State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: Mr. Ulysses Smpardos
Project Engineer

Date: August 25, 2011

File: 07-LA-5-PM 29.8
0700021119 (EA 07-1218w1)
Bridge No. 53-3057
Burbank Blvd OC (Replace)

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Burbank Blvd OC (Replace)

1.0 Scope of Work

The Office of Geotechnical Design South 1 (OGDS-1) has prepared this Memorandum to provide the foundation recommendations for the construction of the Burbank Blvd OC (replacement bridge). The foundation recommendations in this report are based on review of the following sources:

- Subsurface information gathered during the recent foundation investigation (2005 to 2008).
- "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).
- General Plans dated June 22, 2010 provided by Structure Design (SD).
- Preliminary Foundation Report (PFR) date August 31, 2007, prepared by OGDS-1.

2.0 Project Description

The existing Burbank Blvd. Overcrossing is a 3-span structure, consisting of reinforced concrete girders for two spans and a reinforced concrete slab for the remaining span. It is proposed to demolish in its entirety and replace it with a new 2-span cast-in-place pre-stressed box girder structure. The new bridge will be built along the existing alignment of Burbank Blvd., but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps.

3.0 Field Investigation and Testing Program

Site-specific field exploration was performed from August 30, 2005 to March 12, 2008. The field investigation included one hollow stem auger boring, four mud rotary borings and two Cone Penetrometer Tests (CPT). Borings were logged and sampled using a Standard Penetration Test (SPT) sampler and 2-inch tube sampler at selected intervals. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inch. Following drilling, sampling and logging, the borings were backfilled with bentonite chips, and patched with cold asphalt.

A summary of exploratory borings is presented in Table No. 1.

LOTBs (Log of Test Borings) are being prepared by the Office of Geotechnical Support and will be submitted to your office upon completion.

Table No. 1 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
R-08-014	3/11/08-3/12/08	1572+09	156 L	I-5 C/L	615.2	103.2	Not encountered.
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CPT-07-017	12/5/07	1574+32	244.7 L		619.6	61.5	
CPT-07-17A	12/6/07	1574+28	240.6 L		619.7	60.7	
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R-05-005	8/30/05	1572+84	5.9 R		589.3	59.0	

Note: Vertical datum NAVD 88

3.0 Laboratory Testing Program

The following laboratory tests were performed on some selected samples obtained from the borings:

- Particle Size analyses (Sieve Analysis and Mechanical Analysis)
- Atterberg Limits
- Direct Shear Test
- Unconsolidated Undrained Triaxial Test (Triaxial UU)
- Consolidation Test
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 2 below), at the Material Laboratory in Los Angeles and at laboratory selected by the geotechnical consultant URS, Corp.

Table 2 – Laboratory Test Methods

Test	Standard
Sieve Analysis	CTM 202
Mechanical Analysis	CTM 203
Atterberg Limits	CTM 204
Direct Shear Test	ASTM D3080-04
Triaxial UU	ASTM D2850-03
Consolidation	CTM 219
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 Site Geology and Subsurface Condition

5.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of soft to stiff silt and sandy silt and medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness up to approximately 30 feet. The fill consists of poorly graded sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 7.0, Seismic Recommendations).

5.2 Subsurface Conditions

Subsurface soil conditions at the proposed abutment and bent locations was determined based on the five borings drilled for this project and two CPT soundings shown in Table 1. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, the alluvium is composed of soft to stiff silt and sandy silt and loose to dense sand with fine to coarse gravel and cobbles.

5.3 Groundwater

Groundwater was not encountered during the 2005-2009 investigation for this project to the total depth explored of approximately 103 feet below ground surface (elevation +512 feet) (in Boring No. R-08-014). The elevation of the existing ground surface along the proposed bridge alignment ranges from approximately +620 feet to +583 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo . The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

6.0 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 3.

Table No. 3 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
R-08-014	7.0-101.0	9.4	9300	-	-
R-08-015	6.0-102.0	9.4	6700	-	-
A-05-002	15.5	8.8	-	108	60

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:
Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

7.0 Seismic Recommendations

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 4 below.

Table No. 4 - Summary of Faults

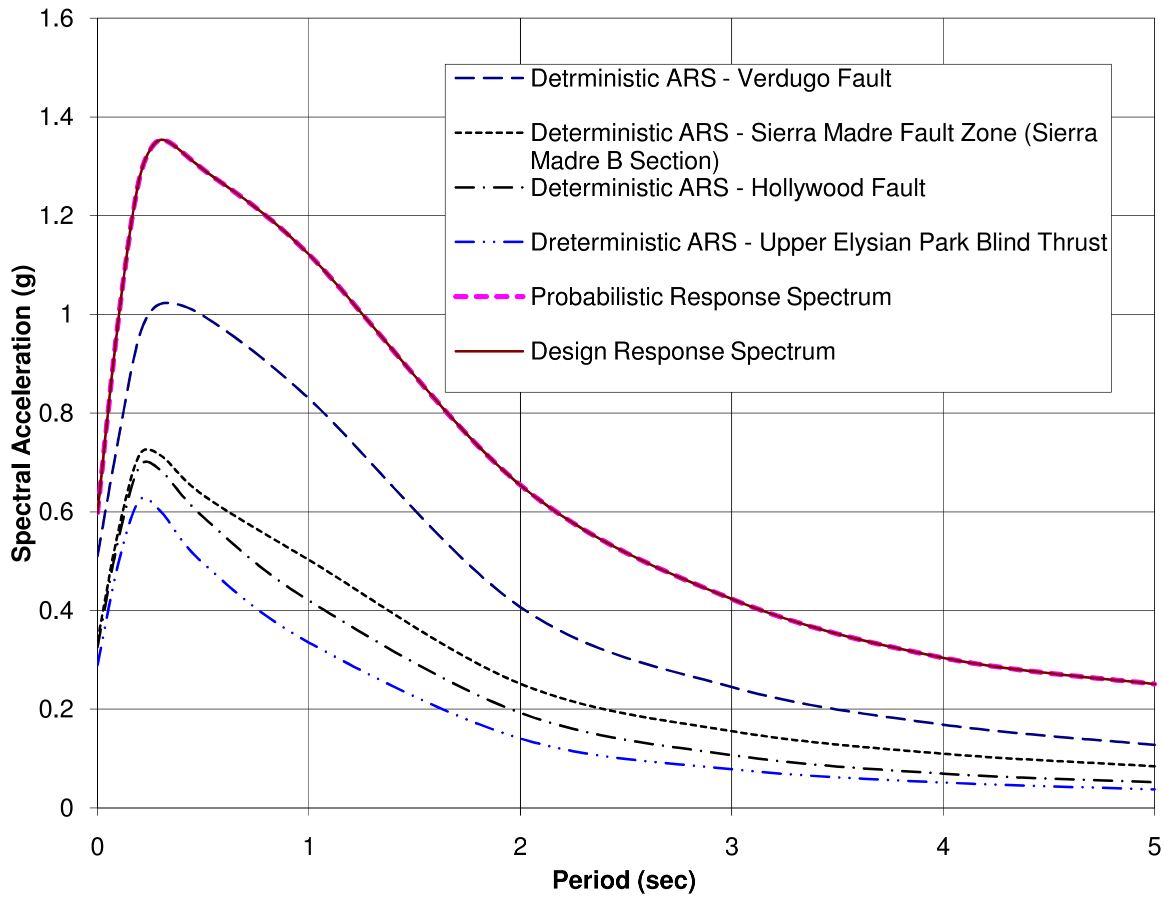
Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/ km)	R_{JB} (mile/ km)	R_{RUP} (mile/ km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure 1. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure 1. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

Figure 1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for Burbank Blvd OC
Damping Ratio = 5%; V_{s30} = 295 m/sec



5.1 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading.

Due to the fact no groundwater was encountered at the site, the liquefaction potential is considered to be low.

6.0 Foundation Recommendations

6.1 Foundation Data Provided by Structural Designers

The foundation design data and foundation loads were provided by the Structural Designers. Table No. 5 shows the foundation design data. Table No.6 shows the foundation design loads.

Table No. 5 – Foundation Design Data Table

Location	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-Off Elevation (ft)	Pile Cap Size		Permissible Settlement Under Service Load (in)	Number of Piles
					B (ft)	L (ft)		
Abut 1 - Lt	WSD	24" Dia x 0.5" Steel Pipe Pile	584.89	574.92	24' - 6.0"	87' - 8.5"	1	66
Abut 1 - Rt	WSD		582.95	573.02	24' - 6.0"	98' - 9.5"	1	84
Bent 2 - Lt	LRFD		587.74	579.82	16' – 0.0"	16' – 0.0"	1	9*
Bent 2 - Rt	LRFD		586.49	578.52	16' – 0.0"	16' – 0.0"	1	9*
Abut 3 - Lt	WSD		591.52	583.32	14' - 6.0"	63' – 6.0"	1	29
Abut 3 - Rt	WSD		590.33	581.82	14' - 6.0"	118' - 5.5"	1	53

*Note: Each pile cap at Bent 2 contains a 3 x 3 arrangement of piles, for a total of 9 piles per cap.
Bent 2 - Lt consists of 3 columns with 3 pilecaps, for a total of 27 piles
Bent 2 - Rt consists of 3 columns with 3 pilecaps, for a total of 27 piles

Table No. 6 – Foundation Design Loads

Location	Service-I Limit State (k)			Controlling Strength Limit State (k)				Controlling Extreme Event Limit State (k)			
	Total Load		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
Abut 1 - Lt	10,510	200	10,132	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 1 - Rt	11,870	200	11,443	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2 - Lt	2,727	N/A	2,167	3,806	423	0	0	2,853	412	0	0
Bent 2 - Rt	2,727	N/A	2,167	3,806	423	0	0	2,853	412	0	0
Abut 3 - Lt	4,026	200	3,738	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 3 - Rt	7,244	200	6,727	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: At Bent 2, the loads shown "per support" are the loads at a single pile cap with a 3 x 3 pile arrangement.

6.2 Axial Pile Capacity

Axial capacity for individual piles and pile group were evaluated using the computer program DRIVEN. Foundation recommendations for the bent 2 and abutments are provided in the Table No. 7 and Table No. 8 respectively below.

Table No. 7 – Foundation Design Recommendations for Bent 2

Support No.	Pile Type	Cut-off elev. (ft)	Service-1 Limit State Load (kips) per Support		Total Permiss. Support Settle. (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Speci. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
						Strength Limit		Extreme Event				
			Total	Perm.		Comp. (ϕ= 0.7)	Tension (ϕ= 0.7)	Comp. (ϕ= 1)	Tension (ϕ= 1)			
Bent 2-Lt	24" Dia x 0.5" Steel Pipe Pile	579.82	2,727	2,167	1	423	0	412	0	534 (a-I) 546 (a-II) 568 (c) xxx (d)	See Note (2)	610
Bent 2-Rt		578.52	2,727	2,167	1	423	0	412	0	532 (a-I) 544 (a-II) 566 (c) xxx (d)	See Note (2)	610

Notes:

1. Design Tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Specified Tip Elevation will be provided in the Final Foundation Report.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

Table No. 8 – Foundation Design Recommendations for Abutments

Support No.	Pile Type	Cut-off elev. (ft)	\ LRFD Service-1 Limit State Load (kips) per Support		Total Permiss. Support Settle. (inches)	LRFD Service-1 Limit State Total Load (kips) per Pile	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specif. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
			Total	Perm.						
Abut1-Lt	24" Dia x 0.5" Steel Pipe Pile	574.92	10510	10132	1	200	400	538 (a) 553 (c) xxx (d)	See Note (2)	400
Abut1-Rt		573.02	11870	11443	1	200	400	538 (a) 558 (c) xxx (d)	See Note (2)	400
Abut3-Lt		583.32	4026	3738	1	200	400	549 (a) 571 (c) xxx (d)	See Note (2)	400
Abut3-Rt		581.82	7244	6727	1	200	400	545 (a) 552 (c) xxx (d)	See Note (2)	400

Notes:

1. Design Tip elevations are controlled by: (a) Compression (Strength Limit), (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Specified Tip Elevation will be provided in the Final Foundation Report.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

6.3 Lateral Pile Capacity

Lateral pile capacity analyses will be performed by Structure Design.

6.4 Pile Data Table

Table No. 14- Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specif. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
		Compression	Tension			
Abut1-Lt	24" Dia x 0.5" Steel Pipe Pile	400	0	538 (a) 553 (c) xxx (d)	See Note (2)	400
Abut1-Rt	24" Dia x 0.5" Steel Pipe Pile	400	0	538 (a) 558 (c) xxx (d)	See Note (2)	400
Bent 2-Lt	24" Dia x 0.5" Steel Pipe Pile	610	0	534 (a) 568 (c) xxx (d)	See Note (2)	610
Bent 2-Rt	24" Dia x 0.5" Steel Pipe Pile	610	0	532 (a) 566 (c) xxx (d)	See Note (2)	610
Abut3-Lt	24" Dia x 0.5" Steel Pipe Pile	400	0	549 (a) 571 (c) xxx (d)	See Note (2)	400
Abut3-Rt	24" Dia x 0.5" Steel Pipe Pile	400	0	545 (a) 552 (c) xxx (d)	See Note (2)	400

Notes:

1. Design Tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Specified Tip Elevation will be provided in the Final Foundation Report.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

6.5 Bridge Approach Embankments

There will be some new embankment fill behind the Abutment1 as Burbank Blvd will be made wider. All of the fill behind the Abutment 3 will be new. Fills should be placed and compacted in accordance with the Section 19-6 of the Caltrans Standard Specifications (2006). Settlement should be fairly rapid at this project site as material is mostly coarse granular. OGDS1 recommends a fill settlement period of up to 30 days for the widening; however, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field.

The downdrag potential on proposed piles in foundation soils due to new fill will be mitigated by building up new embankment material to grade, allowing new embankment and existing soils to settle for the recommended time period (up to 30 days settlement period or as determined by the structure representative), then excavating down to footing grade followed by pile installation.

7.0 Construction Considerations

- Earthwork should be performed in accordance with Section 19 of the latest Caltrans Standard Specifications.

- The pile section for the "24-inch Dia x 0.5-inch" Steel Pipe Piles is generally thick enough to penetrate through hard driving conditions in dense to very dense sand and some gravel layers. Generally open-ended pipe piles with diameter 24 inches or greater tend not to plug. However, if hard driving is encountered, center relief drilling through open-ended steel pipe piles can be used to advance the pile with the approval by Resident Engineer. When center relief drilling is used, the pipe piles should be driven past center relief drilling depth, approximately 4 pile diameters in length, before reaching specified pile tip elevation.

- Groundwater is not anticipated during construction. However, if ground water is encountered within excavations, it is the responsibility of the contractor to control ground water during construction.

- Based on soil types encountered during the recent investigation, OGDS1 recommends a slope ratio of 1:1.5 (V:H) or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations. Any temporary sloping or shoring should be made the contractor's responsibility. For shoring or shoring systems, working drawings and calculations should be submitted to the Resident Engineer prior to placing shoring.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 08-25-2011

Deepa Wathugala, Ph.D., P.E., G.E.
Transportation Engineer
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GS Corporate – Mark Willian (Electronic File)
Structure Construction – RE_Pending_File@dot.ca.gov (Electronic File)
DES Office Engineer, Office of PS&E (Electronic File)
District Material Engineer (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: Mr. Ulysses Smpardos
Project Engineer

Date: March 2, 2012

File: 07-LA-5-PM 29.8
0700021119 (EA 07-1218w1)
Bridge No. 53-3057
Burbank Blvd. Overcrossing
Revision 1

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Revised Foundation Report for Burbank Blvd. Overcrossing

1.0 Scope of Work

On August 25, 2011, the Office of Geotechnical Design South 1 (OGDS-1) has submitted a Foundation Report to provide the foundation recommendations for the Burbank Blvd. Overcrossing (a replacement bridge). After that date, your office informed us that at Abutment 1 Left, some of the proposed pipe piles will be replaced by CIDH piles in order to protect the nearby existing culvert, Stough Canyon Channel. This Memorandum was prepared to provide the revised foundation recommendations for the Burbank Blvd. Overcrossing (OC). These revised foundation recommendations shall have the precedence over the recommendations given in the Foundation Report dated August 25, 2011.

The foundation recommendations in this report are based on review of the following sources:

- Subsurface information gathered during the foundation investigation (2005 to 2008).
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5.2 Subsurface Conditions

The subsurface soil conditions at the proposed abutment and bent locations were determined based on the five borings drilled for this project and two CPT soundings shown in Table No. 1. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles interbedded with medium dense to dense silty sand and sandy silt. Below the fill material, the alluvium is composed of soft to very stiff silt, and sandy silt interbedded with soft to very stiff clay to silty clay and loose to very dense sand with fine to coarse gravel and cobbles.

5.3 Groundwater

Groundwater was not encountered during the 2005-2009 investigation for this project to the total depth explored of approximately 103 feet below ground surface (elevation +512 feet) (in Boring No. R-08-014). The elevation of the existing ground surface along the proposed bridge alignment ranges from approximately +620 feet to +583 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo . The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

6.0 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 3.

Table No. 3 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
R-08-014	7.0-101.0	9.4	9300	-	-
R-08-015	6.0-102.0	9.4	6700	-	-
A-05-002	15.5	8.8	-	108	60

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:
Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

7.0 Seismic Recommendations

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 4 below.

Table No. 4 - Summary of Faults

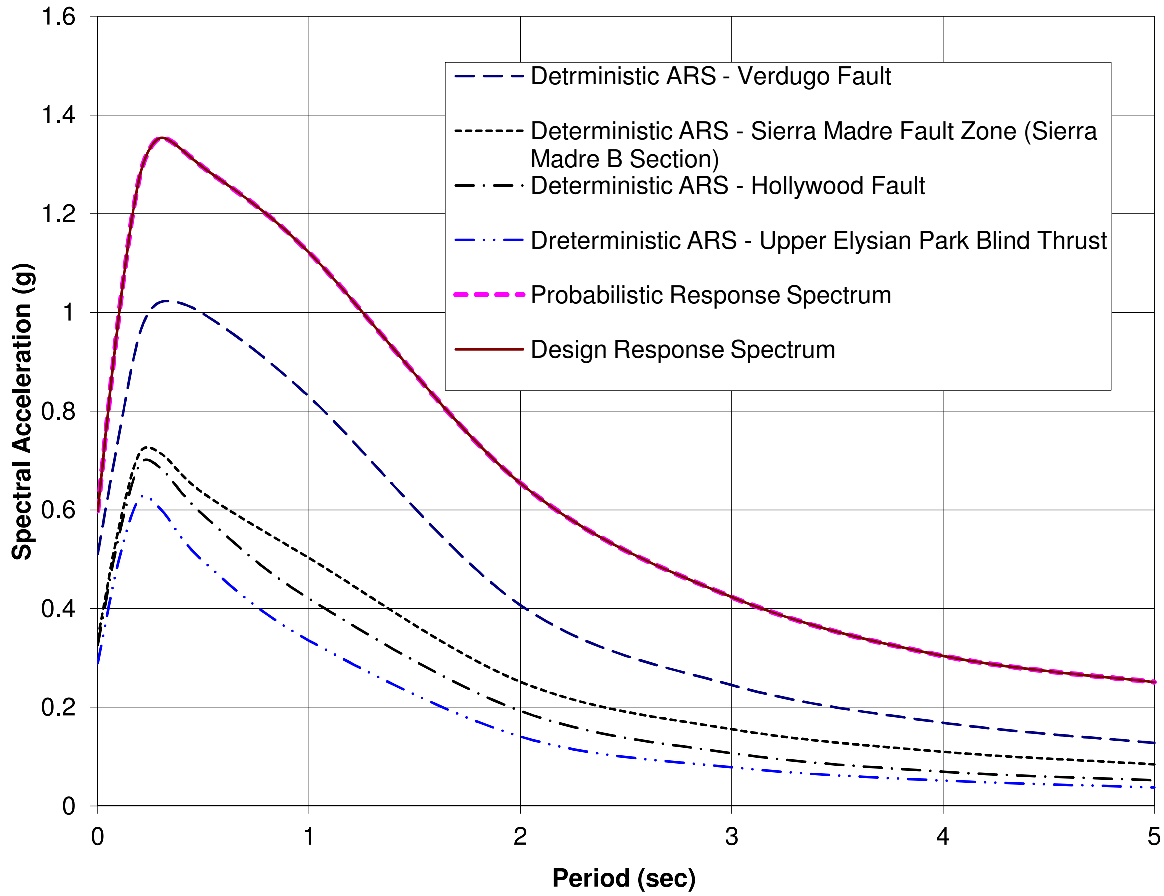
Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/ km)	R_{JB} (mile/ km)	R_{RUP} (mile/ km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure 1. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure 1. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

Figure 1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS) for Burbank Blvd OC Damping Ratio = 5%; V_{s30} = 295 m/sec



5.1 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading.

Due to the fact no groundwater was encountered at the site, the liquefaction potential is considered to be low.

6.0 Foundation Recommendations

6.1 Foundation Data Provided by Structural Designers

The foundation design data and foundation loads were provided by the Structural Designers. Table No. 5 shows the foundation design data. Table No.6 shows the foundation design loads.

Table No. 5 – Foundation Design Data Table

<table>
<tr><th rowspan="2">Location</th><th rowspan="2">Design Method</th><th rowspan="2">Pile Type</th><th rowspan="2">Finished Grade Elevation (ft)</th><th rowspan="2">Cut-Off Elevation (ft)</th><th colspan="2">Pile Cap Size</th><th rowspan="2">Permissible Settlement Under Service Load (in)</th><th rowspan="2">Number of Piles</th></tr>
<tr><th>B (ft)</th><th>L (ft)</th></tr>
<tr><td>Abut 1 - Lt</td><td>WSD</td><td rowspan="6">PP 24X0.500 for all the support locations except Abut 1 –Lt (there are 8 CIDH piles at Abut 1 –Lt)</td><td>587.56</td><td>577.52 for pipe piles, and 577.35 for CIDH piles</td><td>24' - 6.0"</td><td>87' - 8.5"</td><td>1</td><td>70</td></tr>
<tr><td>Abut 1 - Rt</td><td>WSD</td><td>585.26</td><td>575.02</td><td>24' - 6.0"</td><td>98' - 9.5"</td><td>1</td><td>84</td></tr>
<tr><td>Bent 2 - Lt</td><td>LRFD</td><td>587.74</td><td>579.82</td><td>16' – 0.0"</td><td>16' – 0.0"</td><td>1</td><td>9*</td></tr>
<tr><td>Bent 2 - Rt</td><td>LRFD</td><td>586.49</td><td>578.52</td><td>16' – 0.0"</td><td>16' – 0.0"</td><td>1</td><td>9*</td></tr>
<tr><td>Abut 3 - Lt</td><td>WSD</td><td>591.52</td><td>583.32</td><td>14' - 6.0"</td><td>63' – 6.0"</td><td>1</td><td>29</td></tr>
<tr><td>Abut 3 - Rt</td><td>WSD</td><td>590.33</td><td>581.82</td><td>14' - 6.0"</td><td>118' - 5.5"</td><td>1</td><td>53</td></tr>
</table>

*Note: Each pile cap at Bent 2 contains a 3 x 3 arrangement of piles, for a total of 9 piles per cap.
Bent 2 - Lt consists of 3 columns with 3 pilecaps, for a total of 27 piles
Bent 2 - Rt consists of 3 columns with 3 pilecaps, for a total of 27 piles

Table No. 6 – Foundation Design Loads

Location	Service-I Limit State (k)			Controlling Strength Limit State (k)				Controlling Extreme Event Limit State (k)			
	Total Load		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
Abut 1 - Lt	10,510	200	10,132	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 1 - Rt	11,870	200	11,443	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2 - Lt	2,727	N/A	2,167	3,806	423	0	0	2,853	412	0	0
Bent 2 - Rt	2,727	N/A	2,167	3,806	423	0	0	2,853	412	0	0
Abut 3 - Lt	4,026	200	3,738	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Abut 3 - Rt	7,244	200	6,727	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: At Bent 2, the loads shown "per support" are the loads at a single pile cap with a 3 x 3 pile arrangement.

6.2 Axial Pile Capacity

Axial capacity for individual piles and pile group were evaluated using the computer program DRIVEN. Foundation recommendations for the bent 2 are provided in the Table No. 7 below.

Table No. 7 – Foundation Design Recommendations for Bent 2

Support No.	Pile Type	Cut-off elev. (ft)	Service-1 Limit State Load (kips) per Support		Total Permiss. Support Settle. (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Speci. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
						Strength Limit		Extreme Event				
			Total	Perm.		Comp. (ϕ= 0.7)	Tension (ϕ= 0.7)	Comp. (ϕ= 1)	Tension (ϕ= 1)			
Bent 2-Lt	PP 24X0.500	579.82	2,727	2,167	1	423	0	412	0	534 (a-I) 546 (a-II) 568 (c) xxx (d)	See Note (2)	610
Bent 2-Rt		578.52	2,727	2,167	1	423	0	412	0	532 (a-I) 544 (a-II) 566 (c) xxx (d)	See Note (2)	610

Notes:

1. Design Tip elevations are controlled by: (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Design tip elevation for lateral load will be provided and incorporated into the pile data table by Structure Design.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

Foundation recommendations for abutments are provided in the Table No. 8 and Table No. 9 below.

Table No. 8 – Foundation Design Recommendations for Pipe Piles at Abutments

Support No.	Pile Type	Cut-off elev. (ft)	\ LRFD Service-1 Limit State Load (kips) per Support		Total Permiss. Support Settle. (inches)	LRFD Service-1 Limit State Total Load (kips) per Pile	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specif. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
			Total	Perm.						
Abut1-Lt	PP 24X0.500	574.92	10510	10132	1	200	400	538 (a) 553 (c) xxx (d)	See Note (2)	400
Abut1-Rt		573.02	11870	11443	1	200	400	538 (a) 558 (c) xxx (d)	See Note (2)	400
Abut3-Lt		583.32	4026	3738	1	200	400	549 (a) 571 (c) xxx (d)	See Note (2)	400
Abut3-Rt		581.82	7244	6727	1	200	400	545 (a) 552 (c) xxx (d)	See Note (2)	400

Notes:

1. Design Tip elevations are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Design tip elevation for lateral load will be provided and incorporated into the pile data table by Structure Design.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

Table No. 9 – Foundation Design Recommendations for CIDH Piles at Abutments

Support No.	Pile Type	Cut-off elev. (ft)	Total Permiss. Settle. (inches)	LRFD Service-1 Limit State Load per Pile (kips)	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specif. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
					Compression	Tension			
Abut1-Lt	24" Dia CIDH	577.35	1	200	400	0	533 (a) 549 (c) xxx (d)	See Note (2)	N/A

Notes:

1. Design Tip elevations are controlled by: (a) Compression, (b) Tension, , (c) Settlement, (d) Lateral Load.
2. Design Tip Elevation controlled by Lateral Load will be determined by Structure Design. As such, Design tip elevation for lateral load will be provided and incorporated into the pile data table by Structure Design.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

6.3 Lateral Pile Capacity

The p-y data were provided to the bridge designer and the lateral pile capacity analyses will be performed by Structure Design.

6.4 Pile Data Table

Table No. 10 - Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specif. Tip Elev. (ft)	Nominal Driving Resistance Required (kips)
		Compression	Tension			
Abut1-Lt	PP 24X0.500	400	0	538 (a) 553 (c) xxx (d)	See Note (2)	400
Abut1-Lt	24" Dia CIDH Pile	400	0	533 (a) 549 (c) xxx (d)	See Note (2)	N/A
Abut1-Rt	PP 24X0.500	400	0	538 (a) 558 (c) xxx (d)	See Note (2)	400
Bent 2-Lt	PP 24X0.500	610	0	534 (a) 568 (c) xxx (d)	See Note (2)	610
Bent 2-Rt	PP 24X0.500	610	0	532 (a) 566 (c) xxx (d)	See Note (2)	610
Abut3-Lt	PP 24X0.500	400	0	549 (a) 571 (c) xxx (d)	See Note (2)	400
Abut3-Rt	PP 24X0.500	400	0	545 (a) 552 (c) xxx (d)	See Note (2)	400

Notes:
1. Design Tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load.
2. Design tip elevation for lateral load will be provided and incorporated into the pile data table by Structure Design.
3. The specified tip elevation shall not be raised above the design tip elevation for lateral and tolerable settlement.

6.5 Bridge Approach Embankments

There will be some new embankment fill behind the Abutment1 as Burbank Blvd will be made wider. All of the fill behind the Abutment 3 will be new. Fills should be placed and compacted in accordance with the Section 19-6 of the Caltrans Standard Specifications (2006). Settlement should be fairly rapid at this project site as material is mostly coarse granular. OGDS1 recommends a fill settlement period of up to 30 days for the widening; however, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field.

The downdrag potential on proposed piles in foundation soils due to new fill will be mitigated by building up new embankment material to grade, allowing new embankment and existing soils to settle for the recommended time period (up to 30 days settlement period or as determined by the structure representative), then excavating down to footing grade followed by pile installation.

7.0 Notes to Designer

According to the AASHTO LRFD Bridge Design Specification and California Amendment, dynamic formulas such as the Modified Gates Formula should not be used to verify nominal resistance of piles when the required nominal resistance exceeds 600 kips or the pile diameter is greater than 18 inches. Therefore, it is strongly recommended to use pile dynamic testing such as Pile Driving Analyzer (PDA) to verify the required nominal resistances in the plans.

The Special Provisions should specify the following under ***Dynamic Monitoring***.
"The first pile driven at each support location will receive dynamic monitoring."

Before installing driven piles, the Contractor shall provide a driving system submittal, including drivability analysis. The Contract Special Provisions should specify this under the ***Driving System Submittal***.

8.0 Construction Considerations

- Contractor's driving system should be checked to verify that the driving system is capable of installing the proposed piles at the locations of abutments and bent before commencement of driving piles.

- In order to verify the required nominal driving resistance, it is recommended to perform dynamic pile testing such as Pile Driving Analyzer (PDA) instead of using the dynamic formula.

- It is recommended that at least one pile at Abutment 1L, Bent 2 (2L or 2R) and Abutment 3 (3L or 3R) be tested by dynamic pile testing when the pile is initially driven. At Abutment 1L only, the test pile needs to be re-tapped at least five days after the initial driving or installation of adjacent piles. (PDA monitoring usually perform for the first pile in the footprint of the control location). Upon completion of PDA monitoring, FTB (Caltrans Foundation Testing Branch) will generate PDA report with results and pile acceptance criteria (curves). At that time, the production piles within the control location can be released for construction (per results).

- Earthwork should be performed in accordance with Section 19 of the latest Caltrans Standard Specifications.

- The pile section for the "24-inch Dia x 0.5-inch" Steel Pipe Piles is generally thick enough to penetrate through hard driving conditions in dense to very dense sand and some gravel layers. Generally open-ended pipe piles with diameter 24 inches or greater tend not to plug. However, if hard driving is encountered, center relief drilling through open-ended steel pipe piles can be used to advance the pile with the approval by Resident Engineer. When center relief drilling is used, the pipe piles should be driven past center relief drilling depth, approximately 4 pile diameters in length, before reaching specified pile tip elevation.

- Groundwater is not anticipated during construction. However, if ground water is encountered within excavations, it is the responsibility of the contractor to control ground water during construction.

- Based on soil types encountered during the recent investigation, OGDS1 recommends a slope ratio of 1:1.5 (V:H) or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations. Any temporary sloping or shoring should be made the contractor's responsibility. For shoring or shoring systems, working drawings and calculations should be submitted to the Resident Engineer prior to placing shoring.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 3/2/12

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DES Office Engineer, Office of PS&E (Electronic File)
District Material Engineer (Electronic File)

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Date: January 12, 2012

File: 07-LA-5-PM 29.78
07000211191 (EA 07-1218W1)
Abut3-Left Retaining Wall

Attn: MR. ULYSSES SMPARDOS
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Abut3-Left Retaining Wall

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the Abut3-Left Retaining Wall. The foundation recommendations are based on the subsurface information gathered during the foundation investigation (2005) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC.

