FOR CONTRACT NO.: 11-2T0951
EFIS No. 1100000454

INFORMATION HANDOUT

MATERIALS INFORMATION

WATER SOURCE INFORMATION

GEOTECH REPORT_FINAL_DATED, APRIL 2, 2010

PAVEMENT THICKNESS HILLERY DRIVE, DATED 24, 2010

RIGID PAVEMENT BASE DESIGN PPB 09-01, DATED SEPT 8, 2009

REVISED FOUNDATION REPORT FOR HILLERY DR. OC AND DAR, DATED SEPT. 1, 2010.2ND FOUNDATION

REPORT FOR TYPE 1 RETAINING WALLS, DATED MAY 24, 2010

ROUTE: 11-SD-15-M15.0/M16.4 (PM)

THE CITY OF SAN DIEGO

July 12, 2010

Duy N. Hoang
Caltrans, District 11 – MS 333
4050 Taylor Street
San Diego, CA 92110

Dear Mr. Hoang,

Subject: Caltrans Water Availability Letter – I-15, Hillery Drive & Carroll Canyon Road to Mira Mesa Boulevard, 11-SD-15, PM M15.0/M16.4, 11-269-2T0951, Construct Direct Access Ramp

The project is located within the City of San Diego water service area. A City water main is located in the fronting City Street (Hillery Drive, City Fire Hydrant 300' west of Westview Parkway). Note: the City of San Diego is at this time at a 'Level 2 Drought Alert Condition'. San Diego's water customers are encouraged to use water efficiently and not to waste it or use it unreasonably.

Water and water service connections are available. These connections are requested per required demand. All services are governed by City ordinances and regulations concerning connections, construction, capacity charges, permit fees and matters pertaining thereto. The requested demand of this project is approximately 1.2 million gallons of water for the duration of the project (36 months). The City requires the connection to be at the above stated fire hydrant, or an alternate hydrant in the immediate area, subject to approval by the City, through a fire hydrant meter (max. 124 GPM) or a City temporary construction meter (max. 1000 GPM). The service connection temporary construction meter or fire hydrant meter shall be installed by the City.

If further information is required on the service connection, please contact Rudy Benitez at (619) 533-5146.

Sincerely,

Paul J. Buehler, PE

RB

cc: Bobbi Salvini, Senior Civil Engineer, Public Utilities Department

RI-15 Hillery Drive CalTrans Avbl.tr 070710.doc

GEOTECHNICAL DESIGN REPORT

Mira Mesa/Scripps Ranch DAR Project Located in San Diego, on I-15 Between Carroll Canyon Rd and Mira Mesa Blvd (PM R15.2/M16.0).

11-SD-15-R15.2/M16.0

EA 11-2T0951

April 2, 2010

Prepared By:

OFFICE OF GEOTECHNICAL DESIGN-SOUTH 2
7177 OPPORTUNITY ROAD
SAN DIEGO, CA 92111

State of California | Business, Transportation and Housing Agency

Memorandum

To: Mr. Gerard Chadergian
District 11
I-15 Corridor Design Manager

Date: April 2, 2010

File: 11-SD-15
EA 11-2T0951
PM R15.2/M16.0

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2

Subject: Geotechnical Design Report for the I-15 Mira Mesa/Scripps Ranch DAR Project.

Pursuant to your request, the Office of Geotechnical Design-South 2 (OGD-S2) has prepared this Geotechnical Design Report for the roadway section of the 15 Mira Mesa/Scripps Ranch DAR project. This report defines the geotechnical conditions as evaluated from field data and used in the development of the geotechnical design. It provides recommendations and specifications for project design and construction.

OGD-S2 staff will be available for further assistance. Should you have any questions or comments regarding this report, please contact Jeff Tesar at (858) 467-2716 or (858) 945-0458.

J. Tesar

Jeff Tesar, C.E.G.
Associate Engineering Geologist
Office of Geotechnical Design - South 2

REGISTERED GEOLOGIST
JEFF J. TESAR
No. 2137
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

cc:	District Project Manager:	Andrew Rice
	Geotechnical Services Corporate:	Mark_Willian@dot.ca.gov
	District Construction R.E. Pending File:	Gerard Chadergian
	District Materials Engineer:	Art Padilla
	Office Chief, OGD-S2:	Abbas Abghari
	Senior, OGD-S2:	Brian Hinman BH
	OGD-S2 File Room:	

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1.0 INTRODUCTION

This report has been prepared by the Office of Geotechnical Design-South 2 (OGD-S2) to address the geotechnical design considerations for five retaining structures to be constructed adjacent to Interstate 15 on Hillery Drive, in the City of San Diego, California, hereafter referred to as the project.

The geotechnical investigation included: site reconnaissance, research of archived resources, data analysis, and the writing of this report. The project location is depicted in Figure 1.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the roadway portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of changed site conditions.

This report is intended for use by the project design engineer, construction personnel, bidders and contractors.

2.0 EXISTING FACILITIES AND PROPOSED IMPROVEMENT

Currently, the project area stretches between the terminus of Hillery Drive and I-15. It consists of a level asphalt-paved street (Hillery Drive) and a level partially paved access road for the apartment complex. To the south, the site is bounded by a two-stories high commercial structure (San Diego Community College District Distribution Center) and a large level lot where the future Rapid Transit Center is planned to be located. To the north, the project site is bordered by the Park and Ride parking lot and several apartment structures that are three-stories high. To the east, the site is bounded by the I-15 freeway.

A Direct Access Ramp (DAR) system is proposed to connect the I-15 Managed Lanes infrastructure with the Bus Rapid Transit facility in the Mira Mesa and Scripps Ranch communities. In addition, this improvement would provide direct vehicular access from local streets to and from the I-15 Managed Lanes.

The DAR system will consist of an elevated access ramp structure that will be accessed from the Hillery Drive and Westview Parkway intersection. The ramp will ascend eastward along Hillery Drive and connect to a bridge structure to be built over the I-15 freeway. The project Site Plan is depicted in Figures 2 and 3.

Five retaining structures are proposed for this project. Table 1 lists the proposed walls and indicates their station limits, types, and maximum heights. The information contained in Table 1 was provided by the AECOM Transportation Consultant on May 26, 2010. The locations of retaining walls are shown in Figure 4.

- Retaining Wall RW24L will retain the northern flank of the ascending Hillery Drive DAR, and Retaining Wall RW24R will retain the southern flank. Both walls are planned to be 13.87 feet in maximum height. At Station 25+68.95 “HD Line” both retaining walls will connect to the planned bridge structure.
- Retaining Wall RW23L, about 5.33 feet in maximum height, is planned to accommodate the grade separation between Hillery Drive and the current Park and Ride parking lot.

- Retaining Wall RW25R, about 6.67 feet in maximum height, is planned to accommodate the grade separation between Hillery Drive and the lot that is occupied by the San Diego Community College District Distribution Center.
- Retaining Wall RW23R, 5.5 feet in maximum height, is planned to accommodate the grade separation between the future Transit Center lot and the access road to the distribution center.

3.0 PERTINENT REPORTS AND INVESTIGATIONS

Relevant documents reviewed for this report are listed below.

- Tesar, J., Geotechnical Design Report, Interstate 15, Proposed Retaining and Sound Walls, Foundation Recommendations: California Department of Transportation, Division of Engineering Services, March 28, 2002.
- Tesar, J., Interstate 15 (I-15), Managed Lanes Project – Southern Segment, Proposed Retaining Walls: Preliminary Geotechnical Feasibility Evaluation: California Department of Transportation, Division of Engineering Services, January 31, 2005
- Kleinfelder, Preliminary Foundation Report I-15 Managed Lanes, Mira Mesa Boulevard Undercrossing, San Diego County, California, 2008.
- Kleinfelder, Structures Foundation Report I-15 Managed Lanes, South Segment, Mira Mesa Boulevard Undercrossing, San Diego, California, 2007.
- Kennedy, M.P. Geology of the San Diego Metropolitan Area: California Geological Survey, Bulletin 200, 1975.
- Norris, R.M. and Webb, R.W., Geology of California, John Wiley & Sons, Inc., 1990.
- Mualchin, L. A Technical Report to Accompany the Caltrans California Seismic Hazard Map, 1996.
- Soil Survey, San Diego, California, US Department of Agriculture, 1973.
- State of California Department of Transportation, Standard Plans, May 2006.
- Ninyo & Moore, Geotechnical Reconnaissance Mira Mesa/Scripps Ranch Direct Access Ramp Hillery Drive and Galvin Avenue Alternatives, November 9, 2007.
- USGS, Poway Quadrangle, 7.5 Minutes Series Topographic Map, 1967.
- Aerial Photographs, USDA, Flight WAC-89CA, 1-21, January 30, 1989.

4.0 PHYSICAL SETTING

The following section describes the physical setting of the project including: climate; topography and drainage; man-made and natural features of engineering and construction significance; regional geology and seismicity; and soil survey mapping.

4.1 Climate

San Diego has a Mediterranean to semi-arid climate, which is characterized by warm, dry summers and mild winters with some rain. San Diego has mild, mostly dry weather with approximately two hundred (200) days above seventy degrees Farenheit (70°F). The extended summer and dry period lasts from May to October. Temperatures are mild to warm in the summer. The average high and low temperatures during the summer are seventy to seventy-eight degrees Farenheit (70-78°F) and fifty-five to sixty-six degrees Farenheit (55–66°F), respectively. Temperatures exceed ninety degrees Farenheit (90°F) approximately four days a year. Winter is the rainy period and lasts from November to April. Temperatures are mild and somewhat rainy during the winter. The average high and low temperatures during the winter are sixty-six to seventy degrees Farenheit (66–70°F) and fifty to fifty-six degrees Farenheit (50–56°F) respectively. There is approximately ten-inches (10in) of rainfall in San Diego annually. However precipitation may range from three to thirty-inches (3.0-30.0in) during any given year.

4.2 Topography & Drainage

The project is located in a well-developed urban area. The project site lays at the margin between a large terrace landform to the west and low hills to the east. Past grading activities for the structures and appurtenances that are at or adjacent to the location of this project combined with natural landforms have created a near level topography slightly inclined to the west. The location of this project is depicted on topographic map in Figure 5 and on aerial photo in Figure 6..

Natural drainage occurs mainly as a sheet flow flowing to the west. Storm water is conveyed through the existing storm drainage system westward to canyons and arroyos leading the Pacific Coast.

4.3 Man-made and Natural Features of Engineering and Construction Significance

No man-made or natural features that present an unusual engineering or construction challenge were identified during the course of this study.

4.4 Regional Geology and Seismicity

This section describes regional geology and seismicity related to the project location.

4.4.1 Regional Geology

The project site lies within the coastal plain section of the Peninsular Ranges Geomorphic Province of California. The coastal plain generally consists of subdued landforms including mesas underlain by Cenozoic sedimentary formations.

Two principal rock units generally underlie the project area: a Mesozoic igneous and metamorphic rock basement and superjacent late Cretaceous, Eocene, Pliocene, Pleistocene, and Holocene sedimentary succession of strata. The basement is composed of Upper Jurassic Santiago Peak Volcanics, a structurally complex, mildly metamorphosed unit composed of andesitic volcanic and volcanoclastic rocks and mid-Cretaceous granitic rocks of the Southern California Batholith. The post-batholith superjacent sedimentary succession of strata was deposited mainly in a Tertiary and Quaternary periods and includes Eocene Poway Group comprised of Stadium Conglomerate and Pleistocene Lindavista Formation. Alluvium soils were deposited during the Holocene epoch. In addition, artificial fill materials can be found in some areas of this project (Kennedy and Peterson, 1975).

4.4.2. Regional Seismicity

In the San Diego Region the interaction between the North American and Pacific tectonic plates is considered to take place across a wide area extending from the San Andreas Fault in the Imperial Valley to about 100 km offshore to the west. The main fault zones west of the San Andreas Fault include the active San Jacinto and Elsinore fault zones located to the northeast of the project site. Located west of the project site is the active Rose Canyon Fault zone and a complex system of offshore faults. These offshore faults include the Coronado Banks, San Diego Trough, and San Clemente faults. Faults that may produce seismicity with potential to impact the project site are shown in Figure 7.

4.5 Soil Survey Mapping

The Soil Survey of San Diego Area, California, prepared by the U.S. Department of Agriculture (USDA), Soil Conservation Service (1973) was utilized for this project. Although the survey focuses primarily on agricultural issues, the report includes estimated soil properties that are significant in engineering and land use planning.

The review of the Soil Survey report indicates that at the project location the majority of relatively level areas or mesas are classified as having soils characteristic of the Redding association. That association is comprised of well-drained cobbly and gravelly loams that have gravelly and cobbly clay subsoil over a surficial hardpan.

5.0 EXPLORATION

No subsurface field investigation or laboratory testing was conducted specifically for this report. This report was based on a review of archived data, field reconnaissance, and Logs of Test Borings (LOTB) developed by OGD-S2 Branch B staff tasked to perform the field investigation for the DAR Bridge and associated retaining walls. These LOTB were produced during the subsurface exploration program for this project and will be included in the Project Plans.

5.1 Drilling and Sampling

Recent drilling and sampling data utilized in this report was developed by OGD–S2 Branch B working on related project features. Archived drilling data was also utilized in the preparation of this report.

5.2 Geologic Mapping

In order to determine the geological setting of the project site, a field reconnaissance was conducted at the proposed location of the Mira Mesa/Scripps Ranch DAR project. In addition, a review of geologic maps and archived reports pertaining the project site was conducted. A broad scale geologic map encompassing the project area is included in Figure 8 (Kennedy and Peterson, 1975.)

5.3 Geophysical Studies

No geophysical studies were conducted for this report.

5.4 Instrumentation

No instrumentation was established and monitored for this report. However, for the exploration program conducted by the OGD-S2 Branch B, two piezometers were installed at the project location in Borings R-09-020 and R-09-021. These piezometers have been monitored for the groundwater surface elevation.

6.0 GEOTECHNICAL TESTING

No testing was performed specifically for this report. Applicable geotechnical parameters for the various geologic units present have been developed from preceding investigations and local experience. Soil strength parameters utilized in our analyses are presented in Table 2.

6.1 In Situ Testing

No in-situ testing was conducted specifically for this report.

6.2 Laboratory Testing

No laboratory testing was conducted for this report. However, during the exploration program conducted by OGD-S2 Branch B, corrosion testing was conducted on collected soil samples. This testing was performed by the Caltrans Laboratory and in accordance with California Test Methods 643, 417, and 422. Based on the laboratory testing results the soils are considered corrosive due to high levels of sulfates and chlorides. The laboratory corrosion test results may be available in the Mira Mesa/Scripps Ranch DAR foundation report prepared by OGD-S2 Branch B.

7.0 GEOTECHNICAL CONDITIONS

This section describes geotechnical conditions at the project location.

7.1 Site Geology

The project site is capped by a layer of surficial soils comprised of fill materials, topsoil, or (locally) alluvium. This surficial layer is underlain by sedimentary soils consisting of Pleistocene age Lindavista Formation and Eocene age Stadium Conglomerate. Basement rocks of the Santiago Peak Volcanics Formation likely underlie this sedimentary unit at depths far below any project foundation element.

7.1.1 Lithology

Fill

Asphalt pavement that is underlain by road base (sandy gravel).

Lindavista Formation

Alternating layers of cobble conglomerate and sandstone. The cobble conglomerate consists of cobbles ranging in size from three to four inches within gravelly and sandy matrix, often indurated. The sandstone is fine grained, thickly bedded, slightly weathered and poorly indurated.

Stadium Conglomerate

Cobble conglomerate consisting of cobbles and boulders within gravelly and sandy matrix, often indurated.

7.1.2 Structure

All three geologic units present at the project site are in contacts that are horizontal or near horizontal dipping at a very low angle to the west. In addition, no other significant structural features have been documented or were observed during the field reconnaissance for this project.

7.1.3 Natural Slope Stability

This project is located in an area of subdued natural and graded landforms with minimal elevation variance. No significant slopes exist in the project area.

7.2 Soil Conditions

A relatively thin layer of fill material caps most of the project site. This fill layer varies in depth from about one to three feet and may generally be described as sandy gravel. Minor zones and pockets of deeper feel could likely be encountered, especially at locations of underground utilities.

Sedimentary formations underlie the fill. The sedimentary strata consist of alternating layers of cobble conglomerate and sandstone. The conglomerate is massively bedded and consists of three to four-inch diameter cobbles in a gravelly, sandy matrix with an apparent density that is dense to very dense. The sandstone is thickly bedded, fine grained, slightly weathered, medium dense, and locally very dense. The recent boring records developed by OGD-S2 Branch B describe the conglomerate and sandstone as poorly indurated.

