



**GEOTECHNICAL INVESTIGATION
NEW KITCHEN AND WEIGHT ROOM
MODERNIZATION 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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Attention: Ms. Dena Johnson, Project Manager

**Subject: Geotechnical Investigation
New Kitchen and Weight Room Modernization Project
Grossmont High School
1100 Murray Drive
El Cajon, California**

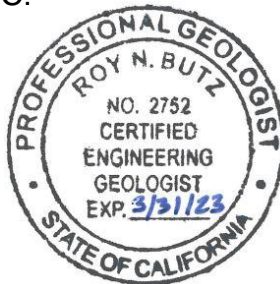
In accordance with your request and authorization, we have conducted a geotechnical investigation for the proposed New Kitchen and Weight Room Modernization Project at Grossmont High School located in El Cajon, San Diego County, California. Based on the results of our study, it is our opinion that the proposed modernization project is 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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in 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.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives 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 this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be, and, in general, if you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying it.* A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may

perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering 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 subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old*.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration*. Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists*.



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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 Kitchen and Weight Room Modernization 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 two geotechnical test pits and seven borings. The geotechnical test pits and boring logs are presented in Appendix B and their approximate locations are shown on Figure 2;
- Laboratory testing of representative soil samples obtained from the subsurface exploration program. Results of these tests are presented in Appendix C;
- Assessment of geologic hazards;
- Development of seismic design parameters based on the 2019 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. The New Kitchen and Weight

Room Modernization Project is proposed to be located along a central portion of the school's campus. In general, the site is bounded by the football stadium, Building 1100, and the swimming pool to the west; Building 1400 and Dennstedt Place to the north; Building 700, Building 600, and Building 400 to the east; and Building 200 and the newly constructed Event Center to the south. A site plan which depicts existing site improvements and 1-foot topographic contours is presented on Figure 3.

Currently, the project site is occupied by the Cafeteria, Building 1200, grass lawn areas, areas of asphalt paved driveways, concrete hardscape features, landscaping, and underground utilities. Site topography within the vicinity of the proposed site improvements is nearly level with surface elevations gently sloping from the northeast to the southwest, ranging from approximately 685 to 681 feet above mean sea level (msl). In addition, there is a minor slope located in the western corner of the site near the proposed press box. This slope ascends up to the existing Cafeteria and is approximately 6 feet in height over a horizontal distance of approximately 12 feet.

Generalized Site Latitude and Longitude

32.7816 ° N

116.9873 ° W

1.3 Proposed Development

Based on our review of the schematic design plan, project documents (Appendix A), and our discussions with you and the project team, we understand that the project will consist of the demolition of the existing Cafeteria, Building 1200, and any associated existing site improvements. After demolition, a new Kitchen Building, Weight Room Building, and New Museum will be constructed in the footprint of the demolished buildings. In addition, site improvements will consist of concrete hardscape, graphics wall, ticket booth, site lighting, and underground utilities. Also, a new press box will be constructed behind the home bleachers at the football stadium. The approximate limits of the proposed improvements for the New Kitchen and Weight Room Modernization Project is depicted on Figure 2.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our exploration performed on August 8, November 23, and November 24, 2021, consisted of excavating two geotechnical test pits (TP-1 and TP-2) and seven 8-inch diameter geotechnical borings (B-1 through B-7) to depths between approximately 4 to 21½ feet below the existing ground surface (bgs). The geotechnical test pits were excavated using conventional hand tools. The geotechnical borings were drilled using a combination of a truck-mounted and a track-mounted limited access drill rigs. Hand tool and auger refusal was encountered in geotechnical exploration locations TP-1, TP-2, B-1, B-2, B-5, B-6, and B-7 due to the presence of large cobbles located within the underlying Stadium Conglomerate. The purpose of our subsurface exploration was to evaluate the underlying stratigraphy, physical characteristics, and specific engineering properties of the soils within the area of the proposed improvements.

During the exploration operations, a geologist from our firm prepared geologic logs and collected bulk and relatively undisturbed samples for laboratory testing and evaluation. Disturbed standard penetration test (SPT) and relatively undisturbed split-barrel soil sampling using a 140-pound automatic-trip hammer free falling 30-inches was performed in accordance with ASTM D 1586 and ASTM D 3550, respectively. After logging, the geotechnical test pits and borings were backfilled with excavated soil cuttings and capped with quick set concrete. The geotechnical test pit and boring logs are provided in Appendix B, laboratory test results are included in Appendix C, and the approximate exploration locations are shown on Figure 2.

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 just east of the eastern margin of the San Diego Embayment.

The Peninsular Ranges are traversed by several major Holocene-active faults (Figure 4). 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 geologic setting is depicted on Figure 5.

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 undocumented artificial fill materials overlying the Tertiary-aged Mission Valley Formation and the Tertiary-aged Stadium Conglomerate. It should be noted that the geologic units underlying the site on the Regional Geology Map (Figure 5) incorrectly map the Tertiary-aged Pomerado Formation as underlying the site. However, based on our site subsurface exploration and geologic mapping, 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 6).

3.2.1 Undocumented Artificial Fill (Afu)

Based on our subsurface exploration, undocumented fill soils were encountered in all our geotechnical exploration locations with thicknesses varying between approximately 2 to 6½ feet bgs. As encountered, the fill soils generally consist of loose, yellow to dark brown, moist, clayey sand and soft, gray, moist, sandy lean clay with variable amounts of scattered gravel and 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. An as-graded report was not available for our review, and it is assumed that no engineering observations of these fill soils were provided at the time of grading. Therefore, these fills are considered undocumented and may settle under the placement of additional fill and/or building loads.

3.2.2 Mission Valley Formation (Tmv)

Underlying the existing undocumented artificial fill soils, formational material consisting of the Tertiary-aged Mission Valley Formation was encountered at the project site. As encountered, 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.

3.2.3 Stadium Conglomerate (Tst)

Underlying the existing undocumented artificial fill soils and the Mission Valley Formation, the Tertiary-aged Stadium Conglomerate was encountered at the project site. As encountered, 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 comprises between approximately 30 and 60 percent of the conglomerate mass, respectively. The cobbles are typically 3 to 6 inches in maximum dimension, but may include larger cobbles and boulders.

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 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 1,200 feet east of the site (Figure 5). 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 vicinity of the proposed site improvements is nearly level with surface elevations gently sloping from the northeast to the southwest, ranging from approximately 685 to 681 feet above mean sea level (msl). In addition, there is a minor slope located in the western corner of the site near the proposed press box. This slope ascends up to the existing Cafeteria and is approximately 6 feet in height over a horizontal distance of approximately 12 feet.

Based on the results of our geotechnical investigation, we anticipate that this existing slope located near the proposed press box is generally comprised of a thin veneer of granular artificial fill overlying a generally massive, near horizontal, very dense, clayey gravel conglomerate identified as the Tertiary-aged Stadium Conglomerate. In addition, there are no planned fill or cut slopes greater than 5 feet in height for the project. Therefore, we have not performed a slope stability analysis. The potential for slope instability at the site is low.

3.6 Expansive Soils

Expansion index testing on representative soil samples taken from test pit TP-1 and from borings B-1, B-4, and B-7 indicates that the onsite soil materials have a very low (14) to high (125) potential for expansion. However, higher expansive soils may be encountered during the grading of the proposed site improvements. Selective grading may be required for isolated zones of highly expansive soils.

3.7 Collapsible and Compressible Soils

Based on the results of our subsurface exploration, the potential for hydro-collapse of the underlying Tertiary-aged Mission Valley Formation and Stadium Conglomerate is considered low at the site. Our opinion is supported by our subsurface exploration which indicated a very dense to hard, non-porous character for the underlying tertiary-aged formational units. The existing undocumented artificial fill materials located at the project site are considered compressible and recommendations for treatment of these materials during construction are provided later in this report.

3.8 Soil Corrosivity

A screening of the onsite soils 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 three representative soil samples obtained during our subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The soil samples tested had a measured pH range of 8.08 to 10.75 and a measured minimum electrical resistivity range of 1,295 to 3,500 ohm-cm. The test results also indicated that the samples had a soluble chloride content range of 40 to 80 parts per million (ppm) and a soluble sulfate content of less 150 ppm.

3.9 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 Mission Valley Formation and the Stadium Conglomerate. Nevertheless, based on the above information, we do not anticipate groundwater will be a constraint to the construction of the proposed improvements.

3.10 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 7). In addition, based on our review of dam inundation and topographic maps, the site is not located within a dam inundation area (Figure 8).

3.11 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.

3.11.1 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.

3.11.2 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.

3.11.3 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.

3.11.4 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.

3.11.5 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.

3.11.6 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 11.3 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 subject 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 11.3 miles west of the site (Figure 4).

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, 2021). 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 $0.093g$.

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 11.3 miles west of the site is the 'Holocene-active' fault considered having the most significant effect at the site from a design standpoint.

4.3.1 Site Class

The site is underlain at shallow depth by the Tertiary-aged Mission Valley Formation and the Tertiary-aged Stadium Conglomerate. Utilizing 2019 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 2019 CBC (CBSC, 2019) and SEAOC/OSHPD Seismic Design Maps Web Application (2021).

Table 1. CBC Mapped Spectral Acceleration Parameters

Site Class	C
Site Coefficients	$F_a = 1.2$
	$F_v = 1.5$
Mapped MCE Spectral Accelerations	$S_s = 0.777g$
	$S_1 = 0.284g$
Site Modified MCE Spectral Accelerations	$S_{MS} = 0.933g$
	$S_{M1} = 0.426g$
Design Spectral Accelerations	$S_{DS} = 0.622g$
	$S_{D1} = 0.284g$

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 0.334g 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 ($PGAM$) is 0.401g 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 1613A.2.5 of the 2019 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 undocumented 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 negligible potential for liquefaction, the potential for lateral spreading flow failure is negligible.

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 681 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 New Kitchen and Weight Room Modernization 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 11.3 miles to the west;
- The peak horizontal ground acceleration associated with the Maximum Considered Earthquake Ground Motion is 0.334g. The peak horizontal ground acceleration associated with the PGA_M is 0.401g;
- The potential for liquefaction and seismic settlement at the site is negligible;
- 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 to high potential for expansion;
- Exceptional geologic hazards are not anticipated to impact the site or the proposed improvements;
- 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. Onsite clay materials may have a high expansion potential, where encountered during grading, highly expansive soils should not be used as fill and exported offsite;
- 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 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 Stadium Conglomerate contains cobbles that range in size from 6 to 8 inches in diameter with isolated cobbles to up to 1 foot in diameter expected. Removed 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 building structures can be supported on conventional reinforced concrete foundations founded on new compacted fill soils. In addition, the proposed press box can be supported on isolated conventional spread footings founded in undisturbed Stadium Conglomerate. Where isolated conventional spread footings for the proposed press box need to be deepened to bear on undisturbed Stadium Conglomerate, a Controlled Low-Strength Material (CLSM) for footing support can be used;
- 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 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.

6.0 PRELIMINARY 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 off-site. 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 recompacted 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 Mission Valley Formation and/or Stadium Conglomerate is encountered. Excavation for utilities may also be difficult in some areas. Where soils are found to have greater than 30 percent oversize particles retained on the ¾-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 variable amount of oversized cobble in the Stadium Conglomerate, a screening process of the removed 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 pad 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.

Due to the general density and soil type characteristics of the 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 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 – New Kitchen, Weight Room, and Museum Buildings

Undocumented fill and the upper weathered portions of the Mission Valley Formation and/or Stadium Conglomerate at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structure loads supported on shallow foundations.

All undocumented fill materials at the site should be completely removed. In addition, weathered portions of the Mission Valley and/or Stadium Conglomerate, if encountered beneath settlement-sensitive improvements and foundations, should be removed. Also, we recommend that at least 2 feet of compacted fill be placed below the proposed building foundations. Based on the results of our subsurface exploration, we anticipate removal depths between 4 to 6½ feet will be necessary within the proposed building pad areas. Horizontally, the lateral limits of the removal excavations should extend at least 3 feet beyond the foundation limits of the settlement-sensitive improvements. However, where the horizontal lateral limits of the removal excavations cannot be achieved due to existing site improvements, a 2:1 (horizontal to vertical) projection off the bottom of the footings from the

existing settlement-sensitive improvements and a 1:1 (horizontal to vertical) projection off existing utilities, hardscape, and site walls should be maintained. Remaining materials should be “benched” during grading. The bottom of all removals should be evaluated by a Certified Engineering Geologist from Leighton to confirm conditions are as anticipated.

6.1.4 Remedial Grading – Concrete Hardscape Areas

Due to the presence of exiting shallow utilities, it may not be practical to scarify and recompact all areas of proposed concrete hardscape. Therefore, we recommended that in these areas soil materials be removed down to proposed subgrade elevations and extend at least 2 feet beyond the limits of the proposed improvements, where possible. Exposing the subgrade will allow for review and testing by our field technician to assess the in-place subgrade uniformity and density. That assessment will allow further subgrade scarification and recompaction recommendations, if needed.

6.1.5 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 5 feet of finish grade within the limits of the proposed building pads or 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. Fill 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, such as beneath restrooms, it may be preferable to utilize a controlled low strength material for backfill. Where used these materials should possess a compressive strength of at least 125 psi as determined by ASTM D 4832 and an average flow diameter of at least 8 inches as determined by ASTM D 6103.

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.

6.1.6 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.

6.1.7 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, to accommodate conventional foundation design, all engineered fill materials placed within the building pads and 3 feet outside the limits of the building foundations, should have a low to medium expansion potential ($El < 50$). Also, 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.

6.1.8 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 2019 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 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 in the 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 Mission Valley Formation and/or Stadium Conglomerate
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.

6.3 Foundation and Slab Considerations

The proposed Kitchen, Weight Room, and Museum Buildings may be constructed with conventional foundations founded in new engineered fill. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a very low to low potential for expansion ($EI < 50$). If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary. Also,

we recommend that isolated conventional spread footings founded in competent undisturbed Stadium Conglomerate be selected to support the proposed press box structure. It should be noted, where isolated conventional spread footings need to be deepened to bear on competent undisturbed Stadium Conglomerate, a CLSM for footing support can be used.