2.0 Project Description

The existing 3-span Burbank Blvd Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. In addition, two Type 1 Retaining walls will be constructed at Burbank OC Abutment 3 Wing Wall locations. Abut3-Left Retaining Wall is one of them. There are three sections of wall, each with different height as shown in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Structure Type	Wall Height 'H' (ft)	Length (ft)	Bottom of Footing Elevation (ft)	Slope in front of footing
Type 1Wall	22	13.5	592.5	Level
Type 1Wall	14	13.5	599.7	Level
Type 1Wall	8	13.5	606.8	Level

3.0 Geotechnical. Exploration

This wall will be located near the Abut3 of the proposed Burbank Blvd OC. Therefore, site specific geotechnical exploration performed for the proposed bridge Abut 3 based on the boring No. BUR-05-2 is applicable for this wall.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
BUR-05-2	8/31/05	1571+04	10.5 R	Existing I-5 C/L	588.3	61.5	Not encountered.

Note: Vertical datum NAVD 88

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

Due to the vicinity of the subject wall to the Abut 3 of the proposed Burbank Blvd OC., Site Geology for proposed Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on “Site Geology” of the Foundation Report for Burbank Blvd OC (Replcace), dated August 25, 2011.

4.2 Subsurface Condition

Due to the vicinity of the subject wall to the Abut 3 of the proposed Burbank Blvd OC., Subsurface Condition for proposed Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on “Subsurface Conditions” of the Foundation Report for Burbank Blvd OC (Replcace), dated August 25, 2011.

4.3 Groundwater

Due to the vicinity of the subject wall to the Abut 3 of the proposed Burbank Blvd OC., ground water condition for proposed Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on “Groundwater” of the Foundation Report for Burbank Blvd OC (Replcace), dated August 25, 2011.

4.4 Corrosion Evaluation

Due to the vicinity of the subject wall to the Abut 3 of the proposed Burbank Blvd OC., Corrosion Evaluation for proposed Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on "Corrosion Evaluation" of the Foundation Report for Burbank Blvd OC (Replcace), dated August 25, 2011.

4.5 Seismicity

Due to the vicinity of the subject wall to the Abut 3 of the proposed Burbank Blvd OC., Seismicity for proposed Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on "Seismicity" of the Foundation Report for Burbank Blvd OC (Replcace), dated August 25, 2011.

5.0 Foundation Recommendations

5.1 Foundation Analysis

This wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). From a geotechnical standpoint, the wall supported on spread footing is feasible.

First GS provided the following information based on the preliminary information provided by SD such as wall heights, bottom of footing elevations, the potential footing width and the permissible settlement limit.

1) Plots of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figures A1, A2 and A3 in Appendix A).
2) Plots of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figures A4, A5 and A6 in Appendix A).
3) Plots of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figures A7, A8 and A9 in Appendix A).
4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for retained soil and 34^0 for backfill).
5) Unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 3) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis.

Table No. 3 – Wall Data Table

Wall Height, H (ft)	Distance from heel of footing to face of wall, B (ft)	Distance from toe of footing to face of wall, C (ft)	Footing width, W (ft)	Footing thickness, F (ft)
22	11.5	4.5	16.0	2.00
14	9.5	3.0	12.5	1.67
8	4.5	2.0	6.5	1.33

5.2 Global Slope Stability

The slope stability analyses were performed to verify the overall stability using the computer program SLOPEW under both static and pseudo-static conditions. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

6.0 Construction Considerations

1. The proposed wall should be founded on properly compacted competent soil. Loose or soft material is not expected at this project site. If clay or loose sand is encountered within the areas to receive the walls, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:
   - (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
   - (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
   - (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope, or shoring may be required.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 1/12/2012

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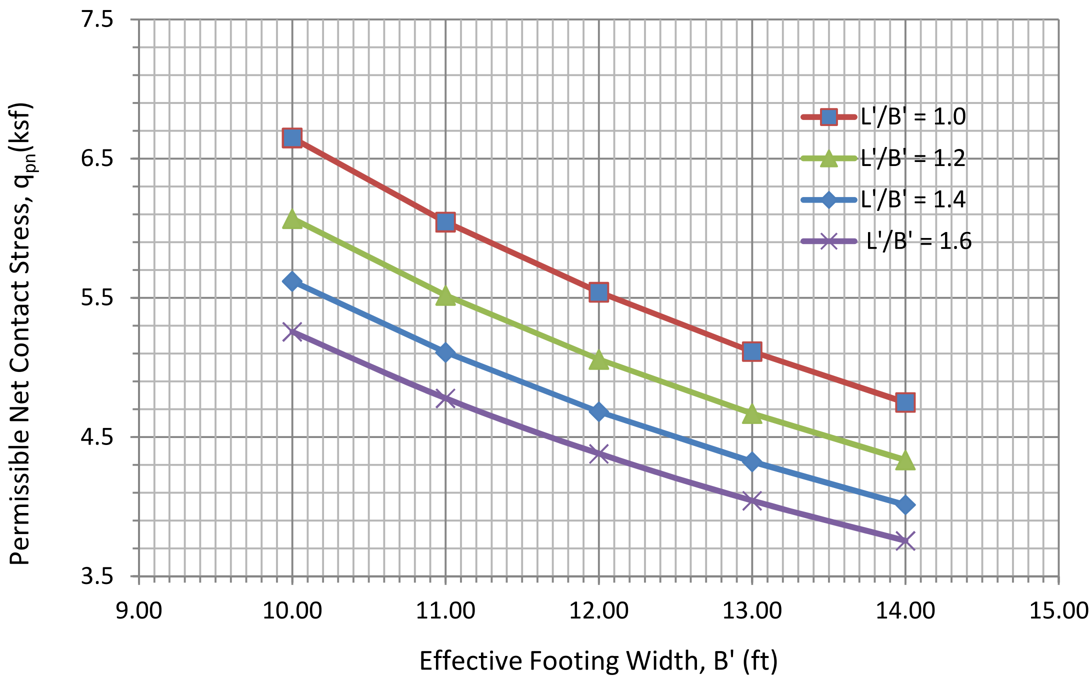
GS Corporate (Mark_Willians@dot.ca.gov)
Structure Construction R.E. Pending File (RE_Pending_File@dot.gov.ca)
DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 -Permissible Net Contact Stress vs. Footing Effective Width (Service Limit State)
For Permissible Settlement = 2"
Wall Height =22'



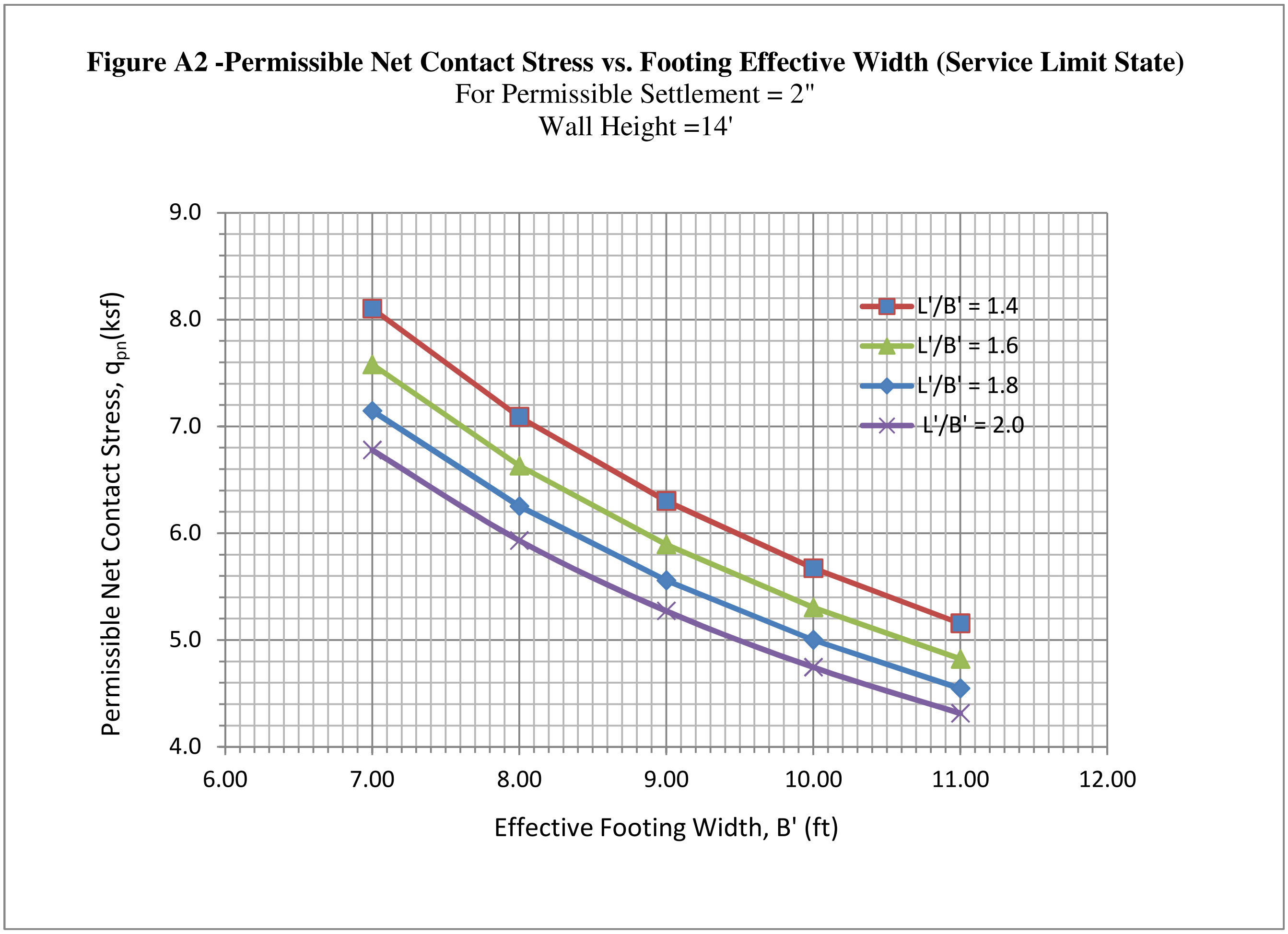


Figure A2 -Permissible Net Contact Stress vs. Footing Effective Width (Service Limit State)
For Permissible Settlement = 2"
Wall Height =14'
Permissible Net Contact Stress, q_{pn}(ksf)
9.0
8.0
7.0
6.0
5.0
4.0
6.00
7.00
8.00
9.00
10.00
11.00
12.00
Effective Footing Width, B' (ft)
L'/B' = 1.4
L'/B' = 1.6
L'/B' = 1.8
L'/B' = 2.0

Figure A3 -Permissible Net Contact Stress vs. Footing Effective Width (Service Limit State)
For Permissible Settlement = 2"
Wall Height =8'

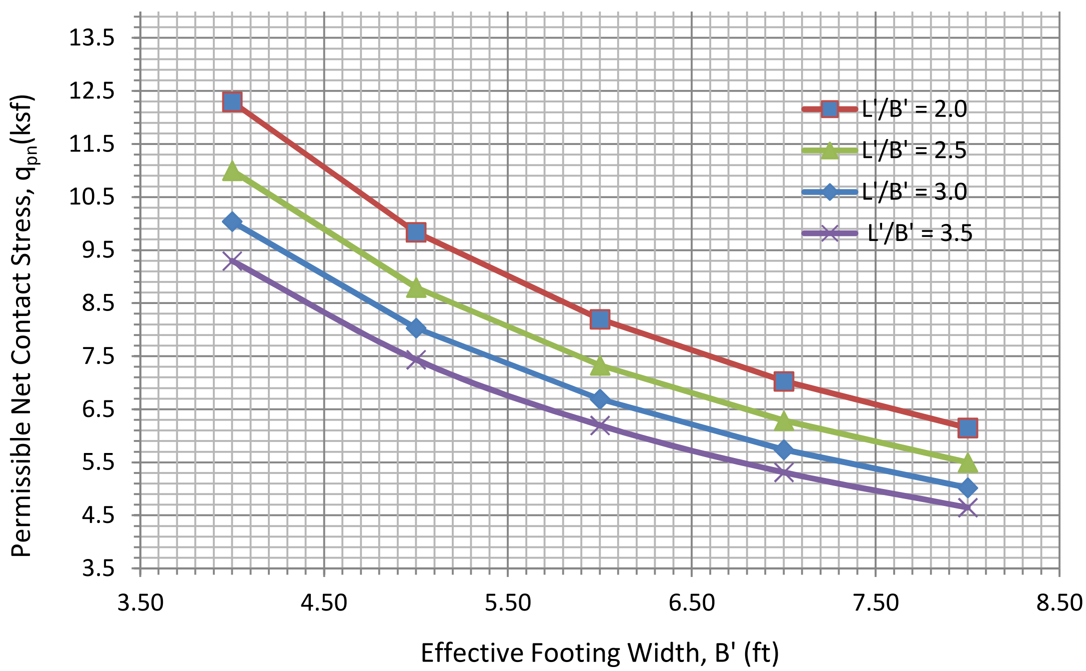
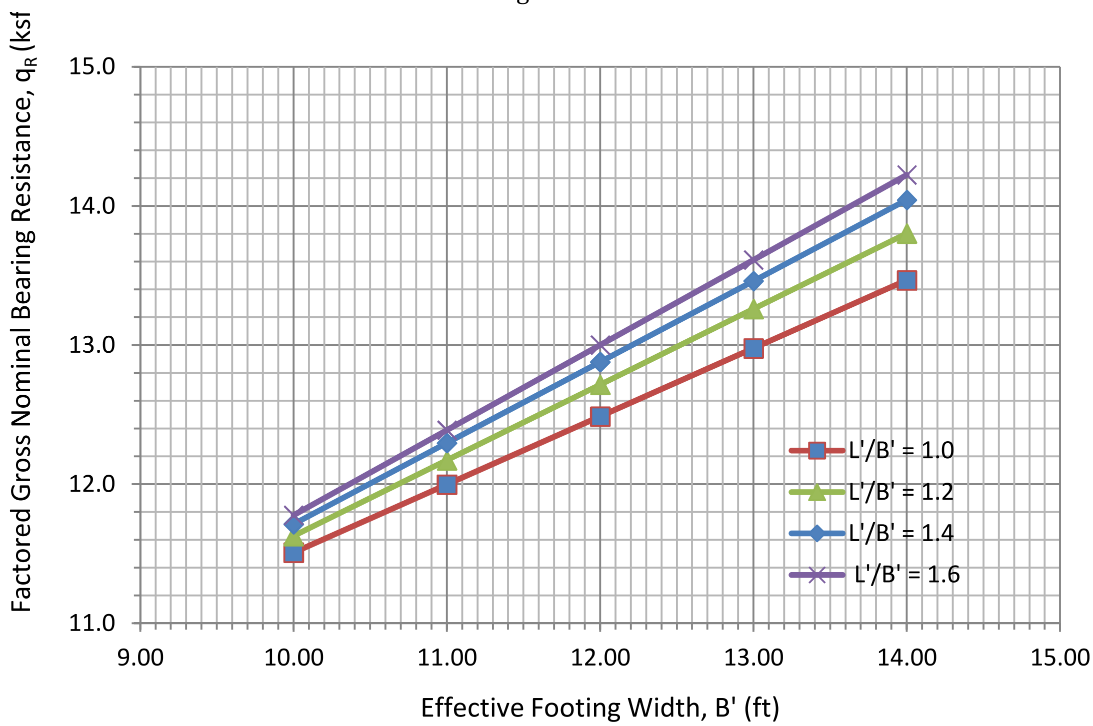


Figure A4 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, φ_b = 0.45)
Wall Height = 22'



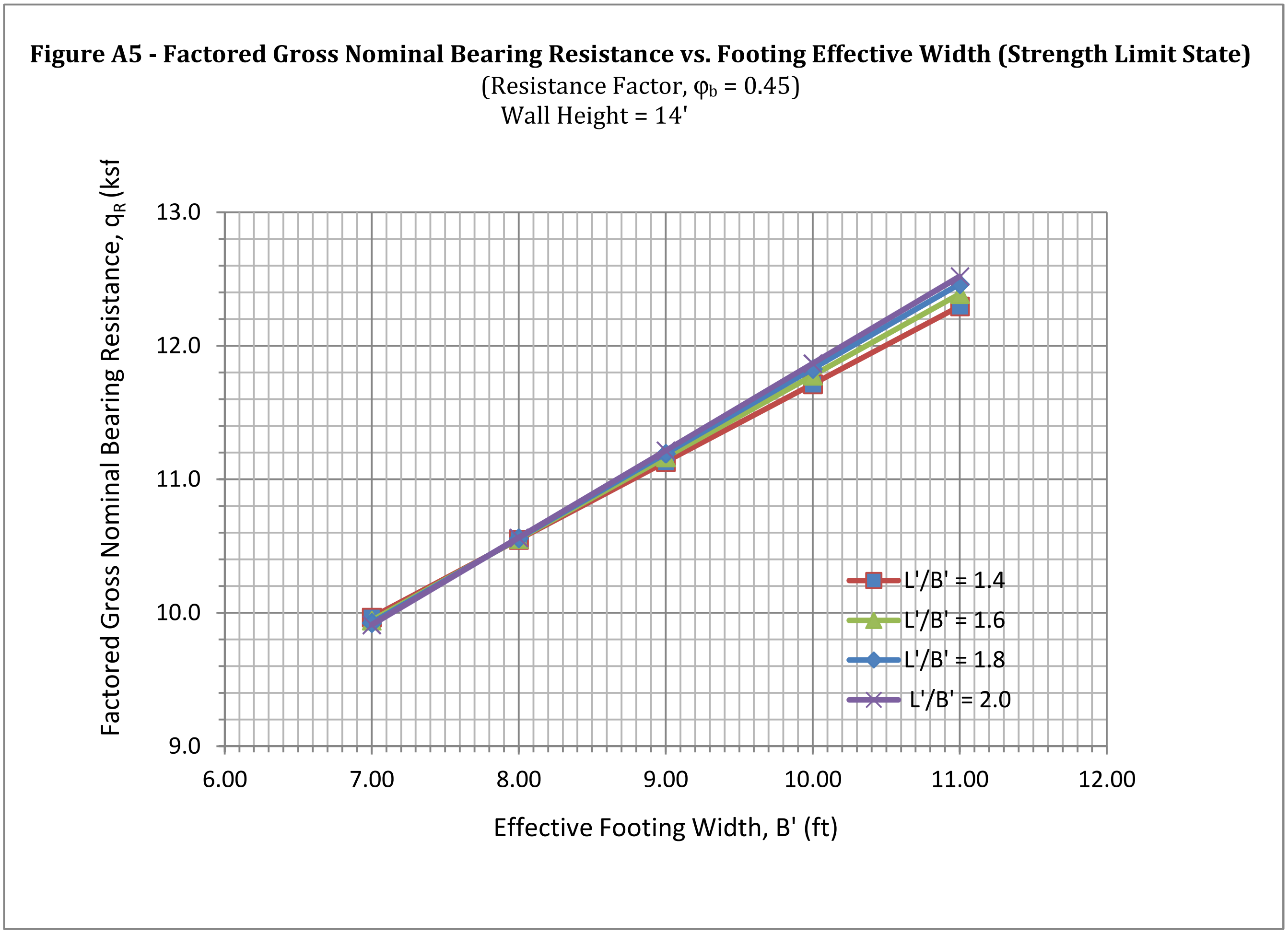


Figure A5 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, φb = 0.45)
Wall Height = 14'
Factored Gross Nominal Bearing Resistance, qR (ksf
13.0
12.0
11.0
10.0
9.0
6.00
7.00
8.00
9.00
10.00
11.00
12.00
Effective Footing Width, B' (ft)
L'/B' = 1.4
L'/B' = 1.6
L'/B' = 1.8
L'/B' = 2.0

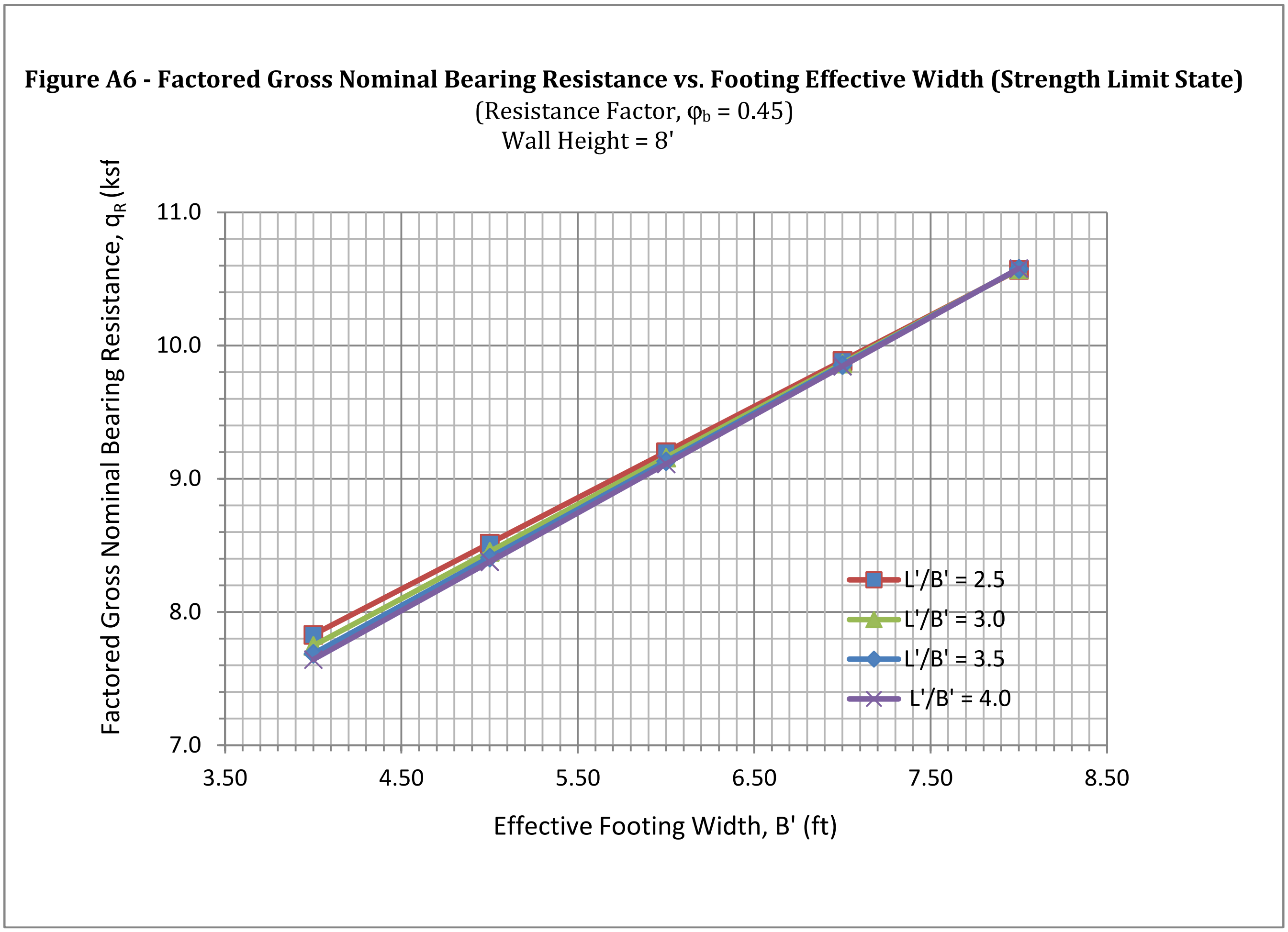


Figure A6 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, φb = 0.45)
Wall Height = 8'
Factored Gross Nominal Bearing Resistance, qR (ksf
11.0
10.0
9.0
8.0
7.0
3.50
4.50
5.50
6.50
7.50
8.50
Effective Footing Width, B' (ft)
L'/B' = 2.5
L'/B' = 3.0
L'/B' = 3.5
L'/B' = 4.0

Figure A7 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width(Extreme Event Limit State)

(Resistance Factor, φ_b = 1.0)

Wall Height = 22'

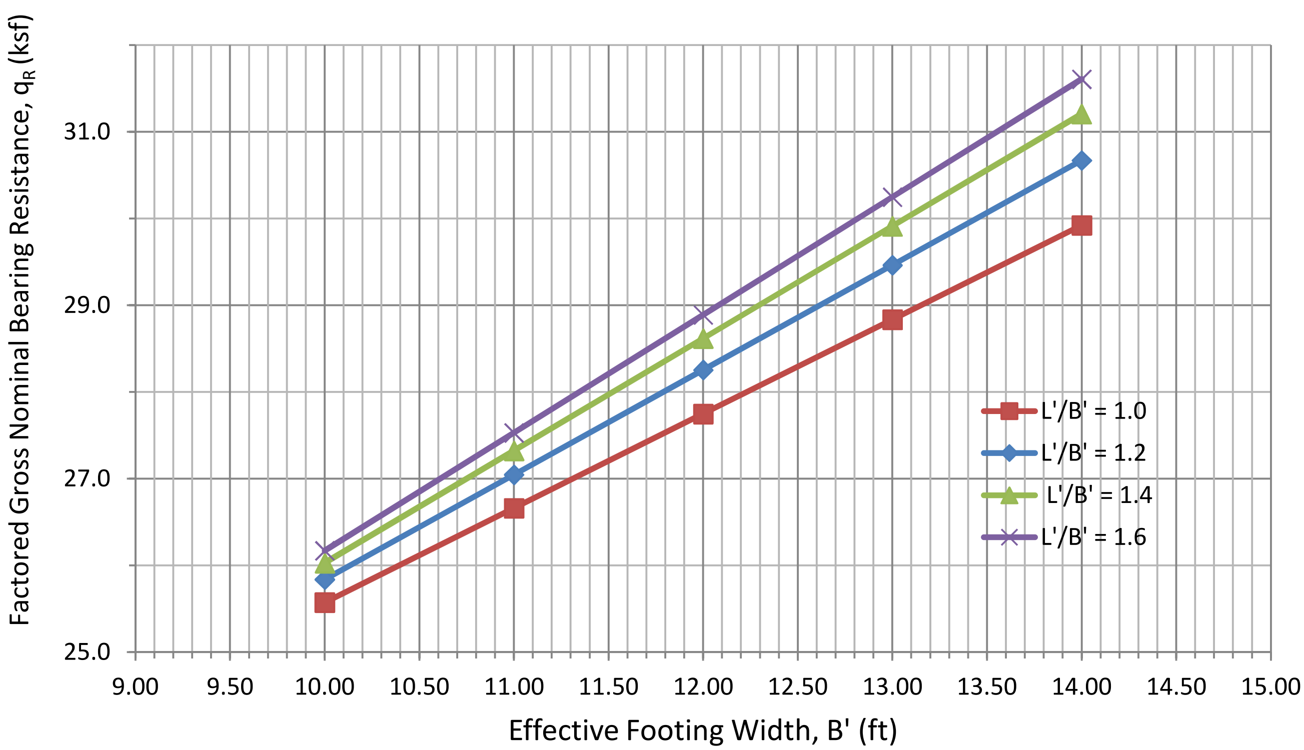
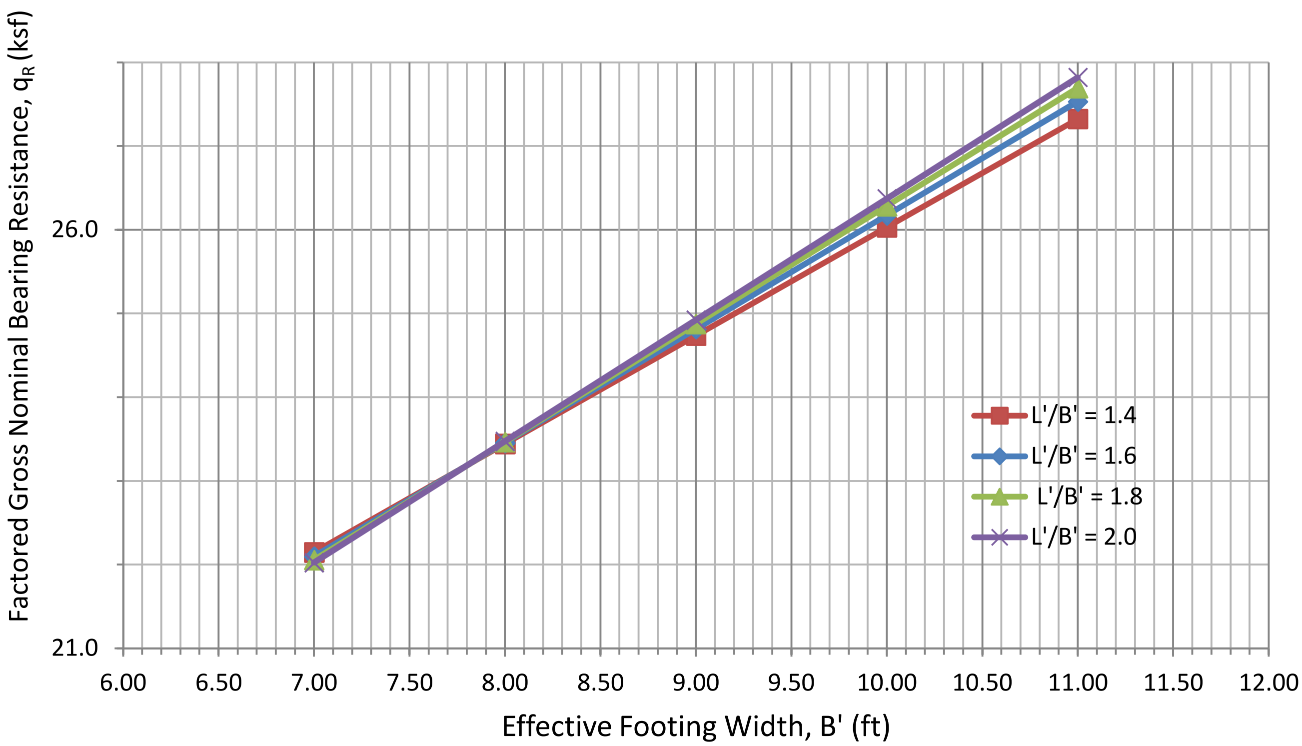


Figure A8 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width(Extreme Event Limit State)
(Resistance Factor, φ_b = 1.0)
Wall Height = 14'



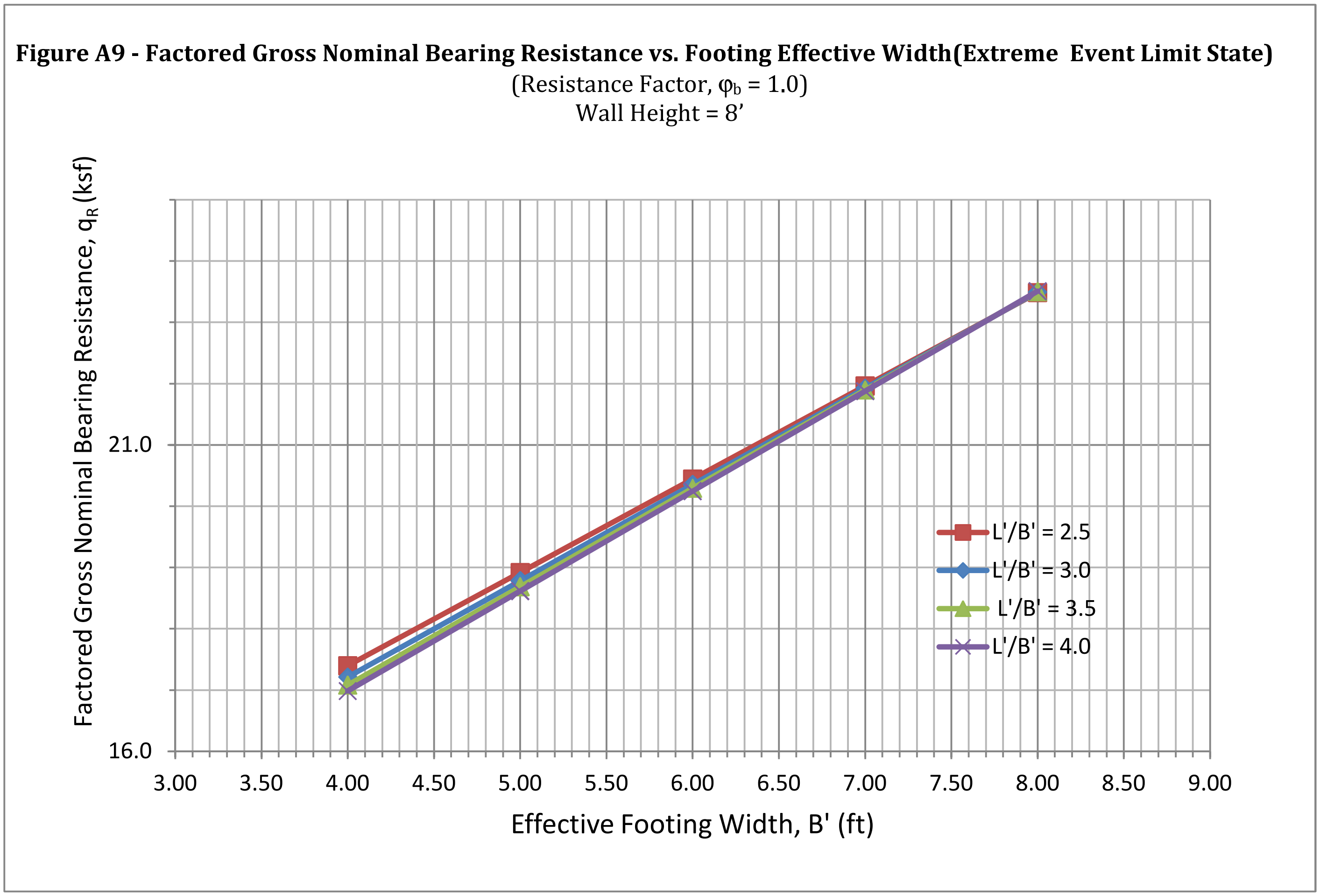


Figure A9 - Factored Gross Nominal Bearing Resistance vs. Footing Effective Width(Extreme Event Limit State)
(Resistance Factor, φb = 1.0)
Wall Height = 8'
Factored Gross Nominal Bearing Resistance, qR (ksf)
21.0
16.0
3.00
3.50
4.00
4.50
5.00
5.50
6.00
6.50
7.00
7.50
8.00
8.50
9.00
Effective Footing Width, B' (ft)
L'/B' = 2.5
L'/B' = 3.0
L'/B' = 3.5
L'/B' = 4.0

M e m o r a n d u m

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: MR. ULYSSES SMPARDOS
Project Engineer

Date: February 2, 2012

File: 07-LA-5-PM 29.8
07000211191 (EA 07-1218W1)
Stough Canyon Culvert Cover
Structure No. 53-3077M

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Stough Canyon Culvert Cover

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the Stough Canyon Culvert Cover. The foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (2007 to 2009) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).

