopes or retaining wall faces. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales at the top and bottom of slopes, and at the top of retaining walls are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed

to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken sprinklers, etc, should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.

- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to unfactored Basic Percolation/Infiltration Rates to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 12 should be applied to lower infiltration rates.
- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of testing, filters, and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, soil crust removal, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.
- Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1:1 (h:v) from the projection upward from the bottom of any foundation, whichever is greater.

5.9 Infiltration

During our field exploration, percolation testing was performed at Borings P-1 and P-2 to depths ranging from 5 feet to 15 below existing ground surface. A perforated PVC casing was placed in the borings and the space between the casing and excavation was filled with gravel. A pre-soak of the borings was performed prior to starting the infiltration test. The percolation test was performed in accordance with the Small Diameter Boring, Shallow Infiltration Test procedure in the Los Angeles County Public Works Geotechnical and Materials Engineering Division manual (June 2021) which is a test procedure performed in a boring and a typical standard of care procedure utilized in the area. The test results were converted empirically per similar LA County methods to infiltration rate for use in Kern County.

Table 10
Infiltration Test Summary

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in/hr)	Design Infiltration Rate (in/hr)
P-1	SM	2.5-5	0.0	0.0
P-2	SM	9-15	0.3	0.1

Based on the small diameter boring infiltration test procedure, the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 2.0. In addition, provided proper maintenance is performed to minimize long-term siltation and clogging, the reduction factor RF_s may be taken as 2.0, for a total reduction of 6. If measures are not taken, RF_s should be taken as 3.0 for a total reduction of 7. The design of any stormwater infiltration system should be based on applicable guidelines and additional reduction factors may be necessary.

The results of the percolation testing indicate that infiltration is not considered feasible at P-1 and P-2, where the subsurface materials have a design infiltration rate of less than 0.3 inch per hour.

Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1H:1V (horizontal to vertical) from the projection upward from the bottom of any foundation, whichever is greater.

5.10 Driveways and Parking Areas

Pavement structural sections for associated drive areas including recommendations for standard asphalt concrete, and Portland cement concrete are provided below and are based upon on-site soils as described in Section 5.1. Soils differing from those described will require differing pavement sections. The appropriate pavement section depends primarily on the shear strength of the subgrade soil exposed after grading in the near finished subgrade elevation and the anticipated traffic over the useful life of the pavement. Based on the on-site soils and our experience in the site vicinity, an 'R-Value' of 25 was selected as a design value for subgrade support beneath pavements at the site. R-value testing or observation of subgrade soils should be performed on near finished subgrade elevation soils to verify and/or modify the preliminary pavement sections presented within this report.

Pavement Area Preparation: In street, drive, and parking areas, the exposed subgrade should be overexcavated as recommended in Section 5.1, moisture conditioned, and compacted. Compaction should be verified by testing. Aggregate base should be compacted to a minimum 95% relative compaction (ASTM D 1557).

Automobile Traffic and Parking Areas: Pavement sections presented in the following table for automobile type traffic areas and are based on a tested R-value and current Caltrans design procedures. Traffic Indices (TI) of 5 and 7 were used to facilitate the design of asphalt concrete pavements for parking and main drives, including fire lanes. The fire lane calculation assumed a

conservative traffic flow of one fire truck per day entering and exiting the site on the same path (20-year life cycle), and a maximum loading of an 88,000-pound Tandem Axle apparatus (approximate 20,000-pound front axle load and two 34,000-pound rear axles loads) which is based upon the *Emergency Vehicle Size and Weight Regulation Guideline*, dated November 22, 2011, prepared by the Fire Apparatus Manufacturers' Association.

Based on the above stated traffic pattern and apparatus loads, a Traffic Index of 4.6 is calculated for fire lanes. For comparison, a 40-year fire lane life cycle analysis results in a Traffic Index of 5. The TI's assumed below should be reviewed by the project Civil Engineer to evaluate the suitability for this project. All design should be based upon an appropriately selected traffic index. Changes in the traffic indices will affect the corresponding pavement section.

Table 11
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils – 25 (Assumed) Design Method - Caltrans

Traffic Index (Assumed)*	Pavement Use	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5	Parking Areas & Fire Lanes ***	3.0	6.5
7	Main Drive Areas	4.0	11.0

*The presented Traffic Indices should be confirmed by the project civil engineer. Changes to the Traffic Index will result in a differing pavement section required.
**Pavement Sections were calculated using Caltrans Design Method.
***Where fire lanes will be a part of a main drive use with other traffic, busses, or trucks, the Main Drive Area pavement section should be used.

Conventional, rigid pavements, i.e., Portland cement concrete (PCC) pavements, are recommended in areas that will be subject to relatively high static wheel loads and/or heavy vehicle loading and unloading and turning areas (i.e., truck/bus lanes). This is due to rutting and shoving that can occur due to the heavy vehicle loads and the repetitious set path which if followed at the bus/delivery truck areas where the same wheel track and stopping occurs generally in the same spot each time. The vehicle load combined with hot summer asphalt (AC) concrete causes the upper surface of the AC to creep forming ruts in conjunction with the braking and accelerating which shove the AC. Turning forces also do the same.

The pavement section below is based upon the American Concrete Institute (ACI) Guide for Construction of Concrete Parking Lots, ACI 330R, utilizing a Jointed Plain Concrete Pavement (no rebar) with load resistance based upon PCC thickness, and the assumptions outlined below. The civil engineer and client may elect to incorporate reinforcing bar into PCC to control the opening of typical fracture faces should it be desired; however, rebar is not needed for load support. ACI 330 R provides reinforcement options.

Table 12
Preliminary Portland Cement Concrete Pavement Sections

Area	Minimum Pavement PCC Thickness* (inches)	Minimum Class 2 Aggregate Base Thickness* (inches)	Minimum 28 Day Flexural Strength (psi)	Concrete Compressive Strength (psi)
Truck/Bus Access or Loading/Unloading Areas/Entrance-Exterior Lanes (Traffic Category B, ADTT = 25)	6.0	4.0	550	4,000

*See Section 5.6 for other slab-on-grade thickness recommendations.

Should the actual traffic category vary from those assumed and listed above, these sections should be modified. All above recommended preliminary pavement sections are contingent on the following recommendations being implemented during construction:

- Pavement should be placed upon compacted fill processed as described in Section 5.1. The upper 12 inches of subgrade soils beneath the asphalt concrete and conventional PCC pavement section should be compacted to a minimum of 95% relative compaction (ASTM D 1557).
- Portland cement concrete pavement may be placed directly on the compacted soil subgrade where soils in the upper 12 inches are confirmed to very low in Expansion Index.
- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to 3% above optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof rolled to verify the absence of soft or unstable zones.
- Aggregate base materials should be compacted at near optimum moisture content to at least 95% relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook" standards (Crushed Aggregate Base class) may be used in lieu of Caltrans and should be used for all offsite work as Per City of Rosamond requirements. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e., fully loaded water trucks).
- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).
- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.

- Portland cement concrete pavements should be constructed with transverse joints at maximum spacing of 12 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (7 inches for a 6-inch pavement) and should taper back to the PCC thickness over a horizontal distance on the order of 3 feet.
- All longitudinal or transverse control joints should be constructed by hand forming or placing pre-molded filler such as "zip strips." Expansion joints should be used to isolate fixed objects abutting or within the pavement area. The expansion joint should extend the full depth of the PCC pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures. In addition, the following is recommended for concrete pavements:
    1. Slope pavement at least ½ percent to provide drainage;
    2. Provide rough surface texture for traction;
    3. Cure PCC concrete with curing compound or keep continuously moist for a minimum of 7 days;
    4. Keep all traffic off concrete until PCC compressive strength exceeds 2,000 pounds per square inch (truck traffic should be limited until the concrete meets the design strength (3,650 psi); and
    5. Consideration should be given to having PCC construction joints keyed or using slip dowels on 24-inch centers to strengthen control and construction joints. Dowels placed within dowel baskets should be incorporated into the concrete at each saw-cut control joint (i.e., dowel baskets and dowels are set in place prior to placement of concrete).
- Portland cement concrete placement and curing should, at a minimum, be in accordance with the American Concrete Institute (ACI) recommendations contained in ACI 211, 304, 305, 308, 309, and 318.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.
- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.
- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at 3% above the optimum moisture content, or 95% if placed under areas designated for pavement.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for

both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.

- Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

The planning and construction process is an integral design component with respect to the geotechnical aspects of this project. Because geotechnical engineering is an inexact science due to the variability of natural processes and because we sample only a small portion of the soil and material affecting the performance of the proposed structure, unanticipated or changed conditions can be disclosed during demolition and construction. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process and to verify that our geotechnical recommendations have been properly interpreted and implemented during construction. Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation or the applicability of our recommendations. The above services can be provided in accordance with our current Fee Schedule.

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our study. The influence(s) of post-construction changes to these conditions such as introduction or removal of water into or from the subsurface will likely influence future performance of the proposed project. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions due to the limitation of data from field studies. The availability and broadening of knowledge and professional standards applicable to engineering services are continually evolving. As such, our services are intended to provide the Client with a source of professional advice, opinions and recommendations based on the information available as applicable to the project location and scope. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems.

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards occur, whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

This report is issued with the understanding that the owner or the owner's representative has the responsibility to bring the information and recommendations contained herein to the attention of the architect and engineers for the project so that they are incorporated into the plans and specifications for the project. The owner or the owner's representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee, express or implied, is made. This report was prepared for the exclusive use of the Client and the Client's authorized agents.

Earth Systems should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations. The owner or the owner's representative has the responsibility to provide the final plans requiring review to Earth Systems' attention so that we may perform our review.

Any party other than the client who wishes to use this report shall notify Earth Systems of such intended use. Based on the intended use of the report, Earth Systems may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Earth Systems from any liability resulting from the use of this report by any unauthorized party.

In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from Earth Systems' engineer that such changes do not affect our recommendations. Failure to do so will vitiate Earth Systems' recommendations.

Although available through Earth Systems, the current scope of our services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services. ***The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.***

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to, the following:

- Consultation during the final design stages of the project;
- A review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design;
- Observation and testing during site preparation, grading, and placement of engineered fill as required by CBC Sections 17 and Appendix J or local grading ordinances;
- Consultation as needed during construction.

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AERIAL PHOTOGRAPHS REVIEWED

Index: ABL-1942; Frames 5K 36, 37, and 38; October 13, 1952; Scale 1:20,000.

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APPENDIX A

Plate 1 – Site Vicinity Map
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Plate 3 – State of California Geomorphic Map
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Plate 5 – Regional Fault Map
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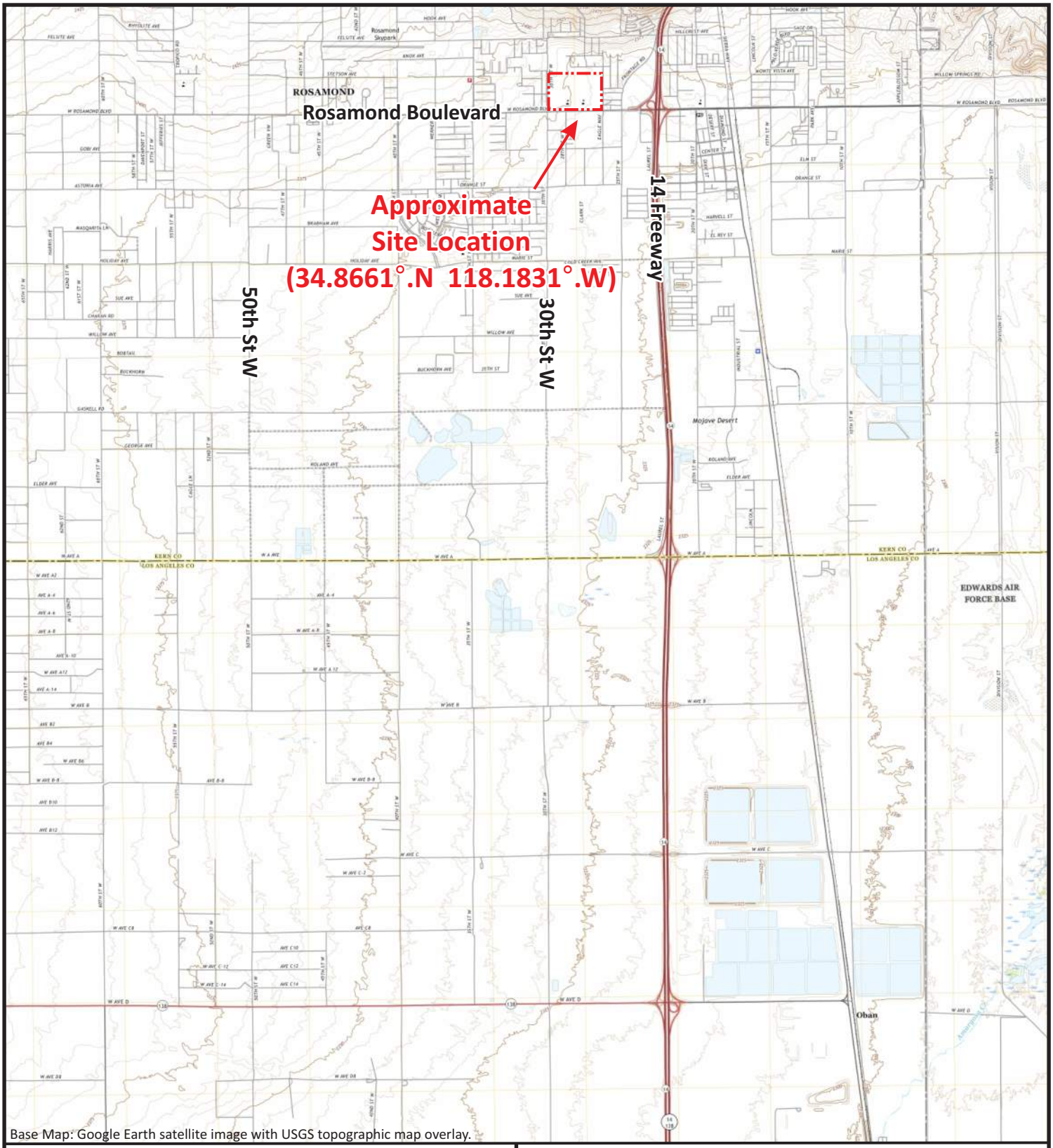
Site Class Estimate
Dry Seismic Settlement



Base Map: Google Earth satellite image with USGS topographic map overlay.

LEGEND

Approximate Site Boundary

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Plate 1
Site Vicinity Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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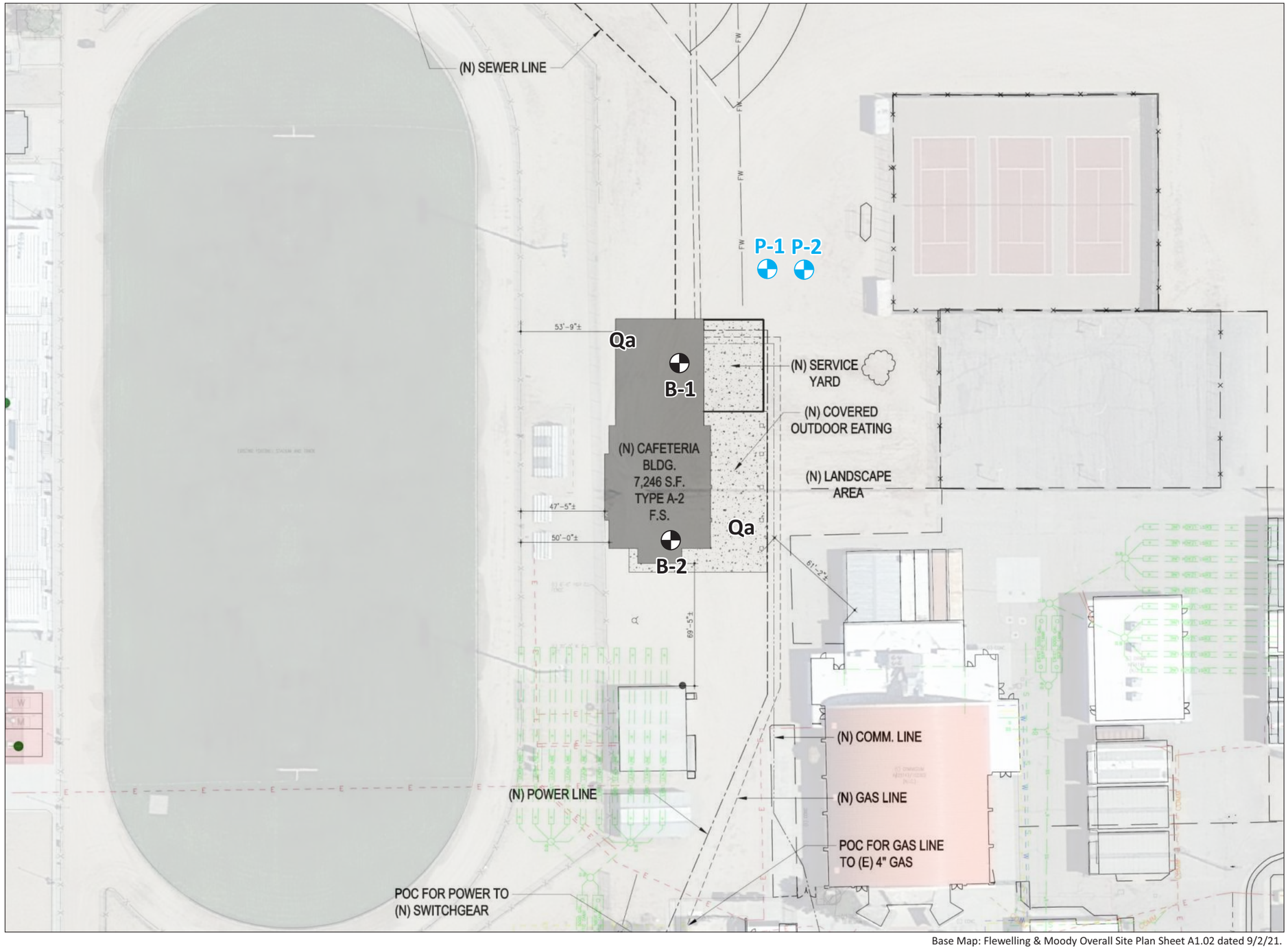

Base Map: Flewelling & Moody Overall Site Plan Sheet A1.02 dated 9/2/21.

LEGEND

B-1	Approximate Exploration Locations
P-1	Approximate Infiltration Test Location
Qa	Quaternary Alluvial Deposits

N

Approximate Scale: 1" = 60'

0 60' 120'

Plate 2
Exploration Location and Geologic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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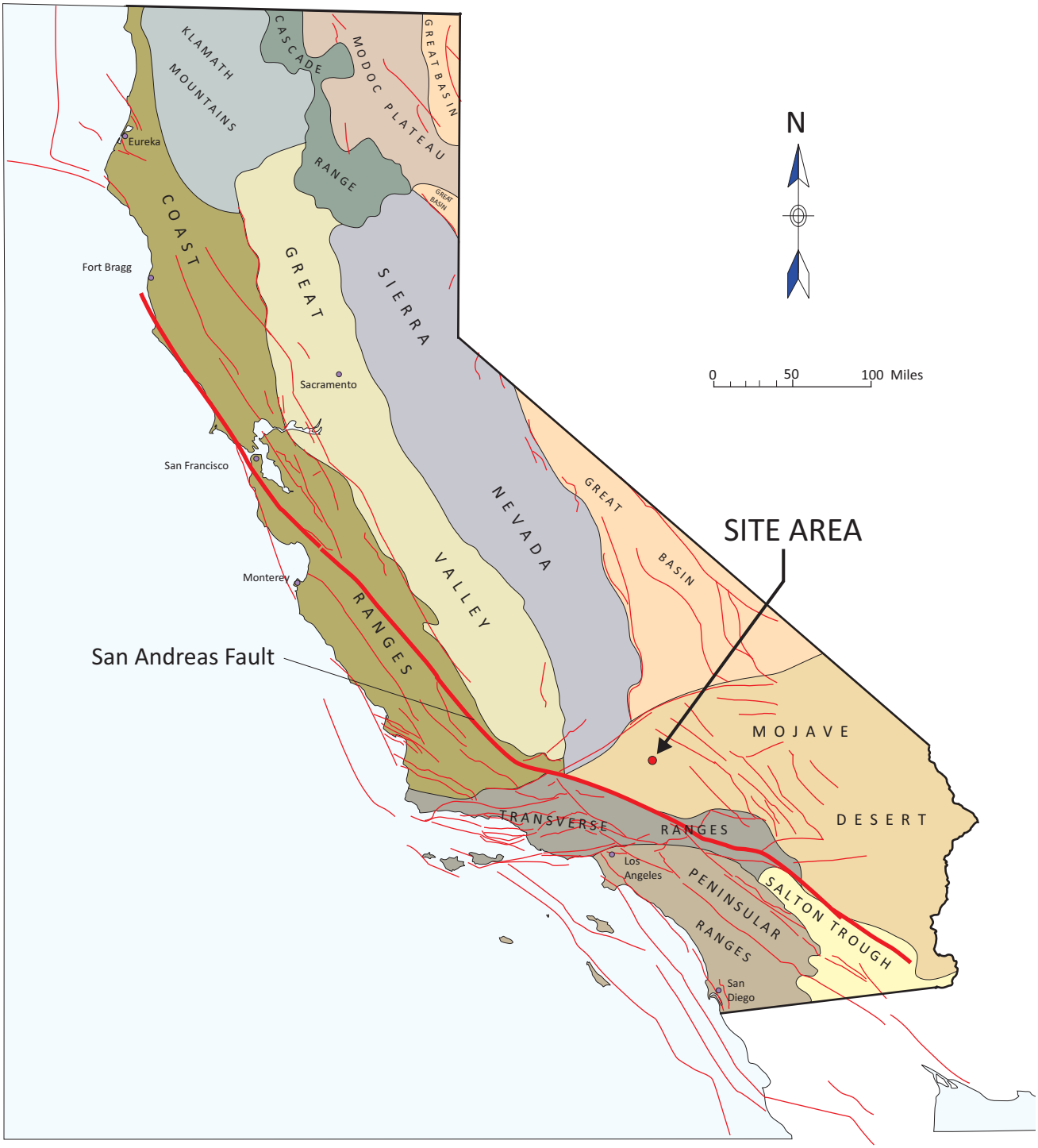

Map showing geomorphic provinces of California and major active and potentially active faults. Fault locations are based on Jennings (1994) and Blake (2000).

Plate 3
State of California Geomorphic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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LEGEND

Qs	Dune sand	Qc	Oligocene nonmarine
Qal	Alluvium	Ec	Eocene nonmarine
Ql	Lake deposits	E	Eocene marine
Qg	Glacial deposits	Ep	Paleocene marine
Qt	River terrace deposits	Tc	Tertiary nonmarine
Qm	Pleistocene marine and marine terrace deposits	Tm	Tertiary marine
Qpv	Pleistocene volcanic rocks Qpv^r-rhyolite Qpv^a-andesite Qpv^b-basalt Qpv^p-pyroclastic rocks	Ti	Tertiary intrusive (hypabyssal) rocks Ti^r-rhyolite Ti^a-andesite Ti^b-basalt
Qc	Pleistocene nonmarine	Tl	Tertiary lake deposits
QP	Plio-Pleistocene nonmarine	Tv	Tertiary volcanic rocks Tv^r-rhyolite Tv^a-andesite Tv^b-basalt Tv^p-pyroclastic rocks
	Quaternary and/or Pliocene cinder cones	Ku	Upper Cretaceous marine
Pc	Undivided Pliocene nonmarine	Ju	Upper Jurasssic marine
Pu	Upper Pliocene marine	gr	Mesozoic granitic rocks
Pmlc	Middle and/or lower Pliocene nonmarine	bi	Mesozoic basic intrusive rocks
Pml	Middle and/or lower Pliocene marine	ub	Mesozoic ultrabasic intrusive rocks
Pv	Pliocene volcanic rocks Pv^r-rhyolite Pv^a-andesite Pv^b-basalt Pv^p-pyroclastic rocks	Jtv	Jurassic-Triassic metavolcanic rocks
Mc	Undivided Miocene nonmarine	ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
Muc	Upper Miocene nonmarine	ms	Pre-Cretaceous metasedimentary rocks
Mu	Upper Miocene marine	mv	Pre-Cretaceous metavolcanic rocks
Mmc	Middle Miocene marine	gr-m	Pre-Cenozoic granitic and metamorphic rocks
Ml	Lower Miocene marine	pCc	Precambrian igneous and metamorphic rock complex
Mv	Miocene volcanic rocks Mv^r-rhyolite Mv^a-andesite Mv^b-basalt Mv^p-pyroclastic rocks	pC	Undivided Precambrian metamorphic rocks pCg=gneiss pCs=schist pCls=limestone and/or dolomite
		pCgr	Undivided Precambrian granitic rocks

– Fault: Dashed where approximate, dotted where concealed

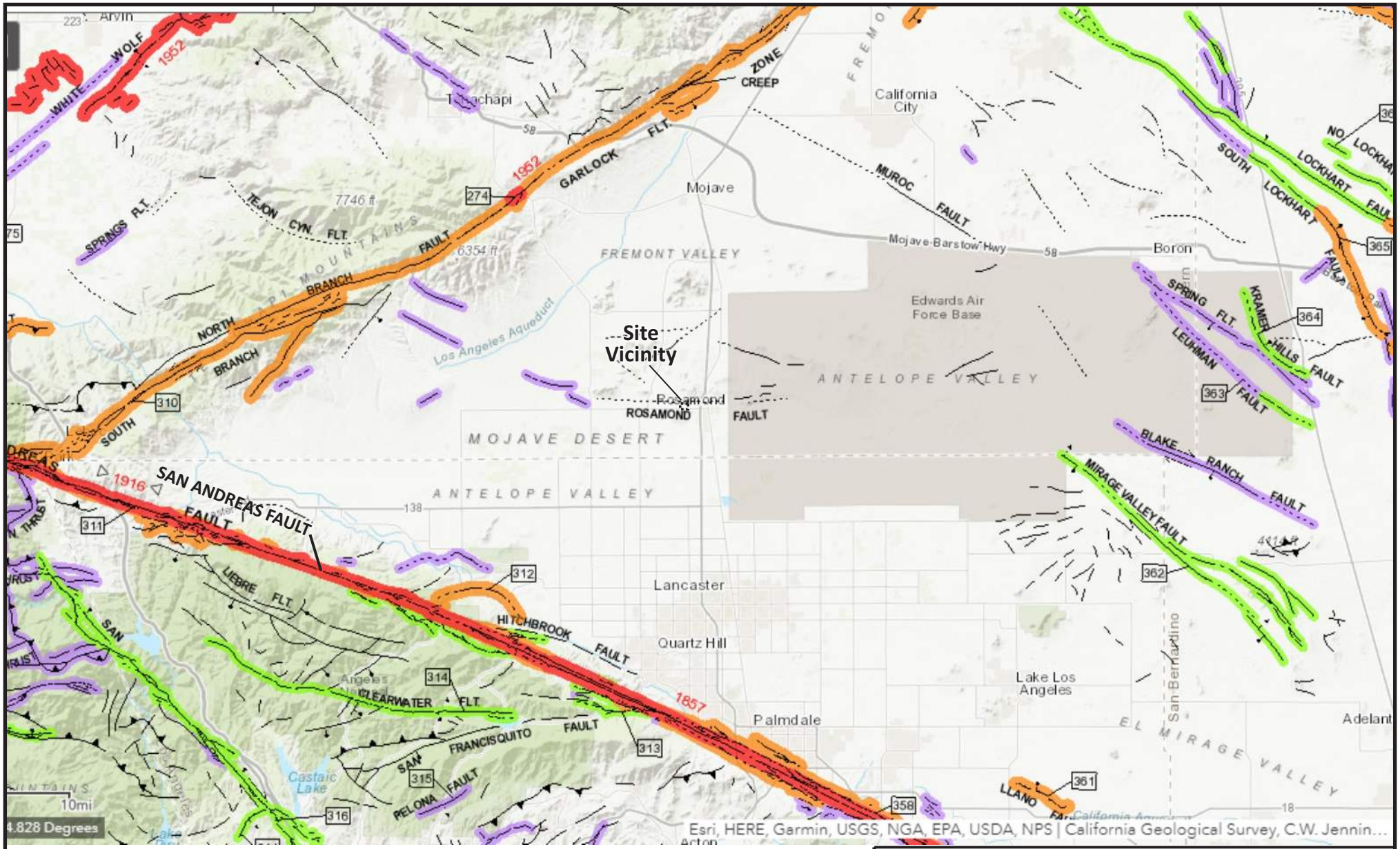
Source: USGS Geologic Map of California, Los Angeles sheet, dated 1969

Plate 4
Regional Geologic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022	File.: 301430-002



Source: CGS Data Map Series, Fault Activity Map of California.

LEGEND

- Fault along which historic (last 200 years) displacement has occurred.
- Fault along which Holocene (last 11,700 years) displacement, has occurred.
- Fault along which Late Quaternary (past 700,000 years) displacement has occurred.
- Fault along which Quaternary (past 1.6 million years) displacement has occurred.

N

Plate 5
Regional Fault Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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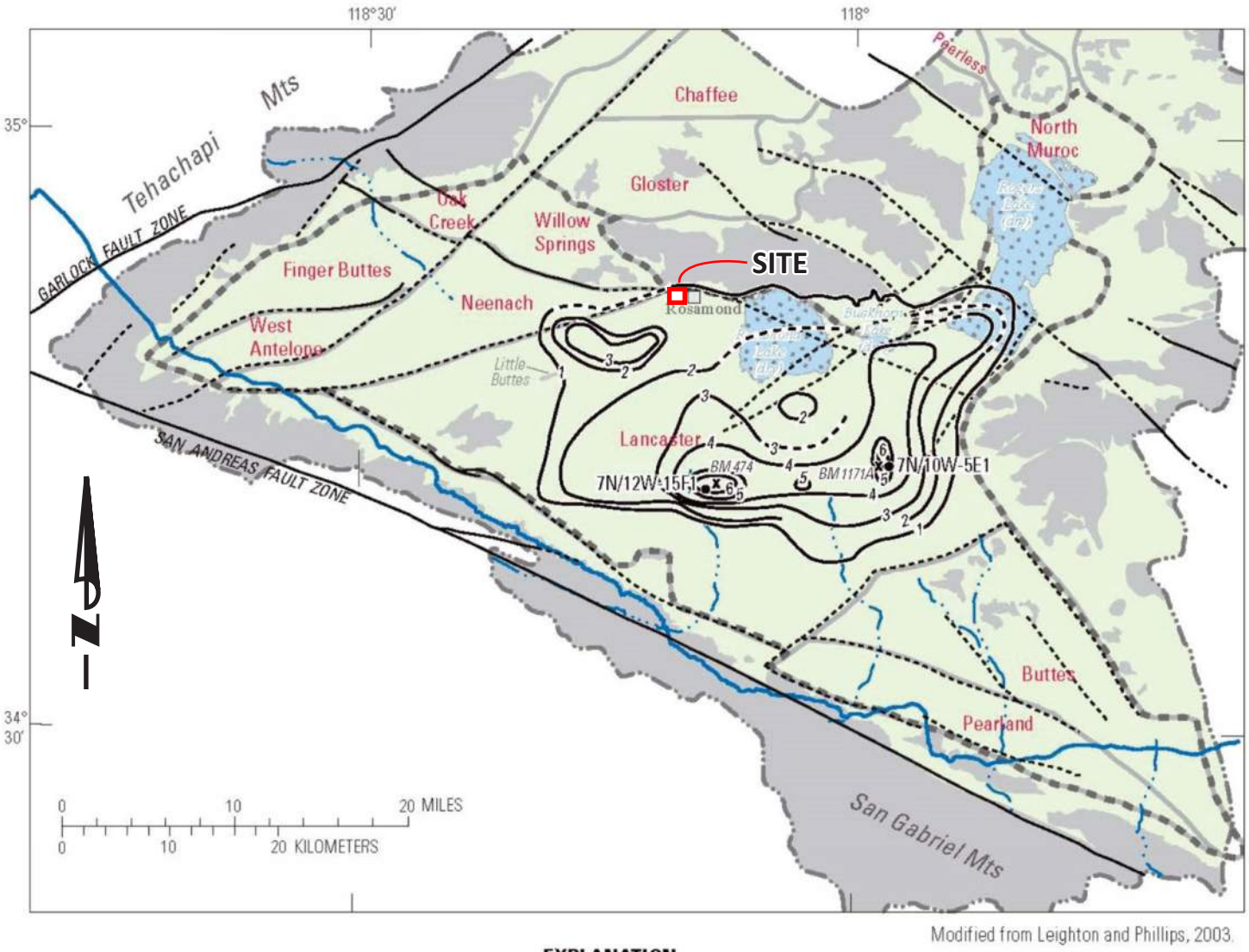


Modified from Leighton and Phillips, 2003.

EXPLANATION

Geologic type

- Alluvial deposits
- Bedrock
- Playa surface

- **Line of equal measured subsidence, in feet**—Dashed where approximately located. Interval 1 foot (modified from Ikehara and Phillips, 1994)
- **Fault**—Dashed where inferred (modified from Dibblee, 1963; Ward and others, 1993; Nishikawa and others, 2001)
- **Antelope Valley drainage basin**
- **Antelope Valley groundwater basin**
- **Groundwater subbasin**
- **California Aqueduct**
- **Stream channel**

7N/12W-15F1 ● **Well and State well number**

BM 474 x **Bench mark and number**

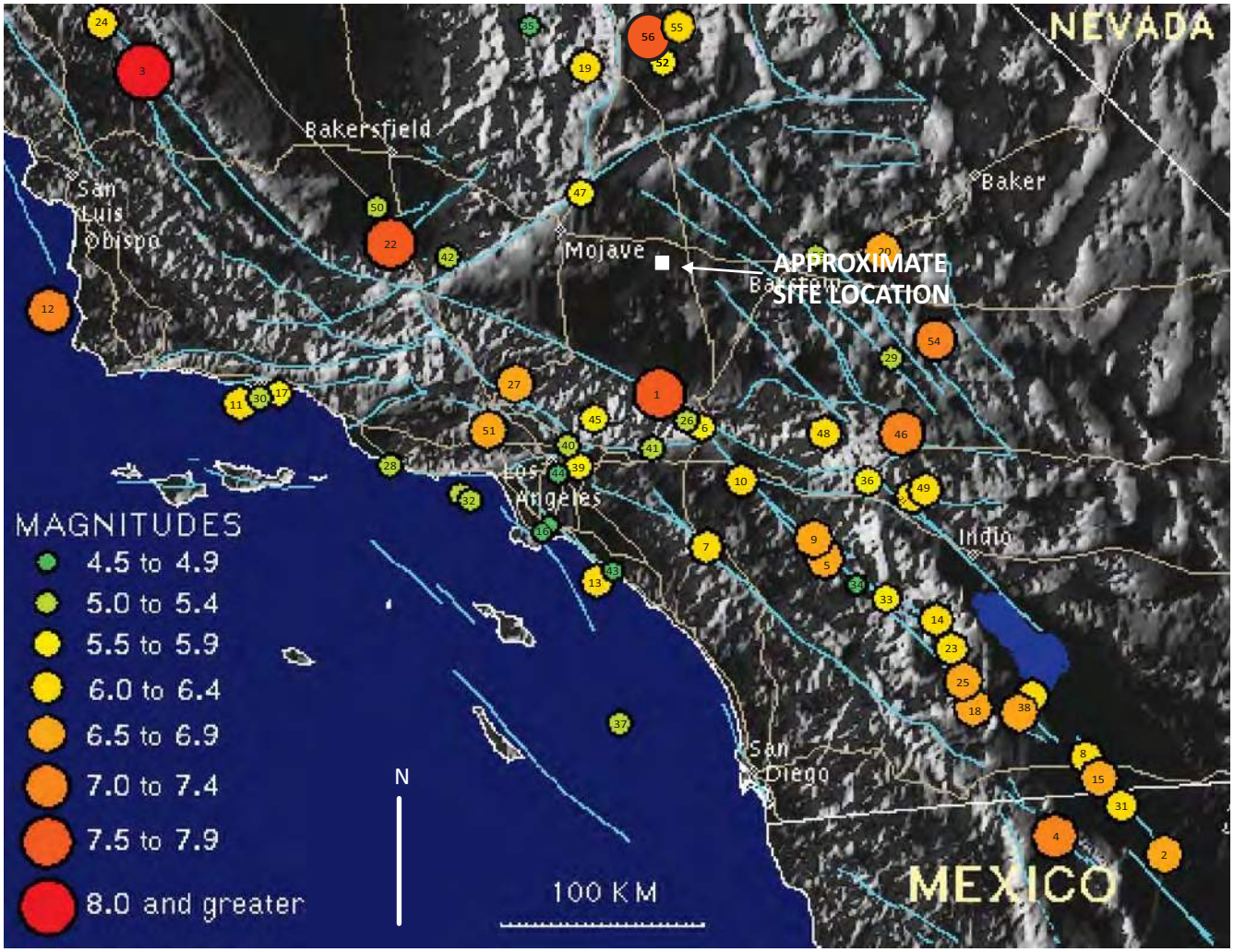
Source: USGS, 2014

Plate 6
Regional Subsidence Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File.: 301430-002



HISTORIC EARTHQUAKES AND EPICENTERS

1. 1812, WRIGHTWOOD
2. 1852, VOLCANO LAKE
3. 1857, FORT TEJON
4. 1892, LAGUNA SALADA
5. 1899, SAN JACINTO
6. 1899, CAJON PASS
7. 1910, ELSINORE
8. 1915, IMPERIAL VALLEY
9. 1918, SAN JACINTO
10. 1923, NORTH SAN JACINTO
11. 1925, SANTA BARBARA
12. 1927, LOMPOC
13. 1933, LONG BEACH
14. 1937, SAN JACINTO
15. 1940, IMPERIAL VALLEY
16. 1941, TORRANCE-GARDENA
17. 1941, SANTA BARBARA
18. 1942, FISH CREEK MOUNTAINS
19. 1946, WALKER PASS
20. 1947, MANIX
21. 1948, DESERT HOT SPRINGS
22. 1952, KERN COUNTY
23. 1954, SAN JACINTO
24. 1966, PARKFIELD
25. 1968, BORREGO MOUNTAINS
26. 1970, LYTLE CREEK
27. 1971, SAN FERNANDO
28. 1973, POINT MAGU
29. 1975, GALWAY LAKE
30. 1978, SANTA BARBARA
31. 1979, IMPERIAL VALLEY
32. 1979, MALIBU
33. 1980, WHITE WASH
34. 1982, ANZA GAP
35. 1983, DURRWOOD MEADOWS SWARM
36. 1986, NORTH PALM SPRINGS
37. 1986, OCEANSIDE
38. 1987, ELMORE RANCH & SUPERSTITION HILLS
39. 1987, WHITTIER NARROWS
40. 1988, PASADENA
41. 1988, UPLAND
42. 1988, TEJON RANCH
43. 1989, NEWPORT BEACH
44. 1989, MONTEBELLO
45. 1991, SIERRA MADRE
46. 1992, LANDERS
47. 1992, MOJAVE
48. 1992, BIG BEAR
49. 1992, JOSHUA TREE
50. 1993, WHEELER RIDGE
51. 1994, NORTHRIDGE
52. 1995, RIDGECREST
53. 1997, CALICO
54. 1999, HECTOR MINE
55. 2019, RIDGECREST
56. 2019, RIDGECREST

SOURCE: SOUTHERN CALIFORNIA EARTHQUAKE CENTER WEB PAGE, 2005

MAP SHOWING LOCATIONS OF SIGNIFICANT HISTORICAL EARTHQUAKES IN SOUTHERN CALIFORNIA FROM 1812 TO 2000

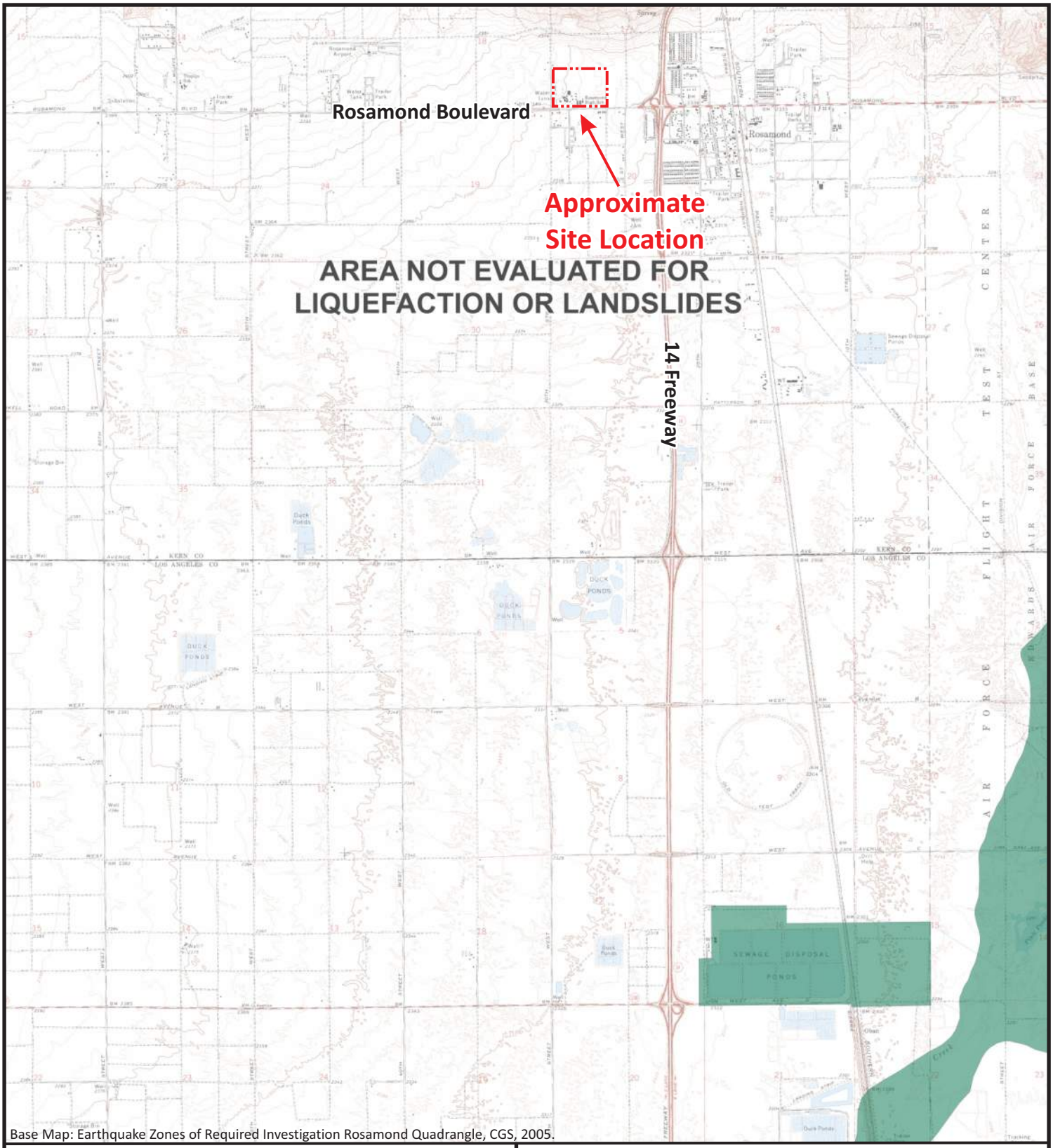
SOURCE: SOUTHERN CALIFORNIA EARTHQUAKE CENTER, WEB PAGE, 2000

Plate 7
Earthquake Epicenter Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002



Base Map: Earthquake Zones of Required Investigation Rosamond Quadrangle, CGS, 2005.

LEGEND

Approximate Site Boundary

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

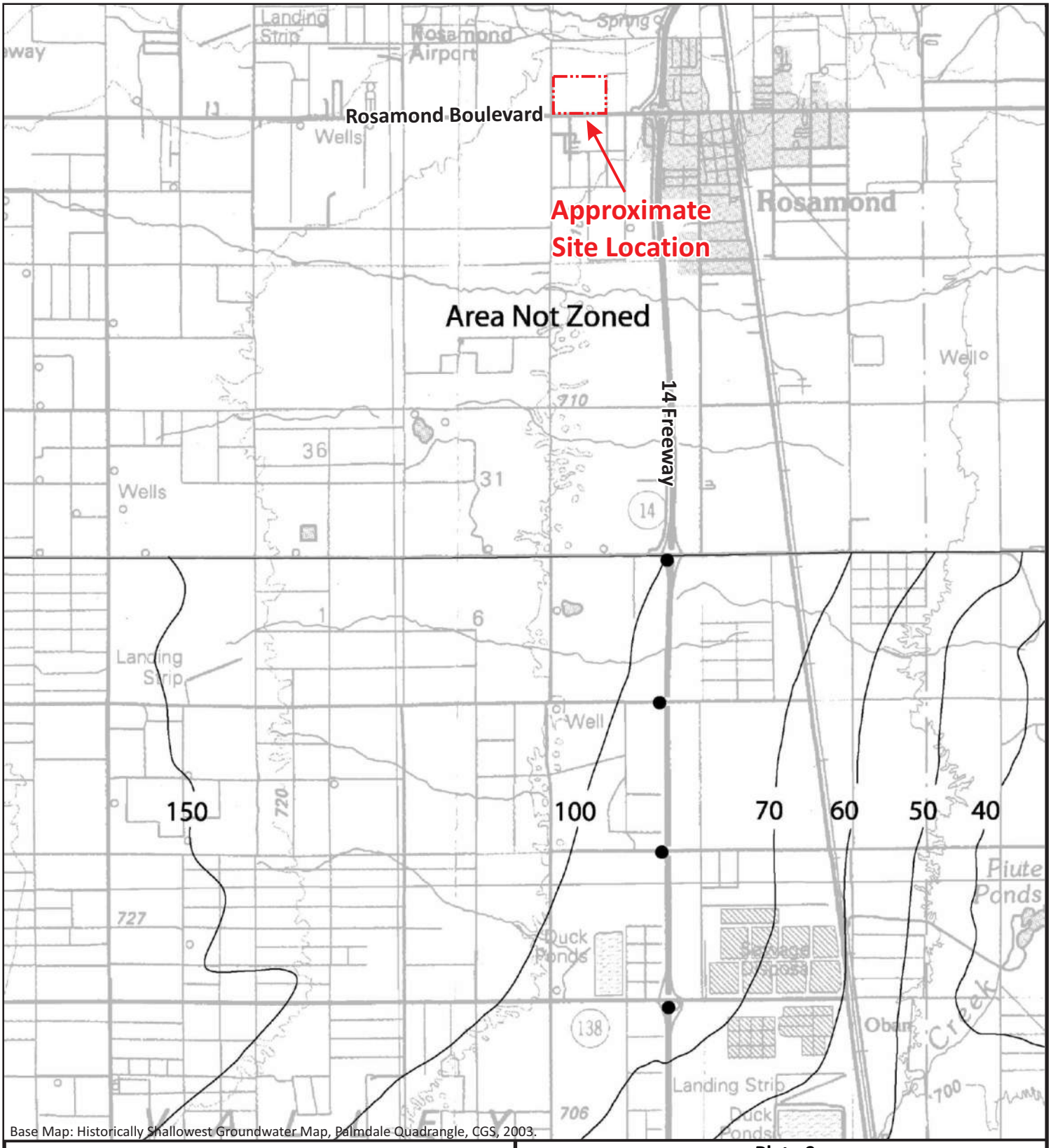
N

Plate 8
Earthquake Hazard Zones

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002



Base Map: Historically Shallowest Groundwater Map, Palmdale Quadrangle, CGS, 2003.

LEGEND

Approximate Site Boundary

0 – 40 Depth to groundwater, in feet

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Plate 9
Historic Groundwater Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002

Fault Parameters

Fault Section Name	Distance (miles)	Distance (km)	Upper Seis. Depth (km)	Lower Seis. Depth (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Garlock (West) FM3.1, 3.2	16.6	26.7	0.0	14.7	90	149	0	98	A	**7.6**	493	5
San Andreas (Mojave S) FM3.1, 3.2	18.3	29.5	0.0	13.1	90	206	180	98	A	**7.7**	102	34
San Andreas (Mojave N) FM3.1, 3.2	21.8	35.1	0.0	15.1	90	199	180	37	A	**7.8**	106	9
Garlock (Central) FM3.1, 3.2	30.2	48.6	0.0	11.5	90	161	0	111	A	**7.6**	523	5
So Sierra Nevada FM3.1, 3.2	30.9	49.8	0.0	13.6	50	92	-90	112	B	**7.5**		0.1
Lenwood-Lockhart-Old Woman Springs FM3.1, 3	33.3	53.7	0.0	13.2	90	43	180	145	B	**7.5**		0.9
San Gabriel FM3.1, 3.2	35.7	57.5	0.0	14.7	61	39	180	71	B	**7.3**		1
White Wolf FM3.1, 3.2	36.4	58.6	0.0	14.6	75	141	60	63	B	**7.1**		2
Holser, alt 2 FM3.2	37.1	59.6	0.0	18.5	58	182	90	17	B'	**6.7**		
Helendale-So Lockhart FM3.1, 3.2	37.1	59.8	0.0	12.8	90	51	180	114	B	**7.4**		0.6
Holser, alt 1, FM 3.1	37.3	60.0	0.0	18.6	58	187	90	20	B	**6.7**		0.4
San Gabriel (Extension) FM3.1, 3.2	38.3	61.6	0.0	14.7	61	6	180	62	B'	**7.2**		
Pine Mtn FM3.1, 3.2	38.4	61.7	0.0	16.3	45	5	na	62	B'	**7.3**		
Santa Susana, alt 2 FM3.2	39.1	62.9	0.0	10.6	53	10	90	43	B'	**6.8**		
White Wolf (Extension) FM3.1, 3.2	39.3	63.2	0.0	14.6	75	136	60	46	B'	**6.9**		
Northridge FM3.1, 3.2	39.6	63.7	7.4	16.8	35	201	90	33	B	**6.8**		1.5
Pleito FM3.1, 3.2	39.7	63.9	0.0	13.6	46	181	90	44	B	**7.1**		2
San Andreas (Big Bend) FM3.1, 3.2	40.3	64.8	0.0	15.1	90	198	180	50	A	**6.8**	89	3.5
Santa Susana East (connector) FM3.1, 3.2	40.3	64.8	0.0	16.3	55	na	na	8	B'	**6.3**		
Breckenridge 2011 FM3.1, 3.2	40.9	65.9	0.0	14.6	60	na	na	11	B'	**6.4**		
Sierra Madre (San Fernando) FM3.1, 3.2	41.1	66.1	0.0	13.0	45	9	90	18	B	**6.6**		2
Sierra Madre FM3.1, 3.2	41.3	66.4	0.0	14.2	53	19	90	57	B	**7.2**		2
Santa Susana, alt 1, FM3.1	41.4	66.7	0.0	16.3	55	9	90	27	B	**6.8**		5
Del Valle FM3.1, 3.2	41.6	67.0	0.0	18.8	73	195	90	9	B'	**6.3**		
Mission Hills FM3.1, 3.2	43.5	69.9	0.0	14.5	55	na	na	13	B'	**6.5**		
Verdugo FM3.1, 3.2	43.9	70.6	0.0	14.5	55	31	90	32	B	**6.8**		0.5
San Cayetano FM3.1, 3.2	44.3	71.3	0.0	16.0	42	3	90	42	B	**7.2**		6
Lake Isabella (Seismicity) FM3.1, 3.2	44.6	71.7	0.0	15.2	90	100	na	60	B'	**7.1**		
Oak Ridge (Onshore) FM3.1, 3.2	44.6	71.7	1.0	19.4	65	159	90	50	B	**7.2**		4
Santa Ynez (East) FM3.1, 3.2	45.3	72.9	0.0	13.3	70	172	0	68	B	**7.2**		2
Kern Canyon (Lake Isabella) FM3.1, 3.2	45.6	73.4	0.0	14.6	60	na	na	35	B'	**6.9**		
Northridge Hills FM3.1, 3.2	46.6	75.0	0.0	14.9	31	19	90	25	B'	**7.0**		
Big Pine (East) FM3.1, 3.2	46.7	75.2	0.0	14.3	73	338	na	23	B'	**6.6**		
Clamshell-Sawpit FM3.1, 3.2	47.0	75.7	0.0	14.0	50	334	90	16	B	**6.6**		0.5
Simi-Santa Rosa FM3.1, 3.2	48.6	78.1	1.0	12.1	60	346	30	39	B	**6.8**		1
Gravel Hills-Harper Lk FM3.1, 3.2	49.2	79.2	0.0	11.4	90	41	180	65	B	**7.0**		0.7
Raymond FM3.1, 3.2	49.8	80.1	0.0	15.6	79	348	60	22	B	**6.7**		1.5
Hollywood FM3.1, 3.2	51.4	82.8	0.0	17.3	70	346	30	19	B	**6.6**		1
San Andreas (San Bernardino N) FM3.1, 3.2	52.3	84.2	0.0	12.8	90	212	180	35	A	**7.5**	103	27
Elysian Park (Upper) FM3.1, 3.2	52.4	84.4	3.0	15.0	50	15	90	20	B	**6.6**		1.3

Reference: USGS OFR 2013-1165 (CGS SP 228)

Based on Site Coordinates of 34.8661 Latitude, -118.1831 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2008-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

34.8661 -118.1831 Lat/Long

Site Specific Spectral Response Values
Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 year MCE Spectrum (1) 2475-year (ASCE 21.2.1)	Max Rotated Probab. 2% in 50 year MCEr Spectrum (2) 2475-year (ASCE 21.2.1.1)	Max 84th Percentile Determ. MCE Spectrum (3) (ASCE 21.2.2)	Determ. Lower Limit MCE Spectrum (4) (3) * 1.39=Scaling (ASCE 21.2.2)	Determ. MCE Spectrum (5) Max (3),(4) (ASCE 21.2.2)	Site Specific MCE, Ground Response (SaM) (6) Min (2),(5) (ASCE 21.2.3)	Site Specific MCE Spectrum Comparator (6b) Max (6),1.5*(8) (ASCE 21.2.3)	2019 CBC MCE Spectrum (7)	Site Specific Design Spectrum (Sa) (8) (ASCE 21.3)	2019 CBC Design Spectrum (9) 2/3*(7)
0.00	0.592	0.594	0.433	0.603	0.603	0.594	0.594	0.466	0.396	0.310
0.05	0.798	0.801	0.480	0.669	0.669	0.669	0.669	0.653	0.446	0.435
0.10	1.004	1.007	0.688	0.958	0.958	0.958	0.958	0.840	0.638	0.560
0.15	1.185	1.189	0.862	1.201	1.201	1.189	1.189	1.026	0.793	0.684
0.20	1.366	1.370	0.981	1.366	1.366	1.366	1.366	1.164	0.911	0.776
0.30	1.528	1.565	1.077	1.500	1.500	1.500	1.500	1.164	1.000	0.776
0.40	1.497	1.531	1.062	1.480	1.480	1.480	1.480	1.164	0.986	0.776
0.50	1.465	1.564	1.012	1.410	1.410	1.410	1.410	1.164	0.940	0.776
0.75	1.221	1.298	0.816	1.137	1.137	1.137	1.137	1.164	0.758	0.776
1.00	0.976	1.145	0.681	0.949	0.949	0.949	0.949	1.088	0.633	0.725
1.50	0.761	0.893	0.503	0.700	0.700	0.700	0.700	0.725	0.467	0.483
2.00	0.547	0.666	0.390	0.544	0.544	0.544	0.544	0.544	0.362	0.363
3.00	0.377	0.476	0.265	0.368	0.368	0.368	0.368	0.363	0.246	0.242
4.00	0.285	0.373	0.195	0.271	0.271	0.271	0.271	0.272	0.181	0.181
5.00	0.222	0.300	0.143	0.200	0.200	0.200	0.200	0.218	0.133	0.145
8.00	0.159	0.215	0.086	0.120	0.120	0.120	0.120	0.136	0.080	0.091
10.00	0.150	0.204	0.048	0.066	0.066	0.066	0.087	0.087	0.058	0.058

C_{RS}:	0.912	
C_{R1}:	0.902	
Site Specific To:	0.164	$= 0.2*S_{D1}/S_{DS}$
Site Specific Ts:	0.819	$= S_{D1}/S_{DS}$

The value of Fa used in Column (3) is defined within ASCE 21.2.2 Supplement 1. This Fa value only applies within Column (3).

Site Coefficients	
F_{PGA}	1.11
F_a	1.06
F_v	2.50

Probabilistic spectrum from 2014 USGS Ground Motion Mapping Program adjusted for site conditions and scaled to represent maximum response in a horizontal plane, in accordance with ASCE 7-16 Section 21.2

Mapped MCE Acceleration Values		
PGA	0.488	g
S_S	1.097	g
S_1	0.435	g

Risk Coefficients have been applied to Column (2); If Method 1 was utilized the Risk Coefficients, CRS and CR1 are presented above, if Method 2 was utilized the Risk Coefficients were obtained from the USGS Risk Targeted Ground Motion Calculator (https://earthquake.usgs.gov/designmaps/rtgm).

Seismic Site Class	**D**
Risk Category	**III**

Reference: ASCE 7-16, Chapters 21.2, 21.3, 21.4, 21.5, 11.4, and 11.8

Calculation Utilized ASCE7-16, Section 21.2.1.1 - Method 1

Site-Specific Design Acceleration Values		
PGA_M	**0.550**	**g**
S_{DS}	**0.900**	**g**
S_{D1}	**0.737**	**g**

Short-Period Seismic Design Category:	1-Second Period Seismic Design Category:
D	D

Site-Specific MCE_R, 5% damped, Spectral Response Acceleration Parameter		
S_{MS}	**1.350**	**g**
S_{M1}	**1.105**	**g**

Vertical Coefficient (C_V)
1.32

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2
PSV (ft/sec) = $32.2(S_a)T/(2p)$

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake

General Procedure Seismic Design Values

2019 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

			CBC Reference	ASCE 7-16 Reference
Seismic Design Category		**D**		
Site Class		**D**	Table 1613.5.6	Table 11.6-1
Latitude:		34.866	Table 1613.5.2	Table 20.3-1
Longitude:		-118.183		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**1.097 g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.435 g**	Figure 1613.5	Figure 22-2
Site Coefficient	F_a	1.06	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.87	Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	1.164 g	$= F_a * S_S$	
	S_{M1}	0.811 g	$= F_v * S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**0.776 g**	$= 2/3 * S_{MS}$	
1 second Spectral Response	S_{D1}	**0.541 g**	$= 2/3 * S_{M1}$	

Site Specific Evaluation May Be Required Due to Site Class = D or E and S1>=0.2. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

To	0.14	sec	$= 0.2 * S_{D1}/S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.70	sec	$= S_{D1}/S_{DS}$
Risk Category	III		Table 1604.5
Seismic Importance Factor	1.25		
F_{PGA}	1.11		
PGA_M	**0.54**		
Vertical Coefficient (C_V)	1.32		Table 11.9-1

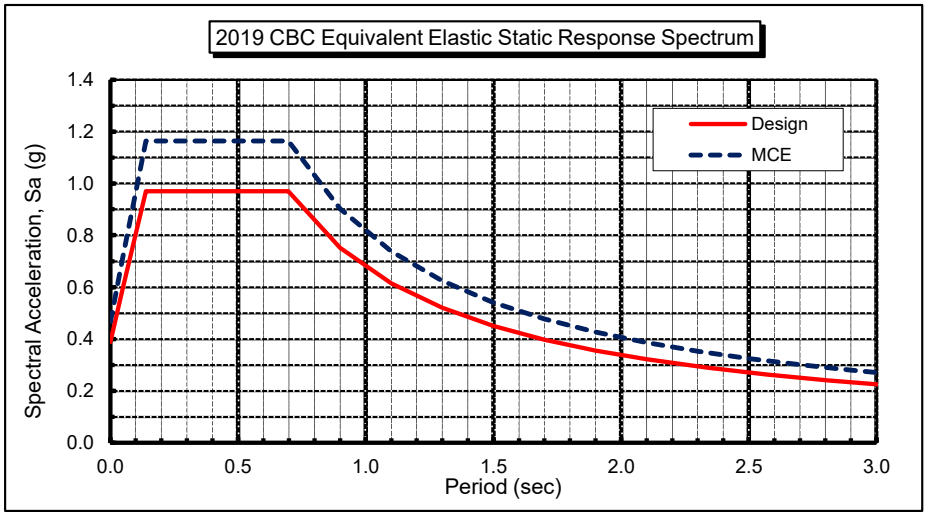


Table 11.5-1 Design

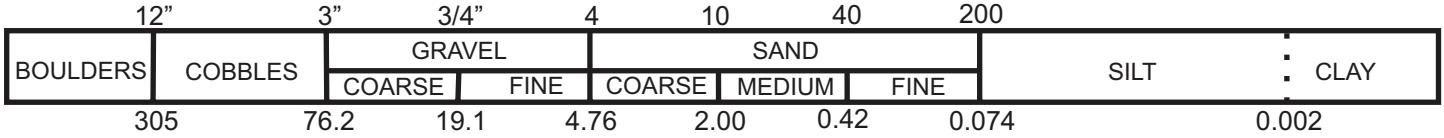
Period T (sec)	Sa (g)
0.00	0.388
0.05	0.597
0.14	0.970
0.70	0.970
0.90	0.751
1.10	0.615
1.30	0.520
1.50	0.451
1.70	0.398
1.90	0.356
2.10	0.322
2.30	0.294
2.50	0.270
2.70	0.250
2.90	0.233
3.10	0.218

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on ASTM Designations D 2487 and D 2488 (Unified Soil Classification System). Information on each boring log is a compilation of subsurface conditions obtained from the field as well as from laboratory testing of selected samples. The indicated boundaries between strata on the boring logs are approximate only and may be transitional.

SOIL GRAIN SIZE

U.S. STANDARD SIEVE



SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND NON-PLASTIC SILTS)

Very Loose	*N=0-4	RD=0-30	Easily push a 1/2-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch reinforcing rod with hammer
Dense	N=31-50	RD=70-90	Drive a 1/2-inch reinforcing rod 1 foot with difficulty by a hammer
Very Dense	N>50	RD=90-100	Drive a 1/2-inch reinforcing rod a few inches with hammer

*N=Blows per foot in the Standard Penetration Test at 60% theoretical energy. For the 3-inch diameter Modified California sampler,140-pound weight, multiply the blow count by 0.63 (about 2/3) to estimate N. If automatic hammer is used, multiply a factor of 1.3 to 1.5 to estimate N. RD=Relative Density (%). C=Undrained shear strength (cohesion).

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

Very Soft	*N=0-1	*C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000	Dented slightly by a pencil point or thumbnail

MOISTURE DENSITY

Moisture Condition: An observational term; dry, damp, moist, wet, saturated.
Moisture Content: The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.
Dry Density: The pounds of dry soil in a cubic foot.

MOISTURE CONDITION

Dry.....Absence of moisture, dusty, dry to the touch
Damp.....Slight indication of moisture
Moist.....Color change with short period of air exposure (granular soil)
Below optimum moisture content (cohesive soil)
Wet.....High degree of saturation by visual and touch (granular soil)
Above optimum moisture content (cohesive soil)
Saturated.....Free surface water

PLASTICITY

DESCRIPTION	FIELD TEST
Nonplastic	A 1/8 in. (3-mm) thread cannot be rolled at any moisture content.
Low	The thread can barely be rolled.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit.
High	The thread can be rerolled several times after reaching the plastic limit.

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

RELATIVE PROPORTIONS

Trace.....minor amount (<5%)
with/some.....significant amount
modifier/and...sufficient amount to influence material behavior (Typically >30%)

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

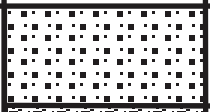
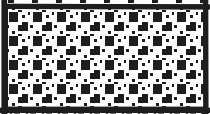
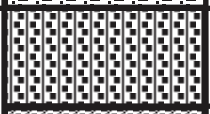
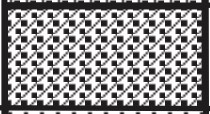
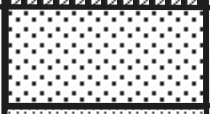
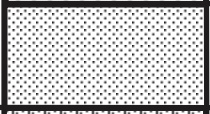
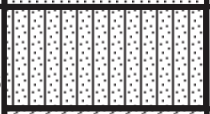
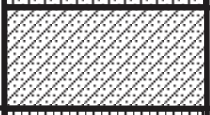
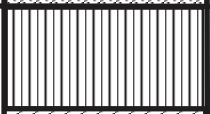
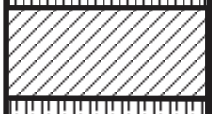
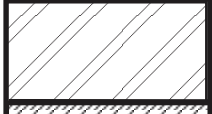
Standard Penetration Split Spoon Sampler (2” outside diameter)

Modified California Sampler (3” outside diameter)

No Recovery

Terms and Symbols Used on Boring Logs

Earth Systems

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is larger than No. 200 sieve size	GRAVEL AND GRAVELLY SOILS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures. Little or no fines
		GRAVELS WITH FINES		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
	SAND AND SANDY SOILS More than 50% of coarse fraction passing No. 4 sieve	CLEAN SAND (Little or no fines)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
		SAND WITH FINES (appreciable amount of fines)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
		LIQUID LIMIT GREATER THAN 50		MH	Inorganic silty, micaceous, or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents
VARIOUS SOILS AND MAN MADE MATERIALS					Fill Materials
MAN MADE MATERIALS					Asphalt and concrete

Soil Classification System

Earth Systems

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
			■	8,13,19			143.1	6.1	SILTY SAND with GRAVEL, brown, damp, medium dense, fine- to coarse-grained sand
5			■	12,14,20		SM	145.6	8.7	SILTY SAND, brown, damp, medium dense, fine- to medium-grained sand
			■	14,18,26			148.3	9.1	SILTY SAND, gray-brown, moist, dense, fine- to medium-grained sand, cementation observed, caliche
10			■	14,25,47			124.9	11.3	very dense, caliche
15			■	7,11,10		SP-SM	105.7	1.9	POORLY GRADED SAND with SILT, light gray, damp, medium dense, fine- to coarse-grained sand
20		■		6,9,9		SM		7.5	SILTY SAND, brown, moist, medium dense, fine- to coarse-grained sand
25			■	20,19,16		SP	110.1	5.5	POORLY GRADED SAND, brown, gray-brown, moist, medium dense, medium- to coarse-grained sand
30		■		5,7,11		SM		14.9	SILTY SAND, brown, moist, medium dense, fine-grained sand
35			■	7,14,31		SM	123.0	11.5	SILTY SAND, brown, moist, dense, fine- to coarse-grained sand

Earth Systems

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PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40		■		11,12,16		SM		13.4	SILTY SAND, brown, moist, medium dense, fine- to coarse-grained sand
45			■	19,29,50(5")		SP	117.5	6.7	POORLY GRADED SAND, brown, moist, very dense, fine- to coarse-grained sand
50		■		11,12,12		SP-SM		5.9	POORLY GRADED SAND with SILT, brown, moist, medium dense, fine- to coarse-grained sand
55									Total depth 51.5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational
60									
65									
70									
75									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B2**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
			■	7,8,10		SP	109.2	2.2	POORLY GRADED SAND, light brown, damp, medium dense, fine- to coarse-grained sand
5			■	13,18,25			125.3	6.7	SILTY SAND, light brown, moist, dense, fine- to coarse-grained sand
			■	8,13,21		SM	137.4	10.0	SILTY SAND, gray-brown, damp, medium dense, fine- to medium-grained sand, cementation, caliche
10			■	8,11,19			142.5	11.4	
		■		6,7,12		CL-ML		17.6	SILTY CLAY, gray-brown, moist, very stiff, some fine-grained sand, white mottles, low to moderate plasticity
15			■	8,13,18		SM	115.9	8.9	SILTY SAND, brown, moist, medium dense, fine- to medium-grained sand
20		■		3,4,6		CL-ML		5.7	SILTY CLAY, gray-brown, moist, medium stiff, low plasticity
25			■	7,9,21		SM	118.4	4.5	SILTY SAND, brown, moist, medium dense, fine- to medium-grained sand
30									Total depth 26.5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
35									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**P1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0						SM			**Alluvium (Qa):** SILTY SAND, brown, damp, fine- to medium-grained sand
5									Total depth 5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
10									
15									
20									
25									
30									
35									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**P2**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
									SILTY SAND, brown, damp, fine- to medium-grained sand
5						SM			
10									
15									Total depth 15 feet.
									Groundwater was not encountered.
20									Backfilled with soil cuttings.
									Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
25									
30									
35									

Boring No.	B-1	Project and Number	Rosamond HS Cafeter	301430-002
		ESSW Field Staff	Notes: Soils Remediated t Upper 5 feet	
		Drilling Company	GP Drilling	
		Drilling Method	6-8" H S A	HSA Inner Diameter 3"
		Site Latitude (North)	Decimal Degrees	34.8661
		Site Longitude (West)	Decimal Degrees	-118.1831

Date Drilled
3/26/2022

Hammer Weight (lbs)
140

Hammer Drop (inches)
30

Hammer Efficiency (E_M)
72

Borehole Correction (Cb)*
1

*inside diameter of Hollow Stem Auger

Sampler Correction Mod Cal to SPT
0.63

Sampler Liner Correction (Cs)
1.2 Applied if SPT Sampler Used
1.0 Applied if Cal Sampler Used

Rod Length Above Ground (ft)
3

Depth to Estimate Vs Over (ft)*
100

*Caltrans Estimation Method

$*N_{sub}$ Value Desired For Column 6
70

*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)

Calculation Results
Ave. SPT N_{60HE}-value (blows/ft)
28
(Based on Upper 55 feet)
Ave. Shear Wave Velocity (ft/sec)
879
(Based on Upper 55 feet)
Soil Profile Type (Site Class)
D
(Based on Upper 55 feet)
Ave. Friction Angle (degrees)
34
(Based on Upper 55 feet)
Estimated Shear Wave Velocity ** Based on Depth Less than 100' ft
884 (ft/sec Upper 100 feet)
Soil Profile Type (Site Class)**
D Based on Ave. Shear Wave Velocity (ft/sec) 270 (m/sec Upper 100 feet)
Ave. Field SPT N-value (blows/ft)
23.2
(Based on Upper 55 feet)
Ave. Field SPT N-value (blows/ft)
27.2
(Based on Upper 100 feet)
Soil Profile Type (Site Class)**
D Based on Ave. Field Blow Count 27 (Upper 100 feet)

	Equipment variable	Typical Correction (%/100)
Energy ratio (Skempton, 1986)	Donut Hammer	0.50 to 1.00
	Safety Hammer	0.70 to 1.20
	Automatic-Trip Donut-type Hammer	0.80 to 1.30

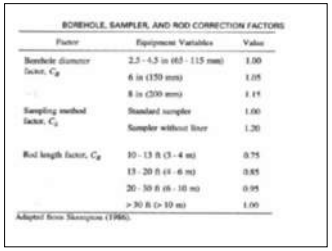
→ Hammer energy as related to the standard 60% delivered energy, i.e. a 72% hammer has and energy ratio of 1.2, i.e. (72/60=1.2)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	Blow Count***	Type of Sampler	d_i (feet)	N_{60} (blows/ft)	N70 (blows/ft)	N_{60HE} (blows/ft)	V_{si}** (m/sec)	V_{si} (ft/sec)	Φ_i (degrees)	d_i/N_{60HEi}	d_i/V_{si}	d_i/Φ_i	Consistency if Coarse Grained (Based on ASTM and Corrected for N60)	Consistency if Fine Grained (Based on ASTM and Corrected for N60)
5.0	32	c	5.0	18.14	15.55	24.19	253.18	830.43	33.20	0.20668	0.00602	0.150624	Medium Dense	Very Stiff
7.5	34	c	2.5	19.28	16.52	25.70	257.67	845.16	33.50	0.09726	0.00296	0.074617	Medium Dense	Very Stiff
10.0	44	c	2.5	24.95	21.38	33.26	277.68	910.78	34.88	0.07516	0.00274	0.071672	Medium Dense	Very Stiff
15.0	72	c	5.0	46.27	39.66	54.43	320.31	1050.60	37.81	0.09186	0.00476	0.132252	Dense	Hard
20.0	21	c	5.0	15.08	12.93	15.88	224.07	734.94	31.19	0.31494	0.00680	0.160325	Medium Dense	Very Stiff
25.0	18		5.0	20.52	17.59	21.60	245.00	803.58	32.63	0.23148	0.00622	0.153228	Medium Dense	Very Stiff
30.0	35	c	5.0	26.46	22.68	26.46	259.85	852.30	33.65	0.18896	0.00587	0.148569	Medium Dense	Very Stiff
35.0	18		5.0	21.60	18.51	21.60	245.00	803.58	32.63	0.23148	0.00622	0.153228	Medium Dense	Very Stiff
40.0	45	c	5.0	34.02	29.16	34.02	279.49	916.73	35.01	0.14697	0.00545	0.142834	Dense	Hard
45.0	28		5.0	33.60	28.80	33.60	278.49	913.44	34.94	0.14881	0.00547	0.143116	Dense	Hard
50.0	100	c	5.0	75.60	64.80	75.60	352.32	1155.61	40.00	0.06614	0.00433	0.125002	Very Dense	Hard
55.0	24		5.0	28.80	24.69	28.80	266.31	873.50	34.10	0.17361	0.00572	0.146631	Medium Dense	Very Stiff
		Total:	55.0	"d" Feet				Total:		1.97335	0.06258	1.602098		

**Used When Boring Depths are less than 100 feet to estimate Shear Wave Velocity over 100 feet. Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009 using N60HE corrected only for Hammer Energy (Empirical Calculation)

*** Uncorrected blowcount not to exceed 100 blows as entry per CBC

Consistency classification based upon ASCE 1996


BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

Spreadsheet Version 2.6, 2019: Prepared by Kevin L. Paul, PE, GE

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Rosamond HS Cafeteria
Job No: 301430-002
Date: 6/12/2022
Boring: B1 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.9 (7.5)
PGA, g: 0.37 (0.42)
MSF: 0.88
GWT: #### feet
Calc GWT: 50.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.30 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

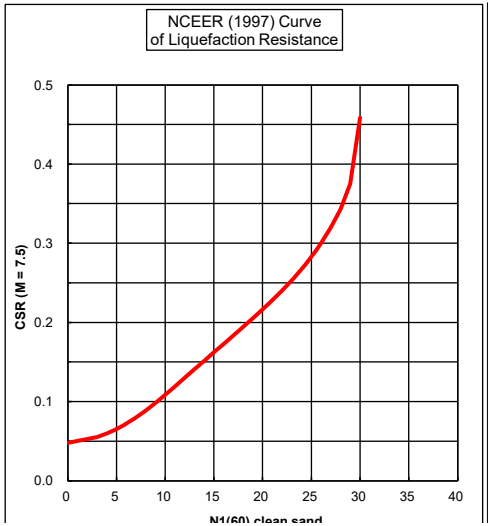
Threshold Acceler., g: #N/A **Minimum Calculated SF:** #N/A **Required SF:** 1.00

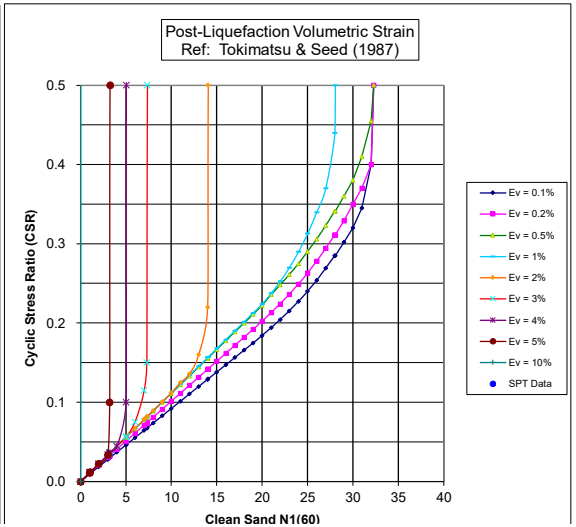
Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
0	0.4

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 19.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
0.0				0				0.000																							
5.0	32	**20**	1	152	10	2.5	5.5	0.190	0.190	1.00	1.70	0.75	1.00	33.4	69	1.6	35.0	1.00	1.200	0.274	**Non-Liq.**	1.6	35.0	0.02	**0.01**	0.127	521	0.045	1.5E-04	7.4E-05	8.3E-05	**0.01**
7.5	34	**21**	1	158	10	5.0	8.0	0.380	0.380	0.99	1.67	0.75	1.00	34.9	71	1.6	36.5	1.00	1.200	0.272	**Non-Liq.**	1.6	36.5	0.02	**0.01**	0.254	748	0.090	1.9E-04	9.2E-05	1.0E-04	**0.01**
10.0	44	28	1	162	10	7.5	10.5	0.577	0.577	0.98	1.35	0.75	1.00	36.6	72	1.7	38.2	1.00	1.200	0.271	**Non-Liq.**	1.7	38.2	0.02	**0.01**	0.387	937	0.137	2.2E-04	1.0E-04	1.1E-04	**0.01**
15.0	72	45	1	139	10	10.0	13.0	0.780	0.780	0.98	1.16	0.76	1.00	52.0	86	2.0	54.0	1.00	1.200	0.269	**Non-Liq.**	2.0	54.0	0.01	**0.01**	0.522	1,221	0.184	2.1E-04	6.4E-05	7.1E-05	**0.01**
20.0	21	13	1	108	10	15.0	18.0	1.127	1.127	0.97	0.97	0.86	1.00	14.4	45	1.2	15.6	0.99	0.168	0.270	**Non-Liq.**	1.2	15.6	0.15	**0.09**	0.755	970	0.263	5.1E-04	6.9E-04	7.7E-04	**0.09**
25.0	0	18	1	113	10	20.0	23.0	1.396	1.396	0.96	0.87	0.93	1.23	23.4	58	1.4	24.7	0.92	0.278	0.286	**Non-Liq.**	1.4	24.7	0.07	**0.04**	0.936	1,260	0.321	4.1E-04	3.2E-04	3.6E-04	**0.04**
30.0	35	22	1	120	10	25.0	28.0	1.679	1.679	0.94	0.79	0.98	1.00	22.4	57	1.4	23.7	0.87	0.263	0.297	**Non-Liq.**	1.4	23.7	0.08	**0.05**	1.125	1,362	0.380	4.5E-04	3.7E-04	4.1E-04	**0.05**
35.0	0	18	1	121	10	30.0	33.0	1.979	1.979	0.92	0.73	1.00	1.21	20.6	54	1.3	21.9	0.83	0.239	0.305	**Non-Liq.**	1.3	21.9	0.10	**0.06**	1.326	1,441	0.438	4.9E-04	4.4E-04	4.9E-04	**0.06**
40.0	45	28	1	137	10	35.0	38.0	2.280	2.280	0.89	0.68	1.00	1.00	25.1	60	1.4	26.5	0.79	0.310	0.308	**Non-Liq.**	1.4	26.5	0.07	**0.04**	1.528	1,648	0.488	4.5E-04	3.2E-04	3.6E-04	**0.04**
45.0	0	28	1	119	10	40.0	43.0	2.623	2.623	0.85	0.64	1.00	1.28	29.5	65	1.5	31.0	0.76	1.200	0.307	**Non-Liq.**	1.5	31.0	0.05	**0.03**	1.757	1,862	0.537	4.2E-04	2.4E-04	2.7E-04	**0.03**
50.0	100	63	1	120	10	45.0	48.0	2.921	2.921	0.80	0.60	1.00	1.00	49.3	84	1.9	51.2	0.67	1.200	0.332	**Non-Liq.**	1.9	51.2	0.02	**0.01**	1.957	2,322	0.565	3.2E-04	1.0E-04	1.1E-04	**0.01**
55.0	0	24	1	111	10	50.0	53.0	3.221	3.221	0.75	0.57	1.00	1.21	21.7	56	1.3	23.1	0.72	0.254	0.289	**Non-Liq.**	1.3	23.1	0.00	**0.00**	2.158	1,869	0.583	4.5E-04			
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									





$N_{1(60)} = C_N*C_E*C_B*C_R*C_S*N$
C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))
$C_N = (1\ atm/p'o)^{0.5}$, max 1.7
C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners
$MSF = 10^{2.24}/M^{2.56}$
z = Depth (m)
pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po
τ_{av} = 0.65*PGA*po*rd
$G_{max} = 447*N_{1(60)CS}^{(1/3)}*p^{0.5}$
a = 0.0389*(p/1)+0.124
$b = 6400*(p/1)^{(-0.6)}$
$\gamma = [1+a*EXP(b*\tau_{av}/G_{max})]/[(1+a)*\tau_{av}/G_{max}]$
$E_{15} = \gamma*(N_{1(60)CS}/20)^{-1.2}$
$E_{nc} = (Nc/15)^{0.45}*E15$

$Nc = (MAG-4)^{2.17}$
S = 2*H*E_{nc}

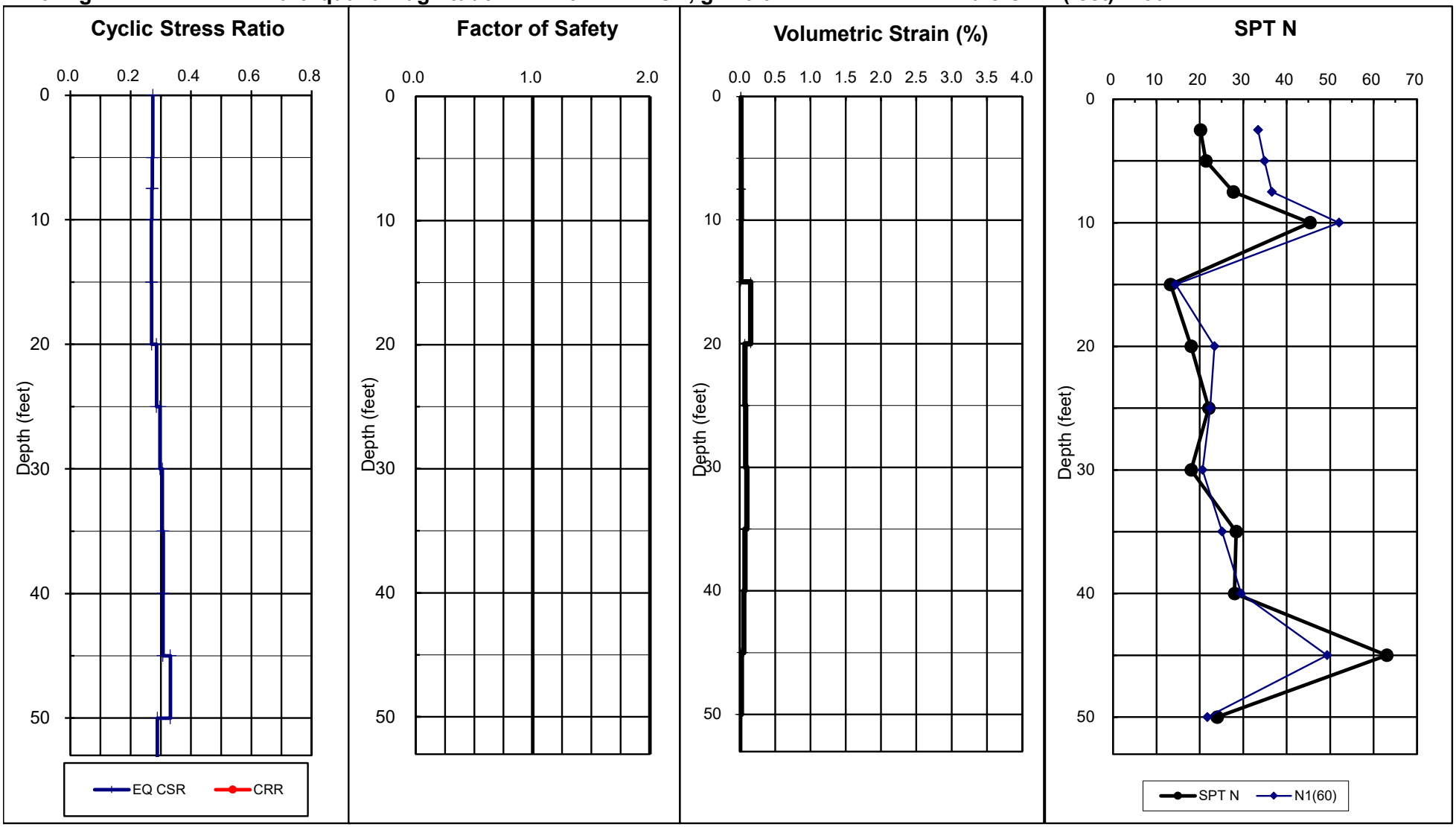
rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))
$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)
$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$
Kσ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$
$Dr = (N_{1(60)}/70)^{0.5}$
CSReq = 0.65*PGA*(po/p'o)*rd
CSR* = CSReq/MSF/Kσ
$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))
N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Rosamond HS Cafeteria **Project No: 301430-002** 1996/1998 NCEER Method