6.3.1 Shallow Spread Footings

The proposed Kitchen Building, Weight Room Building, Museum Building, and press box may be supported by conventional, continuous or isolated spread footings. Footings should extend a minimum of 24 inches beneath the lowest adjacent soil grade. At these depths, footings founded in new engineered fill for the proposed Kitchen, Weight Room, and Museum Buildings may be designed for a maximum allowable bearing pressure of 3,500 pounds per square foot (psf). In addition, the footings founded in competent undisturbed Stadium Conglomerate for the proposed press box may be designed for a maximum allowable bearing pressure of 4,000 psf. The allowable bearing pressures may also be increased by one-third when considering loads of short duration such as wind or seismic forces. The minimum recommended width of footings is 18 inches for continuous footings and 24 inches for square or round footings. Footings should be designed in accordance with the structural engineer's requirements.

6.3.2 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 the 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.

6.3.3 Floor Slabs

Slabs-on-grade should be at least 5 inches thick and be reinforced with No. 4 rebars 18 inches on center each way (minimum) placed at mid-height in the slab. We recommend control joints be provided across the slab at appropriate intervals as designed by the project architect.

For slab areas where vapor control is appropriate, a minimum 15-mil vapor barrier should be provided between the underslab and gravel capillary break. The vapor barrier should have a permeance of less than 0.01 perms across the entire slab area in the final constructed condition. Measures to protect the barrier should be implemented throughout the installation and slab construction process to prevent damage (ASTM E 1643). Vapor barrier materials should conform to ASTM E 1745 Class A. The gravel capillary break should consist of a layer of uniform 3/8-inch to 1/2-inch gravel that is at least 4-inches thick. The mix design of the slab concrete should be proportioned to control bleeding, shrinkage and curling. The project architect should provide waterproofing and underslab insulation designs where appropriate.

Note that moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs. We recommend that the floor covering/insulation installer test the moisture vapor flux rate prior to attempting applications of the flooring/insulation. "Breathable" floor

coverings should be considered if the vapor flux rates are high. A slip-sheet or equivalent should be utilized above the concrete slab if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab. Additional guidance is provided in ACI Publications 302.1R-15 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slab that Receive Moisture-Sensitive Floor Materials.

The potential for slab cracking may be reduced by careful control of water/cement ratios. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to minimize cracking of the slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted tile, marble tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations. If heavy vehicle or equipment loading is proposed for the slabs, greater thickness and increased reinforcing may be required. The additional measures should be designed by the structural engineer using a modulus of subgrade reaction of 150 pounds per cubic inch. Additional moisture/waterproofing measures that may be needed to accomplish desired serviceability of the building finishes and should be designed by the project architect.

6.3.4 Settlement

For conventional footings, the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of $\frac{3}{4}$ inch and $\frac{1}{2}$ -inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists. However, for most cases, differential settlements are considered unlikely to exceed $\frac{1}{2}$ -inch.

6.3.5 Moisture Conditioning

With the assumption of low expansive soils, prior to the placement of slabs and site hardscape, the subgrade soils should be maintained at a moisture content at least 2 percent above optimum. If medium or more highly expansive soil is present at pad grade, alternative moisture conditions and slab recommendations may be necessary.

6.4 Lateral Earth Pressures and Retaining Wall Design

Table 4 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with free draining import soils of low expansion potential (less than 50 per ASTM D 4829) above the groundwater table. Soils used to backfill retaining walls should be classified as one of the following types according to ASTM D 2487: GW, GP, GM, GC, SW, SP, or SM. These backfill soils should be used within horizontal distance behind the wall equal to one-half the wall height.

Table 4. Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	36	55
At-Rest	55	85
Passive	300 (Maximum of 3 ksf)	150 (sloping down)

Walls up to 10 feet in height should be designed for the applicable pressure values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to $0.35q$ should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D 1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Section 6.3.2.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.3 may be used at the concrete and soil interface. This value may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken

as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposite sides differ by more than 6 feet fall under the requirements of 2019 CBC Section 1617A.1.11 and/or ASCE 7-16 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic pressure equal to $9H$ pounds per square foot should be considered for the design of retaining walls with level backfill, where H is the height of the wall in feet. This increment should be added to the unfactored static active earth pressure to obtain the total seismic earth pressure on the wall. For restrained walls the value should be increased by 150%.

6.5 Light Pole Foundations

Bearing resistance for proposed light standard pile foundations may be based on the bearing pressures for "Class of Material 5" in Table 1806A.2 of the 2019 California Building Code (CBC). Lateral bearing may be doubled in accordance with 1806A.3.4, ignoring the upper 12 inches of soil in non-paved areas. This lateral bearing value assumes that the pole can tolerate at least a 0.5-inch deflection at the ground surface due to short term loading. These recommendations assume that the foundations will be embedded against firm intact soil. Any utilities within 5 diameters of the foundation should be relocated or designed to accommodate lateral loading.

We recommend an allowable axial resistance in compression for these foundations consisting of 250 psf for allowable skin friction, ignoring the upper 1 foot. These values are for isolated single piles. A group action reduction in capacity would apply for closely spaced piles.

6.6 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 medium to high corrosion potential to buried uncoated metal conduits (Caltrans, 2018). We recommend measures to mitigate corrosion of uncoated ferrous metal in contact with soil be implemented during design and construction.

6.7 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 exceed 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.

6.8 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.

6.9 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

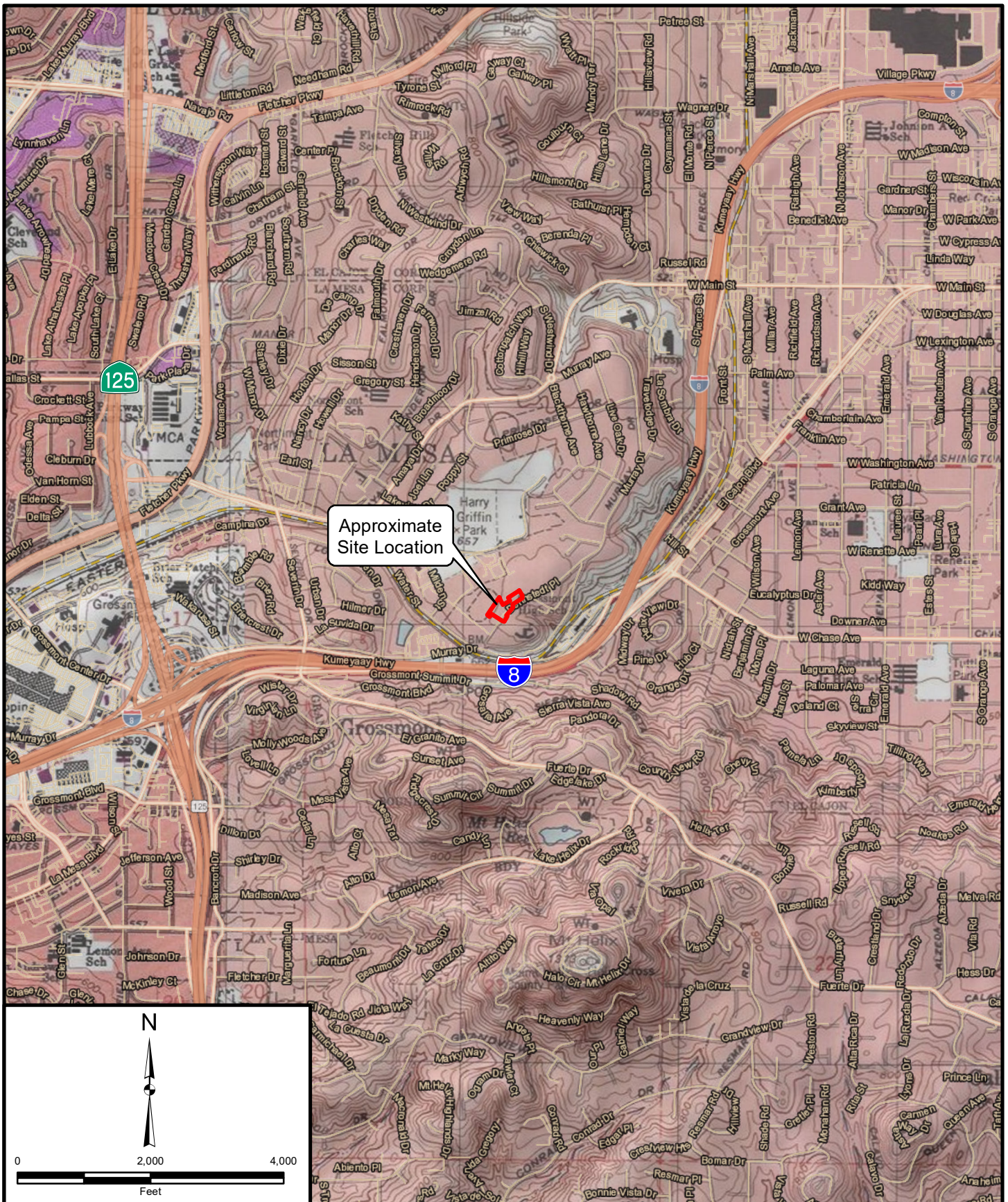
The nature of many sites is such that differing geotechnical or geological conditions can occur over small areal distances and under varying climatic conditions. The conclusions and recommendations in this report are based in part upon data that were obtained from a limited number of observations, site 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 DSA. Anyone using this report before DSA 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 the Grossmont Union High School District and their design team, for application to the design of the proposed New Kitchen and Weight Room Modernization Project at Grossmont High School in accordance with generally accepted California licensed geotechnical engineering practices at this time in California.

FIGURES



Project: 12429.018	Eng/Geol: NJT/RNB/RCS	SITE LOCATION MAP New Kitchen and Weight Room Modernization Project Grossmont High School School El Cajon, California	FIGURE 1
Scale: 1" = 2,000'	Date: January 2022		
Base Map: ESRI ArcGIS Online 2022	Author: (mmurphy)		

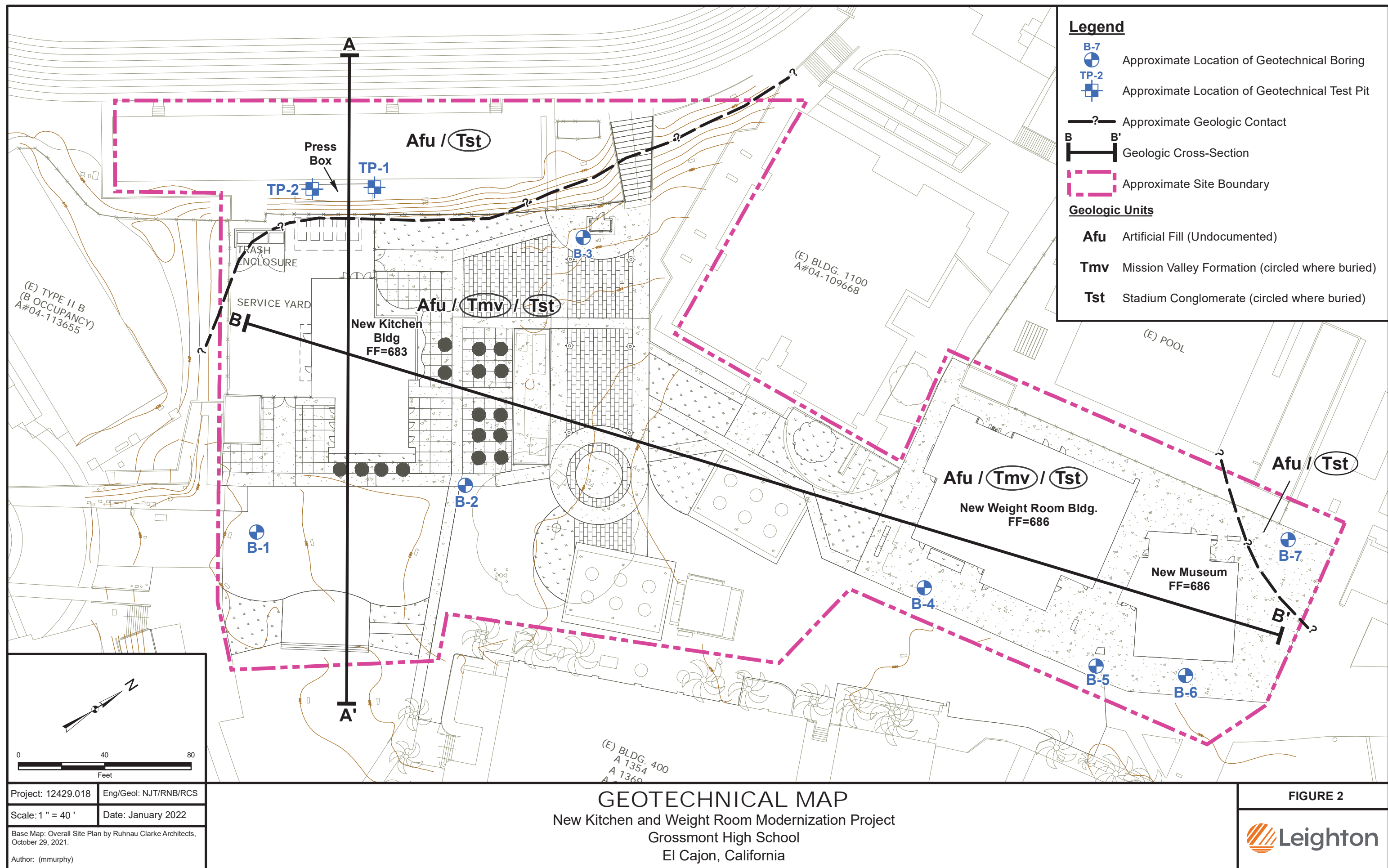
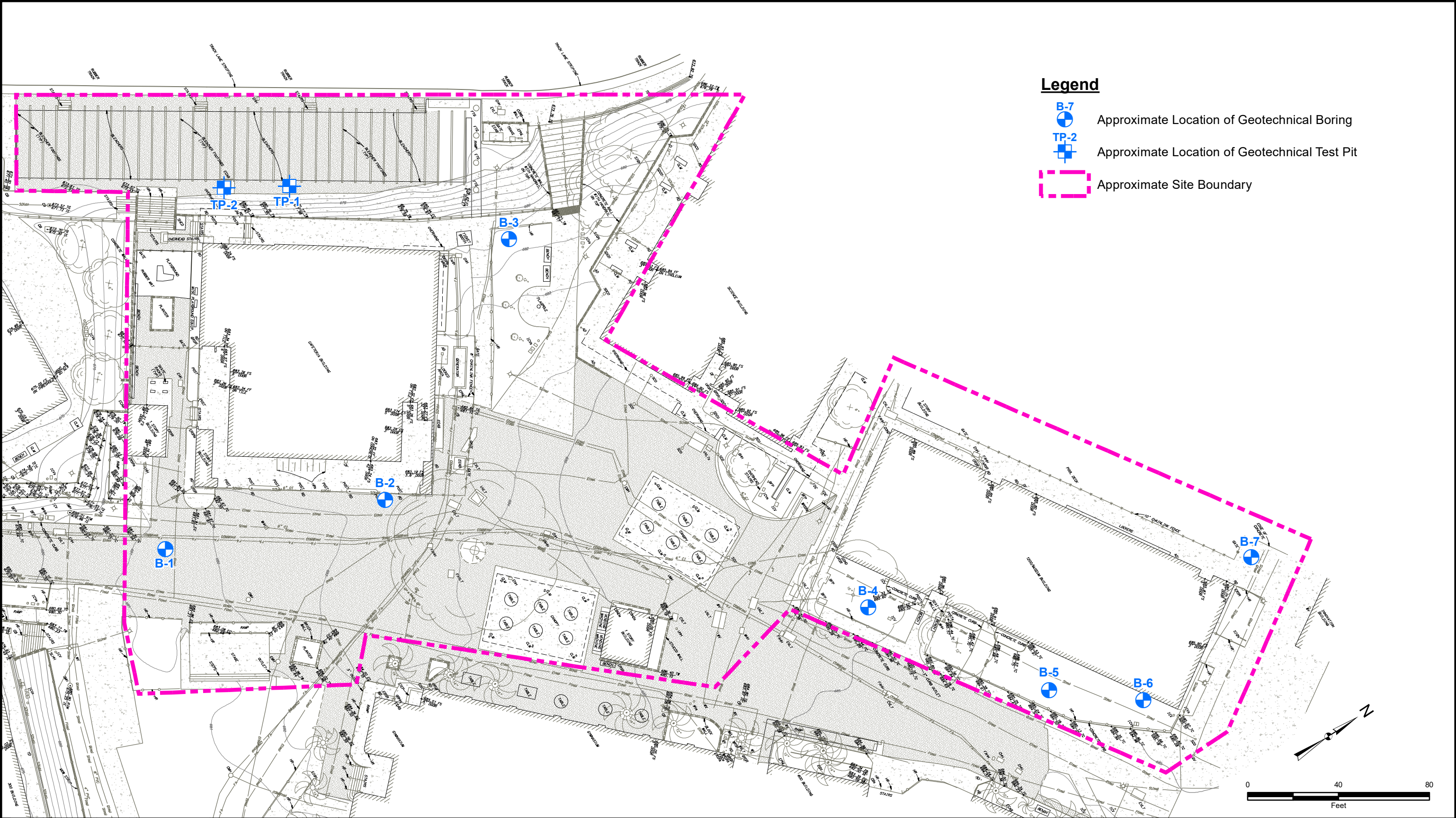