Observations made by OGD-S2 Branch D staff at excavations on adjacent projects reveal that the conglomerate in the project area is often highly indurated and contains large cobbles and small boulders. It is likely that significant zones of highly indurated and cemented soils underlay the planned wall locations.

7.3 Water Condition

This section describes surface and ground water conditions that could impact the design or construction of the project.

7.3.1 Surface Water

Bodies of surface water such as lakes, rivers, or streams do not exist on or adjacent to the project site.

7.3.1.1 Scour

Due to near level topography and absence of watercourses, no scour potential exists at the project location.

7.3.1.2 Erosion

The project is located in a developed urban area that exhibits a well-developed drainage/storm water system. Therefore, the potential for natural erosion at the location of this project is low.

7.3.2 Ground Water

Piezometers were installed in selected borings drilled for the Mira Mesa/Scripps Ranch DAR foundation report. These piezometers have been monitored for groundwater surface elevation. The piezometer data is presented in Table 3, and their locations are presented in Figure 3. Based on the piezometer data, groundwater exists at the project site. However, groundwater occurs at a depth of more that one hundred feet below the ground surface. Therefore, it is not expected that this groundwater would affect the foundations of the proposed retaining structures or the activities related to their construction. As of February 4, 2010, the groundwater surface elevation in Piezometer R-09-020 was found to be at an elevation of 426.44 feet, and the groundwater surface in Piezometer R-09-021 was sounded to be at an elevation of 420.11 feet. The piezometer data suggests that no groundwater exists within the Lindavista Formation (cobble conglomerates and sandstones.) However, our office will continue monitoring the piezometers. Any groundwater-related data that differs with the conclusion of this section of the report will be reported to your office.

Perched groundwater is unconfined groundwater that is trapped by an underlying layer or lens of impermeable soil or rock. No perched water was encountered in the upper 100 feet of soil during the exploration program conducted by OGD-S2 Branch B. However, based on conditions along the wall alignments, the potential for the occurrence of occasional perched water is estimated to be high. A perched water condition may be encountered at the interface of a permeable fill and the underlying Linda Vista Formation; a geologic unit that is locally impermeable.

7.4 Project Site Seismicity

This section describes seismic considerations that must be used for design of project features.

7.4.1 Ground Motions

No known Holocene (active) fault exists within the project area. The nearest known active fault is the Rose Canyon Fault Zone believed to be capable of producing an earthquake with a Maximum Credible Magnitude of 7.0 on the Richter scale. The Rose Canyon Fault Zone is located about 10 miles west from the project site. The Palos Verdes fault zone located approximately 22 miles southwest of the project is capable of producing an earthquake with a Maximum Credible Magnitude of 7.0 on the Richter scale. The Whittier Elsinore Fault laying about 28 miles northeast from the project is capable of producing an earthquake with a Maximum Credible Magnitude of 7.5 on the Richter scale. In addition, the potentially active La Nacion Fault located about 12 miles southwest of the project is capable of producing an earthquake with a Maximum Credible Magnitude of 6.75 on the Richter scale. Fault activity is believed to be capable of generating Peak Ground Acceleration (PGA) of about 0.25 g at the project site (Mualchin, 1996). Seismic activities are estimated to have durations of about 15 to 20 seconds.

7.4.2 Ground Rupture

Surface ground rupture is considered unlikely within the project limits. Active and potentially active faults are not known to exist at the project site. In addition, the project site is not located within the State of California (Alquist –Priolo) Earthquake Fault Special Study Zone. Therefore, the potential for surface ground rupture within the project limits during seismic events is considered unlikely.

7.4.3 Liquefaction

Liquefaction, a sudden large decrease of shearing resistance of a cohesionless soil, can be caused by strong vibratory motion due to earthquakes. Both research and historical data indicate that loose granular soils that are saturated by a relatively shallow groundwater table are most susceptible to liquefaction and dynamic settlement. Liquefaction is generally known to occur in saturated or near-saturated cohesionless materials at depth shallower than about 100 feet.

A review of the piezometer data developed for this project revealed that the groundwater surface (GWS) occurs at depths over 100 feet below the ground surface. In addition, the project site is underlain by formational coarse grained soils that locally are cemented/indurated. Therefore, no potential for liquefaction exists at the project site.

8.0 GEOTECHNICAL ANALYSIS AND DESIGN

8.1 Dynamic Analysis

Dynamic analysis was not performed for this project. However, the parameter selection is provided in the section below.

8.1.1 Parameter Selection

The effective seismic horizontal coefficient, k_h, used in pseudo-static slope stability analyses is specified in Caltrans Guidelines for Foundation Investigations and Reports as one third of the PGA. Therefore, k_h=0.083 should be used for the pseudo-static analyses.

8.1.2 Analysis

In general, the project site is underlain by relatively geotechnically competent sedimentary formations that are not susceptible to adverse behavior during seismic events. In addition, the proposed retaining structures are of relatively low height; the tallest will be 16.5 feet in maximum height. No slopes exist or are planned to be constructed above or below the retaining walls. Therefore, dynamic analyses are not warranted.

8.2 Cuts and Excavations

This section presents the analyses used to determine the stability, rippability, and grading factors of materials in proposed cuts or excavations.

8.2.1 Stability

This project involves no planned or existing slopes. Therefore, slope stability analysis was not warranted for this report. The design and excavation of temporary slopes should follow the guidelines presented in the California Trenching and Shoring Manual.

8.2.2 Rippability

The project site is underlain by Quaternary and Tertiary sedimentary geologic soils. A relative thin mantle of fill materials covers these soils. Fill materials are expected to be rippable with the use of conventional grading equipment. However, the sedimentary soils belonging to the Lindavista and Stadium Conglomerate Formations may yield boulders, indurated gravels, and oversized blocks of rock-like material and extensive zones of concretions. Therefore, prolonged efforts utilizing heavy-duty grading equipment equivalent to a large excavator equipped with a rock breaker may by required to accomplish foundation excavations for planned retaining structures. It is estimated that about 40 percent of the volume of material at retaining wall excavations will require intense effort.

8.2.3 Grading Factors

Earthwork factors relate the in-place volume of material to be excavated to the in-place volume of material after placement as fill. The factors are defined as in-place volume of compacted fill divided by in-place volume of material to be excavated.

$G_f = V_{fill}/V_{exc}$

It is recommended that the following grading factors be applied to the project:

a) Placed at 90% relative compaction: $G_f = 0.96$

b) Placed at 95% relative compaction: $G_f = 0.94$

Based on local experience, on average the volume of soil locally derived from cuts/excavations will shrink during recompaction. However, the presence of cobbles, boulders and cemented concretion zones may result in the excavation of material unsuitable to reuse as compacted fill.

8.3 Embankments

No embankments are planned to be constructed or modified on this project site.

8.4 Earth Retaining Systems

Five Standard Plan retaining walls are addressed by this GDR. Their relevant parameters are presented in Table 1.

<u>Retaining Wall RW23L</u>

This masonry wall will be modified based on a Caltrans Standard Plan Type 6A wall design. Only the location of the upper wall section in relation to the foundation will be modified. However, the wall foundation design will follow the Type 6A Standard Plan. Wall RW23L, 5.33 feet in maximum height, may be designed as a Type 6A (modified) wall supported on a spread footing foundation as shown on sheet B3-11 in the Standard Plans, May 2006. The site foundation soil will easily satisfy the 2.0 ksf Gross Allowable Soil Bearing Pressure capacity for the design of Wall RW23L.

<u>Retaining Wall RW24L</u>

This wall is proposed to be a Caltrans Standard Plan Type 1 retaining wall. It is recommended that Wall RW24L, 13.86 feet in maximum height be designed as a Type 1 wall supported on a spread footing foundation as shown on sheet B3-1 in the Standard Plans, May 2006. With Loading Case I, the site foundation soil will easily satisfy the 3.3 ksf Gross Allowable Soil Bearing Pressure capacity for the design of Wall RW24L.

<u>Retaining Wall RW24R</u>

This wall is proposed to be a Caltrans Standard Plan Type 1 retaining wall. It is recommended that Wall RW24R, 13.87 feet in maximum height be designed as a Type 1 wall supported on a spread footing foundation as shown on sheet B3-1 in the Standard Plans, May 2006. With

Loading Case I, the site foundation soil will easily satisfy 3.3 ksf Gross Allowable Soil Bearing Pressure capacity for the design of Wall RW24R.

Retaining Wall RW25R

This wall is proposed to be a Caltrans Standard Plan Type 6B (modified) retaining wall. Only the location of the upper wall section in relation to the foundation will be modified. However, the wall foundation design will follow the Type 6B Standard Plan. Therefore, it is recommended that Wall RW25R, 6.67 feet in maximum height may be designed as a Type 6B (modified) wall supported on a spread footing foundation as shown on sheet B3-11 in the Standard Plans, May 2006. The site foundation soil will easily satisfy 2.0 ksf Gross Allowable Soil Bearing Pressure capacity for the design of Wall RW25R.

Retaining Wall RW23R

This wall is proposed to be a Caltrans Standard Plan Type 6B retaining wall. It is recommended that Wall RW23R, 5.5 feet in maximum height be designed as a Type 6B wall supported on a spread footing foundation as shown on sheet B3-11 in the Standard Plans, May 2006. The site foundation soil will easily satisfy 2.0 ksf Gross Allowable Soil Bearing Pressure capacity for the design of Wall RW23R.

8.5 Culvert Foundations

The design of culvert foundations was not requested for this report. Additionally, the project plans provided to OGD-S2 and field reconnaissance did not reveal significant excavations for drainage features.

8.6 Minor Structure Foundations

Recommendations for minor structure foundations were not requested for this report. Additionally, the project plans provided to OGD-S2 and field reconnaissance did not reveal significant excavations for minor structure foundations.

9.0 MATERIAL SOURCES

Material generated on site will consist of locally excavated fill materials and sedimentary soils. In general, these soils should be suitable as structure backfill for the retaining walls. However, the anticipated presence of oversized materials (cobbles, boulders) and the occurrence of indurated zones (layers, lenses, or spheroids) within sedimentary soils could render portion of material generated on site unsuitable as backfill. Approximately 40 percent of the total volume of material excavated is likely to require special processing, such as screening and possibly crushing to render the material suitable as fill. Project features should be designed for high corrosion potential; therefore, there may be no need to perform corrosion tests on imported materials.

10.0 MATERIAL DISPOSAL

Within the project limits, no sites are available for the wasting of surplus material. Any excavated materials generated during construction that are found to not be suitable as roadway subgrade, backfill, or topsoil should be properly disposed off-site.

11.0 CONSTRUCTION CONSIDERATIONS

This section describes the project construction considerations including: advisories; considerations that influence design and/or specifications; monitoring and instrumentation; hazardous waste; and differing site conditions.

11.1 Construction Advisories

- The allowable inclination of temporary cut slopes for the foundation excavations in fill materials and formational soils is 1:1 (Horizontal to Vertical). Steeper temporary cut slopes may be permitted based on site-specific soil properties, slope geometry, and stability analyses.
- Temporary slopes should not be allowed to stand unprotected during the rainy season.
- The site soils are expected to be readily excavated using conventional earth-moving equipment. However, it should be anticipated that cobbles, boulders, hard cemented lenses and layers, and concretions will be encountered during excavations for the foundations of the planned retaining structures. It is estimated that 40 percent of the total volume of excavated material will require intense effort equivalent to a large excavator equipped with a rock breaker.
- The likely presence of perched water at the location(s) of the retaining structure(s) may affect the construction activities. However, this hydrogeologic phenomenon may be mitigated by pumping perched water out of the foundation excavations.
- Material generated from onsite excavations will generally be suitable as structure backfill. However, due to the presence of cobbles, boulders, and concretions, significant portions of excavated soils may be deemed unsuitable as structural backfill, and will require special processing.

11.2 Construction Considerations that Influence Design

Site soils have been identified as having high corrosion potential. All project features should be designed to mitigate the effects of corrosive soils.

11.3 Construction Considerations that Influence Specifications

No construction considerations that influence specifications were identified during preparation of this report.

11.4 Construction Monitoring and Instrumentation

Piezometers R-09-020 and R-09-021 have been monitored by our office (OGD-S2) to track the groundwater surface elevation (GWS). This monitoring will continue until the construction phase of this project is completed.

Appropriate personnel should be present during project construction to observe foundation excavations, soils encountered upon excavating, and backfill activities to assure that the provisions set forth in this report are appropriately applied.

11.5 Hazardous Waste Considerations

It is our understanding that no hazardous waste or hazardous site conditions have been discovered at the project site. In addition, no potentially hazardous conditions were found during the preparation of this report.

11.6 Actual vs. Reported Site Conditions

The subsurface soil conditions presented in this report were determined based on the LOTB data developed by the OGD-S2 Branch B following their field investigation of 2009. In addition, the subsurface conditions were based on published material and archived data, site reconnaissance, and local experience. LOTB for this project will be provided in the Project Plans. If subsurface soil conditions encountered during the construction of the planned retaining structures appear to differ materially from those that were described in this report and presented in LOTB sheets, this judgement should be conveyed to the Caltrans Resident Engineer immediately. The Caltrans Resident Engineer, in turn, may establish a contact with Caltrans OGD-S2 staff with regard to the perceived difference. Our Office of Geotechnical Design-South 2 will readily assist the Caltrans Resident Engineer in any geotechnical issue related to this report.

12.0 RECOMMENDATIONS AND SPECIFICATIONS

Recommendations relevant to the design of standard plan retaining walls are contained in Section 8 of this report.

Extraordinary specifications do not apply to this project.

Table 1, Proposed Retaining Structures Data

STRUCTURE NUMBER	TYPE	ORIGIN STATION (“LO” Line)	END STATION (“LO” Line)	MAXIMUM HEIGHT (ft)
RW23L	**6A (MOD)**	**3+05.32**	**5+49.32**	**5.33**
RW24L	**1**	**4+43.57**	**5+68.95**	**13.86**
RW24R	**1**	**4+43.57**	**5+68.98**	**13.86**
RW25R	**6B (MOD)**	**4+55.00**	**7+49.79**	**6.67**
RW23R	**6B**	**1+00.00**	**2+66.58**	**5.50**

MOD = modified

Table 2: Geotechnical Soil Parameters

GEOLOGIC UNIT	COHESION (psf)	ANGLE OF INTERNAL FRICTION (degree)	MAXIMUM DRY DENSITY (psf)
Fill	100	32	125
Lindavista Formation	300	34	125
Stadium Conglomerate	350	38	130

Table 3, Groundwater Surface (GWS) Elevations Measured in Piezometers R-09-020 and R-09-021

	PIEZOMETER R-09-020 TOP ELEV = 529.64 ft DEPTH = 120.00 ft DATE INSTALLED = 6/17/2009		PIEZOMETER R-09-021 TOP ELEV = 525.21 ft DEPTH = 115.5 ft DATE INSTALLED = 6/17/2009	
DATE MEASURED	GWS DEPTH (ft)	**GWS ELEV (ft)**	GWS DEPTH (ft)	**GWS ELEV (ft)**
8/12/2009	102.7	**426.94**	105.7	**419.51**
2/4/2010	103.2	**426.44**	105.1	**420.11**
3/10/2010	103.5	**426.14**	105.2	**420.01**

INDEX OF PLANS

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
11	SD	15	R15.2/M16.0	1	

STATE OF CALIFORNIA

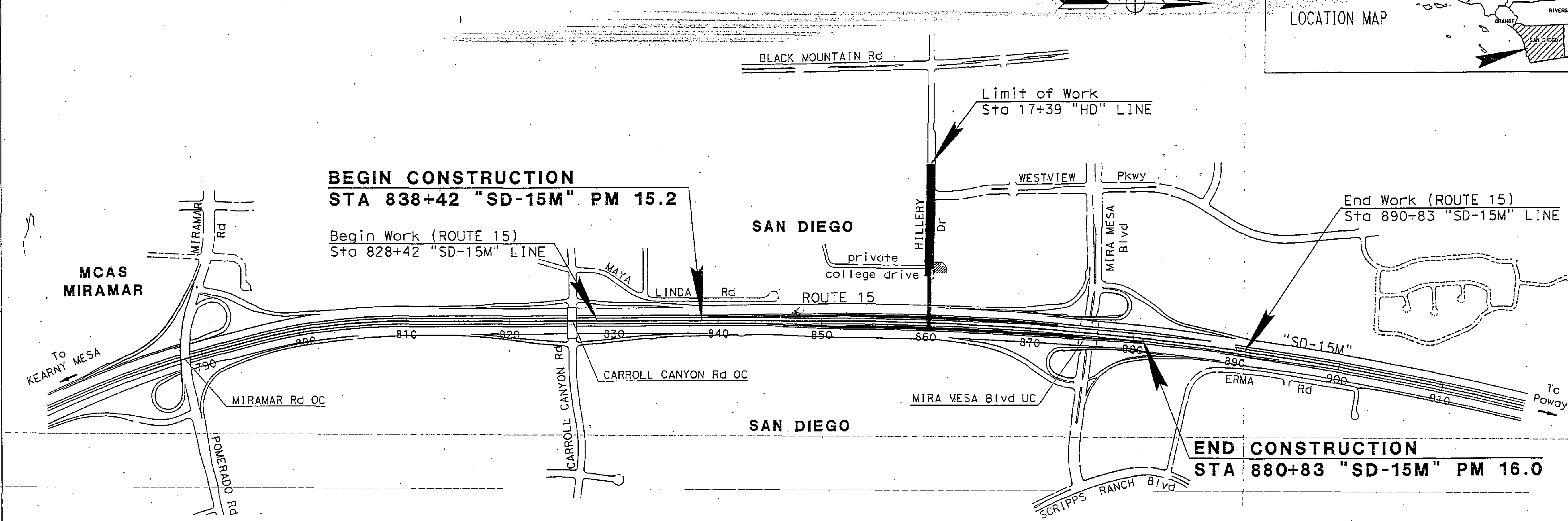
DEPARTMENT OF TRANSPORTATION

PROJECT PLANS FOR CONSTRUCTION ON

STATE HIGHWAY

IN SAN DIEGO COUNTY
IN SAN DIEGO ON ROUTE 15 FROM 0.2 MILE NORTH OF CARROLL CANYON ROAD TO 0.1 MILE NORTH OF MIRA MESA BLVD

TO BE SUPPLEMENTED BY STANDARD PLANS DATED MAY 2006



ESA ENVIRONMENTALLY SENSITIVE AREA

THE CONTRACTOR SHALL POSSESS THE CLASS (OR CLASSES) OF LICENSE AS SPECIFIED IN THE "NOTICE TO BIDDERS."

