REPORT OF GEOTECHNICAL INVESTIGATION
PARKING STRUCTURE NO. 4
MIRAMAR COLLEGE CAMPUS
10440 BLACK MOUNTAIN ROAD
SAN DIEGO, CALIFORNIA
SAN DIEGO COUNTY
APN NO. 318-120-03-00

Prepared for

SAN DIEGO COMMUNITY COLLEGE DISTRICT
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Project No. SD437

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Facilities Management
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July 21, 2015
GDC Project No. SD437
Document No. 15-0098

Attention: Mr. Lance Lareau

SUBJECT: GEOTECHNICAL INVESTIGATION
Parking Structure No. 4
Miramar College Campus
10440 Black Mountain Road
San Diego, California
San Diego County
APN NO. 318-120-03-00

Dear Mr. Lareau:

In accordance with your authorization, we have completed a geotechnical investigation for the proposed Parking Structure No. 4. This report presents the results of our investigation and provides recommendations for the geotechnical aspects of site preparation and earthwork construction, and for the design of foundations, on-grade slabs, and pavements. Based on the results of our investigation, we consider the proposed construction feasible from a geotechnical standpoint provided the recommendations of this report are followed.

We appreciate this opportunity to provide our professional services. If you have any questions or require additional services, please do not hesitate to contact us.

Respectfully submitted,

GROUP DELTA CONSULTANTS, INC.

John R. Theissen, G.E.
Principal Geotechnical Engineer

James C. Sanders, C.E.G.
Senior Engineering Geologist

Distribution: (2) Addressee (llareau@sdccd.edu)

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TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
PARKING STRUCTURE NO. 4
MIRAMAR COLLEGE CAMPUS
10440 BLACK MOUNTAIN ROAD
SAN DIEGO, CALIFORNIA
SAN DIEGO COUNTY
APN NO. 318-120-03-00

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
PARKING STRUCTURE NO. 4
MIRAMAR COLLEGE CAMPUS
10440 BLACK MOUNTAIN ROAD
SAN DIEGO, CALIFORNIA
SAN DIEGO COUNTY
APN NO. 318-120-03-00

TABLE OF CONTENTS (Continued)

TABLES

FIGURES

APPENDICES

GEOTECHNICAL INVESTIGATION
PARKING STRUCTURE NO. 4
MIRAMAR COLLEGE CAMPUS
10440 BLACK MOUNTAIN ROAD
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1 INTRODUCTION

This report presents the results of our Geotechnical Investigation for the proposed Parking Structure No. 4 to be located at Miramar College Campus, 10440 Black Mountain Road in San Diego, California. The purpose of our investigation was to evaluate the existing subsurface conditions at the site as they relate to the proposed development and provide geotechnical design parameters for the planned construction. The conclusions and recommendations presented in this report are based on the subsurface conditions encountered during our field explorations, laboratory testing, engineering evaluations, and our experience with similar soils and geologic conditions.

2 SCOPE OF SERVICES

The scope of services provided during this investigation was generally as described in our Proposal No. SD15-044 dated May 28, 2015 (Document No. 15-0073). Our scope of work did not include evaluation of potentially contaminated soils. Our scope of work included the following items:

- Review of available published geologic maps, topographic maps, aerial photographs, and other literature pertinent to the geotechnical conditions at the site. Pertinent references are listed in Section 12.
- Subsurface explorations consisting of drilling nine borings. The borings were drilled to depths of up to about 4 feet below existing grade using a limited access track mounted, 6-inch auger drill rig and a limited access track mounted, 24-inch bucket auger drill rig. Drilling refusal at depths between about 1½ and 3 feet was encountered in the 6-inch auger borings. Bulk samples were collected for laboratory testing. The borings were logged by our geologist and then backfilled. The boring logs are presented in Appendix A.
- Laboratory testing of selected samples of the on-site soils to assess the pertinent physical characteristics of the materials. The testing included water-soluble sulfate and chloride content, pH, resistivity, remolded shear, Atterberg limits, sieve analysis, maximum density, optimum moisture, R-value, and expansion index. The results of the laboratory tests are presented in Appendix B.

- Evaluation of significant geologic hazards that may affect site development, including the effects of ground shaking due to seismic events on nearby active faults.
- Engineering analysis of field and laboratory data to make recommendations regarding site preparation, earthwork and backfill, remedial earthwork, foundation design parameters, soil bearing capacities, foundation settlement potential, retaining wall design, soil reactivity, and site drainage and moisture protection.
- Preparation of two copies of a bound, illustrated report summarizing our findings, conclusions, and recommendations. This report has been prepared in accordance with the California Building Code and CGS Note 48 and is signed by both a Registered Geotechnical Engineer and a Certified Engineering Geologist. The report includes the following information:

    - A description of the field and laboratory procedures used in the investigation.
    - Logs of exploratory borings summarizing the soil conditions encountered and the results of laboratory testing.
    - A plan showing the approximate locations of the borings and the geology of the site.
    - Cross-sections indicating the estimated soil and groundwater conditions at the site at depth below ground surface.
    - A discussion of the materials (including groundwater) encountered in the borings and their engineering properties.
    - An evaluation of geologic hazards including seismicity and the effects of earthquakes on the proposed structures.
    - Recommendations for mitigation of geotechnical constraints, including groundwater, undocumented fill, collapsible soils, and expansive soils.
    - Recommendations for general site grading and preparation of existing ground, fill and backfill compaction, and imported soil evaluation.
    - An evaluation of the excavation characteristics of materials that may be encountered at the site and recommendations regarding excavation stability.
    - An evaluation of the expansion potential of the on-site soils, including mitigation methods.
    - Analysis and recommendations regarding slope stability.
    - Recommendations for foundation design considering vertical and lateral load supporting capacities, allowable bearing pressure, footing depths below finish grade, passive resistance, and anticipated settlement.
    - Testing of on-site soils as an indication of corrosive or reactive soils, including soluble sulfate, chloride, pH, and resistivity.
    - Recommendations for on-grade floor slabs and for exterior concrete walkways.
    - Recommendations for design of retaining walls, including lateral earth pressures, compaction of backfill, and drainage.
    - Recommendations for pavement sections, including asphalt and portland cement concrete, for automobile parking, fire trucks, buses, service vehicles, and sanitation trucks.

3 SITE DESCRIPTION

The proposed Parking Structure No. 4 site is located on the Miramar Campus of the San Diego Community College District at 10440 Black Mountain Road in the Miramar area of San Diego, California. The new Parking Structure No. 4 will be located in the east central portion of the campus, immediately north of the existing A-1 Building and east of the B-2, B-3, and F-303-04 Buildings. The site is relatively flat, with elevations ranging between approximately 504 to 515 feet WGS84 EGM96 Geoid Datum (approximately NAVD 88 Datum). The proposed building area contains three existing classroom buildings B-2, B-3, and F-303-04 and an unnamed building not in use, as well as concrete walkways, landscape areas, and asphalt driveways. The approximate site location is shown on Figure 1, Site Location Map and Figure 2, Exploration Plan. Approximately 50 feet east of the proposed construction site, a slope descends about 5 feet to a utility access road. East of the utility access road the slope descends another 24 feet below the access road to an adjacent residential development. Total slope height is on the order of 30 feet

4 PROPOSED DEVELOPMENT

We understand the existing classroom buildings B-2, B-3, and F-303-04 and the unnamed abandoned building will be demolished in order to construct the new garage. We understand that the proposed construction will consist of a 45,600 square foot, four-story parking structure. We have assumed the structure will be concrete frame and concrete masonry with a slab-on-grade floor.

5 FIELD INVESTIGATION

Our field investigation consisted of a visual reconnaissance of the site and drilling nine borings. The borings were drilled to depths of up to about 4 feet below existing grade using a limited access track mounted, 6-inch auger drill rig and a limited access track mounted, 24-inch bucket auger drill rig. Drilling refusal at depths between about 1½ and 3 feet was encountered in the 6-inch auger borings. Bulk samples were collected from the borings for laboratory testing. The borings were logged by our geologist and

then backfilled. A description of the field investigation and the boring logs are presented in Appendix A.

6 LABORATORY TESTING

Laboratory testing included sieve analysis, Atterberg Limits, R-value, maximum density and optimum moisture, expansion index, water-soluble sulfate and chloride contents, pH, resistivity, and remolded direct shear. The Atterberg Limits are shown on the boring logs in Appendix A. Descriptions of the laboratory tests and the remaining test results are presented in Appendix B.

7 GEOLOGY AND SUBSURFACE CONDITIONS

The proposed Parking Structure No. 4 site is located within the Peninsular Ranges Geomorphic Province of California. This province extends from the Los Angeles basin to the tip of Baja California, and is characterized by a series of northwest trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and nonmarine sedimentary formations. The general geology in the vicinity of the site is shown on Figure 3, Local Geologic Map. The surficial geology of the site is shown on Figure 2, Exploration Plan.

Specifically, the site is underlain by Quaternary-age sedimentary very old paralic deposits, which are covered by residual soils and fill as discussed below. The approximate locations of our exploratory borings are shown on Figure 2, Exploration Plan. Geologic cross-sections of the site are depicted on Figures 4.1 and 4.2, Geologic Cross-Sections. The boring logs are presented in Appendix A. Generalized descriptions of the units observed, from oldest to youngest, are as follows.

7.1. Very Old Paralic Deposits, Unit 9 ($Qvop_9$)

Very Old Paralic Deposits unconformably underlie the site (Kennedy and Tan, 2005). This formation was previously designated the Lindavista Formation (Qln) (Kennedy, 1975), which is the name familiar to the design and construction industry in San Diego. As observed in the borings, the formation consists of a dense moderately to strongly cemented, cobble conglomerate with a sandy claystone (CL) and clayey sandstone (SC) matrix. The cobbles were observed to be up to approximately 6-inches in diameter. The matrix material was moist and dark yellowish brown to reddish brown in color. The Lindavista Formation was encountered at depths of approximately 1-1/2 to 3 feet. Bucket auger refusal was encountered within approximately 1-1/2 to 2-1/2 feet of penetration into the formation. One Expansion Index test completed on a sample from this formation indicated a very low expansion index. The Very Old Paralic Deposits (*Lindavista Formation)* materials have good bearing capacity and high shear strength.

7.2. Residual Soil

The residual soil observed at the site is a result of in-place weathering of the Very Old Paralic Deposits described above. Residual soil was observed in five of the nine borings drilled for the Parking Garage. The residual soil consists of a fat high plasticity clay (CH) to a low plasticity clay. The clays were dark brown to very dark brown to yellow brown or dark yellow brown, dry to moist. An expansion test completed on a sample from this soil layer indicated a borderline low to medium (45) expansion index.

7.3. Undocumented Fill ($Fill_u$)

Undocumented fill soils were encountered in six of the borings excavated at the site. The fill is likely associated with the original construction of the college campus and varies in thickness across the site from approximately 2½ feet at the southern edge of the proposed building site (B-8), to approximately 2 feet at the northeast corner of the proposed building site (B-3), to approximately 1½ feet at the northern edge of the proposed building site (B-2), to approximately one foot in the central and south eastern areas of the proposed building site (B-4, B-6,

and B-9). As observed in the borings, the fill soils are composed of silty sand (SM), gravelly silty sand (SM), and silty gravel (GM). The fill soils range in color from very dark brown, to yellowish brown to dark yellowish brown. The sandy fill soils are generally fine to coarse grained, and dry to moist. The fill soils encountered at the site may contain gravel and subrounded to subangular cobbles. The fill may be thicker beneath the existing buildings or in other areas of the site not explored. No documentation was provided to verify the engineering characteristics of the fill placement and compaction efforts. In our opinion, the fill soils should be considered compressible and not suitable for the support of new structures or fill loads unless they are removed and recompacted.

7.4. Groundwater

Groundwater was not observed in the exploratory borings drilled during this investigation. Historic groundwater data obtained wells from the United States Geologic Survey, National Water Information System are provided in Appendix D for twelve wells located in the general vicinity of the subject site covering a period from approximately 1969 through 1985. The highest reported groundwater elevation in those twelve borings was 464 feet MSL or approximately 40 feet below the elevation of the site.

Prior investigations performed for the Base Realignment and Closure of Marine Corps Air Station Miramar indicated that standing groundwater was not located in the borings within 30 feet of ground surface although in some borings shallower groundwater seepage was observed on top of the Very Old Paralic Deposit conglomerate materials. Based on a review of the data it appears that the regional groundwater elevation in the vicinity of the site is probably located at least 30 feet below the site. Shallower perched groundwater may be encountered in the future due to rainfall, irrigation, stormwater retention, or broken pipes. Since the prediction of the location of such conditions is difficult, they are typically mitigated if and when they occur.

8 GEOLOGIC HAZARDS

The subject site is not located within an area previously known for significant geologic hazards (City of San Diego, 2008). Evidence of active faulting, landslides, liquefiable soils, or collapsible soils was not encountered during this investigation. This site is located within Geologic Hazard Category 51 as designated on the City of San Diego Seismic Safety Study (Appendix E). This hazard category is described as "Level Mesas – underlain by terrace deposits and bedrock, nominal risk". Seismic hazards at the site may be caused by ground shaking during seismic events on regional active faults. Existing and potential geologic hazards are as follows:

8.1. Seismicity and Ground Motion

Seismic hazards at the site are anticipated to be caused by ground shaking during seismic events on regional active faults. Figure 5 shows the locations of known active faults within 100 kilometers of the site. Commercially-available computer software was used to evaluate potential seismicity at the site. The software determines the distance between the site and known faults based on the latitude and longitude of the site. According to TOPO! (National Geographic Holdings, 2001), the site is located at about latitude 32.90821° north and longitude 117.11861° west using NAD83 coordinates.

Based on the findings of our subsurface explorations, it is our opinion that a 2013 CBC Site Class C is most applicable to the general site conditions. An average shear wave velocity (Vs30) of 640 m/s (2096 ft/sec) was assumed for the site based on the performance of a refraction microtremor (ReMi) survey at an area at the project site crossing the approximate garage location. The results of the refraction microtremor (ReMi) survey are presented in Appendix C.

USGS 2012 IBC

Site Class: C				
Site Coefficients,	F_a:	1.034		
	F_v:	1.444		
Mapped MCE Spectral Accelerations,			S_S:	0.916
			S_1:	0.356
Adjusted MCE Spectral Accelerations,			S_{MS}:	0.947
			S_{M1}:	0.514
Design Spectral Accelerations,			S_{DS}:	0.631
			S_{D1}:	0.343

Site PGA 0.352g.

8.2. CBC Seismic Design Parameters

The following seismic parameters were determined for the on-site soils in accordance with Table 1613A.3.3 (Site Class Definitions) of the 2013 California Building Code (CBC). These parameters assume that the structure will be supported on shallow foundations founded in formational materials or on piers deriving their support in formational materials.

2013 CBC

The following Seismic Design Categories may be used
for structural design for S_1 less than 0.75.
Occupancy Category: All Categories (I through IV)
Seismic Design Category: D

8.3. Site Specific Seismic Design Parameters

Seismic hazards at the site are anticipated to be caused by ground shaking during seismic events on regional active faults. The Fault Location Map, Figure 5, shows the locations of known active faults within 100 kilometers of the site. Commercially-available computer software was used to evaluate potential seismicity at the site. These programs determine the distance between the site and known faults based on the approximate latitude and longitude of the site. According to TOPO! (National Geographic Holdings, 2001), the site is located at about latitude 32.90821° north and longitude 117.11861° west using NAD83 coordinates.

The properties of known active faults within 100 km of the site are shown in Table 1. The median deterministic response spectra for the ten closest known active faults are presented in Figure 9A. These spectra were determined using the Caltrans ARS Online tool and associated spreadsheets (Caltrans 2009). The deterministic site response is controlled by a Magnitude 7.2 event on the Rose Canyon fault zone, which is located about 13.9 km west southwest of the site. The estimated median peak ground acceleration for this event is 0.225g. The median spectral response for this event is shown in red in Figure 9A. Note that this deterministic spectrum does incorporate Caltrans near source factors.

The 84th percentile (MCE) spectrum shown in Figure 9A was obtained by multiplying the upper bound median deterministic seismic response by the scaling factors presented in Table C21.2-1 of the 2009 NEHRP Recommended Seismic Provisions. Note that these scaling factors are used to estimate the 84th percentile of the maximum rotated component of the deterministic spectral response. The 84th percentile spectrum shown in Figure 9A is taken as the 2013 CBC Deterministic MCE Response Spectrum for Site Class C.

A probabilistic seismic hazard analyses was also conducted for the site using the referenced USGS website (USGS, 2009). The uniform hazard response spectra associated with the three risk levels of interest are shown in Figure 9B (Maximum Considered, Upper Bound, and Design Basis Earthquakes). The recurrence intervals for these spectra are approximately 2,475, 975 and 475 years, respectively. Note that per the requirements of the 2013 CBC, these probabilistic spectra have also been modified using the scaling factors from Table C21.2-1 of the 2009 NEHRP Recommended Seismic Provisions. For the probabilistic analysis, the scaling factors are used to model the maximum rotated component of the spectral response. In addition, the probabilistic scaling factors (CRS and CR1) were applied to the MCE spectrum per Section 21.2.1.1 of ASCE 7-10. The 2013 CBC Probabilistic MCE Response Spectrum for Site Class C is shown in Figure 9B.

A site-specific evaluation was conducted in accordance with Section 21.2 of ASCE 7-10. The evaluation involves a comparison of the deterministic and probabilistic MCE response spectra described previously. The 2013 CBC Deterministic MCE Response Spectrum for Site Class C is shown in light blue in Figure 9C. The 2013 CBC Probabilistic MCE Response Spectrum for Site Class D is shown in dark blue in Figure 9C. The 2013 CBC Site Specific MCE Response Spectrum for Site Class C is the lower limit of these two spectra, as shown in green in Figure 9C. The site-specific design spectrum represents two-thirds of the MCE spectrum, although it may not be less than 80 percent of the spectrum from the USGS mapped ordinates. The 2013 CBC Site Specific Design Spectrum for Site Class C is shown in black in Figure 9C.

For purposes of seismic design of the proposed structure, we recommend using the site-specific design spectrum based on the short-period and one-second

spectral ordinates (S_{DS} and S_{D1}). The recommended 2013 CBC Site-Specific Design Spectrum for site Class C is shown with a black line in Figure 9C. The site-specific spectral design parameters S_{DS} and S_{D1} equal 0.625 and 0.274, respectively. The peak ground acceleration associated with the site-specific design spectrum may be taken as 40 percent of S_{DS} or 0.250g. The site-specific MCE parameters S_{MS} and S_{M1} are equal to 1½ times the design values or 0.938 and 0.411, respectively. Note that Table 2 summarizes the pertinent spectral ordinates in Figures 9A through 9C.

8.4. Surface Rupture

Surface rupture is the result of movement on an active fault reaching the surface. Figure 5 shows the relationship between known active faults in the region and the site. The nearest known active fault is the western portion of the Rose Canyon fault zone, which is located about 8.4 miles (13.6 kms) southwest of the site (Kennedy, 1975) (City of San Diego, 2008). We observed no indications of active faulting at or adjacent to the site during our investigation. There are no known active faults underlying the site or projecting toward the site. The site is not located in an Alquist-Priolo Earthquake Fault Zone. In our opinion, the probability of surface rupture due to faulting beneath the site is considered low. However, lurching and ground cracking are a possibility as a result of a significant seismic event on a nearby active fault.

8.5. Liquefaction and Dynamic Settlement

Liquefaction is a process in which soil grains in a saturated deposit lose contact after the occurrence of earthquakes or other sources of ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, the strength of the deposit is greatly diminished, and the soils may experience significant post liquefaction settlement. Liquefiable soils typically consist of cohesionless sands and silts that are loose to medium dense, and saturated. To liquefy, soils must be subjected to ground shaking of sufficient magnitude and duration. Based on previous investigations conducted at nearby sites, as well as the borings drilled for this investigation and the refraction microtremor (ReMi) survey performed at the project site, the site is underlain by dense cemented soils at the anticipated

level of the groundwater table. In our opinion, the potential for liquefaction to occur at the site is negligible.

Dynamic settlement is a process in which a deposit of loose soil densifies during the vibrations induced by the occurrence of earthquakes or other sources of ground shaking. Soils subject to dynamic settlement typically consist of cohesionless sands and silts, but such soils do not have to be saturated. To settle the soils must be subjected to ground shaking of sufficient magnitude and duration. The existing looser shallow fill soils may be subject to minor dynamic settlement. Based on the results of our dynamic seismic settlement analyses presented in Appendix F, the undocumented fills may settle up to approximately ¼ to ½ inch under a seismic event producing a Peak Ground Acceleration equal to 0.4 S_{DS} (0.400g). In order to mitigate such settlement and the potential adverse effects on the proposed structures, the parking garage should be supported on dense cemented Very Old Paralic Deposits at depth and the existing loose shallow fill soils should be compacted during site grading prior to any new construction.

8.6. Landslides and Lateral Spreads

Evidence of ancient landslides or slope instabilities was not observed at the site during our investigation. The site will not contain bluffs, steep slopes, or other topographic features susceptible to landslides after site grading. The site of the proposed Parking Structure No. 4 is located within Geologic Hazard Category 51. This hazard category is described in the City of San Diego Seismic Safety Study as "Level mesas, underlain by terrace deposits and bedrock, nominal risk". The nearest adjacent offsite slope is located within Geologic Hazard Category 53. This hazard category is described in the City of San Diego Seismic Safety Study as "Level or sloping terrain, unfavorable geologic structure, low to moderate risk".

Lateral spreading is the result of liquefaction or plastic deformation occurring on gently sloping ground during an earthquake. Typically, the event requires an unsupported, steep cut or scarp at the toe of the failure area that allows the initial lateral displacement. The site is not liquefiable. Therefore, the developed site is not expected to be susceptible to lateral spreading associated with a seismic event on a nearby active fault.

8.7. Tsunamis, Seiches, and Flooding

The site is not located within a Federal Emergency Management Agency (FEMA) 100-year floodplain or 500-year floodplain (Figure 6, FEMA Flood Insurance Rate Map).

Given the distance between the subject site and the coast (approximately 8 miles or 12.9 kms), and the site's elevation above sea level (approximately 500 feet above mean sea level), the potential for damage due to tsunamis (seismically induced waves) is considered remote. According to the Tsunami Inundation Map, (Cal EMA, 2009), the site is not located within a tsunami inundation area.

The site is not immediately adjacent to any lakes or confined bodies of water; therefore, the possibility of earthquake-induced flooding due to seiches is also negligible. However, the site is located downstream Miramar Lake Dam; therefore, there is a possibility of earthquake-induced inundation in the event of dam failure. Based on the County of San Diego dam inundation maps (SanGIS) as shown on Figure 7, the site is located within a mapped dam inundation zone.

8.8. Subsidence

The subject site is not within an area known for past cases of substantial subsidence due to fluid removal (Alfors et. al., 1973). It is our opinion that the potential for significant subsidence due to the extraction of fluids is negligible.

Soils subject to hydrocollapse, such as loose cemented silty and clayey soils were generally not noted in the borings excavated at the site. The site is not located in an area noted for hydrocollapse. Significant soil settlement associated with wetting of the subgrade materials is not anticipated. Post construction soil settlement associated with landscape irrigation is anticipated to be less than 1 inch.

9 CONCLUSIONS

Based on the results of this investigation, it is our opinion that the proposed construction is feasible from a geotechnical standpoint provided the following recommendations and appropriate construction practices are followed. Other than the location of the parking structure within a mapped dam inundation zone, no geotechnical conditions were encountered that would preclude the proposed construction. Geotechnical design and construction considerations include the following:

- The site is underlain by undocumented fill, residual soils, and Very Old Paralic Deposits. The Very Old Paralic Deposits are considered suitable for support of structural or fill loads. The existing fills and residual soils, however, are considered compressible and unsuitable for support of structural or fill loads. Remedial grading recommendations for compressible soils are provided in this report.

- If the existing fills and residual soils are removed replaced with compacted fills and building foundations lowered to bear on the Very Old Paralic Deposits, new structures may be supported on conventional shallow spread foundations and continuous wall footings with slab-on-grade floors, provided the recommendations of this report are followed. Recommendations for design of shallow foundations and slab-on-grade floors are provided in this report.

- New structures may be located across transitions from fill to dense Very Old Paralic Deposits. To reduce the potential for distress associated with differential settlement, building pads should be graded so that individual structures do not straddle such cut/fill transitions. Remedial grading recommendations to mitigate cut/fill transitions are provided in this report.

- The on-site Very Old Paralic Deposits, in general, appear suitable for re-use in compacted structural fills, for use as finish grade soils, or as landscape fills. The on-site fills and Very Old Paralic Deposits will probably not be suitable for re-use in wall backfill or trench backfill due to the presence of cobbles. Excavated soils with significant amounts of cobble may require processing prior to placement as fill to remove the oversize cobble. The on-site fills and Very Old Paralic Deposits may have a low expansion to medium expansion potential depending on the amount of clay in the soil. The onsite residual soils will probably have a high expansion potential and it is recommended that the residual soils be removed and disposed offsite.

- In general, excavations at the site should be achievable using standard heavy earthmoving equipment in good-working order with experienced operators.

However, localized concretions may require extra excavation effort or may encounter excavation refusal. These excavations may also generate oversized material that will require extra effort to crush or export from the site. In addition, it should be anticipated that excavations in conglomerate will generate a significant amount of rock. Deeper trench excavations for sewers or other utilities may encounter cemented Very Old Paralic Deposits that may require ripping or rock breakers to excavate.

- Shallow groundwater was not encountered in the borings excavated during this investigation. Perched groundwater may develop in the future due to irrigation or leaking utilities. Groundwater is not anticipated to significantly impact the proposed construction.

- Evidence of existing slope instabilities, or landslides, was not encountered during our investigation. Temporary construction slopes and permanent slopes may be susceptible to surficial erosion by wind, rainfall, or runoff. Recommendations to enhance surficial slope stability are provided later in this report.

- There are no known active faults underlying the project site. Potential seismic hazards at the site would more likely be associated with ground shaking associated with seismic events along regional active faults. Seismic shaking hazards are typically mitigated through building designs in accordance with the California Building Code.

- The proposed building(s) may be supported on conventional shallow footings provided the recommendations of this report are followed.

10 RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction as well as geotechnical recommendations for the design of the proposed structures and improvements. These recommendations are based on empirical and analytical methods typical of the standard-of-practice in southern California. If these recommendations appear not to address a specific feature of the project, please contact our office for additions or revisions to the recommendations.

10.1. Plan and Specification Review

Proposed building plans were not available for this investigation. We recommend that grading plans, foundation plans, and earthwork specifications

be reviewed by Group Delta Consultants, Inc. (Group Delta) prior to finalization to evaluate conformance of the plans with the intent of the recommendations of this report. Significant changes in the locations or elevations of the proposed structures or pad grades may require additional geotechnical evaluation.

10.2. Construction Observation and Testing

Group Delta should observe site grading and foundation excavations. Group Delta should provide geotechnical observation and testing services continuously during construction. Such observations are considered essential to identify field conditions that differ from those anticipated by the preliminary investigation, to adjust designs to actual field conditions, and to determine that the grading is accomplished in general accordance with the recommendations of this report. Recommendations presented in this report are contingent upon Group Delta performing such services. Our personnel should perform sufficient observation and testing during construction to support our professional opinion as to compliance with compaction specifications. Should Group Delta not be retained as the geotechnical consultant during construction, Group Delta would cease to be the engineer of record for this project.