FIGURE 2





Legend

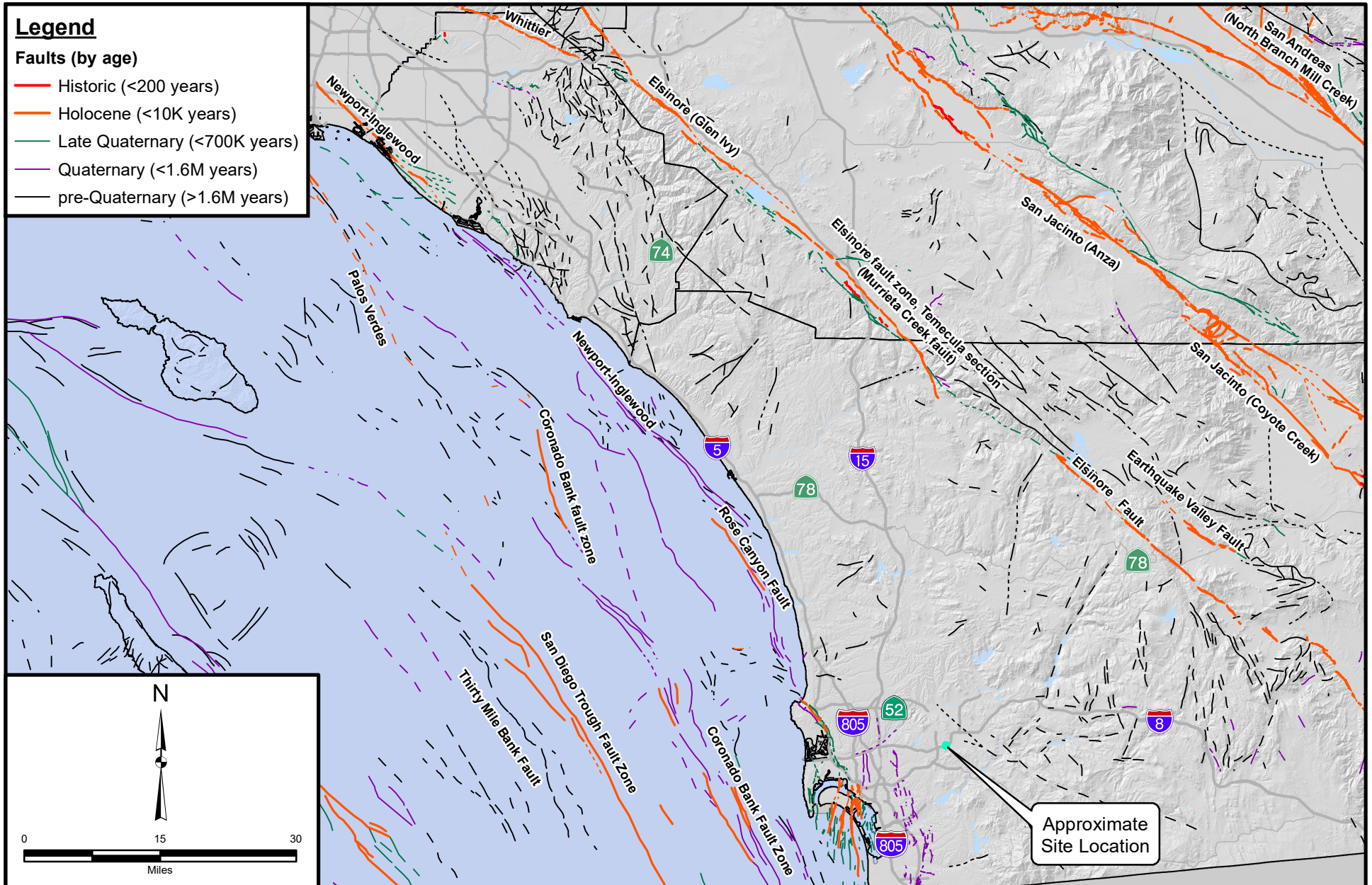
-  B-7
Approximate Location of Geotechnical Boring
-  TP-2
Approximate Location of Geotechnical Test Pit
-  Approximate Site Boundary

Project: 12429.018 Eng/Geol: NJT/RNB/RCS		<div>EXISTING SITE PLAN</div> <div>New Kitchen and Weight Room Modernization Project</div> <div>Grossmont High School</div> <div>El Cajon, California</div>	FIGURE 3
Scale: 1" = 40'	Date: January 2022		
Base Map: Topographic Survey by Ruhnu Clarke Architects October 29, 2021.			
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: 12429.018 Eng/Geol: NJT/RNB/RCS

Scale: 1" = 15 miles Date: January 2022

Faults: Bryant, Bryant CGS 2010

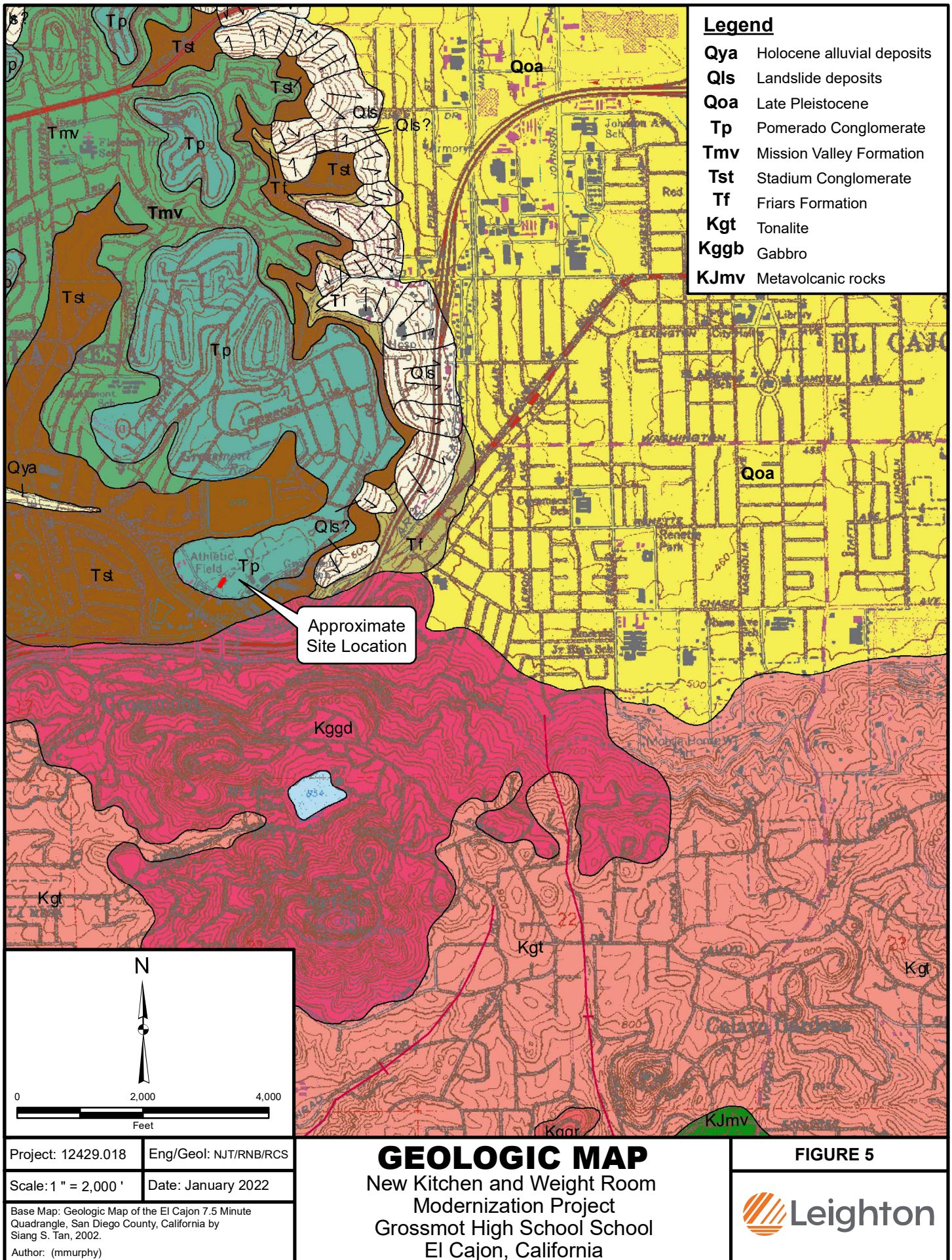
Author: Leighton Geomatics (mmurphy)

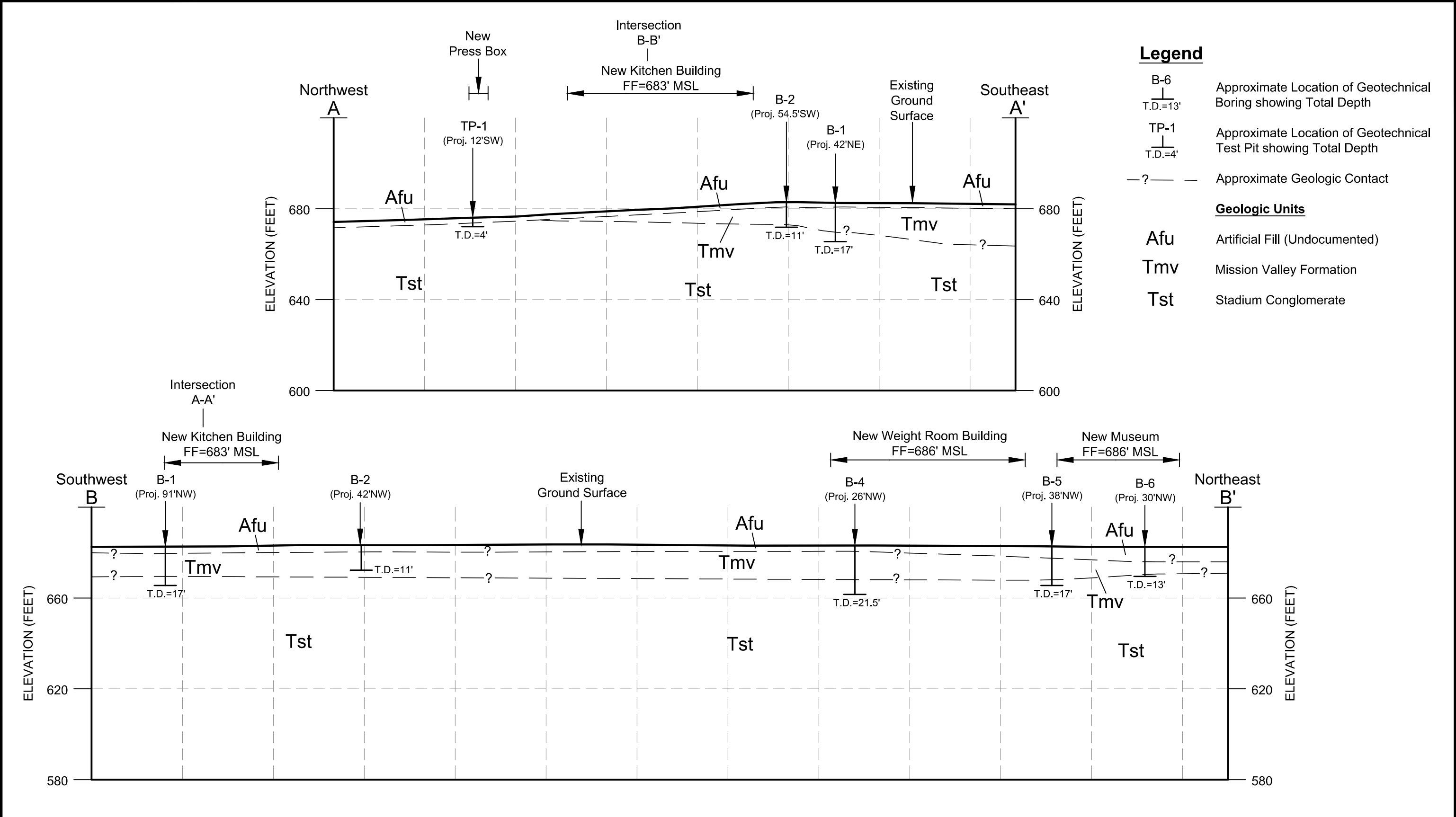
REGIONAL FAULT MAP

New Kitchen and Weight Room Modernization Project
Grossmot High School School
El Cajon, California

