



**GEOTECHNICAL INVESTIGATION
CHILD DEVELOPMENT CENTER
SHADE STRUCTURE PROJECT
GROSSMONT HIGH SCHOOL
1100 MURRAY DRIVE
EL CAJON, CALIFORNIA**

Prepared For **GROSSMONT UNION HIGH SCHOOL
DISTRICT
1100 MURRAY DRIVE
EL CAJON, CA 91920**

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Leighton Consulting, Inc.

A Leighton Group Company

June 28, 2023

Project No. 12429.034

Grossmont Union High School District
1100 Murray Drive
El Cajon, California 92020

Attention: Ms. Dena Johnson, Project Manager

**Subject: Geotechnical Investigation
Child Development Center Shade Structure Project
Grossmont High School
1100 Murray Drive
El Cajon, California**

In accordance with your request and authorization, we have conducted a geotechnical investigation for the proposed Shade Structure Project at the Child Development Center at Grossmont High School located in El Cajon, San Diego County, California. Based on the results of our study, it is our opinion that the proposed site improvements are feasible provided the geotechnical recommendations contained in this report are implemented during design and construction. This report provides a summary of the current investigation and general geotechnical conclusions and recommendations for the project.

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

Respectfully submitted,
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Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply this report for any purpose or project except the one originally contemplated.*

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an

assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by:* the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly

problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time to perform additional study.* Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help

others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.



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1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by Geoprofessional Business Association (GBA) and the Limitations within Section 7.0 of this report.

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the proposed Child Development Center (CDC) Shade Structure Project at Grossmont High School located in El Cajon, San Diego County, California (Figure 1). We have performed a geotechnical investigation to identify and evaluate the geologic hazards and significant geotechnical conditions present at the site in order to provide geotechnical recommendations for the proposed site improvements. Our scope of services included:

- Review of available pertinent, published and unpublished geotechnical literature and maps. References cited are listed in Appendix A;
- Field reconnaissance of the existing on-site geotechnical conditions;
- Subsurface exploration consisting of the excavation, logging, and sampling of three hand auger borings and four dynamic cone penetrometer (DCP) soundings. The exploration logs are presented in Appendix B and their approximate locations are shown on Figures 2;
- Laboratory testing of representative soil samples obtained from the subsurface exploration program. Results of these tests are presented on the boring logs and in Appendix C;
- Assessment of geologic hazards;
- Development of seismic design parameters based on the 2022 California Building Code (CBC);
- Compilation and analysis of the geotechnical data obtained from the field investigation and laboratory testing; and
- Preparation of this report presenting our findings, conclusions, and geotechnical recommendations with respect to the proposed geotechnical design, site grading and general construction considerations.

1.2 Site Location and Description

Grossmont High School is located at 1100 Murray Drive in El Cajon, California. Based on our review of historical aerial photographs, topographic maps, and available background information, Grossmont High School was graded and constructed sometime in the middle to late 1950's. Specifically, the CDC is located on the southeast portion of the school's campus. Importantly, it should be noted that Leighton (Leighton, 2012) provided geotechnical observation and testing services during grading and construction of the CDC. In general, the site is bounded by Dennstedt Place and an asphalt paved parking lot followed by residential developments to the north, building 800 and an asphalt paved parking lot to the west, and a descending slope followed by Murray Drive to the east and south.

Site topography within the vicinity of the proposed site improvements is nearly level with surface elevations gently sloping from the west to the east, ranging from approximately 666 to 668.5 feet above mean sea level (msl). The site lies above and directly adjacent to an approximately 55-foot high descending natural slope to the southwest and a natural slope over a cut slope which descends to Murray Drive on the south and east. The existing slopes range in inclination from about 1.5H:1V (horizontal to vertical) in the natural area to about 0.5H:1V in the cut slope above Murray Drive.

Currently, the project site is occupied by the CDC, which is comprised of Building P and an outdoor patio area consisting of concrete hardscape features, artificial turf and rubber mat areas, existing shade structures, a site retaining wall, and underground utilities. Of particular importance is the distressed concrete flatwork, located directly adjacent to the site retaining wall, and the underlying drainage improvements, located at a depth of approximately 2 feet below the existing ground surface (bgs). Cracks up to half an inch wide were measured between the concrete and the edge of the retaining wall. With respect to the drainage improvements, the 6-inch area drain, located adjacent to the southeast corner of Building P, was observed to be clogged and not functioning properly due to the presence of standing water. Importantly, it should be noted that the underlying 6-inch and 8-inch subdrains parallel the areas of distress. It should also be noted that structural distress was not observed within Building P. The existing site improvements are shown on the base map utilized for our Geotechnical Map (Figure 2).

Generalized Site Latitude and Longitude

32.7814 ° N

116.9846 ° W

1.3 Proposed Development

Based on our discussions with you, we understand that the project will consist of the design and construction of a new shade structure that will be added to the existing CDC at Grossmont High School. The limits of the proposed shade structure are shown on our Geotechnical Map (Figure 2). It is our understanding that the new shade structure will be founded on drilled pile foundations. In addition, the project may include new concrete hardscape to replace the existing distressed areas that appear to be sloping away from the building due to potential settlement. Once project grading and foundation plans are available, they should be sent over to our office so that we can verify that the recommendations provided in our report are still valid with respect to the proposed improvements.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Site Investigation

Our subsurface exploration, performed on June 3, 2023, consisted of excavating three hand auger borings (HA-1 through HA-3) and advancing four DCP soundings (DCP-1 through DCP-4) to depths between approximately 1 to 8.5 feet bgs. The hand auger borings were excavated using conventional hand tools and the DCP soundings were advanced with a Triggs Technologies Wildcat Dynamic Cone Penetrometer using a 35-lb hammer. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils located within the area of the proposed improvements.

During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk samples for laboratory testing and evaluation. After logging and field testing, the hand auger borings and DCP soundings were backfilled with soil cuttings and capped with quick set concrete. The exploration logs are presented in Appendix B and their approximate locations are shown on Figure 2.

2.2 Laboratory Testing

Laboratory testing performed on representative soil samples obtained during the subsurface exploration included expansion index, direct shear, maximum density, optimum moisture content, and geochemical analysis for corrosion. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

3.0 SUMMARY OF GEOLOGIC CONDITIONS

3.1 Geologic and Tectonic Setting

The site is located within the coastal subprovince of the Peninsular Ranges Geomorphic Province, near the western edge of the southern California batholith. Throughout the last 54 million years, the area known as the “San Diego Embayment” has gone through several episodes of marine inundation and subsequent marine regression, resulting in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement rock of the Southern California batholith. The site is located along the eastern margin of the San Diego Embayment.

The Peninsular Ranges are traversed by several major Holocene-active faults (Figure 3). The Whittier-Elsinore, San Jacinto, and the San Andreas faults are major Holocene-active fault systems located northeast of the site and the Rose Canyon, Newport-Inglewood (offshore), Coronado Bank, and San Diego Trough are Holocene-active faults located to the west-southwest. Major tectonic activity associated with these and other faults within this regional tectonic framework is right-lateral strike-slip movement. These faults, as well as other faults in the region, have the potential for generating strong ground motions at the project site. Further discussion of faulting relative to the site is provided in the Faulting and Seismicity section of this report.

Gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, coupled with the lowering of the base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons which characterize the landforms we see in the general site area today. The regional geologic setting is depicted on Figure 4.

3.2 Local Geologic Setting

Based on our subsurface exploration and review of pertinent geologic literature and maps (Appendix A), the geologic units underlying the site consist of surficial documented artificial fill materials overlying the Tertiary-aged Pomerado Conglomerate and Tertiary-aged Mission Valley Formation. At depth, the Tertiary-aged Stadium Conglomerate underlies the entire site. The approximate areal distribution of the geologic units is depicted on the Geotechnical Map (Figure 2).

The approximate vertical distribution of geologic units underlying the site are shown on the geologic cross-sections provided in Cross-Section A-A' and Cross-Section B-B' (Figure 5).

3.2.1 Documented Artificial Fill (Afu)

Based on our subsurface exploration, documented fill soils were encountered in all of our geotechnical exploration locations with thicknesses varying between approximately 1 to 8½ feet bgs. As encountered, the fill soils generally consist of loose to medium dense, brown, slightly moist to moist, silty and clayey sands with gravel and trace amounts of cobble. The cobble located within the fill materials is anticipated to be 6- to 8-inches in diameter with the potential of isolated cobbles up to 1-foot in diameter. It should be noted that the concrete encountered in hand auger borings HA-2 through HA-4 was measured to be 5 inches thick with 1 to 2 inches of pea gravel and no underlying base. As mentioned previously, Leighton (Leighton, 2012) provided geotechnical observation and testing services of these fill soils at the time of grading.

3.2.2 Pomerado Conglomerate (Tp)

Although not encountered, formational material consisting of the Tertiary-aged Pomerado Conglomerate underlies the documented fill. Based on our experience, the Pomerado Conglomerate generally consists of very dense, yellowish brown, moist, clayey gravel conglomerate. In addition, it should be noted that well-rounded gravel and cobble typically comprise between approximately 30 and 60 percent of the conglomerate mass, respectively. The cobbles are typically 3 to 6 inches in maximum dimension; however, larger cobbles and boulders should be anticipated.

3.2.3 Mission Valley Formation (Tmv)

Although not encountered, formational material consisting of the Tertiary-aged Mission Valley Formation underlies the Pomerado Conglomerate at the project site. Based on our experience, the Mission Valley Formation generally consists of interbedded layers of light brown to grayish brown, moist, dense to very dense, moderately-cemented, fine-grained, clayey sandstone; gray, stiff to hard, moist, micaceous, sandy siltstone; and gray, very stiff, moist, oxidized, sandy lean claystone. In addition, minor isolated interbedded layers of gravel and cobble are located within the Mission Valley Formation.

3.2.4 Stadium Conglomerate (Tst)

Although not encountered, the Tertiary-aged Stadium Conglomerate underlies the Mission Valley Formation. Based on our experience, the Stadium Conglomerate generally consists of very dense, yellowish brown, moist, clayey gravel conglomerate. In addition, it should be noted that well-rounded gravel and cobble typically comprise between approximately 30 and 60 percent of the conglomerate mass, respectively. The cobbles are typically 3 to 6 inches in maximum dimension; however, larger cobbles and boulders should be anticipated.

3.3 **Geologic Structure**

Based on our field observations, subsurface exploration, and our review of site topographic and geologic maps, the site is underlain by favorably oriented geologic structure consisting of a thin veneer of artificial fill overlying generally massive, near horizontal, very dense to hard, Tertiary-aged formational units identified as the Pomerado Conglomerate, Mission Valley Formation and the Stadium Conglomerate.

3.4 **Landslides**

In areas of topographic expression, several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No landslides or indications of deep-seated landsliding were indicated at the site during our field exploration or our review of available geologic literature, topographic maps, and historical aerial photographs (Appendix A). However, it should be noted that several mapped landslide deposits which originated from within the Tertiary-aged Friars Formation are located approximately 1,000 feet east of the site (Figure 4). Nevertheless, the potential for significant landslides or large-scale slope instability at the site is considered low due to the lack of slopes and gentle existing topographic relief. In addition, it should be noted that the site is not currently within a mapped California Geologic Survey (CGS) seismic hazard zone. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered low.

3.5 Slope Stability

Site topography within the limits of the proposed development is nearly level with surface elevations ranging from approximately 666 to 668.5 feet msl. However, the site is bounded by a descending slope to the east and south. Specifically, the site lies above and adjacent to an approximately 55-foot high natural slope on the southwest and a natural slope over a cut slope which descends to Murray Drive on the south and east. The existing slopes range in inclination from about 1.5H:1V (horizontal to vertical) in the natural area to about 0.5H:1V in the cut slope above Murray Drive.

Based on previous slope stability analyses performed by Kleinfelder (Kleinfelder, 2010) and Leighton (Leighton, 2011), the hazard posed by static or seismic slope instability at the site is generally considered to be low. In addition, during our recent geotechnical investigation, we noted no indication of slope instability, such as block-type failures, sloughing, or flow type failures associated with poor drainage. Therefore, the potential for slope instability at the site is low.

3.6 Expansive Soils

Expansion index testing on one representative soil sample taken from hand-auger boring HA-3 indicates that the onsite soil materials have a very low (12) potential for expansion. However, higher expansive soils may be encountered during the grading of the proposed site improvements. Expansive soils are not anticipated to significantly impact the proposed site improvements.

3.7 Compressible Soils

Based on the results of our subsurface exploration, the existing documented fill materials located at the project site are considered compressible. Recommendations for treatment of these materials during proposed remediation are provided later in this report.

3.8 Excavation Characteristics

Excavations of the onsite fill materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, local heavy ripping, breaking, or specialty drilling techniques may be required within the underlying very dense formational materials which contain an abundance of cobble.

3.9 Soil Corrosivity

A screening of the onsite materials for corrosivity was performed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on one representative soil sample obtained during our subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had a measured pH of 7.42 and a measured minimum electrical resistivity of 2,200 ohm-cm. The test results also indicated that the sample had a chloride content of 80 parts per million (ppm) and a soluble sulfate content of less than 150 ppm.

3.10 Surface and Groundwater

No indication of surface water or evidence of surface water ponding was encountered during our limited geotechnical investigation at the site. However, surface water may drain as sheet flow across the site during rainy periods.

Groundwater was not encountered during our subsurface exploration at the site. It should be noted that groundwater levels may fluctuate with seasonal variations and irrigation and local perched groundwater conditions may exist within cemented layers of the Pomerado Conglomerate, Mission Valley Formation and Stadium Conglomerate. Nevertheless, based on the above information, we do not anticipate groundwater will be a constraint to the construction of the proposed improvements.

3.11 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the site is not located within a flood zone (Figure 6). In addition, based on our review of dam inundation and topographic maps, the site is not located within a dam inundation area (Figure 7).

3.12 Exceptional Geologic Conditions

Exceptional geologic items are items that are present within the State of California, and occur on a site by site basis. We have addressed the presence or non-presence of these items in the sections below.

3.12.1 Hazardous Materials

Our scope of work has not included evaluation of the site for hazardous materials in the subsurface and we are not aware of any such reports that pertain to the site.

3.12.2 Regional Subsidence

Due to the lack of production groundwater wells in the area and the very dense to hard nature of the underlying Tertiary-aged formational units, the possibility of regional subsidence is considered to be nil.

3.12.3 Non-Tectonic Faulting

Surface expressions of differential settlement, such as ground fissures, can develop in areas affected by ground water withdrawal or banking activities, including geothermal production. The site location is not within an area affected by differential settlement caused by non-tectonic sources.

3.12.4 Volcanic Eruption

The proposed site is not located within or near a mapped area of potential volcanic hazards (Miller, C.D., 1989). The nearest volcanic activity is located in the Salton Sea area of southern California. Therefore, volcanic activity is not considered a hazard at the site.

3.12.5 Asbestos

Due to the lack of proximal sources of serpentinitic or ultramafic rock bodies, naturally-occurring asbestos is not considered a hazard at the site.

3.12.6 Radon-222 Gas

Historically, Radon-222 gas has not typically been recognized as an environmental consideration in San Diego County. In particular, the site area is not mapped as containing organic rich marine shales commonly characterized to potentially contain Radon-222 gas. Therefore, based on our review of the referenced literature, and our site exploration, the potential for the occurrence of Radon-222 gas at the site is considered low.

4.0 FAULTING AND SEISMICITY

4.1 Faulting

The California State Mining and Geology Board (CSMGB) has defined a Holocene-active fault as a fault which has had surface displacement within Holocene time (about the last 11,700 years). In addition, a pre-Holocene fault is a fault that has not moved in the past 11,700 years, and does not meet the criteria of “Holocene-active fault” as defined in the Alquist-Priolo Act and CSMGB regulations.

