FUGRO WEST, INC.

STATEWIDE A&E CONTRACT NO. 59A0284
TASK ORDER NO. 284016

TO

CALIFORNIA STATE DEPARTMENT OF TRANSPORTATION

FOR

RETAINING WALLS:
346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1 & 361R
RETAINING WALL FOUNDATION INVESTIGATIONS AND GEOTECHNICAL DESIGN REPORTS

OF

ROUTE 15/56 SEPARATION MANAGED LANES STAGE 1
MILE POST: 18.9/8.9 to 22.0/9.4
KILOMETER POST: 30.4/14.4 to 35.4/15.2

Expenditure Authorization	FA	AO
11-064811	1	185

November 2002

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December 6, 2002
Project No. 1394.013

California State Department of Transportation
Division of Engineering Services
Geotechnical Services - South
7177 Opportunity Road
San Diego, California 92111

Attention: Mr. Jeff Tesar, C.E.G.

Subject: Geotechnical Design Report: Retaining Walls 346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1, and 361R; Route 15/56 Separation Managed Lanes Stage 1 (EA 11-064811, Mile Post: 1 8.9/8.9 to 22.0/9.4, Kilometer Post: 30.4/14.4 to 35.4/15.2); County of San Diego, California

Dear Mr. Tesar:

Fugro is pleased to present this geotechnical design report for the above referenced retaining walls that will be constructed along Interstate 15 between Carmel Mountain Road and Camino Del Norte as part of the Managed Lanes project in San Diego, California. The attached report presents the results of our field exploration, laboratory testing, geotechnical evaluations, and recommendations for design and construction of the proposed retaining walls.

We appreciate the opportunity to provide our geotechnical services for this project. Please call if you have any questions about this report.

Sincerely,

FUGRO WEST, INC.

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Project Engineer

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Issue 01

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1.0 INTRODUCTION

Fugro West, Inc. (Fugro) has prepared this report for the project, route, mile post, Expenditure Authorization (EA) number, and Caltrans district (EA prefix) listed below:

Retaining Walls Foundation Investigations and Geotechnical Design Reports
Route 15/56 Separation Managed Lanes Stage 1
Retaining Walls: 346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1 and 361R
County of San Diego
Mile Post: 18.9/8.9 to 22.0/9.4
Kilometer Post: 30.4/14.4 to 35.4/15.2

Expenditure Authorization	FA	AO
11-064811	1	185

1.1 GENERAL

This report presents foundation design recommendations for proposed Retaining Walls 346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1, and 361R located along Interstate 15 (I-15) between Carmel Mountain Road and Camino Del Norte in San Diego, California. These walls form part of the Route 15/56 Separation Managed Lanes Stage 1 project. The abbreviated project name is I-15 Managed Lanes, Unit 1.

Appendix I contains various site photographs, site map illustrations, and 13 sheets of LOTBs. Plates I-1 through I-7 are photographs of the project site. Figure I-1 - Vicinity Map, shows the locations of the retaining walls relative to existing roadways. Figure I-2 - Regional Geologic Map, shows the locations of the retaining walls relative to mapped superficial and solid geologic units. Figure I-3 - Seismic Hazard Map, shows the locations of the retaining walls relative to local faults. Figures I-4 through I-7 show the proposed retaining wall layouts and test boring locations.

Appendix II contains the laboratory test summary, data sheets, and results. Appendix III contains results of slope stability and bearing capacity analyses. Appendix IV provides a list of references.

Tables referenced in text appear at the end of their respective section.

1.2 AUTHORIZATION

As part of Task Order No. 284016, this portion of the I-15 Managed Lanes, Unit 1 project was conducted under Fugro's Statewide A&E Contract No. 59A0284 with Caltrans. Caltrans issued a written notice to proceed on April 19, 2002.

1.3 SCOPE OF WORK

1.3.1 Task Order

The purpose of Task Order No. 284016 is to conduct foundation investigations and prepare geotechnical design reports (GDRs) and logs of test borings (LOTBs) for proposed retaining walls. The original task order identified seven (7) retaining walls. Caltrans added two (2) additional retaining walls to the scope of work as the overall design of the Managed Lanes project progressed. The layout and location of some of the walls have also changed.

The executed task order provides a complete overview of the scope of work. A complete list of the contract scope of work/deliverable items is provided in the table below.

Article II, Clause Letter	Item
G	Log of Test Borings (LOTB)
I	Work Plan
J	Safety Plan
K	Traffic Control
L	Drilling
M	Soils and Rock Logging
O	Sampling and Field Tests
P	Temporary Piezometers
R	Case and Grout Boreholes
S	Laboratory Tests
T	Containment, Storage, and Removal of Drilling Waste
U	Borehole Abandonment
V	Borehole Surveys
Y	Permits
Deliverables	Work Plan Safety Plan Geotechnical Design Report (GDR) Log of Test Borings (LOTB)

*Statewide A&E Contract No. 59A0284 A01

1.3.2 Geotechnical Design Report

This report is prepared for the nine retaining walls designated as 346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1, and 361R. All the walls except for 357R will be located within slopes surrounding I-15. Wall 357R will be located within the center median south of the Camino Del Norte undercrossing.

1.4 PROJECT DESCRIPTION

1.4.1 Overall Site Description

The project site is located along I-15 between Carmel Mountain Road and Camino Del Norte in the County of San Diego. The terrain along I-15 slopes very gently towards the south and southwest. Graded slopes ascend from the east and west sides of I-15 up to existing commercial and residential developments, respectively. North of the Carmel Mountain Road overcrossing, slopes descend from the west side of southbound I-15 down to the Carmel Highland Golf Course. The center median, which is about 30 meters wide, includes an asphalt drainage swale and guardrail that trend parallel to I-15. Vegetation in the center median consists of short seasonal grass and sporadic taller weeds. The slopes adjacent to I-15 support a sparse to moderate cover of shrubs, seasonal grass and small trees.

Plates I-1 through I-7 in Appendix I provides photographs of typical site conditions. The following paragraphs describe the surface conditions of areas local to logical groupings of the retaining walls.

Retaining Walls 346L and 349L. Retaining Walls 346L and 349L will be located near the bottom of a west-facing fill slope along the west side of southbound I-15, adjacent to the Carmel Highland Golf Course. The slope is about 3 to 6 meters high and inclined at about 2h:1v (2 horizontal to 1 vertical). The slope surface is moderately to highly disturbed due to burrowing animals. Terrain west of the proposed walls slope gently towards the west and southwest. A small northerly trending ephemeral drainage course is located about 10 meters west of the planned wall alignments. A chain link fence is located along the bottom of the slope, approximately coinciding with the Caltran's Right-of-Way (R/W).

Retaining Walls 353R and 357R1. Retaining Walls 353R and 357R1 will be located at about mid-slope on west-facing cut slopes that ascend from the east side of northbound I-15. The slope inclination is 2h:1v and the height ranges from 20 and 25 meters. The slopes are traversed by an approximately 10-meter-wide terrace and a concrete-lined drainage swale (V-ditch). The area east of the slope crest is occupied by commercial buildings and asphalt-paved parking lots. The existing buildings are set back from the top of the slope by at least 10 meters. Surface runoff from these properties is directed away from the descending slope towards the east, to Avenue of the Science.

Retaining Wall 357R. Retaining Wall 357R will be located about 10 meters east of the "SD15M" line, within the center median of I-15, and about 300 meters south of the Camino Del Norte undercrossing. The northbound lanes of I-15 are topographically situated about 1 to 2 meters higher than the southbound lanes, and are separated by an ascending 1- to 2-meter-high, 2h to 3h:1v cut slope that approximately bisects the center median.

Retaining Walls 358L and 361L1. Retaining Wall 358L will be located about 20 meters west of the crest of the 2h:1v cut slope that ascends from the west side of the Camino Del Norte onramp to southbound I-15. The proposed alignment is close to the R/W. Property to the west slopes gently towards the west and is developed with single-family residences. Surface drainage follows the contours of the slope crest.

Retaining Wall 361L1 will be located about 130 meters north of wall 358L within a cut slope that ascends to the west. This slope is located along the west side of the Camino Del Norte onramp to southbound I-15 and has a maximum height of about 13 meters (at the location of wall 361L1). Surface drainage follows the slope contours.

Retaining Walls 361L and 361R. Retaining Walls 361L and 361R will be located at or near the tops of the fill slopes that ascend from the east side of the Camino Del Norte onramp to southbound I-15 or ascend from the west side of the Camino Del Norte offramp to northbound I-15. The slopes are about 10 meters high and inclined at about 2h:1v. Surface drainage follows the slope contours.

1.4.2 Existing Data

A search of compact discs (CDs) provided by Caltrans that contained archived existing information (Caltrans District 11, Batches 13 and 14, T.O. 11-064811) did not find any data related to the existing infrastructure in the area.

The alignment for I-15 in the project area appears to have been created by excavating into a former east-west-trending ridge that traversed I-15 south of Camino Del Norte. Excavation of the upper 10 to 12 meters of earth materials from the ridge was necessary to achieve the present roadway and median elevations. Earth materials generated from the cut grading are presumed to have been placed as structural fill in topographically low areas flanking the north and south sides of the ridge along the alignment of I-15.

Existing cut and fill slopes in the area appear to have performed satisfactorily. There did not appear to be visual evidence of surface and/or deep-seated slope instability, excessive erosion, seepage or poor surface drainage, with the exception of local areas of wet ground observed near the location proposed for Retaining Wall 357R1.