FIGURE 4







Proj: 12429.018	Eng/Geol: NJT/RNB/RCS
Scale: 1"=40'	Date: January 2022
Base Map:	

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

New Kitchen and Weight Room Modernization Project
Grossmont High School
El Cajon, California

FIGURE 6



Legend

- 500 Year Flood Plain
- 100 Year Flood Plain



Approximate
Site Location

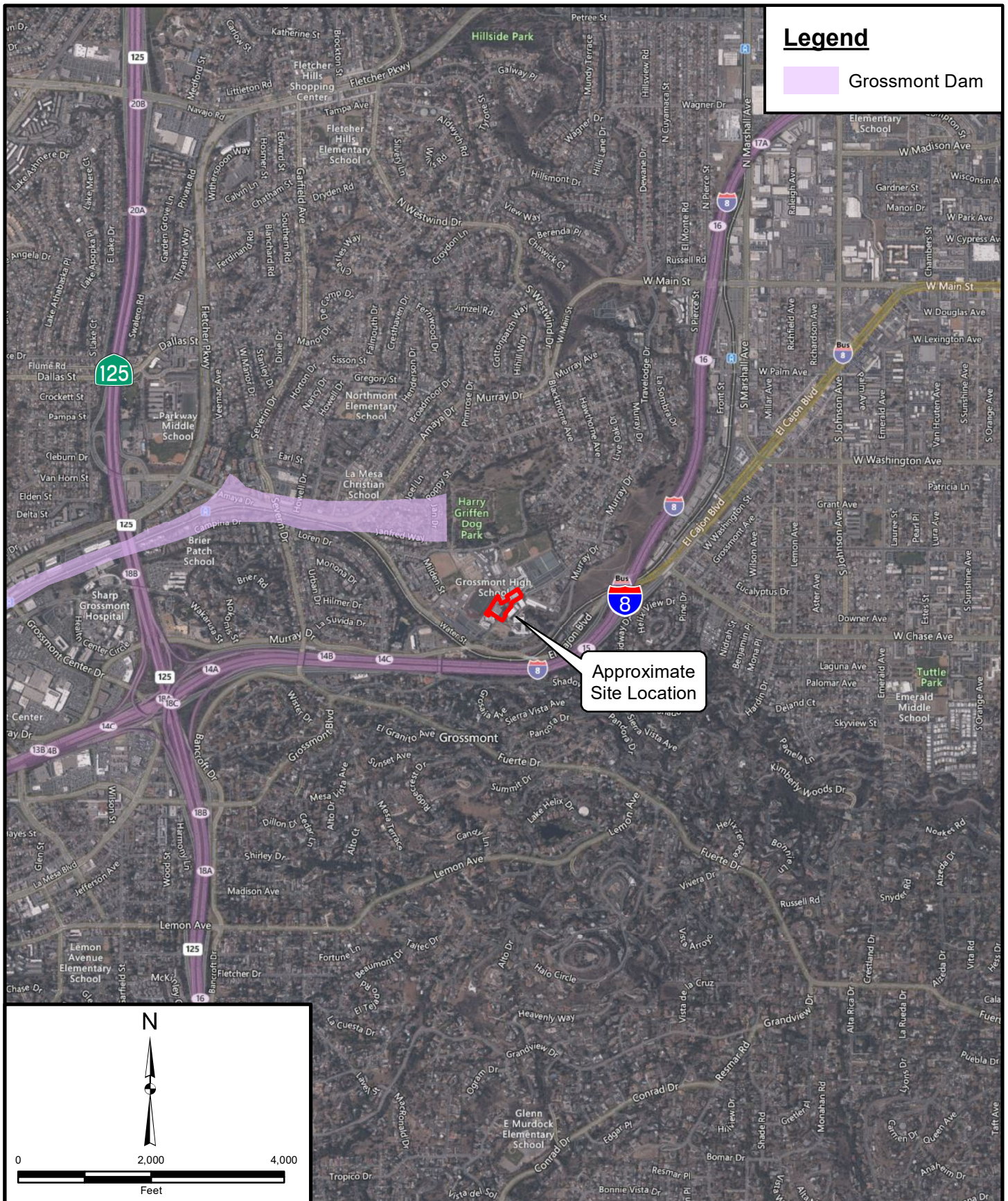
Project: 12429.018	Eng/Geol: NJT/RNB/RCS
Scale: 1" = 2,000'	Date: January 2022
Base Map: ESRI ArcGIS Online 2022 Reference: CADWR, FEMA Author: (mmurphy)	

FLOOD HAZARD ZONE MAP

New Kitchen and Weight Room
Modernization Project
Grossmont High School School
El Cajon, California

FIGURE 7





Project: 12429.018	Eng/Geol: NJT/RNB/RCS
Scale: 1" = 2,000'	Date: January 2022
Base Map: ESRI ArcGIS Online 2022 Reference: CA DWR, FEMA Author: (mmurphy)	

DAM INUNDATION MAP New Kitchen and Weight Room Modernization Project Grossmont High School School El Cajon, California

FIGURE 8



APPENDIX A

References

APPENDIX A

References

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APPENDIX B

Geotechnical Test Pits and Boring Logs

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
		N S							Logged By _____ Sampled By _____	
	0								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.	
				B-1					Bulk Sample 1.	
				C-1					Bulk Sample 2.	
				G-1					Core Sample.	
				R-1					Grab Sample.	
				SH-1					Modified California Sampler (3" O.D., 2.5 I.D.).	
	25			S-1					Shelby Tube Sampler (3" O.D.).	
				PUSH					Standard Penetration Test SPT (Sampler (2" O.D., 1.4" I.D.).	
									Sampler Penetrates without Hammer Blow.	
	30								Bulk Sample 2.	

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 TEST PIT TP-1

Project No.	12429.018	Date Drilled	8-9-21
Project	New Kitchen and Weight Room Modernization Project	Logged By	RNB
Drilling Co.	N/A	Hole Diameter	N/A
Excavation Method	Hand Tools	Ground Elevation	676' msl
Location	See Figure 2	Sampled By	RNB

Elevation Feet	Depth Feet	Graphic Log	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>	
								1" ASPHALT CONCRETE over 4" SAND BASE	
			B-1 (0-4')				SC	ARTIFICIAL FILL (Afu) @ 0-2.5': Clayey SAND with GRAVEL, loose, dark brown (7.5yr, 3/3), moist, fine-grained, gravel well-rounded, trace cobble	
							GC	STADIUM CONGLOMERATE (Tst) @ 2.5'-4': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown (10yr, 5/8), moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
								Hand Tool Refusal on Cobble @ 4 Feet (bgs) No Groundwater or Seepage Encountered Backfilled with Soil Cuttings on 8/9/21	
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

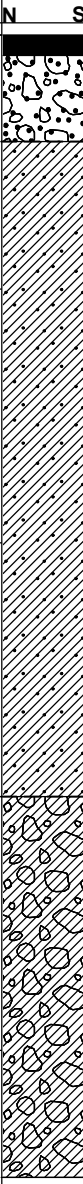
DS DIRECT SHEAR
EI EXPANSION INDEX
H HYDROMETER
MD MAXIMUM DENSITY
PP POCKET PENETROMETER
RV R VALUE

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



GEOTECHNICAL TEST PIT TP-2

Project No.	12429.018	Date Drilled	8-9-21
Project	New Kitchen and Weight Room Modernization Project	Logged By	RNB
Drilling Co.	N/A	Hole Diameter	N/A
Excavation Method	Hand Tools	Ground Elevation	676' msl
Location	See Figure 2	Sampled By	RNB

Elevation Feet	Depth Feet	Graphic Log	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0							<p><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></p> <p>1" ASPHALT CONCRETE over 4" SAND BASE</p> <p>ARTIFICIAL FILL (Afu) @ 0-3': Clayey SAND with GRAVEL, loose, dark brown (7.5yr, 3/3), moist, fine-grained, gravel well-rounded, trace cobble</p> <p>STADIUM CONGLOMERATE (Tst) @ 3'-4.5': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown (10yr, 5/8), moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining</p> <p>Hand Tool Refusal on Cobble @ 4.5 Feet (bgs) No Groundwater or Seepage Encountered Backfilled with Soil Cuttings on 8/9/21</p>	
	5		B-1 (0-4.5')				SC		
							GC		

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 B-1

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. Tri-County Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2

Date Drilled 11-23-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 681' msl
 Sampled By DKJ

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>	
680	0			B-1 (1'-5')				SC	2.5" ASPHALT CONCRETE 12" CONCRETE 4" AGGREGATE BASE UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 1.5': Clayey SAND, loose, yellow, moist, fine- to medium-grained	SA, EI, CR
								ML	MISSION VALLEY FORMATION (Tmv) @ 3': Sandy SILTSTONE, very stiff to hard, light brown, moist, laminated	
675	5			R-1	8 27 34			SC	@ 5': Clayey SANDSTONE, dense, light brown, moist, laminated, mottled	
670	10			S-1	13 18 28				@ 10': becomed oxidized	
								GC	STADIUM CONGLOMERATE (Tst) @ 13': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
665	15			R-2	17 50/5"					
				S-2	50/1"					
660	20								Auger refusal on cobble at 17 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/23/21	
655	25									
	30									

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 B-2

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. Tri-County Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2

Date Drilled 11-23-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 683' msl
 Sampled By DKJ

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>	
								SC	5" ASPHALT CONCRETE	
								SC	3" AGGREGATE BASE	
				B-1 (1.5'-3')				SC	UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 1': Clayey SAND, loose, yellowish brown, moist, asphalt fragments, trace gravel	
680									MISSION VALLEY FORMATION (Tmv) @ 2': Clayey SANDSTONE, very dense, light brown, moist, fine- to medium-grained, white mottling present	
	5			R-1 50/4"					@ 6': becomes fine- to medium-grained, trace cobble	
				B-2 (6'-7')						
675										
	10			S-1 50/1"				GC	STADIUM CONGLOMERATE (Tst) @ 10': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
670									Auger refusal on cobble at 11 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/23/21	
	15									
665										
	20									
660										
	25									
655										
	30									

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 B-3

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. ABC Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 15" Drop
 Location See Figure 2

Date Drilled 11-24-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 682' msl
 Sampled By DKJ

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>	
680	0			B-1 (1'-3')				SC	6" of grass and topsoil UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Clayey SAND with gravel, loose, yellowish brown, moist, fine-grained	
675	5			R-1	13 23 50/6"			SC	MISSION VALLEY FORMATION (Tmv) @ 3': Clayey SANDSTONE, dense, grayish brown, moist, fine- to medium-grained, lenses of friable sand, micaceous	
670	10			S-1	10 16 25				@ 10': becomes very dense and friable	SA
665	15			R-2	50/4"				@ 13': gravel layer encountered	
660	20			S-2	8 17 25			ML	@ 20': Sandy SILTSTONE, hard, gray, moist, friable sand lenses (1-2"), fine-grained, micaceous	
655	25								Bottom of boring at 21.5 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/24/21	
	30									

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 B-4

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. ABC Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 15" Drop
 Location See Figure 2

Date Drilled 11-24-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 687' msl
 Sampled By DKJ

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>	
685	0			B-1 (0.5'-4')				CL	6" of grass and topsoil UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Sandy Lean CLAY, soft, gray, moist, trace gravel, fine-grained	EI, SA
680	5			R-1	10 15 28			CL	MISSION VALLEY FORMATION (Tmv) @ 3': Sandy Lean CLAYSTONE, very stiff, gray, moist, laminated, oxidized	
675	10			S-1	10 16 20				@ 10': becomes hard, micaceous, black mottling observed	
	15			R-2	50/5"				@ 12': gravel layer encountered	
670	20			S-2	32 43 42			GC	STADIUM CONGLOMERATE (Tst) @ 16': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
665	25								Bottom of boring at 21.5 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/24/21	
660	30									

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 B-5

Project No. 12429.018
Project New Kitchen & Weight Room Modernization Project
Drilling Co. ABC Drilling
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 15" Drop
Location See Figure 2

Date Drilled 11-24-21
Logged By DKJ
Hole Diameter 8"
Ground Elevation 686' msl
Sampled By DKJ

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>	
685	0							SC	6" of grass and topsoil UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Clayey SAND with gravel, loose, light brown, moist, fine-grained	
680	5			R-1	4 9 14			ML	MISSION VALLEY FORMATION (Tmv) @ 5': Sandy SILTSTONE, stiff, gray, moist, pink mottling observed	
675	10			S-1	10 15 24			SC	@ 10': Clayey SANDSTONE, very dense, gray, moist, laminated, oxidized, micaceous, fine-grained, black mottling observed	
670	15			R-2 S-2	34 50/3" 42 50/4"			GC	STADIUM CONGLOMERATE (Tst) @ 15': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
665	20								Auger refusal on cobble at 17 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/24/21	
660	25									
	30									

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 B-6

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. ABC Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 15" Drop
 Location See Figure 2