These definitions are used in delineating Special Studies Zones as mandated by the Alquist-Priolo Geologic Hazards Zones Act of 1972. The intent of this act is to assure that unwise urban development does not occur across the traces of Holocene-active faults. As background, Special Publication (SP) 42 was provided for guidance with regard to informing reviewers and practitioners on the locations of the fault rupture hazard zones in California, and was subsequently revised several times. The most recent revision of the document was completed in 2018 and resulted in a significant change from previous versions, as it now provides guidelines (previously included as supplements to SP 42) for both reviewers and practitioners working in Earthquake Fault Zones (EFZs).

The primary seismic risk to the San Diego metropolitan area is the Rose Canyon fault zone located approximately 11.5 miles west of the site. The Rose Canyon fault zone consists predominantly of right-lateral strike-slip faults that extend south-southeast bisecting the San Diego metropolitan area (Figure 3). Various fault strands display strike-slip, normal, oblique, or reverse components of displacement. The Rose Canyon fault zone extends offshore at La Jolla and continues north-northwest subparallel to the coastline. The offshore segments are poorly constrained regarding location and character. South of downtown, the fault zone splits into several splays that underlie San Diego Bay, Coronado, and the ocean floor south of Coronado (Treiman, 1993; Kennedy and Clarke, 1999). Portions of the fault zone in the Mount Soledad, Rose Canyon, and downtown San Diego areas have been designated by the State of California (CGS, 2000 and 2003) as being Earthquake Fault Zones.

Our review of available geologic literature (Appendix A) indicates that there are no known Holocene-active faults transecting, or projecting toward the site.

The site is not located within any State Mapped Earthquake Fault Zones or County of San Diego mapped fault zones. The nearest Holocene-active fault is the Rose Canyon fault zone located approximately 11.5 miles west of the site (Figure 3).

4.1.1 Surface Rupture

As previously discussed, the site is not underlain by a known Holocene-active or pre-Holocene fault. Therefore, the potential for ground rupture due to faulting at the site is considered low. Ground lurching is defined as movement of low density materials on a bluff, steep slope, or embankment due to earthquake shaking. Since the site is relatively flat and removed from any over-steepened slopes, lurching or cracking of the ground surface as a result of nearby or distant seismic events is unlikely.

4.2 **Historical Seismicity**

Historically, the San Diego region has been spared major destructive earthquakes. The most recent earthquake on the Rose Canyon fault in San Diego occurred after A.D. 1523 but before the Spanish arrived in 1769. Studies by Rockwell and Murbach (1999) indicate that the earthquake occurred at A.D. 1650 ± 125 . Two additional earthquakes, the 1800 M6.5 and 1862 M5.9, may have also occurred in the Rose Canyon fault zone. However, no direct evidence of ground rupture within the Rose Canyon fault zone for those events was recorded.

The site location with respect to significant past earthquakes ($>M5.0$) is shown on the Historical Seismicity Map in Appendix D. The historic seismicity for the site has been tabulated utilizing the computer software EQSEARCH (Blake, 2023). The results are presented in Appendix D. The results indicate that the maximum historical site acceleration from 1800 to present has been estimated to be 0.094g.

4.3 **Seismicity**

The site can be considered to lie within a seismically active region, as can all of Southern California. Specifically, the Rose Canyon fault zone located approximately 11.5 miles west of the site is the 'Holocene-active' fault considered having the most significant effect at the site from a design standpoint (USGS, 2008).

4.3.1 Site Class

The site is underlain at shallow depth by the Tertiary-aged Pomerado Conglomerate and the Tertiary-aged Mission Valley Formation, followed by the Tertiary-aged Stadium Conglomerate at depth. Utilizing 2022 California Building Code (CBC procedures), our experience on similar sites, regional shear wave velocity mapping, and the results of our subsurface exploration, we have characterized the site soil profile to be Site Class C.

4.3.2 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided in Table 1 are the spectral acceleration parameters for the project determined in accordance with the 2022 CBC (CBSC, 2022) and SEAOC/OSHPD Seismic Design Maps Web Application (2022).

Table 1. CBC Mapped Spectral Acceleration Parameters

Site Class	C		
Site Coefficients	F_a	=	1.2
	F_v	=	1.5
Mapped MCE Spectral Accelerations	S_s	=	0.775g
	S_1	=	0.284g
Site Modified MCE Spectral Accelerations	S_{MS}	=	0.93g
	S_{M1}	=	0.425g
Design Spectral Accelerations	S_{DS}	=	0.62g
	S_{D1}	=	0.284g

Utilizing ASCE Standard 7-16, in accordance with Section 11.8, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.333g for the site. For a Site Class C, the F_{PGA} is 1.2 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.4g for the site.

Since the mapped spectral response at 1-second period is less than 0.75g, then all structures subject to the criteria in Section 1613.2.5 of the 2022 CBC are assigned Seismic Design Category D.

4.4 Secondary Seismic Hazards

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the site is discussed below.

4.4.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of excess pore-water pressure during strong ground shaking. Liquefaction is associated primarily with loose (low density), granular, saturated soil. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

Due to the presence of shallow hard and very dense Tertiary-aged formational units and the absence of groundwater, the potential for liquefaction at the site is generally considered to be negligible.

4.4.2 Seismically-Induced Settlement

Dynamic settlement of soils can occur as a result of strong vibratory ground shaking. The shallow existing documented fill materials at the site are considered potentially compressible and recommendations for treatment of these materials during construction are provided later in this report. Due to the hard and very dense nature of the underlying Tertiary-aged formational units and the generally shallow depth of existing fills, the potential for dynamic settlement is low at the site.

4.4.3 Surface Manifestation of Liquefaction and Dynamic Settlement

Due to absence of a shallow groundwater table and the generally hard and very dense nature of the underlying Tertiary-aged formational units, the surface manifestation of dynamic settlement is anticipated to be negligible.

4.4.4 Lateral Spreading or Flow Failure

Due to the low potential for liquefaction, the potential for lateral spreading flow failure is very low.

4.4.5 Tsunamis or Seiches

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. A seiche is an oscillation (wave) of a body of water in an enclosed or semi-enclosed basin that varies in period, depending on the physical dimensions of the basin, from a few minutes to several hours, and in height from several inches to several feet. Based on the elevation (approximately 668 feet msl) and inland location of the site, the potential for damage due to either a tsunami or seiche is nil.

5.0 CONCLUSIONS

Based on the results of our investigation of the site, it is our opinion that the proposed Child Development Center Shade Structure Project at Grossmont High School is considered feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications. The following is a summary of the significant geotechnical factors that we expect may affect development of the site.

- As the site is located in the seismically active southern California area, all structures should be designed to tolerate the dynamic loading resulting from seismic ground motions.
- Holocene-active or pre-Holocene active faults do not transect and do not project toward the site. The closest Holocene-active fault is the Rose Canyon fault zone located approximately 11.4 miles to the west.
- The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.333g. The peak horizontal ground acceleration associated with the PGA_M is 0.4g.
- The potential for liquefaction and seismic settlement at the site is low.
- The potential for slope instability at the site is considered to be low.
- Due to the gentle topography at the site and lack of adverse geologic conditions, landsliding and mass movement is considered to be unlikely.
- The existing onsite soils were found to have a very low potential for expansion.
- Exceptional geologic hazards are not anticipated to impact the site or the proposed site development.
- The existing onsite soils are generally suitable for use as engineered fill, provided that they have an expansion index less than 50 and are free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension.
- Based on the results of our subsurface exploration, we anticipate that the onsite materials should be generally excavatable with conventional heavy-duty earthwork equipment. Localized cemented zones within the Pomerado Conglomerate, Mission Valley Formation and/or the Stadium Conglomerate may be difficult to excavate and may require heavy ripping or breaking, which can produce oversized rock fragments. In addition, the Pomerado and Stadium Conglomerate contain cobbles that range in size from 6 to 8 inches in diameter with isolated cobbles to up to 1 foot in diameter expected. Removed Pomerado and Stadium Conglomerate materials may need to be screened of plus sized material to be suitable for reuse as engineered fill.

Unknown objects such as buried concrete footings and debris left from previous site uses should be anticipated and are common on sites where previous structures existed.

- Based on the results of our geotechnical evaluation, it is our opinion that the proposed shade structure can be supported on drilled pile foundations (i.e., drilled CIDH piles).
- A static groundwater table should not be encountered during remedial grading and/or foundation excavation activities. Although not encountered during our exploration, localized seepage along cemented zones and sand lenses within the Pomerado Conglomerate, Mission Valley Formation and the Stadium Conglomerate may occur.
- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on normal concrete. The onsite soils are considered to be medium to highly corrosive to buried uncoated ferrous metals. A corrosion engineer may be retained to design corrosion protection systems and to evaluate the appropriate concrete properties for the project.

Based on the results of our investigation of the site, the distress exhibited by the existing concrete flatwork is a result of a combination of factors which are summarized below:

- The distress exhibited by the existing concrete flatwork appears to be the result of a substandard concrete section (i.e. no aggregate base) over poorly compacted subgrade soils. Due to the adequate setback from the edge of the slope, we do not believe the distress is a result of slope creep.
- The underlying subdrain system parallels the distressed concrete flatwork. Based on the depth of the subdrain system, approximately 2 feet (bgs), and the observed standing water within the 6" area drain, it appears that the subdrain system is not functioning properly to provide positive drainage away from the patio. As a result, it is possible that the surrounding soils have become wet to saturated.
- Recommendations for mitigation of the distressed concrete flatwork is provided in the recommendations section of this report.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, excavation, and fill operations. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill, engineered structures, or hardscape should be cleared of surface and subsurface obstructions, including any existing debris and undocumented, loose, or unsuitable fill soils, and stripped of vegetation. Removed vegetation and debris should be properly disposed offsite. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, brought to at least 2 percent above the optimum moisture content, and recompact to at least 90 percent relative compaction based on ASTM Test Method D 1557.

6.1.2 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. However, local heavy ripping or breaking may be required if cemented zones within the Pomerado Conglomerate, Mission Valley Formation and/or Stadium Conglomerate are encountered. Excavation for utilities may also be difficult in some areas.

Due to the variable amount of oversized cobble in the Pomerado and Stadium Conglomerate, a screening process of the removed Pomerado and Stadium Conglomerate materials may need to be utilized to remove the plus sized cobble prior to its use as engineered fill. From a geotechnical perspective, it is considered acceptable to reuse the gravelly materials below a depth of 12 inches within the limits of the proposed improvements finished grade. Placement of fills with oversize materials shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill.

Oversize material larger than 3 inches should not be placed within 1 vertical feet of finish pad grade, hardscape subgrade, and within 2 feet of future utilities or underground construction. Cobble, or other irreducible material with a maximum dimension greater than 6 inches, should not be placed within engineered fill areas and should be disposed of offsite.

Where soils are found to have greater than 30 percent oversize particles retained on the $\frac{3}{4}$ -inch sieve, corrections using ASTM D 4718 are no longer considered valid. Where materials exceed the oversize fraction allowed by ASTM D 4718, use of a test fill that contains oversize fraction within the allowable limits of the standard can be compacted and tested to develop a field method to obtain the specified compaction. That method should then be applied to subsequent layers that exceed the maximum allowable oversize percentage.

Due to the general density and soil type characteristics of the Pomerado Conglomerate, Mission Valley Formation and the Stadium Conglomerate, temporary shallow excavations less than 5 feet in depth with vertical sides should remain stable for the period required to construct utilities, provided the trenches are free of adverse geologic conditions. Overlying artificial fill soils and beds of friable sands within the Pomerado Conglomerate, Mission Valley Formation and/or Stadium Conglomerate present at the site may ravel during trenching operations. In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back in accordance with Section 6.2 if workers are to enter such excavations.

6.1.3 Remedial Grading

Potentially compressible documented fill at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on shallow foundations.

Within the limits of the proposed remedial improvement areas, we recommend that at least 2 feet of compacted fill be placed below the proposed hardscape and any foundations. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

6.1.4 Engineered Fill

In areas proposed to receive engineered fill, the existing upper 8 inches of subgrade soils should be scarified, then moisture conditioned to a moisture content above the optimum content, and compacted to 90 percent or more of the maximum laboratory dry density, as evaluated by ASTM D 1557. In general, the soil that is removed may be reused and placed as engineered fill provided the material is free of oversized rock, organic materials, and deleterious debris, and moisture conditioned to above optimum moisture content. Soil with an expansion index greater than 50 should not be used within 2 feet of concrete hardscape subgrade and should be disposed of offsite. In addition, oversize material larger than 3 inches shall not be placed within 1 vertical feet of finish pad grade, hardscape subgrade, and within 2 feet of future utilities or underground construction. Cobble, or other irreducible material with a maximum dimension greater than 6 inches, shall not be placed within engineered fill areas and should be disposed of offsite.

The fill materials should be moisture conditioned to at least 2 percent above the optimum moisture content and compacted to 90 percent or more relative compaction, in accordance with ASTM D 1557. Although the optimum lift thickness for fill soils will be dependent on the type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness. Where adjacent utility lines are present and densely configured, it may be preferable to utilize a controlled low strength material (CLSM) for backfill.

Placement and compaction of fill should be performed in general accordance with the current City of El Cajon and also within the County of San Diego grading ordinances, California Building Code, sound construction practice, these recommendations and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.1.5 Import Soils

Import soils, if required, should be granular in nature, have an expansion index less than 50 (per ASTM Test Method D 4829), and have a low corrosion impact to the proposed improvements. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

The contractor should provide evidence that all import materials comply with Department of Toxic Substances Control guidelines for import materials.

6.1.6 Expansive Soils and Selective Grading

Based on our laboratory testing, and observations, we anticipate the onsite soil materials generally possess an expansion index less than 50 (Appendix C). Although not anticipated, should more highly expansive materials be encountered, selective grading may need to be performed. In addition, soil materials located below proposed concrete hardscape areas should have an expansion index less than 50. Testing of pad and subgrade fill materials should be performed to confirm expansion potential according to ASTM D 4829.

6.1.7 Controlled Low-Strength Material

Use of CLSM in lieu of compacted fill is considered acceptable from a geotechnical perspective. Also referred to as a “flowable fill”, the mix design for the CLSM should be appropriate for structural backfill applications in accordance with Section 1804A.6 of the 2022 California Building Code. The mix design should produce a consistency that will result in a flowable product at the time of placement which does not require manual means to move it into place. The CLSM should consist of a two-sack, sand-cement slurry and have and have a minimum compressive strength of 125 psi when tested in accordance with ASTM D4832. Water content in the CLSM should be maintained at a proportion to minimize subsidence and bleed water shrinkage. The CLSM should be placed on competent materials. Any standing water and any loose or soft materials should be removed prior to placement of the CLSM. Sampling of CLSM should be performed in accordance with ASTM D5971 and DSA IR 18-1.

6.2 **Remedial Measures for Distressed Concrete**

Short Term (Concrete): The distressed concrete may continue to be monitored. If left in the current state, additional distress will develop and will worsen. Routine maintenance should be anticipated which may include mitigating the distressed areas with an appropriate joint sealant.

Short Term (Subdrain System): The subdrain system should be thoroughly investigated to ensure that the system is functioning properly.

We recommend that a private utility locator be subcontracted in order to probe the existing subdrains using a push and/or crawler camera equipped with a sensor.

Long Term (Concrete): The concrete flatwork should be replaced in accordance with the recommendations provided in Section 6.

Long Term (Subdrain System): If warranted, the existing subdrain system should be replaced in accordance with Appendix E, Section 5.

