Appendix D

GEOTECHNICAL DESKTOP STUDY

FINAL GEOTECHNICAL DESKTOP STUDY
SAN DIEGO RIVER TRAIL
QUALCOMM STADIUM SEGMENT
SAN DIEGO ASSOCIATION OF GOVERNMENTS

Submitted to:

QUALITY INFRASTRUCTURE CORPORATION
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Prepared By:

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October 19, 2015

AGE

Allied Geotechnical Engineers, Inc.

October 19, 2015

Kirk Bradbury, P.E.
Quality Infrastructure Corporation
7777 Alvarado Road, Suite 606
La Mesa, CA 91942

Subject: **FINAL GEOTECHNICAL DESKTOP STUDY**
SAN DIEGO RIVER TRAIL QUALCOMM STADIUM SEGMENT
SAN DIEGO ASSOCIATION OF GOVERNMENTS
AGE Project No. 54A6

Dear Mr. Bradbury:

In accordance with your request, we are pleased to submit this report which presents the findings, opinions and recommendations of a geotechnical desktop study that we have performed for the above-mentioned subject project. This Final Report incorporates our response to the review comments on our Report dated July 31, 2015 that we have received from Quality Infrastructure Corporation.

We greatly appreciate the opportunity to be of service on this important project for the San Diego Association of Governments. Should you have any questions or need further assistance, please feel free to give us a call.

Sincerely,

ALLIED GEOTECHNICAL ENGINEERS, INC.

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SAN DIEGO RIVER TRAIL QUALCOMM STADIUM SEGMENT
SAN DIEGO ASSOCIATION OF GOVERNMENTS

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS

Page No.

TABLE OF CONTENTS
(continued)

Page No.

Table

Figures

FINAL GEOTECHNICAL DESKTOP STUDY
SAN DIEGO RIVER TRAIL QUALCOMM STADIUM SEGMENT
SAN DIEGO ASSOCIATION OF GOVERNMENTS

1.0 SITE AND PROJECT DESCRIPTION

In accordance with the request of Quality Infrastructure Corporation (QIC), Allied Geotechnical Engineers, Inc. (AGE) has performed a Geotechnical Desktop study for the San Diego River Trail (SDRT) Qualcomm Stadium Segment for the San Diego Association of Governments (SANDAG). The study was performed in conformance with AGE's proposal dated April 10, 2015, and the subconsultant agreement entered into by and between QIC and AGE on June 12, 2015.

The findings and recommendations of this study were presented in a report dated July 31, 2015. At the request of QIC we have prepared this updated report which incorporates our response to the review comments from SANDAG which we received on October 15, 2015.

SANDAG proposes to construct an approximately 0.8-mile segment of the SDRT through Qualcomm Stadium in the Mission Valley community of the City of San Diego. The proposed Qualcomm Segment of the SDRT would extend eastward from the terminus of Fenton Parkway along a vegetated slope behind the Fenton Marketplace shopping center and through the southern portion of the Qualcomm Stadium parking lot to connect with Rancho Mission Road.

The proposed project is located in a developed area with residential and commercial uses to the west, north, and east. The San Diego River is located adjacent to the proposed trail on the south, with commercial office development on the south side of the river. Most of the trail would occur on existing paved surfaces within the stadium parking lot, which is topographically flat. The western end of the proposed trail would occur on a slope behind the shopping center that contains some dense vegetation. The proposed trail would be a constructed as a Class I bikeway, which is a path that provides a separated right-of-way for the exclusive use of people walking and riding bikes.

Existing improvements along and adjacent to the project alignments include landscaped area, the San Diego trolley line (both ground level and overhead tracks), Qualcomm Stadium Recycling Center, a soccer field, Qualcomm Stadium parking lot, Fenton Marketplace Shopping Center driveway behind the IKEA store, Qualcomm Trolley Station, and the 78-inch diameter North Mission Valley Interceptor Sewer (NMVIS).

The proposed pedestrian/ bike pathway will require the construction of retaining walls in the areas between the IKEA driveway and the trolley line fence, and through the low lying area at its eastern terminus. Construction of the retaining walls at the eastern terminus of the project alignment will require removal of existing asphaltic concrete (A.C.) pavement.

2.0 OBJECTIVE AND SCOPE OF STUDY

The objective of this desktop study is to provide general information and to evaluate potential major geologic and geotechnical issues and constraints which could impact the proposed project alignment. The study was performed in conformance with AGE's proposal dated April 10, 2015, and the subconsultant agreement entered into by and between QIC and AGE on June 12, 2015. The scope of the desktop study includes the performance of several tasks/services which are more fully described below.

2.1 Information Review

For this task, we have reviewed information pertaining to the project area that was readily available from a variety of sources which include the following:

- AGE's in-house references and aerial photographs;
- Published geologic literature and maps, including geologic and fault maps published by the City of San Diego, California Geological Survey and United States Geological Survey;
- Pertinent project-related information, including geotechnical reports prepared by others;
- Aerial photography available at Google Earth.

A listing of the references that were reviewed for this study is presented in Section 7.0.

2.2 Site Reconnaissance

The information obtained from our literature review was supplemented with visual observations gathered during our field reconnaissance visit that was conducted on July 14, 2015. The purpose of the site visit was to observe existing site conditions and geologic exposures along the project alignments and in surrounding areas.

2.3 Data Evaluation and Reporting

This task involved a synthesis and evaluation of the data collected during the information review and field reconnaissance phases of this study, particularly with respect to known and anticipated geotechnical conditions and potential geologic hazards, such as faulting and seismicity; seismic-induced hazards, slope stability issues, and landslides. Based on an evaluation of the data, we have prepared this report to present a summary of our preliminary findings and opinions.

3.0 GEOLOGIC CONDITIONS

3.1 Geologic Setting

The project alignment is located within the Peninsular Ranges geomorphic province, a north-south oriented mountain range which extends from the southern edge of the Los Angeles Basin into Baja California, Mexico. Basement rocks of the Peninsular Ranges province include Cretaceous crystalline rocks of the Southern California Batholith and Jurassic metasedimentary and metavolcanic rocks of the Santiago Peak Volcanics.