2.0 Project Description

The existing 3-span Burbank Blvd Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with reconfigured ramps. The structure, Stough Canyon Culvert Cover, is proposed in order not to impose any additional fill loads to the pre-existing drainage structure.

3.0 Geotechnical. Exploration

For this project the geotechnical investigation has been conducted from 2007 to 2009. One exploratory boring has been performed at the location of the proposed structure. A summary of exploratory borings is presented in Table No. 1. Surface elevation, station, and offset of the Boring were provided by District 7 Surveys Branch.

Table No. 1 – Summary of Boring

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
R-08-023	3/12/08	1574+58.86	72.28 R	Existing I-5 C/L	594.07	53.2	Not encountered.

Note: Vertical datum NAVD 88

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

Due to the vicinity of the subject Structure to the proposed Retaining Wall No. 1576, Site Geology for Retaining Wall No. 1576 is applicable for this structure . Therefore, please refer to the Section on "Site Geology" of the Foundation Report for Retaining Wall No. 1576, dated January 12, 2012.

4.2 Subsurface Condition

Due to the vicinity of the subject Structure to the proposed Retaining Wall No. 1576, Subsurface Condition for Retaining Wall No. 1576 is applicable for this wall. Therefore, please refer to the Section on "Subsurface Conditions" of the Foundation Report for Retaining Wall No. 1576, dated January 12, 2012.

4.3 Groundwater

Due to the vicinity of the subject Structure to the proposed Retaining Wall No. 1576, ground water condition for Retaining Wall No. 1576 is applicable for this wall. Therefore, please refer to the Section on "Groundwater" of the Foundation Report for Retaining Wall No. 1576, dated January 12, 2012.

4.4 Corrosion Evaluation

Due to the vicinity of the subject Structure to the proposed Retaining Wall No. 1576, Corrosion Evaluation for Retaining Wall No. 1576 is applicable for this wall. Therefore, please refer to the Section on "Corrosion Evaluation" of the Foundation Report for Retaining Wall No. 1576, dated January 12, 2012.

4.5 Seismicity

Due to the vicinity of the subject Structure to the proposed Retaining Wall No. 1576, Seismicity for Retaining Wall No. 1576 is applicable for this wall. Therefore, please refer to the Section on "Seismicity" of the Foundation Report for Retaining Wall No. 1576, dated January 12, 2012.

5.0 Foundation Recommendations

The proposed structure is supported by 24 inch cast-in-drilled-hole (CIDH) piles. Along the depth of the existing drainage structure the piles will be cased to prevent down drag on the existing drainage structure. The foundation design data and foundation loads were provided by the Office of Structure Design – Branch 15 (SD). Table No. 2 shows the foundation design data. Table No.3 shows the foundation design loads.

Table No. 2 – Foundation Design Data Table

Support Location	Design Method	Pile Type	FG Elev (ft)	Cut-off Elev (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)	Number of Piles per support
					B	L		
Southern Support	LRFD	24-inch CIDH	611.0	582.3	5.0	160.0	0.4	18
Northern Support	LRFD	24-inch CIDH	611.0	582.3	5.0	160.0	0.4	18

Table No. 3 – Foundation Design Loads

Location	Service-I Limit State (k)			Controlling Strength Limit State (k)				Controlling Extreme Event Limit State (k)			
	Total Load		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
Southern Support	3,478	193	3,478	4.366	243	0	0	N/A	N/A	N/A	N/A
Northern Support	3,478	193	3,478	4.366	243	0	0	N/A	N/A	N/A	N/A

Note: N/A = Not Applicable

The pile resistance was estimated using Static Analysis Method in AASHTO LRFD Bridge Design Specification (4^{th} Edition) by employing the software of Shaft 5.0 developed by the Ensoft, Inc. The nominal resistances for each support were calculated by dividing the load in strength limit state by a resistance factor of 0.7.

Our office was informed that the design pile tip elevation for lateral load will not be needed (email from SD on January 18, 2012).

Table No.4 – Foundation Recommendation for Supports

Location	Pile Type	Cut-off Elevation (ft)	Service-1 Limit State Load (kips) per support	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
					Strength		Extreme Event				
					Comp (φ=0.7)	Tension (φ=0.7)	Comp (φ=1.0)	Tension (φ=1.0)			
Southern Support	24" CIDH	582.3	3,478	0.4	243	0	N/A	N/A	556.3 (a-1)	556.3	N/A
Northern Support	24" CIDH	582.3	3,478	0.4	243	0	N/A	N/A	556.3 (a-1)	556.3	N/A

Notes:
1. Design tip elevations are controlled by: (a-1) Compression (Strength Limit), (a-2) Compression (Extreme Event Limit), (b-1) Tension (Strength Limit), (b-2) Tension (Extreme Event Limit), (c) Settlement, and (d) Lateral Load, respectively.
2. There is no design tip elevation for settlement.

Table No. 5 – Pile Data Table

Support Location	Pile Type	Cut-off Elevation (ft)	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
			Compression	Tension			
Southern Support	24-inch CIDH	582.3	350	0	556.3 (a)	556.3	N/A
Northern Support	24-inch CIDH	582.3	350	0	556.3 (a)	556.3	N/A

Notes:
1. Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Load, respectively.
2. There is no design tip elevation for settlement.

CONSTRUCTION CONSIDERATIONS

- In order to prevent deterioration of CIDH piles from caving, contractor may use wet construction method to construct the CIDH piles.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 2/2/12

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STATE OF CALIFORNIA

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DES Office Engineer, Office of PS&E
District Materials Engineer

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Date: January 12, 2012

File: 07-LA-5-PM 29.56
07000211191 (EA 07-1218W1)
Retaining Wall No. 1561

Attn: MR. ULYSSES SMPARDOS
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Retaining Wall No. 1561 (L-shaped Wall)

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the retaining wall No. 1561. The foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (2008 and 2009) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).

2.0 Project Description

Retaining Wall No. 1561 is a part of the project that proposes to replace the Burbank Blvd Overcrossing and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Wall No. 1561 is one of them.

Information of the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1561	SB I-5 On-ramp	L-shaped Wall	From 560+00 to 565+83	583	10 to 13

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed from December 3, 2008 to March 16, 2009. The field investigation included four hollow stem auger borings. Borings were logged and sampled using a Standard Penetration Test (SPT) sampler. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inches.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

LOTBs (Log of Test Borings) are being prepared by the Office of Geotechnical Support and will be submitted to your office upon completion.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
A-08-002	12/9/08	1558+61.66	121.05 L	Existing I-5 C/L	584.01	36.5	Not encountered.
A-08-003	12/3/08	1560+70.49	110.39 L		585.65	41.5	
A-08-004	12/10/08	1562+80.47	120.44 L		587.68	46.5	
A-09-204	3/16/09	1565+10.63	221.88 L		588.63	31.0	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing Program

SPT soil samples and bulk samples obtained from the borings were tested for the following laboratory testing:

- Particle Size analyses (Sieve Analysis and Mechanical Analysis)
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below), at the Material Laboratory in Los Angeles and at laboratory selected by the geotechnical consultant URS, Corp.

Table 3 – Laboratory Test Methods

Test	Standard
Sieve Analysis	CTM 202
Mechanical Analysis	CTM 203
Atterberg Limits	CTM 204
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
A-08-004	5.0 - 45.0	8.09	4800	-	-
A-09-204	9.5 - 14.5	6.9	2850	-	-

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness from approximately o feet to 5 feet. The fill consists of poorly graded sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 4.4, Seismicity).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the four borings drilled for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, the alluvium is composed of loose to dense sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2008-2009 investigation for this project to the total depth explored of approximately 81.5 feet below ground surface (elevation +528 feet) (in Boring No. A-08-008). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +584 feet to +589 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo . The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 5 below.

Table No. 5 - Summary of Faults

Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/ km)	R_{JB} (mile/ km)	R_{RUP} (mile/ km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure A1 in Appendix A. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure A1 in Appendix A. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading. Due to the fact that no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

5.1 Foundation Analysis

The L-shaped wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). From a geotechnical standpoint, the wall supported on spread footing is feasible.

First, GS provided the following information based on the preliminary information provided by SD such as elevation of leveling pad, minimum embedment depth, the potential range of width, B, and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figure A2 in Appendix A).

2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figure A3 in Appendix A).

3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figure A4 in Appendix A).

4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for unreinforced retained soil and 34^0 for reinforced backfill).

5) Total unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 6) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis.

Table No. 6 – Wall Data Table

Design 'H' (feet)	Base Width (feet)	BOF Elevation (feet)	Design Load Case per Standard Plan B3-8	Slope in front of footing	Begin Station	End Station	Distance (feet)
13	10	573.218	I	Level	560+00.000	564+15.388	415.39
12	10	575.218	I	Level	564+15.388	565+27.388	112.00
10	8	578.707	I	Level	565+27.388	565+83.388	56.00

5.2 Global Slope Stability

The slope stability analyses were performed to verify the overall stability using the computer program SLOPEW under both static and pseudo-static conditions. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

6.0 Construction Considerations

1. The proposed wall should be founded on properly compacted competent soil. Loose or soft material is not expected at this project site. If clay or loose sand is encountered within the areas to receive the walls, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:
   - (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
   - (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
   - (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 1/12/2012

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District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for I-5/Burbank Blvd Interchange
Damping Ratio = 5%; V_{s30} = 295 m/sec

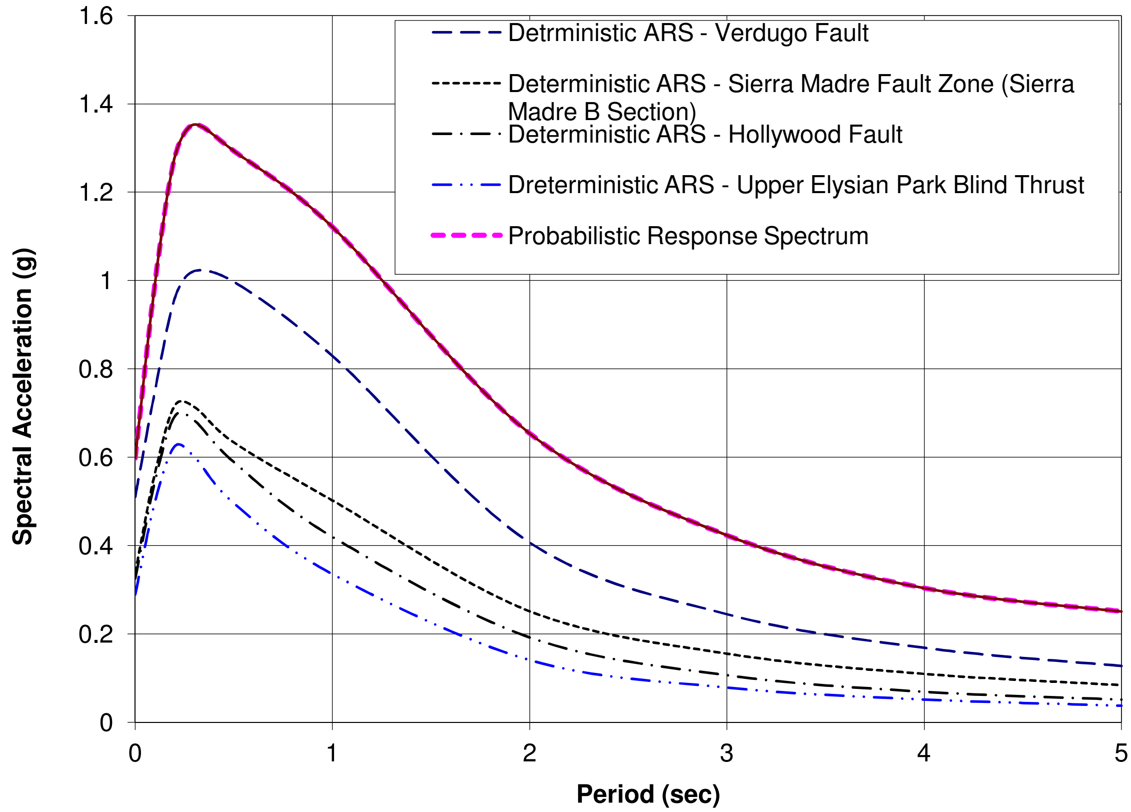
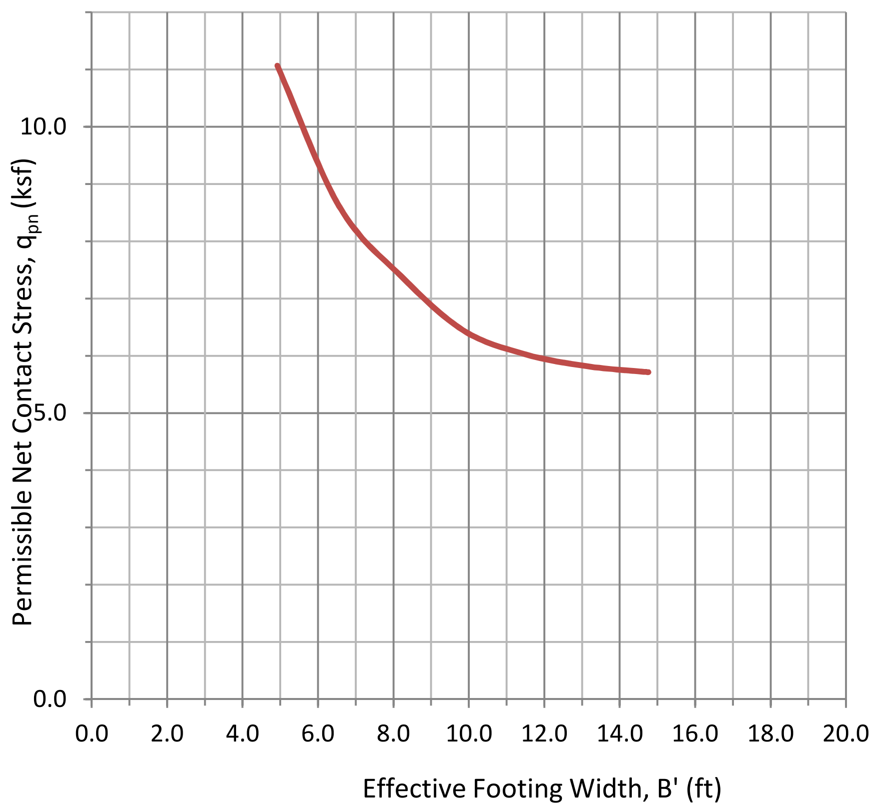


Figure A2 - Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"



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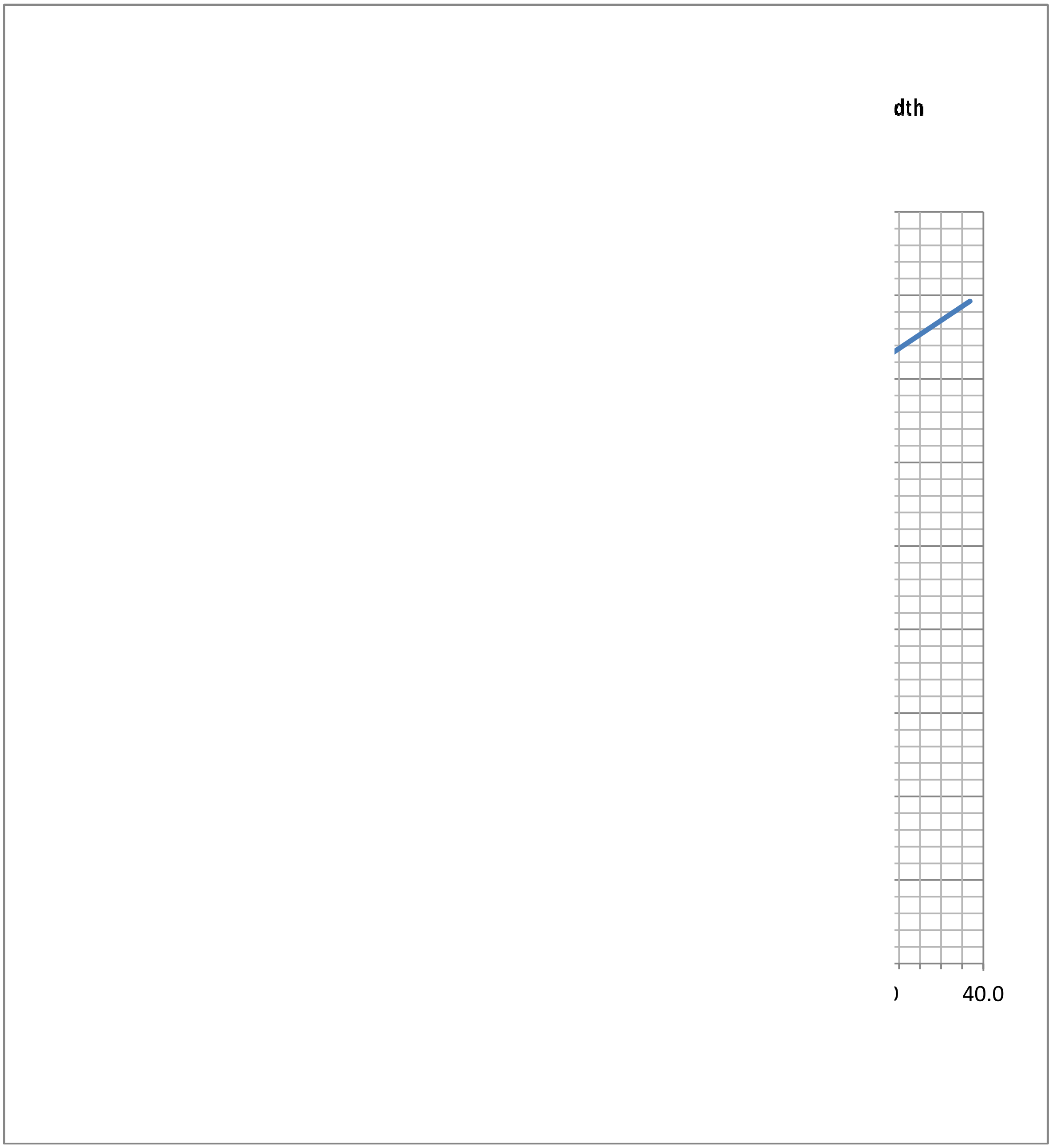
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Memorandum

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Date: February 27, 2012

File: 07-LA-5-PM 29.62
07000211191 (EA 07-1218W1)
Wall No. 1565

Attn: MR. ULYSSES SMPARDOS
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Wall No. 1565

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the MSE wall No. 1565. The foundation recommendations are based on the subsurface information gathered during the foundation investigation (2008) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd. Overcrossing (OC) (Bridge No. 53-1089).

2.0 Project Description

Wall No. 1565 is a part of the project that proposes to replace the Burbank Blvd. OC and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd. OC will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd., but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Wall No. 1565 is one of them.

Information of the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1565	SB I-5 On-ramp	Type 1 Wall	From 564+14 to 564+95	80.25	10.00
		MSE Wall	From 564+95 to 571+50	655.00	11.67 to 44.17

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed on December 2, 2008 and December 3, 2008. The field investigation included two hollow stem auger borings and two mud rotary borings. Borings were logged and sampled using a Standard Penetration Test (SPT) sampler. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inches.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
R-08-005	12/3/08	1564+95.75	148.04 L	Existing I-5 Centerline	589.29	51.5	Not encountered.
R-08-006	12/2/08	1567+10.80	172.19 L		590.13	51.5	
A-08-007	12/2/08	1569+33.64	205.16 L		594.34	71.5	
A-08-008	12/3/08	1571+39.70	221.84 L		609.22	81.5	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing Program

SPT soil samples and bulk samples obtained from the borings were tested for the following laboratory testing:

- Particle Size analyses (Sieve Analysis and Mechanical Analysis)
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods (see Table No. 3 below), at the District 7 Materials Laboratory in Los Angeles.

Table No. 3 – Laboratory Test Methods

Test	Standard
Sieve Analysis	CTM 202
Mechanical Analysis	CTM 203
Atterberg Limits	CTM 204
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
A-08-008	5.0-80.0	8.08	5400	-	-

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:
Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non-corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness up to approximately 30 feet. The fill consists of poorly graded sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 7.0, Seismic Recommendations).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the four borings drilled for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, the alluvium is composed of loose to dense sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2008 investigation for this project to the total depth explored of approximately 81.5 feet below ground surface (elevation +528 feet) (in Boring No. A-08-008). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +610 feet to +585 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo . The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd. OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 5 below.

Table No. 5 - Summary of Faults

Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/ km)	R_{JB} (mile/ km)	R_{RUP} (mile/ km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure 1. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure A1 in Appendix A. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading. Due to the fact that no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

Type1 Wall from 564+14 to 564+95

SD provided Wall Data Table (Table No. 6) given below. Allowable bearing capacity was calculated using Terzaghi's equation. A factor of safety of 3 was used. The allowable bearing capacity obtained was compared against the toe pressure given on the Caltrans Standard Plans.

Due to the granular nature of the underlying granular soils at this portion of the wall, the settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

From a geotechnical standpoint, the wall supported on spread footing is feasible.

Table No. 6 – Wall Data Table

Design 'H' (feet)	Base Width (feet)	BOF Elevation (feet)	Design Load Case per Standard Plan B3-8	Slope in front of footing	Begin Station	End Station	Distance (feet)
10'-0"	6'-3"	574.000	I	Level	564+14.537	564+94.787	80.25

MSE Wall from 564+95 to 571+50

The MSE wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). The MSE wall is being designed as per Section 3-8 (Mechanically Stabilized Embankment) of Caltrans Bridge Design Aids, March, 2009. The Caltrans Standard Drawings (xs13-020-1e to xs13-020-6e) also are being used.

First, GS provided the following information based on the preliminary information provided by SD such as elevation of leveling pad, minimum embedment depth, the potential range of footing effective width (B'), and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figure A2 in Appendix A).

2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figure A3 in Appendix A).

3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figure A4 in Appendix A).

4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for unreinforced retained soil and 34^0 for reinforced backfill).

5) Total unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 9 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 7) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis, using the computer program SLOPEW. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

Table No. 7 – Wall Data Table

Wall Height 'H'	Base Width	Top of Leveling Pad Elevation (ft)	Slope in front of footing	Begin Station	End Station	Distance (ft)
11'-8"	9'-6"	575.578	Level	564+94.787	565+29.787	35.00
14'-2"	11'-6"	575.578	Level	565+29.787	565+79.787	50.00
16'-8"	13'-6"	575.578	Level	565+79.787	566+24.787	45.00
19'-2"	15'-6"	575.578	Level	566+24.787	566+64.787	40.00
21'-8"	17'-6"	575.578	Level	566+64.787	566+99.787	35.00
24'-2"	18'-6"	575.578	Level	566+99.787	567+34.787	35.00
26'-8"	20'-6"	575.578	Level	567+34.787	567+64.787	30.00
29'-2"	21'-6"	575.578	Level	567+64.787	567+99.787	35.00
31'-8"	23'-6"	575.578	Level	567+99.787	568+34.787	35.00
34'-2"	25'-6"	575.578	Level	568+34.787	568+69.787	35.00
36'-8"	26'-6"	575.578	Level	568+69.787	569+04.787	35.00
39'-2"	29'-6"	575.578	Level	569+04.787	569+44.787	40.00
41'-8"	30'-6"	575.578	Level	569+44.787	570+04.787	60.00
41'-8"	30'-6"	578.078	Level	570+04.787	570+74.787	70.00
44'-2"	33'-6"	578.078	Level	570+74.787	571+49.787	75.00

6.0 Construction Considerations

1. The proposed wall (MSE and Type 1 wall sections) should be founded on properly compacted competent soil. Loose or soft material is not expected at this project site. If clay or loose sand is encountered within the areas to receive the wall, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be

placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:

- (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
- (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
- (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 2/27/12

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DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for I-5/Burbank Blvd Interchange
Damping Ratio = 5%; V_{s30} = 295 m/sec

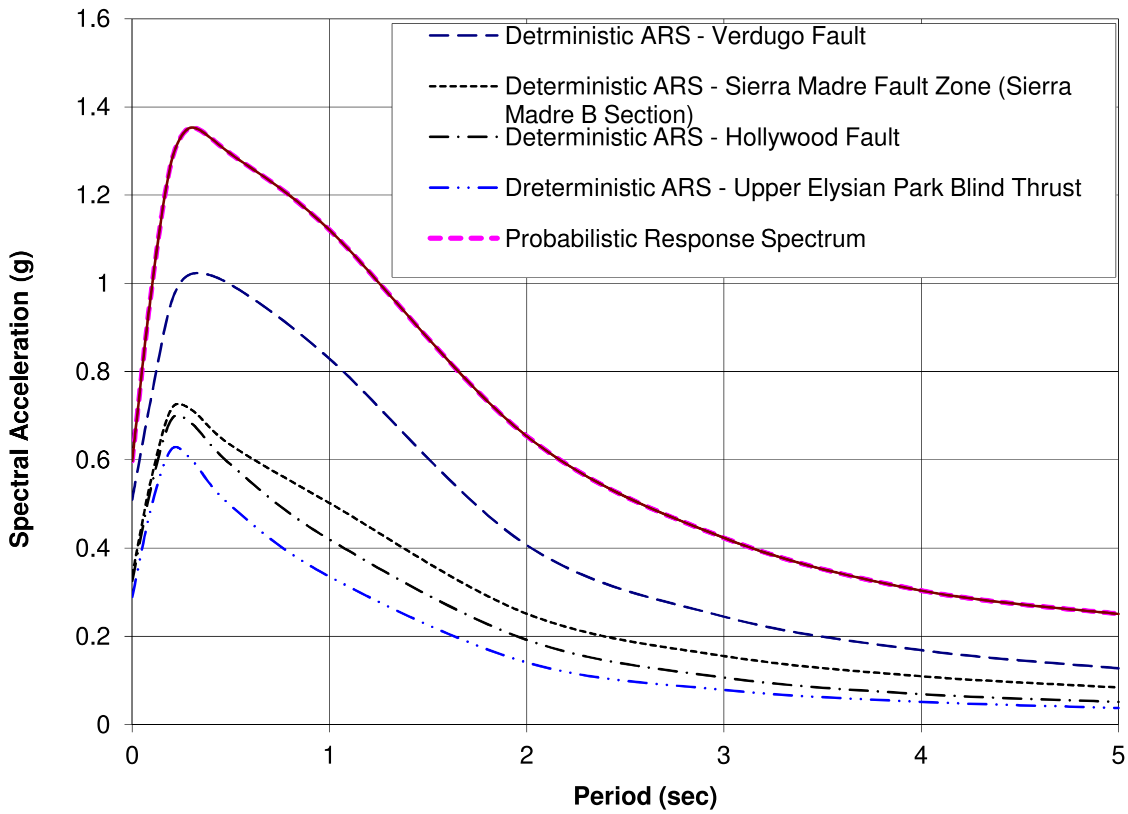
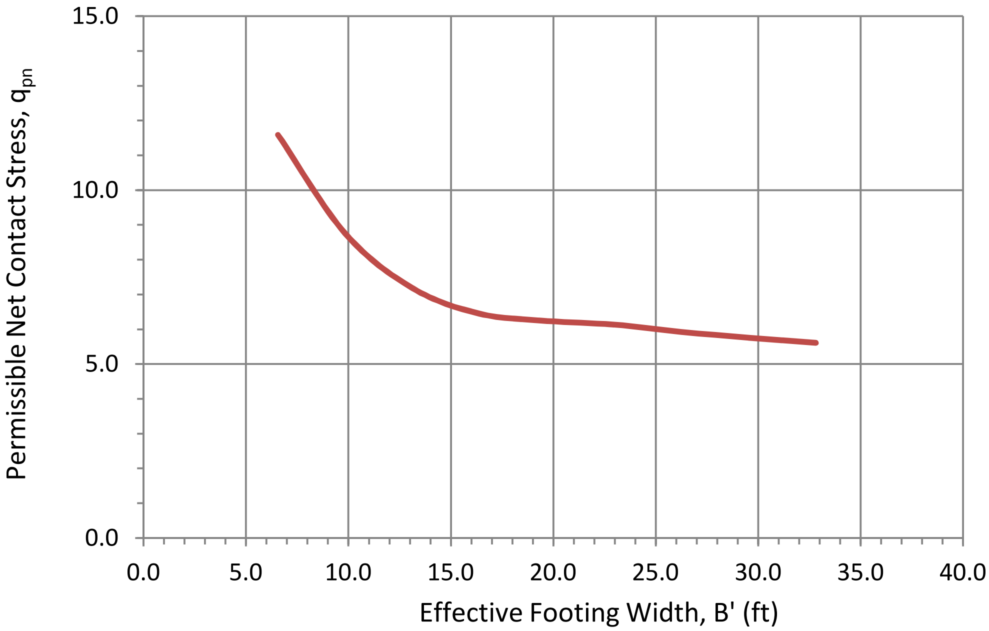


Figure A2 - MSE Wall No. 1565 (from STA 564+95 to 571+50)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"