10.3. Earthwork

Earthwork is anticipated to include remedial grading, excavations for foundations, temporary excavations for underground utilities, and placement and compaction of fill and backfill. Grading and earthwork should be conducted in accordance with the current CBC and with the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by the geotechnical consultant during grading.

10.3.1. Site Preparation

General site preparation should include the removal of unsuitable and deleterious materials, existing structures, or other improvements from areas that will be subjected to structural or fill loads. Clearing and

grubbing should consist of the removal of vegetation including brush, grass, weeds, wood, tree roots, and otherwise deleterious materials from areas to be graded. Clearing and grubbing should extend to the limits of grading. Unsuitable materials include vegetation, trash, construction debris, highly organic soil, highly expansive clays, contaminated soils, or other undesirable materials. Removed materials should be hauled offsite and legally disposed.

The removal of unsuitable materials should be conducted under the observation of the geotechnical consultant to evaluate the competency of the exposed materials for support of structural and fill loads. The excavation of unsuitable materials should be conducted in a way that minimizes the disturbance of competent materials.

Structures or improvements within the grading limits, that are not to be saved for future use, should be demolished and legally disposed off-site. Subsurface improvements or obstructions that are to be removed should be excavated and hauled offsite. The resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Demolition of pipelines may consist of capping or rerouting at the project perimeter, and removal within the project perimeter. If appropriate, abandoned pipelines may be filled with grout or slurry cement as recommended by, under the observation of, the geotechnical consultant. Structures or man-made improvements to be saved should be protected from damage by the contractor.

10.3.2. Remedial Grading

We recommend that remedial grading be performed for compressible soils and expansive soils. Excavation bottoms should be observed by Group Delta personnel to confirm that suitable materials are exposed and to evaluate the need for additional removals. If removals beneath the proposed building(s) result in transitions from formational materials to fill, the formational materials should be over-excavated to provide a uniform fill thickness beneath the entire structure and reduce the potential for differential settlement (Figure 8).

Compressible Soils: Compressible soils should be removed beneath structures, structural fills, exterior flatwork, pavement areas, and wherever the existing soils are disturbed due to demolition of existing structures or improvements. Compressible soils are anticipated to include undocumented fills, residual soils, weathered formational materials, or other soil subject to settlement under increased loads, wetting, or bio-degradation. Compressible soils (fill/residual soil) up to about 3 feet deep were encountered in our borings; deeper compressible soils may exist in areas of the site not explored. Remedial grading should consist of complete removal of compressible soils until competent paralic soils or formational materials are exposed. Remedial excavations should include all areas that will support structures, improvements, or fill. The excavation bottoms should be observed by Group Delta personnel to evaluate the need for additional removals.

Expansive Soils: There is a potential that the onsite fill soils and residual soils, if encountered, may be expansive. Expansive soil heave may cause differential movement and distress to foundations and flatwork. For soils with medium to high expansion potentials, differential movement across control joints of more than 1 inch is not unusual. There are several alternatives available to mitigate the presence of expansive soils beneath heave-sensitive foundations and flatwork. Alternatives include removal of the expansive soil within a select zone beneath structures and flatwork and replacing them with soils having a very low expansion potential, or lime stabilizing the soils. Recommendations for both alternatives follow.

One method consists of removing the shallow expansive soils beneath buildings and flatwork, and replacing them with material having a very low expansion potential. We recommend that soils having an expansion index (EI) greater than 20 not be allowed within the upper 5 feet of finish building pad subgrade or within the upper 2 feet of finish flatwork subgrade. Removals should extend at least 5 feet beyond the perimeter of building foundations and at least 2 feet beyond exterior flatwork areas.

Another method consists of mixing lime with the existing clay soils to reduce their expansion potential and increase their bearing capacity. We recommend that lime stabilization be conducted wherever expansive soils exist within the upper 5 feet of finish building subgrade and within the upper 2 feet of finish flatwork subgrade. The lime stabilization should extend at least 5 feet beyond the perimeter of building foundations and at least 2 feet beyond flatwork areas. Typical lime application rates for permanent lime stabilization range from about 5 to 7 percent hydrated lime by dry weight of soil. Should this alternative be selected, Group Delta should be notified so that the actual lime mix design may be performed in accordance with ASTM D6276 prior to implementation.

10.3.3. Temporary Excavations

Temporary excavations, such as for remedial grading or utility excavations, are anticipated to be no more than 10 feet in depth and are expected to be stable provided they are laid back in accordance with our recommendations or shored. All excavations should conform to Cal-OSHA guidelines. Workmen should be protected from falling rocks and caving soils in accordance with Cal-OSHA requirements. Temporary excavations 3 feet deep or less may be made vertically. Temporary excavations up to 10 feet deep in undocumented fill should be laid back no steeper than 1:1, or shored, prior to allowing workers to enter. Temporary excavations up to 10 feet deep in formational materials should be laid back no steeper than ¾:1. Should deeper temporary excavations be required, Group Delta should be notified so that additional recommendations may be provided.

Where sloped excavations are used, the top of the slope should be barricaded to prevent loads within 7 feet of the top of the slope. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of the slopes to prevent runoff water from entering the excavation and eroding the slope faces. Temporary excavations that encounter seepage or other potentially adverse conditions should be evaluated by the geotechnical consultant on a case-by-case basis during grading. Remedial measures may include shoring or

reducing the inclination of the temporary slope. Where temporary excavations extend below a plane inclined at 1½:1 (horizontal to vertical) downward from the outside bottom edge of existing structures or improvements, shoring is recommended.

For temporary excavations that will be shored, but not braced with tiebacks or struts, we recommend using a triangular pressure distribution for calculating earth pressures. For design of cantilevered shoring, an active earth pressure equal to that of a fluid with a density of 35 pcf may be used for level retained ground, plus any groundwater pressure encountered in the excavation and any surcharge loads resulting from loads placed above the excavation and within a 1:1 plane extending upward from the base of the excavation. If surcharge or groundwater pressures are anticipated, or braced shoring be used, Group Delta should be contacted for additional design values. The active earth pressure condition assumes that the shoring will deflect at the top about 0.2 percent of the shoring height. To reduce deflection, a greater lateral earth pressure may be used. For design of soldier piles embedded in compacted fill or formational materials, an allowable passive pressure of 300 psf per foot of embedment (over twice the pile width) up to a maximum of 5,000 psf may be used. Soldier piles should be spaced at least two pile diameters on center.

10.3.4. Structural Fill Material

Materials being considered for use in compacted fills should be evaluated by Group Delta prior to placement. In general, the onsite soils may be used in the required fills, less any rocks larger than 3 inches in greatest dimension, or unsuitable or deleterious materials described previously. The existing asphalt concrete pavement may be pulverized and incorporated into fills or used as base material. Soils that have an expansion index greater than 20 should not be used within the upper 5 feet of finish building pad subgrade or within the upper 2 feet of finish hardscape subgrade. Soils that have an expansion index greater than 20 should not be used as retaining wall backfill.

During grading operations, soil types other than those evaluated in the geotechnical report may be encountered. Group Delta should be notified to evaluate the suitability of these soils for use as fill and as finish grade soils. Imported fill sources, if needed, should be observed prior to hauling onto the site to determine their suitability for use. Representative samples of imported materials and onsite soils should be tested by the geotechnical consultant to evaluate their engineering properties for the planned use. Imported fill soils should have an expansion index of no more than 20.

10.3.5. Oversized Material

Excavations in the cobble fill and cemented Very Old Paralic Deposits or Friars Formation may generate oversized material. Oversized material is defined as rocks or cemented soil clasts greater than 3 inches in largest dimension. Based on the anticipated construction, there does not appear to be suitable space on-site for disposal of oversized material within the fills. Oversized materials should be broken down to no greater than 3 inches in largest dimension for use in fill, used as landscape material, or disposed off-site.

10.3.6. Fill Compaction

After making the recommended removals and prior to fill placement, the exposed ground surface should be observed by Group Delta. Any remaining dry, loose, or soft materials should also be removed until a stable, unyielding condition under equipment loads is achieved. The exposed ground surface should be scarified to a depth of approximately 8 inches, brought to slightly above optimum moisture content, and compacted to at least 90 percent of the maximum dry density obtained using ASTM D1557 as a guideline. Where the removals expose undisturbed dense cemented Very Old Paralic Deposits, scarifying and recompaction will not be required.

All fill and backfill should be placed at slightly above optimum moisture content using equipment that is capable of producing a uniformly compacted fill throughout the entire fill lift. Fill soils at less than optimum moisture should have water added and the fill mixed to result in material that is uniformly above optimum moisture content. Fill soils that are too wet should be aerated or mixed with drier material to achieve uniformly moisture-conditioned soil. *Flooding or jetting should not be permitted as a method of compacting fill or backfill.*

The fill and backfill should be placed in horizontal lifts at a thickness appropriate for the equipment spreading, mixing, and compacting the material, but generally should not exceed 8 inches in loose thickness. The minimum relative compaction recommended for fill and backfill is 90 percent of the maximum dry density obtained using ASTM D1557. The upper 1 foot of pavement subgrade should be compacted to at least 95 percent of maximum dry density just prior to constructing the pavement section. Fills below building slabs may be compacted to 95 percent of maximum dry density to reduce differential settlement between the slab and foundations supported on formational materials. Group Delta personnel should perform sufficient observation and testing so that an opinion can be rendered as to the compaction achieved.

10.3.7. Bulk/Shrink Estimates

Based on estimated densities of compacted fills, the existing undocumented fills may shrink on the order of 5 to 10 percent when over-excavated and recompacted. The existing Very Old Paralic Deposits may bulk on the order of 0 to 10 percent when over-excavated and recompacted. It should be noted, however, that bulking and shrinking potential can vary considerably based on the variability of the in-situ densities of the materials in question. The estimates for bulking and shrinkage provided are engineers' opinions of the ranges of bulking and shrinkage, which may occur and are not intended to be used to estimate the quantity of fill materials required at the site.

10.3.8. Slopes

All new permanent slopes should be constructed no steeper than 2:1 (horizontal to vertical). All slopes are subject to creep, whether the slopes are natural or man-made. Slope creep is the slow, down-slope movement of the near surface soil along the slope face. The degree and depth of movement is influenced by soil type and the moisture conditions. This movement is typical in slopes and is not considered a hazard. We recommend that settlement sensitive hardscape and improvements not be constructed within 10 feet of the top of slopes.

All slopes may be susceptible to surficial slope failure and erosion given substantial wetting of the slope face. The surficial stability may be enhanced by providing good site drainage. The site should be graded so that water from the surrounding areas is not allowed to flow over the top of the slope. Diversion structures should be provided where necessary. Surface runoff should be confined to gunite-lined swales or other appropriate devices to reduce the potential for erosion. We recommend that slopes be planted with vegetation that will increase their stability. Ice plant is generally not recommended. We recommend that vegetation include woody plants, along with ground cover. All plants should be adapted for growth in semi-arid climates with little or no irrigation. A landscape architect should be consulted in order to develop a specific planting palate suitable for slope stabilization.

10.4. Surface Drainage

Foundation and slab performance depends greatly on how well the runoff waters drain from the site. This is true both during construction and over the entire life of the structure. The ground surface around structures should be graded so that water flows rapidly away from the structures without ponding. The surface gradient needed to achieve this depends on the prevailing landscape. In general, we recommend that pavement and lawn areas within 5 feet of buildings slope away at gradients of at least 2 percent. Densely vegetated areas should have minimum gradients of at least 5 percent away from buildings

in the first 5 feet. Densely vegetated areas are considered those in which the planting type and spacing are such that the flow of water is impeded.

Planters should be built so that water from them will not seep into the foundation, slab, or pavement areas. Roof drainage should be channeled by pipe to storm drains, or discharged at least 10 feet from buildings. Site irrigation should be limited to the minimum necessary to sustain plants. Should excessive irrigation, surface water intrusion, water line breaks, or unusually high rainfall occur, saturated zones or "perched" groundwater may develop in the underlying soils.

10.5. Foundation Recommendations

The recommendations provided herein are considered generally consistent with methods typically used in southern California. Other alternatives may be available. Our recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by the structural engineer. The design of the foundation system should be performed by the project structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

The proposed building may be supported on conventional shallow footings. Covered walkways or other pole-type structures may be supported on cast-in-drilled-hole (CIDH) piles or deepened spread footings.

Foundation excavations should be observed by Group Delta. Such observations are essential to identify field conditions that differ from those anticipated and to adjust designs to actual field conditions. Recommendations presented herein are contingent upon Group Delta performing such services. Loose soils at the bottom of foundations should be removed prior to placing steel and concrete.

The following design parameters assume that the foundations for the proposed building will consist of shallow spread footings or continuous wall footings bearing entirely on compacted fill soils with a very low expansion potential

(Expansion Index of 20 or less) or embedded at least 6 inches into dense cemented Very Old Paralic Deposits.

10.5.1. Conventional Foundations

Allowable Soil Bearing:	2,000 psf for footings supported on fills, compacted as recommended herein, or 4,000 psf for footings imbedded at least 6 inches into dense cemented Very Old Paralic Deposits as recommended herein (estimated factor of safety > 3). Allowable soil bearing may be increased by 500 lbs/ft^2 for each foot of depth below the minimum and 250 lbs/ft^2 for each foot of width beyond the minimum up to a maximum of 7,500 lbs/ft^2 on formation or 4,000 lbs/ft^2 on fill Allow a one-third increase in the allowable bearing for short-term wind or seismic loads.
Minimum Footing Width:	24 inches.
Minimum Footing Depth:	24 inches below adjacent finish soil grade, building slab, or pavement finish grade, whichever is lower.
Lateral Passive Resistance:	350 lbs/ft^2 per foot of embedment in fills compacted as recommended herein or 450 lbs/ft^2 per foot of embedment in dense cemented Very Old Paralic Deposits (estimated factor of safety > 1.5). Allow a one-third increase for short-term wind or seismic loads. The passive resistance of the materials may be combined with the frictional resistance without reduction in evaluating the total lateral resistance.
Coefficient of Friction:	0.35 between the bottom of slab and fills compacted as recommended herein; or between the bottom of footings and either fills, compacted as recommended herein, or dense cemented Very Old Paralic Deposits (estimated factor of safety > 1.5).
Settlement:	Foundations should be designed for 1-inch of differential settlement over a distance of 40 feet.
Minimum Reinforcement:	Two No. 4 bars at both top and bottom in continuous footings. Actual required reinforcement should be determined by the structural engineer.

10.5.2. Deepened Spread Footings

The following design parameters assume that the foundations will be embedded in compacted fill or embedded at least 6 inches into dense cemented Very Old Paralic Deposits at depths between about 4 and 8 feet below finish grade. Deeper foundations will probably require cast-in-drilled-hole (CIDH) pile foundations. For deepened spread footings, uplift loads may be resisted by the weight of the footing and the weight of the compacted backfill overlying the footing. A one-third increase in the soil bearing, frictional uplift, and passive pressure values may be used for short-term wind or seismic loads.

Allowable Soil Bearing:	3,000 lbs/ft^2 for foundations bearing on engineered low expansion fill, compacted as recommended herein, or alternatively, 5,000 lbs/ft^2 for foundations bearing on dense undisturbed cemented Very Old Paralic Deposits (estimated factor of safety > 3). Allowable soil bearing may be increased by 500 lbs/ft^2 for each foot of depth below the minimum and 250 lbs/ft^2 for each foot of width beyond the minimum up to a maximum of 7,500 lbs/ft^2 on formation or 4,000 lbs/ft^2 on fill
Minimum Diameter/Width:	24 inches.
Minimum Depth:	48 inches below lowest adjacent grade.
Soil Unit Weight:	120 lbs/ft^3.
Frictional Pier Uplift:	100 lbs/ft^2 along perimeter of pier below a depth of 2 feet.
Passive Pressure:	300 lbs/ft^2 per foot of embedment based on an estimated lateral deflection up to ½-inch.
Differential Settlement:	½-inch between adjacent piers or footings.

10.6. Slope Setback

Foundation setbacks should conform to Figure 1808A.7.1 of the 2013 CBC. In general, the foundations for all structures should be setback from descending slopes a minimum of 10 feet measured horizontally from the outside bottom edge of the footing to the slope face. The recommended foundation setback for structures located near the tops of slopes may be achieved by deepening the foundation. A 5-foot foundation setback may be used for utility structures situated near minor slopes less than 4 feet in height and for screen wall foundations or fences.

10.7. Conventional On-Grade Slabs

10.7.1. Interior Slabs

On-grade building slabs should be supported on either compacted fill prepared as recommended in this report or on dense undisturbed Very Old Paralic Deposits. The subgrade soils should have a very low expansion potential (Expansion Index less than 20). Slabs should be designed for the anticipated loading. If an elastic design is used for slabs constructed on-grade, a modulus of subgrade reaction of 250 lbs/in^3 may be used for compacted fill prepared as recommended in this report or a modulus of subgrade reaction of 400 lbs/in^3 may be used for dense undisturbed Very Old Paralic Deposits. Slab thickness, control joints, and reinforcement should be designed by the project structural engineer for soils exposed at slab and foundation subgrade in conformance with the requirements of the 2013 CBC and the recommendations contained in the current American Concrete Institute (ACI) *Guide for Concrete Floor and Slab Construction* (ACI 302.1R-04). We recommend that on-grade building slabs be at least 5½ inches in thickness and be reinforced with at least No. 4 bars spaced 18 inches on center, each way. Reinforcing bars should be placed within the upper 1/3 of the slab thickness with a minimum cover of 1½ inches below the top of the slab and 2 inches above the bottom of the slab.

10.7.2. Moisture Protection for Interior Slabs

Concrete slabs constructed on soil ultimately cause the moisture content to rise in the underlying soil. This results from continued capillary rise and the termination of normal evapotranspiration. Because normal concrete is permeable, the moisture will eventually penetrate the slab. Excessive moisture may cause mildewed carpets, lifting or discoloration of moisture sensitive floor coverings such as vinyl, or water-based adhesive degradation. Wood flooring may swell and dome if exposed to excessive moisture transmission. To decrease the likelihood of problems related to damp slabs, suitable moisture protection measures should be used where moisture sensitive floor coverings, moisture sensitive equipment, or other factors warrant.

A commonly used moisture protection in southern California consists of about 2 inches of clean sand covered by at least 10-mil plastic sheeting (Visqueen low-density polyethylene). In addition, 2 inches of clean sand are placed over the plastic to decrease concrete curing problems associated with placing concrete directly on an impermeable membrane. *However, it has been our experience that such systems will transmit from approximately 6 to 12 pounds of moisture per 1000 square feet of floor per day. This transmission rate may be excessive for some applications, particularly for sheet vinyl, wood flooring, vinyl tiles, or carpeting with impermeable backing that use water-soluble adhesives, and may void manufacturers' warranties.* ***The moisture protection system described above is provided only as an example, and is not intended as a recommendation for a specific moisture protection system.*** *Such a system will probably not meet the current requirements for moisture protection systems or manufacturers' warranties*

The current ACI *Guide for Concrete Floor and Slab Construction*, ACI 302.1R-04, provides detailed recommendations regarding moisture protection systems. In general, ACI 302-1R-04 defines a vapor retarder as having a water transmission rate of less than 0.3 perms when tested in accordance with ASTM E96, and defines a vapor barrier as having a water transmission rate of 0.00 perms. In addition, the vapor retarder should

conform to the ASTM E1745 guidelines and if polyethylene plastic sheeting is used, it should be a minimum of 10-mil high-density polyethylene.

Based on a review of ACI 302-1R-04, the moisture protection may consist of placing a 10-mil high-density polyethylene plastic sheeting or equivalent vapor retarder over 4 inches of a granular base material. The vapor retarder should be placed in accordance with ASTM E1643 guidelines. All laps or seams should be overlapped a minimum of 6 inches or as recommended by the manufacturer. Joints and pipe or utility penetrations should be sealed with the manufacturer's recommended adhesive, pressure sensitive tape or both. The granular base material should be a clean, fine graded material with at least 10 to 30% of particles passing No. 100 sieve but not contaminated with clay, silt, or organic material. The granular material should be compacted and proof-rolled prior to placement of the vapor retarder. The vapor retarder should be protected from puncture and if damaged be repaired per the manufacturer's recommendations. It should be noted that placing concrete directly on a low permeable membrane may result in finishing delays. To help speed up the potential finishing delay times and to reduce the potential for curl of the slab, we recommend that the concrete mix be designed for a water cement ratio of between 0.45 and 0.48.

Regardless of the moisture protection method used, the vapor retarder/barrier should extend beneath or be sealed to all foundation elements and grade beams. In addition, it has been shown the lateral migration of water from foundation edges contributes significantly to moisture problems. The membrane should extend above soils grade around the structure's perimeter, and the exposed foundation face should be painted with a latex sealer prior to color coat. ***No surfacing should be placed on the slab until the excess water within the slab has been dissipated. We recommend that vapor transmission tests be performed on the slab surface after operation of HVAC systems and prior to placing floor surfaces to confirm that excessive moisture is not***

still present within the slab and to verify that vapor transmission rates do not exceed the manufacturer's warranty.

The project architect should review the ACI 302.1R-04 document along with the moisture requirements of the proposed flooring system and incorporate an appropriate level of moisture protection based on the allowable moisture transmission rate for the flooring to be used and the manufacturer's warranty. Depending on the floor covering used, it may be necessary to utilize a vapor barrier (with a maximum water vapor transmission rate of 0 perms) and/or waterproof the slab instead of using a vapor retarder.

Additional reduction in vapor transmission through concrete floor slabs may be achieved by the placement of a dense concrete section without cracks. Achieving such a concrete section may be facilitated by the use of low water-cement ratios and a low slump concrete mix with thorough curing. For slab on grade floors, it is recommended that the water-cement ratio of the concrete as placed be between 0.45 and 0.48 and the slump of the concrete not exceed 4½ inches. The concrete should incorporate a well-graded, sound aggregate mix meeting the minimum requirements of the 2013 CBC and should have at least 560 pounds of cement per cubic yard of concrete. High-end water reducing admixtures are not recommended; however, fly ash, not exceeding 15 percent of the total cementitious material by weight may be added to improve workability and reduce shrinkage. In order to decrease permeability, silica fume may be added to the concrete; however, the amount of silica fume should not exceed 10 percent of the total cementitious material by weight. The concrete slab should be thoroughly cured for at least seven days using an accepted curing compound or water.

10.7.3.Exterior Slabs

Exterior slabs should be supported by compacted fill prepared as recommended in this report and having a very low expansion potential. Slabs should be designed for the anticipated loading. We recommend that exterior slabs be at least 4 inches thick and reinforced with at least

6-inch by 6-inch, W2.9 by W2.9 welded wire fabric placed near the top of the slab with at least 1½ inches of cover. Crack control joints should be used on all exterior slabs. A maximum joint spacing of 5-foot centers each way for sidewalks and 10-foot centers each way for slabs is suggested. Actual crack control joint spacing should be designed by the project structural engineer or architect in conformance with the requirements of the 2013 CBC and the PCA Engineering Bulletin for Concrete Floors on Ground.

Exterior slabs constructed directly on expansive soils may experience movement and cracking. One inch of differential movement is not considered unusual, and more is possible. Differential movement and cracking may be decreased by replacing the surficial two feet of expansive subgrade with very low expansive (EI less than 20) soil. Reinforcement and control joints will also help to reduce the cracking and movement potential. Differential movement between buildings and exterior slabs or between sidewalks and curbs may be decreased by dowelling the slabs into the foundations or curbs.

10.8. Cantilevered Earth-Retaining Structures

Retaining walls should be designed by the project structural engineer, using the geotechnical parameters provided. For design of cantilever retaining walls, the following soil parameters may be used. *These soil parameters assume that retaining walls will be backfilled with granular, free-draining cohesionless soil (sand, gravel, or rock) having an expansion index of 20 or less and an angle of friction of at least 32 degrees, and be compacted to at least 90 percent of maximum dry density based on the guidelines of ASTM D1557. The backfill area should include the zone defined by a 1:1 plane projected upward from the heel of the wall for a cantilevered retaining wall.* Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment, which could cause distress to walls, should not be used.

The parameters for the foundation soils are based on previous laboratory testing of the on-site materials. *The parameters for the backfill soils are assumed and laboratory testing should be performed on the materials proposed as backfill to*

confirm the proposed fill soils possess the required strength and expansion index and be free draining. The on-site soils may not meet these requirements for backfill.

Allowable Soil Bearing:	2,000 lbs/ft^2 for foundations bearing on engineered low expansion fill, compacted as recommended herein, or alternatively, 4,000 lbs/ft^2 for foundations bearing on dense undisturbed cemented Very Old Paralic Deposits (estimated factor of safety > 3). Allowable soil bearing may be increased by 500 lbs/ft^2 for each foot of depth below the minimum and 250 lbs/ft^2 for each foot of width beyond the minimum up to a maximum of 7,500 lbs/ft^2 on formation or 4,000 lbs/ft^2 on fill Allow a one-third increase in the allowable bearing for short-term wind or seismic loads.
Coefficient of Friction:	0.35 between the bottom of footings and engineered low expansion fill compacted as recommended herein or between the bottom of footings and dense undisturbed cemented Very Old Paralic Deposits (estimated factor of safety > 1.5).
Passive Pressure:	The passive resistance of engineered low expansion fill, compacted as recommended herein, or alternatively, dense undisturbed cemented Very Old Paralic Deposits may be assumed to be equal to the pressure developed by an equivalent fluid with a density of 350 lbs/ft^2 (estimated factor of safety > 1.5). A one-third increase in the passive value may be used for seismic loads. The passive resistance of the materials may be combined with the frictional resistance without reduction in evaluating the total lateral resistance.

Active Earth Pressure:	Equivalent fluid pressure of 35 lbs/ft^3 for level backfill or 55 lbs/ft^3 for 2:1 sloping backfill (estimated factor of safety = 1). Assumes walls are free to yield at the top at least ½ of the wall height.
At-Rest Earth Pressure:	Equivalent fluid pressure of 60 lbs/ft^3 for level backfill or 85 lbs/ft^3 for 2:1 sloping backfill (estimated factor of safety = 1). Assumes walls are restrained.
Minimum Footing Width:	24 inches.
Minimum Footing Depth:	24 inches below adjacent finish soil grade or pavement grade, whichever is lower.

Foundations should be setback from descending slopes a minimum distance of 10 feet measured horizontally from the outside bottom edge of the foundation to the slope face. In addition to the recommended earth pressure, walls adjacent to vehicular traffic should be designed to resist a uniform lateral pressure of 100 lbs/ft^2, acting as a result of an assumed 300 lbs/ft^2 surcharge behind the wall. If the traffic is kept back at least ten feet from the walls, the traffic surcharge may be neglected.

10.9. MSE Earth-Retaining Structures

MSE reinforced earth retaining walls may experience lateral movement over time prior to mobilizing the tensile resistance of the reinforcement. The retaining wall design engineer should review the configuration of the proposed improvements, and provide recommendations that will help reduce the potential for distress to these improvements from lateral movement.

Soil parameters for the reinforced, foundation, and retained zones are provided below. *These soil parameters assume that retaining walls will be backfilled with granular, free-draining cohesionless soil (sand, gravel, or rock) having an expansion index of 20 or less and an angle of friction of at least 30 degrees, and be compacted to at least 90 percent of maximum dry density based on the guidelines of ASTM D1557. The backfill area should include the reinforced fill zone for a MSE wall. Use of soils with Plasticity Indices as high as 20 may result*

in soils that possess Expansion Indexes exceeding 20 and should not be used. Use of soils with as much as 35% passing the number 200 sieve may result in soils that are not free draining and should not be used. Heavy compaction equipment, which could cause distress to walls, should not be used.