The project alignment is situated within the San Diego Embayment, a deep sedimentary-filled basin which is underlain at depth by the basement rock complex. The sedimentary formations consist of nearly flat-lying to gently southwest dipping, marine and non-marine sediments which range from Cretaceous to Holocene in age.

3.2 Tectonic Setting

Tectonically, the San Diego region is situated in a broad zone of northwest-trending, predominantly right-slip faults that span the width of the Peninsular Ranges and extend offshore into the California Continental Borderland Province west of California and northern Baja California. At the latitude of San Diego, this zone extends from the San Clemente fault zone, located approximately 50 miles to the west, to the San Andreas fault located about 90 miles to the east.

Major active regional faults of tectonic significance include the Coronado Bank, San Diego Trough, San Clemente, and Newport-Inglewood/Rose Canyon fault zones which are located offshore; the faults in Baja California, including the San Miguel-Vallecitos and Agua Blanca fault zones; and the faults located further to the east in Imperial Valley which include the Elsinore, San Jacinto and San Andreas fault zones.

3.3 Geologic Units

For site characterization purposes, the subsurface materials along the project alignment can be categorized into fill materials and Quaternary alluvium. Each geologic unit can be distinguished by its origin or depositional character and has different compositional characteristics. A generalized geologic map is shown on Figure 1.

3.3.1 Fill Materials

Based on a review of the cross-sections and logs of test borings prepared by San Diego Geotechnical Consultants, Inc. for the design of the NMVIS, it appears that the entire project alignment is underlain by man-made fill of variable thickness ranging from approximately 10 feet to over 20 feet (SDGC, 1988). Beginning in World War II, sand and gravel mining operations stripped away most of the natural deposits along the river, down to below the groundwater table. These areas have since been refilled for later development. The composition of the fill materials encountered in the SDGC borings and test borings performed for the Mission Valley West LRT Extension (1999) includes a wide range of materials from silt to gravels and cobbles. Documentation regarding the source and original placement of the fill materials is not available.

3.3.2 Quaternary Alluvium

The published geologic map (Kennedy and Tan, 2005) and aforementioned test borings indicate that the fill materials are underlain by Quaternary alluvial deposits. The alluvium consists of channel, floodplain, and estuary deposits laid down by the San Diego River. These deposits have been disturbed by mining in many places. Based on a review of the Mission Valley West LRT Extension test borings, the depth of the alluvium along the project alignment is estimated to be on the order of 50 feet below the ground surface (bgs) or greater.

3.4 Groundwater

The project study area is located in the Mission San Diego Hydrologic Subarea of the Lower San Diego Hydrologic Area of the San Diego Hydrologic Unit (San Diego Regional Water Quality Control Board (SDRWQCB,1995). Groundwater in this area has beneficial agricultural and industrial service and process supply, and has been exempted by the Regional Board for the municipal use designation under the terms and conditions of the State Board Resolution No. 88-63, “Sources of Drinking Water” Policy.

The San Diego Hydrologic Unit is a triangular shaped area of approximately 440 square miles that includes the San Diego River watershed (SDRWQCB, 1995). Major storage reservoirs within this unit include San Vicente, El Capitan, Jennings, Murray and Cuyamaca reservoirs. These facilities serve portions of the San Diego Metropolitan area and the communities of Santee, El Cajon, Lakeside, Alpine and Julian.

Given the proximity of the project alignment to the San Diego River, shallow to near-surface groundwater is anticipated along proposed project alignment. The SDGC borings reportedly encountered groundwater at an approximate depth varying from 9 to 15 feet bgs. These depths correspond to elevations of 38 feet to 42 feet msl. The project alignment is also located within the 100 year flood zone (EDR, 2015).

4.0 GEOLOGIC HAZARDS

Geologic hazards are those hazards that could impact a site due to local and regional geologic and seismic conditions. Our evaluation of the various geologic hazards and their potential impact on the project alignment are discussed in the following sections.

4.1 Local Faulting

San Diego County is located in a seismically active area, typical of the southern California region. The project alignment is likely to experience moderate to severe ground shaking in response to a local or more distant large magnitude earthquake occurring during the expected life span of the proposed project.

The nearest mapped potentially active fault to the project alignment is the La Nacion fault zone (LNFZ). The LNFZ is comprised of several en echelon faults within a generally north-south trending broad system of faults across the southern San Diego metropolitan area. The faults are generally dip-slip in nature with a down-to-the-west sense of separation. The main fault trace is mapped approximately 2 miles to the east of the project site (Kennedy and Tan, 1977; City of San Diego, 1995).

Geologic studies that have been performed on the LNFZ to date have not discovered any evidence for fault activity within Holocene time (11,000 years BP) (Dowlen, et.al, 1975; Hart, 1974). Based on the California Division of Mines and Geology fault classification criteria, the LNFZ may be considered “potentially active”, meaning that it has documented evidence of movement within Pleistocene time (the last 1.5 to 2 million years) but no movement in Holocene time.

The closest major active fault to the project site is the Rose Canyon fault zone (RCFZ), approximately 4 miles west of the project site. The RCFZ is a complex set of anastomosing and en-echelon, predominantly strike slip faults that extend from off the coast near Carlsbad to offshore south of downtown San Diego. Investigations of the RCFZ in the Rose Creek area (Rockwell et al, 1991) and in downtown San Diego (Patterson et al, 1986; Woodward-Clyde Consultants, 1994) found evidence of multiple Holocene earthquakes. Based on these studies, several fault strands within the RCFZ have been classified as active faults, and are included in Alquist-Priolo Special Studies Zones. A summary of the fault parameters is shown in Table 1 on the next page.