6.3 Temporary Excavations

Sloping excavations may be utilized when adequate space allows. Based on the results of our investigation, we provide the following preliminary recommendations for sloped excavations in fill soils or competent Pomerado Conglomerate, Mission Valley Formation and/or Stadium Conglomerate without seepage conditions.

Table 2. Maximum Slope Ratios

Excavation Depth (feet)	Maximum Slope Ratio in Fill Soils	Maximum Slope Ratio in Competent Formation
0 to 5	1:1 (Horizontal to Vertical)	Vertical
5 to 20	1.5:1 (Horizontal to Vertical)	1:1 (Horizontal to Vertical)

The above values are based on the assumption that no surcharge loading or equipment will be placed within 10 feet of the top of slope. Care should be taken during excavation adjacent to the existing structures so that undermining does not occur. An OSHA “competent person” should observe the slope on a daily basis for signs of instability. An engineered design will be needed where groundwater or surcharge conditions are present.

6.4 Foundation Design

We recommend using CIDH friction piles to support the proposed shade structure. For the analysis and development of the vertical capacity of CIDH friction piles, an allowable skin friction of 300 and 400 psf may be utilized in existing fill and competent formational materials, respectively. For upward loads, a skin friction of 150 psf may be utilized. Skin friction may be combined with end bearing for downwardly loaded piles where the bottom of the drilled pile excavation is embedded a minimum of 2-feet into competent formational material and has been cleaned of any loose accumulation of cuttings.

A value of 4,000 psf may be utilized for allowable end bearing. A one-third increase may be utilized for short term loads as allowed for by CBC Section 1806A, for alternative basic load combinations of Section 1805A.3.2 that include wind and earthquake loads.

Pile settlement is anticipated to be less than ¼-inch under design loads and normal service conditions where capacity is based solely on pile skin friction. Where end bearing is also considered in design, up to ½-inch of settlement may be required to mobilize end bearing. The design skin friction and end bearing are based on center to center pile spacing of at least 3 pile diameters. Where piles are spaced more closely, a reduction in pile capacity is necessary. Construction of piles should be sequenced such that the concrete of constructed piles is allowed to setup prior to construction of piles within 5 diameters.

To resist lateral loads, if any, CIDH piles can be designed in accordance with Section 1807A.3 of the 2022 CBC. For level ground conditions, we recommend lateral soil bearing pressures determined from Table 1806A.2 of 150 psf per foot of depth below the finish grade be used for determination of parameters S1 and S3, in the Non-constrained and Constrained design criteria, respectively. As allowed by Section 1806A.3.4, a two-times increase in lateral bearing pressure may be used for short term loading for isolated poles that are not adversely affected by ½-inch motion at the ground surface. These pressures assume piles spaced at least eight diameters center-to-center. Where sloping ground is present or piles are more closely spaced, revised parameters should be provided.

6.4.1 Pile Installation

All pile installation should be performed under the observation of the geotechnical consultant and consistent with standard practice. Drilling equipment should be powerful enough to drill through the overlying fill soils and into the Tertiary-aged bedrock units to the design penetration depths. The contractor should anticipate difficult drilling conditions in formational materials with cobbles up to 8 inches in maximum dimension. Once a pile excavation has been started, it should be completed within 8 hours, which includes inspection, placement of the reinforcement, and placement of the concrete.

Caving of friable, soft or loose soils may occur where open excavations are made. Therefore, a starter casing may be considered to protect the top of

the borehole to mitigate caving conditions. In addition, the contractor should also be prepared to employ casing or other methods of advancing the drilled pile excavation to temporarily mitigate caving, if needed. Use of casing should be at the contractor's discretion. Loose materials should be removed from the bottom of the pile excavation prior to concrete placement.

If pile excavations become bell-shaped and cannot be advanced due to severe caving, the caved region may be filled with a sand/cement slurry and redrilled. Redrilling may continue when the slurry has reached suitable set and strength. In this case, it may be prudent to utilize casing or other special methods to facilitate continued drilling after the slurry has set.

6.4.2 Foundation Setback

We recommend a minimum horizontal setback distance from the face of slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on Table 3 below. This distance is measured from the outside bottom edge of the footing, horizontally to the slope or retaining wall face, and is based on the overall slope or wall height. The foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 3. Minimum Foundation Setback from Slope Faces

Slope Height	Minimum Recommended Foundation Setback
less than 5 feet	5 feet
5 to 15 feet	7 feet

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a pier and grade beam foundation system to support the improvement.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 2:1 (horizontal to vertical) downward sloping line starting 9 inches above the bottom edge of

the footing and should also not be located closer than 18 inches from the face of the footing.

Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement.

6.5 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as “sulfate attack.” Soluble sulfate results (Appendix C) indicated a negligible exposure potential for sulfate attack on normal concrete. The test results indicate that sulfate contents of the subsurface soils are categorized as having a sulfate exposure Class “S0” utilizing ACI 318R-14 Table 19.3.1.1. Concrete in contact with earth materials should be designed in accordance with Section 4 of ACI 318-14 (ACI, 2014) Table 19.3.2.1.

Minimum resistivity and pH tests were performed on representative samples of subgrade soils (Appendix C). Based on laboratory test results, the site soils have a low corrosion potential to buried uncoated metal conduits (Caltrans, 2021). A corrosion consultant may be contacted for further recommendations.

6.6 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D 1557 prior to the concrete placement.

Control joints should be provided at a distance equal to 24 times the slab thickness in inches, not exceeding 12 feet. Expansion joints should be incorporated where paving abuts a vertical surface, where paving changes direction and at 30 feet maximum spacing, joints should be laid out so as to create square or nearly square areas.

6.7 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked by Leighton Consulting, Inc. in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office.

We recommend that all excavations be mapped by the geotechnical consultant during grading to determine if any potentially adverse geologic conditions exist at the site.

6.8 Plan Review

Final project grading and foundation plans should be reviewed by Leighton Consulting as part of the design development process to ensure that recommendations in this report are incorporated in project plans.

7.0 LIMITATIONS

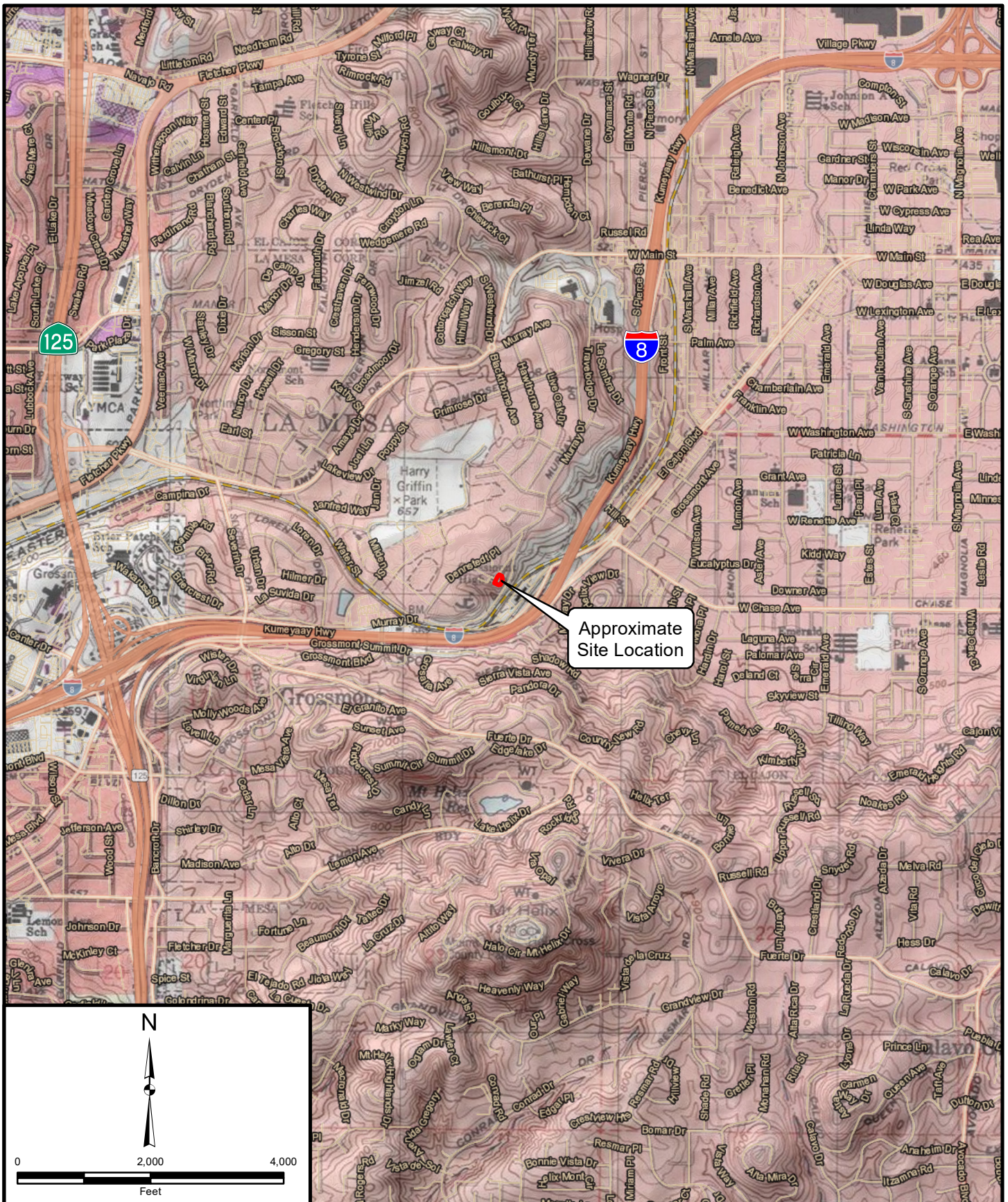
The nature of many sites is such that differing geotechnical or geological conditions can occur over small areal distances and under varying climatic conditions. The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete and therefore preliminary. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are considered preliminary and can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

IMPORTANT: Public school geotechnical reports in California are typically reviewed by the California Geologic Survey (CGS) with oversight by the California Division of the State Architect (DSA). CGS and DSA requirements change and evolve with time. Geologic data in this report is not valid for a public school project until it is reviewed and approved by CGS. Anyone using this report before CGS approval does so at their own risk and by their actions agree to indemnify, defend and hold harmless Leighton Consulting, Inc. from and against any and all alleged or real damage claims, including consequential damages, arising from use of this report before DSA approval.

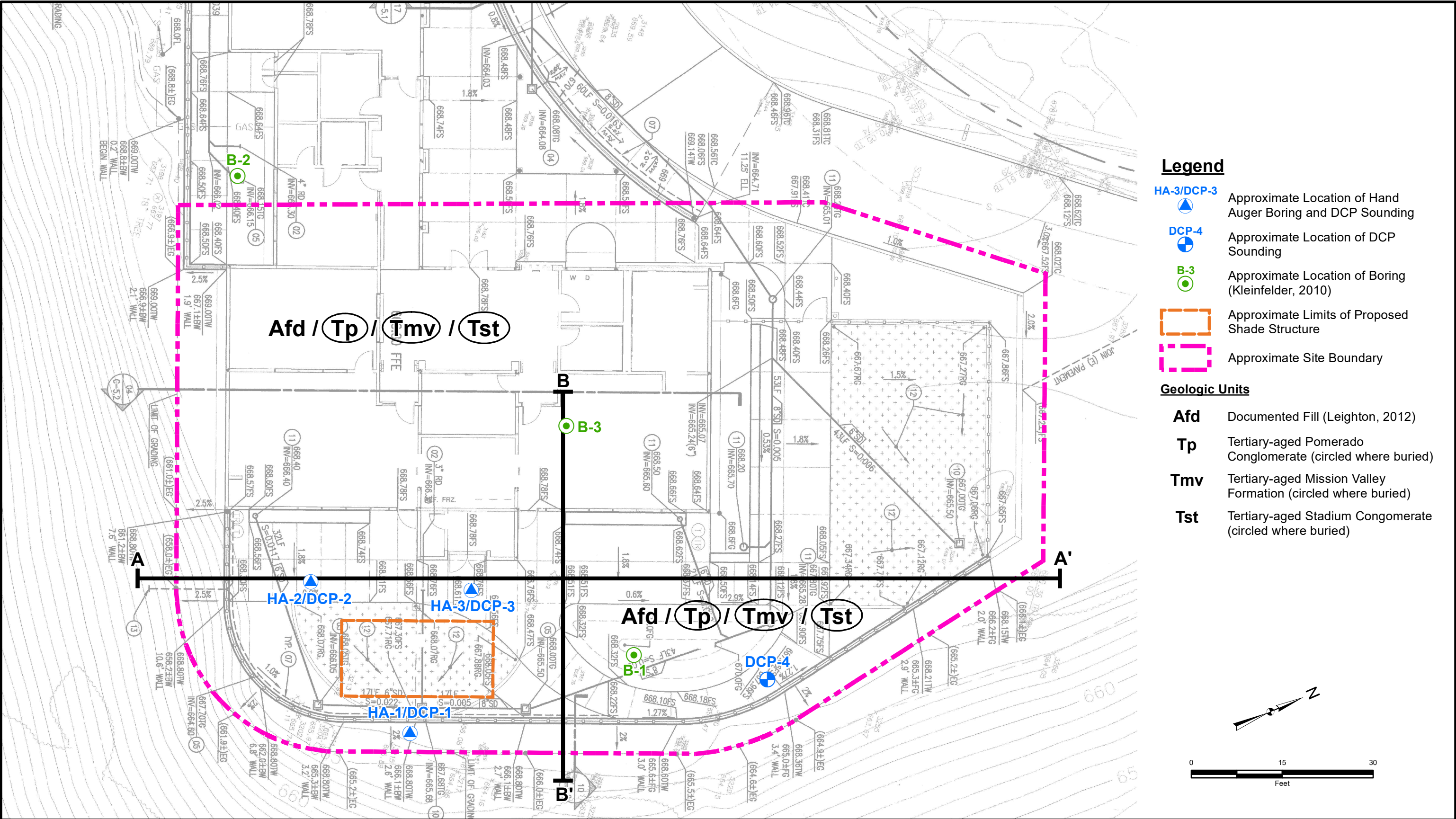
Hazardous materials services were not included as part of this study, nor are they within the scope of this report.

This report was prepared for the sole use of Grossmont Union High School District and their design team, for application to the design of the proposed Child Development Center Shade Structure Project in accordance with generally accepted California licensed geotechnical engineering practices at this time in California.