The parameters for the foundation zone are based on laboratory testing of the on-site materials. *The parameters for the reinforced zones are assumed and laboratory testing should be performed on the materials proposed as reinforced zone fill to confirm the proposed fill soils possess the required strength, expansion index, and drainage values.*

	Infill Reinforced Zone	Backfill Retained Zone	Foundation Soil
Friction Angle:	30°	30°	32°
Cohesion:	0 psf	0 psf	250 psf
Moist Unit Weight:	125 pcf	125 pcf	125 pcf

Foundations should be setback from descending slopes a minimum distance of 10 feet measured horizontally from the outside bottom edge of the foundation to the slope face. In addition to the recommended earth pressure, walls adjacent to vehicular traffic should be designed to resist a uniform lateral pressure of 100 lbs/ft^2, acting as a result of an assumed 300 lbs/ft^2 surcharge behind the wall. If the traffic is kept back at least ten feet from the walls, the traffic surcharge may be neglected.

10.10. Wall Drainage

The wall pressures and wall designs presented above assume no hydrostatic pressures or additional surcharge loads other than the recommended traffic surcharge load described above. Group Delta Consultants should be contacted for additional recommendations if hydrostatic or additional surcharge pressures are applicable. Walls should contain an adequate subdrain to minimize hydrostatic forces. Wall drain details for cantilevered retaining walls are presented in Wall Drain Details, Figure 10. Wall drain details for gravity walls (MSE walls) are given in Figure 11. We recommend that the subdrain consist of a minimum 4-inch diameter perforated PVC pipe, surrounded by at least one cubic

foot (per lineal foot) of ¾-inch crushed rock that is wrapped in a needle punched filter fabric such as Mirafi 140N, or an approved similar product. The perforated PVC pipe should drain through a solid PVC pipe to the storm drain system or a suitable gravity outlet discharging away from the wall. If the pipe is to outlet to grade, a permanent concrete headwall should be constructed to protect the outlet from clogging. The outlet should also be protected from rodent intrusion using a ¼-inch stainless steel mesh cover. If significant water infiltration is anticipated into the reinforced soil fill of the MSE wall, the wall should include a separation geotextile fabric between the reinforced soil fill and the gravel backdrain behind the wall as shown on Figure 11.

10.11. Seismic Wall Design

Per Section 1803.5.12 of the 2013 California Building Code (CBC), seismic design is required for all earth retaining structures over 6 feet in height. Basement walls may also require seismic design. We recommend that a seismic retaining wall design be conducted using the Mononabe-Okabe solution which incorporates a pseudo-static horizontal load. According to the provisions of the 2013 CBC, the design level peak ground acceleration (PGA) for the site may be taken as 40 percent of the short period spectral ordinate (S_{DS} ~ 0.625) or 0.250g, as shown in Figure 9C. One-half of the estimated peak ground acceleration is typically used for pseudo-static seismic wall design. Consequently, we have provided seismic retaining wall design parameters for a pseudo-static load of 0.13g (see Figure 12).

Note that in the Mononabe-Okabe solution, the seismic load is superposed on the classical triangular active pressure wedge. The seismic load may be idealized as an inverted triangular pressure distribution with the resultant acting at a height of 0.6H up from the base of the wall (where H is the retained wall height). The seismic thrust may be calculated using the following equation:

$$F_e = ½(\gamma_e)(H)^2$$

The Mononabe-Okabe solution is based on active earth pressures, and requires that the retaining walls are free to yield. For retained granular soils, active pressures will typically be developed when the walls have yielded about ½

percent of the wall height. For restrained walls, we recommend that static wall design be based on an at-rest earth pressure approximated by a fluid with an equivalent unit weight of 60 pcf (assuming level backfill). We recommend that the equivalent seismic pressures (γ_e) and the earthquake pressure resultants (Fe) shown in Figure 12 (γe ~15 PCF)be added to the at-rest earth pressure for seismic design of any restrained retaining walls at the site which are not free to yield about ½ percent of the wall height. For gravity or MSE retaining walls, we recommend the designer incorporate pseudo-static loads resulting from horizontal accelerations corresponding to the accelerations presented above into the seismic retaining wall design.

10.12. General Wall Construction

Compaction of the wall backfill will be necessary to minimize settlement of the backfill and to minimize settlement of overlying walks and paving. However, some settlement of the backfill should still be anticipated. Any utilities supported therein should be designed to accept differential settlement, particularly at wall penetrations or at points of entry to below grade building walls. In addition, provisions should be made for some settlement of concrete walks on grade supported on backfill. Differential movement between walls and exterior slabs or sidewalks supported on backfill may be decreased by dowelling the slabs or sidewalks into the walls.

Backfilling retaining walls with expansive soils can increase lateral pressures well beyond the active or at-rest pressures indicated above. We recommend that retaining walls be backfilled with free-draining, cohesionless soil (sand, gravel, or rock) having an expansion index of 20 or less. The backfill area should include the zone defined by a 1:1 plane projected upward from the heel of the wall. Retaining wall backfill or MSE wall reinforced wall fill should be mechanically compacted, in layers not more than 8 inches thick, to at least 90 percent of the maximum density obtainable by the ASTM Designation D1557 method of compaction; flooding or jetting should not be permitted. Backfill should not be placed until walls have achieved adequate structural strength. Heavy compaction equipment, which could cause distress to walls, should not be used.

10.13. Pipelines

The proposed improvements may include storm drain, sanitary sewer, and water pipelines. Geotechnical aspects of pipeline design include lateral earth pressures for thrust blocks, modulus of soil reaction, and pipe bedding.

10.13.1. Thrust Blocks

The following design parameters may be used for thrust blocks embedded in either compacted fill or formational materials. One-third increases in the allowable bearing and passive pressure values may be used for short-term wind or seismic loads.

Allowable Bearing: 2,000 psf

Passive Pressure: 350 psf per foot of embedment

Coefficient of Friction: 0.35

10.13.2. Modulus of Soil Reaction

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines. For evaluating deflection due to the load associated with trench backfill over the pipe, a value of 2,000 psi is recommended for the modulus of soil reaction assuming that granular bedding material (Unified Soil Classification GW, GP, SW, or SP) is placed adjacent to the pipe and is compacted to a minimum of 90 percent of maximum dry density based on ASTM D1557 guidelines.

10.13.3. Pipe Bedding

Typical pipe bedding as specified in the Standard Specifications for Public Works Construction "GREENBOOK" may be used. As a minimum, we recommend that pipe be supported on at least 4 inches of granular bedding material, such as 3/4-inch rock. Where pipeline or trench excavation inclinations exceed 15 percent, we recommend that open

graded rock not be used for pipe bedding or backfill because of the potential for piping and internal erosion of the overlying backfill. The recommended bedding in those locations is coarse sand having a sand equivalent value greater than 30. Alternatively, a sand-cement slurry can be used for the bedding. The slurry should consist of at least a 2-sack mix having a slump no greater than 5 inches. If the sand-cement slurry is used both for the pipe bedding and as backfill to at least 1 foot over the top of the pipe, cut-off walls may not be considered necessary. This recommendation should be further evaluated by the project civil engineer designing the pipe system.

10.14. Pavements

Preliminary pavement sections are provided for new pavements that will be associated with the proposed improvements. *Final pavement design should be based on R-value test results of the finish pavement subgrade soils. If pavements are to be designed to City of San Diego Standards, the City of San Diego Testing Laboratory should be notified for a pavement design section.*

Subgrade preparation should be conducted immediately prior to the placement of the pavement section. Prior to paving the site, the exposed pavement subgrade should be scarified to a depth of 12 inches, brought to approximately optimum moisture content, and compacted to at least 95 percent of the maximum dry density, as determined in general accordance with ASTM D1557.

Base Course: If aggregate base is used, the aggregate base should be compacted to at least 95 percent of the maximum dry density, as determined in general accordance with ASTM D1557. Aggregate base materials should conform to Class 2 aggregate base as defined in Section 26-1.02A of the latest edition of the Caltrans Standard Specifications. Alternatively, base material should conform to the specifications for crushed aggregate base, crushed miscellaneous base, or processed miscellaneous base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction, "GREENBOOK", or to the specifications for Class 2 aggregate base as defined in Section 400-2.4 of the Regional Supplement Amendments to the "Greenbook". If a City of San Diego standard roadway section is to be used, the base course should conform to the

City requirements for cement treated base as specified in the Pavement Design Standards Schedule "J" of the current City of San Diego Standard Drawings.

10.14.1. Asphalt Concrete

Alternative paving sections for access roads and parking areas were established using the design criteria of Caltrans Topic 608.4 and the Pavement Design Standards Schedule "J" of the City of San Diego Standard Drawings. For this project, it was assumed that parking areas would be designed as cul-de-sacs with a maximum average daily traffic of 200 vehicles (Maximum Traffic Index = 5.0) and that access roads would be designed as a local street with a maximum average daily traffic of 700 vehicles (Maximum Traffic Index = 5.5). The City may require fire lanes to be designed for a Traffic Index of 7.0 and areas where fire trucks are positioned when responding to a fire to be designed for a Traffic Index of 9.5. R-Value testing performed on a sample of the fill material indicated an R-Value of 24. The actual subgrade R-Value should be confirmed during construction. Based on the indicated Traffic Indexes and a design R-values of at least 24, the following pavement sections were calculated in accordance with the Caltrans design method or City of San Diego standard design method.

	Caltrans Design		City of San Diego Design	
Traffic Index	**Asphalt**	**Aggregate Base**	**Asphalt**	**CTB**
5.0	3 inches	7 inches	3 inches	5½ inches
5.5	3 inches	9 inches	3 inches	7 inches
7.0	4 inches	11 inches	3 inches	10 inches
8.5	5 inches	14 inches	3½ inches	13 inches
9.5	6 inches	15½ inches	4 inches	15 inches

Asphalt concrete should conform to "GREENBOOK" specifications. Asphalt concrete should be compacted to at least 95 percent based on the Hveem unit weight.

10.14.2. Portland Cement Concrete

Concrete pavement design was conducted in accordance with the simplified design procedure of the Portland Cement Association. This methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The portland cement concrete was assumed to have a minimum 28-day flexural strength of 600 psi. A "low" subgrade support (corresponding to a modulus of subgrade reaction between 75 and 120 pci) was assumed for design purposes. Based on these assumptions, we recommend that the pavement section consist of 6½ inches of portland cement concrete over compacted subgrade for a Traffic Indices of 5 or 5.5, 7 inches of portland cement concrete over compacted subgrade for a Traffic Index of 7, and 7½ inches of portland cement concrete over compacted subgrade for a Traffic Index of 8.5. If a City of San Diego standard design method is used, the pavement section would consist of 7 inches of portland cement concrete over compacted subgrade for a Traffic Index of 5, 7 inches of portland cement concrete over compacted subgrade for a Traffic Index of 5.5, 7½ inches of portland cement concrete over compacted subgrade for a Traffic Index of 7, and 8 inches of portland cement concrete over compacted subgrade for a Traffic Index of 8.5.

Crack control joints should be placed on at least 10-foot centers, each way. During construction, the flexural strength of the concrete should be evaluated by making beams in accordance with ASTM C31 and testing for flexural strength in accordance with ASTM C78. Heavy truck traffic areas, such as trash truck aprons and loading dock areas, should be reinforced with at least number 4 bars on 18-inch centers, each way.

10.15. Soil Corrosivity

Selected soil samples were evaluated for water-soluble sulfate content to assess the general degree of sulfate exposure of concrete in contact with the site soils. The test results are presented in Appendix B. The project design engineer may use the test results in conjunction with Table 4.3.1 of ACI 318 to specify a

suitable cement type, water cement ratio, and minimum compressive strength for concrete used on site, which will be in direct contact with soil, including all foundations and slabs. Additional testing of the finish grade materials should be performed to evaluate the final as-graded condition of the site. It should be noted that soluble sulfate in the irrigation water supply, and/or the use of sulfate containing fertilizer may cause the sulfate content in the surficial soils to increase significantly with time. This may result in a higher sulfate exposure than that indicated by the test results reported herein. Studies have shown that the use of improved cements in the concrete, and a low water-cement ratio will improve the resistance of the concrete to sulfate exposure.

Based on the resistivity test results, the on-site soils appear to range from corrosive to extremely corrosive to ferrous metals. Based on the pH test results, the on-site soils appear to be corrosive to metals. Based on the soluble chloride test results, the on-site soils do not appear to be significantly corrosive to reinforcing steel. Based on the soluble sulfate test results, the on-site soils do not appear to be significantly reactive with concrete. A corrosion consultant should be contacted to provide specific corrosion control recommendations.

11 LIMITATIONS OF INVESTIGATION

This report has been prepared for the exclusive use of the San Diego Community College District, for specific application to the proposed Parking Structure No. 4, Miramar College Campus located at 10440 Black Mountain Road in San Diego, California. The recommendations provided in this report are based on our understanding of the described project information and on our interpretation of the data collected during the subsurface exploration. The recommendations apply only to the specific project described in this report. In the event that any changes in the nature, design, or location of the facilities are planned from those described herein, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by Group Delta Consultants. Group Delta Consultants is not responsible for any claims, damages, or liability associated with interpretation of subsurface data or reuse of the subsurface data or engineering analyses without the express written authorization of Group Delta Consultants.

This investigation was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

The analyses and recommendations contained in this report are based on the data obtained from the referenced subsurface explorations. The samples taken and used for testing and the observations made are believed representative of the locations sampled; however, borings indicate subsurface conditions only at the specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations. Soil and geologic conditions can vary significantly between field explorations. The validity of the recommendations is based in part on assumptions about the stratigraphy made by the geotechnical engineer. Such assumptions may be confirmed only during construction operations. In many projects, conditions revealed by excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by Group Delta Consultants and additional recommendations made, if warranted.

This report is issued with the understanding that it is the responsibility of the San Diego Community College District, or of their designated representative, to ensure that the information and recommendations contained herein are brought to the attention of the design consultants for the project and incorporated into the plans, and the necessary steps are taken to see that the contractors carry out such recommendations in the field.

Changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on this or adjacent properties. In addition, changes in applicable or appropriate standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

During site preparation and foundation construction, Group Delta Consultants should observe slab on-grade subgrades, wall backfill, and utility trench backfill to check compaction. The engineer should observe subgrade preparation beneath areas to be recompacted, and observe and test fill compaction as required. The engineer should

also observe building-foundation excavations to verify the presence of a firm bearing surface.

Group Delta Consultants should be retained to observe earthwork construction to help confirm that our assumptions and recommendations are valid or to modify them accordingly. Group Delta Consultants does not assume responsibility or liability for the adequacy of the geotechnical recommendations presented herein if we do not observe construction.

***

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FAULT[1]	DISTANCE TO SITE [KM]	ESTIMATED PEAK GROUND ACCELERATION[2]	MAXIMUM EARTHQUAKE MAGNITUDE[3,5]	ESTIMATED FAULT AREA[4] [CM^2]	SHEAR MODULUS[4] [DYNE/CM^2]	ESTIMATED SLIP RATE[4] [MM/YEAR]
Rose Canyon	14	0.225	7.2	9.10E+12	3.30E+11	1.5
Coronado Bank	35	0.125	7.6	2.41E+13	3.30E+11	3.0
Newport-Inglewood (Offshore)	40	0.085	7.1	8.58E+12	3.30E+11	1.5
Elsinore (Julian)	48	0.072	7.1	1.14E+13	3.30E+11	5.0
San Diego Trough	51	0.094	7.7	3.00E+13	3.30E+11	2.0
Elsinore (Temecula)	53	0.054	6.8	6.45E+12	3.30E+11	5.0
Earthquake Valley	59	0.041	6.5	3.00E+12	3.30E+11	2.0
Elsinore (Coyote Mountain)	71	0.040	6.8	5.85E+12	3.30E+11	4.0
San Clemente	82	0.080	8.1	6.00E+13	3.30E+11	4.0
San Jacinto-Coyote Creek	83	0.034	6.8	6.15E+12	3.30E+11	4.0
San Jacinto-Anza	83	0.040	7.2	1.64E+13	3.30E+11	12.0
Elsinore (Glen Ivy)	85	0.028	6.8	5.40E+12	3.30E+11	5.0
Palos Verdes	87	0.040	7.3	1.25E+13	3.30E+11	3.0
San Joaquin Hills	91	0.024	6.6	4.30E+12	3.60E+11	0.5
San Jacinto - Borrego	92	0.020	6.6	3.48E+12	3.30E+11	4.0
San Jacinto-San Jacinto Valley	94	0.025	6.9	7.74E+12	3.30E+11	12.0

1. Fault activity determined by Blake (2000), CDMG (1992), Wesnousky (1986), and Jennings (1994).

2. Median peak horizontal accelerations from ARS Online for average shear wave velocity (Vs30) ~ 640 m/s (Caltrans, 2015).

3. Moment magnitudes determined from CDMG (2003), Blake (2000), Wesnousky (1986) and Anderson (1984).

4. Estimated fault areas, shear moduli, and slip rates after fault data for EQFAULT and FRISKSP, Blake (2000).

5. The Maximum Earthquake Magnitude is the estimated median moment magnitude that appears capable of occuring given rupture of the entire estimated fault area.

DETERMINISTIC RESPONSE SPECTRA FOR Vs ~ 640 m/s

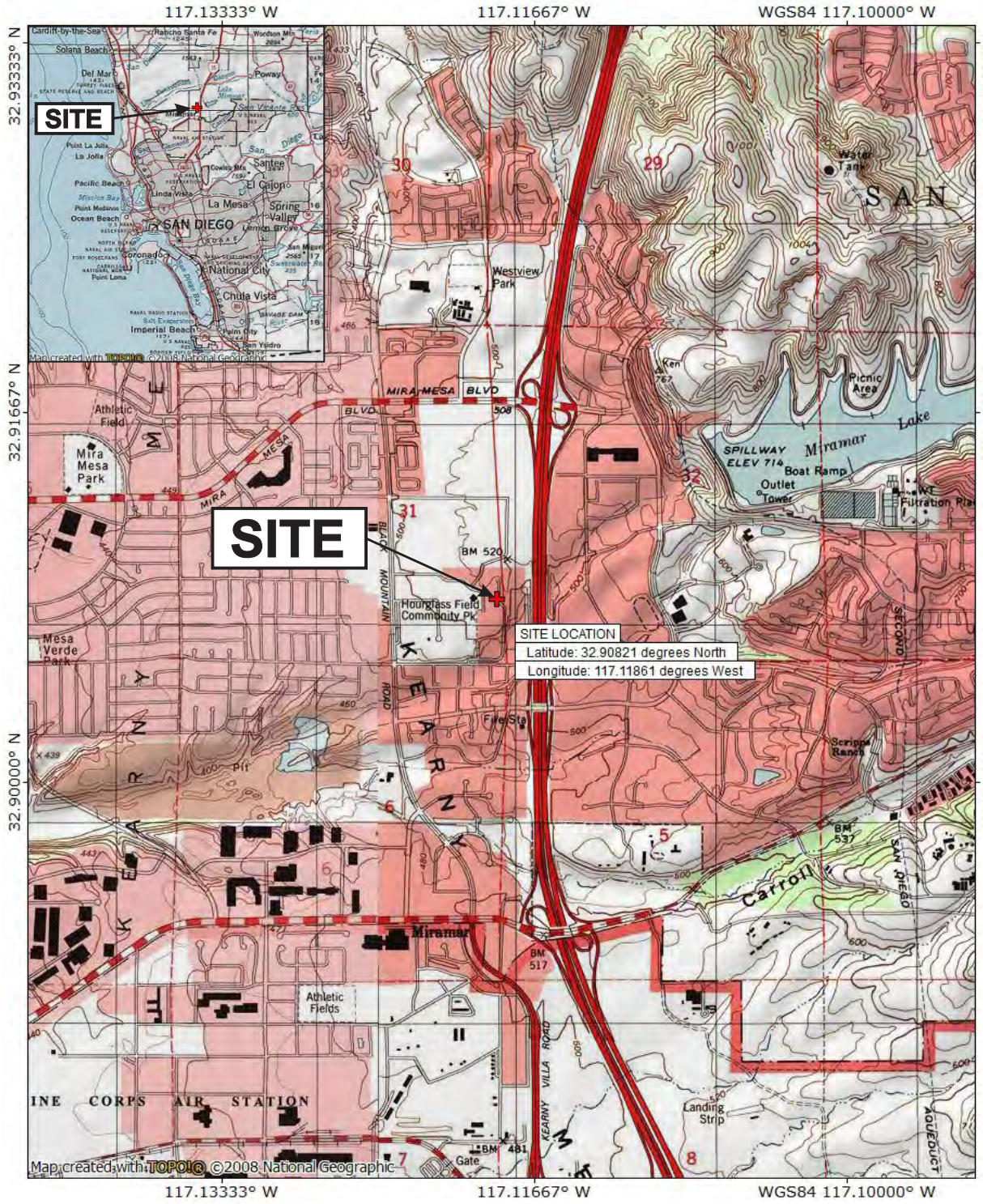
	DISTANCE	$S_{0.0}$	$S_{0.1}$	$S_{0.2}$	$S_{0.3}$	$S_{0.5}$	$S_{1.0}$	$S_{2.0}$	$S_{3.0}$	$S_{4.0}$	$S_{5.0}$
Rose Canyon	13.9 [km]	0.225	0.453	0.536	0.462	0.335	0.216	0.100	0.060	0.042	0.032
Coronado Bank	35 [km]	0.125	0.235	0.286	0.255	0.193	0.113	0.057	0.035	0.025	0.020
Newport-Inglewood (Offshore)	40 [km]	0.085	0.161	0.195	0.172	0.126	0.068	0.032	0.019	0.013	0.010
Elsinore (Julian)	48 [km]	0.072	0.133	0.163	0.146	0.108	0.059	0.027	0.016	0.012	0.009
San Diego Trough	51 [km]	0.094	0.169	0.210	0.193	0.149	0.091	0.047	0.029	0.021	0.017
Elsinore (Temecula)	53 [km]	0.054	0.100	0.123	0.110	0.080	0.042	0.018	0.011	0.007	0.006
Earthquake Valley	59 [km]	0.041	0.075	0.093	0.083	0.060	0.029	0.012	0.007	0.005	0.004
Elsinore (Coyote Mountain)	71 [km]	0.040	0.071	0.090	0.083	0.061	0.032	0.014	0.008	0.006	0.004
San Clemente	82 [km]	0.080	0.133	0.173	0.168	0.137	0.092	0.051	0.033	0.024	0.019
San Jacinto-Coyote Creek	83 [km]	0.034	0.059	0.075	0.071	0.053	0.029	0.013	0.007	0.005	0.004

PROBABILISTIC RESPONSE SPECTRA FOR Vs ~ 640 m/s

	PERIOD	$S_{0.0}$	$S_{0.1}$	$S_{0.2}$	$S_{0.3}$	$S_{0.5}$	$S_{1.0}$	$S_{2.0}$	$S_{3.0}$	$S_{4.0}$	$S_{5.0}$
Design Basis Earthquake (DBE)	475 Years	0.210	0.413	0.491	0.438	0.341	0.212	0.109	0.071	0.052	0.043
Upper Bound Earthquake (UBE)	975 Years	0.281	0.562	0.658	0.585	0.446	0.275	0.142	0.094	0.069	0.058
Maximum Considered Earthquake (MCE)	2,475 Years	0.396	0.813	0.946	0.828	0.624	0.375	0.194	0.129	0.096	0.080

DESIGN RESPONSE SPECTRA FOR Vs ~ 640 m/s

	$S_{0.0}$	$S_{0.1}$	$S_{0.2}$	$S_{0.3}$	$S_{0.5}$	$S_{1.0}$	$S_{2.0}$	$S_{3.0}$	$S_{4.0}$	$S_{5.0}$
2013 CBC Deterministic MCE Spectrum For Site Class C	0.687	1.465	1.500	1.500	1.500	0.780	0.390	0.260	0.195	0.156
2013 CBC Probabilistic MCE Spectrum For Site Class C	0.392	0.805	0.938	0.827	0.634	0.396	0.205	0.136	0.101	0.085
2013 CBC Site Specific MCE Spectrum For Site Class C	0.392	0.805	0.938	0.827	0.634	0.396	0.205	0.136	0.101	0.085
2013 USGS Mapped Spectrum For Site Class C	0.253	0.602	0.631	0.631	0.631	0.343	0.171	0.114	0.086	0.069
2013 CBC Design Spectrum For Class C (Recommended)	0.250	0.625	**0.625**	0.625	0.548	**0.274**	0.137	0.091	0.069	0.055



0 .5 1 MILES

1000 0 1000 2000 3000 4000 5000 FEET

1 .5 0 KILOMETERS 1

1000 0 METERS 1000

NATIONAL GEOGRAPHIC

TN MN

12°

07/22/15

GROUP DELTA

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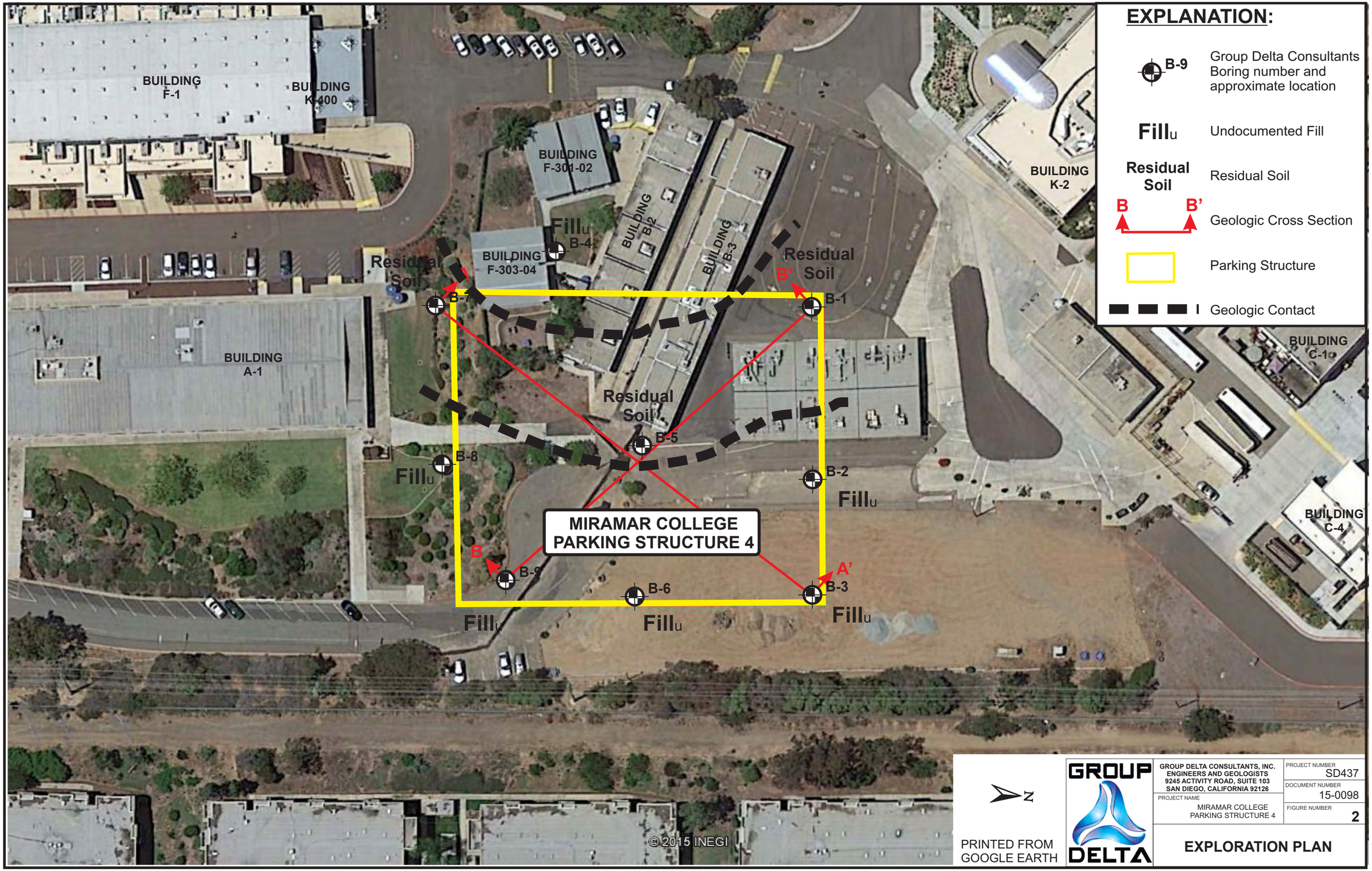
PROJECT NUMBER SD437