Very moist to wet conditions were observed on the surface of the ground below the landscaped slopes descending from the commercial properties east of Retaining Wall 357R1. Fugro staff met the property manager and their geotechnical consultant for the commercial property located at 15330 Avenue of the Science, which is situated east of proposed Retaining Wall 357R1. The purpose of the meeting was to investigate using the property for access to truck-mounted bucket auger test boring sites. During the meeting, the property manager and their geotechnical consultant advised that the site has experienced "elevated" groundwater conditions in the form of excessive moisture leaching through concrete floor slabs of the structures. They further reported that groundwater levels on the property are monitored on a monthly basis. Groundwater was not encountered in the Fugro test borings for wall 357R1.

1.4.3 Structures

The proposed retaining walls will be constructed of cast-in-place, reinforced concrete per Caltrans Standard Plans (2000a) and Caltrans Standard Specifications (1999b). Conventional spread footings will support the walls. The exposed walls will face west or east and perpendicular to the flow of traffic. Table 1-1 and the following paragraphs summarize the pertinent geometric data.

Retaining Walls 346L and 349L. Retaining Walls 346L and 349L will be constructed within existing fill embankments that support I-15. These embankments range from about 3 to 6 meters in height and are inclined at 2h:1v. The retaining walls will support fill slopes with maximum heights ranging from approximately 3 to 5 meters. The maximum retaining wall design height is 4.2 meters. Level ground, sloped to drain away from the walls, will be in front of the walls.

Retaining Wall 353R. Retaining Wall 353R will be constructed within a cut slope ascending from I-15 that has a total height of about 21.5 meters (within the R/W). The highest portion of the slope will form near Station 355+00 at which point the proposed cut slope is about 8.5 meters high and the retaining wall is about 6.6 meters high. The southerly approximately 205 meters of this wall will be constructed within the existing slope by forming an ascending (back) cut slope and a descending (front) cut slope with maximum respective heights of about 8.5 and 4.8 meters. Slope inclinations above and below the retaining wall are not planned to be steeper than about 2.5h:1v and 2h:1v, respectively. Slope inclinations above the wall near Stations 353+80, 354+40, 354+60, 356+00, and 356+18.9 are generally no steeper than 2h:1v.

Retaining Wall 357R. Retaining Wall 357R will be constructed right of the I-15 median centerline (SD15M). Maximum design heights will be about 1.8 meters. The wall will retain an asphalt- or concrete-paved level backslope.

Retaining Wall 357R1. Retaining Wall 357R1 will be constructed within a cut slope that ascends from I-15 and has a total height of about 19.3 meters (within the R/W). The highest portion of the slope will be near Station 358+80 at which point the proposed cut slope is about 11 meters high and the retaining wall is about 8 meters high. The southerly approximately 200 meters of the wall will be constructed within the existing slope by forming a descending (front) cut slope with a maximum height of about 4.5 meters. The ground is level in front of the remaining alignment of the wall. Slope inclinations above and below the retaining wall are not planned to be steeper than 2h:1v.

Retaining Wall 358L. Retaining Wall 358L will be constructed over a natural slope that ascends to the east from the R/W. This slope is about 9.5 meters high. The highest formed (fill) slope will occur near Station 359+80. At this location, the slope will be about 5.5 meters high and the retaining wall will be about 2 meters high. The ground is mostly level to slightly sloping in front of the wall. Fill slope inclinations above the wall are not planned to be steeper than 2h:1v.

Retaining Walls 361L and 361R. Retaining Walls 361L and 361R will be constructed within existing fill embankments that support I-15. The maximum design heights of the walls range from 1.8 meters (361R) to 3.6 meters (361L). The walls will retain asphalt- or concrete-paved level backslopes and have front slopes with maximum heights up to 6.5 meters that descend at a 2h:1v inclination.

Retaining Wall 361L1. Retaining Wall 361L1 will be constructed within a cut slope that ascends from the Camino Del Norte onramp to southbound I-15. This slope is about 13.4 meters high (within the R/W). The highest formed slope will occur near Station 361+60 (relative to the "CN3A" alignment). At this location, the slope will be about 7.6 meters high and the retaining wall will be about 5.5 meters high. The southerly approximately 20 meters of the wall will be constructed within the existing slope by forming an ascending cut (back) slope and a descending cut (front) slope with maximum respective heights of about 7.6 and 5.6 meters. Slope inclinations above and below the retaining wall are not planned to be steeper than 2h:1v.

1.4.4 Design Input

The retaining walls will be constructed according to the following Caltrans Standard Plans:

- Retaining Wall Type 1, H = 1200 through 9100, Revised Standard Plan RSP B3-1, October 26, 2000
- Retaining Wall Type 5, Revised Standard Plan RSP B3-7, October 26, 2000
- Retaining Wall Details No. 1, Revised Standard Plan RSP B3-8, October 26, 2000

Appendix VI provides the design drawings that Caltrans used to adapt each wall to the Standard Plans. These drawings, along with the Standard Plans, form the reference drawings used to prepare this GDR. Caltrans' Memos to Designers were also referred to for insight into the relevant design basis and practice. Pertinent design data obtained from the Standard Plans and design drawings are summarized in Table 1-2.

Table 1-1. Pertinent Geometric Data (Approximate)

Wall No.	Starting Station	Ending Station	Wall Length (m)	Alignment	Location	Maximum Design Height (m)	Retained Slope Angle (h:v) / Maximum Slope Height (m)	Front Slope Angle (h:v) / Maximum Slope Height (m)	Comments
346L	345+82	347+64	182	SD15M	outside left	3.0	2:1 / 3.0	~ Level	Fill slope near left R/W
349L	349+20	350+20	41	SD15M	outside left	4.2	2:1 / 5.0	~ Level	Fill slope near left R/W
353R	353+35	356+18	142	SD15M	outside right	7.9	Level to 2:1 / 8.5	2:1 / 4.8	Outside right cut slope
357R	357+60	359+83	223	SD15M	median right	1.8	Level	Level	Managed lane right of shoulder
357R1	357+77	361+65	388	SD15M	outside right	7.9	2:1/ 11	Level to 2:1 / 4.5	Cut slope approaching NB off-ramp to Camino Del Norte
358L	358+84	359+83	99	SD15M	outside left	3.0	2:1 / 5.5	~ Level	Fill slope near left R/W at SB on-ramp from Camino Del Norte
361L	361+79	363+27	148	SD15M	outside left	3.6	Level	2:1 / 6.0	Outside cut slope at SB onramp from Camino Del Norte
361L1	361+10	362+67	157	CN3A	outside left	5.5	2:1 / 7.5	Level to 2:1 / 5.5	Outside cut slope at SB onramp from Camino Del Norte
361R	361+59	362+40	81	SD15M	outside right	1.8	Level	2:1 / 6.5	NB outside edge of shoulder

R/W = right-of-way; NB = northbound; SB = southbound

Table 1-2. Design Input Data

Retaining Wall Designation	Wall Type	Maximum Design Height (mm)	Corresponding Footing Width (mm)	Loading Case	Toe Pressure (kPa) [1]	Designed Horizontal Footing Embedment (mm) [2]	Designed Vertical Footing Embedment (mm) [3]
346L	Type 1	3000	1900	Case II	110	NA (~)	450
349L	Type 1	4200	2200	Case II	160	NA (~)	450
353R	Type 1	7900	4350	Case II	310	1000 @ 355+20	450
357R	Type 5	1800	1550	Case I	105	NA (~)	500
357R1	Type 1	7900	4350	Case II	310	1000 @ 358+68	450
358L	Type 1	3000	1900	Case II	110	NA (~)	450
361L	Type 1	3600	2200	Case I	135	3500 @ 362+80	450
361L1	Type 1	5500	3050	Case II	200	1000 @ 362+00	450
361R	Type 1	1800	1300	Case I	90	2500 @ 362+00	450

[1] kPa = kilopascals
[2] Distance from slope face to forward edge of wall footing measured from cross section drawings.
[3] Minimum allowable depth from the ground surface to top of footing as specified on footing layout drawings.

2.0 FIELD EXPLORATION

2.1 DRILLING

Subsurface conditions were explored with 32 test borings that were drilled with termination depths ranging from 1.5 to 15.5 meters below the ground surface. This exploration program provided approximately one test boring per 45 meters of retaining wall length. Pertinent test boring exploration and completion data are summarized in Tables 2-1 and 2-2, respectively. Figures I-4 through I-7 and the Log of Test Borings (LOTBs) presented in Appendix I show the various test boring locations. The LOTBs depict an interpretation of the variation in earth materials and groundwater conditions, when encountered, and provides selected laboratory test results.

Variations in coverage and exploration depth below the top of footing arose from such factors as:

- Additional exploration of potential adverse geologic conditions
- Site access restrictions
- Evaluation of slope stability where retaining walls would be constructed in existing slopes
- Deep transmission of stress from foundation loads (toe pressures) from the larger walls retaining sloped ground

The field exploration was completed in two phases that mobilized three drill rigs for each phase. The first phase was for Retaining Walls 346L, 349L, 353R (portion), 357R, 361L, and 361R. This phase commenced on May 29, 2002, and was completed on June 6, 2002. The second phase was for Retaining Walls 353R (portion), 357R1, 358L, and 361L1. This phase commenced on October 8, 2002, and was completed on October 11, 2002. The second phase also included small-scale limited access drilling at walls 346L and 349L. The purpose of this exploration was to improve the interpretation of subsurface conditions by placing test borings close to the wall alignments where access is more restricted.

The test borings for walls 357R, 361L, and 361R were drilled with a truck-mounted, hollow-stem-auger rig. The test borings for walls 346L, 349L, 353R, and 357R1 were drilled with a limited access, track-mounted, hollow-stem-auger rig. Test borings for walls 346L and 349L that were located near the bottom of the slope were drilled with a tripod-mounted, solid-stem rig. A truck-mounted bucket auger rig was originally proposed for Retaining Walls 353R and 357R1, which will be located within large existing cut slopes; however, right-of-entry limitations eventually precluded using this equipment. This type of drill rig permits continuous down-hole geologic observation.