Date Drilled 11-24-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 685' msl
 Sampled By DKJ

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	
<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>											
685	0							SC	6" of grass and topsoil UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 0.5': Clayey SAND, loose, light brown, moist, fine-grained, trace organics		
680	5		B-1 (1.5'-3')	R-1	50/6"			SC	MISSION VALLEY FORMATION (Tmv) @ 6.5': Clayey SANDSTONE, very dense, light brown, moist, fine-grained, trace gravel		
675	10		S-1 B-2 (11'-12')		50/3"			ML	@ 10': Sandy SILTSTONE, hard, gray, moist, micaceous		
							GC	STADIUM CONGLOMERATE (Tst) @ 12': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining			
670	15	Auger refusal on cobble at 13 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/24/21									
665	20										
660	25										
655	30										
SAMPLE TYPES:											
TYPE OF TESTS:											
B	BULK SAMPLE	DS	DIRECT SHEAR	SA	SIEVE ANALYSIS						
C	CORE SAMPLE	EI	EXPANSION INDEX	SE	SAND EQUIVALENT						
G	GRAB SAMPLE	H	HYDROMETER	SG	SPECIFIC GRAVITY						
R	RING SAMPLE	MD	MAXIMUM DENSITY	UC	UNCONFINED COMPRESSIVE STRENGTH						
S	SPLIT SPOON SAMPLE	PP	POCKET PENETROMETER								
T	TUBE SAMPLE	RV	R VALUE								

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

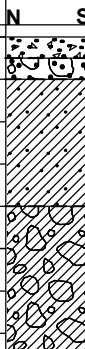
SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG B-7

Project No. 12429.018
 Project New Kitchen & Weight Room Modernization Project
 Drilling Co. Tri-County Drilling
 Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
 Location See Figure 2

Date Drilled 11-23-21
 Logged By DKJ
 Hole Diameter 8"
 Ground Elevation 685' msl
 Sampled By DKJ


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
<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>										
685	0								6" CONCRETE 3" AGGREGATE BASE UNDOCUMENTED ARTIFICIAL FILL (Afu) @ 1': Clayey SAND with gravel, loose, light brown, moist	SA, EI, CR
				B-1 (1'-5')				SC		
680	5			R-1 B-1 (6'-7')	50/3"			GC	STADIUM CONGLOMERATE (Tst) @ 4': Clayey GRAVEL CONGLOMERATE, very dense, yellowish brown, moist, fine-grained, few cobble, gravel and cobble well-rounded, trace oxidation staining	
									Auger refusal on cobble at 7.5 Feet (bgs) No groundwater or seepage encountered during drilling Backfilled with soil cuttings on 11/23/21	
675	10									
670	15									
665	20									
660	25									
655	30									

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



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



APPENDIX C

Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

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

Sample Location	Description	Expansion Index	Expansion Potential
B-1 @ 1-5 Feet	Yellow to Light Brown Clayey Sand	30	Low
B-4 @ ½-5 Feet	Gray Sandy Lean Clay	125	High
B-7 @ 1-5 Feet	Light Brown Clayey Sand w/ Gravel	14	Very Low
TP-1 @ 0-4 Feet	Clayey Sand with Gravel	17	Very Low

Particle Size Analysis (ASTM D 1140): Particle size analysis was performed by mechanical sieving methods according to ASTM D6913 and ASTM D1140. This test was performed to assist in the classification of the soil and to determine grain size distributions of the tested soil. The plot of a sieve result is provided as a Figure in this appendix and the percent fine particles from these analyses are summarized below:

Sample Location	Percent Passing No. 200 Sieve
B-1 @ 1-5 Feet	29
B-3 @ 10 Feet	19
B-4 @ ½-5 Feet	62
B-7 @ 1-5 Feet	23

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

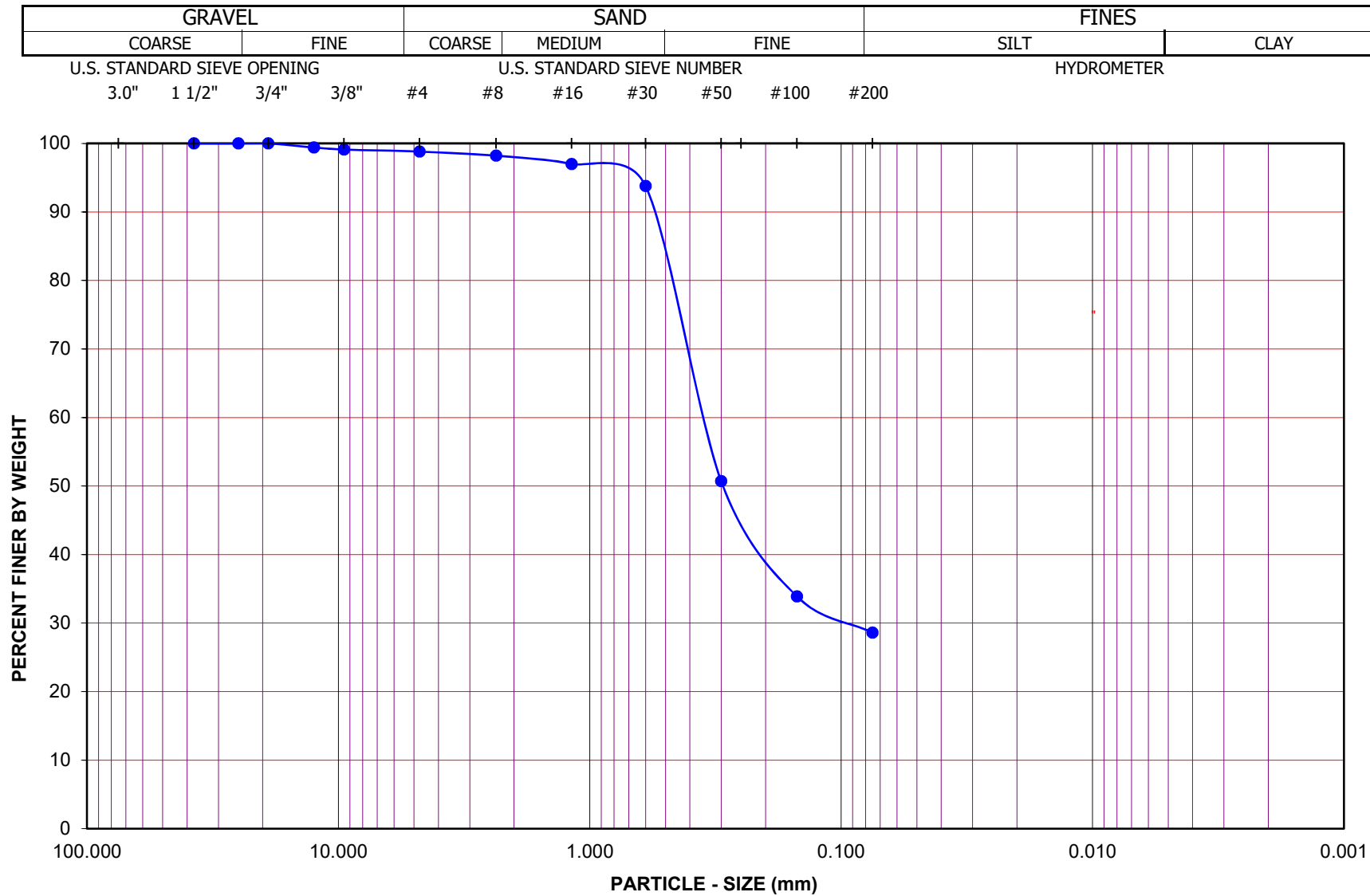
Sample Location	Sulfate Content, ppm
B-1 @ 1-5 Feet	<150
B-7 @ 1-5 Feet	<150
TP-1 @ 0-4 Feet	<150

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

Sample Location	Chloride Content, ppm
B-1 @ 1-5 Feet	80
B-7 @ 1-5 Feet	40
TP-1 @ 0-4 Feet	50

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)
B-1 @ 1-5 Feet	10.75	1,295
B-7 @ 1-5 Feet	10.16	1,410
TP-1 @ 0-4 Feet	8.08	3,500



Project Name: GHS Modernization Project

Project No.: 12429.018

**PARTICLE - SIZE
DISTRIBUTION**

ASTM D 6913

Boring No.: B-1

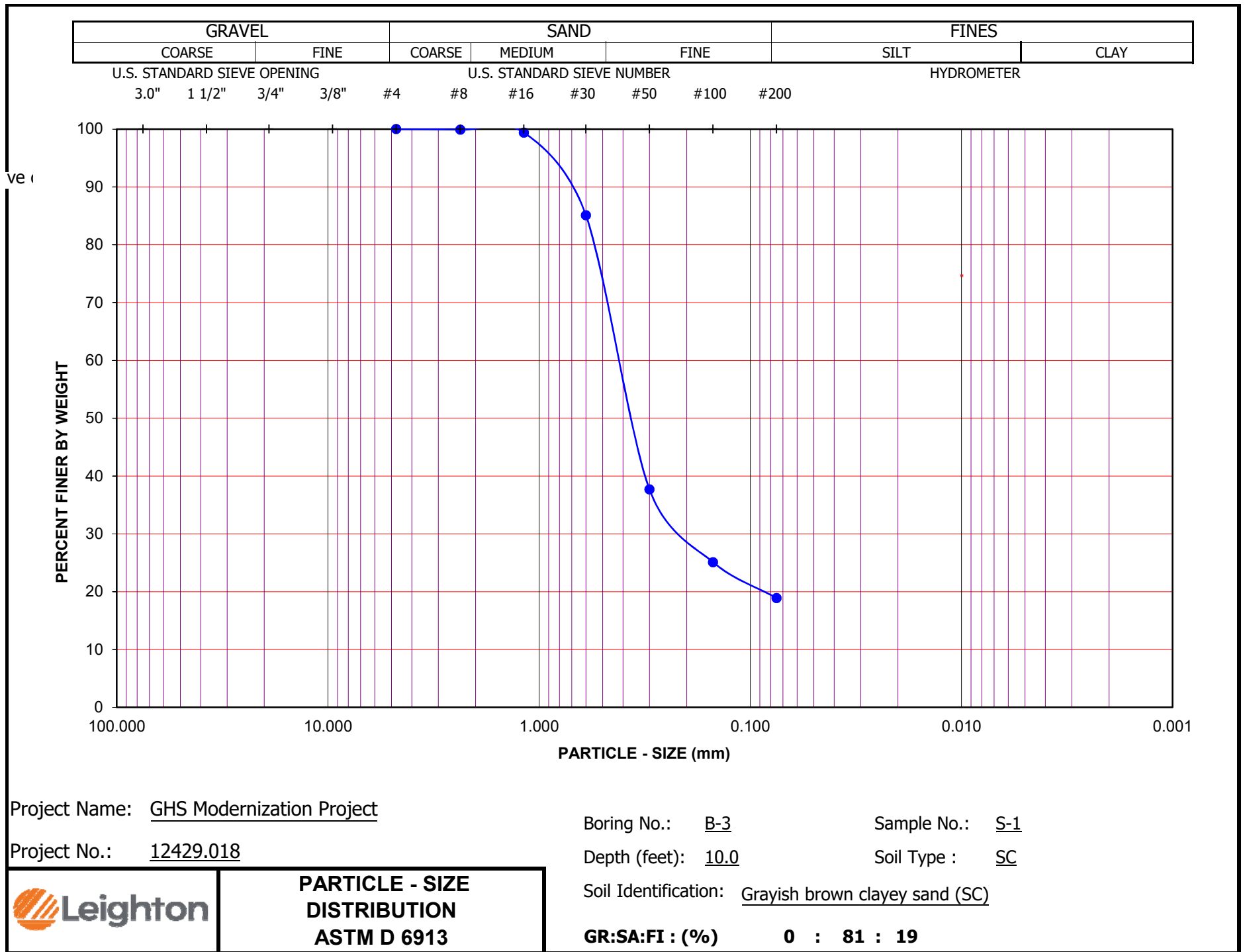
Sample No.: 1

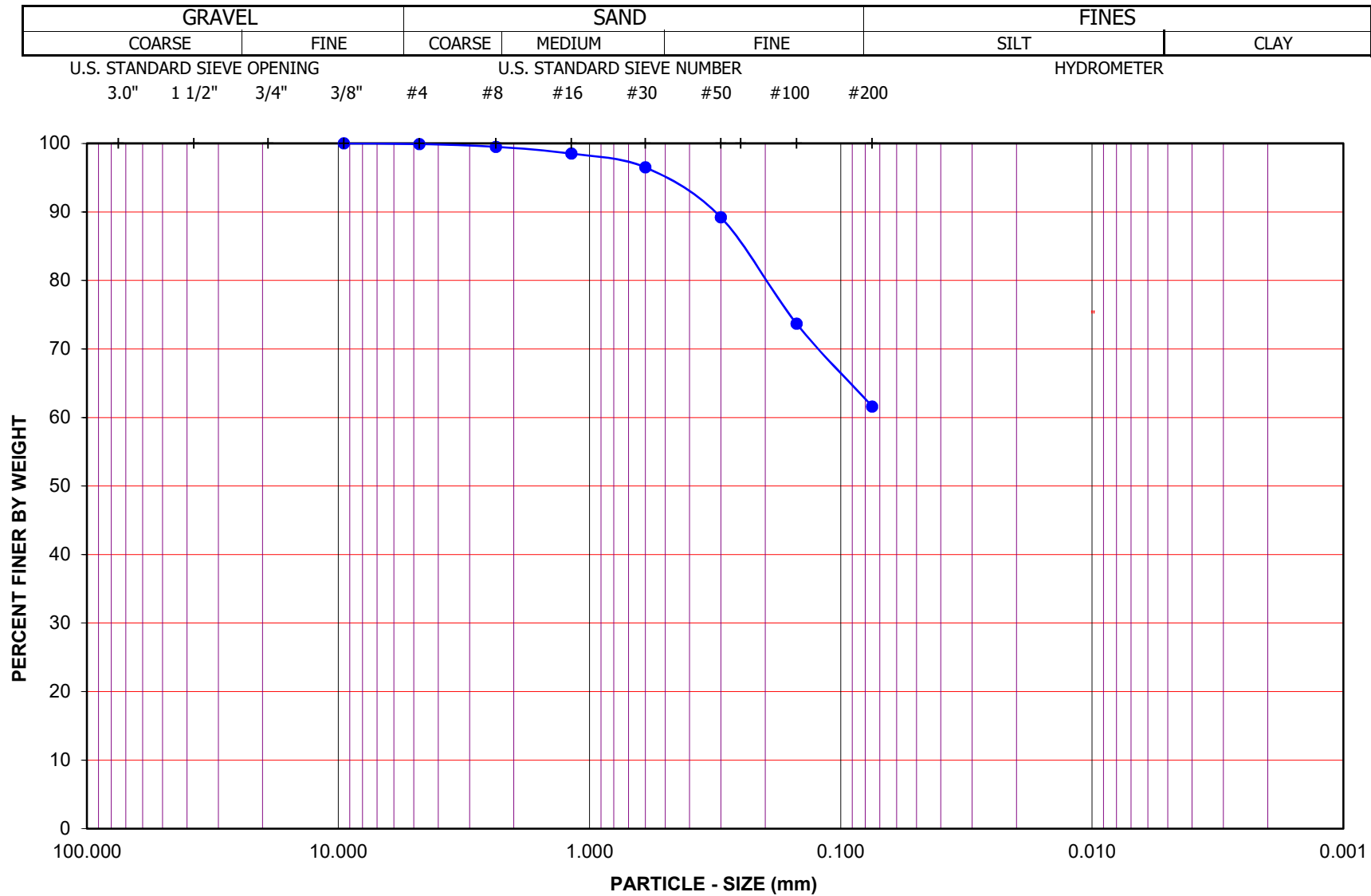
Depth (feet): 1-5

Soil Type : SC

Soil Identification: Yellow to light brown clayey sand (SC)