DOCUMENT NUMBER 15-0098

PROJECT NAME
MIRAMAR COLLEGE
PARKING STRUCTURE 4

FIGURE NUMBER 1

REGIONAL SITE LOCATION MAP



EXPLANATION:

- B-9 — Group Delta Consultants Boring number and approximate location
- Fill u — Undocumented Fill
- Residual Soil — Residual Soil
- B — B' — Geologic Cross Section
- Parking Structure
- Geologic Contact

N

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SAN DIEGO, CALIFORNIA 92126

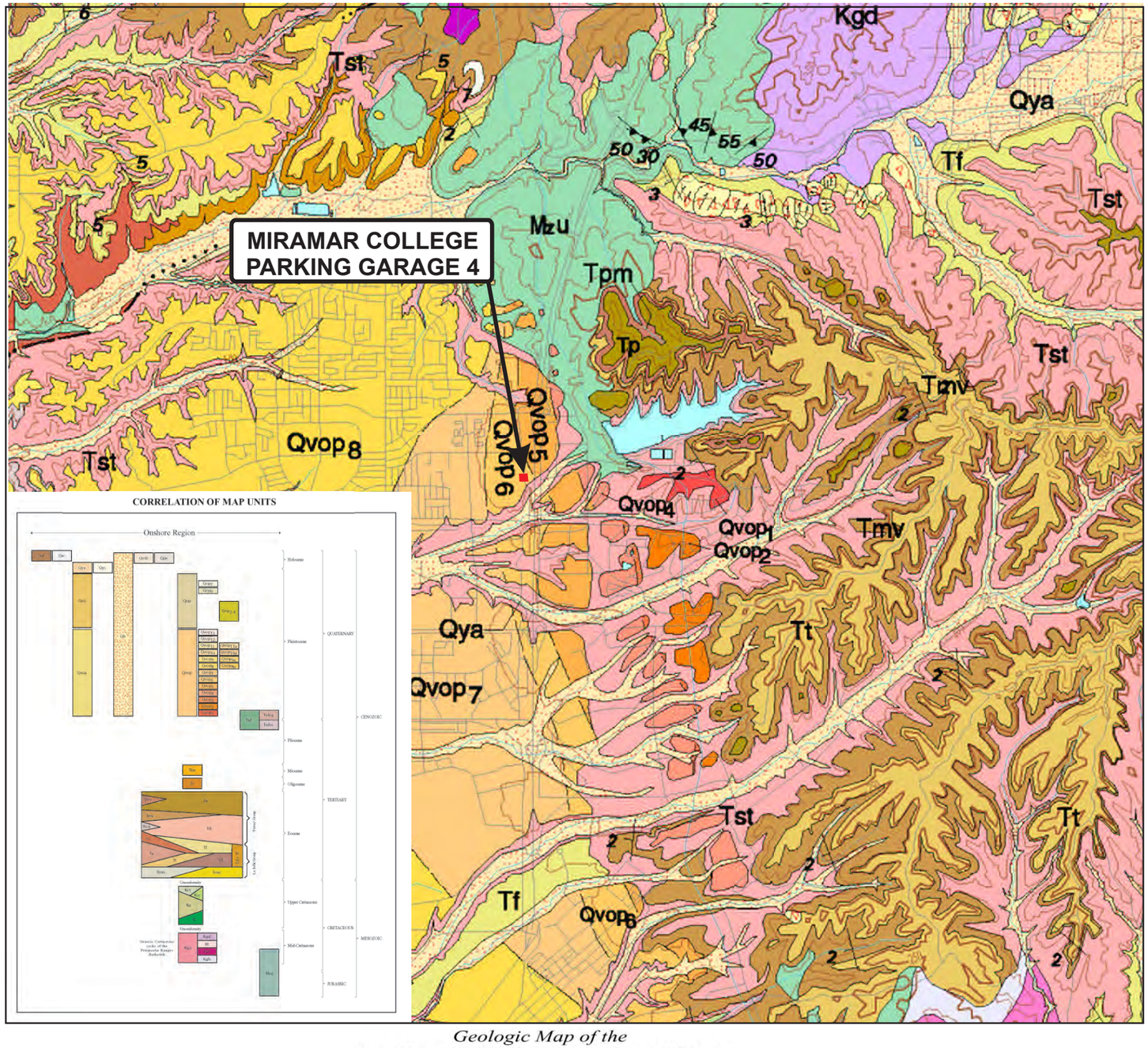
PROJECT NAME: MIRAMAR COLLEGE PARKING STRUCTURE 4

PROJECT NUMBER: SD437

DOCUMENT NUMBER: 15-0098

FIGURE NUMBER: 2

EXPLORATION PLAN



ONSHORE MAP SYMBOLS

Contact—Contact between geologic units; dotted where concealed.

Contact—Contact between paralic deposits and their associated marine abrasion platforms. This contact is approximate and generally buried by 1-5 m of marine and/or nonmarine sediment.

Fault—Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Anticline—Solid where accurately located; dashed where approximately located, dotted where concealed. Arrow indicates direction of axial plunge.

Syncline—Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge.

Landslide—Arrows indicate principal direction of movement.

Fault Zone—Area of extensively sheared rock within a zone defined by multiple faults.

Strike and dip of beds

Inclined

Strike and dip of igneous joints

Inclined

Vertical

Strike and dip of metamorphic foliation

Inclined

Very old paralic deposits, Unit 9 (middle to early Pleistocene)—Mostly poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate. These deposits rest on the 113-115 m Linda Vista terrace (Fig. 3)

Friars Formation (middle Eocene)—Mostly yellowish-gray, medium-grained, massive, poorly indurated nonmarine and lagoonal sandstone and claystone with tongues of cobble conglomerate. It contains an early Uintan vertebrate assemblage (Walsh and others, 1996) and was named for exposures along the north side of Mission Valley near Friars Road (Kennedy and Moore, 1971). The Friars Formation reaches a maximum thickness of 50 m between Mission Valley and Carmel Valley

Stadium Conglomerate (middle Eocene)—Massive cobble conglomerate with a dark yellowish-brown, coarse-grained sandstone matrix. The formation consists predominantly (up to 85%) of slightly metamorphosed rhyolitic to dacitic volcanic and volcaniclastic rocks and up to 20% percent quartzite. It yields an early and late Uintan vertebrate assemblage (Walsh and others, 1996) and Tejon stage mollusks (Givens and Kennedy, 1979). The Stadium Conglomerate is 50 m thick at its type section, located between the La Jolla and La Mesa quadrangles along the northern wall of Mission Valley near the San Diego Stadium (Kennedy and Moore, 1971)

Friars Formation (middle Eocene)—Mostly yellowish-gray, medium-grained, massive, poorly indurated nonmarine and lagoonal sandstone and claystone with tongues of cobble conglomerate. It contains an early Uintan vertebrate assemblage (Walsh and others, 1996) and was named for exposures along the north side of Mission Valley near Friars Road (Kennedy and Moore, 1971). The Friars Formation reaches a maximum thickness of 50 m between Mission Valley and Carmel Valley

Scripps Formation (middle Eocene)—The Scripps Formation (Tsc) is mostly pale yellowish-brown, medium-grained sandstone containing occasional cobble-conglomerate interbeds. It contains a middle Eocene Molluscan fauna (Givens and Kennedy, 1979). The Scripps Formation is 56 m thick at its type section which is 1 km north of Scripps Pier, on the north side of the mouth of Blacks Canyon (Kennedy and Moore, 1971). Both the basal contact with the Ardath Shale and the upper contact with the Friars Formation are conformable. In upper Carroll Canyon a tongue of the Scripps Formation (Tscu) exists above an intervening part of the Stadium Conglomerate. This "upper" tongue is difficult to separate from the main body of the Scripps Formation where the Statium Conglomerate is absent

Geologic Map of the
San Diego 30' X 60' Quadrangle, California

Compiled by
Michael P. Kennedy and Siang S. Tan

2005

Digital Preparation by
Kelly R. Bovard[1], Anne G. Garcia[1] and Diane Burns[1]
1: U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

N

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PROJECT NAME
MIRAMAR COLLEGE
PARKING STRUCTURE 4

FIGURE NUMBER 3

LOCAL GEOLOGIC MAP

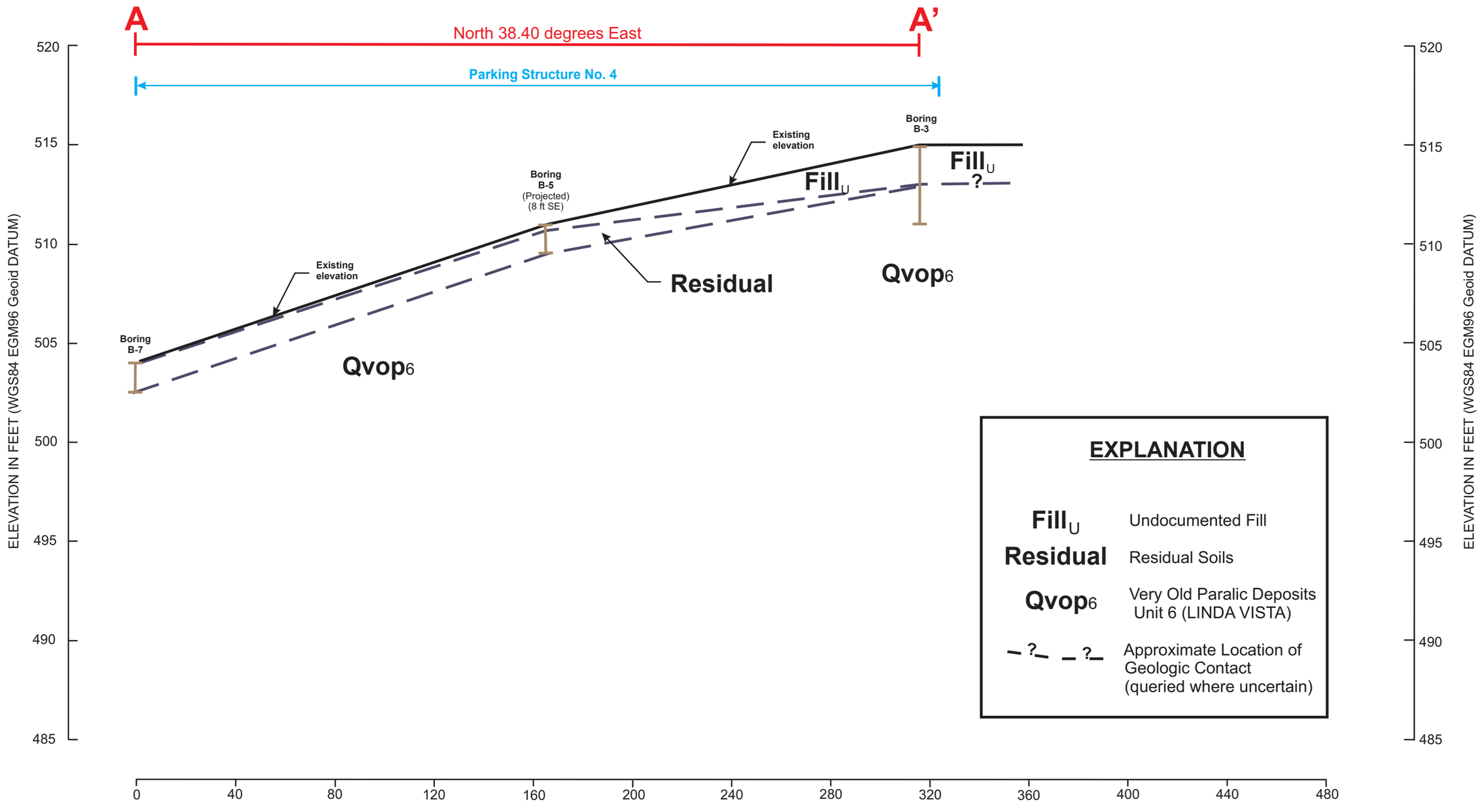

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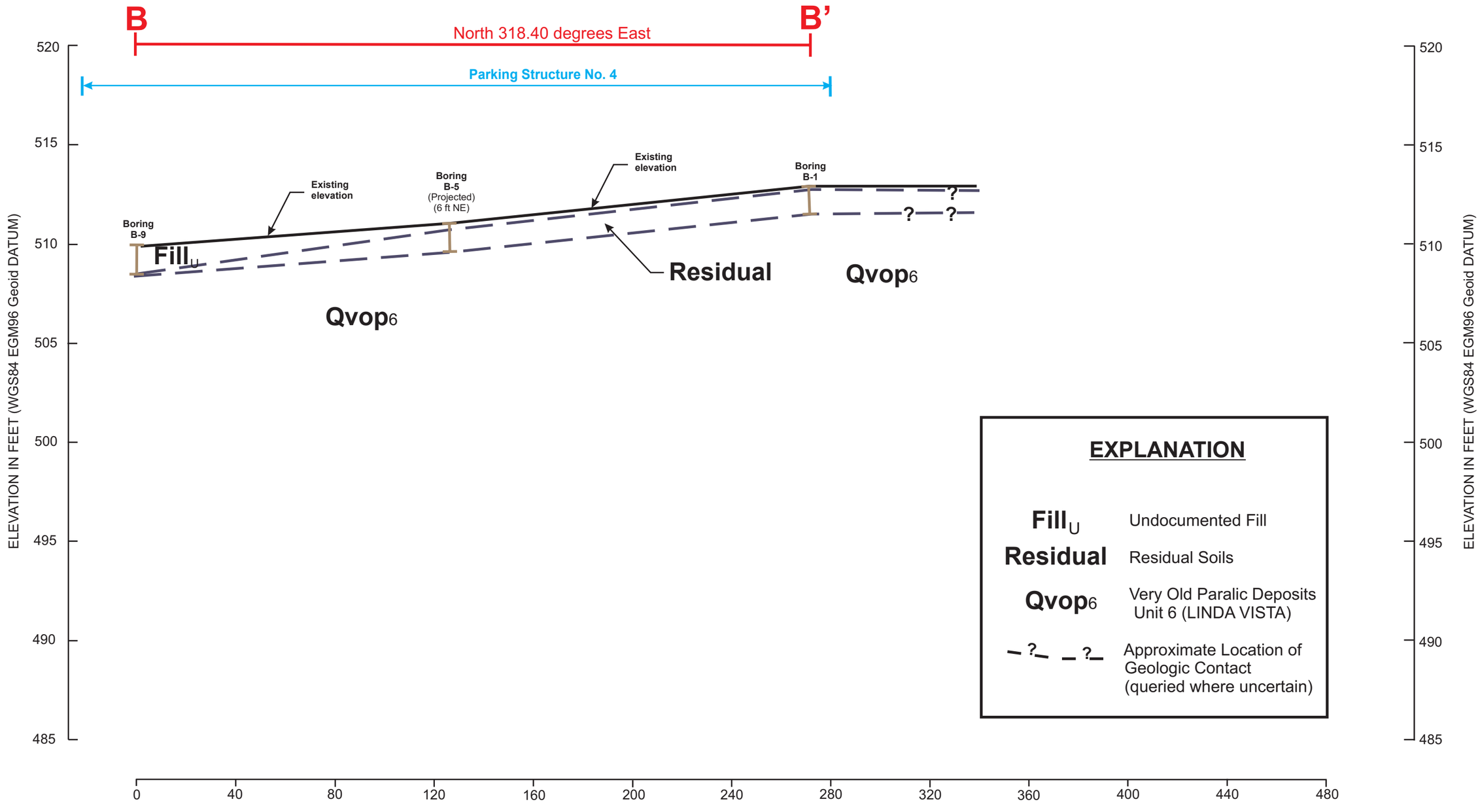
FIGURE NUMBER: 4.1

GEOLOGIC CROSS SECTION



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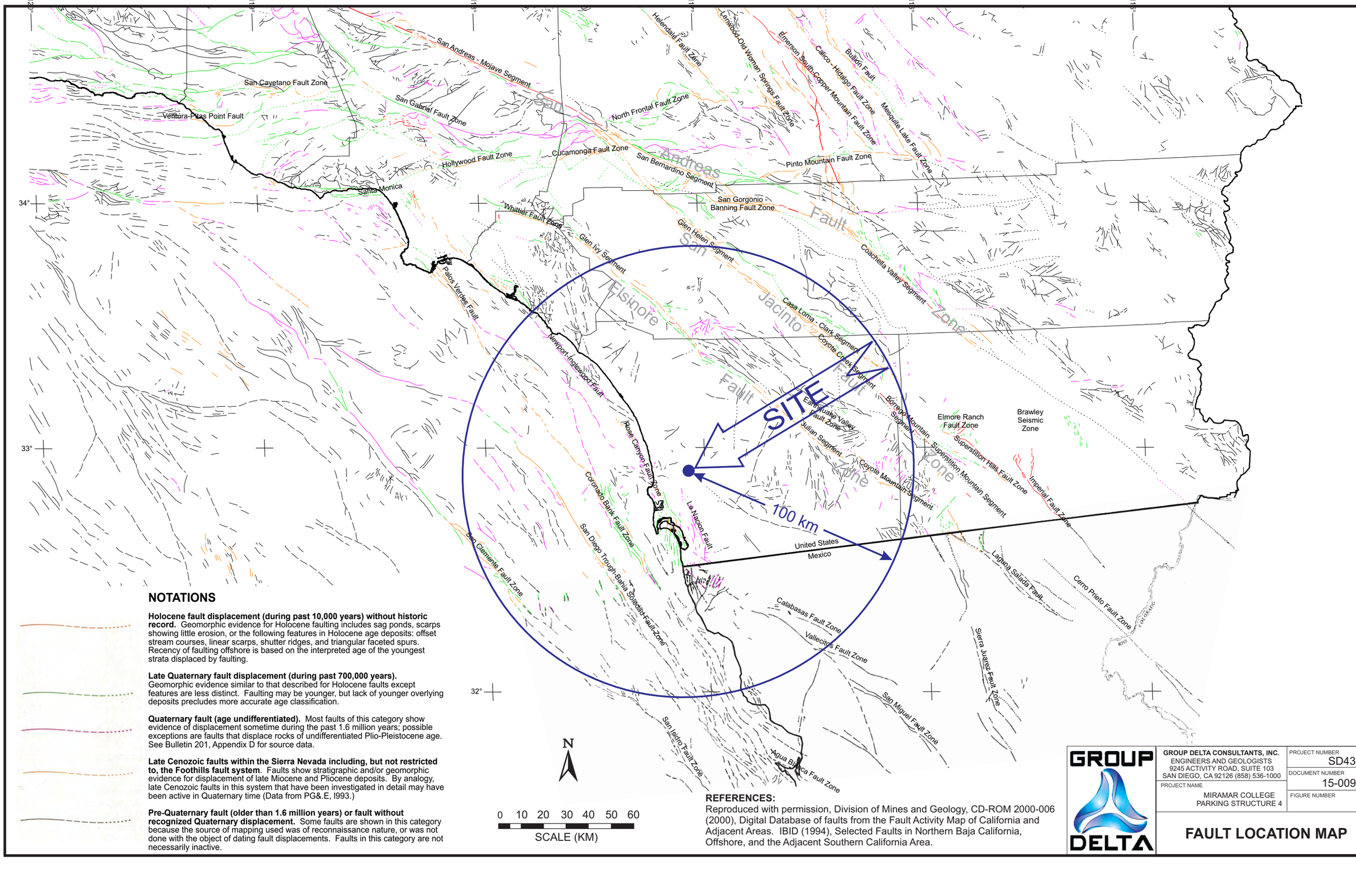
PROJECT NAME
MIRAMAR COLLEGE
PARKING STRUCTURE 4

PROJECT NUMBER
SD437

DOCUMENT NUMBER
15-0098

FIGURE NUMBER
4.2

GEOLOGIC CROSS SECTION



NOTATIONS

Holocene fault displacement (during past 10,000 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults that displace rocks of undifferentiated Plio-Pleistocene age. See Bulletin 201, Appendix D for source data.

Late Cenozoic faults within the Sierra Nevada including, but not restricted to, the Foothills fault system. Faults show stratigraphic and/or geomorphic evidence for displacement of late Miocene and Pliocene deposits. By analogy, late Cenozoic faults in this system that have been investigated in detail may have been active in Quaternary time (Data from PG&.E, I993.)

Pre-Quaternary fault (older than 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissance nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

REFERENCES:
Reproduced with permission, Division of Mines and Geology, CD-ROM 2000-006 (2000), Digital Database of faults from the Fault Activity Map of California and Adjacent Areas. IBID (1994), Selected Faults in Northern Baja California, Offshore, and the Adjacent Southern California Area.

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PROJECT NAME: MIRAMAR COLLEGE PARKING STRUCTURE 4
FIGURE NUMBER: 5

FAULT LOCATION MAP

NFIP

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 1361G

FIRM

FLOOD INSURANCE RATE MAP

SAN DIEGO COUNTY, CALIFORNIA

AND INCORPORATED AREAS

PANEL 1361 OF 2375

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY	NUMBER	PANEL	SUFFIX
SAN DIEGO, CITY OF	060295	1361	G

Notice to User: The **Map Number** shown below should be used when placing map orders; the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
06073C1361G

MAP REVISED
MAY 16, 2012

Federal Emergency Management Agency

NOTE: MAP AREA SHOWN ON THIS PANEL IS LOCATED WITHIN TOWNSHIP 14 SOUTH, RANCE 2 WEST AND THE RANCHO LOS PENASQUITOS LAND GRANT.

MIRO CIR

Private Rd

ZONE A

HIBERT ST

32

15

DR

TREENA ST

SCRIPPS LAKE DR

SCRIPPS LAKE D

ZONE X

DC1168

DC1169

MIRAMAR COLLEGE PARKING GARAGE 4

ZONE A

1% ANNUAL CHANCE FLOOD DISCHARGE CONTAINED IN STORM SEWER

MAYALINDA RD

Private Rd

15

ZONE A

ZONE A

SCRIPPS RANCH BLVD

MEANLEY DR

PARK DR

HOYT

1% ANNUAL CHANCE FLOOD DISCHARGE CONTAINED IN STORM SEWER

VISTA

VISTA LAGO PL

CANYON LAKE DR

PINETREE

LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual chance flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

ZONE A No Base Flood Elevations determined.

ZONE AE Base Flood Elevations determined.

ZONE AH Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined.

ZONE AO Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined.

ZONE AR Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood.

ZONE A99 Areas to be protected from 1% annual chance flood event by a Federal flood protection system under construction; no Base Flood Elevations determined.

ZONE V Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined.

ZONE VE Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

ZONE X Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

ZONE X Areas determined to be outside the 0.2% annual chance floodplain.

ZONE D Areas in which flood hazards are undetermined, but possible.

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths, or flood velocities

513 — Base Flood Elevation line and value; elevation in feet*

(EL 987) — Base Flood Elevation value where uniform within zone; elevation in feet*

* Referenced to the North American Vertical Datum of 1988

A — A — Cross section line

23 — 23 — Transect line

97°07'30", 32°22'30" — Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere

4275000mE — 1000-meter Universal Transverse Mercator grid ticks, zone 11

6000000 FT — 5000-foot grid values: California State Plane coordinate system, Zone VI (FIPSZONE = 406), Lambert projection

DX5510 — Bench mark (see explanation in Notes to Users section of this FIRM panel)

M1.5 — River Mile

MAP REPOSITORIES
Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTYWIDE FLOOD INSURANCE RATE MAP
June 19, 1997

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL
June 16, 1999

July 2, 2002

May 16, 2012 – to update corporate limits, to add roads and road names, to incorporate previously issued Letters of Map Revision, and to update map elevations to North American Vertical Datum of 1988.

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

MAP SCALE 1" = 500'

0 250 500 750 1,000 FEET

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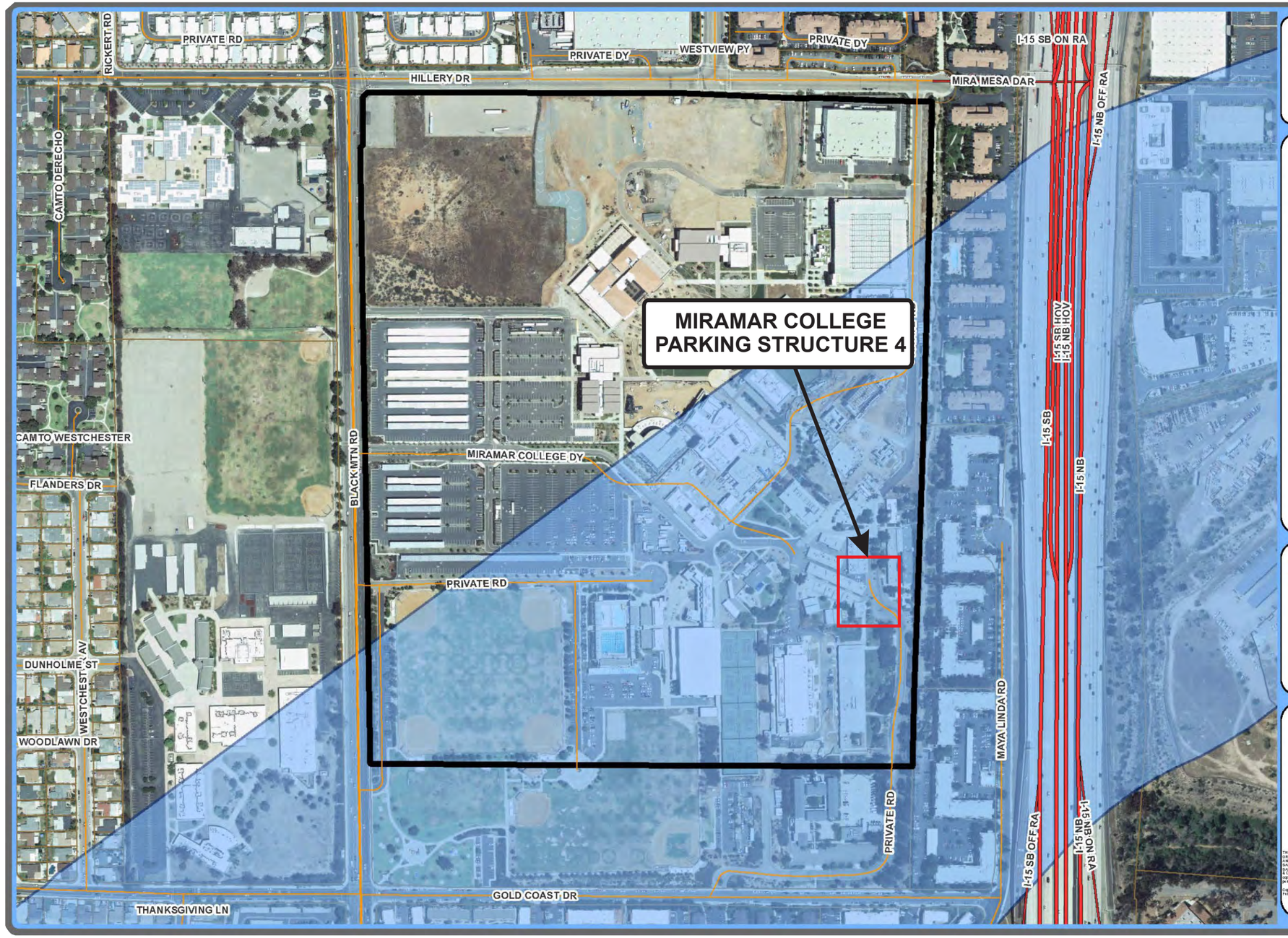
DOCUMENT NUMBER	15-0098
PROJECT NAME	MIRAMAR COLLEGE PARKING STRUCTURE 4
FIGURE NUMBER	6

FLOOD INSURANCE RATE MAP



Miramar Campus

Legend

- Miramar Campus
- Dam Inundation

0 75 150 300 Feet

1 inch = 300 feet

Vicinity Map

SanGIS
We Have San Diego Covered!