Table 1
Summary of Fault Parameters

	Rose Canyon fault zone (San Diego Section)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	6.432 km
Rjb*	6.432 km
Rx*	6.432 km
Fnorm*	0
Frev*	0

	Rose Canyon Fault Zone (Silver Strand Section - Downtown Graben Fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	6.822 km
Rjb*	6.822 km
Rx*	3.420 km
Fnorm*	0
Frev*	0

Table 1 (Continued)
Summary of Fault Parameters

	Rose Canyon fault zone (Silver Strand section-Spanish Bight fault)
Maximum Moment Magnitude	6.8
Fault Type	Strike-Slip (SS)
Fault Dip Angle	90 degree
Dip Direction	Vertical
Bottom of Rupture Plane	8 km
Top of Rupture Plane	0
Rrup*	7.201 km
Rjb*	7.201 km
Rx*	7.198 km
Fnorm*	0
Frev*	0

*** Definition of Terms in Table 1**

Rrup - Closest distance (km) to the fault rupture plane.
Rjb - Joyner-Boore distance: The shortest horizontal distance to the surface projection of the rupture area. Rjb is zero if the site is located within that area.
Rx - Horizontal distance to the fault trace or surface projection of the top of rupture plane. It is measured perpendicular to the fault (or the fictitious extension of the fault).
Fnorm - Fault normal
Frev - Fault reverse

The project alignment is subject to moderate to severe ground shaking in response to a major earthquake occurring on the RCFZ or on one of the major regional active faults. The closest active regional faults to the project alignment with recurring magnitude 4.0 and greater earthquakes are the Coronado Bank, the Vallecitos-San Miguel, and the Elsinore fault zones. Other more distant, active regional faults that are considered potential sources of seismic activity include the offshore located San Diego Trough and San Clemente fault zones and some of the faults in Imperial Valley which include the San Jacinto and San Andreas fault zones.

The location of the project alignment in relation to the active faults in the region is shown on the Regional Fault Map (Figure 2). The computer program EQFAULT (Blake, 2000, updated 2004) was used to approximate the distance of known faults to the project alignment. Seven (7) known active faults are identified within a search radius of 50 miles from the project alignment. A summary of seismic source characteristics for faults that present the most significant seismic hazard potential to the pipeline are presented in Table 2 below.

Table 2
Summary of Seismic Source Characteristics

Fault	Maximum Magnitude (Mw)	Peak Site Acceleration (g)	Closest Distance to Site (miles)
Rose Canyon	6.9	0.444	3.9
Coronado Bank	7.4	0.236	16.8
Newport-Inglewood (offshore)	6.9	0.080	31.5
Elsinore - Julian	7.7	0.077	37.0
Elsinore - Temecula	7.7	0.051	41.9
Earthquake Valley	6.5	0.039	42.2
Elsinore - Coyote Mountain	7.7	0.045	46.3

4.2 Historical Seismicity

EQSEARCH is a program that performs automated searches of a catalog of historical Southern California earthquakes. As the program searches the catalog, it computes and prints the epicentral distance from a selected site to each of the earthquakes within a specified radius (100 kilometers). From the computed distance, the program also estimates (using an appropriate attenuation relation) the peak horizontal ground acceleration that may have occurred at the site due to each earthquake. Based on the blow counts which are shown on the logs of borings which were performed for the NMVIS, a V_{S30} of 250 m/s was estimated for the project alignment

Based on the estimated shear wave velocities and our visual observations of the on-site geologic units, site Class D attenuation was used for all of our analysis. We used a combined earthquake catalog for magnitude 5.0 or larger events which occurred within 100 kilometers of the project alignment between 1800 and December 1999. The earthquake catalog for events prior to about 1933 is limited to the higher magnitude events.

The search results indicate that the nearest earthquake of magnitude 5.0 occurred on May 25, 1803 about 2.1 miles from the project alignment on an unmapped fault in the Allied Gardens area of San Diego. The seismic event resulted in a calculated ground acceleration of 0.227g. The largest site acceleration generated from this search is 0.256 g which was the result of a 6.5 magnitude earthquake which occured on November 22, 1800 on a strand of the Rose Canyon Fault Zone (Del Mar Section). The largest magnitude earthquake reported was a magnitude 7.0 event in 1858, located 78.9 miles of the project alignment on a strand of the Fontana Fault in the Riverside area of California which resulted in a calculated ground acceleration of 0.021 g.

It is our opinion that the major seismic hazard affecting the project alignment would be seismic-induced ground shaking. The alignment will likely be subject to moderate to severe ground shaking in response to a local or more distant large magnitude earthquake occurring during the life of the proposed project. For project design purposes, we recommend that the RCFZ be considered as the dominant seismic source.

4.3 Seismic Design Parameters

For structural design in accordance with the 2010 ASCE 7 procedures, the United States Geological Survey Design Maps (USGS, 2013) were used to calculate ground motion parameters for the project alignment. The Risk-Targeted Maximum Considered Earthquake (MCE_R) ground motion response acceleration is calculated based on the most severe earthquake effects considered by ASCE 7-10 determined for the orientation that resulted in the largest maximum response to the horizontal ground motions and with adjustment to the targeted risk. The Maximum Considered Earthquake Geometric Mean (MCE_G) is determined for the geometric peak ground acceleration and without adjustment for the targeted risk. The MCE_G Peak Ground Acceleration (PGA) adjusted for site effects (PGA_M) should be used for design and evaluation of liquefaction, lateral spreading, seismic settlements, and other soil related issues.

The calculated seismic design parameters are presented in Table 3 on the next page. The design criteria are based on the soil profile type as determined by existing subsurface geologic conditions, on the proximity of the site to a nearby fault and on the maximum moment magnitude and slip rate of the nearby fault. The Design Response Spectrum and Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum are shown on Figures 3 and 4, respectively.

Table 3
Summary of Seismic Design Parameters

REFERENCE	PARAMETER
Table 20.3-1 Site Classification	Site Class = D
Figure 22-1	Ss = 1.038 g
Table 11.4-1 Site Coefficient Fa	Fa = 1.085
Figure 22-2	S_1 = 0.398 g
Table 11.4-2 Site Coefficient Fv	Fv = 1.604
Equation 11.4-1	S_{MS} = 1.126 g
Equation 11.4-2	S_{M1} = 0.638 g
Equation 11.4-3	S_{DS} = 0.751 g
Equation 11.4-5	S_{D1} = 0.426 g
Figure 22-12	T_L = 8 seconds
Figure 22-7	PGA = 0.436 g
Equation 11.8-1	PGA_M = 0.464 g
Figure 22-17	C_{RS} = 0.899
Figure 22-18	C_{R1} = 0.958

Figure 22-1	Ss Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-2	S1Risk-Targeted Maximum Considered Earthquake (MCER) Ground Motion Parameter for the Conterminous United States for 1.0 s Spectral Response Acceleration (5% of Critical Damping), Site Class B.
Figure 22-12	Mapped Long-Period Transition Period, TL (s), for the Conterminous United States.
Figure 22-7	Maximum Considered Earthquake Geometric Mean (MCEG) PGA, %g, Site Class B for the Conterminous United States.
Figure 22-17	Mapped Risk Coefficient at 0.2 s Spectral Response Period, CRS.
Figure 22-18	Mapped Risk Coefficient at 1.0 s Spectral Response Period, CR1.