Greg Drilling of Signal Hill, California, provided truck-mounted and limited access track-mounted drill rigs. Limited Access Unlimited, Inc. (Pacific Drilling Company) of San Diego, California, provided limited access track-mounted and tripod-mounted drill rigs.

2.2 BOREHOLE SURVEYS

The test borings were located by taped measurement and hand leveling from Caltran's staked positions to within ±0.15 meter. Due to existing access constraints, the test boring locations sometimes varied from about 5 to 20 meters horizontally from the wall alignment.

2.3 SOILS AND ROCK LOGGING

An engineer or geologist accompanied each drill rig to technically supervise and log the test borings. The fieldwork was supervised by either a California Registered Geologist (C.E.G.) or an Engineering Geologist (R.G.).

2.4 SAMPLING AND FIELD TESTS

The upper 1.5 meters of many of the test borings were hand augered to avoid damaging buried utilities. Samples were usually obtained at approximately 1.5-meter intervals, alternating between California (60-millimeter- [mm-] diameter) and Standard Penetration Test (SPT, 30-mm-diameter) samplers. Samplers were driven using a 63.6-kilogram automatic trip hammer falling a distance of 0.76 meter. This sampling interval was modified locally as summarized in Table 2-3.

2.5 TEMPORARY PIEZOMETERS

Temporary piezometers were placed in test borings 346L/HA/01, 346L/HA/02, and 346L/LA/04 to measure groundwater levels.

2.6 BOREHOLE ABANDONMENT

The test borings were terminated near the planned target depth or at refusal, whichever was shallower. Refusal was defined as when more than 50 blows per 150 mm were encountered in two consecutive SPT drives or the inability of the drilling equipment to penetrate any farther.

The test borings were backfilled with full-depth cement grout or compacted drill spoil capped with 0.3 meter of activated bentonite chips. Test borings that were not near the traveled way (e.g., shoulder and median of I-15) but were within undeveloped sloped areas were backfilled with tamped drill spoil capped with 0.3 meter of activated bentonite chips. Test borings that encountered groundwater, regardless of their location, were backfilled with full depth cement grout. The ground surfaces were reinstated following borehole completion.

2.7 CONTAINMENT, STORAGE, AND REMOVAL OF DRILLING WASTE

Drill spoil was placed in drums and properly disposed offsite.

Table 2-1. Test Boring Exploration Data (Relative to Proposed Retaining Walls)

Wall No.	Wall Length (m)	No. of Test Borings	Coverage (Test Borings Per Meter of Wall)	Deepest Top of Footing Elevation (m)	Deepest Termination Elevation (m)	Exploration Depth Below Top of Footing (m)
346L	182	4	45	191.4	186.6	4.8
349L	41	3	14	199.4	197.4	2.0
353R	142	5	28	224.7	225.9	1.2
357R	223	5	45	228.3	221.6	6.7
357R1	388	5	78	226.8	219.6	7.2
358L	99	2	50	227.6	222.4	5.2
361L	148	3	50	225.0	218.7	6.3
361L1	157	3	52	221.6	219.2	2.4
361R	81	2	40	226.9	216.4	10.5
Total	1461	32	--	--	--	--
Average	--	--	45	--	--	5

Table 2-2. Test Boring Completion Data

Test Boring No.	Drilling Method	Surface Elevation (m)	Termination Elevation/Total Depth (m)	Backfill Construction	Completion Date
346L/HA/01	Hollow Stem / Track Rig	196.0	186.6 / 9.4	Cement Grout	May 31, 2002
346L/HA/02	Hollow Stem / Track Rig	197.6	186.6 / 11.0	Cement Grout	May 31, 2002
346L/HA/03	Hollow Stem / Track Rig	198.5	190.9 / 7.6	Cement Grout	May 30, 2002
346L/LA/04	Solid Stem / Tripod Rig	192.5	189.0 / 3.5	Cement Grout	Oct 09, 2002
349L/HA/01	Hollow Stem / Track Rig	205.5	197.4 / 8.1	Cement Grout	May 30, 2002
349L/HA/02	Hollow Stem / Track Rig	206.5	197.4 / 9.1	Cement Grout	May 30, 2002
349L/LA/03	Solid Stem / Tripod Rig	201.5	200.0 / 1.5	Cement Grout	Oct 09, 2002
353R/LA/01	Hollow Stem / Track Rig	231.8	227.1 / 4.7	Bentonite Cap	Oct 08, 2002
353R/LA/02	Hollow Stem / Track Rig	236.8	234.8 / 2.0	Bentonite Cap	Oct 08, 2002
353R/LA/02B	Hollow Stem / Track Rig	237.2	233.3 / 3.9	Bentonite Cap	Oct 08, 2002
353R/LA/03	Hollow Stem / Track Rig	238.8	225.9 / 12.9	Bentonite Cap	Oct 09, 2002
353R/LA/04	Hollow Stem / Track Rig	236.6	226.7 / 9.9	Cement Grout	Jun 06, 2002
357R/LA/01	Hollow Stem / Truck Rig	229.5	224.5 / 5.0	Cement Grout	Jun 06, 2002
357R/LA/02	Hollow Stem / Truck Rig	230.3	225.3 / 5.0	Cement Grout	Jun 06, 2002
357R/LA/03	Hollow Stem / Truck Rig	230.8	222.7 / 8.1	Cement Grout	Jun 06, 2002
357R/LA/04	Hollow Stem / Truck Rig	231.1	221.6 / 9.5	Cement Grout	Jun 06, 2002
357R/LA/05	Hollow Stem / Truck Rig	231.3	224.8 / 6.5	Cement Grout	Jun 06, 2002
357R1/LA/01	Hollow Stem / Track Rig	236.4	222.6 / 13.8	Bentonite Cap	Oct 09, 2002
357R1/LA/02	Hollow Stem / Track Rig	249.2	226.6 / 22.6	Bentonite Cap	Oct 09, 2002
357R1/LA/03	Hollow Stem / Track Rig	235.8	221.6 / 14.2	Bentonite Cap	Oct 11, 2002
357R1/LA/04	Hollow Stem / Track Rig	232.7	222.7 / 10.0	Bentonite Cap	Oct 11, 2002
357R1/LA/05	Hollow Stem / Track Rig	229.2	219.6 / 9.6	Bentonite Cap	Oct 10, 2002
358L/LA/01	Hollow Stem / Track Rig	228.8	222.4 / 6.4	Bentonite Cap	Oct 11, 2002
358L/LA/02	Hollow Stem / Track Rig	230.5	225.8 / 4.7	Bentonite Cap	Oct 10, 2002
361L/HA/01	Hollow Stem / Truck Rig	230.0	221.9 / 8.1	Cement Grout	May 29, 2002
361L/HA/02	Hollow Stem / Truck Rig	229.4	221.3 / 8.1	Cement Grout	May 31, 2002
361L/HA/03	Hollow Stem / Truck Rig	228.8	218.7 / 10.1	Cement Grout	May 29, 2002
361L1/LA/01	Hollow Stem / Truck Rig	241.7	227.9 / 13.8	Bentonite Cap	Jun 04, 2002
361L1/LA/02	Hollow Stem /Track Rig	240.6	225.1 / 15.5	Bentonite Cap	Jun 04, 2002
361L1/LA/03	Hollow Stem /Track Rig	224.0	219.2 / 4.8	Bentonite Cap	Jun 04, 2002
361R/HA/01	Hollow Stem / Truck Rig	228.8	218.7 / 10.1	Cement Grout	May 29, 2002
361R/HA/02	Hollow Stem / Truck Rig	229.4	221.3 / 8.1	Cement Grout	Jun 04, 2002
32 = total number of test borings			277.6 = total meters of drilling		

Table 2-3. Sampling Intervals

Test Boring No.	Purpose [1]	Sample Type	Frequency (m)	Depth Interval (m)
346/LA/04	1	SPT or California Barrel	0.5	Entire test boring
349/LA/03	1	SPT or California Barrel	0.5	Entire test boring
353R/LA/03	2	California Barrel	0.75	15 to 20 (bottom)
357R1/LA/01	2	California Barrel	0.75	9 to 13 (bottom)
357R1/LA/02	2	California Barrel	0.75	17 to 25 (bottom)
357R1/LA/04	2	California Barrel	0.75	6 to 10 (bottom)
361L1/LA/01	3	SPT or California Barrel	3	0 to 5
361L1/LA/02	2	California Barrel	0.75	12 to 16 (bottom)

[1] 1 = Investigation of possible remnant alluvium below proposed foundation level.
2 = Obtaining "intact" rock samples at and below foundation level for Unconfined Compressive Strength (UCS) testing.
3 = Larger sampling interval possible since material within specified depth interval will be removed to form new cut slope.

3.0 LABORATORY TESTING

The primary purpose of the laboratory testing was to evaluate the physical characteristics of the earth materials encountered and to correlate and reconcile field descriptions with more accurate laboratory assessments. Additional testing was completed where necessary to evaluate strength and deformation characteristics, corrosion potential, and the suitability of potential cut materials for reuse as compacted fill. Table 3-1 provides the type, purpose, percent of samples, and number of tests completed for this task order. Appendix II provides the laboratory test results.