NO SCALE

FIGURE 1	EA 11-2T0951
Project Location	4/2/2010

APPROVED AS TO IMPACT ON STATE FACILITIES AND CONFORMANCE WITH APPLICABLE STATE STANDARDS AND PRACTICES AND THAT TECHNICAL OVERSIGHT WAS PERFORMED.

CONSULTANT DESIGN ENGINEER	CALTRANS DESIGN OVERSIGHT APPROVAL	REGISTRATION No.	LICENSE Exp DATE	DATE SIGNED
F. R. CLARK FERNON	DUY NGOC HOANG	C66033	06/30/2010	

PROJECT ENGINEER DATE
REGISTERED CIVIL ENGINEER

REGISTERED PROFESSIONAL ENGINEER F.R. CLARK FERNON No. 47273 Exp. 12/31/09 CIVIL STATE OF CALIFORNIA

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS ...RS OR AGENTS SHALL NOT BE ...SIBLE FOR THE ACCURACY OR ...TENESS OF SCANNED COPIES OF THIS PLAN SHEET.

AECOM
7807 CONVOY COURT, SUITE 200
SAN DIEGO, CA 92111

CONTRACT No. 11-2T0954

DATE PLOTTED => 7/28/2009
TIME PLOTTED => 2:50:37 PM

LAST REVISION 06-26-09

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
11	SD	15	R15.2/M16.0		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER MICHAEL R. CRULL No. 34522 Exp. 09/30/09 CIVIL STATE OF CALIFORNIA

AECOM
7807 CONVOY COURT, SUITE 200
SAN DIEGO, CA 92111

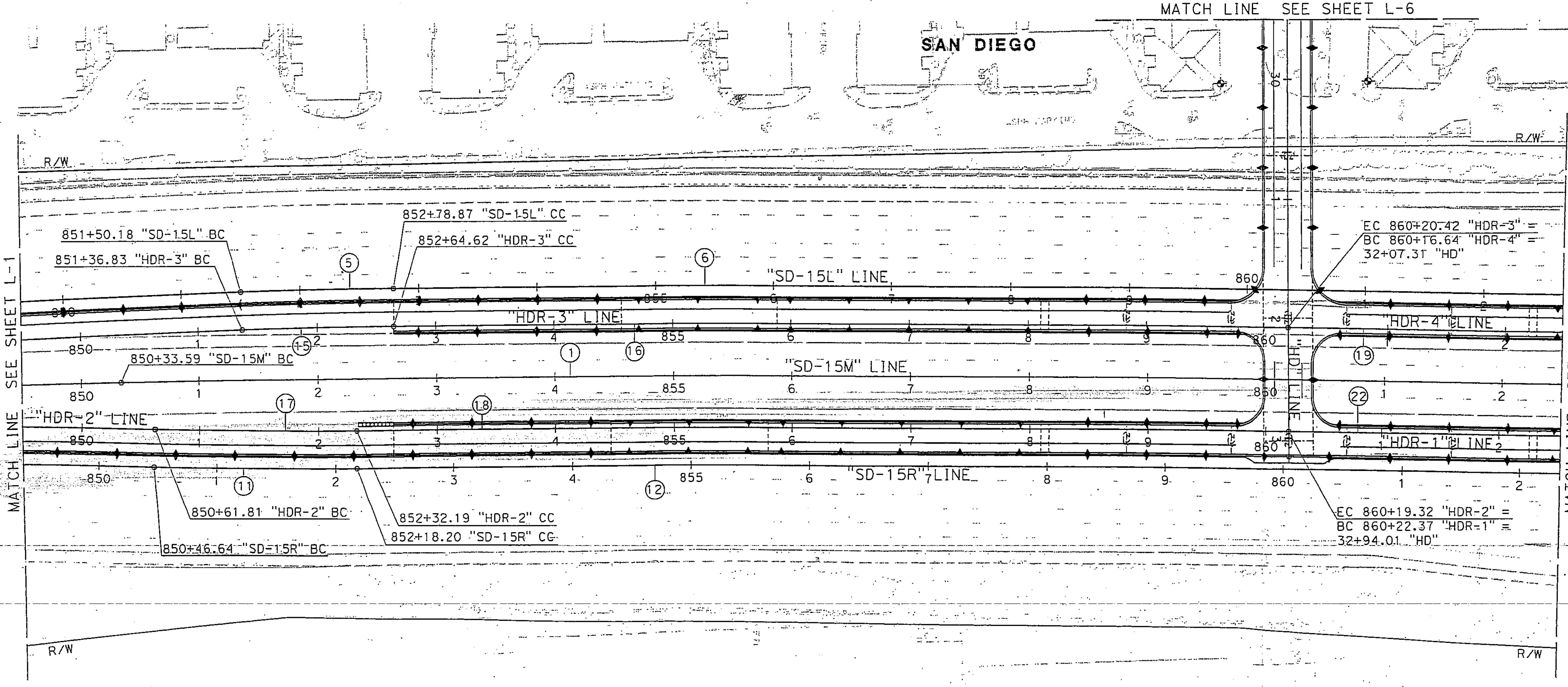
NOTE:

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

CURVE DATA

No.	R	Δ	T	L
1	28015.55'	6°35'30"	1613.35'	3223.15'
5	4559.08'	1°37'02"	64.35'	128.69'
6	28090.42'	3°04'51"	755.41'	1510.45'
11	4559.08'	2°09'22"	85.79'	171.56'
12	27940.68'	3°09'27"	770.11'	1539.82'
15	4527.55'	1°37'02"	63.90'	127.80'
16	28058.89'	1°32'36"	377.92'	755.79'
17	4527.55'	2°09'22"	85.20'	170.38'
18	27972.21'	1°36'44"	393.59'	787.13"
19	28058.89'	1°32'15"	376.51'	752.97'
22	27972.21'	1°32'43"	377.24'	754.43"

MATCH LINE SEE SHEET L-6



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

FIGURE 2	EA 11-2T0951
Layout L-2	4/2/2010

Dist	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS
11	SD	15	R15.2/M16.0		

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

MICHAEL R. CRULL No. 34522 Exp 09/30/09 CIVIL STATE OF CALIFORNIA REGISTERED PROFESSIONAL ENGINEER

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

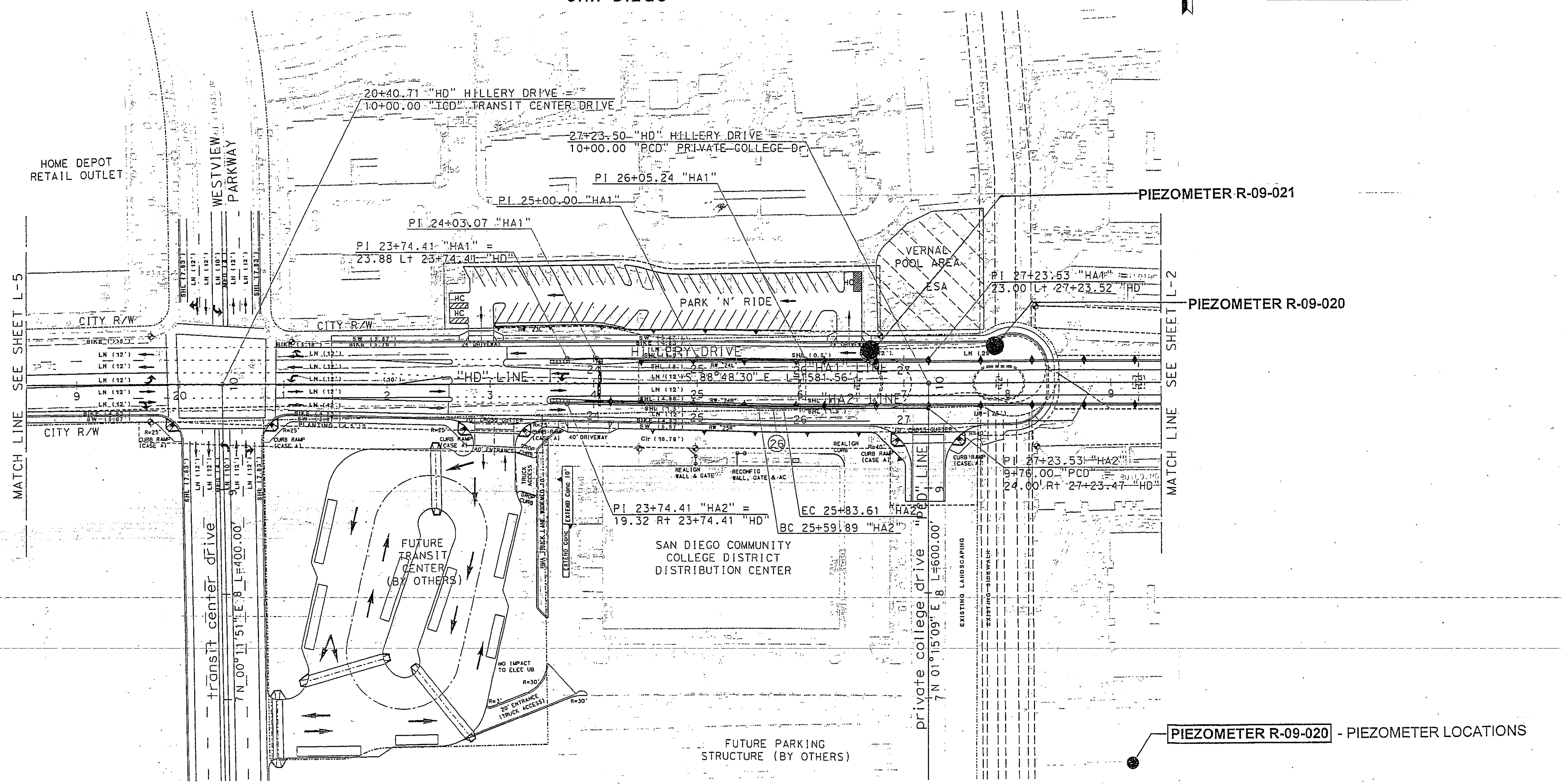
AECOM
7807 CONVOY COURT, SUITE 200
SAN DIEGO, CA 92111

NOTE:

FOR COMPLETE RIGHT OF WAY AND ACCURATE ACCESS DATA, SEE RIGHT OF WAY RECORD MAPS AT DISTRICT OFFICE.

CURVE DATA

No.	R	Δ	T	L
26	1000.00'	1°21'32"	11.86'	23.72'



ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE SHOWN

FIGURE 3	**EA 11-2T0951**
Layout L-6	**4/2/2010**

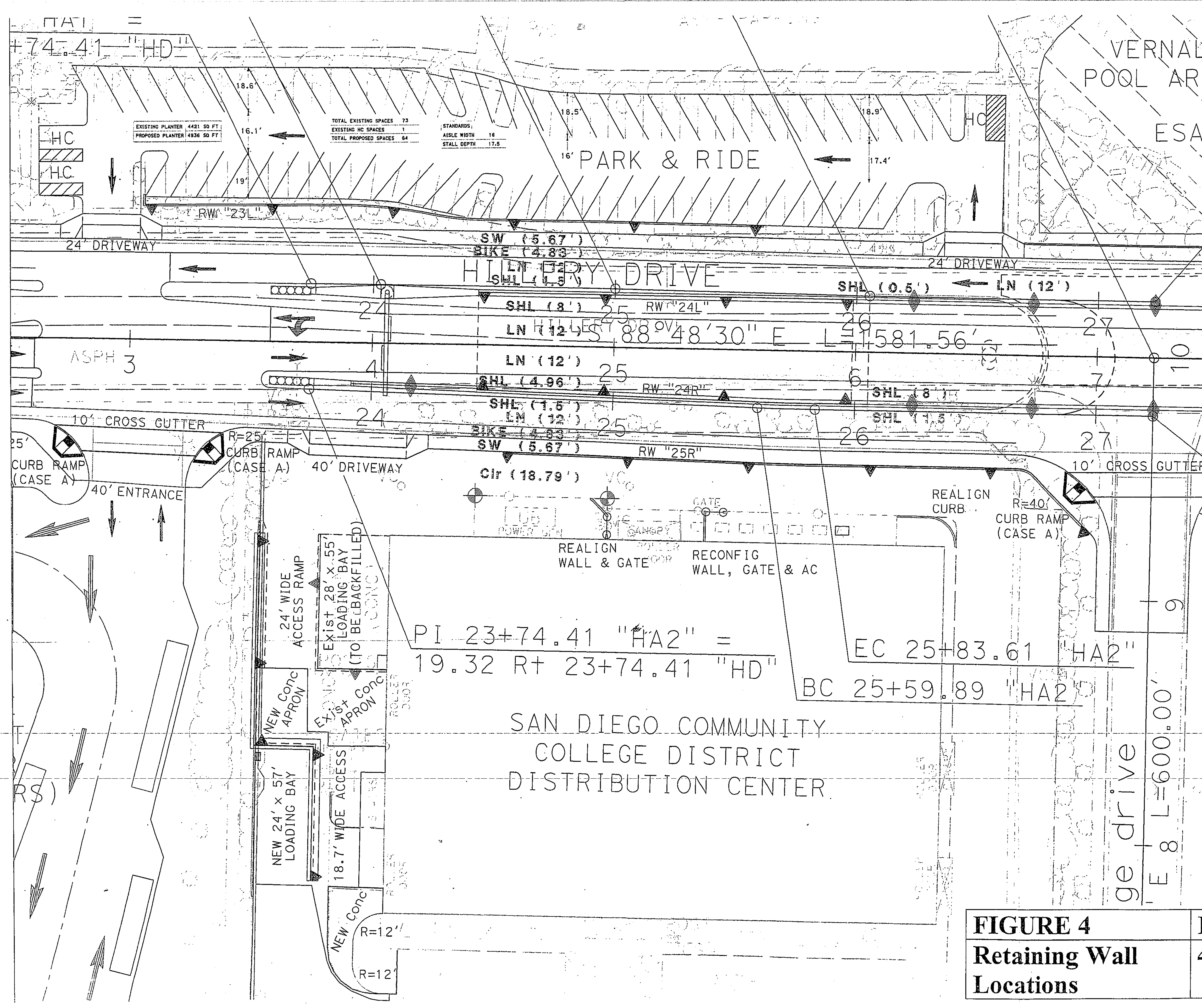


FIGURE 4	**EA 11-2T0951**
Retaining Wall Locations	**4/2/2010**

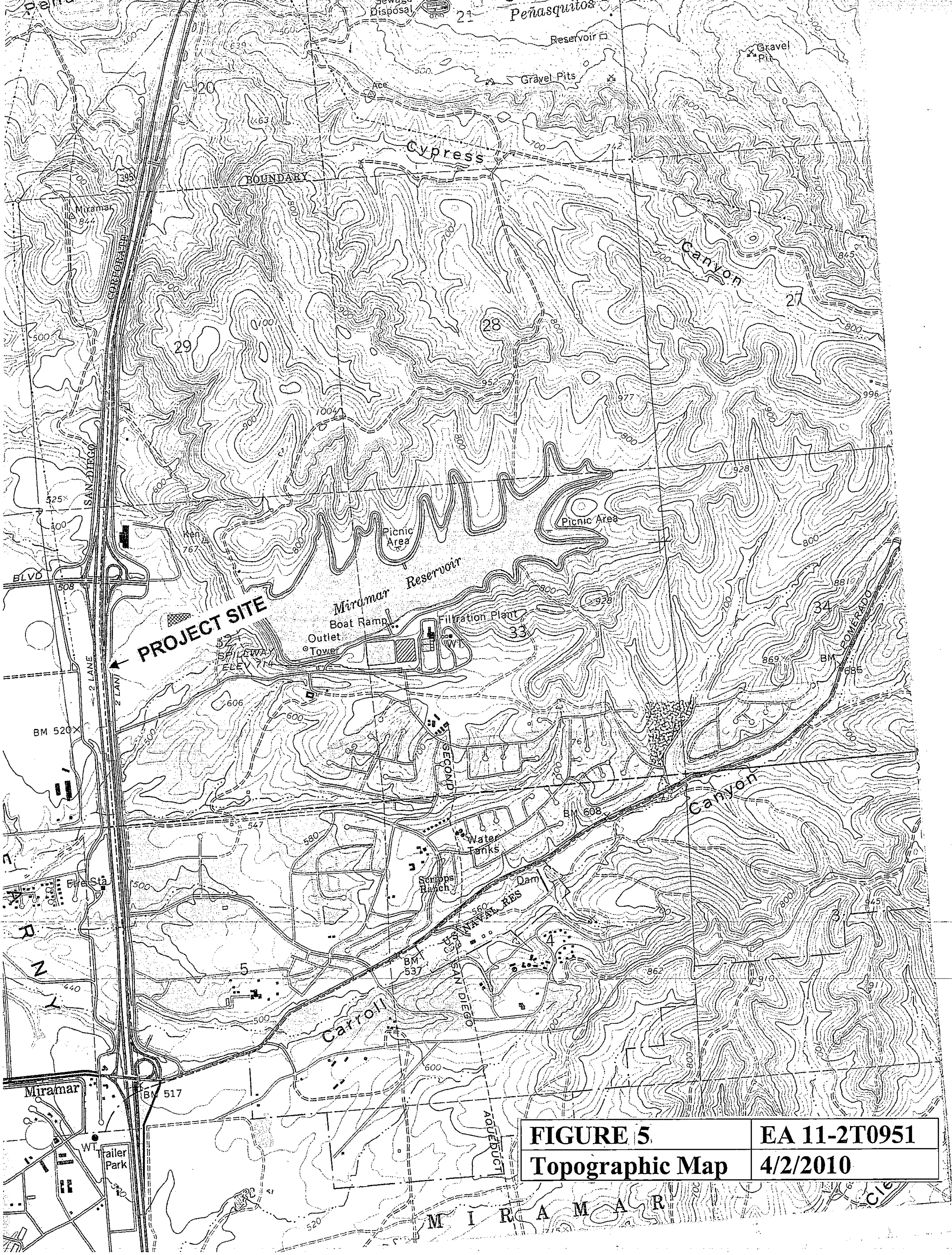


FIGURE 5 | EA 11-2T0951
Topographic Map | 4/2/2010

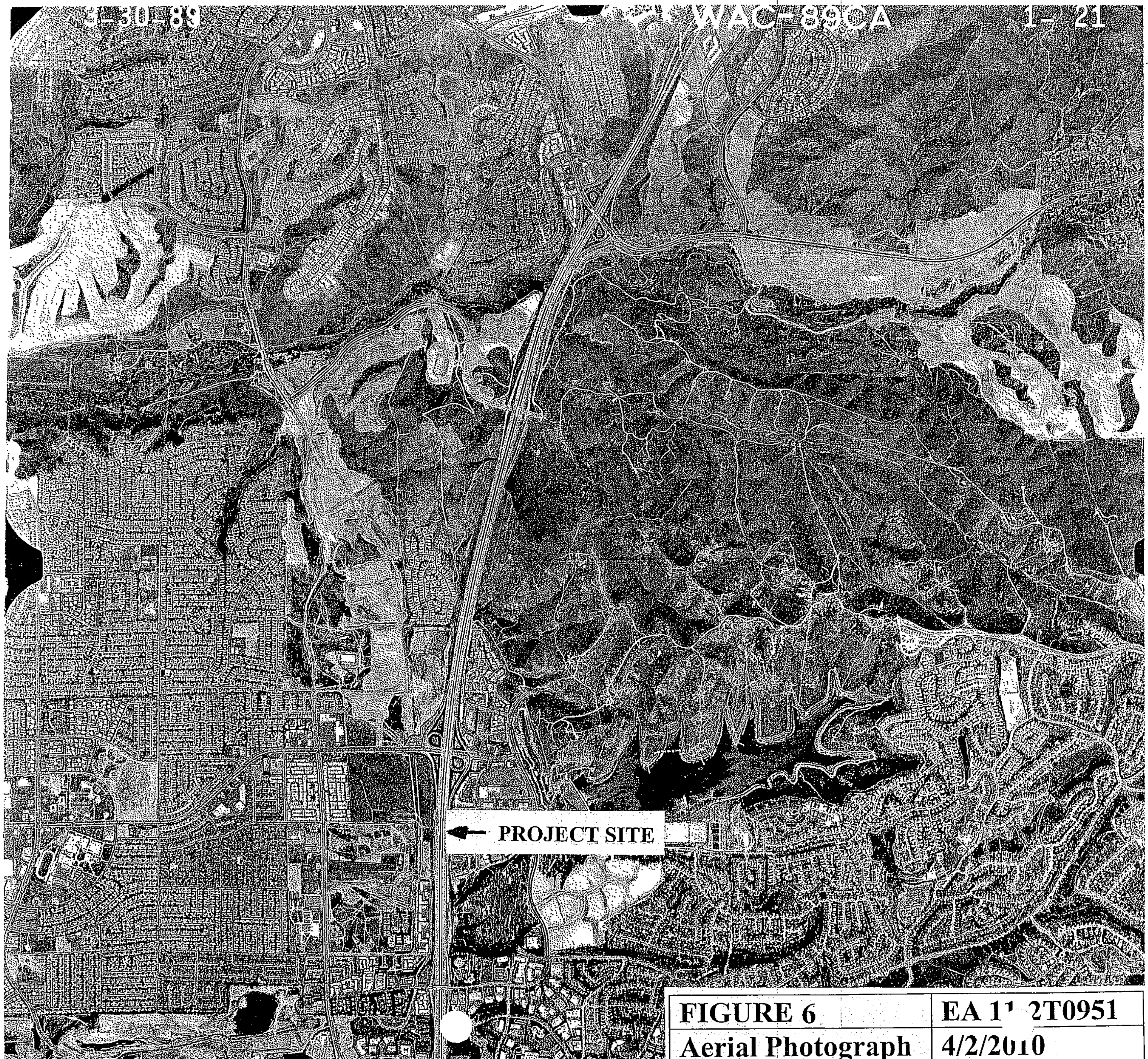


FIGURE 6	**EA 11-2T0951**
Aerial Photograph	**4/2/2010**

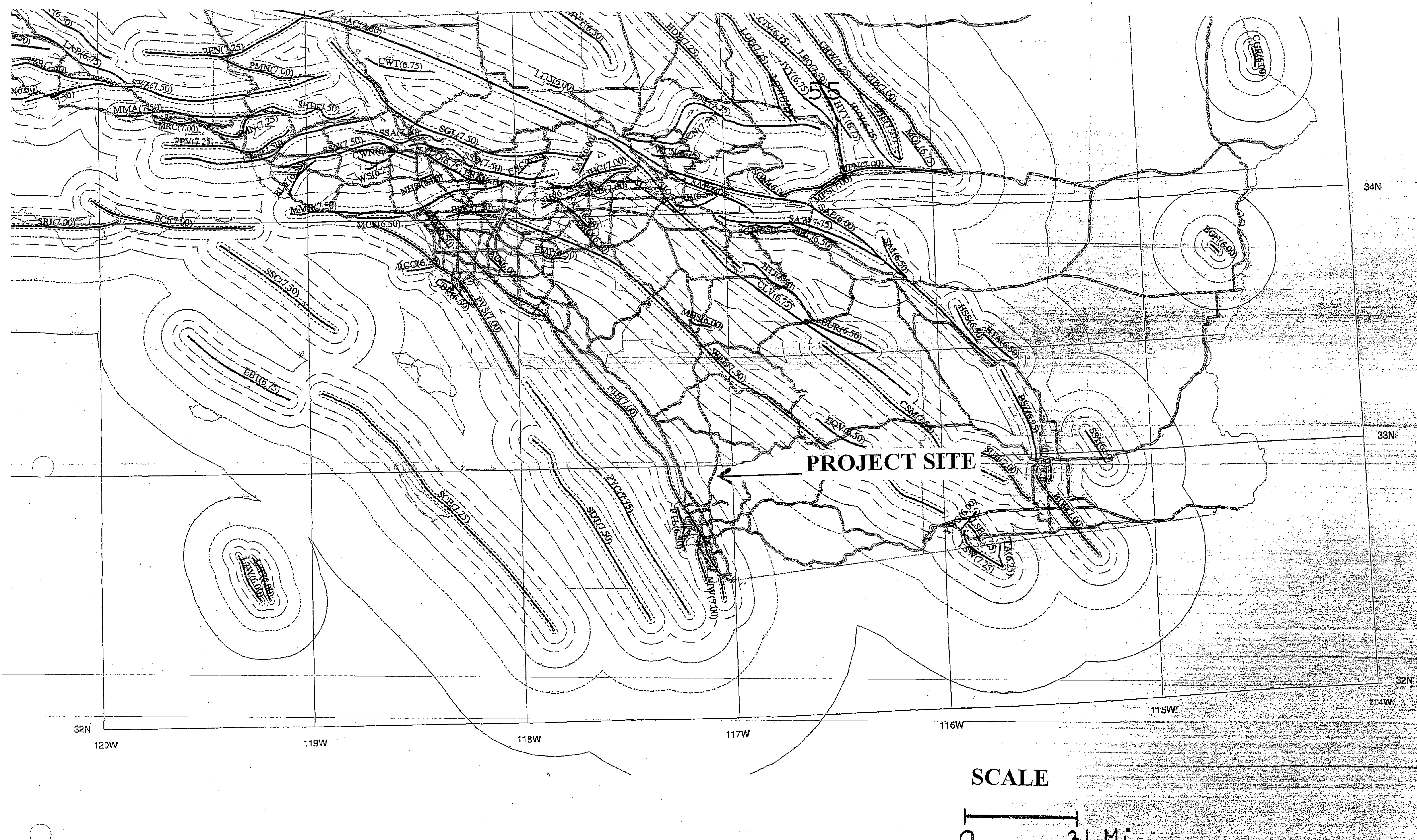


FIGURE 7	EA 11-2T0951
California Seismic Hazard Detail Index Map 1996	4/2/2010

A
B
7°00′
33°00′
3651

GEOLOGY OF THE POWAY QUADRANGLE SAN DIEGO COUNTY, CALIFORNIA

by Michael P. Kennedy and G. L. Peterson

SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 20 FEET
DATUM IS MEAN SEA LEVEL

T. 13 S.
T. 14 S.
3650
300 000
FEET

1975

EXPLANATION

QUATERNARY

Holocene

Qsw
Qal + Qsw

Alluvium and Slopewash

Qsw, Slopewash; Qal & Qsw, Alluvium and Slopewash undifferentiated.

Qls

Landslide deposits

Pleistocene

Qt

Stream-terrace deposits

Ql

Lindavista Formation

3648

TERTIARY

Eocene

Tpm
Tp
Tmv
Tst

Poway Group

Tp, Pomerado Conglomerate; Tpm, Mirmar Sandstone Tongue of Pomerado Conglomerate; Tmv, Mission Valley Formation; Tst, Stadium Conglomerate.

3647

Tf

Friars Formation

57′30″

CRETACEOUS

Upper Cretaceous

Kl

Lusardi Formation

3646

Kg

Granite rocks

Undifferentiated granitic rocks of the southern California batholith.

FIGURE 8	EA 11-2T0951
Geologic Map	4/2/2010

JURASSIC

Jsp

Santiago Peak Volcanics

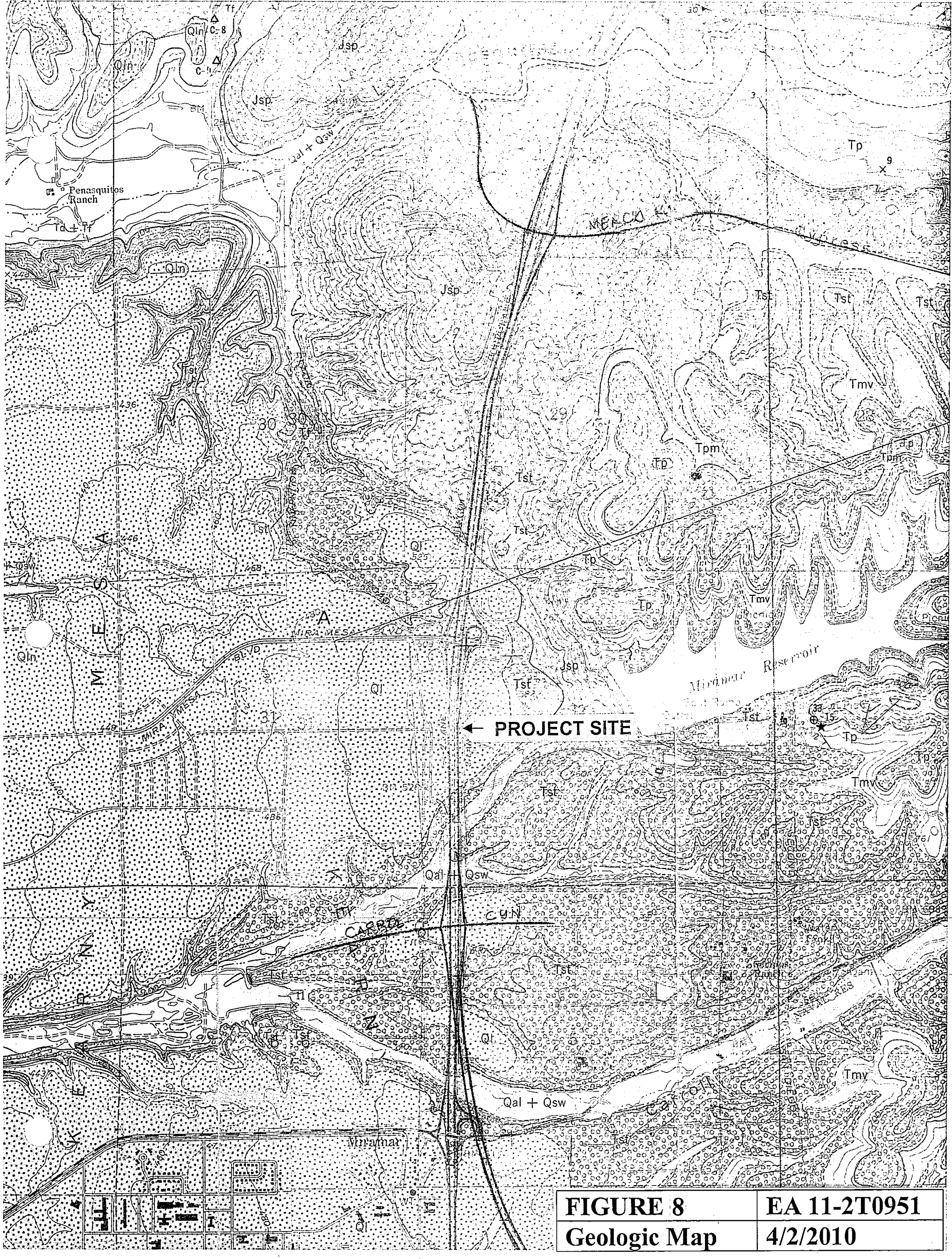


FIGURE 8	EA 11-2T0951
Geologic Map	4/2/2010

Gerard Chadergian/D11/Caltrans/CAGov

03/24/2010 11:16 AM

To duy_ngoc_hoang@dot.ca.gov

cc

bcc

Subject Fw: Pavement Thickness Hillery Drive

please file.