Map made exclusively for our client by the
San Diego Geographic Information Source
5510 Overland Ave., Suite 230
San Diego, CA 92123
(858) 874-7000
www.sangis.org

For reprints, mapping or additional information please contact SanGIS.

Plot Date: 07/01/15

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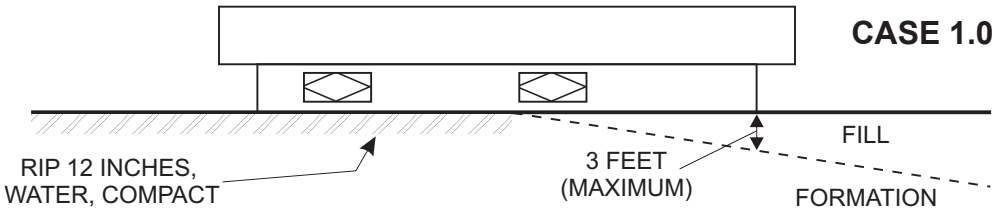
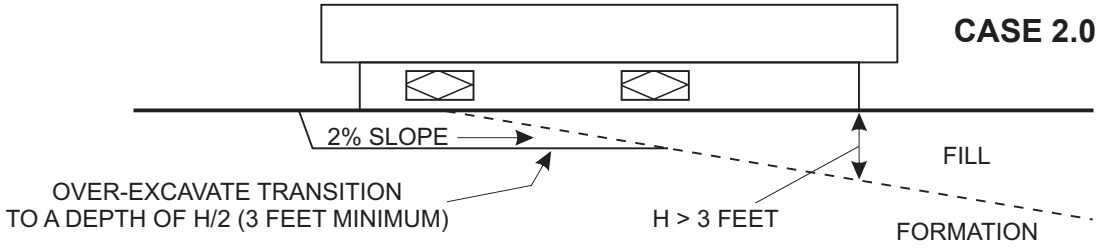
DOCUMENT NUMBER: 15-0098

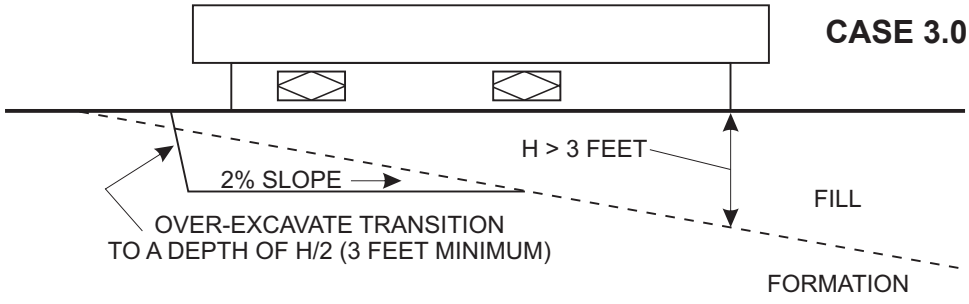
PROJECT NAME: MIRAMAR COLLEGE PARKING STRUCTURE 4

FIGURE NUMBER: 7

DAM INUNDATION MAP







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PARKING STRUCTURE 4

PROJECT NUMBER
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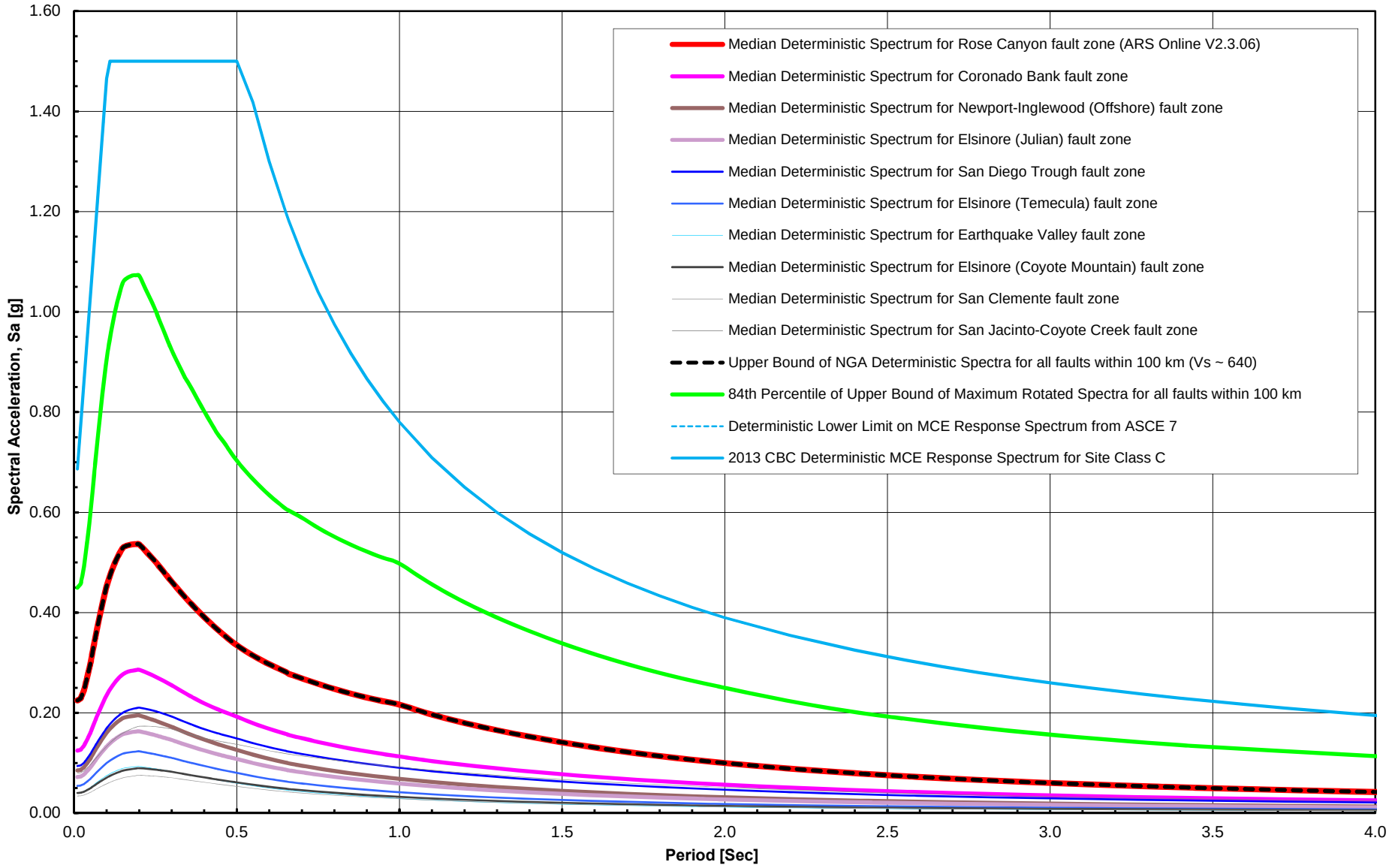
DOCUMENT NUMBER
15-0098

FIGURE NUMBER
8

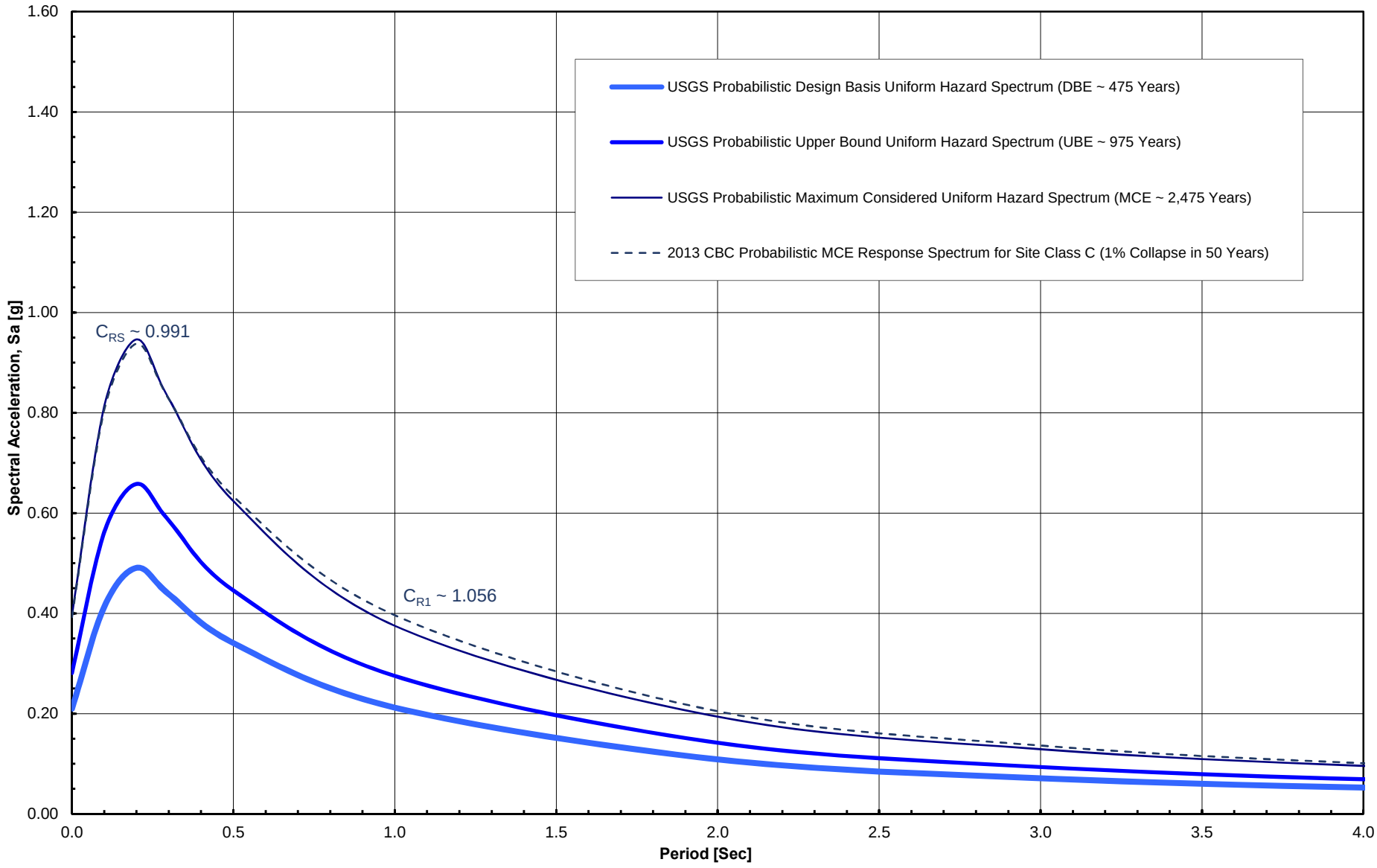
TRANSITION DETAILS



GROUP DELTA

DETERMINISTIC SPECTRA

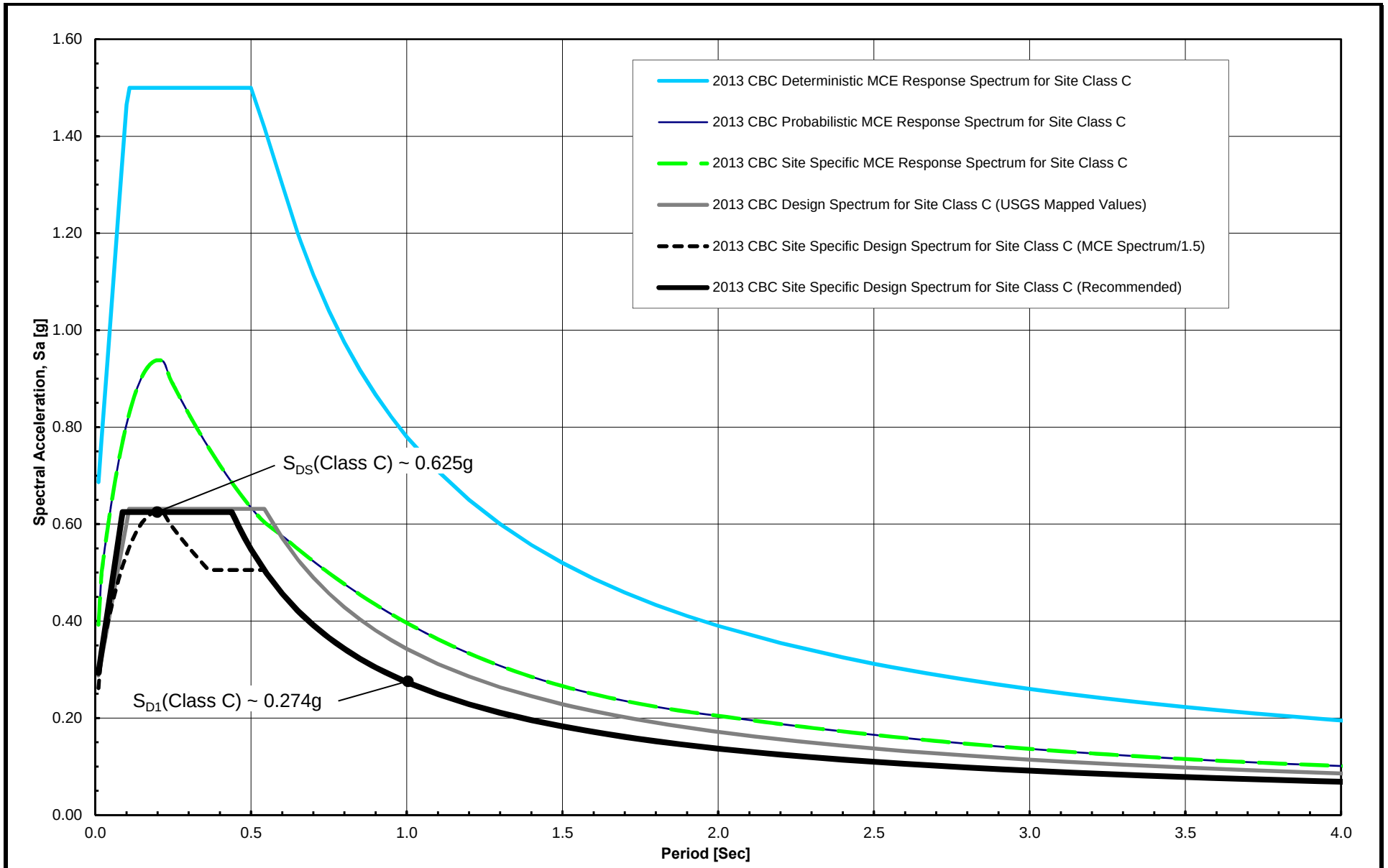
Document No. 15-0098
Project No. SD437
FIGURE 9.1



GROUP DELTA

PROBABILISTIC SPECTRA

Document No. 15-0098
Project No. SD437
FIGURE 9.2

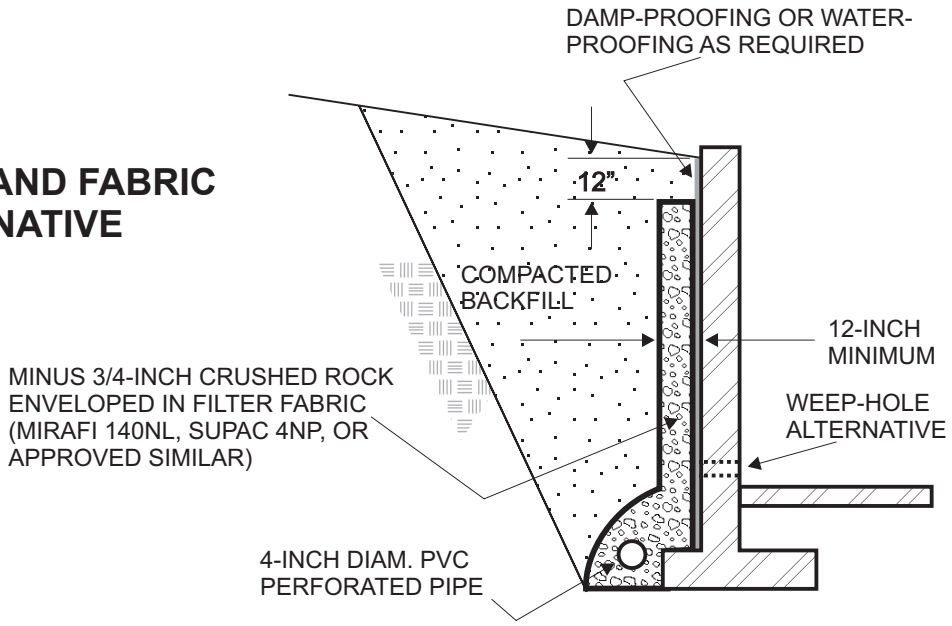


GROUP DELTA

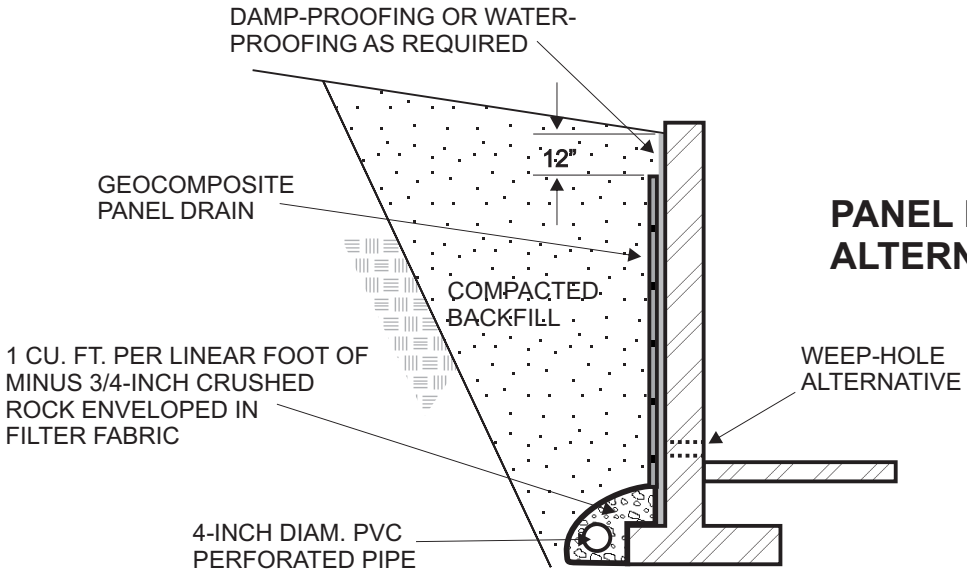
DESIGN SPECTRA

Document No. 15-0098
Project No. SD437
FIGURE 9.3

ROCK AND FABRIC ALTERNATIVE



PANEL DRAIN ALTERNATIVE



NOTES

1) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

2) As an alternative to the perforated pipe and outlet, weep-holes may be constructed. Weep-holes should be at least 2 inches in diameter, spaced no greater than 8 feet, and be located just above grade at the bottom of wall.

3) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599, or similar approved fabric. Filter fabric should be overlapped at least 6-inches.

4) Geocomposite panel drain should consist of Miradrain 6000, J-DRain 400, Supac DS-15, or approved similar product.

5) Drain installation should be observed by the geotechnical consultant prior to backfilling.

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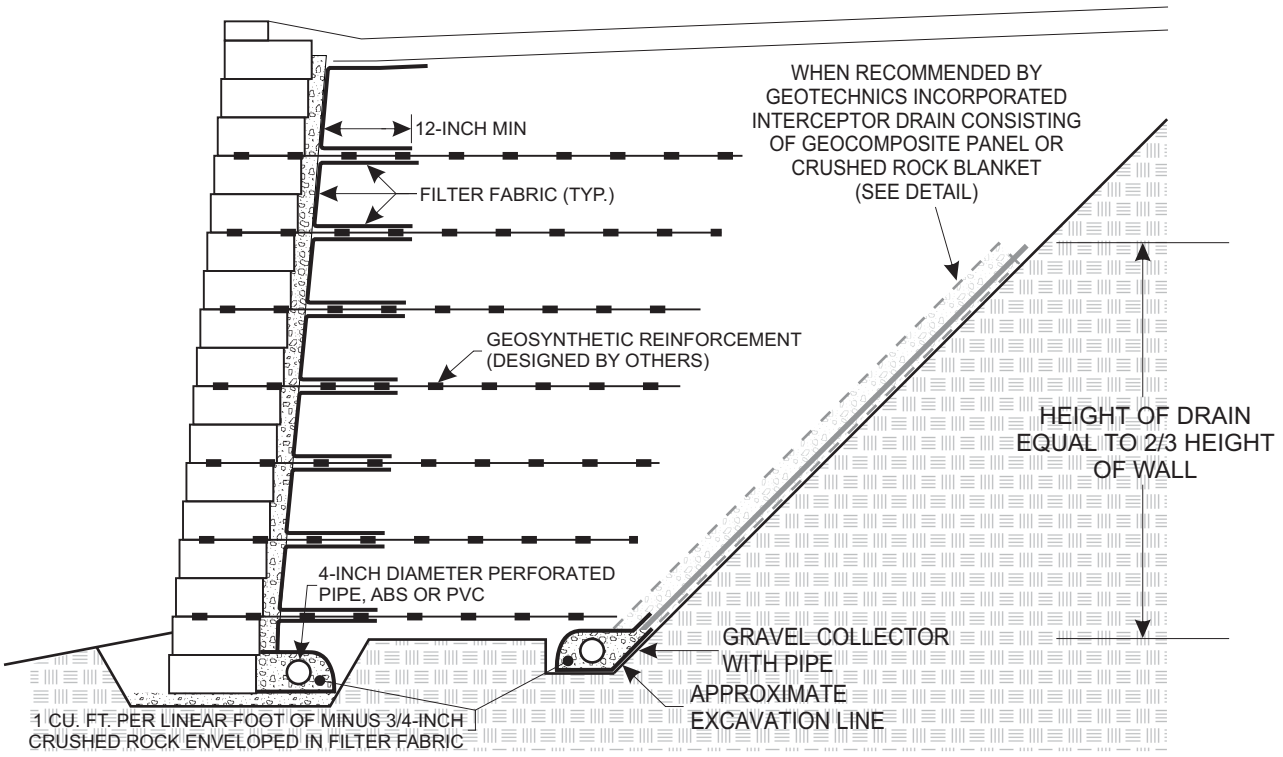
PARKING STRUCTURE 4

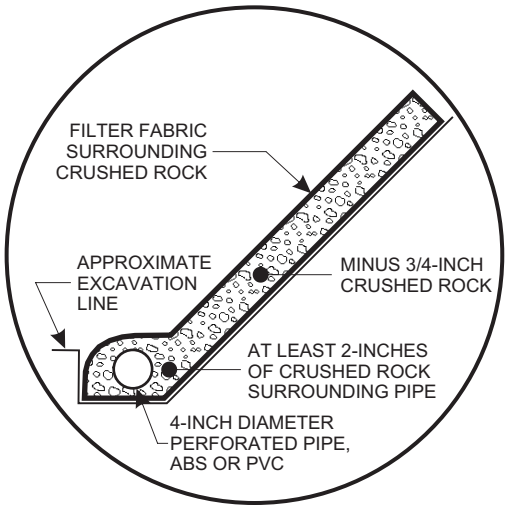
PROJECT NUMBER
SD437

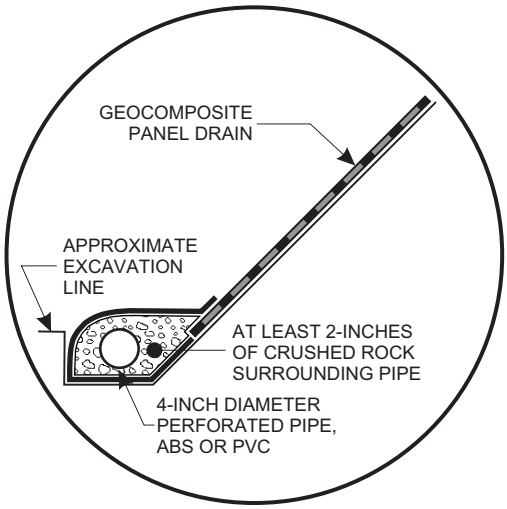
DOCUMENT NUMBER
15-0098

FIGURE NUMBER
10

WALL DRAIN DETAILS







ROCK BLANKET ALTERNATIVE **GEOCOMPOSITE PANEL ALTERNATIVE**

NOTES

1) Geocomposite panel should consist of Miradrain 6000, J-DRain 400, Supac DS-15 or approved similar product.

2) Interceptor drain at excavation backcut may be recommended based on conditions observed during construction.

3) Perforated pipe should outlet through a solid pipe to a free gravity outfall. Perforated pipe and outlet pipe should have a fall of at least 1%.

4) Filter fabric should consist of Mirafi 140N, Supac 5NP, Amoco 4599 or similar approved product. Filter fabric should be overlapped at least 6-inches.

5) Drain installation should be observed by the geotechnical consultant prior to backfilling.