Boring: B1 **Earthquake Magnitude: 7.9** **PGA, g: 0.37** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.4 inches

APPENDIX B

Laboratory Test Results

File No.: 301430-002 6/12/2022

Lab No.: 22-186

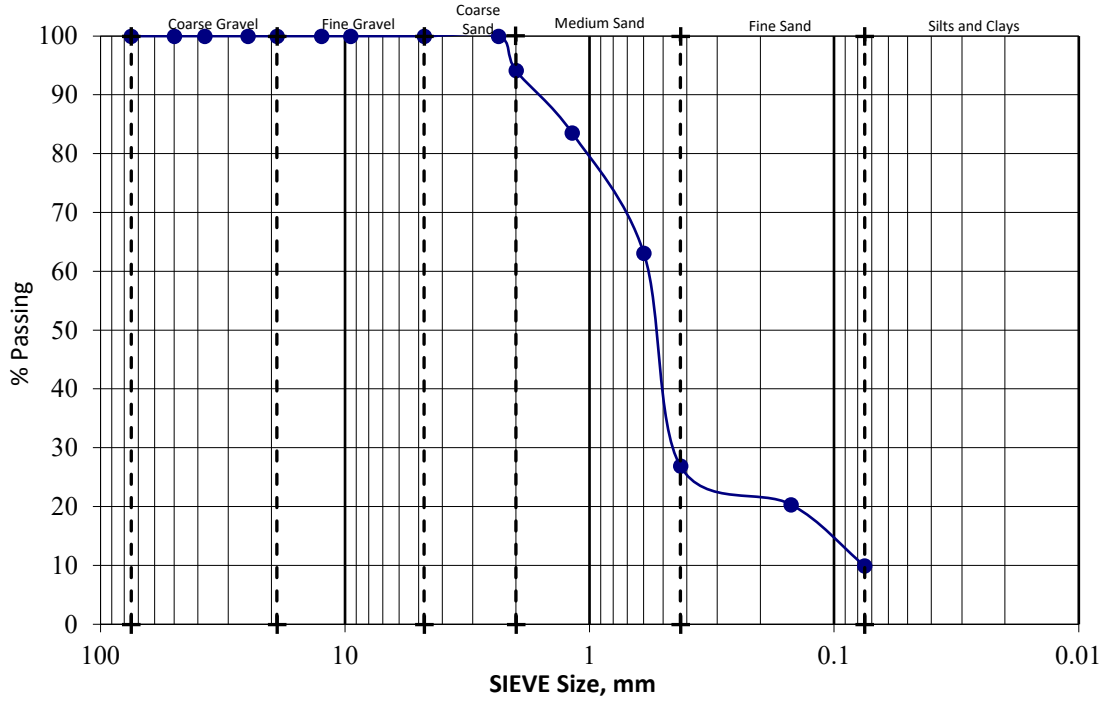
SIEVE ANALYSIS ASTM D6913

Job Name: Rosamond High School Cafeteria

Sample ID: B-1 @ 0-5'

Description: Silty Sand (SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#10	94
#16	84
#30	63
#40	27
#50	100.0
#100	20.3
#200	9.9



% Coarse Gravel:	0	% Coarse Sand:	6	**FM=**	**1.3**	
% Fine Gravel:	0	% Medium Sand:	67	Cu:	2.81	
		% Fine Sand:	17	Cc:	1.67	Gradation
% Total Gravel	0	% Total Sand	90	% Fines:	9.9	Poorly Graded

File No.: 301430-002 May 20, 2022

Job Name: Rosamond High School Cafeteria

Lab Number: 22-186

ASTM D-1140 or Earth Systems Method (circle one)

AMOUNT PASSING NO. 200 SIEVE (Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B-1	30	10.0	SM	10

File No.: 301430-002 May 20, 2022
Lab No.: 22-186

EXPANSION INDEX **ASTM D-4829**

Job Name: Rosamond High School Cafeteria
Sample ID: B-1 @ 0-5'
Soil Description: Silty Sand (SM)

Initial Moisture, %:	12.0	
Initial Compacted Dry Density, pcf:	112.8	
Initial Saturation, %:	66	*
Final Moisture, %:	19.8	
Volumetric Swell, %:	-0.3	
Expansion Index, EI:	**0**	**Very Low**

EI	ASTM Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 2.5'

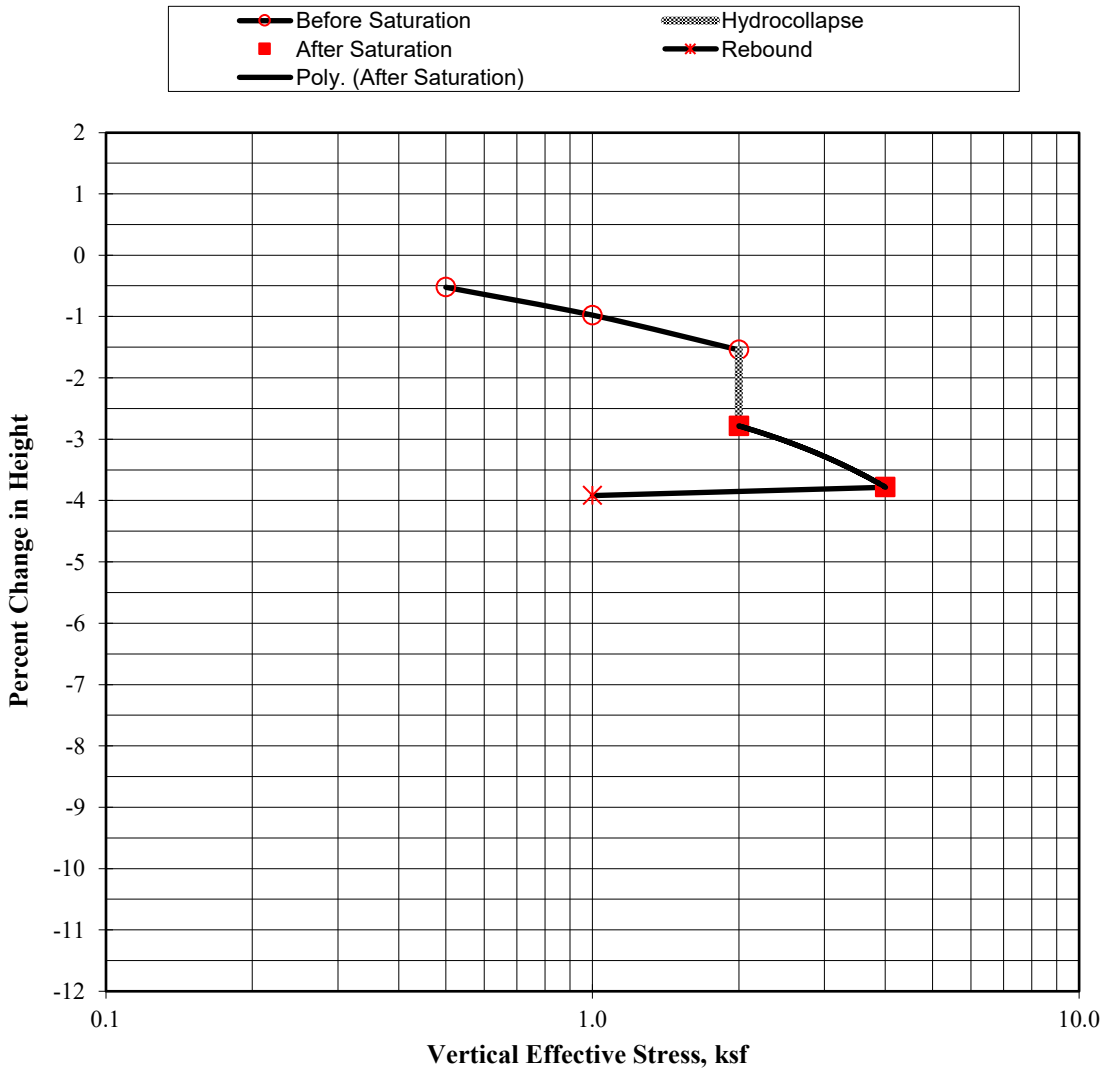
Silty Sand (SM)

Ring Sample

Initial Dry Density: 109.0 pcf
Initial Moisture: 4.6%
Specific Gravity: 2.67
Initial Void Ratio: 0.529

Hydrocollapse: 1.2% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 5'

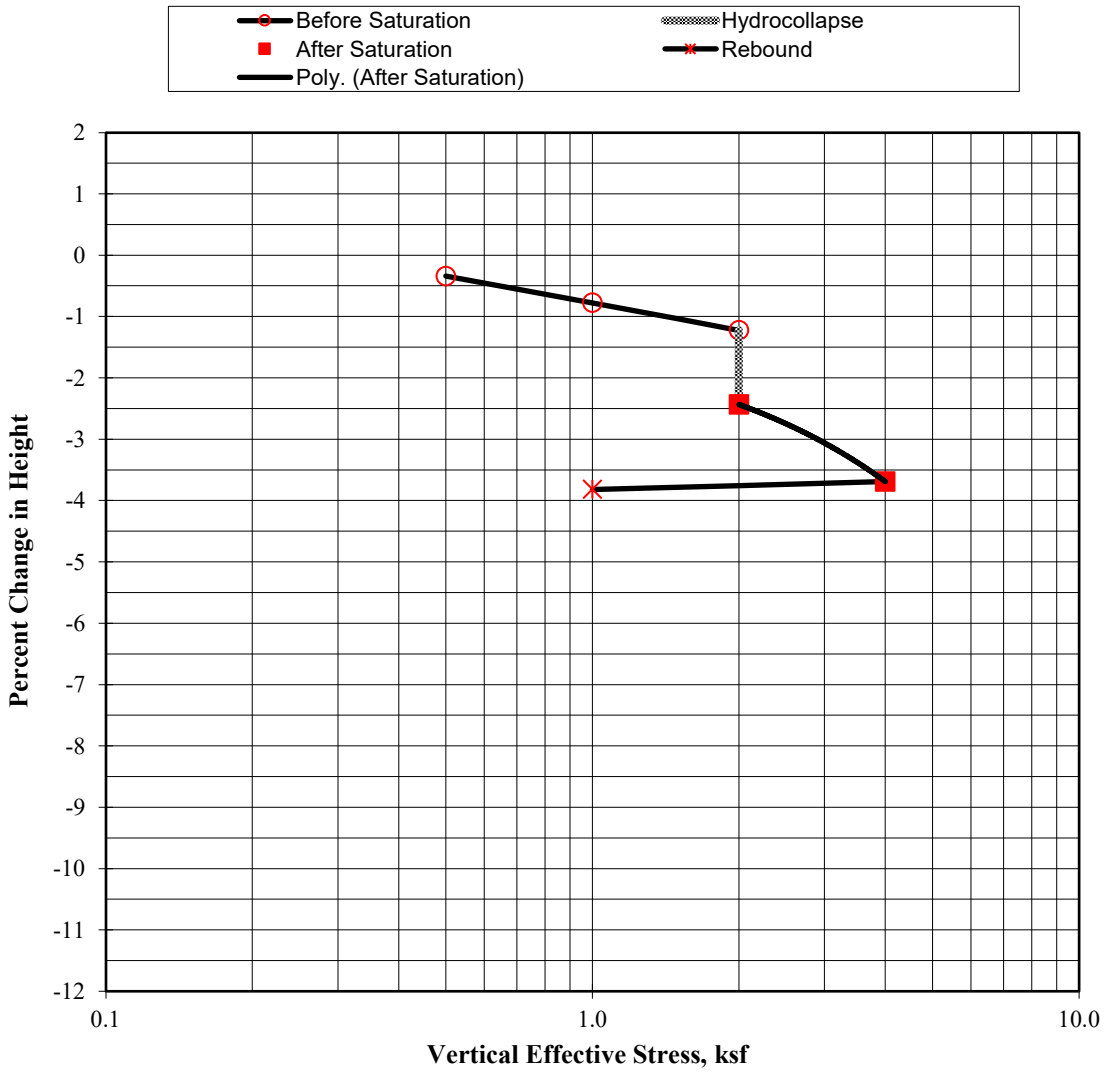
Silty Sand (SM)

Ring Sample

Initial Dry Density: 124.7 pcf
Initial Moisture: 9.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.336

Hydrocollapse: 1.2% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 7.5'

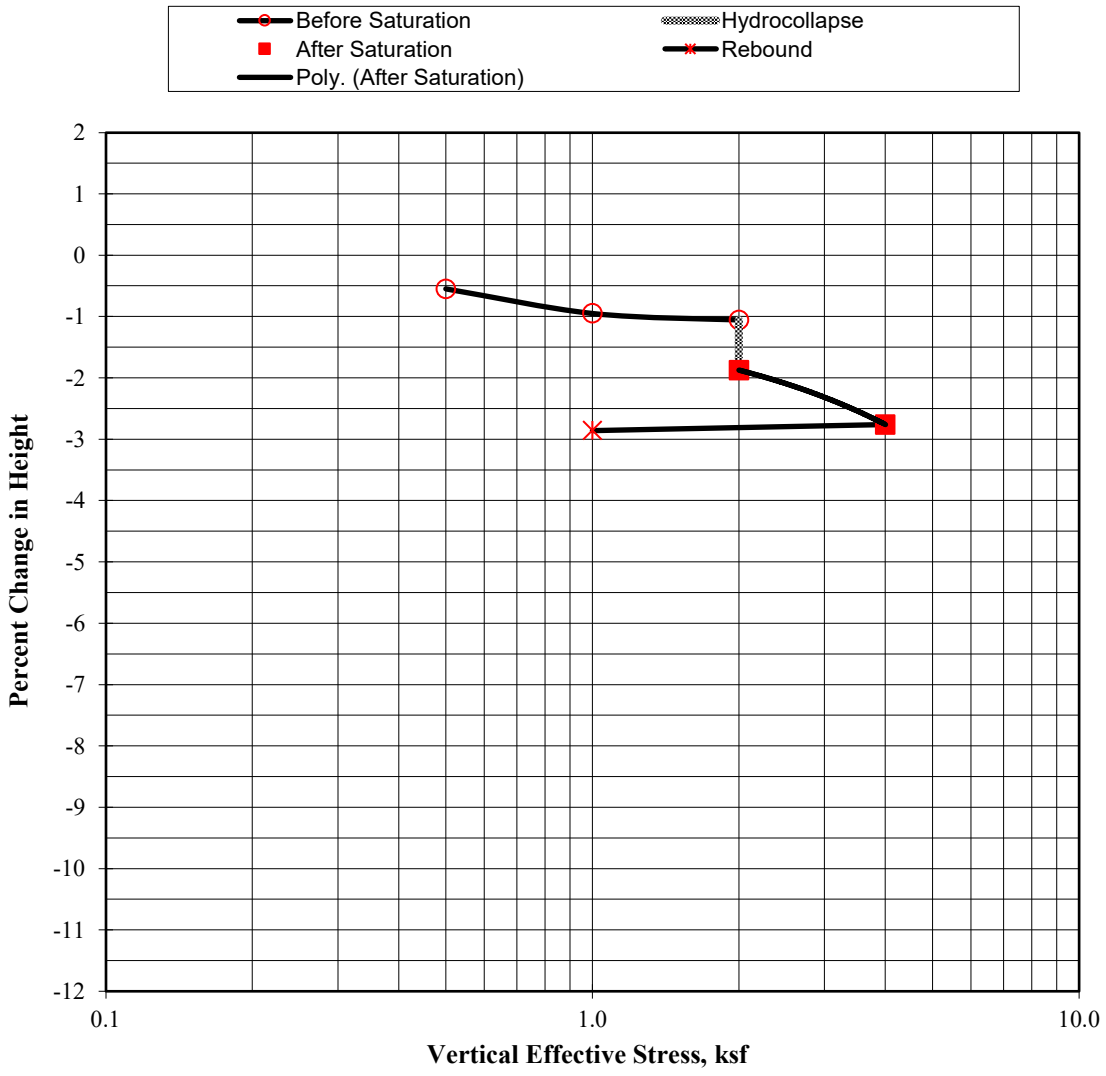
Silty Sand (SM)

Ring Sample

Initial Dry Density: 114.7 pcf
Initial Moisture: 13.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.453

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-2 @ 7.5'

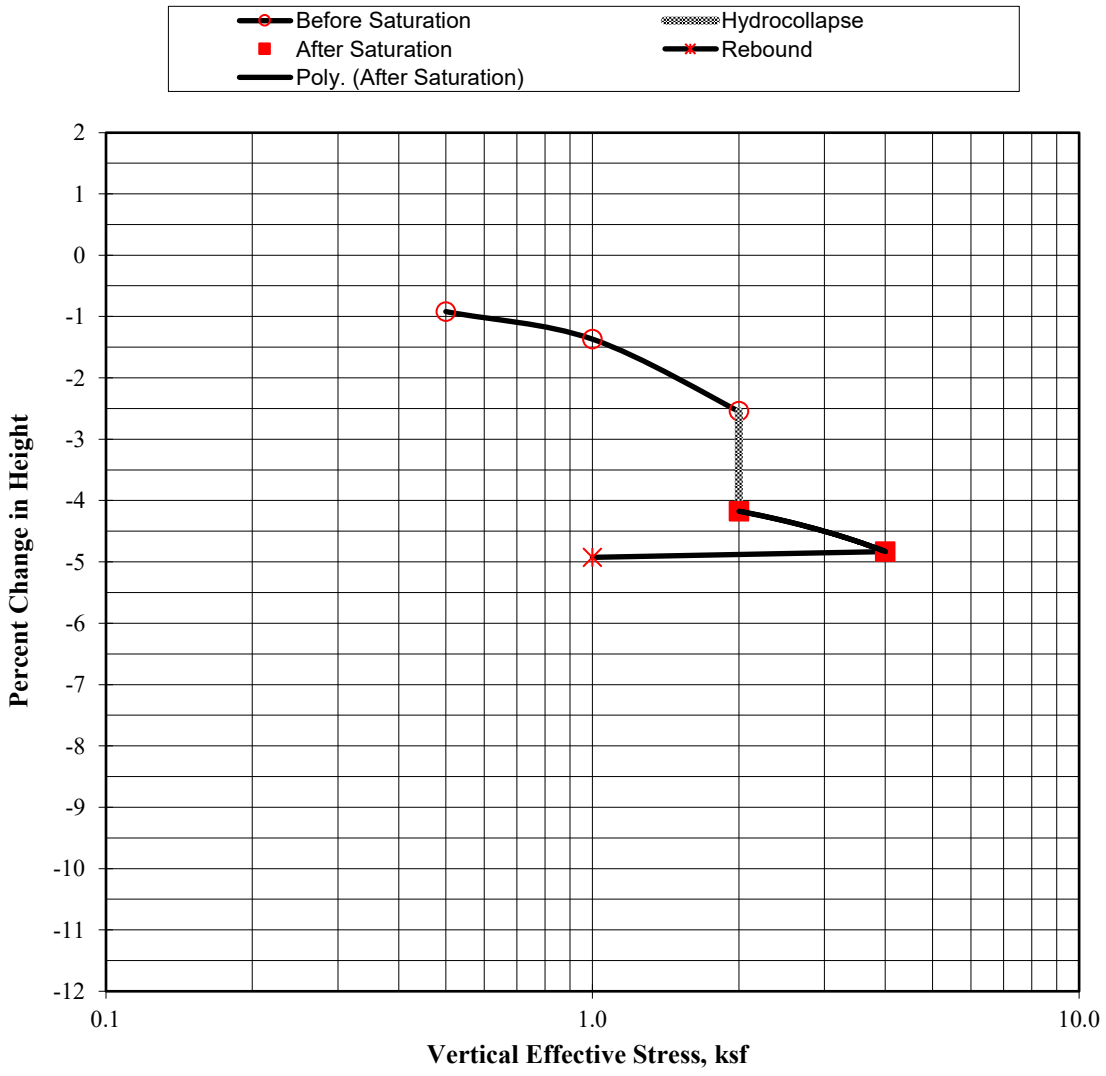
Silty Sand (SM)

Ring Sample

Initial Dry Density: 124.6 pcf
Initial Moisture: 10.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.337

Hydrocollapse: 1.6% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-2 @ 10'

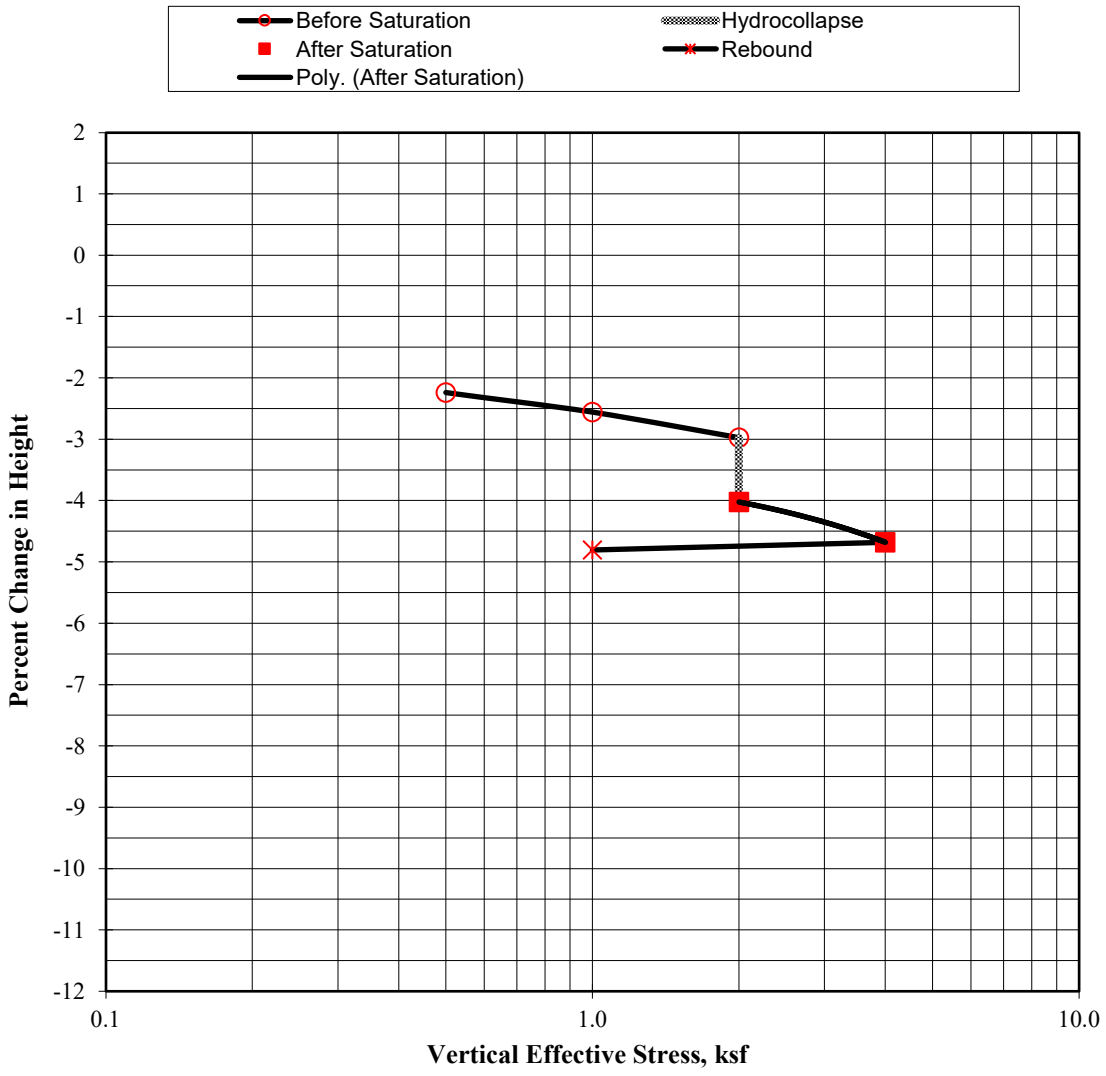
Silty Sand (SM)

Ring Sample

Initial Dry Density: 113.0 pcf
Initial Moisture: 9.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.476

Hydrocollapse: 1.1% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

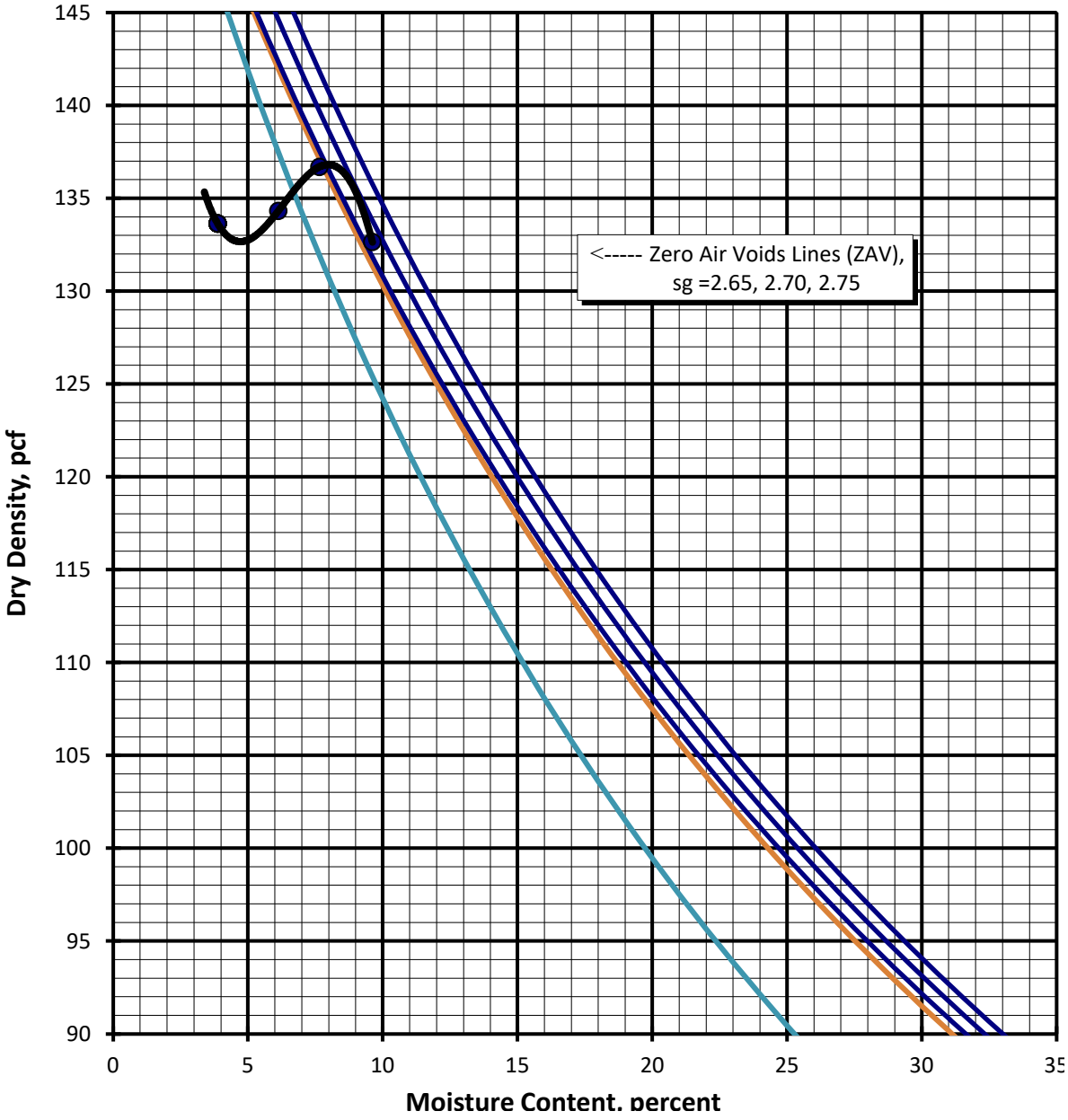
Job Name: Rosamond High School Cafeteria
Sample ID: B-1 @ 0-5'
Location: N/A

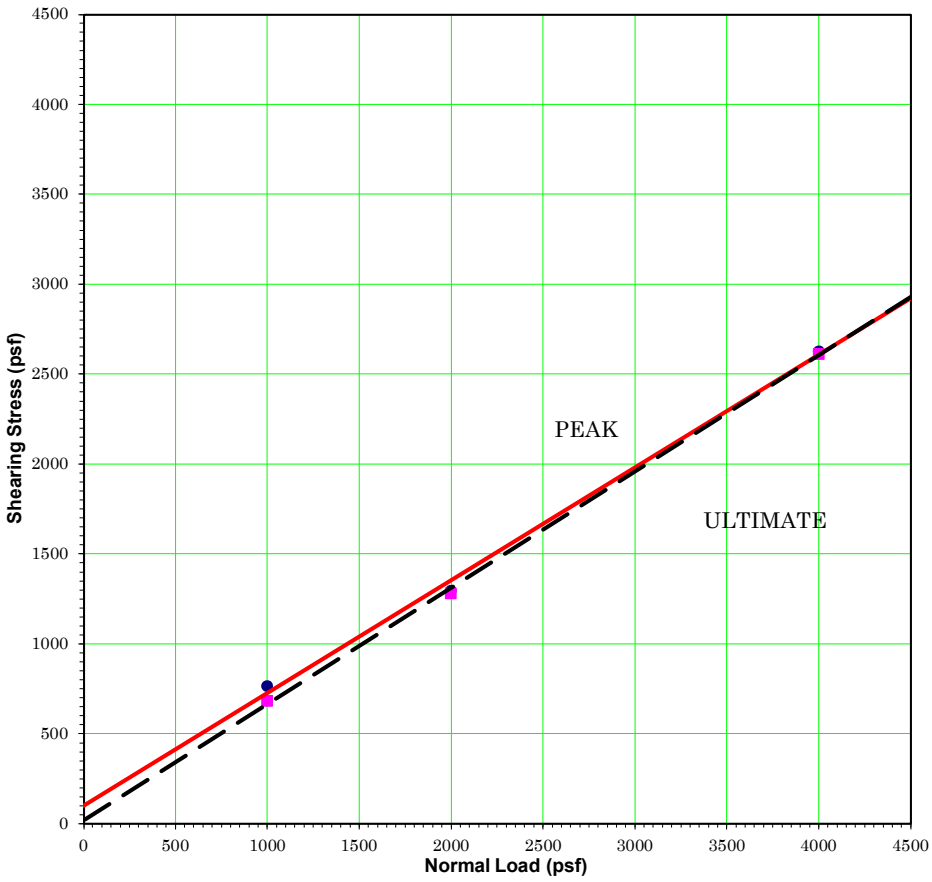
Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical

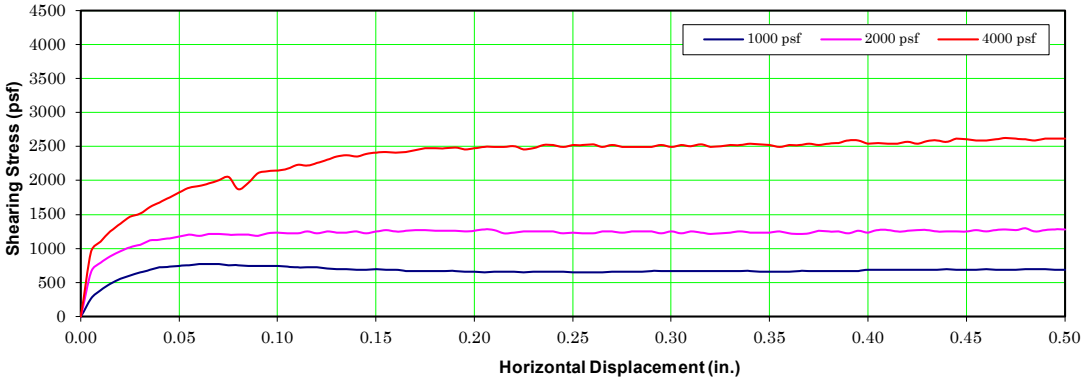
Description: Silty Sand (SM)

Maximum Dry Density: 136.8 pcf
Optimum Moisture: 8%
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	0.0
#4	5.7







DIRECT SHEAR DATA*

Sample Location: B1 @ 0-5'
Material: Silty Sand (SM)
Dry Density (pcf): 118.8

	Initial	Final
Moisture Content (%):	9	15.0
Saturation (%):	60	100
	Peak	Ultimate
ϕ Angle of Friction (degrees):	32	33
c Cohesive Strength (psf):	100	10

Test Type: Peak and Ulitimate
Shear Rate (in/min): 0.01

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Rosamond High School Cafeteria

Rosamond, Kern County, California

Earth Systems Pacific

5/24/2022 | 301430-002

TRANSMITTAL LETTER

DATE: May 27, 2022

ATTENTION: **Josh Thomas**

TO: Earth Systems Pacific - Southwest
79-811 B Country Club Drive
Bermuda Dunes, CA 92203

SUBJECT: Laboratory Test Data
Rosamond HS Cafeteria
Your #301430-002, HDR Lab #22-0538LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

Table 1 - Laboratory Tests on Soil Samples

Earth Systems Pacific - Southwest
Rosamond HS Cafeteria
Your #301430-002, HDR Lab #22-0538LAB
27-May-22

Sample ID			B-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	>4,400,000
saturated		ohm-cm	2,080
pH			7.3
Electrical Conductivity		mS/cm	0.20
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	17
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	185
potassium	K^{1+}	mg/kg	41
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	59
bicarbonate	HCO_3^{1-}	mg/kg	220
fluoride	F^{1-}	mg/kg	6.1
chloride	Cl^{1-}	mg/kg	13
sulfate	SO_4^{2-}	mg/kg	83
nitrate	NO_3^{1-}	mg/kg	54
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

SECTION 066400
FIBERGLASS REINFORCED PROTECTIVE WALL COVERING

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes: Fiberglass reinforced plastic (FRP) paneling for wall surfaces, including trim accessories.

1.2 REFERENCES

A. General: Standards listed by reference form a part of this specification section. Standards listed are identified by issuing authority, abbreviation, designation number, title or other designation. Standards subsequently referenced in this Section are referred to by issuing authority abbreviation and standard designation.

B. ASTM International:

1. ASTM D2583 - Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a Barcol Impressor.

2. ASTM D5319 – Standard Specification for Glass-Fiber Reinforced Polyester Wall and Ceiling Panels.

3. ASTM D5420 - Standard Test Method for Impact Resistance of Flat, Rigid Plastic Specimen by Means of a Striker Impacted by a Falling Weight (Gardner Impact).

4. ASTM E84 - Standard Test Method for Surface Burning Characteristics of Building Materials.

D. Crane Composities (Inspired by Kemlite):

1. Installation Guide For FRP Panels #6876.

1.3 ADMINISTRATIVE REQUIREMENTS

A. Preinstallation Meetings: Conduct preinstallation meeting to clarify Project requirements, substrate conditions, manufacturer’s installation instructions and manufacturer’s warranty requirements.

1.4 ACTION SUBMITTALS

A. Product Technical Data: For each type of product required.

B. Shop Drawings: Showing layout, profiles and product components, including anchorage, accessories, finish colors, patterns and textures. Indicate location and dimension of joints and fastener attachment.

C. Samples: Selection and verification samples for finishes, colors and textures. Submit two samples of each type of panel, trim and fastener.

D. Certificates: Product certificates signed by manufacturer certifying materials comply with specified performance characteristics, criteria and physical requirements.

E. Test and Evaluation Reports: Showing compliance with specified performance characteristics and physical properties.

F. Manufacturer's Instructions: Manufacturer's Installation Guide for FRP #6876.

G. Qualifications Statements: For manufacturer and installer.

1.5 CLOSEOUT SUBMITTALS

A. Operation and Maintenance Data: For installed products including maintenance methods and precautions against cleaning materials and methods detrimental to finishes and performance.

B. Warranty: Warranty documents required in this section.

1.6 MAINTENANCE MATERIAL

A. Extra Materials: Deliver to Owner extra materials from same production run as products installed. Package products with protective covering and identify with descriptive labels. Comply with Division 01 Closeout Submittals Section.

1. Quantity: Furnish quantity of units equal to 5% percent of amount installed.
2. Delivery, Storage and Protection: Comply with Owner's requirements for delivery, storage and protection of extra materials.

1.7 QUALITY ASSURANCE

A. Manufacturer Qualifications:

1. Provider of advanced installer training.

B. Installer Qualifications:

1. At least five years experience in the installation of fiberglass reinforced plastic panels.

2. Experience on at least five projects of similar size, type and complexity as this Project.

3. Employer of workers for this Project who are competent in techniques required by manufacturer for installation indicated.

C. Surface-Burning Characteristics: Determined by testing identical products according to ASTM E84 by a testing agency acceptable to authorities having jurisdiction.

1. Flame-Spread Index: 25 (Class A)] or less.

2. Smoke-Developed Index: [450] or less.

1.8 DELIVERY, STORAGE AND HANDLING

A. Delivery: Deliver materials in manufacturer's original, unopened, undamaged containers with identification labels intact. Package sheets on skids or pallets for shipment to project site.

B. Storage and Handling: Store materials protected from exposure to harmful weather conditions and at temperature and humidity conditions recommended by manufacturer. Store panels in a dry indoor location at Project site. Remove any foreign matter from face of panel by using a soft bristle brush, avoiding abrasive action.

1.9 PROJECT CONDITIONS

A. Ambient Conditions:

1. Do not begin installation until building is enclosed, permanent heating and cooling equipment is in operation, and residual moisture from plaster, concrete or terrazzo work has dissipated.

2. During installation, and within 48 hours prior to installation, maintain ambient temperature and relative humidity within limits required by type of panel adhesive used and recommendation of panel adhesive manufacturer.

1.10 WARRANTY

A. Special Warranty: Manufacturer's standard form in which manufacturer agrees to repair or replace FRP panels that fail within specified warranty period.

1. Failures shall include, but not be limited to substantial defects in material and workmanship, rotting, rusting, corrosion, development of structural surface cracks, or requiring painting or refinishing.

2. Warranty Period: Ten years from date of Substantial Completion.

B. Special Warranty: Installer's standard form in which installer agrees to repair or replace FRP panels that fail due to poor workmanship or faulty installation within the specified warranty period.

1. Warranty Period: 1 years from date of Substantial Completion.

PART 2 - PRODUCTS

2.1 FIBERGLASS REINFORCED PLASTIC (FRP) PANELS

A. General: Fiberglass reinforced plastic panels complying with ASTM D5319.

1. Low-Emitting Materials: Comply with testing and product requirements of California Department of Health Services standards for Volatile Organic Emissions.

B. Basis of Design Product: Subject to compliance with requirements provide Crane Composites, Inc.; Class A Fiberglass Reinforced Plastic (FRP) Panels, or approved equal.

C. Product Options:

1. Color: As selected by Architect manufactures full color line of Class A FRP products.

2. Surface Finish: As Approved.

3. Nominal Thickness: 0.09 inch (2.3 m)

4. Wall Panel Size: 4 feet (1.2 m) by 8 feet (2.4 m)

2.2 ACCESSORIES

A. Moldings, Trim and Caps: One-piece extruded polypropylene or PVC, configured to cover panel edges and corners.

1. Color: 85 White.

B. Panel Adhesive: As recommended by panel manufacturer for the required substrates.

1. Adhesive shall have a VOC content of 50 g/L or less.

C. Panel Sealant: As recommended by panel manufacturer.

1. Sealant shall have a VOC content of 250 g/L or less.

2.3 SOURCE QUALITY CONTROL

A. Obtain fiberglass reinforced panels, moldings and other accessories from a single manufacturer.

PART 3 - EXECUTION

3.1 EXAMINATION

A. General: Comply with manufacturer's product data, including product technical bulletins, and installation instructions in product catalogs and product packaging.

B. Verify that substrates previously installed under other sections are acceptable for product installation in accordance with FRP manufacturer's instructions.

1. Examine substrate surfaces to determine that corners are plumb and straight, that surfaces are smooth, sound and uniform, that nails or screw fasteners are countersunk, and that joints and cracks are filled flush and smooth with adjoining surfaces.

2. Do not begin panel installation until substrate surfaces are in satisfactory condition.

3.2 PREPARATION

A. Clean substrates to remove substances that could impair bond of adhesive, including oil, grease, dirt, dust or other contamination.

B. Condition panels by unpacking and placing in installation space no less than 24 hours before installation.

C. Lay out paneling before beginning installation. Locate panel joints to provide equal panel widths at ends of walls and so that trimmed panels at corners are not less than 12 inches (300 mm) wide.

3.3 INSTALLATION

A. General: Comply with panel manufacturer's Installation Guide #6876.

B. Cut and drill panels with carbide tipped saw blades or drill bits, or cut with snips.

C. Install panels with manufacturer's recommended gap for panel field and corner joints.

1. Pre-drill fastener holes in panels, 1/8 inch (3.2 mm) greater in diameter than fastener.

2. Install panels in a full spread of adhesive. For trowel type and application of adhesive, follow adhesive manufacturer's recommendations.

D. Install trim accessories with adhesive and nails or staples. Do not fasten through panels.

E. Sealant:

1. Fill grooves in trim accessories with sealant before installing panels and bed inside corner trim in a bead of sealant.

2. Remove excess sealant and smears as paneling is installed. Clean with solvent recommended by sealant manufacturer and then wipe with clean dry cloths.

3.5 CLEANING

A. Remove temporary coverings and protection of adjacent work areas.

B. Repair or replace any installed products that have been damaged.

C. Clean installed panels in accordance with manufacturer's instructions prior to Owner's acceptance.

D. Remove and lawfully dispose of construction debris from project site.

3.6 PROTECTION

A. Protect installed product and finish surfaces from damage during construction.

END OF SECTION

Southern Kern Unified School District
P.O. Drawer CC
Rosamond, California 93560

Geotechnical Engineering and Geohazards Report
Southern Kern Unified School District
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

June 14, 2022

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File No.: 300234-002
Doc. No.: 22-06-705

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June 14, 2022

File No.: 301430-002
Doc. No.: 22-06-705

Southern Kern Unified School District
P.O. Drawer CC
Rosamond, California 93560

Attention: Ms. Barbara Gaines

Subject: **Geotechnical Engineering and Geohazards Report**

Project: **Southern Kern Unified School District**
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems Pacific (Earth Systems) presents this geotechnical and geohazards report for the new planned cafeteria and associated outdoor eating area at the existing Rosamond High School campus located at 2925 Rosamond Boulevard in Rosamond, Kern County, California. This report presents our findings and recommendations for site grading and foundation design. From a geotechnical engineering standpoint, the site is suitable for the proposed development, provided the recommendations in this report are followed in design and construction. This report should stand as a whole, and no part of the report should be excerpted or used to the exclusion of any other part.

This report completes our scope of services in accordance with our proposal (PAL-22-3-003) dated March 9, 2022. Other geotechnical related services that may be required, such as plan reviews, responses to agency inquiries, and grading observation and testing are additional services and will be billed according to the Fee Schedule in effect at the time services are provided. The Client is responsible to distribute the report to the appropriate governing agency and other members of the design team. The Limitations (Section 6) of this report should be reviewed as they are vital to the understanding of this report.

We appreciate the opportunity to provide our professional services. Please contact our office if there are any questions or comments concerning this report or its recommendations.

Respectfully submitted,
Earth Systems Pacific

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Geotechnical Engineering and Geohazards Report
Southern Kern Unified School District
Rosamond High School Cafeteria Project
2925 Rosamond Boulevard
Rosamond, Kern County, California

Section 1
INTRODUCTION

1.1 Project and Site Description

This Geotechnical Engineering and Geohazards Report has been prepared for the proposed Rosamond High School Cafeteria Project to be constructed at the existing Rosamond High School campus located at 2925 Rosamond Boulevard, in Rosamond, Kern County, California. The proposed improvements will include a new cafeteria building with a footprint of approximately 7,246 square feet. The new building will be supported on concrete conventional spread foundations. A service yard and covered outdoor eating area are also planned, as well as other improvements necessary for site development. The approximate location of the campus is shown on Plate 1, Site Vicinity Map. The locations of the proposed improvements are shown on Plate 2, Exploration Location Map and Site Geology. From Google Earth, the site elevations range from approximately 2,347 feet and 2,415 feet (MSL) and the site is located at approximate site coordinates of 34.8661°N, 118.1831°W.

Currently, the proposed building site is generally flat and level. The proposed improvement areas are currently vacant of existing improvements except for an existing chain link fence. The ground surface is unpaved in the proposed improvement areas.

The project will be designed and constructed under the jurisdiction of the Division of the State Architect (DSA), and the 2019 California Building Code (CBC). We understand that the new building will be single-story, of steel and/or stud-frame construction. The building will be founded on conventional continuous and spread footings. For the purposes of this report, we have assumed maximum continuous wall loads of 3 kips per linear foot (klf) and maximum isolated column loads of 100 kips. The buildings will be served by the existing campus utility systems. A CMU wall trash enclosure and pedestrian PCC flatwork are also be included as part of the project. No preliminary design loading was provided by the structural engineer. If actual structural loading exceeds these assumed values, the given recommendations should be re-evaluated.

Grading has been assumed to consist of cuts and fills on the order of 5 feet or less from existing grade. New slopes are not proposed, and existing slopes are minimal as the improvement sites are relatively flat. Public roadways (Rosamond Boulevard and 30th Street West) are located south and west, respectively, and vacant land and single-family residences are present to the north and east, respectively, of the campus.

1.2 Purpose and Scope of Services

The purpose for our services was to evaluate the site soil and geologic conditions at our exploration locations and to provide professional opinions and recommendations, from a geologic and geotechnical point of view, regarding the proposed development of the site. We understand that these proposed school site improvements will be developed under the regulation of the Division of the State Architect (DSA) and the current California Building Code (2019).

The conclusions and recommendations included in this report are based upon the data collected for this commission. The scope of services performed for this study included:

Task 1 - Literature and Photograph Reviews

We reviewed select geologic and geotechnical literature pertaining to the project, including various hazard, fault, and geologic maps prepared by the California Geological Survey (CGS), the U.S. Geological Survey (USGS), Kern County and other governmental agencies as they relate to the project area. Select historical aerial photographs were reviewed using the Google Earth Pro website and Historical Aerials website. Select information contained in these reports was utilized in the preparation of this report.

Task 2 – Utility Clearance, USA Dig Alert

Each of our proposed field exploration locations was located and marked in the field and cleared with known utility lines as identified by Underground Service Alert (USA), “Dig Alert”. Our exploration locations were located in the field by a hand-held GPS or sighting from landmarks identified in the field (i.e., trees or existing fences).

Task 3 – Field Exploration

We evaluated the general subsurface conditions at the site by drilling two small-diameter hollow-stem auger borings to depths of approximately 26½ feet to 51½ feet below the existing ground surface (bgs). The CGS Note 48 for school facilities requires a minimum of one exploration point within the near vicinity of the building footprint for every 5,000 square feet of first floor area, with a minimum of two borings per structure. Should the proposed structure be moved from the presently proposed and discussed locations, the current boring locations may not satisfy the required criteria and additional borings may be required. The field exploration also included a site reconnaissance of the project area and immediate surroundings. Plate 2 shows the surface geology and the approximate location of each boring.

Task 4 - Laboratory Testing

Laboratory tests were performed on selected samples to evaluate the physical characteristics of the materials encountered during our field exploration. Laboratory testing included moisture content, dry unit weight, maximum dry density/optimum moisture content, sieve analysis, consolidation/collapse potential, direct shear, and Expansion Index. The testing was performed in general accordance with American Society for Testing and Materials (ASTM) or appropriate test procedures. Selected samples were also tested for a preliminary screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates and water-soluble chlorides). Earth

Systems does not practice corrosion engineering; however, these test results may be used by a qualified engineer in designing an appropriate corrosion plan for the project.

Task 5 – Analysis and Report

Earth Systems analyzed the field data obtained, performed engineering analyses, and provided recommended design parameters for earthwork and foundations for the structures as described within. Our report includes:

- A description of the proposed project including a site plan showing the approximate boring locations;
- A description of the surface and subsurface site conditions including groundwater conditions, as encountered in our field exploration;
- A description of the site geologic setting and possible associated geology-related hazards, including liquefaction, subsidence, and seismic settlement analysis;
- A site-specific geologic map;
- A discussion of regional geology and site seismicity, including a regional geology map;
- A description of local and regional active faults, their distances from the site, their potential for future earthquakes;
- A discussion of other geologic hazards such as ground shaking, landslides, flooding, and tsunamis;
- A discussion of site conditions, including the geotechnical suitability of the site for the general type of construction proposed;
- A “General Procedure” and “Site Specific” seismic analysis including recommendations for geotechnical seismic design coefficients and soil profile type in accordance with the 2019 California Building Code;
- Recommendations for imported fill for use in compacted fills;
- Recommendations for foundation design including parameters for shallow foundations, and subgrade preparation;
- Anticipated total and differential settlements for the recommended foundation system;
- Preliminary recommendations for the mitigation of seismic induced settlement including liquefaction;
- Recommendations for lateral load resistance (earth pressures and drainage);
- Recommendations for site preparation, earthwork, and fill compaction specifications;
- Discussion of anticipated excavation conditions;
- Recommendations for underground utility trench backfill;
- Recommendations for stability of temporary trench excavations;
- Recommendations for slabs-on-grade;
- Recommendations for collapsible or expansive soils (if applicable);
- A discussion of the corrosion potential of the near-surface soils encountered during our field exploration;

- An appendix, which includes a summary of the field exploration (computer generated boring logs) and laboratory testing program (computer generated plots).

Not Contained in This Report: Although available through Earth Systems, the current geotechnical scope of our services does not include:

- A Phase 1 Environmental Site Assessment.
- An investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

Section 2
METHODS OF EXPLORATION AND TESTING

2.1 Field Exploration

The subsurface exploration program included advancing two exploratory borings to depths on the order of 26½ to 51½ feet bgs. The borings were drilled using a Mobile B-61 truck-mounted drill rig provided by GP Drilling of Apple Valley, California and equipped with 8-inch hollow-stem augers. Borings were advanced to observe soil profiles and obtain samples for laboratory testing. The approximate boring locations are shown on Plate 2, Exploration Location Map. The locations shown are approximate, established by a hand-held GPS and relative landmark locations.

An engineer from Earth Systems maintained a log of the subsurface conditions encountered and obtained samples for visual observation, classification and laboratory testing. Subsurface conditions encountered in the borings were categorized and logged in general accordance with the Unified Soil Classification System [USCS] and ASTM D 2487 and 2488 (current edition). Our typical sampling interval within the borings was approximately every 2½ or 5 feet to the full depth explored; however, sampling intervals were adjusted depending on the materials encountered onsite. Samples were obtained within the test borings using a California type ring sampler (ASTM D 3550 with those similar to ASTM D 1586). The ring sampler has an approximate 3-inch outside diameter and 2.4-inch inside diameter. Further sampling and collection of disturbed soil samples was accomplished using the Standard Penetration Test (SPT) sampler in accordance with ASTM D 1586. The SPT sampler is a split barrel sampler with a 1-3/8 inch inside diameter. The SPT and ring sampler was mounted on a drill rod and driven using a rig-mounted 140-pound automatic hammer falling for a height of 30 inches. The number of blows necessary to drive the samplers within the borings was recorded.

Design parameters provided by Earth Systems in this report have considered an estimated 72% hammer efficiency based on data provided by the drilling subcontractor. The N-values (blow count) using the California type sampler can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. In general, a conversion factor of approximately 0.63 from a study at the Port of Los Angeles (Zueger and McNeilan, 1998 per SP 117A) is considered satisfactory. A value of 0.63 was applied in our calculations for this project.

Bulk samples were also obtained from the drill auger cuttings, representing a mixture of soils encountered at the depths noted. The depth to groundwater, if any, was measured in the boreholes. Following drilling, sampling, and logging, the borings were backfilled with the cuttings and tamped upon completion. Our field exploration was provided under the direction of a State of California Registered Geotechnical Engineer from our firm.

The logs of the borings represent our interpretation of the soils observed in the field and the results of laboratory testing performed on the samples obtained; the logs are included in Appendix A of this report. The stratification lines represent the approximate boundaries between soil types, although the transitions may be gradual. In reviewing the logs and legend, the reader should recognize the legend is intended as a guideline only, and there are a number of conditions that may influence the soil characteristics observed during drilling. These include, but are not limited to cementation, variations in soil moisture, presence of groundwater, and other factors.

The boring logs present field blow counts per 6 inches of driven embedment (or portion thereof) for a total driven depth attempted of 18 inches. The blow counts on the logs are uncorrected (i.e., not corrected for overburden, type of sampler etc.). Consequently, the user must correct the blow counts per standard methodology if they are to be used for design and exercise judgment in interpreting soil characteristics, possibly resulting in soil descriptions that vary somewhat from the legend.

2.2 Laboratory Testing

Samples were reviewed along with field logs to select those that would be analyzed further. Those selected for laboratory testing include, but were not limited to, soils that would be exposed and those deemed to be within the influence of the proposed structures. Test results are presented in graphic and tabular form in Appendix B of this report. Testing was performed in general accordance with American Society for Testing and Materials (ASTM) or another appropriate test procedure. Selected samples were also tested for a screening level of corrosion potential (pH, electrical resistivity, water-soluble sulfates, and water-soluble chlorides). Earth Systems does not practice corrosion engineering; however, these test results may be used by a qualified corrosion engineer in designing an appropriate corrosion control plan for the project.

Our testing program consisted of the following:

- Density and Moisture Content of select samples of the site soils (ASTM D 2937 & 2216).
- Maximum Dry Density/Optimum Moisture Content tests to evaluate the moisture-density relationship of typical soils encountered (ASTM D 1557).
- Particle Size Analysis to classify and evaluate soil composition. The gradation characteristics of selected samples were made by sieve analysis procedures (ASTM D 6913).
- Washed Sieve Analyses were performed on select samples of the site soils to determine the percent soil fraction passing the No. 200 sieve (ASTM D 1140).
- Direct Shear testing was performed on a representative sample remolded to 90 percent relative compaction (ASTM D 3080).
- Consolidation and Collapse Potential to evaluate the compressibility and hydroconsolidation (collapse) potential of the soil upon wetting (ASTM D 5333).
- Expansion Index tests to evaluate the expansive nature of the soil. The samples were surcharged under 144 pounds per square foot at moisture contents of near 50% saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator (ASTM D 4829).
- Screening Level Chemical Analyses (Soluble Sulfates and Chlorides (ASTM D 4327), pH (APHA 2320-B), and Electrical Resistivity/Conductivity (ASTM G 187) to evaluate the potential for adverse effects of the soil on concrete and steel.

Section 3
DISCUSSION

3.1 Soil Conditions

The field exploration and literature review indicate that site geologic materials consist generally of Quaternary Alluvial (Qa) deposits. The alluvial soils consisted of silty sands, poorly graded sands, and silty clays with sand (SM, SP, SP-SM, and CL-ML per the Unified Soil Classification System) with a medium dense to very dense consistency for the coarse-grained soils and medium stiff to very stiff consistency for the fine-grained soils. In general, the upper five feet of soils were damp. Detailed descriptions of the observed fill and alluvial soils are provided on the boring logs in Appendix A. Due to the flat site, geologic cross sections are not provided.

3.2 Groundwater

Groundwater was not encountered within the upper 51½ feet of the soil profile. Historically shallowest groundwater levels were not mapped by the California Geological Survey in the site vicinity; however, historically shallowest groundwater levels are mapped by CGS approximately 3 miles to the south of the site at depths ranging from of 100 to 150 feet below ground surface.

Nearby state monitored wells were researched for their most recent and historic well readings. The following is a summary of our findings for the two wells closest to the campus.

- Well No. 348672N1181846W001 was located in the northwest portion of the campus. In that State Well, depth to groundwater was measured at approximately 83 feet below the ground surface in 1951.
- Well No. 348644N1181846W001 was located in the southwest portion of the campus. In that State Well, depth to groundwater varied between approximately 99 feet to 160 feet below the ground surface between 1951 and 1967, indicating an increased depth to water with time. The most recent depth to groundwater was measured at approximately 160 feet below the ground surface in 1967.

Fluctuations in groundwater levels may occur due to variations in rainfall, regional climate, and in response to landscape irrigation. However, groundwater in the vicinity of the Rosamond High School campus is expected to be more than 50 feet below the existing ground surface. Perched water conditions were not observed, despite the site being surrounded by nearby residential housing and associated irrigation for decades.

3.3 Collapse and Consolidation Potential

Collapsible soil deposits generally exist in regions of moisture deficiency. Collapsible soils are generally defined as soils that have potential to suddenly decrease in volume upon increase in moisture content even without an increase in external loads. Soils susceptible to collapse include loess, weakly cemented sands and silts where the cementing agent is soluble (e.g., soluble gypsum, halite), valley alluvial deposits within semi-arid to arid climate, and certain granite residual soils above the groundwater table. In arid climatic regions, granular soils may have a potential to collapse upon wetting. Collapse (hydro-consolidation) may occur when the soils are

lubricated or the soluble cements (carbonates) in the soil matrix dissolve, causing the soil to densify from its loose configuration from deposition.

The degree of collapse of a soil can be defined by the Collapse Potential (CP) value, which is expressed as a percent of collapse of the sample using the Collapse Potential Test (ASTM Standard Test Method D 5333). Based on the Naval Facilities Engineering Command (NAVFAC) Design Manual 7.1, the severity of collapse potential can be evaluated by the following Table 1, Collapse Potential Values.

Table 1
Collapse Potential Values

Collapse Potential Value	Severity of Problem
0-1%	No Problem
1-5%	Moderate Problem
5-10%	Trouble
10-20%	Severe Trouble
> 20%	Very Severe Trouble

Table 1 can be combined with other factors such as the probability of ground wetting to occur on-site and the extent or depth of potential collapsible soil zone to evaluate the potential hazard by collapsible soil at a specific site. A hazard ranking system associated with collapsible soil as developed by Hunt (1984) is presented in Table 2, Collapsible Soil Hazard Ranking System.

Table 2
Collapsible Soil Hazard Ranking System

Degree of Hazard	Definition of Hazard
No Hazard	No hazard exists where the potential collapse magnitudes are non-existent under any condition of ground wetting.
Low Hazard	Low hazards exist where the potential collapse magnitudes are small and tolerable, or the probability of significant ground wetting is low.
Moderate Hazard	Moderate hazards exist where the potential collapse magnitudes are undesirable, or the probability of substantial ground wetting is low, or the occurrence of the collapsible unit is limited.
High Hazard	High hazard exists where potential collapse magnitudes are undesirably high and the probability of occurrence is high.

The results of collapse potential tests performed on five (5) selected single ring samples from depths ranging from 2½ to 10 feet below the ground surface in native soils indicated a collapse potential range of 0.8 to 1.6 percent. The goal of the collapse testing was to identify soils and densities where the potential for collapse decreased to accepted levels. This accepted level is defined as where on-site soils had collapse potential less than 2% or the estimated relative compaction is greater or equal to 85%, which is the typical standard of care based on the above Table 1 or where soil collapse becomes a concern for structural soils (2%) (County of Los Angeles, 2013). Plotting and analysis of the results of the five tests from whole stack ring samples indicates that collapse potential at the site appears to be directly related to in-place density (relative

compaction). Typically, it is assumed in engineering practice that at least 1% of total tested collapse is due to sample disturbance. This is demonstrated via Table 1.

Based on the field and laboratory testing performed, Earth Systems provides key items of interest that supports Earth Systems recommendations regarding collapse potential at this site:

1. Soils are generally granular in nature; however, cementation was observed in soils from 6.5 to 14 feet below ground surface.
2. Voiding was not observed other than minor occurrences.
3. Soil collapse at the site appears to be related to in-place density (relative compaction).
4. In-place densities as measured for a whole soil sample stack (6 rings) generally indicated higher densities, such that collapsible zones may be limited to thinner soil layers with corresponding lower density (i.e., it is considered low potential for a whole soil wetted zone to collapse of the same magnitude through the whole zone).

For some deposits without cementation, studies suggest some sites with densities above 103 pcf are “not likely to collapse” and N60 values > 10 do not fit into the category of “Likely Collapsible” (Lommler, C.J. and Bandini). In addition, soils with greater than 85 percent relative compaction are compact, and it is accepted that they are not likely to settle, especially after initial inundation. Eighty-five percent of the maximum density for the silty sand type soils is 116 pcf.

Based on the above criteria and our field and laboratory findings, we estimate there is “Low” to “Moderate” collapse potential from soil layers between 2½ and 10 ft bgs at the locations tested. Without collapse mitigation efforts, the collapse potential is estimated to be approximately over 1½ inches. Assuming the recommended grading is accomplished according to Section 5.1 of this report, we estimate the collapse potential is on the order of approximately ½ inch (1 inch in hardscape areas with the lesser overexcavation).

3.4 Expansive Soils

Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors, and may cause unacceptable settlement or heave of structures, concrete slabs supported-on-grade, or pavements supported over these materials. Depending on the extent and location below finished subgrade, expansive soils can have a detrimental effect on structures. Based on our laboratory testing and experience with the project, the expansion potential of the on-site soils tested is generally “very low” as defined by ASTM D 4829 and the 2019 California Building Code.

Testing and/or observation of the subgrade soils during grading within the building pad and at the footing grade should be performed to further evaluate the expansion potential and confirm or modify the recommendations presented herein.

3.5 Corrosivity

One sample of the near-surface soils within the site was tested for potential corrosion of concrete and ferrous metals. A representative soil sample collected between 0 and 5 feet bgs was tested in general accordance with the ASTM Standard Test Methods to evaluate pH, resistivity, and water-soluble sulfate and chloride content. The test results are presented in Appendix B. These tests should be considered as only an indicator of corrosivity for the samples tested. Other earth materials found on site may be more, less, or of a similar corrosive nature.

Water-soluble sulfates in soil can react adversely with concrete. ACI 318 provides the relationship between corrosivity to concrete and sulfate concentration, presented in the table below:

Table 3

Water-Soluble Sulfate in Soil (ppm)	Corrosivity to Concrete
0-1,000	Negligible
1,000 – 2,000	Moderate
2,000 – 20,000	Severe
Over 20,000	Very Severe

In general, the lower the pH (the more acidic the environment), the higher the soil corrosivity will be with respect to ferrous structures and utilities. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures, due to protective surface films, which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried steel or reinforced concrete structures. Soil resistivity is a measure of how easily electrical current flows through soils and is the most influential factor. Based on the findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (ASTM, 1989), the approximate relationship between soil resistivity and soil corrosivity was developed as shown in Table 4.

Table 4

Soil Resistivity (Ohm-cm)	Corrosivity to Ferrous Metals
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Test results show a pH value of 7.3, chloride content of 13 ppm, sulfate content of 83 ppm and minimum resistivity of 2,080 Ohm-cm. Although Earth Systems does not practice corrosion engineering, the corrosion values from the soil tested are normally considered as being "Severely" corrosive to buried metals and as possessing a "negligible" exposure to sulfate attack for concrete as defined in American Concrete Institute (ACI) 318, Section 4.3. The results of all chemical testing have been provided in Appendix B. The above values can potentially change based on several factors, such as importing soil from another job site and the quality of construction water used during grading and subsequent landscape irrigation.

3.6 Geologic Setting

Regional Geology: The project site is located at the western portion of the Mojave Desert Geomorphic Province (see Plates 3 and 4), a broad interior region of isolated mountain ranges separated by desert plains and basins. The Western Mojave Desert is a wedge-shaped structural block bounded on the northwest by the Garlock fault and along the southwest by the San Andreas rift zone. The Sierra Nevada and Tehachapi Mountains are north of the Garlock fault. The Transverse Ranges and coastal basins are southwest of the San Andreas fault. Regional features of the western Mojave Desert include the alluvial-filled Antelope Valley, San Gabriel and San Bernardino Mountains, the Rosamond Hills, Rosamond Lake, and scattered buttes.

The Rosamond High School campus is located near the Rosamond Hills in the Antelope Valley. The site is underlain by a deep section of Quaternary sediments. Tertiary volcanic and sedimentary rocks and pre-Tertiary igneous rocks are exposed in the hills to the northwest and north.

The Garlock fault that forms the northwest boundary of the Antelope Valley is the closest active fault to the campus that has significant earthquake potential. The nearest reach of the fault is approximately 16.6 miles to the northwest of the campus.

No known active or potentially active faults trend through the campus. However, it should be noted that Jennings and Bryant (2010) show a fault identified as the Willow Springs fault located approximately 4.5 miles west of the campus and projecting in a trend slightly south of the site. It is characterized as a Quaternary fault (displacement within the last 1.6 million years).

In addition, a "pre-Quaternary fault", i.e., an inactive fault that is concealed by alluvium, trends along the north side of Rosamond Avenue and close to the campus. This fault has been identified on other geologic maps as the Rosamond fault.

3.7 Geologic Hazards

Geologic hazards that may affect the region include seismic shaking, surface fault rupture, soil liquefaction, landsliding and rock fall, flooding, expansive soils, ground subsidence, and erosion. A discussion follows on the specific hazards to this site.

3.7.1 Seismic Shaking

The primary seismic hazard to the site is strong ground shaking from earthquakes along regional faults such as the San Andreas fault, the Garlock fault, the Sierra Nevada fault zone, the Eastern California shear zone, the White Wolf fault zone, and the many faults within the Los Angeles basin and Inland Empire. The Garlock fault is located approximately 16.6 miles northwest of the site. The Mojave segment of the San Andreas fault is located approximately 17.3 miles southwest of the campus. About 40 other active faults or seismic zones lie within 52 miles of the project site (see Table A-1).

The site is located within an active seismic area in southern California where large numbers of earthquakes are recorded each year. Approximately 35 magnitude 5.5 or greater earthquakes have occurred within 60 miles of the site since 1812. Plate 7, Earthquake Epicenter Map, depicts epicenters of significant seismic events greater than magnitude 5.5 that have occurred in southern California between 1812 and 1999. Magnitudes that are above 6 and prior to accurate instrumental measurements (after 1933) are based on moment magnitudes (M_W). Magnitudes that are below 6 or earthquakes prior to 1933 are based on local magnitudes (M_L).

Many of the major historic earthquakes felt in the vicinity of the Rosamond area have originated from faults located outside the area. These include the 1812 Wrightwood, 1857 Fort Tejon, 1872 Owens Valley, 1933 Long Beach, 1952 Arvin-Tehachapi, 1971 San Fernando, 1987 Whittier Narrows, 1992 Landers, 1994 Northridge, and 1999 Hector Mine earthquakes. Of recent note, the July 6, 2019 7.1 Mw Ridgecrest earthquake was about 73 miles northeast of Rosamond.

Seven historic seismic events (5.9 M or greater) have significantly affected the Antelope Valley in the last 70 years. They include:

- Fort Tejon (San Andreas) Earthquake – On January 9, 1857, a magnitude 8.0Mw earthquake occurred with an epicenter near Parkfield, California. The earthquake was felt throughout much of California with the greatest damage in Fort Tejon, California.
- Owens Valley (Sierra Nevada) – A magnitude 7.6Mw earthquake occurred on March 26, 1872, in Lone Pine, CA. This event caused massive structural damage, 56 injuries, and 27 deaths.
- Arvin-Tehachapi (White Wolf) – On July 21, 1952, a magnitude 7.5Mw earthquake occurred on the White Wolf fault near Wheeler Ridge. The town of Tehachapi suffered the greatest damage and loss of life.
- San Fernando – On February 9, 1971, a magnitude 6.6Mw earthquake occurred in the foothills of the San Gabriel Mountains in Southern California. The earthquake resulted in extensive damage to structures in parts of San Fernando and the Santa Clarita Valley. Uplift and other effects affected private homes and business. The epicenter of the earthquake was located near Soledad Junction. Strong motion accelerographs recorded

ground accelerations as high as 1.25g at Pacoima Dam near the epicenter of the earthquake. Some structures designed in accordance with the Building Code in affect at the time were extensively damaged.

- Northridge – The 1994 Northridge earthquake (6.7Mw) and related aftershocks significantly shook the Palmdale area. This event was produced by a buried thrust fault that underlies portions of the San Fernando Valley and the Santa Susana Mountains. No actual fault rupture associated with the main thrust faulting occurred at the surface. Primary fault rupture terminated approximately 3 to 4.3 miles (5 to 7 km) below the ground surface.
- Landers and Big Bear Earthquakes – Early on June 28, 1992, a magnitude 7.3Mw earthquake occurred near Landers, the largest seismic event in Southern California in 40 years. Surface rupture occurred just south of the town of Yucca Valley and extended some 43 miles toward Barstow. About three hours later, a magnitude 6.4Mw earthquake occurred near Big Bear Lake. No significant structural damage from these earthquakes was reported in the Antelope Valley area.
- Hector Mine Earthquake – On October 16, 1999, a 7.1Mw earthquake occurred on the Lavic Lake and Bullion Mountain faults north of Twentynine Palms. While this event was widely felt, no significant structural damage has been reported in the Antelope Valley.
- Ridgecrest Earthquake – The last major earthquake to affect the Lancaster area was the July 2019 Ridgecrest earthquake. The epicenter of this 7.1(Mw) earthquake was approximately 80 miles north-northeast of Lancaster. Surface rupture occurred near segments of the Little Lake fault zone near Ridgecrest. Geologic evaluation of this earthquake is underway.

One measure of ground shaking is intensity. The Modified Mercalli Intensity Scale of ground shaking ranges from I to XII with XII indicating the maximum possible intensity of ground movement. Structural damage begins to occur when the intensity exceeds a value of VI. It is estimated that the intensity of earthquake response was at least IX on the Modified Mercalli Scale during the Fort Tejon earthquake of 1957. Based on an AGU Geophysical Research Paper by Katherine M. Scharer of the USGS Earthquake Research Center, and Doug Yule of California State University Northridge (2020), historic and paleoseismic research data supports a magnitude 7.9 even for the southern San Andreas. Larger rupture events are not supported in the paleoseismic and historic records. As such we have utilized a magnitude 7.9 for our analyses.

It is assumed that the 2019 CBC and ASCE 7-16 guidelines will apply for the seismic design parameters used in design. The 2019 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The “general procedure” (i.e., probabilistic) seismic design parameters presented below were determined by the U.S. Seismic Design Maps "risk-targeted" calculator on the SEAOC/OSHPD website for ASCE 7-16 for the site coordinates (34.8661° North Latitude and 118.1831° West Longitude, Soil Site Class D (for stiff soils), for Occupancy (Risk) Category III (which includes public school structures). (A listing of the calculated 2019 CBC and ASCE 7-16 Seismic Parameters is presented below and again in Appendix A.)

Seismic Design Criteria: This site is subject to strong ground shaking due to potential fault movements along regional faults including the San Andreas and Garlock fault zones. Engineered design and earthquake-resistant construction increase safety and allow development of seismic areas. The minimum seismic design should comply with the 2019 edition of the California Building Code and ASCE 7-16 using the seismic coefficients given in the table below. The site is not within Alquist-Priolo or other hazard zones. A site-specific response spectrum is provided per CGS guidelines. General Procedure and site-specific seismic parameters are presented below considering a Site Class D (results in Appendix A). Our site-specific analysis considered the proximity of the San Andreas and Garlock faults and the results are listed in the Seismic Hazards section of this report. See Appendix A for tables.

Earthquake Performance Statement: Depending upon the extent of structural design of structures, exterior flatwork, walls, utilities, roadways, and other similar site improvements, some damage due to seismic events will occur. Note that all of southern California in general is in earthquake country. Site developments in southern California are typically not designed to mitigate anticipated seismic events without some damage. In fact, the Building Code is intended to provide Life-Safety performance, not complete damage-free design. In other words, some damage from earthquakes in the form of structural damage, settlement, cracking, and disruption of utilities can occur. It is not the current standard of care for site designers to fully mitigate all anticipated earthquake induced hazards.

Table 5
Summary of Seismic Parameters – 2019 CBC "General Procedure"

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	III
Seismic Design Category	D
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	1.097 g
Spectral Response Acceleration at 1 sec. – S_1	0.435 g
Site Coefficient – F_a	1.06
Site Coefficient – F_v	1.87
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.164 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	0.811 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	0.776 g
One Second Spectral Response – S_{D1}	0.541 g
Site Modified Peak Ground Acceleration - PGA_M	0.543 g
Values appropriate for a 2% probability of exceedance in 50 years	

The seismic factor S_1 is greater than 0.2 g and the Site Class is "D" which is based on an average blow count of 39. If the structural engineer determines that the exemptions of Section 11.4.8 of ASCE 7-16 do not apply, a site-specific (i.e., deterministic) ground motion hazard analysis is required. The site-specific study takes into account soil amplification effects. The United States Geological Survey (USGS, 2009) has undertaken a probabilistic earthquake analysis that covers the continental United States. A reasonable site-specific spectral response curve may be developed from USGS Unified Hazard Tool web page, which adjusts for site-specific ground factors. The interactive webpage appears to be a precise calculation based on site coordinates. For the purposes of this study, the Dynamic: Conterminous U.S. 2014 (Update) (Version 4.20) values have been chosen for use in the analysis. The seismic shear velocity (Vs) used in the analysis for Site Class D was 270 meters per second.

NGA West2 2014 attenuation relationships were used in the analyses. These attenuations included those of Kumai, Abrahamson, and Silva (2013), Boore, Stewart, Seyhand, and Atkinson (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014).

Table 6
Summary of Seismic Parameters – 2019 CBC "Site-Specific Procedure"

Site Class (ASCE 7-16)	D
Occupancy (Risk) Category	III
Seismic Design Category	D
Maximum Considered Earthquake (MCE) Ground Motion	
Spectral Response Acceleration, Short Period – S_s	1.097 g
Spectral Response Acceleration at 1 sec. – S_1	0.435 g
Site Coefficient – F_a	1.06
Site Coefficient – F_v	2.50
Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.350 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	1.105 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	0.900 g
One Second Spectral Response – S_{D1}	0.737 g
Site Modified Peak Ground Acceleration - PGA_M	0.550 g
Values appropriate for a 2% probability of exceedance in 50 years	

The Fault Parameters table in Appendix A lists the significant "active" and "potentially active" faults within an approximate 52-mile radius of the project site. The distance between the project site and the nearest portion of each fault is shown as well as the respective estimated maximum earthquake magnitudes.

Surface Fault Rupture: Surficial displacement along a fault trace is known as fault rupture. Fault rupture typically occurs along previously existing fault traces. No faults considered to be "active" or "potentially active" were observed to be located on or trending into the campus during the field study, during reviews of the referenced geologic literature, or during review of the aerial photographs taken of the site.

It should be noted that the Rosamond fault trends nearly east-west and near the campus (Jennings and Bryant, 2010, and Ken County Safety Element of the General Plan, 2009), but this fault was classified by the Jennings and Bryant work as a pre-Quaternary fault, which is also known as an inactive fault. As a result, it is the opinion of this firm that the potential for fault rupture on this site is low.

Landsliding and Rock Fall: The campus is nearly level. As a result, it appears that the hazards posed by landsliding and rock fall are nil.

3.7.2 Secondary Hazards

Secondary seismic hazards related to ground shaking include soil liquefaction, lateral spreading, seiches, and tsunamis.

Soil Liquefaction: Liquefaction is the loss of soil strength from sudden shock (usually earthquake shaking), causing the soil to become a fluid mass. In general, for the effects of liquefaction to be manifested at the surface, groundwater levels must be within 50 feet of the ground surface and the soils within the saturated zone must also be susceptible to liquefaction. Our exploration did not reveal a shallow groundwater table or a perched water table (despite decades of irrigation). Our research of nearby water well data indicates the groundwater table is deeper than 50 feet below the ground surface. Based on the above, the potential hazard posed by liquefaction at the site is considered "low".

Lateral Spreading: Lateral spreading (lateral migration) is caused when a loose and saturated soil layer is liquefied by a nearby earthquake of sufficient magnitude and a sloping ground, free face conditions, or sufficient building/embankment stresses (Yazdi, J.S. and Moss, R.E. S., 2016) exist near the site. Currently, these factors are not present at this site. The exploration and laboratory work revealed that the soils within 50 feet of the surface are not liquefiable. Therefore, lateral spreading at this site has a low potential.

Dry Seismic Settlement: The amount of dry seismic settlement is dependent on relative density of the soil, ground motion, and earthquake duration. In accordance with current CGS policy (Earth Systems discussion with Jennifer Thornburg, CGS May 2014), we used a site peak ground acceleration of ⅔ PGA_M (PGA_M = 0.55) and an earthquake magnitude of 7.9 to evaluate dry seismic settlement potential. The design peak ground acceleration values were obtained from the SEAOC online application (https://seismicmaps.org/) and adjusted per the site-specific evaluation. The reasoning for using ⅔ PGA_M has stemmed from the limitations of the Pradel (modified Tokimatsu and Seed) method of calculating dry seismic settlement. This issue manifested during the code change from ASCE 7-05 to ASCE 7-10 (2013) and has been continued into the ASCE 7-16 document, where it was no longer codified to evaluate liquefaction and seismic settlement as a design value ($S_{DS/2.5}$) but to evaluate it at full PGA. PGA_M equated to a maximum considered value and the design spectral response, S_{DS}. Is equal to ⅔ S_{MS}, where SMS

is the Site Class adjusted maximum considered earthquake value. When using the full PGA_M in Pradel's method, (Pradel's method is the Standard of Care methodology of calculating dry sand settlement), the potential settlements increased dramatically resulting in unreasonable amounts of assumed settlement (stress/strain curves defaulted to 10 percent vertical strain as high acceleration) which did not correlate with field observations of earthquake events. Given these unreasonably high potential settlements and after much discussion with agencies and CGS, the PGA used for dry seismic analysis was returned to a "design" value in practice. This is on the basis that since dry seismic shaking will typically not cause bearing failure (like liquefaction will) but is only settlement related deformation. Earth Systems is in concurrence with this methodology and thus the method utilized for our analysis and that ⅔ PGA_M may be used as an appropriate acceleration in the analysis of seismic settlement of unsaturated soils.

Based upon methods presented by Tokimatsu and Seed (1987), the potential for seismically induced dry settlement of soils for the depths explored was calculated for our borings at the site and estimated to be less than ½-inch (Boring B-1). Seismic settlement is based on post grading recommendations provided in Section 5.1. Due to the general uniformity of the soils encountered and the relative minor difference in settlement potential between the borings evaluated, seismic settlement is expected to occur on an areal basis and as such per Special Publication 117A (CGS, 2008), the differential settlement is estimated to be approximately ¼-inch considering the grading as recommended in Section 5.1.

Ground Subsidence: Subsidence is the gradual settling or sudden sinking of the Earth's surface due to subsurface movement of earth materials. It can result in the loss of aquifer storage, increased flooding, cracks and fissures at the land surface, damage to man-made structures, and associated economic costs. According to the Scientific Investigations Report 2014-5166 "Groundwater-Flow and Land-Subsidence Model of Antelope Valley, California" (Siade et al. 2014) the project site is not within an area of concentrated land subsidence (Plate 6). Therefore, the potential for subsidence settlement from groundwater removal is considered low. Changes in pumping regimes can affect localized groundwater depths, related cones of depression, and associated subsidence such that the prediction of where fissures might occur in the future is difficult. In the project area, groundwater depths remain fairly deep and we consider the current subsidence potential low. However, in the event of future nearby aggressive groundwater pumping and utilizations, the occurrence of deep subsidence cannot be ruled out. Changes in regional groundwater pumping could result in areal subsidence. The risk of areal subsidence in the future is more a function of whether groundwater recharge continues and/or over-drafting stopes, than geologic processes, and therefore the future risk cannot be predicted or quantified from a geotechnical perspective.

Slope Instability: The surface soils were found to be mostly silty sand. Correlations using Standard Penetration Testing and lab testing indicates the soils have an internal friction angle of 33 degrees. Any proposed 2(H):1(V) or steeper slopes have a high potential for surficial failures, the factor of safety being less than 1.0 when the slopes are saturated to a depth of 4 feet.

Flooding: According to the Flood Emergency Management Agency (FEMA) website, the project site lies within a zone defined by FEMA as Zone "X", which is defined as an area of 0.2% annual chance flood. Thus, the hazard posed by storm-induced flooding appears to be low.

The site is far inland, so the hazard from tsunamis is non-existent.

According to the Kern County Safety Element of the General Plan (2009), the campus is not within any inundation zones for reservoirs or seiche hazard zones. As a result, it appears that the hazards posed by dam failure and seiche are low.

Section 4
CONCLUSIONS

The following is a summary of our conclusions and professional opinions based on the data obtained from the field and laboratory testing, geotechnical analysis, and our experience with similar projects in the general area.

Based on our field exploration, laboratory testing, and geotechnical analyses conducted for this study, it is our professional opinion that the site is suitable, from a geotechnical standpoint, for construction of the improvements and buildings as proposed, provided the recommendations presented in this report are incorporated into project design and construction.

The recommendations presented in this report may change pending a review of final grading plans and foundation plans. Recommendations presented in this report should not be extrapolated to other areas or be used for other projects (beyond those expressly identified within) without our prior review and comment.

Geotechnical Constraints and Mitigation:

- The primary geologic hazard is moderate to severe ground shaking from earthquakes originating on regional southern California faults. A major earthquake originating on the nearby segments of the San Andreas or Garlock fault zones and other southern California faults would be the critical seismic events that may affect the site within the design life of the campus. Engineered design and earthquake-resistant construction increase safety and allow development within seismic areas.

- The underlying geologic condition for seismic design is Site Class D. The site is about 16.6 miles from the Garlock fault and about 17.3 miles from the San Andreas fault, which are "active" faults of significance. A qualified professional should design any permanent structure constructed on the site. The ***minimum*** seismic design should comply with the 2019 edition of the California Building Code.

- This site will have structures on Site Class "D" and the seismic coefficient S_1 (0.435) is greater than 0.2. A site-specific response spectrum is provided.

- The site is not within a currently designated Alquist-Priolo Earthquake Fault Zone or liquefaction zone. No known faults or obvious lineaments cross the site. Therefore, the potential for surface fault rupture at the site is considered very low.

- The potential for ground subsidence and liquefaction settlement hazards are considered to be very low to low for this project. The site is not within an area of documented areal subsidence due to groundwater. Dry seismic settlement potential is low.

- Other geologic hazards, including flooding, and landslides, pose low or negligible potential hazards to the proposed project.

- Based on current conditions, groundwater is not anticipated to be encountered during construction, however moist soils may be encountered.

- Much of the existing on-site fill and alluvial soils are very low in expansion potential and suitable for location under structures or hardscape after remedial grading. Building structure recommendations provided within are based upon using a very low to low in expansion potential fill material for the building pad.

- Hydro-consolidation is a potential hazard in this site. Adherence to the grading and structural recommendations in this report should reduce potential settlement problems from seismic forces, increased wetting of the soil profile, and the weight of the intended structures.

- Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1H:1V from the projection upward from the bottom of any foundation, whichever is greater.

- Laboratory testing of one sample showed potentially “Severely” corrosivity to buried metallic elements and “negligible” for sulfate exposure to concrete. See Section 3.5 for further information. Site soils should be reviewed by an engineer competent in corrosion evaluation.

- In our professional opinion, structures can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on “very low” expansion category soils.

- Due to the project complexity, this report and its recommendations are contingent on a structural and grading plan review by Earth Systems to confirm or modify applicability.

Section 5
RECOMMENDATIONS

5.1 Site Development and Grading

A representative of Earth Systems should observe site clearing, grading, and the bottoms of excavations before placing fill. Local variations in soil conditions may warrant increasing or decreasing the depth of recompaction and over-excavation. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process, to verify that our geotechnical recommendations have been properly interpreted and implemented during construction and is required by the 2019 California Building Code. Preventative measures to reduce seasonal flooding and erosion should be incorporated into site grading plans. Dust control should also be implemented during construction. Site grading should be in strict compliance with the requirements of the local Air Quality Management District.

Observation of fill placement by the Geotechnical Engineer of Record should be in conformance with Section 17A of the 2019 California Building Code. California Building Code requires full-time observation by the geotechnical consultant during site grading (fill placement). Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to provide testing and observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. Additionally, the California Building Code requires the testing agency to be employed by the project owner or representative (i.e., architect) to avoid a conflict of interest if employed by the contractor. Unless noted otherwise, grading should be performed in general accordance with Appendix J of the 2019 CBC.