Gerard Chadergian
Design Manager/Project Manager
District 11 - I-15 Corridor Design
13560 Evening Creek Dr. N
San Diego, CA 92128 / MS 93
Office - (858) 748-7935
Mobile - (858) 752-0336
Fax - (858) 748-7915

----- Forwarded by Gerard Chadergian/D11/Caltrans/CAGov on 03/24/2010 11:16 AM -----

"Lundquist, Jim" <JLundquist@sandiego.gov>

03/24/2010 10:59 AM

To "gerard_chadergian@dot.ca.gov" <gerard_chadergian@dot.ca.gov>, "andrew_rice@dot.ca.gov" <andrew_rice@dot.ca.gov>, "Crull, Michael" <Michael.Crull@aecom.com>, "gustavo.dallarda@dot.ca.gov" <gustavo.dallarda@dot.ca.gov>

cc "Castillo, Jose" <JCastillo@sandiego.gov>, "Gefrom, Walter" <WGefrom@sandiego.gov>, "Van Wanseele, Deborah" <DVanWanseele@sandiego.gov>, "Yousef, Hasan" <HYousef@sandiego.gov>, "Pazargadi, Siavash" <SPazargadi@sandiego.gov>

Subject RE: Pavement Thickness Hillery Drive

Gerard, Andrew, Gustavo and Michael:

We have approved the installation of rubberized asphalt on Hillary Drive east of Black Mountain Road for your project.

As you may be aware, the widening of Hillary Drive in this same location is expected to be a City project using funding from this project. Close coordination between the Caltrans project and the City project will be required.

If you have any questions, please feel free to contact me.

Jim Lundquist
Associate Engineer - Traffic
Bike Coordinator & Associate Caltrans Liaison
City of San Diego
Program Management, Engineering & Capital Projects Dept.
1010 2nd Ave, Ste 800 ; MS 609
San Diego, CA 92101-4907

phone: 619/533-3045
Fax: 619/533-3651
e-mail: JLundquist@SanDiego.gov

From: Yousef, Hasan
Sent: Wednesday, March 24, 2010 10:31 AM
To: Lundquist, Jim
Cc: Castillo, Jose; Gefrom, Walter; Van Wanseele, Deborah
Subject: RE: Pavement Thickness Hillery Drive

Yes. This will be the only street within the City with rubberized asphalt and we will use it as a test site.

Thanks
Hasan

From: Bartholomae, Barbara [mailto:Barbara.Bartholomae@aecom.com]
Sent: Monday, March 01, 2010 4:44 PM
To: Lundquist, Jim
Cc: Crull, Michael
Subject: FW: Pavement Thickness Hillery Drive

Attached are our calculations and supporting documentation for the additional layer of AC pavement (3.25") needed to handle the additional traffic load along Hillery Drive. Also attached is an email from Caltrans stating how the use of Rubberized HMA (hot mixed asphalt) could provide basically the same load support at half the thickness and Caltrans' standard specification for Rubberized asphalt.

Please forward to the proper City staff to determine whether or not the City will allow Rubberized HMA to be installed along Hillery Drive and if the City can maintain it.

Thank you.

<<Duy Hoang pavement thickness email.pdf>> <<Rubberized AC Spec.doc>>

From: Crull, Michael
Sent: Monday, March 01, 2010 3:35 PM
To: Bartholomae, Barbara
Subject: FW: Pavement Thickness Hillery Drive

From: Crull, Michael
Sent: Thursday, January 28, 2010 6:41 AM
To: 'Gerard Chadergian'
Subject: Pavement Thickness Hillery Drive

Hi Gerard,

We found a plan that shows the originally constructed pavement thickness for Hillery Drive from Black Mountain Road to Westview Parkway. This is attached.

<<Pvmt Thick Hillery Dr BMR - WVP 011810.pdf>> <<Traffic on Hillery Dr BMR - WVP 011810.pdf>> <<City Pvmt Sched J.pdf>> <<Hillery Drive E of BMR Calcs 012810.pdf>> <<Hillery Drive E of BMR Scetions 012810.pdf>> <<Hillery Drive E of BMR Cost Backup 012810.pdf>>

Also attached is a table from the traffic study. It shows the future year 2030 with project average daily traffic as 38,267 and the future year 2030 no build average daily traffic as 24,823, which means that the project is expected to add 13,444 (average) vehicles per day to the road.

Gerard, I remember that Jim Lundquist wanted to check to see if this portion of Hillery Drive would need an additional layer of AC pavement to handle the additional traffic. Using the existing pavement thickness (from the above plans) and the City's Pavement Design Standards (Schedule "J") (copy attached), I came up with a quick calc, which is also attached.

This quick calc shows that we would need to add about 3.25" of AC to get a pavement section equal to the City Standard section. I expect that we would have to cold plane the edges to match the existing curb and gutter as shown on the cross section sketch. A rough cost for this would be about $210,000. Gerard, what do you think about adding this to the project? Please let me know. We could also get a better determination for the additional AC thickness (either from the Caltrans lab or Kleinfelder). If you would like Kleinfelder to look at this for us, just let me know.

Thanks, Mike.

State of California

Business, Transportation and Housing Agency

{PRIVATE }

Memorandum

To : Gerard Chadergian (MS 93)
Design Manager
I-15 Corridor Design

Date: September 8, 2009

File: Pavement Policy Bulletin
PPB 09-01

From : **DEPARTMENT OF TRANSPORTATION - DISTRICT 11**
PAVEMENT ENGINEERING SECTION

Subject: **Rigid Pavement Base Design**

In accordance with Pavement Policy Bulletin, PPB 09-01, Rigid Pavement Base Design, projects that have not yet completed PS&E shall be updated and estimated to reflect the new base thickness of 0.25’ (75 mm) HMA-A under the new PCC pavement.

PPB 09-01, dated August 27, 2009, from Shakir Shatnawi, State Pavement Engineer, supercedes the HMA base thickness previously used on this project.

Please remember to consider all of the effects that reducing the HMA thickness will have on other aspects of the roadbed construction. Adjusting the subgrade elevation will be required unless, as an option, the concrete pavement layer is increased as long as the concrete thickness is no more than 1.15 feet (345 mm) thick.

If you have questions with regards to this memorandum, please contact me at 858-467-4056 or FAX at 858-467-4063.

David Evans
District Pavement Engineer
District 11 Materials Lab

cc: A Padilla (DME)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. NORBERT GEE
Office of Special Funded Projects
Division of Engineering Services

Date: September 1, 2010

File: 11-SD-15-PM R15.7
11-2T0951
Hillery Drive O.C.: Br. #57-1213
Northbound On-ramp: Br. #57-1214
Northbound Off-ramp: Br. #57-1215
Southbound On-ramp: Br. #57-1216
Southbound Off-ramp: Br. #57-1217

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: Revised Foundation Report for Hillery Dr. OC and DAR

This Revised Foundation Report supercedes the "original" Foundation Report, dated May 4, 2010, and all consultant generated Preliminary Foundation Reports for the proposed Hillery Drive O.C. (Main Access Ramp, Br. #57-1213) and its connecting Direct Access Ramps: Northbound On-ramp (Br. # 57-1214), Northbound Off-ramp (Br. #57-1215), Southbound On-ramp (Br. #57-1216), and Southbound Off-ramp (Br. #57-1217). This Revised Foundation Report is in response to changes in design loads and pile diameters at the Bent locations of the Main Access Ramp and the Direct Access Ramps. The Office of Geotechnical Design-South 2, Branch B (OGDS2B) completed a foundation investigation pursuant to a request by the Office of Special Funded Projects (OSFP) for foundation recommendations for the proposed structures. The Hillery Drive O.C. (Main Access Ramp) and its connecting Direct Access Ramps are being designed by the consultant AECOM which has provided the Office of Geotechnical Design, South-2 the design information used in this report to provide foundation recommendations.

The following foundation recommendations are based on subsurface information gathered during a foundation investigation conducted from April 2009 through October 2009. With regards to the current foundation recommendations given in this report, all elevations referenced within this report and shown on the Log of Test Borings (LOTB) sheets are based on the North American Vertical Datum (NAVD 88).

Project Description/History

The proposed bridge sites for the five structures are located on Interstate 15 in the northern part of the city of San Diego. These structures are part of the I-15 Managed Lanes Project aimed at improving traffic mobility on Route 15 between the Escondido area and San Diego. The proposed structures will allow access to the I-15 managed lanes from Hillery Drive.

The Hillery Drive O.C. (Main Access Ramp, Br. #57-1213) will consist of a six span, cast-in-place, reinforced concrete and pre-stressed, box girder type structure measuring 712 feet long and 42 feet wide.

The proposed Hillery Drive Northbound On-ramp, Northbound Off-ramp, Southbound On-Ramp and Southbound Off-Ramp structures (Br. Nos. 57-1214, 57-1215, 57-1216 and 57-1217, respectively), will measure 182.3 ft long and 26.6 ft wide, and will provide commuters, using proposed managed lanes, access to the Hillery Drive O.C. (Main Access Ramp). The proposed Northbound and Southbound On-Ramp and Off-Ramp structures will each consist of a three span, cast-in-place, pre-stressed, box girder type structure, which will connect to the Main Access Ramp by seatless hinges.

Geology

The foundation investigation completed in October 2009 consisted of 24 mud rotary borings (Borings R-09-001 through R-09-024) and 7 auger borings (borings A-09-025 through A-09-031).

The proposed bridge site is located in an area of ancient sedimentary marine terraces cut by creeks, which generally flow east to west. The geologic map "Geology of the Poway Quadrangle, San Diego County, California (1975)" indicates that the site is underlain by the Quaternary Lindavista Formation at the surface, which is described as sandstone and conglomerate. Below the Lindavista Formation lie the sedimentary facies of the Tertiary Poway Group, specifically the Stadium Conglomerate.

The 2009 foundation investigation revealed the site is generally underlain by sedimentary formational material consisting of interbedded layers of sandstone and cobble conglomerate. The sandstone is typically very soft and poorly indurated. The cobble conglomerate consists of rounded igneous and metamorphic clasts within a very soft, poorly indurated gravel and sand matrix.

For more specific details regarding the sedimentary formation descriptions from the 2009 foundation investigation, refer to the LOTB sheets for the proposed new bridges.

Ground Water

At the proposed bridge site, ground water was attempted to be measured in some of the borings drilled for both the Main Access Ramp as well as the Direct Access Ramps and associated retaining walls. Generally, the ground water was determined to be relatively deep, however, in boring R-09-005, ground water was measured at two feet below the ground surface on July 9, 2009. To determine if there was perched ground water in the area, seven auger borings were drilled from October 6 to 8, 2009 across the site. Ground water was not encountered in any of the auger borings. At the nearby Mira Mesa Rd. OC, during the 2000 subsurface investigation for the widening of this structure, water was found to be flowing just beneath the highway pavement. This water was seeping/flowing out from beneath both bridge abutments, causing the embankments to erode and flow down onto the adjacent city sidewalks below. It was observed that the water would be present and flowing, and then not present, at irregular intervals. Due to this irregular presence of water, the abutments for the widening were supported on deep foundations. Additionally, during construction of the most recent widening of the Mira Mesa Rd. OC, water was also encountered at shallow depths beneath the pavement, requiring a Contract Change Order (CCO) to divert water away from the site. The source of the flowing ground water still has not been determined. Due to this irregular presence of water at shallow depths, it is possible that water may be encountered during construction of the footings for the abutment supports, as well as the Cast-In-Drilled-Hole (CIDH) piles.

Recorded ground water information from the 2009 subsurface investigation is presented in Table 1, below.

Table 1 – Ground Water Summary

Boring No.	Top of Boring Elev. (ft)	Total Depth of Boring (ft)	Date Measured	GWS depth (ft)	GWS elev. (ft)
R-09-021	525.2	115.5	2/4/2010	105.1	420.1
R-09-020	529.6	120.0	2/4/2010	103.2	426.4
R-09-003	528.4	131.9	7/9/2009	108.6	419.8
R-09-014	529.3	90.0	7/9/2009	dry	dry
R-09-006	528.2	60.7	7/9/2009	dry	dry
R-09-016	529.1	65.5	7/9/2009	dry	dry
R-09-005	524.4	25.7	7/9/2009	2	522.4

Measured ground water elevations are also shown on the LOTB sheets. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions.

Scour Potential

There is no scour potential at the site, since the structures do not span any watercourse.

Corrosion

Corrosion test results for soil samples collected from borings R-09-001, R-09-004, R-09-017, are shown below in Table 2. The site is considered corrosive by current Caltrans standards. Reinforced concrete (including piles) requires corrosion mitigation in accordance with *Bridge Design Specifications, Article 8.22.*

Table 2 – Corrosion Test Summary

Location	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
Boring R-09-001 (Elev. 521.5 – 505.7 ft)	4.71	1343	N/A	N/A
Boring R-09-004 (Elev. 528.8 – 478.8 ft)	5.16	1151	N/A	N/A
Boring R-09-017 (Elev. 525.7 – 500.7 ft)	6.03	779	219	151

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Fault and Seismic Data

The structure site is potentially subject to strong ground motions from nearby earthquake sources during the design life of the new structure. The site is located about 9 miles northeast of the Newport-Inglewood-Rose Canyon/E Fault (Fault ID 224, M_{max}=7.5, strike-slip). The peak horizontal bedrock acceleration (PBA) at the site is estimated at 0.25g. The Office of Geotechnical Design, South 2, will provide Final Seismic Design Recommendations in a separate memorandum, which will be forwarded to your office when completed.

Liquefaction Potential

Due to the dense nature of the underlying sedimentary formational material, and deep ground water elevation, the potential for soil liquefaction due to strong ground shaking is considered low at the proposed bridge site.

Foundation Recommendations

The following recommendations are for the proposed Hillery Drive O.C. (Main Access Ramp, Br. #57-1213) as shown on the General Plans dated March 8, 2010, and its connecting Direct Access Ramps: Northbound On/Off-Ramp (Br. No. 57-1214 & 57-1215) and Southbound On/Off-Ramps (Br. No. 57-1216 & 57-1217), as shown on the General Plans dated January 22, 2010. At the Abutment support locations of all the bridges spread footings are recommended for support. At the Bent support locations of all the bridges Cast-In-Drilled-Hole (CIDH) piles are recommended for support.

Abutment Footing Locations

Hillery Drive O.C. (Main Access Ramp, Br. No. 57-1213)

At the Abutment 1 support location spread footings are recommended for support. The Abutment 1 footing may be supported on the underlying undisturbed sedimentary formational material. Table 3, below, presents the Abutment 1 spread footing design information provided by the structure designer, AECOM.

Table 3: Abutment 1 Footing (Br. #57-1213)
Spread Footing Design Information Provided by the Structure Designer, AECOM

Support Location	Bottom of Footing Elevation	Footing Size		Permissible Settlement Under Service Load	Service Limit State I						
					Total Load				Permanent Load		
					Vertical Load	Effective Dimensions		Horizontal Load in Long. Direction	Vertical Load	Effective Dimensions	
		B	L			B'	L'			B'	L'
Abutment 1	518.8 ft	16.0 ft	44.0 ft	1 in	2032 kips	14.5 ft	44.0 ft	420 kips	1691 kips	14.2 ft	44.0 ft

The recommended Allowable Bearing Capacity and bottom of footing elevation are listed below in Table 4.

Table 4: Foundation Design Recommendations for Abutment 1 Spread Footing (Br. #57-1213)

Support Location	Footing Size		Bottom of Footing Elevation	Minimum Footing Embedment Depth	Total Permissible Support Settlement	WSD (LRFD Service I Limit State Load Combination)	
	B'	L'				Permissible Gross Contact Stress	Allowable Gross Bearing Capacity
Abutment 1	14.5 ft	44.0 ft	518.8 ft	5.0 ft	1.0 in	N/A*	20.0 ksf

Notes: 1) Recommendations are based on the foundation geometry and the load provided by AECOM in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2) See MTD 4-1 for definitions and applications of the recommended design parameters.
*Settlement is N/A due to bottom of footing founded on dense sedimentary formational material.

Hillery Drive Northbound On-Ramp (HDR-1) (Br. No. 57-1214)

At Abutment 3 support location spread footings are recommended for support. The Abutment 3 footing may be supported on the underlying undisturbed sedimentary formational material. Table 5, below, presents the Abutment 3 spread footing design information provided by the structure designer, AECOM.

Table 5: Abutment 3 Footing (Br. #57-1214)
Spread Footing Design Information Provided by the Structure Designer, AECOM

Support Location	Bottom of Footing Elevation	Footing Size		Permissible Settlement Under Service Load	Service Limit State I						
					Total Load				Permanent Load		
					Vertical Load	Effective Dimensions		Horizontal Load in Long. Direction	Vertical Load	Effective Dimensions	
		B	L			B'	L'			B'	L'
Abutment 3	524.8 ft	14.0 ft	29.0 ft	1 in	1397 kips	11.9 ft	29.0 ft	362 kips	973 kips	11.6 ft	29.0 ft

The recommended Allowable Bearing Capacity and bottom of footing elevation are listed below in Table 6.