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PARKING STRUCTURE 4

PROJECT NUMBER
SD437

DOCUMENT NUMBER
15-0098

FIGURE NUMBER
11

WALL DRAIN DETAILS

INPUT PARAMETERS

Unit Weight of Soil (γ) [pcf]:	125
Backfill Soil Friction Angle (ϕ) [°]:	30
Wall Friction Angle (δ) [°]:	10
Soil Backfill Angle (α) [°]:	0
Wall Batter Angle (β) [°]:	90
Horizontal Acceleration (K_h) [g's]:	0.13
Vertical Acceleration (K_v) [g's]:	0.00

CALCULATED PARAMETERS

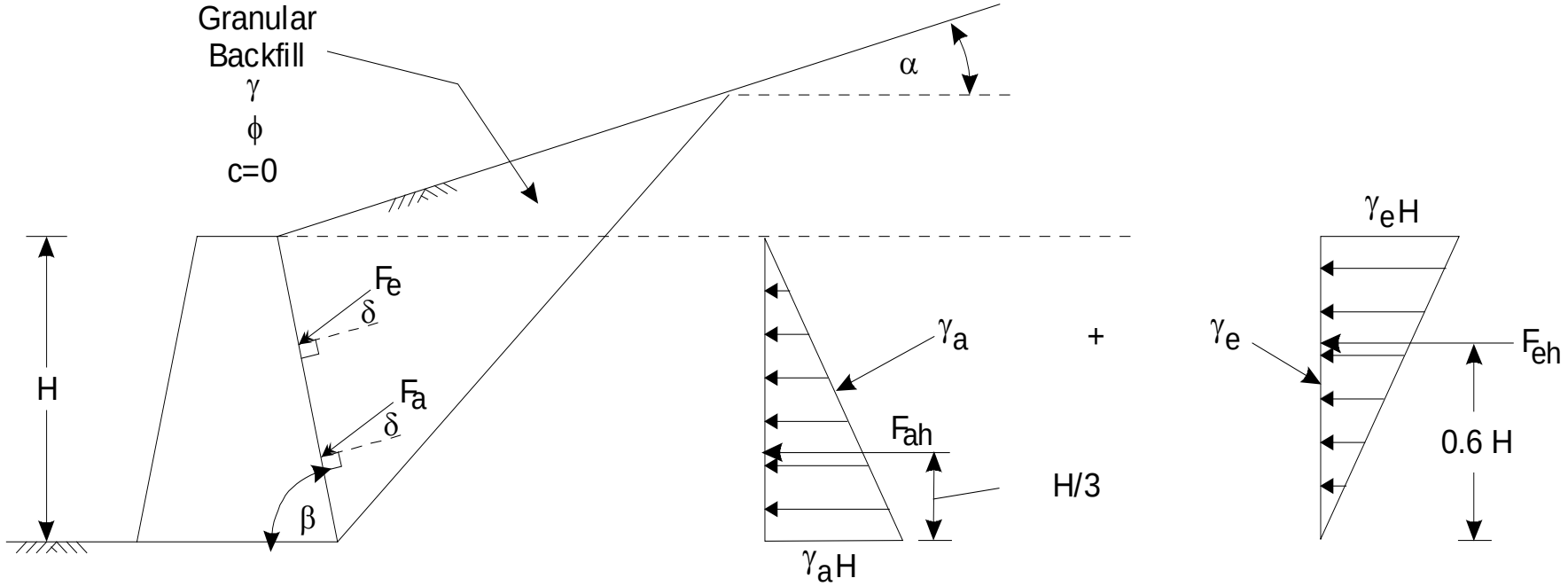
Active Pressure Coefficient (K_a):	0.31
Equivalent Fluid Pressure (γ_a) [pcf]:	39
Seismic Pressure Coefficient (K_{ae}):	0.40
Equivalent Fluid Pressure (γ_{ae}) [pcf]:	49
Equivalent Seismic Pressure (γ_e) [pcf]:	11

Active Pressure Resultant: $F_a = 1/2\ \gamma_a H^2$

Earthquake Pressure Resultant: $F_e = 1/2\ \gamma_e H^2$

Horizontal Component of Active Pressure Resultant $F_{ah} = F_a \cos(\delta+90-\beta)$

Horizontal Component of Seismic Pressure Resultant $F_{eh} = F_e \cos(\delta+90-\beta)$



GROUP DELTA

SEISMIC WALL LOADING (Kh = 0.13g)

Project No. SD437
Document No. 15-0098
FIGURE 12

APPENDIX A
FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

Our field investigation consisted of a visual reconnaissance of the site and drilling nine borings on July 6 and 7, 2015. The borings were drilled to depths of up to about 4 feet below existing grade using a limited access track mounted, 6-inch auger drill rig and a limited access track mounted, 24-inch bucket auger drill rig. Drilling refusal at depths between about 1½ and 3 feet was encountered in the 6-inch auger borings. The approximate locations of the borings are shown on Figure 2. Logs describing the subsurface conditions encountered are presented on the following Figures A-1 through A-5.

Sampling was attempted in the borings using a Standard Penetration Test (SPT) Sampler. The SPT sampler is a split tube with an outside diameter of 2 inches and an inside diameter of 1⅜ inches. No usable samples were obtained from the Standard Penetration Test (SPT) Sampler. The hammer used to drive the SPT samplers weighed 140 pounds, with a free fall of 30 inches. The number of blows needed to drive the SPT samplers is shown on the boring logs. SPT refusal (REF) was encountered when 50 blows were applied during any one of the three 6-inch intervals, a total of 100 blows was applied, or there was no discernible sampler advancement during the application of 10 successive blows. Bulk samples were also obtained from auger cuttings. The approximate locations of the bulk samples are indicated on the logs with shading.

Boring locations were established in the field by taping distances from landmarks shown on the plans provided. The locations shown should not be considered more accurate than is implied by the method of measurement used. The lines designating the interface between soil units on the boring logs are determined by interpolation and are therefore approximations. The transition between the materials may be abrupt or gradual. Further, soil conditions at locations between the borings may be substantially different from those at the specific locations explored. It should be recognized that the passage of time can result in changes in the soil conditions reported in our logs.

APPENDIX A
FIELD EXPLORATION (Continued)

Subsurface Exploration Summary

Boring No.	Ground Surface Elevation (feet)	Total Depth (feet)	Groundwater Depth (feet)
B-1	513	1½	Not encountered
B-2	513	4	Not encountered
B-3	515	4	Not encountered
B-4	510	3	Not encountered
B-5	511	1½	Not encountered
B-6	513	4	Not encountered
B-7	504	1½	Not encountered
B-8	505	2½	Not encountered
B-9	510	1½	Not encountered

Note: Ground surface elevations estimated to nearest 1 foot from topographic survey data provided by Google Earth.

LOG OF EXPLORATION BORING NO. 1

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 513 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
0		4-inch asphalt concrete	
1		**RESIDUAL SOIL:** Fat clay (CH) to lean clay (CL) , dark brown, medium to high plasticity, moist. Refusal on cemented Paralic Deposits	
2		SPT Blow counts: 50 for 0 inches at 1½ feet	
3		Total depth 1½ feet No groundwater encountered	
4			
5			
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 2

Logged by: RCF **Date Drilled:** 7/7/2015

Equipment Used: 24-inch bucket auger limited access rig **Elevation:** 513 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
0		**FILL:** Silty sand (SM), dark yellowish brown, fine to coarse, dry to moist.	
1			
2		**PARALIC DEPOSITS:** Clayey sand to lean clay (SC/CL) with cobble, dark yellowish brown, moist, strongly cemented, cobbles up to 8 inch diameter.	
3			
4		Refusal on cemented Paralic Deposits	
5		Total depth: 4 feet No groundwater encountered	
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 3

Logged by: RCF **Date Drilled:** 7/7/2015

Equipment Used: 24-inch bucket auger limited access rig **Elevation:** 515 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
1		**FILL:** Silty sand (SM), dark yellowish brown, fine to medium, dry.	Maximum Density Optimum Moisture Direct Shear Expansion Index
2		**PARALIC DEPOSITS:** Clayey sand to lean clay (SC/CL) with cobbles, dark yellowish brown, moist, strongly cemented, cobbles up to 6 inch diameter.	
3			
4		Refusal on cemented Paralic Deposits	
5		SPT Blow counts: 50 for 6 inches at 4 feet	
6		Total depth: 4 feet No groundwater encountered	
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 4

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 510 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
1		**FILL:** Silty sand (SM), very dark brown, fine to medium, moist.	Atterberg Limits LL=35, PL=12, PI=23 Resistivity & pH Sulfate Chloride Expansion Index
2		**RESIDUAL SOIL:** Lean clay (CL), very dark brown, medium plasticity, moist.	
3		(LL=35, PL=23) Refusal on cemented Paralic Deposits	
4		Total depth: 3 feet No groundwater encountered	
5			
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 5

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 511 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
		4-inch asphalt concrete	
1	■	**RESIDUAL SOIL:** Gravelly fat clay (CH), very dark brown, high plasticity, moist. Refusal on cemented Paralic Deposits	
2		Total depth: 1½ feet No groundwater encountered	
3			
4			
5			
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 6

Logged by: RCF **Date Drilled:** 7/7/2015

Equipment Used: 24-inch bucket auger limited access rig **Elevation:** 513 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
1	■	**FILL:** Silty sand (SM), yellowish brown, fine to medium, dry.	
2	■	**RESIDUAL SOIL:** Gravelly lean clay (CL), reddish brown, medium plasticity, dry to moist.	
3	■	**PARALIC DEPOSITS:** Clayey sand to lean clay (SC/CL) with cobble, dark yellowish brown to reddish brown, moist, strongly cemented, cobbles up to 6 inch diameter. Refusal on cemented Paralic Deposits	Resistivity & pH Sulfate Chloride
4		SPT Blow counts: 50 for 4 inches at 3 feet	
5		Total depth: 4 feet No groundwater encountered	
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 7

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 504 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
1		**RESIDUAL SOIL:** Gravelly lean clay (CL), yellowish brown, dry to moist.	
		Refusal on cemented Paralic Deposits	
2		Total depth: 1½ feet	
3		No groundwater encountered	
4			
5			
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 8

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 505 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
1		**FILL:** Silty gravel (GM), yellowish brown, fine to medium, dry to moist.	Sieve analysis with hydro R-Value
2		Gravelly silty sand (SM), dark yellowish brown, fine to medium, dry. Refusal on cemented Paralic Deposits	
3		Total depth: 2½ feet	
4		No groundwater encountered	
5			
6			
7			
8			
9			
10			

LOG OF EXPLORATION BORING NO. 9

Logged by: RCF **Date Drilled:** 7/6/2015

Equipment Used: 6-inch auger limited access track rig **Elevation:** 510 Ft.

DEPTH (FT)	BULK SAMPLE	DESCRIPTION	LAB TESTS
		4-inch asphalt concrete	Resistivity & pH Sulfate Chloride R-Value
1		**FILL:** Gravelly silty sand (SM), dark yellowish brown, fine to medium, dry to moist. Refusal on cemented Paralic Deposits	
2		Total depth: 1½ feet No groundwater encountered	
3			
4			
5			
6			
7			
8			
9			
10			

APPENDIX B
LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Selected samples of soils encountered during the investigation were tested using generally accepted testing standards. The soils selected for testing are believed to be generally representative of the materials encountered during the investigation at the site; however, variations may occur in the soils at the site, and the materials tested may not be representative of the materials encountered during construction.

Laboratory testing was conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions and in the same locality. No warranty, express or implied, is made as to the correctness or serviceability of the test results or the conclusions derived from these tests. Where a specific laboratory test method has been referenced, such as ASTM or California Test, the reference applies only to the specified laboratory test method and not to associated referenced test method(s) or practices, and the test method referenced has been used only as a guidance document for the general performance of the test and not as a "Test Standard". A brief description of the tests performed follows:

Classification: Soils were classified visually according to the Unified Soil Classification System as described in ASTM D2488. Visual classification was supplemented by laboratory testing of selected soil samples and classification in general accordance with the laboratory soil classification tests outlined in ASTM D2487. The resultant soil classifications are shown on the boring logs in Appendix A.

Atterberg Limits: The liquid limit, plastic limit, and plasticity index of a selected soil sample were estimated in general accordance with ASTM D4318. The results are shown on the Boring Logs in Appendix A.

Maximum Density/Optimum Moisture: The maximum dry density and optimum moisture content for a selected soil sample were estimated in general accordance with the laboratory procedures outlined in ASTM test method D1557, modified Proctor. The test results are summarized in Figure B-1.1.

R-Value: An R-value test was performed on a sample of the upper soils in general accordance with California Test 301. The test results are shown in Figure B-1.1.

Sulfate Content: To assess the potential for reactivity with below grade concrete, selected soil samples were tested for water-soluble sulfate content. The soluble sulfate was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The extracted solution was then tested for water-soluble sulfate in general accordance with ASTM D516. The test results are shown in Figure B-1.2.

APPENDIX B

LABORATORY TESTING (Continued)

pH and Resistivity: To assess the potential for reactivity with buried metal pipe and below grade ferrous materials, selected soil samples were tested for pH and resistivity in general accordance with California Test 643. The test results are shown in Figure B-1.2.

Chloride Content: Selected soil samples were tested for water-soluble chloride content. The soluble chloride was extracted from the soil under vacuum using a 10:1 (water to dry soil) dilution ratio. The extracted solution was then tested for water-soluble chloride using a calibrated ion specific probe (9617BN) in general accordance with ASTM D512. The test results are shown in Figure B-1.2.

Expansion Index: The expansion index of selected soil samples was estimated in general accordance with ASTM D4829. The test results are shown in Figure B-1.3.

Sieve Analysis: A sieve analysis were performed in general accordance with the laboratory procedures outlined in ASTM test method D422. The grain size distribution was used to estimate presumptive soil strength parameters and foundation design criteria. The results are given in Figure B-2.

Remolded Direct Shear: The shear strength of a selected recompacted soil sample was assessed through direct shear testing performed in general accordance with the laboratory procedures outlined in ASTM test method D3080. The samples were fabricated by compacting bulk samples of the onsite soils to approximately 90 percent of maximum dry density as determined in general accordance with the laboratory procedures outlined in ASTM test method D1557, modified Proctor. The samples were confined under vertical pressures in the laboratory approximately equal to the anticipated range of design effective overburden pressures. The samples were then inundated under water and sheared at a strain rate selected to approximate drained shear conditions. The test results are shown in Figure B-3.

List of Attached Figures

The following figures are attached and complete this appendix:

Laboratory Test Results	Figures B-1.1 through B-1.3
Sieve Analysis Test Results	Figure B-2
Remolded Direct Shear Test Results	Figure B-3

MAXIMUM DENSITY/OPTIMUM MOISTURE CONTENT
(ASTM D 1557)

SAMPLE	SAMPLE DESCRIPTION	MAXIMUM DRY DENSITY (PCF)	OPTIMUM MOISTURE (%)
B-3A@ 0-3½ ft	Dark yellowish brown silty sand to clayey sand (SM/SC) with 0% rock	124	11
B-3B@ 0-3½ ft	Dark yellowish brown silty sand to clayey sand (SM/SC) with 5% rock	125.5	10.5
B-3C@ 0-3½ ft	Dark yellowish brown silty sand to clayey sand (SM/SC) with 10% rock	127	10
B-3D@ 0-3½ ft	Dark yellowish brown silty sand to clayey sand (SM/SC) with 15% rock	128.5	9.5

R-VALUE TEST RESULTS
(California Test 301)

SAMPLE	SAMPLE DESCRIPTION	R-VALUE
Combined B-8 and B-9	Dark yellowish brown gravelly silty sand (SM)	24

pH, RESISTIVITY, SULFATE AND CHLORIDE TEST RESULTS

SAMPLE NO.	WATER-SOLUBLE SULFATE CONTENT (% of Dry Soil Weight) [ASTM D 516]	PH [CALTRANS 643]	RESISTIVITY (ohm-cm) [CALTRANS 643]	WATER-SOLUBLE CHLORIDE CONTENT (% of Dry Soil Weight) [SMEWW4500CL-C]
B-4@ 0-3 ft	0.03	7.04	488	0.02
B-6@ 2½-4 ft	0.03	6.45	1067	0.01
B-9@ 0-1½ ft	0.01	5.17	4066	<0.01

Soil Resistivity in ohm-cm	General Degree of Corrosivity to Ferrous Metal
Less than 1,000	Extremely Corrosive
1,000 to 3,000	Highly Corrosive
3,000 to 5,000	Corrosive
5,000 to 10,000	Moderately Corrosive
10,000 to 20,000	Mildly Corrosive
Greater than 20,000	Essentially Non-Corrosive

Reference: Corrosion Basics: An Introduction, 2nd Edition, by Pierre R. Roberge

Water Soluble Chloride (Cl) Content in % of Dry Soil Weight	General Degree of Chloride Exposure to Reinforcing Steel
Over 0.5%	Very Severe
0.1% to 0.5%	Severe
0.05% to 0.1%	Moderate
Up to 0.05%	Negligible

Reference: SBC 2008, Section R4.4, Table 4.4.2

Water Soluble Sulfate (SO_4) Content in % of Dry Soil Weight	General Degree of Reactivity with Concrete
$SO_4 > 2.00\%$	Very Severely Reactive
$0.20\% \leq SO_4 \leq 2.00\%$	Severely Reactive
$0.10\% \leq SO_4 < 0.20\%$	Moderately Reactive
$0.00\% \leq SO_4 < 0.10\%$	Negligible

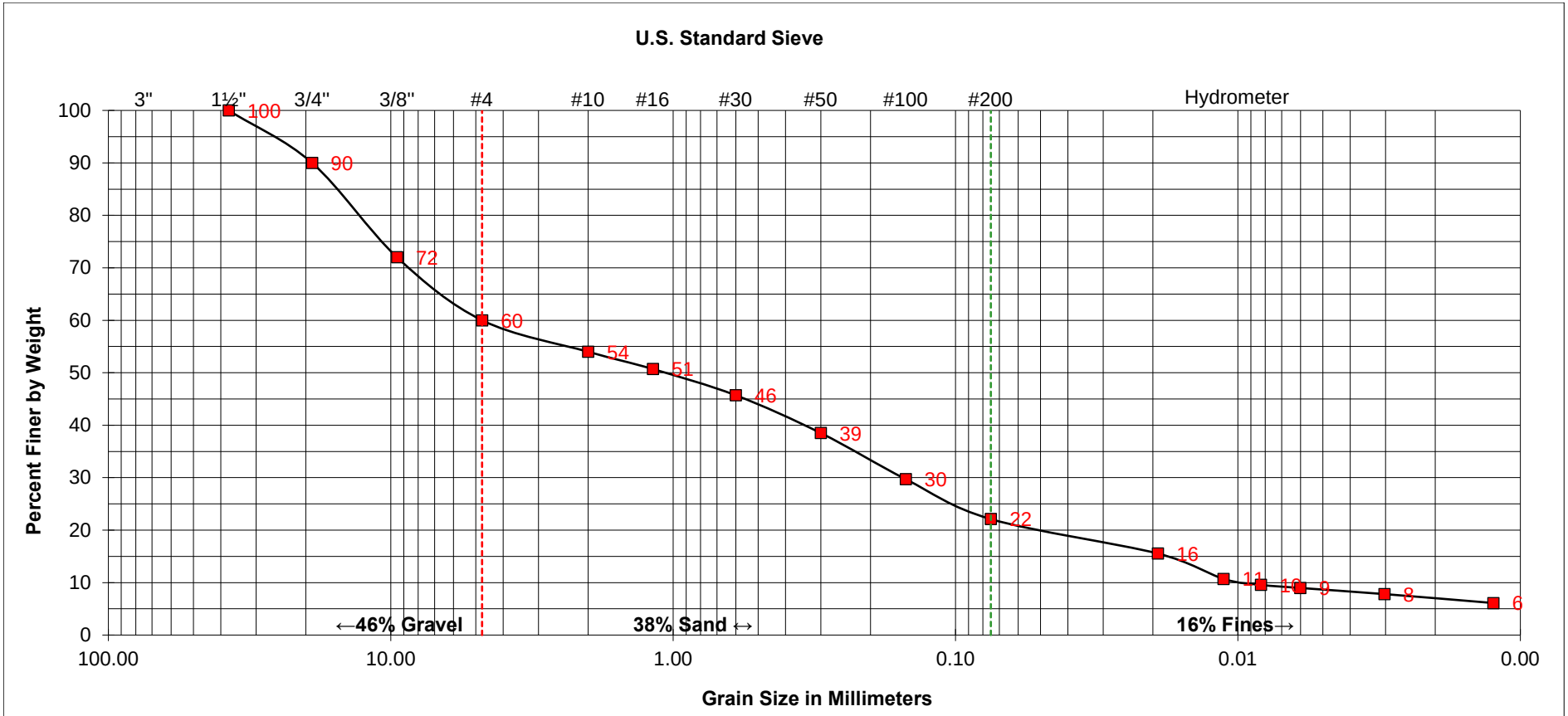
Reference: ACI 318, Table 4.3.1

EXPANSION INDEX TEST RESULTS
(ASTM D4829)

SAMPLE NO.	SAMPLE DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
B-4@ 0-3 ft	Very dark brown lean clay (CL)	45	Low
B-3@ 0-3½ ft	Dark yellowish brown silty sand to clayey sand (SM/SC)	8	Very low

EXPANSION INDEX CRITERIA

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very High



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	
EXPLORATION NUMBER:	B-8
SAMPLE DEPTH:	0-2½ ft

UNIFIED SOIL CLASSIFICATION SYSTEM	
GROUP SYMBOL:	GM
GROUP NAME:	Silty gravel with sand (GM)

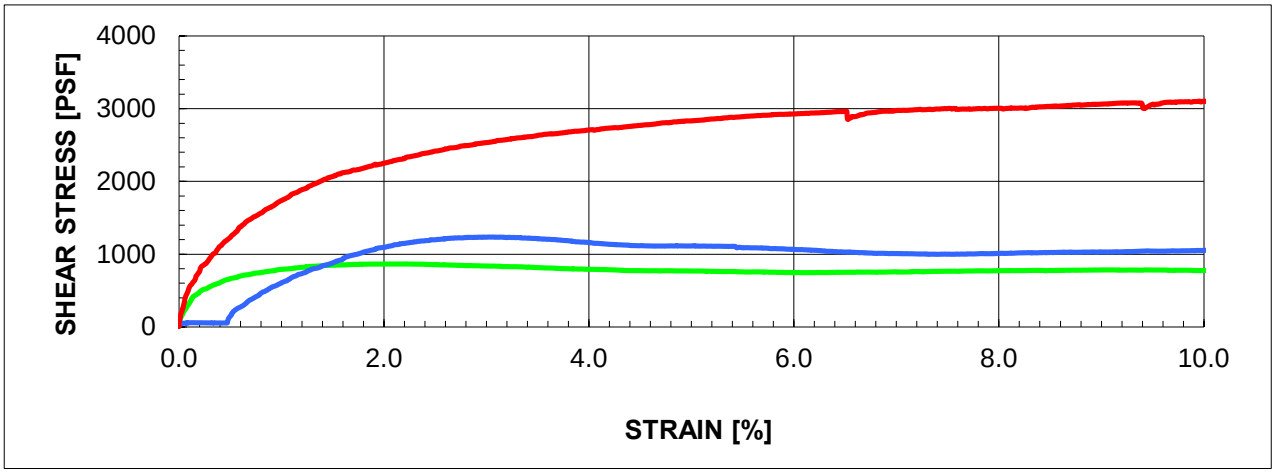
PARTICLE PERCENTAGE	
40%	GRAVEL
38%	SAND
22%	FINES

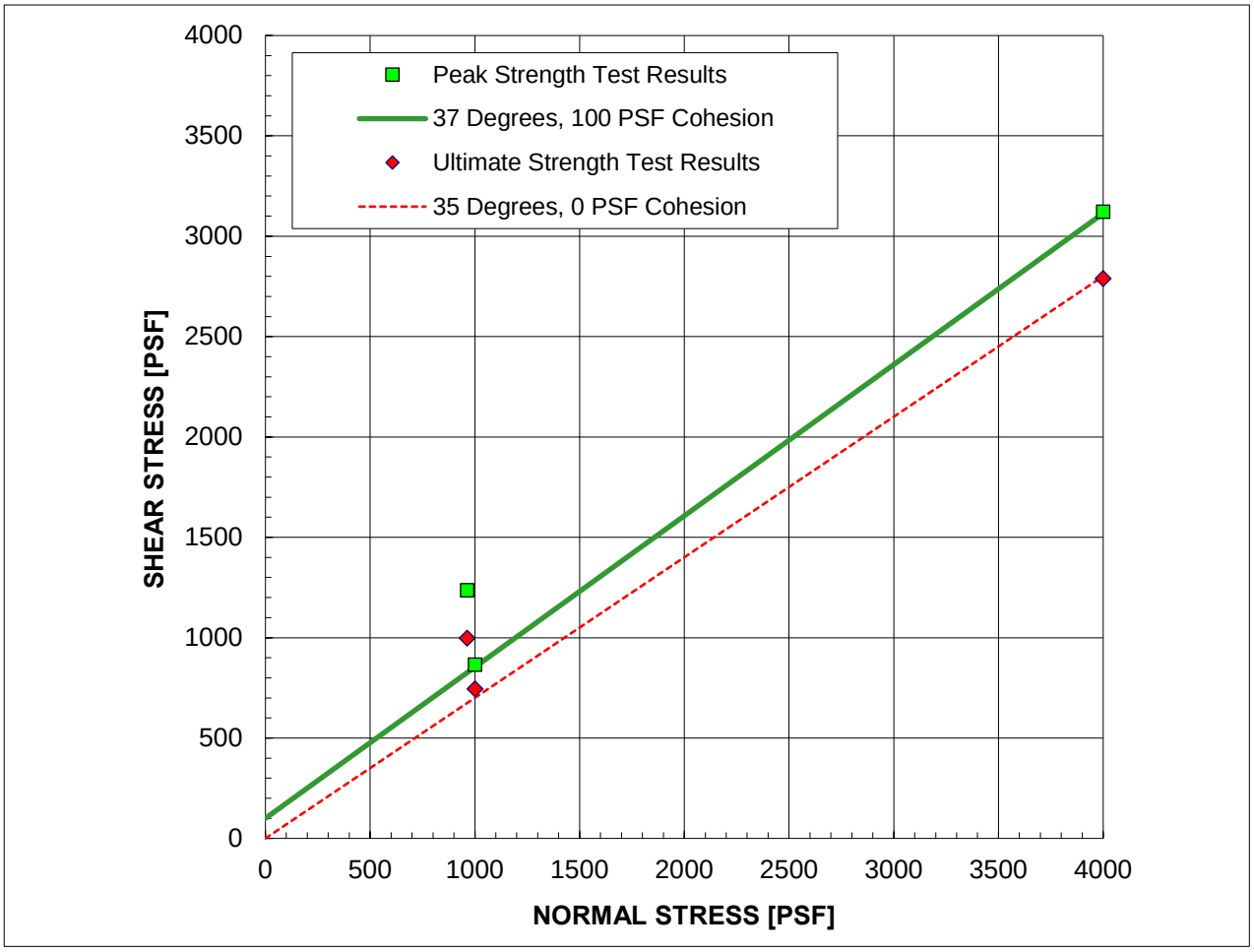
ATTERBERG LIMITS	
LIQUID LIMIT:	N/A
PLASTIC LIMIT:	N/A
PLASTICITY INDEX:	N/A

GROUP DELTA

SOIL CLASSIFICATION

Project No. SD437
Document No. 15-0098
FIGURE B-2





SAMPLE: Boring B3 @ 0-3½ Ft.

Dark Yellow Brown Silty Sand (SM)
(Sample Compacted to 90% ASTM D1557)

	PEAK	ULTIMATE
ϕ'	37.0 °	35.0 °
C'	100 PSF	0 PSF

STRAIN RATE: 0.0020 IN/MIN
(Sample was inundated, consolidated, and drained)

	IN-SITU	AS-TESTED
γ_d	111.7 PCF	111.7 PCF
w_c	10.7 %	18.2 %

GROUP DELTA

REMOLDED DIRECT SHEAR TEST RESULTS

Project No. SD437
Document No. 15-0098
FIGURE B-3

APPENDIX C
SEISMIC SHEAR WAVE SURVEY

APPENDIX C

SEISMIC SHEAR WAVE SURVEY

A Seismic Shear Wave Survey was performed by SouthWest Geophysics, Inc. on June 16, 2015 for the proposed Parking Structure No. 4 at Miramar College in San Diego, California. The scope of services consisted on the performance of a refraction microtremor (ReMi) survey at a preselected area at the project site. The purpose of the survey was to develop a Shear-Wave Velocity Profile for the project site. The attached report presents the survey methodology, equipment, used, analysis, and findings from the study.