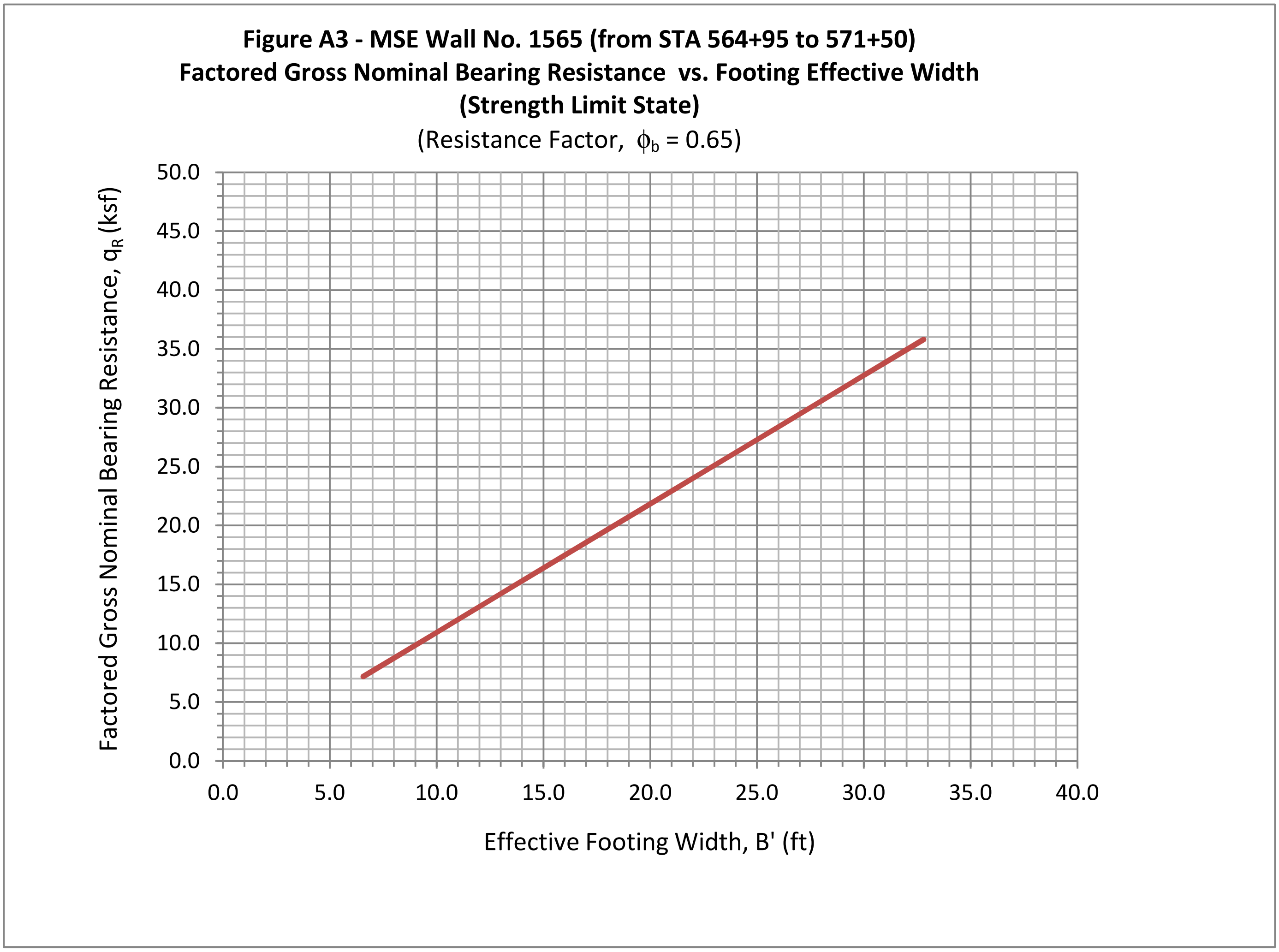


Figure A3 - MSE Wall No. 1565 (from STA 564+95 to 571+50)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)
Factored Gross Nominal Bearing Resistance, q_R (ksf)
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Effective Footing Width, B' (ft)

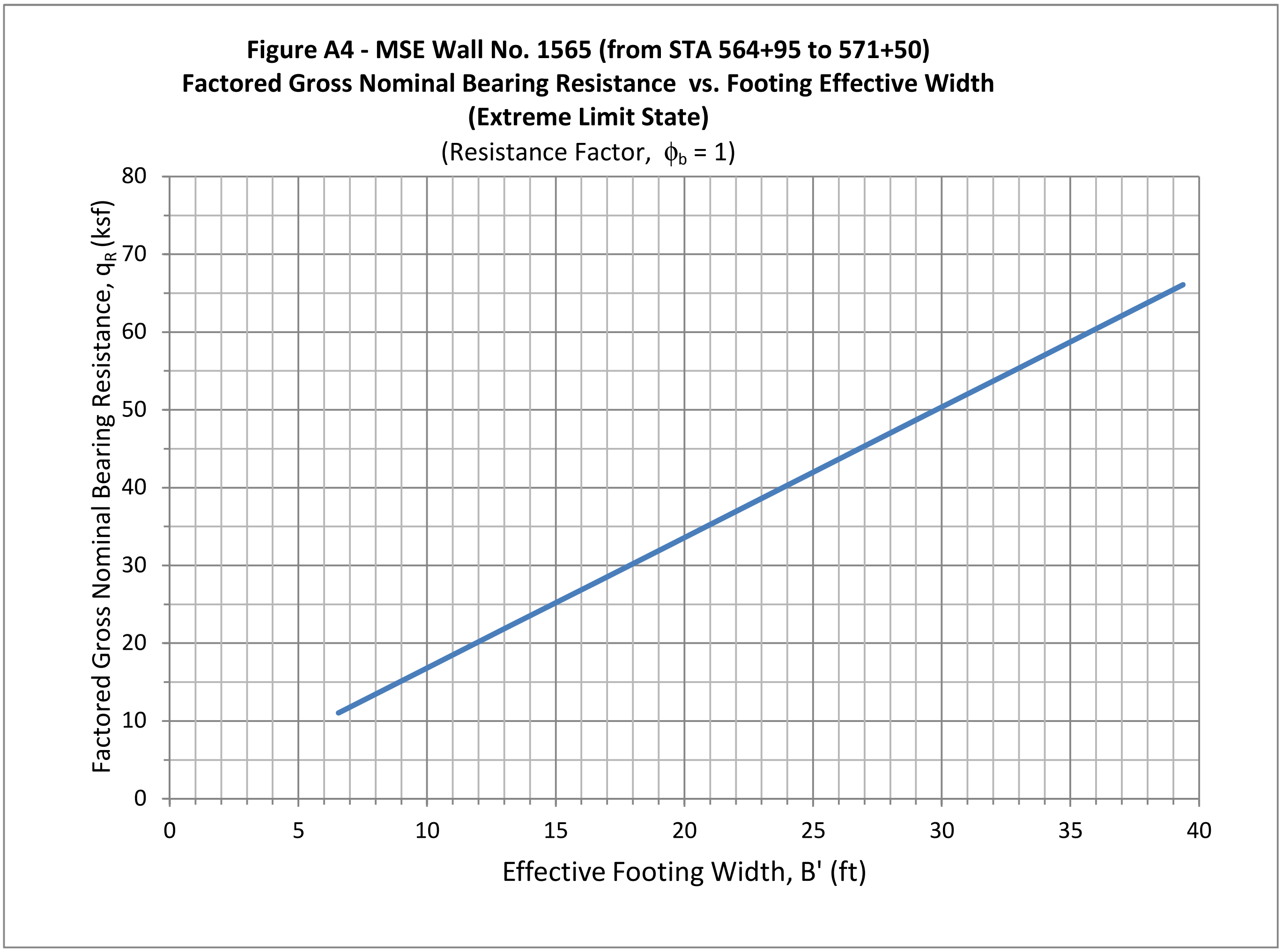


Figure A4 - MSE Wall No. 1565 (from STA 564+95 to 571+50)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Extreme Limit State)
(Resistance Factor, ϕ_b = 1)
Factored Gross Nominal Bearing Resistance, q_R (ksf)
0
10
20
30
40
50
60
70
80
0
5
10
15
20
25
30
35
40
Effective Footing Width, B' (ft)

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Date: January 12, 2012

File: 07-LA-5-PM 29.69
07000211191 (EA 07-1218W1)
MSE Wall No. 1567

Attn: MR. ULYSSES SMPARDOS
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for MSE Wall No. 1567

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the MSE wall No. 1567. The foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (2008 to 2009)) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).

2.0 Project Description

Wall No. 1567 is a part of the project that proposes to replace the Burbank Blvd Overcrossing and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. MSE Wall No. 1567 is one of them.

Information of the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1567	SB I-5 On-ramp	MSE Wall	From 566+31 to 572+38	606	6.67 to 39.17

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed from March 10, 2008 to March 16, 2009. The field investigation included three hollow stem auger borings and one mud rotary boring. Borings were logged and sampled using a Standard Penetration Test (SPT) sampler. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inches.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

LOTBs (Log of Test Borings) are being prepared by the Office of Geotechnical Support and will be submitted to your office upon completion.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
R-08-015	3/10/08	1573+31.08	253.64 L	Existing I-5 C/L	584.01	36.5	Not encountered.
A-09-201	3/13/09	1570+46.89	279.57 L		585.82	61.0	
A-09-202	3/16/09	1568+77.71	260.44 L		588.03	51.5	
A-09-203	3/13/09	1567+32.48	232.30 L		589.78	41.0	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing Program

SPT soil samples and bulk samples obtained from the borings were tested for the following laboratory testing:

- Particle Size analyses (Sieve Analysis and Mechanical Analysis)
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below), at the Material Laboratory in Los Angeles and at laboratory selected by the geotechnical consultant URS, Corp.

Table No. 3 – Laboratory Test Methods

Test	Standard
Sieve Analysis	CTM 202
Mechanical Analysis	CTM 203
Atterberg Limits	CTM 204
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
R-08-015	6.0-102.0	9.4	6700	-	-
A-09-201	4.5-14.5	6.9	5150	-	-
A-09-202	4.5-14.5	7	5150	-	-
A-09-203	4.5-14.5	7	2250	-	-

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness from approximately 4 feet to 21 feet. The fill consists of poorly graded sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 4.4, Seismicity).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the four borings drilled for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, the alluvium is composed of loose to dense sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2008-2009 investigation for this project to the total depth explored of approximately 81.5 feet below ground surface (elevation +528 feet) (in Boring No. A-08-008). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +619 feet to +588 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo . The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 5 below.

Table No. 5 - Summary of Faults

Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/ km)	R_{JB} (mile/ km)	R_{RUP} (mile/ km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure A1 in Appendix A. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure A1 in Appendix A. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading. Due to the fact that no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

5.1 Foundation Analysis

The MSE wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). The MSE wall is being designed as per Section 3-8 (Mechanically Stabilized Embankment) of Caltrans Bridge Design Aids, March, 2009. The Caltrans Standard Drawings (xs13-020-1e to xs13-020-6e) also are being used.

First GS provided the following information based on the preliminary information provided by SD such as elevation of leveling pad, minimum embedment depth, the potential range of width, B, and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figure A2 in Appendix A).

2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figure A3 in Appendix A).

3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figure A4 in Appendix A).

4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for unreinforced retained soil and 34^0 for reinforced backfill).

5) Total unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 6) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis.

Table No. 6 – Wall Data Table

Wall Height 'H'	Base Width	Top of Leveling Pad Elevation (ft)	Slope in front of footing	Begin Station	End Station	Distance (ft)
6'-8"	9'-6"	585.438	Level	566+31.114	566+46.114	15.00
9'-2"	9'-6"	585.438	Level	566+46.114	566+81.114	35.00
11'-8"	9'-6"	585.438	Level	566+81.114	567+16.114	35.00
14'-2'	11'-6"	585.438	Level	567+16.114	567+51.114	35.00
16'-8'	13'-6"	585.438	Level	567+51.114	567+81.114	30.00
19'-2"	15'-6"	585.438	Level	567+81.114	567+96.114	15.00
21'-8"	17'-6"	582.938	Level	567+96.114	568+16.114	20.00
24'-2"	18'-6"	582.938	Level	568+16.114	568+51.114	35.00
26'-8"	20'-6"	582.938	Level	568+51.114	568+86.114	35.00
29'-2"	21'-6"	582.938	Level	568+86.114	569+21.114	35.00
31'-8"	23'-6"	582.938	Level	569+21.114	569+71.114	50.00
34'-2"	25'-6"	582.938	Level	569+71.114	570+36.114	65.00
36'-8"	26'-6"	582.938	Level	570+36.114	571+06.114	70.00
39'-2"	29'-6"	582.938	Level	571+06.114	571+61.891	55.78
39'-2"	29'-6"	582.938	Level	571+61.891	572+37.608	75.72

5.2 Global Slope Stability

The slope stability analyses were performed to verify the overall stability using the computer program SLOPEW under both static and pseudo-static conditions. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

6.0 Construction Considerations

1. The proposed MSE wall should be founded on properly compacted competent soil. During the field investigation, from STA 571+00 to 572+44 (based on wall LOL), sandy silty clay was encountered at the bottom of footing elevation; and from STA 569+25 to STA 571+00, loose silty sand was encountered at the bottom of footing elevation. Therefore, from STA 569+25 to 572+44 STA, soil should be over-excavated up to five feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:

(i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
(ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall (horizontal extension in front of the walls should be at least equal to the width of the footing, in the case of over-excavation).
(iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 1/12/2012

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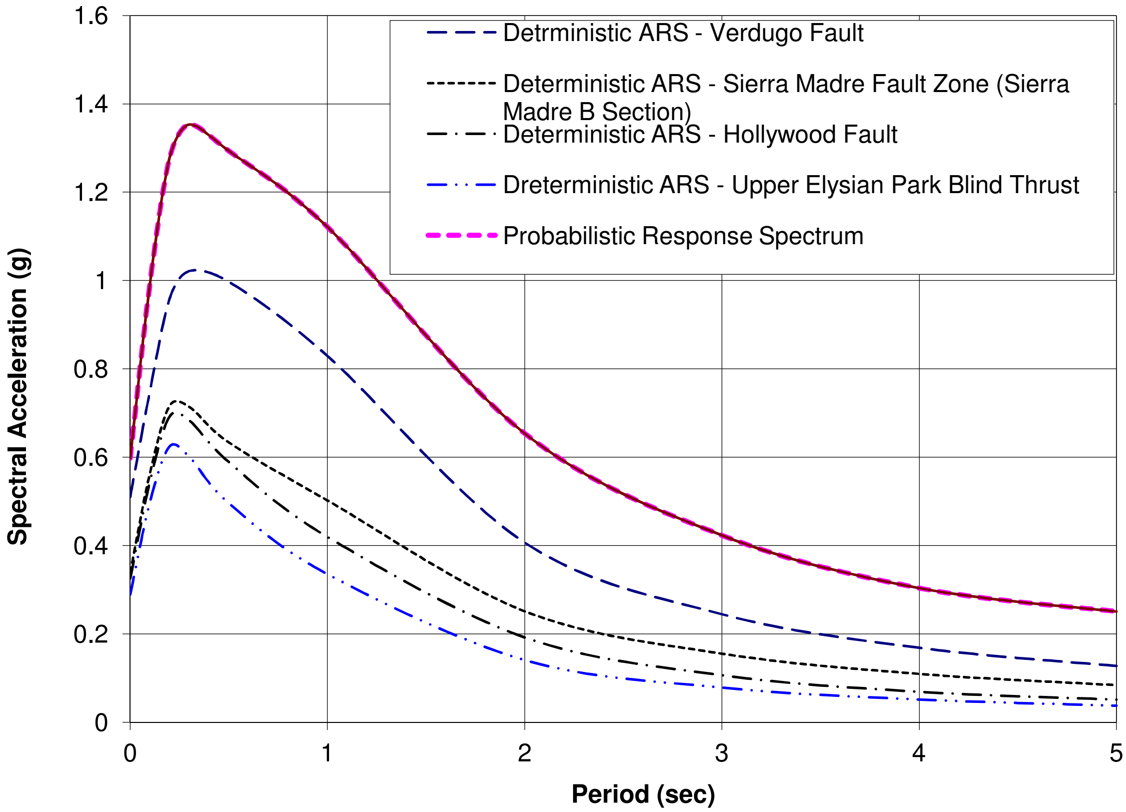
GS Corporate (Mark_Willians@dot.ca.gov)
Structure Construction R.E. Pending File (RE_Pending_File@dot.gov.ca)
DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for Burbank Blvd OC
Damping Ratio = 5%; V_{s30} = 295 m/sec



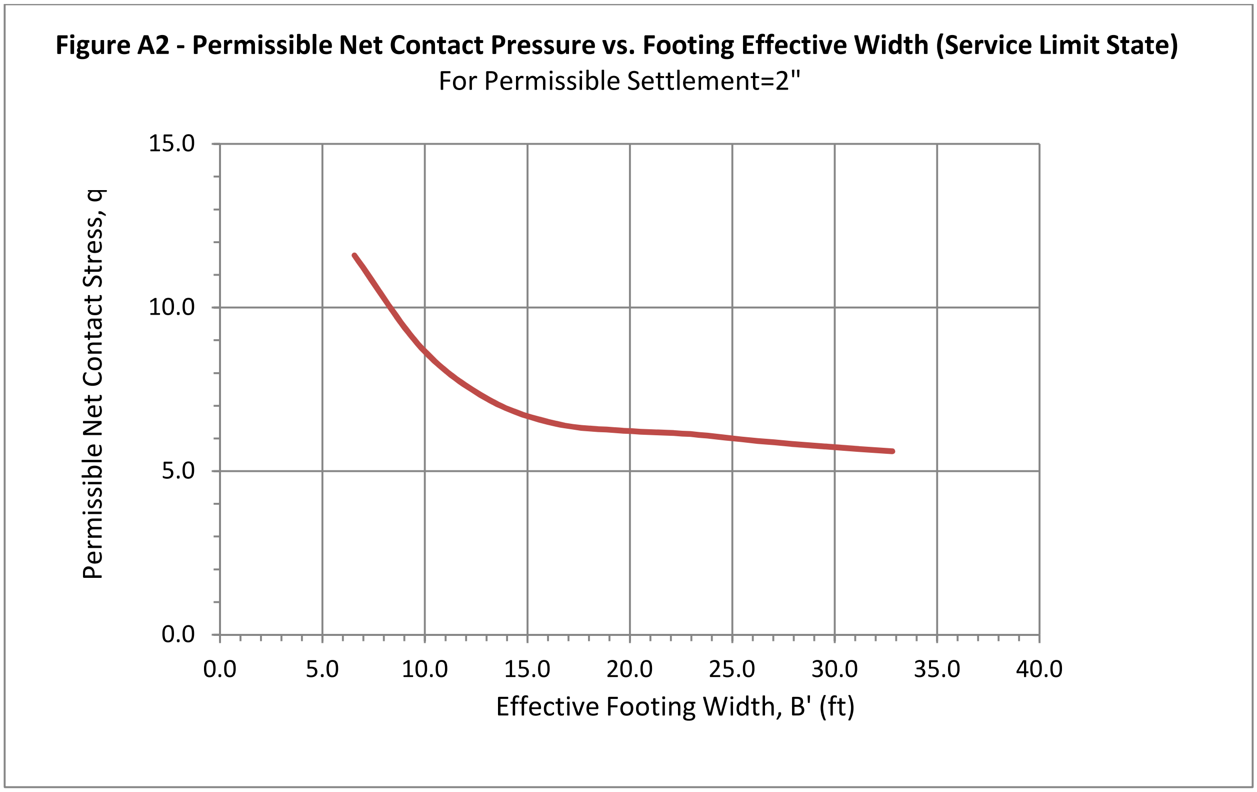
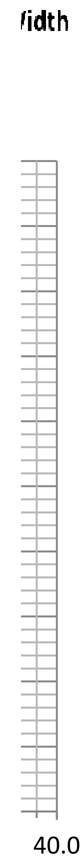
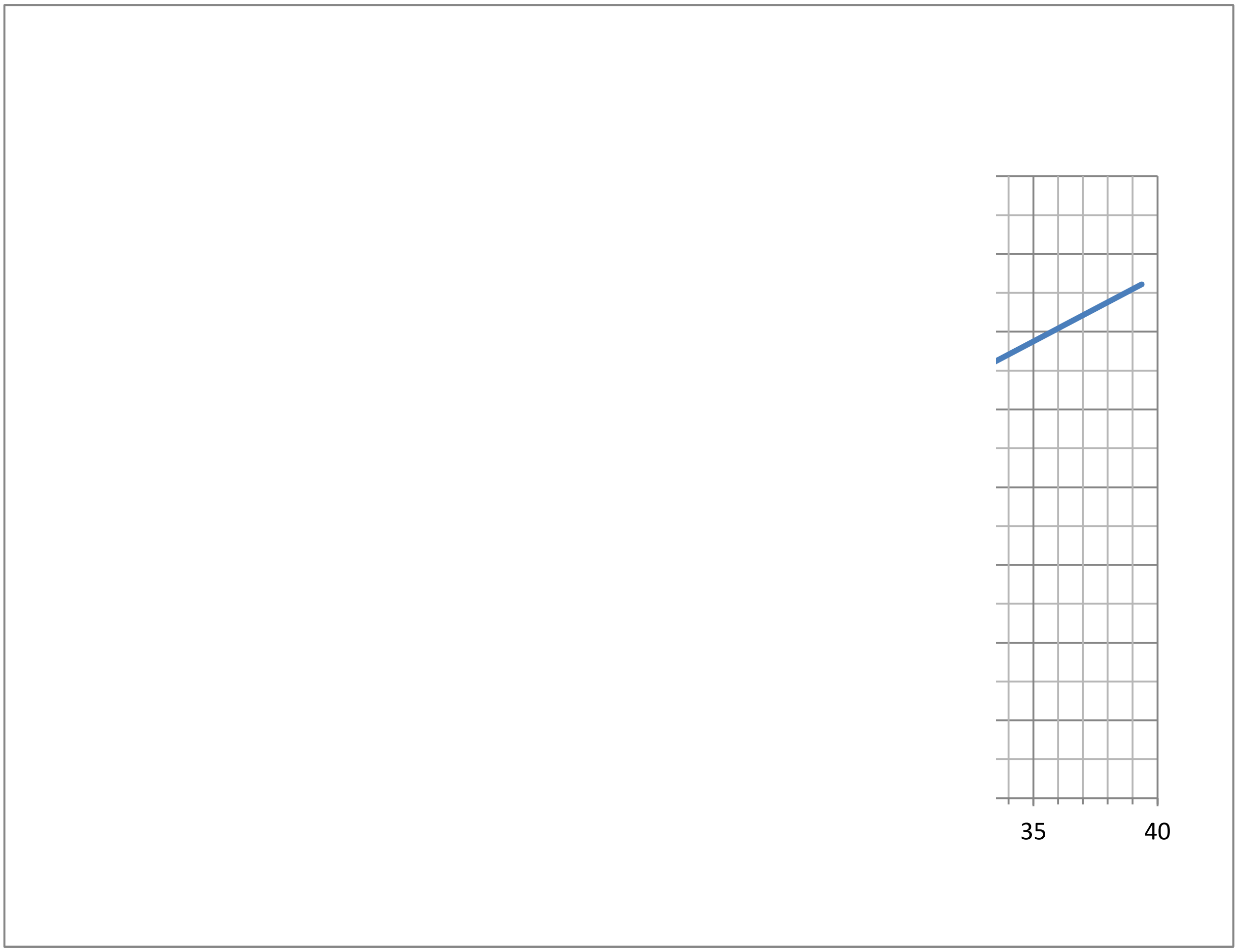


Figure A2 - Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"
Permissible Net Contact Stress, q
15.0
10.0
5.0
0.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Effective Footing Width, B' (ft)


40.0


35
40

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Date: February 28, 2012

File: 07-LA-5-PM 29.83
07000211191 (EA 07-1218W1)
Wall No. 1575

Attn: MR. ULYSSES SMPARDOS
Project Engineer

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Retaining Wall No. 1575 from STA 575+68 to STA 580+58

1.0 Scope of Work

Limits of Retaining Wall No. 1575 are from STA 573+73 to STA 580+58. Our office submitted a Foundation Report dated February 15, 2012 to your office to provide foundation recommendations for the portion of the the wall that will be constructed over two existing culverts, from STA 573+73 to STA 575+68.

Our office has prepared this Memorandum to provide the foundation recommendations for the wall No. 1575 from STA 575+68 to STA 580+58. The foundation recommendations are based on the subsurface information gathered during the foundation investigation (2007 to 2009) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd. Overcrossing (OC) (Bridge No. 53-1089).

2.0 Project Description

Wall No. 1575 is a part of the project that proposes to replace the Burbank Blvd. OC and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd. OC will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd., but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Wall No. 1575 is one of them.

Information for the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1575	SB I-5 Off-ramp	Modified Type1 Wall on piles	From 573+73 to 575+68	194.5	32 to 36
		MSE wall	From 575+68 to 579+93	425	12 to 32
		Type 1 Wall	From 579+93 to 580+58	65	10

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed from November 29, 2007 to December 5, 2007. The field investigation included three hollow stem auger borings and one Cone Penetration Testing (CPT). Borings were logged and sampled using a Standard Penetration Test (SPT) sampler and 2-inch California Modified sampler at selected intervals. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inch. Following drilling, sampling and logging, the borings were backfilled with bentonite chips, and patched with cold asphalt.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
A-07-019	11/29/07 - 11/30/07	1577+23.61	296.91 L	Existing I-5 C/L	615.01	51.5	Not encountered.
A-07-021	11/30/07	1579+20.72	243.03 L		607.19	51.5	
A-07-022	12/3/07-12/4/07	1580+12.67	175.20 L		602.05	51.5	
CPT-07-018	12/5/07	1576+20.30	242.75 L		608.52	78.0	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing Program

SPT soil samples and bulk samples obtained from the borings were tested for the following laboratory testing:

- Particle Size analyses (Sieve Analysis and Mechanical Analysis)
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below), at the Material Laboratory in Los Angeles and at laboratory selected by the geotechnical consultant URS, Corp.

Table No. 3 – Laboratory Test Methods

Test	Standard
Sieve Analysis	CTM 202
Mechanical Analysis	CTM 203
Atterberg Limits	CTM 204
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
A-07-019	5-50	8.13	2200	-	-

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:
Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non-corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness up to approximately 30 feet. The fill consists of poorly graded sand and silty sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 4.4, Seismicity).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the three borings and one cone penetrometer test conducted for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to very dense, fine to coarse sand and silty sand with gravel and occasional cobbles. Below the fill material, the alluvium is composed of medium stiff to very stiff sandy silty clay and loose to dense sand and silty sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2008-2009 investigation for this project to the total depth explored of approximately 103.2 feet below ground surface (elevation +512 feet in Boring No. A-08-014 for Bridge 53-3057). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +615 feet to +600 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo. The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

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Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
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R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure A1 in Appendix A. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure A1 in Appendix A. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading. Due to the fact that no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

<u>Type1 Wall from STA 579+93 to 580+58</u>

SD provided Wall Data Table (Table No. 6) given below. Allowable bearing capacity was calculated using Terzaghi's equation. A factor of safety of 3 was used. The allowable bearing capacity obtained was compared against the toe pressure given on the Caltrans Standard Plans.

Due to the granular nature of the underlying granular soils at this portion of the wall, the settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

From a geotechnical standpoint, the wall supported on spread footing is feasible.

Table No. 6 – Wall Data Table

Design 'H' (feet)	Base Width (feet)	BOF Elevation (feet)	Design Load Case per Standard Plan B3-8	Slope in front of footing	Begin Station	End Station	Distance (feet)
10.000	6.250	594.400	I	Level	579+93.159	580+58.159	65.00

<u>MSE Wall from 575+68 to 579+93</u>

The MSE wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). The MSE wall is being designed as per Section 3-8 (Mechanically Stabilized Embankment) of Caltrans Bridge Design Aids, March, 2009. The Caltrans Standard Drawings (xs13-020-1e to xs13-020-6e) also are being used.

First, GS provided the following information based on the preliminary information provided by SD such as elevation of leveling pad, minimum embedment depth, the potential range of footing effective width (B'), and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figures A2 and A3).
2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figures A4 and A5).
3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figures A6 and A7).
4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for unreinforced retained soil and 34^0 for reinforced backfill).
5) Unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 7) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis, using the computer program SLOPEW. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

Table No. 7 – Wall Data Table

Wall Height 'H'	Base Width	Top of Leveling Pad Elevation (ft)	Slope in front of footing	Begin Station	End Station	Distance (ft)
31.667	23.500	590.000	Level	575+68.159	576+23.159	55.00
29.167	21.500	590.000	Level	576+23.159	576+53.159	30.00
26.667	20.500	592.500	Level	576+53.159	577+03.159	50.00
24.167	18.500	592.500	Level	577+03.159	577+58.159	55.00
21.667	17.500	592.500	Level	577+58.159	577+98.159	40.00
19.167	15.500	595.000	Level	577+98.159	578+18.159	20.00
16.667	13.500	595.000	Level	578+18.159	578+83.159	65.00
14.167	11.500	595.000	Level	578+83.159	579+63.159	80.00
11.667	9.500	595.000	Level	579+63.159	579+93.159	30.00

6.0 Construction Considerations

1. The proposed wall (MSE and Type 1 Wall Sections) should be founded on properly compacted competent soil. Loose or soft material is not expected at the bottom of footing elevation at this project site. If clay or loose sand is encountered within the areas to receive the walls, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:
   (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
   (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
   (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 2/28/12

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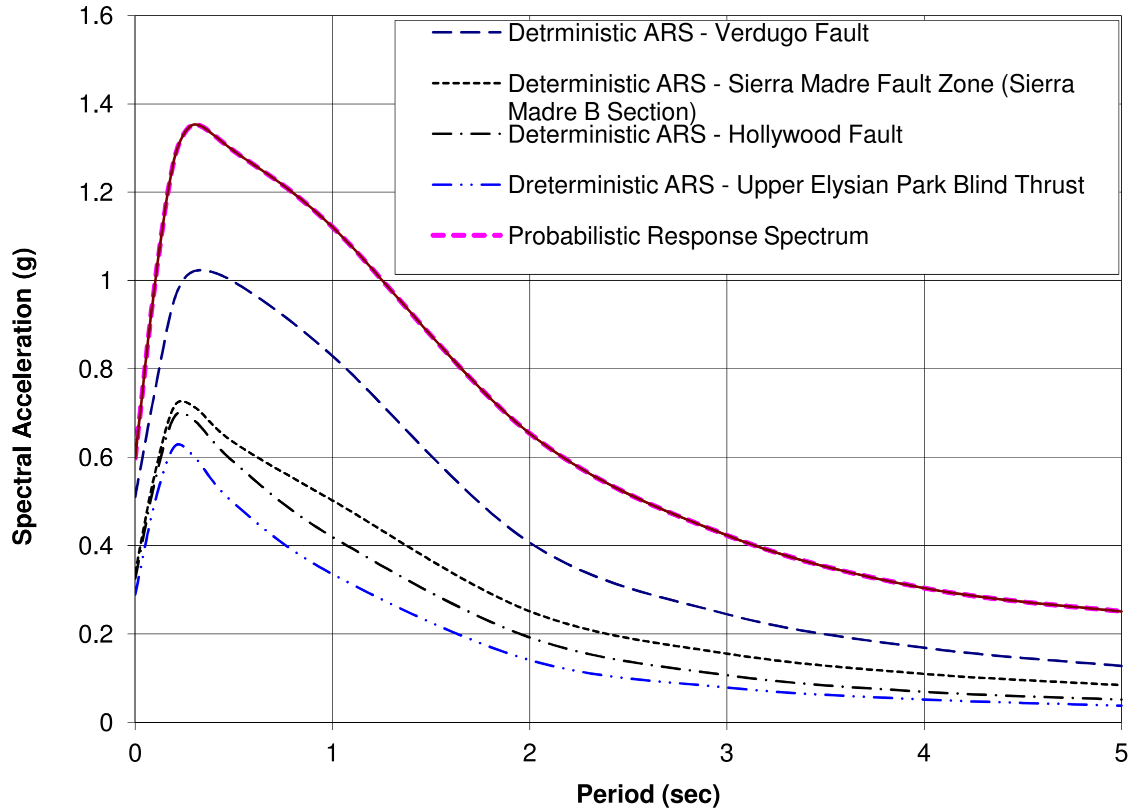
Structure Construction R.E. Pending File (RE_Pending_File@dot.gov.ca)
DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for Burbank Blvd OC
Damping Ratio = 5%; V_{s30} = 295 m/sec