Based on the calculated S_{DS} of 0.751 g and S_{D1} of 0.426 g, a Seismic Design Category of "D" may be used for design of facilities with risk categories I, II and III.

4.4 Liquefaction

Seismic-induced soil liquefaction is a phenomenon during which loose, saturated granular materials undergo matrix rearrangement, develop high pore water pressure, and lose shear strength due to cyclic ground vibrations induced by earthquakes. Manifestations of soil liquefaction can include loss of bearing capacity below foundations, surface settlements and tilting in level ground, and instabilities in areas of sloping ground. Soil liquefaction can also result in increased lateral and uplift pressures on buried structures.

Based on a review of the logs of the borings performed for the NMVIS, it appears that the fill materials have low to moderate liquefaction potential whereas the alluvial deposits have low to high potential for liquefaction. Liquefaction phenomena along the project alignment most likely would manifest itself as local ground subsidence and settlement.

4.5 Landslides

A review of the published geologic maps indicates that the project site is not located on or below any known (mapped) ancient landslides. Furthermore, a review of the State of California Seismic Hazard Zones (2009) and City of San Diego Seismic Safety Study Geologic Hazards and Faults map (1995) indicate that the site is not located in an area that is susceptible to landslide hazards.

4.6 Lateral Spread Displacement

Due to the relatively level ground surface elevation and the absence of nearby mapped faults, the project alignment is not considered susceptible to seismic-induced lateral spreading.

4.7 Differential Seismic-Induced Settlement

Differential seismic settlement occurs when seismic shaking causes one type of soil to settle more than another type. It may also occur within a soil deposit with largely homogenous properties if the seismic shaking is uneven due to variable geometry or thickness of the soil deposit.

The project alignment is generally underlain by fill materials of variable thickness, and alluvium to a depth of approximately 50 feet bgs. There may be areas with low to moderate potential of differential settlement along the project alignment where sharp transitions between fill thickness and consistency of the alluvium occur.

4.8 Ground Lurching

Ground lurching is a permanent displacement or shift of the ground in response to seismic shaking. Ground lurching occurs in areas with high topographic relief, and usually occurs near the source of an earthquake. These displacements can result in permanent cracks in the ground surface. Considering the absence of nearby active faults, it is our opinion that ground lurching does not represent a potential hazard along the proposed project alignment.

4.9 Expansive Soils

Based on a review of the laboratory test results performed for the NMVIS, the onsite fill materials are considered to possess low expansion potential.

4.10 Compressible Soils

Based on a review of the logs of borings performed for the NMVIS and the Mission Valley West LRT Extension, it appears that the fill materials and alluvial deposits are not compressible.

4.11 Secondary Hazards

Given the elevation of the project alignment and the absence of nearby large bodies of water, the risk of damage resulting from seismic-induced seiches or tsunamis is considered negligible. In the event of failure or spillage of one of the upstream dams, the site will be subject to damage due to flooding and erosion.

The project alignment is located within the 100-year flood plain. Seasonal flooding caused by overflows of Murphy Canyon Creek and/or San Diego River from heavy rainfall events should be anticipated.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Earthwork

5.1.1 General Requirements

The earthwork operations for the project should be performed in accordance with the approved plans and specifications for the project, the applicable provisions of the City of San Diego Grading Ordinance, and Section 300 of the latest edition of Standard Specifications for Public Works Construction (SSPWC, known as the "Green Book").

5.1.2 Soil Excavation Characteristics

Based on our experience with similar geologic units, we anticipate that excavations in the on-site soil materials can be easily accomplished using conventional heavy-duty excavation equipment.

5.1.3 Fill Materials

The soil materials generated from excavations at the project site are considered suitable for use and placement as structural fill in the proposed building areas. Fill materials should be free of biodegradable materials, hazardous substance contamination, or other deleterious debris. If the fill materials contain rocks or hard lumps, at least 70 percent (by weight) of its particles shall pass a U.S. Standard $^3/_4$-inch sieve. Fill materials should consist of predominantly granular soil (less than 50 percent passing the U.S. Standard #200 sieve) with Expansion Index of less than 30. Biodegradable materials, hazardous substance contamination, or other deleterious debris will need to be segregated, stockpiled and removed from the project site.

5.1.4 Fill Placement and Compaction

Prior to placement of fill materials, the firm competent ground which is determined to be satisfactory for the support of filled ground shall be plowed or scarified to a depth of at least 6 inches until the surface is free from ruts, hummocks, or other uneven features which would tend to prevent uniform compaction by the equipment to be used.

The fill materials should then be moisture-conditioned, placed and uniformly compacted in layers until final elevations are reached. Each layer should be no thicker than will allow for adequate bonding and compaction, but shall not exceed 8 inches in loose (uncompacted) thickness. Unless otherwise specified, all fills shall be compacted to at least 90 percent of maximum dry density as determined in the laboratory by the ASTM D1557 test method. Fill placed within 24 inches below the bottom of the bikeway pavement section should be compacted to at least 95 percent of maximum dry density. Field density testing shall be performed in accordance with either the Sand Cone Method (ASTM D1556) or the Nuclear Gauge Method (ASTM D2922 and D3017).

5.2 Retaining Walls

Based on the information provided by QIC, it is our understanding that the proposed retaining walls will be designed using the Caltrans Standard Plans (2010 Edition) for Type 1 and Type 5 retaining walls. The wall heights have not been determined at the time of the preparation of this report. However, based on the information provided by QIC, it is anticipated that maximum height of the proposed retaining wall is on the order of 8 feet.