Clearing and Grubbing: At the start of site grading, existing vegetation, trees (including the entire root ball), large roots, overly wet and/or soft soil, undocumented fill, pavements, foundations, construction debris, septic tanks, leach fields, deleterious material, trash, and abandoned underground utilities should be removed from the proposed building areas. Organic growth should be stripped off the surface and removed from the construction area. Areas disturbed during demolition and clearing should be properly backfilled and compacted as described below.

Buried utilities may be located in the vicinity of the planned structures and within other areas of the project site. All buried structures which are removed should have the resultant excavation backfilled with soil compacted as engineered fill described herein or with a minimum 2-sack cement-sand slurry (minimum of 100 psi compressive strength) approved by the project geotechnical engineer. Abandoned utilities should be removed entirely, or pressure-filled with concrete or grout and be capped. Abandoned buried utilities structures, or foundations should not extend under building limits.

After stripping and grubbing operations, areas to receive fill should be stripped of loose or soft earth materials until a firm subgrade is exposed, as evaluated by the geotechnical engineer or geologist (or their representative). Before the placement of fill or after cut, the existing surface soils within the building pads and improvement areas should be over-excavated as follows:

Structure Building Pad Preparation: Due to the estimated settlements from collapse soils analysis and assumed structural loading on the soil profile, the existing soils within the building pad and foundation (including covered eating area) will need over-excavation for use as a soil mitigation measure. We have provided recommendations in this report for soil densification (mat of compacted fill) which is one of the accepted methods of reducing localized differential settlement provided in SP117A (2008, page 57) and *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California*, Martin and Lew, 1999.

The over-excavation should extend a minimum of 4 feet below existing or finished grade, or 2 feet below the bottom of foundations, whichever is deeper. The over-excavation should extend for at least 5 feet beyond the outer edge of the building pad or the depth of overexcavation, whichever is greater in length, and include all exterior footings or slabs and include any overhead canopy/walkway areas, where possible. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. A sample of the excavation bottom should be evaluated for maximum density (ASTM D 1557). Deeper over-excavation may be recommended if the over-excavation bottom densities are not met. Once the bottom subgrade is attained and approved, the surface should be scarified an additional 6 inches, moisture conditioned to 3% above the optimum moisture content, and recompacted to a minimum of 90% compaction relative to ASTM D 1557.

After the approved subgrade has been scarified, moisture conditioned, and compacted as required above, engineered fill should be placed in 8-inch loose lifts at 3% above the optimum moisture content and compacted to at least 90% relative compaction (ASTM D 15557) prior to the placement of subsequent lifts until the remaining pad elevation is achieved.

Utilities and lifeline services provided from outside the structure could still suffer disruption unless mitigation measures are employed that would account for the soil deformations that could occur between the structure and the supporting soils.

Auxiliary Structures Subgrade Preparation: Auxiliary structures such as fence or retaining walls, etc. should have the foundation areas overexcavated similar to Building Pad Preparation section noted described above. The overexcavation should extend laterally a distance of 3 feet from the edge of footings. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. The exposed soils should then be moisture conditioned to 3% above the optimum moisture content, and recompacted to at least 90 percent relative compaction (ASTM D 1557). Moisture conditioned engineered fill may then be placed to finished subgrade in suitable, compacted lifts. Compaction should be verified by testing.

Masonry fence walls should include construction joints every 10 feet to minimize the potential for distress cracks.

Subgrade Preparation: In areas to receive fill, but not supporting structures or hardscape, the subgrade should be scarified; moisture conditioned and compacted to at least 90% relative

compaction (ASTM D 1557) for a depth of 1 foot below existing grade, or finished subgrade, whichever is deeper. Compaction should be verified by testing.

Hardscape Area Preparation (Including non-structure building areas which have increased settlement sensitivity): In hardscape areas, the subgrade should be over-excavated a minimum depth of 1 foot below existing grade or finish grade (whichever is deeper) and to a lateral extent of 2 feet beyond the outside edge. The excavation bottom should be flat across the pad. The exposed and undisturbed bottom of the over-excavation should be observed and tested by the geotechnical engineer or their representative to verify that in-place densities are at least 85% relative compaction. The excavation bottom should be scarified 6 inches, moisture conditioned to 3% over the optimum moisture content and be recompacted to at least 90% relative compaction. Engineered fill should then be moisture conditioned, placed in suitable lifts, and compacted to a minimum of 90% relative compaction to finish grade prior to the placement of subsequence lifts, with the upper 1 foot compacted to at least 95% relative compaction in parking drive and track areas. Compaction should be verified by testing. Re-leveling may be required if soil collapse occurs in hardscape areas due to nearby irrigation. Irrigation is recommended to be minimal adjacent to hardscape.

Slope Construction: Please see Section 5.5 for detailed slope preparation recommendations.

All over-excavations should extend to a depth where the project geologist, engineer or his/her representative has deemed the exposed soils as being suitable for receiving compacted fill. The materials exposed at the bottom of excavations should be observed by a geotechnical engineer or geologist from our office prior to the placement of any compacted fill soils to verify that a competent bottom has been attained. Additional removals may be required as a result of observation and/or testing of the exposed subgrade subsequent to the required over-excavation.

Engineered Fill Soils: The existing fill and native soils when processed appropriately are considered to be suitable for use as engineered fill. Engineered fill should be generally free from expansive soil (Expansive defined as Expansion Index (EI) greater than 20), vegetation, trash, large roots, overly wet and/or soft soil, clods larger than 3 inches, construction debris, oversized rock (greater than 6 inches) and other deleterious material as determined by the geotechnical engineer or his representative. Deleterious materials should be hauled offsite. Engineered fill soils should have a “very low” Expansion Index.

Engineered fill (and any import) should be placed in maximum 8-inch lifts (loose) and compacted to at least 90 percent relative compaction (ASTM D 1557)(unless noted otherwise) at 3% above the optimum moisture content prior to placement of a subsequent loose lift. Within pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction (ASTM D 1557). Compaction should be verified by testing. Rocks larger than 6 inches in greatest dimension should be removed from fill or backfill material, with the exception of playfield areas, where criteria necessitating a smaller oversize allowance may apply. Typically, in play field areas, the maximum oversize allowed is 1 inch.

Imported fill soils should be “very low” expansion potential granular soils meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches and 5 to 35 percent passing the No. 200 sieve (unless otherwise approved by the geotechnical engineer). The geotechnical engineer should evaluate the import fill soils before hauling to the site. However,

because of the potential variations within the borrow source, import soil will not be prequalified by Earth Systems.

A program of compaction testing, including frequency and method of test, should be developed by the project geotechnical engineer at the time of grading. Acceptable methods of testing may include Nuclear methods such as those outlined in ASTM D 6938 (Standard Test Methods for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods). Alternative methods may include methods outlined in ASTM D 1556 (Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method), or correlated hand probing to verify firm conditions.

All soils should be moisture conditioned prior to application of compactive effort and prior to foundation, slab-on-grade and pavement placement. Moisture conditioning of soils refers to adjusting the soil moisture to 3% above the optimum moisture content. If the soils are overly moist so that instability occurs, or if the minimum recommended compaction cannot be readily achieved, it may be necessary to aerate to dry the soil to 3% above the optimum moisture content or use other means to address soft soils (as approved by the geotechnical engineer prior to use).

Shrinkage and Bulking: The shrinkage factor for earthwork for the alluvial materials is expected to range from 5 to 10 percent for the upper excavated or scarified site soils. This estimate is based on compactive effort to achieve a weighted average relative compaction of about 93 percent.

Shrinkage is highly dependent on and may vary with contractor methods for compaction. Losses from site clearing, oversize rock removal, and removal of existing site improvements, as well as the addition of excavated soil (footings, piers, etc.) may significantly affect earthwork quantity calculations and should be considered.

Dust Control: The proposed site lies within an area of moderate potential for wind erosion. The site soils have a fine-grained component of their composition. As such, exposed soil surfaces may be subject to disturbed fine particulate matter (PM_{10}) which can create airborne dust if the soil surface or roadways are not maintained. During construction, watering the soil surface can reduce airborne dust. Alternatively, a dust control palliative may be spray applied to the soil surface to act as a tackifier to contain loose soil particles. Palliatives must be reapplied periodically as they weather and degrade. Further guidance for dust palliatives can be found in reviewing the United States Department of Agriculture Publication *Dust Palliative Selection and Application Guide*, Document No. 9977-1207-SDTDC. The recommended soil input parameters are Plasticity Index <8, and fines content between 10 and 25 percent, for typical onsite fills.

5.2 Excavations and Shoring

Excavations should be made in accordance with Cal/OSHA requirements. Using the Cal/OSHA standards and general soil information obtained from the field exploration, classification of the near surface on-site soils will likely be characterized as Type C. Actual classification of site-specific soil type per Cal/OSHA specifications as they pertain to trench safety should be based on real-time observations and determinations of exposed soils by the contractors *Competent Person* (as defined by OSHA) during grading and trenching operations.

Our site exploration and knowledge of the general area indicates there is a moderate potential for caving and sloughing of site excavations (over excavation areas, utilities, footings, etc.) due to dry and also overly moist/wet conditions. Where excavations in soils over 4 feet deep are planned, lateral bracing or appropriate cut slopes of 1.5:1 (horizontal/vertical) should be provided. No surcharge loads from stockpiled soils or construction materials should be allowed within a horizontal distance measured from the top of the excavation slope and equal to the depth of the excavation. Excavations should be protected from water flow over the exposed surface and saturation.

Excavations which parallel structures, pavements, or other flatwork, should be planned so that they do not extend into a plane having a downward slope of 1:1 (horizontal: vertical) from the bottom edge of the footings, pavements, or flatwork. Shoring or other excavation techniques may be required where these recommendations cannot be satisfied due to space limitations or foundation layout. Where overexcavation will be performed adjacent to existing structures, ABC slot cutting techniques may be used as pre-approved by the project geotechnical engineer.

Shoring: Shoring may be required where soil conditions, space, or other restrictions do not allow a sloped excavation or slot cutting is not an option. A braced or cantilevered shoring system may be used. Trench boxes should not be placed below or within the pipe zone elevation as their removal may loosen compacted backfill. Positive trench shoring may be required (jacks and plates).

A temporary cantilevered shoring system should be designed to resist an active earth pressure equivalent to a fluid weighing as shown in the table below. Braced or restrained excavations above the groundwater table should be designed to resist a uniform horizontal equivalent soil pressure as presented in the table below.

Table 7
Temporary Cantilevered and Braced Shoring System Parameters

Equivalent Fluid Pressure pounds per cubic foot (pcf)	
Cantilevered	**Braced**
40	60

The values provided above assume a level ground surface adjacent to the top of the shoring and do not include a factor of safety. Fifty percent of an aerial surcharge placed adjacent to the shoring may be assumed to act as an additional uniform horizontal pressure against the shoring. Special cases such as combinations of slopes and shoring or other surcharge loads may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical or shoring engineer on a case-by-case basis. Retaining walls subjected to traffic loads should include a uniform surcharge load equivalent to at least 240 psf for auto or delivery truck (2 axle) traffic kept at least 3 feet from the back of the wall. Retaining walls with closer traffic or heavier traffic loads should be designed for a 450 psf surcharge load. Retaining walls should be designed with a minimum factor of safety of 1.5.

The wall pressures above the groundwater do not include hydrostatic pressures; it is assumed that drainage will be provided. If drainage is not provided, shoring extending below the groundwater level should be evaluated on a case-by-case basis.

Cantilevered shoring must extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. We recommend required embedment depths be determined using methods for evaluating sheet pile walls and based on the principles of force and moment equilibrium. For this method, the allowable passive pressure against shoring, which extends below the level of excavation, may be assumed to be equivalent to a fluid weighing 400 pcf. Additionally, we recommend a factor of safety of at least 1.2 be applied to the calculated embedment depth and that passive pressure be limited to 2,000 psf.

The contractor should be responsible for the structural design and safety of all temporary shoring systems. The contractor should carefully review the exploration logs in this report, and perform their own assessment of potential construction difficulties, and methods should be selected accordingly. Shoring should be sealed to prevent the piping of soil material and potential soil loss conditions which can cause settlement. The method of excavation and support is ultimately left to the contractor.

5.3 Utility Trenches and Backfill

Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.). Utility trench backfill within private property should be placed in conformance with the provisions of this report. Backfill operations should be observed and tested to monitor compliance with these recommendations.

Trench Width and Vertical Loads on Pipelines: Vertical loads to the pipeline are highly dependent upon the geometry of the trench. In general, the narrower the trench is at the top of the pipe/conduit with respect to the diameter of the conduit, the less vertical load is applied to the conduit. This is because as the trench backfill and bedding compress or consolidate over time, the weight of the soil mass is partially offset by the frictional resistance along the trench sidewalls. In addition, the type of bedding supporting the pipeline affects the bearing strength of the conduit. This is accounted by a load factor that is multiplied to the design strength of the conduit. The pipe manufacturer recommendations for trench installation and maximum width should be followed to reduce the potential for overloading the pipe due to excess backfill load.

Pipe Subgrade and Bedding: Pipeline subgrade should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative for a depth of 6 inches below any bedding. Bedding material shall consist of sand with 100 percent passing a No. 4 sieve and less than 5 percent fines (passing a No. 200 sieve), and a sand equivalent of 30 or more or as approved by the project inspector and geotechnical engineer.

Pipe-Zone, Trench–Zone, Trench Backfill and Compaction: Backfill of utilities should be placed in conformance with the requirements of the specifications. Backfill of utilities within roads or public right-of-ways should be placed in conformance with the requirements of the governing agency (water district, public works department, etc.).

Pipe-zone backfill material (the pipe area from the bedding to 12 inches above the top of pipe) may consist of sandy native soils screened to a ¾" maximum particle size or import sand (as described above for bedding) as dictated by the pipe designer or manufacturer. The pipe zone backfill material should be placed in maximum 8-inch lifts (loose) and compacted near its optimum moisture content prior to the placement of subsequent lifts. Pipe zone backfill should be compacted to a minimum of 90% relative compaction (ASTM D 1557) or to a firm condition as evaluated by the geotechnical engineer or his representative. Compaction should be assured in the pipe haunches.

The native soil is suitable for use as trench zone and street zone (and manholes) backfill (from the top of pipe zone up to finished grade), provided it is free of significant organic or deleterious matter and oversize materials. This backfill shall contain no particles larger than 3 inches in greatest dimension. The final backfill material should be placed in maximum 8-inch lifts (loose) and compacted to at least 90% relative compaction (ASTM D 1557) at 3% above the optimum moisture content for the trench zone and 95% for the street zone (upper 12 inches) when below pavement. Compaction or firmness should be verified by testing.

Backfill materials should be brought up at substantially the same rate on both sides of the pipe or conduit. Reduction of the lift thickness may be necessary to achieve the above recommended compaction. Care should be taken to not overstress the piping during compaction operations. Mechanical compaction is recommended; ponding or jetting is not recommended.

Alternatively, if the utility cannot accommodate the increased stress, or if compaction is difficult, we recommend the pipe be encased by at least 1 foot of 2-sack cement-sand slurry (at least 1 foot as measured from the top of pipe). Backfill operations should be observed and tested to monitor compliance with these recommendations.

In general, coarse-grained sand and/or gap graded gravel (i.e., ¾-inch rock or pea-gravel, etc.) should not be used for pipe or trench zone backfill due to the potential for soil migration into the relatively large void spaces present in this type of material and water seepage along trenches backfilled with coarse-grained sand and/or gravel. Gravel should be separated from backfill with a filter fabric such as Mirafi 140N or equivalent as approved by the soils engineer. Water seepage or soil migration will cause settlement of the overlying soils.

Compaction should be verified by testing. Backfill operations should be observed and tested to monitor compliance with these recommendations. Trench backfill compacted per these requirements can be expected to settle 0.1 to 0.3 percent of the trench depth. This can cause an elevation difference between backfilled trenches and the surrounding soil or pavement. Increased relative compaction can reduce the settlement if the potentials presented are not acceptable. The geotechnical engineer should be consulted on a case-by-case basis to provide further recommendations to reduce the settlement potential.

STRUCTURES

In our professional opinion, structure foundations can be supported on shallow foundations bearing on a zone of properly prepared and compacted soils placed as recommended in Section 5.1. The recommendations that follow are based on "very low" expansion category soils.

5.4 Foundations

Footing design of widths, depths, and reinforcing are the responsibility of the Structural Engineer, considering the structural loading and the geotechnical parameters given in this report. A minimum footing depth of 12 to 18 inches (below lowest adjacent grade) should be maintained and considers a bearing soil with a "very low" expansion potential. Lowest adjacent grade is the lowest grade within 5 feet laterally of the footing edge. A representative of Earth Systems should observe foundation excavations to verify compaction (minimum 90% per ASTM D 1557) before placement of reinforcing steel or concrete. Loose soil or construction debris should be removed from footing excavations before placement of concrete. *All footing excavations should be probed for uniformity. Soft or loose zones should be excavated and recompacted to finish foundation bottom subgrade. The bottom of all foundations should be tested to confirm compaction effort and moisture contents as stated in Section 5.1 of this report are met.* The moisture contents should be at least the indicated moisture content 24 hours prior to and immediately prior to placing concrete for a depth of at least 12 inches below the foundation subgrade. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Minimum Slope Setback for Foundations: Earth Systems recommends a minimum setback distance of 5 feet. The 2019 California Building Code provides setback distances for foundations along slopes. Setback distances are measured differently for foundations located above the slope and those located below the slope. For foundations located at the top of the slope, the measurement is taken horizontally from the outside face of the foundation footing to the face of the slope. For foundations located below the slope, the horizontal distance is measured from the face of the structure foundation to the toe of the slope. For pools and slopes steeper than 1H:1V (horizontal to vertical), please contact Earth System for these setbacks with submittal of detailed information using plan form.

Conventional Spread Foundations: Allowable soil bearing pressures are given below for foundations bearing on recompacted soils as described in Section 5.1. Allowable bearing pressures are net (weight of footing and soil surcharge may be neglected).

- Continuous foundations, 1-foot minimum width and 3-foot maximum width embedded a minimum of 12 inches below finished grade:

  1,500 psf for dead plus design live loads

- Pad foundations, 2 x 2-foot minimum and 4 x 4-foot maximum in plan embedded a minimum of 18 inches below finished grade:

  2,000 psf for dead plus design live loads

- Foundations at Grade, 12-inch minimum width, 600 psf for Dead & Live loads. Bearing pressure is based on a soil having a minimum Soil Friction Angle (Phi) of 33 degrees with a dry unit weight 123 pcf or greater.

As the allowable bearing pressure is settlement governed, one-third (⅓) increase in the allowable bearing pressure may be used when calculating resistance to wind or seismic loads.

If the actual structural loads exceed the assumed values stated in Section 1.1 (100 kips for isolated pad footings and 3 kips/linear foot for continuous wall footings), the geotechnical engineer must reevaluate the allowable bearing values as the allowable bearing was controlled

by the allowable total differential settlement from dry seismic, collapse, and static loads. Underground utilities should be designed for an anticipated settlement within the building areas.

The spacing between any large spread footings should be evaluated by the geotechnical engineer during the plan review stage to confirm or modify the settlement estimates and bearing capacity due to large footings and the influences from adjacent footings. A preliminary analysis suggests spacing the footings (adjacent edge to adjacent edge) a lateral distance from one another of the width of the largest footing from any adjacent footing, such that influence effects are minor. A 1:1 projection downward from the bottom edge of footings may be considered in the evaluation of foundation influence zone.

Maximum foundation sizes given above are based on settlement due to Dead + Live loads. Transient loads such as earthquake or wind loads are not subject to the stated size limitations; however, the allowable bearing pressure (including ⅓ increase) should be followed considering the relevant foundation sizes given above.

An average modulus of subgrade reaction, k, of 150 pounds per cubic inch (pci) can be used to design lightly loaded footings, beams, pavement, and slabs founded upon compacted fill. Other foundations such as mat slabs, will require the use of differing modulus of subgrade reaction values than used for lightly loaded slabs. Please contact Earth Systems for k values used for mat foundations.

The table below is based upon the above presented allowable, short term, and ultimate bearing pressures. Short Term allowable bearing may use the values presented below (based on Allowable Stress Design) or be based on Code mandated structural reductions, whichever is less. Ultimate bearing capacities consider a factor of safety of 3 (ASD design) to control settlement and bearing failure considering deep groundwater (4,500 to 6,000 psf ultimate) and a safety factor of 2.25 on transient loads (2,000 to 2,667 psf). Ultimate bearing to soil failure depends on foundation size and could be greater than 6,000 psf. The restrictions of Section 1605A.1.1 apply to the cited bearing values for Allowable Stress Design (ASD).

Table 8

	Allowable Bearing Capacity (psf) (FS = 3)	**Short Term (Wind/Seismic) (FS = 2.25)**	**Ultimate Bearing Capacity (FS = 1)**
Continuous Foundations	1,500	2,000	4,500
Isolated Pad Foundations	2,000	2,667	6,000

FS = Factor of Safety

Footings should be designed and reinforced by the structural engineer for the specific loading, settlement, or collapse soil conditions defined herein.

Stepped foundations should be designed in accordance with the 2019 CBC. CBC 2019 and ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength. Seismic Design Category for compressive strength determination is 'D'. Due to the negligible sulfates in the site soils, normal cements may be and should be proportioned in accordance with ACI recommendations considering the time of year for placement. Hot weather proportions should be used during high ambient heat days during placement and curing.

Expected Settlement: Estimated total static settlement should be less than ½ inch, based on footings founded on compacted soils as recommended. Differential static settlement between similarly loaded members should be less than ¼ inch.

As such, considering static settlement applied over a typical foundation distance of 30 feet, we recommend the structural engineer design for the typical angular distortion of 1:480 (¾ inch in 30 feet). Settlement will not result in the complete loss of soil support but will be manifested as a tilting of the structure over the applied distance.

We estimate that up to ½ of the estimated collapse settlement could occur (¼ inch of differential collapse settlement with mitigation).

Estimated total static differential (¼ inch), seismic differential (¼ inch), and collapse differential (¼ inch) settlement should be less than ¾ inch, based on footings founded on compacted soils as recommended.

As such, considering static, seismic, and collapse settlement we recommend the structural engineer design for an angular distortion of 1:360. Settlement will not result in the complete loss of soil support but will be manifested as a tilting of the structure over the applied distance. The estimated differential settlement does not exceed the Differential Threshold of ASCE7-16 Table 12.13-3 for Risk Category III, "Other single-story structures".

Settlement calculations are presented in Appendix A and collapse results are provided in Section 3.3. The actual settlement of large spread footings should be evaluated by the geotechnical engineer during the plan review stage based on the actual column loads to confirm or modify the settlement estimates presented.

Minor Structures Deep Foundations: Although no specific elements were identified by the architect, for miscellaneous structural components such as light poles, small shade structures, gate posts, temporary retaining walls, and flag poles, may be supported on cast-in-place piles, or direct embed in drilled holes filled with concrete, and the design be based on parameters presented in the subsequent sections of this report. Construction employing poles or posts may utilize design methods presented in Section 1806A and 1807A of the CBC for Silty Sand (SM) material class and utilize the strength parameters of Table 1806A.2. For designs utilizing allowable frictional resistance, Earth Systems recommends the use of Section 1810A.3.3.1.4 and 5 of the CBC. For piles with an axial and/or lateral load, these design methods apply for piles spaced at least 3 and 6 pile diameters, respectively, center to center as graded in accordance with Section 5.1. Piles spaced closer than these limits could have soil strength reduction and should be evaluated on a case-by-case basis by geotechnical engineer. Piles should be designed for frictional resistance only.

For piers founded in areas with native soil at the surface, an additional 1.5 feet should be added to the calculated pile embedment due to the potential effects of long-term surficial disturbance and erosion. Additionally, where piers are constructed adjacent to the tops of slopes, there should be a minimum distance between the top of the slope and the closest edge of the pier of H/3, where 'H' is the height of the slope, otherwise a lateral resistance reduction must be applied. For piers founded closer than a distance H/3 to the crest or within the slope area itself, the calculated lateral resistance of the soil should be reduced by 30 percent. The above

recommendations have considered slopes no steeper than 2H:1V (horizontal to vertical). Piles within H distance from a slope or within a slope (where H is the slope Height) slopes will require additional analysis by the geotechnical engineer and may change the recommendations presented.

Drilled piers should have a minimum clearance of 3 inches between the embedded post and the soil side wall to allow for adequate placement and flow of concrete. Drill holes may end up oversize. Casing or other means may be required in a drilled hole. Any "slough" or loose soils at the bottom of the shaft must be removed or tamped prior to setting rebar cages and placing concrete. Extreme care must be exercised to carefully position reinforcing steel cages and place concrete without disturbing the sidewalls of the drilled shafts. We recommend centralizers be used to positively locate rebar cages within the pier shaft. It is recommended that pier excavations that have not received concrete, not be left open and concrete should be placed immediately. Caving is a high concern.

Normally, drilled pier excavations should be made without the use of water. If necessary, water may be used to facilitate removal of cuttings unless it aggravates caving problems. Added water that may accumulate at the bottom of the hole should be removed from the drilled hole prior to placing the concrete. Sidewalls which have softened from the addition of water should be cleaned of the soft/loose material. Each excavation should be completed in a continuous operation and the concrete should be placed without undue delay. The contractor should use appropriate means to clean the bottom of the excavation so that no loose material is present at the base of the pier. We do not recommend over drilling beyond specified pier tip elevations to eliminate the need for bottom cleaning in order to account for slough or loose materials at the excavation bottom. To reduce the potential for caving and sidewall sloughing which may contaminate concrete during placement, and segregation, concrete should be placed by tremie methods and not directly chute-dumped into the hole.

Where casing is used with drilled holes and cannot be withdrawn, the skin friction capacity is theoretically reduced, as are passive resistance and stiffness. The amount of reduction is subject to assessment by the geotechnical consultant. The use of casing with drilled holes should be approved prior to use by the geotechnical engineer.

If casing is required, it should be withdrawn as the concrete is being placed, maintaining a 3-foot minimum head of concrete within the casing. This is to prevent reduction in the diameter of the drilled shaft due to earth pressure on the fresh concrete and to prevent extraneous material from falling in from the sides and mixing with the concrete. Concrete placement should continue in this manner until suitable concrete extends to the top of the excavation or forms. The upper eight feet of the pier should be consolidated by vibratory means.

Pier capacity is greatly dependent on the soil conditions at the location of the pier and upon contractor means and methods of placement. It is recommended that drilling operations and concrete placement be performed in the continuous presence of the geotechnical consultant or his representative to confirm that suitable materials for pier support are penetrated, that the dimensions of the installed piers meet the design dimensions, and that the installation has been performed as specified by the 2019 California Building Code. Observation during drilling is required by the 2019 California Building Code on a full-time basis by the geotechnical engineer or their representative. If subsurface conditions noted during drilled pier installation are

significantly different than those encountered in our borings, it may be necessary to adjust the overall length of the pier.

Prior to the placement of steel, and again prior to and during the placement of concrete, the excavation must be examined by the geotechnical consultant before proceeding with construction. The contractor should provide all aid and assistance required by the geotechnical and geologic consultants for field monitoring of the drilled pier operations.

Piers are accepted or rejected based on visual observation and testing during construction. The contractor should not allow nor cause any of this work to be permanently enclosed or covered up until it has been observed, tested, and accepted by the geotechnical engineer and all legally constituted authorities having jurisdiction.

5.5 Slope Construction

Slopes are not generally proposed for this project; however, minor slopes (less than 5 feet in height) may be constructed. Compacted fill slopes protected against erosion (per approved methods such as significant planting, facing, or erosion blankets, etc.) may be constructed at 2H:1V (horizontal to vertical) or flatter inclinations. Unprotected slopes with exposed native soils or compacted fill at the surface should be expected to require repair after heavy nuisance or storm runoff occurs due to significant erosion. Slope recommendations may change pending a more in-depth geotechnical evaluation once design plans are developed.

Compacted fill should be placed at 3% above the optimum moisture content and compacted to a minimum 90 percent of the maximum dry unit weight, as measured in relation to ASTM D 1557 test procedures. The exposed face of any cut or fill slope (upper 12 inches) should have a minimum relative compaction of 90 percent, as measured in relation to ASTM D 1557 test procedures and be compacted at 3% above the optimum moisture content. Due to the erodible site soils, slope faces should be protected with facing or densely spaced vegetation to reduce the erosion potential.

Surficial Slope Failures: All slopes will be exposed to weathering, resulting in decomposition of surficial earth materials, thus potentially reducing shear strength properties of the surficial soils. In addition, these slopes become increasingly susceptible to rodent burrowing. As these slopes deteriorate, they can be expected to become susceptible to surficial instability such as soil slumps, erosion, soil creep, and debris flows. Development areas immediately adjacent to ascending or descending slopes should address future surficial sloughing of soil material and erosion. Such measures may include debris fences, slope facing, catchment areas or walls, diversion ditches or berms, soil planting, velocity reducers or other techniques to contain soil material away from developed areas and reduce erosion. Additionally, foundations should be set back at least 5 feet from the face of slope (to daylight) or as per the 2019 CBC, whichever is greater.

Operation and maintenance inspections should be done after a significant rainfall event and on a time-based criterion (annually or less) to evaluate distress such as erosion, slope condition, rodent infestation burrows, etc. Inspections should be recorded, and photographs taken to document current conditions. The repair procedure should outline a plan for fixing and maintaining surficial slope failures, erosional areas, gullies, animal burrows, etc. Repair methods

could consist of excavating and infilling with compacted soil erosional features, track walking the slope faces with heavy equipment, as determined by the type and size of repair. These repairs should be performed in a prompt manner after their occurrence. Slope inclinations should be maintained, and a maintenance program should include identifying areas where slopes begin to steepen. Where future maintenance is not possible, slopes should be faced to reduce the erosion and degradation potential.

Slope faces are highly erodible even if compacted and will gradually erode and move down slope presenting maintenance issues and debris deposited in drainage devices and flatwork areas. The minimum material necessary to support landscaping should be specified by the landscape consultant (typically less than 6 inches).

5.6 Slabs-on-Grade

Subgrade: Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with Section 5.1 of this report. The moisture content below slabs should be at least 3% above the optimum moisture content 24 hours prior to and immediately prior to placing concrete for a depth 12 inches. If the moisture condition is less than indicated, it shall be brought up to or above the indicated moisture content.

Vapor Retarder: In areas of moisture-sensitive floor coverings, coatings, adhesives, underlayment, goods or equipment stored in direct contact with the top of the slab, bare slabs, humidity-controlled environments, or climate-controlled cooled environments, an appropriate vapor retarder that maintains a permeance of 0.01 perms or less after ASTM E1745's mandatory conditioning tests should be installed to reduce moisture transmission from the subgrade soil to the slab. For these areas, a vapor retarder (Stego wrap 15-mil thickness or equal) should underlie the floor slabs. If a Class A vapor retarder (ASTM E 1745) is specified, the retarder can be placed directly on non-expansive soil and be covered with a minimum 2 inches of clean sand.

Clean sand is defined as well or poorly graded sand (ASTM D 2488) of which less than 5 percent passes the No. 200 sieve and all the material passes a No. 4 sieve. The site soils do not fulfill the criteria to be considered clean sand. Alternatively, the slab designer may consider the use of other vapor retarder systems that are recommended by the American Concrete Institute.

Low-slump concrete should be used to help reduce the potential for concrete shrinkage. The effectiveness of the membrane is dependent upon its quality, the method of overlapping, its protection during construction, the successful sealing of the membrane around utility lines, and sealing the membrane at perimeter terminations and of all penetrations. Capillary breaks, if any, beneath slabs should consist of a minimum of at least four inches of permeable base material (Caltrans Class 2 Permeable Base) with the following specified gradation.

Table 9
Percent Passing Sieve Size

Sieve Size	Percent Passing
1 inch	100
¾ Inch	90-100
3/8 Inch	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Where vapor retarders are placed directly on a gravel capillary break, they should be a minimum of 15-mil thickness.

Where the 2-inch sand layer overlying the vapor retarder is omitted, and concrete is placed directly on the vapor retarder "plastic", proper curing techniques are essential to minimizing the potential of slab edge curl and shrinkage cracking. The edges of slabs can curl upward because of differential shrinkage when the top of the slab dries to lower moisture content than the bottom of the slab. Curling and cracking are caused by the difference in drying shrinkage between the top and bottom of the slab. Curling and cracking can be exacerbated by hot weather, or dry condition concrete placement, even with proper curing techniques.

The following minimum slab recommendations are intended to address geotechnical concerns such as potential variations of the subgrade and are not to be construed as superseding any structural design. A design engineer should be retained to provide building specific systems to handle subgrade moisture to ensure compliance with SB800 with regards to moisture and moisture vapor.

Slab Thickness and Reinforcement: Slabs should be a minimum of 4 inches in actual thickness and be reinforced with # 3 bars at 18 inches on center both ways. Slabs in contact with earth should use closer joints to control cracking or be thickened to allow adequate earth to rebar clearance. Reinforcing bars should extend at least 40 bar diameters into the footings and slabs. Concrete slabs-on-grade and flatwork should be supported by compacted and moisture conditioned soil placed in accordance with this report. If slabs are structural, they should be designed for the specific settlement conditions presented within.

Slab thickness and reinforcement of slabs-on-grade are contingent on the recommendations of the structural engineer or architect and the Expansion Index of the supporting soil. Based upon our findings, a modulus of subgrade reaction of approximately 150 pounds per cubic inch can be used in concrete lightly loaded (not mat) slab design for the expected compacted subgrade. Mat slab design will require differing modulus values. ACI Section 4.3, Table 4.3.1 should be followed for recommended cement type, water cement ratio, and compressive strength.

If heavily loaded flatwork is proposed (forklift drive areas, heavy racking, etc.), the actual thickness should be designed by the structural engineer utilizing techniques of the American Concrete Institute (ACI) and may be greater than 4 inches in thickness. Concrete floor slabs may either be monolithically placed with the foundations or doweled (No. 4 bar embedded at least 40 bar diameters) after footing placement. The thickness and reinforcing given are not intended to supersede any structural requirements provided by the structural engineer. The project architect or concrete inspector should continually observe all reinforcing steel in slabs during placement of concrete to check for proper location within the slab. The minimum concrete rebar cover should be as per the project architect or structural engineer.

Slab-On-Grade Control Joints: Control joints should be provided in all regular concrete slabs-on-grade at a maximum spacing of 26 to 36 times the slab thickness (12 feet maximum on-center each way, 4 to 6 feet for sidewalks) as recommended by American Concrete Institute [ACI] guidelines. All joints should form approximately square patterns to reduce the potential for randomly oriented shrinkage cracks. Control joints in the slabs should be tooled at the time of the concrete placement or saw cut (¼ of slab depth) as soon as practical but not more than 8 hours from concrete placement.

Construction (cold) joints should consist of thickened butt joints with ¾-inch dowels at 18 inches on center embedded per ACI or a thickened keyed-joint to resist vertical deflection at the joint. All control joints in exterior flatwork should be sealed to reduce the potential of moisture or foreign material intrusion. These procedures will reduce the potential for randomly oriented cracks but may not prevent them from occurring.

Curing and Quality Control: The contractor should take precautions to reduce the potential of curling and cracking of slabs in this arid desert region using proper batching, placement, and curing methods. Curing is highly affected by temperature, wind, and humidity.

Quality control procedures should be used, including trial batch mix designs, batch plant inspection, and on-site special inspection and testing. Curing should be in accordance with ACI recommendations contained in ACI 211, 304, 305, 308, 309, and 318. Additionally, the concrete should be vibrated during placement. Concrete should be wet cured for at least 7 days with burlap or plastic and not allowed to dry out to minimize surface cracking.

5.7 Retaining Walls and Lateral Earth Pressures

Walls which are restrained at the top such as retaining wall returns, below-grade walls and walls tied to floor slabs should be designed with “at rest” earth pressures. Retaining walls, free to tilt (0.1% of wall height) at the top, may be designed for “active” earth pressures.

The following list presents lateral earth pressures for use in wall design. The values are given as equivalent fluid pressures ***without*** surcharge loads or hydrostatic pressure. Clay soils are not suitable for wall backfill as they are not free draining. Native sand material may be used for backfill or free draining material imported as wall backfill. For native or import free draining material, active and restrained walls equivalent fluid pressures are as follows:

- Conventional cantilever retaining walls may be backfilled with compacted on-site soils verified by the contractor to be “very low” in expansion potential. Provided the wall is backfilled at a 1:1 projection upward from the heels of the wall footings with granular

backfill, an active pressure of 40 pcf of equivalent fluid weight for well-drained, level backfill may be used. Similarly, an active pressure of 50 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 2H:1V (horizontal to vertical). For the restrained level backfill condition, a pressure of 62 pcf of equivalent fluid weight should be used.

- In addition to the active or at rest soil pressure, the proposed wall structures should be designed (where not excepted) to include forces from dynamic (seismic) earth pressure. Dynamic pressures are additive to active and at-rest earth pressure, follow their pressure distribution, and should be considered as 0 pcf for flexible walls, and 2 pcf for rigid walls. Seismic pressures are based on PGA_M of 0.55 g, Friction Soil Angle of 33°, and a maximum total density of 136 pcf.
- Retaining wall foundations should be placed upon compacted fill described in Section 5.1.
- A backdrain or an equivalent system of backfill drainage should be incorporated into the wall design, whereby the collected water is conveyed to an approved point of discharge. Design should be in accordance with the 2019 California Building Code. Drain rock should be wrapped in filter fabric such as Mirafi 140N as a minimum and should have a volume of 1 cubic foot per foot of length. Backfill immediately behind the retaining structure should be a free-draining granular material. Waterproofing should be according to the designer's specifications. Water should not be allowed to pond or infiltrate near the top of the wall. To accomplish this, the final backfill grade should divert water away from retaining walls.
- Compaction on the retained side of the wall within a horizontal distance equal to one wall height (to a maximum of 6 feet) should be performed by hand-operated or other lightweight compaction equipment (90% compaction relative to ASTM D 1557 at 3% above the optimum moisture content). This is intended to reduce potential locked-in lateral pressures caused by compaction with heavy grading equipment or dislodging modular block type walls.
- The above recommended values do not include compaction or truck-induced wall pressures. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained a distance of at least 3 feet away from the walls while the backfill soils are placed. Upward sloping backfill or surcharge loads from nearby footings can create larger lateral pressures. Should any walls be considered for retaining sloped backfill or placed next to foundations, our office should be contacted for recommended design parameters. Surcharge loads should be considered if they exist within a zone between the face of the wall and a plane projected 45 degrees upward from the base of the wall. The increase in lateral earth pressure should be taken as 50% of the surcharge load within this zone. Retaining walls subjected to traffic loads should include a minimum uniform surcharge load equivalent of 240 psf for auto and 450 psf for truck traffic kept back at least 3 feet from the wall back edge. Retaining walls should be designed with a minimum factor of safety of 1.5.

<u>Frictional and Lateral Coefficients</u>:

- Resistance to lateral loads (including those due to wind or seismic forces) may be provided by frictional resistance between the bottom of concrete foundations and the underlying

soil, and by passive soil pressure against the foundations. An allowable coefficient of friction of 0.35 (Factor of Safety = 1.2) may be used between cast-in-place concrete foundations and slabs and the underlying soil. An allowable coefficient of friction of 0.30 may be used between pre-cast or formed concrete foundations and slabs and the underlying soil.

- Allowable passive pressure may be taken as equivalent to the pressure exerted by a fluid weighing 350 pounds per cubic foot (pcf)(inherent Factor of Safety = 1.5). The upper 1 foot of soil should not be considered when calculating passive pressure unless confined by overlying asphalt concrete pavement or Portland cement concrete slab. The soils pressures presented have considered onsite fill soils. Testing or observation should be performed during grading by the soils engineer or his representative to confirm or revise the presented values.
- Passive resistance for thrust blocks bearing against firm natural soil or properly compacted backfill can be calculated using an equivalent fluid pressure of 350 pcf. The maximum passive resistance should not exceed 2,000 psf.
- The passive resistance of the subsurface soils will diminish or be non-existent if trench sidewalls slough, cave, or are over widened during or following excavations. If this condition is encountered, our firm should be notified to review the condition and provide remedial recommendations, if warranted.

5.8 Surface and Subsurface Site Drainage and Maintenance

Positive drainage should be maintained away from the structures and undersides of raised floor structures (5 percent for 10 feet minimum) to prevent ponding and subsequent saturation of the foundation soils. Gutters and downspouts in conjunction with a 1 to 2% hardscape grade can be considered as a means to convey water away from foundations if increased fall is not provided. Drainage should be maintained for paved areas. Water should not pond on or near paved areas or foundations. Ponded water can saturate subgrade soils and lead to pavement failure. The following recommendations are provided in regard to site drainage and structure performance:

- Water control and conveyance is a critical aspect of project design. It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Landscaping grades should be lowered and sloped such that water drains to appropriate collection and disposal areas. All runoff water should be controlled, collected, and drained into proper drain outlets. Control methods may include curbing, ribbon gutters, 'V' ditches, or other suitable containment and redirection devices.
- It is highly recommended that landscape irrigation or other sources of water be collected and conducted to an approved drainage device. Site drainage should be devised such that runoff should be directed away from the tops of all graded slopes. Water should not freely flow over slopes or retaining wall faces. Diversion and conveyance structures which can accommodate water and eroded soil should be constructed at the tops and toes of all slopes. Lined swales at the top and bottom of slopes, and at the top of retaining walls are recommended.
- In no instance should water be allowed to flow or pond against structures, slabs or foundations or flow over unprotected slope faces. Adequate provisions should be employed

to control and limit moisture changes in the subgrade beneath foundations or structures to reduce the potential for soil saturation. Landscape borders should not act as traps for water within landscape areas. Potential sources of water such as piping, drains, over-spray broken sprinklers, etc, should be frequently examined. Any such leakage, over-spray, or plugging should be immediately repaired.

- Maintenance of drainage systems and infiltration structures can be the most critical element in determining the success of a design. They must be protected and maintained from sediment-laden water both during and after construction to prevent clogging of the surficial soils any filter medium. The potential for clogging can be reduced by pre-treating structure inflow through the installation of maintainable forebays, biofilters, or sedimentation chambers. In addition, sediment, leaves, and debris must be removed from inlets and traps on a regular basis. Since these and other factors (such as varying soil conditions) may affect the rate of water infiltration, it is imperative to apply a conservative factor of safety [FOS] to unfactored Basic Percolation/Infiltration Rates to provide a reliable basis for design. In order to account not only for the unknown factors above but also for changes of conditions during the use of the structures such as potential clogging effects due to washing in of soil fines, a FOS between 3 and 12 should be applied to lower infiltration rates.
- The factor of safety should be selected by the project drainage engineer and may be dependent on agency guidelines and the presence of testing, filters, and sedimentation structures. If these measures are provided, the factor of safety can be reduced.
- The drainage pattern should be established at the time of final grading and maintained throughout the life of the project. Additionally, drainage structures should be maintained (including the de-clogging of piping, basin bottom scarification, soil crust removal, etc.) throughout their design life. Maintenance of these structures should be incorporated into the facility operation and maintenance manual. Structural performance is dependent on many drainage-related factors such as landscaping, irrigation, lateral drainage patterns and other improvements.
- Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1:1 (h:v) from the projection upward from the bottom of any foundation, whichever is greater.

5.9 Infiltration

During our field exploration, percolation testing was performed at Borings P-1 and P-2 to depths ranging from 5 feet to 15 below existing ground surface. A perforated PVC casing was placed in the borings and the space between the casing and excavation was filled with gravel. A pre-soak of the borings was performed prior to starting the infiltration test. The percolation test was performed in accordance with the Small Diameter Boring, Shallow Infiltration Test procedure in the Los Angeles County Public Works Geotechnical and Materials Engineering Division manual (June 2021) which is a test procedure performed in a boring and a typical standard of care procedure utilized in the area. The test results were converted empirically per similar LA County methods to infiltration rate for use in Kern County.

Table 10
Infiltration Test Summary

Boring	Soil Type	Infiltration Depth (ft)	Measured Percolation Rate (in/hr)	Design Infiltration Rate (in/hr)
P-1	SM	2.5-5	0.0	0.0
P-2	SM	9-15	0.3	0.1

Based on the small diameter boring infiltration test procedure, the reduction factor RF_t may be taken as 2.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 2.0. In addition, provided proper maintenance is performed to minimize long-term siltation and clogging, the reduction factor RF_s may be taken as 2.0, for a total reduction of 6. If measures are not taken, RF_s should be taken as 3.0 for a total reduction of 7. The design of any stormwater infiltration system should be based on applicable guidelines and additional reduction factors may be necessary.

The results of the percolation testing indicate that infiltration is not considered feasible at P-1 and P-2, where the subsurface materials have a design infiltration rate of less than 0.3 inch per hour.

Drywells and infiltration underground chambers should be located at least 50 feet away from any structure or settlement sensitive areas or be setback outside a projection of 1H:1V (horizontal to vertical) from the projection upward from the bottom of any foundation, whichever is greater.

5.10 Driveways and Parking Areas

Pavement structural sections for associated drive areas including recommendations for standard asphalt concrete, and Portland cement concrete are provided below and are based upon on-site soils as described in Section 5.1. Soils differing from those described will require differing pavement sections. The appropriate pavement section depends primarily on the shear strength of the subgrade soil exposed after grading in the near finished subgrade elevation and the anticipated traffic over the useful life of the pavement. Based on the on-site soils and our experience in the site vicinity, an 'R-Value' of 25 was selected as a design value for subgrade support beneath pavements at the site. R-value testing or observation of subgrade soils should be performed on near finished subgrade elevation soils to verify and/or modify the preliminary pavement sections presented within this report.

Pavement Area Preparation: In street, drive, and parking areas, the exposed subgrade should be overexcavated as recommended in Section 5.1, moisture conditioned, and compacted. Compaction should be verified by testing. Aggregate base should be compacted to a minimum 95% relative compaction (ASTM D 1557).

Automobile Traffic and Parking Areas: Pavement sections presented in the following table for automobile type traffic areas and are based on a tested R-value and current Caltrans design procedures. Traffic Indices (TI) of 5 and 7 were used to facilitate the design of asphalt concrete pavements for parking and main drives, including fire lanes. The fire lane calculation assumed a

conservative traffic flow of one fire truck per day entering and exiting the site on the same path (20-year life cycle), and a maximum loading of an 88,000-pound Tandem Axle apparatus (approximate 20,000-pound front axle load and two 34,000-pound rear axles loads) which is based upon the *Emergency Vehicle Size and Weight Regulation Guideline*, dated November 22, 2011, prepared by the Fire Apparatus Manufacturers' Association.

Based on the above stated traffic pattern and apparatus loads, a Traffic Index of 4.6 is calculated for fire lanes. For comparison, a 40-year fire lane life cycle analysis results in a Traffic Index of 5. The TI's assumed below should be reviewed by the project Civil Engineer to evaluate the suitability for this project. All design should be based upon an appropriately selected traffic index. Changes in the traffic indices will affect the corresponding pavement section.

Table 11
Preliminary Flexible Pavement Section Recommendations
On-site/Interior Automobile Drive Areas

R-Value of Subgrade Soils – 25 (Assumed) Design Method - Caltrans

Traffic Index (Assumed)*	Pavement Use	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5	Parking Areas & Fire Lanes ***	3.0	6.5
7	Main Drive Areas	4.0	11.0

*The presented Traffic Indices should be confirmed by the project civil engineer. Changes to the Traffic Index will result in a differing pavement section required.
**Pavement Sections were calculated using Caltrans Design Method.
***Where fire lanes will be a part of a main drive use with other traffic, busses, or trucks, the Main Drive Area pavement section should be used.

Conventional, rigid pavements, i.e., Portland cement concrete (PCC) pavements, are recommended in areas that will be subject to relatively high static wheel loads and/or heavy vehicle loading and unloading and turning areas (i.e., truck/bus lanes). This is due to rutting and shoving that can occur due to the heavy vehicle loads and the repetitious set path which if followed at the bus/delivery truck areas where the same wheel track and stopping occurs generally in the same spot each time. The vehicle load combined with hot summer asphalt (AC) concrete causes the upper surface of the AC to creep forming ruts in conjunction with the braking and accelerating which shove the AC. Turning forces also do the same.

The pavement section below is based upon the American Concrete Institute (ACI) Guide for Construction of Concrete Parking Lots, ACI 330R, utilizing a Jointed Plain Concrete Pavement (no rebar) with load resistance based upon PCC thickness, and the assumptions outlined below. The civil engineer and client may elect to incorporate reinforcing bar into PCC to control the opening of typical fracture faces should it be desired; however, rebar is not needed for load support. ACI 330 R provides reinforcement options.

Table 12
Preliminary Portland Cement Concrete Pavement Sections

Area	Minimum Pavement PCC Thickness* (inches)	Minimum Class 2 Aggregate Base Thickness* (inches)	Minimum 28 Day Flexural Strength (psi)	Concrete Compressive Strength (psi)
Truck/Bus Access or Loading/Unloading Areas/Entrance-Exterior Lanes (Traffic Category B, ADTT = 25)	6.0	4.0	550	4,000

*See Section 5.6 for other slab-on-grade thickness recommendations.

Should the actual traffic category vary from those assumed and listed above, these sections should be modified. All above recommended preliminary pavement sections are contingent on the following recommendations being implemented during construction:

- Pavement should be placed upon compacted fill processed as described in Section 5.1. The upper 12 inches of subgrade soils beneath the asphalt concrete and conventional PCC pavement section should be compacted to a minimum of 95% relative compaction (ASTM D 1557).
- Portland cement concrete pavement may be placed directly on the compacted soil subgrade where soils in the upper 12 inches are confirmed to very low in Expansion Index.
- Subsequent to utility installation, the entire pavement (including PCC) final subgrade should be scarified 12 inches, moisture conditioned to 3% above optimum moisture content, and compacted to a minimum 95% relative compaction immediately prior (within a few days) to the placement and compaction of aggregate base to re-establish proper moisture content and compaction in site soils.
- Subgrade soils and aggregate base should be in a stable, non-pumping condition at the time of placement and compaction. Exposed subgrades should be proof rolled to verify the absence of soft or unstable zones.
- Aggregate base materials should be compacted at near optimum moisture content to at least 95% relative compaction (ASTM D 1557) and should conform to Caltrans Class II criteria. Standard Specifications for Public Works Construction "Greenbook" standards (Crushed Aggregate Base class) may be used in lieu of Caltrans and should be used for all offsite work as Per City of Rosamond requirements. Compaction efforts should include rubber tire proof-rolling of the aggregate base with heavy compaction-specific equipment (i.e., fully loaded water trucks).
- All concrete curbs separating pavement from landscaped areas should extend at least 6 inches into the subgrade soils to reduce the potential for movement of moisture into the aggregate base layer (this reduces the risk of pavement failures due to subsurface water originating from landscaped areas).
- Asphaltic concrete should be ½-in. or ¾-in. grading and compacted to a minimum of 95% of the 75-blow Marshall density (ASTM D 1559) or equivalent.

- Portland cement concrete pavements should be constructed with transverse joints at maximum spacing of 12 feet. A thickened edge should be used where possible and, as a minimum, where concrete pavements abut asphalt pavements. The thickened edge should be 1.2 times the thickness of the pavement (7 inches for a 6-inch pavement) and should taper back to the PCC thickness over a horizontal distance on the order of 3 feet.
- All longitudinal or transverse control joints should be constructed by hand forming or placing pre-molded filler such as "zip strips." Expansion joints should be used to isolate fixed objects abutting or within the pavement area. The expansion joint should extend the full depth of the PCC pavement. Joints should run continuously and extend through integral curbs and thickened edges. We recommend that joint layout be adjusted to coincide with the corners of objects and structures. In addition, the following is recommended for concrete pavements:
    1. Slope pavement at least ½ percent to provide drainage;
    2. Provide rough surface texture for traction;
    3. Cure PCC concrete with curing compound or keep continuously moist for a minimum of 7 days;
    4. Keep all traffic off concrete until PCC compressive strength exceeds 2,000 pounds per square inch (truck traffic should be limited until the concrete meets the design strength (3,650 psi); and
    5. Consideration should be given to having PCC construction joints keyed or using slip dowels on 24-inch centers to strengthen control and construction joints. Dowels placed within dowel baskets should be incorporated into the concrete at each saw-cut control joint (i.e., dowel baskets and dowels are set in place prior to placement of concrete).
- Portland cement concrete placement and curing should, at a minimum, be in accordance with the American Concrete Institute (ACI) recommendations contained in ACI 211, 304, 305, 308, 309, and 318.
- Within the structural pavement section areas, positive drainage (both surface and subsurface) should be provided. In no instance should water be allowed to pond on the pavement. Roadway performance depends greatly on how well runoff water drains from the site. This drainage should be maintained both during construction and over the entire life of the project.
- Proper methods, such as hot-sealing or caulking, should be employed to limit water infiltration into the pavement base course and/or subgrade at construction/expansion joints and/or between existing and reconstructed asphalt concrete sections (if any). Water infiltration could lead to premature pavement failure.
- To reduce the potential for detrimental settlement, excess soil material, and/or fill material removed during any footing or utility trench excavation, should not be spread or placed over compacted finished grade soils unless subsequently compacted to at least 90% of the maximum dry unit weight, as evaluated by ASTM D 1557 test procedure, at 3% above the optimum moisture content, or 95% if placed under areas designated for pavement.
- Where new roadways will be installed against existing roadways, the repaired asphalt concrete pavement section should be designed and constructed to have at least the pavement and aggregate base section as the original pavement section thickness (for

both AC and base) or upon the newly calculated pavement sections presented within, whichever is greater.

- Pavement designs assume that heavy construction traffic will not be allowed on base cap or finished pavement sections.

Section 6
LIMITATIONS AND ADDITIONAL SERVICES

6.1 Uniformity of Conditions and Limitations

Our findings and recommendations in this report are based on selected points of field exploration, laboratory testing, and our understanding of the proposed project. Furthermore, our findings and recommendations are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil or groundwater conditions could exist between and beyond the exploration points. The nature and extent of these variations may not become evident until construction. Variations in soil or groundwater may require additional studies, consultation, and possible revisions to our recommendations.

The planning and construction process is an integral design component with respect to the geotechnical aspects of this project. Because geotechnical engineering is an inexact science due to the variability of natural processes and because we sample only a small portion of the soil and material affecting the performance of the proposed structure, unanticipated or changed conditions can be disclosed during demolition and construction. Proper geotechnical observation and testing during construction is imperative to allow the geotechnical engineer the opportunity to verify assumptions made during the design process and to verify that our geotechnical recommendations have been properly interpreted and implemented during construction. Therefore, we recommend that Earth Systems be retained during the construction of the proposed improvements to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those assumed while completing this study. If we are not accorded the privilege of performing this review, we can assume no responsibility for misinterpretation or the applicability of our recommendations. The above services can be provided in accordance with our current Fee Schedule.

Our evaluation of subsurface conditions at the site has considered subgrade soil and groundwater conditions present at the time of our study. The influence(s) of post-construction changes to these conditions such as introduction or removal of water into or from the subsurface will likely influence future performance of the proposed project. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions due to the limitation of data from field studies. The availability and broadening of knowledge and professional standards applicable to engineering services are continually evolving. As such, our services are intended to provide the Client with a source of professional advice, opinions and recommendations based on the information available as applicable to the project location and scope. If the scope of the proposed construction changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions of this report are modified or approved in writing by Earth Systems.

Findings of this report are valid as of the issued date of the report. However, changes in conditions of a property can occur with passage of time, whether they are from natural processes or works of man, on this or adjoining properties. In addition, changes in applicable standards occur, whether they result from legislation or broadening of knowledge. Accordingly, findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year.

This report is issued with the understanding that the owner or the owner's representative has the responsibility to bring the information and recommendations contained herein to the attention of the architect and engineers for the project so that they are incorporated into the plans and specifications for the project. The owner or the owner's representative also has the responsibility to verify that the general contractor and all subcontractors follow such recommendations. It is further understood that the owner or the owner's representative is responsible for submittal of this report to the appropriate governing agencies.

Earth Systems has striven to provide our services in accordance with generally accepted geotechnical engineering practices in this locality at this time. No warranty or guarantee, express or implied, is made. This report was prepared for the exclusive use of the Client and the Client's authorized agents.

Earth Systems should be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, we can assume no responsibility for misinterpretation of our recommendations. The owner or the owner's representative has the responsibility to provide the final plans requiring review to Earth Systems' attention so that we may perform our review.

Any party other than the client who wishes to use this report shall notify Earth Systems of such intended use. Based on the intended use of the report, Earth Systems may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Earth Systems from any liability resulting from the use of this report by any unauthorized party.

In addition, if there are any changes in the field to the plans and specifications, the Client must obtain written approval from Earth Systems' engineer that such changes do not affect our recommendations. Failure to do so will vitiate Earth Systems' recommendations.

Although available through Earth Systems, the current scope of our services does not include an environmental assessment or an investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or adjacent to the subject property.

6.2 Additional Services

This report is based on the assumption that an adequate program of client consultation, construction monitoring, and testing will be performed during the final design and construction phases to check compliance with these recommendations. Maintaining Earth Systems as the geotechnical consultant from beginning to end of the project will provide continuity of services. ***The geotechnical engineering firm providing tests and observations shall assume the responsibility of Geotechnical Engineer of Record.***

Construction monitoring and testing would be additional services provided by our firm. The costs of these services are not included in our present fee arrangements but can be obtained from our office. The recommended review, tests, and observations include, but are not necessarily limited to, the following:

- Consultation during the final design stages of the project;
- A review of the building and grading plans to observe that recommendations of our report have been properly implemented into the design;
- Observation and testing during site preparation, grading, and placement of engineered fill as required by CBC Sections 17 and Appendix J or local grading ordinances;
- Consultation as needed during construction.

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AERIAL PHOTOGRAPHS REVIEWED

Index: ABL-1942; Frames 5K 36, 37, and 38; October 13, 1952; Scale 1:20,000.

Google Earth: 1985 – 2018
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APPENDIX A

Plate 1 – Site Vicinity Map
Plate 2 – Exploration Location and Geologic Map
Plate 3 – State of California Geomorphic Map
Plate 4 – Regional Geologic Map
Plate 5 – Regional Fault Map
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Spectral Response Values
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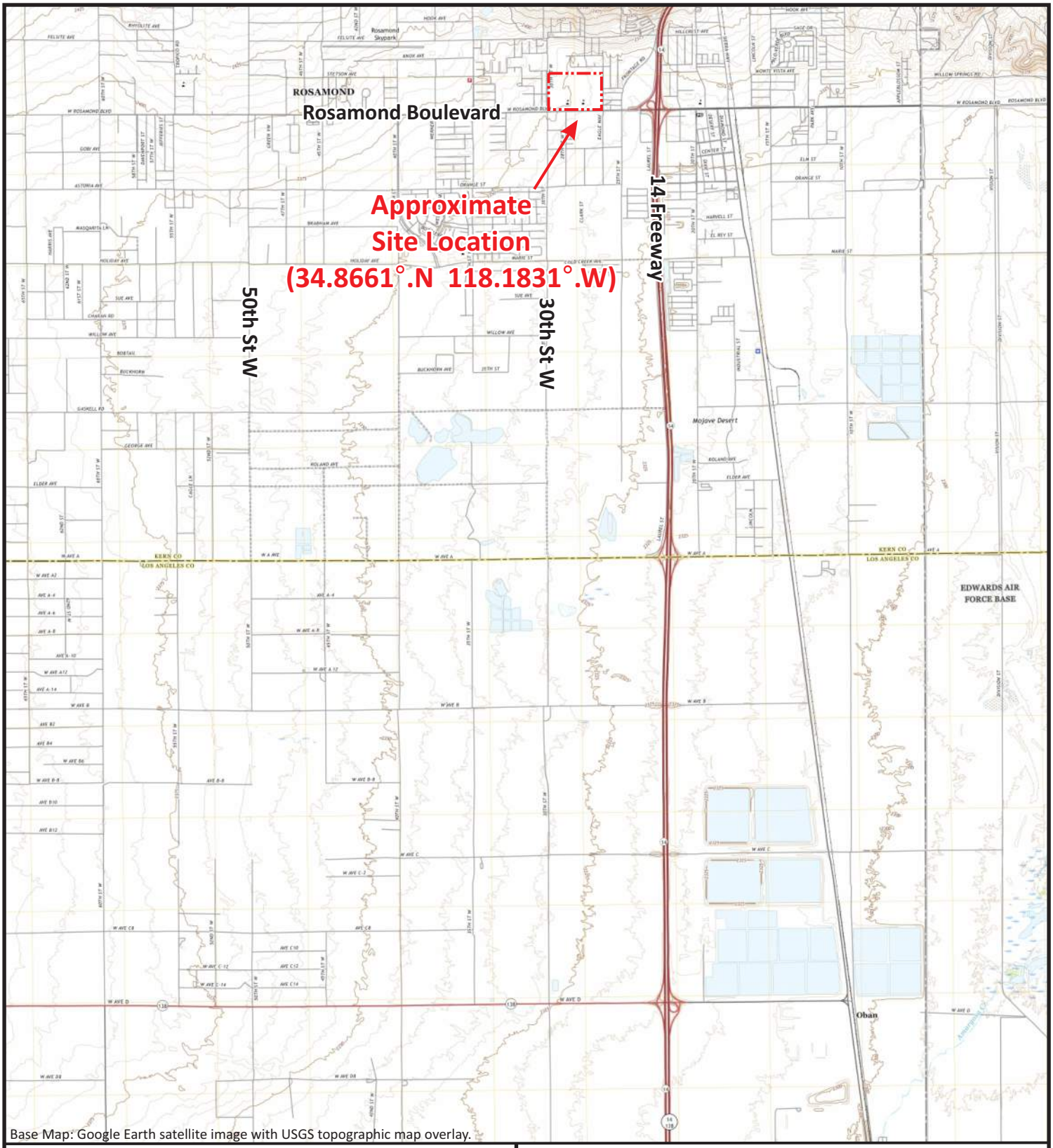
General Procedure Seismic Output
Terms and Symbols Used on Boring Logs
Soil Classification System
Logs of Geotechnical Borings (5)
Site Class Estimate
Dry Seismic Settlement



Base Map: Google Earth satellite image with USGS topographic map overlay.

LEGEND

Approximate Site Boundary

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Plate 1
Site Vicinity Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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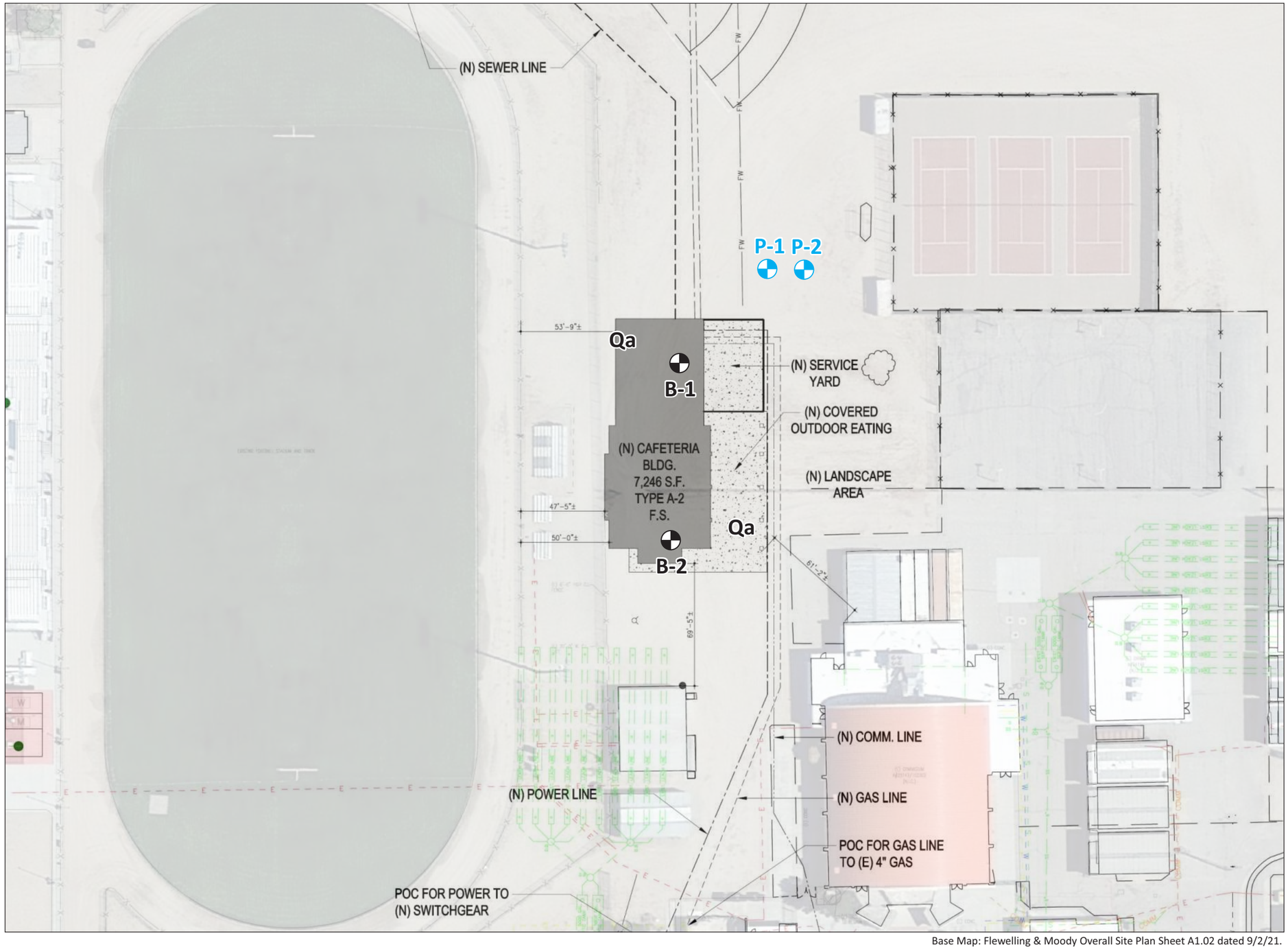


Base Map: Flewelling & Moody Overall Site Plan Sheet A1.02 dated 9/2/21.

LEGEND

B-1	Approximate Exploration Locations
P-1	Approximate Infiltration Test Location
Qa	Quaternary Alluvial Deposits

N

Approximate Scale: 1" = 60'

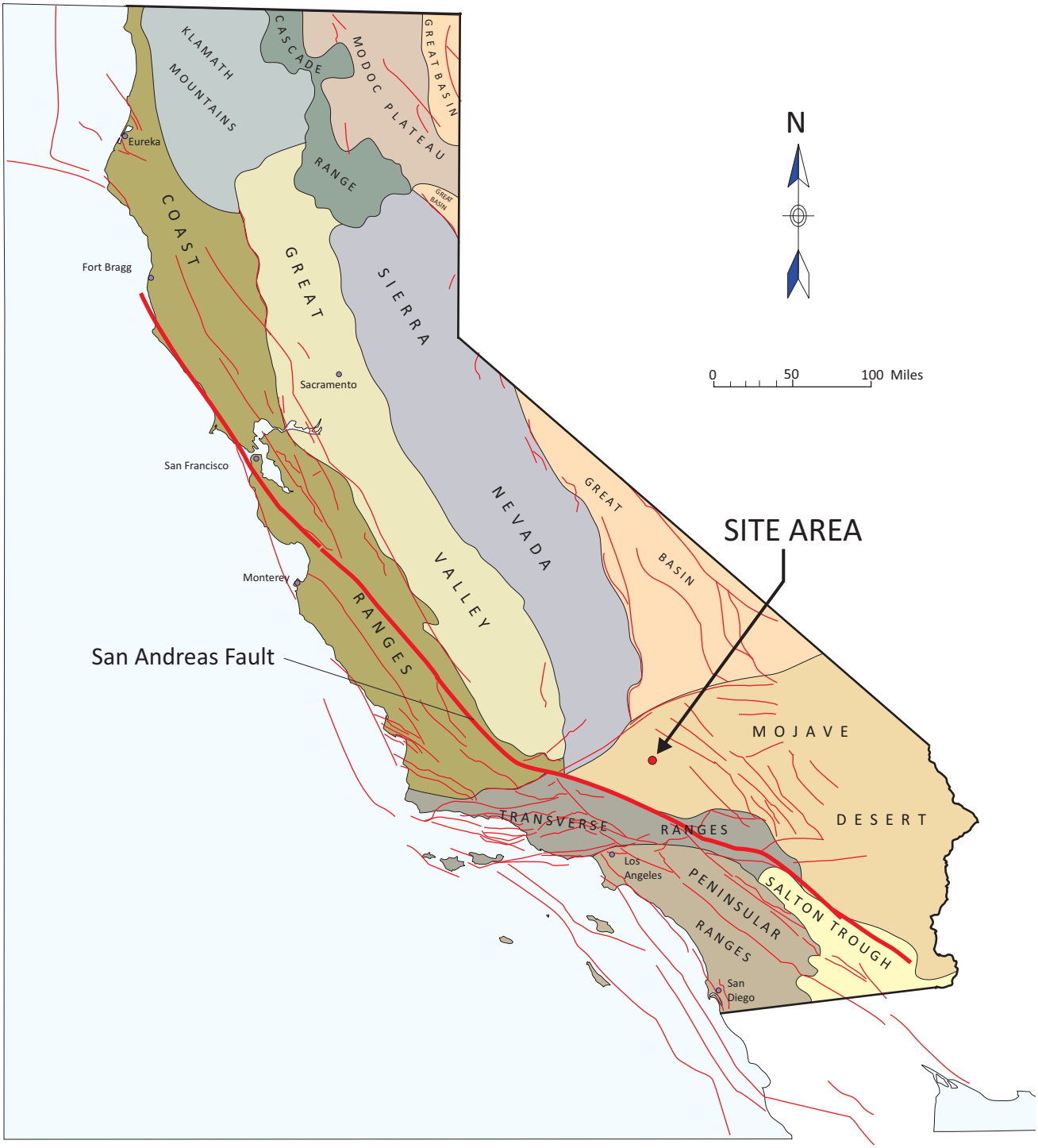
0 60' 120'

Plate 2
Exploration Location and Geologic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002



Map showing geomorphic provinces of California and major active and potentially active faults. Fault locations are based on Jennings (1994) and Blake (2000).

Plate 3
State of California Geomorphic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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LEGEND

Qs	Dune sand	Qc	Oligocene nonmarine
Qal	Alluvium	Ec	Eocene nonmarine
Ql	Lake deposits	E	Eocene marine
Qg	Glacial deposits	Ep	Paleocene marine
Qt	River terrace deposits	Tc	Tertiary nonmarine
Qm	Pleistocene marine and marine terrace deposits	Tm	Tertiary marine
Qpv	Pleistocene volcanic rocks Qpv^r-rhyolite Qpv^a-andesite Qpv^b-basalt Qpv^p-pyroclastic rocks	Ti	Tertiary intrusive (hypabyssal) rocks Ti^r-rhyolite Ti^a-andesite Ti^b-basalt
Qc	Pleistocene nonmarine	Tl	Tertiary lake deposits
QP	Plio-Pleistocene nonmarine	Tv	Tertiary volcanic rocks Tv^r-rhyolite Tv^a-andesite Tv^b-basalt Tv^p-pyroclastic rocks
	Quaternary and/or Pliocene cinder cones	Ku	Upper Cretaceous marine
Pc	Undivided Pliocene nonmarine	Ju	Upper Jurasssic marine
Pu	Upper Pliocene marine	gr	Mesozoic granitic rocks
Pmlc	Middle and/or lower Pliocene nonmarine	bi	Mesozoic basic intrusive rocks
Pml	Middle and/or lower Pliocene marine	ub	Mesozoic ultrabasic intrusive rocks
Pv	Pliocene volcanic rocks Pv^r-rhyolite Pv^a-andesite Pv^b-basalt Pv^p-pyroclastic rocks	Jtv	Jurassic-Triassic metavolcanic rocks
Mc	Undivided Miocene nonmarine	ls	Pre-Cretaceous metamorphic rocks (ls=limestone)
Muc	Upper Miocene nonmarine	ms	Pre-Cretaceous metasedimentary rocks
Mu	Upper Miocene marine	mv	Pre-Cretaceous metavolcanic rocks
Mmc	Middle Miocene marine	gr-m	Pre-Cenozoic granitic and metamorphic rocks
Ml	Lower Miocene marine	pCc	Precambrian igneous and metamorphic rock complex
Mv	Miocene volcanic rocks Mv^r-rhyolite Mv^a-andesite Mv^b-basalt Mv^p-pyroclastic rocks	pC	Undivided Precambrian metamorphic rocks pCg=gneiss pCs=schist pCls=limestone and/or dolomite
		pCgr	Undivided Precambrian granitic rocks

– Fault: Dashed where approximate, dotted where concealed

Source: USGS Geologic Map of California, Los Angeles sheet, dated 1969

Plate 4
Regional Geologic Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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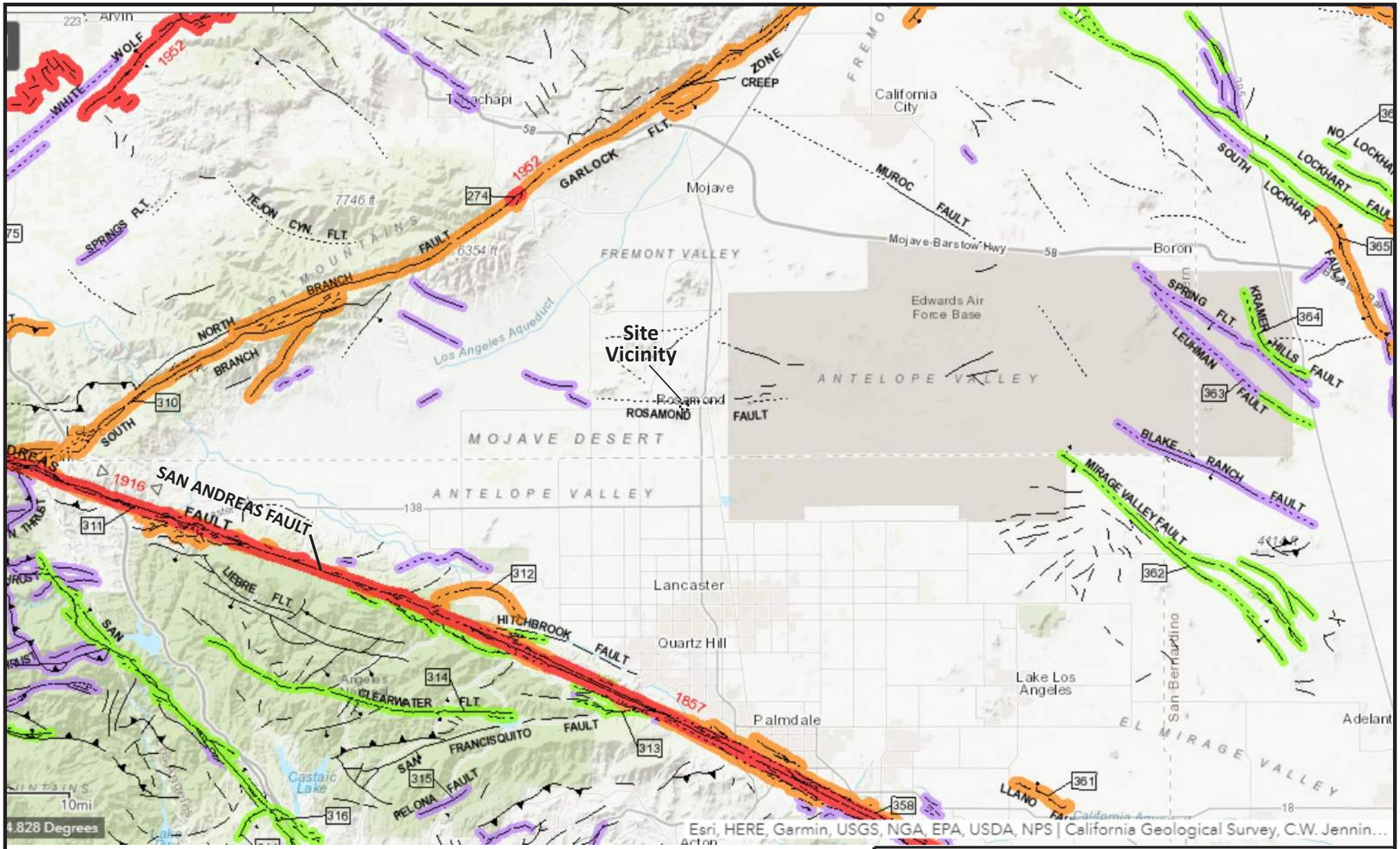

Source: CGS Data Map Series, Fault Activity Map of California.

LEGEND

- Fault along which historic (last 200 years) displacement has occurred.
- Fault along which Holocene (last 11,700 years) displacement, has occurred.
- Fault along which Late Quaternary (past 700,000 years) displacement has occurred.
- Fault along which Quaternary (past 1.6 million years) displacement has occurred.

N

Plate 5
Regional Fault Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

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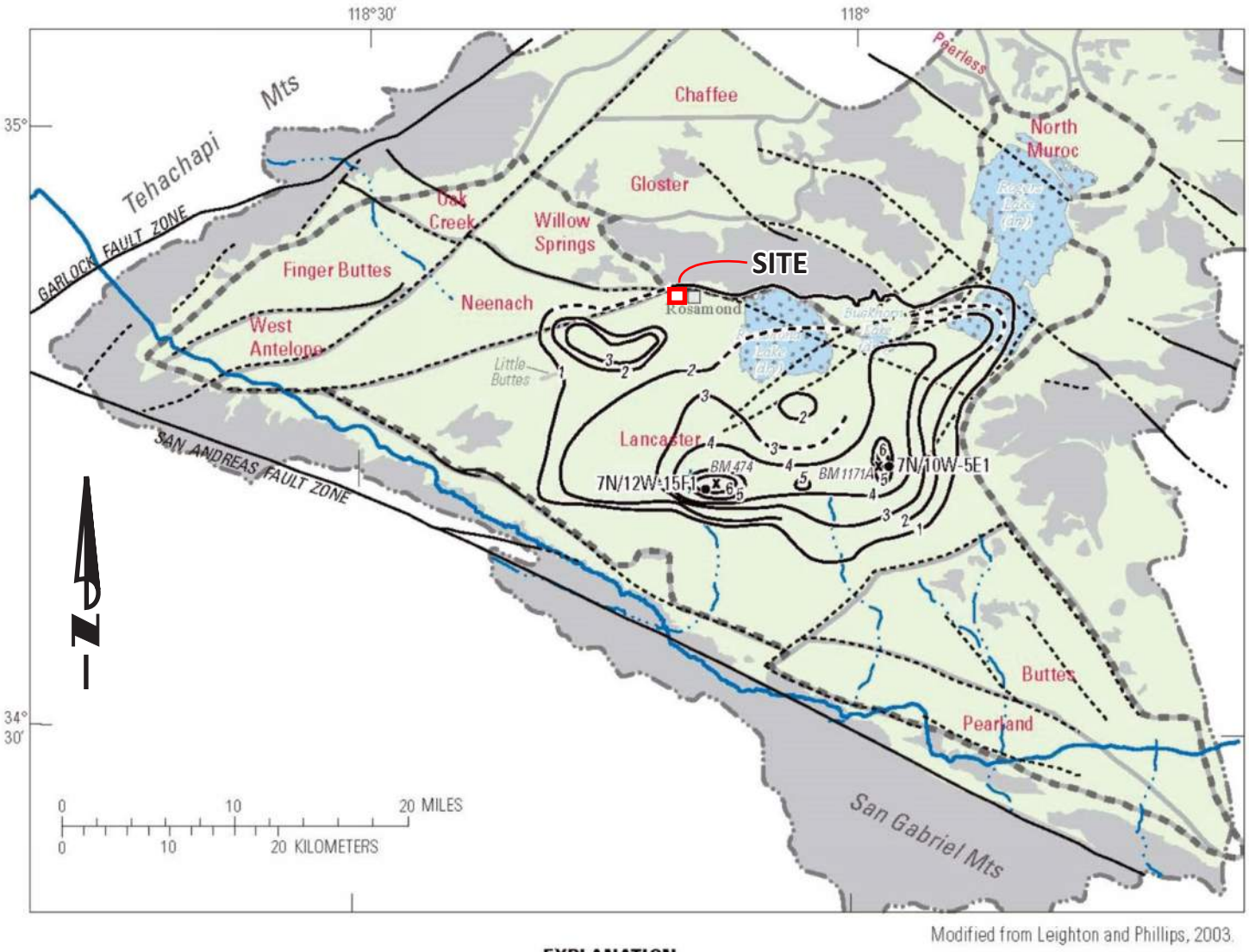


Modified from Leighton and Phillips, 2003.