GR:SA:FI : (%) 1 : 70 : 29





Project Name: GHS Modernization Project

Project No.: 12429.018

Boring No.: B-4

Sample No.: 1

Depth (feet): .5-5

Soil Type : (CL)

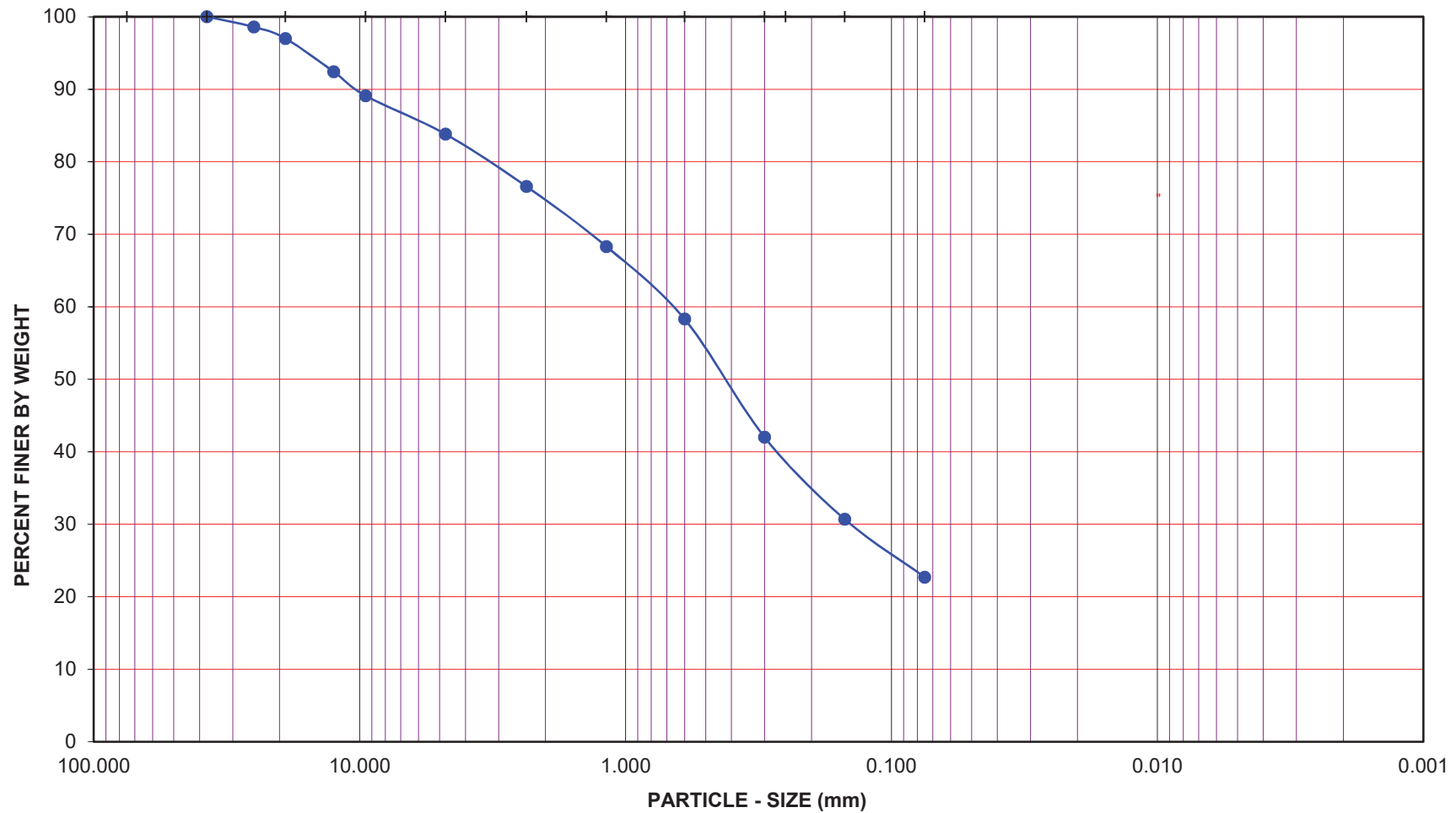
Soil Identification: Gray sandy lean clay s(CL)

GR:SA:FI : (%) 0 : 38 : 62



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

GRAVEL				SAND							FINES		
COARSE		FINE		COARSE	MEDIUM		FINE			SILT		CLAY	
U.S. STANDARD SIEVE OPENING				U.S. STANDARD SIEVE NUMBER							HYDROMETER		
3.0"	1 1/2"	3/4"	3/8"	#4	#8	#16	#30	#50	#100	#200			



Project Name: GUHSD/GHHS

Project No.: 12429.018

Boring No.: B-7

Depth (feet): 1-5

Sample No.: 1

Soil Type : (SC)g

Soil Identification: Light brown clayey sand with gravel (SC)g

GR:SA:FI : (%) **16 : 61 : 23**

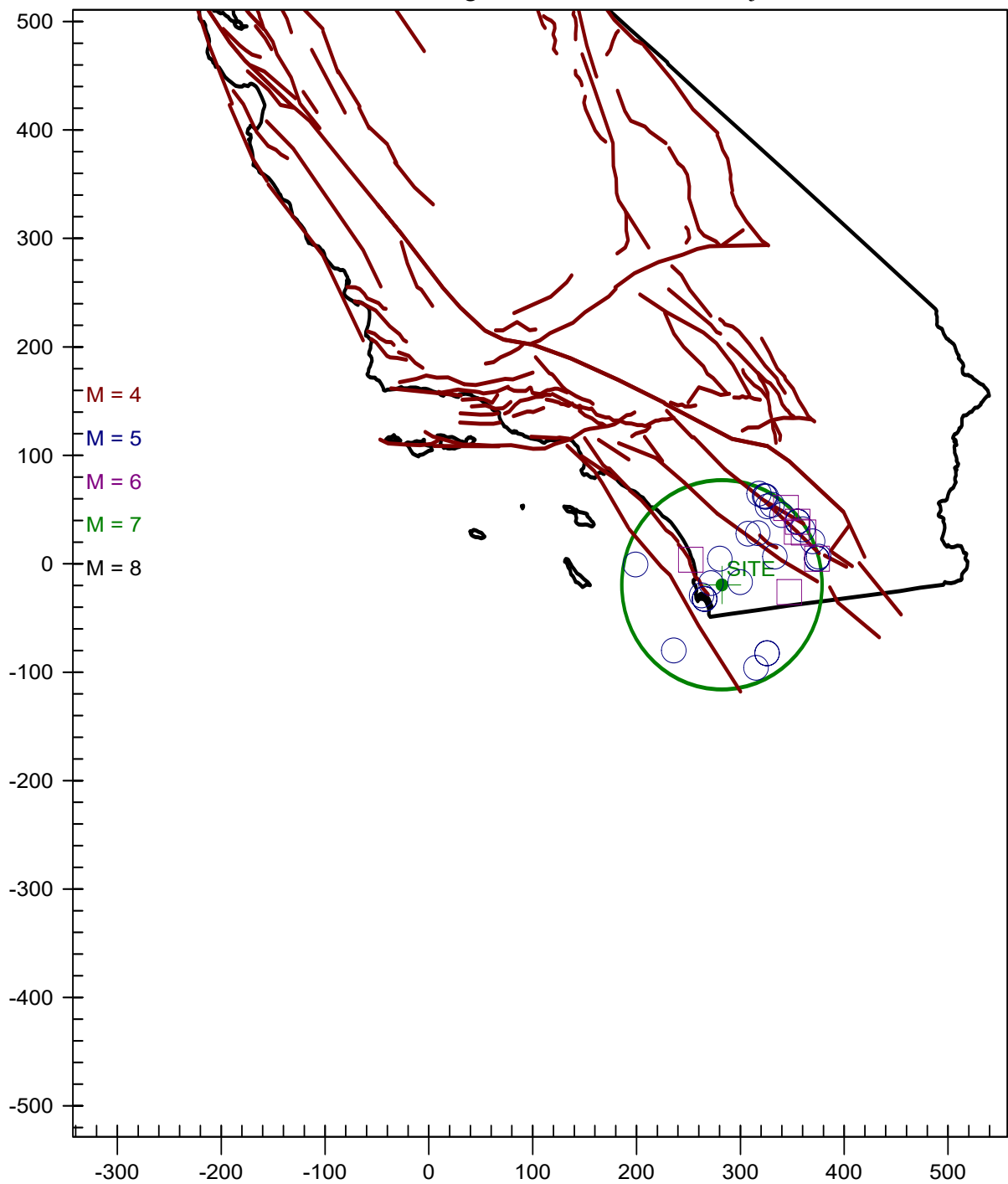


**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

APPENDIX D

Seismic Hazard Analysis

EARTHQUAKE EPICENTER MAP
GHS Kitchen and Weight Room Modernization Project



* * *

* E Q S E A R C H *

* * *

* Version 3.00 *

* * *

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 12429.018

DATE: 12-23-2021

JOB NAME: GHS Kitchen and Weight Room Modernization Project

MAGNITUDE RANGE:

MINIMUM MAGNITUDE: 5.00

MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:

SITE LATITUDE: 32.7816

SITE LONGITUDE: 116.9873

SEARCH DATES:

START DATE: 1800

END DATE: 2021

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	DEPTH	QUAKE	SITE	SITE	APPROX.
CODE	NORTH	WEST		(UTC)	(km)	MAG.	ACC.	MM	DISTANCE
-----+-----+-----+-----+-----+-----+-----+-----+-----+-----				H M Sec			g	INT.	mi [km]
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.090	VII	6.7(10.7)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.093	VII	10.9(17.6)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.057	VI	13.1(21.1)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.057	VI	13.1(21.1)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.057	VI	13.1(21.1)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.089	VII	13.6(21.8)
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	23.6(37.9)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.028	V	33.3(53.6)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.028	V	35.5(57.1)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	36.6(58.8)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.059	VI	40.3(64.9)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.021	IV	47.3(76.1)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.022	IV	47.5(76.5)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.031	V	47.5(76.5)
DMG	32.0830	116.6670	11/25/1934	818 0.0	0.0	5.00	0.020	IV	51.7(83.2)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.026	V	52.7(84.7)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.023	IV	52.8(85.0)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.030	V	53.7(86.3)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.038	V	54.0(86.9)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.021	IV	54.8(88.2)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.024	V	56.7(91.3)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.021	IV	56.9(91.6)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.018	IV	57.0(91.8)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.019	IV	57.1(91.9)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.039	V	57.1(92.0)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.026	V	57.9(93.1)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.019	IV	58.0(93.4)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.024	IV	58.0(93.4)

EARTHQUAKE SEARCH RESULTS

FILE	LAT.	LONG.	DATE	TIME	DEPTH	QUAKE	SITE	SITE	APPROX.
CODE	NORTH	WEST		(UTC)	(km)	MAG.	ACC.	MM	DISTANCE
				H M Sec			g	INT.	mi [km]
----	+	+	+	+	+	+	+	+	+
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.018	IV	58.0(93.4)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.034	V	58.0(93.4)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.036	V	58.3(93.9)
DMG	32.9670	116.0000	10/21/1942	162519.0	0.0	5.00	0.018	IV	58.7(94.4)
DMG	32.9670	116.0000	10/22/1942	181326.0	0.0	5.00	0.018	IV	58.7(94.4)
DMG	32.9670	116.0000	10/21/1942	162654.0	0.0	5.00	0.018	IV	58.7(94.4)
DMG	32.9670	116.0000	10/21/1942	162213.0	0.0	6.50	0.040	V	58.7(94.4)
DMG	33.1130	116.0370	04/09/1968	3 353.5	5.0	5.20	0.020	IV	59.6(95.9)
DMG	32.9830	115.9830	05/23/1942	154729.0	0.0	5.00	0.018	IV	59.9(96.3)

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

TIME PERIOD OF SEARCH: 1800 TO 2021

LENGTH OF SEARCH TIME: 222 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 6.7 MILES (10.7 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.7

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.093 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

a-value= 0.879

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	0.17195
4.5	38	0.17195
5.0	38	0.17195
5.5	15	0.06787
6.0	7	0.03167
6.5	3	0.01357



GHS Kitchen and Weight Room Modernization Project

Latitude, Longitude: 32.7816, -116.9873



Date	12/23/2021, 9:54:48 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	0.777	MCE_R ground motion. (for 0.2 second period)
S_1	0.284	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.933	Site-modified spectral acceleration value
S_{M1}	0.426	Site-modified spectral acceleration value
S_{DS}	0.622	Numeric seismic design value at 0.2 second SA
S_{D1}	0.284	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.334	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.401	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	0.777	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	0.847	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	1.5	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.284	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	0.308	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.6	Factored deterministic acceleration value. (1.0 second)
$PGAd$	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.917	Mapped value of the risk coefficient at short periods
C_{R1}	0.923	Mapped value of the risk coefficient at a period of 1 s

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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.