Table 3-1. Laboratory Testing Program

Test Type	Purpose	Retaining Walls [1]																	
		346L		349L		353R		357R		357R1		358L		361L		361L1		361R	
		No.	%	No.	%	No.	%	No.	%	No.	No.	%	%	No.	%	No.	%	No.	%
Moisture Content (ASTM D2216)	Physical Characteristics	20	74	8	44	15	54	17	70	25	12	80	50	5	38	12	80	11	38
Total and Dry Densities (ASTM D2937)	Physical Characteristics	10	37	3	17	2	7	8	32	11	6	40	23	4	31	6	40	4	14
Atterberg Limits (ASTM D4318)	Physical Characteristics	4	15	3	17	3	11	3	12	13	6	40	27	3	23	6	40	5	17
Percent Passing #200 Sieve (ASTM D1140)	Physical Characteristics	8	30	4	22	7	25	9	36	25	8	53	50	7	22	8	53	11	38
Sand Equivalent (ASTM 2419)	Reuse as Compacted Fill	1	4	- -	- -	2	7	1	1	4	- -	- -	8	1	3	- -	- -	4	14
Expansion Index	Reuse as Compacted Fill	- -	- -	- -	- -	- -	- -	- -	- -	2	1	7	4	1	3	1	7	1	3
Direct Shear, Multi-Stage (ASTM D3080)	Shear Strength	2	7	- -	- -	1	4	2	8	4	1	7	8	1	3	1	7	2	7
Unconfined Compression (ASTM 2938)	Compressive Strength	- -	- -	- -	- -	1	4	- -	- -	5	- -	- -	10	1	3	- -	- -	1	3
pH, Chloride, Sulfate, Resistivity	Corrosivity	3	11	1	5	3	11	2	8	7	1	7	15	2	6	1	7	4	14

[1] % = Percent of samples; No. = Number of tests completed for this task order

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 REGIONAL GEOLOGY

The Geologic Map of the Poway Quadrangle (scale=1:24000; Kennedy and Peterson, 1975) indicates that alluvium, marine sandstone assigned to the Mission Valley Formation (Tmv), and volcanic rocks of the Santiago Peak Volcanics (Jsp) underlie the project area. Figure I-2 shows the approximate location of the retaining walls on the above referenced geologic map.

4.2 GEOLOGIC STRUCTURE

The findings from our subsurface investigation and geologic observations are in general agreement with regional trends and geologic data shown on the published maps. However, detailed geologic mapping of the existing cut slopes was not possible due to the presence of vegetation, soil cover, and surface disturbance of the exposed bedrock units due to weathering and burrowing animals. Therefore, geologic mapping of the temporary excavations and permanent cut slopes will be required as part of the final evaluation for slope instability.

Published geologic maps (scale=1:24000; Kennedy and Peterson, 1975) indicate that bedding within the Mission Valley Formation near the project site and surrounding areas is inclined towards the southwest at very shallow angles ranging from 2 to 3 degrees. The findings from the test borings and limited surface geologic mapping suggest that the Mission Valley Formation at the project site is thick to very thickly bedded.

Regional geologic maps show a contact between the Mission Valley Formation and Santiago Peak Volcanics (volcanics) near the south end of wall 353R. Foliation with the volcanics is shown to strike northwest-southeast and dip moderately (about 40 degrees) to the north. Joints exposed in an existing cut slope at the south end of wall 353R are highly variable in orientation and dip moderately to steeply. Foliation within the exposed volcanics was not observed.

4.3 GEOLOGIC HAZARDS

No historical and/or potential geologic hazards (e.g., landslides, slope instability, ground subsidence, heave, liquefaction, and faulting) are known to exist at the project site within the vicinity of the retaining walls and, based on the data obtained from this study, none have been interpreted to have a potential adverse impact.

4.4 SUBSURFACE CONDITIONS

Test borings encountered a typical sequence of fill, native soil, and weathered bedrock consisting of sandstone, claystone or volcanics. Since it occurs within the R/W, the fill is assumed to have been properly processed, placed, and compacted. Groundwater was only encountered locally at one wall location. The following paragraphs describe the subsurface conditions encountered within logical groupings of the retaining walls.

4.4.1 Retaining Walls 346L and 349L

The test borings for Retaining Walls 346L and 349L encountered a sequence of fill, native soil (at wall 346L), and weathered volcanic bedrock. The fill encountered in test borings 346L/LA/04 and 349L/LA/03, which are located near the toe of an existing embankment that approximately follows the proposed wall alignments, is about 1 meter thick. The fill is about 4 to 5 meters thick at the top of the embankment that forms the shoulder of southbound I-15. Approximately 2 meters of native soil underlie the fill at wall 346L. Volcanic bedrock is below the native soil at wall 346L and directly underlies the fill at wall 349L.

The fill at both sites consists of stiff to very stiff sandy lean clay (CL). Occasional angular gravel, which caused resistance to sample driving, was encountered in the test borings at wall 349L.

The native soil encountered at wall 346L consists of stiff to very stiff fat clay with sand (CH). This material is interpreted to be residual soil resulting from in-place weathering of the underlying volcanic bedrock.

The weathered volcanic bedrock encountered at both walls consists of moderately weathered to decomposed volcanics. The bedrock is medium gray to reddish brown in color, highly fractured (locally), and soft to moderately hard.

4.4.2 Retaining Walls 353R and 357R1

Test borings for Retaining Walls 353R and 357R1 encountered a sequence of native soil over weathered sandstone and claystone bedrock, and lesser amounts of intrusive weathered volcanic bedrock that extends to the maximum depth explored (about 14 meters). These various types of bedrock are the predominant earth materials that could influence the design and construction of these retaining walls.

The native soil consists of stiff sandy lean clay (CL) and medium dense clayey sand (SC). The native soils vary from 1 to 3 meters thick at the test boring locations. This material is interpreted to be residual soil resulting from in-place weathering of the underlying bedrock.

Weathered volcanic bedrock was observed in test borings at the southern end of wall 353R. Weathered sandstone and claystone bedrock underlie the native soil along the northern portion of wall 353R and along the entire length of wall 357R1.

The volcanic bedrock consists of moderately weathered to decomposed volcanics that vary from medium gray to reddish brown in color. The volcanic bedrock is highly fractured (locally) and soft to moderately hard.

The sandstone and claystone bedrock consist of very thickly bedded to massive, moderately weathered to decomposed sandstone and occasional interbedded claystone. The sandstone is light to medium gray in color, fine- to medium-grained, poorly to moderately cemented, soft to moderately hard, and locally friable. The claystone is typically greenish gray in color and is soft to moderately hard.

4.4.3 Retaining Wall 357R

Test borings drilled along the alignment of Retaining Wall 357R encountered less than 1.1 meters of fill overlying weathered sandstone and claystone bedrock, extending to the maximum depth explored (i.e., 9.5 meters). The depth of fill appears to increase as the alignment progresses south.

The fill consists mostly of stiff to very stiff sandy lean clay (CL).

A 1-meter-thick layer of fat clay with sand (CH) was observed below the fill in test boring 357R/HA/01, the southernmost test boring. This material is interpreted to be residual soil resulting from in-place weathering of the bedrock.

Weathered bedrock was encountered at the ground surface (northerly portion) and underlying the artificial fill and residual soil. The bedrock appears to be similar to that encountered in test borings for wall 357R1.

4.4.4 Retaining Walls 358L and 361L1

Test borings for Retaining Walls 358L and 361L1 encountered a sequence of native soil underlying weathered sandstone and claystone bedrock extending to the maximum depth explored (about 6.4 meters at wall 358L and 16 meters at wall 361L1). Weathered bedrock is the predominant earth material that could influence the design and construction of these retaining walls.

The native soil consists predominantly of stiff sandy lean clay (CL). These materials vary from 1 to 2.5 meters thick at the test boring locations. They are interpreted to be residual soil resulting from the in-place weathering of the underlying bedrock.

Bedrock consists of very thickly bedded to massive, moderately weathered to decomposed sandstone and claystone. The sandstone is light olive gray to yellowish brown in color, fine-to medium-grained, poorly to moderately cemented, soft to moderately hard, and locally friable. The claystone is light yellowish gray to olive green in color and is moderately soft to moderately hard.

4.4.5 Retaining Walls 361L and 361R

Test borings for these walls encountered a sequence of fill, native soil, and weathered sandstone and claystone bedrock to the maximum depths explored (about 10 meters at wall 361L and 12.5 meters at wall 361R). Fill and native soils are the predominant earth materials that could influence the design and construction of these retaining walls.

The fill consists mostly of medium dense clayey sand (SC) and lesser stiff fat clay (CH). The depth of fill increases from about 2.5 meters at the south end of the wall alignment to about 12 meters at the north end (closest to the Camino Del Norte undercrossing). Fill was the only material encountered in test boring 361I/HA/03 at the north end of wall 361L.

At test boring 361L/HA/01, the fill is underlain by an approximately 3- to 4-meter-thick layer of native soil consisting of stiff to very stiff sandy lean clay (CL) and isolated fat clay (CH). These materials are interpreted to be a combination of residual soils resulting from in-place weathering of the underlying bedrock and alluvial deposits.

Weathered bedrock consists of very thickly bedded to massive, moderately weathered to decomposed sandstone and occasional interbedded claystone. The sandstone is light to medium gray in color, fine- to medium-grained, poorly to moderately cemented, soft to moderately hard, and locally friable. The claystone is typically greenish gray in color and is soft to moderately hard.

4.5 GROUNDWATER

Groundwater was encountered in the test borings at wall 346L. Temporary piezometers were installed to monitor groundwater levels for a period of at least 24 hours. A summary of the depth and elevation of groundwater at each boring is provided in the following table.

Test Boring Number	Depth of Groundwater (m)	Elevation of Groundwater (m)	Total Depth Drilled (m)
346L/HA/01	4.4	191.6	9.4
346L/LA/02	4.6	186.6	11.0
346L/LA/04	1.8	190.7	3.5

Groundwater may occur at different levels and location in the futures from additional rain, seasonal fluctuations, and changes in land use. Seepage could occur in rock fractures or joints.