EXPLANATION

Geologic type

- Alluvial deposits
- Bedrock
- Playa surface

- **Line of equal measured subsidence, in feet**—Dashed where approximately located. Interval 1 foot (modified from Ikehara and Phillips, 1994)
- **Fault**—Dashed where inferred (modified from Dibblee, 1963; Ward and others, 1993; Nishikawa and others, 2001)
- **Antelope Valley drainage basin**
- **Antelope Valley groundwater basin**
- **Groundwater subbasin**
- **California Aqueduct**
- **Stream channel**

7N/12W-15F1 ● **Well and State well number**

BM 474 x **Bench mark and number**

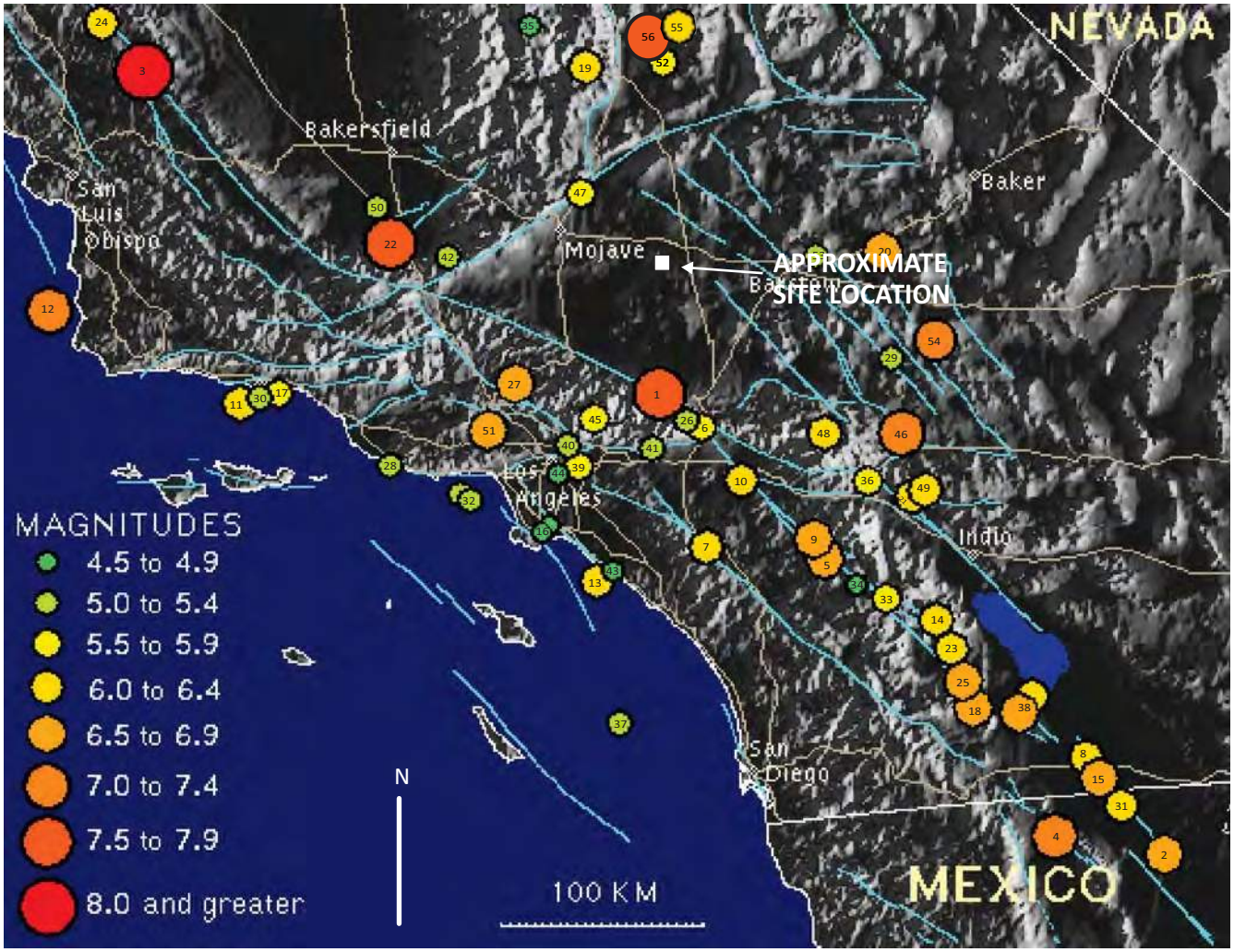
Source: USGS, 2014

Plate 6
Regional Subsidence Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File.: 301430-002



HISTORIC EARTHQUAKES AND EPICENTERS

1. 1812, WRIGHTWOOD
2. 1852, VOLCANO LAKE
3. 1857, FORT TEJON
4. 1892, LAGUNA SALADA
5. 1899, SAN JACINTO
6. 1899, CAJON PASS
7. 1910, ELSINORE
8. 1915, IMPERIAL VALLEY
9. 1918, SAN JACINTO
10. 1923, NORTH SAN JACINTO
11. 1925, SANTA BARBARA
12. 1927, LOMPOC
13. 1933, LONG BEACH
14. 1937, SAN JACINTO
15. 1940, IMPERIAL VALLEY
16. 1941, TORRANCE-GARDENA
17. 1941, SANTA BARBARA
18. 1942, FISH CREEK MOUNTAINS
19. 1946, WALKER PASS
20. 1947, MANIX
21. 1948, DESERT HOT SPRINGS
22. 1952, KERN COUNTY
23. 1954, SAN JACINTO
24. 1966, PARKFIELD
25. 1968, BORREGO MOUNTAINS
26. 1970, LYTLE CREEK
27. 1971, SAN FERNANDO
28. 1973, POINT MAGU
29. 1975, GALWAY LAKE
30. 1978, SANTA BARBARA
31. 1979, IMPERIAL VALLEY
32. 1979, MALIBU
33. 1980, WHITE WASH
34. 1982, ANZA GAP
35. 1983, DURRWOOD MEADOWS SWARM
36. 1986, NORTH PALM SPRINGS
37. 1986, OCEANSIDE
38. 1987, ELMORE RANCH & SUPERSTITION HILLS
39. 1987, WHITTIER NARROWS
40. 1988, PASADENA
41. 1988, UPLAND
42. 1988, TEJON RANCH
43. 1989, NEWPORT BEACH
44. 1989, MONTEBELLO
45. 1991, SIERRA MADRE
46. 1992, LANDERS
47. 1992, MOJAVE
48. 1992, BIG BEAR
49. 1992, JOSHUA TREE
50. 1993, WHEELER RIDGE
51. 1994, NORTHRIDGE
52. 1995, RIDGECREST
53. 1997, CALICO
54. 1999, HECTOR MINE
55. 2019, RIDGECREST
56. 2019, RIDGECREST

SOURCE: SOUTHERN CALIFORNIA EARTHQUAKE CENTER WEB PAGE, 2005

MAP SHOWING LOCATIONS OF SIGNIFICANT HISTORICAL EARTHQUAKES IN SOUTHERN CALIFORNIA FROM 1812 TO 2000

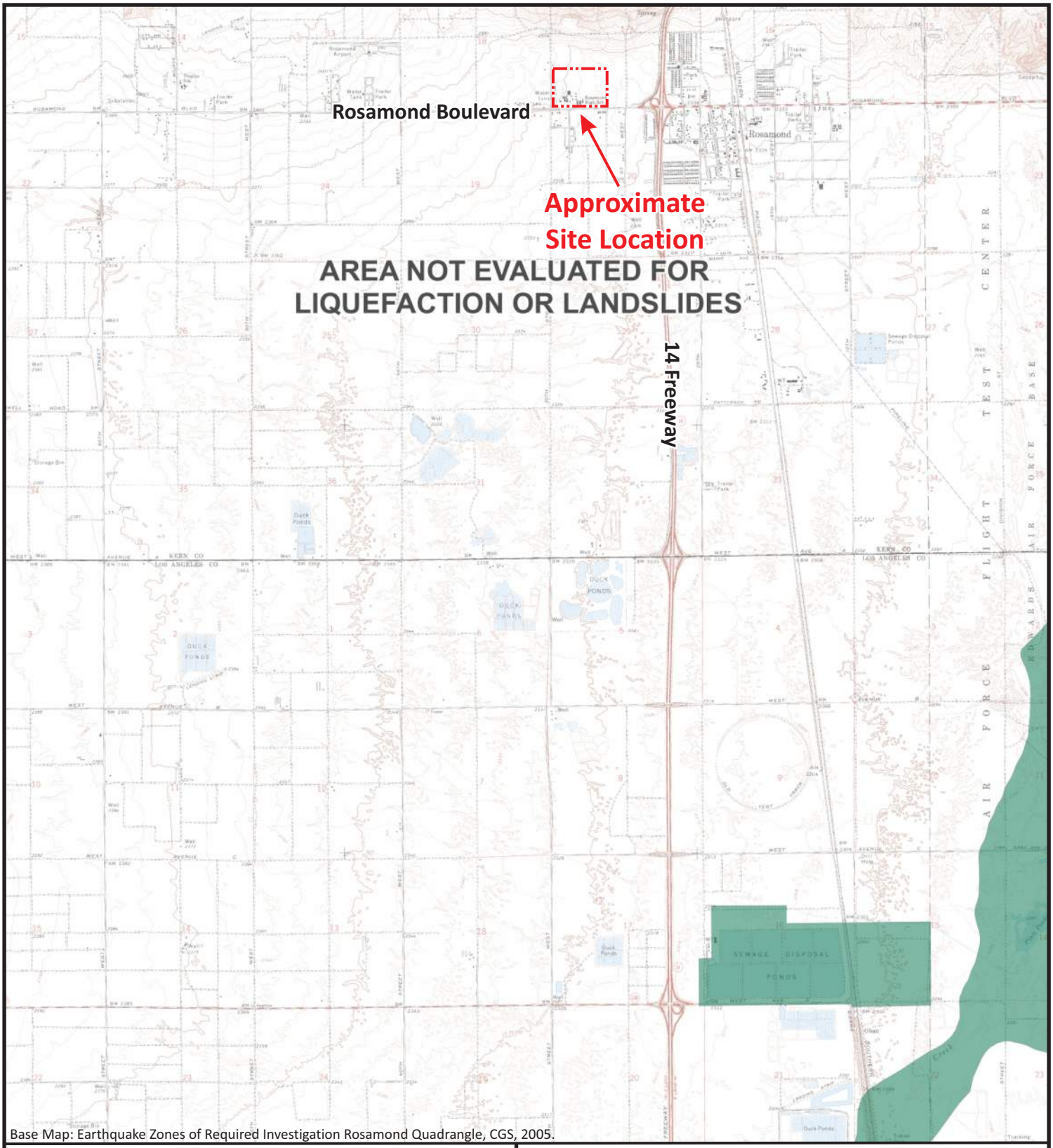
SOURCE: SOUTHERN CALIFORNIA EARTHQUAKE CENTER, WEB PAGE, 2000

Plate 7
Earthquake Epicenter Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002



Base Map: Earthquake Zones of Required Investigation Rosamond Quadrangle, CGS, 2005.

LEGEND

Approximate Site Boundary

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

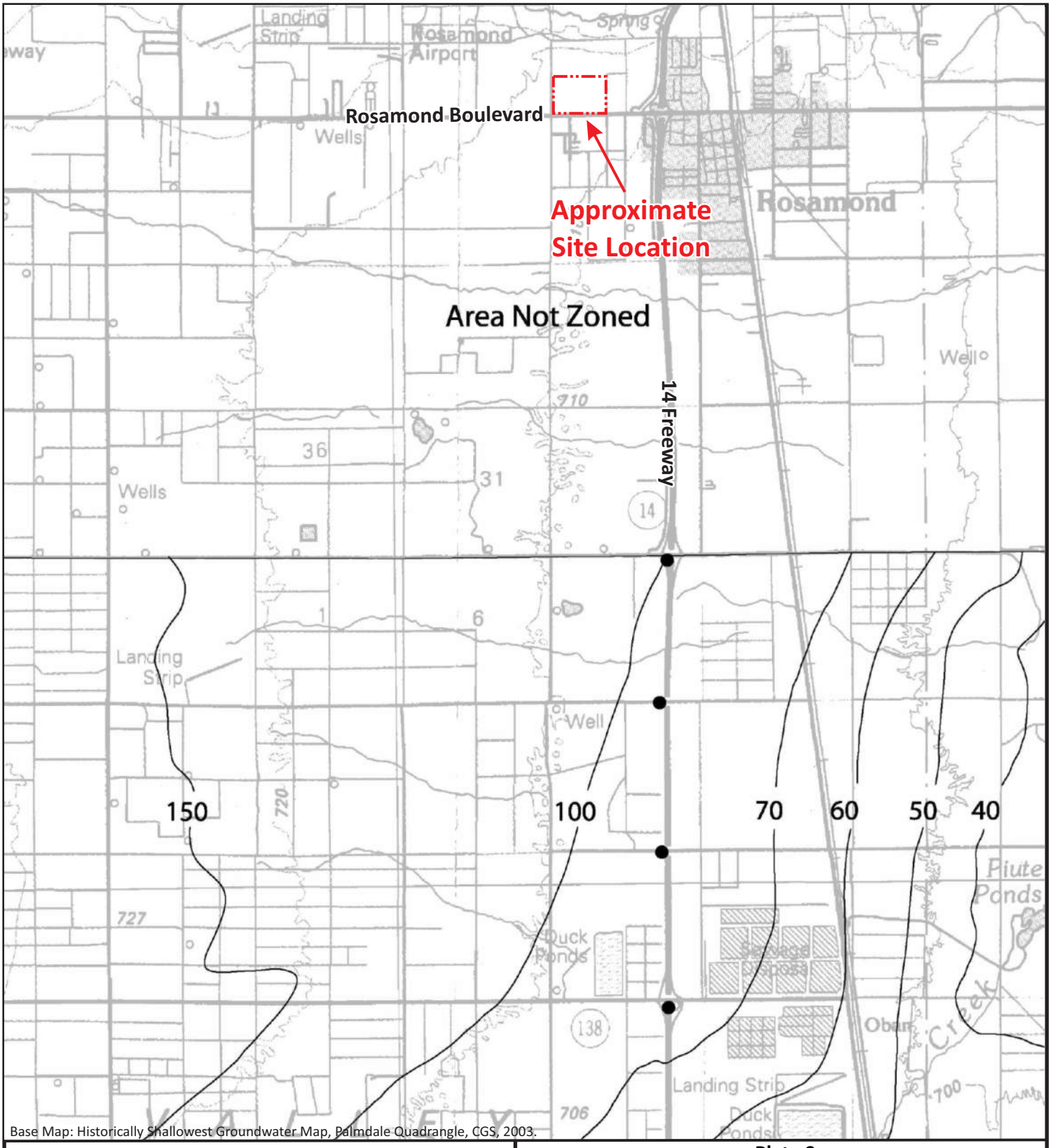
N

Plate 8
Earthquake Hazard Zones

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002



Base Map: Historically Shallowest Groundwater Map, Palmdale Quadrangle, CGS, 2003.

LEGEND

Approximate Site Boundary

0 – 40 Depth to groundwater, in feet

Approximate Scale: 1" = 1 Mile

0 1 Mile 2 Miles

N

Plate 9
Historic Groundwater Map

Rosamond High School Cafeteria
2925 Rosamond Boulevard
Rosamond, Kern County, California

Earth Systems

May 2022 | File No.: 301430-002

Fault Parameters

Fault Section Name	Distance (miles)	Distance (km)	Upper Seis. Depth (km)	Lower Seis. Depth (km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Garlock (West) FM3.1, 3.2	16.6	26.7	0.0	14.7	90	149	0	98	A	**7.6**	493	5
San Andreas (Mojave S) FM3.1, 3.2	18.3	29.5	0.0	13.1	90	206	180	98	A	**7.7**	102	34
San Andreas (Mojave N) FM3.1, 3.2	21.8	35.1	0.0	15.1	90	199	180	37	A	**7.8**	106	9
Garlock (Central) FM3.1, 3.2	30.2	48.6	0.0	11.5	90	161	0	111	A	**7.6**	523	5
So Sierra Nevada FM3.1, 3.2	30.9	49.8	0.0	13.6	50	92	-90	112	B	**7.5**		0.1
Lenwood-Lockhart-Old Woman Springs FM3.1, 3	33.3	53.7	0.0	13.2	90	43	180	145	B	**7.5**		0.9
San Gabriel FM3.1, 3.2	35.7	57.5	0.0	14.7	61	39	180	71	B	**7.3**		1
White Wolf FM3.1, 3.2	36.4	58.6	0.0	14.6	75	141	60	63	B	**7.1**		2
Holser, alt 2 FM3.2	37.1	59.6	0.0	18.5	58	182	90	17	B'	**6.7**		
Helendale-So Lockhart FM3.1, 3.2	37.1	59.8	0.0	12.8	90	51	180	114	B	**7.4**		0.6
Holser, alt 1, FM 3.1	37.3	60.0	0.0	18.6	58	187	90	20	B	**6.7**		0.4
San Gabriel (Extension) FM3.1, 3.2	38.3	61.6	0.0	14.7	61	6	180	62	B'	**7.2**		
Pine Mtn FM3.1, 3.2	38.4	61.7	0.0	16.3	45	5	na	62	B'	**7.3**		
Santa Susana, alt 2 FM3.2	39.1	62.9	0.0	10.6	53	10	90	43	B'	**6.8**		
White Wolf (Extension) FM3.1, 3.2	39.3	63.2	0.0	14.6	75	136	60	46	B'	**6.9**		
Northridge FM3.1, 3.2	39.6	63.7	7.4	16.8	35	201	90	33	B	**6.8**		1.5
Pleito FM3.1, 3.2	39.7	63.9	0.0	13.6	46	181	90	44	B	**7.1**		2
San Andreas (Big Bend) FM3.1, 3.2	40.3	64.8	0.0	15.1	90	198	180	50	A	**6.8**	89	3.5
Santa Susana East (connector) FM3.1, 3.2	40.3	64.8	0.0	16.3	55	na	na	8	B'	**6.3**		
Breckenridge 2011 FM3.1, 3.2	40.9	65.9	0.0	14.6	60	na	na	11	B'	**6.4**		
Sierra Madre (San Fernando) FM3.1, 3.2	41.1	66.1	0.0	13.0	45	9	90	18	B	**6.6**		2
Sierra Madre FM3.1, 3.2	41.3	66.4	0.0	14.2	53	19	90	57	B	**7.2**		2
Santa Susana, alt 1, FM3.1	41.4	66.7	0.0	16.3	55	9	90	27	B	**6.8**		5
Del Valle FM3.1, 3.2	41.6	67.0	0.0	18.8	73	195	90	9	B'	**6.3**		
Mission Hills FM3.1, 3.2	43.5	69.9	0.0	14.5	55	na	na	13	B'	**6.5**		
Verdugo FM3.1, 3.2	43.9	70.6	0.0	14.5	55	31	90	32	B	**6.8**		0.5
San Cayetano FM3.1, 3.2	44.3	71.3	0.0	16.0	42	3	90	42	B	**7.2**		6
Lake Isabella (Seismicity) FM3.1, 3.2	44.6	71.7	0.0	15.2	90	100	na	60	B'	**7.1**		
Oak Ridge (Onshore) FM3.1, 3.2	44.6	71.7	1.0	19.4	65	159	90	50	B	**7.2**		4
Santa Ynez (East) FM3.1, 3.2	45.3	72.9	0.0	13.3	70	172	0	68	B	**7.2**		2
Kern Canyon (Lake Isabella) FM3.1, 3.2	45.6	73.4	0.0	14.6	60	na	na	35	B'	**6.9**		
Northridge Hills FM3.1, 3.2	46.6	75.0	0.0	14.9	31	19	90	25	B'	**7.0**		
Big Pine (East) FM3.1, 3.2	46.7	75.2	0.0	14.3	73	338	na	23	B'	**6.6**		
Clamshell-Sawpit FM3.1, 3.2	47.0	75.7	0.0	14.0	50	334	90	16	B	**6.6**		0.5
Simi-Santa Rosa FM3.1, 3.2	48.6	78.1	1.0	12.1	60	346	30	39	B	**6.8**		1
Gravel Hills-Harper Lk FM3.1, 3.2	49.2	79.2	0.0	11.4	90	41	180	65	B	**7.0**		0.7
Raymond FM3.1, 3.2	49.8	80.1	0.0	15.6	79	348	60	22	B	**6.7**		1.5
Hollywood FM3.1, 3.2	51.4	82.8	0.0	17.3	70	346	30	19	B	**6.6**		1
San Andreas (San Bernardino N) FM3.1, 3.2	52.3	84.2	0.0	12.8	90	212	180	35	A	**7.5**	103	27
Elysian Park (Upper) FM3.1, 3.2	52.4	84.4	3.0	15.0	50	15	90	20	B	**6.6**		1.3

Reference: USGS OFR 2013-1165 (CGS SP 228)

Based on Site Coordinates of 34.8661 Latitude, -118.1831 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2008-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

34.8661 -118.1831 Lat/Long

Site Specific Spectral Response Values
Probabilistic and Deterministic Response Spectra for MCE compared to Code Spectra for 5% Viscous Damping Ratio

Natural Period T (seconds)	GeoMean Probab. 2% in 50 year MCE Spectrum (1) 2475-year (ASCE 21.2.1)	Max Rotated Probab. 2% in 50 year MCEr Spectrum (2) 2475-year (ASCE 21.2.1.1)	Max 84th Percentile Determ. MCE Spectrum (3) (ASCE 21.2.2)	Determ. Lower Limit MCE Spectrum (4) (3) * 1.39=Scaling (ASCE 21.2.2)	Determ. MCE Spectrum (5) Max (3),(4) (ASCE 21.2.2)	Site Specific MCE, Ground Response (SaM) (6) Min (2),(5) (ASCE 21.2.3)	Site Specific MCE Spectrum Comparator (6b) Max (6),1.5*(8) (ASCE 21.2.3)	2019 CBC MCE Spectrum (7)	Site Specific Design Spectrum (Sa) (8) (ASCE 21.3)	2019 CBC Design Spectrum (9) 2/3*(7)
0.00	0.592	0.594	0.433	0.603	0.603	0.594	0.594	0.466	0.396	0.310
0.05	0.798	0.801	0.480	0.669	0.669	0.669	0.669	0.653	0.446	0.435
0.10	1.004	1.007	0.688	0.958	0.958	0.958	0.958	0.840	0.638	0.560
0.15	1.185	1.189	0.862	1.201	1.201	1.189	1.189	1.026	0.793	0.684
0.20	1.366	1.370	0.981	1.366	1.366	1.366	1.366	1.164	0.911	0.776
0.30	1.528	1.565	1.077	1.500	1.500	1.500	1.500	1.164	1.000	0.776
0.40	1.497	1.531	1.062	1.480	1.480	1.480	1.480	1.164	0.986	0.776
0.50	1.465	1.564	1.012	1.410	1.410	1.410	1.410	1.164	0.940	0.776
0.75	1.221	1.298	0.816	1.137	1.137	1.137	1.137	1.164	0.758	0.776
1.00	0.976	1.145	0.681	0.949	0.949	0.949	0.949	1.088	0.633	0.725
1.50	0.761	0.893	0.503	0.700	0.700	0.700	0.700	0.725	0.467	0.483
2.00	0.547	0.666	0.390	0.544	0.544	0.544	0.544	0.544	0.362	0.363
3.00	0.377	0.476	0.265	0.368	0.368	0.368	0.368	0.363	0.246	0.242
4.00	0.285	0.373	0.195	0.271	0.271	0.271	0.271	0.272	0.181	0.181
5.00	0.222	0.300	0.143	0.200	0.200	0.200	0.200	0.218	0.133	0.145
8.00	0.159	0.215	0.086	0.120	0.120	0.120	0.120	0.136	0.080	0.091
10.00	0.150	0.204	0.048	0.066	0.066	0.066	0.087	0.087	0.058	0.058

C_{RS}:	0.912	
C_{R1}:	0.902	
Site Specific To:	0.164	$= 0.2*S_{D1}/S_{DS}$
Site Specific Ts:	0.819	$= S_{D1}/S_{DS}$

The value of Fa used in Column (3) is defined within ASCE 21.2.2 Supplement 1. This Fa value only applies within Column (3).

Site Coefficients	
F_{PGA}	1.11
F_a	1.06
F_v	2.50

Probabilistic spectrum from 2014 USGS Ground Motion Mapping Program adjusted for site conditions and scaled to represent maximum response in a horizontal plane, in accordance with ASCE 7-16 Section 21.2

Mapped MCE Acceleration Values		
PGA	0.488	g
S_S	1.097	g
S_1	0.435	g

Risk Coefficients have been applied to Column (2); If Method 1 was utilized the Risk Coefficients, CRS and CR1 are presented above, if Method 2 was utilized the Risk Coefficients were obtained from the USGS Risk Targeted Ground Motion Calculator (https://earthquake.usgs.gov/designmaps/rtgm).

Seismic Site Class	**D**
Risk Category	**III**

Reference: ASCE 7-16, Chapters 21.2, 21.3, 21.4, 21.5, 11.4, and 11.8

Calculation Utilized ASCE7-16, Section 21.2.1.1 - Method 1

Site-Specific Design Acceleration Values		
PGA_M	**0.550**	**g**
S_{DS}	**0.900**	**g**
S_{D1}	**0.737**	**g**

Short-Period Seismic Design Category:	1-Second Period Seismic Design Category:
D	D

Site-Specific MCE_R, 5% damped, Spectral Response Acceleration Parameter		
S_{MS}	**1.350**	**g**
S_{M1}	**1.105**	**g**

Vertical Coefficient (C_V)
1.32

1 g = 980.6 cm/sec^2 =32.2 ft/sec^2
PSV (ft/sec) = $32.2(S_a)T/(2p)$

Key: Probab. = Probabilistic, Determ. = Deterministic, MCE = Maximum Considered Earthquake

General Procedure Seismic Design Values

2019 California Building Code (CBC) (ASCE 7-16) Seismic Design Parameters

(Values presented should only be used by a Structural Engineer to determine if the exception in 11.4.8 (ASCE 7-16) can be used)

			CBC Reference	ASCE 7-16 Reference
Seismic Design Category		**D**		
Site Class		**D**	Table 1613.5.6	Table 11.6-1
Latitude:		34.866	Table 1613.5.2	Table 20.3-1
Longitude:		-118.183		
Maximum Considered Earthquake (MCE) Ground Motion				
Short Period Spectral Reponse	S_S	**1.097 g**	Figure 1613.5	Figure 22-1
1 second Spectral Response	S_1	**0.435 g**	Figure 1613.5	Figure 22-2
Site Coefficient	F_a	1.06	Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.87	Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	1.164 g	$= F_a * S_S$	
	S_{M1}	0.811 g	$= F_v * S_1$	
Design Earthquake Ground Motion				
Short Period Spectral Reponse	S_{DS}	**0.776 g**	$= 2/3 * S_{MS}$	
1 second Spectral Response	S_{D1}	**0.541 g**	$= 2/3 * S_{M1}$	

Site Specific Evaluation May Be Required Due to Site Class = D or E and S1>=0.2. The Presented SDS and SD1 are NOT Valid Unless the Exception of ASCE7-16, Section 11.4.8 Applies

To	0.14	sec	$= 0.2 * S_{D1}/S_{DS}$
Ts (11.4.8 ASCE 7-16 Exception Assumed)	0.70	sec	$= S_{D1}/S_{DS}$
Risk Category	III		Table 1604.5
Seismic Importance Factor	1.25		
F_{PGA}	1.11		
PGA_M	**0.54**		
Vertical Coefficient (C_V)	1.32		Table 11.9-1

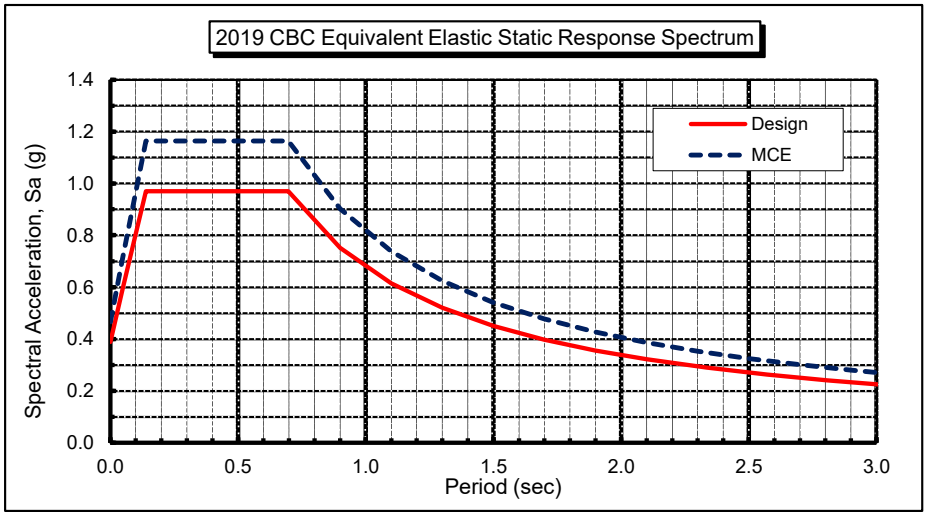


Table 11.5-1 Design

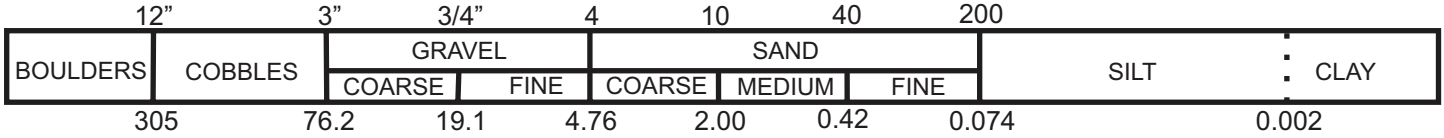
Period T (sec)	Sa (g)
0.00	0.388
0.05	0.597
0.14	0.970
0.70	0.970
0.90	0.751
1.10	0.615
1.30	0.520
1.50	0.451
1.70	0.398
1.90	0.356
2.10	0.322
2.30	0.294
2.50	0.270
2.70	0.250
2.90	0.233
3.10	0.218

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on ASTM Designations D 2487 and D 2488 (Unified Soil Classification System). Information on each boring log is a compilation of subsurface conditions obtained from the field as well as from laboratory testing of selected samples. The indicated boundaries between strata on the boring logs are approximate only and may be transitional.

SOIL GRAIN SIZE

U.S. STANDARD SIEVE



SOIL GRAIN SIZE IN MILLIMETERS

RELATIVE DENSITY OF GRANULAR SOILS (GRAVELS, SANDS, AND NON-PLASTIC SILTS)

Very Loose	*N=0-4	RD=0-30	Easily push a 1/2-inch reinforcing rod by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch reinforcing rod by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch reinforcing rod with hammer
Dense	N=31-50	RD=70-90	Drive a 1/2-inch reinforcing rod 1 foot with difficulty by a hammer
Very Dense	N>50	RD=90-100	Drive a 1/2-inch reinforcing rod a few inches with hammer

*N=Blows per foot in the Standard Penetration Test at 60% theoretical energy. For the 3-inch diameter Modified California sampler,140-pound weight, multiply the blow count by 0.63 (about 2/3) to estimate N. If automatic hammer is used, multiply a factor of 1.3 to 1.5 to estimate N. RD=Relative Density (%). C=Undrained shear strength (cohesion).

CONSISTENCY OF COHESIVE SOILS (CLAY OR CLAYEY SOILS)

Very Soft	*N=0-1	*C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented slightly by finger pressure
Hard	N>30	C>4000	Dented slightly by a pencil point or thumbnail

MOISTURE DENSITY

Moisture Condition: An observational term; dry, damp, moist, wet, saturated.
Moisture Content: The weight of water in a sample divided by the weight of dry soil in the soil sample expressed as a percentage.
Dry Density: The pounds of dry soil in a cubic foot.

MOISTURE CONDITION

Dry.....Absence of moisture, dusty, dry to the touch
Damp.....Slight indication of moisture
Moist.....Color change with short period of air exposure (granular soil)
Below optimum moisture content (cohesive soil)
Wet.....High degree of saturation by visual and touch (granular soil)
Above optimum moisture content (cohesive soil)
Saturated.....Free surface water

PLASTICITY

DESCRIPTION	FIELD TEST
Nonplastic	A 1/8 in. (3-mm) thread cannot be rolled at any moisture content.
Low	The thread can barely be rolled.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit.
High	The thread can be rerolled several times after reaching the plastic limit.

GROUNDWATER LEVEL

Water Level (measured or after drilling)

Water Level (during drilling)

RELATIVE PROPORTIONS

Trace.....minor amount (<5%)
with/some.....significant amount
modifier/and...sufficient amount to influence material behavior (Typically >30%)

LOG KEY SYMBOLS

Bulk, Bag or Grab Sample

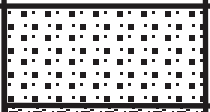
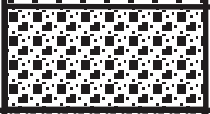
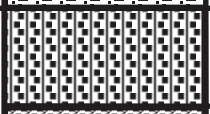
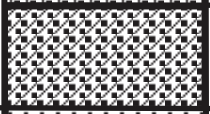
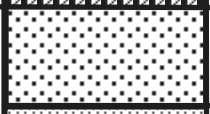
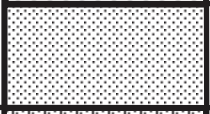
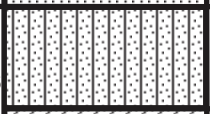
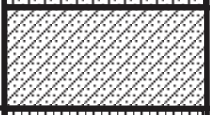
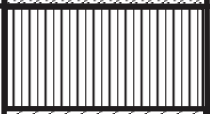
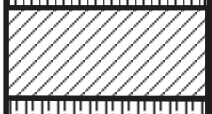
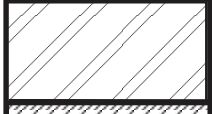
Standard Penetration Split Spoon Sampler (2” outside diameter)

Modified California Sampler (3” outside diameter)

No Recovery

Terms and Symbols Used on Boring Logs

Earth Systems

MAJOR DIVISIONS			GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS More than 50% of material is larger than No. 200 sieve size	GRAVEL AND GRAVELLY SOILS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly-graded gravels, gravel-sand mixtures. Little or no fines
		GRAVELS WITH FINES		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
	SAND AND SANDY SOILS More than 50% of coarse fraction passing No. 4 sieve	CLEAN SAND (Little or no fines)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly-graded sands, gravelly sands, little or no fines
		SAND WITH FINES (appreciable amount of fines)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 sieve size	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty low clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
		LIQUID LIMIT GREATER THAN 50		MH	Inorganic silty, micaceous, or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents
VARIOUS SOILS AND MAN MADE MATERIALS					Fill Materials
MAN MADE MATERIALS					Asphalt and concrete

Soil Classification System

Earth Systems

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
			■	8,13,19			143.1	6.1	SILTY SAND with GRAVEL, brown, damp, medium dense, fine- to coarse-grained sand
5			■	12,14,20		SM	145.6	8.7	SILTY SAND, brown, damp, medium dense, fine- to medium-grained sand
			■	14,18,26			148.3	9.1	SILTY SAND, gray-brown, moist, dense, fine- to medium-grained sand, cementation observed, caliche
10			■	14,25,47			124.9	11.3	very dense, caliche
15			■	7,11,10		SP-SM	105.7	1.9	POORLY GRADED SAND with SILT, light gray, damp, medium dense, fine- to coarse-grained sand
20		■		6,9,9		SM		7.5	SILTY SAND, brown, moist, medium dense, fine- to coarse-grained sand
25			■	20,19,16		SP	110.1	5.5	POORLY GRADED SAND, brown, gray-brown, moist, medium dense, medium- to coarse-grained sand
30		■		5,7,11		SM		14.9	SILTY SAND, brown, moist, medium dense, fine-grained sand
35			■	7,14,31		SM	123.0	11.5	SILTY SAND, brown, moist, dense, fine- to coarse-grained sand

Earth Systems

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PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
40		■		11,12,16		SM		13.4	SILTY SAND, brown, moist, medium dense, fine- to coarse-grained sand
45			■	19,29,50(5")		SP	117.5	6.7	POORLY GRADED SAND, brown, moist, very dense, fine- to coarse-grained sand
50		■		11,12,12		SP-SM		5.9	POORLY GRADED SAND with SILT, brown, moist, medium dense, fine- to coarse-grained sand
55									Total depth 51.5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational
60									
65									
70									
75									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**B2**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
			■	7,8,10		SP	109.2	2.2	POORLY GRADED SAND, light brown, damp, medium dense, fine- to coarse-grained sand
5			■	13,18,25			125.3	6.7	SILTY SAND, light brown, moist, dense, fine- to coarse-grained sand
			■	8,13,21		SM	137.4	10.0	SILTY SAND, gray-brown, damp, medium dense, fine- to medium-grained sand, cementation, caliche
10			■	8,11,19			142.5	11.4	
		■		6,7,12		CL-ML		17.6	SILTY CLAY, gray-brown, moist, very stiff, some fine-grained sand, white mottles, low to moderate plasticity
15			■	8,13,18		SM	115.9	8.9	SILTY SAND, brown, moist, medium dense, fine- to medium-grained sand
20		■		3,4,6		CL-ML		5.7	SILTY CLAY, gray-brown, moist, medium stiff, low plasticity
25			■	7,9,21		SM	118.4	4.5	SILTY SAND, brown, moist, medium dense, fine- to medium-grained sand
30									Total depth 26.5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
35									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**P1**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0						SM			**Alluvium (Qa):** SILTY SAND, brown, damp, fine- to medium-grained sand
5									Total depth 5 feet. Groundwater was not encountered. Backfilled with soil cuttings. Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
10									
15									
20									
25									
30									
35									

Earth Systems

1024 W. Avenue M-4, Palmdale, CA 93551
Phone: (661) 948-7538 Fax: (805) 781-0180

PROJECT NAME:	Rosamond High School Cafeteria	BORING NO:	**P2**
PROJECT NUMBER:	301430-002	DRILL RIG:	B-61
DRILLING DATE:	3/26/2022	DRILLING METHOD:	8-inch hollow-stem auger
BORING LOCATION:	See Map	LOGGED BY:	RSM

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	Penetration Resistance (Blows/6-inches)	Symbol	USCS Classification	Unit Dry Weight (pcf)	Moisture Content (%)	DESCRIPTION OF UNITS
0									**Alluvium (Qa):**
									SILTY SAND, brown, damp, fine- to medium-grained sand
5						SM			
10									
15									Total depth 15 feet.
									Groundwater was not encountered.
20									Backfilled with soil cuttings.
									Note: The stratification lines shown represent the approximate boundaries between soil and/or rock and may be gradational.
25									
30									
35									

Boring No.	B-1	Project and Number	Rosamond HS Cafeter	301430-002
		ESSW Field Staff	Notes: Soils Remediated t Upper 5 feet	
		Drilling Company	GP Drilling	
		Drilling Method	6-8" H S A	HSA Inner Diameter 3"
		Site Latitude (North)	Decimal Degrees	34.8661
		Site Longitude (West)	Decimal Degrees	-118.1831

Date Drilled
3/26/2022

Hammer Weight (lbs)
140

Hammer Drop (inches)
30

Hammer Efficiency (E_M)
72

Borehole Correction (Cb)*
1

*inside diameter of Hollow Stem Auger

Sampler Correction Mod Cal to SPT
0.63

Sampler Liner Correction (Cs)
1.2 Applied if SPT Sampler Used
1.0 Applied if Cal Sampler Used

Rod Length Above Ground (ft)
3

Depth to Estimate Vs Over (ft)*
100

*Caltrans Estimation Method

$*N_{sub}$ Value Desired For Column 6
70

*Only Used for Calculating Nsub otherwise not used by program (i.e.N50, N70, N80, etc)

Calculation Results
Ave. SPT N_{60HE}-value (blows/ft)
28
(Based on Upper 55 feet)
Ave. Shear Wave Velocity (ft/sec)
879
(Based on Upper 55 feet)
Soil Profile Type (Site Class)
D
(Based on Upper 55 feet)
Ave. Friction Angle (degrees)
34
(Based on Upper 55 feet)
Estimated Shear Wave Velocity ** Based on Depth Less than 100' ft
884 (ft/sec Upper 100 feet)
Soil Profile Type (Site Class)**
D Based on Ave. Shear Wave Velocity (ft/sec) 270 (m/sec Upper 100 feet)
Ave. Field SPT N-value (blows/ft)
23.2
(Based on Upper 55 feet)
Ave. Field SPT N-value (blows/ft)
27.2
(Based on Upper 100 feet)
Soil Profile Type (Site Class)**
D Based on Ave. Field Blow Count 27 (Upper 100 feet)

	Equipment variable	Typical Correction (%/100)
Energy ratio (Skempton, 1986)	Donut Hammer	0.50 to 1.00
	Safety Hammer	0.70 to 1.20
	Automatic-Trip Donut-type Hammer	0.80 to 1.30

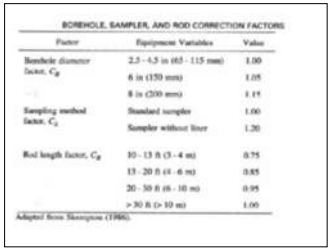
→ Hammer energy as related to the standard 60% delivered energy, i.e. a 72% hammer has and energy ratio of 1.2, i.e. (72/60=1.2)

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Bottom of Layer Depth (ft)	Blow Count***	Type of Sampler	d_i (feet)	N_{60} (blows/ft)	N70 (blows/ft)	N_{60HE} (blows/ft)	V_{si}** (m/sec)	V_{si} (ft/sec)	Φ_i (degrees)	d_i/N_{60HEi}	d_i/V_{si}	d_i/Φ_i	Consistency if Coarse Grained (Based on ASTM and Corrected for N60)	Consistency if Fine Grained (Based on ASTM and Corrected for N60)
5.0	32	c	5.0	18.14	15.55	24.19	253.18	830.43	33.20	0.20668	0.00602	0.150624	Medium Dense	Very Stiff
7.5	34	c	2.5	19.28	16.52	25.70	257.67	845.16	33.50	0.09726	0.00296	0.074617	Medium Dense	Very Stiff
10.0	44	c	2.5	24.95	21.38	33.26	277.68	910.78	34.88	0.07516	0.00274	0.071672	Medium Dense	Very Stiff
15.0	72	c	5.0	46.27	39.66	54.43	320.31	1050.60	37.81	0.09186	0.00476	0.132252	Dense	Hard
20.0	21	c	5.0	15.08	12.93	15.88	224.07	734.94	31.19	0.31494	0.00680	0.160325	Medium Dense	Very Stiff
25.0	18		5.0	20.52	17.59	21.60	245.00	803.58	32.63	0.23148	0.00622	0.153228	Medium Dense	Very Stiff
30.0	35	c	5.0	26.46	22.68	26.46	259.85	852.30	33.65	0.18896	0.00587	0.148569	Medium Dense	Very Stiff
35.0	18		5.0	21.60	18.51	21.60	245.00	803.58	32.63	0.23148	0.00622	0.153228	Medium Dense	Very Stiff
40.0	45	c	5.0	34.02	29.16	34.02	279.49	916.73	35.01	0.14697	0.00545	0.142834	Dense	Hard
45.0	28		5.0	33.60	28.80	33.60	278.49	913.44	34.94	0.14881	0.00547	0.143116	Dense	Hard
50.0	100	c	5.0	75.60	64.80	75.60	352.32	1155.61	40.00	0.06614	0.00433	0.125002	Very Dense	Hard
55.0	24		5.0	28.80	24.69	28.80	266.31	873.50	34.10	0.17361	0.00572	0.146631	Medium Dense	Very Stiff
		Total:	55.0	"d" Feet				Total:		1.97335	0.06258	1.602098		

**Used When Boring Depths are less than 100 feet to estimate Shear Wave Velocity over 100 feet. Caltrans Geotechnical Services Design Manual, Version 1.0, August 2009 using N60HE corrected only for Hammer Energy (Empirical Calculation)

*** Uncorrected blowcount not to exceed 100 blows as entry per CBC

Consistency classification based upon ASCE 1996


BOREHOLE, SAMPLER, AND ROD CORRECTION FACTORS

Factor	Equipment Variables	Value
Borehole diameter factor, C_B	2.5 - 4.5 in (65 - 115 mm)	1.00
	6 in (150 mm)	1.05
	8 in (200 mm)	1.15
Sampling method factor, C_S	Standard sampler	1.00
	Sampler without liner	1.20
Rod length factor, C_R	10 - 13 ft (3 - 4 m)	0.75
	13 - 20 ft (4 - 6 m)	0.85
	20 - 30 ft (6 - 10 m)	0.95
	> 30 ft (> 10 m)	1.00

Adapted from Skempton (1986).

Spreadsheet Version 2.6, 2019: Prepared by Kevin L. Paul, PE, GE

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Rosamond HS Cafeteria
Job No: 301430-002
Date: 6/12/2022
Boring: B1 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.9 (7.5)
PGA, g: 0.37 (0.42)
MSF: 0.88
GWT: #### feet
Calc GWT: 50.0 feet
Remediate to: 0.0 feet

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.30 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: #N/A **Minimum Calculated SF:** #N/A **Required SF:** 1.00

Total (ft) Liquefied Thickness	Total (in.) Induced Subsidence
0	0.4

SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 19.2

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	*Trigger FC Adj.* $\Delta N_{1(60)}$	*Equiv. Sand* $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	**Liquefac. Safety Factor**	*Post FC Adj.* $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric *Strain (%)*	**Induced Subsidence (in.)**	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	Shear Strain γ	Strain E_{15}	Strain Enc	**Dry Sand Subsidence (in.)**
0.0				0				0.000																							
5.0	32	**20**	1	152	10	2.5	5.5	0.190	0.190	1.00	1.70	0.75	1.00	33.4	69	1.6	35.0	1.00	1.200	0.274	**Non-Liq.**	1.6	35.0	0.02	**0.01**	0.127	521	0.045	1.5E-04	7.4E-05	8.3E-05	**0.01**
7.5	34	**21**	1	158	10	5.0	8.0	0.380	0.380	0.99	1.67	0.75	1.00	34.9	71	1.6	36.5	1.00	1.200	0.272	**Non-Liq.**	1.6	36.5	0.02	**0.01**	0.254	748	0.090	1.9E-04	9.2E-05	1.0E-04	**0.01**
10.0	44	28	1	162	10	7.5	10.5	0.577	0.577	0.98	1.35	0.75	1.00	36.6	72	1.7	38.2	1.00	1.200	0.271	**Non-Liq.**	1.7	38.2	0.02	**0.01**	0.387	937	0.137	2.2E-04	1.0E-04	1.1E-04	**0.01**
15.0	72	45	1	139	10	10.0	13.0	0.780	0.780	0.98	1.16	0.76	1.00	52.0	86	2.0	54.0	1.00	1.200	0.269	**Non-Liq.**	2.0	54.0	0.01	**0.01**	0.522	1,221	0.184	2.1E-04	6.4E-05	7.1E-05	**0.01**
20.0	21	13	1	108	10	15.0	18.0	1.127	1.127	0.97	0.97	0.86	1.00	14.4	45	1.2	15.6	0.99	0.168	0.270	**Non-Liq.**	1.2	15.6	0.15	**0.09**	0.755	970	0.263	5.1E-04	6.9E-04	7.7E-04	**0.09**
25.0	0	18	1	113	10	20.0	23.0	1.396	1.396	0.96	0.87	0.93	1.23	23.4	58	1.4	24.7	0.92	0.278	0.286	**Non-Liq.**	1.4	24.7	0.07	**0.04**	0.936	1,260	0.321	4.1E-04	3.2E-04	3.6E-04	**0.04**
30.0	35	22	1	120	10	25.0	28.0	1.679	1.679	0.94	0.79	0.98	1.00	22.4	57	1.4	23.7	0.87	0.263	0.297	**Non-Liq.**	1.4	23.7	0.08	**0.05**	1.125	1,362	0.380	4.5E-04	3.7E-04	4.1E-04	**0.05**
35.0	0	18	1	121	10	30.0	33.0	1.979	1.979	0.92	0.73	1.00	1.21	20.6	54	1.3	21.9	0.83	0.239	0.305	**Non-Liq.**	1.3	21.9	0.10	**0.06**	1.326	1,441	0.438	4.9E-04	4.4E-04	4.9E-04	**0.06**
40.0	45	28	1	137	10	35.0	38.0	2.280	2.280	0.89	0.68	1.00	1.00	25.1	60	1.4	26.5	0.79	0.310	0.308	**Non-Liq.**	1.4	26.5	0.07	**0.04**	1.528	1,648	0.488	4.5E-04	3.2E-04	3.6E-04	**0.04**
45.0	0	28	1	119	10	40.0	43.0	2.623	2.623	0.85	0.64	1.00	1.28	29.5	65	1.5	31.0	0.76	1.200	0.307	**Non-Liq.**	1.5	31.0	0.05	**0.03**	1.757	1,862	0.537	4.2E-04	2.4E-04	2.7E-04	**0.03**
50.0	100	63	1	120	10	45.0	48.0	2.921	2.921	0.80	0.60	1.00	1.00	49.3	84	1.9	51.2	0.67	1.200	0.332	**Non-Liq.**	1.9	51.2	0.02	**0.01**	1.957	2,322	0.565	3.2E-04	1.0E-04	1.1E-04	**0.01**
55.0	0	24	1	111	10	50.0	53.0	3.221	3.221	0.75	0.57	1.00	1.21	21.7	56	1.3	23.1	0.72	0.254	0.289	**Non-Liq.**	1.3	23.1	0.00	**0.00**	2.158	1,869	0.583	4.5E-04			
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
0.0	0	0	0	0	0	0.0																									
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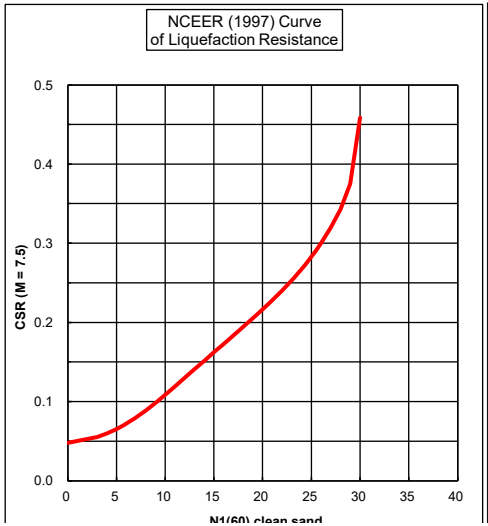
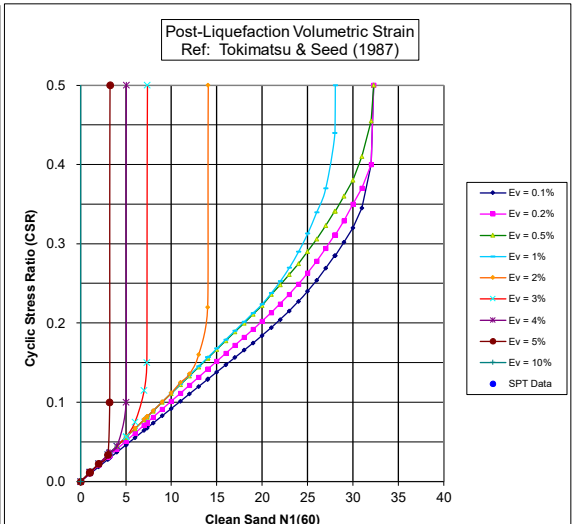




$N_{1(60)} = C_N*C_E*C_B*C_R*C_S*N$
C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))
$C_N = (1\ atm/p'o)^{0.5}$, max 1.7
C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners
$MSF = 10^{2.24}/M^{2.56}$
z = Depth (m)
pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po
τ_{av} = 0.65*PGA*po*rd
$G_{max} = 447*N_{1(60)CS}^{(1/3)}*p^{0.5}$
a = 0.0389*(p/1)+0.124
$b = 6400*(p/1)^{(-0.6)}$
$\gamma = [1+a*EXP(b*\tau_{av}/G_{max})]/[(1+a)*\tau_{av}/G_{max}]$
$E_{15} = \gamma*(N_{1(60)CS}/20)^{-1.2}$
$E_{nc} = (Nc/15)^{0.45}*E15$

$Nc = (MAG-4)^{2.17}$
S = 2*H*E_{nc}

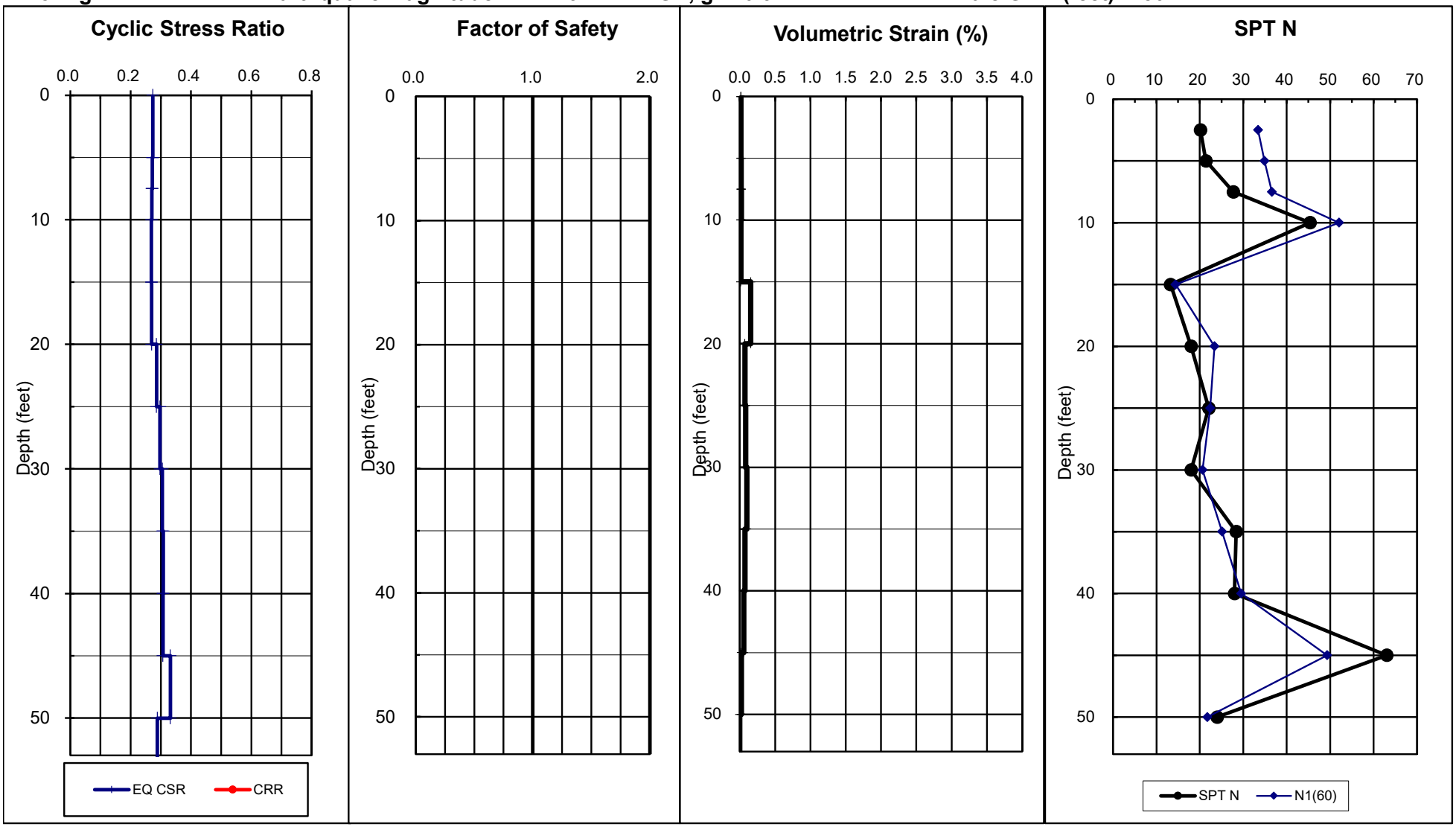
rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))
$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)
$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$
Kσ = min of 1.0 or $(p'o/1.058)^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$
$Dr = (N_{1(60)}/70)^{0.5}$
CSReq = 0.65*PGA*(po/p'o)*rd
CSR* = CSReq/MSF/Kσ
$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))
N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

EARTH SYSTEMS - EVALUATION OF LIQUEFACTION POTENTIAL AND INDUCED SUBSIDENCE

Rosamond HS Cafeteria **Project No: 301430-002** 1996/1998 NCEER Method

Boring: B1 **Earthquake Magnitude: 7.9** **PGA, g: 0.37** **Calc GWT (feet): 50**



Total Thickness of Liquefiable Layers: 0.0 feet

Estimated Total Ground Subsidence: 0.4 inches

APPENDIX B

Laboratory Test Results

File No.: 301430-002 6/12/2022

Lab No.: 22-186

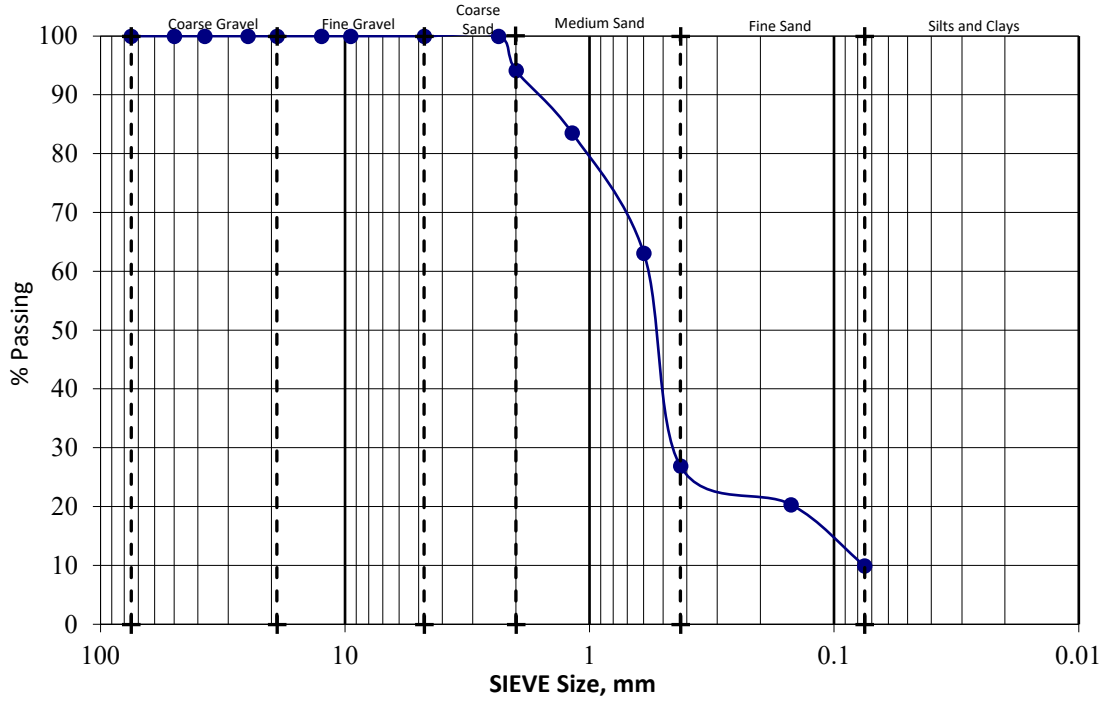
SIEVE ANALYSIS ASTM D6913

Job Name: Rosamond High School Cafeteria

Sample ID: B-1 @ 0-5'

Description: Silty Sand (SM)

Sieve Size	% Passing
3"	100
2"	100
1-1/2"	100
1"	100
3/4"	100
1/2"	100
3/8"	100
#4	100
#8	100
#10	94
#16	84
#30	63
#40	27
#50	100.0
#100	20.3
#200	9.9



% Coarse Gravel:	0	% Coarse Sand:	6	**FM=**	**1.3**	
% Fine Gravel:	0	% Medium Sand:	67	Cu:	2.81	
		% Fine Sand:	17	Cc:	1.67	Gradation
% Total Gravel	0	% Total Sand	90	% Fines:	9.9	Poorly Graded

File No.: 301430-002 May 20, 2022

Job Name: Rosamond High School Cafeteria

Lab Number: 22-186

ASTM D-1140 or Earth Systems Method (circle one)

AMOUNT PASSING NO. 200 SIEVE (Earth Systems Method Transfers Sample until water runs clear)

Sample Location	Depth (feet)	Fines Content (%)	USCS Group Symbol	Soaking Time
B-1	30	10.0	SM	10

File No.: 301430-002 May 20, 2022
Lab No.: 22-186

EXPANSION INDEX **ASTM D-4829**

Job Name: Rosamond High School Cafeteria
Sample ID: B-1 @ 0-5'
Soil Description: Silty Sand (SM)

Initial Moisture, %:	12.0	
Initial Compacted Dry Density, pcf:	112.8	
Initial Saturation, %:	66	*
Final Moisture, %:	19.8	
Volumetric Swell, %:	-0.3	
Expansion Index, EI:	**0**	**Very Low**

EI	ASTM Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 2.5'

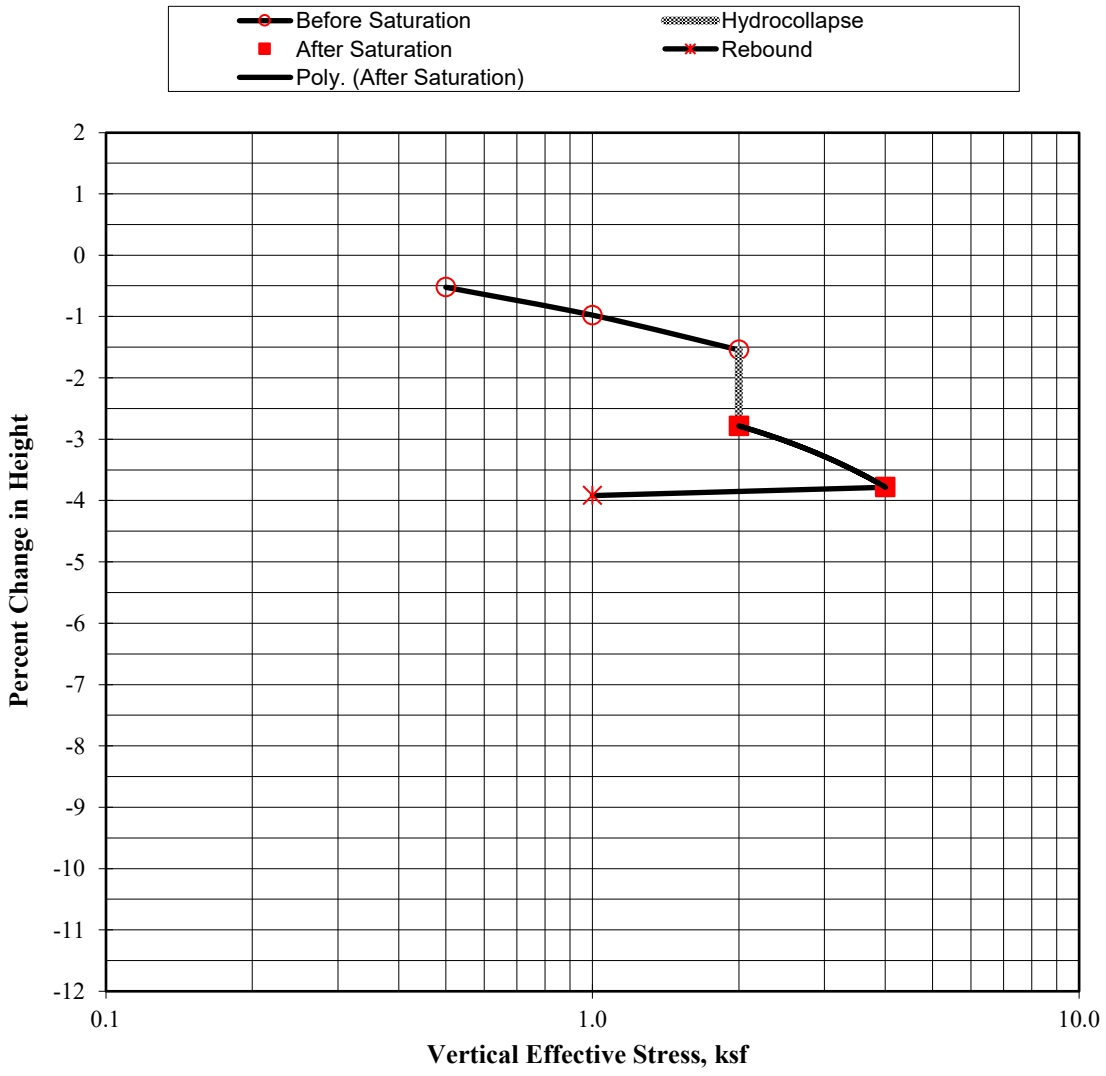
Silty Sand (SM)

Ring Sample

Initial Dry Density: 109.0 pcf
Initial Moisture: 4.6%
Specific Gravity: 2.67
Initial Void Ratio: 0.529

Hydrocollapse: 1.2% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 5'

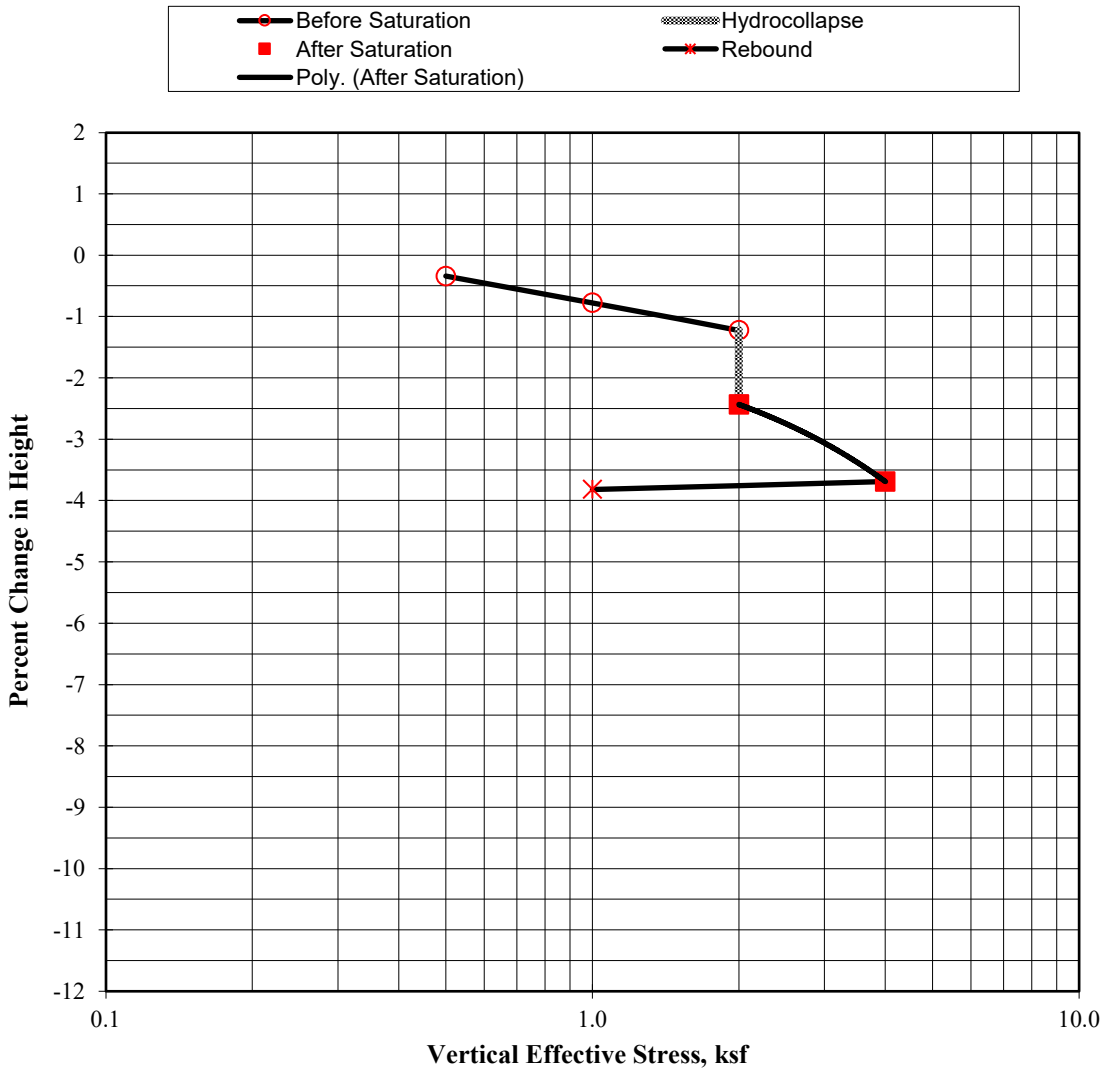
Silty Sand (SM)

Ring Sample

Initial Dry Density: 124.7 pcf
Initial Moisture: 9.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.336

Hydrocollapse: 1.2% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-1 @ 7.5'

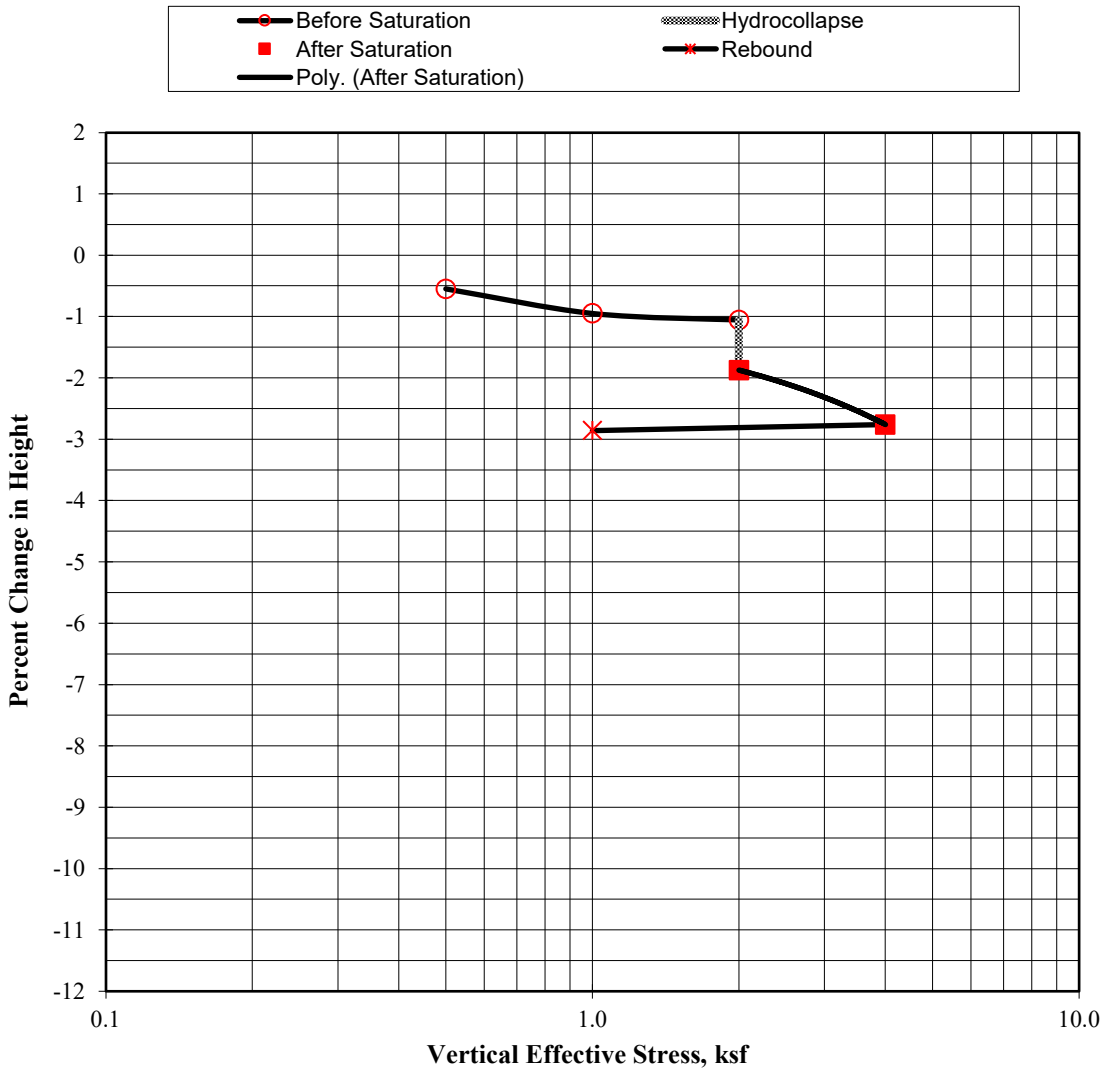
Silty Sand (SM)

Ring Sample

Initial Dry Density: 114.7 pcf
Initial Moisture: 13.1%
Specific Gravity: 2.67
Initial Void Ratio: 0.453

Hydrocollapse: 0.8% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-2 @ 7.5'

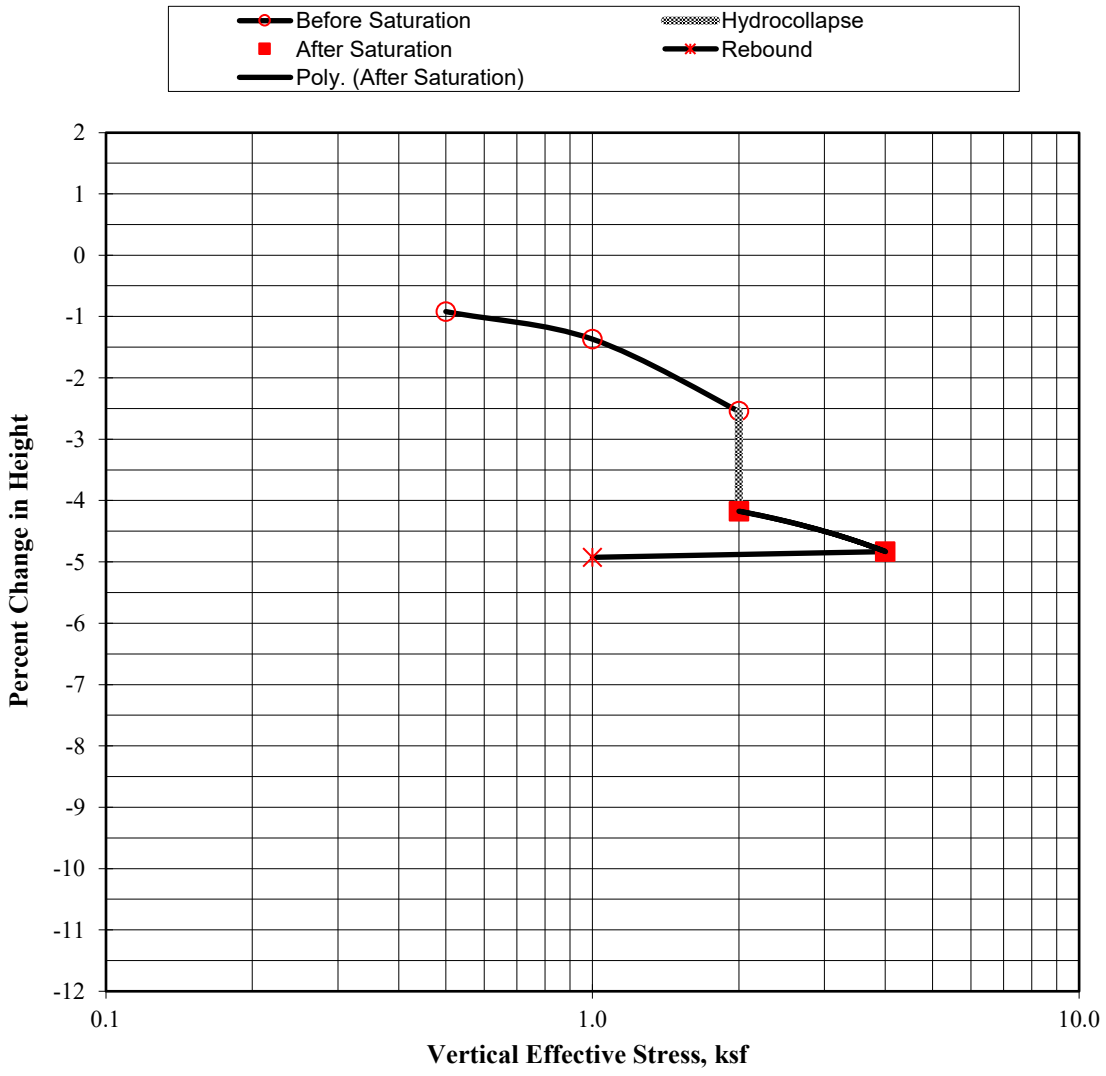
Silty Sand (SM)

Ring Sample

Initial Dry Density: 124.6 pcf
Initial Moisture: 10.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.337

Hydrocollapse: 1.6% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

CONSOLIDATION TEST **ASTM D 2435 & D 5333**

Rosamond High School Cafeteria
B-2 @ 10'

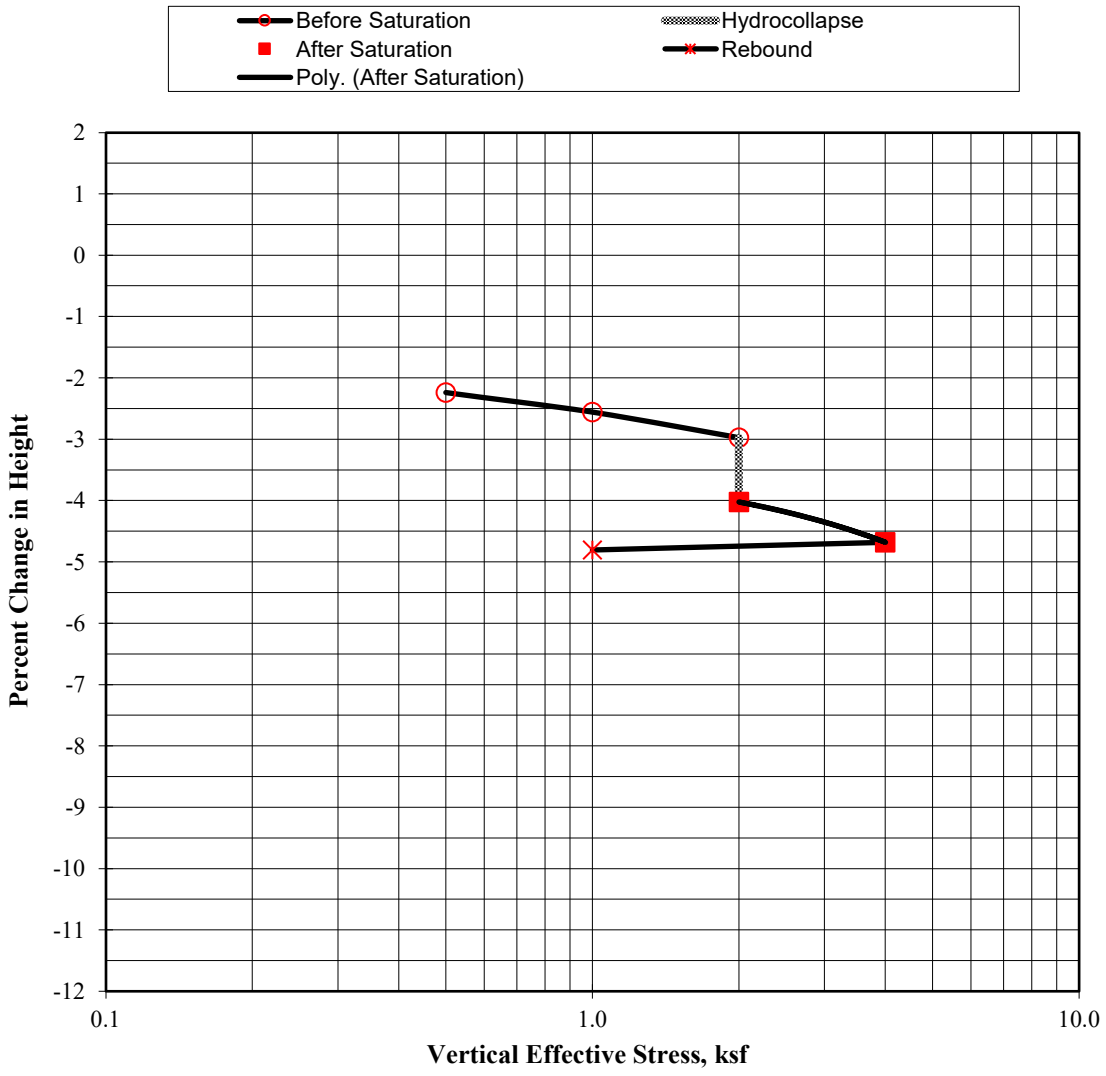
Silty Sand (SM)

Ring Sample

Initial Dry Density: 113.0 pcf
Initial Moisture: 9.2%
Specific Gravity: 2.67
Initial Void Ratio: 0.476

Hydrocollapse: 1.1% @ 2.0 ksf

% Change in Height vs Normal Pressure Diagram



File No.: 301430-002
Lab No.: 22-186

May 20, 2022

MAXIMUM DRY DENSITY / OPTIMUM MOISTURE

ASTM D 1557 (Modified)

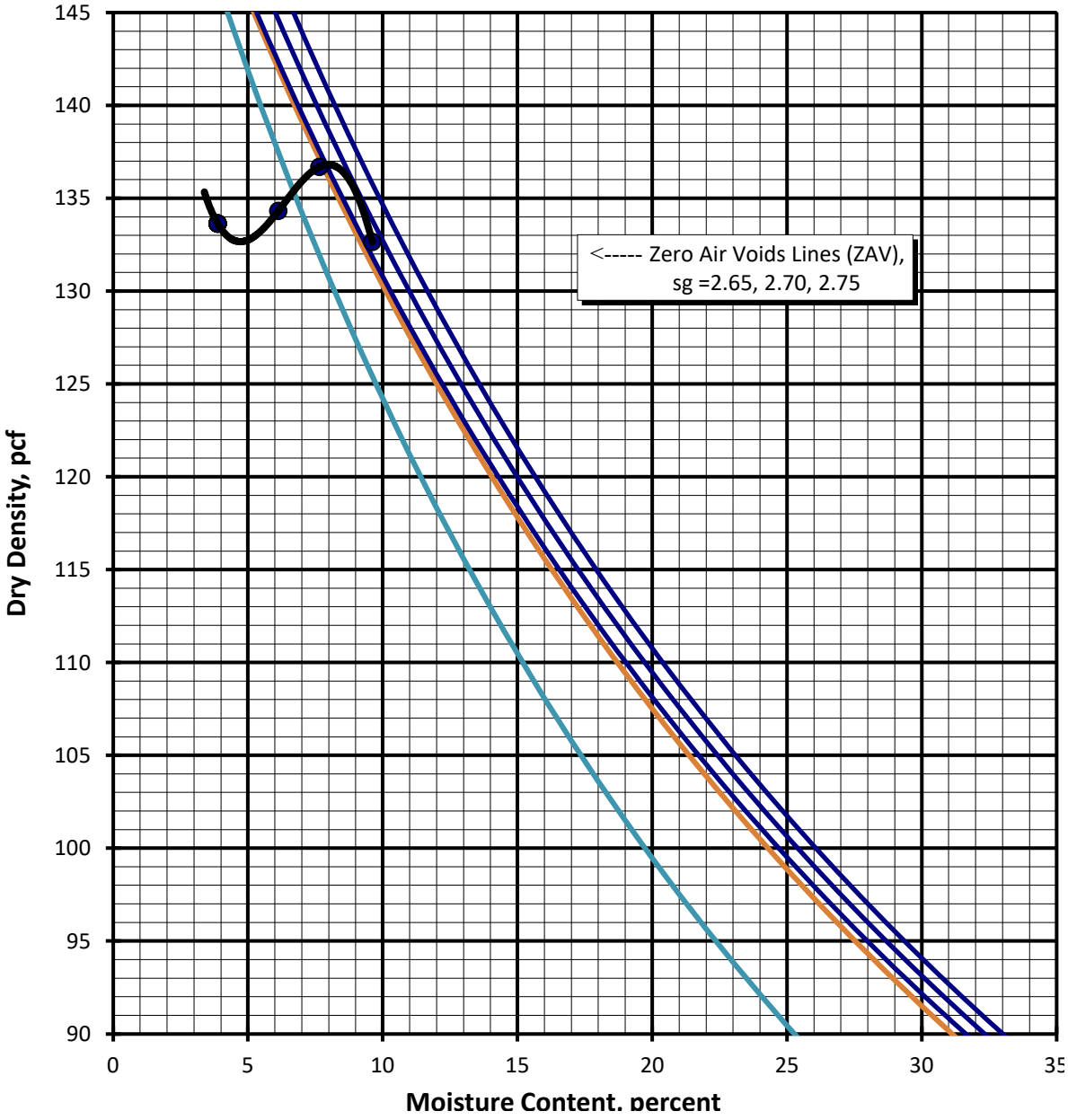
Job Name: Rosamond High School Cafeteria
Sample ID: B-1 @ 0-5'
Location: N/A

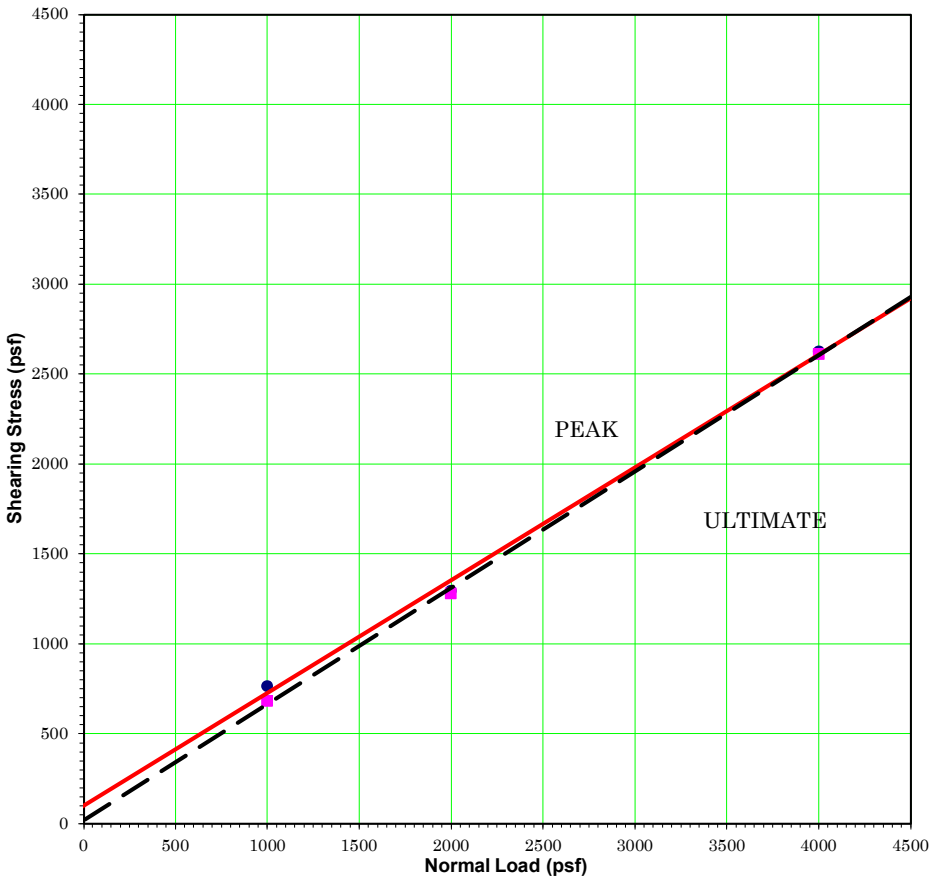
Procedure Used: A
Preparation Method: Moist
Rammer Type: Mechanical

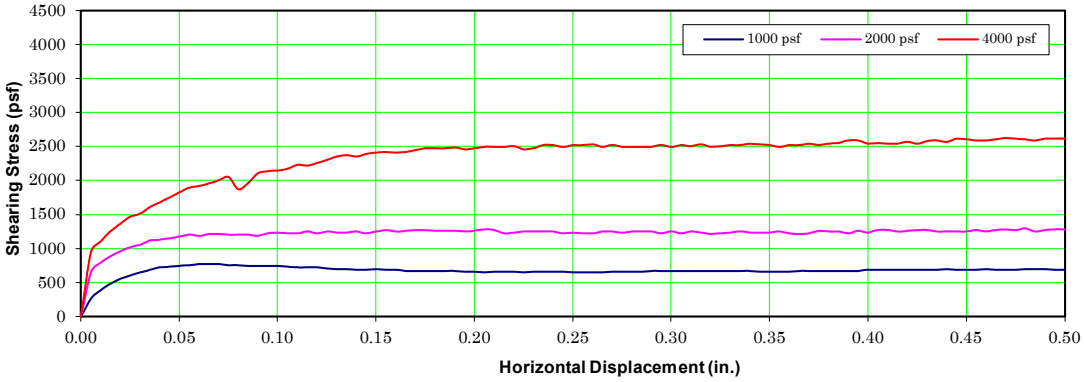
Description: Silty Sand (SM)

Maximum Dry Density: 136.8 pcf
Optimum Moisture: 8%
Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained (Cumulative)
3/4"	0.0
3/8"	0.0
#4	5.7







DIRECT SHEAR DATA*

Sample Location: B1 @ 0-5'
Material: Silty Sand (SM)
Dry Density (pcf): 118.8

	Initial	Final
Moisture Content (%):	9	15.0
Saturation (%):	60	100
	Peak	Ultimate
ϕ Angle of Friction (degrees):	32	33
c Cohesive Strength (psf):	100	10

Test Type: Peak and Ulitimate
Shear Rate (in/min): 0.01

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Rosamond High School Cafeteria

Rosamond, Kern County, California

Earth Systems Pacific

5/24/2022 | 301430-002

TRANSMITTAL LETTER

DATE: May 27, 2022

ATTENTION: **Josh Thomas**

TO: Earth Systems Pacific - Southwest
79-811 B Country Club Drive
Bermuda Dunes, CA 92203

SUBJECT: Laboratory Test Data
Rosamond HS Cafeteria
Your #301430-002, HDR Lab #22-0538LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

Table 1 - Laboratory Tests on Soil Samples

Earth Systems Pacific - Southwest
Rosamond HS Cafeteria
Your #301430-002, HDR Lab #22-0538LAB
27-May-22

Sample ID			B-1 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	>4,400,000
saturated		ohm-cm	2,080
pH			7.3
Electrical Conductivity		mS/cm	0.20
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	17
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	185
potassium	K^{1+}	mg/kg	41
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	59
bicarbonate	HCO_3^{1-}	mg/kg	220
fluoride	F^{1-}	mg/kg	6.1
chloride	Cl^{1-}	mg/kg	13
sulfate	SO_4^{2-}	mg/kg	83
nitrate	NO_3^{1-}	mg/kg	54
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

SECTION 066400
FIBERGLASS REINFORCED PROTECTIVE WALL COVERING

PART 1 - GENERAL

1.1 SUMMARY

A. Section Includes: Fiberglass reinforced plastic (FRP) paneling for wall surfaces, including trim accessories.

1.2 REFERENCES

A. General: Standards listed by reference form a part of this specification section. Standards listed are identified by issuing authority, abbreviation, designation number, title or other designation. Standards subsequently referenced in this Section are referred to by issuing authority abbreviation and standard designation.