FIGURES



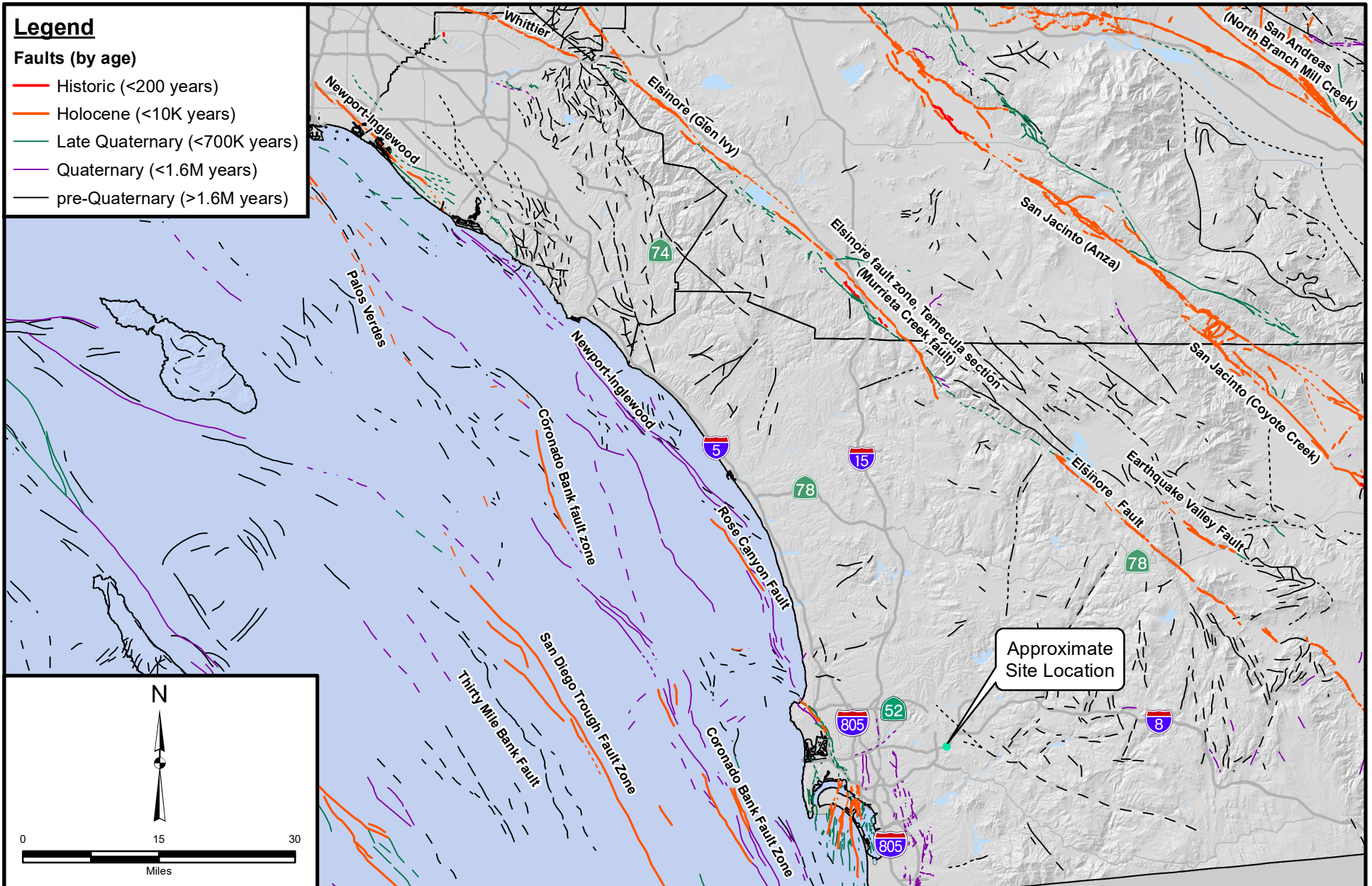
Project: 12429.034	Eng/Geol: NJT/RNB	SITE LOCATION MAP CDC Shade Structure Project Grossmont High School El Cajon, California	FIGURE 1
Scale: 1" = 2,000'	Date: June 2023		
Reference: Esri, HERE, Garmin, (c) OpenStreetMap contributors Author: (mmurphy)			



Legend

Faults (by age)

- Historic (<200 years)
- Holocene (<10K years)
- Late Quaternary (<700K years)
- Quaternary (<1.6M years)
- pre-Quaternary (>1.6M years)



Project: 12429.034 Eng/Geol: NJT/RNB

Scale: 1" = 15 miles Date: June 2023

Faults: Bryant, Bryant CGS 2010

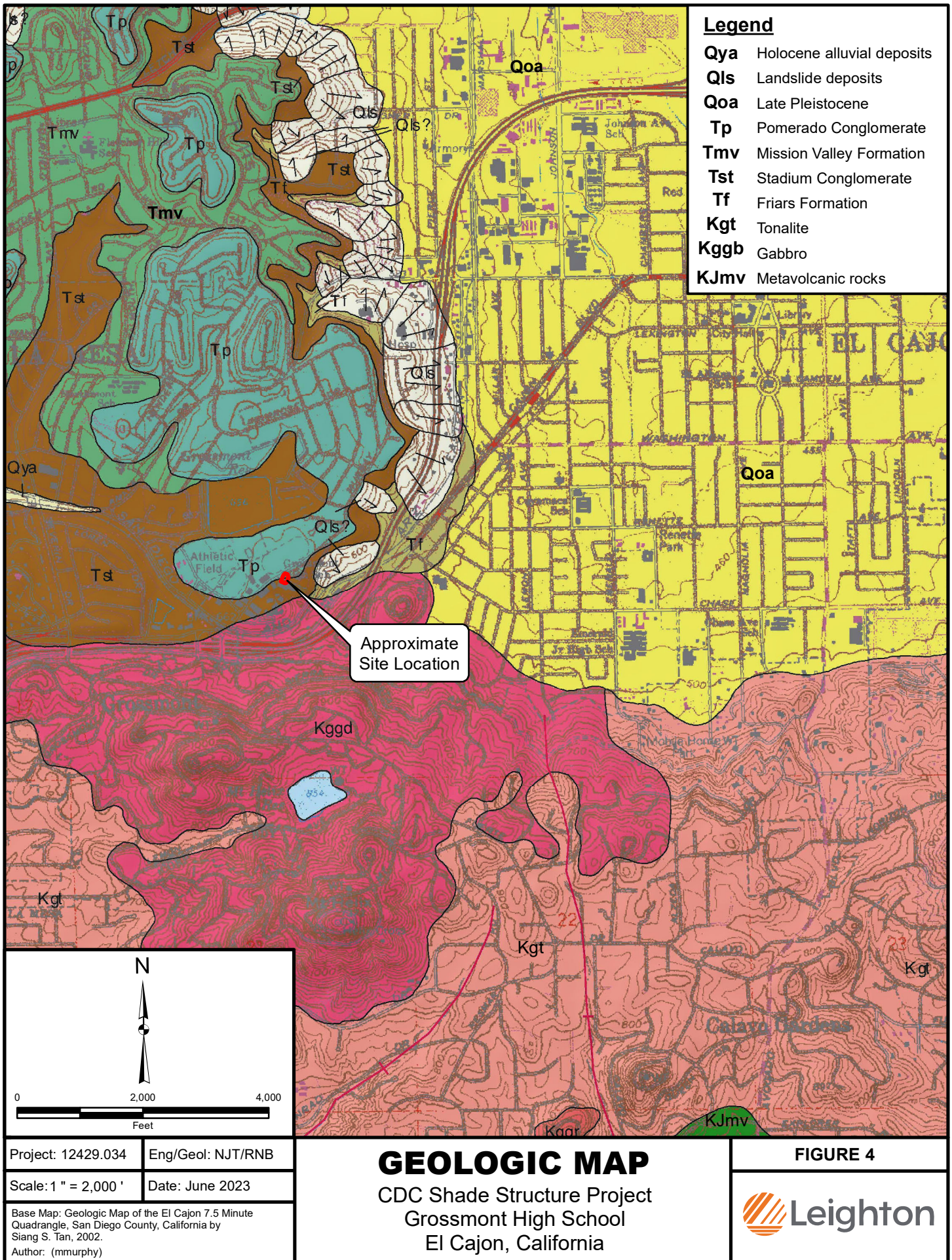
Author: Leighton Geomatics (mmurphy)

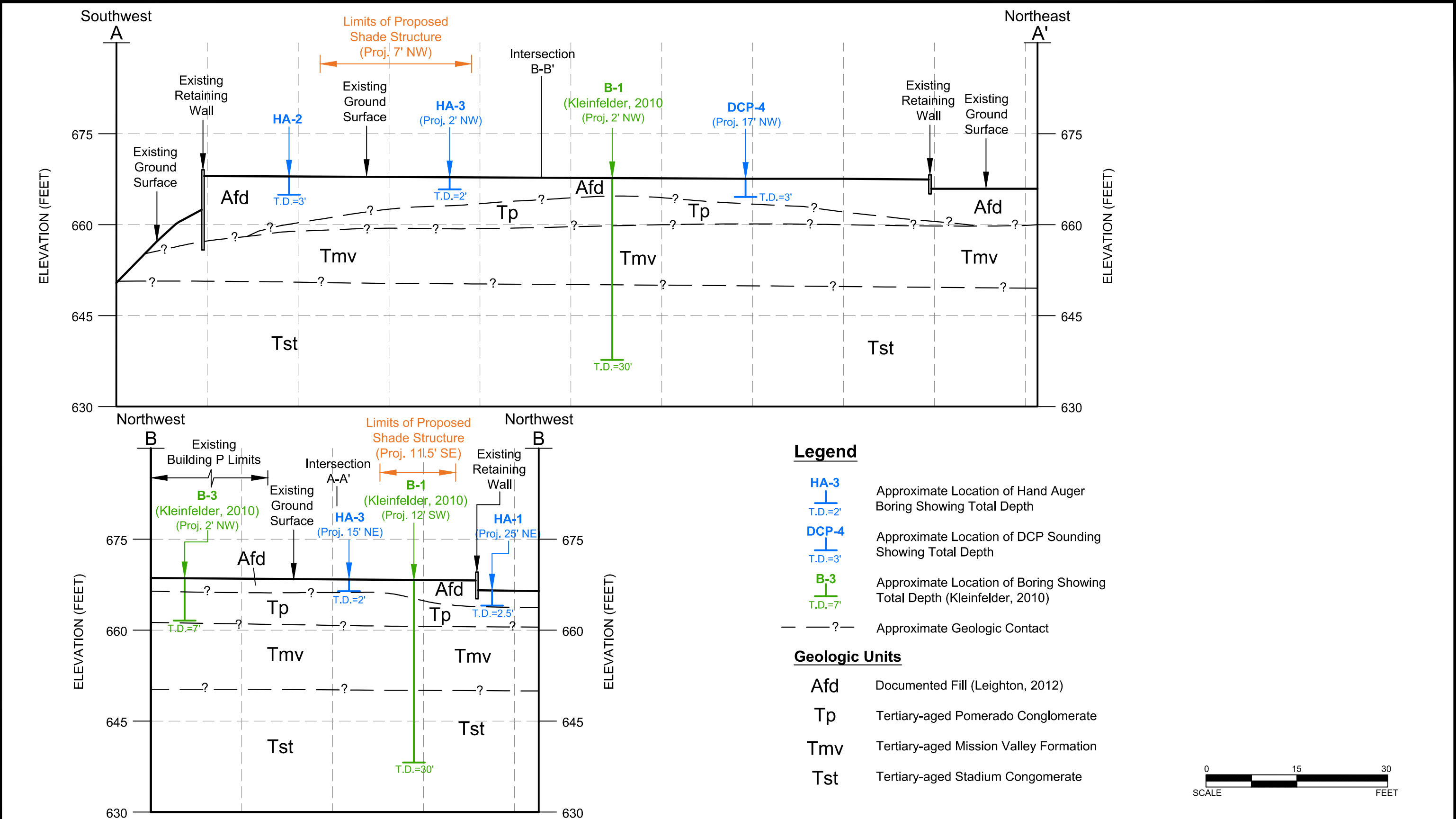
REGIONAL FAULT MAP

CDC Shade Structure Project
Grossmont High School
El Cajon, California

FIGURE 3







Proj: 12429.034	Eng/Geol: NJT/RNB
Scale: 1"=15'	Date: June 2023
Base Map:	

GEOLOGIC CROSS-SECTIONS A-A' & B-B'

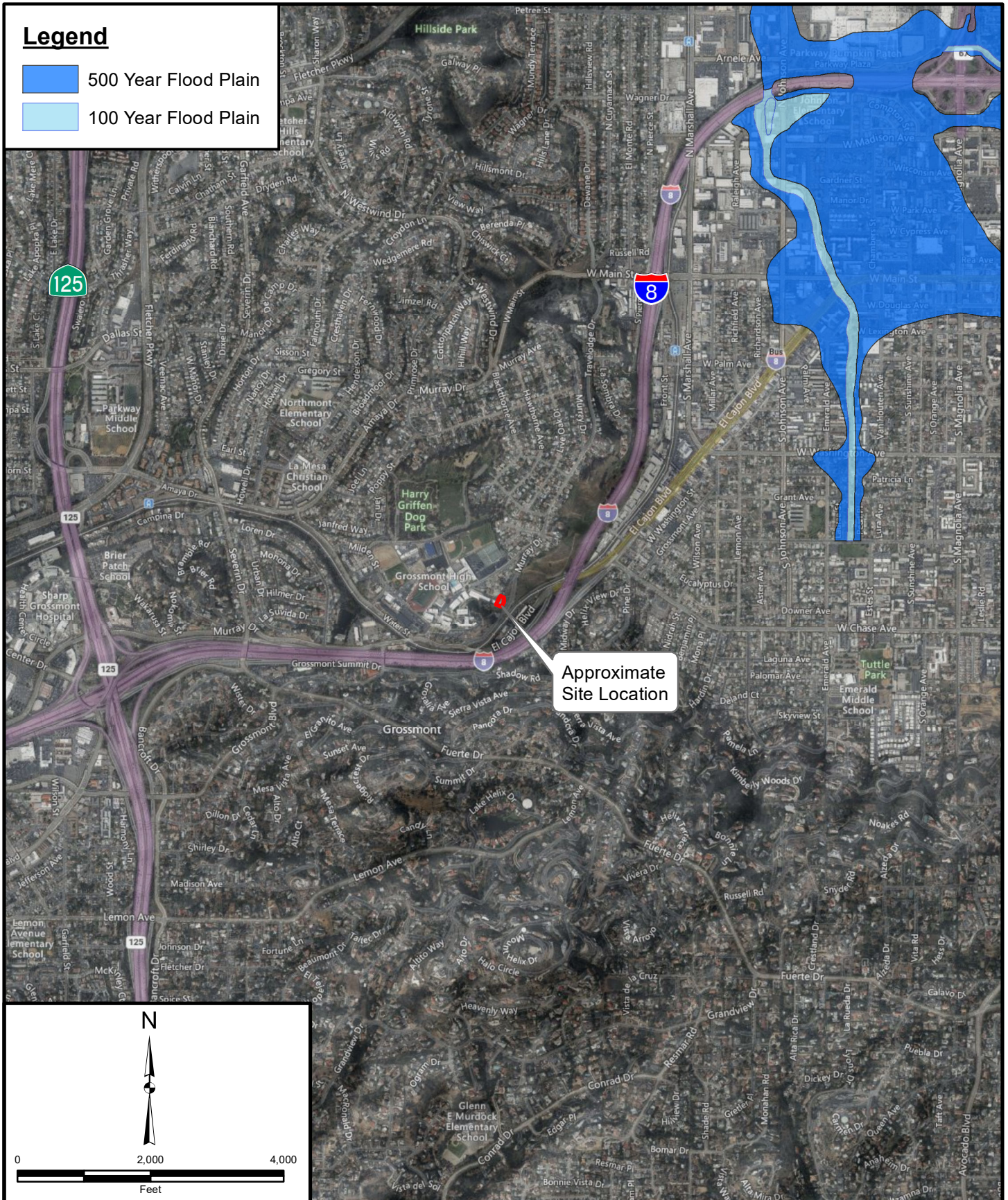
CDC Shade Structure Project
Grossmont High School
El Cajon, California

FIGURE 5



Legend

- 500 Year Flood Plain
- 100 Year Flood Plain



Approximate
Site Location

Project: 12429.034 Eng/Geol: NJT/RNB

Scale: 1" = 2,000' Date: June 2023

Reference: © 2023 Microsoft Corporation © 2023
Maxar ©CNES (2023) Distribution Airbus DS © 2023
TomTom