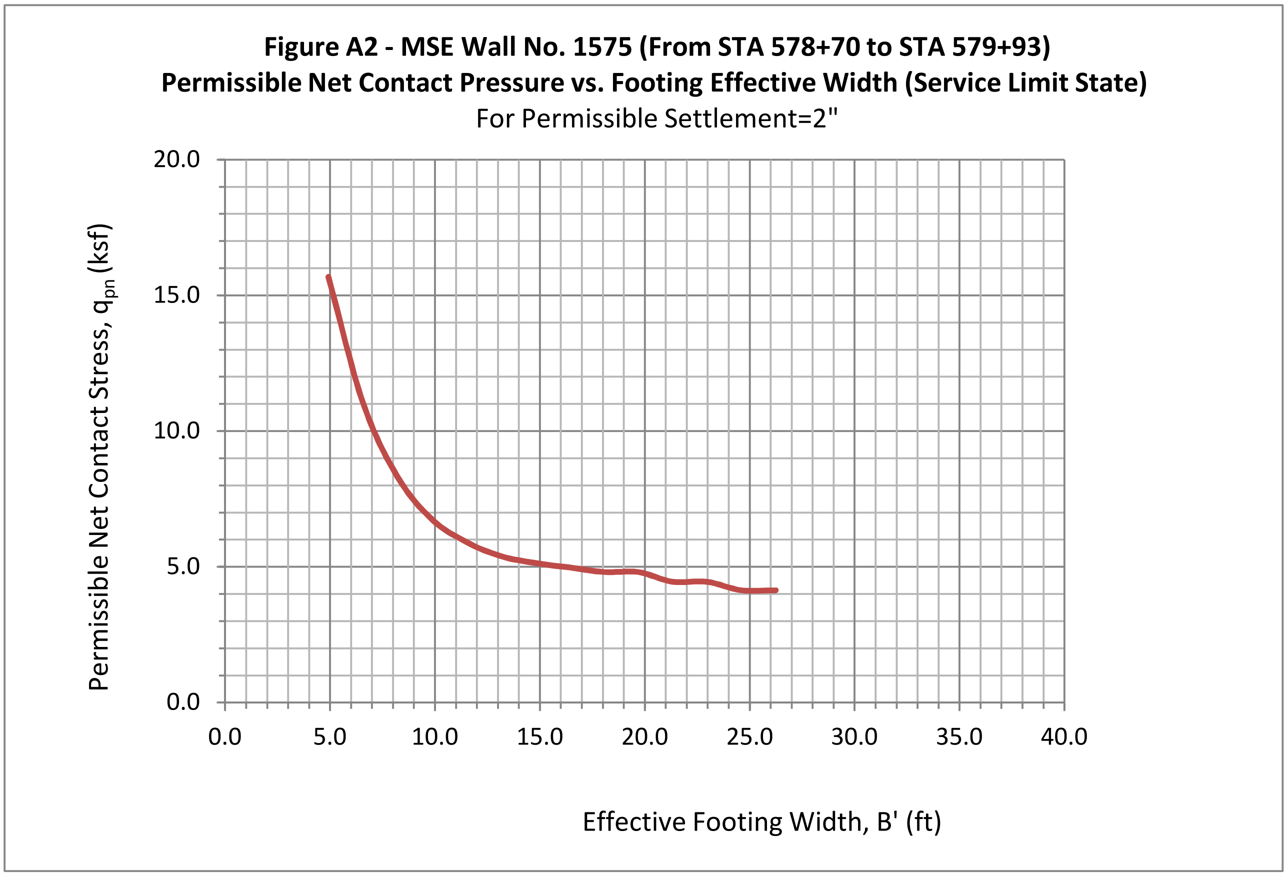


Figure A2 - MSE Wall No. 1575 (From STA 578+70 to STA 579+93)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"
Permissible Net Contact Stress, qpn (ksf)
20.0
15.0
10.0
5.0
0.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Effective Footing Width, B' (ft)

Figure A3 - MSE Wall No. 1575 (From STA 575+68 to STA 578+70)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"

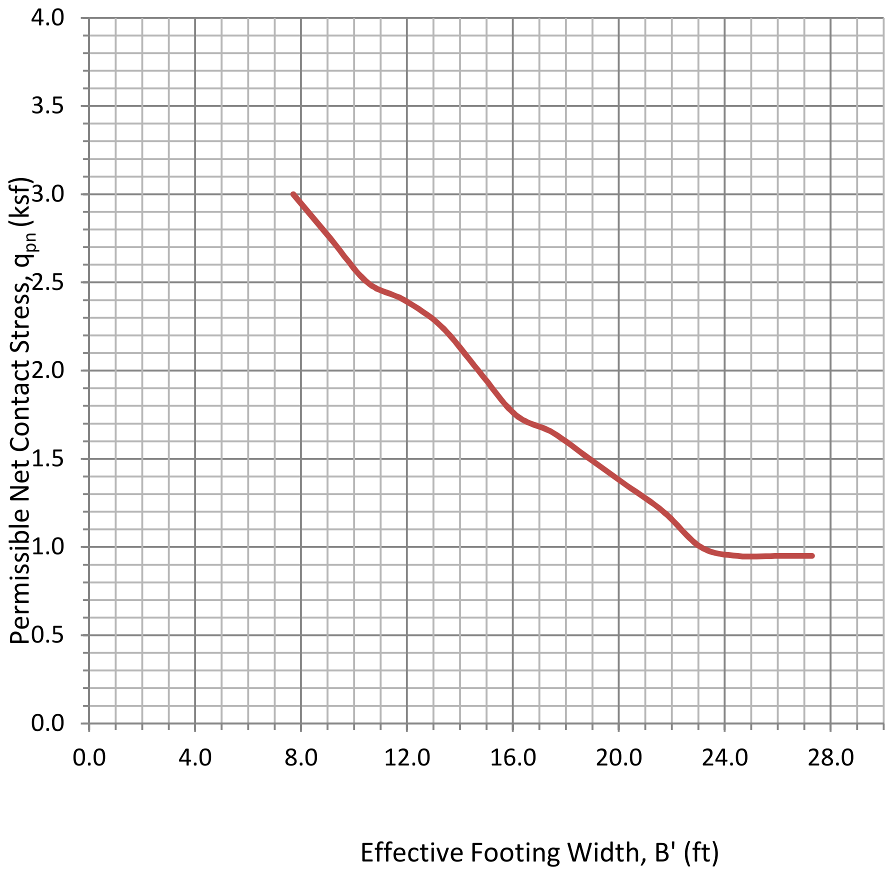
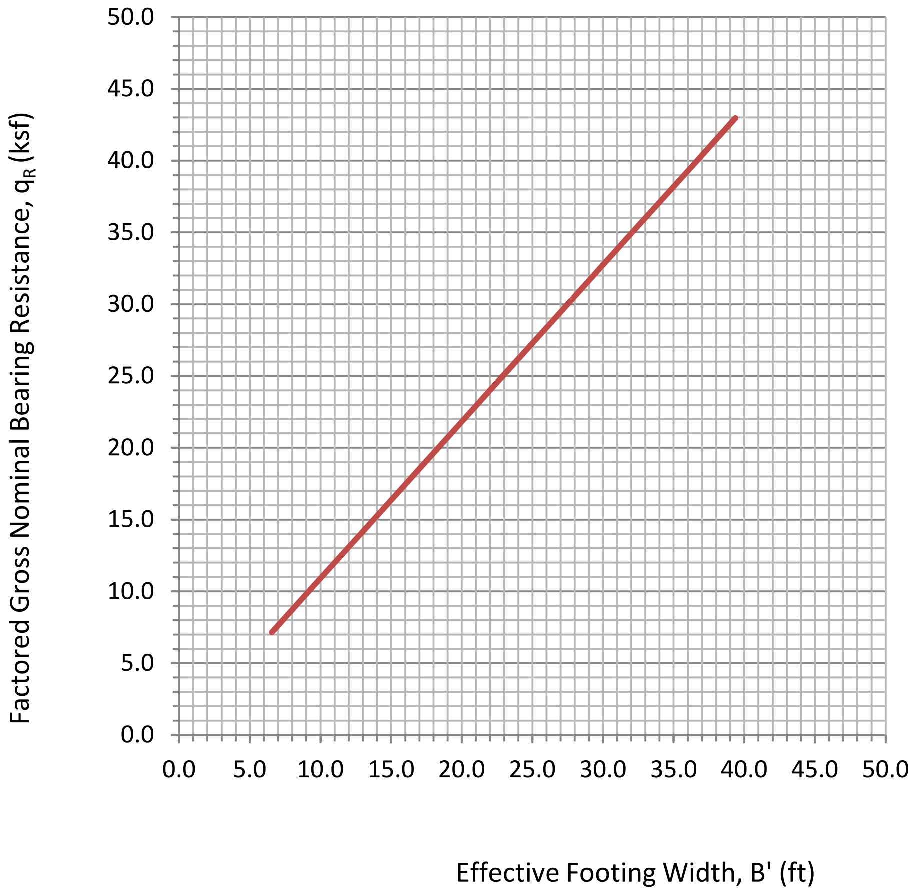


Figure A4 - MSE Wall No. 1575 (From STA 578+70 to STA 579+93)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)



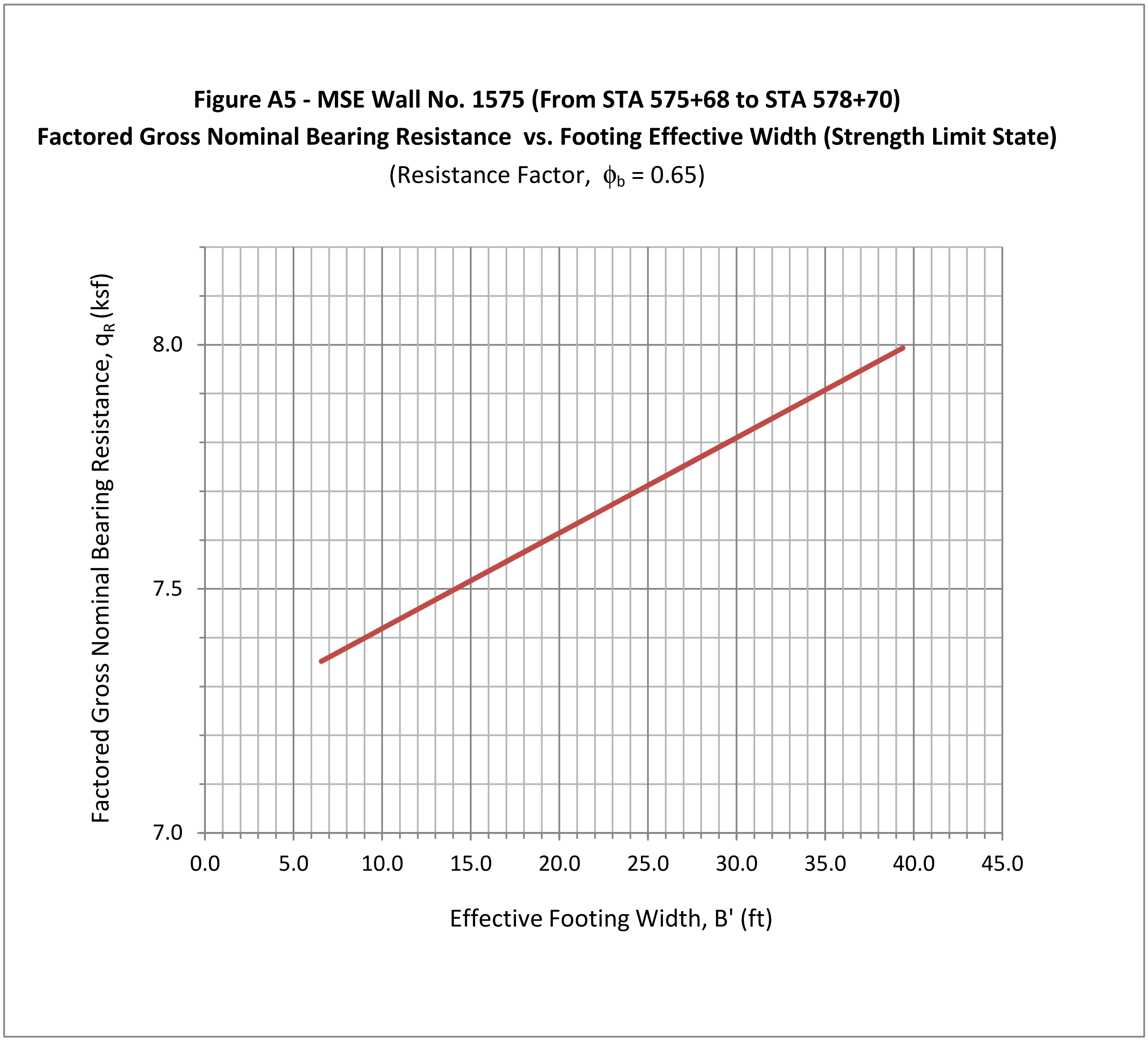


Figure A5 - MSE Wall No. 1575 (From STA 575+68 to STA 578+70)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)
Factored Gross Nominal Bearing Resistance, q_R (ksf)
8.0
7.5
7.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
Effective Footing Width, B' (ft)

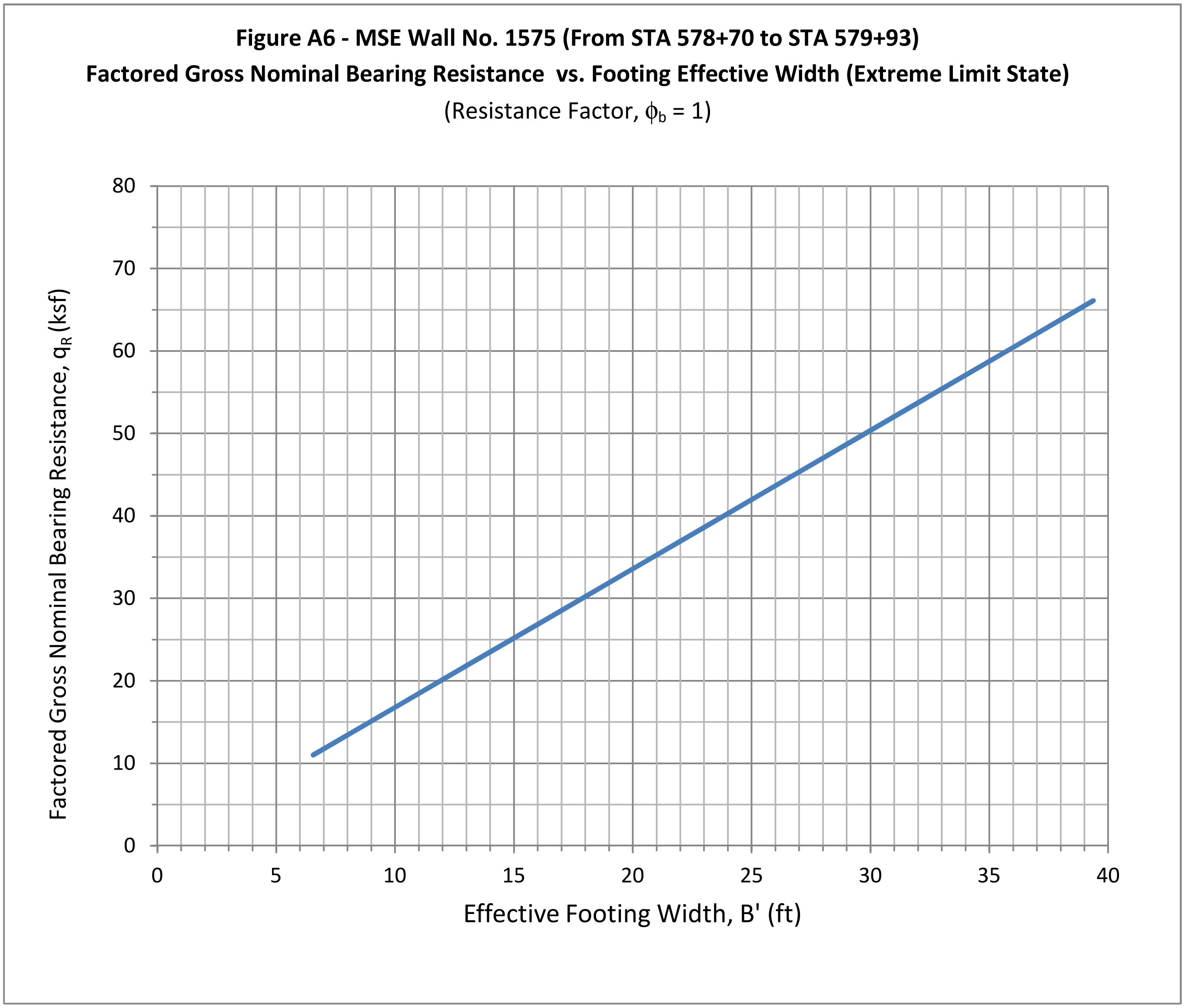


Figure A6 - MSE Wall No. 1575 (From STA 578+70 to STA 579+93)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Extreme Limit State)
(Resistance Factor, ϕ_b = 1)
Factored Gross Nominal Bearing Resistance, q_R (ksf)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
Effective Footing Width, B' (ft)

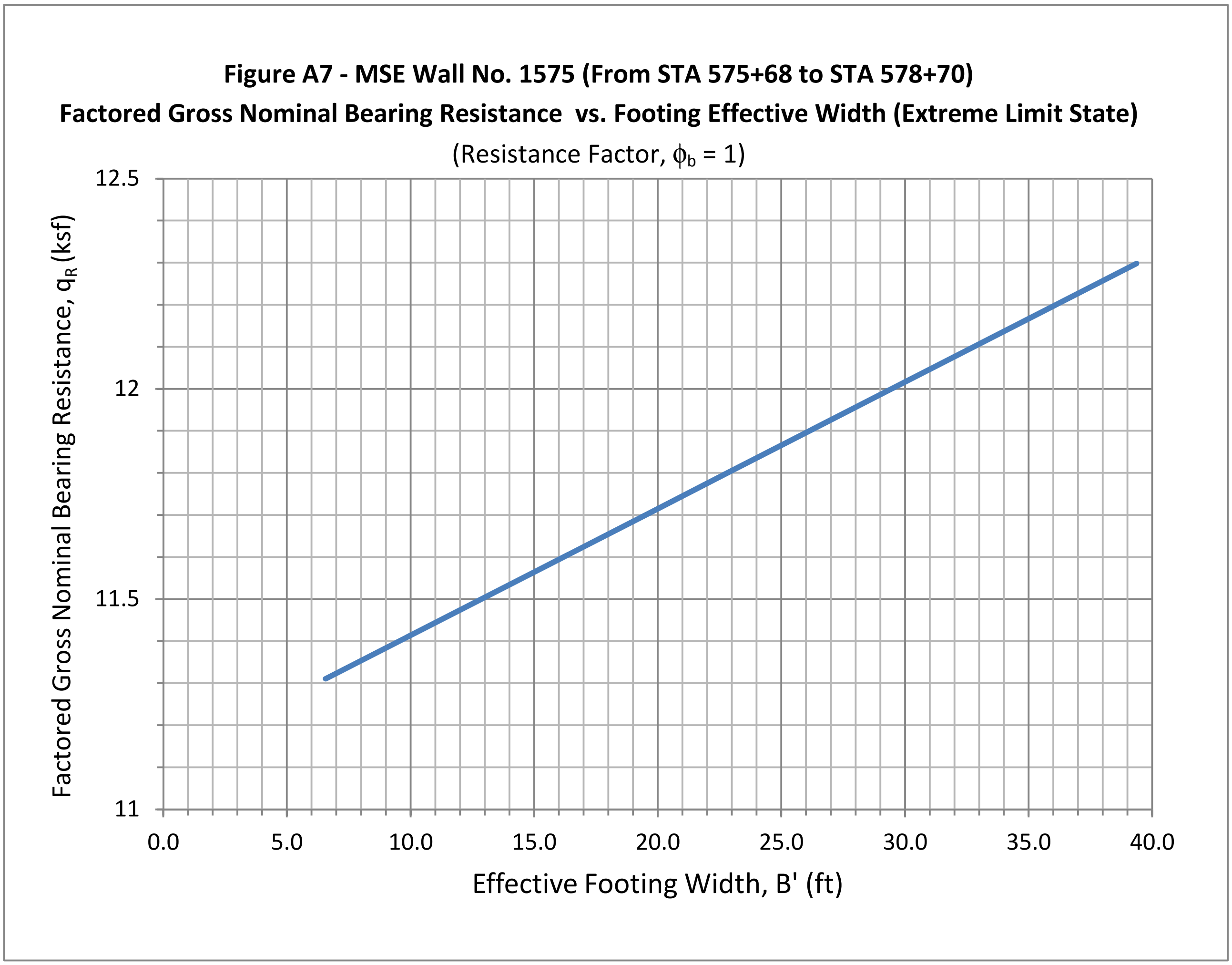


Figure A7 - MSE Wall No. 1575 (From STA 575+68 to STA 578+70)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Extreme Limit State)
(Resistance Factor, ϕ_b = 1)
Factored Gross Nominal Bearing Resistance, q_R (ksf)
12.5
12
11.5
11
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Effective Footing Width, B' (ft)

State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
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To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: MR. ULYSSES SMPARDOS
Project Engineer

Date: February 15, 2012

File: 07-LA-5-PM 29.83
07000211191 (EA 07-1218W1)

Retaining Wall 1575 over
Stough Canyon Channel and
Burbank Western Channel

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Retaining Wall 1575 over Stough Canyon Channel and Burbank Western Channel, from STA 573+73 to 575+68

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of a portion of the Retaining Wall 1575, from STA 573+73 to 575+68, over two existing culverts. The foundation recommendations are based on the subsurface information gathered during the foundation investigation (2007 to 2009) along with the review of “As-Built” Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).

2.0 Project Description

Wall No. 1575 is a part of the project that proposes to replace the Burbank Blvd. Overcrossing and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd. Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd., but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Wall No. 1575 is one of them. In order to reduce the load from the embankment fill over two existing culverts, from STA 573+73 to 575+68, use of lightweight fill - EPS (expanded polystyrene material) block is proposed as the embankment fill.

3.0 Geotechnical Exploration

For this project the geotechnical investigation has been conducted from 2007 to 2009. Two Cone Penetrometer Tests (CPT) have been performed at the location of the proposed structure. A summary of CPTs is presented in Table No. 1. Surface elevation, station, and offset of the Boring were provided by District 7 Surveys Branch.

Table No. 1 – Summary of Boring

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
CPT-07-018	12/5/07	1576+20.30	242.75 L	Existing I-5 C/L	608.52	78.0	Not encountered.
CPT-07-17A	12/6/07	1574+28	240.6 L		619.7	60.7	

Note: Vertical datum NAVD 88

4.0 Site Geology and Subsurface Condition

4.1 Site Geology

Due to the vicinity of the subject Structure to the proposed Burbank Blvd OC, Site Geology for Burbank Blvd OC is applicable for this structure. Therefore, please refer to the Section on "Site Geology" of the Foundation Report for Burbank Blvd OC, dated August 25, 2011.

4.2 Subsurface Conditions for Wall 1575 from STA 573+73 to 575+68

From elevation 603 to 583, soil consists of sand, silty sand, sand with silt and clayey silt. From elevation 583 to 568, soil consists of clay, silty clay and clayey silt. From elevation 568 to 563, soil consists of sand, silty sand, sand with silt and clayey silt. From elevation 563 to 552 soil consists of clay, silty clay and clayey silt. The invert of the bottom of the culvert in the vicinity of CPT 18 is approximately elevation 563. The approximate base of the fill in the vicinity of CPT 18 is elevation 552.

From elevation 552 to 538, soil consists of sand, silty sand, sand with silt, clayey silt, clay, silty clay and clayey silt. From elevation 538 to 528, soil consists of sand, silty sand, sand with silt and clayey silt.

The subject structure is very close to the proposed Burbank Blvd OC. The geotechnical exploration conducted for the proposed Burbank Blvd OC includes several exploratory borings, and they indicate occasional presence of gravel and cobbles in soil layers (Foundation Report for Burbank Blvd OC, dated August 25, 2011).

4.3 Groundwater

Due to the vicinity of the subject Structure to the proposed Burbank Blvd OC, ground water condition for Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on "Groundwater" of the Foundation Report for Burbank Blvd OC, dated August 25, 2011.

4.4 Corrosion Evaluation

Due to the vicinity of the subject Structure to the proposed Burbank Blvd OC, Corrosion Evaluation for Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on "Corrosion Evaluation" of the Foundation Report for Burbank Blvd OC, dated August 25, 2011.

4.5 Seismicity

Due to the vicinity of the subject Structure to the proposed Burbank Blvd OC, Seismicity for Burbank Blvd OC is applicable for this wall. Therefore, please refer to the Section on "Seismicity" of the Foundation Report for Burbank Blvd OC, dated August 25, 2011.

5.0 Foundation Recommendations

The proposed structure is supported by 24 inch cast-in-drilled-hole (CIDH) piles. Along the depth of the existing drainage structures the piles will be cased to prevent down drag on the existing drainage structures. The foundation design data and foundation loads were provided by the Office of Structure Design – Branch 15 (SD). Table No. 2 shows the foundation design data and foundation loads.

Table No. 2 – Foundation Design Data Table

Location	Design Method	Pile Type	Factored Load (kips)	
			Compression	Tension
Wall over culverts	LFD	24-inch CIDH	200	50

The pile resistance was estimated using the software, Shaft 5.0, developed by the Ensoft, Inc. Table No.3 shows the foundation design recommendations.

Table No. 3 – Foundation Design Recommendations

Location	Pile Type	Approximate Cut-off Elevation (ft)	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
			Compression	Tension			
Section AA (LH side)	24” CIDH	577	400	100	534 (a) 556 (b)	534	N/A
Section AA (RH side)	24” CIDH	580	400	100	536 (a) 558 (b)	536	N/A
Section CC (pile with sleeve)	24” CIDH	581	400	100	534 (a) 557 (b)	534	N/A
Section CC (pile without sleeve)	24” CIDH	581	400	100	539 (a) 562 (b)	539	N/A
Section DD	24” CIDH	581	400	100	539 (a) 562 (b)	539	N/A
Section EE (LH side)	24” CIDH	580	400	100	530 (a) 559 (b)	530	N/A
Section EE (RH side)	24” CIDH	583	400	100	530 (a) 559 (b)	530	N/A
Section FF (LH side)	24” CIDH	583	400	100	530 (a) 559 (b)	530	N/A
Section FF (RH side)	24” CIDH	580	400	100	529 (a) 558 (b)	529	N/A
Section GG	24” CIDH	583	400	100	530 (a) 559 (b)	530	N/A
Section HH (pile with sleeve)	24” CIDH	583	400	100	529 (a) 560 (b)	529	N/A
Section HH (pile without sleeve)	24” CIDH	583	400	100	539 (a) 562 (b)	539	N/A

Notes:
1. N/A = Not Applicable
2. Design tip elevations are controlled by: (a) Compression, (b) Tension.
3. Designer may use the controlling (lowest) specified tip elevation for all piles located within Stations 573+73 to 575+68."

Lightweight Fill – (EPS Geofoam)

In order to reduce the load from the embankment fill over the existing culverts, use of lightweight fill - EPS (expanded polystyrene material) geofoam is recommended as the embankment fill. District Material Engineer needs to evaluate minimum thickness of compacted soil backfill between the pavement structure section and EPS geofoam in order to reduce load demand on EPS geofoam. A compacted soil backfill layer can be placed above the EPS geofoam for the roadway geometry design purpose to maintain super elevations or sliver fill.

Lightweight fill (EPS geofoam) shall be fabricated as blocks measuring approximately 2-ft x 4-ft x 8-ft. Manufacturer's standard size blocks will be acceptable. Special-size blocks will be required at the edges of the lightweight fill section to fill the volume shown on the contract plans. Except as specified herein, EPS geofoam block material need to meet or exceed the requirements of the ASTM D6817, Type EPS29.

EPS geofoam block must comply with:

Physical Property	ASTM Designation	Acceptance Value
Density	C303	1.5 lb/ft^3 Minimum 2 lb/ft^3 Maximum
Compressive Strength (at 5% deformation)	D1621	14.5 psi Minimum
Flexural Strength	C203	43.5 psi Minimum
Tensile Strength	D1623	20 psi Minimum
Water Absorption	C 272	2.0% Maximum by Volume

The EPS geofoam shall not be exposed to hydrocarbons contamination or ultraviolet light. Between stages, the lightweight embankment fill (EPS geofoam) shall be protected with geomembrane (Type A-gasoline resistant) and weighted down with sufficient sand bags to keep it in place. Exact location of geomembrane (Type A-gasoline resistant) will be shown on the plan detail sheets and discussed in the special provisions.

Geomembrane (gasoline resistant) shall consist of reinforced or unreinforced tri-polymer membrane consisting of polyvinyl chloride (PVC), ethylene interpolymer alloy, and polyurethane or a comparable polymer combination. The geomembrane shall be suitable for the containment of spilled liquid hydrocarbons, including gasoline, diesel fuel, kerosene, hydraulic fluid, methanol, ethanol, mineral spirits, and naptha. The geomembrane shall be sufficiently flexible to cover and closely conform to 90 degree edges and corners of lightweight fill (EPS geofoam) subgrade material at ambient temperatures as low as 45°F without application of heat.

6.0 Construction Considerations

- All work should be performed in accordance with the Caltrans Standard Specifications (2006) except as indicated in Special Provisions prepared for this project.
- Earthwork should be performed in accordance with Section 19 of the latest Caltrans Standard Specifications.
- Proposed additional fills and EPS geofoam for pavement widening should be keyed into the existing embankments and placed as specified in Section 19-3 of the Caltrans Standard Specifications (2006).
- In order to prevent deterioration of CIDH piles from caving, contractor may use wet construction method to construct the CIDH piles.

.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 2/15/2012

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DES Office Engineer, Office of PS&E
District Materials Engineer

State of California Business, Transportation and Housing Agency
DEPARTMENT OF TRANSPORTATION

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: MR. ULYSSES SMPARDOS
Project Engineer

Date: January 12, 2012

File: 07-LA-5-PM 29.85
07000211191 (EA 07-1218W1)
Type 1 Wall No. 1576

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Type 1 Wall No. 1576

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the MSE Wall No. 1576. The foundation recommendations are based on the subsurface information gathered during the recent foundation investigation (2007 to 2008) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd OC (Bridge No. 53-1089).

2.0 Project Description

Type 1 Wall No. 1576 is a part of the project that proposes to replace the Burbank Blvd Overcrossing and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd Overcrossing will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Type 1 Wall No. 1576 is one of them.

Information of the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1576	NB I-5 On-ramp	Type 1 Wall	From 574+40 to 577+21	281	4 to 14

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed from November 28, 2007 to March 12, 2008. The field investigation included one hollow stem auger boring and one mud rotary boring. Borings were logged and sampled using a Standard Penetration Test (SPT) sampler. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inches.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

LOTBs (Log of Test Borings) are being prepared by the Office of Geotechnical Support and will be submitted to your office upon completion.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
R-08-023	3/12/08	1574+58.86	72.28 R	Existing I-5 C/L	594.07	53.2	Not encountered.
A-07-024	11/28/07 - 11/29/07	1576+84.79	51.54 R		593.23	61.0	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing

SPT soil samples and bulk samples obtained from borings were tested for the following laboratory testing:

- Mechanical Analysis
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below), at the Material Laboratory in Los Angeles.

Table 3 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 202, 203
Atterberg Limits of Soils	CTM 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
R-08-023	7-52	9.23	6400	-	-
A-07-024	4.5-59.5	8.14	3600	-	-

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Conditions

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness up to approximately 10 feet. The fill consists of poorly graded sand and silty sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 4.4, Seismicity).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the two borings drilled for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense, fine to coarse sand and sandy silt with gravel and occasional cobbles. Below the fill material, the alluvium is composed of medium stiff to stiff sandy silty clay and loose to dense sand and silty sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2007-2008 investigation for this project to the total depth explored of approximately 103.2 feet below ground surface (elevation +512 feet in Boring No. A-08-014 for Bridge 53-3057). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +593 feet to +595 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo. The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 5 below.

Table No. 5 - Summary of Faults

Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (km)	R_{JB} (km)	R_{RUP} (km)
Verdugo Fault	418	R	6.9	1.7	1.7	1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	8.7	8.7	8.7
Hollywood fault	282	LLSS	6.6	9.1	2.9	8.6
Upper Elysian Park Blind Thrust	239	R	6.4	4.4	7.0	8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure A1 in Appendix A. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure A1 in Appendix A. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading.

Due to the fact no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

5.1 Foundation Analysis

This wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). From a geotechnical standpoint, the wall supported on spread footing is feasible.

First GS provided the following information based on the preliminary information provided by SD such as bottom of footing elevation, the potential footing width, and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figures A2 and A3).
2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figures A4 and A5).
3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figures A6 and A7).
4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for retained soil and 34^0 for backfill).
5) Unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 6) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis.