We recommend that the wall foundations be supported on properly compacted filled ground. Where applicable, it is recommended that a setback of at least 6 feet be observed for retaining wall foundations from the top of any slope. Where walls are closer than 6 feet from the top of a slope, it is recommended that the foundations in those areas be deepened such that the exterior face of the footing at its bottom level is at least 6 feet away from the face/surface of the slope at the same level. No reduction in friction and passive pressure is required for retaining walls designed as above.

Gross nominal bearing resistance of 6 ksf for footings up to 6 feet wide, 8 ksf for footings between 6 feet and 10 feet wide, 10 ksf for footings up to 16 feet wide (or mat foundation) may be used for the design of the retaining wall foundations. The permissible net contact stress is estimated to be on the order of 8 ksf for one inch settlement and an assumed soil angle of friction of 30°.

An active soil pressure equivalent to that generated by a fluid weighing 32 and 48 pounds per cubic foot, for level and 2:1 (horizontal : vertical) sloped backfill, respectively, may be used for design of these walls assuming that they are free to rotate at the top at least 0.001H (where H is the height of the wall). An at-rest soil pressure equivalent to that generated by a fluid weighing 60 pounds per cubic foot may be used for design of restrained walls.

For seismic loading, an inverted triangular pressure distribution of 5 pcf (equivalent fluid pressure, Level II seismic event) may be used in addition to the static earth pressures. This seismic earth pressures may be assumed to act at 0.6H from the bottom of the wall and are applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and/or walls retaining less than 5 feet of backfill materials, and may be ignored in estimating the seismic lateral earth pressure.

Lateral loads may be resisted by a passive pressure equivalent to that generated by a fluid weighing 250 pounds per cubic foot, and maximum passive resistance is limited to 2,500 pounds per square foot. A coefficient of friction of 0.35 between concrete and soil may be used to calculate the resistance to sliding provided that the concrete is placed directly against the soil. Passive pressure and friction may be used in combination, without reduction, in determining the total resistance to lateral loads.

The above recommendations assume properly drained granular backfill with no surcharge. Surcharge and foundation loads occurring within a horizontal distance equal to the wall height should be added to the lateral pressures.

All retaining walls should be properly waterproofed and provided with a gravel and perforated pipe drain system to reduce the potential for hydrostatic pressure build-up behind the walls. The discharge from the wall subdrain should be directed to flow into a nearby located on-site drainage facility. As an alternative to a pipe and gravel drain system, a pre-manufactured drainage product such as "Mirafi G100N" or equivalent, may be used.

We recommend that the wall be backfilled with soil materials which have less than 40 percent passing the standard #200 sieve and not less than 70 percent passing the U.S. standard 3/4-inch sieve, and expansion index of less than 20 and sand equivalent of at least 30. In addition, the backfill materials should not contain any organic debris, rocks or hard lumps greater than 6 inches, or other deleterious materials. All wall backfill soils should be compacted to at least 90 percent of maximum dry density as determined in the laboratory by the ASTM D1557 testing procedures. Small, hand-operated compacting equipment should be used for compaction of wall backfill soils. If large equipment is used, additional lateral pressures due to equipment weight should be considered in the design of the wall.

5.3 Fill Slope

Construction of the elevated portion of the bikeway at the eastern terminus and along the slope between IKEA and the trolley line at the western terminus of the project alignment may require the construction of fill slopes. The new fill slopes are anticipated to be less than 6 feet in height and have a slope gradient of 2 : 1 (horizontal : vertical) or gentler.

Provided that the new fill slopes are constructed with structural fill which meet the criteria presented in Section 5.1.3, the new fill slopes should have a factor of safety greater than 1.5 against deep-seated and shallow failures under static loading and a factor of safety greater than 1.1 under pseudo-static loading. All slopes should be planted, drained and maintained to reduce erosion. It is recommended that the frequency and amount of irrigation be kept to a minimum to support the vegetation cover without overwatering. Surface drainage should not be allowed to flow directly over the surface of the slopes.

Where fills are to be placed on hillsides or slopes with gradients greater than 10 percent, horizontal benches shall be cut into firm undisturbed natural ground in order to provide both lateral and vertical stability. This is to provide a horizontal base so that each layer is placed and compacted on a horizontal plane. The initial bench at the toe of the fill shall be at least 10 feet in width and 2 feet in depth on firm undisturbed natural ground at the elevation of the toe stake placed by the surveyor. The Engineer shall determine the width and frequency of all succeeding benches which will vary with the soil conditions and the steepness of the slope.

5.4 Temporary Excavation

Excavation and safety during construction are the sole responsibility of the contractor. Excavations should be performed in accordance with applicable Local, State, and prevailing Federal and Cal OSHA safety regulations to prevent excessive ground movement and failure.

Unsupported temporary excavations along the project alignment may be constructed at an inclination no steeper than 1.5 : 1 (horizontal to vertical), or flatter, up to a maximum height of 15 feet. Temporary construction slopes are considered to have a factor of safety against deep-seated failure in excess of 1.2 under static conditions.

Observations will need to be performed during site grading to check that no adverse conditions, geologic features or discontinuities are exposed in the excavation which may necessitate shoring or tie-backs. The contractor should exercise caution and provide adequate safety measures during excavations to protect equipment and/or personnel working directly below any excavation. Adequate safety measures include, but are not limited to, providing proper drainage control above and below the excavation, and elimination of any surcharge within a lateral distance equal to the height of the excavations.

5.5 Temporary Shoring

Prevailing Federal and Cal OSHA safety regulations require that any excavations 4 feet or deeper be either sloped (if sufficient construction space or easement is available), shored, braced, or protected with an approved sliding trench shield.

The safety of the excavation and the design and construction of the shoring system shall be the sole responsibility of the contractor. Prior to commencement of the excavation work, it is recommended that all existing structures within a horizontal distance that is at least two times the depth of the excavation be inspected to document their existing conditions. Documentation should include photographs of the existing building and site conditions as well as field surveys of the building floors and exterior pavements.

During the course of the construction, deflection of the shoring system should be regularly monitored. In addition, the structures should be periodically inspected for signs of distress. Continued field surveys should be performed to check and document any movement of slabs and pavements resulting from the construction activities. In the event that distress or settlement is noted, an investigation should be performed and appropriate corrective measures should be taken to protect the structures from any further distress/movement.