B. ASTM International:

1. ASTM D2583 - Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a Barcol Impressor.

2. ASTM D5319 – Standard Specification for Glass-Fiber Reinforced Polyester Wall and Ceiling Panels.

3. ASTM D5420 - Standard Test Method for Impact Resistance of Flat, Rigid Plastic Specimen by Means of a Striker Impacted by a Falling Weight (Gardner Impact).

4. ASTM E84 - Standard Test Method for Surface Burning Characteristics of Building Materials.

D. Crane Composities (Inspired by Kemlite):

1. Installation Guide For FRP Panels #6876.

1.3 ADMINISTRATIVE REQUIREMENTS

A. Preinstallation Meetings: Conduct preinstallation meeting to clarify Project requirements, substrate conditions, manufacturer’s installation instructions and manufacturer’s warranty requirements.

1.4 ACTION SUBMITTALS

A. Product Technical Data: For each type of product required.

B. Shop Drawings: Showing layout, profiles and product components, including anchorage, accessories, finish colors, patterns and textures. Indicate location and dimension of joints and fastener attachment.

C. Samples: Selection and verification samples for finishes, colors and textures. Submit two samples of each type of panel, trim and fastener.

D. Certificates: Product certificates signed by manufacturer certifying materials comply with specified performance characteristics, criteria and physical requirements.

E. Test and Evaluation Reports: Showing compliance with specified performance characteristics and physical properties.

F. Manufacturer's Instructions: Manufacturer's Installation Guide for FRP #6876.

G. Qualifications Statements: For manufacturer and installer.

1.5 CLOSEOUT SUBMITTALS

A. Operation and Maintenance Data: For installed products including maintenance methods and precautions against cleaning materials and methods detrimental to finishes and performance.

B. Warranty: Warranty documents required in this section.

1.6 MAINTENANCE MATERIAL

A. Extra Materials: Deliver to Owner extra materials from same production run as products installed. Package products with protective covering and identify with descriptive labels. Comply with Division 01 Closeout Submittals Section.

1. Quantity: Furnish quantity of units equal to 5% percent of amount installed.
2. Delivery, Storage and Protection: Comply with Owner's requirements for delivery, storage and protection of extra materials.

1.7 QUALITY ASSURANCE

A. Manufacturer Qualifications:

1. Provider of advanced installer training.

B. Installer Qualifications:

1. At least five years experience in the installation of fiberglass reinforced plastic panels.

2. Experience on at least five projects of similar size, type and complexity as this Project.

3. Employer of workers for this Project who are competent in techniques required by manufacturer for installation indicated.

C. Surface-Burning Characteristics: Determined by testing identical products according to ASTM E84 by a testing agency acceptable to authorities having jurisdiction.

1. Flame-Spread Index: 25 (Class A)] or less.

2. Smoke-Developed Index: [450] or less.

1.8 DELIVERY, STORAGE AND HANDLING

A. Delivery: Deliver materials in manufacturer's original, unopened, undamaged containers with identification labels intact. Package sheets on skids or pallets for shipment to project site.

B. Storage and Handling: Store materials protected from exposure to harmful weather conditions and at temperature and humidity conditions recommended by manufacturer. Store panels in a dry indoor location at Project site. Remove any foreign matter from face of panel by using a soft bristle brush, avoiding abrasive action.

1.9 PROJECT CONDITIONS

A. Ambient Conditions:

1. Do not begin installation until building is enclosed, permanent heating and cooling equipment is in operation, and residual moisture from plaster, concrete or terrazzo work has dissipated.

2. During installation, and within 48 hours prior to installation, maintain ambient temperature and relative humidity within limits required by type of panel adhesive used and recommendation of panel adhesive manufacturer.

1.10 WARRANTY

A. Special Warranty: Manufacturer's standard form in which manufacturer agrees to repair or replace FRP panels that fail within specified warranty period.

1. Failures shall include, but not be limited to substantial defects in material and workmanship, rotting, rusting, corrosion, development of structural surface cracks, or requiring painting or refinishing.

2. Warranty Period: Ten years from date of Substantial Completion.

B. Special Warranty: Installer's standard form in which installer agrees to repair or replace FRP panels that fail due to poor workmanship or faulty installation within the specified warranty period.

1. Warranty Period: 1 years from date of Substantial Completion.

PART 2 - PRODUCTS

2.1 FIBERGLASS REINFORCED PLASTIC (FRP) PANELS

A. General: Fiberglass reinforced plastic panels complying with ASTM D5319.

1. Low-Emitting Materials: Comply with testing and product requirements of California Department of Health Services standards for Volatile Organic Emissions.

B. Basis of Design Product: Subject to compliance with requirements provide Crane Composites, Inc.; Class A Fiberglass Reinforced Plastic (FRP) Panels, or approved equal.

C. Product Options:

1. Color: As selected by Architect manufactures full color line of Class A FRP products.

2. Surface Finish: As Approved.

3. Nominal Thickness: 0.09 inch (2.3 m)

4. Wall Panel Size: 4 feet (1.2 m) by 8 feet (2.4 m)

2.2 ACCESSORIES

A. Moldings, Trim and Caps: One-piece extruded polypropylene or PVC, configured to cover panel edges and corners.

1. Color: 85 White.

B. Panel Adhesive: As recommended by panel manufacturer for the required substrates.

1. Adhesive shall have a VOC content of 50 g/L or less.

C. Panel Sealant: As recommended by panel manufacturer.

1. Sealant shall have a VOC content of 250 g/L or less.

2.3 SOURCE QUALITY CONTROL

A. Obtain fiberglass reinforced panels, moldings and other accessories from a single manufacturer.

PART 3 - EXECUTION

3.1 EXAMINATION

A. General: Comply with manufacturer's product data, including product technical bulletins, and installation instructions in product catalogs and product packaging.

B. Verify that substrates previously installed under other sections are acceptable for product installation in accordance with FRP manufacturer's instructions.

1. Examine substrate surfaces to determine that corners are plumb and straight, that surfaces are smooth, sound and uniform, that nails or screw fasteners are countersunk, and that joints and cracks are filled flush and smooth with adjoining surfaces.

2. Do not begin panel installation until substrate surfaces are in satisfactory condition.

3.2 PREPARATION

A. Clean substrates to remove substances that could impair bond of adhesive, including oil, grease, dirt, dust or other contamination.

B. Condition panels by unpacking and placing in installation space no less than 24 hours before installation.

C. Lay out paneling before beginning installation. Locate panel joints to provide equal panel widths at ends of walls and so that trimmed panels at corners are not less than 12 inches (300 mm) wide.

3.3 INSTALLATION

A. General: Comply with panel manufacturer's Installation Guide #6876.

B. Cut and drill panels with carbide tipped saw blades or drill bits, or cut with snips.

C. Install panels with manufacturer's recommended gap for panel field and corner joints.

1. Pre-drill fastener holes in panels, 1/8 inch (3.2 mm) greater in diameter than fastener.

2. Install panels in a full spread of adhesive. For trowel type and application of adhesive, follow adhesive manufacturer's recommendations.

D. Install trim accessories with adhesive and nails or staples. Do not fasten through panels.

E. Sealant:

1. Fill grooves in trim accessories with sealant before installing panels and bed inside corner trim in a bead of sealant.

2. Remove excess sealant and smears as paneling is installed. Clean with solvent recommended by sealant manufacturer and then wipe with clean dry cloths.

3.5 CLEANING

A. Remove temporary coverings and protection of adjacent work areas.

B. Repair or replace any installed products that have been damaged.

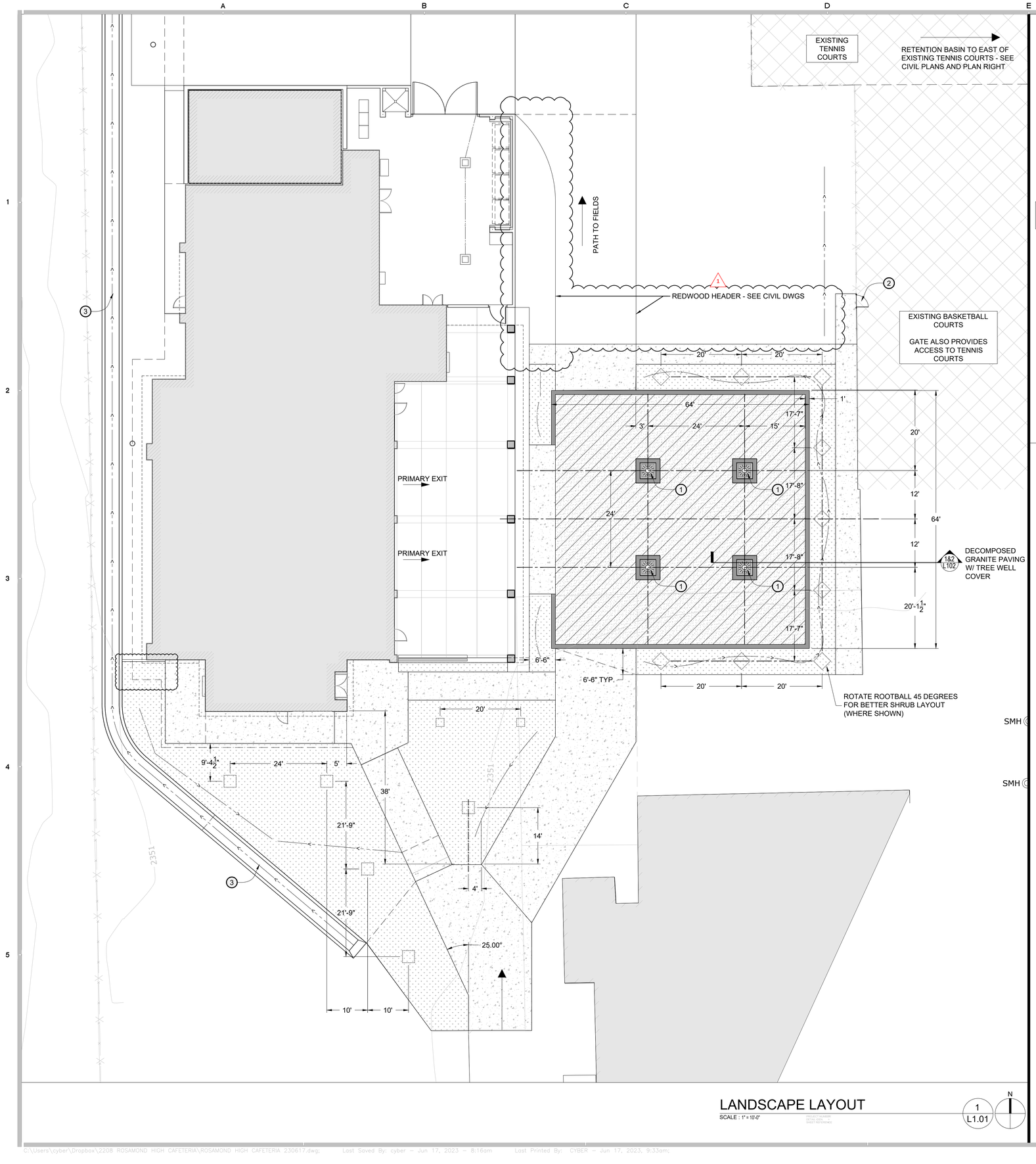
C. Clean installed panels in accordance with manufacturer's instructions prior to Owner's acceptance.

D. Remove and lawfully dispose of construction debris from project site.

3.6 PROTECTION

A. Protect installed product and finish surfaces from damage during construction.

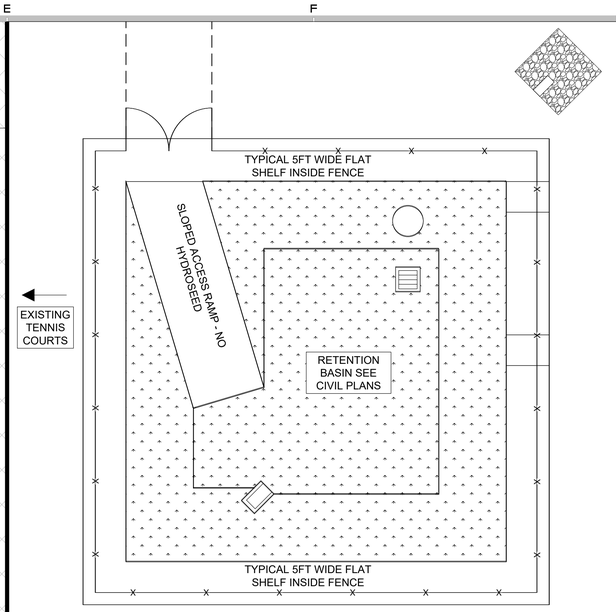
END OF SECTION



LANDSCAPE LAYOUT

SCALE : 1" = 10'-0"

1 / L1.01



TOTAL HYDROSEED AREA: 3,353sf

RETENTION BASIN

SCALE : 1" = 10'-0"

2 / L1.01

LANDSCAPE LEGEND

1. TREE PLANTING WITH PRECAST CONCRETE TREE WELL - SEE DETAIL 2 / SHEET L1.02
2. EXISTING GATE
3. DRAINAGE CHANNEL - SEE CIVIL DWGS

LANDSCAPE LEGEND

- STABILIZED DECOMPOSED GRANITE PAVING - SEE ARCH DETAIL 1 / A1.02
- LANDSCAPE AREAS
- HYDROSEED AREA
- TREE BOX LOCATIONS

TREE WELL COVERS

"A" STYLE SQUARE PRECAST CONCRETE TREE WELL COVER
MODEL: QS-48A-TG - QTY: 4 - COLOR: NATURAL
SEE DETAIL 1 / L1.02

TREE GRATES BY QCP CORPORATION, NORCO, CALIFORNIA
WEBSITE: https://qcp-corp.com/detail/style-square/1148
WEBSITE: https://qcp-corp.com/detail/style-round/1146
PHONE: 1 (866) 703 - 3434
EMAIL: SALES @QCP-CORP.COM
SALES REP: JUSTIN WAIT - JUSTIN.WAIT@QCP-CORP.COM

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. L1.01, (A#03-122536, DATED 03/24/2023)

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architecture planning interiors

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ARCHITECT

LICENSED ARCHITECT
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

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E-mail: info @ mbeall.com

LICENSED LANDSCAPE ARCHITECT
MARK D. BEALL
CA NO 3640
REN. 04/30/23
STATE OF CALIFORNIA

Drawn by

Checked by

Revisions

No.	Date	Description
1	06-30-2023	BID CLARIFICATION NO.1

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ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

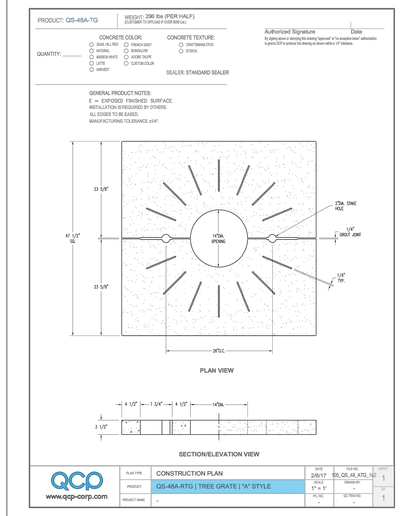
LANDSCAPE LAYOUT

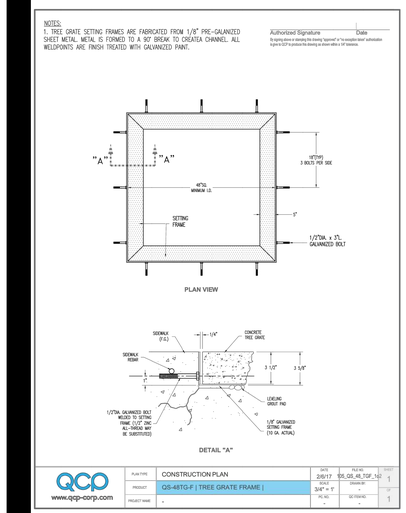
Job No. 2990

Date 06-07-22

L1.01

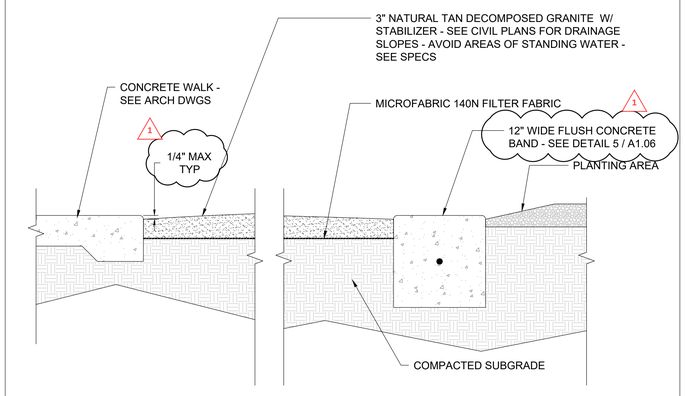
BID CLARIFICATION NO.1 (06-20-2023)





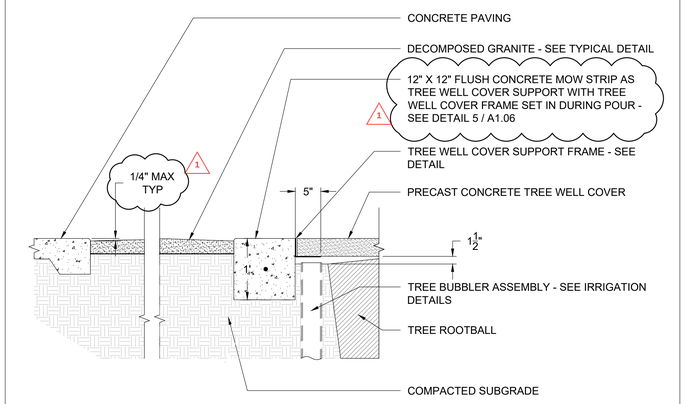
3 48" SQUARE PRECAST TREE GRATE

AS NOTED



1 STABILIZED DECOMPOSED GRANITE PAVING (BASE BID)

1-1/2" = 1'-0"



2 CONCRETE TREE WELL COVER IN DG AREA (BASE BID)

1" = 1'-0"

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C-26053
Rnw. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

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STATE OF CALIFORNIA

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

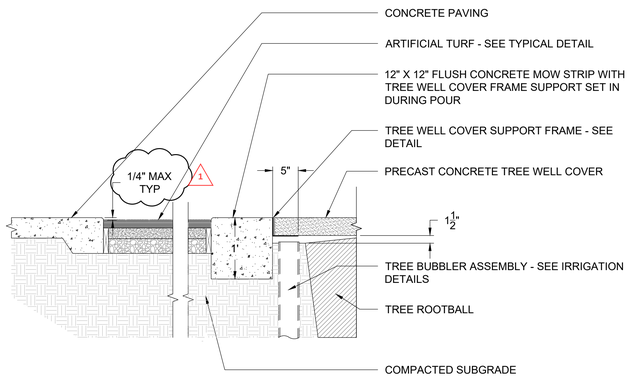
2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

BASE BID DETAILS

Job No.
2990

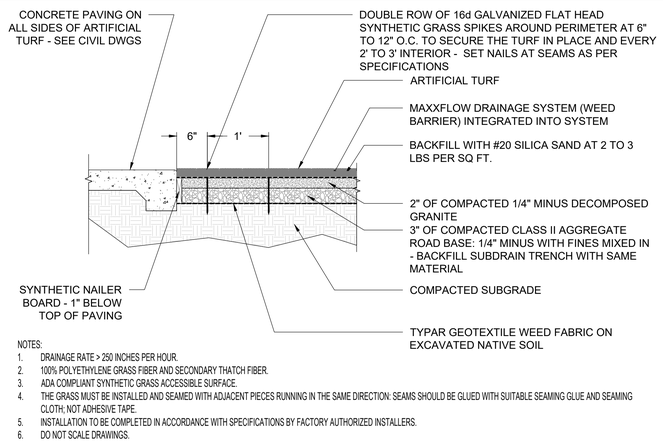
Date
06-07-22

L1.02



CONCRETE TREE WELL IN ARTIFICIAL TURF (BID ALTERNATE NO. 1)

3 — 1" = 1'-0"



NOTES:
1. DRAINAGE RATE > 250 INCHES PER HOUR.
2. 100% POLYETHYLENE GRASS FIBER AND SECONDARY THATCH FIBER.
3. ADA COMPLIANT SYNTHETIC GRASS ACCESSIBLE SURFACE.
4. THE GRASS MUST BE INSTALLED AND SEAMED WITH ADJACENT PIECES RUNNING IN THE SAME DIRECTION; SEAMS SHOULD BE GLUED WITH SUITABLE SEAMING GLUE AND SEAMING CLOTH; NOT ADHESIVE TAPE.
5. INSTALLATION TO BE COMPLETED IN ACCORDANCE WITH SPECIFICATIONS BY FACTORY AUTHORIZED INSTALLERS.
6. DO NOT SCALE DRAWINGS.

ARTIFICIAL TURF (BID ALTERNATE NO. 1)

4 — 1" = 1'-0"

LANDSCAPE LEGEND

ARTIFICIAL TURF - SEE SPECS

LANDSCAPE AREAS

TREE BOX LOCATIONS

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. L1.03, (A#03-122536, DATED 03/24/2023)

BENCHES (BID ALTERNATE NO. 2)

COATED STEEL PLAYERS BENCH WITHOUT BACK - IN GROUND BENCH INSTALLATION ADVANTAGE FINISH

MODEL: BYNB6G-PA - QTY: 2 - COLOR: BLACK

TABLE/BENCH SET (BID ALTERNATE NO. 2)

46" SQUARE TOP TABLE W/ PERFORATED METAL AND 4 BENCH SEATS ADVANTAGE FINISH - IN GROUND MOUNT

MODEL: TSQP464G-PS - QTY: 10 - COLOR: BLACK

46" SQUARE TOP TABLE W/ PERFORATED METAL AND 3 BENCH SEATS (ADA ACCESSIBLE) ADVANTAGE FINISH - IN GROUND MOUNT

MODEL: TSQP463HG-PS - QTY: 4 - COLOR: BLACK

46" SQUARE TOP TABLE W/ PERFORATED METAL AND 2 BENCH SEATS ADVANTAGE FINISH - IN GROUND MOUNT

MODEL: TSQP462G-PS - QTY: 2 - COLOR BLACK

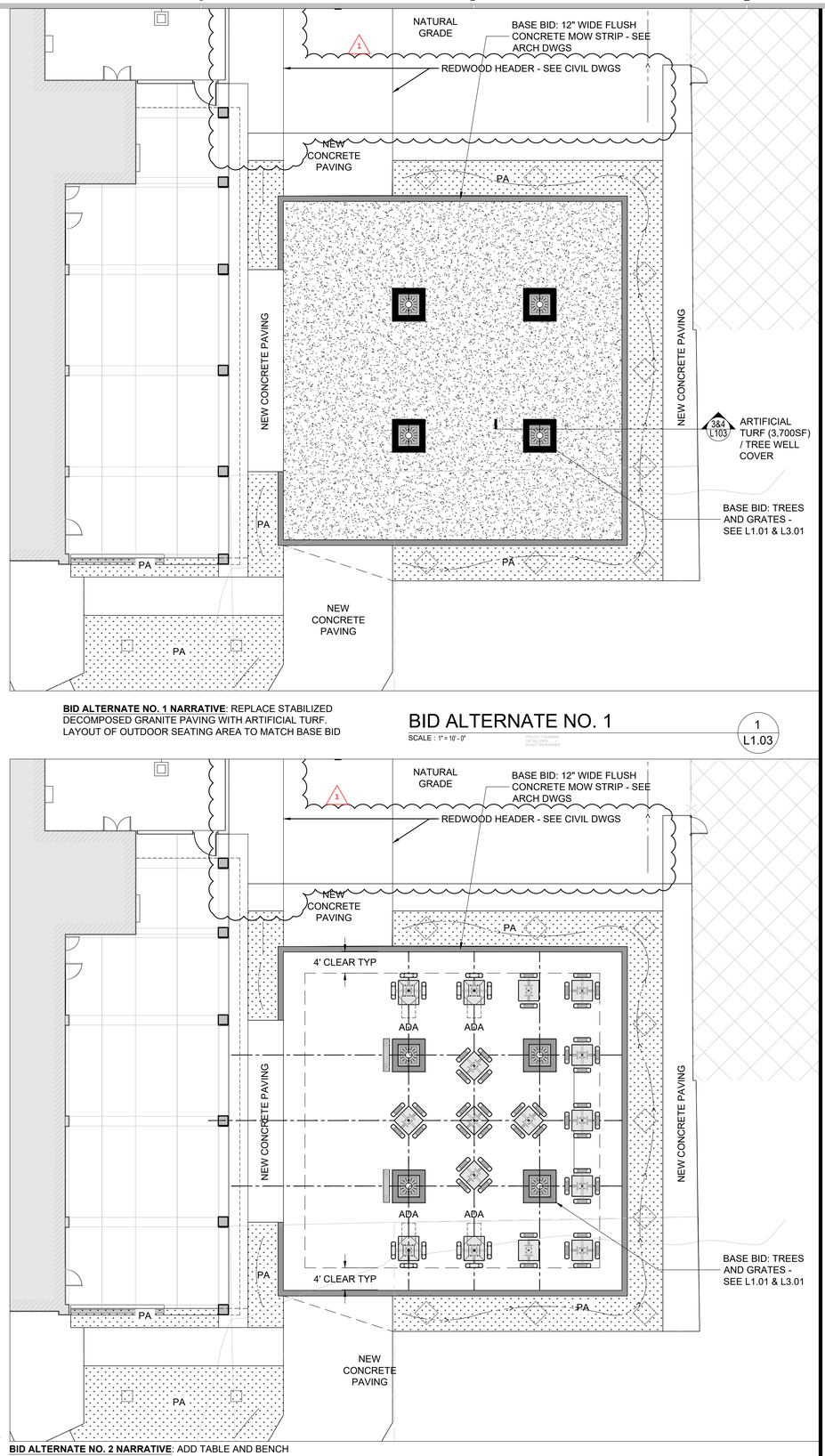
BID ALTERNATE NO. 2 OPTIONS

BID ALTERNATE NO. 2 AS PART OF BASE BID: IN GROUND FURNITURE INSTALLATION IN DECOMPOSED GRANITE - SEE DETAIL

BID ALTERNATE NO. 2 AS PART OF BID ALTERNATE NO. 1: IN GROUND FURNITURE INSTALLATION IN ARTIFICIAL TURF

ALL FURNITURE BY BELSON OUTDOORS

BENCH WEBSITE: https://www.belson.com/Coated-Steel-Players-Benches-without-Back-Punched-Steel
TABLE WEBSITE: https://www.belson.com/Square-Coated-Steel-Pedestal-Tables-Punched-Steel
CONTACT: 1 (800) 323 - 5664 / HOURS: M-F, 8AM TO 4:30PM CT



BID ALTERNATE NO. 1 NARRATIVE: REPLACE STABILIZED DECOMPOSED GRANITE PAVING WITH ARTIFICIAL TURF. LAYOUT OF OUTDOOR SEATING AREA TO MATCH BASE BID

BID ALTERNATE NO. 1

SCALE : 1" = 10'-0" — 1 / L1.03

BID ALTERNATE NO. 2 NARRATIVE: ADD TABLE AND BENCH FURNITURE AS INDICATED. IN GROUND MOUNT - SEE DETAIL

BASE BID: INSTALLATION TO BE IN STABILIZED DECOMPOSED GRANITE PAVING.

BID ALTERNATE NO. 1 ACCEPTANCE: FURNITURE INSTALLATION TO BE IN ARTIFICIAL TURF AREA IF BID ALTERNATE NO. 1 IS ACCEPTED.

BID ALTERNATE NO. 2

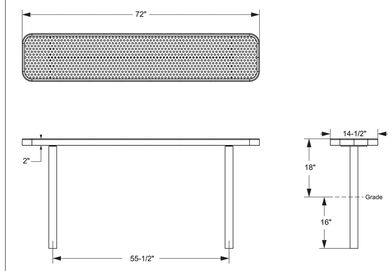
SCALE : 1" = 10'-0" — 1 / L1.03 — N

627 Amersale Drive
Naperville, IL 60563
Phone: (800) 323-5664
Fax: (630) 897-0573
sales@belson.com

BELSON OUTDOORS

Model # BYNB6G-PA — **Dimension Sheet**

6' Coated Steel Players Benches without Back



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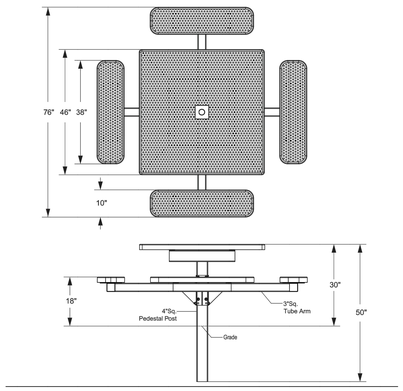
Naperville, IL 60563
Phone: (800) 323-5664
Fax: (630) 897-0573
sales@belson.com

BELSON OUTDOORS

Model # TSQP464G-PS — **Dimension Sheet**

Square Coated Steel Pedestal Tables



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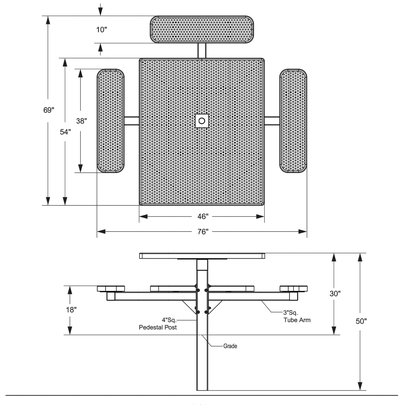
Naperville, IL 60563
Phone: (800) 323-5664
Fax: (630) 897-0573
sales@belson.com

BELSON OUTDOORS

Model # TSQP463HG-PS — **Dimension Sheet**

Square Coated Steel Pedestal Tables



www.belson.com

FURNITURE SPEC SHEETS (BID ALTERNATE NO. 2)
ROSA 'RED MEIDILAND' (RED CARPET ROSE)

AGENCY

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architecture planning interiors

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LICENSED ARCHITECT — C-26053 — Ren. Date 08/31/2023 — STATE OF CALIFORNIA

CONSULTANT

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landscape architecture
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Los Angeles, CA 90028-7525
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Fax: 323-465-1458
E-mail: info @ mbeall.com

LICENSED LANDSCAPE ARCHITECT — MARK D. BEALE — CA NO 3640 — REN. 04/30/23 — STATE OF CALIFORNIA

Drawn by

Checked by

Revisions

No.	Date	Description
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ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

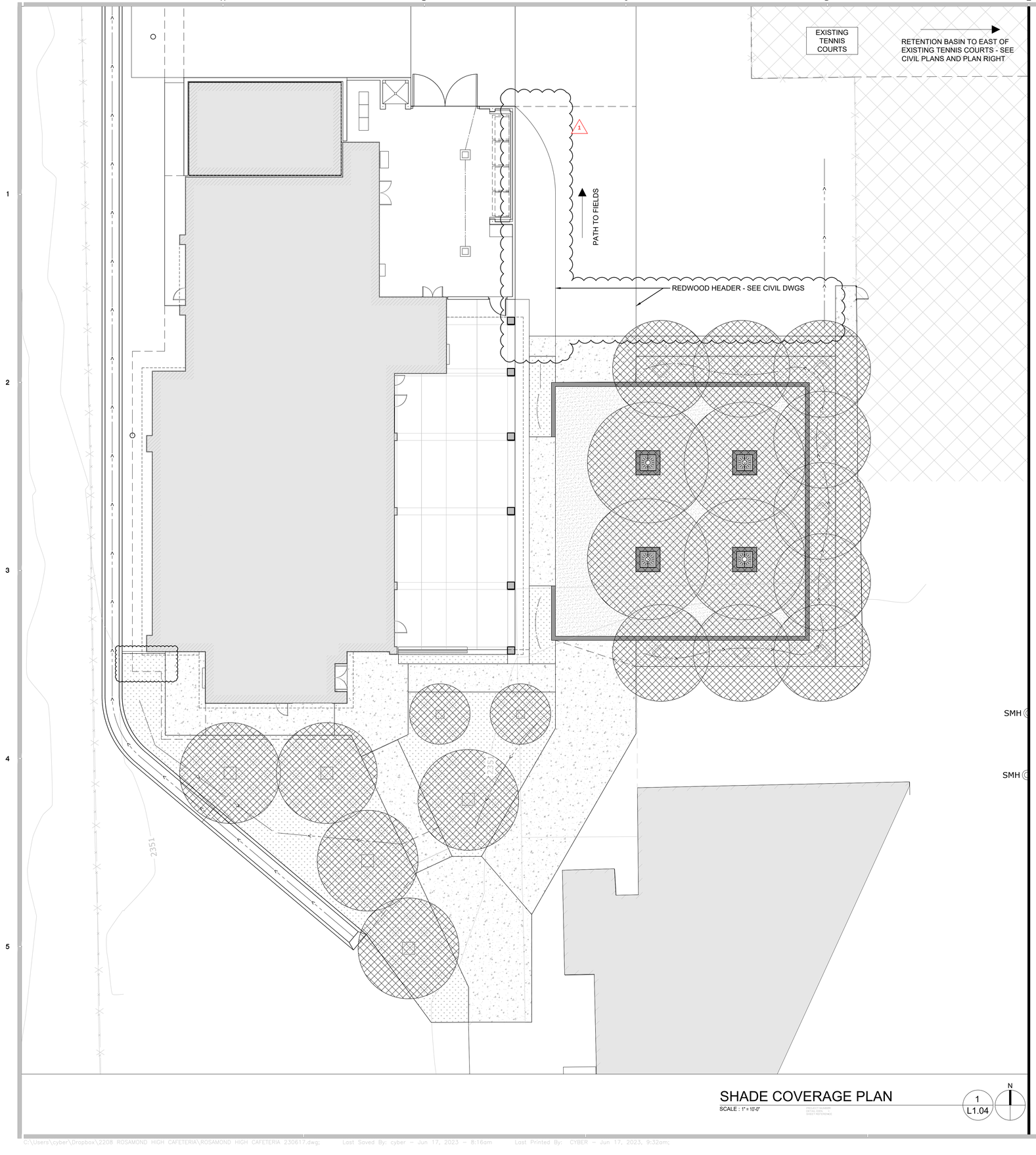
BID ALTERNATES NO. 1 & 2

Job No. 2990

Date 06-07-22

L1.03

BID CLARIFICATION NO.1 (06-20-2023)



SHADE COVERAGE PLAN

SCALE : 1" = 10'-0"

1 / L1.04

N

AREA CALCULATIONS - BASE BID

TOTAL HARDSCAPE AND LANDSCAPE AREA 13,906
(HARDSCAPE = CONCRETE, DECOMPOSED GRANITE)

SHADE AREA 8,736

RATIO OF SHADE AREA TO HARDSCAPE/LANDSCAPE: 63%

EXCEEDS 20% MINIMUM CAL GREEN SECTION 5.106.12 SHADE REQUIREMENT

SHADE TREE LEGEND

BASE BID

BOTANICAL NAME	PROJECTED 15YR CANOPY	QTY
CHITALPA 'PINK CLOUD'	25FT	9
COTINUS COGGYGRIA	15FT	2
GLEDITSIA 'SHADEMASTER'	30FT	4
OLEA EUROPAEA 'SWAN HILL'	25FT	5

TOTAL NUMBER OF TREES: 20

TOTAL SHADE SQUARE FOOTAGE: 8,601

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. L1.04, (A#03-122536, DATED 03/24/2023)

AGENCY

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architecture planning interiors

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ARCHITECT

LICENSED ARCHITECT
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

Mark Beall associates
landscape architecture
7033 W Sunset Blvd, Suite 216
Los Angeles, CA 90028-7525
Cell: 323-683-1450
Fax: 323-465-1458
E-mail: info @ mbeall.com

LICENSED LANDSCAPE ARCHITECT
MARK D. BEALL
CA NO 3640
REN. 04/30/23
STATE OF CALIFORNIA

Drawn by

Checked by

Revisions

No.	Date	Description
1	06-30-2023	BID CLARIFICATION NO.1

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

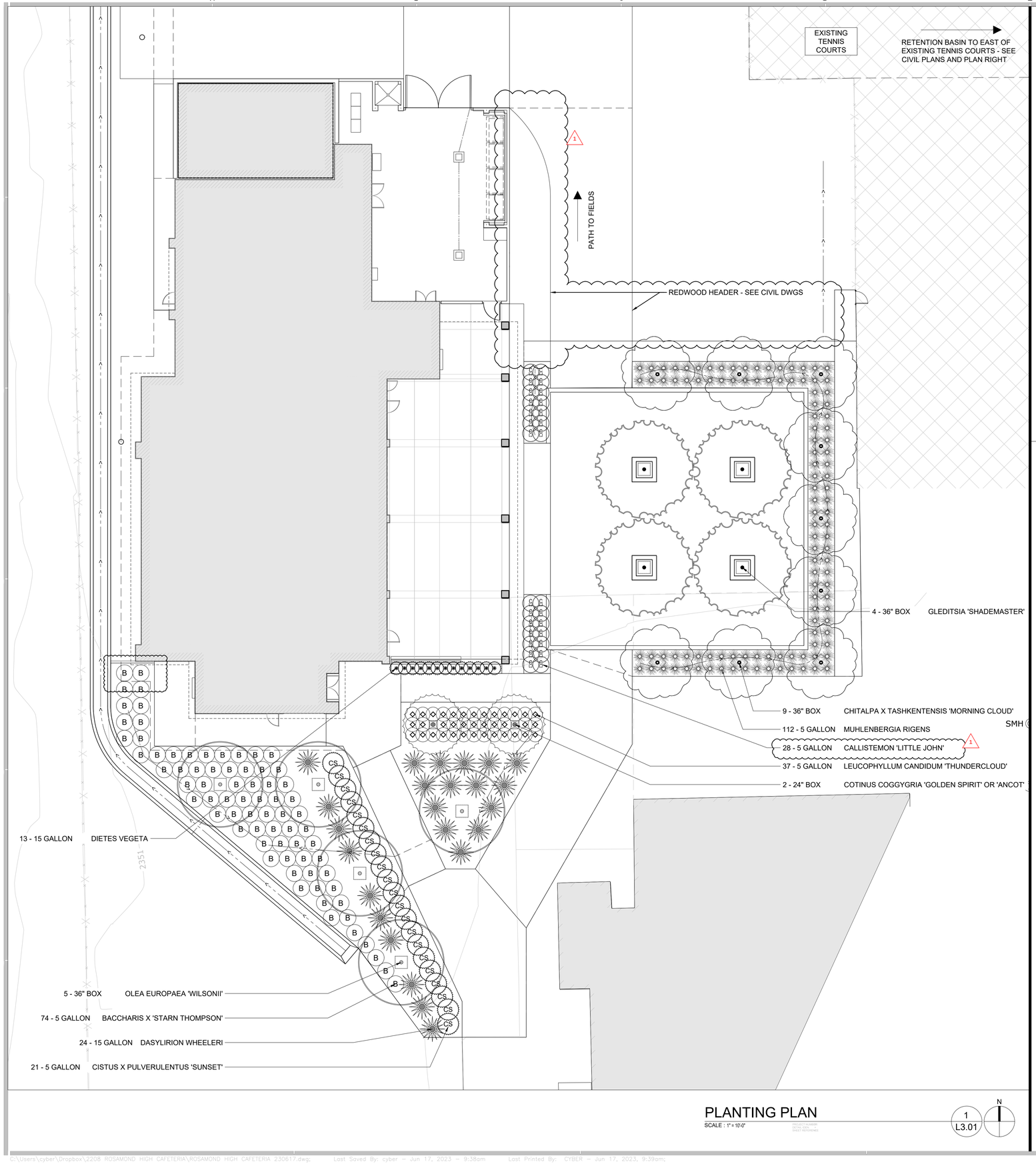
CAL GREEN
SHADE CALCULATION

Job No. 2990

Date 06-07-22

L1.04

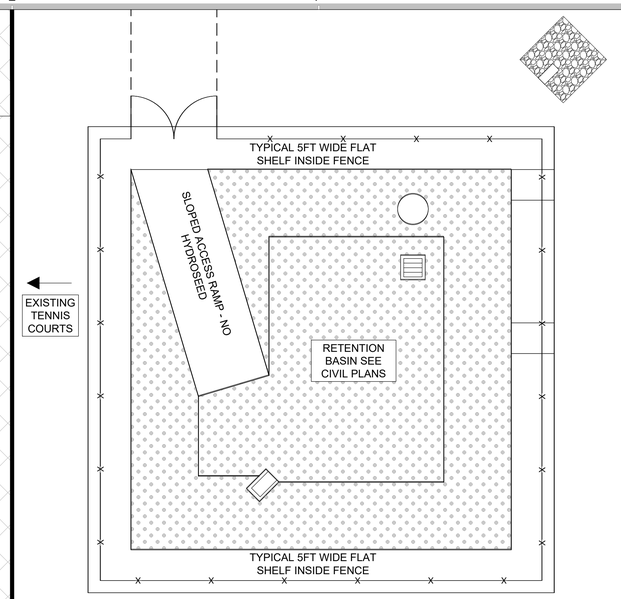
BID CLARIFICATION NO.1 (06-20-2023)



PLANTING PLAN

SCALE : 1" = 10'-0"

1 / L3.01



TOTAL HYDROSEED AREA: 3,353sf

BIOSWALE TEMPORARY or NON IRRIGATED MIX

This composition of native California grasses with a flowering component will formulate a very effective, filtering, functional bioswale when planted into drainage basins, bioswales and other open areas that are on occasion inundated with water for a very short term basis.

SPECIES	COMMON NAME	BULK #'s/ACRE	MIN % PLS*
Agrostis exarata	Spike bentgrass	2.00	76
Anemopsis californica	Yerba mansa	1.00	44
Deschampsia danthoniodes	Annual hairgrass	2.00	72
Elymus triticoides 'Rio'	Rio creeping wild rye	5.00	72
Eschscholzia californica	California poppy	1.00	83
Hordeum brachyantherum	Meadow barley	6.00	72
Hordeum intercedens	Little barley	4.00	72
Lasthenia glabrata	Goldfields	0.50	89
Melica imperfecta	Coast melic	2.00	60
Muhlenbergia rigens	Deergrass	0.50	32
Plantago insularis	Plantain	20.00	74
Sisyrinchium bellum	Blue eyed grass	1.00	78
Stipa pulchra	Purple needle grass	4.00	73
		49.00	

* MIN % PLS (Pure Live Seed) = Seed Parity x Germination Rate

Seeding rate: 49.00 lbs per acre
Height: 12-36 inches
Emergence: 10-21 days
Establishment: 55 days to 80% cover after emergence

WILDFLOWER AND NATIVE GRASS SEED: AS PROVIDED BY S&S SEEDS, CARPENTERIA, CA (805) 684-0436. FRESH, CLEAN, AND DRY NEW SEED, OF MIXED SPECIES
(SEE SPECIFICATIONS FOR HYDROSEED PROCEDURES)

CALLISTEMON 'LITTLE JOHN'

RETENTION BASIN - PLANTING

SCALE : 1" = 10'-0"

1 / L3.01

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LICENSED ARCHITECT C-26053 Renewal Date: 08/31/2023 STATE OF CALIFORNIA

CONSULTANT

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LICENSED LANDSCAPE ARCHITECT MARK D. BEALL CA NO 3640 REN. 04/30/23 STATE OF CALIFORNIA

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

PLANTING PLAN

Job No. 2990

Date 06-07-22

L3.01

BID CLARIFICATION NO.1 (06-20-2023)

GENERAL NOTES

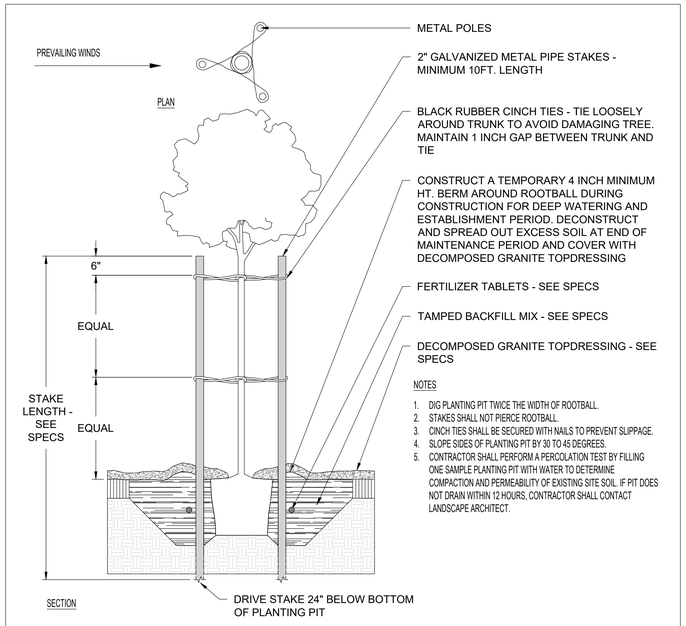
1. See architect/civil engineer's drawings for additional construction and utility routing information.
2. Landscape Contractor shall be responsible for making himself familiar with all understructure utilities, pipes, and structural conditions. Landscape Contractor shall take sole responsibility for any cost incurred due to damage of said utilities.
3. Landscape Contractor shall not willfully proceed with construction as designed when it is obvious that unknown obstructions exist that may not have been known during design or if signage is not located as indicated on the plans. Such conditions shall be immediately brought to the attention of the Landscape Architect (323) 465-1450. The Landscape Contractor shall assume full responsibility for all necessary revisions due to failure to give such notification.
4. Landscape Contractor shall be responsible for any coordination with other subcontractors, as required, to accomplish irrigation and planting operations. Landscape Contractor shall coordinate with the General Contractor for all necessary sleeving and conduits, as shown on plan.
5. Contractor shall submit a source list of nursery names and addresses for all plant materials for approval from the Landscape Architect.

CONSTRUCTION NOTES

1. All vaults, boxes, and other existing in-ground structures that are designated to be inside of new planting areas shall have their locations adjusted or modified as needed to prevent trip hazards or avoid conflict with planting design.
2. Install root barriers along edges of paving that are within 8' of new trees.
3. Grades for lawn and shrub areas adjacent to paving areas shall be set 1" below top of paving. All paved areas shall drain away from buildings and drainage shall be directed to planting areas or existing drains as directed by civil engineer.

PLANTING NOTES

1. Contractor shall break up, rip, turn over and otherwise cultivate to a depth of at least 12" either mechanically or by hand the existing soils that are heavily compacted to achieve 80% compaction levels for planting areas.
2. Soil amendments within specifications are for bidding purposes only. A soil report's recommendations shall govern.
3. Install root barriers along edges of paving that are within 48" of new trees.
4. Grades for lawn and shrub areas adjacent to paving areas shall be set 1" below top of paving. All paved areas shall drain away from buildings and drainage shall be directed to planting areas or existing drains as directed by civil engineer.
5. All new and existing planting areas shall be free of construction debris, rocks, form boards, weeds and any and all existing plants or turf designated to be removed.
6. Tree locations shown on plans with dimensions shall be placed accurately and shall not be adjusted without approval of the Landscape Architect.
7. A representative sample of the botanical name tag from the nursery shall be provided for each species of plant to correctly indentify it and shall remain attached to the plant until final inspection.
8. All planting areas shall have a wood or mineral mulch or topdressing as designated on the plans and as per the specifications.



1 TRIPLE TREE STAKING - METAL POLES
1/2" = 1'-0"

2 SHRUB PLANTING DETAIL
NOT TO SCALE

3 STABILIZED DECOMPOSED GRANITE PATH (NOT FIRE LANE)
1-1/2" = 1'-0"

PLANTING LEGEND

NOTE: THE PLANT LIST IS FOR THE CONTRACTOR'S CONVENIENCE ONLY. IF DISCREPANCIES EXIST BETWEEN THE PLANT LIST AND PLANTING PLANS,THE PLANS SHALL GOVERN.

BASE BID (SEE SHEET L3.01)

SYM	QTY	SIZE	HEIGHT	WIDTH	CALIPER	BOTANICAL NAME	COMMON NAME	REMARKS	WUCOLS H20
		TREES							
	9	36" BOX				CHITALPA X TASHKENTENSIS 'PINK CLOUD'	PINK CLOUD CHITALPA	MATCHING STANDARD	L
	2	24" BOX				COTINUS COGGYGRIA 'GOLDEN SPIRIT'	GOLDEN LEAF SMOKE TREE	MATCHING LOW BRANCH	L
	4	36" BOX				GLEDITSIA TRIANCANTHOS 'SHADEMASTER'	SHADEMASTER HONEYLOCUST	MATCHING STANDARD	L
	5	36" BOX				OLEA EUROPAEA 'SWAN HILL'	SWAN HILL FRUITLESS OLIVE	LOW BRANCHING	L
		SHRUBS							
1	74	5 GALLON				BACCHARIS 'STARN THOMPSON'	STERILE HYBRID COYOTE BRUSH		L
	28	5 GALLON				CALLISTEMON 'LITTLE JOHN'	DWARF LITTLE JOHN BOTTLEBRUSH		M
	21	5 GALLON				CISTUS X PULVERULENTUS 'SUNSET'	SUNSET ROCKROSE		M
	24	15 GALLON				DASYLIRION WHEELERI	DESERT SPOON		L
	13	5 GALLON				DIETES VEGETA	FORTNIGHT LILY		M
	37	5 GALLON				LEUCOPHYLLUM CANDIDUM 'THUNDERCLOUD'	THUNDERCLOUD TEXAS SAGE		L
	72	5 GALLON				MUHLENBERGIA RIGENS	DEER GRASS		M
		HYDROSEED	3,353 SQUARE FEET			CUSTOMIZED SEE MIX - SEE SHEET L3.01		BY S&S SEED	

ALL TREES SHALL BE TAGGED BY THE LANDSCAPE ARCHITECT - TREE NURSERY SOURCES SHALL BE AT THE LANDSCAPE ARCHITECT'S DISCRETION

WATER USE BASED ON STATE OF CALIFORNIA WATER USE CLASSIFICATION OF LANDSCAPE SPECIES (WUCOLS)

VL - VERY LOW USE, L = LOW, M = MEDIUM, H = HIGH

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C-26053
Ren. Date 08/31/2023
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LICENSED LANDSCAPE ARCHITECT
MARK D. BEALL
CA NO 3640
REN. 04/30/23
STATE OF CALIFORNIA

Drawn by

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL
CAFETERIA BUILDING
2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

PLANTING LEGENDS, DETAILS, AND NOTES

Job No. 2990

Date 06-07-22

L3.02

MATCHLINE - SEE SHEET C3.02

GRADING AND DRAINAGE CONSTRUCTION NOTES

1. SAWCUT PCC OR AC PAVEMENT TO LIMITS SHOWN.
2. CONSTRUCT PCC WALKWAY (4" THICK) PER DETAIL "A" ON SHEET C5.01.
3. CONSTRUCT VEHICULAR CONCRETE (6" THICK) PER DETAIL "E" ON SHEET C5.01.
4. CONSTRUCT 5'-WIDE, 4" DEEP CONCRETE GUTTER PER DETAIL "F" ON SHEET C5.01.
5. CONSTRUCT 5'-WIDE, VARIABLE DEPTH CONCRETE GUTTER PER DETAIL "G" ON SHEET C5.01.
6. CONSTRUCT NEW CONCRETE DRIVEWAY TYPE "B" PER KERN COUNTY DEVELOPMENT STANDARD PLATE NO. R-54. SEE DETAIL "QQ" ON SHEET C5.03.
7. CONSTRUCT DETENTION BASIN PER KERN COUNTY DRAINAGE DEVELOPMENT STANDARD PLATE NO. D-1. SEE DETAIL "JJ" ON SHEET C5.02.
8. CONSTRUCT CHAIN LINK FENCE AROUND RETENTION BASIN PER KERN COUNTY DRAINAGE DEVELOPMENT STANDARD PLATE NO. D-10. SEE DETAIL "LL" ON SHEET C5.03. MODIFIED TO BE BLACK VINYL CHAIN LINK PER ARCHITECT'S PLAN. CONSTRUCT CURBING AT BASE OF FENCE PER PER KERN COUNTY DRAINAGE DEVELOPMENT STANDARD PLATE NO. D-2. SEE DETAIL "KK" ON SHEET C5.02. OMIT CURB WHERE FENCE IS ON TOP OF CONCRETE SPILLWAY.
9. CONSTRUCT DOUBLE CHAIN LINK GATE PER KERN COUNTY DRAINAGE DEVELOPMENT STANDARD PLATE NO. D-11. SEE DETAIL "MM" ON SHEET C5.03. MODIFIED TO BE BLACK VINYL CHAIN LINK PER ARCHITECT'S PLAN.
10. CONSTRUCT DECOMPOSED GRANITE PER ARCHITECT'S PLAN. SEE ARCHITECT'S PLAN FOR CONCRETE BANDING WHERE APPLICABLE.
11. CONSTRUCT LANDSCAPING PER LANDSCAPE ARCHITECT'S PLAN.
12. CONSTRUCT FENCE/GATE PER ARCHITECT'S PLAN. SEE ARCHITECT'S PLAN FOR LOCATION OF MOW STRIP UNDER FENCE.
13. CONSTRUCT 14'-WIDE CONCRETE RIBBON GUTTER PER DETAIL "H" ON SHEET C5.01.
14. CONSTRUCT 10'X10' GROUTED RIP RAP PAD PER DETAIL "J" ON SHEET C5.01.
15. CONSTRUCT DECOMPOSED GRANITE FIRE LANE WITH REDWOOD HEADER EDGE PER DETAIL 3 ON ARCHITECTURAL SHEET A1.04FA.
16. CONSTRUCT NEW AC AND BASE TO MATCH EXISTING AC AND BASE THICKNESSES.
17. CONSTRUCT NEW STANDARD SIDEWALK TYPE 2 PER KERN COUNTY DEVELOPMENT STANDARD PLATE NO. R-65. SEE DETAIL "RR" ON SHEET C5.03.
18. CONSTRUCT CONCRETE TRANSITION BETWEEN WALKWAY AND VEHICULAR CONCRETE SECTIONS PER DETAIL "K" ON SHEET C5.01.

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EXISTING UTILITY COVER ADJUSTMENT NOTE

CONTRACTOR SHALL ADJUST ALL EXISTING UTILITY PULL BOXS, CLEANOUTS, VAULTS AND OTHER SURFACE LIDS/COVERS WITHIN THE PROJECT AREA TO FINISH SURFACE ELEVATION.

*EXISTING SLOPES AND ELEVATIONS NOTE

ALL EXISTING SLOPES AND ELEVATIONS DENOTED WITH A "*" WERE TAKEN FROM AN AERIAL TOPO DONE BY OTHERS. NO DETAILED TOPO WAS DONE IN THIS AREA. IMPROVEMENTS IN THESE AREAS ARE INTENDED TO BE CONSTRUCTED TO MATCH EXISTING GRADES. CONTRACTOR TO FIELD VERIFY ELEVATIONS AND NOTIFY ENGINEER OF ANY CONFLICTS WITH DESIGN.

N W E S

SCALE: 1"=20'
0 20 40 60

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ARCHITECT

LICENSED ARCHITECT C-26053 Ren. Date 08/31/2025 STATE OF CALIFORNIA

CONSULTANT

REGISTERED PROFESSIONAL ENGINEER JOSIAH D. JENSON NO 77454 CIVIL STATE OF CALIFORNIA

ECG Encompass Consultant Group
25115 AVE STANFORD, SUITE #330, SANTA CLARITA, CA 91355
PHONE: 661.600.9367 WEBSITE: WWW.ECGCIVIL.COM

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

GRADING AND PAVING PLAN

Job No. 2990

Date 01.30.2023

C3.01

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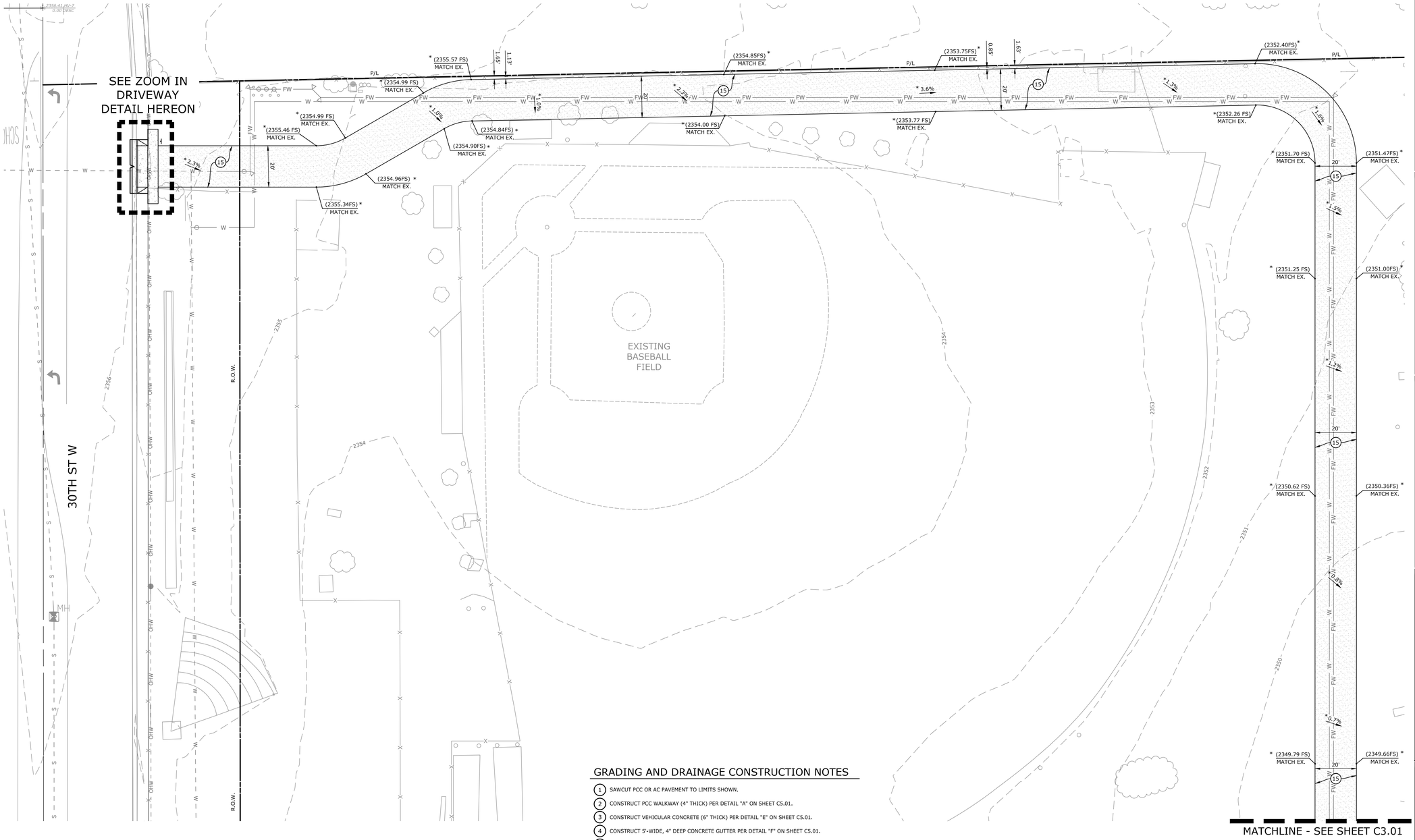

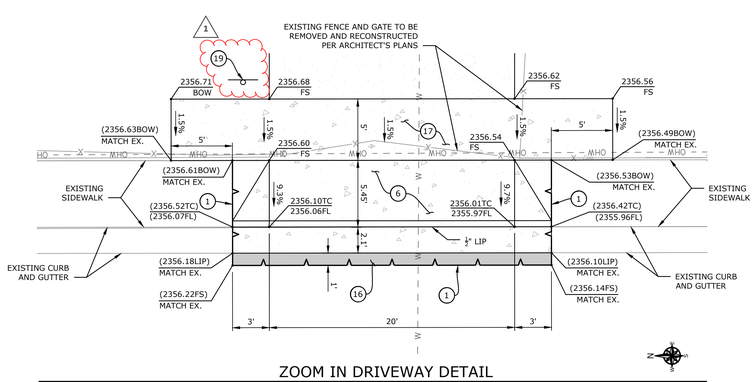

ZOOM IN DRIVEWAY DETAIL

SCALE: 1"=5'

GRADING AND DRAINAGE CONSTRUCTION NOTES

1. SAWCUT PCC OR AC PAVEMENT TO LIMITS SHOWN.
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18. CONSTRUCT CONCRETE TRANSITION BETWEEN WALKWAY AND VEHICULAR CONCRETE SECTIONS PER DETAIL "K" ON SHEET C5.01.
19. CONSTRUCT "RIGHT TURN ONLY" SIGN AND POST.

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N W E S

SCALE: 1"=20'

0 20 40 60

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CONSULTANT

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ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

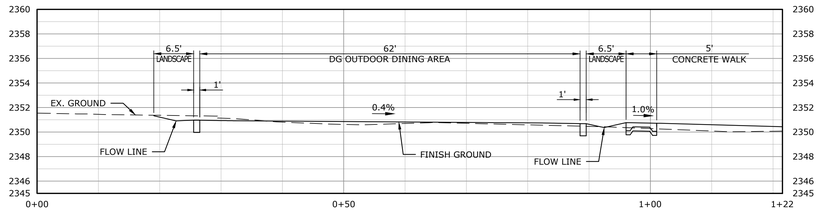
GRADING AND PAVING PLAN

Job No. 2990

Date 06.16.2023

C3.02

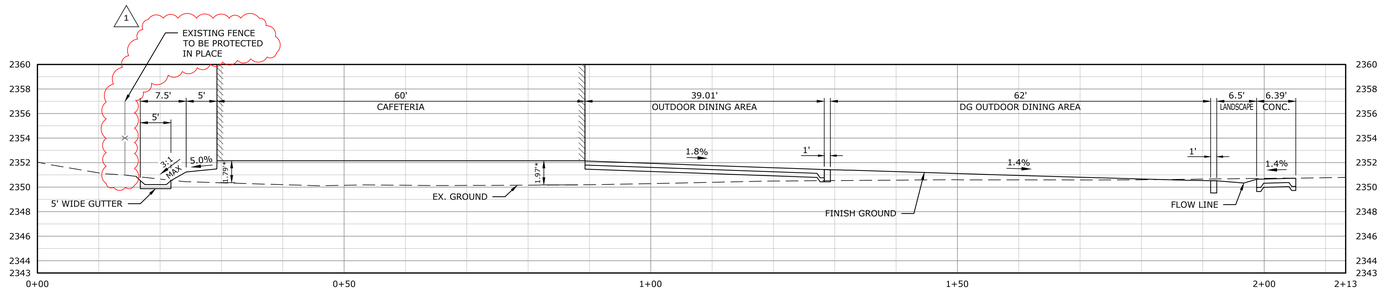
BID CLARIFICATION NO.1 (06-20-2023)



SECTION "A"

SCALE: 1"=10' HORZ
1"=5' VERT

A



SECTION "B"

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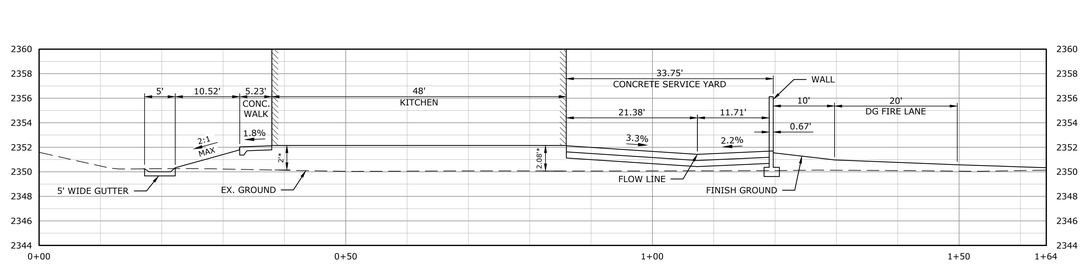
1"=5' VERT

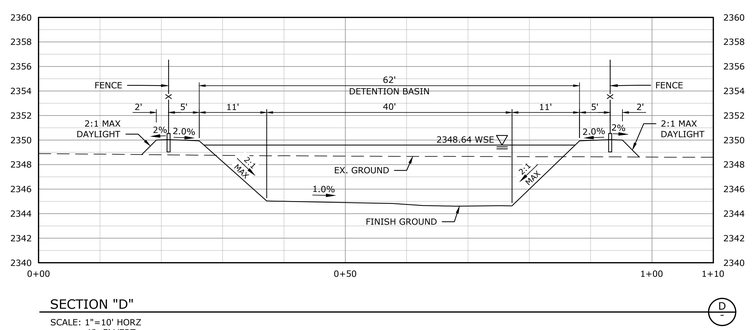
B



SECTION "C"

SCALE: 1"=10' HORZ
1"=5' VERT

C



SECTION "D"

SCALE: 1"=10' HORZ
1"=5' VERT

D

*PROPOSED BLDG FF ELEVATION NOTE

THIS PROJECT IS LOCATED IN THE DASHED X FEMA ZONE CLASSIFIED AS "OTHER AREAS OF FLOOD HAZARD". PER KERN COUNTY FLOOD CONTROL REQUIREMENTS, NEW BUILDING FF ELEVATIONS MUST BE RAISED A MINIMUM OF 1.5' ABOVE ADJACENT EXISTING GRADE.

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. C3.03, (A#03-122536, DATED 03/24/2023)

Know what's **below.**
Call before you dig.

AGENCY

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architecture planning interiors

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C-26053
Ren. Date 08/31/2025
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CONSULTANT

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PHONE: 661.600.9367 WEBSITE: WWW.ECGCIVIL.COM

Drawn by

Checked by

Revisions

No.	Date	Description
1	06-20-23	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

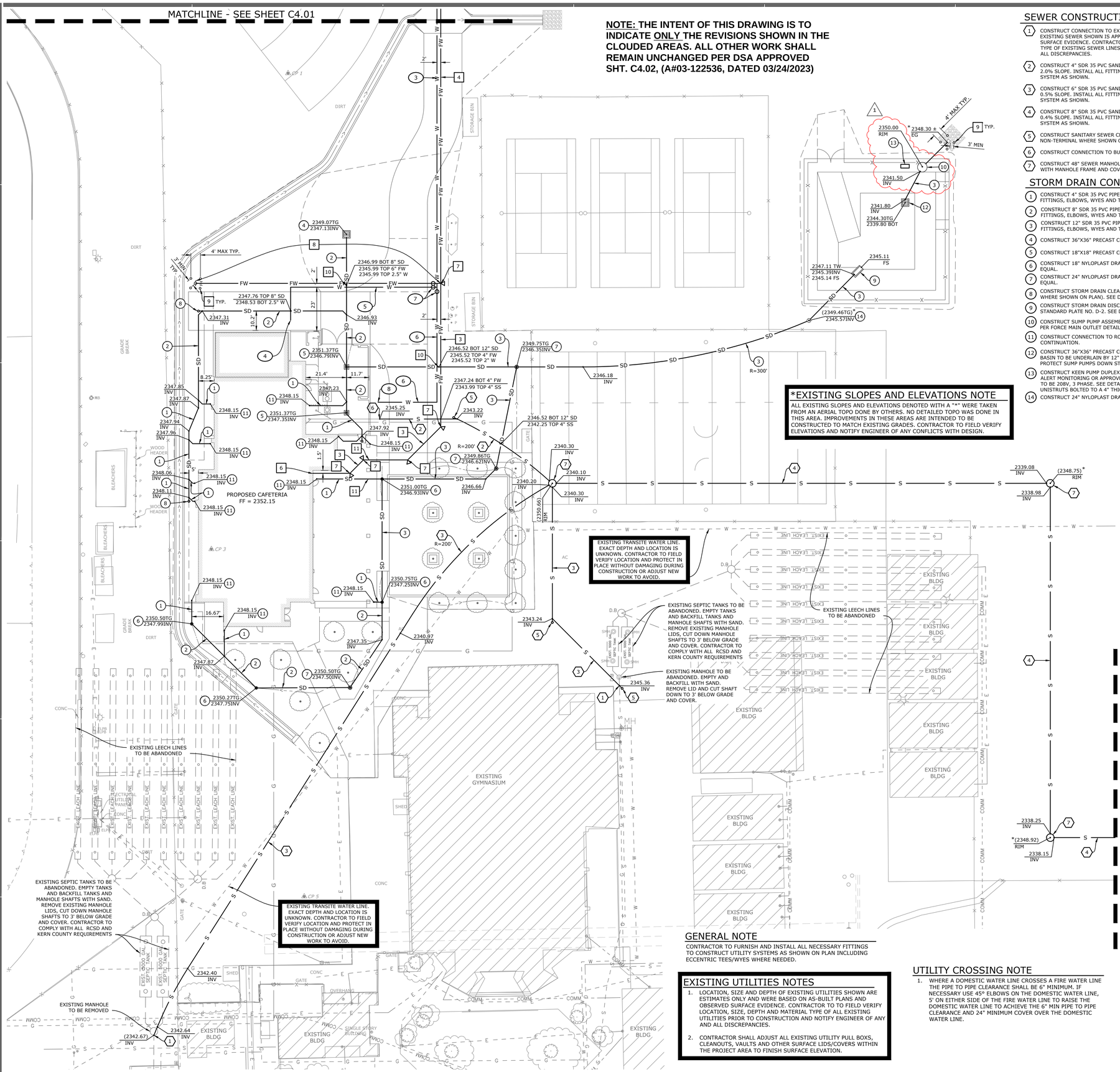
SITE SECTIONS

Job No.
2990

Date
01.30.2023

C3.03

BID CLARIFICATION NO.1 (06-20-2023)



*EXISTING SLOPES AND ELEVATIONS NOTE

ALL EXISTING SLOPES AND ELEVATIONS DENOTED WITH A "*" WERE TAKEN FROM AN AERIAL TOPO DONE BY OTHERS. NO DETAILED TOPO WAS DONE IN THIS AREA. IMPROVEMENTS IN THESE AREAS ARE INTENDED TO BE CONSTRUCTED TO MATCH EXISTING GRADES. CONTRACTOR TO FIELD VERIFY ELEVATIONS AND NOTIFY ENGINEER OF ANY CONFLICTS WITH DESIGN.

GENERAL NOTE

CONTRACTOR TO FURNISH AND INSTALL ALL NECESSARY FITTINGS TO CONSTRUCT UTILITY SYSTEMS AS SHOWN ON PLAN INCLUDING ECCENTRIC TEES/WYES WHERE NEEDED.

EXISTING UTILITIES NOTES

1. LOCATION, SIZE AND DEPTH OF EXISTING UTILITIES SHOWN ARE ESTIMATES ONLY AND WERE BASED ON AS-BUILT PLANS AND OBSERVED SURFACE EVIDENCE. CONTRACTOR TO TO FIELD VERIFY LOCATION, SIZE, DEPTH AND MATERIAL TYPE OF ALL EXISTING UTILITIES PRIOR TO CONSTRUCTION AND NOTIFY ENGINEER OF ANY AND ALL DISCREPANCIES.
2. CONTRACTOR SHALL ADJUST ALL EXISTING UTILITY PULL BOXS, CLEANOUTS, VAULTS AND OTHER SURFACE LIDS/COVERS WITHIN THE PROJECT AREA TO FINISH SURFACE ELEVATION.

UTILITY CROSSING NOTE

1. WHERE A DOMESTIC WATER LINE CROSSES A FIRE WATER LINE THE PIPE TO PIPE CLEARANCE SHALL BE 6" MINIMUM. IF NECESSARY USE 45° ELBOWS ON THE DOMESTIC WATER LINE, 5' ON EITHER SIDE OF THE FIRE WATER LINE TO RAISE THE DOMESTIC WATER LINE TO ACHIEVE THE 6" MIN PIPE TO PIPE CLEARANCE AND 24" MINIMUM COVER OVER THE DOMESTIC WATER LINE.

SEWER CONSTRUCTION NOTES

1. CONSTRUCT CONNECTION TO EXISTING SEWER LINE AS SHOWN. LOCATION, SIZE AND DEPTH OF EXISTING SEWER SHOWN IS APPROXIMATE AND WAS BASED ON AS-BUILT PLANS AND OBSERVED SURFACE EVIDENCE. CONTRACTOR TO TO FIELD VERIFY LOCATION, SIZE, DEPTH AND MATERIAL TYPE OF EXISTING SEWER LINES PRIOR TO CONSTRUCTION AND NOTIFY ENGINEER OF ANY AND ALL DISCREPANCIES.
2. CONSTRUCT 4" SDR 35 PVC SANITARY SEWER LINE PER DETAIL "AA" ON SHEET C5.01. MINIMUM 2.0% SLOPE. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT SEWER SYSTEM AS SHOWN.
3. CONSTRUCT 6" SDR 35 PVC SANITARY SEWER LINE PER DETAIL "AA" ON SHEET C5.01. MINIMUM 0.5% SLOPE. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT SEWER SYSTEM AS SHOWN.
4. CONSTRUCT 8" SDR 35 PVC SANITARY SEWER LINE PER DETAIL "AA" ON SHEET C5.01. MINIMUM 0.4% SLOPE. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT SEWER SYSTEM AS SHOWN.
5. CONSTRUCT SANITARY SEWER CLEANOUT PER SPPWC STD PLAN 204-2 (MODIFIED TO BE NON-TERMINAL WHERE SHOWN ON PLAN). SEE DETAIL "HH" ON SHEET C5.02.
6. CONSTRUCT CONNECTION TO BUILDING SEWER LINE. SEE PLUMBING PLAN FOR CONTINUATION.
7. CONSTRUCT 48" SEWER MANHOLE PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARD S-7 WITH MANHOLE FRAME AND COVER PER STANDARD S-6. SEE DETAIL "II" ON SHEET C5.02.

STORM DRAIN CONSTRUCTION NOTES

1. CONSTRUCT 4" SDR 35 PVC PIPE PER GRAVITY PIPE TRENCH DETAIL "AA", ON SHEET C5.01. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT STORM DRAIN SYSTEM AS SHOWN.
2. CONSTRUCT 8" SDR 35 PVC PIPE PER GRAVITY PIPE TRENCH DETAIL "AA", ON SHEET C5.01. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT STORM DRAIN SYSTEM AS SHOWN.
3. CONSTRUCT 12" SDR 35 PVC PIPE PER GRAVITY PIPE TRENCH DETAIL "AA", ON SHEET C5.01. INSTALL ALL FITTINGS, ELBOWS, WYES AND TEES NECESSARY TO CONSTRUCT STORM DRAIN SYSTEM AS SHOWN.
4. CONSTRUCT 36"X36" PRECAST CONCRETE CATCH BASIN WITH TRAFFIC RATED GRATE.
5. CONSTRUCT 18"X18" PRECAST CONCRETE CATCH BASIN WITH TRAFFIC RATED GRATE.
6. CONSTRUCT 18" NYLOPLAST DRAIN BASIN 2818AG WITH 18" STANDARD GRATE 1899CGS OR APPROVED EQUAL.
7. CONSTRUCT 24" NYLOPLAST DRAIN BASIN 2824AG WITH 24" STANDARD GRATE 2499CGS OR APPROVED EQUAL.
8. CONSTRUCT STORM DRAIN CLEANOUT PER SPPWC STD PLAN 204-2 (MODIFIED TO BE NON-TERMINAL WHERE SHOWN ON PLAN). SEE DETAIL "HH" ON SHEET C5.02.
9. CONSTRUCT STORM DRAIN DISCHARGE STRUCTURE PER KERN COUNTY DRAINAGE DEVELOPMENT STANDARD PLATE NO. D-2. SEE DETAIL "KK" ON SHEET C5.02. H=24", W=24".
10. CONSTRUCT SUMP PUMP ASSEMBLY PER DETAILS ON SHEET C5.04. OUTLET FORCE MAIN TO THE SURFACE PER FORCE MAIN OUTLET DETAIL ON SHEET C5.04.
11. CONSTRUCT CONNECTION TO ROOF DRAIN OR DOWN SPOUT PER ARCHITECT'S PLAN. SEE ROOF PLANS FOR CONTINUATION.
12. CONSTRUCT 36"X36" PRECAST CONCRETE CATCH BASIN WITH OPEN BOTTOM AND STANDARD GRATE. CATCH BASIN TO BE UNDERLAIN BY 12" OF 3/4" CRUSHED ROCK. CATCH BASIN TO ACT AS DESILTING BASIN TO PROTECT SUMP PUMPS DOWN STREAM.
13. CONSTRUCT KEEN PUMP DUPLEX CONTROL PANEL OR APPROVED EQUAL WITH REMOTE MONITORING, STATION ALERT MONITORING OR APPROVE EQUAL. ENCLOSURE SHALL BE NEMA 4X FIBERGLASS. ELECTRICAL SERVICE TO BE 208V, 3 PHASE. SEE DETAILS ON SHEET C5.04. MOUNT CONTROL PANEL BETWEEN TWO VERTICAL UNISTRUTS BOLTED TO A 4" THICK CONCRETE PAD.
14. CONSTRUCT 24" NYLOPLAST DRAIN BASIN 2824AG WITH 24" SOLID COVER 2499CGC OR APPROVED EQUAL.

FIRE WATER CONSTRUCTION NOTES

1. CONSTRUCT CONNECTION TO EXISTING 8" ROSAMOND COMMUNITY SERVICES DISTRICT WATER LINE PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARDS AND SPECIFICATIONS.
2. CONSTRUCT 8" DOUBLE CHECK DETECTOR ASSEMBLY WITH FIRE DEPARTMENT CONNECTION PER DETAIL "EE" ON SHEET C5.01.
3. CONSTRUCT 4" C900 CL 305 (DR 14) PVC WATER LINE PER DETAIL "CC " ON SHEET C5.01.
4. CONSTRUCT 8" C900 CL 305 (DR 14) PVC WATER LINE PER DETAIL "CC " ON SHEET C5.01.
5. CONSTRUCT GATE VALVE AND VALVE BOX PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARD W-9. SEE DETAIL "FF" ON SHEET C5.02.
6. CONSTRUCT CONNECTION TO BUILDING FIRE RISER. SEE DETAIL "SS" ON SHEET C5.03.
7. CONSTRUCT HORIZONTAL BEND AND CONCRETE THRUST BLOCK PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARD W-11. SEE DETAIL "GG" SHEET ON C5.02. ANGLE, SIZE, AND MATERIAL AS REQUIRED.
8. CONSTRUCT FIRE HYDRANT ASSEMBLY PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARD W-6. SEE DETAIL "OO" ON SHEET C5.03.
9. CONSTRUCT BOLLARD PER DETAIL "PP" ON SHEET C5.03. NUMBER AND SPACING AS SHOWN ON PLAN.
10. DIP WATER AND FIRE WATER LINES UNDER STORM DRAIN LINE WITH MINIMUM 12" CLEARANCE BETWEEN CROSSING PIPES.
11. CONSTRUCT PIPE UNDERCROSSING AT GRADE BEAM WITH CONCRETE ENCASEMENT PER DETAIL "TT" ON SHEET C5.03.

DOMESTIC WATER CONSTRUCTION NOTES

1. CONSTRUCT CONNECTION TO EXISTING 6" WATER LINE AS SHOWN.
2. CONSTRUCT GATE VALVE AND VALVE BOX PER ROSAMOND COMMUNITY SERVICES DISTRICT STANDARD W-9. SEE DETAIL "FF" ON SHEET C5.02.
3. CONSTRUCT 3" SCH 40 PVC WATER LINE WITH ALL NECESSARY FITTINGS PER DETAIL "CC" ON SHEET C5.01.
4. CONSTRUCT CONNECTION TO BUILDING WATER LINE. SEE PLUMBING PLAN FOR CONTINUATION.
5. CONSTRUCT 2.5" SCH 40 PVC WATER LINE WITH ALL NECESSARY FITTINGS PER DETAIL "BB" ON SHEET C5.01.
6. CONSTRUCT 2" SCH 40 PVC WATER LINE WITH ALL NECESSARY FITTINGS PER DETAIL "BB" ON SHEET C5.01.
7. CONSTRUCT VALVE AND VALVE BOX PER DETAIL "DD" ON SHEET C5.01.
8. CONSTRUCT CONNECTION TO SITE IRRIGATION SYSTEM. SEE IRRIGATION PLANS FOR CONTINUATION.

LEGEND

PROPOSED FLOWLINE
PROPOSED SEWER LINE
PROPOSED STORM DRAIN LINE
PROPOSED WATER LINE
PROPOSED FIRE WATER LINE
PROPOSED GAS LINE (SEE PLUMBING PLANS)
PROPOSED CHAIN LINK FENCE
EXISTING RIGHT OF WAY
EXISTING EASEMENT
EXISTING FENCE
EXISTING OVER HEAD WIRE
EXISTING ELECTRICAL LINE
EXISTING GAS LINE
EXISTING WATER LINE
EXISTING STORM DRAIN LINE
EXISTING FIREWATER LINE
EXISTING SEWER LINE
EXISTING COMMUNICATION LINE

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Revisions

No.	Date	Description
1	06-20-23	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. Drawings & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

WET UTILITY PLAN

Job No. 2990

Date 01.30.2023

C4.02

BID CLARIFICATION NO.1 (06-20-2023)

Pump Data Sheet - Keen Pump Inc.