Table 6: Foundation Design Recommendations for Abutment 3 Spread Footing (Br. #57-1214)

Support Location	Footing Size		Bottom of Footing Elevation	Minimum Footing Embedment Depth	Total Permissible Support Settlement	WSD (LRFD Service I Limit State Load Combination)	
	B'	L'				Permissible Gross Contact Stress	Allowable Gross Bearing Capacity
Abutment 3	11.9 ft	29.0 ft	524.8 ft	5.0 ft	1.0 in	N/A*	10.0 ksf

Notes: 1) Recommendations are based on the foundation geometry and the load provided by AECOM in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2) See MTD 4-1 for definitions and applications of the recommended design parameters.
*Settlement is N/A due to bottom of footing founded on dense sedimentary formational material.

Hillery Drive Northbound Off-Ramp (HDR-2) (Br. No. 57-1215)

At Abutment 1 support location spread footings are recommended for support. The Abutment 1 footing may be supported on the underlying undisturbed sedimentary formational material. Table 7, below, presents the Abutment 1 spread footing design information provided by the structure designer, AECOM.

Table 7: Abutment 1 Footing (Br. #57-1215)
Spread Footing Design Information Provided by the Structure Designer, AECOM

Support Location	Bottom of Footing Elevation	Footing Size		Permissible Settlement Under Service Load	Service Limit State I						
					Total Load				Permanent Load		
					Vertical Load	Effective Dimensions		Horizontal Load in Long. Direction	Vertical Load	Effective Dimensions	
		B	L			B'	L'			B'	L'
Abutment 1	523.5 ft	14.0 ft	29.0 ft	1 in	1274 kips	11.1 ft	29.0 ft	352 kips	988 kips	10.7 ft	29.0 ft

The recommended Allowable Bearing Capacity and bottom of footing elevation are listed below in Table 8.

Table 8: Foundation Design Recommendations for Abutment 1 Spread Footing (Br. #57-1215)

Support Location	Footing Size		Bottom of Footing Elevation	Minimum Footing Embedment Depth	Total Permissible Support Settlement	WSD (LRFD Service I Limit State Load Combination)	
	B'	L'				Permissible Gross Contact Stress	Allowable Gross Bearing Capacity
Abutment 1	11.1 ft	29.0 ft	523.5 ft	5.0 ft	1.0 in	N/A*	10.0 ksf

Notes: 1) Recommendations are based on the foundation geometry and the load provided by AECOM in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2) See MTD 4-1 for definitions and applications of the recommended design parameters.
*Settlement is N/A due to bottom of footing founded on dense sedimentary formational material.

Hillery Drive Southbound On-Ramp (HDR-3) (Br. No. 57-1216)

At Abutment 1 support location spread footings are recommended for support. The Abutment 1 footing may be supported on the underlying undisturbed sedimentary formational material. Table 9, below, presents the Abutment 1 spread footing design information provided by the structure designer, AECOM.

Table 9: Abutment 1 Footing (Br. #57-1216)
Spread Footing Design Information Provided by the Structure Designer, AECOM

Support Location	Bottom of Footing Elevation	Footing Size		Permissible Settlement Under Service Load	Service Limit State I						
					Total Load				Permanent Load		
					Vertical Load	Effective Dimensions		Horizontal Load in Long. Direction	Vertical Load	Effective Dimensions	
		B	L			B'	L'			B'	L'
Abutment 1	522.5 ft	14.0 ft	29.0 ft	1 in	1334 kips	10.6 ft	29.0 ft	443 kips	1122 kips	10.0 ft	29.0 ft

The recommended Allowable Bearing Capacity and bottom of footing elevation are listed below in Table 10.

Table 10: Foundation Design Recommendations for Abutment 1 Spread Footing (Br. #57-1216)

Support Location	Footing Size		Bottom of Footing Elevation	Minimum Footing Embedment Depth	Total Permissible Support Settlement	WSD (LRFD Service I Limit State Load Combination)	
	B'	L'				Permissible Gross Contact Stress	Allowable Gross Bearing Capacity
Abutment 1	10.6 ft	29.0 ft	522.5 ft	5.0 ft	1.0 in	N/A*	10.0 ksf

Notes: 1) Recommendations are based on the foundation geometry and the load provided by AECOM in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2) See MTD 4-1 for definitions and applications of the recommended design parameters.
*Settlement is N/A due to bottom of footing founded on dense sedimentary formational material.

Hillery Drive Southbound Off-Ramp (HDR-4) (Br. No. 57-1217)

At Abutment 3 support location spread footings are recommended for support. The Abutment 3 footing may be supported on the underlying undisturbed sedimentary formational material. Table 11, below, presents the Abutment 3 spread footing design information provided by the structure designer, AECOM.

Table 11: Abutment 3 Footing (Br. #57-1217)
Spread Footing Design Information Provided by the Structure Designer, AECOM

Support Location	Bottom of Footing Elevation	Footing Size		Permissible Settlement Under Service Load	Service Limit State I						
					Total Load				Permanent Load		
					Vertical Load	Effective Dimensions		Horizontal Load in Long. Direction	Vertical Load	Effective Dimensions	
		B	L			B'	L'			B'	L'
Abutment 3	524.8 ft	14.0 ft	29.0 ft	1 in	1424 kips	12.1 ft	29.0 ft	400 kips	1212 kips	11.7 ft	29.0 ft

The recommended Allowable Bearing Capacity and bottom of footing elevation are listed below in Table 12.

Table 12: Foundation Design Recommendations for Abutment 3 Spread Footing (Br. #57-1217)

Support Location	Footing Size		Bottom of Footing Elevation	Minimum Footing Embedment Depth	Total Permissible Support Settlement	WSD (LRFD Service I Limit State Load Combination)	
	B'	L'				Permissible Gross Contact Stress	Allowable Gross Bearing Capacity
Abutment 3	12.1 ft	29.0 ft	524.8 ft	5.0 ft	1.0 in	N/A*	10.0 ksf

Notes: 1) Recommendations are based on the foundation geometry and the load provided by AECOM in the Foundation Design Data Sheet. The footing contact area is taken as equal to the effective footing area, where applicable.
2) See MTD 4-1 for definitions and applications of the recommended design parameters.
*Settlement is N/A due to bottom of footing founded on dense sedimentary formational material.

The recommended Allowable Gross Bearing Capacities to be used for design, provided in Tables 4, 6, 8, 10 and 12, above, are based on the following design criteria:

1) The spread footings have minimum widths (B) as shown in Tables 3, 5, 7, 9, and 11.

2) The spread footings are to be constructed at or below the recommended bottom of footing elevations shown in Tables 4, 6, 8, 10 and 12.

If any of the above minimum footing widths or embedment depths are reduced, or bottom of footing elevations raised, the Office of Geotechnical Design-South 2, Branch B, is to be contacted for reevaluation.

Bent Support Locations

Hillery Drive O.C. (Main Access Ramp, Br. No. 57-1213)

At Bents 2 through 7 support locations, Cast-In-Drilled-Hole (CIDH) piles may be used for support of the proposed structure. Tables 13 and 14, below, show the foundation design information provided by the consultant AECOM.

Table 13: Hillery Drive O.C. (Main Access Ramp, Br. #57-1213)
General Foundation Information Provided by Structure Designer, AECOM

Support Location	Pile Type	Finished Grade Elevation	Pile Cut-off Elevation	Permissible Settlement Under Service Load
Bent 2	72 in Type I CIDH	525.0 ft	513.6 ft	1 in
Bent 3	72 in Type I CIDH	528.9 ft	521.4 ft	1 in
Bent 4	72 in Type I CIDH	532.7 ft	523.1 ft	1 in
Bent 5L	90 in Type II CIDH	531.7 ft	527.9 ft	1 in
Bent 5R	90 in Type II CIDH	531.2 ft	527.9 ft	1 in
Bent 6L	90 in Type II CIDH	529.1 ft	527.1 ft	1 in
Bent 6R	90 in Type II CIDH	529.0 ft	527.1 ft	1 in
Bent 7L	90 in Type II CIDH	529.8 ft	526.0 ft	1 in
Bent 7R	90 in Type II CIDH	529.7 ft	526.0 ft	1 in

Table 14: Hillery Drive O.C. (Main Access Ramp, Br. #57-1213)
Factored Loads Provided by Structure Designer, AECOM

Support Location	Service 1 Limit State			Strength Limit State (Controlling Group)				Extreme Event Limit State (Controlling Group)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Bent 2	2445 kips	2445 kips	1970 kips	3420 kips	3420 kips	0	0	2035 kips	2035 kips	0	0
Bent 3	2895 kips	2895 kips	2385 kips	3970 kips	3970 kips	0	0	2455 kips	2455 kips	0	0
Bent 4	3030 kips	3030 kips	2525 kips	4140 kips	4140 kips	0	0	2690 kips	2690 kips	0	0
Bent 5L	1465 kips	1465 kips	2415 kips	2410 kips	2410 kips	0	0	1250 kips	1250 kips	0	0
Bent 5R	1465 kips	1465 kips	2415 kips	2410 kips	2410 kips	0	0	1250 kips	1250 kips	0	0
Bent 6L	1370 kips	1370 kips	2255 kips	2270 kips	2270 kips	0	0	1190 kips	1190 kips	0	0
Bent 6R	1370 kips	1370 kips	2255 kips	2270 kips	2270 kips	0	0	1190 kips	1190 kips	0	0
Bent 7L	1320 kips	1320 kips	2185 kips	2160 kips	2160 kips	0	0	1235 kips	1235 kips	0	0
Bent 7R	1320 kips	1320 kips	2185 kips	2160 kips	2160 kips	0	0	1235 kips	1235 kips	0	0

The specified pile tip elevations for Bent 2 through Bent 7 CIDH piles are shown below in Table 15.

Table 15: Hillery Drive O.C. (Main Access Ramp, Br. #57-1213)
Foundation Design Recommendations for Bent 2 through Bent 7 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 2	72 in Type I CIDH	513.6 ft	2450 kips	1 in	4890 kips	0	2040 kips	0	439.0 ft (a-I)	439.0 ft
Bent 3	72 in Type I CIDH	521.4 ft	2900 kips	1 in	5680 kips	0	2460 kips	0	438.0 ft (a-I)	438.0 ft
Bent 4	72 in Type I CIDH	523.1 ft	3030 kips	1 in	5920 kips	0	2690 kips	0	439.0 ft (a-I)	439.0 ft
Bent 5L	90 in Type II CIDH	527.9 ft	1470 kips	1 in	3450 kips	0	1250 kips	0	477.0 ft (a-I)	477.0 ft

Table 15 (continued): Hillery Drive O.C. (Main Access Ramp, Br. #57-1213)
Foundation Design Recommendations for Bent 2 through Bent 7 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 5R	90 in Type II CIDH	527.9 ft	1470 kips	1 in	3450 kips	0	1250 kips	0	477.0 ft (a-I)	477.0 ft
Bent 6L	90 in Type II CIDH	527.1 ft	1370 kips	1 in	3250 kips	0	1190 kips	0	475.0 ft (a-I)	475.0 ft
Bent 6R	90 in Type II CIDH	527.1 ft	1370 kips	1 in	3250 kips	0	1190 kips	0	475.0 ft (a-I)	475.0 ft
Bent 7L	90 in Type II CIDH	526.0 ft	1320 kips	1 in	3090 kips	0	1240 kips	0	479.0 ft (a-I)	479.0 ft
Bent 7R	90 in Type II CIDH	526.0 ft	1320 kips	1 in	3090 kips	0	1240 kips	0	479.0 ft (a-I)	479.0 ft

Note: Design tip elevations are controlled by (a-1) Compression (Strength Limit)

Hillery Drive Northbound On-Ramp (HDR-1) (Br. No. 57-1214)

At Bents 1 and 2 support locations, Cast-In-Drilled-Hole (CIDH) piles may be used for support of the proposed structure. Tables 16 and 17, below, show the foundation design information provided by the consultant AECOM.

Table 16: Hillery Drive Northbound On-Ramp (HDR-1), Br. #57-1214
General Foundation Information Provided by Structure Designer, AECOM

Support Location	Pile Type	Finished Grade Elevation	Pile Cut-off Elevation	Permissible Settlement Under Service Load
Bent 1	90 in Type II CIDH	529.9 ft	528.0 ft	1 in
Bent 2	90 in Type II CIDH	530.0 ft	524.8 ft	1 in

Table 17: Hillery Drive Northbound On-Ramp (HDR-1), Br. #57-1214
Factored Loads Provided by Structure Designer, AECOM

Support Location	Service 1 Limit State			Strength Limit State (Controlling Group)				Extreme Event Limit State (Controlling Group)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Bent 1	1292 kips	1292 kips	974 kips	1925 kips	1925 kips	0	0	1217 kips	1217 kips	0	0
Bent 2	1187 kips	1187 kips	881 kips	1753 kips	1753 kips	0	0	1061 kips	1061 kips	0	0

The specified pile tip elevations for Bents 1 and 2 CIDH piles are shown below in Table 18.

Table 18: Hillery Drive Northbound On-Ramp (HDR-1), Br. #57-1214
Foundation Design Recommendations for Bents 1 and 2 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 1	90 in Type II CIDH	528.0 ft	1300 kips	1 in	2750 kips	0	1220 kips	0	484.0 ft (a-I)	484.0 ft
Bent 2	90 in Type II CIDH	524.8 ft	1190 kips	1 in	2510 kips	0	1070 kips	0	482.0 ft (a-I)	482.0 ft

Note: Design tip elevations are controlled by (a-1) Compression (Strength Limit)

Hillery Drive Northbound Off-Ramp (HDR-2) (Br. No. 57-1215)

At Bents 2 and 3 support locations, (CIDH) piles may be used for support of the proposed structure. Tables 19 and 20, below, show the foundation design information provided by the consultant AECOM.

Table 19: Hillery Drive Northbound Off-Ramp (HDR-2), Br. #57-1215
General Foundation Information Provided by Structure Designer, AECOM

Support Location	Pile Type	Finished Grade Elevation	Pile Cut-off Elevation	Permissible Settlement Under Service Load
Bent 2	90 in Type II CIDH	529.0 ft	523.3 ft	1 in
Bent 3	90 in Type II CIDH	529.5 ft	527.0 ft	1 in

Table 20: Hillery Drive Northbound Off-Ramp (HDR-2), Br. #57-1215
Factored Loads Provided by Structure Designer, AECOM

Support Location	Service 1 Limit State			Strength Limit State (Controlling Group)				Extreme Event Limit State (Controlling Group)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Bent 2	1191 kips	1191 kips	885 kips	1859 kips	1859 kips	0	0	1055 kips	1055 kips	0	0
Bent 3	1297 kips	1297 kips	979 kips	1930 kips	1930 kips	0	0	1211 kips	1211 kips	0	0

The specified pile tip elevations for Bents 2 and 3 CIDH piles are shown below in Table 21.

Table 21: Hillery Drive Northbound Off-Ramp (HDR-2), Br. #57-1215
Foundation Design Recommendations for Bents 1 and 2 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 2	90 in Type II CIDH	523.3 ft	1200 kips	1 in	2660 kips	0	1060 kips	0	479.0 ft (a-I)	479.0 ft
Bent 3	90 in Type II CIDH	527.0 ft	1300 kips	1 in	2760 kips	0	1220 kips	0	482.0 ft (a-I)	482.0 ft

Note: Design tip elevations are controlled by (a-1) Compression (Strength Limit)

Hillery Drive Southbound On-Ramp (HDR-3) (Br. No. 57-1216)

At Bents 2 and 3 support locations, Cast-In-Drilled-Hole (CIDH) piles may be used for support of the proposed structure. Tables 22 and 23, below, show the foundation design information provided by the consultant AECOM.

Table 22: Hillery Drive Southbound On-Ramp (HDR-3), Br. #57-1216
General Foundation Information Provided by Structure Designer, AECOM

Support Location	Pile Type	Finished Grade Elevation	Pile Cut-off Elevation	Permissible Settlement Under Service Load
Bent 2	90 in Type II CIDH	528.3 ft	522.0 ft	1 in
Bent 3	90 in Type II CIDH	528.8 ft	526.0 ft	1 in

Table 23: Hillery Drive Southbound On-Ramp (HDR-3), Br. #57-1216
Factored Loads Provided by Structure Designer, AECOM

Support Location	Service 1 Limit State			Strength Limit State (Controlling Group)				Extreme Event Limit State (Controlling Group)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Bent 2	1176 kips	1176 kips	860 kips	1860 kips	1860 kips	0	0	1018 kips	1018 kips	0	0
Bent 3	1346 kips	1346 kips	1024 kips	1988 kips	1988 kips	0	0	1208 kips	1208 kips	0	0

The specified pile tip elevations for Bents 2 and 3 CIDH piles are shown below in Table 24.