5.0 GEOLOGIC PROFILE AND ENGINEERING PARAMETERS

5.1 REPRESENTATIVE GEOLOGIC PROFILES

Geologic profiles for analysis and design have been developed that consider the variation in subsurface conditions encountered at the project site. Geologic conditions can be grouped into the four general soil/rock profiles presented in Table 5-1. Table 5-2 summarizes the interpreted soil/rock profile at each wall.

5.2 ENGINEERING PARAMETERS

Table 5-3 summarizes the engineering parameters interpreted for the soil/rock units encountered at the project site. Interpretations of the engineering parameters are discussed in the following paragraphs.

5.2.1 Penetration Index

After correcting for sample type, elevation and automatic trip hammer (energy ratio of 75 percent), estimated or equivalent SPT $N_{1(60)}$ blow counts of about 16 and 17 blows per 300 mm were obtained for the fill and native soils encountered at the project site (Retaining Walls 346L, 349L, 357R, 361L, and 361R).

The average uncorrected (field) blow count for the California sampler (60-mm diameter) were 25 and 26 blows per 300 mm for the fill and native soils, excluding the drives where the blow counts exceed 50 blows per 100 mm.

The average uncorrected (field) blow counts for the California sampler (60-mm diameter) for the weathered bedrock were typically greater than 50 blows per 150 mm and often greater than 50 blows per 75 mm.

DMG (1997) advises that N-values obtained from a California sampler can be roughly correlated to SPT N-values using a conversion factor of 0.63 (ranging from 0.5 to 0.7). They further advise that the equivalent SPT N-value calculated from this conversion should be used for comparison with intervening SPT N-values and not relied upon as the sole source of penetration index (blow count) data. This correlation is provided for reference purposes. Users of the data should review the reference for conversion suitability to their particular analysis or design application.

5.2.2 Index Properties

The surface materials (fill and native soils) are fine to coarse grained, and the finer fraction generally possesses a "medium" degree of plasticity. Sandstone weathered bedrock that has been reworked to soil with fines generally possess a "medium" degree of plasticity. Reworked claystone weathered bedrock generally possess a "high" degree of plasticity. Tests on two samples of reworked volcanic weathered bedrock produced results indicating a "medium" to "high" degree of plasticity. Table 5-4 summarizes the range of index data for the soil/rock units encountered.

5.2.3 Shear Strength

Table 5-5 summarizes the effective stress shear strength parameters (c' and ϕ') interpreted from the envelope of direct shear test (consolidated, drained, and saturated) results on intact samples. One sample of fill was remolded to a unit weight and moisture content that approximately represented 90-percent relative compaction (California Test Method 216 [California Impact]). Appendix II provides the test results with plots of data envelopes from multiple samples obtained in the same soil/rock unit and data sets from individual samples.

5.2.4 Compression Potential

The fill, native soil, and weathered bedrock should possess relatively low elastic compressibility characteristics based on their in situ density/consistency, the deformation modulus interpreted from the slope of compression (consolidation) tests, and an evaluation using common correlations between in situ deformation modulus and penetration index (e.g., $E \cong f$ SPT N).

Time-dependent deformation (consolidation) is expected to be negligible to low considering the soil types, their age, relatively low moisture contents, and other factors. These materials are not expected to settle (collapse) when inundated

5.2.5 Expansion Potential

The Expansion Index provides a qualitative assessment of the swelling characters of compacted soils. The table below summarizes the Expansion Index test data. Appendix II provides a summary of the individual test data.

Soil / Rock Unit	Maximum Expansion Index [1]
Native Soil	36 (Low)
Weathered Bedrock: Claystone	141 (Very High)
Weathered Bedrock: Volcanics	54 (Medium)

[1] Uniform Building Code (1997)

Existing fill should have a "low" to "medium" degree of expansion based on evaluations (Holtz and Kovac, 1981, as referenced by the Canadian Geotechnical Society, 1992) that consider soil type and plasticity index/clay content. Using the same evaluations, locally occurring fat clay (CH) native soil should have a high to very high degree of expansion.

Sandstone weathered bedrock that has been mined and reworked to a soil should possess a low to medium degree of expansion based on the above described evaluations.

5.2.6 Suitability for Reuse as Fill

The Sand Equivalent test qualitatively evaluates the suitability of excavated materials for reuse as fill. Fill is defined as Structure Backfill according to Section 19-3.06 of the Standard Specifications. The table below summarizes the Sand Equivalent test data. Appendix II provides a summary of the individual test data.

Soil / Rock Unit	Range of Sand Equivalent
Native Soil	3 - 36
Weathered Bedrock: Claystone	3 - 8

Portions of the sandstone weathered bedrock should be suitable for reuse as structure backfill based on fines content and plasticity characteristics.

Weathered bedrock comprising severely to moderately fractured volcanics is not expected to be suitable for structure backfill due to the potential for large quantities of material that exceed the specification. This material should generally excavate as a silty gravel (GM) with little or no fines and sizeable quantities of material greater than 4.75 mm. Moderately to slightly fractured volcanics produced from heavy ripping or blasting should consist of a high percentage of oversize and angular rock.

Table 5-1. General Soil /Rock Profiles

General Soil / Rock Profile	Representative of Retaining Walls:
Fill	361L and 361R
Fill Over Weathered Bedrock (sometimes with an intervening layer of native soil)	346L, 349L, and 357R
Weathered Bedrock: Sandstone and Claystone	353R (northern half), 357R1, 358L, and 361L1
Weathered Bedrock: Volcanics	353R (southern half)

Table 5-2. Geologic Profiles for Each Wall

Wall	Soil / Rock Unit	Description	Approximate Thickness at Test Boring Locations (m)
346L	Fill	Stiff to very stiff sandy lean clay (CL)	1
	Native Soil	Stiff to very stiff fat clay with sand (CH)	2
	Weathered Bedrock	Volcanics: soft to moderately hard, highly fractured (locally)	--
349L	Fill	Sandy lean clay (CL): stiff to very stiff	1
	Weathered Bedrock	Volcanics: soft to moderately hard, highly fractured (locally)	--
353R	Native Soil	Stiff sandy lean clay (CL) and medium dense clayey sand (SC)	1 - 3
	Weathered Bedrock	Volcanics: soft to moderately hard, highly fractured (locally)	--
357R	Fill	Stiff to very stiff sandy lean clay (CL)	1
	Weathered Bedrock	Sandstone and occasional interbedded claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--
357R1	Native Soil	Stiff sandy lean clay (CL) and medium dense clayey sand (SC)	1 - 3
	Weathered Bedrock	Sandstone and occasional interbedded claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--
358L	Weathered Bedrock	Sandstone and claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--
361L	Fill	Stiff to very stiff sandy lean clay (CL)	2 - >9
	Native Soil	Stiff to very stiff sandy lean fat clay (CL) and fat clay (CH)	2 - >3
	Weathered Bedrock	Sandstone and occasional interbedded claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--
361L1	Weathered Bedrock	Sandstone and claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--
361R	Fill	Stiff to very stiff sandy lean clay (CL)	2 - 6
	Native Soil	Stiff to very stiff sandy lean fat clay (CL) and fat clay (CH)	3 - 4
	Weathered Bedrock	Sandstone and occasional interbedded claystone: soft to moderately hard, very thickly bedded to massive, locally friable	--

Table 5-3. Interpreted Engineering Parameters and Behavior

Soil / Rock Unit	SPT N or UCS (MPa)	Moisture Content/Fines (%)	Atterberg Limits (LL/PL/PI)	Bulk Unit Weight (kN/m^3)	Assigned Shear Strength [1]		Expansion Potential	Compression Potential	Potential Suitability for Use as Fill [4,5]
					C' (kPa)	ϕ' (°)			
Fill	15	15/40	48/18/30	18	20	27	High	Low	NA
Native Soil	15	19/70	39/17/22	19	20	24	High	Low (Locally Moderate)	Low
Weathered Bedrock: Volcanics 2	0.8 – 1.5	16/47	50/20/30 [6]	20	30	35	Low to High	Very Low	Low to Moderate
Weathered Bedrock: Sandstone 1,3	< 0.1	15/30	35/19/16 [6]	21	25	30	Low	Very Low	Moderate
Weathered Bedrock: Claystone [1]	0.8 - 1.5	17/86	49/23/26 [6]	20	50	24	High	Very Low	Low

[1] The shear strength parameters in the table above assume the intensity of the weathering within the bedrock is such that material behaves as a soil and relict discontinuities of the parent rock do not influence geotechnical engineering behavior.
[2] Insufficient samples for testing since material encountered locally.
[3] Material poorly cemented (very friable), resulting in low unconfined compressive strength.
[4] Fill defined as Structure Backfill, Section 19-3.06 of the Standard Specifications.
[5] Based on plasticity and fines content along with expansion index and sand equivalent test results.
[6] Samples reworked to a soil.