Reference: FEMA, DWR


FLOOD HAZARD MAP

CDC Shade Structure Project
Grossmont High School
El Cajon, California

FIGURE 6



Legend

 Grossmont Dam



Approximate
Site Location

Project: 12429.034 Eng/Geol: NJT/RNB

Scale: 1" = 2,000' Date: June 2023

Reference: © 2023 Microsoft Corporation © 2023
Maxar ©CNES (2023) Distribution Airbus DS © 2023
TomTom

Reference: CA DWR, FEMA

DAM INUNDATION MAP

CDC Shade Structure Project
Grossmont High School
El Cajon, California

FIGURE 7



APPENDIX A

References

APPENDIX A

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APPENDIX B
Exploration Logs

**Current Exploration Logs
(This Study)**

GEOTECHNICAL BORING LOG KEY

Date _____ Sheet 1 of 1
 Project _____ KEY TO BORING LOG GRAPHICS Project No. _____
 Drilling Co. _____ Type of Rig _____
 Hole Diameter _____ Drive Weight _____ Drop _____
 Elevation Top of Elevation _____ Location _____

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per Foot	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	DESCRIPTION	Type of Tests
	0	N S							Logged By _____ Sampled By _____	
									Asphaltic concrete.	
									Portland cement concrete.	
								CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay.	
								CH	Inorganic clay; high plasticity, fat clays.	
	5							OL	Organic clay; medium to plasticity, organic silts.	
								ML	Inorganic silt; clayey silt with low plasticity.	
								MH	Inorganic silt; diatomaceous fine sandy or silty soils; elastic silt.	
								ML-CL	Clayey silt to silty clay.	
								GW	Well-graded gravel; gravel-sand mixture, little or no fines.	
	10							GP	Poorly graded gravel; gravel-sand mixture, little or no fines.	
								GM	Silty gravel; gravel-sand-silt mixtures.	
								GC	Clayey gravel; gravel-sand-clay mixtures.	
								SW	Well-graded sand; gravelly sand, little or no fines.	
								SP	Poorly graded sand; gravelly sand, little or no fines.	
	15							SM	Silty sand; poorly graded sand-silt mixtures.	
								SC	Clayey sand; sand-clay mixtures.	
									Bedrock.	
	20			B-1					Ground water encountered at time of drilling.	
				C-1					Bulk Sample 1.	
				G-1					Core Sample.	
				R-1					Grab Sample.	
				SH-1					Modified California Sampler (3" O.D., 2.5 I.D.).	
				S-1					Shelby Tube Sampler (3" O.D.).	
	25			PUSH					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.).	
									Sampler Penetrates without Hammer Blow.	
	30									

SAMPLE TYPES:

S SPLIT SPOON
 R RING SAMPLE
 B BULK SAMPLE
 T TUBE SAMPLE

G GRAB SAMPLE
 SH SHELBY TUBE

TYPE OF TESTS:

DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 CN CONSOLIDATION
 CR CORROSION

SA SIEVE ANALYSIS
 AT ATTERBURG LIMITS
 EI EXPANSION INDEX
 RV R-VALUE



GEOTECHNICAL BORING LOG HA-1

Project No.	12429.034	Date Drilled	6-3-23
Project	CDC Shade Structure Project Grossmont High School	Logged By	CA
Drilling Co.	N/A	Hole Diameter	3.25"
Drilling Method	Hand Auger	Ground Elevation	665' msl
Location	See Figure 2	Sampled By	CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
		N S							<i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	
665	0			B-1 (0-2.5')				SM	<p>DOCUMENTED ARTIFICIAL FILL (Afd)</p> <p>@ 0'-2': Silty SAND with GRAVEL, medium dense, brown (7.5YR, 4/2), slightly moist, fine to medium SAND, fine to coarse gravel, trace cobble, trace rootlets, trace asphalt concrete debris.</p> <p>@ 2'-2.5': Increase in gravel noted.</p>	
									<p>Hand Auger Refusal on Gravel-Cobble @ 2.5 Feet (bgs)</p> <p>No Groundwater or Seepage Encountered</p> <p>Backfilled on 6/3/23</p>	
660	5									


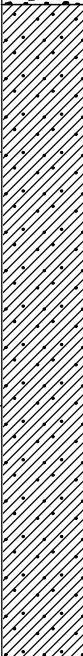
SAMPLE TYPES: B BULK SAMPLE C CORE SAMPLE G GRAB SAMPLE R RING SAMPLE S SPLIT SPOON SAMPLE T TUBE SAMPLE	TYPE OF TESTS: -200 % FINES PASSING AL ATTERBERG LIMITS CN CONSOLIDATION CO COLLAPSE CR CORROSION CU UNDRAINED TRIAXIAL	DS DIRECT SHEAR EI EXPANSION INDEX H HYDROMETER MD MAXIMUM DENSITY PP POCKET PENETROMETER RV R VALUE
		SA SIEVE ANALYSIS SE SAND EQUIVALENT SG SPECIFIC GRAVITY UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG HA-2

Project No. 12429.034
 Project CDC Shade Structure Project Grossmont High School
 Drilling Co. N/A
 Drilling Method Hand Auger
 Location See Figure 2

Date Drilled 6-3-23
 Logged By CA
 Hole Diameter 3.25"
 Ground Elevation 668.5' msl
 Sampled By CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0								5" of CONCRETE	
				B-1 (0.5-3')				SC	<p>DOCUMENTED ARTIFICIAL FILL (Afd)</p> <p>@ 0.5'-1.5': Clayey SAND with GRAVEL, medium dense, brown (7.5YR, 4/2), moist, fine to medium SAND, fine to coarse gravel, trace cobble.</p> <p>@ 1.5': Becomes loose.</p>	EI, CR
665									<p>Hand Auger Refusal on Gravel-Cobble @ 3 Feet (bgs)</p> <p>No Groundwater or Seepage Encountered</p> <p>Backfilled on 6/3/23</p>	
5										

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG HA-3

Project No. 12429.034
 Project CDC Shade Structure Project Grossmont High School
 Drilling Co. N/A
 Drilling Method Hand Auger
 Location See Figure 2

Date Drilled 6-3-23
 Logged By CA
 Hole Diameter 3.25"
 Ground Elevation 668.5' msl
 Sampled By CA

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	
									5" of CONCRETE	
				B-1 (0.5-2')				SC	DOCUMENTED ARTIFICIAL FILL (Afd) @ 0.5'-1.5': Clayey SAND with GRAVEL, loose to medium dense, brown (7.5YR, 4/2), moist, fine to medium SAND, fine to coarse gravel, trace cobble. @ 1.5': Becomes loose.	MD, DS
665									Hand Auger Refusal on Gravel-Cobble @ 2 Feet (bgs) No Groundwater or Seepage Encountered Backfilled on 6/3/23	
5										

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 12429.034
DATE STARTED: 06-03-2023
DATE COMPLETED: 06-03-2023

HOLE #: DCP-1
CREW: CA
PROJECT: CDC Shade Structure Project Grossmont High School
ADDRESS: 1100 Murray Drive, El Cajon, California
LOCATION: See Figure 2

SURFACE ELEVATION: 665' (msl)
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
-	17	75.5	21	MEDIUM DENSE	VERY STIFF
-	15	66.6	19	MEDIUM DENSE	VERY STIFF
- 1 ft	12	53.3	15	MEDIUM DENSE	STIFF
-	13	57.7	16	MEDIUM DENSE	VERY STIFF
-	12	53.3	15	MEDIUM DENSE	STIFF
- 2 ft	25	111.0	25+	DENSE	HARD
-	50	222.0	25+	VERY DENSE	HARD
-	60	266.4	25+	VERY DENSE	HARD
- 3 ft	55	244.2	25+	VERY DENSE	HARD
- 1 m						
-						
- 4 ft						
-						
-						
- 5 ft						
-						
-						
- 6 ft						
-						
- 2 m						
-						
- 7 ft						
-						
-						
- 8 ft						
-						
-						
- 9 ft						
-						
- 3 m						
- 10 ft						
-						
-						
- 11 ft						
-						
-						
- 12 ft						
-						
- 4 m						
- 13 ft						

WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 12429.034
DATE STARTED: 06-03-2023
DATE COMPLETED: 06-03-2023

HOLE #: DCP-2
CREW: CA
PROJECT: CDC Shade Structure Project Grossmont High School
ADDRESS: 1100 Murray Drive, El Cajon, California
LOCATION: See Figure 2

SURFACE ELEVATION: 668.5' (msl)
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
-	12	53.3	15	MEDIUM DENSE	STIFF
-	11	48.8	13	MEDIUM DENSE	STIFF
- 1 ft	8	35.5	10	LOOSE	STIFF
-	10	44.4	12	MEDIUM DENSE	STIFF
-	8	35.5	10	LOOSE	STIFF
- 2 ft	8	35.5	10	LOOSE	STIFF
-	8	35.5	10	LOOSE	STIFF
-	14	62.2	17	MEDIUM DENSE	VERY STIFF
- 3 ft	15	66.6	19	MEDIUM DENSE	VERY STIFF
- 1 m	17	75.5	21	MEDIUM DENSE	VERY STIFF
-	14	54.0	15	MEDIUM DENSE	STIFF
- 4 ft	16	61.8	17	MEDIUM DENSE	VERY STIFF
-	14	54.0	15	MEDIUM DENSE	STIFF
-	19	73.3	20	MEDIUM DENSE	VERY STIFF
- 5 ft	14	54.0	15	MEDIUM DENSE	STIFF
-	13	50.2	14	MEDIUM DENSE	STIFF
-	10	38.6	11	MEDIUM DENSE	STIFF
- 6 ft	16	61.8	17	MEDIUM DENSE	VERY STIFF
-	30	115.8	25+	DENSE	HARD
- 2 m	31	119.7	25+	DENSE	HARD
- 7 ft	52	177.8	25+	DENSE	HARD
-	24	82.1	23	MEDIUM DENSE	VERY STIFF
-	20	68.4	19	MEDIUM DENSE	VERY STIFF
- 8 ft	22	75.2	21	MEDIUM DENSE	VERY STIFF
-	50	171.0	25+	DENSE	HARD
- 9 ft						
- 3 m 10 ft						
- 11 ft						
- 12 ft						
- 4 m 13 ft						

WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 12429.034

DATE STARTED: 06-03-2023

DATE COMPLETED: 06-03-2023

HOLE #: DCP-3

CREW: CA

PROJECT: CDC Shade Structure Project Grossmont High School

ADDRESS: 1100 Murray Drive, El Cajon, California

LOCATION: See Figure 2

SURFACE ELEVATION: 668.5' (msl)

WATER ON COMPLETION:	N/A
----------------------	-----

HAMMER WEIGHT: 35 lbs.

CONE AREA: 10 sq. cm

DEPTH		BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
						NON-COHESIVE	COHESIVE
1 ft		8	35.5	10	LOOSE	STIFF
		18	79.9	22	MEDIUM DENSE	VERY STIFF
		50	222.0	25+	VERY DENSE	HARD
2 ft							
3 ft							
4 ft							
5 ft							
6 ft							
7 ft							
8 ft							
9 ft							
10 ft							
11 ft							
12 ft							
13 ft							

WILDCAT DYNAMIC CONE LOG

Page 1 of 1

Leighton

PROJECT NUMBER: 12429.034
DATE STARTED: 06-03-2023
DATE COMPLETED: 06-03-2023

HOLE #: DCP-4
CREW: CA
PROJECT: CDC Shade Structure Project Grossmont High School
ADDRESS: 1100 Murray Drive, El Cajon, California
LOCATION: See Figure 2

SURFACE ELEVATION: 668' (msl)
WATER ON COMPLETION: N/A
HAMMER WEIGHT: 35 lbs.
CONE AREA: 10 sq. cm

DEPTH	BLOWS PER 10 cm	RESISTANCE Kg/cm ²	GRAPH OF CONE RESISTANCE 0 50 100 150	N'	TESTED CONSISTENCY	
					NON-COHESIVE	COHESIVE
-	7	31.1	8	LOOSE	MEDIUM STIFF
-	8	35.5	10	LOOSE	STIFF
- 1 ft	6	26.6	7	LOOSE	MEDIUM STIFF
-	4	17.8	5	LOOSE	MEDIUM STIFF
-	5	22.2	6	LOOSE	MEDIUM STIFF
- 2 ft	11	48.8	13	MEDIUM DENSE	STIFF
-	17	75.5	21	MEDIUM DENSE	VERY STIFF
-	15	66.6	19	MEDIUM DENSE	VERY STIFF
- 3 ft	50	222.0	25+	VERY DENSE	HARD
- 1 m						
- 4 ft						
-						
- 5 ft						
-						
- 6 ft						
- 2 m						
- 7 ft						
-						
- 8 ft						
-						
- 9 ft						
- 3 m						
- 10 ft						
-						
- 11 ft						
-						
- 12 ft						
-						
- 4 m						
- 13 ft						

**Previous Exploration Logs
(Kleinfelder, 2010)**

DATE DRILLED: 4/5/2010
 DRILLING COMPANY: TRI-COUNTY DRILLING
 DRILLING METHOD: CME-75, 140lb Auto Hammer, 30" Drop,
 Hollow Stem Auger (HSA) & Air Rotary
 HOLE DIAMETER: 8" HSA & 7" Air Hammer

WATER DEPTH: Not Encountered
 DATE OBSERVED: 4/5/2010
 GROUND ELEVATION: ± 667.5 Feet MSL*
 LOGGED BY: EK
 REVIEWED BY: DH

ELEVATION (feet)	DEPTH (feet)	BULK SAMPLES	DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
							3 inches of Asphalt Concrete			
							ARTIFICIAL FILL (Qaf):			
					1		Silty SAND (SM), brown, moist, loose to medium dense, fine to coarse grained sand, non-plastic, trace gravel and cobble			
-665				**50/6"	2					
							POMERADO CONGLOMERATE (Tp):			
							Drill action indicates presence of gravel/cobble at approximately 3 to 3-1/2 ft bgs			
	5			**50/6"	NR		No soil matrix recovered from the sampler, broken pieces of gravel/cobble			Switched to Air Rotary drilling method
-660							MISSION VALLEY FORMATION (Tmv):			
							Sandy SILT (ML), light brownish gray, moist, very stiff to hard, fine to medium grained sand, trace clay, Sandy SILTSTONE excavates as ML			
	10			64	3				11.9	
-655							Fat CLAY (CH), light brownish gray, moist, hard, trace fine grained sand, high plasticity, CLAYSTONE excavates as CH			
	15			37	4					
-650							STADIUM CONGLOMERATE (Tst):			
							Sandy SILT with Gravel (ML), light brownish gray, moist, hard, fine grained sand, fine to coarse grained gravel, abundant gravel/cobble			




PROJECT NO. 110515

CHILD DEVELOPMENT BUILDING
 PHASE 1A IMPROVEMENTS
 GROSSMONT HIGH SCHOOL
 EL CAJON, CALIFORNIA
 LOG OF BORING B-1

PLATE

A3

ELEVATION (feet)	DEPTH (feet)	SAMPLES		BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION (Continued From Previous Page)	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
		BULK	DRIVEN							
				**50/6"	5					
645										
	25			**50/5"	6		Drill action indicates presence of gravel/cobble at approximately 24 to 30 ft bgs			
640										
	30			**50/2"	NR		* Elevations obtained from topography shown on "Phase 1A Grading and Drainage Plan-C3.1" dated 2/9/10, prepared by LPA Architecture. ** Blowcount may not be representative due to gravel/cobble content. -Bottom of boring at approximately 30 ft bgs. -No groundwater observed. Caving observed at approximately 7 ft bgs when augers were withdrawn. -Boring backfilled with soil cuttings, bentonite chips, and patched at the surface with concrete on 4/5/10.			
635										
	35									
630										
	40									
625										



KLEINFELDER
Bright People. Right Solutions.