Company: Signa Services, Inc. Data Sheet prepared for ECG, specifically for listed project by S
Name: Rosamond High School Cafeteria
Date: 07/18/2022

KEEN PUMP

Pump:

Size:	K3RN	Dimensions:	
Type:	Submersible Sewage	Suction:	3 in
Synch Speed:	1200 rpm	Discharge:	3 in
Dia:	5.5625 in		
Curve:	---		

Fluid:

Name:	Water		
SG:	1	Vapor Pressure:	0.256 psi a
Density:	62.4 lb/ft³	Atm Pressure:	14.7 psi a
Viscosity:	1.1 cP		
Temperature:	60 °F	Margin Ratio:	1

Search Criteria:

Flow:	125 US gpm	Near Miss:	---
Head:	10.7 ft	Static Head:	9.3 ft

Pump Limits:

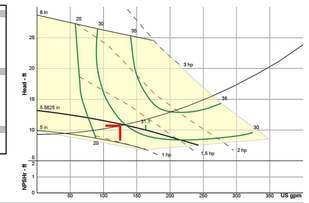
Temperature:	---	Sphere Size:	3 in
Wkg Pressure:	---		

Motor:

Standard:	NEMA	Size:	2 hp
Enclosure:	TEFC	Speed:	1200 rpm
Frame:	184T		
Sizing Criteria:	Max Power on Design Curve		

Pump Selection Warnings:
None

--- Duty Point ---	
Flow:	131 US gpm
Head:	10.8 ft
Eff:	29.4%
Power:	1.21 hp
NPSHr:	---
Speed:	1150 rpm
--- Design Curve ---	
Shutoff Head:	13.2 ft
Shutoff dP:	5.71 psi
Min Flow:	--- US gpm
BEP: 31.1% @ 163 US gpm	
NOL Power:	
1.64 hp @ 243 US gpm	
--- Max Curve ---	
Max Power:	
2.99 hp @ 174 US gpm	



Performance Evaluation:

Flow US gpm	Speed rpm	Head ft	Efficiency %	Power hp	NPSHr ft
150	1150	10.4	30.6	1.28	---
125	1150	11	29.4	1.21	---
100	1150	11.5	24	1.19	---
75	1150	12	19.6	1.17	---
50	1150	12.4	15.2	1.14	---

Selected from catalog: Keen.60, Vers 1.3.0

Solids-Handling Pumps

KEEN PUMP

K3RN(X)

FM APPROVED

3" Submersible, Recessed Impeller
(Class 1, Div. 1, Groups C & D Hazardous Location)

Optional :
All Stainless
Steel Construction

CERAMIC COATED RECESSED IMPELLER
- **Superior Abrasion Resistance**
- **Allows The Pump To Operate Better In Any Pressure Or Flow Condition**

CERAMIC COATED VOLUTE
- **Holds Up to Wear On Inside of Volute Casting Case**

STANDARD MOTOR CONSTRUCTION
- **Class H Laminations (Standard)**

MOTOR GROUND RING PROTECTION
- **Allows Motor To Operate With VFD Without Damaging Bearings and Motor**
- **Ground Ring Safely Diverts Harmful Voltages Away From Bearings**

DOUBLE ROW LOWER BEARINGS
- **Heavy Duty Bearings Handle The Toughest Applications of High Pressure and Flow Rates**

BEST MOTOR CONSTRUCTION
- **Class "N" (392° F) Highest Temperature**
- **Motor Will Not Overheat In Continuous Applications**

CAPABILITIES:
- **Flows---------- to 625 GPM**
- **Heads---------- to 71 Feet**
- **HP Range---------- 1 - 10 HP**
- **Voltage / Phase Options---------- 208/230V, 1 Phase - 208/230/460 V, 3 Phase**
- **Discharge Connections---------- 3" ANSI Class 125 Horizontal Flange**
- **Motor Speeds---------- 1150 / 1750 RPM**
- **Solids-Handling---------- 2.5"**
- **Impeller---------- Recessed, Vortex**
- **Motor Service Factor---------- 1.20**

Features and Benefits

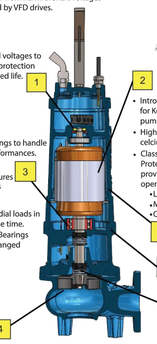
Keen Pump's Bearing Protection Program

- Keen's Motor Grounding Ring protects the motor from harmful shaft voltages produced by VFD drives.
- Safely conducts the harmful voltages to the grounding ring adding protection for the bearings and extended life.

Double Row Bearings

- Heavy Duty Bearings to handle the toughest performances.
- High Pressures
- High Flows
- Handles both Axal and Radial loads in both directions at the same time.
- Double Row Angular Ball Bearings have two rows of balls arranged back-to-back.

Ceramic Coating
The ultimate indestructable ceramic coated impellers and volutes for Keen's Solids Handling pumps. This allows the pump to operate in any condition without any problems! Impressive sand, grit, abrasion and cavitation resistance!



Heavy Duty Motor

- Introducing New Premium Class "N" Motors for Keen's Large Grinders and Solids Handling pumps.
- Highest temperature construction (200 degrees celcius) for motors
- Class "N" Construction is the best in the industry. Protects your motor from over heating and provides extended motor life. Motors are operating at higher teperatures due to:
  - Longer run times
  - Motor housing not submerged
  - Continuous

5 Castings
- ASTM - Class 35 high tensile strength castings.

6 Durable Powder Coat Finish
- Holds up to the toughest environmental conditions.

7 Dual Shaft Seals
- Twice the protection quality silicon carbide faces, viton elastomers, stainless steel construction.

KEEN PUMP
471 US HWY 250 East Ashland, Ohio 44805
419.207.9400 fax 419.207.8031
www.keenpump.com

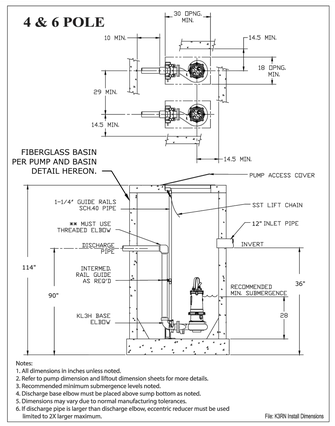
File : K3RN CUTS
Rev. 04/2021

KEEN PUMP *Section* VORTEX *Date* NOV 2016

Pump Series	K3RN	3" SUBMERSIBLE SEWAGE PUMPS

DUPLEX INSTALLATION DIMENSIONAL DATA

4 & 6 POLE



Notes:
1. All dimensions in inches unless noted.
2. Refer to pump dimension and liftout dimension sheets for more details.
3. Recommended minimum submergence levels noted.
4. Discharge base elbow must be placed above sump bottom as noted.
5. Dimensions may vary due to normal manufacturing tolerances.
6. If discharge pipe is larger than discharge elbow, eccentric reducer must be used limited to 2X larger maximum.

File: K3RN Install Dimensions

KEEN PUMP *Section* SOLIDS-HANDLING *Dated* April 2021

Pump Model: **K3RN**

Physical Data:

Discharge Size	ANSI 3" Horizontal
Solids Size	2.5"
Impeller Type	Balanced, Recessed, 8 Vane
Power/Control Cable Length	40' Standard
Paint	Blue, Water Reducible Enamel, One Coat, Air Dried – Impeller & Inside Volute - Ceramic Coated For Abrasion Resistance

Motor Construction:

Motor Type	Enclosed Submersible Oil Filled
NEMA Insulation Code	Class H (**Optional** - Class N)
Service Factor	1.2
NEMA Design Type	B (50) L (10)
Single Phase Configuration	External Start and Run Components
Motor Protection	Thermal Sensors Embedded in the Windings
Maximum Stator Temperature	Class H: 356°F (180°C) Class N: 392° F (200°C) - **Optional**
Power Cord Type	SOOW - 600V, 90° C, Type W - 2000V, 90° C
Control Cord Type	16-4 or 18-5 - SOOW - 600V, 90° C

Materials of Construction:

Cord Entry	Cast Iron, ASTM A48, Class 35
Motor Housing	Cast Iron, ASTM A48, Class 35
Bearing Housing	Cast Iron, ASTM A48, Class 35
Volute	Cast Iron, ASTM A48, Class 35 / Ceramic Coated Inside
Impeller	Ductile Iron, ASTM A536, 60-40-18 / Ceramic Coated
Shaft	ANSI 400 Stainless Steel
Inboard Mechanical Seal	Silicone Carbide / Silicone Carbide / Viton Elastomers
Outboard Mechanical Seal	Silicone Carbide / Silicone Carbide / Viton Elastomers
Fasteners	ANSI 18-8 Stainless Steel
O-Rings	Nitrile Rubber
Upper Bearing	Conrad Style Single Row Deep Groove Ball Bearing
Lower Bearing	Conrad Style Double Row Angular Contact Ball Bearing
Line Bearing	Bronze, CDA 836

K3RN_M246_PG1

KEEN PUMP *Section* VORTEX *Dated* MAR 2017

Pump Model: **K3RN – 1150 RPM**

Thermal Data:

Maximum Liquid		140° F (60° C)
Maximum Stator		311° F (155° C)
Heat Sensor	**Open:**	257° F (125° C) Max. / 239° F (115° C) Min.
	Closed:	194° F (90° C) Max. / 119° F (48° C) Min.
Oil Flash Point		390° F (199° C)

Electrical Data:

RPM		**1150**		
Electrical Ratings	Heat Sensor	24VDC 5AMPS	115VAC 5AMPS	230VAC 5AMPS
	Seal Fail	300VAC 5mAMPS		
Voltage Tolerance		± 10%		

HP	Voltage	Phase	NEC Code	Service Factor	Full Load AMPS	SF Amps	Locked Rotor AMPS	Run KW	Start KVA	Run KVA
1	208	1	H	1.2	9.6	11.5	33.6	2.0	7.0	2.6
	230				8.6	10.4	30.3			
	208	3	J	1.2	6.6	7.9	21.0	1.7	7.6	2.1
1	230				6.2	7.4	18.9			
	460				3.1	3.7	9.5			
	208	1	H	1.2	18.9	22.7	66.2	13.8	5.1	5.1
2	230				17.0	20.4	59.6			
	208	3	G	1.2	9.9	11.9	34.7	2.8	12.5	3.5
2	230				8.9	10.7	31.2			
	460				4.5	5.4	15.6			
	208	1	F	1.0	22.7	22.7	79.5	4.7	16.5	6.1
3	230				20.5	20.5	71.6			
	208	3	G	1.0	14.3	14.3	50.1	4.1	18.0	6.0
	230				12.9	12.9	45.1			
	460				6.4	6.4	22.5			

Motor Efficiencies & Power Factor

HP	Phase	Motor Efficiency % Service Factor Load	100% Load	75% Load	50% Load	Power Factor % Service Factor Load	100% Load	75% Load	50% Load
1	1	60	60	59	53	75	73	66	57
1	3	65	62	56	52	75	72	67	61
2	1	63	61	60	50	76	74	70	62
2	3	65	64	60	52	78	76	71	60
3	1	68	65	59	55	75	70	64	54
3	3	65	64	60	57	79	78	71	59

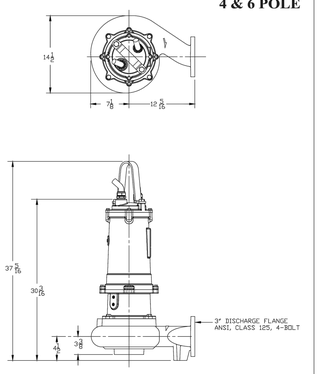
K3RN_M6_PG2

KEEN PUMP *Section* VORTEX *Date* NOV 2016

Pump Series	K3RN	3" SUBMERSIBLE SEWAGE PUMPS

DIMENSIONAL DATA

4 & 6 POLE



Note: All dimensions in inches unless noted.
Dimensions may vary due to normal manufacturing tolerances.

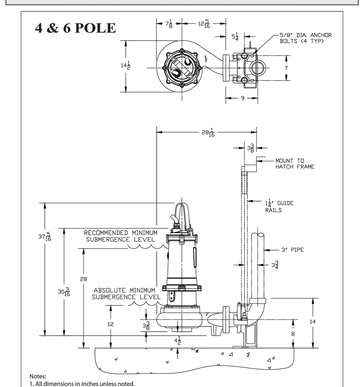
File: K3RN Dimensions

KEEN PUMP *Section* VORTEX *Date* NOV 2016

Pump Series	K3RN	3" SUBMERSIBLE SEWAGE PUMPS

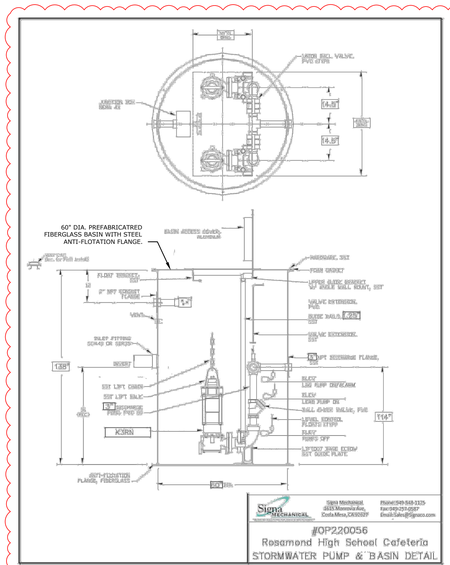
LIFTOUT DIMENSIONAL DATA

4 & 6 POLE



Notes:
1. All dimensions in inches unless noted.
2. Discharge flange 3" NPT.
3. Absolute minimum and recommended minimum submergence levels noted.
4. Dimensions may vary due to normal manufacturing tolerances.
5. If discharge pipe is larger than discharge elbow, eccentric reducer must be used limited to 2X larger maximum.

File: K3RN Liftout Dimensions



1

KEEN PUMP

Duplex Control Panels
4 Float Operation

KEEN Pump duplex control panels are custom designed for use with KEEN grinder and sewage pumps and packaged systems.

All control panels are built to Underwriters Laboratories UL508 standard to ensure the highest level of quality and safety.

All panels can carry the UL label.

There are many optional features available, consult the "Control Panel Options Page".

Standard Features – Duplex Panels

Single Phase
- **NEMA 4X Fiberglass Enclosure**
- **Stainless Steel Piano Hinge**
- **(2) Pump Circuit Breakers (2 Pole)**
- **(2) Capacitor Start Kits**
- **Control Circuit Breaker**
- **(2) Motor Contactors**
- **Alternator (switch able w/ lights)**
- **External Alarm Light**
  - **Flashing – High Water Level**
- **Audible Alarm – Sonalert Buzzer**
- **Alarm Test Switch**
- **(3) Control Circuit Relays**
- **DIN Rail Mounted Components**
- **Aluminum Back plate**
- **(2) Internal HOA Switches**
- **External Lockable Hasps**
- **Easy Wiring Terminal Blocks**
- **Grounding Lugs**
- **Wiring in Enclosed Wire way**
- **Transformer (460 volt panels only)**

Three Phase
- **NEMA 4X Fiberglass Enclosure**
- **Stainless Steel Piano Hinge**
- **(2) Pump Circuit Breakers (3 Pole)**
- **Control Circuit Breaker**
- **(2) Motor Starters**
- **Alternator (switch able w/ lights)**
- **External Alarm Light**
  - **Flashing – High Water Level**
- **Audible Alarm – Sonalert Buzzer**
- **Alarm Test Switch**
- **(3)Control Circuit Relays**
- **DIN Rail Mounted Components**
- **Aluminum Back plate**
- **(2) Internal HOA Switches**
- **External Lockable Hasps**
- **Easy Wiring Terminal Blocks**
- **Grounding Lugs**
- **Wiring in Enclosed Wire way**
- **Transformer (460 volt panels only)**

SAM

Protect Your Pump

SAM is your pump station's constant companion, providing enhanced remote monitoring capabilities to protect your system. Compatible with a variety of water and wastewater pumps, SAM can be added to an existing system or configured when ordering new equipment.

Features
- Collects data on pump run times and cycles
- Map overview of installed systems and statuses
- Automatic notifications (via text or email) for alert conditions
- Scheduled, customizable reports
- Web platform access to review pump performance and history of alerts
- Monitors up to 4 pumps in one station

Benefits of SAM
-Reduce unnecessary service callouts for "false" alarms
-Replaces service contract for on-site preventative maintenance and inspections
-Troubleshooting data is available to service provided prior to arrival on site
-Homeowner Peace of Mind, especially for vacation or second homes
-Instant notification for critical statuses and resolution

How does SAM work?

A cellular modem is wired into the pump system's control panel, using one of SAM's monitoring component kits.

As the panel receives electrical signals from the pump, the modem collects and sends the data to a web platform, where it can be accessed for active monitoring.

When an alert condition occurs, the web platform will send an automatic notification, immediately advising you of critical system statuses via text or email.

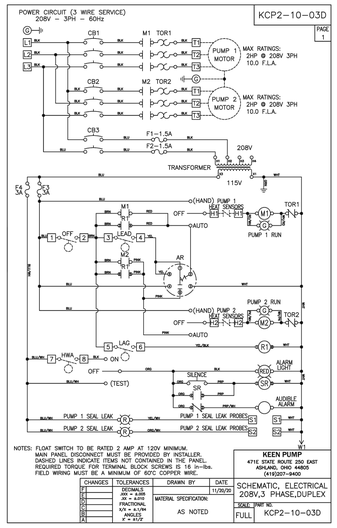
STATION ALERT MONITORING

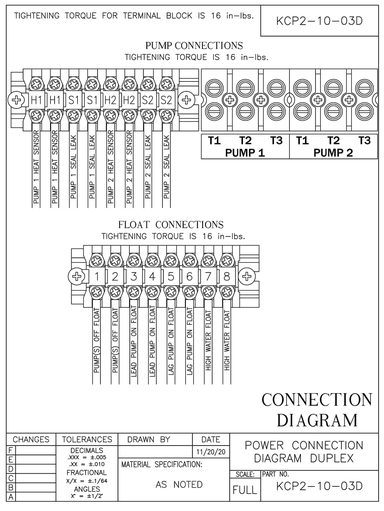
Local Service Centers:

Signa MECHANICAL

Southern CA, Las Vegas Metro & Eastern NV
(949) 548-1135
1615 Monrovia Ave,
Costa Mesa, CA 92627

Northern CA, Reno Metro & Western NV
(855) 747-4462
29441 Kohoutek Way,
Union City, CA 94587





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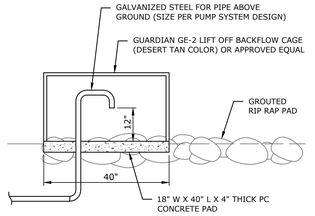

FORCE MAIN OUTLET
N.T.S.

811 Know what's **below.** **Call** before you dig.

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LICENSED ARCHITECT JOSHUA D. JENISON C-26053 Ren. Date 08/31/2025 STATE OF CALIFORNIA

CONSULTANT

REGISTERED PROFESSIONAL ENGINEER JOSHUA D. JENISON NO 77454 CIVIL STATE OF CALIFORNIA

ECG Encompass Consultant Group
25115 AVE STANFORD, SUITE #320, SANTA CLARITA, CA 91355
PHONE: 661.600.9367 WEBSITE: WWW.ECGCIVIL.COM

Drawn by

Checked by

Revisions

No.	Date	Description
1	06-20-23	BID CLARIFICATION NO.1

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 ROSAMOND BLVD. ROSAMOND, CA 93560

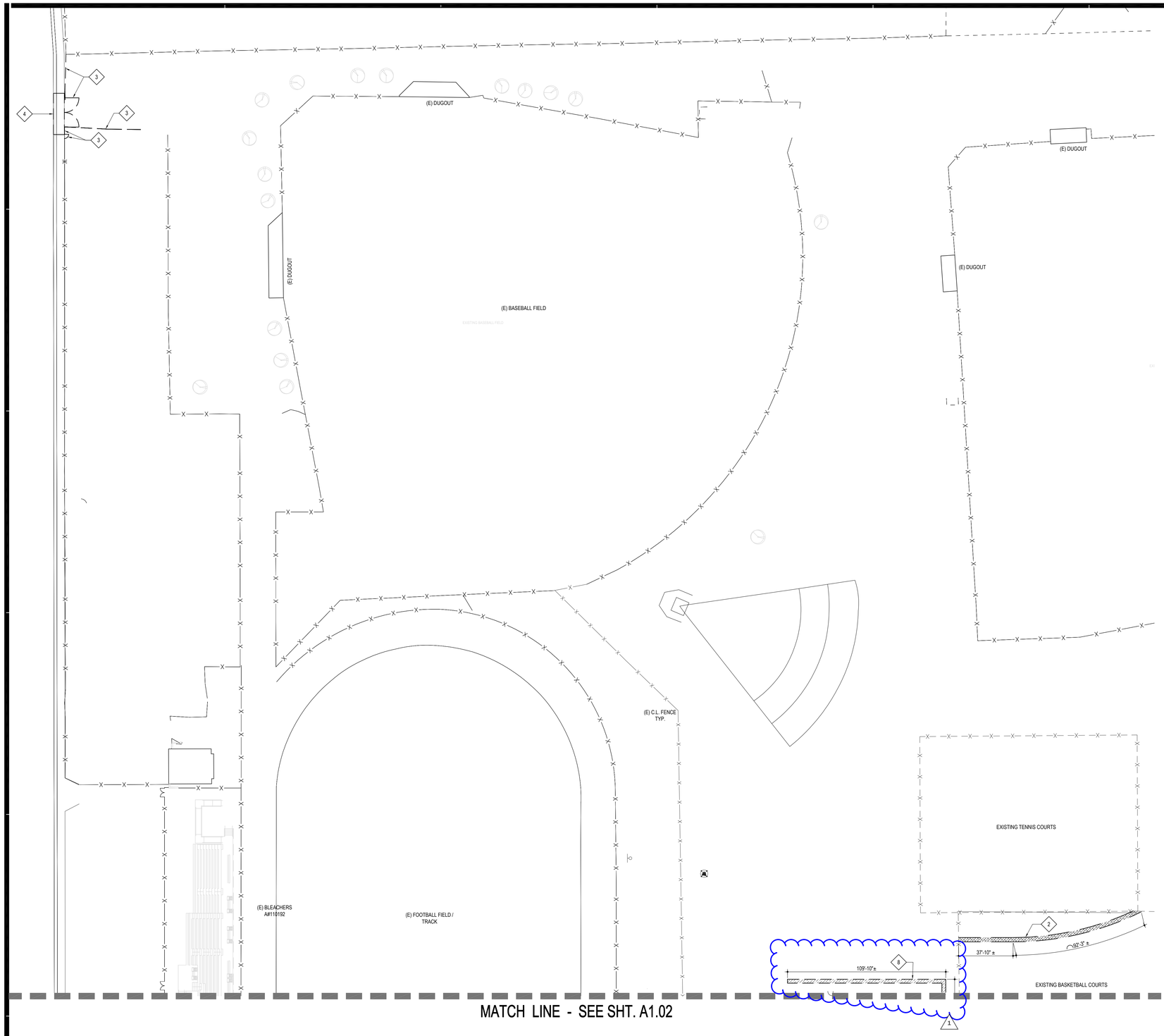
SUMP PUMP DETAILS

Job No. 2990

Date 01.30.2023

C5.04

BID CLARIFICATION NO.1 (06-20-2023)



PARTIAL DEMOLITION SITE PLAN

SCALE : 1" = 30'-0"

PROJECT NUMBER
DETAIL IDEN.
SHEET REFERENCE

1 / A1.01

N

SITE PLAN LEGEND

DEMO EXISTING AC PAVING AND BASE IN ITS ENTIRETY. CONTRACTOR SHALL PROTECT IN PLACE ALL EXISTING BASKETBALL POST. CONTRACTOR SHALL REMOVE ALL DEBRIS AND DISPOSE OF IN A LEGAL MANNER. REFER TO SHT. C3.01 & C3.02 FOR NEW SCOPE OF WORK

DEMO EXISTING PORTION OF CONCRETE AND BASE IN ITS ENTIRETY. CONTRACTOR SHALL REMOVE ALL DEBRIS AND DISPOSE OF IN A LEGAL MANNER. REFER TO SHT. C3.01 & C3.03 FOR NEW SCOPE OF WORK

DEMO EXISTING TURF AND IRRIGATION IN ITS ENTIRETY. DISCONNECT IRRIGATION MAIN LINE FOR NEW WORK. PROTECT IN PLACE EXISTING RCSD WATER AND SEWER MAIN LINES. CONTRACTOR SHALL REMOVE ALL DEBRIS AND DISPOSE OF IN A LEGAL MANNER. REFER TO SHT. A1.02 FOR NEW PLAYFIELD AREA

SITE DEMOLITION KEYNOTES

1. DEMO (E) A.C. PAVING IN ITS ENTIRETY INCLUDING ALL (E) BASE. SCARIFY AND PREP SOIL FOR NEW PAVING PER CIVIL SHT. C3.01 & C3.02. (E) BASKETBALL POST SHALL REMAIN IN PLACE
2. DEMO (E) PORTION OF CONCRETE AND BASE IN ITS ENTIRETY. CONTRACTOR SHALL REMOVE ALL DEBRIS AND DISPOSE OF IN A LEGAL MANNER. REFER TO SHT. C3.01 & C3.03 FOR NEW SCOPE OF WORK
3. DEMO (E) GATES AND FENCING AS REQUIRED FOR REWORK OF DRIVEWAY. SEE DETAIL ON CIVIL SHEET C3.02.
4. DEMO (E) CONCRETE DRIVEWAY AND PREPARE FOR NEW WORK. SEE DETAIL ON CIVIL SHEET C3.02
5. DEMO (E) STORAGE BINS, N.I.C.
6. DEMO (E) RELO BUILDING, N.I.C.
7. DEMO EXISTING CHAINLINK FENCE, POST AND FOUNDATION IN ITS ENTIRETY. CONTRACTOR SHALL REMOVE ALL DEBRIS AND DISPOSE OF IN A LEGAL MANNER.
8. TRENCH (E) DIRT AND A.C. PAVING FOR NEW GAS LINE. SEE PLUMBING SHEET P1.0.

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ARCHITECT

LICENSED ARCHITECT
SCOTT POHLIG
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

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Checked by _

Revisions

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

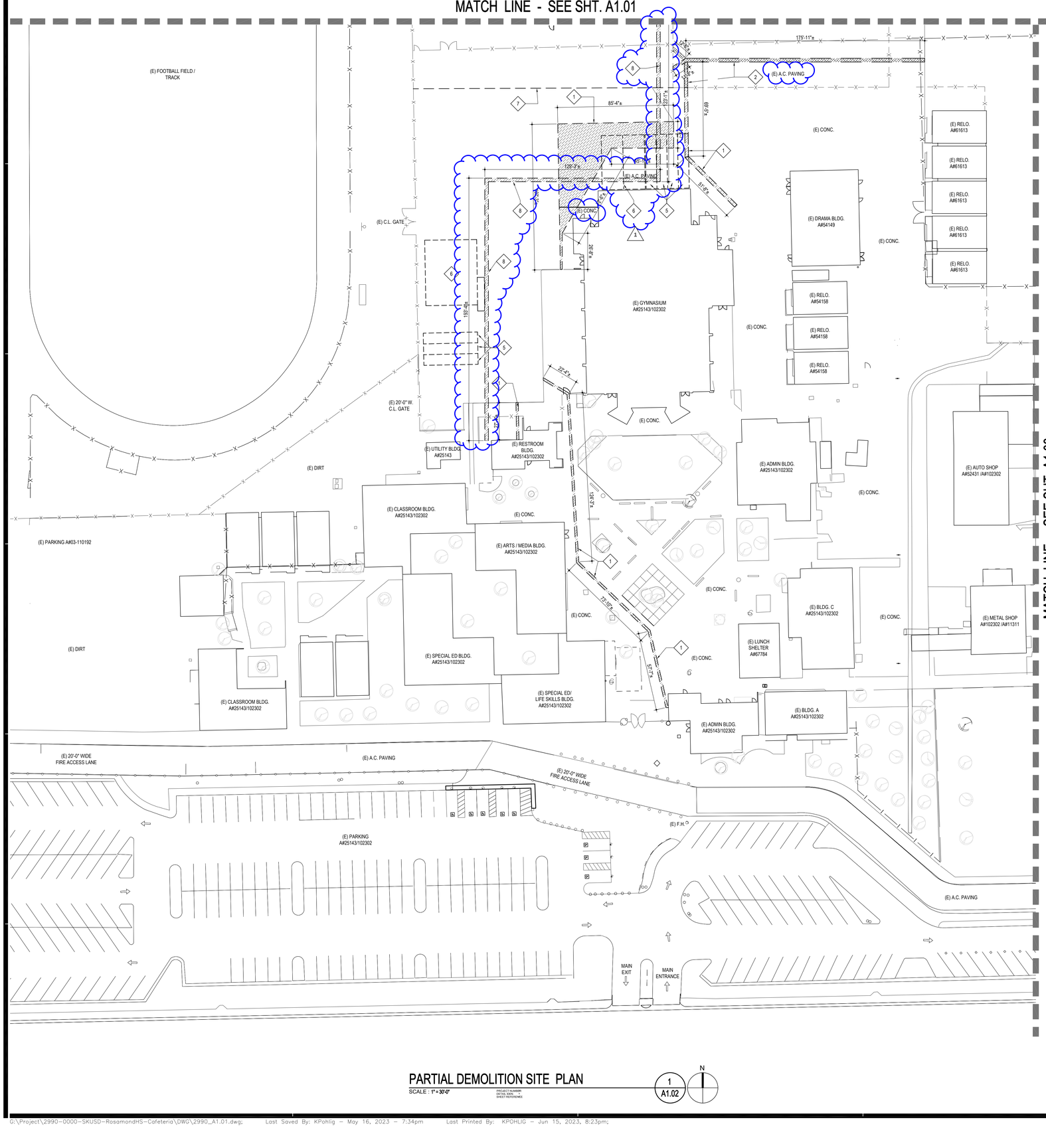
SITE DEMOLITON PLAN

Job No. 2990

Date 01.30.2023

A1.01

BID CLARIFICATION NO.1 (06-20-2023)



PARTIAL DEMOLITION SITE PLAN

SCALE : 1" = 30'-0"

1 / A1.02

N

SITE PLAN LEGEND

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SITE DEMOLITION KEYNOTES

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LICENSED ARCHITECT
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

Drawn by: --

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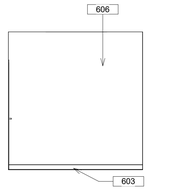
SITE DEMOLITON PLAN

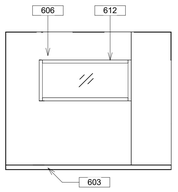
Job No. 2990

Date 01.30.2023

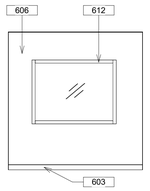
A1.02



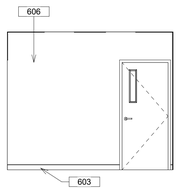
1 OFFICE 11 - NORTH ELEVATION
1/4" = 1'-0"



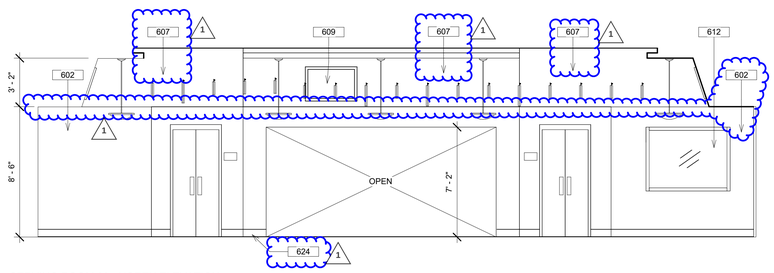
2 OFFICE 11 - EAST ELEVATION
1/4" = 1'-0"



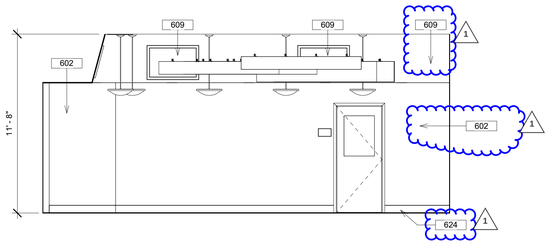
3 OFFICE 11 - SOUTH ELEVATION
1/4" = 1'-0"



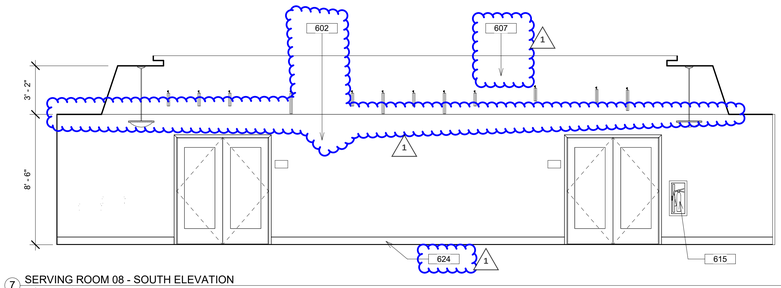
4 OFFICE 11 - WEST ELEVATION
1/4" = 1'-0"



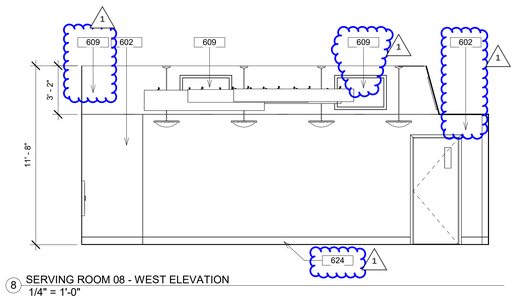
5 SERVING ROOM 08 - NORTH ELEVATION
1/4" = 1'-0"



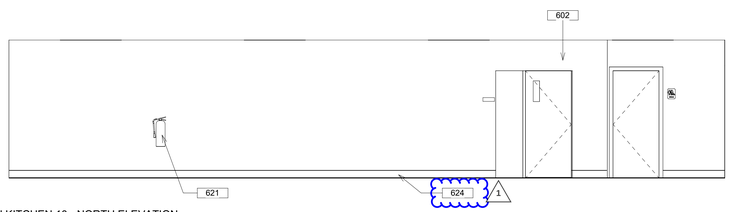
6 SERVING ROOM 08 - EAST ELEVATION
1/4" = 1'-0"



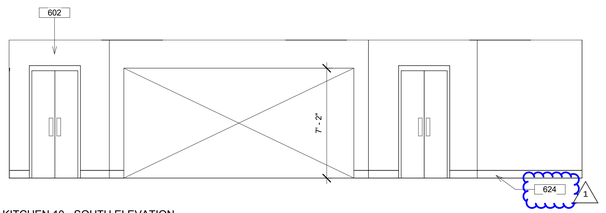
7 SERVING ROOM 08 - SOUTH ELEVATION
1/4" = 1'-0"



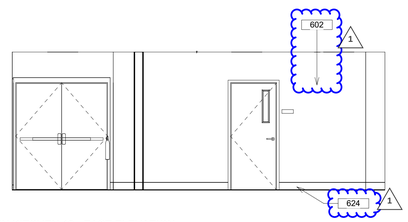
8 SERVING ROOM 08 - WEST ELEVATION
1/4" = 1'-0"



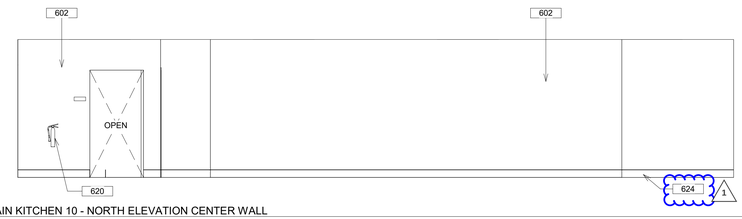
9 MAIN KITCHEN 10 - NORTH ELEVATION
1/4" = 1'-0"



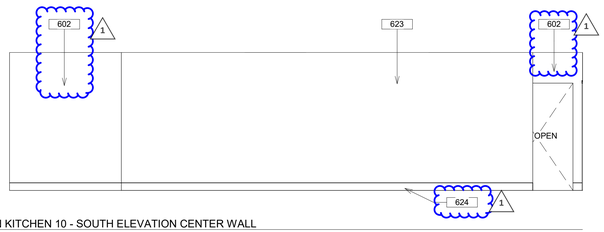
10 MAIN KITCHEN 10 - SOUTH ELEVATION
1/4" = 1'-0"



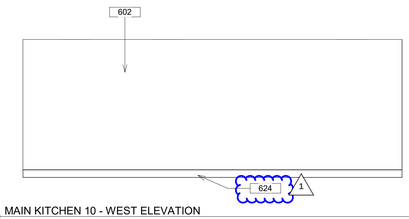
11 MAIN KITCHEN 10 - EAST ELEVATION
1/4" = 1'-0"



12 MAIN KITCHEN 10 - NORTH ELEVATION CENTER WALL
1/4" = 1'-0"



13 MAIN KITCHEN 10 - SOUTH ELEVATION CENTER WALL
1/4" = 1'-0"



14 MAIN KITCHEN 10 - WEST ELEVATION
1/4" = 1'-0"

INTERIOR ELEVATION KEYNOTES

602	FRP PANELING
603	4" HIGH TOPSET BASE, COLOR TO BE SELECTED BY ARCHITECT.
606	FABRIC TACKABLE PANELS ON 1/2" TACKABLE SUBSTRATE TO 6" ABOVE CEILING. COLOR TO BE SELECTED BY ARCHITECT.
607	GYPSUM BOARD, PAINTED. COLOR TO BE SELECTED BY ARCHITECT.
609	32" DIAGONAL WALL-MOUNTED TELEVISION. TELEVISION AND MOUTING BRACKET SHALL BE PROVIDED BY DISTRICT. CONTRACTOR INSTALLED. SEE DETAIL 16/A9.02
612	DOUBLE GLAZED ALUMINUM WINDOW SYSTEM. TYP. SEE WINDOW SCHEDULE ON SHEET A7.01 FOR SIZE.
615	FURNISH AND INSTALL CLASS 2A:10B-C FIRE EXTINGUISHER IN SEMI-RECESSED FIRE EXTINGUISHER CABINET, SEE DETAIL 4/A9.02
620	FURNISH AND INSTALL CLASS 2A:10B-C FIRE EXTINGUISHER. TOP OF CONTROLS SHALL BE 48" MAX. A.F.F. MOUNT PER CFC 906.7.
621	FURNISH AND INSTALL CLASS 2A:K FIRE EXTINGUISHER. TOP OF CONTROLS SHALL BE 48" MAX. A.F.F. MOUNT PER CFC 906.7.
623	20 GA.STAINLESS STEEL PANELING GLUED TO GYP BD., FLOOR UP TO 2" ABOVE THE BOTTOM OF THE HOOD.
624	6X6 SLIM FOOT CERAMIC WALL SANITARY COVE BASE, BY DALTILE.

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. A5.03, (A#03-122536, DATED 03/24/2023)

AGENCY

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An Employee Owned Corporation

ARCHITECT
LICENSED ARCHITECT
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

Drawn by:

Check by:

Revisions

No.	Date	Description
1	06-20-2023	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

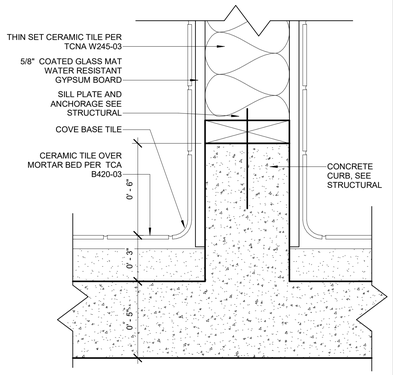
Drawing Title

INTERIOR ELEVATIONS

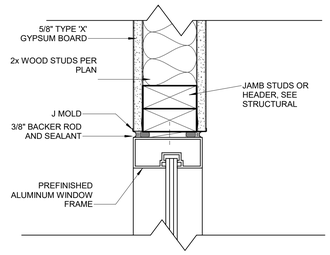
Job No. 2990.000

Sheet No. **A5.03**

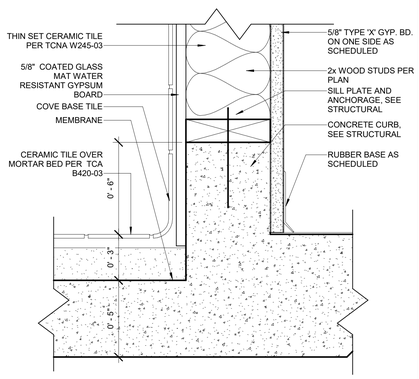
Date 01.30.2023



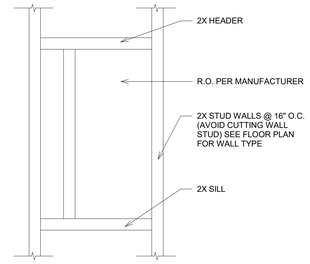
14 INTERIOR WALL CURB
3" = 1'-0"



13 INTERIOR WILDOW JAMB/ HEAD (SIM)
3" = 1'-0"

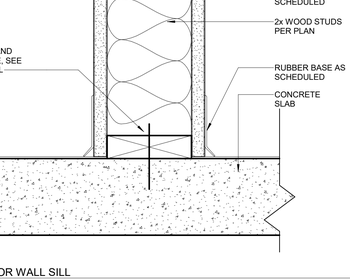


6 RESTROOM SILL
3" = 1'-0"

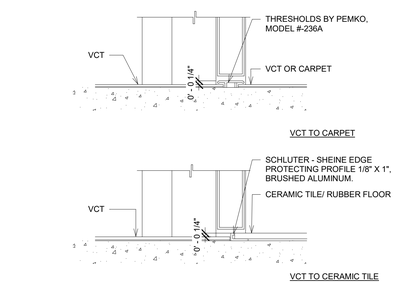


5 FIRE EXTINGUISHER CABINET FRAMING
1 1/2" = 1'-0"

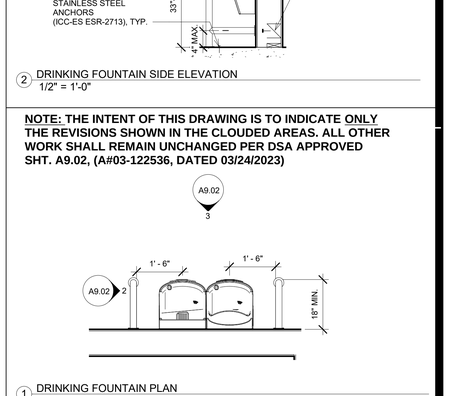
15 SERVING ROOM GUARDRAIL
3/4" = 1'-0"



12 INTERIOR WILDOW SILL
3" = 1'-0"



7 HM INTERIOR DOOR HEAD DETAIL
3" = 1'-0"



4 FIRE EXTINGUISHER CABINET
1 1/2" = 1'-0"

16 TV MOUNTING DETAIL
1 1/2" = 1'-0"

11 INTERIOR PARTITION TOP PLATE
3" = 1'-0"

8 HM INTERIOR DOOR JAMB DETAIL
3" = 1'-0"

3 DRINKING FOUNTATIN ELEVATION
1/2" = 1'-0"

2 DRINKING FOUNTAIN SIDE ELEVATION
1/2" = 1'-0"

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. A9.02, (A#03-122536, DATED 03/24/2023)

1 DRINKING FOUNTAIN PLAN
1/2" = 1'-0"

9 INTERIOR DOOR THRESHOLD
3" = 1'-0"

10 INTERIOR WALL SILL
3" = 1'-0"

AGENCY

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ARCHITECT

LICENSED ARCHITECT — C-26053 — Ren. Date 08/31/2023 — STATE OF CALIFORNIA

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No.	Date	Description
1	06-20-2023	BID CLARIFICATION NO.1

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

Drawing Title

INTERIOR DETAILS

Job No. 2990.000

Sheet No. **A9.02**

Date 01.30.2023

SIMPSON PRODUCT EVALUATION REPORT	
PRODUCT	ICC-ES ESR #
A34 , A35	112
BP	1161
CBSQ	3050
CTS218	2105
DTT1Z	2330
H2 , H2.5	2613
HDU	2330
'HGA'	112
HU , HUC	2549
LTP4	112
'MST'	2105
PDPAWL	2138
SET-XP	2508
SET-3G	4057
'ST12'	2105
TITEN HD	2713 CONCRETE - RR 25741 CMU: RR 25741
'U' HNGR	2549

STRUCTURAL STEEL:

1. STRUCTURAL STEEL:
   PROVIDE STRUCTURAL STEEL COMPLYING WITH THE FOLLOWING ASTM STANDARD SPECIFICATIONS, UNLESS NOTED OTHERWISE:
   ANGLES AND CHANNELS..... ASTM A36
   PLATES..... ASTM A36
   HOLLOW STRUCTURAL SECTIONS..... ASTM A500, GRADE B (46 KSI)
   STRUCTURAL STEEL NOTED THUS (50)..... ASTM A572 OR A992, GRADE 50
   ANCHOR BOLTS / THREADED ROD..... ASTM F1554, GRADE 36 U.N.O
   BOLTS (SS)..... ASTM F593
   BOLTS..... ASTM A325
   REINFORCING STEEL..... SEE REINFORCING STEEL SECTION

   WHERE NOTED ON PLANS AS (SS), SHALL BE STAINLESS STEEL. ASTM A709 GRADE 50R, STAINLESS STEEL MECHANICAL TUBING SHALL BE PER ASTM A554 STANDARD SPECIFICATION, AND ALL STAINLESS BARS AND SHAPES SHALL BE PER ASTM A276 STANDARD SPECIFICATIONS.

   FURNISH READILY IDENTIFIABLE STRUCTURAL STEEL COMPLYING WITH CBC 2019 SECTION 2202A.

2. WELDING:
   A. WELDING STANDARD: AWS D1.1 AND AISC 360 (AWS D1.8 AND AISC 341 FOR SEISMIC FORCE RESISTING SYSTEM)
   B. WELDING PROCESS: WELDING PROCEDURES ARE "MEANS AND METHODS" SELECTED BY THE CONTRACTOR FROM AN APPROVED PROCEDURE IN THE AWS D1.1.
   C. PRE-QUALIFIED & NON PRE-QUALIFIED WELDS: WELDS SHALL BE PRE-QUALIFIED COMPLYING WITH WELDING STANDARD. WHERE NON PRE-QUALIFIED WELDS ARE SPECIFICALLY INDICATED, QUALIFY BY TESTING AND PROCEDURE QUALIFICATION TEST RECORD COMPLYING WITH WELDING STANDARD.
   D. WELDER CERTIFICATION: WELDERS SHALL BE CERTIFIED AS REQUIRED BY GOVERNING CODE AUTHORITY.
   E. SPECIAL INSPECTION OF FABRICATED ITEMS: WHERE FABRICATION OF STRUCTURAL, LOAD-BEARING OR LATERAL LOAD-RESISTING MEMBERS OR ASSEMBLIES IS BEING CONDUCTED ON THE PREMISES OF A FABRICATOR'S SHOP, SPECIAL INSPECTIONS OF THE FABRICATED ITEMS SHALL BE PERFORMED DURING FABRICATION PER CBC 2019 SECTION 1704A.2.5
   F. MINIMUM FILLET WELD SIZE: PROVIDE MINIMUM FILLET WELD COMPLYING WITH AISC 360 SECTION J2 AND TABLE J2.4 UNLESS A LARGER WELD IS INDICATED ON DRAWINGS.
   G. SHOP AND FIELD WELDING INDICATION ON DRAWINGS: NO ATTEMPT IS MADE TO DIFFERENTIATE BETWEEN SHOP AND FIELD WELDED CONNECTIONS.
3. WELD INSPECTION:
   A. LEAD WELDING INSPECTOR CERTIFICATION: LEAD WELDING INSPECTOR SHALL BE A CERTIFIED WELDING INSPECTOR (CWI) COMPLYING WITH AWS-QC1 STANDARDS, SHALL BE RECOGNIZED BY THE BUILDING OFFICIAL AS A REGISTERED DEPUTY INSPECTOR FOR STRUCTURAL STEEL WELDING (ICC CERTIFICATION) AND SHALL POSSES A MINIMUM LEVEL OF UT LEVEL II CERTIFICATION.
   B. OTHER WELDING INSPECTORS: WELDING INSPECTORS PERFORMING VISUAL INSPECTION UNDER THE SUPERVISION OF THE LEAD WELDING INSPECTOR SHALL POSSES ICC CERTIFICATION, AND PERSONS PERFORMING NON-DESTRUCTIVE TESTING SHALL POSSES UT LEVEL II CERTIFICATION. FOUR NON-CERTIFIED WELDING INSPECTORS MAXIMUM SHALL BE UNDER THE SUPERVISION OF A CWI.
   C. WELD INSPECTIONS: PROVIDE WELD INSPECTIONS AS REQUIRED BY CBC SECTION 1705.2 AND AISC 360 CHAPTER N (AISC 341 CHAPTER J FOR SEISMIC FORCE RESISTING SYSTEM). SEE STATEMENT OF SPECIAL INSPECTION AND QUALITY ASSURANCE SECTION.
4. THE STRUCTURAL FABRICATOR SHALL FURNISH SHOP DRAWINGS OF ALL STEEL FOR THE ENGINEERS REVIEW PRIOR TO FABRICATION. ALL STEEL FABRICATION SHALL BE PERFORMED IN A SHOP APPROVED BY THE BUILDING DEPARTMENT.
5. EXCEPT WHERE ENCASED IN CONCRETE, MASONRY, OR SPRAYED-ON FIREPROOFING, ALL STEEL SHALL BE PRIMERED UNLESS NOTED OTHERWISE ON THE DRAWINGS. PAINTING OF STRUCTURAL STEEL MEMBERS SHALL COMPLY WITH THE REQUIREMENTS CONTAINED IN AISC 360.
6. OPENINGS SHALL NOT BE PLACED IN STEEL MEMBERS UNLESS SPECIFICALLY DETAILED. STEEL MEMBERS SHALL NOT BE CORED WHEN PERMISSIBLE HOLES ARE CUT WITH A TORCH AFTER STEEL IS ERECTED. THE SHORES SHALL REMAIN IN PLACE UNTIL THE STEEL TEMPERATURE HAS RETURNED TO AIR TEMPERATURE.
7. STRUCTURAL STEEL SHALL BE DELIVERED TO THE JOB SITE FREE OF EXCESSIVE RUST, MILL SCALE, GREASE, ETC. AND SHALL BE PRIMED.
8. NON-SHRINK GROUTS OR DRY-PACKS SHALL BE 6000 PSI MIN. ASTM C109 W/ SPECIAL INSPECTION REQUIRED TYP. BASE PLATES SHALL BE GROUTED PRIOR TO PLACEMENT OF CONCRETE ON ANY UPPER FLOORS.
9. ALL ARCHITECTURALLY EXPOSED STRUCTURAL STEEL (AESS) SHALL MEET THE MINIMUM SPECIFICATIONS IN THE AISC CODE OF STANDARD PRACTICE CHAPTER 10 UNLESS NOTED OTHERWISE ON THE ARCHITECTURAL DRAWINGS.
10. ANCHOR ROD HOLES IN BASE PLATES:
    A. ANCHOR ROD HOLES IN BASE PLATES W/O WASHER PLATES, MAXIMUM HOLE SIZE SHALL BE THE ANCHOR DIAMETER +⅛" MAX.
    B. ANCHOR ROD HOLES IN BASE PLATES W/ WASHER PLATES:

MAXIMUM SIZES FOR ANCHOR-ROD HOLES IN BASE PLATES

ANCHOR ROD DIAMETER, IN.	MAX. HOLE DIAMETER, IN.	MIN. WASHER SIZE, IN.	MIN. WASHER THICKNESS	ANCHOR ROD DIAMETER, IN.	MAX. HOLE DIAMETER, IN.	MIN. WASHER SIZE, IN.	MIN. WASHER THICKNESS
¾	1 1/16	2	¼	1½	2 5/16	3½	½
⅞	1 1/16	2½	5/16	1¾	2¾	4	⅝
1	1 3/16	3	⅜	2	3¼	5	¾
1¼	2 5/16	3	½	2½	3¾	5½	⅞

NOTES:
1. CIRCULAR OR SQUARE WASHERS MEETING THE WASHER SIZE ARE ACCEPTABLE.
2. CLEARANCE MUST BE CONSIDERED WHEN CHOOSING AN APPROPRIATE ANCHOR ROD HOLE LOCATION, NOTHING EFFECTS SUCH AS THE POSITION OF THE ROD IN THE HOLE WITH RESPECT TO THE COLUMN, WELD SIZE AND OTHER INTERFERENCES.
3. WHEN BASE PLATES ARE LESS THAN 1½ IN. THICK, PUNCHING OF HOLES MAY BE AN ECONOMICAL OPTION. IN THIS CASE, ¾-IN. ANCHOR RODS AND 1 5/16-IN. DIAMETER PUNCHED HOLES MAY BE USED WITH ASTM F844 (USS STANDARD) WASHERS IN PLACE OF FABRICATED PLATE WASHERS.
4. FOR USE AT LFRS MEMBERS, WELDING OF WASHER ₹ TO BASE ₹ REQUIRED. (FILLET WELD = WASHER ₹ THK - 1/16 IN. U.N.O. SUBJECT TO E.O.R. APPROVAL.)

MASONRY:

1. ALL MASONRY SHALL COMPLY WITH ANGELUS BLOCK SPECIFICATIONS.
2. MIN. SPECIFIED COMPRESSIVE STRENGTH SHALL BE f'm = 2,000 PSI, UNLESS OTHERWISE SPECIFIED ON THE PLANS.
3. BLOCK SHALL BE MEDIUM WEIGHT UNITS CONFORMING TO ASTM C-90 GRADE N-1. USE UNITS OPEN ONE END, AND BOND BEAM UNITS AT HORIZONTAL REINFORCING. WHEN BLOCKS ARE EXPOSED OBTAIN APPROVAL OF SUBMITTAL FROM ARCHITECT.
4. MORTAR (TYPE M OR S) SHALL ATTAIN A MINIMUM COMPRESSIVE STRENGTH EQUAL TO THE BLOCK STRENGTH OR 2,000 PSI, WHICHEVER IS GREATER.
5. CEMENT: ASTM C-150, LOW ALKALI, TYPE I OR II PORTLAND CEMENT. (MASONRY CEMENT AND PLASTIC CEMENT SHALL NOT BE USED.)
6. MORTAR:
   A. CONFORMING TO ASTM C-270, TYPE M OR S.
   B. MIX PROPORTIONS SHALL CONFORM TO ASTM C-270.
   C. AGGREGATES SHALL CONFORM TO ASTM C-144.
7. GROUT:
   A. CONFORMING TO ASTM C-476.
   B. ATTAINS THE MASONRY COMPRESSIVE STRENGTH f'm OR 2,000 PSI AT 28 DAYS, WHICHEVER IS GREATER.
   C. MIX PROPORTIONS SHALL CONFORM TO ASTM C-476.
   D. AGGREGATES SHALL CONFORM TO ASTM C-404.
   E. DO NOT USE ADMIXTURES UNLESS ACCEPTABLE. FIELD ADDITION OF ADMIXTURES IS NOT PERMITTED IN SELF-CONSOLIDATING GROUT. TMS 602-13, 2.2C
8. REINFORCING STEEL: USE ASTM A 706 OR ASTM A 615 GRADE 60.
9. MINIMUM GROUTING: SOLID GROUT ALL CELLS UNLESS NOTED OTHERWISE.
10. VERTICAL EXPANSION JOINTS SHALL BE PROVIDED AT 26'-0" o/c MAXIMUM. EXPANSION JOINT MATERIALS SHALL BE PER ASTM C90, D994, OR D1056 CLASS 2A1.
11. SPECIAL INSPECTION OF WORK IS REQUIRED FOR ALL MASONRY STRUCTURES PER TMS 402/ACI 530/ASCE 5 LEVEL B QUALITY ASSURANCE, UNLESS NOTED OTHERWISE ON THE DRAWINGS.
12. ALL BOLTS SHALL BE GROUTED IN PLACE WITH AT LEAST 1 INCH OF GROUT BETWEEN THE BOLT AND THE MASONRY.
13. THE CLEAR DISTANCE BETWEEN PARALLEL BARS SHALL NOT BE LESS THAN THE NOMINAL DIAMETER OF THE BARS, NOR LESS THAN 1 IN. (25.4mm).
14. IN COLUMNS AND PILASTERS, THE CLEAR DISTANCE BETWEEN VERTICAL BARS SHALL NOT BE LESS THAN ONE AND
    ONE-HALF TIMES THE NOMINAL BAR DIAMETER, NOR LESS THAN 1½ IN. (38.1mm).
15. THE CLEAR DISTANCE LIMITATIONS BETWEEN BARS REQUIRED IN SECTIONS 1.13.3.1 AND 1.13.3.2 SHALL ALSO APPLY TO THE CLEAR DISTANCE BETWEEN A CONTACT LAP SPLICE AND ADJACENT SPLICES OR BARS.
16. GROUPS OF PARALLEL REINFORCING BARS BUNDLED IN CONTACT TO ACT AS A UNIT SHALL BE LIMITED TO TWO IN ANY ONE BUNDLE. INDIVIDUAL BARS IN A BUNDLE CUT OFF WITHIN THE SPAN OF A MEMBER SHALL TERMINATE AT POINTS AT LEAST 40 BAR DIAMETER APART.
17. REINFORCEMENT EMBEDDED IN GROUT SHALL HAVE A THICKNESS OF GROUT BETWEEN THE REINFORCEMENT AND MASONRY UNITS NOT LESS THAN ¼ IN. (6.4mm) FOR FINE GROUT OR ½ IN. (12.7mm) FOR COARSE GROUT.
18. REINFORCING BARS SHALL HAVE A MASONRY COVER NOT LESS THAN THE FOLLOWING:
    A. MASONRY FACE EXPOSED TO EARTH OR WEATHER: 2 IN. (50.8mm) FOR BARS LARGER THAN NO.5 (M #16); 1½ IN. (38.1mm) FOR NO.5 (M #16) BARS OR SMALLER.
    B. MASONRY NOT EXPOSED TO EARTH OR WEATHER: 1½ IN. (38.1mm).
19. LONGITUDINAL WIRES OF JOINT REINFORCEMENT SHALL BE FULLY EMBEDDED IN MORTAR OR GROUT WITH A MINIMUM COVER OF ⅝ IN. (15.9mm) WHEN EXPOSED TO EARTH OR WEATHER AND ½ IN. (12.7mm) WHEN NOT EXPOSED TO EARTH OR WEATHER. JOINT REINFORCEMENT SHALL BE STAINLESS STEEL OR PROTECTED FROM CORROSION BY HOT-DIPPED GALVANIZED COATING OR EPOXY COATING WHEN USED IN MASONRY EXPOSED TO EARTH OR WEATHER AND IN INTERIOR WALLS EXPOSED TO A MEAN RELATIVE HUMIDITY EXCEEDING 75 PERCENT. ALL OTHER JOINT REINFORCEMENT SHALL BE MILL GALVANIZED, HOT-DIP GALVANIZED, OR STAINLESS STEEL.
20. WALL TIES, SHEET-METAL ANCHORS, STEEL PLATES AND BARS, AND INSERTS EXPOSED TO EARTH OR WEATHER, OR EXPOSED TO A MEAN RELATIVE HUMIDITY EXCEEDING 75 PERCENT SHALL BE STAINLESS STEEL OR PROTECTED FROM CORROSION BY HOT-DIP GALVANIZED COATING OR EPOXY COATING. WALL TIES, ANCHORS, AND INSERTS SHALL BE MILL GALVANIZED, HOT-DIP GALVANIZED, OR STAINLESS STEEL FOR ALL OTHER CASES. ANCHOR BOLTS, STEEL PLATES, AND BARS NOT EXPOSED TO EARTH, WEATHER, NOR EXPOSED TO A MEAN RELATIVE HUMIDITY EXCEEDING 75 PERCENT, NEED NOT BE COATED.
21. REINFORCEMENT SHALL BE SUPPORTED AND FASTENED TOGETHER TO PREVENT DISPLACEMENTS BEYOND THE TOLERANCES ALLOWED BY SECTION 3.4 OF ACI 530.1 PRIOR TO GROUTING.
22. THE MAXIMUM HEIGHT OF A POUR, WHICH IS CONSIDERED AS THE ENTIRE HEIGHT OF GROUT FILL PLACED IN ONE DAY AND IS COMPOSED OF A NUMBER OF SUCCESSIVELY PLACED GROUT LIFTS (DSA IR 21-2 SECTION 1.3), SHALL NOT EXCEED 12 FEET WHEN THE MASONRY HAS CURED FOR 4 HOURS, THE GROUT SLUMP IS MAINTAINED BETWEEN 10 AND 11 INCHES AND NO INTERMEDIATE REINFORCED BOND BEAMS ARE PLACED BETWEEN THE TOP AND THE BOTTOM OF THE POUR HEIGHT, OTHERWISE, PLACE GROUT IN LIFTS NOT EXCEEDING 5 FEET. (SECTION 3.5 OF ACI 530.1)
23. PIPES AND CONDUITS EMBEDDED IN MASONRY SHALL NOTE REDUCE THE REQUIRED STRENGTH. (SECTION 1.16.2 OF ACI 530)
24. SUPPORT AND FASTEN REINFORCEMENT TOGETHER TO PREVENT DISPLACEMENT BEYOND THE TOLERANCES ALLOWED BY CONSTRUCTION LOADS OR BY PLACEMENT OF GROUT OR MORTAR.
25. COMPLETELY EMBED REINFORCING BARS IN GROUT IN ACCORDANCE WITH ARTICLE 3.5 ACI 350.
26. MAINTAIN CLEAR DISTANCE BETWEEN REINFORCING BARS AND ANY FACE OF MASONRY UNIT OR FORMED SURFACE, BUT NOT LESS THAN ¼ IN. (6.4mm) FOR FINE GROUT OR ½ IN. (12.7mm) FOR COARSE GROUT.
27. SPLICE ONLY WHERE INDICATED ON THE PROJECT DRAWINGS, UNLESS OTHERWISE ACCEPTABLE. WHEN SPLICING BY WELDING, PROVIDE WELDS IN CONFORMANCE WITH THE PROVISIONS OF AWS D1.4.
28. UNLESS ACCEPTED BY THE ARCHITECT/ENGINEER, DO NOT BEND REINFORCEMENT AFTER IT IS EMBEDDED IN GROUT OR MORTAR.
29. NON-CONTACT LAP SPLICES - POSITION BARS SPLICED BY NON-CONTACT LAP SPLICE NO FARTHER APART TRANSVERSELY THAN ONE-FIFTH THE SPECIFIED LENGTH OF LAP NOR MORE THAN 8 IN. (203mm).
30. TOLERANCES FOR THE PLACEMENT OF REINFORCING BARS IN WALLS AND FLEXURAL ELEMENTS SHALL BE ± ½ IN. (12.7mm) WHEN THE DISTANCE FROM THE CENTERLINE OF REINFORCING BARS TO THE OPPOSITE FACE OF MASONRY, d, IS EQUAL TO 8 IN. (203mm) OR LESS, ± 1 IN. (25.4mm) FOR d EQUAL TO 24 IN. (610mm) OR LESS BUT GREATER THAN 8 IN. (203mm), AND ± 1¼ IN. (31.8mm) FOR d GREATER THAN 24 IN. (610mm).
31. PLACE VERTICAL BARS WITHIN 2 IN. (50.8mm) OF THE REQUIRED LOCATION ALONG THE LENGTH OF THE WALL.
32. IF IT IS NECESSARY TO MOVE BARS MORE THAN ONE BAR DIAMETER OR A DISTANCE EXCEEDING THE TOLERANCE STATED ABOVE TO AVOID INTERFERENCE WITH OTHER REINFORCING STEEL, CONDUITS, OR EMBEDDED ITEMS, NOTIFY THE ARCHITECT/ENGINEER FOR ACCEPTANCE OF THE RESULTING ARRANGEMENT OF BARS.
33. FOUNDATION DOWELS THAT INTERFERE WITH UNIT WEBS ARE PERMITTED TO BE BENT TO A MAXIMUM OF 1 IN. (25mm) HORIZONTALLY FOR EVERY 6 IN. (152mm) OF VERTICAL HEIGHT.
34. EMBED HEADED AND BENT-BAR ANCHOR BOLTS LARGER THAN ¼ IN. (6.4mm) DIAMETER IN GROUT THAT IS PLACED IN ACCORDANCE WITH ARTICLE 3.5 A AND ARTICLE 3.5 B ACI 350. ANCHOR BOLTS OF ¼ IN. (6.4mm) DIAMETER OR LESS ARE PERMITTED TO BE PLACED IN GROUT OR MORTAR BED JOINTS THAT HAVE A SPECIFIED THICKNESS OF AT LEAST ½ IN. (12.7mm) THICKNESS.
35. MAINTAIN CLEAR DISTANCE BETWEEN ANCHOR BOLTS AND ANY FACE OF MASONRY UNIT OR FORMED SURFACE OF AT LEAST ¼ IN. (6.4mm) WHEN USING FINE GROUT, AND OF AT LEAST ½ IN. (12.7mm) WHEN USING COARSE GROUT.
36. PLACE ANCHOR BOLTS WITH A CLEAR DISTANCE BETWEEN PARALLEL ANCHOR BOLTS NOT LESS THAN THE NOMINAL DIAMETER OF THE ANCHOR BOLT, NOR LESS THAN 1 IN. (25.4mm).
37. OWNER WILL ENGAGE A QUALIFIED INDEPENDENT TESTING AGENCY TO PERFORM PRECONSTRUCTION TESTING INDICATED BELOW. RETESTING OF MATERIALS THAT FAIL TO COMPLY WITH SPECIFIED REQUIREMENTS SHALL BE DONE AT CONTRACTOR'S EXPENSE.
38. CONCRETE MASONRY UNIT TEST: FOR EACH TYPE OF UNIT REQUIRED, ACCORDING TO ASTM C 140 FOR COMPRESSIVE STRENGTH.
39. MORTAR TEST (PROPERTY SPECIFICATION): FOR EACH MIX REQUIRED, ACCORDING TO ASTM C 109/C 109M FOR COMPRESSIVE STRENGTH[, ASTM C 1506 FOR WATER RETENTION, AND ASTM C 91 FOR AIR CONTENT].
40. MORTAR TEST (PROPERTY SPECIFICATION): FOR EACH MIX REQUIRED, ACCORDING TO ASTM C 780 FOR COMPRESSIVE STRENGTH.
41. GROUT TEST (COMPRESSIVE STRENGTH): FOR EACH MIX REQUIRED, ACCORDING TO ASTM C 1019.
42. COMPRESSIVE STRENGTH OF MASONRY BASED ON THE COMPRESSIVE STRENGTH OF CONCRETE MASONRY UNITS AND TYPE OF MORTAR USED IN CONSTRUCTION.

NET AREA COMPRESSIVE STRENGTH OF CONCRETE MASONRY, PSI (MPa)	NET AREA COMPRESSIVE STRENGTH OF CONCRETE MASONRY UNITS, PSI (MPa)	
	TYPE M OR S MORTAR	TYPE N MORTAR
1,700 (11.72)	---	1,900 (13.10)
1,900 (13.10)	1,900 (13.10)	2,350 (14.82)
2,000 (13.79)	2,000 (13.79)	2,650 (18.27)
2,250 (15.51)	2,600 (17.93)	3,400 (23.44)
2,500 (17.24)	3,250 (22.41)	4,350 (28.96)
2,750 (18.96)	3,900 (26.89)	----
3,000 (20.69)	4,500 (31.03)	----

1. FOR UNITS OF LESS THAN 4 IN. (102 mm) NOMINAL HEIGHT, USE 85 PERCENT OF THE VALUES LISTED.

1 CHECK & VERIFY ALL FOOTING ELEVATIONS AGAINST FOOTING SCHEDULE AND ARCHITECTURAL DRAWINGS FOR FOOTING ELEVATIONS FOR PROPER COURSING ELEVATIONS.

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. S0.03, (A#03-122536, DATED 03/24/2023)

BID CLARIFICATION NO.1 (06-20-2023)

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An Employee Owned Corporation

ENGINEER

REGISTERED PROFESSIONAL ENGINEER No. 5152 STATE OF CALIFORNIA STRUCTURAL

LICENSED ARCHITECT C-26053 Ren. Date 08/31/2023 STATE OF CALIFORNIA

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

GENERAL NOTES

Job No.
22005

Date
06-07-22

S0.03

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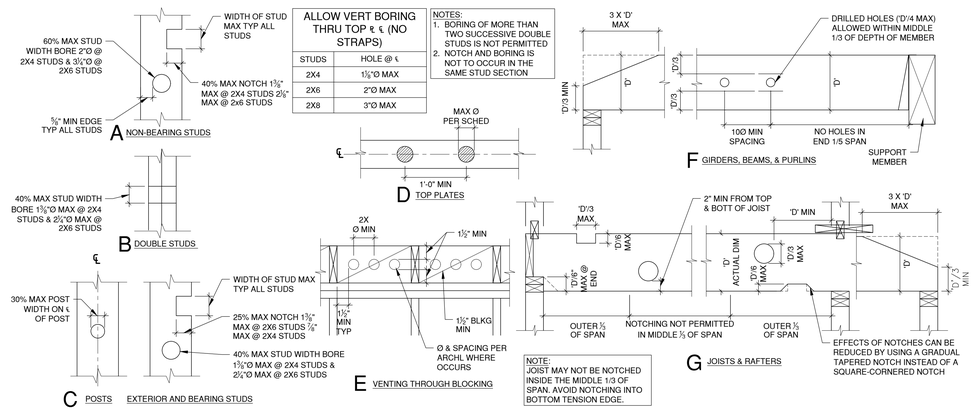

ALLOW VERT BORING THRU TOP ℄ ℄ (NO STRAPS)	
STUDS	HOLE @ ℄
2X4	1⅜"Ø MAX
2X6	2"Ø MAX
2X8	3"Ø MAX

NOTES:
1. BORING OF MORE THAN TWO SUCCESSIVE DOUBLE STUDS IS NOT PERMITTED
2. NOTCH AND BORING IS NOT TO OCCUR IN THE SAME STUD SECTION

NOTE:
JOIST MAY NOT BE NOTCHED INSIDE THE MIDDLE 1/3 OF SPAN. AVOID NOTCHING INTO BOTTOM TENSION EDGE.

ALLOWABLE NOTCHING & HOLES

SCALE:1"=1'-0" | 9

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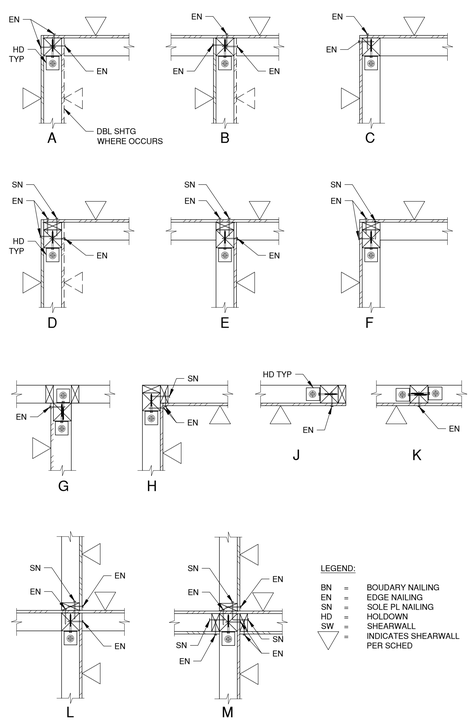
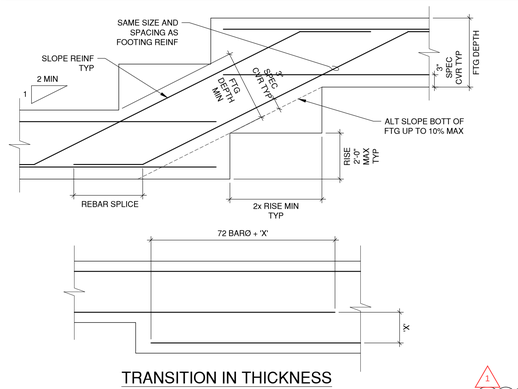
SCALE:1"=1'-0" | 14

TYP SW INTERSECTIONS (W/ HOLDOWNS)

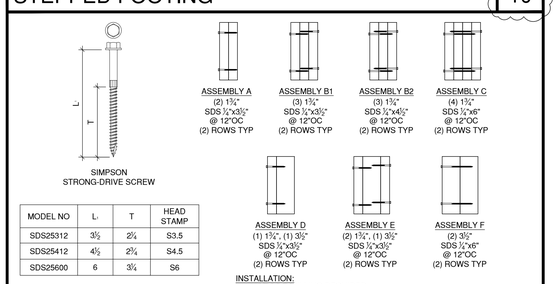
SCALE:1"=1'-0" | 16





SCALE:1"=1'-0" | 10

STEPPED FOOTING

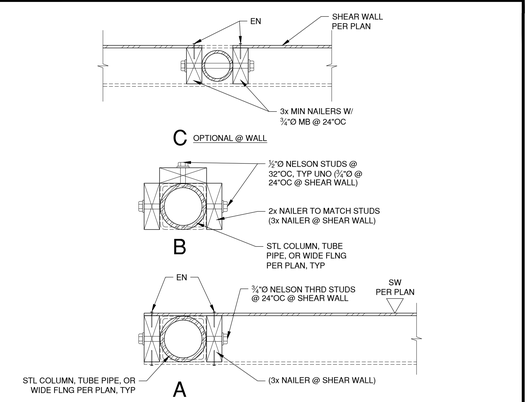


MODEL NO	L	T	HEAD STAMP
SDS25312	3½	2¼	S3.5
SDS25412	4½	2¾	S4.5
SDS25600	6	3¼	S6

INSTALLATION:
- NO PRE-DRILLING REQUIRED.
- SEE ILLUSTRATIONS FOR SDS POSITIONING ON DIFFERENT ASSEMBLIES
- INSTALL WITH HIGH-TORQUE, LOW-SPEED DRILL (5 AMP+).
- DO NOT OVER-DRIVE THE SDS SCREWS.

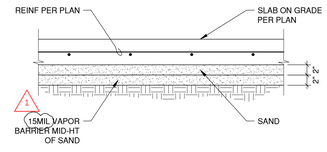
ENG WD & STRUCT COMPOSITE LUMBER CONN

SCALE:1"=1'-0" | 11



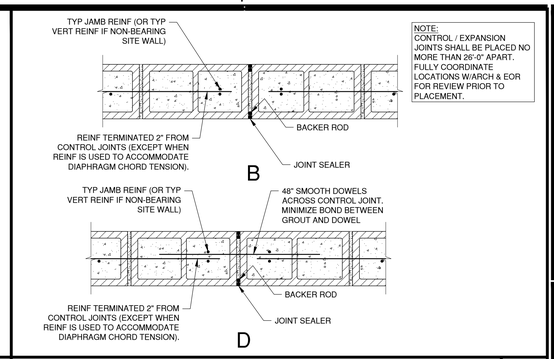
COLUMN NAILER

SCALE:1"=1'-0" | 12



TYPICAL SLAB ON GRADE

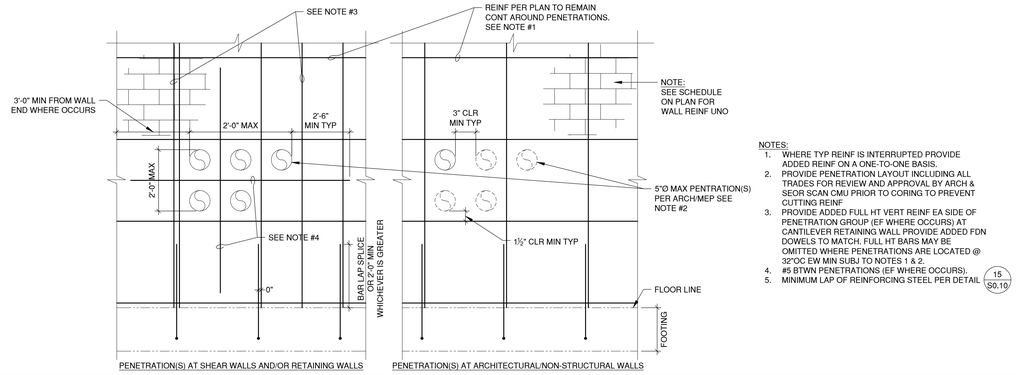
SCALE:1"=1'-0" | 5



NOTE:
CONTROL / EXPANSION JOINTS SHALL BE PLACED NO MORE THAN 26'-0" APART. FULLY COORDINATE LOCATIONS W/ARCH & EOR FOR REVIEW PRIOR TO PLACEMENT.

CMU CONTROL/EXPANSION JOINT DETAILS

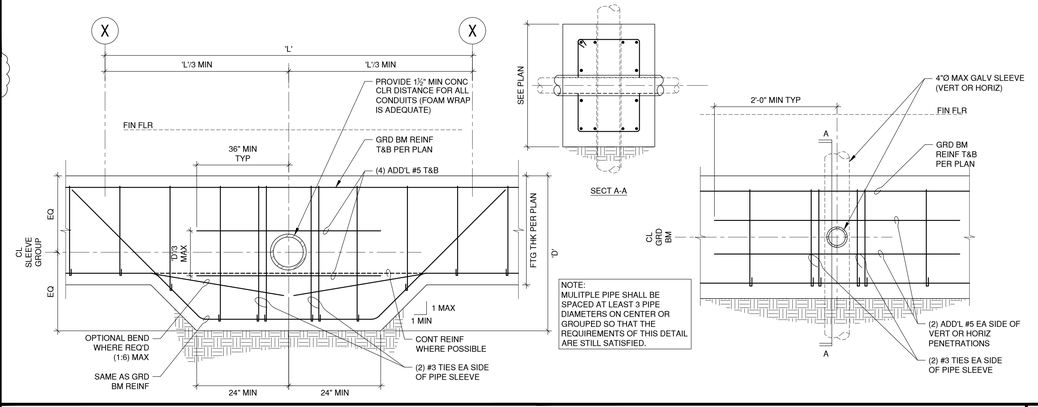
SCALE:1"=1'-0" | 1



NOTES:
1. WHERE TYP REINF IS INTERRUPTED PROVIDE ADDED REINF ON A ONE-TO-ONE BASIS.
2. PROVIDE PENETRATION LAYOUT INCLUDING ALL TRADES FOR REVIEW AND APPROVAL BY ARCH & SEOR SCAN CMU PRIOR TO CORING TO PREVENT CUTTING REINF
3. PROVIDE ADDED FULL HT VERT REINF EA SIDE OF PENETRATION GROUP (EF WHERE OCCURS) AT CANTILEVER RETAINING WALL PROVIDE ADDED FDN DOWELS TO MATCH. FULL HT BARS MAY BE OMITTED WHERE PENETRATIONS ARE LOCATED @ 32"OC EW MIN SUBJ TO NOTES 1 & 2.
4. #5 BTWN PENETRATIONS (EF WHERE OCCURS).
5. MINIMUM LAP OF REINFORCING STEEL PER DETAIL 15/S0.10

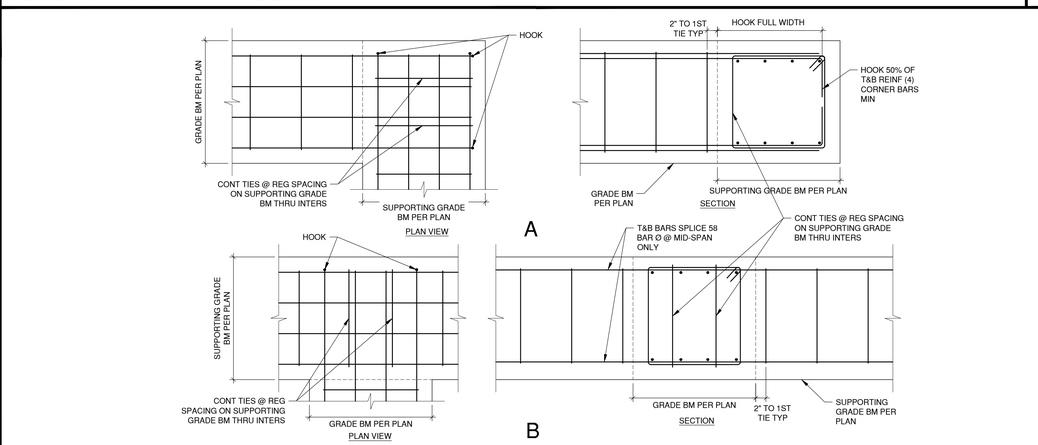
TYPICAL CMU WALL DETAIL WITH PENETRATIONS

SCALE:1"=1'-0" | 2



PENETRATIONS AT GRADE BM

SCALE:1"=1'-0" | 3



GRADE BEAM INTERSECTIONS

SCALE:1"=1'-0" | 4

BID CLARIFICATION NO.1 (06-20-2023)

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ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

TYPICAL DETAILS

Job No. 22005

Date 06-07-22

S0.20

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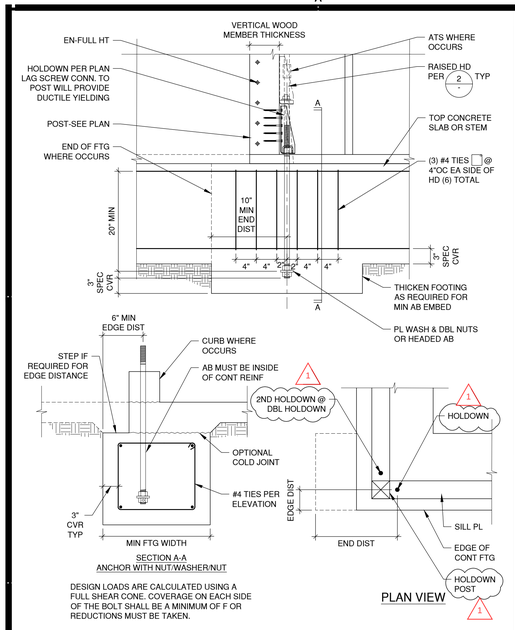


SECTION A-A
ANCHOR WITH NUT/WASHER/NUT

PLAN VIEW

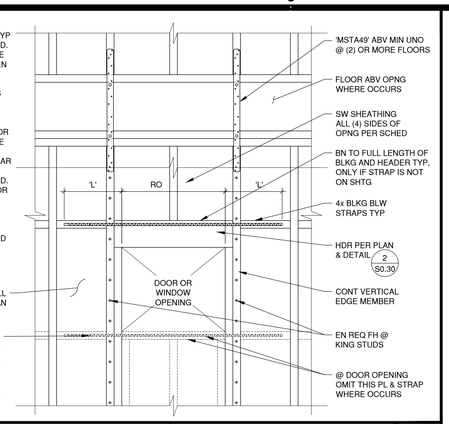
DESIGN LOADS ARE CALCULATED USING A FULL SHEAR CONE. COVERAGE ON EACH SIDE OF THE BOLT SHALL BE A MINIMUM OF F OR REDUCTIONS MUST BE TAKEN.

1. ANCHOR EMBEDMENT LENGTH IS BASED ON A SINGLE POUR CONCRETE FOUNDATION WITHIN THE FOOTING DIMENSIONS de. DOUBLE POUR FOUNDATION SYSTEMS, MASONRY WALLS AND MASONRY FOOTINGS MUST BE EVALUATED BY THE DESIGNER.
2. ANCHOR BOLT MUST BE ASTM A307 OR A36 STEEL.
3. ANCHOR HEAD REQUIRED TWO HEX NUTS WITH A 3x3x⁵⁄₁₆" PLATE AT TYPE 'G' HD. USE OF SIMPSON STRONG-TIE BEARING PLATE BP1 MAY BE USED AS A SUBSTITUTE FOR 1" DIAMETER BOLTS AND SIMPSON STRONG-TIE BEARING PLATE BP⅝ MAY BE USED AS A SUBSTITUTE PLATE FOR ¾" DIAMETER BOLTS.
4. AT ATS LOCATIONS, ANCHOR BOLT DIAMETER & GRADE TO MATCH ATS DIAMETER & GRADE.
5. DESIGN VALUES ARE BASED ON ACI 318-14 CH. 17. SEISMIC LOADS DUCTILITY REQUIREMENTS PER D.3.3.4 ARE CONSIDERED.
6. FOR HOLDOWNS, PER ASTM TEST STANDARDS, ANCHOR BOLT NUT SHOULD BE FINGER-TIGHT PLUS ⅓ TO ½ TURN WITH A HAND WRENCH, WITH CONSIDERATION GIVEN TO POSSIBLE FUTURE WOOD SHRINKAGE. CARE SHOULD BE TAKEN TO NOT OVER-TORQUE THE NUT. IMPACT WRENCHES SHOULD NOT BE USED.
7. TENSION VALUES ARE VALID FOR HOLDOWNS FLUSH OR RAISED OFF SILL PLATE.
8. REQUIRES HEAVY HEX ANCHOR NUT TO ACHIEVE TABULATED LOADS (SUPPLIED WITH HOLDOWN).