Table 5-4. Summary of Index Properties

Soil / Rock Unit	Bulk Unit Weight (kN/m^3)		Moisture Content (%)		Atterberg Limits (Max./Min.)		Fines (%)	
	Max.	Min.	Max.	Min.	LL	PL	Max.	Min.
Fill	21	15	20	6	84/38	21/16	73	13
Native Soil	21	14	34	8	44/33	24/14	92	25
Weathered Bedrock: Volcanics	23	19	26	5	61/40	21/20	90	11
Weathered Bedrock: Sandstone	22	18	21	1	41/33	23/17	50	20
Weathered Bedrock: Claystone	22	16	20	11	60/35	24/15	97	60

Table 5-5. Interpreted Shear Strengths

Soil / Rock Unit	Peak Cohesion (kPa)	Peak Friction Angle (degrees)	Ultimate Cohesion (kPa)	Ultimate Friction Angle (degrees)
Fill	30	28	20	27
Native Soil	45	24	20	24
Weathered Bedrock: Sandstone	75	35	25	30
Weathered Bedrock: Claystone	100	28	50	24

6.0 SLOPE STABILITY ANALYSIS

6.1 METHODOLOGY

Slope stability analyses were performed using SLOPE/W, a computer program produced by Geo-Slope International Ltd, to evaluate a factor of safety against overall rotational, static, and pseudostatic failure mechanisms. The analyses were conducted on cross sections obtained from Caltrans' design drawings using the soil, bedrock, and groundwater conditions interpreted from the site investigation. The analyses searched for the most-critical failure surfaces. Appendix III provides cross sections and results.

Conventional limit-equilibrium methods of analyses (e.g., Janbu, Spencer, and Morgenstern and Price) were used to calculate the static and pseudostatic factors of safety against deep-seated failure. With these methods, the free body above the failure surface is divided into vertical slices and the equilibrium of each slice is considered. The shear resistance of soil necessary for equilibrium is calculated and compared to the available shear strength of the soil along the failure surface, giving a calculated factor of safety.

6.2 SHEAR STRENGTH PARAMETERS

The strength parameters used in the stability analyses were interpreted from direct shear tests performed on intact samples. To model potential long-term groundwater conditions, the samples were placed in water and saturated until fully "soaked." Section 5.2 summarizes the shear strength parameters used in the analyses.

6.3 RESULTS OF ANALYSES

The analyses indicate that the proposed slopes have adequate factors of safety against deep-seated slope failure. Calculated factors of safety exceed 1.5 for static conditions and 1.1 for seismic consideration. The ten most critical failure surfaces for each slope analyzed are shown on the slope stability cross sections presented in Appendix III. Table 6-1 provides a summary of the results of the slope stability analyses.

The slope stability analyses assume that the cut slopes will be formed in weathered bedrock comprised of massive materials, where the stability is not controlled by bedding, discontinuities (relict or recent), or pre-existing shear zones (e.g., bedding parallel shear zones [Hart, 2000]). In addition, portions of the slope below the southern half of Retaining Wall 353R may expose relatively unweathered volcanic rock. The stability of rock cut slopes is a function of discontinuities (e.g., jointing, fracturing and/or foliation) and their orientation to the slope face. Discontinuities oriented out-of-slope (i.e., dip angle parallel or shallower than the cut slope inclination) can cause deep-seated failures (i.e., rock glides or surface failures like rockfalls, spalling or exfoliation). Based on the mapping of existing cut slope exposures interpreted to represent intact materials, there appears to be a low potential for such failures. However, due to vegetation, weathering, and soil cover, it is difficult to evaluate the orientation of discontinuities with a high degree of certainty before mass excavation. Therefore, thorough geologic mapping of the cut slopes and other mass excavations, whether they are in sandstone, claystone or volcanics, is essential to prepare the final evaluation of the potential for slope instability and to provide timely recommendations for slope stabilization measures, if needed. Such measures could include stabilization fills, buttress fills, soil nails, rock dowels, and/or rock bolts.

Table 6-1. Results of Slope Stability Analyses

Wall No.	Station No.	Factor of Safety	Analysis/Failure Location	Notes
353R	355+00	2.4	Static / below footing	--
		1.7	Seismic / below footing	Seismic Coefficient: 0.2
		2.4	Static / at toe of slope	--
		1.6	Seismic / at toe of slope	Seismic Coefficient: 0.2
357R1	358+80	1.9	Static / below footing	--
		1.3	Seismic / below footing	Seismic Coefficient: 0.2
361L	363+13	2.6	Static / below footing	20 kPa Surcharge
		1.9	Seismic / below footing	20 kPa Surcharge, Seismic Coefficient: 0.2
		2.2	Static / at toe of slope	20 kPa Surcharge
		1.5	Seismic / at toe of slope	20 kPa Surcharge, Seismic Coefficient: 0.2
361L1	361+60	2.1	Static / below footing	--
		1.5	Seismic / below footing	--
361R	362+40	3.3	Static / below footing	20 kPa Surcharge
		2.3	Seismic / below footing	20 kPa Surcharge, Seismic Coefficient: 0.2
		2.5	Static / at toe of slope	20 kPa Surcharge
		1.6	Seismic / at toe of slope	20 kPa Surcharge, Seismic Coefficient: 0.2

7.0 SEISMIC STUDY

The project site is located within a seismically active region of southern California. The project is near a number of faults that are considered active or potentially active. Seismic design criteria have been developed based on the Caltrans California Seismic Hazard Map (Mualchin, 1996) in conjunction with recommendations presented in Caltrans (2001).

7.1 CONTROLLING FAULT

As shown on Figure I-3, the Newport-Inglewood-Rose Canyon East (NIE) fault is the controlling fault near the project. Other nearby faults include the Newport-Inglewood-Rose Canyon West (NIW), Point Loma (PTL), and Whittier-Elsinore (WEE). The Newport-Inglewood-Rose Canyon East fault zone is located about 21 kilometers southwest of the project.

With the exception of the Point Loma fault zone, whose style of faulting is unknown or unpublished, Mualchin (1996) indicates that the fault zones referenced above have a strike-slip (ST) style of faulting. According to Mualchin (1996), the maximum anticipated moment magnitudes (M_w) for the WEE, NIE/NIW, and PTL fault zones are 7.5, 7.0, and 6.5, respectively.

7.2 PEAK HORIZONTAL GROUND ACCELERATION

The maximum peak horizontal ground acceleration (PGA) of bedrock is estimated to be about 0.3g from Mualchin (1996), as shown on Figure I-3. However, we note that there is a potential for peak bedrock accelerations greater than 0.3g to occur in response to an earthquake on one of the discussed nearby faults due to local variation in geologic conditions and statistical variation of attenuation relationships.

8.0 CORROSION EVALUATION

Corrosivity testing, which was performed on selected samples obtained from the test borings, suggests the area is not a corrosive environment (Caltrans, 1999a). The corrosion tests were performed in accordance with Caltrans test methods. Laboratory test results are presented in Appendix II. Table 8-1 presents a summary of the pertinent test data.

Table 8-1. Corrosion Evaluation

Retaining Wall	Maximum or Minimum Test Value (as appropriate) [1]			
	Chlorides, ppm (Threshold = >500)	Sulfates, ppm (Threshold = >2,000)	Resistivity, ohm-cm (Threshold = <1,000)	pH (Threshold = <5.5)
346L	115	60	9438	8.09
349L	314	413	1307	7.05
353R	341	98	1488	8.39
357R	80	25	8894	8.14
357R1	176	271	1670	7.76
358L	55	54	1634	7.96
361L	23	8	11253	8.49
361L1	228	61	1670	7.64
361R	100	34	8168	8.88

[1] ppm = parts per million; ohm-cm = ohm-centimeter

9.0 FOUNDATION RECOMMENDATIONS

The geotechnical conditions encountered along the retaining wall alignments are suitable for design using Caltrans Standard Plans and Specifications (Caltrans, 2000b and 1999b). Continuous spread footings may be used to support the retaining walls.

The following sections discuss relevant geotechnical design issues related to the adoption of Caltrans Standard Plans and Specifications.

9.1 FOOTING DEPTH AND BEARING

The majority of footings should bear on earth materials with suitable strength and compressibility characteristics. However, the existing site investigation data suggest a potential exists for encountering different earth materials that may require local modification of the footing depth. Based on the interpretations of the existing site investigation data, this condition could occur but is not limited to the following locations:

- **Retaining Wall 346L** - The proposed footing elevation is interpreted to be near a contact between fill and native soil.
- **Retaining Wall 349L** - The proposed footing elevation within the north end of the wall is interpreted to be near a contact between fill and weathered bedrock.
- **Retaining Wall 357R** - The proposed footing elevation within the south portion of the wall is interpreted to be near a contact between fill and native soil (south end) and fill and weathered bedrock.
- **Retaining Wall 357R1** - The proposed footing elevation within the center of the wall is interpreted to be near a contact between native soils and weathered bedrock.
- **Retaining Wall 358L** - The proposed footing elevation at the north end of the wall is interpreted to be near a contact between native soils and weathered bedrock.
- **Retaining Wall 361L** - The proposed footing elevation at the south end of the wall is interpreted to be near a contact between fill and native soil.
- **Retaining Wall 361L1** - The proposed footing elevation at the north end of the wall is interpreted to be near a contact between native soil and weathered bedrock.
- **Retaining Wall 361R** - The proposed footing elevation at the southern half of the wall is interpreted to be near a contact between fill and native soil.

The Resident Geotechnical Engineer (Engineer) should observe the footing excavations. Once the footings are excavated, minor excavation and testing (i.e., pot-holing, rodding, and probing) may be needed to evaluate foundation bearing. The contract documents should include special provisions to accommodate such changes along with a separate letter that reminds the contractor of the possibility of foundation modifications during construction. These provisions should allow the Engineer to instruct the contractor to:

1. Locally remove and recompact (or replace with Class C concrete) potentially unsuitable foundation soils where the thickness of this material is not large.
2. Install vertical construction joints at specific locations, which may not coincide with those shown on the drawings, if the footing excavation encounters a significant thickness of potentially unsuitable soils.
3. Remove the potentially unsuitable soils along a substantial length of the alignment to expose competent earth materials, such as weathered bedrock, and replace the potentially unsuitable foundation soils with structure backfill or compacted gravel covered with filter fabric.