**CHILD DEVELOPMENT BUILDING
PHASE 1A IMPROVEMENTS
GROSSMONT HIGH SCHOOL
EL CAJON, CALIFORNIA**

LOG OF BORING B-1

PLATE

A4

PROJECT NO. 110515

DATE DRILLED: 4/5/2010
 DRILLING COMPANY: TRI-COUNTY DRILLING
 DRILLING METHOD: CME-75, 140lb Auto Hammer, 30" Drop,
 Hollow Stem Auger (HSA)
 HOLE DIAMETER: 8" HSA

WATER DEPTH: Not Encountered
 DATE OBSERVED: 4/5/2010
 GROUND ELEVATION: ± 670 Feet MSL*
 LOGGED BY: EK
 REVIEWED BY: DH

ELEVATION (feet)	DEPTH (feet)	SAMPLES BULK DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
						3 inches of Asphalt Concrete			
						ARTIFICIAL FILL (Qaf):			
				1		Silty SAND (SM), brown to light brown, moist, loose to medium dense, fine to coarse grained sand, trace clay, non-plastic, 2-inch broken AC piece	96	7.6	
			16	2					
						trace fine grained gravel, 2-inch clay lumps, pieces of plastic and AC			
			16	3					
-665	5					Drill action indicates presence of gravel/cobble at approximately 7.5 ft bgs			
						MISSION VALLEY FORMATION (Tmv):			
						Sandy SILT (ML), light brownish gray, moist, very stiff, fine grained sand, trace clay, piece of broken rock in sampler shoe, Sandy SILTSTONE excavates as ML		33.1	
-660	10	**50/3"		4					
				5					
						STADIUM CONGLOMERATE (Tst):			
-655	15	**50/4"	NR			Drill action and cuttings indicate presence of gravel/cobble at approximately 13 ft bgs			
						No soil matrix recovered from the sampler, broken pieces of gravel/cobble			
						* Elevations obtained from topography shown on "Phase 1A Grading and Drainage Plan-C3.1" dated 2/9/10, prepared by LPA Architecture. ** Blowcount may not be representative due to gravel/cobble content.			
-650	20					-Auger refusal at approximately 17 ft bgs on gravel/cobble. -No groundwater or caving observed. -Boring backfilled with soil cuttings, bentonite chips, and patched at the surface with asphalt on 4/5/10.			



PROJECT NO. 110515

CHILD DEVELOPMENT BUILDING
 PHASE 1A IMPROVEMENTS
 GROSSMONT HIGH SCHOOL
 EL CAJON, CALIFORNIA
 LOG OF BORING B-2

PLATE

A5

DATE DRILLED: 4/5/2010
 DRILLING COMPANY: TRI-COUNTY DRILLING
 DRILLING METHOD: CME-75, 140lb Auto Hammer, 30" Drop,
 Hollow Stem Auger (HSA)
 HOLE DIAMETER: 8" HSA

WATER DEPTH: Not Encountered
 DATE OBSERVED: 4/5/2010
 GROUND ELEVATION: ± 667.8 Feet MSL*
 LOGGED BY: EK
 REVIEWED BY: DH

ELEVATION (feet)	DEPTH (feet)	BULK SAMPLES DRIVEN	BLOW COUNTS (blows/foot)	SAMPLE NUMBER	GRAPHIC LOG	SOIL DESCRIPTION AND CLASSIFICATION	DRY UNIT WEIGHT (pcf)	MOISTURE CONTENT (%)	COMMENTS/ ADDITIONAL TESTS
667	1	X				3 inches of Asphalt Concrete			
666	2	X	**50/3"	1 NR		ARTIFICIAL FILL (Qaf): Silty SAND (SM), brown to light brown, moist, fine to coarse grained sand, non-plastic, trace gravel and cobble			
665	3	X				POMERADO CONGLOMERATE (Tp): Drill action indicates presence of gravel/cobble at approximately 2 to 7 ft bgs			
664	4	X	**50/2"	NR		No soil matrix recovered from the sampler, broken pieces of gravel/cobble			
663	5	X		2					
662	6	X	**50/4"	NR					
661	7								
660	8								
659	9					* Elevations obtained from topography shown on "Phase 1A Grading and Drainage Plan-C3.1" dated 2/9/10, prepared by LPA Architecture. ** Blowcount may not be representative due to gravel/cobble content.			
658	10					-Auger refusal at approximately 7 ft bgs on gravel/cobble. -No groundwater or caving observed.			
657	11					-Boring backfilled with soil cuttings, bentonite chips, and patched at the surface with asphalt on 4/5/10.			
656	12								
655	13								
654	14								
653									



PROJECT NO. 110515

CHILD DEVELOPMENT BUILDING
 PHASE 1A IMPROVEMENTS
 GROSSMONT HIGH SCHOOL
 EL CAJON, CALIFORNIA
 LOG OF BORING B-3

PLATE

A6

APPENDIX C

Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Expansion Index Test: The expansion potential of a selected material was evaluated by the Expansion Index Test, ASTM Test Method D 4829. The specimen was molded under a given compactive energy to approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimen was then loaded to an equivalent 144 psf surcharge and was inundated with water until volumetric equilibrium was reached. The result of this test is presented in the table below:

Sample Location	Description	Expansion Index	Expansion Potential
HA-2 @ 0.5-3 Feet	Clayey Sand with Gravel	16	Very Low

Soluble Sulfates: The soluble content of a selected sample was determined by standard geochemical methods. The test result is presented in the table below:

Sample Location	Sulfate Content (ppm)
HA-2 @ 0.5-3 Feet	<150

Chloride Content: Chloride content was tested in accordance with DOT Test Method No. 422. The result is presented below:

Sample Location	Chloride Content, ppm
HA-2 @ 0.5-3 Feet	80

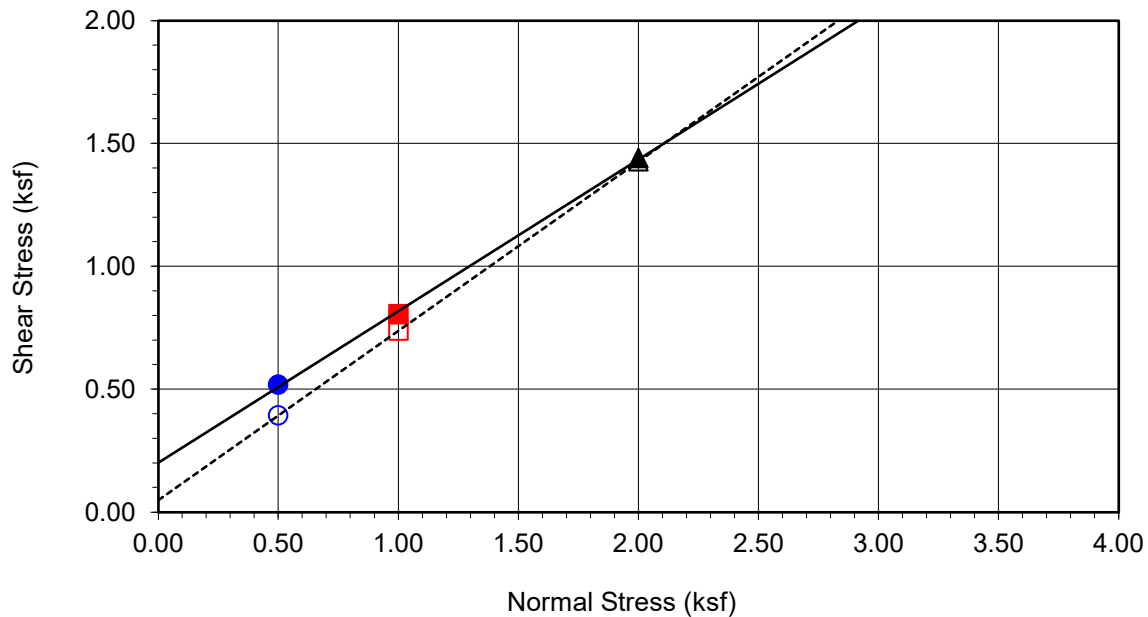
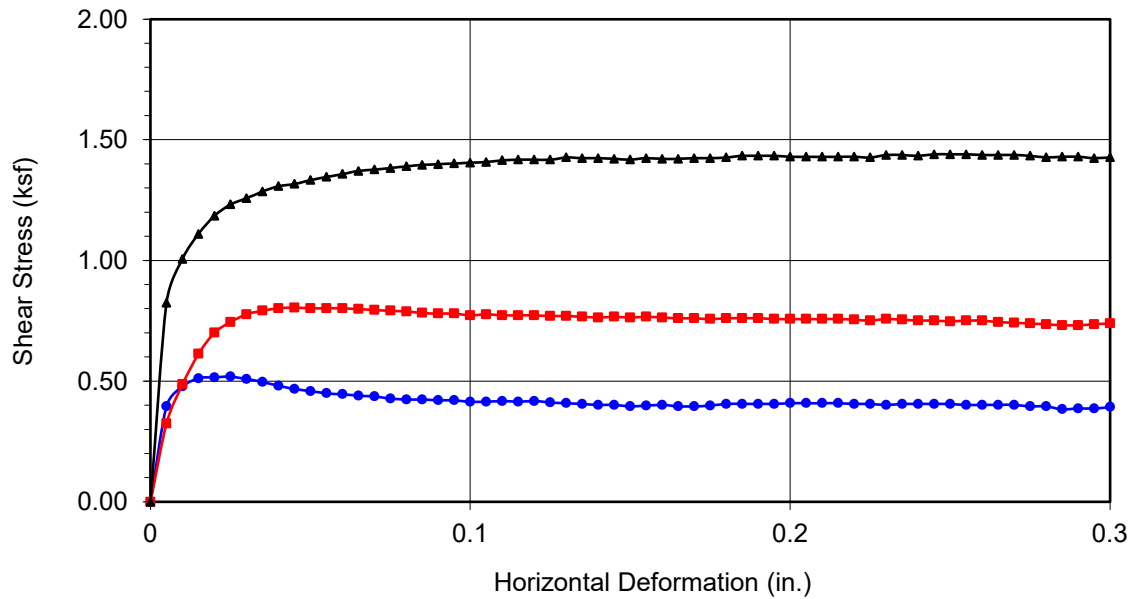
Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with California Test Method 643. The results are presented in the table below:

Sample Location	pH	Minimum Resistivity (ohms-cm)
HA-2 @ 0.5-3 Feet	7.42	2,200

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D 1557. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HA-3 @ 0.5-2 Feet	Clayey Sand with Gravel	128.0	9.0

Direct Shear Test: A direct shear test was performed on a selected remolded sample which was soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box and reloading of the sample, the pore pressures set up in the sample (due to the transfer) were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The sample was tested under various normal loads utilizing a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less 0.05 inches per minute. The test result is presented on the attached figure.



Boring No.	HA-3	
Sample No.	B-1	
Depth (ft)	0.5-2.0	
<u>Sample Type:</u> 90% Remold		
<u>Soil Identification:</u> Dark brown clayey sand with gravel (SC)g		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	202	32
Ultimate	49	35

Normal Stress (kip/ft ²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft ²)	● 0.519	■ 0.805	▲ 1.440
Shear Stress @ End of Test (ksf)	○ 0.393	□ 0.739	△ 1.427
Deformation Rate (in./min.)	0.0025	0.0025	0.0025
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	8.80	8.80	8.80
Dry Density (pcf)	116.1	116.1	116.2
Saturation (%)	52.5	52.5	52.7
Soil Height Before Shearing (in.)	1.0037	0.9925	0.9872
Final Moisture Content (%)	16.4	15.9	14.9



DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 12429.034

Grossmont High School CDC

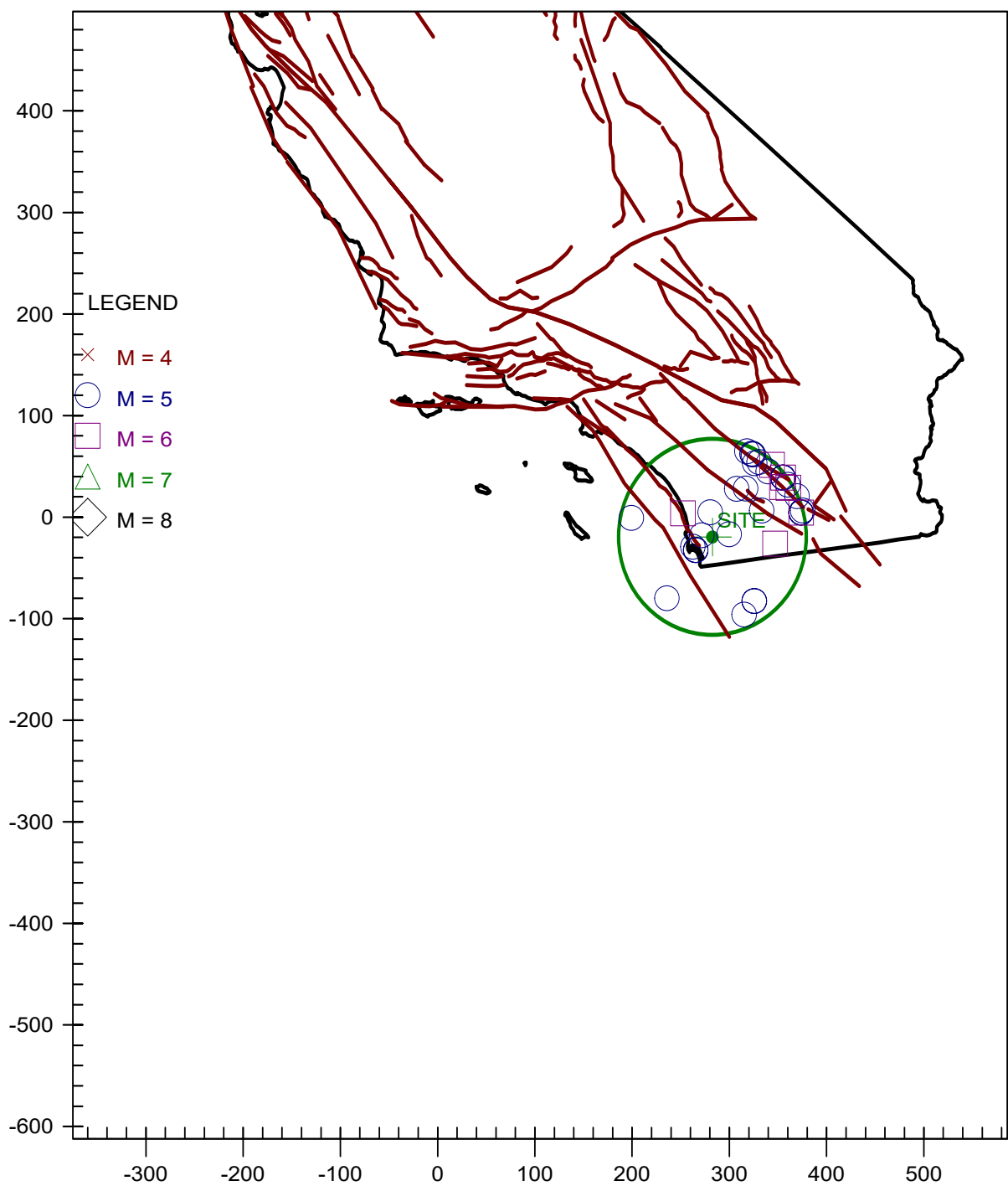
06-23

APPENDIX D

Seismic Hazard Analysis

EARTHQUAKE EPICENTER MAP

GHS CDC Shade Structure



*
* E Q S E A R C H *
*
* Version 3.00 *
*

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 12429.034

DATE: 6-22-2023

JOB NAME: GHS CDC Shade Structure Project

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 5.00

MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 32.7814

SITE LONGITUDE: 116.9846

SEARCH DATES:

START DATE: 1800

END DATE: 2023

SEARCH RADIUS:

60.0 mi

96.6 km

ATTENUATION RELATION: 2) Boore et al. (1997) Horiz. - NEHRP C (520)

UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0

ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]

SCOND: 0 Depth Source: A

Basement Depth: 5.00 km Campbell SSR: Campbell SHR:

COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 0.0

EARTHQUAKE SEARCH RESULTS

FILE	LAT.	LONG.	DATE	TIME (UTC)	DEPTH	QUAKE	SITE	SITE	APPROX.
CODE	NORTH	WEST		H M Sec	(km)	MAG.	ACC.	MM	DISTANCE
							g	INT.	mi [km]
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.088	VII	6.8 (11.0)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.094	VII	10.8 (17.4)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.056	VI	13.2 (21.3)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.056	VI	13.2 (21.3)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.056	VI	13.2 (21.3)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.088	VII	13.7 (22.1)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.051	VI	15.1 (24.3)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.080	VII	23.7 (38.1)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.028	V	33.3 (53.5)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.028	V	35.4 (56.9)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	36.5 (58.7)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.059	VI	40.1 (64.6)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.021	IV	47.4 (76.3)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.022	IV	47.5 (76.4)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.031	V	47.5 (76.4)
DMG	32.0830	116.6670	11/25/1934	818 0.0	0.0	5.00	0.020	IV	51.6 (83.1)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.026	V	52.6 (84.6)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.023	IV	53.0 (85.3)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.030	V	53.6 (86.2)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.038	V	53.8 (86.7)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.021	IV	54.7 (88.1)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.024	V	56.7 (91.2)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.021	IV	56.9 (91.5)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.018	IV	57.0 (91.7)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.039	V	57.0 (91.8)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.019	IV	57.1 (91.8)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.026	V	57.8 (92.9)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.019	IV	57.9 (93.2)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.024	IV	57.9 (93.2)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.018	IV	57.9 (93.2)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.034	V	57.9 (93.2)

EARTHQUAKE SEARCH RESULTS

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	ACC. g	SITE MM INT.	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.036	V		58.2 (93.7)
DMG	32.9670	116.0000	10/21/1942	162519.0	0.0	5.00	0.018	IV		58.5 (94.2)
DMG	32.9670	116.0000	10/22/1942	181326.0	0.0	5.00	0.018	IV		58.5 (94.2)
DMG	32.9670	116.0000	10/21/1942	162654.0	0.0	5.00	0.018	IV		58.5 (94.2)
DMG	32.9670	116.0000	10/21/1942	162213.0	0.0	6.50	0.040	V		58.5 (94.2)
DMG	33.1130	116.0370	04/09/1968	3 353.5	5.0	5.20	0.020	IV		59.5 (95.7)
DMG	32.9830	115.9830	05/23/1942	154729.0	0.0	5.00	0.018	IV		59.7 (96.1)

-END OF SEARCH- 38 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2023

LENGTH OF SEARCH TIME: 224 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 6.8 MILES (11.0 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.7

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.094 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 0.875

b-value= 0.375

beta-value= 0.863

 TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake	Number of Times	Cumulative
Magnitude	Exceeded	No. / Year
-----+-----+-----		
4.0	38	0.17040
4.5	38	0.17040
5.0	38	0.17040
5.5	15	0.06726
6.0	7	0.03139
6.5	3	0.01345

APPENDIX E

General Earthwork and Grading Specifications

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant

prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

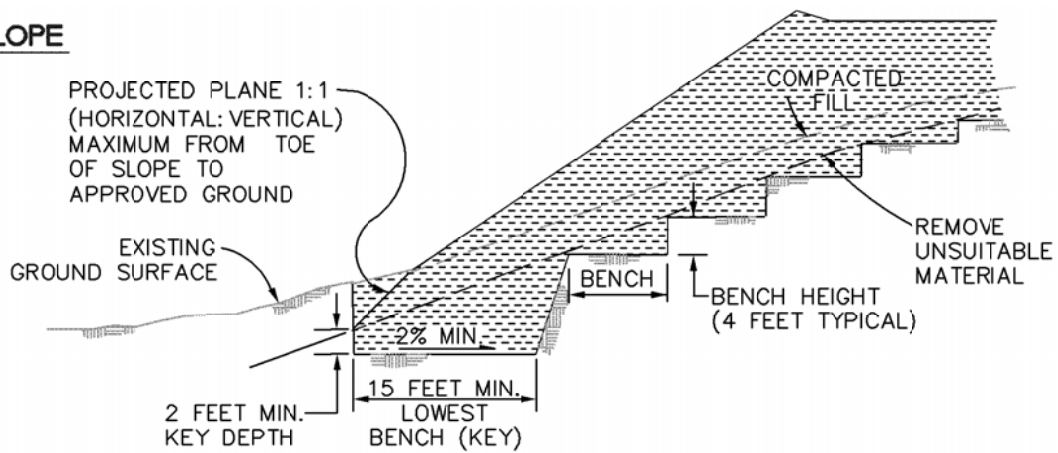
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

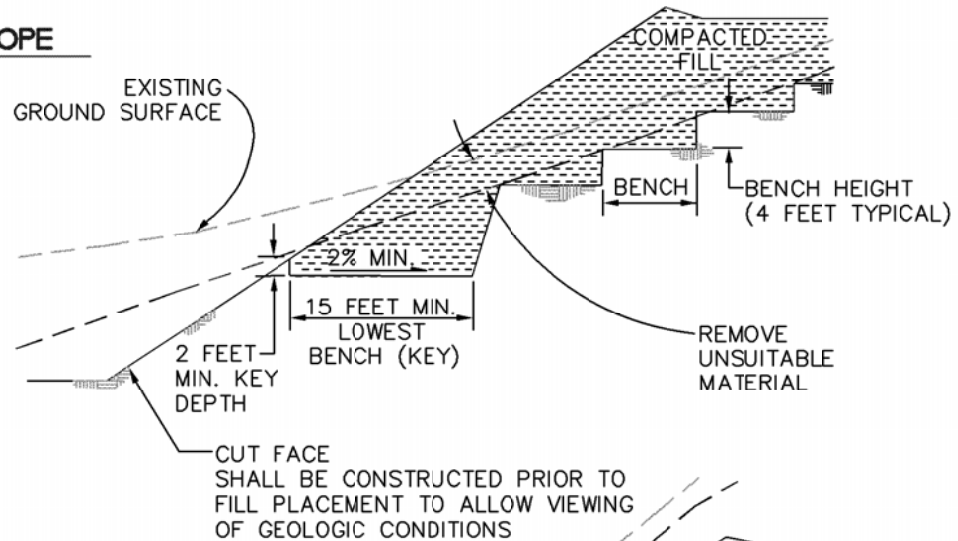
7.4 Observation and Testing

The densification of the bedding around the conduits shall be observed by the Geotechnical Consultant.

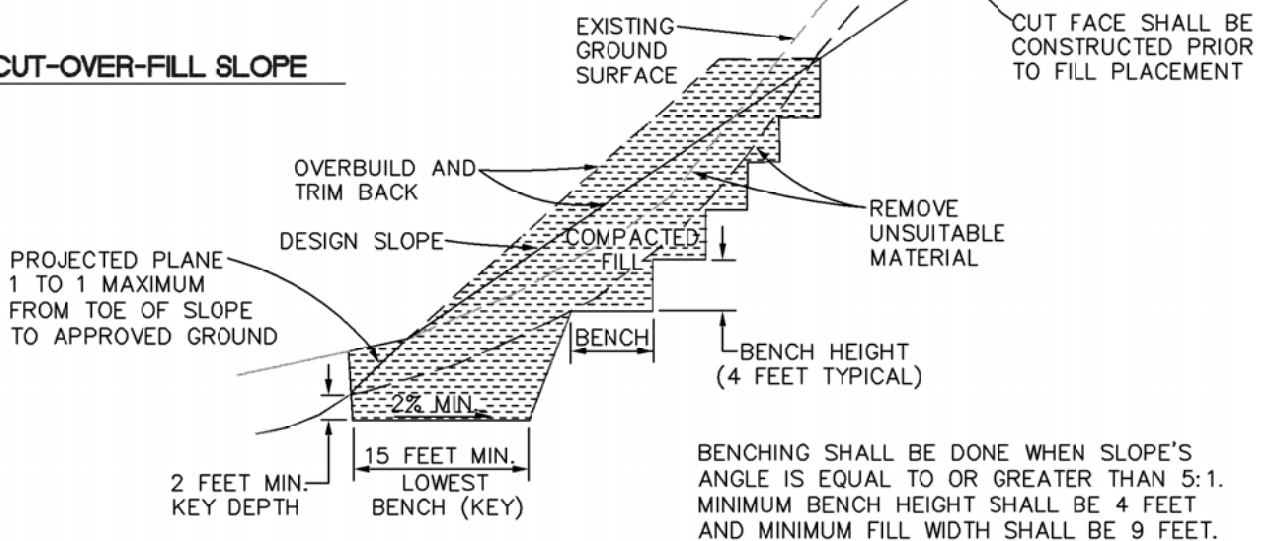
FILL SLOPE



FILL-OVER-CUT SLOPE



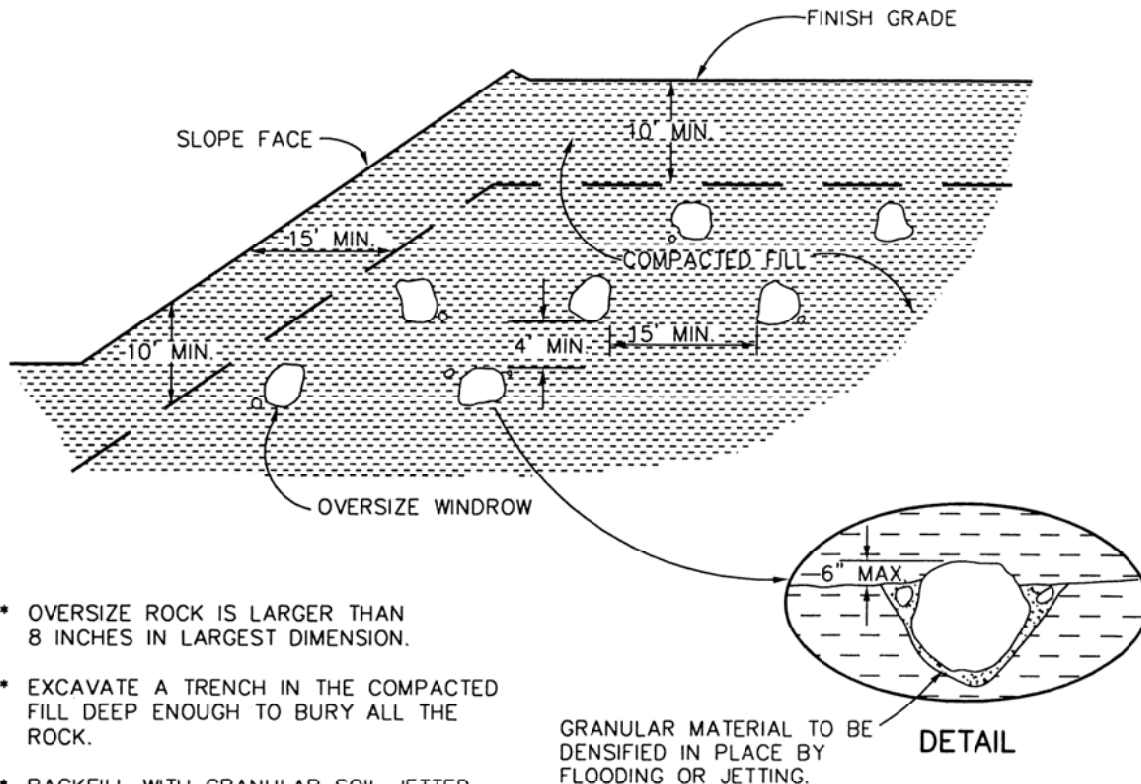
CUT-OVER-FILL SLOPE



KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

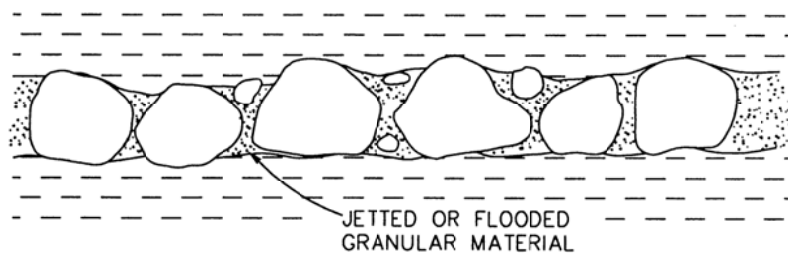




- * OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- * EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- * BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- * DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- * WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.

GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

DETAIL

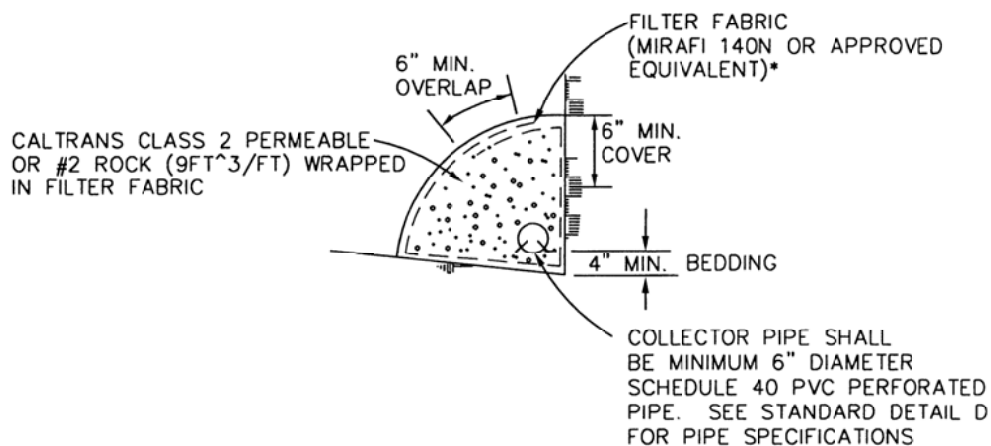
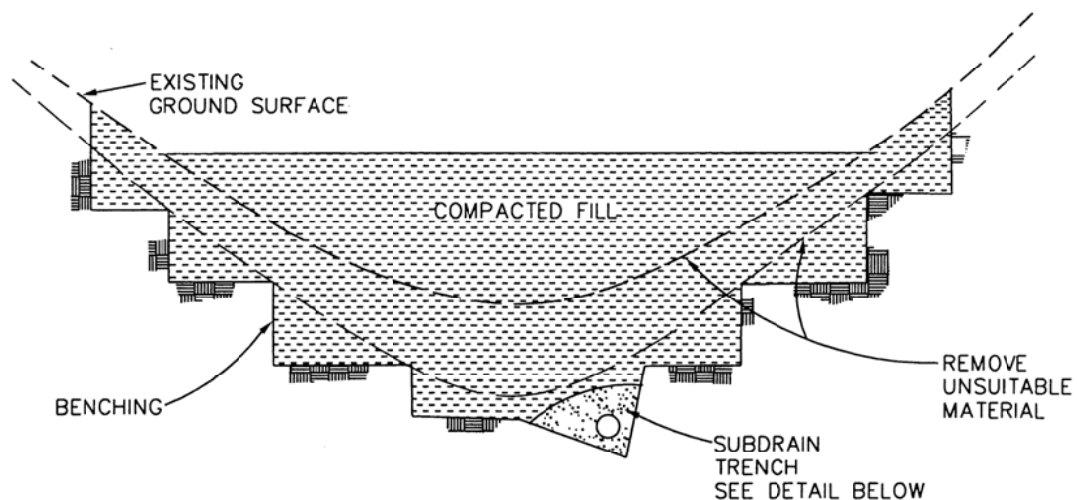