HD OR ATS ANCHOR DETAIL (30K ALLOW)

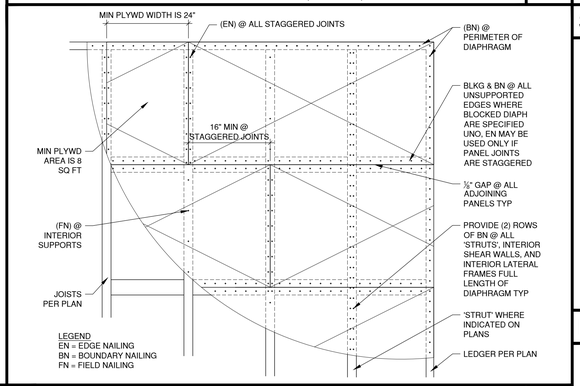
SCALE:1"=1'-0" | 14

NOTES:
1. EN AROUND OPENING TYP SEE SHEAR WALL SCHED.
2. IF SHTG IS ON OPPOSITE SIDE OF STRAPS, ADD EN ALL BLKG.
3. IF SHTG IS ON BOTH SIDES, INSTALL STRAPS TO BOTH SIDES OVER SHTG.
4. L=R.O./2, 24" MIN.
5. WHERE SIMPSON 'A35' OR 'LTP4' OCCUR, THEY ARE TO BE CONTINUOUS THROUGHOUT THE SHEAR WALL W/ SPACING PER THE SHEAR WALL SCHED.
6. SILL NAILING OR ANCHOR BOLTS ARE TO BE CONTINUOUS THROUGHOUT SHEAR WALL W/ SPACING PER THE SHEAR WALL SCHED INCLUDING BLW OPNG.



TYP FRMG AT SW OPENING (SWO)

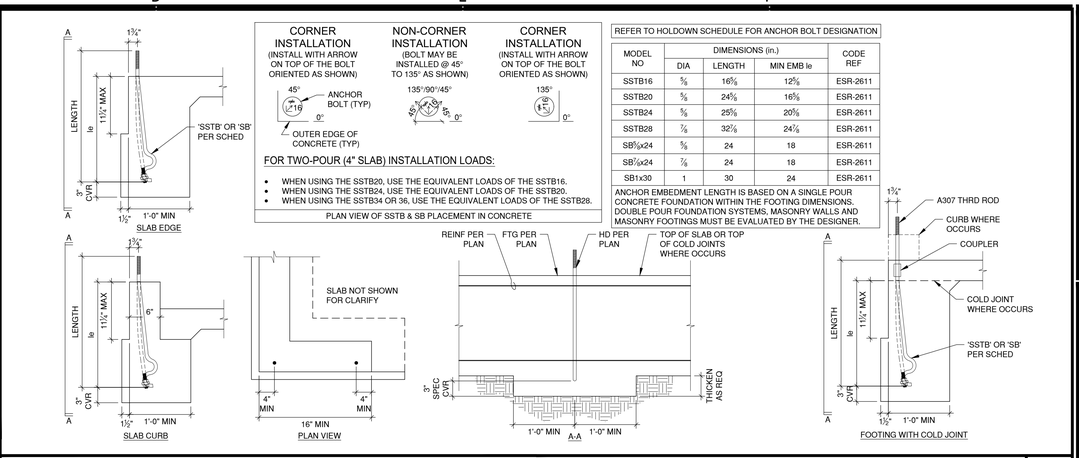
SCALE:1"=1'-0" | 9



LEGEND
EN = EDGE NAILING
BN = BOUNDARY NAILING
FN = FIELD NAILING

FLOOR & ROOF SHTG DETAIL

SCALE:1"=1'-0" | 10



CORNER INSTALLATION (INSTALL WITH ARROW ON TOP OF THE BOLT ORIENTED AS SHOWN)
NON-CORNER INSTALLATION (BOLT MAY BE INSTALLED @ 45° TO 135° AS SHOWN)
CORNER INSTALLATION (INSTALL WITH ARROW ON TOP OF THE BOLT ORIENTED AS SHOWN)

FOR TWO-POUR (4" SLAB) INSTALLATION LOADS:

- WHEN USING THE SSTB20, USE THE EQUIVALENT LOADS OF THE SSTB16.
- WHEN USING THE SSTB24, USE THE EQUIVALENT LOADS OF THE SSTB20.
- WHEN USING THE SSTB34 OR 36, USE THE EQUIVALENT LOADS OF THE SSTB28.

PLAN VIEW OF SSTB & SB PLACEMENT IN CONCRETE

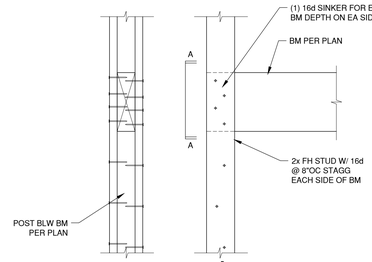
REFER TO HOLDOWN SCHEDULE FOR ANCHOR BOLT DESIGNATION

MODEL NO	DIMENSIONS (in.)			CODE REF
	DIA	LENGTH	MIN EMB le	
SSTB16	⅝	16⅝	12⅝	ESR-2611
SSTB20	⅝	24⅝	16⅝	ESR-2611
SSTB24	⅝	25⅝	20⅝	ESR-2611
SSTB28	⅞	32⅝	24⅞	ESR-2611
SB⅝x24	⅝	24	18	ESR-2611
SB⅞x24	⅞	24	18	ESR-2611
SB1x30	1	30	24	ESR-2611

ANCHOR EMBEDMENT LENGTH IS BASED ON A SINGLE POUR CONCRETE FOUNDATION WITHIN THE FOOTING DIMENSIONS. DOUBLE POUR FOUNDATION SYSTEMS, MASONRY WALLS AND MASONRY FOOTINGS MUST BE EVALUATED BY THE DESIGNER.

SSTB & SB BOLTS AT SLAB ON GRADE

SCALE:1"=1'-0" | 1



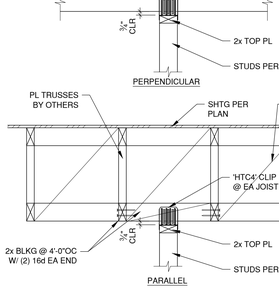
FOUNDATIONS - MANDATORY MINIMUM FORMWORK (UNLESS FULLY FORMED)

TO ENSURE UNIFORMITY IN BIDDING, THE FOLLOWING PERMISSIVE PROCEDURE SHOULD BE NOTED IN THE SPECIFICATIONS, IMMEDIATELY FOLLOWING THE SECTION ON FORMWORK. THE SKETCH SHOULD BE SHOWN ON THE DRAWINGS.

"FOUNDATION CONCRETE MAY BE PLACED DIRECTLY INTO NEAT EXCAVATIONS PROVIDED THE FOUNDATION TRENCH WALLS ARE STABLE AS DETERMINED BY THE ARCHITECT (STRUCTURAL ENGINEER), SUBJECT TO THE APPROVAL OF THE OFFICE OF THE STATE ARCHITECT. IN SUCH CASE, THE MINIMUM FORMWORK SHOWN ON THE DRAWINGS IS MANDATORY TO ENSURE CLEAN EXCAVATIONS IMMEDIATELY PRIOR TO AND DURING THE PLACING OF CONCRETE."

DSA STARTER WALL DETAIL

SCALE:1"=1'-0" | 6



HOLDOWN RAISED OFF SILL PLATE

SCALE:1"=1'-0" | 2

A WALL TO SAWN LUMBER

B WALL TO TJI'S

C WALL TO TRUSSES

PARTITION BRACING

SCALE:1"=1'-0" | 3

NOTE:
1. NAIL 'A34' W/ (8) 8d COMMON X 1½" LONG NAILS
2. NAILS SHALL BE AT LEAST 2 INCHES APART

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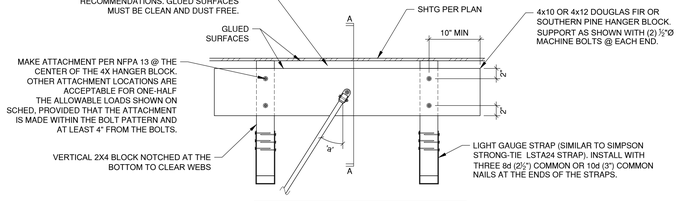
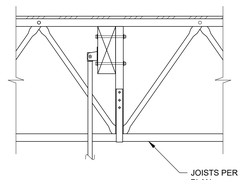
BEAM POCKET

SCALE:1"=1'-0" | 4

FLAT BLOCK. GLUE TOP AND BOTT SURFACES WITH (4) ½" BEADS OF APPROVED ELASTOMERIC FLOOR ADHESIVE. FOLLOW MANUFACTURER'S RECOMMENDATIONS. GLUED SURFACES MUST BE CLEAN AND DUST FREE.



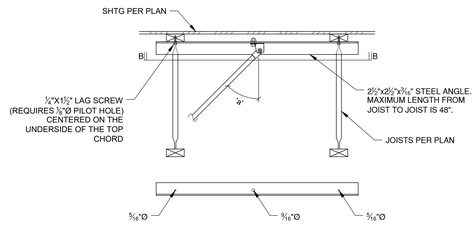


SECTION A-A

ALLOWABLE HORIZONTAL SEISMIC FORCES (lbs)		
ANGLE TO VERTICAL 'a'		
30°	45°	60°
800	1400	1400

- THE LOADS IN THE TABLE ARE BASED ON THE CONTROLLING CONNECTION TO THE TRUSS.
- THE CAPACITY OF THE BRACE FASTENER TO THE WOOD BLOCK MAY LIMIT THE CAPACITY OF THE DETAIL (SEE NFPA 13).
- THE FORCES INCLUDE A 1.33 DURATION FACTOR ADJUSTMENT.

FOR FORCES PERPENDICULAR TO OPEN-WEB TRUSSES

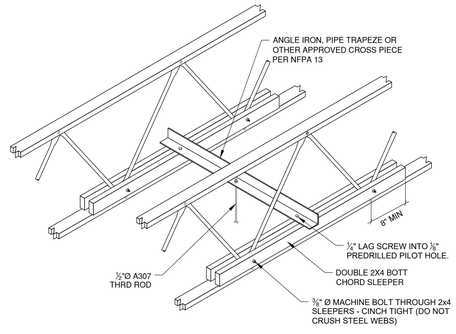


SECTION B-B

ALLOWABLE HORIZONTAL SEISMIC FORCES (lbs)		
ANGLE TO VERTICAL 'a'		
30°	45°	60°
190	240	240

- THE LOADS IN THE TABLE ARE BASED ON THE CONTROLLING CONNECTION TO THE TRUSS.
- THE FORCES INCLUDE A 1.33 DURATION FACTOR ADJUSTMENT.

FOR FORCES PARALLEL OR PERPENDICULAR TO OPEN-WEB TRUSSES



ROD TO OPEN WEB

OPEN-WEB TRUSS SEISMIC FORCE DETAILS

SCALE:1"=1'-0" | 12

AGENCY

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REGISTERED PROFESSIONAL ENGINEER No. 5602 CIVIL STRUCTURAL STATE OF CALIFORNIA

LICENSED ARCHITECT C-26053 Ren. Date 08/31/2023 STATE OF CALIFORNIA

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ROSAMOND HIGH SCHOOL

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TYPICAL DETAILS

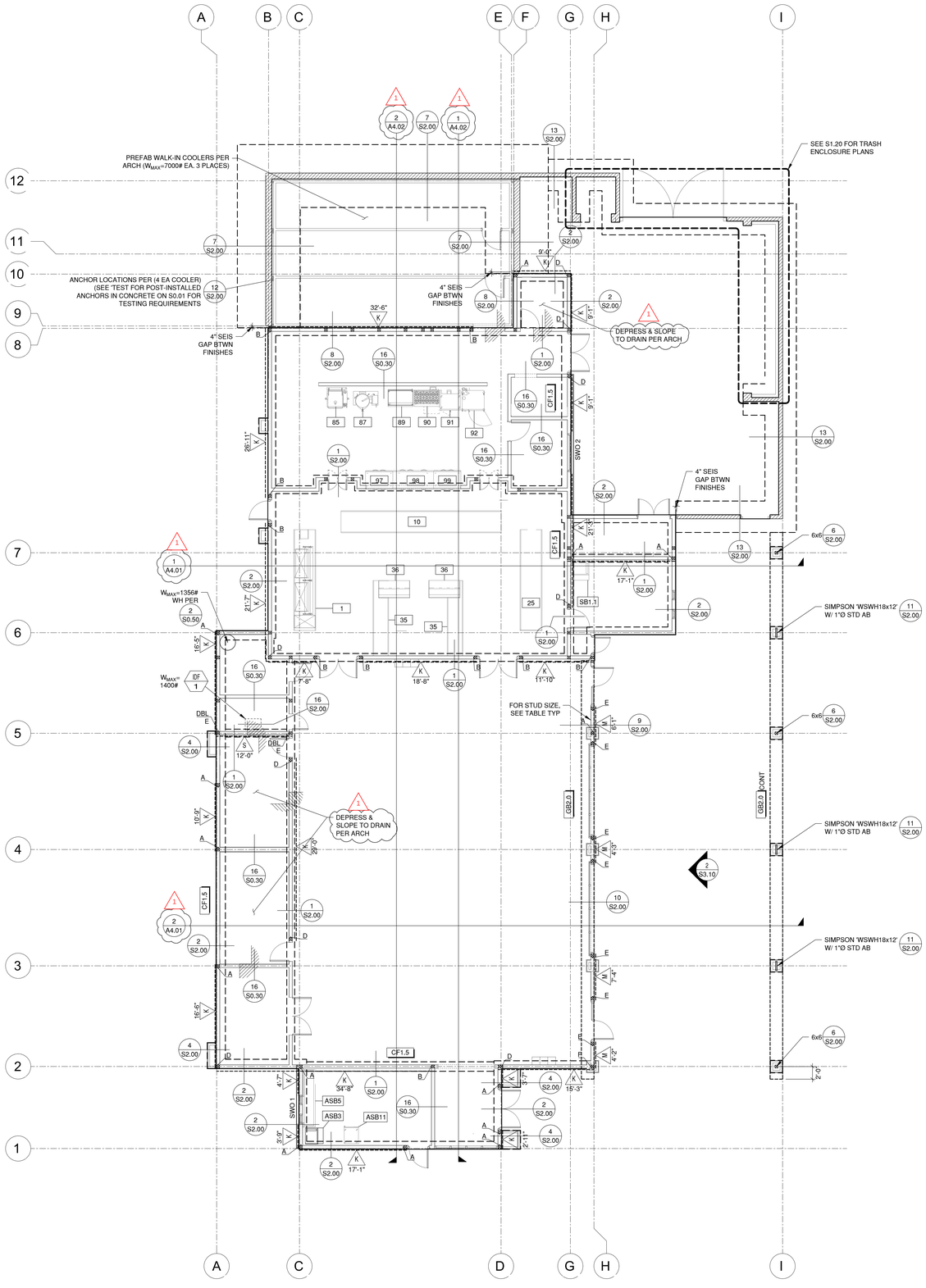
Job No.
22005

Date
06-07-22

S0.40

BID CLARIFICATION NO.1 (06-20-2023)

6/14/2023 4:54:23 PM



CONTINUOUS FOOTING SCHEDULE				
MARK	WIDTH	THICKNESS	REINFORCEMENT	DEPTH
CF1.5	1'-6"	20" MIN	(3) #6 CONT BARS @ T&B	12" MIN BLW LOWEST ADJ GRD
NOTES:				

GRADE BEAM SCHEDULE			
MARK	SIZE	REINF	TIES
GB2.0	24"W X 24"THK	(4) #6 T&B	#3 TIES @ 12"OC

EXTERIOR WALL STUD SIZE	
MAX PLATE OR LEDGER HT	MIN STUD SIZE
13'-6"	1¾ x 5½ VERSA-LAM LVL 2.1E 2800 @ 16"OC (OR 2 x 6 @ 16"OC) 1
15'-6"	1¾ x 5½ VERSA-LAM LVL 2.1E 2800 @ 16"OC
19'-6"	1¾ x 7¼ VERSA-LAM LVL 2.1E 2800 @ 16"OC
27'-6"	1¾ x 9¼ VERSA-LAM LVL 2.1E 2800 @ 16"OC
NOTE: ENGINEERED STUDS PER ICC-ES ESR-1387	

HOLDOWN SCHEDULE							
MARK	ALLOW. LOAD	ICC ESR #	MIN. WOOD MEMBER THICKNESS	HOLDOWN	POST FASTENERS	FDN. BOLT	DETAIL @ SLAB ON GRADE
A	2460 lb.	2330, 2611	(2) 2X	SIMPSON HDU2-SDS2.5	(6) SDS ¼" X 2½" SCREWS	SSTB16/ SB⅝X24	1 S0.40
B	3652 lb.	2330, 2611	4X	SIMPSON HDU4-SDS2.5	(10) SDS ¼" X 2½" SCREWS	SSTB20/ SB⅝X24	1 S0.40
D	5116 lb.	2330, 2611	4X	SIMPSON HDU8-SDS2.5	(20) SDS ¼" X 2½" SCREWS	SSTB28/ SB⅞X24	1 S0.40
E	7628 lb.	2330, 2611	4X6	SIMPSON HDU11-SDS 2.5	(30) SDS ¼" X 2½" SCREWS	SB1X30	1 S0.40
F	11175 lb.	2330, 2611	4X8	SIMPSON HDU11-SDS 2.5	(30) SDS ¼" X 2½" SCREWS	SB1X30	1 S0.40
DBL E	15256 lb.	2330, 2611	4X12 OR 6X6	(2) SIMPSON HDU11-SDS 2.5	(30) SDS ¼" X 2½" SCREWS EA HDU	SB1X30 HDU	14 S0.40

NOTES:
1. MAXIMUM HOLDOWN SYSTEM DEFLECTION PER FLOOR SHALL BE .2" TYP. U.N.O.
2. WHEN ALTERNATIVE CONTINUOUS ROD SYSTEMS ARE USED TO RESIST LIGHT-FRAME SHEAR WALL OVERTURNING FORCES, CALCULATIONS MUST BE SUBMITTED TO THE CODE OFFICIAL CONFIRMING THAT THE TOTAL VERTICAL DISPLACEMENT, WHICH WOULD INCLUDE STEEL ROD ELONGATION AND THE SHRINKAGE COMPENSATING DEVICE DEFLECTION, IS LESS THAN OR EQUAL TO 0.200 INCH (5 mm) FOR EACH STORY, OR BETWEEN RESTRAINTS, WHICHEVER IS MORE RESTRICTIVE. USING ALLOWABLE STRESS DESIGN (ASD), SHEAR WALL DRIFT LIMIT CALCULATIONS MUST CONSIDER THE 0.200 INCH VERTICAL DISPLACEMENT LIMIT.
3. HOLDOWN CONNECTION BOLTS AND NUTS SHALL BE TORQUED ⅓ TURN BEYOND FINGER TIGHT OR AS REQUIRED BY THE MANUF. (INSPECTOR SHALL VERIFY BY RANDOM INSPECTION PRIOR TO COVERING THE WALLS).
4. ALL HOLDOWNS ANCHOR BOLTS SHALL BE TIED IN PLACE PRIOR TO POURING CONCRETE.
5. A.T.R. INDICATES ALL THREADED ROD.

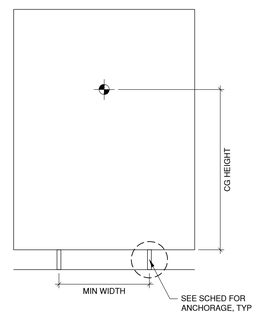
SHEAR WALL SCHEDULE										
MARK	ALLOW. SHEAR	STUD OR BLKG AT ADJOINING PANEL EDGES	PERIODIC SPEC. INSP. PER 1705.11.2	APA RATED OSB	(EN) EDGE NAILING	(FN) FIELD NAILING	(SN) FASTENERS @ 2x SOLE PLATE TO BLKG.1,2	MUDSILL ANCHORS		'A35' OR 'LTP4' WHERE "PER SCHED." NOTED ON DET.	'URFP' OR 'FRFP'3; ICC-ES ESR 2616
								2X MUDSILL	3X MUDSILL4		
K	280 #/'	2x	NO	15/32" STR 1	8d @ 6"OC	12"OC	16d COMMON @ 9"OC	⅝"Ø @ 4'-0"	⅝"Ø @ 4'-0"	19"OC MAX	@ 4'-6"OC
M	550 #/'	3x	YES	15/32" STR 1	8d @ 3"OC	12"OC	16d COMMON @ 8"OC	⅝"Ø @ 2'-8"	⅝"Ø @ 3'-5"	9"OC MAX	@ 2'-4"OC
S	1740 #/'	3x	YES	15/32" STR 1 BOTH SIDES	10d @ 2"OC BOTH SIDES	8"OC	N/A	N/A	⅝"Ø @ 1'-1"	6"O.C. MAX BOTH SIDES	@ 9"O.C.

1. 3x SILL PL ONLY OCCURS AT FOUNDATIONS AND STRUCTURAL SLABS, AND NOT ON UPPER FLOORS.
2. USE 1½"X MIN BLKG BLW MIN WHERE 16d < 4"OC OR SDWS LAGS OCCUR.
3. 'FRFP' SHALL USE TOTAL OF (5) ¼" SDS SCREW PER MANUF SPECIFICATION.
4. FOR INFO NOT SHOWN SEE DETAIL 2 S0.30

EQUIPMENT LIST						
MARK	DESCRIPTION	CG HEIGHT (ft)	MIN WIDTH (ft)	# OF ANCHORAGE POINTS PER SIDE	MAX WEIGHT IN LBS.	DETAIL
1	COUNTER	1.8961	2.375	5	2000	15 S2.00
10	COUNTER	1.8961000	2.375	6	2000	15 S2.00
25	COUNTER	1.8961	2.375	5	2000	15 S2.00
35	POS COUNTER	1.8961	2.375	4	1500	15 S2.00
36	REFRIGERATED SELF-SERVICE CASE	3.0708	2.333	2	1300	15 S2.00
85	TILTING BRAISING PAN	2.4567	2.6667	2	800	15 S2.00
87	TILTING, STEAM JACKETED KETTLE	2.2892	2	2	800	15 S2.00
89	THERMOSTATIC GRIDDLE	3	2	2	680	15 S2.00
90	8 BURNER RANGE W/ OVEN	2.375	1.9167	2	800	15 S2.00
91	DOUBLE CONVECTION OVEN	4.1317	2	2	1300	15 S2.00
92	COMBI OVEN GAS	4.2992	2.75	2	2000	15 S2.00
97	HEATED HOLDING CABINET, PASS-THRU	4.7458	2	2	700	15 S2.00
98	REFRIGERATOR, PASS-THRU	4.7458	2	2	1200	15 S2.00
99	HEATED HOLDING CABINET, PASS-THRU	4.7458	2	2	700	15 S2.00
ASB3	ICE BIN	2.4567	2.0833	2	500	15 S2.00
ASB5	SERVING COUNTER	1.8961	2	3	400	15 S2.00
ASB11	FREEZER, REACH-IN, GLASS DOOR	4.7458	2	2	600	15 S2.00
SB1.1	ICE BIN	2.7917	2.0833	2	500	15 S2.00

FOUNDATION NOTES:

1. SEE FOUNDATION NOTES ON SHEET S0.01 FOR DESIGN SOIL BEARING PRESSURE, ETC.
2. C1 -INDICATES COLUMN MARK, FOR SIZE SEE SCHEDULE SHEET S1.00.
   F1 -INDICATES FOOTING MARK, FOR SIZE SEE SCHEDULE SHEET S1.00.
3. K INDICATES SHEAR WALL MARK, SEE SCHEDULE SHEET S1.00.
4. A INDICATES A HOLDOWN MARK, SEE SCHEDULE SHEET S1.00.
5. 'SWO' INDICATES SHEAR WALL OPENING PER 9 S0.40
6. S-------S INDICATES STEPPED FOOTING PER DETAIL 1 S0.10
7. NEW TYPICAL CONTINUOUS FOOTINGS ARE 15" WIDE X 20" MIN THK X 12" DEEP BELOW LOWEST ADJACENT GRADE WITH (3) #4 T&B (6) TOTAL, U.N.O. CFXX INDICATES CONTINUOUS FOOTING MARK, SEE SCHEDULE SHEET S1.00
8. GB2.0 INDICATES GRADE BEAM MARK, SEE SCHEDULE SHEET S1.00.
9. INTERIOR LOAD BEARING WALLS AND ALL EXTERIOR STUD WALLS WITH NO SHEAR WALL DESIGNATION, USE: ⅝" Ø ANCHOR BOLTS @ 48" OC U.N.O.
10. SEE ARCH. FOR ALL DIMENSIONS & CONDITIONS NOT SHOWN.
11. PRIOR TO THE CONTRACTOR REQUESTING A BUILDING DEPT. FOUNDATION INSPECTION, THE SOILS ENGINEER SHALL ADVISE THE BUILDING OFFICIAL IN WRITING THAT:
    A. THE BUILDING PAD WAS PREPARED IN ACCORDANCE WITH THE SOILS REPORT.
    B. THE UTILITY TRENCHES HAVE BEEN PROPERLY BACK FILLED & COMPACTED.
    C. THE FOUNDATION EXCAVATIONS COMPLY WITH THE INTENT OF THE SOILS REPORT.
12. ALL HOLDOWNS AND ANCHOR BOLTS AT SHEARWALL SHALL BE SET IN PLACE BY TEMPLATE PRIOR TO FOUNDATIONS INSPECTION AND RETIGHTENED PRIOR TO COVERING OF WALLS.
13. TYPICAL CONCRETE SLAB ON GRADE SHALL BE 4" THICK WITH #4 @ 18"OC EA. WAY MID HT. OVER
    1 4" OF SAND W/ 15MIL VISQUEEN VAPOR BARRIER MID. HT. PER 5 S0.20 THE MAXIMUM SPACING OF THE CONTROL JOINTS SHALL BE 30X THE SLAB THICKNESS BEFORE 7 DAYS OF CURING. SLAB CONTROL JOINTS SHALL BE PER DETAIL 8 S0.10 FULLY COORDINATED WITH THE ARCHITECT. SEE SOILS REPORT FOR SPECIAL SLAB REQUIREMENTS.
14. ALL FOOTINGS ARE CENTERED UNDER COLUMNS AND BEARING WALLS UNLESS NOTED OTHERWISE ON PLANS AND DETAILS.
15. SEE ARCHITECTURAL PLANS FOR LOCATIONS OF ALL WALL OPENINGS, SLOPED AND DEPRESSED SLABS, CONCRETE CURBS, ADDITIONAL EMBEDDED ITEMS NOT SHOWN ON THESE DRAWINGS. VERIFY ALL BUILDING DIMENSIONS, SLOPES AND DEPRESSED SLAB DIMENSIONS WITH ARCHITECTURAL PLANS BEFORE BEGINNING WORK. REPORT ANY DISCREPANCIES TO THE ARCHITECT FOR RESOLUTION.
16. SEE GENERAL NOTES AND SPECIFICATIONS FOR SPECIAL GRADING REQUIREMENTS UNDER FOOTINGS.
17. COLUMNS ARE ONE SIZE FROM FOUNDATION TO ROOF UNLESS OTHERWISE NOTED ON PLANS
18. ALL DIMENSIONS SHOWN ARE TO EDGE OF CONCRETE SLAB, CENTER OF INTERIOR WALL, OR CENTERLINE OF POST OR COLUMN. REFER TO ARCHITECTURAL PLANS FOR OTHER DIMENSIONS.
19. ALL COLUMNS ARE CENTERED ON THE GRID LINES UNLESS NOTED OTHERWISE.
20. ALL CONCRETE FLOORS TO BE FLAT U.N.O.
21. SURFACE WATER SHALL DRAIN AWAY FROM THE BUILDING. COORDINATE WITH CIVIL & LANDSCAPE.
22. CONTRACTOR SHALL COORDINATE ALL UNDERGROUND UTILITY WORK TO AVOID CONFLICTS WITH FOOTINGS.
23. CONTRACTOR SHALL SUBMIT PROPOSED FIRE RISER, CONDUIT, PIPING SLEEVE LOCATION FOR ENGINEER'S REVIEW PRIOR TO FOOTING CONSTRUCTION.
24. SEE ARCH'L AND CIVIL DRAWINGS FOR LOCATION OF MOISTURE BARRIER, CURBS, TRASH ENCLOSURES, EXTERIOR SLABS, DRAINAGE, RAMPS, WALKS, ETC.
25. FOR FOOTINGS OUTSIDE THE BUILDING ENVELOPE, COORDINATE TOP OF FOOTING ELEVATION WITH THE STRUCTURAL DETAILS AND WITH CIVIL AND LANDSCAPE.
26. BUILDING SLAB IS NOT DESIGNED TO SUPPORT CRANE LOADS, CONCRETE MIXING TRUCKS, OR OTHER SPECIFIC CONSTRUCTION LOADING. IF SUCH LOADS OCCUR, THE CONTRACTOR SHALL BE RESPONSIBLE FOR UPGRADING THE SLAB THICKNESS AND PROVIDING REINFORCING AS REQUIRED. ANY DAMAGE CAUSED TO THE SLAB SHALL BE REPAIRED AT THE CONTRACTOR'S EXPENSE PER THE OWNER'S REQUEST.
27. EXCAVATIONS SHALL BE MADE IN COMPLIANCE WITH CAL/OSHA REGULATIONS.
28. BACKFILLING OF WALLS WITH RETAINED SOIL SHALL BE OF NON-EXPANSIVE GRANULAR FILL IN ACCORDANCE WITH THE SOILS REPORT. WALLS RETAINING SOIL SHALL NOT BE BACKFILLED PRIOR TO PLACING SLAB ON GRADE AND SECOND FLOOR SLAB.
29. ALL CONTINUOUS FOUNDATIONS SHALL BE EMBEDED A MINIMUM OF 12" BELOW LOWEST ADJACENT GRADE UNO, ALL PAD FOOTINGS SHALL BE EMBEDED A MINIMUM OF 18" BELOW LOWEST ADJACENT GRADE UNO.



EQUIPMENT ELEVATION

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. S1.00, (A#03-122536, DATED 03/24/2023)

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PROFESSIONAL ENGINEER No. 5152 STATE OF CALIFORNIA STRUCTURAL

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Drawn by

Checked by

Revisions

No.	Date	Description
1	06-30-2023	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

FOUNDATION PLAN

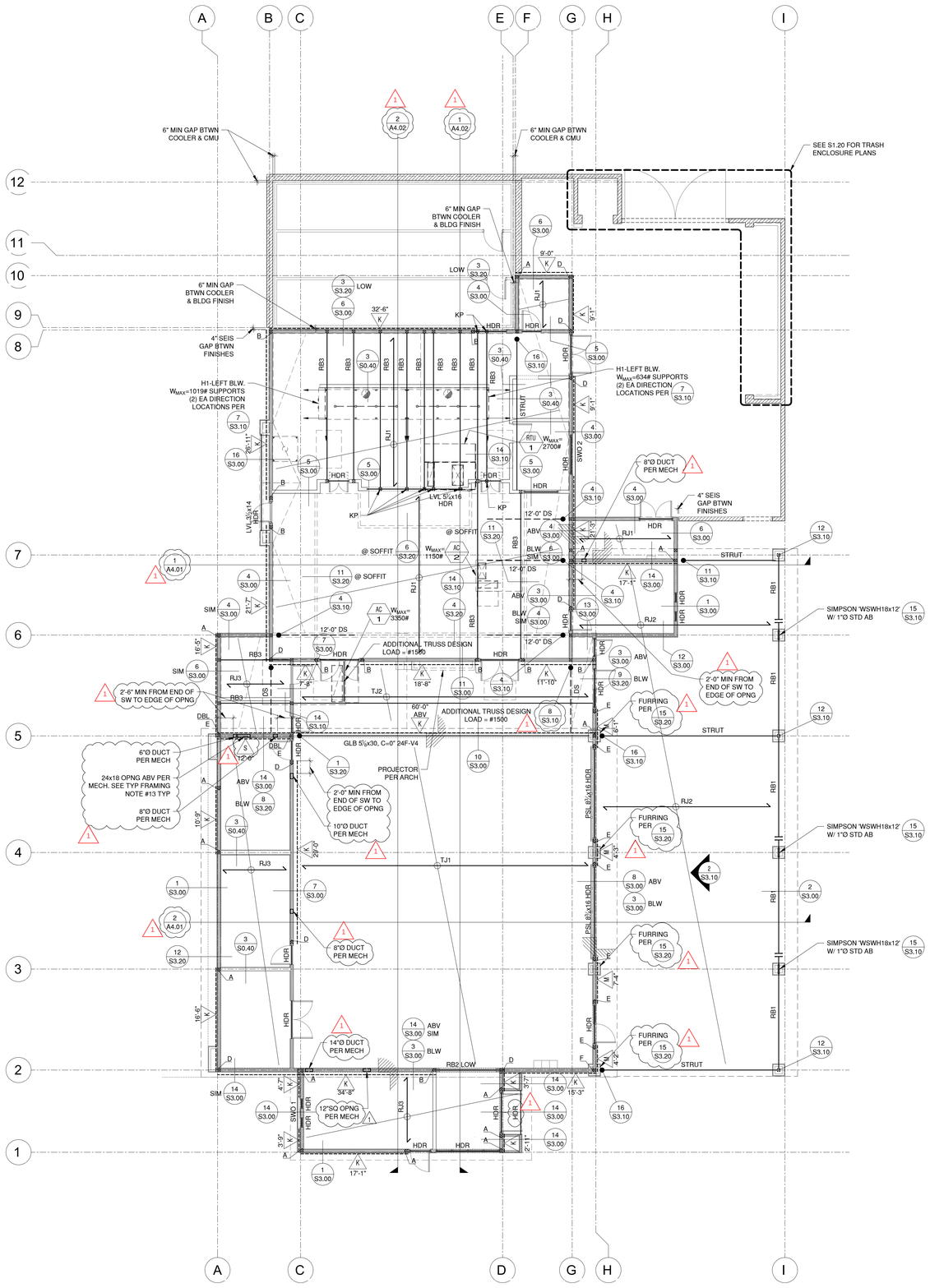
Job No. 22005

Date 06-07-22

S1.00

BID CLARIFICATION NO.1 (06-20-2023)

6/15/2023 10:55:51 AM



FRAMING NOTES:

1. PROVIDE 2X TRIMMER @ 7¼ OR LESS HEADER (2) 2X TRIMMER @ 9½ OR GREATER & 4X POST TRIMMER @ LVL/GLB HEADER PER DETAIL 2/S0.30
2. ALL EXTERIOR WALLS TO BE PER THE STUD SCHEDULE ON S1.00.
3. ALL INTERIOR WALLS TO BE 2x6 @ 16"OC STUDS U.N.O.
4. ALL EXTERIOR WALLS SHALL BE 100% SHEATHED. EXTERIOR WALLS WITHOUT SHEAR WALL SYMBOL SHALL HAVE SHEATHING TYPE 'K' PER SHEAR WALL SCHEDULE U.N.O. ON ARCHITECTURAL DRAWINGS.
5. FOR TYPICAL STUD WALL FRAMING SEE 2/S0.30 HEADER SIZES SHALL BE PER 2/S0.30 U.N.O.
6. PROVIDE MULTIPLE STUDS UNDER MULTIPLE JOISTS.
7. "C" = INDICATES CAMBER BETWEEN SUPPORTS ONLY @ GLUED, LAMINATED BEAMS. (i.e. NO CAMBER @ CANTILEVERS)
8. PROVIDE TWO ROWS BOUNDARY NAILING ALONG ALL STRUT MEMBERS, INTERIOR SHEAR WALLS, AND INTERIOR LATERAL FRAMES U.N.O.
9. CONTRACTOR SHALL COORDINATE ALL MECH'L. UNIT LOCATIONS, SIZES, OPENINGS, ETC. WITH MECH'L. DRAWINGS.
10. SEE ARCH. FOR ALL DIMENSIONS & CONDITIONS NOT SHOWN.
11. K INDICATES SHEAR WALL MARK FOR WALLS BELOW U.N.O., SEE SCHEDULE SHEET S1.00.
12. A INDICATES A HOLDOWN MARK FOR SHEAR WALLS FOR THE LEVEL BELOW U.N.O., SEE SCHEDULE SHEET S1.00.
13. 'SWO' INDICATES SHEAR WALL OPENING PER DETAIL 9/S0.40, DUCT OPENINGS GREATER THAN 4½" SHALL BE PER SHEAR WALL WITH OPENING DETAIL. 1
14. TYPICAL ROOF SHEATHING SHALL BE ½" APA RATED SHEATHING, STUD-T-FLOOR, OR SINGLE FLOOR (EXPOSURE-1) (32/16) W/ 10d COMMON NAILS @ 6"O.C. BOUNDARIES, 6"O.C. OTHER PANEL EDGES, AND 12"O.C. FIELD NAILING. (UN-BLOCKED, T&G @ UNSUPPORTED PANEL EDGES OR WITH PANEL SHT'G CLIPS (ALLOW SHEAR 215 PLF))
15. UNLESS SPECIFICALLY NOTED ON THE PLANS, FRAMING SHALL NOT BE CUT OR RELOCATED WITHOUT PRIOR APPROVAL OF THE STRUCTURAL ENGINEER. CONTRACTOR SHALL OBTAIN APPROVAL OF JOIST MFR. SHOP DRAWINGS FROM BOTH MECHANICAL AND STRUCTURAL ENGINEERS PRIOR TO ERECTION OF JOIST FRAMING.
16. ALL MULTIPLE JOISTS SHALL HAVE EACH JOIST NAILED TO THE ADJACENT ONE WITH 16d @ 16"O.C. STAGGERED.
17. ALL MULTIPLE STUDS SHALL HAVE EACH STUD NAILED TO THE ADJACENT ONE WITH SOLE PLATE NAILING PER SHEAR WALL SCHEDULE, 10d @ 9"O.C. STAGGERED MINIMUM.
18. FOR HEADER SIZES NOT NOTED, REFER TO TYPICAL DETAIL SCHEDULE.
19. LOW SPOTS CREATED BY THE ROOF PITCH SHALL BE PROVIDED WITH CRICKETS AS REQUIRED TO SLOPE.
20. BALLOON FRAME ALL GABLE END WALLS U.N.O.
21. ALL ARCHITECTURALLY EXPOSED STRUCTURAL STEEL (AESS) SHALL MEET THE MINIMUM SPECIFICATIONS IN THE AISC CODE OF STANDARD PRACTICE CHAPTER 10 UNLESS NOTED OTHERWISE ON THE ARCHITECTURAL DRAWINGS.
22. ⊠ 6X FOR 6X WALLS, 4X FOR 4X WALLS. WIDTH TO MATCH BEAM U.N.O.
23. WOOD TO WOOD, SAWN LUMBER USE 'HU' HANGER U.N.O.
    WOOD TO WOOD, PSL/LVL/LSL/GLB USE 'HGU' HANGER U.N.O.
    WOOD TO STEEL USE 'WPU' HANGER U.N.O.
24. POST W/ 'ECCG' U.N.O.
25. POST W/ 'CCQ' U.N.O.
26. 3" MIN WIDE PSL, GLB, OR DBL 2X OR 4X DF STRUT UNO W/ CONNECTOR PER PLAN.
27. DRAG STRUT W/ CONNECTOR PER PLAN U.N.O.
28. POST IN WALL TO MATCH BM. WIDTH U.N.O. PER DETAIL 9/S0.50
29. INDICATES WALLS REQUIRING TOP PLATE SPLICES PER DETAIL 7/S0.30
30. INDICATES BRACE LOCATIONS PER DETAIL 7/S3.10
31. INDICATES SPLICE LOCATION PER DETAIL 15/S3.10
32. "STRUT (XX.XK)" INDICATES TRUSS STRUT DESIGN FORCE IN KIPS (ASD). THE TRUSS DESIGNER SHALL DESIGN THE OPEN WEB TRUSS FOR THE AXIAL STRUT FORCE.

FRAMING SCHEDULE:

RJ1 = 14" TJI210 @ 16"OC BY WEYERHAEUSER

RJ2 = LVL 1¾ x 14 @ 16"OC

RJ3 = 2 x 10 DF #2 @ 16"OC

RB1 = GLB 5½ x 15 C=0" 24F-V4

RB2 = PSL 6¼ x 9¼ 1

RB3 = LVL 3½ x 14

TJ1 = 32" DEEP OPEN WEB TRUSSES @ 32"OC. TRUSSES ARE A DEFERRED SUBMITTAL

TJ2 = 32" DEEP OPEN WEB TRUSSES @ 32"OC. TRUSSES ARE A DEFERRED SUBMITTAL

FOR LEGENDS, TABLES, & SCHEDULES NOT SHOWN, SEE S1.00

ROOF FRAMING PLAN

SCALE: 1/8"=1'-0" (0, 4', 8', 16')

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. S1.10, (A#03-122536, DATED 03/24/2023)

BID CLARIFICATION NO.1 (06-20-2023)

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Drawn by

Checked by

Revisions

No.	Date	Description
1	06-30-2023	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

ROOF FRAMING PLAN

Job No. 22005

Date 06-07-22

S1.10

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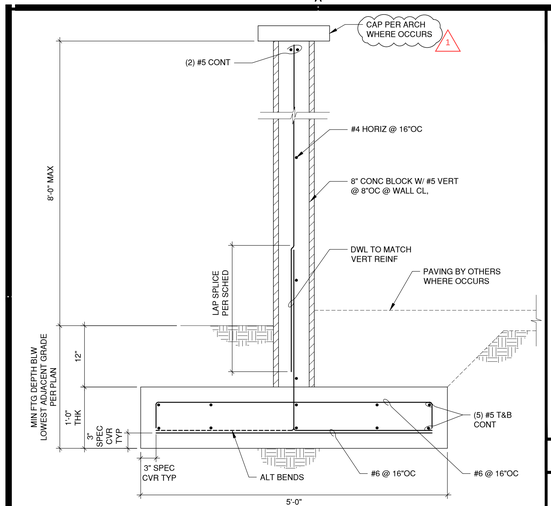
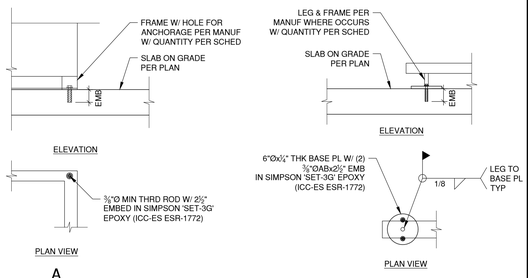
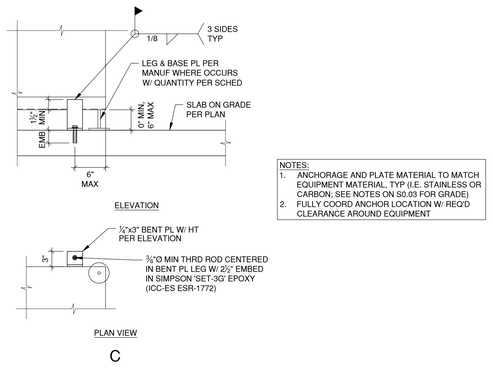


SECTION AT CMU WALL AT FREE STANDING WALL BALANCE

SCALE:1"=1'-0" 13

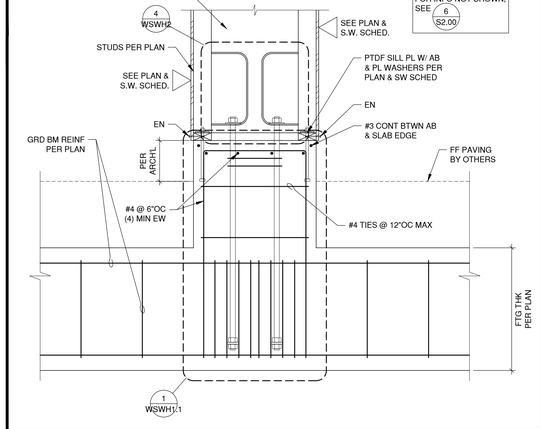




NOTES:
1. ANCHORAGE AND PLATE MATERIAL TO MATCH EQUIPMENT MATERIAL, TYP (I.E. STAINLESS OR CARBON; SEE NOTES ON S0.03 FOR GRADE)
2. FULLY COORD ANCHOR LOCATION W/ REQ'D CLEARANCE AROUND EQUIPMENT

MECH ANCHORAGE

SCALE:1"=1'-0" 15



SECTION AT GRD BM

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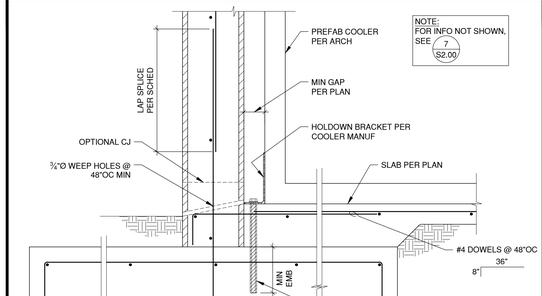
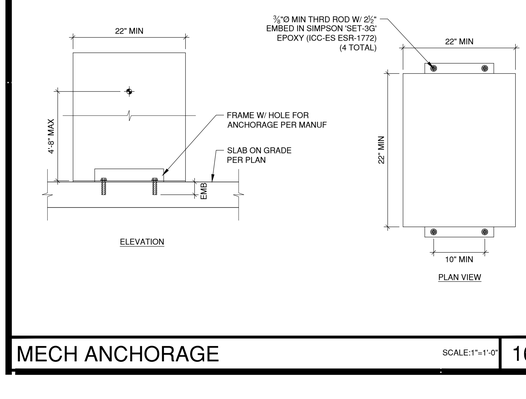


SECTION AT GRD BM

SCALE:1"=1'-0" 10

WSWH AT GRD BM

SCALE:1"=1'-0" 11

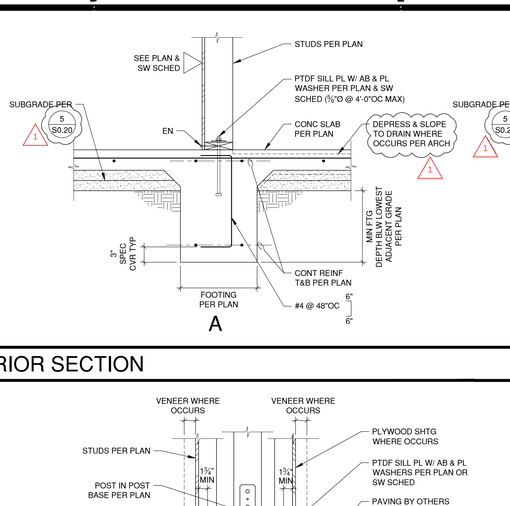


MECH ANCHORAGE

SCALE:1"=1'-0" 16

COOLER ANCHORAGE

SCALE:1"=1'-0" 12



INTERIOR SECTION

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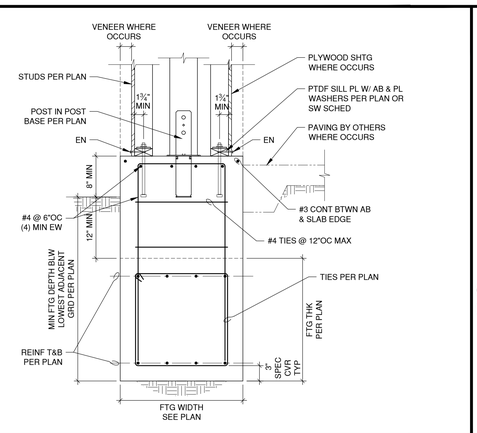

SECTION AT FTG

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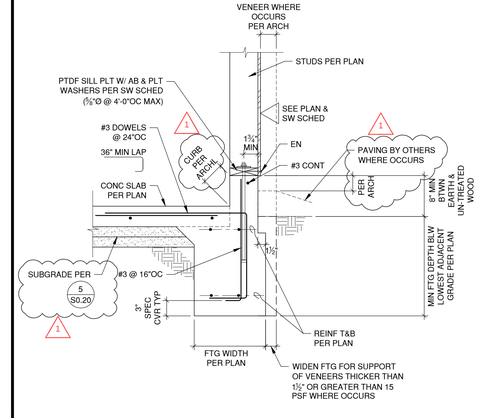

EXTERIOR FOOTING SECTION

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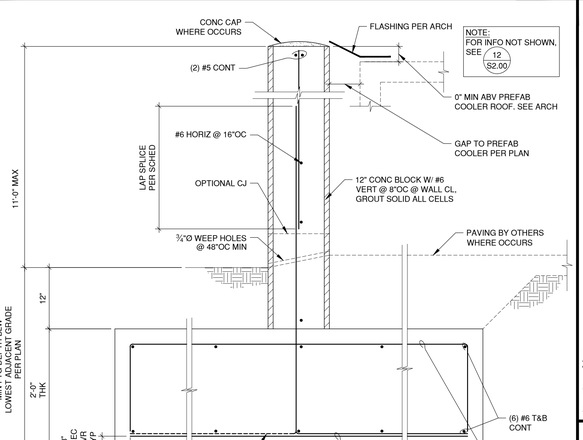

SECTION AT CMU WALL AT COOLER

SCALE:1"=1'-0" 7

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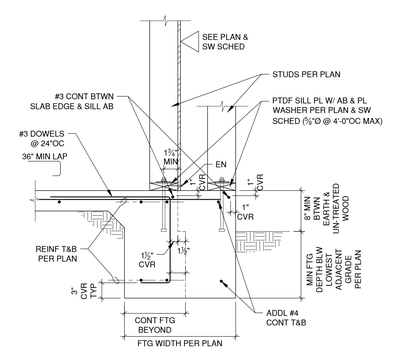
SECTION AT OPNG

SCALE:1"=1'-0" 3

EXTERIOR SECTION

SCALE:1"=1'-0" 8



SECTION AT DBL WALL

SCALE:1"=1'-0" 4

AGENCY

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LICENSED ARCHITECT C-26053 Ren. Date 08/31/2023 STATE OF CALIFORNIA

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Drawn by

Checked by

Revisions

No.	Date	Description
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SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL
CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

FOUNDATION DETAILS

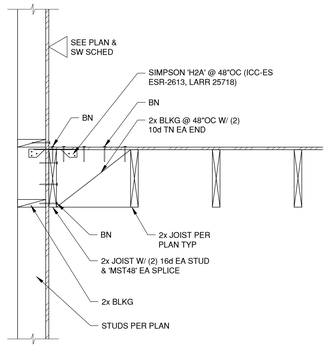
Job No. 22005

Date 06-07-22

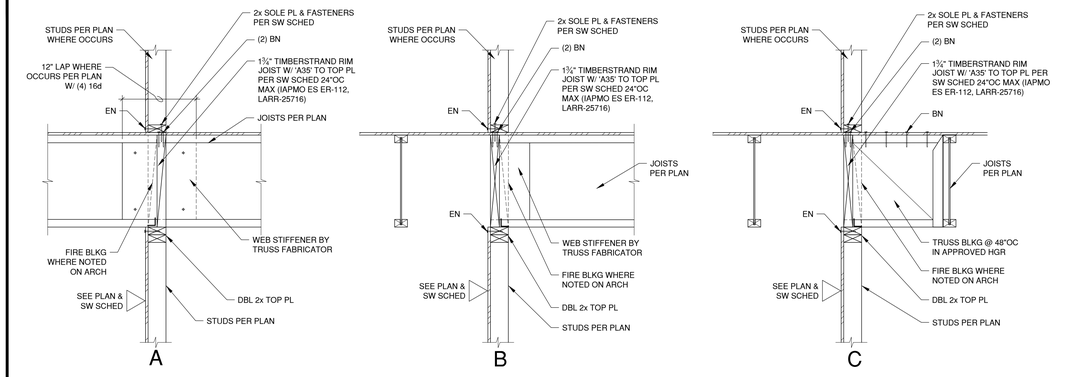
S2.00

BID CLARIFICATION NO.1 (06-20-2023)

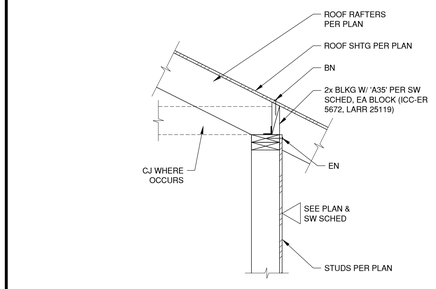
6/14/2023 4:54:27 PM



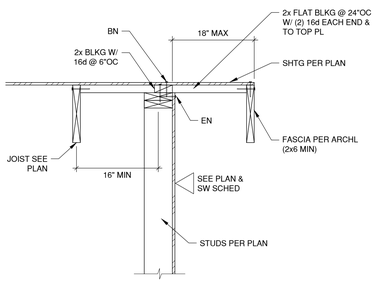
LOW ROOF SECTION — SCALE:1"=1'-0" — 13



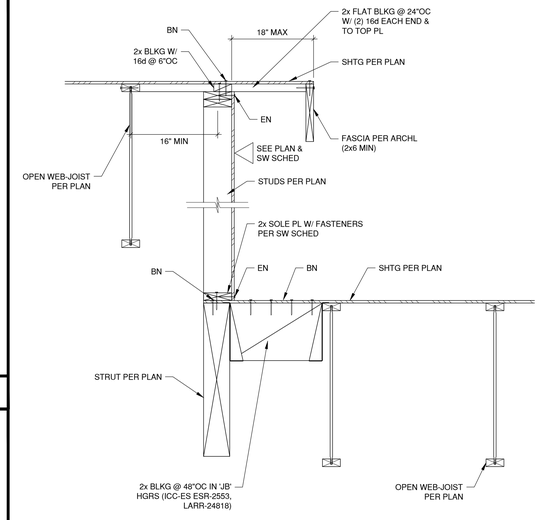
SECTION AT WALL — SCALE:1"=1'-0" — 5



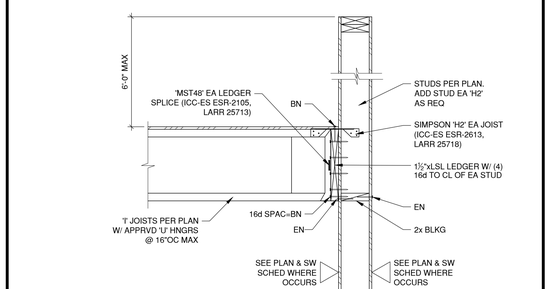
EAVE SECTION — SCALE:1"=1'-0" — 1



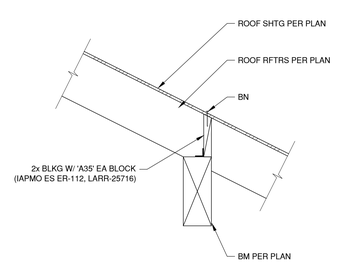
RAKE DETAIL — SCALE:1"=1'-0" — 14



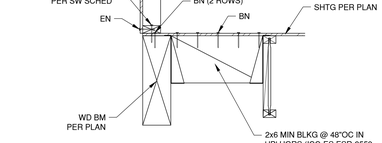
RAKE DETAIL — SCALE:1"=1'-0" — 10



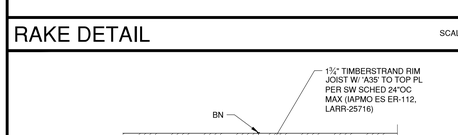
LOW ROOF SECTION — SCALE:1"=1'-0" — 6



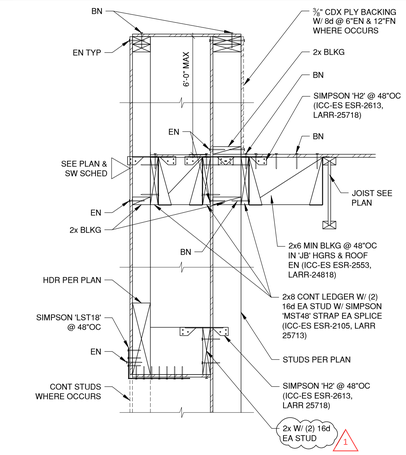
EAVE SECTION — SCALE:1"=1'-0" — 2



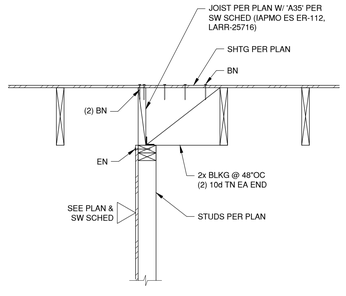
SECTION AT BEAM — SCALE:1"=1'-0" — 15



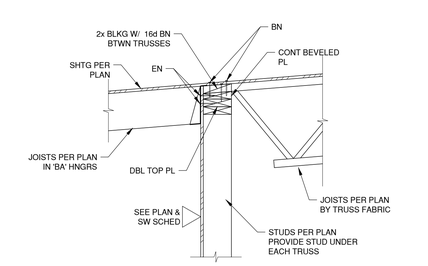
SECTION AT WALL — SCALE:1"=1'-0" — 11



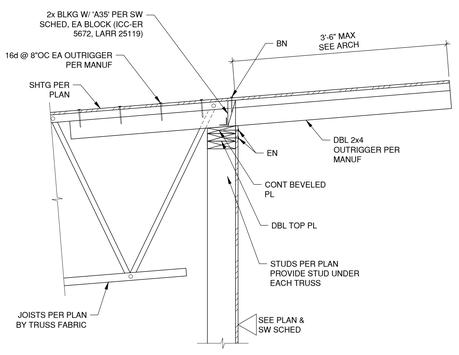
SECTION AT PARAPET — SCALE:1"=1'-0" — 16



SECTION AT WALL — SCALE:1"=1'-0" — 12



SECTION AT WALL — SCALE:1"=1'-0" — 7



SECTION AT WALL — SCALE:1"=1'-0" — 8

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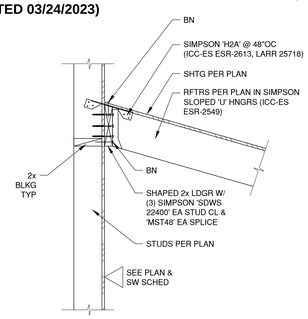
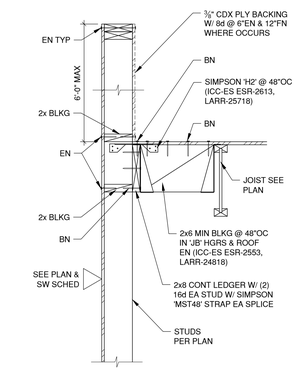


SECTION AT LOW ROOF — SCALE:1"=1'-0" — 3



SECTION AT PARAPET — SCALE:1"=1'-0" — 4

AGENCY

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LICENSED ARCHITECT — C-26053 — Ren. Date 08/31/2023 — STATE OF CALIFORNIA

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Revisions

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ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

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ROSAMOND, CA 93560

FRAMING DETAILS

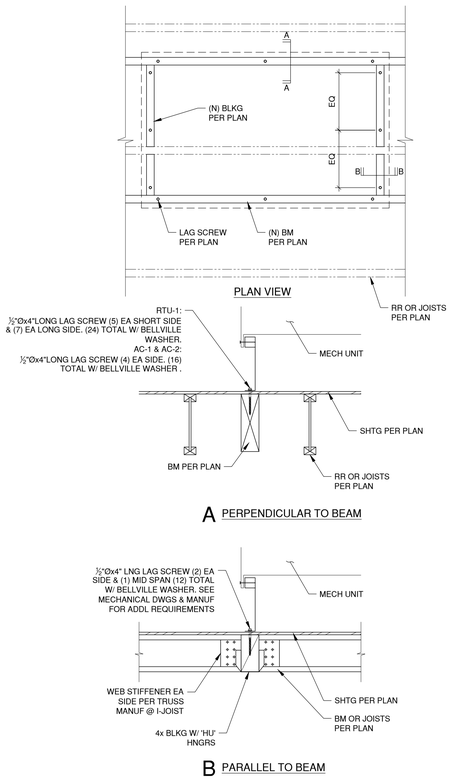
Job No. 22005

Date 06-07-22

S3.00

BID CLARIFICATION NO.1 (06-20-2023)

6/14/2023 4:54:28 PM

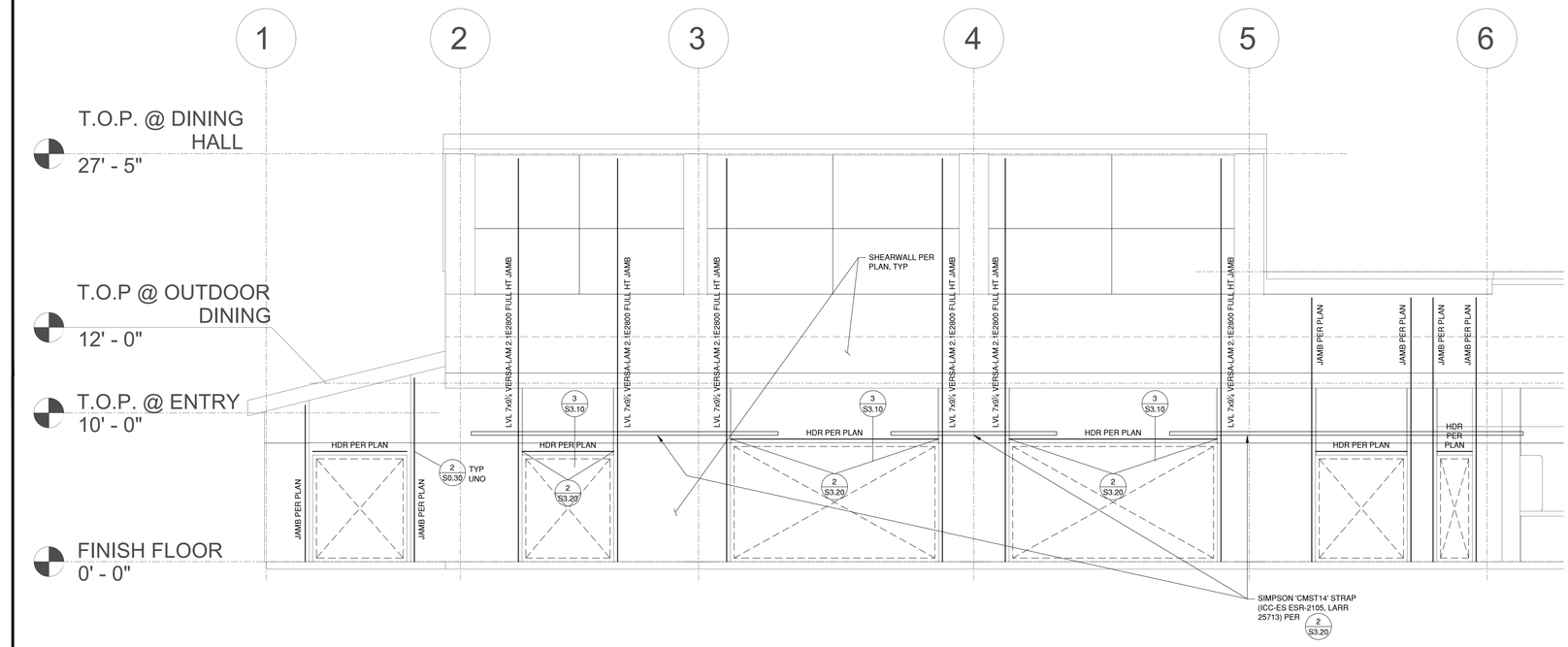


A PERPENDICULAR TO BEAM

B PARALLEL TO BEAM

MECH ANCHORAGE DETAILS

SCALE:1"=1'-0" | 14



EAST ELEVATION

SCALE:1/4"=1'-0" | 2

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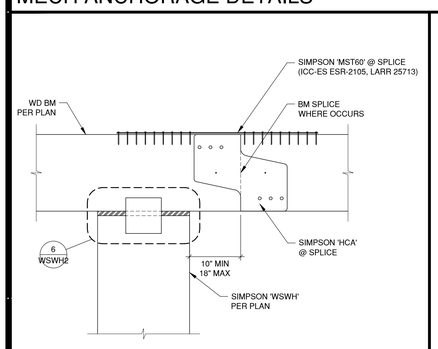
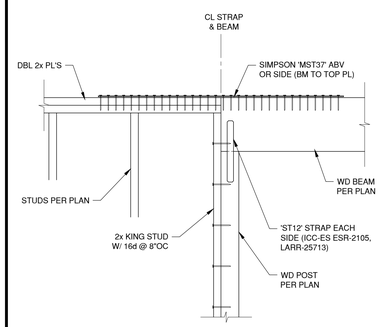


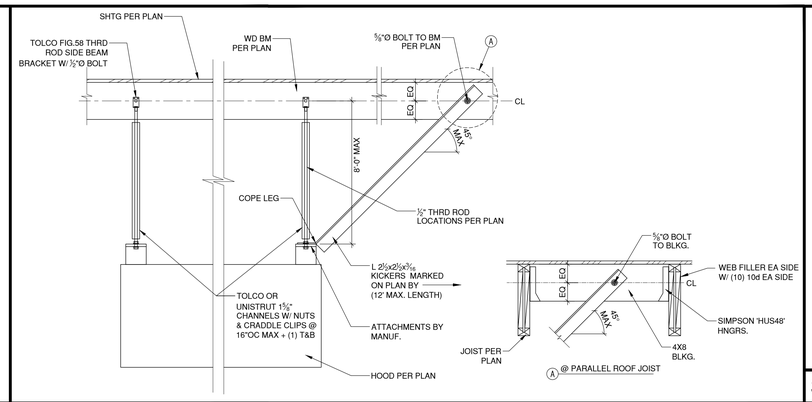
SECTION AT 'WSWH'

SCALE:1"=1'-0" | 15



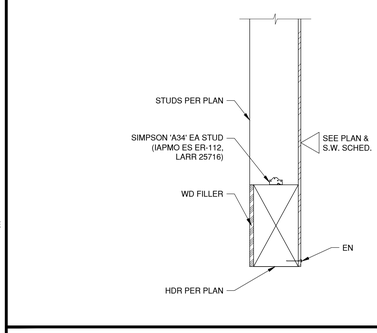
STRUT DETAIL

SCALE:1"=1'-0" | 11



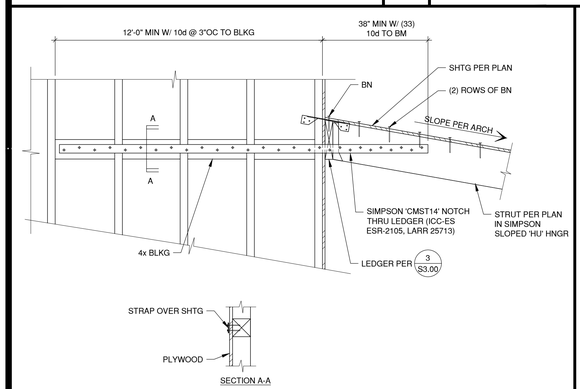
HOOD DETAIL

SCALE:1"=1'-0" | 7



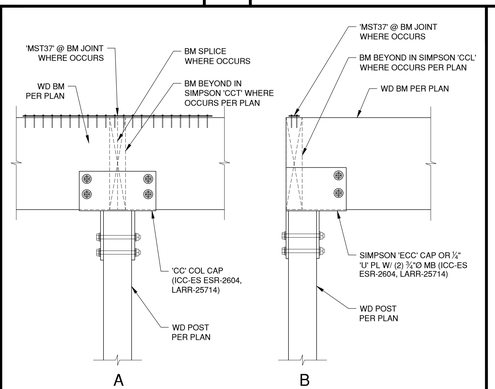
SECTION AT HDR

SCALE:1"=1'-0" | 3



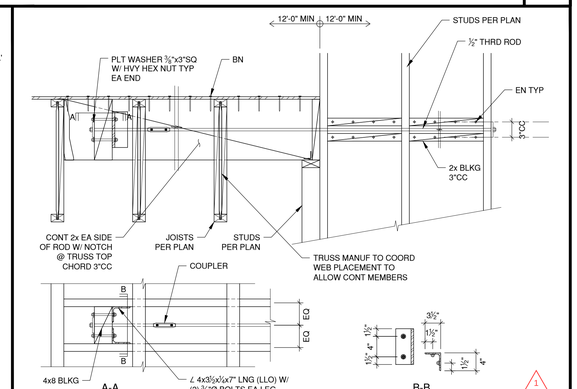
STRUT DETAIL

SCALE:1"=1'-0" | 16



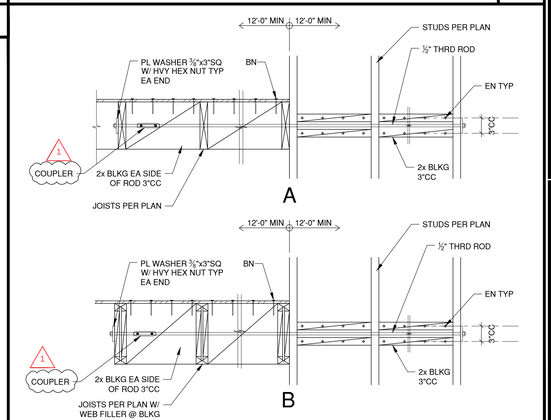
SECTION AT POST

SCALE:1"=1'-0" | 12



DRAG STRUT TO DRAG STRUT

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



DRAG STRUT TO DRAG STRUT

SCALE:1"=1'-0" | 4

BID CLARIFICATION NO.1 (06-20-2023)

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architecture planning interiors

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SOUTHERN KERN UNIFIED
SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

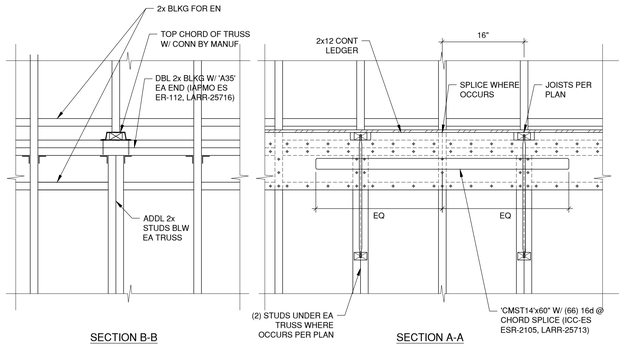
FRAMING
DETAILS

Job No.
22005

Date
06-07-22

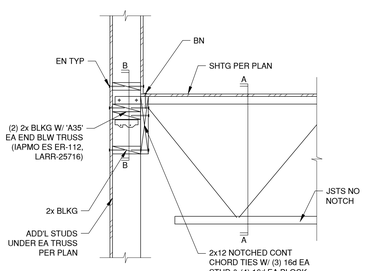
S3.10

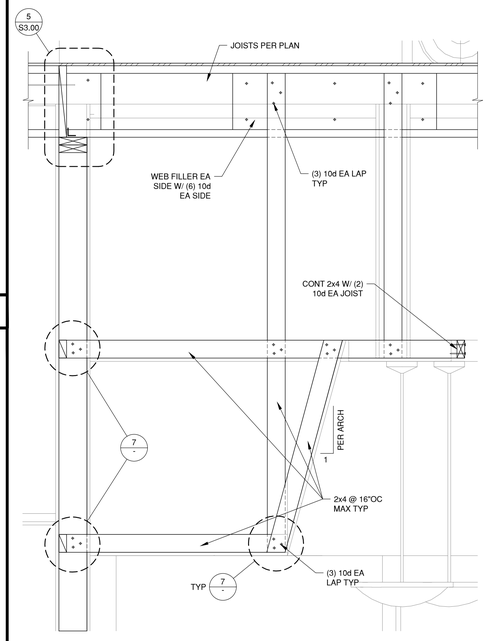
6/15/2023 10:55:53 AM



SECTION AT LEDGER

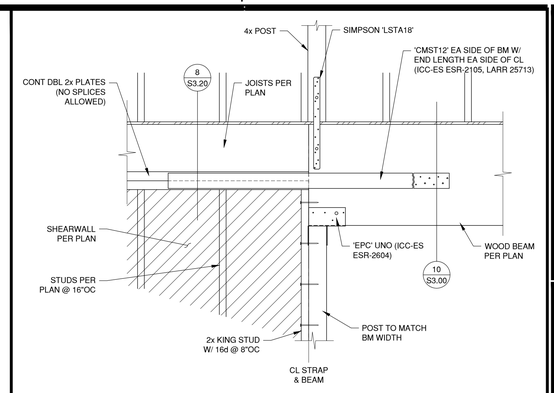
SCALE:1"=1'-0" | 9





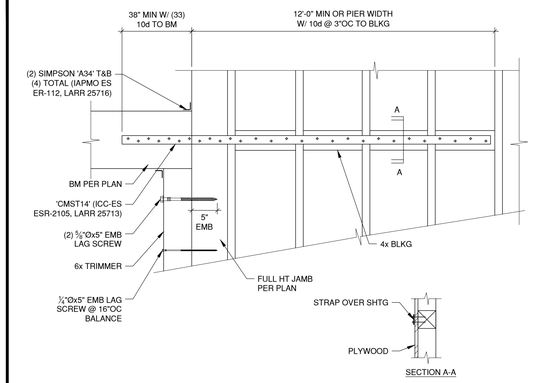
SECTION AT SOFFIT

SCALE:1"=1'-0" | 6



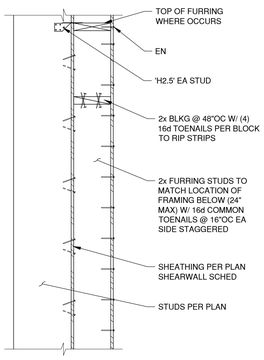
STRUT BM TO PLATE

SCALE:1"=1'-0" | 1



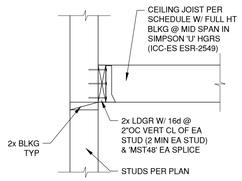
STRUT DETAIL

SCALE:1"=1'-0" | 2



FURRING STUDS

SCALE:1"=1'-0" | 15



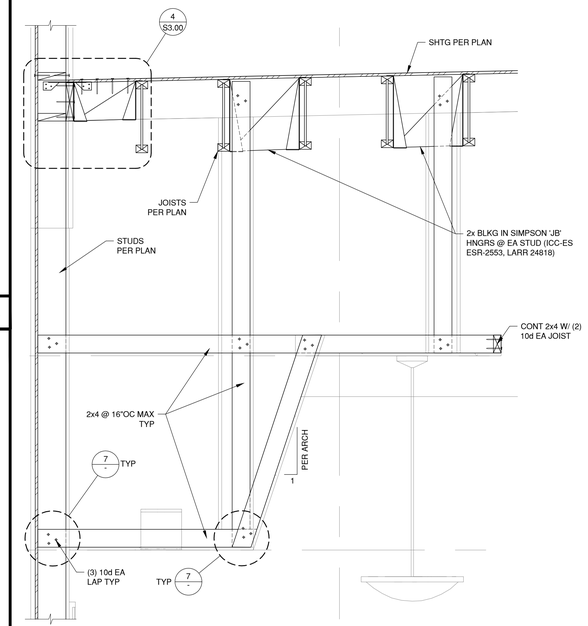
CEILING JOIST SCHEDULE (WITHOUT STORAGE)

JOIST SIZE	JOIST SPACING	SPAN IN FEET (MAX)
2 x 4	16"OC	11'-3"
	24"OC	9'-10"
2 x 6	16"OC	17'-8"
	24"OC	14'-10"
2 x 8	16"OC	23'-0"
	24"OC	18'-9"

NOTES:
1. CEILING SPANS ARE IN ACCORDANCE WITH CBC TABLE 2308.7.
2. 15 PSF MAX TOTAL LOAD (NO STORAGE ALLOWANCE).
3. DEFLECTION LIMIT PER SPAN = L/240.
4. VALUES ARE FOR SIMPLE SPANS.

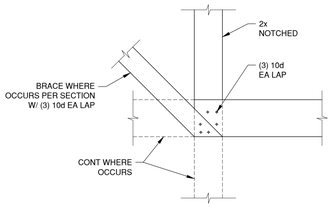
WOOD CEILING JOIST DETAILS & SCHED

SCALE:1"=1'-0" | 14



SECTION AT SOFFIT

SCALE:1"=1'-0" | 11



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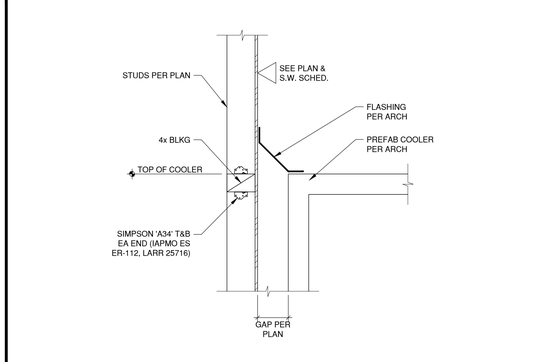
TYPICAL SOFFIT DETAIL

SCALE:1"=1'-0" | 7

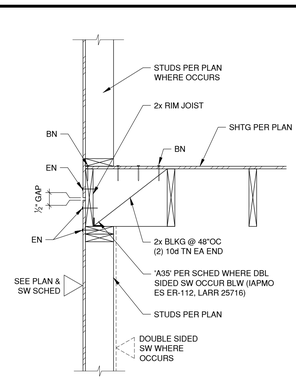


SECTION AT WALL

SCALE:1"=1'-0" | 3

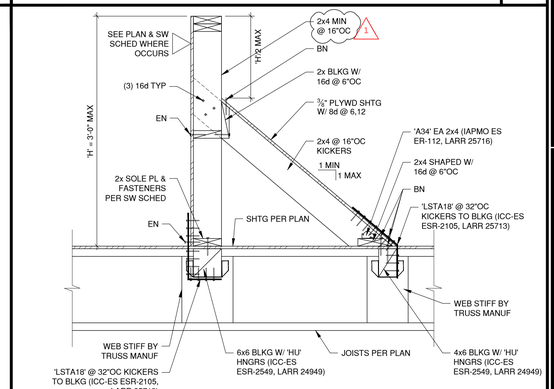
TYPICAL SOFFIT DETAIL

SCALE:1"=1'-0" | 12



SECTION AT WALL

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



SECTION AT KNEE WALL

SCALE:1"=1'-0" | 4

-

SCALE:1"=1'-0" | 16

AGENCY

FLEWELLING & MOODY
architecture planning interiors

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An Employee Owned Corporation

ENGINEER

REGISTERED PROFESSIONAL ENGINEER VICTORIOUS MOQET T. LIMJUCO No. 24152 STRUCTURAL STATE OF CALIFORNIA

LICENSED ARCHITECT VICTORIOUS MOQET T. LIMJUCO C-26053 Ren. Date 08/31/2023 STATE OF CALIFORNIA

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Drawn by

Checked by

Revisions

No.	Date	Description
1	06-30-2023	BID CLARIFICATION NO.1

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT
ROSAMOND HIGH SCHOOL

CAFETERIA BUILDING

2925 ROSAMOND BLVD.
ROSAMOND, CA 93560

FRAMING DETAILS

Job No. 22005

Date 06-07-22

S3.20

BID CLARIFICATION NO.1 (06-20-2023)

6/14/2023 4:54:29 PM

ELECTRICAL SYMBOLS

A 5a A 5a	LIGHTING FIXTURES, "5" DENOTES CIRCUIT NUMBER, LOWER CASE "a" DENOTES CONTROLLING SWITCH, UPPER CASE "A" DENOTES FIXTURE TYPE.
	LED STRIP LIGHTS.
B	RECESSED CEILING MOUNTED FIXTURE: UPPER LETTER INDICATES TYPE (U.O.N.).
H	WALL MOUNTED LIGHT FIXTURE: UPPER LETTER INDICATES TYPE.
	EXIT SIGN WITH JUNCTION BOX APPROVED FOR THROUGH WIRING. UNIVERSAL MOUNT. DIRECTIONAL ARROW AS SHOWN ON PLAN.
	LED LIGHTING FIXTURE WITH EMERGENCY BATTERY PACK.
	LED LIGHTING FIXTURES ON EMERGENCY UNIT INVERTER SYSTEM.
A	PANEL DESIGNATION, LETTER IDENTIFIES THE PANEL.
	DUPLEX RECEPTACLE: 125V., 15 AMP., NEMA 5-15R. +18" A.F.F. TO BOTTOM OF BOX UNLESS OTHERWISE NOTED.
	CEILING MOUNTED DUPLEX RECEPTACLE: 20A, NEMA 5-20R.
WP	WEATHERPROOF "GFI" RECEPTACLE: 125V., 15 AMP., NEMA 5-15R. +18" A.F.F. TO BOTTOM OF BOX.
WP L	WEATHERPROOF WHILE IN USE LOCKABLE "GFI" RECEPTACLE: 125V., 15 AMP., NEMA 5-15R. +18" A.F.F. TO BOTTOM OF BOX.
	GROUND FAULT INTERRUPTER TYPE DUPLEX RECEPTACLE: 125V., 15 AMP., NEMA 5-15R. +18" UNLESS OTHERWISE NOTED. +42" NEAR SINK.
	DUPLEX RECEPTACLE: 220V, 20 AMP., NEMA 6-20R, +18" A.F.F. UNLESS OTHERWISE NOTED.
	TWIST LOCK PLUG NEMA L5-30P.
J	JUNCTION BOX: MOUNT +18" UNLESS NOTED OTHERWISE.
J	JUNCTION BOX: MOUNTED IN ATTIC SPACE OR UNDER CANOPY.
	IDF RACK "TRIPP-LITE #SR4POST1224 WITH VERTICAL CABLE MANAGER "TRIPP-LITE #SRCABLEDUCTVRT". UPS "CYBER POWER # PR3000RTXL2UHVAN" AND EXTENDED BATTERY "#BP48VP2U03
C	CAMERA - FLUSH SINGLE GANG DEEP TYPE BOX, + A.F.F. PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
CB	CLOCK-BELL - "VALCOM #VIP-429A-D-IC", FLUSH BACKBOX "#VB-R22". PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
H	EXTERIOR HORN "VALCOM #VIP480AL-IC", FLUSH BACKBOX "#V-9805". PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
WAP	WIRELESS ACCESS POINT, WALL OR CEILING MOUNT. "RACKUS #R650" AND "#T750SE" WITH MOUNTING ACCESSORY. PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
2D	DATA/TELEPHONE OUTLET, FLUSH 4S BOX WITH 3/4" CONDUIT TO CEILING SPACE. PROVIDE "TELEPHONE #CISCO #8811". PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
4	QUAD CAT-6a JACK WITH CAT-6a CABLE. +18" A.F.F., U.O.N.
P	PENDANT MOUNT PROJECTOR "EPSON #V11HA68820" WITH COMPLETE MOUNTING ACCESSORIES AND RECESSED CEILING MOUNT PROJECTOR SCREEN "DA-LITE #21818L". - OUTDOOR TV DISPLAY "SAMSUNG #OM32H" WITH MOUNTING ACCESSORY "#FPWARTS1". - INDOOR TV DISPLAY "SAMSUNG #OM43B" WITH MOUNTING ACCESSORY "#FPWARTS1". - DINING HALL TV DISPLAY "SAMSUNG #BH65T" WITH MOUNTING ACCESSORY "#FPWARTS1".
1D	HDMI - FLUSH SINGLE GANG DEEP TYPE BOX WITH 3/4" CONDUIT TO CEILING SPACE AND "1D" CABLE TO IDF U.O.N. +18" A.F.F. U.O.N. HDMI OUTLET WITH CAT-6a CABLE TO IDF RACK.
X	XLR - FLUSH SINGLE GANG DEEP TYPE BOX WITH 3/4" C.O. TO CEILING SPACE.
X	XLR FLUSH TWO GANG DEEP TYPE BOX WITH 3/4" C.O. TO CEILING SPACE.
X2	XLR FLUSH TWO GANG DEEP TYPE BOX WITH (2) 3/4" C.O. TO CEILING SPACE.
1D AV	AV FLUSH SINGLE GANG DEEP TYPE BOX WITH 3/4" CONDUIT TO CEILING SPACE. PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
AV	AV FLUSH CEILING MOUNT SINGLE GANG DEEP TYPE BOX. PROVIDE CAT-6a JACK WITH CAT-6a CABLE TO IDF RACK.
	SECURITY SYSTEM MOTION DETECTOR.
	POWER PACK NX "NXRCFX2-1RD".
	WALL MOUNTED LINE VOLTAGE OCCUPANCY SENSOR AT +48" A.F.F.
ab	WALL MOUNTED OCCUPANCY SENSOR WITH DUAL SWITCHING. +48" A.F.F.
H	NXSW2-ORLO NX LOW VOLTAGE DIMMER SWITCH STATION.
H	NXSW2 NX 8 BUTTON LOW VOLTAGE SWITCH STATION.
H	NXSW2-KEY NX LOW VOLTAGE KEY SWITCH STATION.
K	NXOS-LODT NX WALL MOUNT OCCUPANCY SENSOR.
K	NXOS-LODT NX WALL MOUNT OCCUPANCY SENSOR.
LV	3/4"C, (1) CAT-5e CABLE.
DC	2#16 TSP DIMMING CONTROL CABLE.
A-1,3,5	HOMERUN TO PANEL "A", CIRCUITS 1, 3, 5.
	CONDUIT: EXPOSED IN UNFINISHED AREAS; CONCEALED ABOVE CEILING OR IN WALL IN FINISHED AREAS.
	CONDUIT: IN OR BELOW FLOOR OR BELOW GRADE.