5.5.1 Settlement

Settlement of existing street improvements and/or utilties adjacent to the shoring may occur in proportion to both the distance between shoring system and adjacent structures or utilities and the amount of horizontal deflection of the shoring system. Vertical settlement will be maximum directly adjacent to the shoring system, and decreases as the distance from the shoring increases. At a distance equal to the height of the shoring, settlement is expected to be negligible. Maximum vertical settlement is estimated to be on the order of 75 percent of the horizontal deflection of the shoring system. It is recommended that shoring be designed to limit the maximum horizontal deflection to ½-inch or less where structures or utilities are to be supported.

5.5.2 Lateral Earth Pressures

Temporary shoring should be designed to resist the pressure exerted by the retained soils and any additional lateral forces due to loads placed near the top of the excavation. For design of braced shorings supporting fill materials and topsoil the recommended lateral earth pressure should be 30H psf, where H is equal to the height of the retained earth in feet. For braced shoring supporting bedrock, the recommended lateral earth pressures may be reduced to 20H psf. Any surcharge loads would impose uniform lateral pressure of 0.3q, where "q" equals the uniform surcharge pressure. The surcharge pressure should be applied starting at a depth equal to the distance of the surcharge load from the top of the excavation.

The recommended lateral earth pressures have been developed based on several assumptions, as follows: the shored earth is level at the surface; there are no hydrostatic pressures above the bottom of the excavation; and the shoring system is temporary in nature. In the event that the water table rises above the bottom of the excavation, it is recommended that a hydrostatic pressure equal to $62.4h_w$ psf, where "h_w" is equal to the height of groundwater above the bottom of the excavation, be added to the lateral pressure.

5.5.3 Lateral Bearing Capacity

Resistance to lateral loads will be provided by passive soil resistance. The same allowable passive resistance presented in Section 5.2 may also be used to calculate lateral bearing capacity for design of temporary shorings.

5.6 Dewatering

Proposed excavations for the subject project are anticipated to be less than 5 feet in depth and under normal conditions not expected to extend below the groundwater table. We therefore do not anticipate the need for dewatering of foundation and trenched excavations made during construction. The contractor should, however, anticipate the possible need for sump pumps in the event that localized perched water conditions are encountered during construction. The design, installation, and operation of any construction dewatering measures necessary for the project shall be the sole responsibility of the contractor.

It must be noted that the project alignment is located adjacent to the San Diego River and subject to seasonal flooding and high groundwater level. In the event that project construction is performed during a wet season, the use of shallow well points in combination with sump pump may be require to keep the project excavations dry.

5.7 North Mission Valley Interceptor Sewer

The retaining walls that will be constructed on the slope between IKEA and the trolley line at the western terminus of the project alignment may be located over the 78-inch diameter City of San Diego North Mission Valley Interceptor Sewer. Considering the dimension and age of the sewer pipeline, we recommend that an analysis be performed to evaluate the loading impact from the retaining walls' foundation on the existing sewer pipeline.

5.8 Additional Geotechnical Studies

The findings of this desktop study are based on a cursory evaluation of readily available information which is generally very limited and contain data gaps in many areas. We therefore recommend that additional geotechnical studies be performed for final design of the proposed project. Subsurface field exploration for the subject project should at a minimum include the performance of soil borings or test pits in the area of the proposed retaining walls.

6.0 LIMITATIONS

The information presented in this report is intended for the sole use of QIC and the SANDAG in their planning and design of the subject project. Our firm did not perform an investigation to evaluate the subsurface conditions along the project alignment. This report is based on a review and evaluation of readily available information, various assumptions to bridge over data gaps, and our previous experience in the general project study area.

This study was performed in accordance with the authorized scope of work for this project. The findings and professional opinions presented in this report were developed in general conformance with the current practices and standard of care exercised by local geotechnical engineering consultants performing similar tasks at the present time. No other warranty, either expressed or implied, is made with regard to the findings and professional opinions presented in this report.