SOUTHWEST GEOPHYSICS, INC.
YOUR SUBSURFACE SOLUTION

June 24, 2015
Project No. 115283

Mr. Jim Sanders
Group Delta
9245 Activity Road, Suite 103
San Diego, CA 92126

Subject: Geophysical Evaluation
Proposed Parking Structure, Miramar College
San Diego, California

Dear Mr. Sanders:

In accordance with your authorization, we have performed geophysical survey services pertaining to the proposed parking structure for Miramar College in San Diego, California (Figure 1). The purpose of our survey was to develop a Shear-wave velocity profile for the project site. Our services were performed on June 16, 2015. This report presents the survey methodology, equipment used, analysis, and findings from our study.

Our scope of services included the performance of a refraction microtremor (ReMi) survey at a preselected area at the project site (Figure 2). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The ReMi survey included the use of a 24-channel Geometrics Geode seismograph and 24 4.5-Hz vertical component geophones. The geophones were spaced 10 feet apart for a line length of 230 feet. Fifteen records, each 30 seconds long, were recorded and then downloaded to a computer. The data were later processed using SeisOpt® ReMi™ software. Figure 3 depicts the general site conditions in the survey area.

Figure 4 and Table 1 present the results from our survey. Based on our analysis of the collected data, the average characteristic site Shear-wave velocity down to a depth of 100 feet is 2,096 feet per second (CBC, 2010). This value corresponds to a site classification of **C**.

TABLE 1
ReMi Results

Line No. (Long/Lat)[1]	Depth (feet)	Shear Wave Velocity (feet/second)
RL-1 (-117.118202874 / 32.908056971) (-117.118531585 / 32.908624932)	0 – 3	902
	3 – 15	1,164
	15 – 35	1,427
	35 – 79	2,966
	79 – 100	3,998

1 – Coordinate System WGS 1984

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Patrick Lehrmann, P.G., P.Gp.
Principal Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

PFL/HV/hv

Attachments: Figure 1 – Site Location Map
Figure 2 – Line Location Map
Figure 3 – Site Photograph
Figure 4 – ReMi Results, RL-1

Distribution: Addressee (electronic)

SITE LOCATION MAP

N

Proposed Parking Structure, Miramar College
San Diego, California

Project No.: 115283

Date: 06/15

SOUTHWEST GEOPHYSICS INC.

Figure 1

LINE LOCATION MAP

N

Proposed Parking Structure, Miramar College
San Diego, California

Project No.: 115283 | Date: 06/15

SOUTHWEST GEOPHYSICS INC.

Figure 2

0 100 200

approximate scale in feet

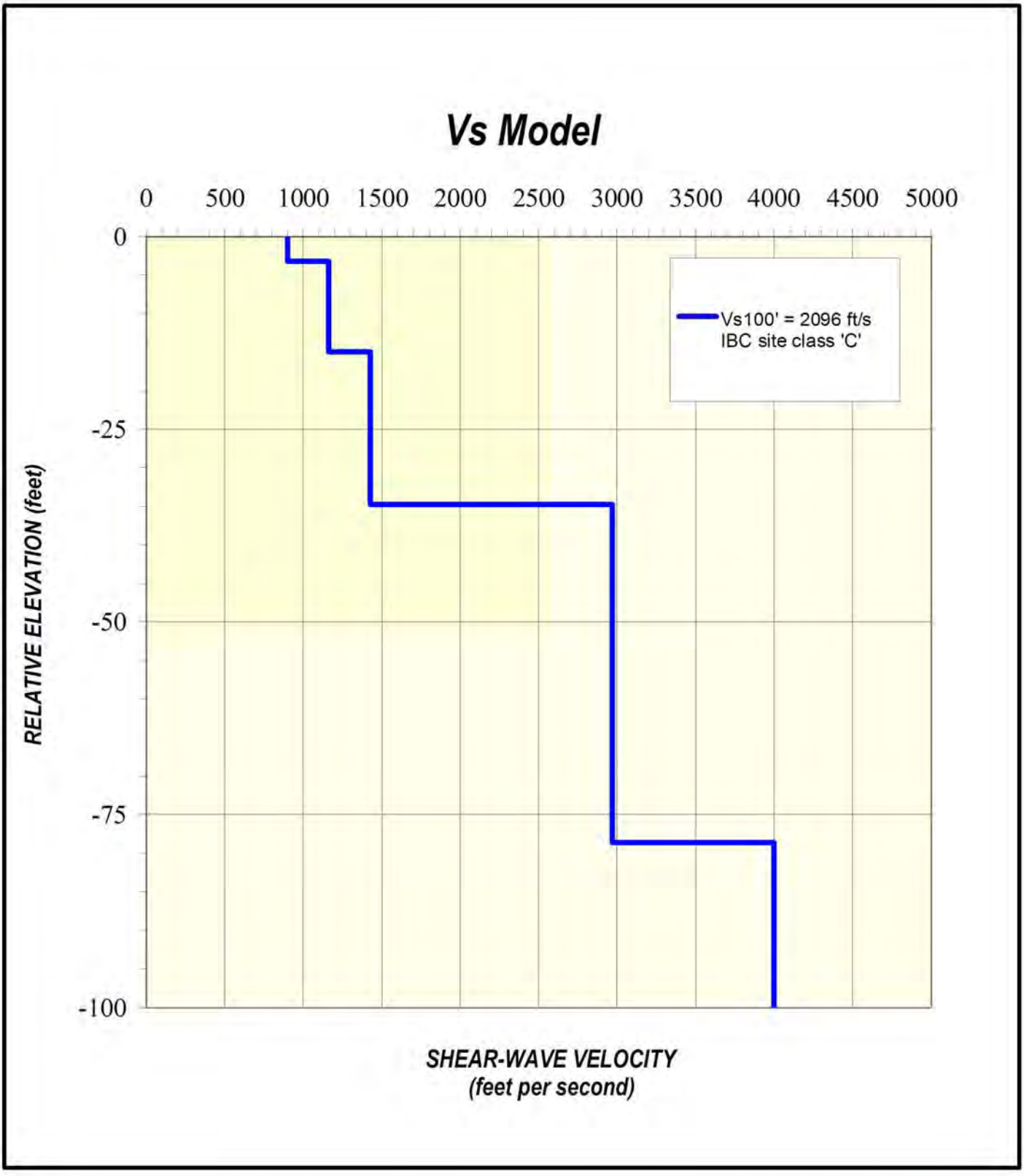
SITE PHOTOGRAPH

Proposed Parking Structure, Miramar College
San Diego, California

Project No.: 115283 | Date: 06/15

SOUTHWEST GEOPHYSICS INC.

Figure 3



ReMi RESULTS **RL-1**	Proposed Parking Structure, Miramar College San Diego, California	SOUTHWEST GEOPHYSICS INC.
	Project No.: 115283 | Date: 06/15	Figure 4

APPENDIX D
GROUND WATER LEVELS

APPENDIX D

GROUND WATER LEVELS

Historic groundwater data obtained wells from the United States Geologic Survey, National Water Information System are provided for twelve wells located within about 5 miles of the subject site and covering a period from approximately 1969 through 1985. Based on a review of the data it appears that the regional groundwater elevation in the vicinity of the site is probably currently located at least 40 feet below the site.

Groundwater levels for California

Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325723117034401

Minimum number of levels = 1

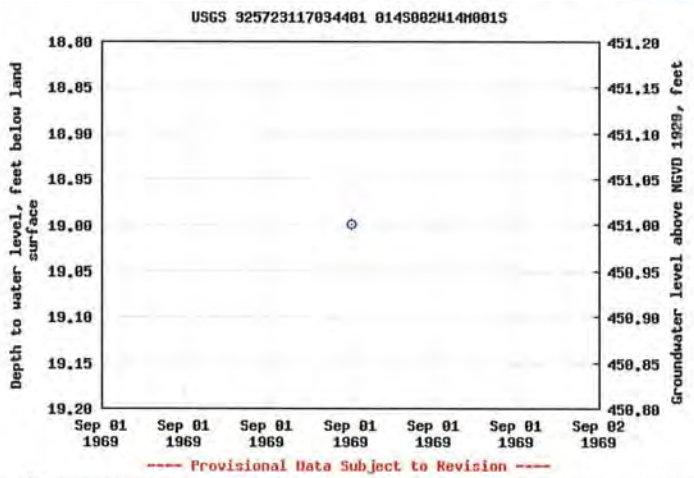
Save file of selected sites to local disk for future upload

USGS 325723117034401 014S002W14M001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°57'23", Longitude 117°03'44" NAD27
Land-surface elevation 470.00 feet above NGVD29
The depth of the well is 19.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



Breaks in the plot represent a gap of at least one year between field measurements.
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Groundwater levels for California

Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325625117105501

Minimum number of levels = 1

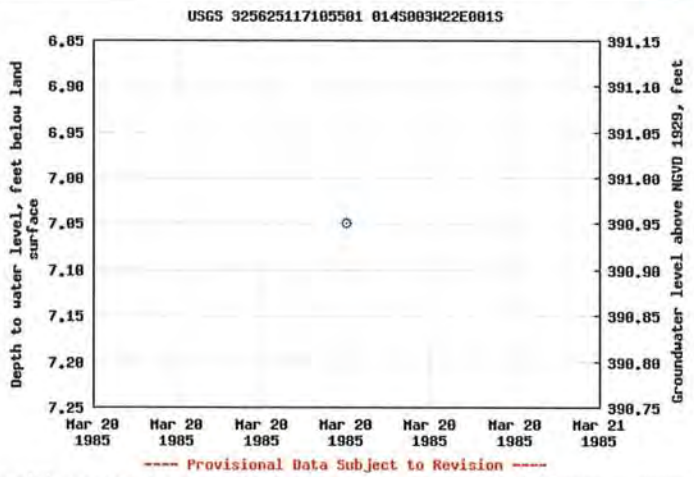
Save file of selected sites to local disk for future upload

USGS 325625117105501 014S003W22E001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°56'25", Longitude 117°10'55" NAD27
Land-surface elevation 398.00 feet above NGVD29
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



Breaks in the plot represent a gap of at least one year between field measurements.
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U.S. Department of the Interior | U.S. Geological Survey
Title: Groundwater for California: Water Levels
URL: http://nwis.waterdata.usgs.gov/ca/nwis/gwlevels?

USA.gov
Take Pride in America

Page Contact Information: California Water Data Maintainer
Page Last Modified: 2011-08-11 17:03:18 EDT
0.25 0.23 nadww01

Groundwater levels for California

Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325623117062601

Minimum number of levels = 1

Save file of selected sites to local disk for future upload

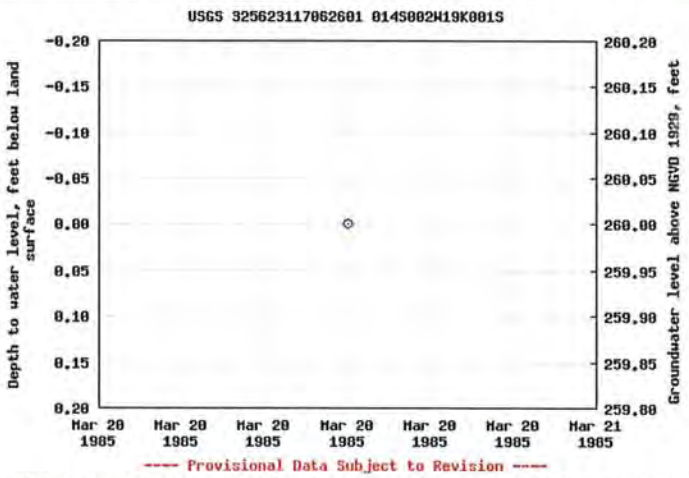
USGS 325623117062601 014S002W19K001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°56'23", Longitude 117°06'26" NAD27
Land-surface elevation 260.00 feet above NGVD29
The depth of the well is 6.0 feet below land surface.
The depth of the hole is 6.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats

- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Groundwater levels for California

Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325557117032502

Minimum number of levels = 1

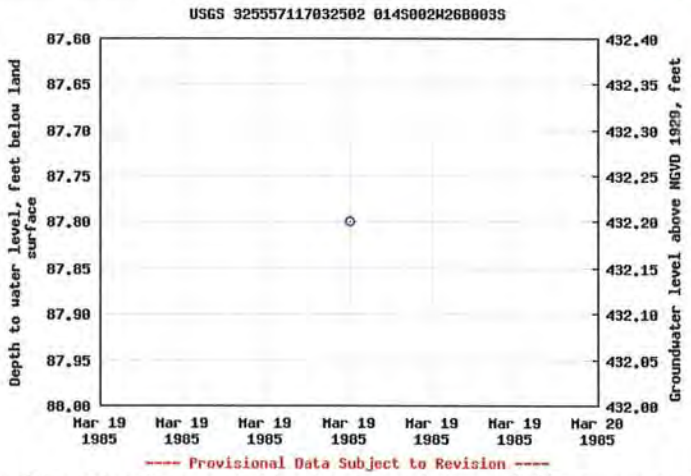
Save file of selected sites to local disk for future upload

USGS 325557117032502 014S002W26B003S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°55'57", Longitude 117°03'25" NAD27
Land-surface elevation 520.00 feet above NGVD29
The depth of the well is 216 feet below land surface.
The depth of the hole is 216 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325520117035101

Minimum number of levels = 1

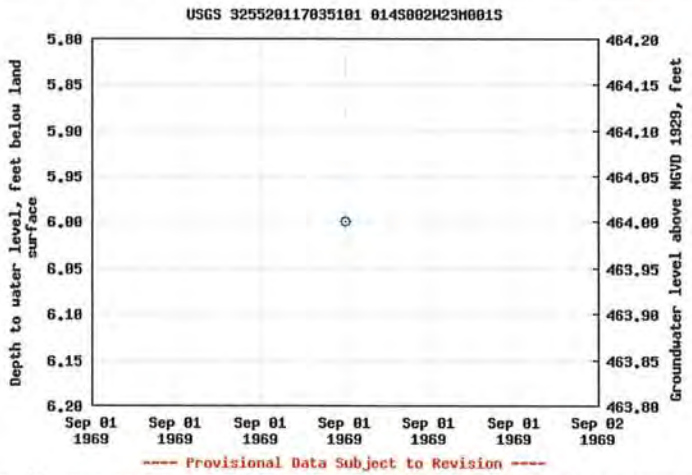
Save file of selected sites to local disk for future upload

USGS 325520117035101 014S002W23M001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°55'20", Longitude 117°03'51" NAD27
Land-surface elevation 470.00 feet above NGVD29
The depth of the well is 13.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Search Criteria

Agency code = usgs
site_no list =
- 325723117034401

Minimum number of levels = 1

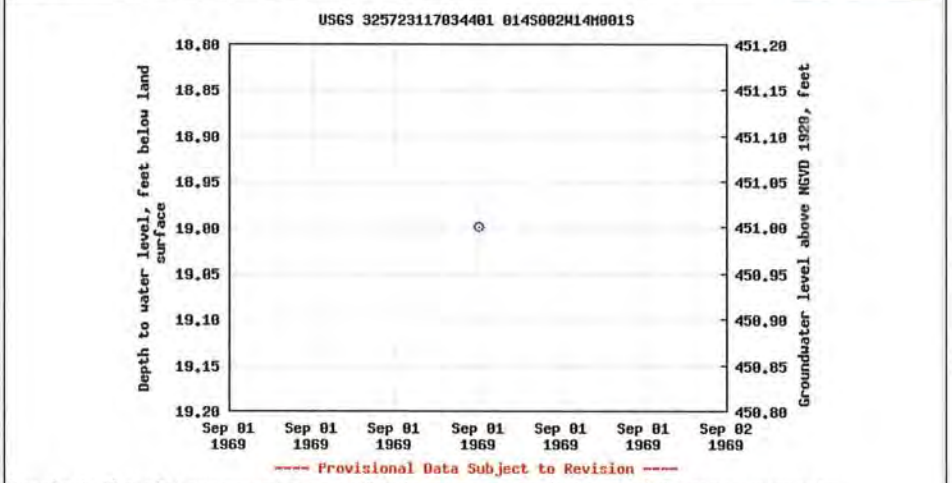
Save file of selected sites to local disk for future upload

USGS 325723117034401 014S002W14M001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°57'23", Longitude 117°03'44" NAD27
Land-surface elevation 470.00 feet above NGVD29
The depth of the well is 19.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325714117034601

Minimum number of levels = 1

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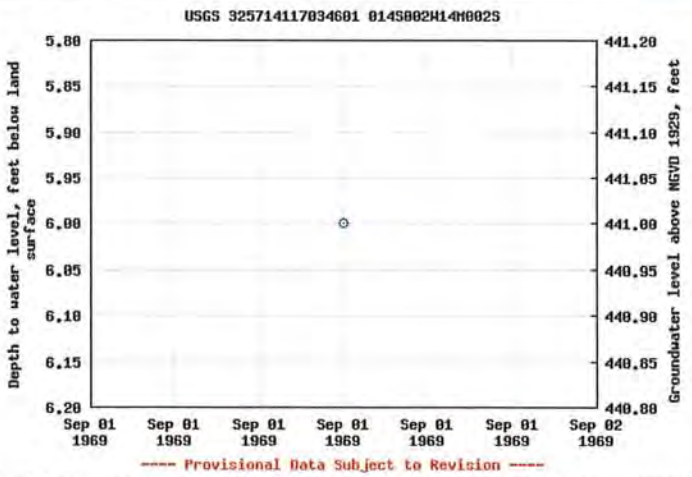
USGS 325714117034601 014S002W14M002S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°57'14", Longitude 117°03'46" NAD27
Land-surface elevation 447.00 feet above NGVD29
The depth of the well is 19.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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0.24 0.23 nadww01

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Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325700117040701

Minimum number of levels = 1

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USGS 325700117040701 014S002W15R001S

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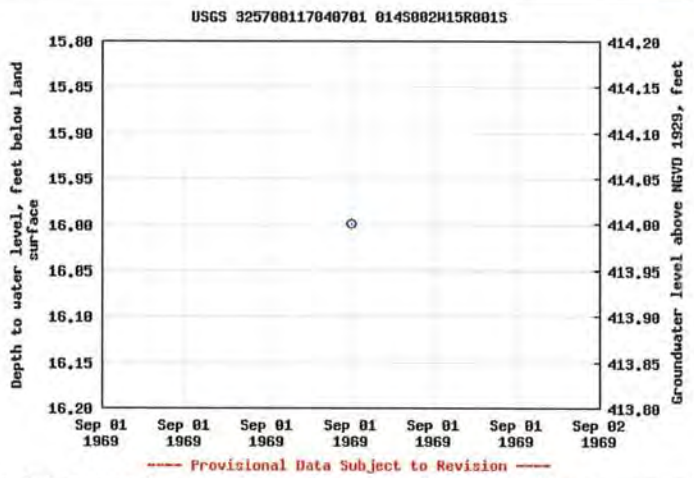
San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°57'00", Longitude 117°04'07" NAD27
Land-surface elevation 430.00 feet above NGVD29
The depth of the well is 39.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325700117033601

Minimum number of levels = 1

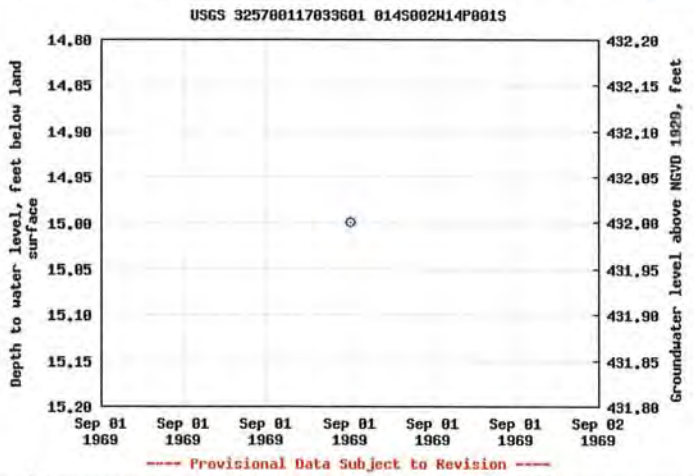
Save file of selected sites to local disk for future upload

USGS 325700117033601 014S002W14P001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°57'00", Longitude 117°03'36" NAD27
Land-surface elevation 447.00 feet above NGVD29
The depth of the well is 59.0 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Search Criteria

Agency code = usgs
site_no list =
- 325650117040201

Minimum number of levels = 1

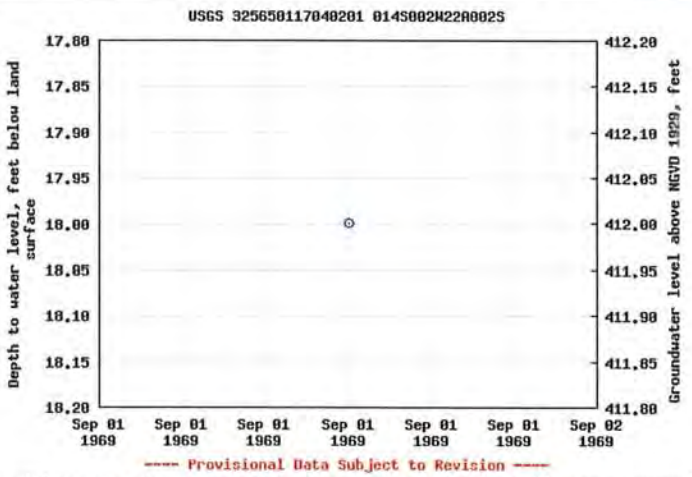
Save file of selected sites to local disk for future upload

USGS 325650117040201 014S002W22A002S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°56'50", Longitude 117°04'02" NAD27
Land-surface elevation 430.00 feet above NGVD29
The depth of the well is 180 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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0.24 0.22 nadww01

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Search Results -- 1 sites found

Search Criteria

Agency code = usgs
site_no list =
- 325636117035501

Minimum number of levels = 1

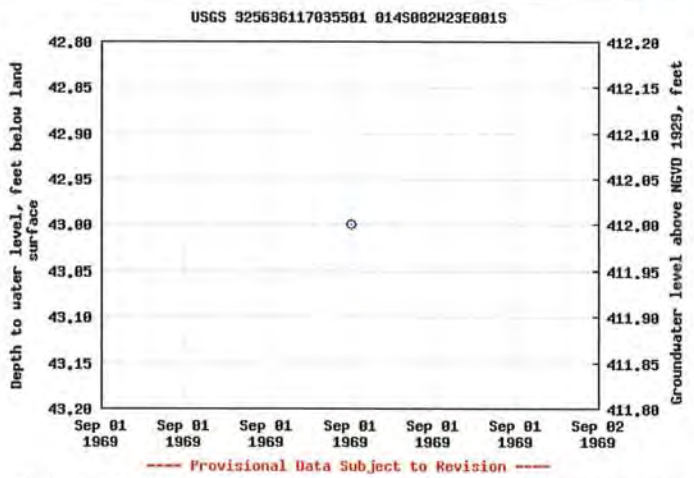
Save file of selected sites to local disk for future upload

USGS 325636117035501 014S002W23E001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°56'36", Longitude 117°03'55" NAD27
Land-surface elevation 455.00 feet above NGVD29
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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Agency code = usgs
site_no list =
- 325646117043701

Minimum number of levels = 1

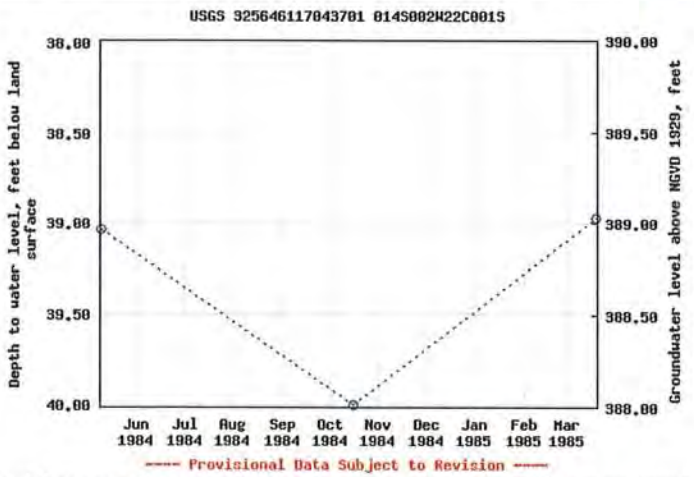
Save file of selected sites to local disk for future upload

USGS 325646117043701 014S002W22C001S

Available data for this site | Groundwater: Field measurements | GO

San Diego County, California
Hydrologic Unit Code 18070304
Latitude 32°56'46", Longitude 117°04'37" NAD27
Land-surface elevation 428.00 feet above NGVD29
The depth of the well is 350 feet below land surface.
This well is completed in the Other aquifers (N9999OTHER) national aquifer.

Output formats
- Table of data
- Tab-separated data
- Graph of data
- Reselect period



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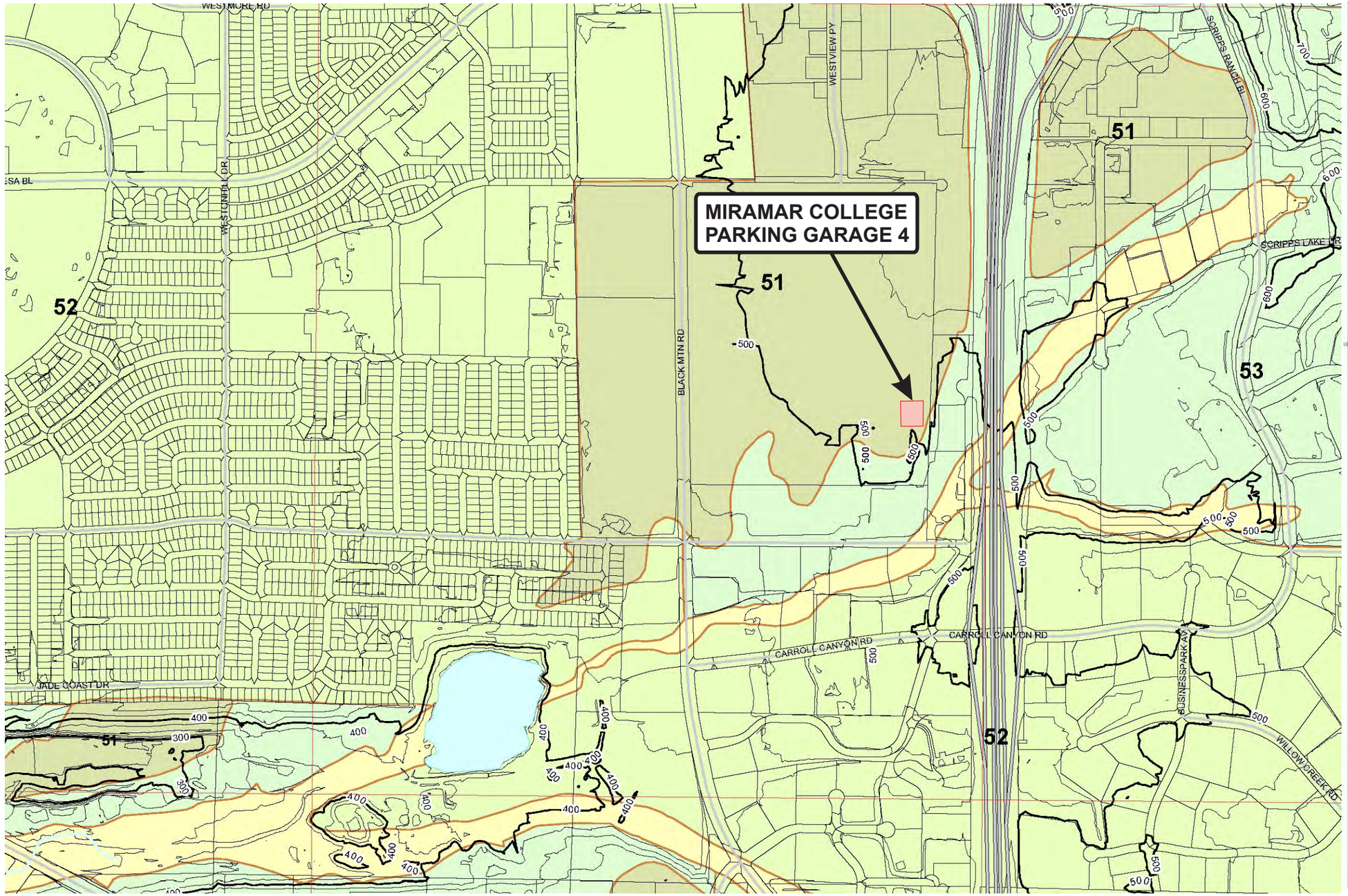
0.25 0.23 nadww01

APPENDIX E
CITY OF SAN DIEGO SEISMIC SAFETY STUDY

APPENDIX E

CITY OF SAN DIEGO
SEISMIC SAFETY STUDY

Figure E-1 presents a portion of Map 35 from the City of San Diego Seismic Safety Study with the site of the proposed Parking Structure No. 4, Miramar College Campus indicated on the map. The site is located within Geologic Hazard Category 51. This hazard category is described in the City of San Diego Seismic Safety Study as "Level mesas, underlain by terrace deposits and bedrock, nominal risk". The nearest adjacent offsite slope is located within Geologic Hazard Category 53. This hazard category is described in the City of San Diego Seismic Safety Study as "Level or sloping terrain, unfavorable geologic structure, Low to moderate risk".