Where the design toe pressures are relatively high and/or the footing will be embedded in a slope that descends away from the wall (e.g., Retaining Walls 353R, 357R1, and 361L1), construction modifications should embed the footing uniformly within one competent earth material, such as weathered bedrock, along the entire alignment.

Where the design toe pressures are relatively low and/or the footing will be embedded in level ground in front of the wall (e.g., Retaining Walls 349L, 357R, 361L, and 361R), more local removal with the special placement of construction joints (i.e., options 1 and 2 above) should be suitable.

The Engineer should prescribe the above measures based on interpretations of the soil/rock conditions exposed from the footing excavations and the design guidance in this report. However, the presence of groundwater at the proposed footing level for Retaining Wall 346L and the locally low SPT N-values (less than 10) indicate that additional footing excavation is needed to provide acceptable long-term performance for settlement and stability. Site-specific recommendations for Retaining Wall 346L, which could also apply to Retaining Wall 349L, depending on conditions encountered during construction are provided below:

- The footing excavation should remove the native soils to the underlying weathered bedrock. This removal should occur along the entire alignment and should conform to the standard specifications for structure backfill (Section 19-3.06). The removal limits should extend horizontally to a distance equal to a 1:1 line projected down and outward from the bottom forward edge of the wall footing to the top of the "approved" weathered bedrock.
- The removed soils should be replaced with structure backfill (or compacted gravel covered with filter fabric). Class II aggregate base could be used in the upper 0.5 meter of the structure backfill to facilitate formwork construction. The gravel fill could also ease the dewatering needed for footing excavation and construction.
- Alternatively, the footing excavation could be deepened to embed within weathered bedrock along the entire alignment. This embedment could increase the design height another 1,500 to 2,000 mm. However, the corresponding increased toe pressure should not exceed the ultimate soil/rock-bearing capacity of the weathered bedrock by a factor of 3 or more, as described in Section 9.3 of this report.

9.2 EXTERNAL WALL STABILITY

External wall stability comprises the overall (slope stability, refer to Section 6.0), overturning and sliding limit states.

Based on the current interpretation of the slope geologic conditions, slope stability analyses indicate that the retaining walls should be stable when positioned within sloping ground above and below the wall (i.e., overall stability of the slope is satisfied).

Overturning stability should be satisfied since the design toe pressures are less than the interpreted allowable bearing capacity of the foundation soils (refer to Section 9.3). This assessment assumes that the footing embedment is modified as needed for the actual soil/rock conditions exposed during construction (refer to Section 9.1).

Sliding stability should be satisfied with the minimum vertical footing embedment (450 to 500 mm) shown on the Standard Plans. However, where sloping ground descends away from the front of the wall, the footing should be deepened as follows:

- <u>A vertical distance from the lowest adjacent grade that provides "infinite" level ground above the top forward edge of the wall footing</u>. Such conditions occur but may not be limited to Retaining Walls 361L1 (e.g., Stations 361+40 to 362+00), 357R1 (e.g., Stations 358+80 to 359+00), and 353R (e.g., Stations 355+60 to 355+80). At these or other locations with similar geometry, it should be economically feasible to deepen the footings to provide infinite level ground in front of the wall.
- <u>A horizontal distance of at least 2.4 meters from the slope face to the top forward edge of the wall footing</u>. Such conditions occur, but may not be limited to Retaining Walls 361L1, 357R1 and 353R, where relatively substantial cut slopes will be formed below the footings.

Caltrans Bridge Design Specifications (2000c) recommends a minimum horizontal distance of 1.2 meters between the near face of the footing and the face of the finished slope. The additional horizontal setback described above is recommended considering the potential disturbance created by forming the cut slope below the wall footing (possible reduction in passive resistance) and the relatively large driving forces created by retaining sloping ground behind Retaining Walls 361L1, 357R1, and 353R. Further, the weathered bedrock materials should be able to accommodate the increased toe pressures created by additional embedment (i.e., increase in design height) as described in the Section 9.3 below.

9.3 TOE PRESSURE

The ultimate soil/rock-bearing capacity of the earth materials expected to support the retaining wall footings should exceed the design toe pressures by a factor of 3 or more, considering typical methods (e.g., Terzaghi, Hansen) of bearing capacity assessment. The following paragraphs discuss the assessment of soil/rock-bearing capacity (a summary of which is presented in Table 9-1). Appendix III provides typical calculations.

Effective stress methods were used to calculate the bearing capacity for soil units (fill and native soil). This method was also used to evaluate the bearing capacity of the weathered bedrock. However, the evaluation of the bearing capacity of weathered bedrock referenced published values and methods commonly used for sound rock, as described below.

Published allowable (presumed) bearing capacities are within the range of required design toe pressures (maximum of 310 kPa). Published bearing capacities for soft, poorly cemented sandstone and cemented mudstone range from 250 to 500 kPa (Tomlinson, 1996). Published bearing pressures for very dense sand and gravel (SPT N > 50) range from 500 to 600 kPa for footings that are 2 to 4 meters wide (Tomlinson, 1996). These capacities can be considered applicable to the soft, poorly cemented (friable) sandstone encountered at the project site.

Allowable bearing pressure for rock is often assessed using the unconfined compressive strength (UCS). The Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1992) recommends estimating the allowable bearing pressure of sound rock as 10 to 40 percent of the unconfined compressive strength, depending on the spacing of discontinuities. This method is generally not applicable to the weathered bedrock materials. However, unconfined compressive strength testing was performed on a few intact samples for additional interpretative information to assess bearing capacity.

9.4 SETTLEMENT

The Caltrans Standard Plans and Specifications (Caltrans, 2000a and 1999b) do not identify allowable settlement tolerances. However, the wall should not tilt excessively since the design toe pressures are less than the interpreted bearing capacity of the foundation soils.

The potential for adverse differential settlement along the length of the retaining walls should be low since their footings should bear on earth materials with relatively similar compressibility characteristics. In addition, there are several measures that can be adopted during construction to mitigate the potential for differential settlement if unsuitable or dissimilar materials are encountered during footing excavation (refer to Section 9.1).

If settlement occurs, most of it should take place during construction.

9.5 DESIGN AND DRAINAGE

Weepholes and a wall drain with filter/drainage materials should be provided as shown on Standard Plan B3-8. Prefabricated drain composites may be substituted subject to approval by the Engineer.

Excessive groundwater, existing or proposed water bearing utilities, and/or surface conditions that could require additional subsurface drainage provisions were not encountered or known to be planned as part of the project development, except where noted in the following paragraphs. However, as noted in Section 1.4.2, the manager (and his geotechnical consultant) reported problems with groundwater at a commercial property above Retaining Wall 357R1. The contract documents should include special provisions that allow the Engineer to instruct the contractor to place additional subsurface drainage if geologic mapping of the wall backcut exposes seeps or other similar forms of groundwater that have the potential to exceed the capacity of the standard wall drain.

Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Site-specific recommendations for Retaining Wall 346L, which could also apply to Retaining Wall 349L, depending on conditions encountered during construction are provided below:

- Groundwater was encountered near the proposed footing elevations for Retaining Wall 346L. In addition, because this wall is located near a small northerly trending ephemeral drainage course, there is potential for groundwater to regularly reach levels that are above the wall footings and/or exceed the capacity of the standard wall drain.
- The drainage of this wall should be amended to provide base and inclined drainage systems with detail similar to the wall drain shown on Standard Plan B3-8. The wall, base, and inclined drains should form one continuous drainage system. The wall drain should extend down to the top of footing and connect to a base drain placed along the top of the footing. The top of footing should be formed to provide a fall of at least 1:100 towards the wall stem. The inclined drain should connect to the base drain and be placed along the backcut formed to construct the wall. The height of the inclined drain should match the wall drain.
- Additional weepholes should be provided at the base of the wall stem/top of footing. These weepholes should be formed with a plastic pipe that extends (daylights) to the ground surface in front of the wall. The plastic pipe should be above finished grade at the outlet. For long-term maintenance, it would be prudent to provide suitable protection and a permanent marker for easy locating in the future.
- Figures I-8 and I-9 (in Appendix I) illustrate the proposed wall drainage.

9.6 CEMENT

In accordance with Caltrans (1999a), Type I-P (MS) modified or Type II modified cement may be used for cast-in-place concrete.

9.7 RETAINING WALL BACKFILL MATERIAL

Soils placed behind retaining walls should consist of fill materials conforming to Section 19-3.06, Structure Backfill, of the Standard Specifications. Those involved in contract development may want to consider including a special provision that would bring a bidding contractors' attention to the need for 95 percent relative compaction of the retaining wall structure backfill when it is placed below surfacing (e.g., Retaining Walls 346L, 349L, 357R, 361L, and 361R).

9.8 SEISMIC DESIGN

The soils that will surround the retaining wall foundations may be characterized as "competent" using the indicators provided by Caltrans (2001).

Table 9-1. Ultimate Soil/Rock Bearing Capacity

Wall No.	Footing Width (mm)	Soil / Rock Unit Interpreted to Predominately Occur at Proposed Footing Level	Design Toe Pressure (kPa)	Ultimate Bearing Capacity (kPa)
346L	1900	Native Soil	110	650
349L	2200	Weathered Bedrock: Volcanics	160	2690
353R	4350	Weathered Bedrock: Volcanics (south end) to Sandstone (north end)	310	1700
357R	1550	Weathered Bedrock: Sandstone	105	1300
357R1	4350	Weathered Bedrock: Claystone	310	1430
358L	1900	Native Soil (south end) to Weathered Bedrock: Sandstone (north end)	110	650
361L	2200	Fill	135	890
361L1	3050	Weathered Bedrock: Claystone (north end) to Weathered Bedrock: Sandstone (south end)	200	1350
361R	1300	Fill with Native Soil (south end)	90	630

NOTE: All footings are assumed to be either embedded 450 mm below the lowest adjacent grade with "infinite" level ground in front of the wall or set back from descending slopes in front of the wall that approximates "infinite" level ground.