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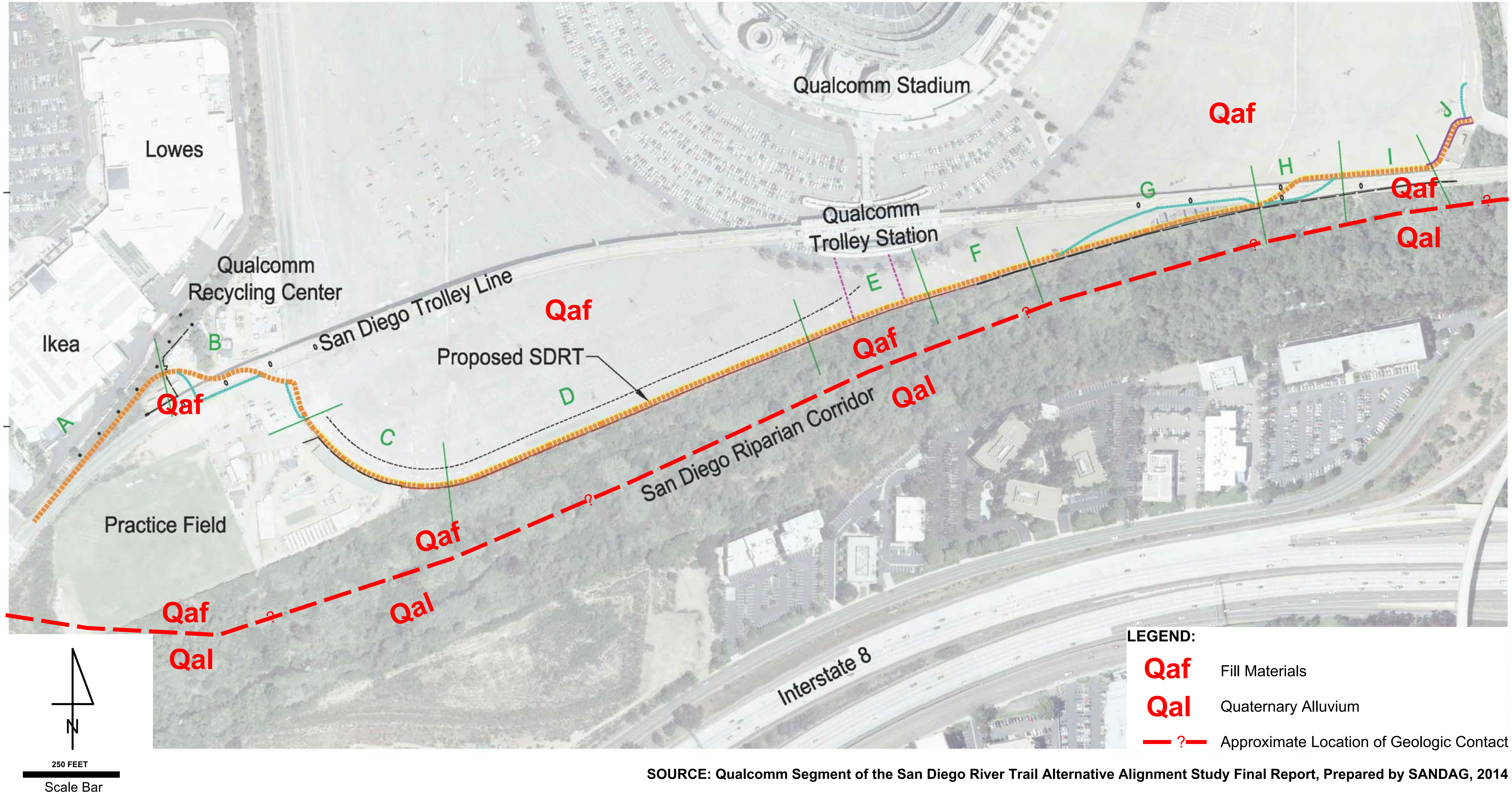
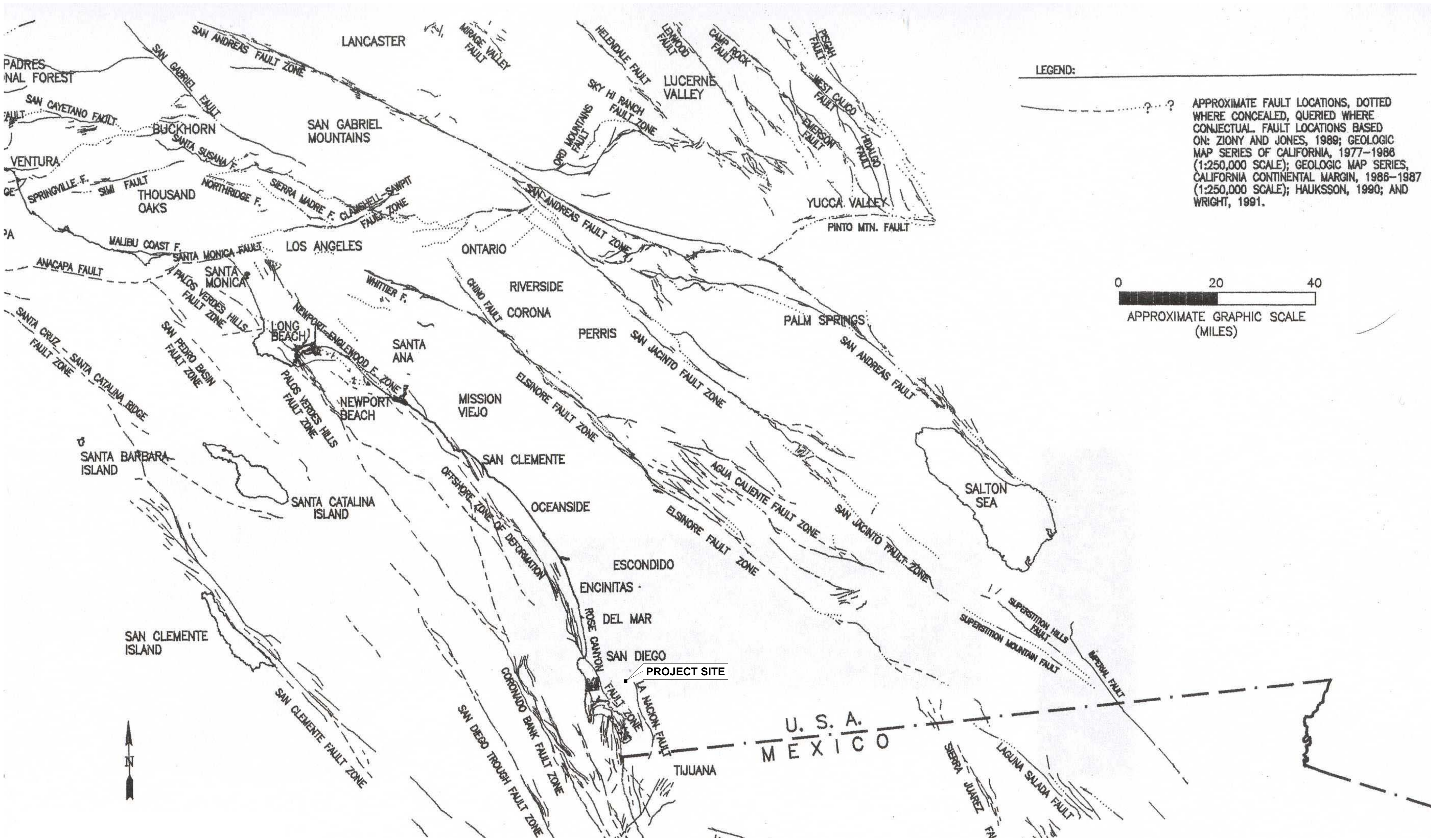


SOURCE: Qualcomm Segment of the San Diego River Trail Alternative Alignment Study Final Report, Prepared by SANDAG, 2014