Table No. 6 – Wall Data Table

Wall Height 'H' (ft)	Bottom of Footing Elevation (ft)	Design Load Case per Standard Plan B3-8	Slope in front of footing	Begin Station	End Station
14	593.50	I	Level	574+40	574+80
12	593.50	I	Level	574+80	575+40
10	593.50	I	Level	575+40	576+40
6	592.00	I	Level	576+40	577+00
4	592.00	I	Level	577+00	577+21.25

5.2 Global Slope Stability

The slope stability analyses were performed to verify the overall stability using the computer program SLOPEW under both static and pseudo-static conditions. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

6.0 Construction Considerations

1. The proposed wall should be founded on properly compacted competent soil. Loose or soft material is not expected at this project site. If clay or loose sand is encountered within the areas to receive the walls, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:
   - (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
   - (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
   - (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 1/12/2012

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DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for Burbank Blvd OC
Damping Ratio = 5%; V_{s30} = 295 m/sec

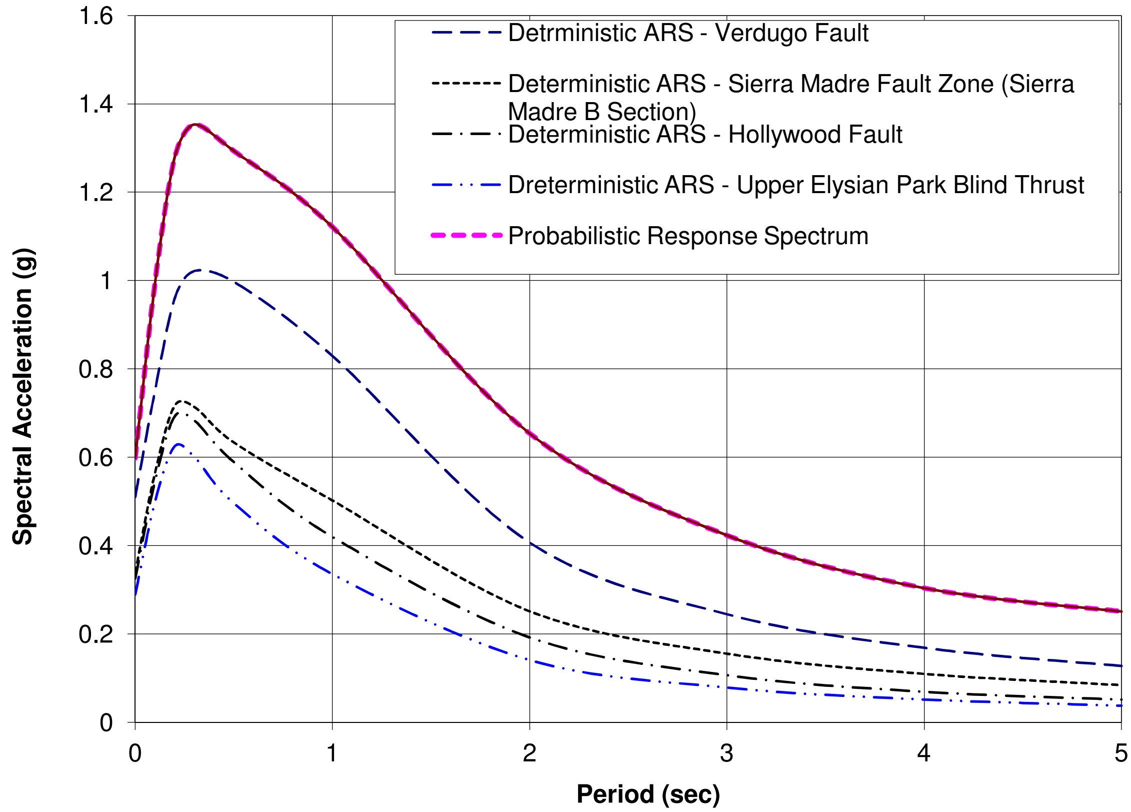


Figure A2 – Type 1 Wall No. 1576 (from STA 574+40 to STA 575+40)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"

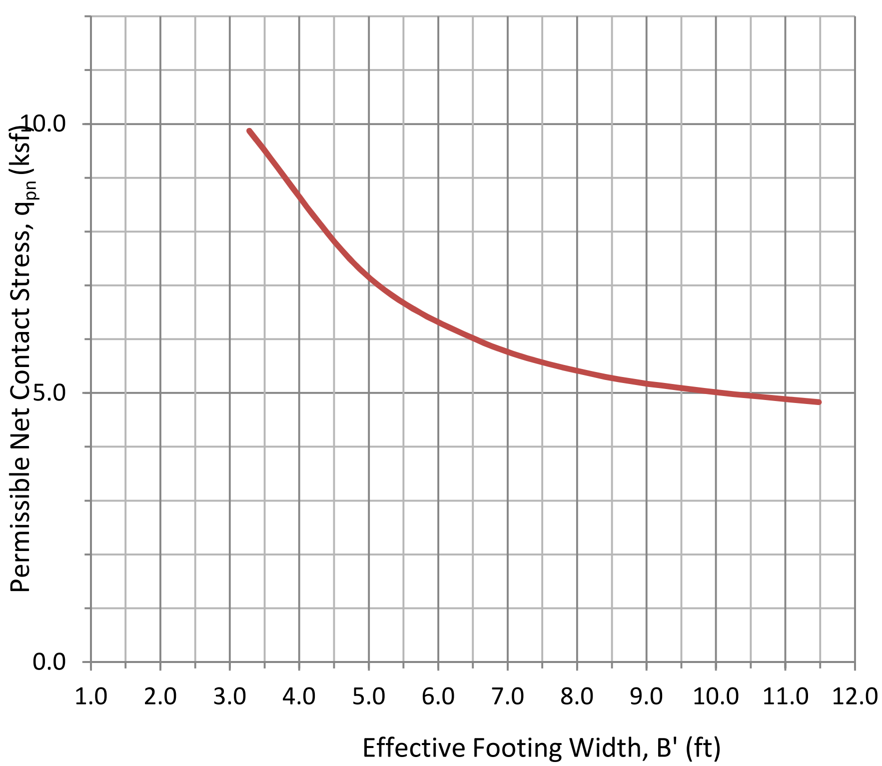


Figure A3 – Type 1 Wall No. 1576 (from STA 575+40 to STA 577+21)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"

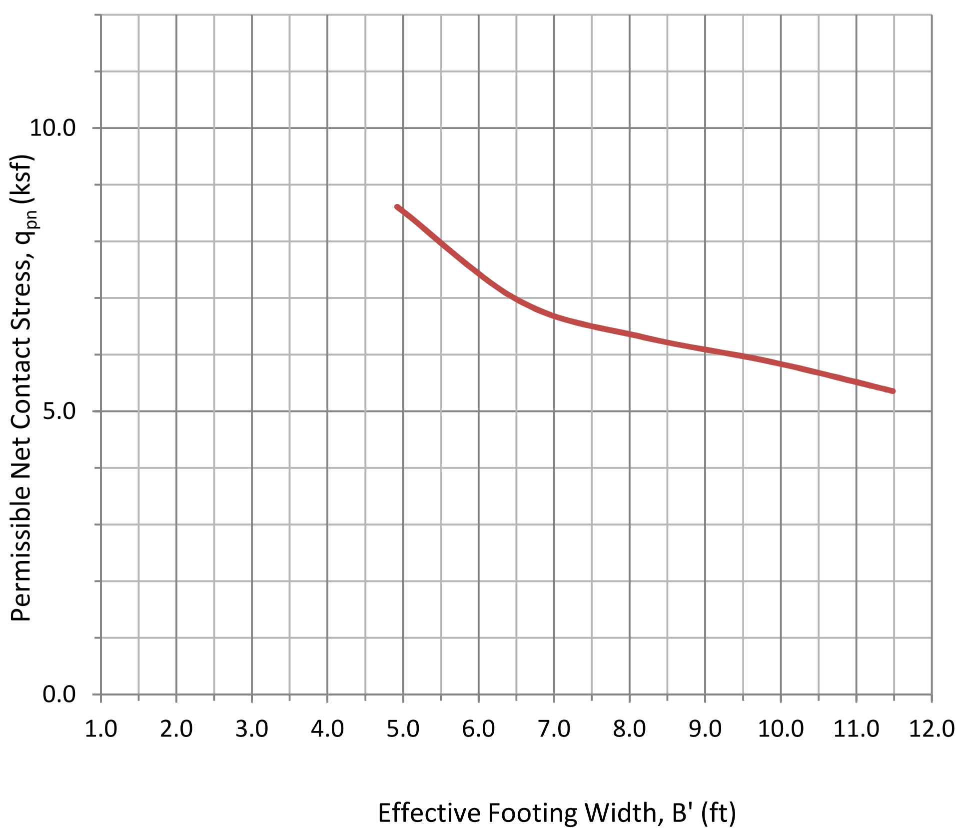


Figure A4 – Type 1 Wall No. 1576 (from STA 574+40 to STA 575+40)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)

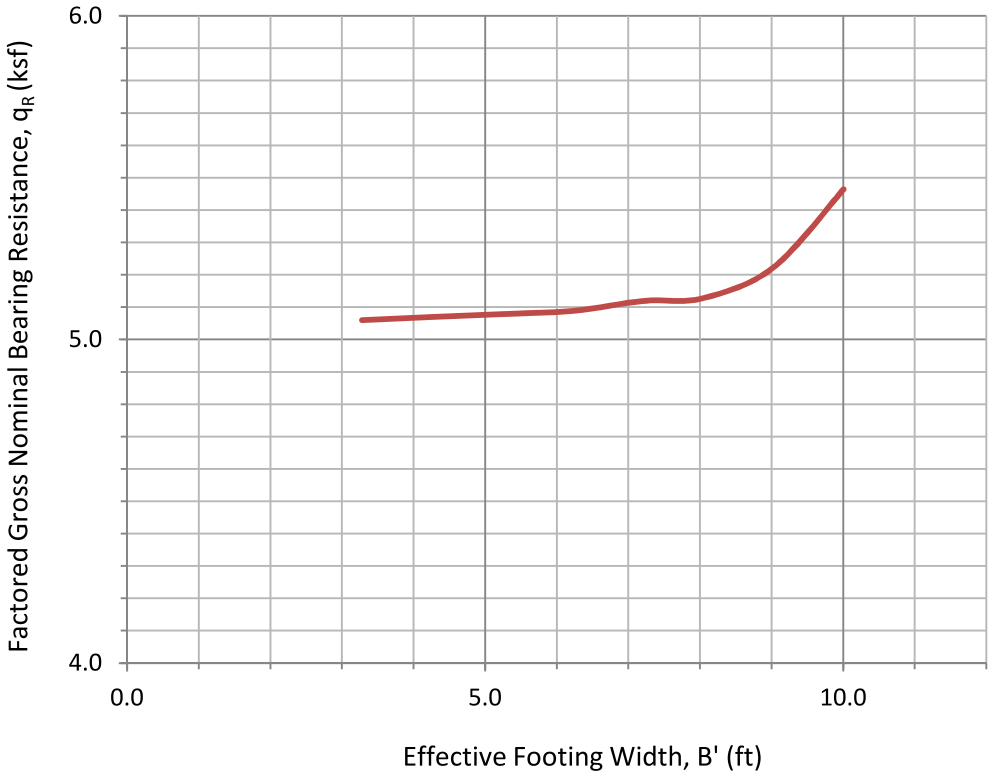


Figure A5 – Type 1 Wall No. 1576 (from STA 575+40 to STA 577+21)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)

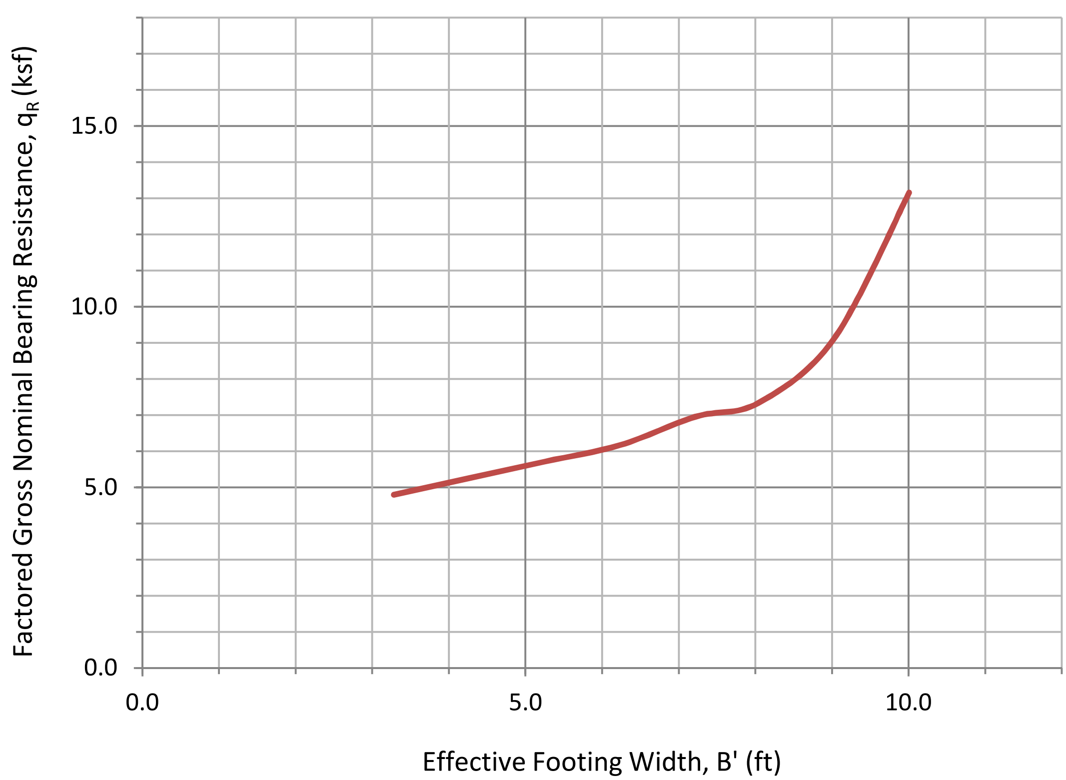


Figure A6 – Type 1 Wall No. 1576 (from STA 574+40 to STA 575+40)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Extreme Limit State)
(Resistance Factor, $\phi_b = 1$)

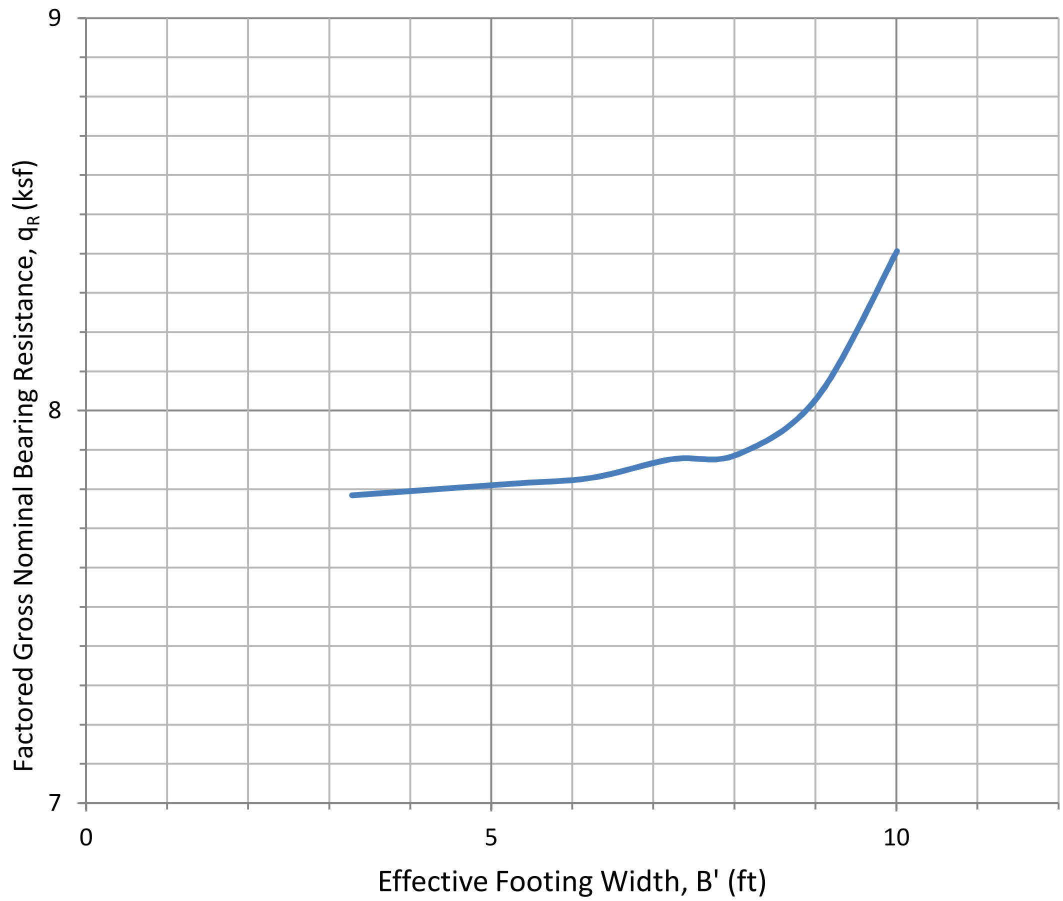
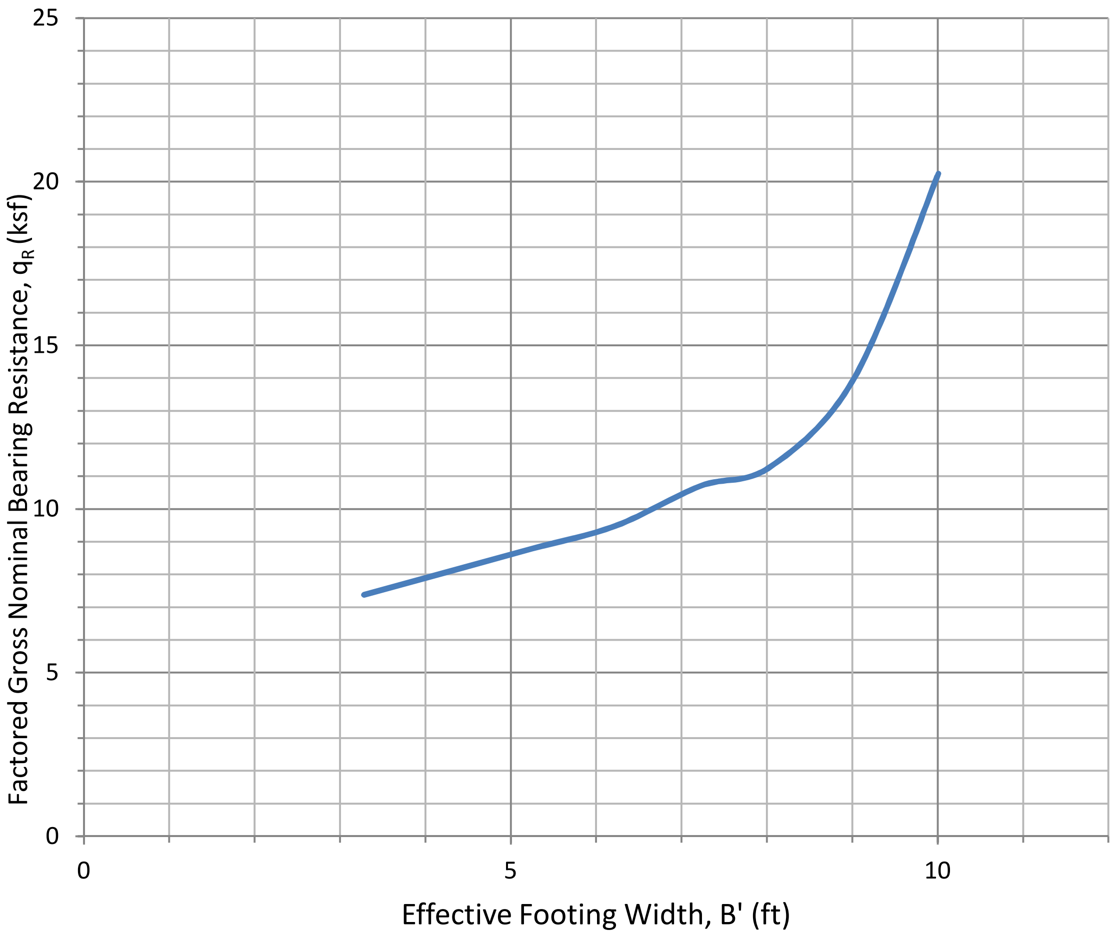


Figure A7 – Type 1 Wall No. 1576 (from STA 575+40 to STA 577+21)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Extreme Limit State)
(Resistance Factor, ϕ_b = 1)



State of California
DEPARTMENT OF TRANSPORTATION

Business, Transportation and Housing Agency

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Flex your power!
Be energy efficient!

To: MS. TRACI MENARD
Chief, Bridge Design Branch 15

Attn: MR. ULYSSES SMPARDOS
Project Engineer

Date: February 27, 2012

File: 07-LA-5-PM 29.92
07000211191 (EA 07-1218W1)
Wall No. 1585

From: **DEPARTMENT OF TRANSPORTATION**
Division of Engineering Services
Geotechnical Services
Office of Geotechnical Design – South 1
Branch C

Subject: Foundation Report for Retaining Wall No. 1585

1.0 Scope of Work

The Office of Geotechnical Design South 1 has prepared this Memorandum to provide the foundation recommendations for the construction of the Wall No. 1585. The foundation recommendations are based on the subsurface information gathered during the foundation investigation (2005 and 2007) along with the review of "As-Built" Log of Test Borings (LOTB) for the existing Burbank Blvd. Overcrossing (OC) (Bridge No. 53-1089).

2.0 Project Description

Wall No. 1585 is a part of the project that proposes to replace the Burbank Blvd. OC and ramps at I-5/Burbank Blvd Interchange. The existing 3-span Burbank Blvd. OC will be replaced with a new 2-span bridge. The new bridge will be built along the existing alignment of Burbank Blvd, but will be shifted about 144 feet to the west to allow the realignment of I-5 beneath the bridge. The new bridge spans will be longer to accommodate the new I-5 HOV lanes, and the replacement bridge will be wider as well. In addition to the replacement of the existing bridge, the four existing ramps at the interchange will be removed and replaced with the reconfigured ramps, which will include the construction of five new retaining structures at the replacement ramps. Wall No. 1585 is one of them.

Information for the proposed retaining wall is given in the Table No. 1 below.

Table No. 1- Retaining Wall Data

Wall No.	Location	Structure Type	Stations (Based on Wall LOL)	Length (ft)	Wall Height (ft)
1585	SB I-5 Off-ramp	MSE Wall	From 578+61 to 585+76	715.00	9.17 to 26.67
		Type 1 Wall	From 585+76 to 588+21	239.25	4.00 to 8.00

3.0 Geotechnical Exploration

3.1 Field Exploration Program and Testing Program

Site-specific field exploration was performed in August, 2005 and in November and December, 2007. The field investigation included three hollow stem auger borings and one mud rotary boring. Borings were logged and sampled using a Standard Penetration Test (SPT) sampler. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 1.4 inch I.D. sampler with a 140-lb hammer dropped 30-inches.

A summary of exploratory borings is presented in Table No. 2. Surface elevations, stations, and offsets of the Borings were provided by District 7 Surveys Branch.

Table No. 2 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (ft)	Reference Line	Surface Elevation (ft)	Total Depth (ft)	Groundwater Elevation (ft)
05-024	8/11/05-8/12/05	1585+38.7	59 L	Proposed I-5 C/L	601.17	66.5	Not encountered.
05-042	8/22/05-8/23/05	1582+11.3	70 L		599.15	65.8	
A-07-021	11/30/07	1578+41.5	186 L		607.19	51.5	
A-07-022	12/3/07-12/4/07	1579+41.0	130 L		602.05	51.5	

Note: Vertical datum NAVD 88

3.2 Laboratory Testing

SPT soil samples and bulk samples obtained from borings were tested for the following laboratory testing:

- Mechanical Analysis
- Atterberg Limits
- Corrosion

Laboratory tests were performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below), at the Material Laboratory in Los Angeles and at laboratory selected by the geotechnical consultant URS, Corp.

Table 3 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 202, 203
Atterberg Limits of Soils	CTM 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

3.3 Corrosion Evaluation

A summary of corrosion test results is presented in Table No. 4.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (ft)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
05-024	5-10	7.9	1000	123	75

Note: * The Corrosion Technology Branch policy states that if the minimum resistivity is greater than 1000 ohm-cm the area is considered to be non-corrosive and sulfate and chloride contents are not tested.

The Department considers a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

Based on the on the results of corrosion analyses, the site is considered non corrosive to metal and reinforced concrete.

4.0 Site Geology and Subsurface Conditions

4.1 Site Geology

The entire project (including the existing fill embankments) is directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Verdugo Hills and the San Gabriel Mountains to the north of the San Fernando Valley adjacent to the project location. The alluvium consists of predominantly medium dense to dense sand that in some areas include sparse to abundant gravel and cobbles. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. Fill ranges in thickness up to approximately 25 feet. The fill consists of poorly graded sand and silty sand and silty clayey sand with some gravel.

The closest fault to the site is the Verdugo fault oriented in a northwest-southeast direction and it has been included on maps by Mualchin (1996) and Dibblee (1991) approximately 1.06 miles north of the proposed project (Please see also Section 4.4, Seismicity).

4.2 Subsurface Conditions

Subsurface soil conditions along the proposed wall alignment was determined based on the four borings drilled for this project and the as-built LOTB for Bridge 53-1089. The subject area generally consists of artificial fill that overlies alluvium. This artificial fill material is composed of poorly graded medium dense to very dense, fine to coarse sand and silty sand and silty clayey sand with gravel and occasional cobbles. Below the fill material, the alluvium is composed of medium stiff to stiff silt with sand and sandy silt and loose to dense sand and silty sand with fine to coarse gravel and cobbles.

4.3 Groundwater

Groundwater was not encountered during the 2007-2009 investigation for this project to the total depth explored of approximately 103.2 feet below ground surface (elevation +512 feet in Boring No. A-08-014 for Bridge 53-3057). Groundwater was not encountered during the 1957 investigation for Bridge 53-1089, Burbank Blvd OC. The elevation of the existing ground surface along the proposed wall alignment ranges from approximately +599 feet to +607 feet. Ground water level data in the area has been obtained from the Los Angeles County Department of Public Works web site, www.ladpw.org/wrd/wellinfo. The closest well to the site well number 3871H, located approximately 0.6 mile west of the project site, had a maximum reading from 1994 to 1997 as an elevation of 488.0 feet above mean sea level (MSL).

4.4 Seismicity

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. An analysis was performed to develop and recommend ground motion parameters for the seismic design of the I-5/Burbank Blvd OC. This analysis was performed in accordance with requirements specified in Appendix B of the Caltrans' 2009 Seismic Design Criteria (SDC, Version 1.5, August 2009) for ordinary bridge structures, and utilizing the "Caltrans ARS Online" and other tools available at the internet sites. The average shear wave velocity (V_{s30}) for the upper 100 feet of the subsurface profile was estimated to be about 295 m/sec based on SPT blow counts.

The significant faults and fault zones for the bridge site are summarized in the Table No. 5 below.

Table No. 5 - Summary of Faults

Fault Name	Fault ID #	Type of Fault	M_{max}	R_X (mile/km)	R_{JB} (mile/km)	R_{RUP} (mile/km)
Verdugo Fault	418	R	6.9	1.1/ 1.7	1.1/1.7	1.1/ 1.7
Sierra Madre Fault Zone (Sierra Madre B Section)	248	N	7.2	5.4/ 8.7	5.4/ 8.7	5.4/ 8.7
Hollywood fault	282	LLSS	6.6	5.6/ 9.1	1.8/ 2.9	5.3/ 8.6
Upper Elysian Park Blind Thrust	239	R	6.4	2.7/ 4.4	4.3/ 7.0	5.5/ 8.8

Notes: R_X = Horizontal distance to the fault trace
R_{JB} = Shortest horizontal distance to the surface projection of the rupture area
R_{RUP} = Closest distance to the fault rupture plane

The deterministic as well as the probabilistic acceleration response spectrum (ARS) curves developed are shown in the Figure 1. The probabilistic ARS curve corresponds to a ground motion return period (RP) of 975-year (i.e., 5% probability of exceedance in 50 years). ARS curves were developed according to the Caltrans Geotechnical Services-Design Manual (Version 1.0, Aug. 2009). The design Peak Ground Acceleration (PGA) for the project site is 0.65g.

The Design ARS curve recommended for design is also shown in Figure 1. This Design ARS curve was developed by enveloping the deterministic and the probabilistic ARS curves.

4.5 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, fine grained granular soils behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy soils and (3) high-intensity ground motion. Saturated, loose and medium dense, near surface cohesionless soils exhibit the greatest liquefaction potential, while dense cohesionless soil and cohesive soil exhibit the lowest, negligible liquefaction potential. Effects of liquefaction on ground surface include sand boils, settlement and lateral spreading. Due to the fact no groundwater was encountered at the site, the liquefaction potential is considered to be low.

5.0 Foundation Recommendations

Type1 Wall from 585+76 to 588+21

SD provided Wall Data Table (Table No. 6) given below. Allowable bearing capacity was calculated using Terzaghi's equation. A factor of safety of 3 was used. The allowable bearing capacity obtained was compared against the toe pressure given on the Caltrans Standard Plans.

Due to the granular nature of the underlying granular soils at this portion of the wall, the settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

From a geotechnical standpoint, the wall supported on spread footing is feasible.

Table No. 6 – Wall Data Table

Design 'H' (feet)	Base Width (feet)	BOF Elevation (feet)	Design Load Case per Standard Plan B3-8	Slope in front of footing	Begin Station	End Station	Distance (feet)
8'-0"	5'-3"	594.200	I	Level	585+76.162	586+27.579	51.42
6'-0"	4'-3"	595.100	I	Level	586+33.329	587+36.162	102.83
4'-0"	3'-3"	596.600	I	Level	587+36.162	588+21.162	85.00

MSE Wall from 578+61 to 585+76

The MSE wall is being designed by the Office of Structures Design – Branch 15 (SD), based on the information provided by our office (GS). The MSE wall is being designed as per Section 3-8 (Mechanically Stabilized Embankment) of Caltrans Bridge Design Aids, March, 2009. The Caltrans Standard Drawings (xs13-020-1e to xs13-020-6e) also are being used.

First GS provided the following information based on the preliminary information provided by SD such as elevation of leveling pad, minimum embedment depth, the potential range of width, B, and the permissible settlement limit.

1) A plot of Permissible Net Contact Stress (Service I Limit State) vs. the effective footing width (B') for permissible settlement (Figures A2 and A3).
2) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Strength Limit State design (Figures A4 and A5).
3) A plot of Factored Gross Nominal Bearing Resistance vs. B' for Extreme Event Limit State design (Figures A6 and A7).
4) Total unit weight (120 pcf) and effective friction angle for the retained fill (32^0 for unreinforced retained soil and 34^0 for reinforced backfill).
5) Unit weight (120 pcf) and effective friction angle of the foundation soil (32^0; if clay is found at the bottom of footing elevation, item no. 1 in the Section 6 (Construction Consideration) of this report should be referred).

Then SD selected the wall parameters to meet the service, strength and seismic design requirements using this information. SD is responsible for sliding and overturning/ rotational failure checks.

Once SD provided the updated Wall Data Table (Table No. 7) given below, GS performed the static global stability analysis pseudo-static (seismic) global stability analysis, using the computer program SLOPEW. The slope stability analysis under pseudo-static condition was performed using a seismic coefficient equal to one-third of the horizontal ground acceleration and not exceeding 0.2g. The slope stability analyses were performed using the Bishop, Ordinary and Jambu methods for circular slip surfaces. Analyses indicate that the wall meets the required minimum factors of safety, 1.5 for the static condition and 1.0 for the pseudo-static condition.

Table No. 7 – Wall Data Table

Wall Height 'H'	Base Width	Top of Leveling Pad Elevation (ft)	Slope in front of footing	Begin Station	End Station	Distance (ft)
26'-8"	20'-6"	586.711	Level	578+61.162	578+86.162	25.00
24'-2"	18'-6"	586.711	Level	578+86.162	579+81.162	95.00
21'-8"	17'-6"	586.711	Level	579+81.162	580+86.162	105.00
16'-8"	13'-6"	589.271	Level	580+86.162	582+51.162	165.00
14'-2"	11'-6"	591.771	Level	582+51.162	583+76.162	125.00
11'-8"	9'-6"	591.771	Level	583+76.162	584+46.162	70.00
9'-2"	9'-6"	594.271	Level	584+46.162	585+76.162	130.00

6.0 Construction Considerations

1. The proposed wall should be founded on properly compacted competent soil. Loose or soft material is not expected at this project site. If clay or loose sand is encountered within the areas to receive the walls, soil should be over-excavated for 5 feet and replaced with compacted fill. The compacted fill beneath the wall should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have less than 50% of material passing No. 200 sieve size. The compacted fill beneath the wall should be placed in horizontal loose layers of approximately 8-inch thick, and compacted to at least 95% relative compaction. The limits of compacted fill beneath the wall are as follows:
   - (i) Depth below the bottom of footing elevation is two feet (or five feet, in the case of over-excavation).
   - (ii) Horizontal extension is at least two feet away from the outer edge of the footprint of the wall.
   - (iii) Slope of excavation for the compacted fill should not be steeper than 1:1 slope.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications.