10.0 CONSTRUCTION CONSIDERATIONS

10.1 EXCAVATION CHARACTERISTICS

10.1.1 Footing Excavation

Trench excavation in fill and native soil, and the weathered bedrock encountered at footing elevations planned for Retaining Walls 346L, 349L, 357R, is expected to encounter no unusual difficulty using modern trenching machines or backhoes. The contractor's choice of excavation method should not disturb the earth materials at the planned footing elevation or allow the surface to become contaminated with slough and/or water. Footing excavations at the other walls are likely to be part of mass excavation, the characteristics of which are described below.

10.1.2 Mass Excavation

Conventional earth moving equipment (dozers, scrapers, etc.) should be able to excavate the fill and native soil units encountered at the walls with no unusual difficulty.

Excavation characteristics of the weathered bedrock vary according to rock type, degree of weathering and fracturing, depth of cut, and method of excavation. However, modern excavating equipment (e.g., D-9 and D-8 Caterpillar Tractor with No. 9 Series D Ripper and No. 8 Series D Ripper) should be able to excavate (rip) the sandstone and claystone weathered bedrock to the planned footing depths with no unusual difficulty. Some of the sandstone may be locally resistant. In these materials, hollow-stem-auger test borings were able to terminate at depths below the planned footing elevations (north portion of Retaining Walls 353R, 357R1, 358L, and 361L1).

Modern excavating equipment may also be able to excavate the volcanic weathered bedrock with no unusual difficulty where it is decomposed to moderately weathered (and moderately to intensely fractured). However, hard ripping and local blasting may be needed to reach the planned footing elevations in this area. Moderately to slightly fractured volcanic rock could require heavy ripping or local blasting for removal. Hollow-stem-auger test borings terminated at sampler/auger refusal at depths above the planned footing elevations along the south end of Retaining Wall 353R, where intruded volcanics where encountered. The existing cut slope in the volcanics near the southern end of wall 353R appears to have been excavated by ripping.

Rippable conditions are typically defined as the excavation needed to produce practical production quantities of fill. Rippability assessment assumes that the excavating equipment is well maintained and operating at factory-specified efficiencies. The choice of excavation method, ripping or blasting, is often a function of economics, level of desired effort, logistics, quality of machinery used, permit conditions, and contractor convenience.

10.2 GROUNDWATER CONTROL

Based on the existing site investigation data, groundwater may need to be diverted or removed to construct the footings for Retaining Wall 346L. Control of groundwater should comprise "unwatering," which is the removal of water from the excavation by gravity drainage, sump pumping, or other similar means. The contractor is responsible for groundwater control. They may use the factual information provided in this report to assess groundwater control needs, and any additional data they may acquire to further evaluate groundwater conditions relative to their proposed method of construction.

10.3 REUSE AS FILL

Surficial organic materials, native soil, and properly processed weathered bedrock materials are not expected to be suitable for reuse as structure backfill behind retaining walls, according to the criteria of Section 19-3.06, Structure Backfill, of the Standard Specifications. Selective mining of portions of the sandstone may be possible to provide structure backfill, subject to testing and approval by the Engineer.

10.4 TEMPORARY SLOPES

The design and excavation of temporary slopes as well as their maintenance during construction is the responsibility of the contractor. The contractor should have a geotechnical or geological professional evaluate the soil/rock conditions encountered during excavation to determine permissible temporary slope inclinations and other measures as required by California OSHA (Cal/OSHA).

Based on the existing data interpreted from the test borings, the design of temporary slopes and benches for planning purposes may assume the conditions summarized in Table 10-1.

The contractor's geotechnical or geological professional may use the information provided in this report to assess the stability of temporary slopes, as well as any additional data they may need to acquire, to prepare a specific temporary slope analysis and design. Existing infrastructure that is within a 2h:1v line projected up from the bottom edge (toe) of temporary slopes should be monitored during construction.

The contractor should note that the materials encountered in construction excavations may vary significantly across the site. The above assessment of soil type for temporary excavations is based on preliminary engineering classifications of material encountered in widely spaced excavations. The contractor's geotechnical or geological professional should observe and map mass excavations and temporary slopes at regular intervals during excavation and assess the stability of temporary slopes, as necessary.

10.5 CLEARANCE FOR CONSTRUCTION OPERATIONS

Lane closures and the redirection of traffic to accommodate construction of the retaining wall should consider the plan limits of the potential footing excavations. The working face between the traffic faces of temporary railings can be developed by combining the width of the footing and a projection of the temporary slope inclinations from the edge of the footing to the existing ground surface. Such closures/redirections may be need for Retaining Walls 346L, 349L, 357R, 361L, and 361R.

Table 10-1. Temporary Slopes

Retaining Wall	Soil/Rock Unit Interpreted as Predominately Occurring Within the Excavation	Maximum Temporary Slope Height (m) [1,2]	Interpreted Cal/OSHA Soil Type
346L	Fill	5 - 7	C
349L	Fill	5 - 7	B [3]
353R	Weathered Bedrock: Volcanics (south), Sandstone and Claystone (north)	13 - 18	B [4]
357R	Weathered Bedrock: Sandstone (with a local intervening layer of Native Soil)	<2	B [3]
357R1	Weathered Bedrock: Sandstone and Claystone	13 - 18	B [4]
358L	Weathered Bedrock: Sandstone and Claystone	>2	B [4]
361L	Fill	5 - 7	B [3]
361L1	Weathered Bedrock: Sandstone and Claystone	12 - 15	A
361R	Fill	2 - 4	B [3]

[1] Approximate height considering existing site conditions and assuming temporary slopes formed at 1h:1v to 1h:1.5v.
[2] Sloped excavations with heights greater than 6 meters require a design by a California Registered Civil Engineer.
[3] Material subject to vibration from traffic.
[4] Material is part of a sloped layered system where the layers dip in the excavation on a slope less steep than 4h:1v.

11.0 SLOPE PLANTING AND MAINTENANCE

Normal deterioration of slope surfaces may be reduced by landscaping upon completion of grading activities. Proper vegetative cover, watering, and drainage control, along with adequate maintenance will reduce the potential for surficial slope instability. Burrowing rodent activity on the slopes should not be permitted.

The slopes should be planted with deep-rooting, lightweight, drought-resistant varieties of grasses or groundcover as recommended by a landscape architect. The use of any sprinkler system should be restricted to provide minimum water necessary for plant growth. Overwatering must be avoided.

All surface drainage should be controlled and directed away from the slopes to avoid slope saturation and the potential for slope instability. Ponded water at the tops of slopes and sheetflow over slope surfaces must not be allowed. Adequate surface and subsurface drainage provisions should be provided. Hillsides will require periodic maintenance to ensure that drainage devices are kept in working condition.

12.0 LIMITATIONS

12.1 REPORT USE

The conclusions and professional opinions presented in this report were developed by Fugro solely for Caltrans for use during the design of Retaining Walls 346L, 349L, 353R, 357R, 357R1, 358L, 361L, 361L1, and 361R along Interstate 15 between Carmel Mountain Road and Camino Del Norte in San Diego, California.

Although information contained in this report may be of some use for other purposes, it may not contain sufficient information for other parties or uses. If any changes are made to the project as described in this report, the conclusions and recommendations in this report shall not be considered valid unless the changes are reviewed and the conclusions and recommendations of this report are modified or validated in writing by Fugro.

12.2 POTENTIAL VARIATION IN SUBSURFACE CONDITIONS

Earth materials can vary in type, strength, and other engineering properties between points of observations and exploration. Additionally, groundwater and soil moisture conditions also can vary seasonally or for other reasons. Therefore, complete knowledge of the subsurface conditions underlying the site is not known. The findings, conclusions, and recommendations presented in this report are based on the findings at the points of exploration as well as the interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed during construction.

12.3 HAZARDOUS MATERIALS

Caltrans advised that areas within their right-of-way were safe for site investigation from a hazardous waste perspective. The scope of services for this task order did not include any environmental assessments for the presence or absence of hazardous/toxic materials in the soil, surface water, groundwater, or atmosphere. Any statements or absence of statements in this report or data presented herein regarding odors, unusual or suspicious items, or conditions observed are strictly for descriptive purposes and are not intended to convey engineering judgment regarding potential hazardous/toxic assessment.

12.4 LOCAL PRACTICE

In performing our professional services, Fugro has used generally accepted geologic and geotechnical engineering principles and has applied that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers currently practicing in this or similar localities. No other warranty, express or implied, is made as to the professional advice included in this report.

12.5 PLAN REVIEW

Users of this report should check that the retaining wall design drawings and the date and/or revisions referenced in this report are current and remain applicable to the project. The recommendations in this report may not be applicable to drawings with dates and/or revisions that differ from those referenced in this report.

12.6 CONSTRUCTION MONITORING

Users of this report should recognize that the construction process is an integral design component with respect to the geotechnical aspects of a project, and that geotechnical engineering is inexact due to the variability of natural and man-induced processes that can produce unanticipated or changed conditions. Proper geotechnical observation and testing during construction are imperative in allowing the Geotechnical Engineer the opportunity to verify assumptions made during the design process. A resident Geotechnical Engineer (either Caltrans or Fugro West staff, as appropriate) should be onsite during construction to observe compliance with the design concepts and geotechnical recommendations, and to allow design changes in the event that subsurface conditions or methods of construction differ from those anticipated.