LEGEND

Geologic Hazard Categories

FAULT ZONES
- 11 Active, Alquist-Priolo Earthquake Fault Zone
- 12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown
- 13 Downtown special fault zone

LANDSLIDES
- 21 Confirmed, known, or highly suspected
- 22 Possible or conjectured

SLIDE-PRONE FORMATIONS
- 23 Friars: neutral or favorable geologic structure
- 24 Friars: unfavorable geologic structure
- 25 Ardath: neutral or favorable geologic structure
- 26 Ardath: unfavorable geologic structure
- 27 Otay, Sweetwater, and others

LIQUEFACTION
- 31 High Potential -- shallow groundwater major drainages, hydraulic fills
- 32 Low Potential -- fluctuating groundwater minor drainages

COASTAL BLUFFS
- 41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure
- 42 Generally unstable Unfavorable bedding plains, high erosion
- 43 Generally unstable Unfavorable jointing, local high erosion
- 44 Moderately stable Mostly stable formations, local high erosion
- 45 Moderately stable Some minor landslides, minor erosion
- 46 Moderately stable Some unfavorable geologic structure, minor or no erosion
- 47 Generally stable Favorable geologic structure, minor or no erosion, no landslides
- 48 Generally stable Broad beach areas, developed harbor

OTHER TERRAIN
- 51 Level mesas -- underlain by terrace deposits and bedrock nominal risk
- 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
- 53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk
- 54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk
- 55 Modified terrain (graded sites) Nominal risk

Water (Bays and Lakes)

FAULTS
- Fault
- Inferred Fault
- Concealed Fault
- Shear Zone

City of San Diego
SEISMIC SAFETY STUDY
Geologic Hazards and Faults

N / S

0 400 800 1,600 2,400 Feet

0 120 240 480 720 960

Modified from: City of San Diego Seismic Safety Study Geologic Hazard and Fault Map page 35

GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

PROJECT NUMBER: SD437
DOCUMENT NUMBER: 15-0098
PROJECT NAME: MIRAMAR COLLEGE PARKING GARAGE 4
FIGURE NUMBER: E-1

CITY OF SAN DIEGO
SEISMIC SAFETY STUDY

APPENDIX F
DYNAMIC SETTLEMENT ANALYSIS

APPENDIX F

DYNAMIC SETTLEMENT ANALYSES

Dynamic seismic analyses were performed on the SPT data obtained from Group Delta Consultants Inc. borings B-3 and B-8 drilled for the Miramar Parking Structure No. 4 using a Peak Ground Acceleration equal to 0.4 S_{DS} (0.250g). The dynamic seismic analysis was based on the simplified techniques originally presented by Seed and Idriss (1982), with recent improvements from the 1996 and 1998 NCEER workshops as summarized by Youd et al (2001). The dynamic seismic analysis was conducted in general accordance with the recommended procedures for implementation of DMG special publication 117 (SCES, 1999).

The SPT data $(N_1)_{60CS}$ was normalized for overburden pressure and corrected for fines content. The fines correction was based on the laboratory test results and the methods described in Youd et al (2001). The SPT data was also corrected for rod energy, borehole diameter, rod length, and sampling method. The results of the dynamic seismic analyses are presented in Figures F-1.1 through F-2.2, corresponding to borings B 3 and B 8, respectively. The top chart of the figure shows the relationship between the cyclic resistance ratio (CRR) and the corrected standard penetration resistance $(N_1)_{60CS}$ for the SPT data. The bottom chart in the figure shows the average shear stresses induced in the soil profile due to a design PGA of 0.250g (S_{DS}/2.5), as well as the shear stress required to cause dynamic seismic in the soil profile. Both of these parameters are plotted as a function of depth. Where the earthquake-induced shear stress exceeds the stress required to cause dynamic seismic, dynamic seismic is possible. As shown on the figures, soil dynamic seismic is likely at the site.

Seismic settlement analysis was performed using the safety factor against Dynamic seismic for each soil layer and the corrected SPT data based on methods by Ishihara (1992). The results of our analysis indicate total seismic settlements for borings B 3 and B 8 of about ⅔ to 1⅛ inch.

GROUP DELTA

LIQUEFACTION AND DYNAMIC SETTLEMENT ANALYSIS

GDC Project No. SD437
Document No. 15-0098
FIGURE F-1.1

After Seed & Idriss (1967, 1971, 1982), Idriss (1997), NCEER Workshop (1997), SCEC (1999), Youd et al. (2001)

SOIL PARAMETERS

Soil Type Number	Average Moist Weight [PCF]	Percent Passing No. 200
1	120	37%
2	125	62%
3	130	22%

SITE PARAMETERS

Peak Site Acceleration $a_{(max)}/g$	Moment Magnitude M_W	Depth to Water [FT]
0.25	7.2	50

Magnitude Scaling Factor
1.06

CORRECTION FACTORS

Rod Energy Correction C_E	Borehole Diameter C_B	Sampling Method C_S
1.00	1.15	1.00

[1]**E**arthquake **I**nduced **S**hear **S**tress
[2]**S**tress **R**equired to **C**ause **L**iquefaction

SOIL PROFILE

Depth [FT]	Soil Type Number	Est. Field SPT (N)	$(N_1)_{60CS}$
0	1	5	14
1	1	5	14
2	3	50	84
3	3	50	84
4	3		
5	3		
6	3		
7	3		
8	3		
9	3		
10	3		
11	3		
12	3		
13	3		
14	3		
15	3		
16	3		
17	3		
18	3		
19	3		
20	3		
21	3		
22	3		
23	3		
24	3		
25	3		
26	3		
27	3		
28	3		
29	3		
30	3		
31	3		
32	3		
33	3		
34	3		
35	3		
36	3		
37	3		
38	3		
39	3		
40	3		
41	3		
42	3		
43	3		
44	3		
45	3		
46	3		
47	3		
48	3		
49	3		
50	3		

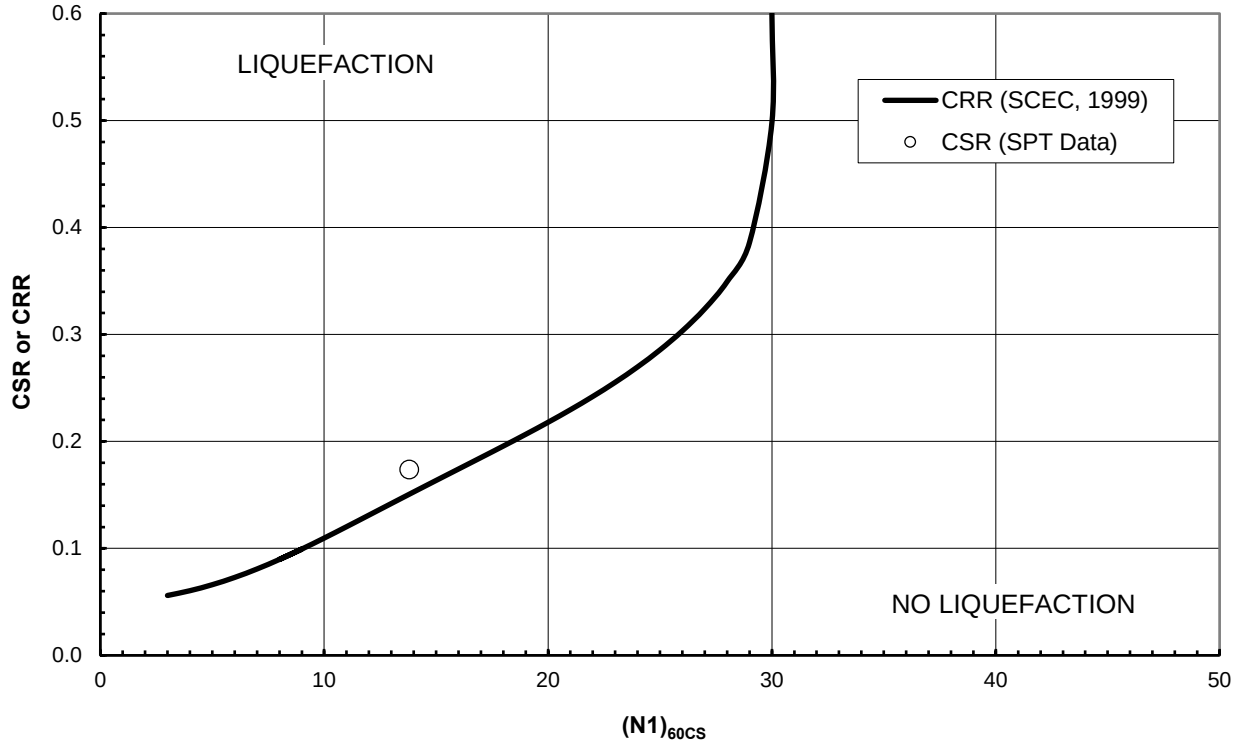
STATIC STRESS

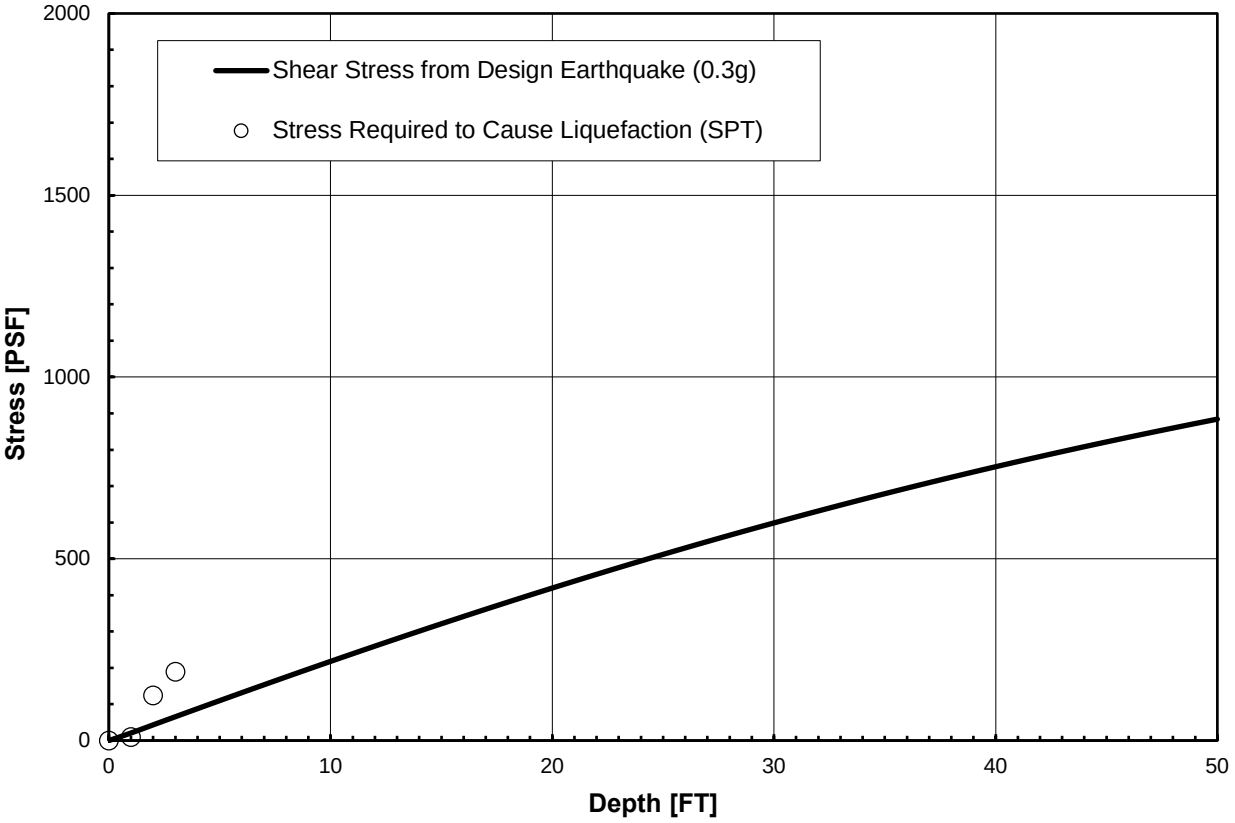
Total [PSF]	Effective [PSF]	Reduction R_d
0	0.0	1.01
120.0	120.0	1.00
250.0	250.0	1.00
380.0	380.0	1.00
510.0	510.0	1.00
640.0	640.0	0.99
770.0	770.0	0.99
900.0	900.0	0.99
1030.0	1030.0	0.98
1160.0	1160.0	0.98
1290.0	1290.0	0.98
1420.0	1420.0	0.97
1550.0	1550.0	0.97
1680.0	1680.0	0.97
1810.0	1810.0	0.96
1940.0	1940.0	0.96
2070.0	2070.0	0.95
2200.0	2200.0	0.95
2330.0	2330.0	0.95
2460.0	2460.0	0.94
2590.0	2590.0	0.94
2720.0	2720.0	0.93
2850.0	2850.0	0.93
2980.0	2980.0	0.92
3110.0	3110.0	0.92
3240.0	3240.0	0.91
3370.0	3370.0	0.91
3500.0	3500.0	0.90
3630.0	3630.0	0.90
3760.0	3760.0	0.90
3890.0	3890.0	0.89
4020.0	4020.0	0.89
4150.0	4150.0	0.88
4280.0	4280.0	0.88
4410.0	4410.0	0.87
4540.0	4540.0	0.87
4670.0	4670.0	0.86
4800.0	4800.0	0.86
4930.0	4930.0	0.85
5060.0	5060.0	0.84
5190.0	5190.0	0.84
5320.0	5320.0	0.83
5450.0	5450.0	0.83
5580.0	5580.0	0.82
5710.0	5710.0	0.82
5840.0	5840.0	0.81
5970.0	5970.0	0.81
6100.0	6100.0	0.80
6230.0	6230.0	0.80
6360.0	6360.0	0.79
6490.0	6490.0	0.79

SEISMIC STRESS

EISS[1] [PSF]	SRCL[2] [PSF]	Settlement [IN]
0.0	0.0	0.00
20.8	10.0	0.66
43.3	124.5	0.66
65.6	189.2	0.66
87.8		0.66
109.9		0.66
131.8		0.66
153.6		0.66
175.2		0.66
196.6		0.66
217.9		0.66
238.9		0.66
259.8		0.66
280.5		0.66
301.0		0.66
321.3		0.66
341.4		0.66
361.3		0.66
380.9		0.66
400.4		0.66
419.6		0.66
438.6		0.66
457.3		0.66
475.8		0.66
494.1		0.66
512.1		0.66
529.9		0.66
547.5		0.66
564.8		0.66
581.9		0.66
598.7		0.66
615.3		0.66
631.6		0.66
647.6		0.66
663.5		0.66
679.1		0.66
694.4		0.66
709.5		0.66
724.3		0.66
738.9		0.66
753.3		0.66
767.4		0.66
781.3		0.66
794.9		0.66
808.3		0.66
821.5		0.66
834.5		0.66
847.2		0.66
859.8		0.66
872.1		0.66
884.2		0.66

Settlement in inches 0.66





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LIQUEFACTION ANALYSIS (B-3)

GDC Project No. SD437
Document No. 15-0098
FIGURE F-1.1

GROUP DELTA

LIQUEFACTION AND DYNAMIC SETTLEMENT ANALYSIS

GDC Project No. SD437
Document No. 15-0098
FIGURE F-1.2

After Seed & Idriss (1967, 1971, 1982), Idriss (1997), NCEER Workshop (1997), SCEC (1999), Youd et al. (2001)

SOIL PARAMETERS

Soil Type Number	Average Moist Weight [PCF]	Percent Passing No. 200
1	120	23%
2	120	38%
3	130	22%

SITE PARAMETERS

Peak Site Acceleration $a_{(max)}/g$	Moment Magnitude M_W	Depth to Water [FT]
0.25	7.2	50

Magnitude Scaling Factor
1.06

CORRECTION FACTORS

Rod Energy Correction C_E	Borehole Diameter C_B	Sampling Method C_S
1.00	1.15	1.00

[1]**E**arthquake **I**nduced **S**hear **S**tress
[2]**S**tress **R**equired to **C**ause **L**iquefaction

SOIL PROFILE

Depth [FT]	Soil Type Number	Est. Field SPT (N)	$(N_1)_{60CS}$
0	1	5	12
1	1	5	12
2	3	50	84
3	3	50	84
4	3		
5	3		
6	3		
7	3		
8	3		
9	3		
10	3		
11	3		
12	3		
13	3		
14	3		
15	3		
16	3		
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22	3		
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24	3		
25	3		
26	3		
27	3		
28	3		
29	3		
30	3		
31	3		
32	3		
33	3		
34	3		
35	3		
36	3		
37	3		
38	3		
39	3		
40	3		
41	3		
42	3		
43	3		
44	3		
45	3		
46	3		
47	3		
48	3		
49	3		
50	3		

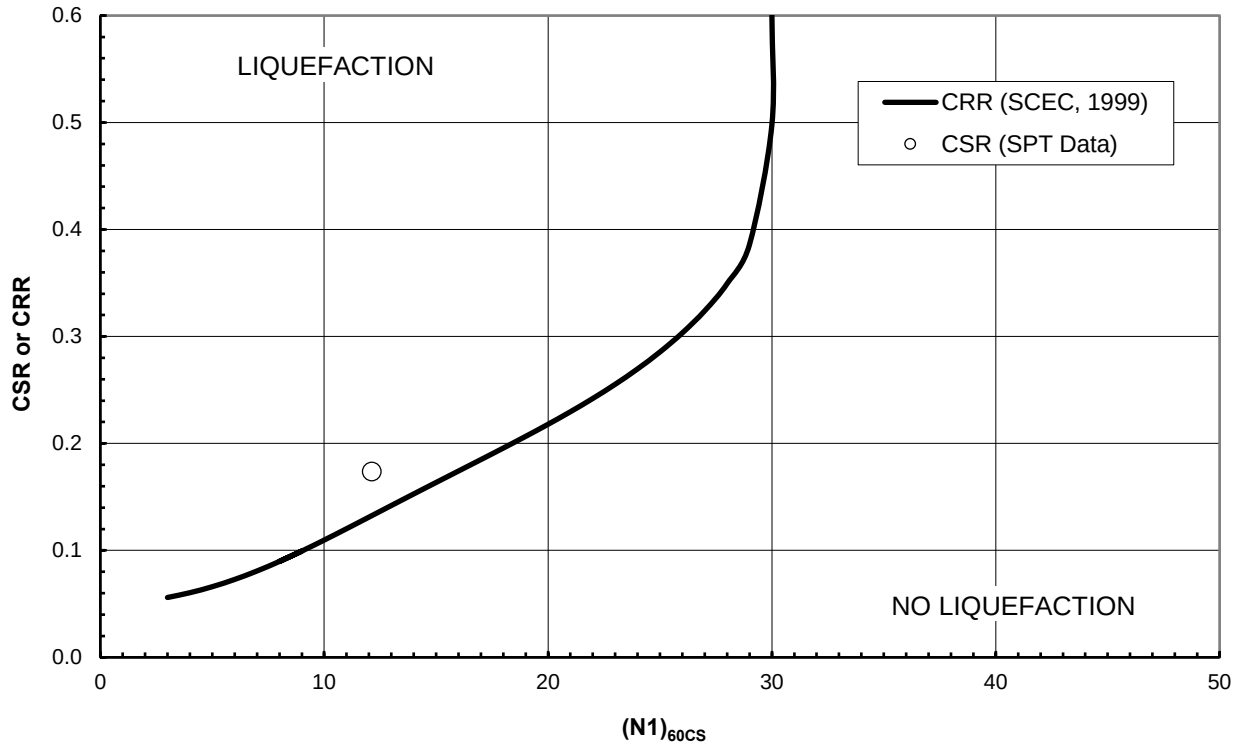
STATIC STRESS

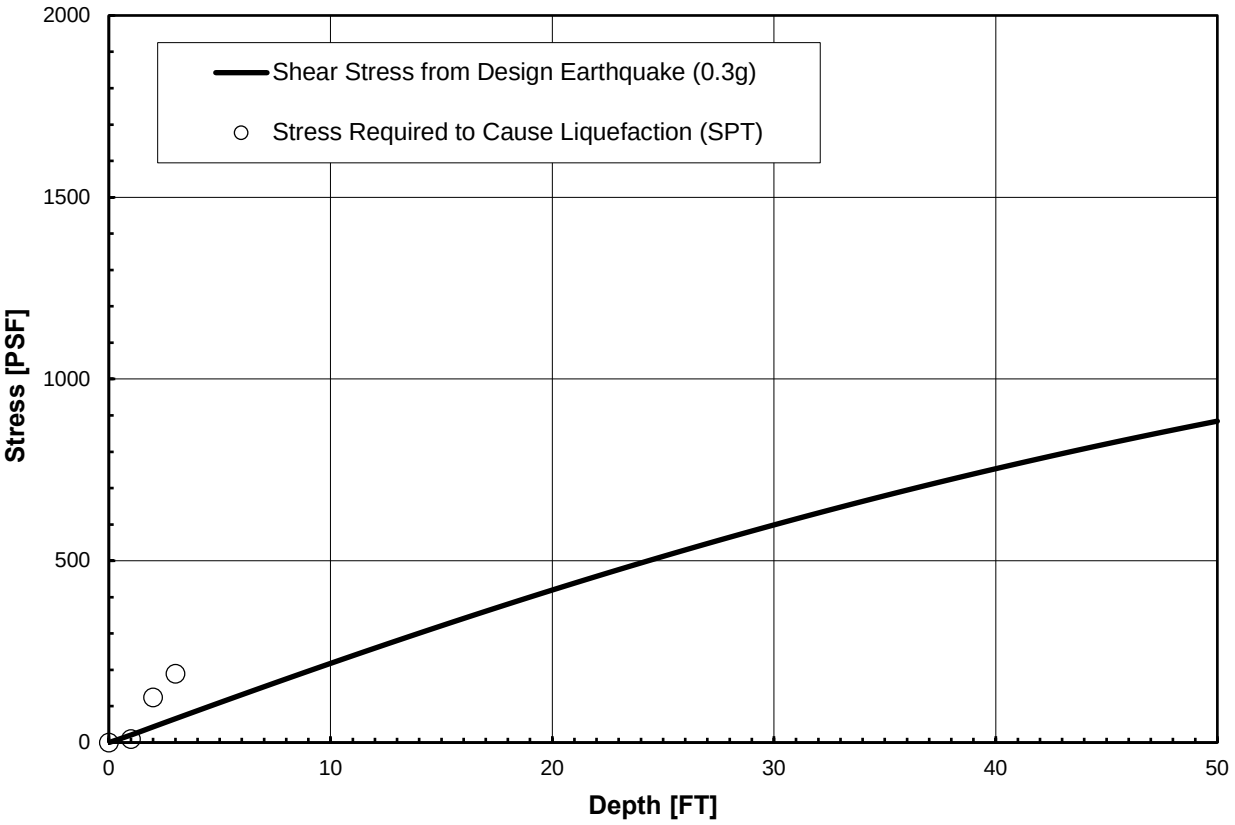
Total [PSF]	Effective [PSF]	Reduction R_d
0	0.0	1.01
120.0	120.0	1.00
250.0	250.0	1.00
380.0	380.0	1.00
510.0	510.0	1.00
640.0	640.0	0.99
770.0	770.0	0.99
900.0	900.0	0.99
1030.0	1030.0	0.98
1160.0	1160.0	0.98
1290.0	1290.0	0.98
1420.0	1420.0	0.97
1550.0	1550.0	0.97
1680.0	1680.0	0.97
1810.0	1810.0	0.96
1940.0	1940.0	0.96
2070.0	2070.0	0.95
2200.0	2200.0	0.95
2330.0	2330.0	0.95
2460.0	2460.0	0.94
2590.0	2590.0	0.94
2720.0	2720.0	0.93
2850.0	2850.0	0.93
2980.0	2980.0	0.92
3110.0	3110.0	0.92
3240.0	3240.0	0.91
3370.0	3370.0	0.91
3500.0	3500.0	0.90
3630.0	3630.0	0.90
3760.0	3760.0	0.90
3890.0	3890.0	0.89
4020.0	4020.0	0.89
4150.0	4150.0	0.88
4280.0	4280.0	0.88
4410.0	4410.0	0.87
4540.0	4540.0	0.87
4670.0	4670.0	0.86
4800.0	4800.0	0.86
4930.0	4930.0	0.85
5060.0	5060.0	0.84
5190.0	5190.0	0.84
5320.0	5320.0	0.83
5450.0	5450.0	0.83
5580.0	5580.0	0.82
5710.0	5710.0	0.82
5840.0	5840.0	0.81
5970.0	5970.0	0.81
6100.0	6100.0	0.80
6230.0	6230.0	0.80
6360.0	6360.0	0.79
6490.0	6490.0	0.79

SEISMIC STRESS

EISS[1] [PSF]	SRCL[2] [PSF]	Settlement [IN]
0.0	0.0	0.00
20.8	10.0	0.66
43.3	124.5	1.32
65.6	189.2	1.32
87.8		1.32
109.9		1.32
131.8		1.32
153.6		1.32
175.2		1.32
196.6		1.32
217.9		1.32
238.9		1.32
259.8		1.32
280.5		1.32
301.0		1.32
321.3		1.32
341.4		1.32
361.3		1.32
380.9		1.32
400.4		1.32
419.6		1.32
438.6		1.32
457.3		1.32
475.8		1.32
494.1		1.32
512.1		1.32
529.9		1.32
547.5		1.32
564.8		1.32
581.9		1.32
598.7		1.32
615.3		1.32
631.6		1.32
647.6		1.32
663.5		1.32
679.1		1.32
694.4		1.32
709.5		1.32
724.3		1.32
738.9		1.32
753.3		1.32
767.4		1.32
781.3		1.32
794.9		1.32
808.3		1.32
821.5		1.32
834.5		1.32
847.2		1.32
859.8		1.32
872.1		1.32
884.2		1.32

Settlement in inches 1.32





GROUP DELTA

LIQUEFACTION ANALYSIS (B-3)

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FIGURE F-1.2