Table 24: Hillery Drive Southbound On-Ramp (HDR-3), Br. #57-1216
Foundation Design Recommendations for Bents 2 and 3 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 2	90 in Type II CIDH	522.0 ft	1180 kips	1 in	2660 kips	0	1020 kips	0	478.0 ft (a-I)	478.0 ft
Bent 3	90 in Type II CIDH	526.0 ft	1350 kips	1 in	2840 kips	0	1210 kips	0	482.0 ft (a-I)	482.0 ft

Note: Design tip elevations are controlled by (a-1) Compression (Strength Limit)

Hillery Drive Southbound Off-Ramp (HDR-4) (Br. No. 57-1217)

At Bents 2 and 3 support locations, Cast-In-Drilled-Hole (CIDH) piles may be used for support of the proposed structure. Tables 25 and 26, below, show the foundation design information provided by the consultant AECOM.

Table 25: Hillery Drive Southbound Off-Ramp (HDR-4), Br. #57-1217
General Foundation Information Provided by Structure Designer, AECOM

Support Location	Pile Type	Finished Grade Elevation	Pile Cut-off Elevation	Permissible Settlement Under Service Load
Bent 1	90 in Type II CIDH	529.2 ft	527.0 ft	1 in
Bent 2	90 in Type II CIDH	529.5 ft	523.3 ft	1 in

Table 26: Hillery Drive Southbound Off-Ramp (HDR-4), Br. #57-1217
Factored Loads Provided by Structure Designer, AECOM

Support Location	Service 1 Limit State			Strength Limit State (Controlling Group)				Extreme Event Limit State (Controlling Group)			
	Total Loads		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max Per Pile	Per Support	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile
Bent 1	1299 kips	1299 kips	981 kips	1932 kips	1932 kips	0	0	1208 kips	1208 kips	0	0
Bent 2	1194 kips	1194 kips	888 kips	1860 kips	1860 kips	0	0	1053 kips	1053 kips	0	0

The specified pile tip elevations for Bents 2 and 3 CIDH piles are shown below in Table 27.

Table 27: Hillery Drive Southbound Off-Ramp (HDR-4), Br. #57-1217
Foundation Design Recommendations for Bents 2 and 3 Locations

Support Location	Pile Type	Cut-Off Elevation	Service-I Limit State Load per Column	Total Permissible Support Settlement	Required Nominal Resistance				Design Tip Elevation	Specified Tip Elevation
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 1	90 in Type II CIDH	527.0 ft	1300 kips	1 in	2760 kips	0	1210 kips	0	485.0 ft (a-I)	485.0 ft
Bent 2	90 in Type II CIDH	523.3 ft	1200 kips	1 in	2660 kips	0	1060 kips	0	477.0 ft (a-I)	477.0 ft

Note: Design tip elevations are controlled by (a-1) Compression (Strength Limit)

The Pile Data Tables for the proposed Hillery Drive O.C. (Main Access Ramp, Br. #57-1213) and its connecting Direct Access Ramps: Northbound On/Off-Ramp (Br. No. 57-1214 & 57-1215) and Southbound On/Off-Ramps (Br. No. 57-1216 & 57-1217), are presented in Tables 28 through 32, below. The ultimate geotechnical pile capacity for the CIDH piles will meet or exceed the required nominal resistance in compression.

Table 28: Pile Data Table
Hillery Drive O.C. (Main Access Ramp, Br. #57-1213)

Location	Pile Type	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
		Compression	Tension		
Bent 2	72 in Type I CIDH	4890 kips	0	439.0 ft (a-I)	439.0 ft
Bent 3	72 in Type I CIDH	5680 kips	0	438.0 ft (a-I)	438.0 ft
Bent 4	72 in Type I CIDH	5920 kips	0	439.0 ft (a-I)	439.0 ft
Bent 5L	90 in Type II CIDH	3450 kips	0	477.0 ft (a-I)	477.0 ft
Bent 5R	90 in Type II CIDH	3450 kips	0	477.0 ft (a-I)	477.0 ft
Bent 6L	90 in Type II CIDH	3250 kips	0	475.0 ft (a-I)	475.0 ft
Bent 6R	90 in Type II CIDH	3250 kips	0	475.0 ft (a-I)	475.0 ft
Bent 7L	90 in Type II CIDH	3090 kips	0	479.0 ft (a-I)	479.0 ft
Bent 7R	90 in Type II CIDH	3090 kips	0	479.0 ft (a-I)	479.0 ft

Notes: 1) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit)

Table 29: Pile Data Table
Hillery Drive Northbound On-Ramp (HDR-1), Br. #57-1214

Location	Pile Type	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
		Compression	Tension		
Bent 1	90 in Type II CIDH	2750 kips	0	484.0 ft (a-I)	484.0 ft
Bent 2	90 in Type II CIDH	2510 kips	0	482.0 ft (a-I)	482.0 ft

Notes: 1) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit)

Table 30: Pile Data Table
Hillery Drive Northbound Off-Ramp (HDR-2), Br. #57-1215

Location	Pile Type	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
		Compression	Tension		
Bent 2	90 in Type II CIDH	2660 kips	0	479.0 ft (a-I)	479.0 ft
Bent 3	90 in Type II CIDH	2760 kips	0	482.0 ft (a-I)	482.0 ft

Notes: 1) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit)

Table 31: Pile Data Table
Hillery Drive Southbound On-Ramp (HDR-3), Br. #57-1216

Location	Pile Type	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
		Compression	Tension		
Bent 2	90 in Type II CIDH	2660 kips	0	478.0 ft (a-I)	478.0 ft
Bent 3	90 in Type II CIDH	2840 kips	0	482.0 ft (a-I)	482.0 ft

Notes: 1) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit)

Table 32: Pile Data Table
Hillery Drive Southbound Off-Ramp (HDR-4), Br. #57-1217

Location	Pile Type	Nominal Resistance		Design Tip Elevation	Specified Tip Elevation
		Compression	Tension		
Bent 1	90 in Type II CIDH	2760 kips	0	485.0 ft (a-I)	485.0 ft
Bent 2	90 in Type II CIDH	2660 kips	0	477.0 ft (a-I)	477.0 ft

Notes: 1) Design tip elevations for Bents are controlled by: (a-I) Compression (Strength Limit)

General Notes:

1) All abutment and bent CIDH pile locations are to be plotted in plan view on the Log of Test Borings as stated in "Memo to Designers" 4-2. The plotting of support locations should be made prior to requesting a final foundation review.

2) When applicable, the structure engineer shall show on the plans, in the pile data table, the design pile tip elevation required to meet the lateral load demands. If the design pile tip elevation required to meet lateral load demands exceeds the specified pile tip elevations given within this report, the Office of Geotechnical Design-South 2, Branch B shall be contacted for further recommendations.

3) At the abutment locations of the Main Access Ramp, as well as the Northbound and Southbound Direct Access Ramps, it is possible that the contractor may encounter ground water while excavating to the bottom of footing elevations. Structure Excavation Type "D" is to be shown on the plans at these locations.

Construction Considerations:

1) Due to the irregular presence of water at shallow depths, as described in the Ground Water section, above, it is possible that water may be encountered during excavation and construction of the footings for the abutment supports, as well as the Cast-In-Drilled-Hole (CIDH) piles. Therefore, the contractor should expect, and be prepared to deal with, wet footing excavations as well as the need to control water flowing into the CIDH pile borings. Ground water levels indicated on the LOTB sheets reflect the measured ground water levels at the time of the Caltrans investigation. At the time of construction, the ground water surface elevations may be higher or lower than those shown on the LOTB due to seasonal fluctuations.

Cores Samples

1) Core samples from the 2009 Caltrans foundation investigation are available for viewing by bidders at the California Department of Transportation, Transportation Laboratory, 5900 Folsom Blvd., Sacramento, CA. The bidders are to allow the State five (5) working days to prepare and display the cores.

Spread Footings

1) At the abutment support locations, concrete for the proposed support footings shall be placed neat against the undisturbed sedimentary formational material at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the bottom of the footing excavation shall be extended down at 0.5 ft intervals until undisturbed formational material is observed and approved by the Engineer. The subexcavated material is to be replaced with either lean or Class 3 concrete.

2) At the abutment support locations, the excavations are to be inspected and approved by a representative of the Office of Geotechnical Design-South 2, Branch B, prior to placing any steel and/or structural, lean or Class 3 concrete. The required inspection is to verify that the concrete is placed on top undisturbed sedimentary formational material. Once the excavation has been completed to the specified elevations, the contractor is to allow the Office of Geotechnical Design–South 2, Branch B, five (5) working days to perform the inspection. The structures representative is to provide the Office of Geotechnical Design–South 2, Branch B, a one-week notification prior to beginning the five-day contractor waiting period.

CIDH Piles

1) Beneath the proposed bridge site, the sedimentary formational material mainly consists of interbedded layers of sandstone and cobble conglomerate. The cobble conglomerate consists of very hard, rounded igneous and metamorphic clasts within a very soft, poorly indurated, non-cemented gravel and sand matrix. In the sandstone and conglomerate sedimentary formational units, the contractor should anticipate varying drilling conditions (alternating soft and hard drilling) across all the pier locations. The variations in conditions (described above) will occur from one pile location to the next pile location. The contractor should also be prepared for potential caving conditions within the conglomerate formational unit. The amount of difficulty and caving the contractor will experience will be dependent upon the methods and means the contractor chooses to use to construct the CIDH piles.

2) Should the contractor choose to use slurry displacement methods to construct the CIDH piles, the contractor should use care while drilling the shafts for the piles. Due to the poorly indurated, non-cemented nature of portions of the conglomerate formational unit, rapid insertion and removal of the drilling tools during the drilling process can cause excessive scouring and caving of the walls of the drilled shaft.

3) Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity of all CIDH piles is based on skin friction and no end-bearing was considered. The skin friction zones used to calculate geotechnical capacity of the CIDH piles are summarized below in Tables 33 through 37, below.

Table 33: CIDH Pile Skin Friction Zone Elevations
Hillery Drive OC (Main Access Ramp, Br. #57-1213)

Location	Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
Bent 2	507.6 ft	444.0 ft
Bent 3	514.0 ft	443.0 ft
Bent 4	517.1 ft	444.0 ft
Bent 5L	520.9 ft	482.0 ft
Bent 5R	520.9 ft	482.0 ft
Bent 6L	520.0 ft	480.0 ft
Bent 6R	520.0 ft	480.0 ft
Bent 7L	519.0 ft	484.0 ft
Bent 7R	519.0 ft	484.0 ft

Table 34: CIDH Pile Skin Friction Zone Elevations
Hillery Drive Northbound On-Ramp (HDR-1), Br. #57-1214

Location	Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
Bent 1	521.0 ft	489.0 ft
Bent 2	517.8 ft	487.0 ft

Table 35: CIDH Pile Skin Friction Zone Elevations
Hillery Drive Northbound Off-Ramp (HDR-2), Br. #57-1215

Location	Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
Bent 2	516.3 ft	484.0 ft
Bent 3	520.0 ft	487.0 ft

Table 36: CIDH Pile Skin Friction Zone Elevations
Hillery Drive Southbound On-Ramp (HDR-3), Br. #57-1216

Location	Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
Bent 2	515.0 ft	483.0 ft
Bent 3	519.0 ft	487.0 ft

Table 37: CIDH Pile Skin Friction Zone Elevations
Hillery Drive Southbound Off-Ramp (HDR-4), Br. #57-1217

Location	Skin Friction Zone Start Elevation	Skin Friction Zone End Elevation
Bent 1	520.0 ft	490.0 ft
Bent 2	516.3 ft	482.0 ft

4) If the CIDH piles are to be constructed using slurry displacement method, the slurry shall consist of mineral or synthetic slurry only. Use of water shall not be allowed as slurry.

5) At the Type II shaft locations, if a required or optional construction joint is shown on the contract plans, at the bottom of the column cage, the Standard Special Provision, SSP 49-310_E_B03-13-09, needs to be included in the Special Provisions. The contractor is to install a permanent steel casing and the casing is to extend 5 feet below the bottom of the column cage elevation. Because the elevation of the beginning of the skin friction zone is above the elevation of the bottom of the column cages, the permanent steel casing is to consist of a corrugated metal pipe (CMP). Impact and or vibratory hammers are not to be allowed to install the CMP. The use of "Slurry Cement Backfill", item # 15 of SSP 49-310_E_B03-13-09 is to be deleted and Grout is to be used to fill the annular space between to CMP and the borehole wall.

The recommendations contained in this report are based on specific project information regarding structure type, support locations, and design loads that have been provided by the consultant AECOM. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B, should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565, D. Te-Ming Liao, (916) 227-5756, or Mark DeSalvatore, (916) 227-5391, at the Office of Geotechnical Design-South 2, Branch B.

Prepared by: Date: 9/1/10

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cc: R.E. Pending File
Kelly Holden - Specs & Estimates (4)
Andrew Rice – District 11 (Project Manager)
Art Padilla – District 11 Materials Engineer
Mike Crull - AECOM
Abbas Abghari – OGDS-2
Mark Willian – GS Corporate

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. NORBERT GEE
Office of Special Funded Projects
Division of Engineering Services

Date: May 24, 2010

File: 11-2T0951
Type 1 Retaining Walls for
Hillery Drive O.C.
Main Access Ramp: Br. #57-1213
and Direct Access Ramps:
Br. #57-1214, Br. #57-1215,
Br. #57-1216, Br. #57-1217

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 2 MS #5
Design Branch B

Subject: 2nd Revised Foundation Report

This 2nd Revised Foundation Report supercedes the "original" Foundation Report, dated April 26, 2010, and the Revised Foundation Report, dated April 30, 2010, for the proposed Type 1 Retaining Walls associated with the Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213) and its connecting Direct Access Ramps (Br. No. 57-1214, 57-1215, 57-1216, and 57-1217). This 2nd Revised Foundation Report reflects changes in stationing and bottom of footing elevations for the proposed structures. The Office of Geotechnical Design-South 2, Branch B (OGDS2B) completed a foundation investigation pursuant to a request by the Office of Special Funded Projects (OSFP) for foundation recommendations for the proposed structures. The Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213) and its connecting Direct Access Ramps (Br. No. 57-1214, 57-1215, 57-1216, and 57-1217) are being designed by the consultant AECOM which has provided the Office of Geotechnical Design, South-2 the design information used in this report to provide foundation recommendations.

The following foundation recommendations are based on subsurface information gathered during a foundation investigation conducted from April 2009 through October 2009. With regards to the current foundation recommendations given in this report, all elevations referenced within this report and shown on the Log of Test Borings (LOTB) sheets are based on the North American Vertical Datum (NAVD 88).

Project Description/History

The proposed bridge sites for the five structures are located on Interstate 15 in the northern part of the city of San Diego. These structures are part of the I-15 Managed Lanes Project aimed at improving traffic mobility on Route 15 between the Escondido area and San Diego. The proposed structures will allow access to the I-15 managed lanes from Hillery Drive.

The Hillery Drive O.C. Main Access Ramp will consist of a six span, cast-in-place, reinforced concrete and pre-stressed, box girder type structure measuring approximately 745 feet long and 43 feet wide.

The proposed Hillery Drive Direct Access Ramps: Northbound and Southbound On-Ramp and Off-Ramp structures (Br. Nos. 57-1214, 57-1215, 57-1216 and 57-1217), which measure 182.3

ft long and 26.6 ft wide, will provide commuters, using proposed managed lanes, access to the Hillery Drive O.C. Main Access Ramp. The proposed Northbound and Southbound On-Ramp and Off-Ramp structures will each consist of a three span, cast-in-place, pre-stressed, box girder type structure, which will connect to the Main Access Ramp by seatless hinges.

Geology

The foundation investigation completed in October 2009 consisted of 24 mud rotary borings (Borings R-09-001 through R-09-024) and 7 auger borings (borings A-09-025 through A-09-031).

The proposed bridge site is located in an area of ancient sedimentary marine terraces cut by creeks, which generally flow east to west. The geologic map "Geology of the Poway Quadrangle, San Diego County, California (1975)" indicates that the site is underlain by the Quaternary Lindavista Formation at the surface, which is described as sandstone and conglomerate. Below the Lindavista Formation lie the sedimentary facies of the Tertiary Poway Group, specifically the Stadium Conglomerate.

The 2009 foundation investigation revealed the site is generally underlain by sedimentary formational material consisting of interbedded layers of sandstone and cobble conglomerate. The sandstone is typically very soft and poorly indurated. The cobble conglomerate consists of rounded igneous and metamorphic clasts within a very soft, poorly indurated gravel and sand matrix.