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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

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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 sheepfoot 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

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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.

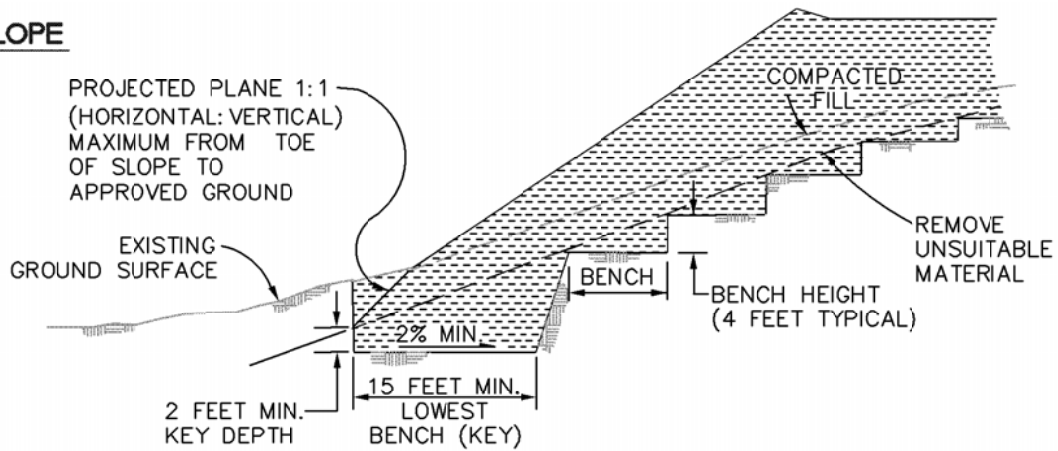
7.3 Lift Thickness

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.

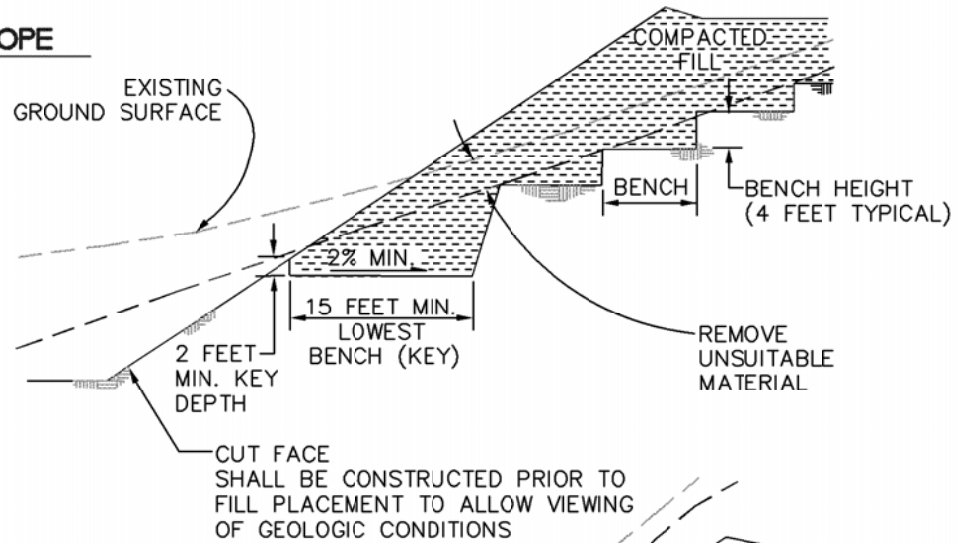
7.4 Observation and Testing

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

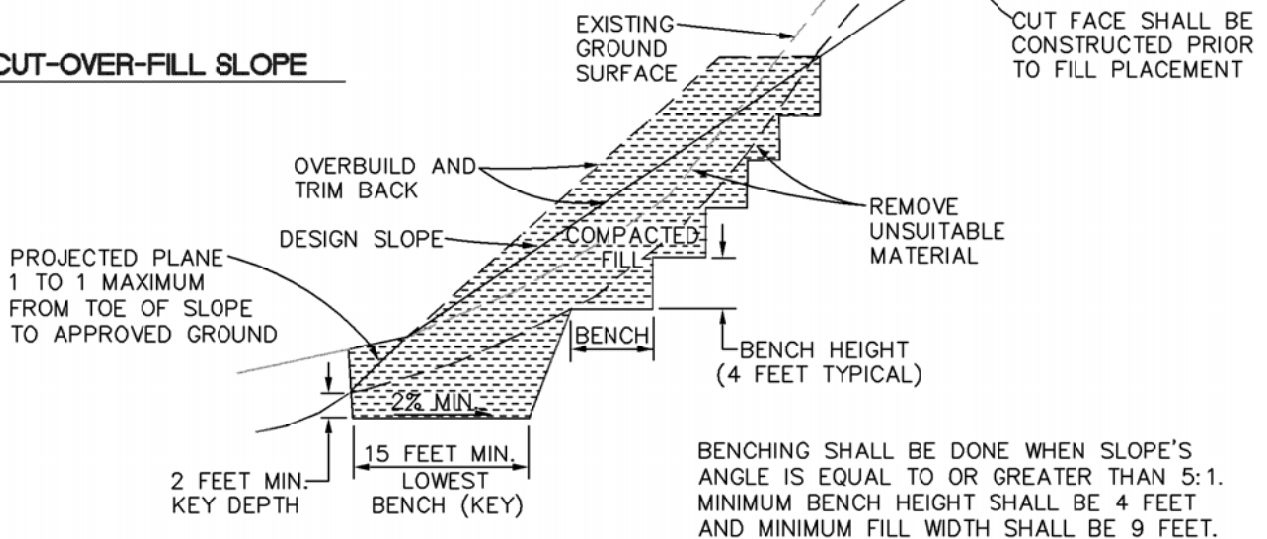
FILL SLOPE



FILL-OVER-CUT SLOPE



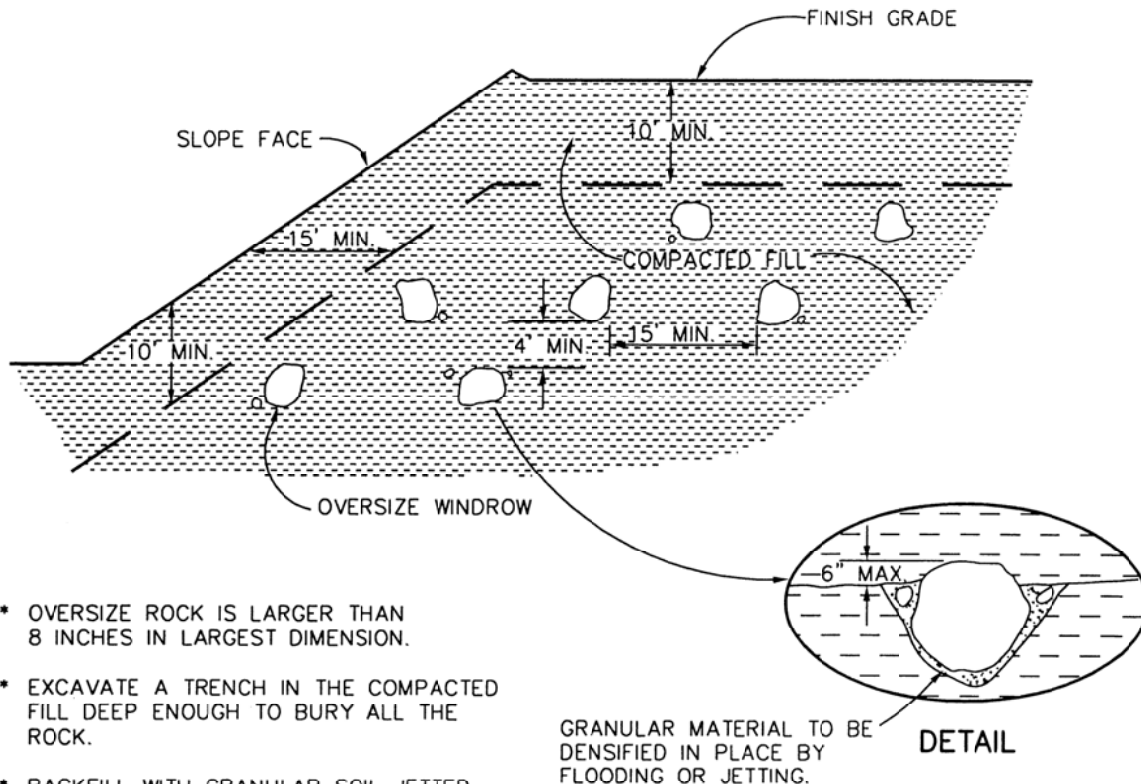
CUT-OVER-FILL SLOPE



KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

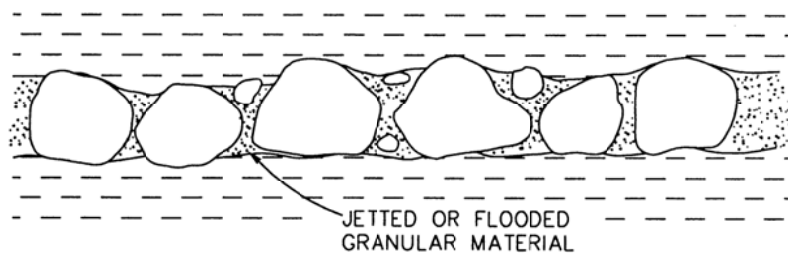




- * OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
- * EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
- * BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
- * DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
- * WINDROW OF BURIED ROCK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

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

DETAIL

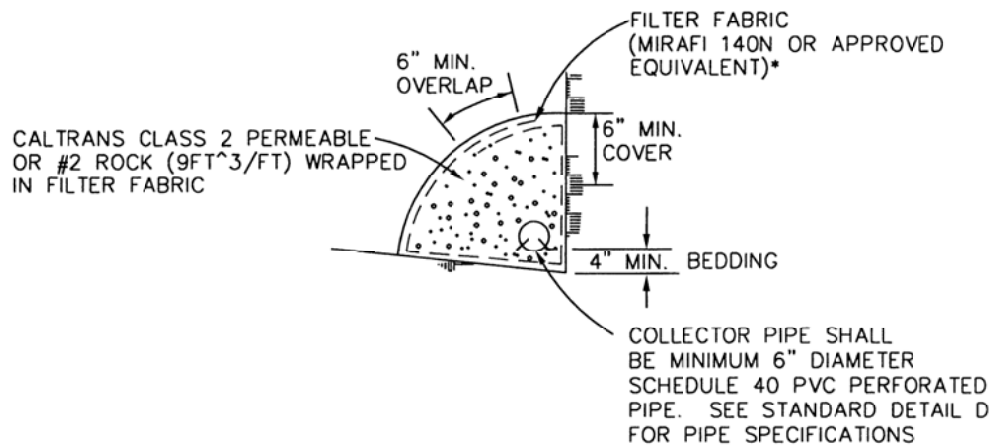
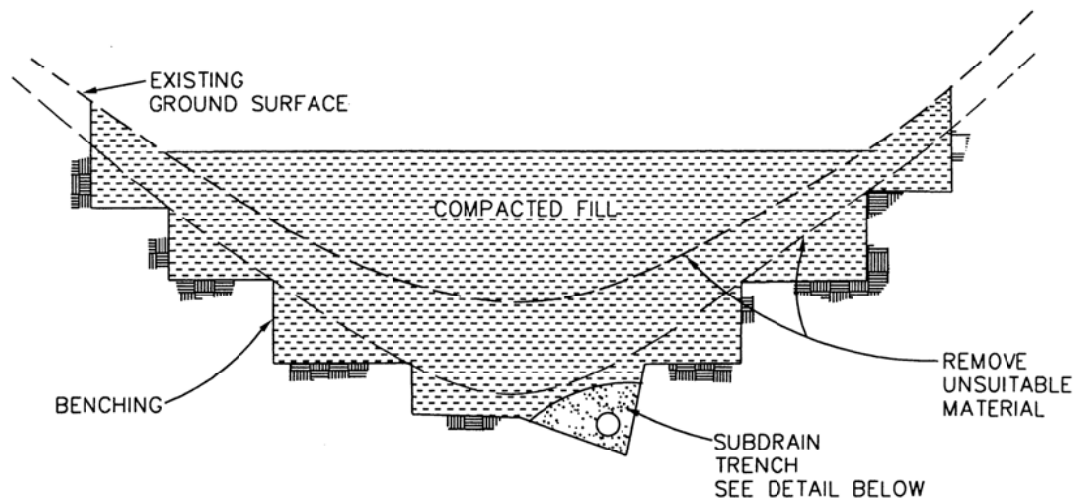