TYPICAL PROFILE ALONG WINDROW

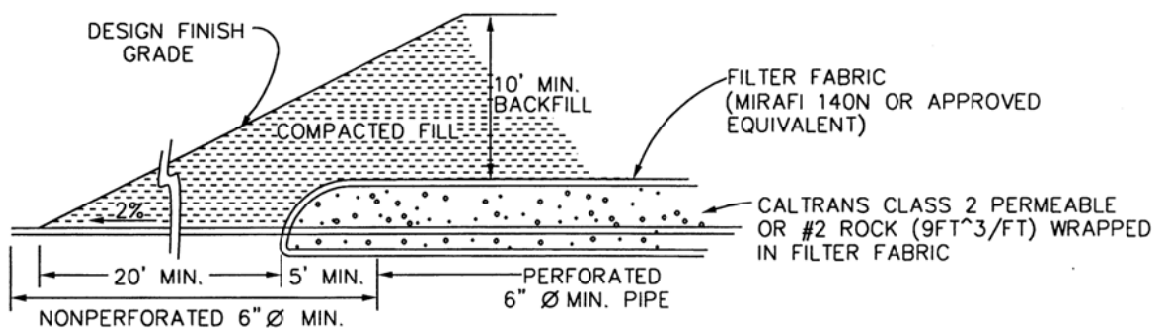
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





SUBDRAIN DETAIL

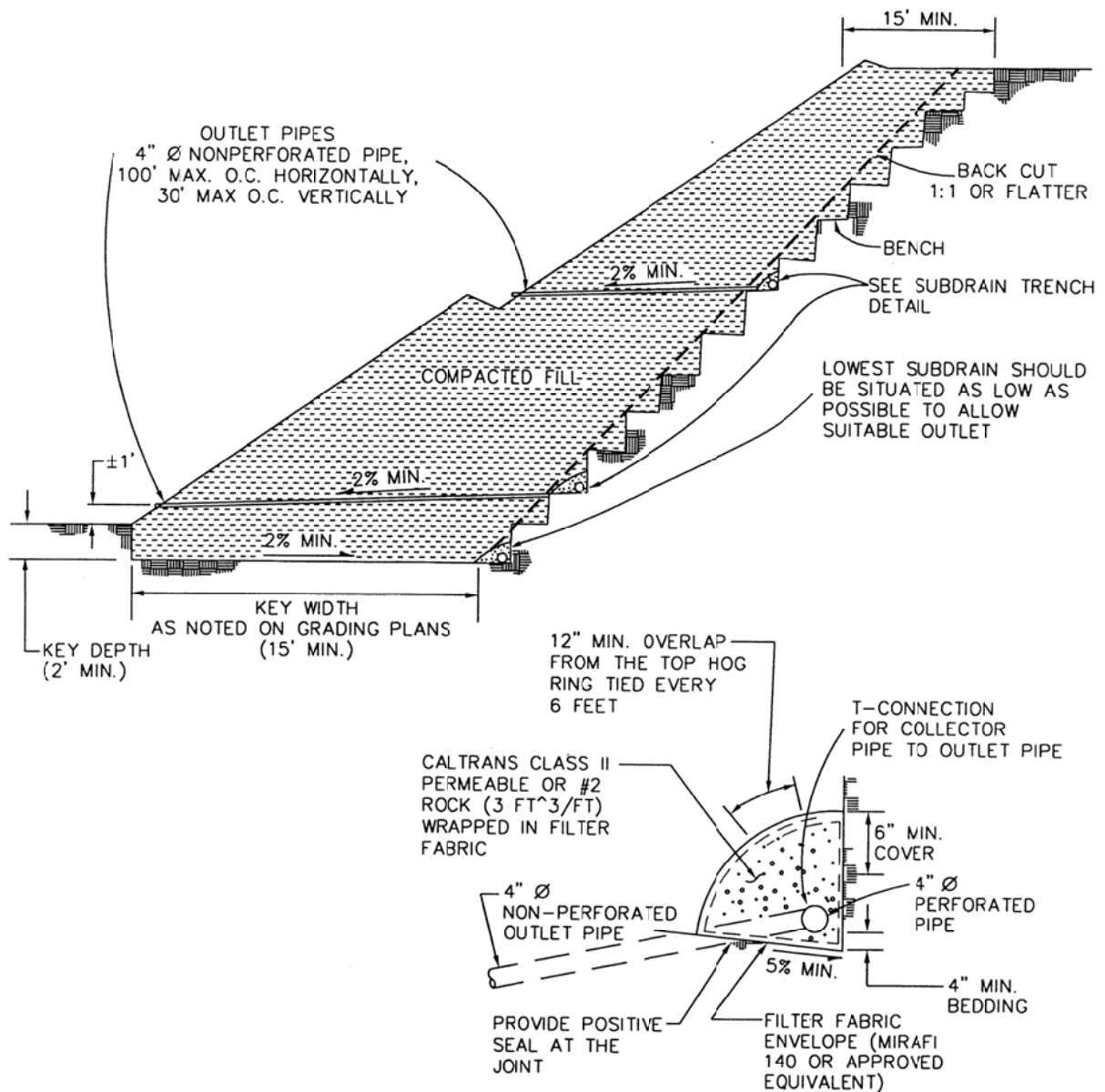


DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

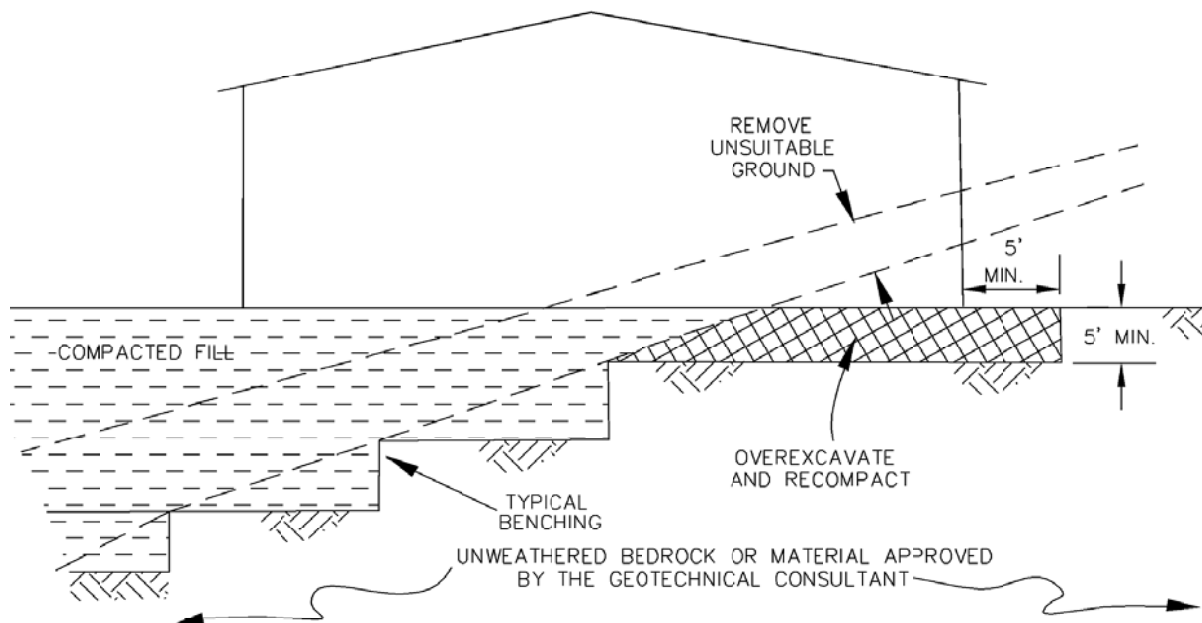
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

**BUTTRESS OR
REPLACEMENT
FILL SUBDRAINS**

**GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D**



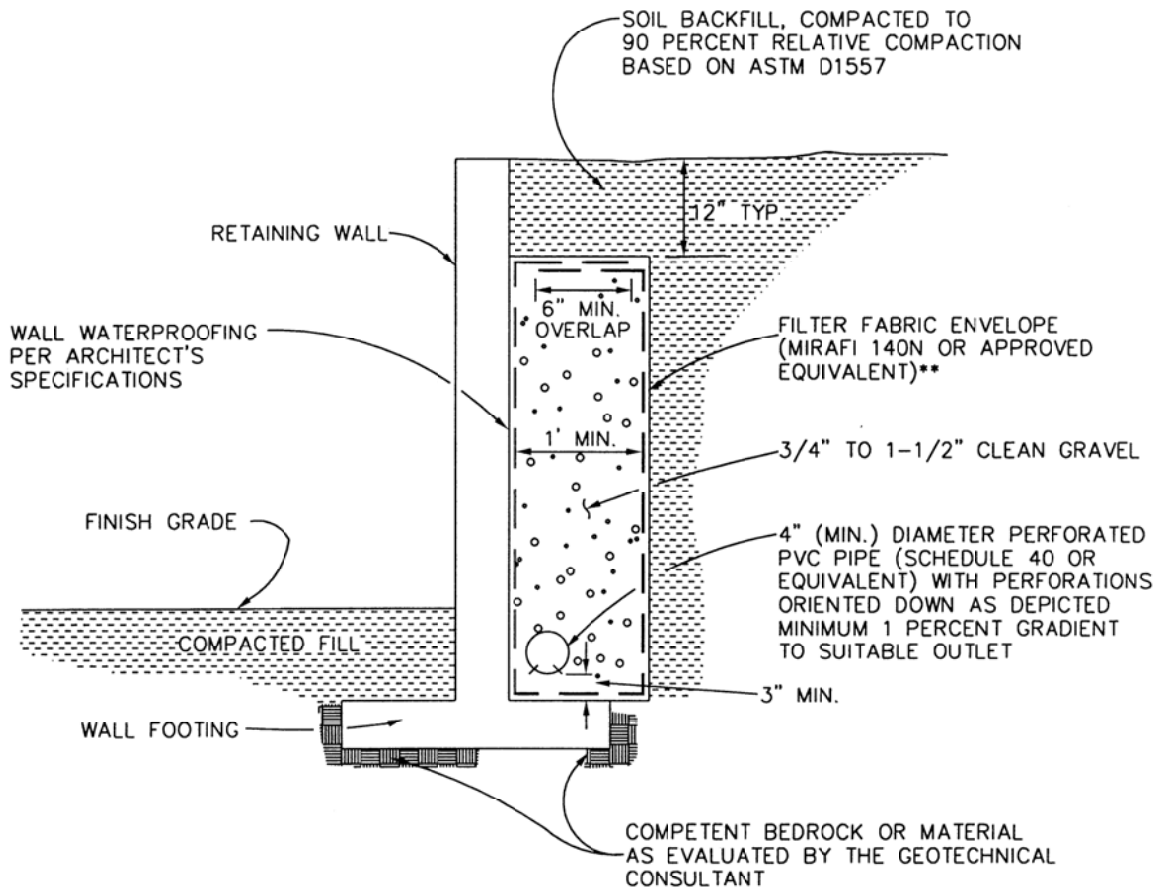
CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



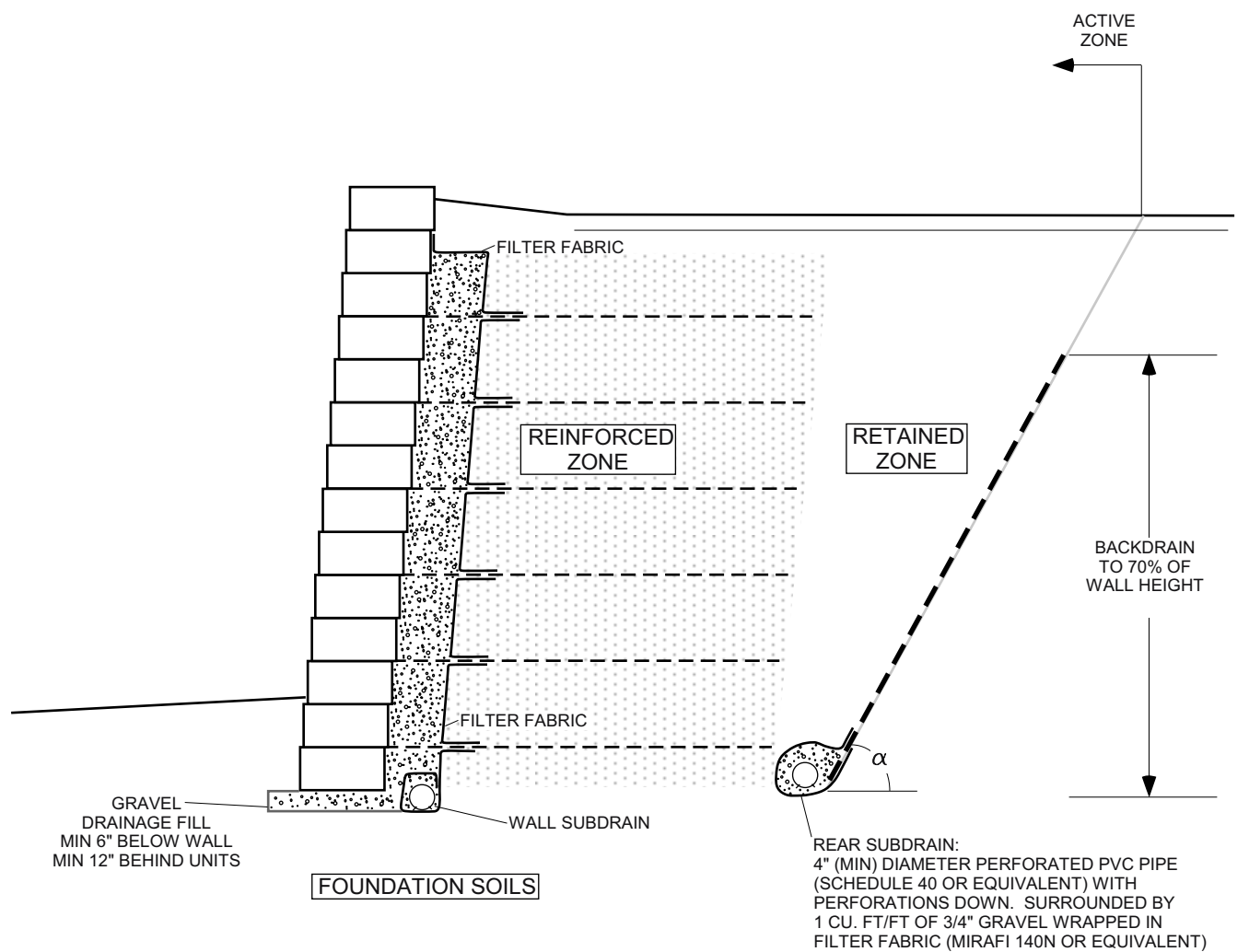


NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL F





NOTES:

1) MATERIAL GRADATION AND PLASTICITY
REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

- CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.
- GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.
- GEOGRID TO BE PRETENSIONED DURING INSTALLATION.
- IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45 + \phi/2$, WHERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.
- BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL G