3. On-site material may be used as replacement material. However, oversized material (greater than 8-inch in the widest dimension) should be excluded from the replacement fill material.

If you have any questions or comments, please call Deepa Wathugala at (213) 620-2134, or Ted Liu at or (213) 620-2136.

Prepared by: Date: 2/27/12

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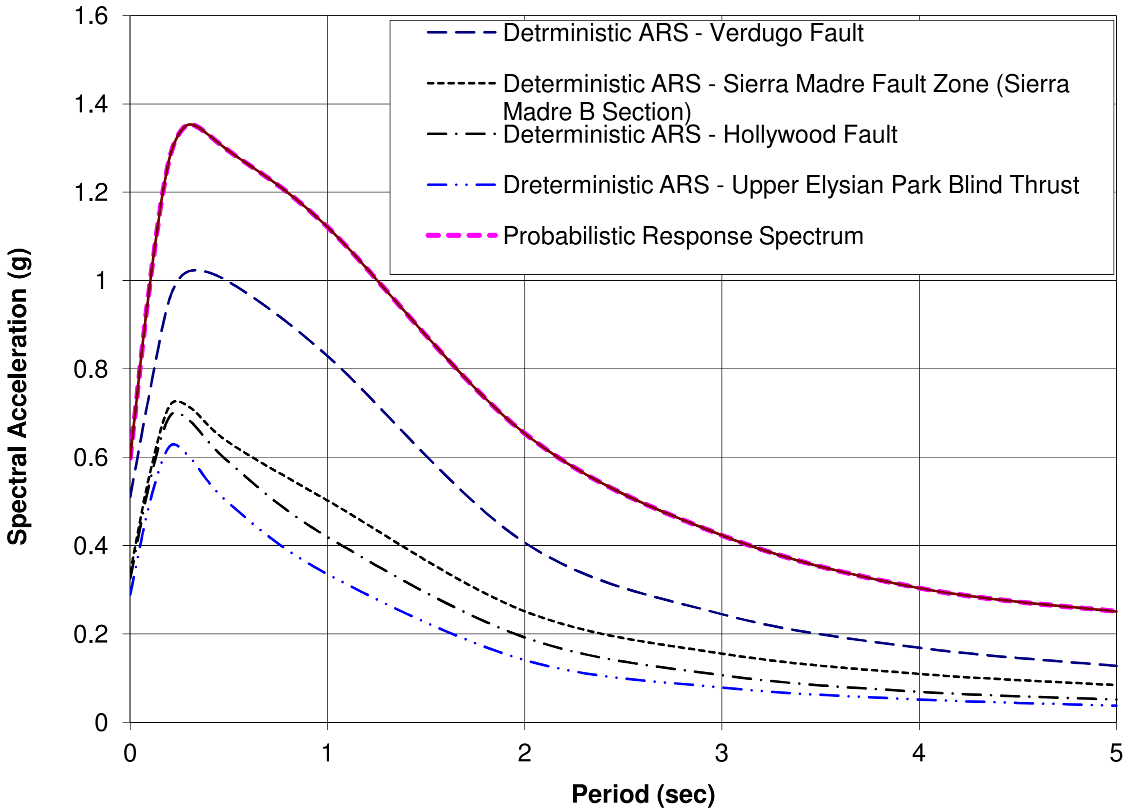
Structure Construction R.E. Pending File (RE_Pending_File@dot.gov.ca)
DES Office Engineer, Office of PS&E
District Materials Engineer

APPENDIX A

Figure A1 - RECOMMENDED DESIGN ACCELERATION RESPONSE SPECTRUM (ARS)
for Burbank Blvd OC
Damping Ratio = 5%; V_{s30} = 295 m/sec



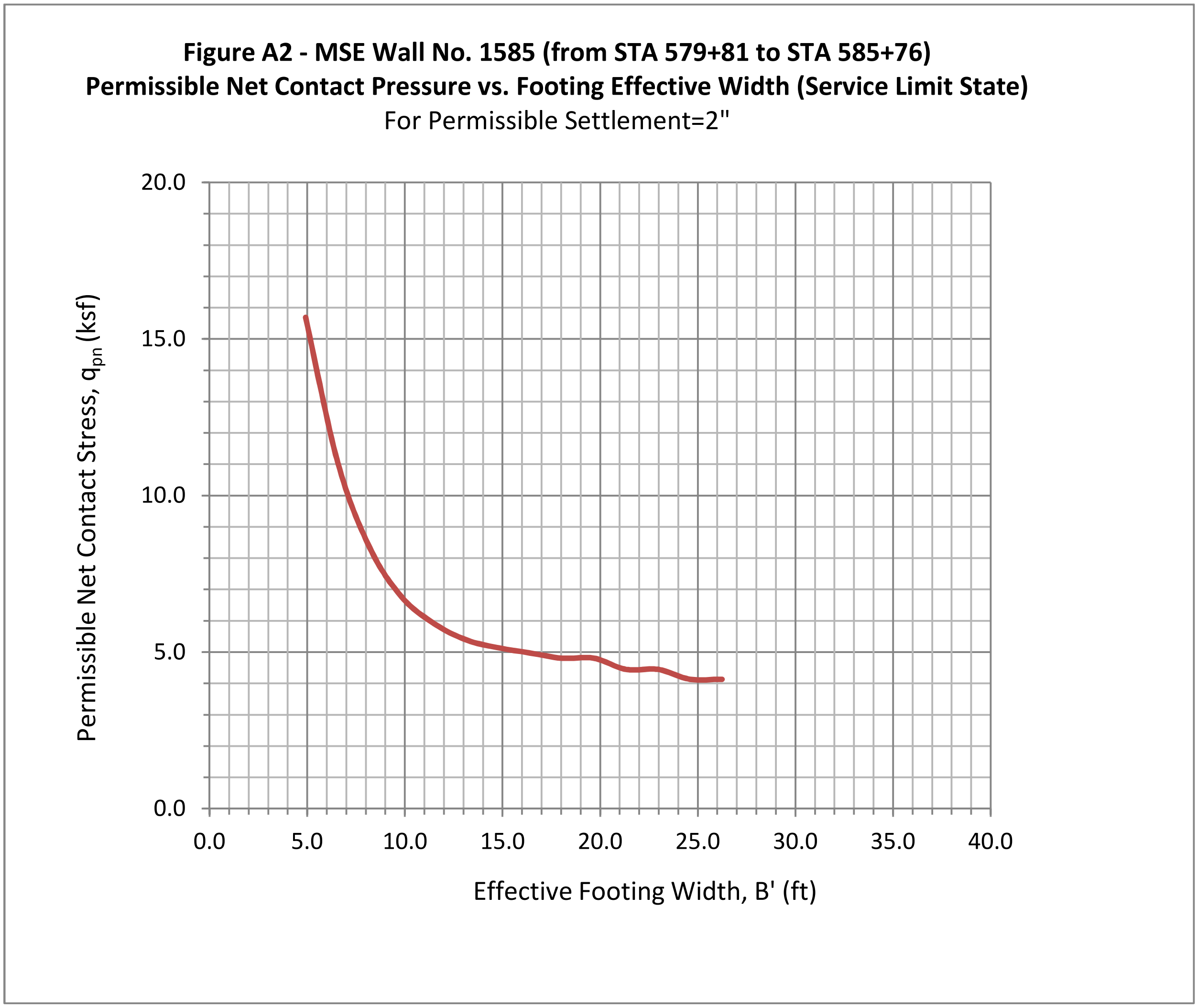


Figure A2 - MSE Wall No. 1585 (from STA 579+81 to STA 585+76)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"
20.0
15.0
10.0
5.0
0.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Permissible Net Contact Stress, q_pn (ksf)
Effective Footing Width, B' (ft)

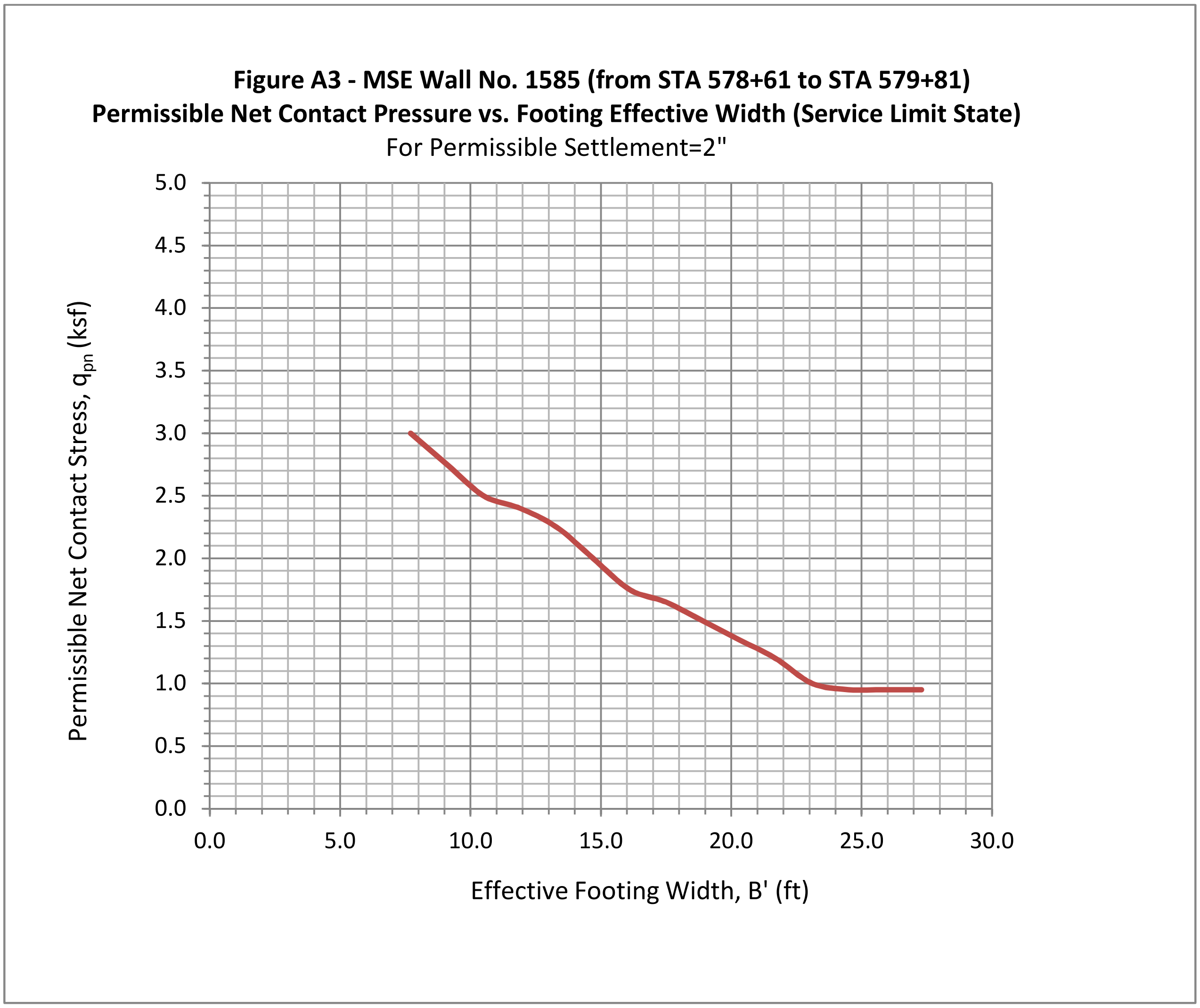


Figure A3 - MSE Wall No. 1585 (from STA 578+61 to STA 579+81)
Permissible Net Contact Pressure vs. Footing Effective Width (Service Limit State)
For Permissible Settlement=2"
Permissible Net Contact Stress, q_{pn} (ksf)
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Effective Footing Width, B' (ft)

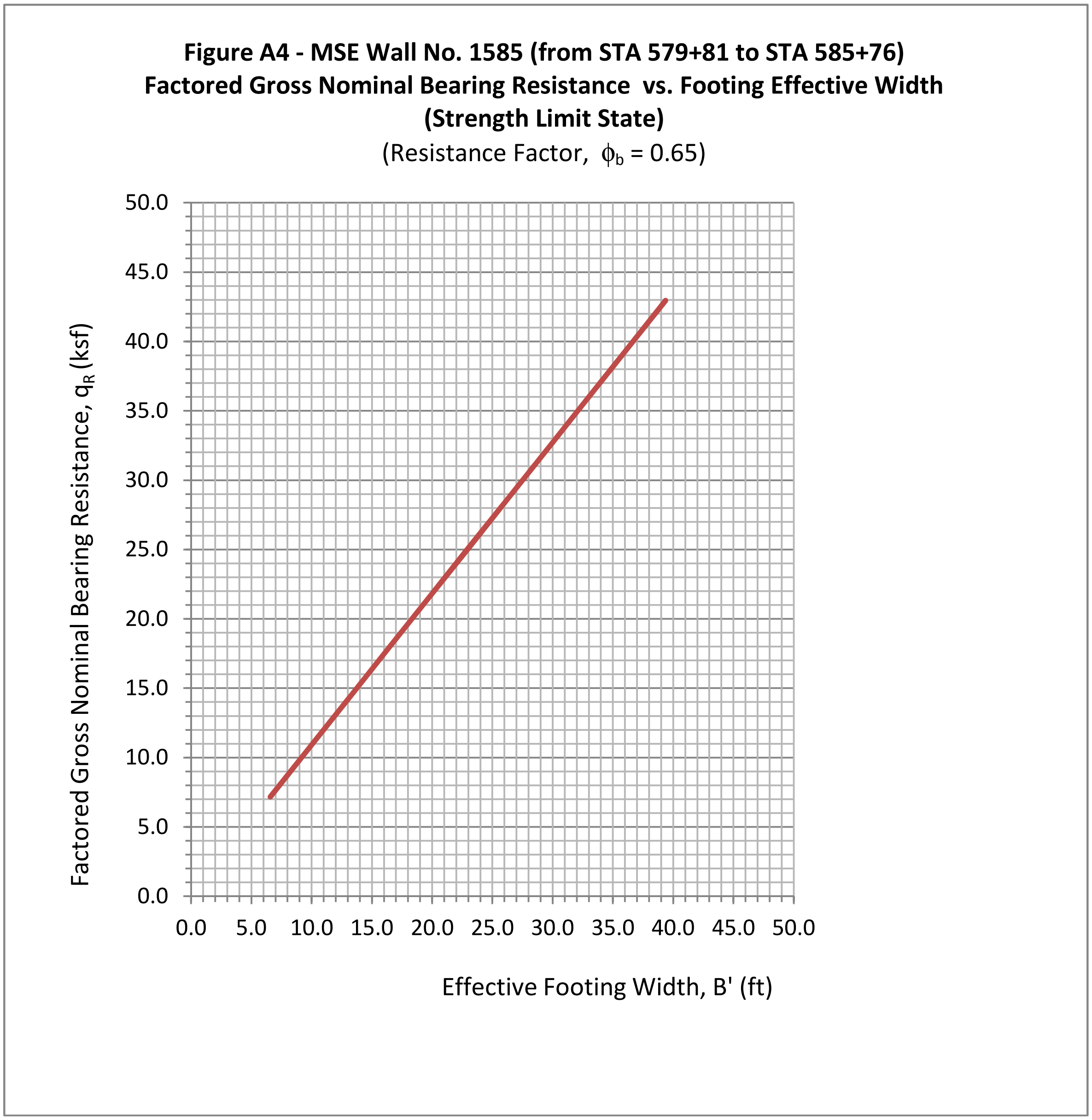
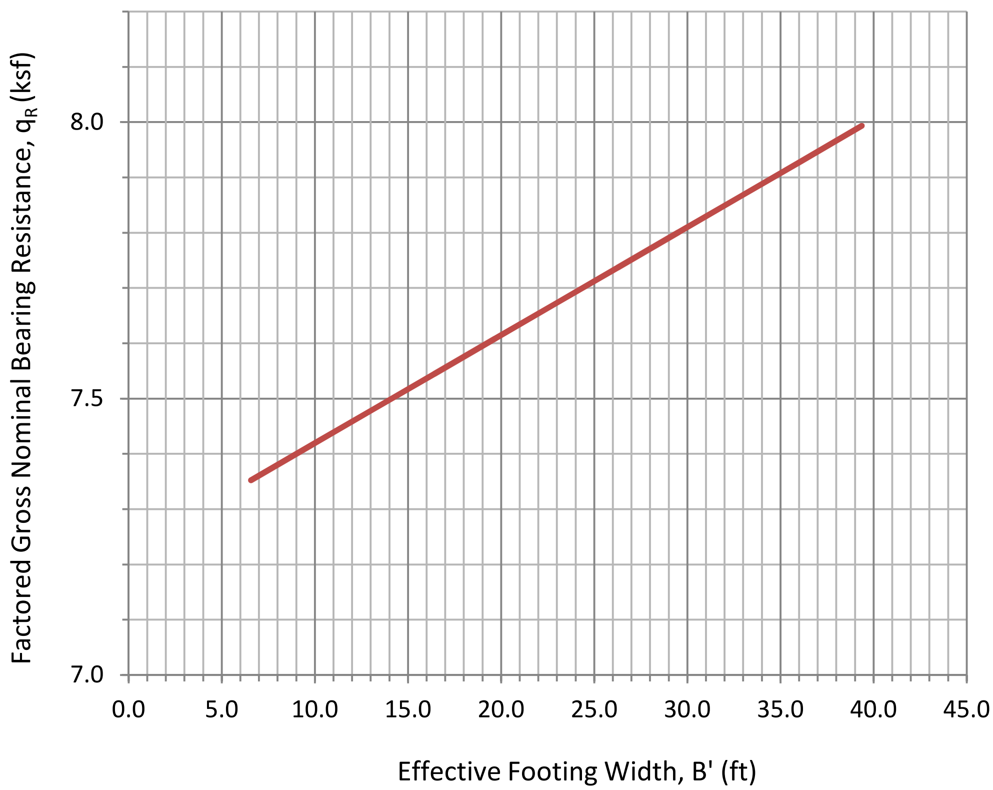


Figure A4 - MSE Wall No. 1585 (from STA 579+81 to STA 585+76)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)
50.0
45.0
40.0
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
Factored Gross Nominal Bearing Resistance, q_R (ksf)
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
Effective Footing Width, B' (ft)

Figure A5 - MSE Wall No. 1585 (From STA 578+61 to STA 579+81)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Strength Limit State)
(Resistance Factor, ϕ_b = 0.65)



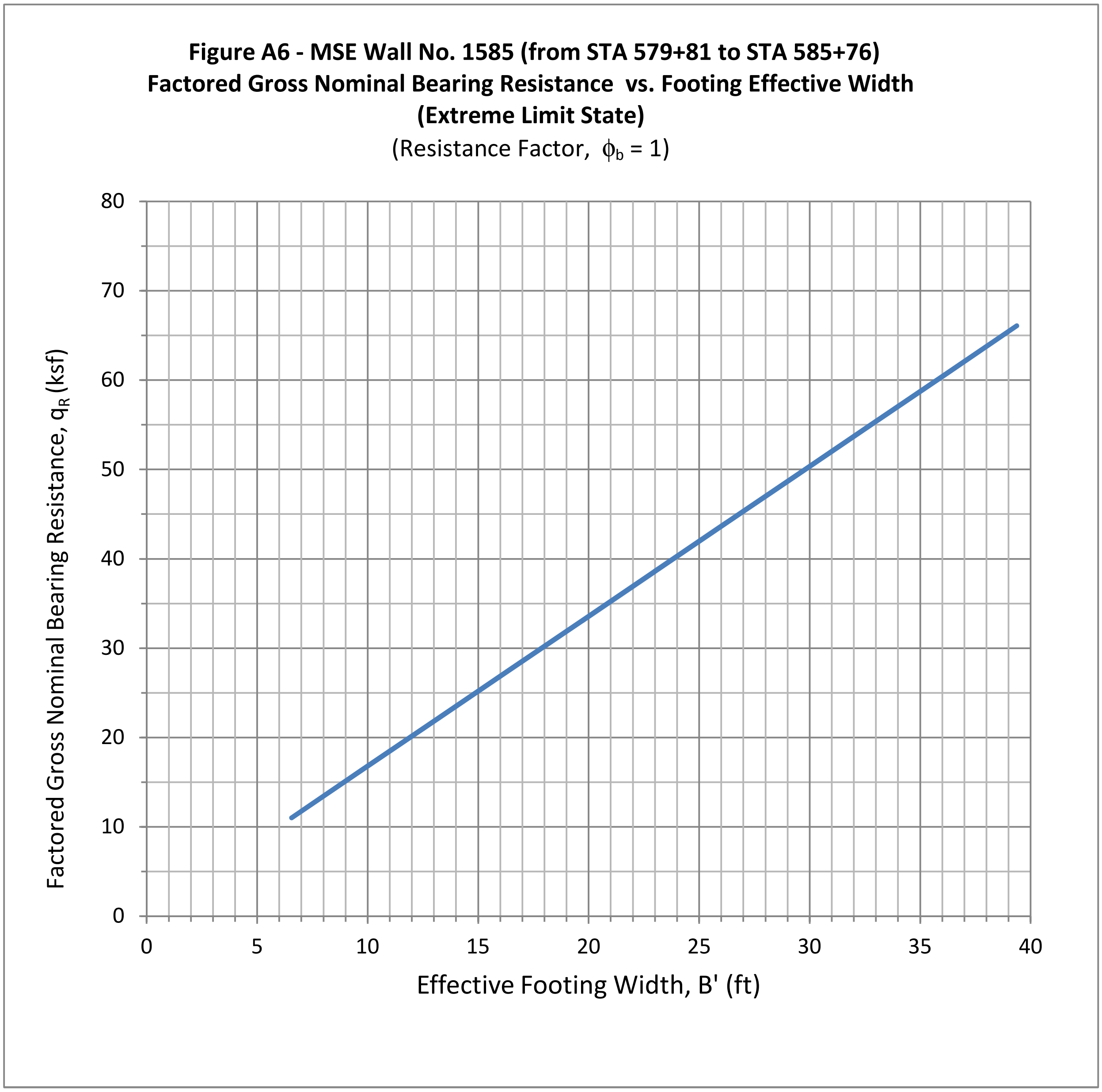


Figure A6 - MSE Wall No. 1585 (from STA 579+81 to STA 585+76)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width
(Extreme Limit State)
(Resistance Factor, ϕ_b = 1)
80
70
60
50
40
30
20
10
0
0
5
10
15
20
25
30
35
40
Factored Gross Nominal Bearing Resistance, q_R (ksf)
Effective Footing Width, B' (ft)

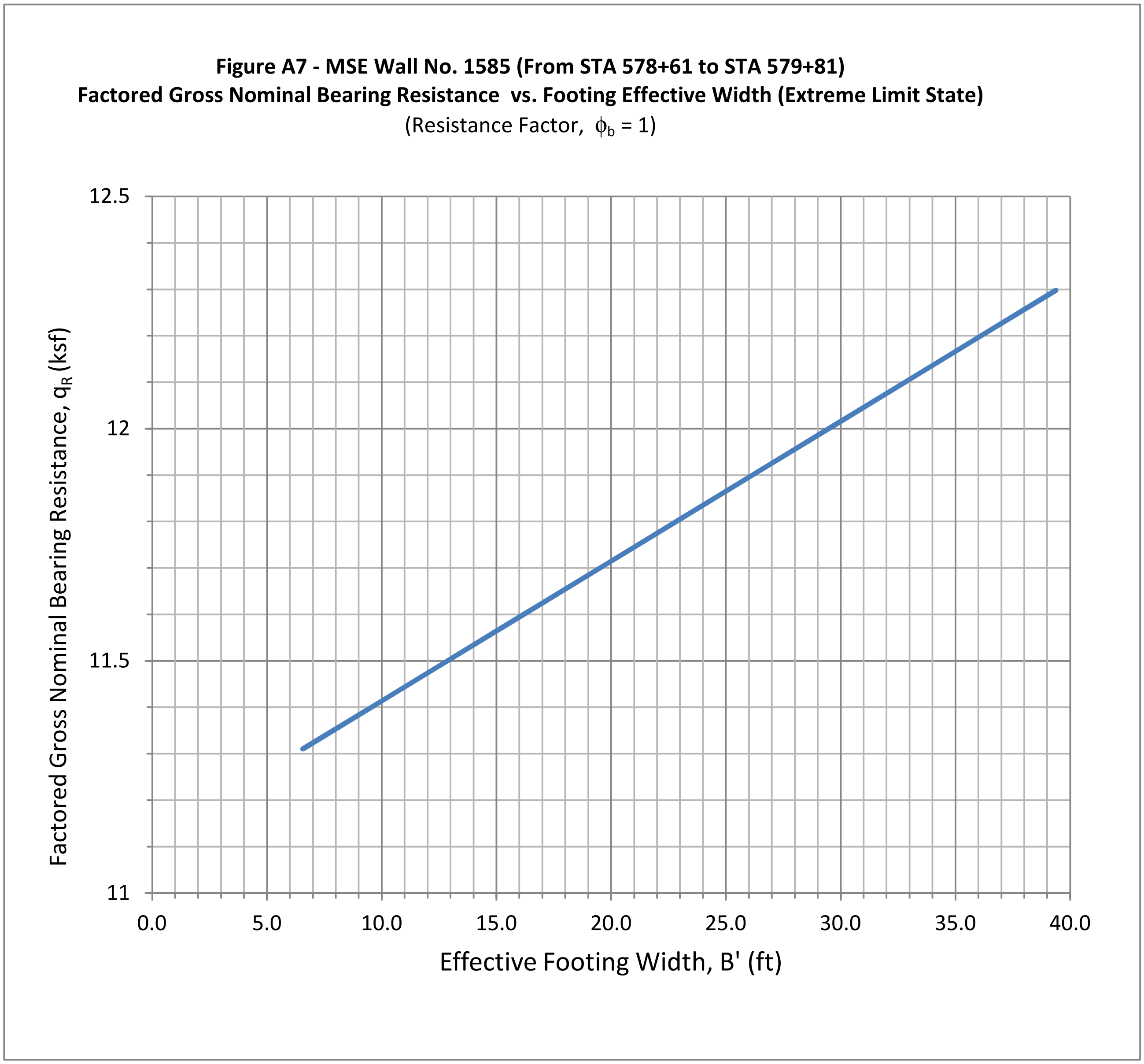


Figure A7 - MSE Wall No. 1585 (From STA 578+61 to STA 579+81)
Factored Gross Nominal Bearing Resistance vs. Footing Effective Width (Extreme Limit State)
(Resistance Factor, ϕb = 1)
Factored Gross Nominal Bearing Resistance, qR (ksf)
12.5
12
11.5
11
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Effective Footing Width, B' (ft)

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Bridge Design Branch 18
Office of Bridge Design South-1

Date: January 31, 2012

File: 07-LA-5-PM 31.23
07-1218W1
Buena Vista-Winona UC (Wdn)
Bridge No. 53-1110

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Buena Vista-Winona Undercrossing (Left Side Widen), Bridge No. 53-1110

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated September 27, 2011 and Project General Plan and Foundation Plans (plotted September 27, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed left side widening of the subject six span (Left) and five span (Right) bridge as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed bridge widening. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed bridge site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed bridge from the recent 2004/2006 Caltrans drilling program and As-Built Log of Test Borings (LOTB); and

f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed bridge widening.

This Foundation Report supersedes the previous Foundation Recommendations for Buena Vista-Winona UC widening (based on updated metric plans) dated August 31, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the City of Burbank. The Empire interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metrolink railroad tracks, and add high occupancy vehicle (HOV) lanes on Route 5 (one lane in each direction).

The existing structure (built in 1960) is a 6 span reinforced concrete box girder and precast prestressed inverted T girder bridge with a bin type diaphragm abutment (Abutments 1 and 1a) and a high cantilever abutment (Abutment 6), according to Mr. Jorge Estrada (October 6, 2005, Caltrans Structure Type Selection Memorandum). Bents are multicolumn. The total length of the existing original bridge is 428.5 ft. In 1987 the bridge underwent a minor right side widening. The existing bridge is founded predominantly on plumb, 16 in diameter, 45 ton design load, cast-in-drilled-hole (CIDH) piles and minor Class 45 driven concrete piles. The existing bridge also underwent some retrofitting in 1996.

The proposed left side widening (approximately 465.8 ft length measured along Proposed Centerline realigned Rte. 5) will be composed of a cast-in-place (CIP) prestressed box girder bridge supported on 70 ton design load, open ended vertical and/or battered driven pipe piles (PP16X0.5, Class 200, alternative W). Abutments will roughly match the existing types. Bents will be pinned at the bottom. The existing multicolumn bents will have steel jackets added where needed for additional seismic retrofitting.

All English unit elevations shown on the As-Built Plans/LOTB and within the As-Built Foundations section (below) are based on NGVD29 datum. Based on District 07 Survey Data, the necessary shift amount (add) for the 1961 and 1987 As-Built plan elevations to correct to the current NAVD88 plan elevations is +2.58 ft for this bridge structure.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from June 21, 2004 through January 30, 2006. The field investigation included drilling three 8-inch outer diameter hollow-stem auger and four 4.5-inch mud rotary borings. Standard Penetration Tests (SPT's) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at the three - 8 in. diameter hollow stem auger (HAS) boring locations. Caltrans drill rigs were utilized for the four – 4.5 in.

diameter mud/polymer rotary (MPR) boring locations. Caltrans geologists/engineers and a URS engineer performed the logging of the soil borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

Table 1 – Summary of Borings

Boring No.	Prop. C/L Rte 5 Stationing	Offset from Prop. C/L Rte 5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-14 (MPR)	1647+53.1	49.6 Lt.	693.0	66.5	08-01-05
06-93 (MPR)	1648+53.6	104.3 Lt.	667.9	105.2	01-26/31-06
05-63 (HSA)	1650+19.8	67.5 Lt.	670.6	80.8	11-10/11-05
04-6 (MPR)	1651+32.1	76.2 Lt.	672.2	100.4	06-24-04
04-5 (MPR) piezo	1652+25.3	63.8 Lt.	673.4	100.0	06-21/23-04
05-38 (HSA)	1653+36.5	111.1 Rt.	692.3	51.5	08-17/18-05
05-13 (HSA)	1655+22.6	46.3 Lt.	695.5	76.5	08-01-05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Minor soil testing was completed at Caltrans D07 Laboratory. Soil samples were tested for corrosivity, mechanical analysis/hydrometer, moisture content, and minor Atterberg Limits and compaction testing (modified Proctor). Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 2, below.

Table 2 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Sieve Analysis	TM 202/ASTM D422 (#200 by ASTM D1140	17
Mechanical Analysis (Hydrometer)	CTM 203/ASTM D4318	14
Atterberg Limits (Plasticity Index)	CTM 204/ASTM D4318	2
Moisture Content	CTM 226/ASTM D2937 or D2216	1
Corrosion – Sulfate Content	CTM 417	10
Corrosion – Chloride Content	CTM 422	10
Corrosion – Resistivity	CTM 532	15
Corrosion – pH	CTM 643	15
Compaction (modified Proctor)	ASTM D1557	1

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Subsurface Conditions

The proposed left side widening of the existing Buena Vista – Winona UC (Br. No. 53-1110) is bounded by I-5 to the east and northeast, San Fernando Blvd. and the SCRRA Railroad to the west and southwest, and will span over Buena Vista Street and Winona Avenue. Existing I-5 embankment ranges from approximately 26.0 ft at Abutment 1 and 22.3 ft at Abutment 6. Existing embankment side slopes have a 1(V):2(H) gradient and end slopes at abutments show a 1(V):1.5(H) or variable slope. The top and toe of the existing Left Side Rte. 5 embankment ranges in approximate elevations from 693 to 668 ft at Abutment 1 and 695 to 673 ft at Abutment 6, respectively. Embankment slopes are partially shrub and leaf covered with some sporadic trees at or near the base of the slope. Significant erosion of embankment is not apparent and the roadbed PCC pavement has been repaired/replaced during summer/fall 2005 due to extremely heavy concentration of truck and vehicle traffic.