For more specific details regarding the sedimentary formation descriptions from the 2009 foundation investigation, refer to the LOTB sheets for the proposed new bridges.

Ground Water

At the proposed bridge site, ground water was attempted to be measured in some of the borings drilled for both the Main Access Ramp as well as the Direct Access Ramps and associated retaining walls. Generally, the ground water was determined to be relatively deep, however, in boring R-09-005, ground water was measured at two feet below the ground surface on July 9, 2009. To determine if there was perched ground water in the area, seven auger borings were drilled from October 6 to 8, 2009 across the site. Ground water was not encountered in any of the auger borings. At the nearby Mira Mesa Rd. OC, during the 2000 subsurface investigation for the widening of this structure, water was found to be flowing just beneath the highway pavement. This water was seeping/flowing out from beneath both bridge abutments, causing the embankments to erode and flow down onto the adjacent city sidewalks below. It was observed that the water would be present and flowing, and then not present, at irregular intervals. Due to this irregular presence of water, the abutments for the widening were supported on deep foundations. Additionally, during construction of the most recent widening of the Mira Mesa Rd. OC, water was also encountered at shallow depths beneath the pavement, requiring a Contract Change Order (CCO) to divert water away from the site. The source of the flowing ground water still has not been determined. Due to this irregular presence of water at shallow depths, it is possible that water may be encountered during construction of the footings for the retaining walls.

Recorded ground water information from the 2009 subsurface investigation is presented in Table 1, below.

Table 1 – Ground Water Summary

Boring No.	Top of Boring Elev. (ft)	Total Depth of Boring (ft)	Date Measured	GWS depth (ft)	GWS elev. (ft)
R-09-021	525.2	115.5	2/4/2010	105.1	420.1
R-09-020	529.6	120.0	2/4/2010	103.2	426.4
R-09-003	528.4	131.9	7/9/2009	108.6	419.8
R-09-014	529.3	90.0	7/9/2009	Dry	Dry
R-09-006	528.2	60.7	7/9/2009	Dry	Dry
R-09-016	529.1	65.5	7/9/2009	Dry	Dry
R-09-005	524.4	25.7	7/9/2009	2.0	522.4

Measured ground water elevations are also shown on the LOTB sheets. Ground water levels indicated in this report and shown on the LOTB sheets reflect the measured ground water level in the borehole on the specified date. Ground water surface elevations are subject to seasonal fluctuations and will be encountered at higher or lower elevations depending on seasonal conditions.

Scour Potential

There is no scour potential at the site, since the structures do not span any watercourse.

Corrosion

Corrosion test results for soil samples collected from borings R-09-001, R-09-004, R-09-017, are shown below in Table 2. The site is considered corrosive by current Caltrans standards. Reinforced concrete (including piles) requires corrosion mitigation in accordance with *Bridge Design Specifications, Article 8.22.*

Table 2 – Corrosion Test Summary

Location	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
Boring R-09-001 (Elev. 521.5 – 505.7 ft)	4.71	1343	N/A	N/A
Boring R-09-004 (Elev. 528.8 – 478.8 ft)	5.16	1151	N/A	N/A
Boring R-09-017 (Elev. 525.7 – 500.7 ft)	6.03	779	219	151

Note: Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 2000 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Foundation Recommendations

The following recommendations are for the proposed Type 1 Retaining Walls located on the left and right sides of the Hillery Drive O.C. Main Access Ramp (Br. #57-1213) and its connecting

Direct Access Ramps: Northbound On/Off-Ramp (Br. No. 57-1214 & 57-1215) and Southbound On/Off-Ramps (Br. No. 57-1216 & 57-1217). Specific design information for the retaining walls was provided to Caltrans by the consultant AECOM on February 8, 2010 for the ramp structures, and on March 30, 2010 for the Hillery Drive O.C. Main Access Ramp structure. Updated information was also provided on May 23, 2010.

Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213)

The left and right side Type 1 Retaining Walls at the Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213) may be supported on spread footings founded on the underlying undisturbed sedimentary formational material. The recommendations for bottom of footing elevations and Allowable Bearing Capacities to be used for design are shown in Tables 3 and 4, below.

Table 3: Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213)
LEFT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "24L" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+43.57	Sta. 4+61.45	6	519.92	1.9	N/A
Sta. 4+61.45	Sta. 4+89.45	8	519.92	2.2	N/A
Sta. 4+89.45	Sta. 5+17.45	10	519.92	2.5	N/A
Sta. 5+17.45	Sta. 5+45.45	12	519.92	2.8	N/A
Sta. 5+45.45	Sta. 5+68.95	14	519.75	3.3	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Table 4: Hillery Drive O.C. Main Access Ramp (Br. No. 57-1213)
RIGHT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "24R" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+43.57	Sta. 4+61.48	6	519.92	1.9	N/A
Sta. 4+61.48	Sta. 4+89.48	8	519.92	2.2	N/A
Sta. 4+89.48	Sta. 5+17.48	10	519.92	2.5	N/A
Sta. 5+17.48	Sta. 5+45.48	12	519.92	2.8	N/A
Sta. 5+45.48	Sta. 5+68.98	14	519.75	3.3	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Hillery Drive Northbound On-Ramp (HDR-1) (Br. No. 57-1214)

The left and right side Type 1 Retaining Walls at the Hillery Drive Northbound On-Ramp (Br. No. 57-1214) may be supported on spread footings founded on the underlying undisturbed sedimentary formational material. The recommendations for bottom of footing elevations and Allowable Bearing Capacities to be used for design are shown in Tables 5 and 6, below.

Table 5: Hillery Drive Northbound On-Ramp (HDR-1) (Br. No. 57-1214)
LEFT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR1L" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 2+31.61	Sta. 2+41.61	20	526.50	4.3	N/A
Sta. 2+41.61	Sta. 2+81.61	18	526.50	4.0	N/A
Sta. 2+81.61	Sta. 3+21.61	16	526.50	3.5	N/A
Sta. 3+21.61	Sta. 3+61.61	14	526.50	3.3	N/A
Sta. 3+61.61	Sta. 4+01.61	12	526.67	2.8	N/A
Sta. 4+01.61	Sta. 4+41.61	10	526.67	2.5	N/A
Sta. 4+41.61	Sta. 4+93.61	8	526.67	2.2	N/A
Sta. 4+93.61	Sta. 5+23.05	6	526.67	1.9	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Table 6: Hillery Drive Northbound On-Ramp (HDR-1) (Br. No. 57-1214)
RIGHT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR1R" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 2+31.61	Sta. 2+41.61	20	526.50	4.3	N/A
Sta. 2+41.61	Sta. 2+81.61	18	526.50	4.0	N/A
Sta. 2+81.61	Sta. 3+21.61	16	526.50	3.5	N/A
Sta. 3+21.61	Sta. 3+61.61	14	526.50	3.3	N/A
Sta. 3+61.61	Sta. 4+01.61	12	526.67	2.8	N/A
Sta. 4+01.61	Sta. 4+41.61	10	526.67	2.5	N/A
Sta. 4+41.61	Sta. 4+93.61	8	526.67	2.2	N/A
Sta. 4+93.61	Sta. 5+22.78	6	526.67	1.9	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Hillery Drive Northbound Off-Ramp (HDR-2) (Br. No. 57-1215)

The left and right side Type 1 Retaining Walls at the Hillery Drive Northbound Off-Ramp (Br. No. 57-1215) may be supported on spread footings founded on the underlying undisturbed sedimentary formational material. The recommendations for bottom of footing elevations and Allowable Bearing Capacities to be used for design are shown in Tables 7 and 8, below.

Table 7: Hillery Drive Northbound Off-Ramp (HDR-2) (Br. No. 57-1215)
LEFT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR2L" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+98.00	Sta. 5+16.15	6	521.67	1.9	N/A
Sta. 5+16.15	Sta. 5+60.15	8	521.67	2.2	N/A
Sta. 5+60.15	Sta. 6+00.15	10	521.67	2.5	N/A
Sta. 6+00.15	Sta. 6+38.15	12	521.67	2.8	N/A
Sta. 6+38.15	Sta. 6+74.15	14	521.50	3.3	N/A
Sta. 6+74.15	Sta. 7+08.15	16	521.50	3.5	N/A
Sta. 7+08.15	Sta. 7+14.15	18	521.50	4.0	N/A
Sta. 7+14.15	Sta. 7+70.15	18	523.50	4.0	N/A
Sta. 7+70.15	Sta. 8+00.15	20	523.33	4.3	N/A
Sta. 8+00.15	Sta. 8+10.15	22	523.33	4.6	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Table 8: Hillery Drive Northbound Off-Ramp (HDR-2) (Br. No. 57-1215)
RIGHT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR2R" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+98.00	Sta. 5+25.86	6	521.67	1.9	N/A
Sta. 5+25.86	Sta. 5+69.86	8	521.67	2.2	N/A
Sta. 5+69.86	Sta. 6+09.86	10	521.67	2.5	N/A
Sta. 6+09.86	Sta. 6+47.86	12	521.67	2.8	N/A
Sta. 6+47.86	Sta. 6+83.86	14	521.50	3.3	N/A
Sta. 6+83.86	Sta. 7+13.86	16	521.50	3.5	N/A
Sta. 7+13.86	Sta. 7+45.86	16	523.50	3.5	N/A
Sta. 7+45.86	Sta. 7+69.86	18	523.50	4.0	N/A
Sta. 7+69.86	Sta. 7+99.86	20	523.33	4.3	N/A
Sta. 7+99.86	Sta. 8+09.86	22	523.33	4.6	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Hillery Drive Southbound On-Ramp (HDR-3) (Br. No. 57-1216)

The left and right side Type 1 Retaining Walls at the Hillery Drive Southbound On-Ramp (Br. No. 57-1216) may be supported on spread footings founded on the underlying undisturbed sedimentary formational material. The recommendations for bottom of footing elevations and Allowable Bearing Capacities to be used for design are shown in Tables 9 and 10, below.

Table 9: Hillery Drive Southbound On-Ramp (HDR-3) (Br. No. 57-1216)
LEFT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR3L" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+77.36	Sta. 5+10.42	6	520.92	1.9	N/A
Sta. 5+10.42	Sta. 5+54.42	8	520.92	2.2	N/A
Sta. 5+54.42	Sta. 5+94.42	10	520.92	2.5	N/A
Sta. 5+94.42	Sta. 6+32.42	12	520.92	2.8	N/A
Sta. 6+32.42	Sta. 6+43.42	14	520.75	3.3	N/A
Sta. 6+43.42	Sta. 6+92.42	14	522.50	3.3	N/A
Sta. 6+92.42	Sta. 7+22.42	16	522.50	3.5	N/A
Sta. 7+22.42	Sta. 7+52.42	18	522.50	4.0	N/A
Sta. 7+52.42	Sta. 7+82.42	20	522.50	4.3	N/A
Sta. 7+82.42	Sta. 8+11.42	22	522.33	4.6	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Table 10: Hillery Drive Southbound On-Ramp (HDR-3) (Br. No. 57-1216)
RIGHT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR3R" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 4+77.36	Sta. 4+99.11	6	520.92	1.9	N/A
Sta. 4+99.11	Sta. 5+43.11	8	520.92	2.2	N/A
Sta. 5+43.11	Sta. 5+83.11	10	520.92	2.5	N/A
Sta. 5+83.11	Sta. 6+23.11	12	520.92	2.8	N/A
Sta. 6+23.11	Sta. 6+43.11	14	520.75	3.3	N/A
Sta. 6+43.11	Sta. 6+87.11	14	522.50	3.3	N/A
Sta. 6+87.11	Sta. 7+19.11	16	522.50	3.5	N/A
Sta. 7+19.11	Sta. 7+49.11	18	522.50	4.0	N/A
Sta. 7+49.11	Sta. 7+77.11	20	522.33	4.3	N/A
Sta. 7+77.11	Sta. 8+01.11	22	522.33	4.6	N/A
Sta. 8+01.11	Sta. 8+11.11	24	522.17	4.9	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Hillery Drive Southbound Off-Ramp (HDR-4) (Br. No. 57-1217)

The left and right side Type 1 Retaining Walls at the Hillery Drive Southbound Off-Ramp (Br. No. 57-1217) may be supported on spread footings founded on the underlying undisturbed sedimentary formational material. The recommendations for bottom of footing elevations and Allowable Bearing Capacities to be used for design are shown in Tables 11 and 12, below.

Table 11: Hillery Drive Southbound Off-Ramp (HDR-4) (Br. No. 57-1217)
LEFT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR4L" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 2+25.87	Sta. 2+41.87	20	526.50	4.3	N/A
Sta. 2+41.87	Sta. 2+81.87	18	526.50	4.0	N/A
Sta. 2+81.87	Sta. 3+21.87	16	526.50	3.5	N/A
Sta. 3+21.87	Sta. 3+41.87	14	526.50	3.3	N/A
Sta. 3+41.87	Sta. 3+81.87	12	526.67	2.8	N/A
Sta. 3+81.87	Sta. 4+17.87	10	526.67	2.5	N/A
Sta. 4+17.87	Sta. 4+67.87	8	526.67	2.2	N/A
Sta. 4+67.87	Sta. 5+03.41	6	526.67	1.9	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

Table 12: Hillery Drive Southbound Off-Ramp (HDR-4) (Br. No. 57-1217)
RIGHT SIDE Type 1 Retaining Walls Spread Footing Data

Wall Locations (Referenced From "HDR4R" Line)		Design Height of Wall "H" (ft)	Bottom Footing Elevation (ft)	Recommended Bearing Limits (ksf)	
				WSD[1]	LFD[2]
Beginning Station (ft)	End Station (ft)			Allowable Bearing Capacity (q_{all})	Nominal Bearing Resistance (q_n)
Sta. 2+25.87	Sta. 2+41.87	20	526.50	4.3	N/A
Sta. 2+41.87	Sta. 2+81.87	18	526.50	4.0	N/A
Sta. 2+81.87	Sta. 3+21.87	16	526.50	3.5	N/A
Sta. 3+21.87	Sta. 3+41.87	14	526.50	3.3	N/A
Sta. 3+41.87	Sta. 3+81.87	12	526.67	2.8	N/A
Sta. 3+81.87	Sta. 4+31.87	10	526.67	2.5	N/A
Sta. 4+31.87	Sta. 4+81.87	8	526.67	2.2	N/A
Sta. 4+81.87	Sta. 5+03.15	6	526.67	1.9	N/A

Notes: **1)** Working Stress Design (WSD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Bearing Capacity, (q_{all}). **2)** Load Factor Design, (LFD). The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the Nominal Bearing Resistance, (q_n).

The recommended Allowable Bearing Capacities provided in Tables 3 through 12, above, are based upon the following design criteria:

1) All retaining walls will be Standard Type 1 retaining walls as shown in the "Standard Plans (May 2006)" on sheet B3-1 for Loading Case 1.

2) All spread footings shall be constructed at or below the recommended bottom of footing elevations as shown in Tables 3 through 12, above.

If any of the above vertical embedment depths are reduced, the Loading Case changed, or wall heights increased, the Office of Geotechnical Design-South 2, Branch B is to be contacted for reevaluation.

General Notes:

1) At the Type 1 Retaining Wall locations of the Northbound and Southbound Direct Access Ramps, it is possible that the contractor may encounter ground water while excavating to the bottom of footing elevations. Structure Excavation Type "D" is to be shown on the plans at these locations.

Construction Considerations:

1) Due to the irregular presence of water at shallow depths, as described in the Ground Water section, above, it is possible that water may be encountered during excavation and construction of the footings for the retaining walls. Therefore, the contractor should expect and be prepared to deal with wet footing excavations.

2) At all proposed Type 1 retaining wall locations the concrete for the retaining wall support footings shall be placed neat against the undisturbed formational material at the bottom of footing elevations. Should the bottom of footing excavations be disturbed, then the bottom of the footing excavations shall be extended down at 0.5 ft intervals until undisturbed formational material is observed and approved by the Engineer. The subexcavated material is to be replaced with lean concrete or structure backfill compacted to at least 95% relative compaction. The disturbed native material is not to be recompacted. If lean concrete is used to backfill the subexcavation, the contractor is to form a key-way in the top of the lean concrete to allow for construction of the retaining wall footing shear key.

The recommendations contained in this report are based on specific project information regarding structure type, support locations, and design loads that have been provided by the consultant AECOM. If any conceptual changes are made during final project design, the Office of Geotechnical Design-South 2, Design Branch B, should review those changes to determine if these foundation recommendations are still applicable. Any questions regarding the above recommendations should be directed to the attention of Erich Neupert, (916) 227-4565; D. Te-Ming Liao, (916) 227-5756; or Mark DeSalvatore, (916) 227-5391 at the Office of Geotechnical Design-South 2, Branch B.

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