	3/4"C, 2#12 & 1#12 GND.		3/4"C, 3#12 & 1#12 GND.		3/4"C, 4#12, 1#12 GND.
	3/4"C, 5#12 & 1#12 GND.		1"C, 6#12 & 1#12 GND.		1"C, 8#12 & 1#12 GND.
#10	3/4"C, 2#10 & 1#10 GND.	#10	3/4"C, 3#10 & 1#10 GND.	#10	3/4"C, 4#10, 1#10 GND.
#10	3/4"C, 5#10 & 1#10 GND.	#10	1"C, 6#10 & 1#10 GND.	#10	1"C, 8#10 & 1#10 GND.
#6	3/4"C, 3#6 & 1#8 GND.	#4	1"C, 3#4 & 1#8 GND.		

	GROUND ROD IN GROUND YARD BOX.
	LIGHTING PANEL.
	POWER PANEL.
	MAIN SERVICE SWITCHBOARD.
OR	TRANSFORMER.
S_T	MANUAL MOTOR STARTER: TOGGLE TYPE WITHOUT THERMAL OVERLOAD PROTECTION.
S_P	TOGGLE TYPE SWITCH WITH PILOT LIGHT.
S_M	HORSEPOWER RATED, DISCONNECT SWITCH WITHOUT THERMAL OVERLOAD PROTECTION.
S_{2M}	HORSEPOWER RATED, 2P DISCONNECT SWITCH WITHOUT THERMAL OVERLOAD PROTECTION.
	FUSED DISCONNECT SWITCH H.P. RATED WITH CLASS "R" FUSECLIPS
	COMBINATION MAGNETIC STARTER AND FUSIBLE DISCONNECT SWITCH.
XFMR	TRANSFORMER.

A.I.C.	AMPERE INTERRUPTING CURRENT.	SWBD	SWITCHBOARD.	N.C.	NORMALLY CLOSED.
DPDT	DOUBLE POLE DOUBLE THROW.	C/B	CIRCUIT BREAKER.		
(E)	EXISTING.				
C.O.	CONDUIT ONLY WITH #12 PULL WIRE.				
(N)	NEW.	A.F.F	ABOVE FINISHED FLOOR.		
U.O.N.	UNLESS OTHERWISE NOTED.	W.P.	WEATHERPROOF.		
(TYP)	TYPICAL.				

CABLE TYPE

"S" CABLE - "WEST PENN" NO. 240, 2 PAIR #22 - SECURITY ZONE & POWER FEED.

"SW" CABLE - "WEST PENN" NO. AQC240, 2 PAIR #22 - SECURITY ZONE & POWER FEED (UNDERGROUND).

"SC" CABLE - "WEST PENN" NO. 240, 4#22 - SECURITY COMM. BUS., AQC 240 (WET LOC. TYPE).

"D" CABLE - 4 PAIR #24 UTP CATEGORY 6a CABLE AS MANUFACTURED BY COMSCOPE.

"DW" CABLE - 4 PAIR #24 UTP CATEGORY 6a CABLE AS MANUFACTURED BY COMSCOPE (WET LOCATION TYPE).

"FO" CABLE - FIBER OPTIC, 6 FIBERS SINGLE MODE.

SECURITY CABLE AND WIRING

—— 1S ——	3/4"CONDUIT WITH ONE "S" CABLE.
—— 2S ——	3/4"CONDUIT WITH TWO "S" CABLES.
—— 3S ——	3/4"CONDUIT WITH THREE "S" CABLES.
—— 4S ——	1"CONDUIT WITH FOUR "S" CABLES.

COMPUTER DATA/TELEPHONE/ PA/CLOCK/CCTV CABLE & WIRING

— 1D —	3/4" CONDUIT WITH ONE "D" CABLE.
— 2D —	3/4" CONDUIT WITH TWO "D" CABLES.
— 3D —	3/4" CONDUIT WITH THREE "D" CABLES.
— 4D —	3/4" CONDUIT WITH FOUR "D" CABLES.
— 5D —	1" CONDUIT WITH (5) "D" CABLES.
— 6D —	1" CONDUIT WITH (6) "D" CABLES.

GENERAL NOTES

1) ANY DISCREPANCIES BETWEEN THE DRAWINGS AND THE CODE(S), OR ANY RECOGNIZED STANDARDS SHALL BE BROUGHT TO THE ATTENTION OF SCHOOL DISTRICT ARCHITECT/ENGINEER OF RECORD PRIOR TO COMMENCING ANY WORK.

2) SPECIFIC COMPONENTS ON CATALOG CUT SHEETS MUST BE HIGHLIGHTED, OR IDENTIFIED.

3) PANELS MUST NOT BE MOUNTED HIGHER THAN 6 FEET AND SYSTEM STATUS DISPLAYS ARE TO BE AT EYE LEVEL (+60" AFF). NO EQUIPMENT OR RACEWAY MAY BE LOCATED UNDER A CABINET CONTAINING BATTERIES.

4) CONSULT WITH THE DISTRICT'S ELECTRICAL INSPECTOR BEFORE STARTING WORK.

5) ALL EXPOSED CONDUITS AND BOXES SHALL BE PAINTED TO MATCH THE SURFACES WHERE INSTALLED.

6) ALL EXISTING MATERIALS REMOVED FROM BUILDINGS SHALL BE REMOVED FROM SITE AS DIRECTED BY THE DISTRICT ELECTRICAL INSPECTOR.

7) WHERE EXISTING STRUCTURAL WALLS ARE CORED FOR NEW CONDUIT RUNS, SEPARATION BETWEEN CORED HOLES SHALL BE THREE INCHES FROM NEW OR EXISTING HOLES, UNLESS DIRECTED OTHERWISE BY THE ENGINEER.

8) THE REPRESENTATION OF PHYSICAL PLACEMENT OF EXISTING CONDUITS HAS BEEN DEVELOPED FROM THE BEST INFORMATION AVAILABLE TO THE DISTRICT AT THE TIME THE DRAWINGS WERE PREPARED. THE DISTRICT PROVIDES THIS ONLY AS A GENERAL GUIDELINE FOR THE CONVENIENCE OF BIDDERS/CONTRACTORS AND DOES NOT GUARANTEE OR WARRANT IN ANY WAY EXPRESSLY OR IMPLIEDLY, THE ACCURACY OF THESE REPRESENTATIONS. NOTHING IN THIS DISCLAIMER AFFECTS IN ANY WAY THE DUTY OF THE CONTRACTOR TO FURNISH ACCURATE "AS BUILT" DRAWINGS AFTER THE COMPLETION OF THE CONTRACT.

9) IN EXISTING BUILDINGS, CONTRACTORS SHALL NOT WORK IN AREAS CONTAMINATED BY MATERIALS MADE OF ASBESTOS UNTIL THE ASBESTOS MATERIALS HAVE BEEN REMOVED OR ENCAPSULATED.

10) REFER TO THE SPECIFICATIONS FOR ADDITIONAL REQUIREMENTS AND EQUIPMENT AND MATERIAL APPROVED FOR USE UNDER THIS CONTRACT.

11) EXISTING FIRE ALARM SYSTEM AND SECURITY SYSTEM MUST REMAIN IN OPERATION UNTIL NEW SYSTEM IS COMPLETE, APPROVED AND OPERATIONAL.

12) QUANTITY OF WIRES SHOWN IN ALL CONDUITS IS FOR GENERAL GUIDELINE. SUPPLIER OF AUXILIARY SYSTEM (FA, TEL, SECURITY) SHALL PREPARE CONSTRUCTION DRAWINGS SHOWING ALL NECESSARY WIRES AND CABLES AND VERIFY SIZES OF ALL CONDUITS SHOWN. PROVIDE ALL SPARE WIRES BETWEEN DRAWINGS DO NOT SHOW ALL THE NECESSARY J-BOXES AND PULL BOXES WHICH WILL BE REQUIRED THROUGHOUT.

13) IT IS THE CONTRACTOR'S RESPONSIBILITY TO PROVIDE ALL THESE BOXES AS NECESSARY TO TERMINATE CONDUITS AND RACEWAYS. PAINT BOXES TO MATCH COLOR OF THE FINISHED SURFACE THAT THE BOXES ARE ATTACHED BUILDINGS.

14) ALL JUNCTION BOXES AND DEVICES INDICATED ON BUILDING EXTERIORS SHALL BE WEATHERPROOF TYPE.

15) ALL SIGNAL WIRING IN UNDERGROUND CONDUITS SHALL BE WET LOCATION TYPE.

16) COORDINATION:
A. THE GENERAL CONTRACTOR SHALL COORDINATE LAYOUT DIMENSIONS INDICATED ON ELECTRICAL. ALL DISCREPANCIES SHALL BE REPORTED TO THE ENGINEER BEFORE BID TIME, OR BEFORE PROCEEDING WITH THE WORK.

B. IN THE EVENT THAT CERTAIN FEATURES OF THE CONSTRUCTION ARE NOT FULLY SHOWN ON THE CONSTRUCTION DOCUMENTS, THEN THEIR CONSTRUCTION SHALL BE OF THE SAME CHARACTER AS SIMILAR CONDITIONS THAT ARE SHOWN.

C. THE CONTRACTOR SHALL VERIFY ALL ELECTRICAL, COMMUNICATION AND SECURITY REQUIREMENTS BEFORE CONSTRUCTION BEGINS.

17) ALL DIMENSIONS, ELEVATIONS AND EXISTING CONDITIONS SHALL BE CHECKED AND VERIFIED ON THE SITE BY GENERAL CONTRACTOR, AND EACH SUB-CONTRACTOR BEFORE THE WORK BEGINS. ERRORS, OMISSIONS AND DISCREPANCIES SHALL BE BROUGHT TO THE ENGINEER'S ATTENTION BEFORE CONSTRUCTION BEGINS.

18) THE ENGINEER HAS PREPARED THESE DOCUMENTS ONLY FOR IMPROVEMENTS SPECIFIED, DETAILED OR SHOWN AS NEW WORK, AND ASSUMES NO RESPONSIBILITY FOR OTHER CONSTRUCTION, MATERIAL OR EQUIPMENT NOTED AS "EXISTING" OR AS "PROVIDED BY OTHERS".

19) IF USE OF WIREMOLD IS DESIRED, CONTRACTOR SHALL OBTAIN A LETTER FROM AOR AUTHORIZING USE OF WIREMOLD PRIOR TO INSTALLATION.

20) DRAWINGS ARE BASED ON AVAILABLE AS-BUILT PLANS AND FIELD OBSERVATIONS. NOTIFY THE AUTHORIZED OWNER REPRESENTATIVE (AOR) IMMEDIATELY WHEREVER (E) CONDITIONS ENCOUNTERED DEVIATE FROM THESE DRAWINGS AND EXISTING EQUIPMENT MUST BE RELOCATED DUE TO NEW CONSTRUCTION EFFORTS.

21) PROVIDE GROUND BUSHING ON NEW AND EXISTING PULLBOXES.

22) IT SHALL BE THE CONTRACTORS RESPONSIBILITY TO VERIFY THE TYPE OF CEILING CONSTRUCTION AND TO PROVIDE THE PROPER TYPE OF BOX MOUNTING AND SUPPORT FOR FIRE ALARM INITIATION DEVICES AND ALL OTHER LOW VOLTAGE/SIGNAL SYSTEMS.

23) ANY DAMAGE CAUSED BY DEMOLITION OPERATIONS TO ADJACENT FACILITIES SHALL BE PROMPTLY REPAIRED AT NO ADDITIONAL COST TO THE OWNER. ALL ITEMS MOVED OR TEMPORARILY DISASSEMBLED SHALL BE REPLACED OR REASSEMBLED TO AT LEAST THE CONDITION IN LIKE QUALITY PRIOR TO REMOVAL OR DISASSEMBLY.

24) GENERAL CONTRACTOR TO HIRE HVAC CONTRACTOR TO MODIFY/INTERFACE HAVC EQUIPMENT TO THE NEW FIRE ALARM.

25) THE COLOR CODE FOR WIRE INSULATION IN THE LOW VOLTAGE SPECIFICATIONS SHOULD BE FOLLOWED.

26) SEAL ALL SPACE AROUND CONDUIT PENETRATION THROUGH FIRE RATED WALL WITH A UL LISTED FIRE BARRIER COMPOUND "3M" CAULKING OR EQUAL.

LIGHTING FIXTURE SCHEDULE

TYPE	DESCRIPTION	FINISH	LED	DRIVER	WATTAGE	MANUFACTURER & PART NUMBER	REMARKS
A1	8FT INDIRECT DIRECT LED LINEAR 7IN WIDE BY 2IN DEEP. CENTER WHITE ACRYLIC LENS 60% UP AND 40% DOWN WITH LOW PEAK ANGLE UPLIGHT	WHITE	1 - 82.4W LED 4K	0 - 10V DIMMING 1%	82.4	LITE CONTROL - SAE-104-P-LPA-8-8-SOF-C1-40K-130-4D-DO1-IC-UNV-FA3	PENDANT 26 LBS
A2	8FT INDIRECT DIRECT LED LINEAR 7IN WIDE BY 2IN DEEP. CENTER WHITE ACRYLIC LENS 60% UP AND 40% DOWN WITH LOW PEAK ANGLE UPLIGHT	WHITE	1 - 106.4 LED 4K	0 - 10V DIMMING 1%	106.4	LITE CONTROL - SAE-104-P-LPA-8-8-SOF-C1-40K-155-2D-DO1-IC-UNV-FA3	PENDANT 24LBS
B	2X4 NSF LISTED TROFFER IP66 ALUMINUM HOUSING AND STAINLESS STEEL DOOR 304 STAINLESS STEEL. 125 PRISMATIC POLYCARBONATE LENS INVERTER	WHITE	1 - 55W LED 4K	0 - 10V DIMMING	55	KURTZON - FP-R-4-2X4-2-LEDR-840-UNV-LEX-F2NC PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 25LBS
B1	2X4 NSF LISTED TROFFER IP66 ALUMINUM HOUSING AND STAINLESS STEEL DOOR 304 STAINLESS STEEL. 125 PRISMATIC POLYCARBONATE LENS INVERTER	WHITE	1 - 35W LED 4K	0 - 10V DIMMING	35	KURTZON - FP-R-4-2X4-2-LEDR-840-UNV-LEX-F2NC-P35W PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 25LBS
C	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 59W LED 4K	0 - 10V DIMMING	59	COLUMBIA - SRP14-40-VL-G-ED-U PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
CE	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 59W LED 4K	0 - 10V DIMMING	59	COLUMBIA - SRP14-40-VL-G-ED-U-ELL14 PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
C1	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 43W LED 4K	0 - 10V DIMMING	43	COLUMBIA - SRP14-40-HL-G-ED-U PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
C2	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 39W LED 4K	0 - 10V DIMMING	39	COLUMBIA - SRP14-40-ML-G-ED-U PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
C2E	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 39W LED 4K	0 - 10V DIMMING	39	COLUMBIA - SRP14-40-ML-G-ED-U-ELL14 PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
C3	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 32W LED 4K	0 - 10V DIMMING	32	COLUMBIA - SRP14-40-LW-G-ED-U PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
C4	1X4 RECESSED FLAT PANEL LIGHTING WITH SIDE LITE LEDS FOR LOW GLARE	WHITE	1 - 27W LED 4K	0 - 10V DIMMING	27	COLUMBIA - SRP14-40-MW-G-ED-U PROVIDE SURFACE VERSION IF REQUIRED BY ARCHITECT	RECESSED 20 LBS
D1	4FT LED LINEAR WITH SATIN LENS NO BREAK IN LENS	WHITE	1 - 22.4W LED 4K	0 - 10V DIMMING 1%	22.4	PRUDENTIAL - BPRO4-REC-FLSH-LED4-MO-4-TMW-SAL-LP-SC-UNV-X(TBD)-DM01	RECESSED 13 LBS
D2	8FT LED LINEAR WITH SATIN LENS NO BREAK IN LENS	WHITE	1 - 30.4W LED 4K	0 - 10V DIMMING 1%	30.4	PRUDENTIAL - BPRO4-REC-FLSH-LED4-LO-8-TMW-SAL-LP-SC-UNV-X(TBD)-DM01	RECESSED 29 LBS
D3	12FT LED LINEAR WITH SATIN LENS NO BREAK IN LENS	WHITE	1 - 45.6W LED 4K	0 - 10V DIMMING 1%	45.6	PRUDENTIAL - BPRO4-REC-FLSH-LED4-LO-12-TMW-SAL-LP-SC-UNV-X(TBD)-DM01	RECESSED 42 LBS
EX	DIE CAST ALUMINUM EXIT SIGN	STANDARD FINISH	1 - 1W LED GREEN	NON DIM	1	EVENLITE - CCDS-AC-G-1/2-TBD	TBD 5.25 LBS EM 4.51LBS AC
F	10IN WIDE BY 13.5 IN TALL PENDANT WITH DUAL LAYER ACRYLIC LENS WITH OUTER GLOW DAMP LISTED	STANDARD FINISH	1 - 25W LED 4K	0 - 10V DIMMING 1%	25	OCL - LE1-TBD-10-MW-FINISH-LED2-40K-UNV-48-DM1-ULD	PENDANT 8 LBS
G	6IN LED DOWNLIGHT WITH CLEAR POLYCARBONATE LENS	WHITE FLANGE	1 - 21.6W LED 4K	0 - 10V DIMMING	21.6	VANTAGE LIGHTING - A6VFLED-U-15-40K-L6011P-SGC-ZDM	RECESSED 6LBS
H	12.5IN ROUND EXTERIOR OPAL LENS ADA	STANDARD FINISH	1 - 17W LED 4K	0 - 10V DIMMING	17	NEW STAR - GW-R-O-MD-40-UN-FINISH	SURFACE 15 LBS

CODES, STANDARDS & GUIDES

PARTIAL LIST OF APPLICABLE CODES AS OF January 1, 2020*
2019 California Administrative Code (CAC), Part 1, Title 24 CCR*
2019 California Building Code (CBC), Part 2, Title 24 CCR
(2018 International Building Code, Vol. 1 & 2, and 2019 California amendments)
2019 California Electrical Code (CEC), Part 3, Title 24 CCR
(2017 National Electrical Code and 2019 California Amendments)
2019 California Mechanical Code (CMC), Part 4, Title 24 CCR
(2018 IAPMO Uniform Mechanical Code and 2019 California amendments)
2019 California Plumbing Code (CPC), Part 5, Title 24 CCR
(2018 IAPMO Uniform Plumbing Code and 2019 California amendments)
2019 California Energy Code (CEC), Part 6, Title 24 CCR
2019 California Fire Code (CFC), Part 9, Title 24 CCR
(2018 International Fire Code and 2019 California Amendments)
2019 California Existing Building Code (CEBC), Part 10, Title 24 CCR
(2018 International Existing Building Code and 2019 California Amendments)
2019 California Green Building Standards Code (CALGreen), Part 11, Title 24 CCR
2019 California Referenced Standards Code, Part 12, Title 24 CCR
Title 19 CCR, Public Safety, State Fire Marshal Regulations

PARTIAL LIST OF APPLICABLE STANDARDS
NFPA 13 - Standard for the Installation of Sprinkler Systems (CA amended).....2016 Edition
NFPA 14 - Standard for the Installation of Standpipe and Hose Systems (CA amended)....2016 Edition
NFPA 17 - Standard for Dry Chemical Extinguishing Systems.....2017 Edition
NFPA 17A - Standard for Wet Chemical Extinguishing Systems.....2017 Edition
NFPA 20 - Standard for the Installation of Stationary Pumps for Fire Protection.....2016 Edition
NFPA 72 - National Fire Alarm and Signaling Code (CA amended).....2016 Edition
NFPA 80 - Standard for Fire Doors and Other Opening Protectives.....2016 Edition
NFPA 2001 - Standard on Clean Agent Fire Extinguishing Systems (CA amended).....2015 Edition
UL 300 - Standard for Fire Testing of Fire Extinguishing Systems for Protection of Commercial Cooking Equipment.....2005 (R2010)
UL 464 - Audible Signaling Devices for Fire Alarm and Signaling Systems, Including Accessories.....2003 Edition
UL 521 - Standard for Heat Detectors for Fire Protective Signaling Systems.....1999 Edition
UL 1971 - Standard for Signaling Devices for the Hearing Impaired.....2002 (R2010)

For a complete list of applicable NFPA standards refer to 2019 CBC (SFM) Chapter 35 and California Fire Code Chapter 80.

See California Building Code Chapter 35 for State of California amendments to the NFPA Standards.

*All parts of the 2019 California Building Code become effective January 1, 2020 except the effective date for the use of the 2019 Building Energy Efficiency Standards (Title 24, Part 1, Chapter 10) is January 8, 2019 and the effective date for the use of the California Administrative Code (Title 24, Part 1, Chapter 4) is January 8, 2019.

MEP Component Anchorage Note
All mechanical, plumbing, and electrical components shall be anchored and installed per the details on the DSA-approved construction documents. The following components shall be anchored or braced to meet the force and displacement requirements prescribed in the 2019 CBC Sections 1617A.1.18 through 1617A.1.26 and ASCE 7-16 Chapters 13, 26 and 30:

1. All permanent equipment and components.
2. Temporary, movable or mobile equipment that is permanently attached (e.g. hard wired) to the building utility services such as electricity, gas or water. "Permanently attached" shall include all electrical connections except plugs for 110/220 volt receptacles having a flexible cable.
3. Temporary, movable or mobile equipment which is heavier than 400 pounds or has a has a center of mass located 4 feet or more above the adjacent floor or roof level that directly support the component is required to be restrained in a manner approved by DSA.

The following mechanical and electrical components shall be positively attached to the structure but need not demonstrate design compliance with the references noted above. These components shall have flexible connections provided between the component and associated ductwork, piping, and conduit. Flexible connections must allow movement in both transverse and longitudinal directions:

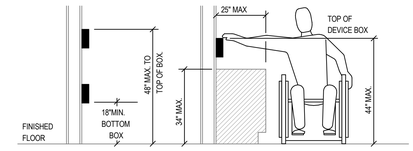
A. Components weighing less than 400 pounds and having a center of mass located 4 feet or less above the adjacent floor or roof level that directly support the component.
B. Components weighing less than 20 pounds, or in the case of distributed systems, less than 5 pounds per foot, which are suspended from a roof or floor or hung from a wall.

The anchorage of all mechanical, electrical and plumbing components shall be subject to the approval of the design professional in general responsible charge or structural engineer delegated responsibility and acceptance by DSA. The project inspector will verify that all components and equipment have been anchored in accordance with the above requirements.

Piping, Ductwork, and Electrical Distribution System Bracing Note
Piping, ductwork, and electrical distribution systems shall be braced to comply with the forces and displacements prescribed in ASCE 7-16 Section 13.3 as defined in ASCE 7-16 Sections 13.6.5, 13.6.6, 13.6.7, 13.6.8; and 2019 CBC, Sections 1617A.1.24, 1617A.1.25 and 1617A.1.26.

The method of showing bracing and attachments to the structure for the identified distribution system are as noted below. When bracing and attachments are based on a preapproved installation guide (e.g., OSHPD OPM for 2013 CBC or later), copies of the bracing system installation guide or manual shall be available on the jobsite prior to the start of and during the hanging and bracing of the distribution systems. The Structural Engineer of Record shall verify the adequacy of the structure to support the hanger and brace loads.

Mechanical Piping (MP), Mechanical Ducts (MD), Plumbing Piping (PP), Electrical Distribution Systems (E):
MP☐ MD☐ PP☐ E☑ - Option 1: Detailed on the approved drawings with project specific notes and details.
MP☐ MD☐ PP☐ E☐ - Option 2: Shall comply with the applicable OSHPD Pre-Approval (OPM #) #____.



MOUNTING HEIGHTS
N.T.S. OUTLETS, INTERCOM, CONTROLS, ETC.

TYPICAL MOUNTING HEIGHTS ABOVE FINISHED FLOOR (UNLESS OTHERWISE NOTED ON DRAWINGS)

+48" A.F.F. TO TOP OF BOX:	LIGHT SWITCHES, DIMMER SWITCHES, FIRE ALARM PULL STATION, DUCT DETECTOR TEST PANEL, INTERCOM CALL SWITCH, SPEAKER VOLUME CONTROL, T-STATS, BY-PASS TIMER WALL TELEPHONE.
+18" A.F.F. TO BOTTOM OF BOX:	ALL DUPLEX RECEPTACLES, WALL OUTLET FOR DESK TELEPHONE, COMPUTER OUTLET, UNLESS OTHERWISE NOTED.

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. E102, (A#03-122536, DATED 03/24/2023)

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LICENSED ARCHITECT C-26053 Ren. Date 08/31/2023 STATE OF CALIFORNIA

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Check by: AF
Revisions

No.	Date	Description
1	06-20-2023	BID CLARIFICATION NO.1

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH SCHOOL CAFETERIA

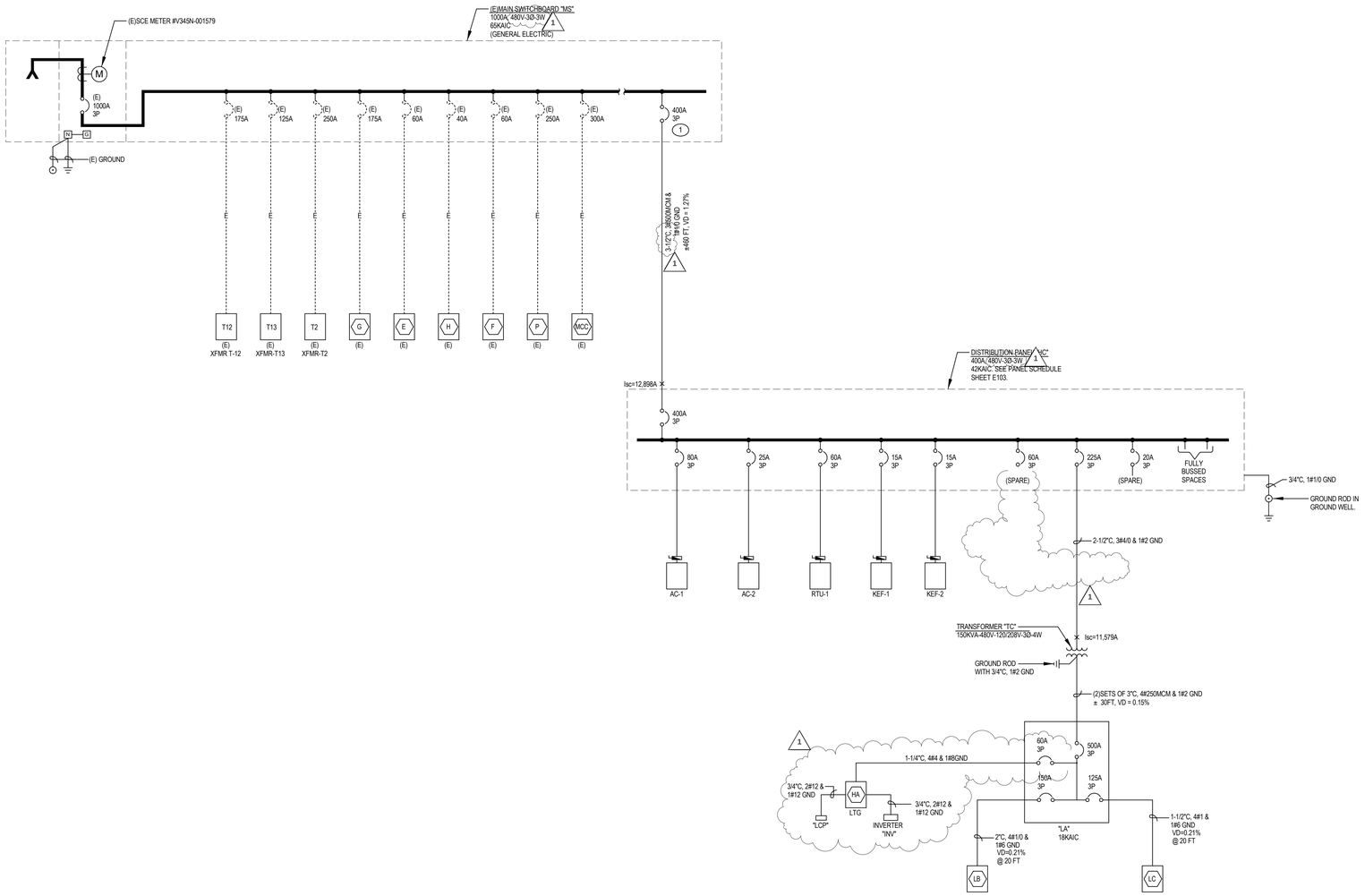
2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

Drawing Title

ELECTRICAL SYMBOL LIST, NOTES & DETAIL

Job No. 2990.000
Date 01.26.2022
Sheet No. **E101**

BID CLARIFICATION NO.1 (06-20-2023)



SINGLE LINE DIAGRAM

N.T.S.

KEYED NOTES

1. PROVIDE NEW (1)400 AMPS, 3 POLE, 480 VOLTS CIRCUIT BREAKER WITH MOUNTING ACCESSORY. NEW CIRCUIT BREAKER TO MATCH EXISTING AND A.I.C. RATING.

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH SCHOOL CAFETERIA

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

Drawing Title

SINGLE LINE DIAGRAM

Job No. 2990.000

Sheet No. **E102**

Date 01.26.2022

BID CLARIFICATION NO.1 (06-20-2023)

DISTRIBUTION PANEL HC

480 VOLTS
3 PHASE
3 WIRE

MAIN BKR: 400A, 3P, 480V CB
LOCATION ELEC ROOM 17
FEEDER
400 A. BUSSING
ENTER CAB'T. AT
SURFACE MTG.

LOCATION	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LTG	REC	MIS	CIR	BKR		BKR	CIR	MIS	REC	LTG	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LOCATION
AC-1	16406					1	1	80-3	@ @	20-3	2	1			2809			AC-1 POWER EXHAUST 4HP
-----		16406				-	3	-		-	4	-				2809		-----
-----			16406			-	5	-		-	6	-					2809	-----
AC-2	4786					1	7	25-3	@ @	15-3	8	1			775			AC-2 POWER EXHAUST 1HP
-----		4786				-	9	-		-	10	-				775		-----
-----			4786			-	11	-		-	12	-					775	-----
RTU-1	7747					1	13	60-3	@ @	15-3	14	1			1053			
-----		7747				-	15	-		-	16	-				1053		-----
-----						-	17	-		-	18	-					1053	-----
KEF-1	1053					1	19	15-3	@	20-3	20	1						SPARE
-----		1053				-	21	-		-	22	-						-----
-----			1053			-	23	-		-	24	-						-----
SPACE							25	SPC		SPC	26							SPACE
SPACE							27	SPC		SPC	28							SPACE
SPACE							29	SPC		SPC	30							SPACE
SPACE							31	SPC		SPC	32							SPACE
SPACE							33	SPC		SPC	34							SPACE
SPACE							35	SPC		SPC	36							SPACE
SPARE							37	60-3		225-3	38	1			46608			SUBFEED XFMR "TC" PANEL "LA"
-----							39	-		-	40	-				43955		-----
-----							41	-		-	42	-					43197	-----
PH - A= 81237 VA							PH - B= 78584 VA											PH - C= 70079 VA
TOTAL CONNECTED LOAD: 229900 VA OR 276.7 AMPS @ 480 VOLTS - 3 Ø																		
LCL: 0 VA X 125% = 0 VA																		
FDL: 229900 VA + 0 VA (LCL) = 229900 VA OR 276.7																		

@ - USE HACR TYPE CIRCUIT BREAKER.

PANEL HA

120/208 VOLTS
3 PHASE
4 WIRE

MAIN BKR: MAIN LUGS ONLY
LOCATION ELEC ROOM 17
FEEDER
125 A. BUSSING
ENTER CAB'T. AT
SURFACE MTG.

LOCATION	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LTG	REC	MIS	CIR	BKR		BKR	CIR	MIS	REC	LTG	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LOCATION
LTG DINING HALL RM 02	1154			14			1	20-1	A	20-1	2			8	393			LTG RMS 09,11,12,13 & 17
LTG DINING HALL RM 02		243		7			3	20-1	A A	20-1	4			10		653		LTG SERVING & MAIN KITCHEN
LTG SERVING AREA RM 08			534	23			5	20-1	A	20-1	6			10			440	LTG RMS 03,04,05,06,07 & 18
LTG EXTERIOR DINING AREA	459			17			7	20-1	A	20-1	8	1			300			LCP
LTG EXTERIOR WALL		187		11			9	20-1		20-1	10	SPR						SPARE
INVERTER			765			I	11	20-1		20-1	12	SPR						SPARE
SPARE						SPR	13	20-1		SPC	14							SPACE
SPARE						SPR	15	20-1		SPC	16							SPACE
SPARE						SPR	17	20-1		SPC	18							SPACE
SPACE							19	SPC		SPC	20							SPACE
SPACE							21	SPC		SPC	22							SPACE
SPACE							23	SPC		SPC	24							SPACE
PH - A= 2306 VA							PH - B= 1083 VA											PH - C= 1739 VA
TOTAL CONNECTED LOAD: 5128 VA OR 14.24 AMPS @ 120/208 VOLTS -- 3 Ø																		
LCL: 4063 VA X 125% = 5079 VA																		
FDL: 1065 VA + 5079 VA (LCL) = 6143.8 VA OR 17.07																		

NOTE: PROVIDE TIE BARS DISCONNECTING MEANS TO CIRCUIT BREAKERS SERVING MULTI-WIRE CIRCUITS (SHARED NEUTRAL CONDUCTOR).
A - CIRCUIT IS CONTROLLED BY "LCP".

EMERGENCY INVERTER "INV"

120 VOLTS INPUT
120 VOLTS OUTPUT
1 PHASE
3 WIRE

MAIN BRK: 20A, 1P, 120V
BUS RATING/TYPE: 70A, 14KAIC
LOCATION: ELECTRICAL ROOM
FEEDER: 2#12 & 1#12G
MOUNTING: FLOOR

CK #	LOCATION	WATTAGE	LTG	REC	MIS	BKR
1	LTG DINING & MAIN ENTRY	240	5			20-1
2	LTG SERVING & KITCHEN	323	4			20-1
3	LTG EXT DINING	108	4			20-1
4	LTG EXTERIOR ENTRY	90	5			20-1
5	EXIT SIGN	4	4			20-1
6	SPARE					20-1
	TOTAL (VA)	765				

TOTAL CONNECTED LOAD: 765 VA OR 6.375 AMPS

NOTE: PROVIDE TIE BARS DISCONNECTING MEANS TO CIRCUIT BREAKERS SERVING MULTI-WIRE CIRCUITS (SHARED NEUTRAL CONDUCTOR).
A - CIRCUIT IS CONTROLLED BY "LCP".

INVERTER INV SHALL BE "EVENLITE #LM 1600-1P-IA-OA-D6-TA"

PANEL LA

120/208 VOLTS
3 PHASE
4 WIRE

MAIN BKR: 500A, 3P, 240V CB
LOCATION ELEC ROOM 17
FEEDER
600 A. BUSSING
ENTER CAB'T. AT
SURFACE MTG.

LOCATION	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LTG	REC	MIS	CIR	BKR		BKR	CIR	MIS	REC	LTG	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LOCATION
AIR COOLED CONDENSER & EVAPORATOR	2640					1	1	35-3	@ @	60-3	2	1			4164			AIR COOLED CONDENSER & EVAPORATOR
-----		2640				-	3	-		-	4	-				4164		-----
-----			2640			-	5	-		-	6	-					4164	-----
SUMP PUMP	1872					1	7	30-3	@	60-3	8	1			5520			DISHWASHING MACHINE
-----		1872				-	9	-		-	10	-				5520		-----
-----			1872			-	11	-		-	12	-					5520	-----
FREEZER LTG	120				4		13	20-1		35-2	14	1			2640			ROOF HP-3
COOLER LTG		120			4		15	20-1		-	16	-				2640		-----
DRY STORAGE LTG			120		4		17	20-1		35-2	18	1					2640	ROOF HP-2
EXHAUST FAN-1	500					1	19	20-1		-	20	-			2640			-----
RECEPTACLE - IT ROOM 06		180			1		21	20-1		35-2	22	1				2640		ROOF HP-1
INTRUSION CONTROL PANEL			500			1	23	20-1		-	24	-					2640	-----
EXHAUST FAN-2	120					1	25	20-1		20-1	26							SPARE
EXHAUST FAN-3		150				1	27	20-1		20-1	28					900		IRRIGATION CONTROLLER
EXHAUST HOOD CONTROL PANEL			500			1	29	20-1	*	20-1	30						1176	TOTAL PAC X-DRY SYS.
SUB-FEED PANEL-HA	2306					1	31	60-3		20-1	32							SPARE
-----		1083				-	33	-		20-1	34							SPARE
-----			1739			-	35	-		20-1	36							SPARE
SUBFEED PANEL "LB"	13338						37	150-3		125-3	38	1			10748			SUBFEED PANEL "LC"
-----		14898					39	-			40	-				7148		-----
-----			10734				41	-		-	42	-					8952	-----
PH - A= 46608 VA							PH - B= 43955 VA											PH - C= 43197 VA
TOTAL CONNECTED LOAD: 133760 VA OR 371.6 AMPS @ 120/208 VOLTS -- 3 Ø																		
LCL: 360 VA X 125% = 450 VA																		
FDL: 133400 VA + 450 VA (LCL) = 133850 VA OR 371.8																		

@ - USE HACR TYPE CIRCUIT BREAKER.
* - PROVIDE LOCK-ON DEVICE AND PAINT RED PER NFPA REQUIREMENT.

PANEL LB

120/208 VOLTS
3 PHASE
4 WIRE

MAIN BKR: MAIN LUGS ONLY
LOCATION ELEC ROOM 17
FEEDER
225 A. BUSSING
ENTER CAB'T. AT
SURFACE MTG.

LOCATION	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LTG	REC	MIS	CIR	BKR		BKR	CIR	MIS	REC	LTG	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LOCATION
ICE MACHINE RM 9	1920						1	20-2		$ 20-2	2	1			450			COMB. OVEN GAS
-----		1920					3			-	4	-				450		-----
REACH-IN REFRIGERATOR RM 9			900				5	20-1		$ 20-2	6	1					450	COMB. OVEN GAS
ULTRA NX - RM 9	1800						7	20-1		-	8	-			450			-----
ULTRA NX - RM 9		1800					9	20-1		20-1	10		2			1800		RECEPTACLE
RECEPTACLE RM 9			540		3		11	20-1		20-1	12		2				1200	RECEPTACLE
HEATED HOLDING CABINET	1560					1	13	20-2		20-1	14		2		360			RECEPTACLE
-----		1560				-	15	-		20-1	16		4			720		RECEPTACLE
REFRIGERATOR, PASS-THRU			1020			1	17	20-1		20-1	18	1					1728	AIR CURTAIN
HEATED HOLDING CABINET	1560					1	19	20-2		20-1	20		2		1200			RECEPTACLE
-----		1560				-	21			20-1	22		2			1200		RECEPTACLE
HOT FOOD DROP-IN WELL			1044			1	23	20-2		20-2	24	1					1044	HOT FOOD DROP-IN WELL
-----	1044					-	25	-		-	26	-			1044			-----
HOT FOOD DROP-IN WELL		1044				1	27	20-2		20-2	28	1				1044		HOT FOOD DROP-IN WELL
-----			1044			-	29	-		-	30	-					1044	-----
REFRIGERATED SELF SVC	1800					1	31	20-1	*	20-1	32	1			150			LIGHTING CONTACTOR
REFRIGERATED SELF SVC		1800				1	33	20-1		20-1	34							SPARE
ROOF RECEPTACLE			720		4		35	20-1		20-1	36							SPARE
SPACE							37	SPC		20-1	38							SPARE
SPACE							39	SPC		20-1	40							SPARE
SPACE							41	SPC		20-1	42							SPARE
PH - A= 13338 VA							PH - B= 14898 VA											PH - C= 10734 VA
TOTAL CONNECTED LOAD: 38970 VA OR 108.3 AMPS @ 120/208 VOLTS -- 3 Ø																		
LCL: 0 VA X 125% = 0 VA																		
FDL: 38970 VA + 0 VA (LCL) = 38970 VA OR 108.3																		

* - PROVIDE LOCK-ON DEVICE AND PAINT RED PER NFPA REQUIREMENT.
$ - USE GFCI TYPE CIRCUIT BREAKER.

PANEL LC

120/208 VOLTS
3 PHASE
4 WIRE

MAIN BKR: MAIN LUGS ONLY
LOCATION ELEC ROOM 17
FEEDER
125 A. BUSSING
ENTER CAB'T. AT
SURFACE MTG.

LOCATION	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LTG	REC	MIS	CIR	BKR		BKR	CIR	MIS	REC	LTG	WATTAGE PH - A	WATTAGE PH - B	WATTAGE PH - C	LOCATION
RECEPTACLE RM 11	720				4		1	20-1		20-1	2		4		720			RECEPTACLE, TV DINING HALL
RECEPTACLE POS		360			2		3	20-1		20-1	4		4			720		RECEPTACLE, TV, PROJECTOR DINING HALL
RECEPTACLE TV, SERVING AREA			540		3		5	20-1		20-1	6		6				1080	RECEPTACLE, DINING HALL
RECEPTACLE TV, SERVING AREA	540				3		7	20-1		20-1	8		1		1920			PLUG, IDF
REMOTE FA AMPLIFIER		600				1	9	20-1	*	30-1	10		1			1920		PLUG, IDF
FCPS FA			600			1	11	20-1	*	20-1	12		1				1320	DISPLAY CABINET
THERMAL SERVER COFFEE BREWER	1800				1		13	20-1		20-1	14		1		900			FREEZER
SINGLE HOT FOOD DISPENSER		816			1		15	20-1		20-1	16		1			900		REFRIGERATOR
MICROWAVE, COMMERCIAL			1728		1		17	20-1		20-1	18	1					864	AIR CURTAIN
MICROWAVE, COMMERCIAL	1728				1		19	20-1		20-1	20	1			500			DRINKING FOUNTAIN
ROLLER GRILL		1152			1		21	20-1		20-1	22	1	1			500		CIRC. PUMP & WH IGNITER
ICE MACHINE			1920			1	23	20-2		20-1	24		4				720	RECEPTACLE OUTDOOR
-----	1920					-	25			20-1	26							SPARE
RECEPTACLE RM 17		180			1		27	20-1		20-1	28							SPARE
EXTERIOR RECEPTACLE			180		1		29	20-1		20-1	30							SPARE
SPARE							31	20-1		20-1	32							SPARE
SPARE							33	20-1		20-1	34							SPARE
SPARE							35	20-1		20-1	36							SPARE
SPACE							37	SPC		SPC	38							SPACE
SPACE							39	SPC		SPC	40							SPACE
SPACE							41	SPC		SPC	42							SPACE
PH - A= 10748 VA							PH - B= 7148 VA											PH - C= 8952 VA
TOTAL CONNECTED LOAD: 26848 VA OR 74.58 AMPS @ 120/208 VOLTS -- 3 Ø																		
LCL: 0 VA X 125% = 0 VA																		
FDL: 26848 VA + 0 VA (LCL) = 26848 VA OR 74.58																		

* - PROVIDE LOCK-ON DEVICE AND PAINT RED PER NFPA REQUIREMENT.

PANEL SCHEDULE KEY	
HC	LA
HA	LB
INV	LC

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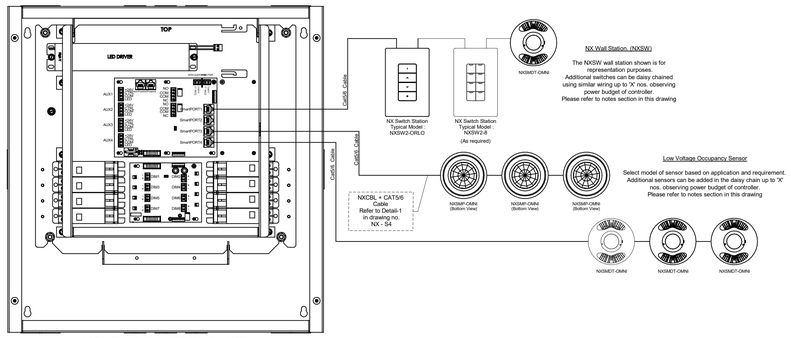
ROSAMOND, CA 93560

Drawing Title

PANEL SCHEDULES

Job No. 2990.000
Sheet No. **E103**
Date 01.26.2022



NX Wall Station. (NXSW)

The NXSW wall station shown is for representation purposes. Additional switches can be daisy chained using similar wiring up to 'X' nos. observing power budget of controller. Please refer to notes section in this drawing

Low Voltage Occupancy Sensor

Select model of sensor based on application and requirement. Additional sensors can be added in the daisy chain up to 'X' nos. observing power budget of controller. Please refer to notes section in this drawing

LIGHTING CONTROL PANEL
TYPICAL MODEL :
NXP2-24

Notes:

1. The NXP2 panels all come with 4 female RJ45 connectors (FX/SmartPORTs)
2. The NXP2 panel supports connections to Cat5 cables terminated in a T568B configuration male RJ45 connector.
3. These (4) NX FX/SmartPORTs are to be used to connect with optional NX compatible devices to increase functionality.
4. FX/SmartPORT connections are typically in a daisy chain manner to the next optional compatible device.
5. Time clock functionality included
6. Can control normal and EM loads

NX CONTROLS LIGHTING CONTROL DIAGRAM

LIGHTING CONTROL PANEL (LCP) SCHEDULE			
RELAY #	PANEL CKT #	LOCATION SERVICE	SWITCH
R1	HA-1	DINING HALL 02	DIMMER SW./OCCUPANCY SENSOR
R2	HA-3	DINING HALL 02	DIMMER SW./OCCUPANCY SENSOR
R3	HA-5	SERVING AREA 08	DIMMER SW./OCCUPANCY SENSOR
R4	HA-5	SERVING AREA 08	DIMMER SW./OCCUPANCY SENSOR
R5	HA-4	SERVING AREA 08	DIMMER SW./OCCUPANCY SENSOR
R6	HA-4	MAIN KITCHEN	KEY SW./DIMMER SW./OCCUPANCY SENSOR
R7	HA-7	EXT. DINING AREA	KEY SW./DIMMER SW./TIME CLOCK
R8	HA-9	EXT. LIGHTING	TIME CLOCK
R9	SPARE		
R10	SPARE		
R11	SPARE		
R12	SPARE		BARRIER EMERGENCY AND NORMAL
R13	INV-1	DINING HALL 02	DIMMER SW/OCCUPANCY SENSOR
R14	INV-2	SERVING AREA 08	DIMMER SW./OCCUPANCY SENSOR
R15	INV-2	MAIN KITCHEN 10	DIMMER SW./OCCUPANCY SENSOR
R16	INV-1	MAIN ENTRY 01	DIMMER SW./OCCUPANCY SENSOR
R17	INV-3	EXT. DINING AREA	KEY SW/TIME CLOCK
R18	INV-4	EXT. LIGHTING	TIME CLOCK
R19	SPARE		
R20	SPARE		

NOTES

1. THE NX NETWORK IS A POWERED ETHERNET NETWORK. IT UTILIZES CAT5 IN/OUT DAISY CHAIN CONNECTION BETWEEN DEVICES. UP TO 32-NX DEVICES PER FX ZONE SEGMENT. UP TO 1,000 FEET TOTAL CABLE LENGTH PER FX ZONE SEGMENT. MAXIMUM OF 500 TOTAL NX DEVICES OF ALL TYPES PER NX ZONE.

2. CONNECT NX SWITCHES, SENSORS AND OTHER DEVICES ON ONE SMARTPORT.

3. THE NX SELECT SYSTEM COMPONENTS COME EMBEDDED WITH TIME CLOCK AND SCHEDULING FUNCTIONALITY.

4. FX ZONE SEGMENTS PROVIDE FOR CONNECTION OF AND COMMUNICATIONS BETWEEN NX ZONE DEVICES.

5. SMARTPORT DEVICES SEGMENTS PROVIDE FOR CONNECTION OF AND COMMUNICATION BETWEEN NX DEVICES (SENSORS, SWITCHES, ACCESSORIES) WITHIN A NX ZONE. SMARTPORT AUTOMATIC CONFIGURATION SUPPORT FOR NX OCCUPANCY SENSORS AND SMART WALL STATIONS OR CONTROLLED RECEPTACLES.

6. THE NX IS PROGRAMMED VIA A MOBILE APP. IT IS AVAILABLE FOR DOWNLOAD FREE OF CHARGE FROM THE APPLE (IOS VERSION) OR GOOGLE PLAY (ANDROID VERSION) STORES. THE APP IS INTENDED AS A PROGRAMMING TOOL FOR CONFIGURATION OF VARIOUS NX DEVICES ON A ROOM BY ROOM BASIS. ONLY ONE SMART DEVICE SUCH AS A PHONE OR TABLET CAN CONNECT TO ONE BLUETOOTH® ENABLED NX DEVICE AT A TIME.

7. LIGHTING CONTROL PANELS MAY HAVE MULTIPLE VOLTAGES AND POWER FEEDS ENTERING THE ENCLOSURE. WHERE MULTIPLE VOLTAGES/FEEDS EXIST, ONE OR MORE VOLTAGE BARRIERS ARE REQUIRED.

8. NX LIGHTING CONTROL PANELS COME EITHER IN 8, 16,24, 32 OR 48 RELAY OPTIONS WITH BOTH SWITCHES OR 0-10 VOLT DIMMING.

9. NX ROOM CONTROLLERS COME WITH 1 OR 2 RELAYS WITH 0-10 VOLT DIMMING OUTPUTS.

10. OCCUPANCY SENSORS UTILIZE DIGITALLY CONFIGURABLE DUAL TECHNOLOGY ULTRASONIC (US) AND PASSIVE INFRARED FOR DETECTION. ADJUSTABLE COVERAGE AREA UP TO 2,000 SQUARE FEET. IT ALSO INTEGRATES HYBRID DAYLIGHT SENSOR FOR DAYLIGHT HARVESTING.

LEGEND

LV - Use CAT-5e CABLE.

LIGHTING CONTROL - CURRENT NX DEVICES

"LCP" "LCP" NXP2-PNL-32-24-U-S
NX LIGHTING CONTROL PANEL 32 SIZE CABINET WITH (24) 0-10 VOLT DIMMING RELAYS WITH EM UL924

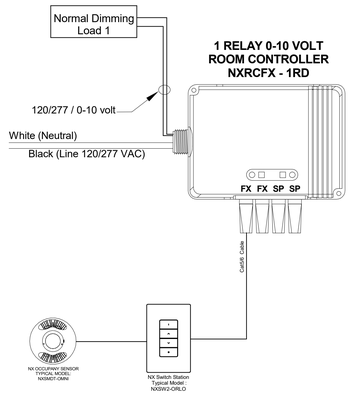
NXSW2
NX 8 BUTTON LOW VOLTAGE SWITCH STATION

NXSW2-ORLO
NX LOW VOLTAGE DIMMER SWITCH STATION

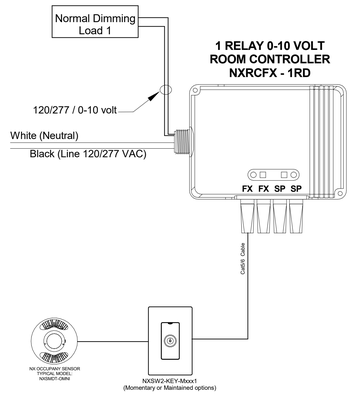
NXSW2-KEY
NX LOW VOLTAGE KEY SWITCH STATION

NXOS-LODT
NX WALL MOUNT OCCUPANCY SENSOR

NXOS-LODT
NX WALL MOUNT OCCUPANCY SENSOR



ASB SNACK BAR & MAIN ENTRY



BOY'S & GIRL'S TOILET

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Revisions

No.	Date	Description
1	06-20-2023	BID CLARIFICATION NO.1

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH SCHOOL CAFETERIA

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

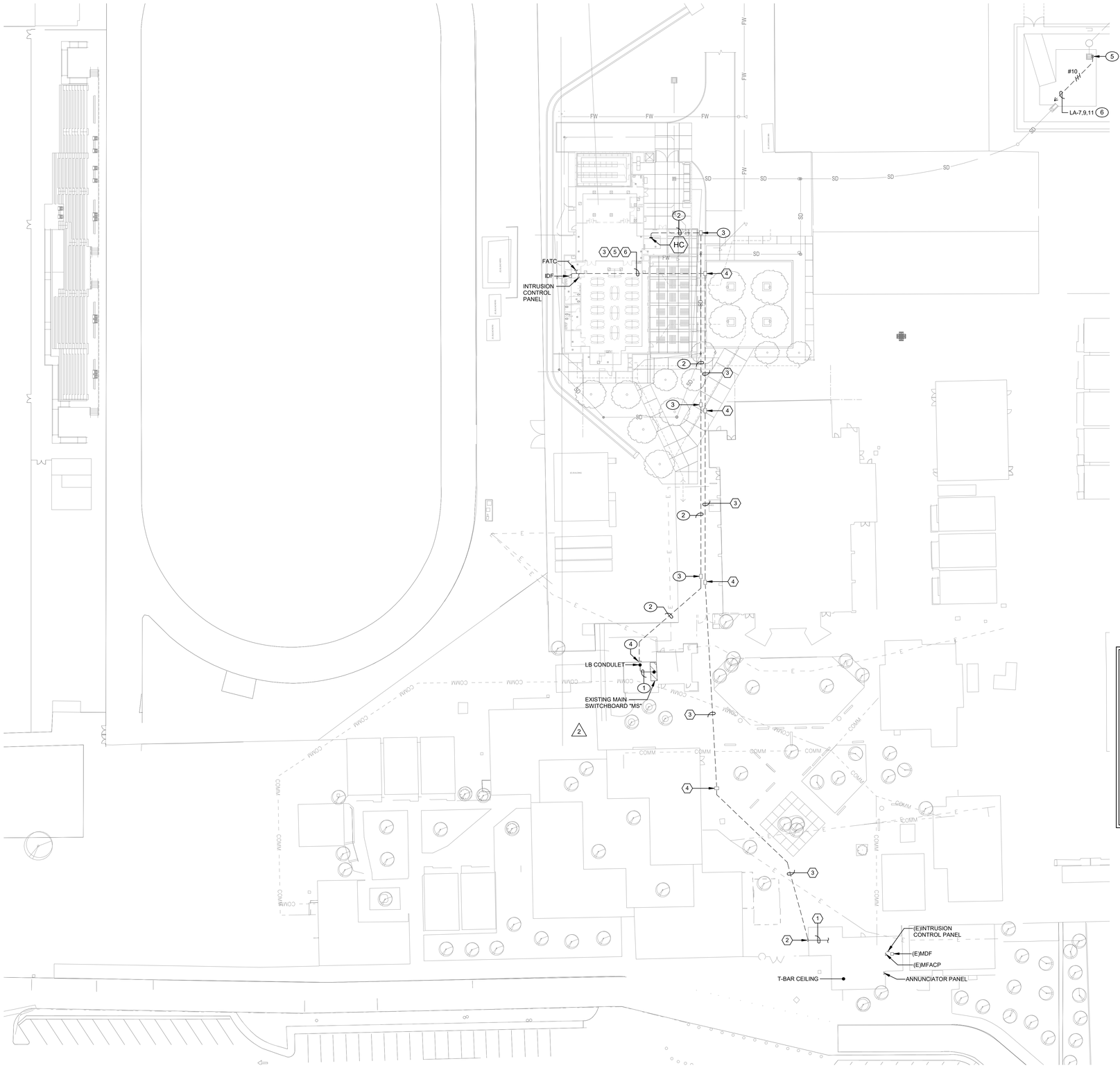
Drawing Title

LIGHTING CONTROL WIRING DIAGRAM

Job No. 2990.000

Sheet No. **E104**

Date 01.26.2022



1 ELECTRICAL SITE PLAN
1" = 30'-0"

POWER KEYED NOTES

1. 3-1/2"C, 3#500MCM & 1#1/0 GND. RUN CONDUIT HIGH ON WALL.
2. 3-1/2"C, 3#500MCM & 1#1/0 GND.
3. 2'x3'x3' DEEP CONCRETE PULLBOX, TRAFFIC TYPE COVER MARK "ELECTRIC". SIMILAR OR EQUAL TO "JENSEN #PB2436".
4. WEATHERPROOF PULLBOX 24"x24"x8" DEEP, MOUNT HIGH ON WALL.
5. SUMP PUMP MOTOR CONTROLLER SUITABLE FOR 208 VOLTS, 3 PHASE, 3 WIRES POWER SUPPLY.
6. RUN CONDUIT ALONG STORM DRAIN LINE, BURY 24" BELOW FINISH GRADE WITH CONCRETE ENCASEMENT AND DETECTABLE WARNING TAPE.

SIGNAL KEYED NOTES

1. 2"C, (1)FO (DATA/TEL/PA);
   2"C, (1)FW, (1)AW, (1)VW (FA);
   2"C, (1)SC (INTRUSION);
   RUN CONDUIT IN CEILING SPACE. TERMINATE TO RESPECTIVE SIGNAL HEADEND.
2. WEATHERPROOF PULLBOX, MOUNT HIGH ON WALL
   (a)24"x24"x6"DEEP (DATA, TEL, PA, INTRUSION);
   (b)12"x12"x6"DEEP (FIRE ALARM).
3. 2"C, (1)FO (DATA/TEL/PA);
   2"C, (1)FW, (1)AW, (1)VW (FA);
   2"C, (1)SC (INTRUSION);
   2"C.O. (SPARE).
4. 2'x3'x3' DEEP CONCRETE PULLBOX, TRAFFIC TYPE COVER MARK "SIGNAL". SIMILAR OR EQUAL TO "JENSEN #PB2436".
5. TERMINATE TO RESPECTIVE SIGNAL CABINET AND IDF.
6. STUB-UP SPARE CONDUIT 12" ABOVE GROUND ALONG FATC WALL. CAP AND TAG CONDUIT.

NOTE:

SAW CUT EXISTING CONCRETE PAVEMENT AND RESTORE TO ORIGINAL CONDITION AFTER NEW CONDUIT PATHWAYS ARE INSTALLED.

EXCAVATION NOTES

CONTRACTOR SHALL FIELD VERIFY UNDERGROUND UTILITIES PRIOR TO WORK (ELECTRICAL CONDUITS, SEWER LINES, WATER LINES, PLUMBING, ETC.). CONTRACTOR SHALL BE RESPONSIBLE FOR COORDINATION AND ROUTING OF NEW TRENCHING IN AREAS OF EXISTING UNDERGROUND LINES. CONTRACTOR SHALL VERIFY EXACT LOCATIONS OF ALL EXISTING LINES WHERE NEW TRENCHING OCCURS BY USING ELECTRONIC LOCATING DEVICES, DIG ALERT SERVICES, ETC. CONTRACTOR SHALL INCLUDE THE COST OF LOCATING THE EXISTING UNDERGROUND LINES IN THE BID. CONTRACTOR SHALL BE RESPONSIBLE FOR REPAIRING ANY AND ALL DAMAGE TO EXISTING LINES.

TRENCHING FOR NEW UNDERGROUND CONDUITS IN AREAS WHERE EXISTING UNDERGROUND LINES ARE PRESENT SHALL BE EXCAVATED BY HAND AND WITH EXTREME CAUTION. UNDERGROUND POWER AND SIGNAL PULLBOXES WHERE SHOWN ON THE SITE PLAN DETERMINED TO BE IN CONFLICT WITH EXISTING UNDERGROUND LINES AFTER EXCAVATION BY HAND SHALL BE RELOCATED TO A POSITION AS CLOSE AS POSSIBLE TO THE LOCATION SHOWN ON THE DRAWINGS.

NOTE: THE INTENT OF THIS DRAWING IS TO INDICATE ONLY THE REVISIONS SHOWN IN THE CLOUDED AREAS. ALL OTHER WORK SHALL REMAIN UNCHANGED PER DSA APPROVED SHT. E201, (A#03-122536, DATED 03/24/2023)

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SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH SCHOOL CAFETERIA

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

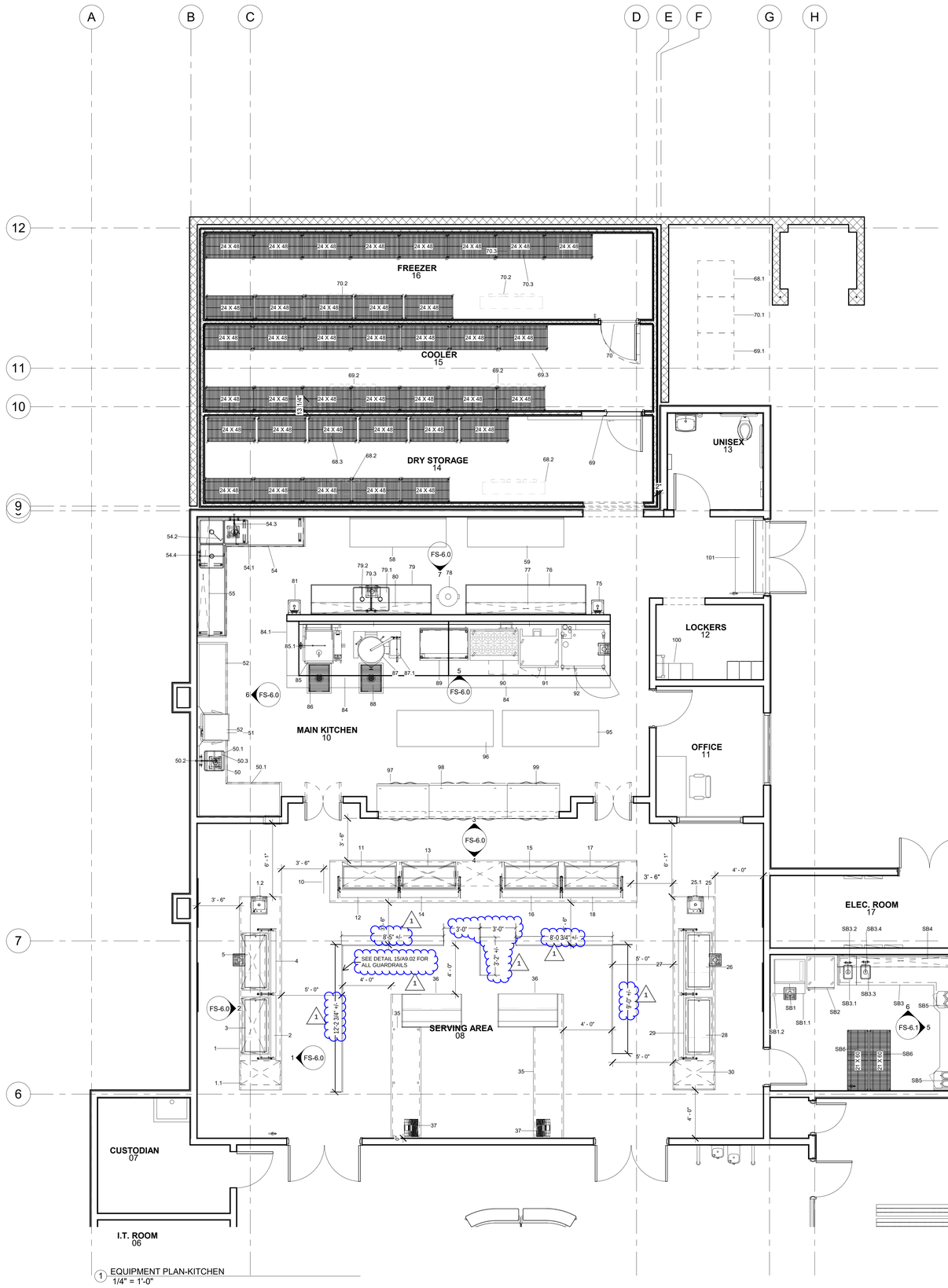
Drawing Title

ELECTRICAL SITE PLAN

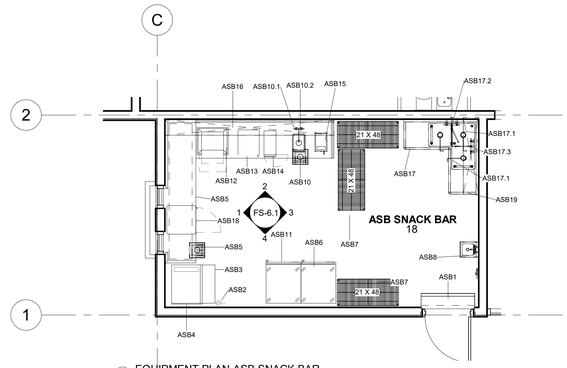
Job No. 2990.000

Date 01.26.2022

Sheet No. **E201**



1 EQUIPMENT PLAN-KITCHEN
1/4" = 1'-0"



2 EQUIPMENT PLAN-ASB SNACK BAR
1/4" = 1'-0"

Applicable Code: 2019 CBC

MEP Component Anchorage Note

All mechanical, plumbing, and electrical components shall be anchored and installed per the details on the DSA-approved construction documents. The following components shall be anchored or braced to meet the force and displacement requirements prescribed in the 2019 CBC Sections 1617A.1.18 through 1617A.1.26 and ASCE 7-16 Chapters 13, 26, and 30:

1. All permanent equipment and components.
2. Temporary, movable or mobile equipment that is permanently attached (e.g., hard wired) to the building utility services such as electricity, gas or water. "Permanently attached" shall include all electrical connections except plugs for 110/220 volt receptacles having a flexible cable.
3. Temporary, movable or mobile equipment which is heavier than 400 pounds or has a center of mass located 4 feet or more above the adjacent floor or roof level that directly support the component is required to be restrained in a manner approved by DSA.

The following mechanical and electrical components shall be positively attached to the structure but need not demonstrate design compliance with the references noted above. These components shall have flexible connections provided between the component and associated ductwork, piping, and conduit. Flexible connections must allow movement in both transverse and longitudinal directions:

A. Components weighing less than 400 pounds and having a center of mass located 4 feet or less above the adjacent floor or roof level that directly support the component.
B. Components weighing less than 20 pounds, or in the case of distributed systems, less than 5 pounds per foot, which are suspended from a roof or floor or hung from a wall.

The anchorage of all mechanical, electrical and plumbing components shall be subject to the approval of the design professional in general responsible charge or structural engineer delegated responsibility and acceptance by DSA. The project inspector will verify that all components and equipment have been anchored in accordance with the above requirements.

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HEALTH DEPARTMENT NOTES

1. ALL EQUIPMENT SHALL MEET NATIONAL SANITATION FOUNDATION STANDARDS.
2. FLOOR SURFACE IN KITCHEN, SERVING AREA AND RESTROOMS SHALL BE SMOOTH WASHABLE CONTINUOUS APPROVED MATERIAL, COVED UP AT WALL 6 INCHES WITH 3/8" MINIMUM RADIUS COVE. SAMPLE OF MATERIAL TO BE SUBMITTED FOR APPROVAL.
3. WALLS AND CEILINGS OF KITCHEN, SERVING AREA AND RESTROOMS SHALL BE OF A DURABLE, SMOOTH, NONABSORBENT, LIGHT COLORED AND WASHABLE SURFACE. SAMPLES OF MATERIAL AND COLOR TO BE SUBMITTED FOR APPROVAL.
4. ALL EQUIPMENT SHALL BE MOUNTED ON 6" LEGS, CASTER OR MINIMUM 4" HIGH CURB, AND BE CONSTRUCTED OF NONTOXIC, NON-CORROSIVE MATERIALS. DURABLE, SMOOTH, NONABSORBENT AND WASHABLE.
5. ALL UTENSIL AND FOOD PREPARATION SINKS SHALL DRAIN TO A FLOOR SINK.
6. ALL HAND SINKS TO HAVE PERMANENTLY INSTALLED SOAP AND TOWEL DISPENSERS.
7. WATER HEATER TO BE PROVIDED. SIZED TO PROVIDE ADEQUATE CAPACITY FOR ALL HOT WATER REQUIREMENTS AT A MINIMUM OF 120 DEGREES FAHRENHEIT.
8. ALL SINKS TO BE EQUIPPED WITH HOT AND COLD MIXING FAUCETS.
9. ALL EXPOSED PLUMBING AND ELECTRICAL LINES SHALL BE MOUNTED SO AS TO FACILITATE CLEANING.
10. GREASE TRAP OR INTERCEPTOR SHALL BE LOCATED OUTSIDE OF ESTABLISHMENT.
11. ALL EXPOSED ELECTRICAL LINES SHALL BE RUN IN RIGID OR SEALTIGHT CONDUIT AND TO BE MOUNTED SO AS TO FACILITATE CLEANING.
12. SHATTER PROOF SHIELDS WILL BE PROVIDED ON ALL LIGHTS IN KITCHEN AND FOOD SERVING AREAS.
13. WALK-IN COOLER TO CONFORM WITH N.S.F. STANDARDS OR EQUIVALENT AND TO MEET WITH HEALTH DEPT. REQUIREMENTS.
14. ALL EXTERIOR DOORS TO HAVE SELF CLOSING DEVICES.
15. TRASH DUMPSTER LOCATED OUTSIDE ON HARD CLEANABLE SURFACE.
16. PROVIDE MINIMUM 50 FOOT CANDLES OF LIGHT IN ALL FOOD PREPARATION AND STORAGE AREAS, UTENSIL WASH, BARS AND RESTROOM AREAS.
17. BUILDING TO BE RODENT PROOFED, WITH ALL DOORS COMING TO WITHIN 1/16 INCH OF FLOOR OR THRESHOLD.
18. ALL EQUIPMENT, FLASHING AND BACK SPLASHES, UNLESS EASILY MOVABLE, SHALL BE SEALED TO THE WALL AND/OR ABUTTING EQUIPMENT.
19. ALL TOILETS TO HAVE PERMANENTLY INSTALLED TISSUE DISPENSERS.
20. REFRIGERATION EQUIPMENT SHALL HAVE CONDENSATE EVAPORATOR OR DRAIN TO FLOOR SINK.
21. ALL FLOOR SINKS SHALL BE HALF EXPOSED OR OTHERWISE READILY ACCESSIBLE FOR INSPECTION AND CLEANING. FLOOR SINK SHALL NOT POSE A TRIPPING HAZARD.
22. HOODS TO EXTEND A MINIMUM OF 6" BEYOND EDGE OF COOKING SURFACE AS MEASURED FROM INSIDE EDGE.
23. RESTROOM DOORS SHALL BE TIGHT FITTING AND EQUIPPED WITH SELF CLOSING DEVICE.
24. ALL RESTROOMS SHALL BE PROVIDED WITH VENTILATION TO OUTSIDE.
25. MOP SINK TO BE PROVIDED WITH HOT AND COLD WATER PROVIDED WITH BACK FLOW PROTECTION.
26. ALL EQUIPMENT WHICH GENERATES CONDENSATE AND LIQUID WASTES FROM STEAM TABLES, ICE MACHINES AND BINS, UTENSIL WASH SINKS, FOOD PREPARATION SINKS, DISPLAY CASES, ETC. SHALL BE DRAINED BY INDIRECT WASTE LINE TO FLOOR SINK OR APPROVED RECEPTOR. FLOOR DRAINS ARE NOT TO BE USED IN LIEU OF FLOOR SINK.
27. PUBLIC SEWAGE SYSTEM IS USED.
28. ALL EQUIPMENT TO BE N.S.F. APPROVED OR FABRICATED TO N.S.F. STANDARDS.

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FOR CAPTIVEAIRE KITCHEN HOOD AND ANSUL SYSTEM SEE SHEETS M5.01 TO M5.08.

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Drawn by:

Check by:

Revisions

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Owner

ROSAMOND HS

2925 WEST ROSAMOND BLVD
ROSAMOND, CA

Drawing Title

EQUIPMENT

Job No.: Project Number

Sheet No.: **FS-1.0**

Date: Issue Date

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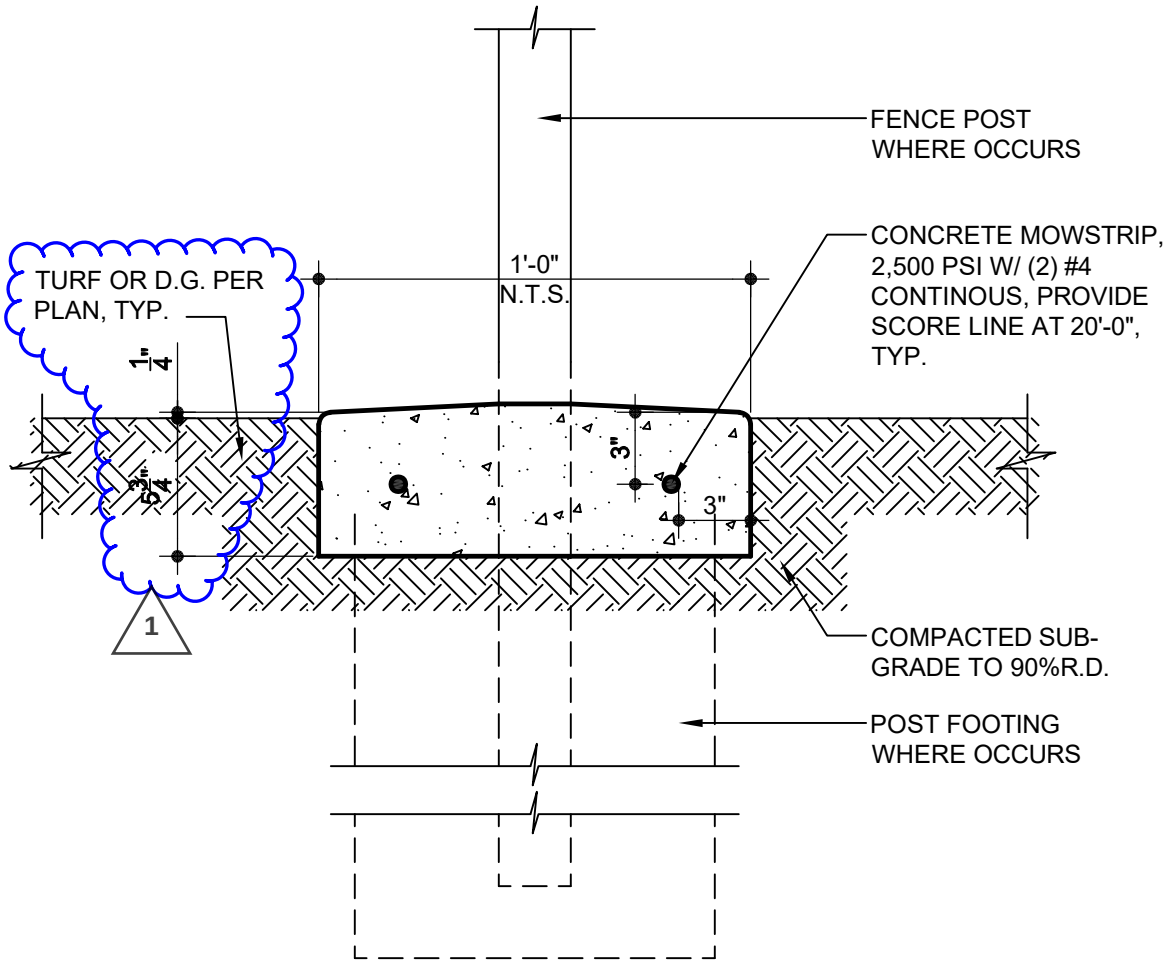

CONCRETE BAND 1

SCALE : 1-1/2"=1'-0"

PROJECT NUMBER =
DETAIL IDEN. =
SHEET REFERENCE =

5
A1.06

LICENSED ARCHITECT
STEVEN BUCHMAN
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

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BID CLARIFICATION NO. 1

CONCRETE BAND

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COLLEGE CAMPUS
for
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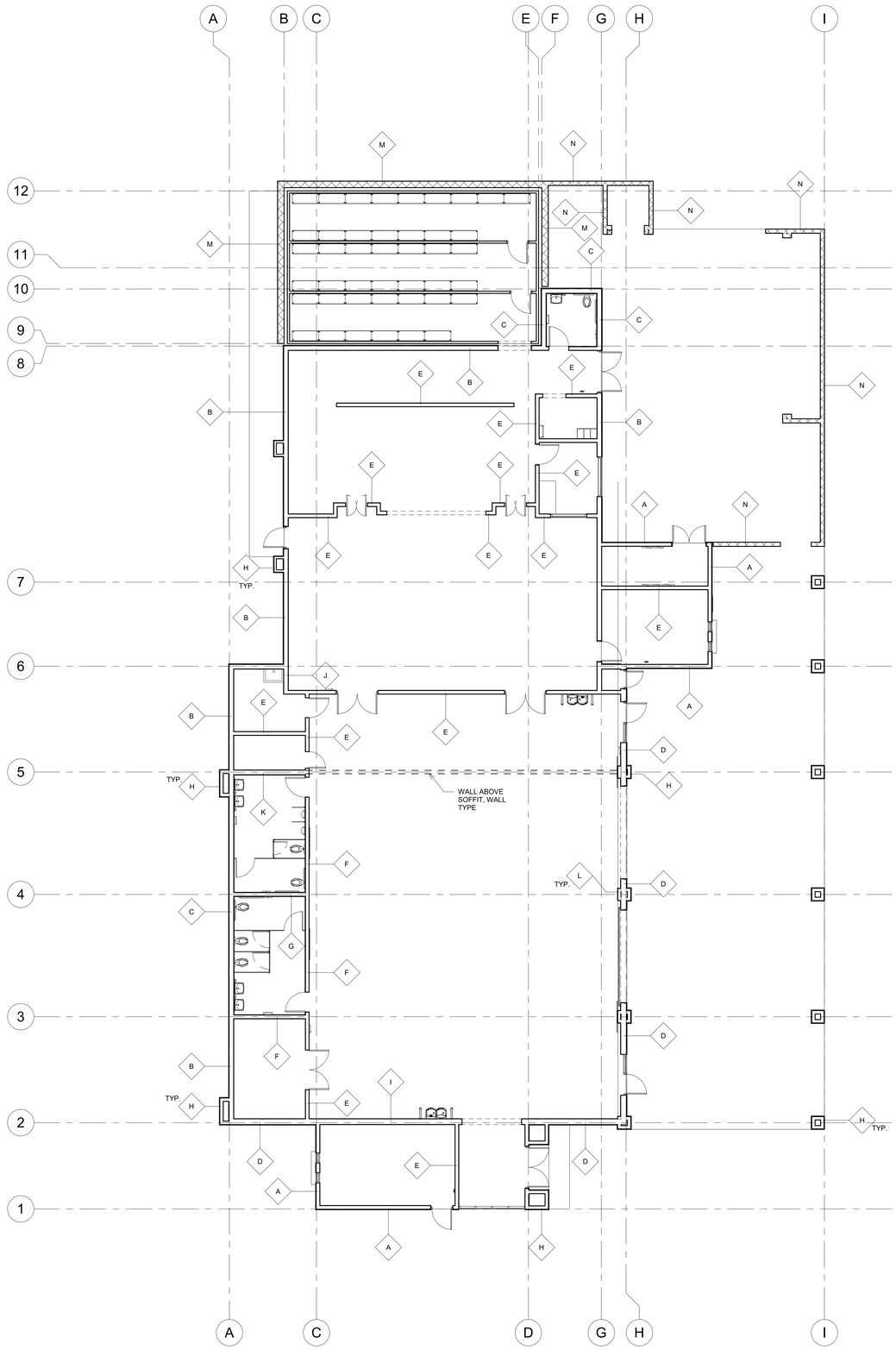
ROSAMOND, CA 93560

Date 06-20-23
Job No. 2990.000
Sheet No. BD-01



1 WALL TYPE FLOOR PLAN
1/8" = 1'-0"

WALL TYPES

A	2X6 STUDS @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD (INTERIOR) AND PLASTER STUCCO OVER PLYWOOD SHEATHING (EXTERIOR), PER STRUCTURAL PLAN.	
B	1 ¾ X 7 ¼ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD (INTERIOR) AND PLASTER STUCCO OVER PLYWOOD SHEATHING (EXTERIOR), PER STRUCTURAL PLAN.	
C	1 ¾ X 7 ¼ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ ½" CERAMIC TILE, GROUTED, OVER ½" DUROC BOARD (INTERIOR) AND PLASTER STUCCO OVER PLYWOOD SHEATHING (EXTERIOR), PER STRUCTURAL PLAN.	
D	1 ¾ X 9 ¼ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD (INTERIOR) AND PLASTER STUCCO OVER PLYWOOD SHEATHING (EXTERIOR), PER STRUCTURAL PLAN.	
E	2X6 STUDS @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD ON BOTH SIDES. FINISHES PER FINISH SCHEDULE.	
F	2X6 STUDS @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD ON ONE SIDE, ¼" CERAMIC TILE, GROUTED OVER ½" DUROC BOARD ON THE OTHER SIDE. FINISHES PER FINISH SCHEDULE.	
G	2X6 STUDS @ 16" O.C. W/ ½" CERAMIC TILE, GROUTED OVER ½" DUROC BOARD ON THE BOTH SIDES.	
H	2X6 STUDS @ 16" O.C. FURRED WALL W/ PLASTER STUCCO OVER PLYWOOD SHEATHING, PER STRUCTURAL DETAIL X/X.X	
1	1 ¾ X 9 ¼ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD ON BOTH SIDES. FINISHES PER FINISH SCHEDULE.	
J	1 ¾ X 7 ½ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD BOTH SIDES. FINISHES PER FINISH SCHEDULE.	
K	1 ¾ X 7 ¼ VERSA-LAM LVL 2.1E 2800 @ 16" O.C. W/ 5/8" TYPE 'X' GYPSUM BOARD ON ONE SIDE, ½" CERAMIC TILE, GROUTED OVER ½" DUROC BOARD ONE OTHER SIDE.	
L	2X6 STUDS @ 16" O.C. FURRED WALL W/ 5/8" TYPE 'X' GYPSUM BOARD. FINISH PER FINISH SCHEDULE.	
M	12" PRECISION CMU BLOCK WALL	
N	8" PRECISION CMU BLOCK WALL	

NOTE: THE INTENT OF THIS DRAWING IS TO INCORPORATE THE WORK SHOWN HEREIN INTO THE CONTRACT. ALL WORK HEREIN INDICATED SHALL HEREBY BE MADE A PART OF THE CONTRACT.

AGENCY

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An Employee Owned Corporation

ARCHITECT

LICENSED ARCHITECT
C-26053
Ren. Date 08/31/2023
STATE OF CALIFORNIA

CONSULTANT

Drawn by:

Check by:

Revisions

No.	Date	Description

All dimensions must be checked at the job by the contractor who accepts full responsibility for their accuracy under the contract. These plans & the specifications in connection therewith have been prepared for a specific site. Any and all responsibility for their use in whole or in part on any other site is hereby disclaimed by Flewelling & Moody.

SOUTHERN KERN UNIFIED SCHOOL DISTRICT

ROSAMOND HIGH EARLY COLLEGE CAMPUS

2925 WEST ROSAMOND BLVD.
ROSAMOND, CA 93560

Drawing Title

WALL TYPES FLOOR PLAN

1

Job No. 2990.000

Sheet No. **A2.00A**

Date 01.30.2023