TYPICAL PROFILE ALONG WINDROW

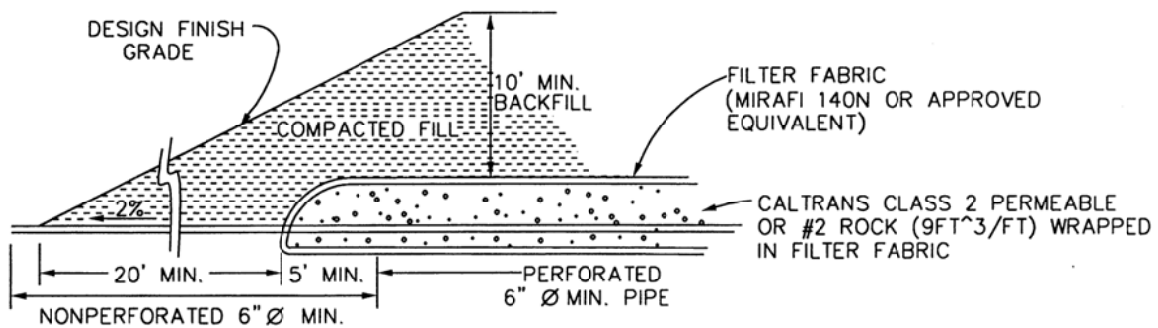
OVERSIZE ROCK DISPOSAL

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL B





SUBDRAIN DETAIL

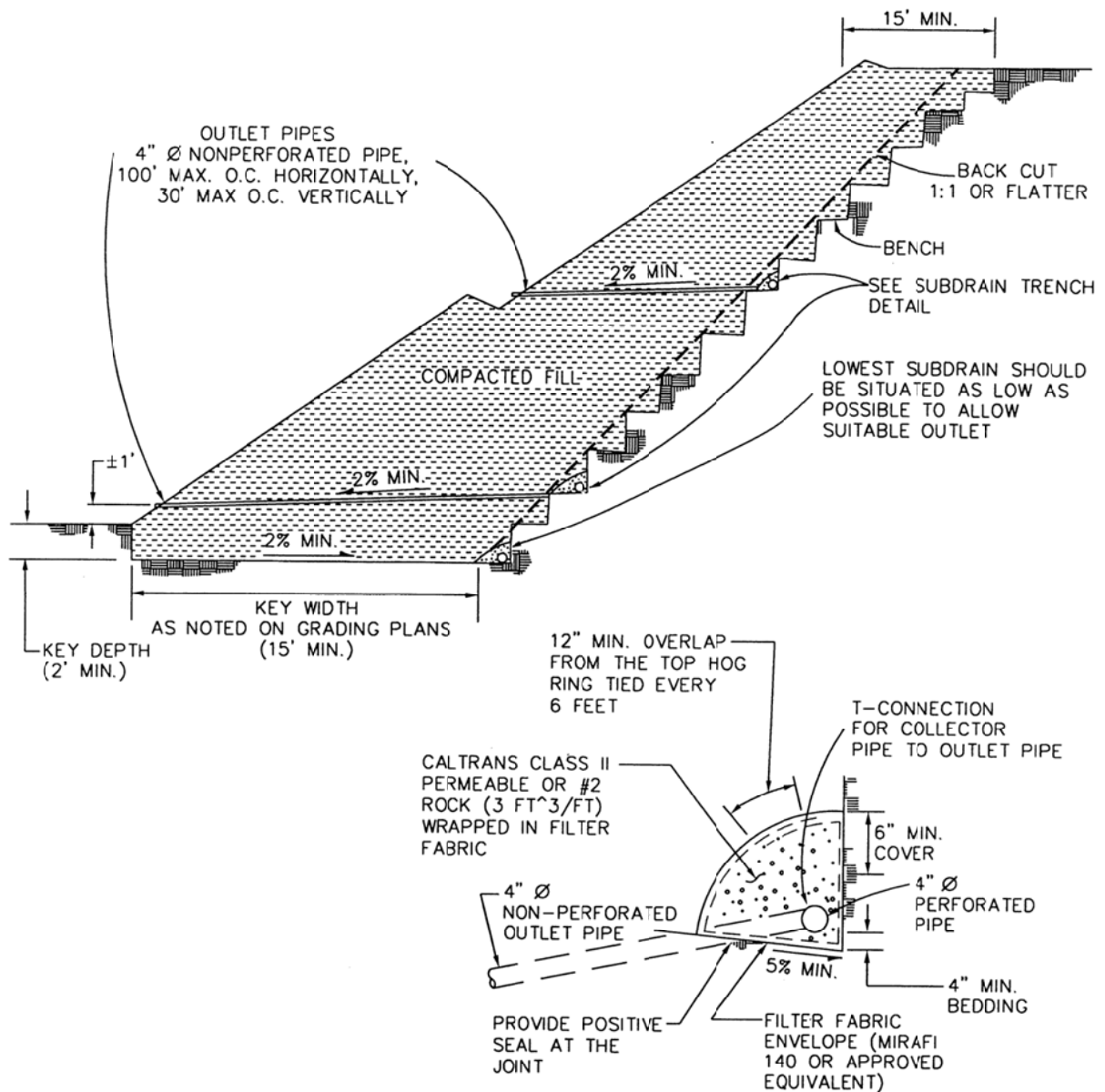


DETAIL OF CANYON SUBDRAIN OUTLET

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.

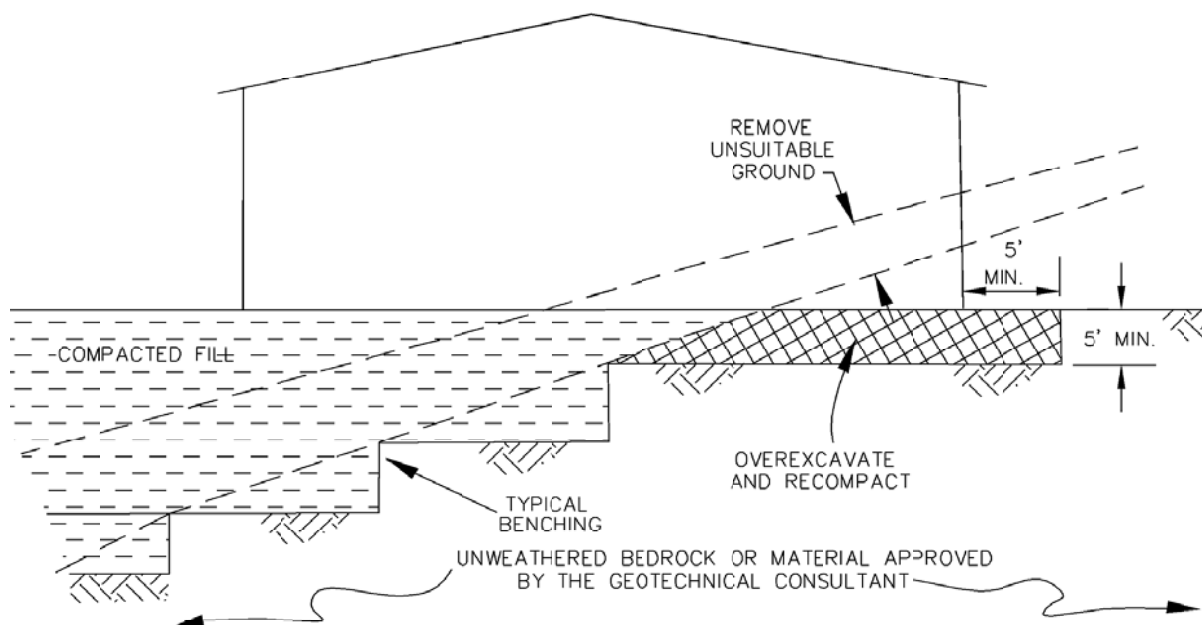
All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

**BUTTRESS OR
REPLACEMENT
FILL SUBDRAINS**

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



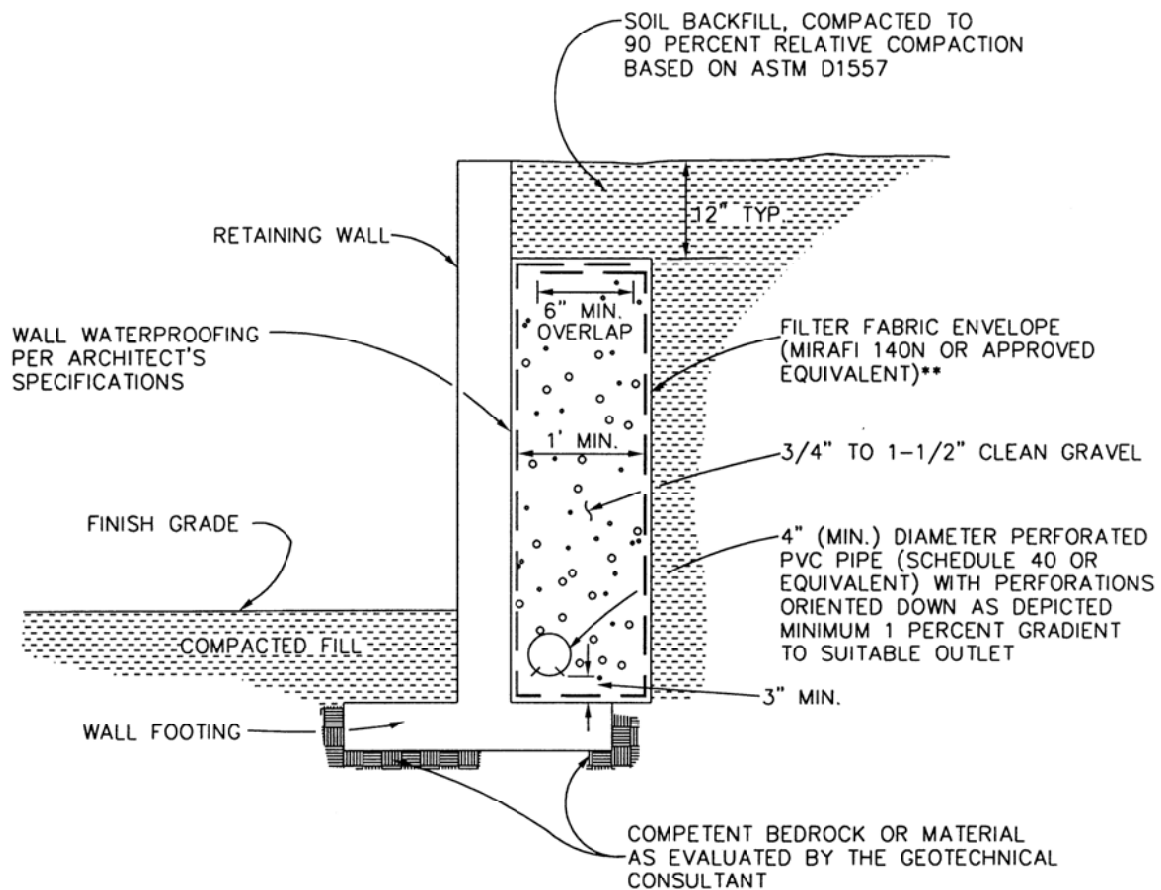
CUT-FILL TRANSITION LOT OVEREXCAVATION



TRANSITION LOT FILLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



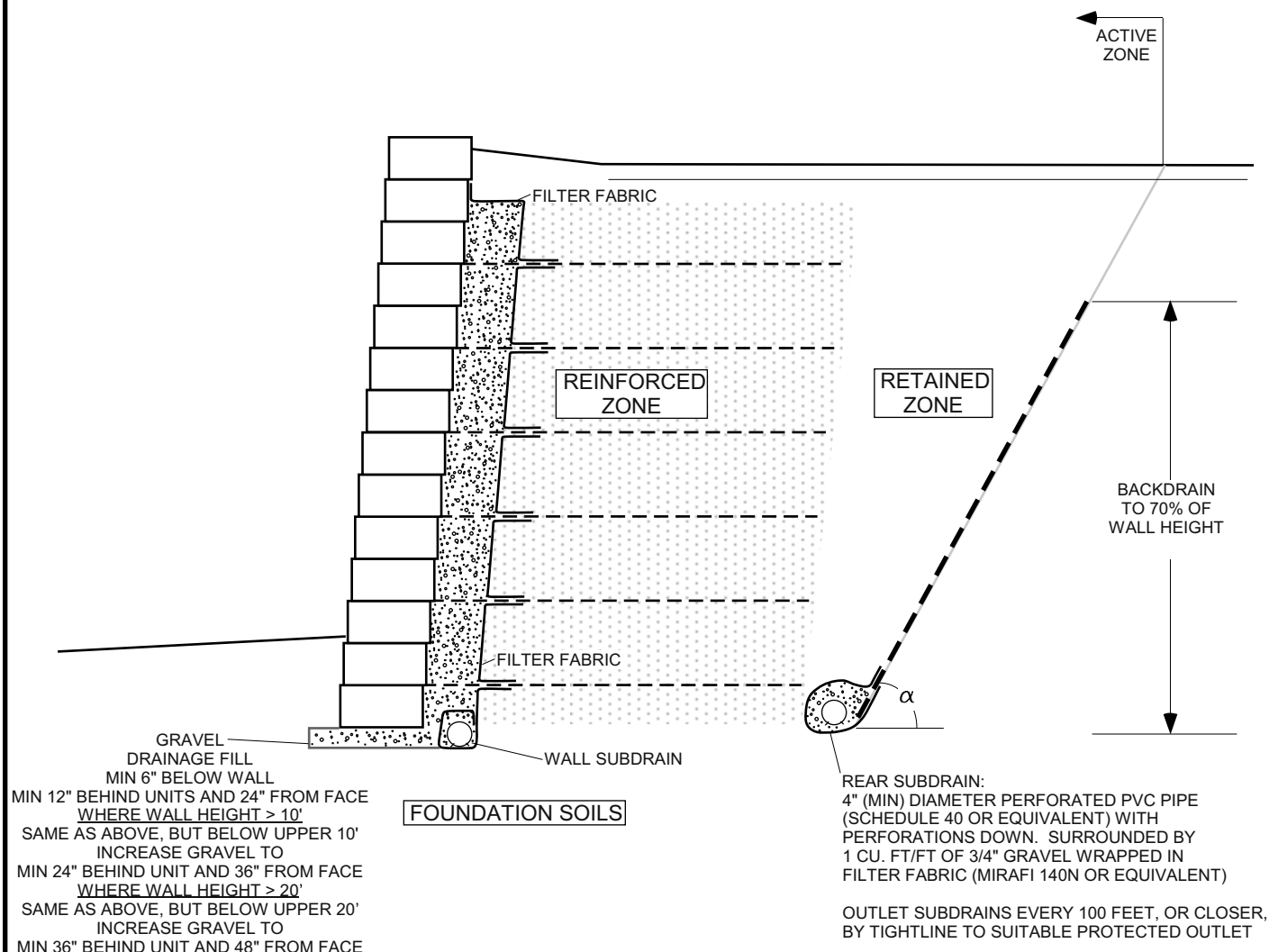


NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

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 AND LIQUID LIMIT < 40

FOR WALL HEIGHT 10 FEET OR TALLER, PLASTICITY INDEX < 6

FOR TIERED WALLS, USE COMBINED WALL HEIGHTS

FOR WALL HEIGHT > 20 FEET, REDUCE ALLOWABLE RANGE % PASSING NO. 200 SIEVE TO 0-15

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

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH AND UNIT WEIGHT REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) 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.

5) 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