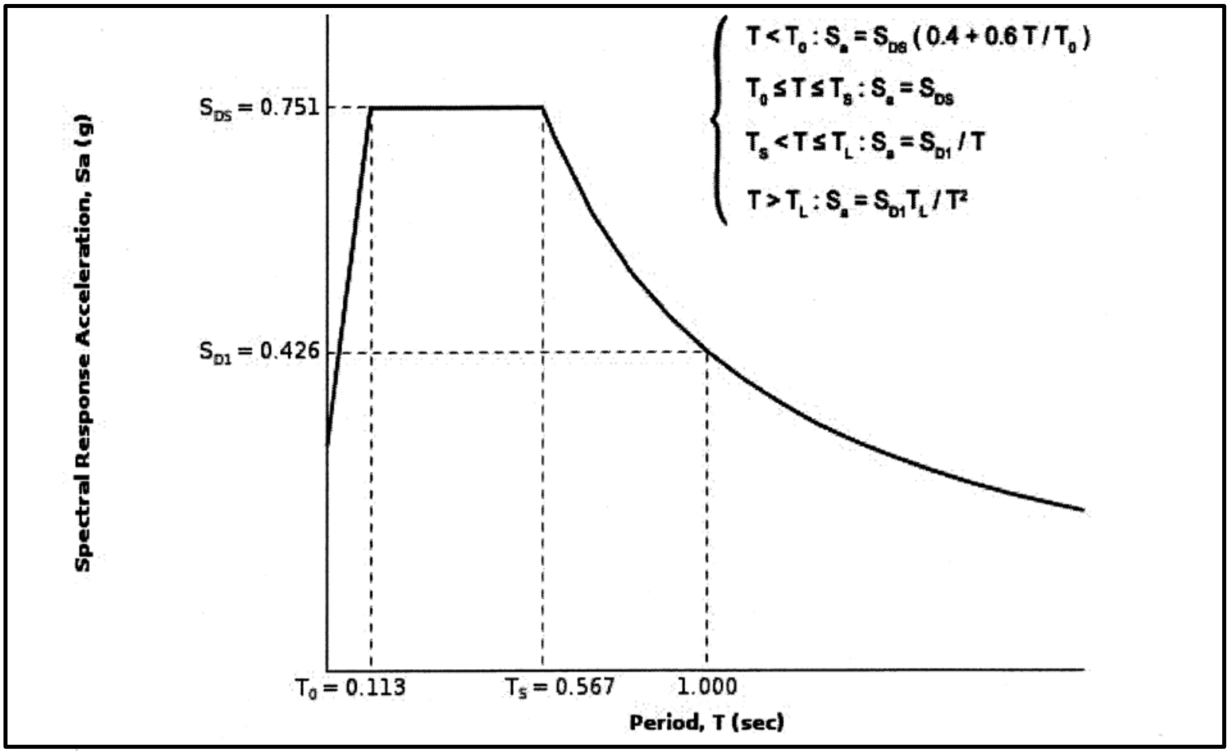
SAN DIEGO RIVER TRAIL QUALCOMM SEGMENT	ALIGNMENT/GENERALIZED GEOLOGIC MAP

PROJECT NO. 54A6	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 1



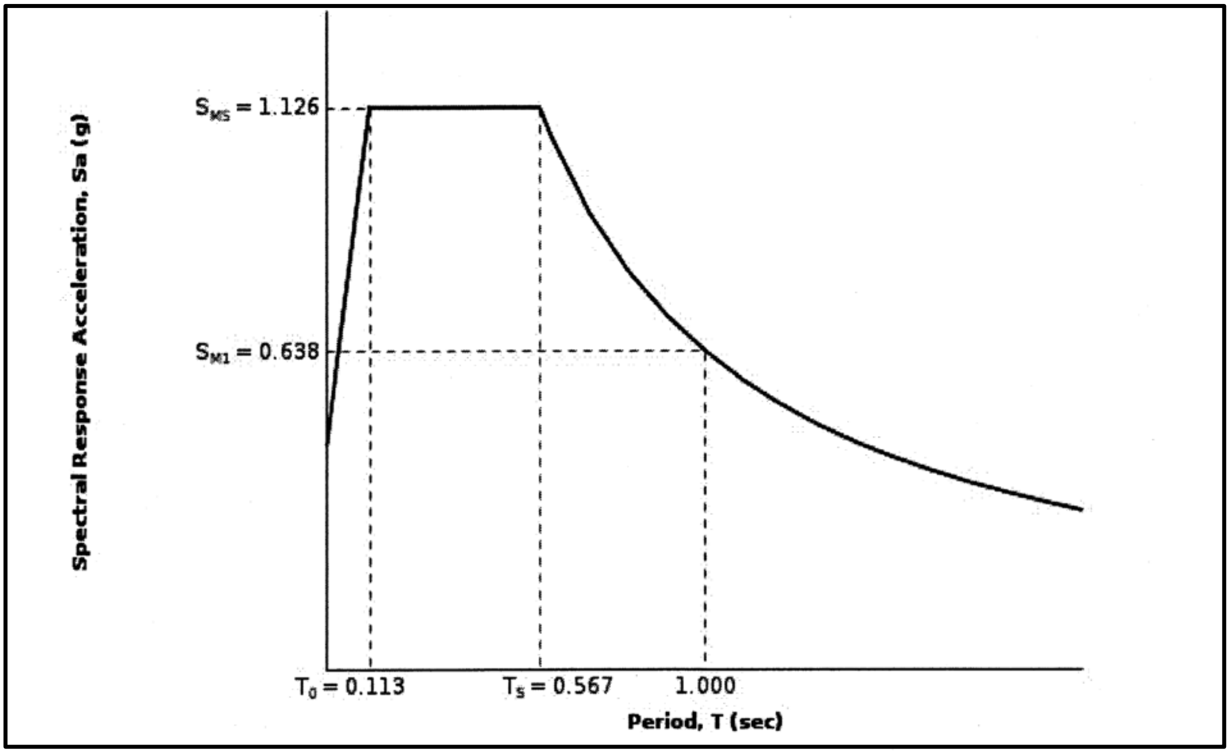
SAN DIEGO RIVER TRAIL QUALCOMM SEGMENT	REGIONAL FAULT MAP

PROJECT NO. 54A6	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 2



DESIGN RESPONSE SPECTRUM
SAN DIEGO RIVER TRAIL QUALCOMM SEGMENT

PROJECT NO. 54A6	**ALLIED GEOTECHNICAL ENGINEERS, INC.**	**FIGURE 3**



RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R)
SAN DIEGO RIVER TRAIL QUALCOMM SEGMENT

PROJECT NO. 54A6	ALLIED GEOTECHNICAL ENGINEERS, INC.	FIGURE 4