It is OGDS1's understanding that no noise constraints are applicable at the site. Currently the abandoned Caltrans owned Buena Vista Landscape Mtce. Yard underlies much of the existing bridge and a major portion of the footprint for the proposed left side bridge widening. No additional inhabited buildings or businesses are present within the immediate vicinity of the widening, but existing businesses to the southwest, west, and northwest range from estimated distances of 260 to 80 ft (business to northwest) away. District 07 Project Development/Design/Right of Way will actually determine if noise constraints are applicable and whether noise reduction or possible relocations are required.

Subsurface Conditions

Embankment fill is underlain by Holocene alluvium. The underlying Holocene alluvium (Qa unit of Dibblee, 1991a) may be underlain by undifferentiated alluvial fan gravel derived from the Verdugo Mountains (Qf unit of Dibblee, 1991a) or older Pleistocene alluvium. Most recent deeper borings have likely terminated within older Pleistocene alluvium or fan gravel.

Embankment fill consists predominantly of medium dense to dense (minor very dense or loose) silty sand with gravel. Undifferentiated Holocene or older fan gravel/Pleistocene alluvium can be separarated into approximately four units. The upper unit is composed of loose to medium dense, sand to silty sand with gravel and minor scattered cobbles (up to 6 in diameter) and minor sandy silt from elevations ranging from +667.5 and +673 ft down to elevations ranging from +658 and +664 ft. The underlying second alluvial unit, ranges between approximate elevations +658 and +664 ft down to approximate elevations +623 and +616 ft, consists of medium dense to dense (rare loose), silty sand and sand with gravel and gravel interbeds containing minor cobbles/rare boulders (up to 12 in diameter) and rare sandy silt and clayey sand. The underlying third alluvial unit, ranges between approximate elevations +623 and +616 ft down to approximate elevations +594.5 to +601 ft, consists of medium dense to very dense, silty sand and sand with gravel containing minor cobbles interbedded with rare sandy silt and minor stiff to firm, sandy lean clay to lean clay with sand interbeds. The underlying lower alluvial unit, ranges between approximate elevations +594.5 to +601 ft down to approximate elevation +562.7, consists of very dense, gravel with sand containing cobbles (ranging from 4 to 8 in length) and scattered boulders (estimated up to 18 in. length) and silty sand with gravel. The deepest recent boring for the proposed bridge left side widen, Boring 06-93 (drilled late January 2006, near proposed Abutment 1a and Bent 2, was drilled 105.2 ft below the surface to elevation +562.7 ft. The LOTB should be reviewed for more specific details.

For additional subsurface information, the September 1961 As-Built Log of Test Borings (LOTB) for the Buena Vista – Winona Undercrossing, Br. No. 53-1110, shows four 3 in. diameter rotary sample borings and four 2.25 in. diameter cone penetration tests were completed. The one As-Built LOTB sheet will be included within the new contract plans for the newly proposed bridge widening. As Built LOTB information was incorporated in the above discussion of sedimentary units.

Groundwater

Groundwater was not encountered during the recent field exploration for the subject bridge widening. Perforated pipe was installed within boring 04-5 and successive measurements taken from July 2004 through August 2005 revealed the boring to be dry to the bottom of the hole at 98.5 ft depth below surface or elevation +574.9 ft. All auger borings completed for the approximate 2 mile length of the Empire Interchange project also showed no groundwater was encountered.

Borehole geophysical measurements were completed within Boring 05-47 for the proposed Empire Avenue UC (New), Br. No. 53-2920 (approximately 4000 ft southwest of the subject bridge widening) including natural gamma, formation conductivity and resistivity, and primary compression – shear wave suspension log records. According to Mr. Dave Hughes (March 17, 2006) engineering geologic "interpretations for the increased conductivity measurements at approximately 131 to 138 ft depth may be the result of brackish perched vadose groundwater but do not appear to be a saturated condition or groundwater zones are too thin to appear on the 1.6 ft sampling interval of the P-S suspension log. Primary wave velocity (Vp)>4920 ft/s (1500 m/s) in an otherwise poorly-consolidated sedimentary material with no apparent material changes suggest saturated material. The Vp data is questionable at the base of the measured section due to apparent poor grout/PVC bonding and is therefore inconclusive. However, based on interpretating the (poor) signal where it appears through the PVC overprinting, it does not appear saturated conditions were encountered in the measured section (measured depth 179.3 ft, elevation +444.7 ft). A general increase in velocity at the base of the measured section may be the result of more competent material (noted in the lithologic log as silty sand and sand, trace fine gravel). The increase in Vp may also be the result of approaching a saturated zone."

Groundwater was also not encountered during the 1957 field investigation (As-Built LOTB plan dated September 1961) down to adjusted NAVD88 elevation +594.6 ft., the maximum penetration depth obtained. Also no ground water was encountered on tape measured down to caving depth of 68.2 ft. at adjusted elevation +602.3 ft within cone penetrometer boring B-6.

Historic Records

The closest historical water wells on record from the Department of Water Resources (DWR, 01N14W03F03S and 01N14W03F06S) are located at approximately 700 ft to the north of the proposed bridge widening. The DWR wells located approximately near the Buena Vista Street/Winona Avenue intersection) show groundwater measurements below the surface vary from 211.8 to 167.5 ft ranging between approximate elevations +471 to +515.5 ft adjusted NAVD88 elevation. No measurement dates were provided but the wells had between 35 to 14 measurements taken.

The above measurements indicate that groundwater level fluctuates between different locations, years, and seasons. All groundwater measurements taken reasonably close to the project area show groundwater levels well below any probable foundation type contemplated for the Empire Interchange Project including Buena Vista – Winona UC (Widen), Br. No. 53-1110.

SCOUR

There is no potential scour at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS and Caltrans D07 Soils Laboratories. Corrosion test results, presented in Table 3 show subsurface soils are non-corrosive to metal and reinforced concrete.

Table 3 - Corrosion Test Summary for Buena Vista – Winona UC (Widen), Br. No. 53-1110

Boring No	Depth Interval (ft)	pH*	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
04-5	5.0 to 26.5	7.58	7500	NA	NA
04-5	26.5 to 41.5	7.58	4000	NA	NA
04-5	41.5 to 61.5	7.58	8900	NA	NA
04-5	61.5 to 71.5, 75.0 to 81.5	7.39	2400	NA	NA
04-5	85.0 to 92.5	7.63	9100	NA	NA
05-13	10.0 to 15.0	8.3	7400	12	75

05-13	35.0 to 40.0	8.3	9500	7	120
05-14	5.0 to 25.0	8.3	4000	0	60
05-14	25.0 to 51.5	8.5	5000	42	60
05-14	51.5 to 66.5	8.2	7200	24	45
05-63	5.0 to 31.5	8.3	17,000	ND	45
05-63	31.5 to 61.5	8.6	17,000	3	45
05-63	61.5 to 80.8	8.3	4200	27	45
06-93	62.0 to 70.5, 75.5 to 81.8	8.4	5300	216	120
06-93	85.0 to 95.2	8.6	#4900	12	105
Caltrans Corrosion Guidelines		**≤5.5**	<1000	**≥2000**	**≥500**

ND = not detectable
NA = not applicable
# = Value for resistivity derived from the reciprocal of conductivity.
*It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be noncorrosive. For structural elements, the California Department of Transportation considers a site to be corrosive if one or more of the following conditions exist for representative soil and/or water samples taken at the site: Chloride concentration ≥500 ppm, sulfate concentration ≥2000 ppm, or the pH is ≤5.5. Corrosion mitigation is required if one or more of the 3 conditions noted above exists where structural elements are involved (Caltrans Corrosion Guidelines, September 2003). Since resistivity serves only as an indicator parameter for the possible presence of soluble salts, it isn't included to define a corrosive area.

SEISMIC EVALUATION

Faulting and Seismicity

The following faulting and seismicity section and ARS curve (in Appendix A) was provided by Dr. Mohammed Islam of OGDS1 on March 23, 2006. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map or CSHM (CALTRANS, 1996) the Verdugo Fault (VDO), a reverse/oblique type fault is the nearest active seismic source from the site. Based on the CSHM, this fault is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w6.75. Based on the California Geological Survey (CGS, 2006) 2002 fault database, VDO is a reverse fault and capable of generating a maximum earthquake of Mw6.9. Based on Weber et al (1980), this fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

For seismic evaluation, the soil profile is assigned soil type D based on recommendations in Caltrans Seismic Design Criteria (SDC, v 1.3). The recommended ARS curve was developed based on Figure B.7 of the Seismic Design Criteria by proportionably adjusting the values by a factor of 0.8/0.6 =1.33.

SURFACE GROUND RUPTURE

The site is not located within an Earthquake Fault Zone (EFZ) as defined by the California Department of Conservation (Special Publication 42, 1997). As stated above the nearest known fault is located at a distance of about 0.4 miles from the site. Based on this information, the potential for ground rupture hazard at the site due to primary fault movement is considered low.

LIQUEFACTION

This site is not located in an area shown as potentially liquefiable on the Special Studies Zones Map of the Burbank Quadrangle (Davis, 1999). Since groundwater was not encountered (dry to at least 98.5 ft. depth) and soils were generally dense, the potential for liquefaction at the site is considered low. The potential for other seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

AS BUILT FOUNDATIONS

The original 1960 bridge and the 1987 right side widen is supported on a combination of plumb 16 in diameter, 45 ton design load, cast-in-drilled-hole (CIDH) piles and 16 in, 45 ton design load driven concrete piles (plumb or placed with 3V:1H batter) placed within alluvial material. OGDS1's review of the 1961 and 1987 As-Built Plans and LOTB allowed calculation of geotechnical support for the existing piles. Pile cap lateral dimensions, original design loading, elevations of the bottom of pile caps, and average pile tip elevations are provided on the 1961 and 1987 As-Built Plans. Existing grade at each support is estimated from the 1961 As-Built Foundation Plan, General Plan, and LOTB and the current Layout Plans with topographic contors for the Empire Interchange Project. Based on the information provided by D07 Surveys on October 24, 2011, the necessary elevation shift (add) from 1961 and 1987 As-Built plans (based on NGVD29 elevations) to the current plans is +2.58 ft.

Axial Pile Geotechnical Capacity

CIDH pile geotechnical capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-IF-99-025) published August 1999. Driven concrete pile geotechnical capacities were calculated using the Federal Highway Administration's Design and Construction of Driven Pile Foundations (Pub. No. FHWA-HI-97-013) revised November 1998 and the Driven pile program. Pile Data Tables for each pile type are shown below.

An elevation shift (add) of +2.58 ft should be applied to the As-Built plan elevations (NGVD29 datum) to convert to the current elevations (NAVD88 datum) in the following Table No.'s 4 through 7.

Table 4- 1961 As-Built Pile Data for Buena Vista–Winona UC, Br. No. 53-1110

Support Location/ Type & Diameter	Design Loading	Ultimate Soil Resistance*		Elevations Based On NGVD 29 Datum (1961 Contract Plans)			
	Compression (tons)	Compression (tons)	Tension (tons)	Approx. Exist. Grade Elev. (ft)	Bottom Pile Cap (ft)	Begin Pile Bearing Elevation (ft)	Approx. Pile Tip Elevation (ft)
Abut 1/Lt. side CIDH 16 in	45	136	13	+692	+683	+666	+633
Abut 1/ Middle CIDH 16 in	45	98	13	+691	+681.4	+666	+640
Abut 1/Rt. side CIDH 16 in	45	84	13	+690	+679.5	+666	+643
Abut 1a/Lt. side CIDH 16 in	45	106	40	+666	+643.0	+641.0	+625
Abut 1a/Lt. Mid side CIDH 16 in	45	103	36	+666	+647.5	+645.5	+630
Abut1a/Rt. Mid. side CIDH 16 in	45	119	36	+667.3	+650.5	+648.5	+630
Abut1a /Rt. side CIDH 16 in	45	101	32	+667.3	+654.0	+652.0	+636
Bent 2/Lt. side CIDH 16 in	45	102	36	+668.5	+650.5	+648.5	+633
Bent 2/Lt. Mid side CIDH 16 in	45	94	34	+668.9	+653.5	+651.5	+635.6
Bent 2/Rt. Mid side CIDH 16 in	45	111	31	+669.0	+656.5	+654.5	+635.6
Bent 2/Rt. side CIDH 16 in	45	87	28	+670.0	+659.5	+657.5	+642.0
Bent 3/Lt. side CIDH 16 in	45	105	22	+669.5	+664.0	+662.0	+640.5
Bent 3a/Rt side CIDH 16 in	45	108	22	+670.6	+665.0	+663.0	+641.0
Bent 4/Lt side CIDH 16 in	45	113	27	+670.6	+665.0	+663.0	+640.0
Bent 5/ CIDH 16 in	45	110	22	+671.5	+666.0	+664.0	+642.0
Abut 6/Lt. side CIDH 16 in	45	89	21	+672.2	+667.0	+665	+645.0
Abut 6/Rt. Mid CIDH 16 in	45	89	21	+672	+667.0	+665	+645.0
Abut 6/Rt. side CIDH 16 in	45	90	21	+672	+667.25	+665.25	+645.0

Note: CIDH piles were reinforced 12 ft minimum below the bottom of pile cap elevation or the top of original ground surface when piles are drilled through embankment constructed by contractor. CIDH piles constructed in 1960 are generally unreinforced below the 12 ft depth noted above. Tension capacity is substantially reduced due to the unreinforced lower part of the pile to pile tip.

Table 5- 1961 As-Built Pile Data for Buena Vista–Winona UC, Br. No. 53-1110

Support Location/ Type & Size	Design Loading	Ultimate Soil Resistance*		Elevations Based On NGVD 29 Datum (1961 Contract Plans)			
	Compression (tons)	Compression (tons)	Tension (tons)	Approx. Exist. Grade Elevation (ft)	Bottom Pile Cap (ft)	Begin Pile Bearing Elevation (ft)	Approx. Pile Tip Elevation (ft)
Abut 1/Rt. Middle Driven Concrete 16 in	45	140	45	+691	+681.4	+666	+640
Abut 1/Rt. side Driven Concrete 16 in	45	124	34	+690	+679.5	+666	+643
Abut 1a/Lt. side Driven Concrete 16 in	45	141	75	+666	+643.0	+641.0	+625
Abut 1a/Lt. Middle side Driven Concrete 16 in	45	129	65	+666	+647.5	+645.5	+630
Abut 1a/Rt. Middle side Driven Concrete 16 in	45	140	73	+667.3	+650.5	+648.5	+630
Abut 1a/Rt. side Driven Concrete 16 in	45	107	50	+667.3	+654.0	+652.0	+636
Bent 2/Lt. side Driven Concrete 16 in	45	121	60	+668.5	+650.5	+648.5	+633
Bent 2/Lt. Middle side Driven Concrete 16 in	45	109	60	+668.9	+653.5	+651.5	+635.6
Bent 2/Rt. Middle side Driven Concrete 16 in	45	99	64	+669.0	+656.5	+654.5	+635.6
Bent 2/Rt. Side Driven Concrete 16 in	45	97	43	+670.0	+659.5	+657.5	+642.0
Bent 3/Left side Driven Concrete 16 in	45	118	58	+669.5	+664.0	+662.0	+640.5
Bent 3a/Right side Driven Concrete 16 in	45	111	53	+670.6	+665.0	+663.0	+641.0
Bent 4/Left side Driven Concrete 16 in	45	121	60	+670.6	+665.0	+663.0	+640.0
Bent 5/ Driven Concrete 16 in	45	114	55	+671.5	+666.0	+664.0	+642.0
Abut 6/Lt. side Driven Concrete 16 in	45	99	45	+672.2	+667.0	+665	+645.0
Abut 6/Rt. Middle side Driven Concrete 16 in	45	99	45	+672	+667.0	+665	+645.0
Abut 6/Rt. side Driven Concrete 16 in	45	98	44	+672	+667.25	+665.25	+645.0

Notes: *Ultimate Soil Resistance calculated at $\leq$0.5 in displacement at top of pile.

Axial resistance to compression noted in the tables above is based on combined skin friction and end bearing at the supports within foundation soils.

Table 6- 1987 As-Built Pile Data for Buena Vista–Winona UC (Right Widen), Br. No. 53-1110

	Design Loading	Ultimate Soil Resistance*		Elevations Based On NGVD 29 Datum (1961 Contract Plans)			
Support Location/ Type & Diameter	Compression (tons)	Compression (tons)	Tension (tons)	Approx. Exist. Grade Elevation (ft)	Bottom Pile Cap (ft)	Begin Pile Bearing Elevation (ft)	Approx. Pile Tip Elevation (ft)
Bent 5/Right side widen CIDH 16 in	45	93	45	+671.5	+665.5	+664.0	+645.0
Abut 6/Rt. side widen CIDH 16 in	45	90	47	+672	+667.25	+665.25	+645.0

Note: 1987 CIDH piles are fully reinforced for the length of the pile. Tension capacity is substantially higher for the 1987 CIDH piles versus the 1961 CIDH piles due to the additional reinforcement length.

Table 7- 1987 As-Built Pile Data for Buena Vista–Winona UC (Right Widen), Br. No. 53-1110

	Design Loading	Ultimate Soil Resistance*		Elevations Based On Probable NGVD 29 Datum (1961 Contract Plans)			
Support Location/ Type & Diameter	Compression (tons)	Compression (tons)	Tension (tons)	Approx. Exist. Grade Elevation (ft)	Bottom Pile Cap (ft)	Begin Pile Bearing Elevation (ft)	Approx. Pile Tip Elevation (ft)
Bent 5/Rt side widen Driven Concrete 16 in	45	87	35	+671.5	+665.5	+664.0	+645.0
Abut 6/Rt. side widen Driven Concrete 16 in	45	88	36	+672	+667.25	+665.25	+645.0

Note: Assumed 12 in square driven concrete pile such as Alternative X.

Lateral Geotechnical Capacity

Results of LPILE analysis for both the abutment and bent locations for the original 1960 structure are summarized in the following Table 8. OGDS1 assumed a free-head condition.

Table 8 – As-Built Pile Lateral Capacity for Buena Vista-Winona UC, Br. No. 53-1110

Support Location	Lateral Load per Pile (kips)	Pile Head Deflection (in)	Maximum Bending Moment (in-kips)
Abutment 1 Lt. side	78.5	0.25	1108
	314.1	1.00	4432
Bent 2 Lt. Middle	55.2	0.25	925.1
	220.9	1.00	3700

FOUNDATION RECOMMENDATIONS

The following recommendations are based on the Caltrans field investigation completed February 2006, Updated Layout Plans (received September 27, 2011), Updated Buena Vista – Winona Undercrossing (Widen), Br. No. 53-1110, General Plans and Foundation Plans (received September 27, 2011), and the Updated Information for Pile Data Table (dated August 24, 2009, with no significant recent design changes) from Mr. Jorge Estrada (Structures Design, Branch 18) which provided the basis for the foundation recommendations in this report.

The proposed Left Side Bridge Widening (Br. No. 53-1110) can best be supported on vertical and/or battered 70 ton design load open ended pipe piles (PP16X0.5, Class 200, alternative W). Plumb, 16 in. diameter, 70 ton design load CIDH were also reviewed for feasibility but pile lengths generally exceeded the recommended maximum 30:1 ratio (pile length: pile diameter) for constructibility.

Axial and Lateral Pile Geotechnical Capacity and Pile Data

Driven 16 in. diameter open ended steel pipe piles (PP16X0.5, Class 200, alternative W) are recommended to support the bridge widening. Open ended pipe pile geotechnical resistances were calculated using the Federal Highway Administration's Design and Construction of Driven Pile Foundations (Pub. No. FHWA-HI-97-013, revised November 1998) and the American Petroleum Institute's (1993) guidelines for comparison.

Lateral load resistances for the proposed piles were analyzed using the computer program LPILE. The evaluation was based on the free and fixed condition at the top of the piles, for pile head deflection of 0.25 inch. Lateral load demands per pile for Buena Vista–Winona UC (Widen) were provided by Bridge Design Branch 18 on January 31st, 2012. OGDS1 also calculated Lateral Design Pile Tips using the program LPILE.

A Pile Data Table (Table 9) for driven 16 in. diameter (0.5 in. thick) pipe piles is provided below.

Table 9- Pile Data for Buena Vista–Winona UC (Left Side Widen), Br. No. 53-1110

	Design Loading	Nominal Resistance			Elevations Based On NAVD88 datum				
Support Location/ Pile Type & Diam. (in.)	Compression (tons)	Compression (tons)	Tension (tons)	Lateral (kips)	Approx. Finish Grade Elev. (ft)	Bot. Pile Footing Elev. (ft)	Begin Pile Bearing Elev. (ft)	Design Pile Tip Elev. (ft)	Specified Pile Tip Elevation[3] (ft)
Abutment 1 Lt side Widen/ PP16X0.5	70	140	0	15	+694.2	+685.0	+668.0	+638.0 (1) +655.0 (2a) +654.0 (2b)	+638.0
Abutment 1a Lt side Widen/ PP16X0.5	70	140	0	14	+668.3	+655.0	+653.2	+623.0 (1) +629.0 (2a) +628.0 (2b)	+623.0

Bent 2 Lt side widen PP16X0.5	70	140	0	20	+669.0	+661.0	+659.1	+623.0 (1) +633.0 (2a) +633.0 (2b)	+623.0
Bent 3 Lt side widen PP16X0.5	70	140	0	20	+670.9	+665.0	+663.0	+623.0 (1) +636.0 (2a) +636.0 (2b)	+623.0
Bent 4 Lt side widen PP16X0.5	70	140	0	18	+672.1	+666.0	+664.0	+623.0 (1) +637.0 (2a) +637.0 (2b)	+623.0
Bent 5 Lt side widen / PP16X0.5	70	140	0	18	+673.7	+667.5	+665.5	+622.0 (1) +638.0 (2a) +638.0 (2b)	+622.0
Abutment 6 Lt side Widen/ PP16X0.5	70	140	0	13	+672.9	+668.5	+666.5	+622.0 (1) +639.0 (2a) +639.0 (2b)	+622.0

Notes: Design Tip is controlled by the following demands:
(1) Nominal Resistance in Compression ($\leq$ 0.5 in. vertical deflection at top of pile)
(2a) Nominal Lateral Resistance – Fixed Head Condition (specified 0.25 in. lateral deflection at top of pile)
(2b) Nominal Lateral Resistance – Free Head Condition (specified 0.25 in. lateral deflection at top of pile)
(3) Specified Pile Tip Elevation is controlled by the maximum pile length necessary to satisfy resistance demands from cases (1), (2a), and (2b).
(4) Nominal Resistance in Tension is assumed to be 0.

Driven open ended pipe pile/geotechnical resistance capacity (axial nominal resistance in compression, Rnc), noted in Table 10 above, is based on skin friction resistance along the length of the pile from begin pile bearing elevation down to pile tip elevation plus the end bearing derived from the actual pile end area assuming no soil plug.

APPROACH FILL

Estimated Settlement

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. At the Abutments 1 and 6 areas, additional approach fill is estimated to range from 26.0 to 22.3 ft., respectively. Calculated maximum settlements (Hough's Method) range from 2.7 to 1.7 in. at Abutments 1 and 6, respectively. OGDS1 recommends a fill settlement period of up to 30 days for the widening; however, the actual settlement period will be determined by the structure representative on the basis of settlement data in the field. Settlement should be fairly rapid at the site as material is mostly coarse granular.

The downdrag potential on proposed piles in foundation soils and new fill will be mitigated by building up new embankment material to grade, allowing new embankment and existing soils to settle for the recommended time period (up to 30 days settlement period or as determined by structure representative), then excavating down to footing grade followed by pile installation.

Approach Slabs

Structure approach slab types N(9D), R(9D), N(9S), and R(9S) will be incorporated within the existing bridge and the proposed bridge widening as shown on the General Plan No. 1 for the Buena Vista – Winona UC (Widen), Br. No. 53-1110 (received September 27, 2011). Structure approach slabs are required for geotechnical reasons.

SLOPE STABILITY ANALYSES

The global stability of the new fill and existing embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. Results of slope stability analysis are shown in Table 10 below. For the purpose of slope stability analysis, ground water was at least 98.5 ft below the ground surface (elevation +574.9 ft). A 2 ft level surcharge of 240 psf was assumed at the bridge abutments. The slope stability analysis at abutments 1 and 6 embankments yielded a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static (global) stability and seismic condition, respectively.

Table 10- Slope Stability at Abutments 1 and 6

Support Location	Factor of Safety				
	Rotational Failure		Translational Failure		Surficial Failure
	Static	Pseudostatic	Static	Pseudostatic	
Abutment 1	1.53	1.16	N/A	N/A	N/A
Abutment 6	1.79	1.35	N/A	N/A	N/A

Based on subsurface information from the recent field investigation and As-Built LOTB, the soil profile with corresponding strength parameters used in performing the stability analysis are given in Table Nos. 11 and 12 below for Abutment 1 and 6, respectively. The proposed fill material is assumed to have a friction angle of 32 degrees and unit weight of 120 pcf, based on material compacted to at least 90 percent relative compaction.

Table 11-Idealized Soil Parameters for Slope Stability Analysis and/or Temporary Excavations/Shoring at Abutment 1

Idealized Soil Type	Approximate Elevation Range ft	Thickness ft	Unit Weight pcf	Internal Friction Angle (degrees)	Cohesion psf
silty sand with gravel (fill)	+693 to +668	25	120	32	0
silty sand (alluvium)	++668 to +665	3	120	32	0
sandy silt (alluvium)	+665 to 660.0	5	110	31	0
silty sand with gravel (alluvium)	+660.0 to +655	5	125	33	0
sandy silt (alluvium)	+655 to +650	5	105	29	0
silty sand to sand with gravel (alluvium)	+650 to +640.0	10	125	34	0

Table 12 - Idealized Soil Parameters for Slope Stability Analysis and/or Temporary Excavations/Shoring at Abutment 6

Idealized Soil Type	Approximate Elevation Range ft	Thickness ft	Unit Weight pcf	Internal Friction Angle (degrees)	Cohesion psf
silty sand with gravel (fill)	+697 to +673	24	120	32	0
silty sand, sand, and gravel/cobbles (alluvium)	+673 to +657	16	125	33	0
silty sand (alluvium)	+657 to +652	5	115	31	0
sand with gravel, gravel with cobbles (alluvium)	+652 to +641	11	129	34	0
silty sand (alluvium)	+641 to +625	16	110	31	0

CONSTRUCTION CONSIDERATIONS

1. The bottom of all excavations should be cleaned of loose debris before placing concrete.
2. The axial geotechnical capacities of proposed pipe piles are based dominantly on skin friction and minor end bearing within alluvial soils from below Begin Pile Bearing Elevation through specified tip elevation. End bearing is derived from the actual pile end area assuming no soil plug.
3. Driven open-ended steel pipe piles may require center relief drilling if hard/dense layers are encountered. If center relief drilling is necessary, the pipe piles should be driven past center relief drilling depth, approximately 4 pile diameters in length, before reaching specified pile tip elevation.
4. Driven open ended pipe piles can generally be driven through sporadic hard/dense layers to reach specified pile tip elevation. The pile section for the PP16X0.5 pipe piles (Class 200, alternative W) is generally thick enough to penetrate through hard driving conditions in dense to very dense sand and some gravel layers. These piles have an advantage over CIDH piles where heavy caving conditions might be present. Sporadic hard driving may be anticipated above and down to specified pile tip elevation from approximate elevations +666 to +622 ft. Existing driven concrete piles were specified at a minimum of 15 ft depth below pile cap. At Abutment 1 Left for the original bridge, a CCO (contract change order) was issued and only vertical CIDH piles were installed. This was probably due to hard driving conditions.
5. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (July 2002). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive or soils having low expansion potential (EI less than 50) should be used in the Soil Expansion Exclusion Zone in bridge approach embankment and within 3 m of the roadbed subgrade elevation.
6. Free water shall not be allowed to stand in any excavations. If excavations become flooded, at least the bottom 0.15 m of soils shall be removed and replaced or recompacted per Caltrans specifications.
7. Based on soil types encountered during the recent investigation, OGDS1 recommends a slope ratio of 1V:1H or flatter for the temporary back cut slope and excavations for construction. If

there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations.

If significant future design changes are made from that shown on referenced plans/information within this report, OGDS1 should be notified. OGDS1 should review the changes to verify that the foundation recommendations provided within this report remain applicable.

If you have any questions or need further information, please contact Joe Pratt at (213) 620-2313, or Shiva Karimi at (213) 620-2146.

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Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Bridge Design Branch 18
Office of Bridge Design South-1

Date: January 31, 2012

File: 07-LA-5-PM 30.45/30.49
07-1218W1
Empire Ave UC (Replace)
Bridge No. 53-2920

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Empire Ave Undercrossing, Bridge No. 53-2920

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 10, 2011 and Wall General Plan and Foundation Plans (plotted October 10, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Empire Avenue Undercrossing Replacement as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed bridge replacement. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed bridge site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed bridge; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed bridge replacement.

This Foundation Report supersedes the previous Foundation Recommendations for Empire Avenue UC (Replace) (based on updated metric plans) dated July 7, 2008 (Revised April 9, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the City of Burbank. The Empire interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metro link railroad tracks, and add high occupancy vehicle (HOV) lanes on Route 5 (one lane in each direction).

The proposed Empire Ave UC Bridge is 193.375 feet long (along I-5 "A" Line) and 194 feet wide with $50^0\ 30^{'}\ 01^{''}$ skew, and will be a single span cast-in-place prestressed box girder bridge with open-end seated abutments supported on 24 inch diameter Cast-In-Drilled-Hole (CIDH) piles. Northbound bridge Begin Station is 1607+62.49, 11 ft Rt. "A" Line with elevation of +640.35 ft and End Station is 1609+54.05, 11 ft Rt. "A" Line with elevation of 643.03 ft. Southbound bridge Begin Station is 1607+89.23, 11 ft Lt. "A" Line with elevation of +638.54 ft and End Station is 1609+84.13, 11 ft Lt. "A" Line with elevation of 641.25 ft. Elevations provided on current plans and recommendations are based on NAVD88 datum.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from September 30, 2005 through March 2, 2006. The field investigation included drilling seven 8-inch outer diameter hollow-stem auger and three 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County/Caltrans Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

Table 1 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-47 P-S	1609+02.93	138.179 Lt.	624.0	181	09/27,30/05
05-41	1612+23.2	64.0 Lt.	641.5	36.5	8/23/05
05-46	1614+45.9	194.7 Lt.	628.6	120	08/25-26/05
05-46A	1614+49.44	196.302 Lt.	628.5	96.5	11/9-10/05
05-21A	1605+68.6	63.743 Lt.	636.4	61.5	08/10/05
05-34	1605+91.54	60.43 Rt.	635.4	36.5	08/16-17/05
05-7	1606+41.69	137.258 Rt.	621.0	101.2	07/20-21/05
05-33	1610+17.55	122.733 Rt.	641.1	36.5	08/16-17/05

06-98	1611+26.82	123.2 Lt.	624.6	100.3	03/01-02/06
06-99	1608+07.78	126.91 Rt.	621.0	100.7	02/07-08/06

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 2, below.

Table 2 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Sieve Analysis	TM 202/ASTM D422 (#200 by ASTM D1140	3
Mechanical Analysis (Hydrometer)	CTM 203/ASTM D4318	3
Atterberg Limits (Plasticity Index)	CTM 204/ASTM D4318	3
Moisture Content	CTM 226/ASTM D2937 or D2216	0
Corrosion – Sulfate Content	CTM 417	26
Corrosion – Chloride Content	CTM 422	26
Corrosion – Resistivity	CTM 532	26
Corrosion – pH	CTM 643	26
Compaction (modified Proctor)	ASTM D1557	0

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Route 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Subsurface Conditions

The upper 1 to 20 feet (elevation +605 to +620 ft) of the borings consist of fill which is generally composed of loose to very dense silty sand and sand with gravel and cobbles. The top of native material was logged at an elevation of about 623 feet in the borings. The native alluvium was generally composed of loose to very dense silt, silty sand, poorly graded gravel sand, and sand with gravel lenses and cobbles throughout. Schist rock fragments encountered at elevation +550 to +570 ft.

Groundwater

Groundwater was not encountered during the Caltrans 2005 field exploration. Perforated pipe was installed within boring 05-46A and successive measurements taken during November 2005 and January 2006 revealed the boring to be dry to the bottom to 96.5 ft below surface (elevation +532.1 feet). Ground water was not encountered in any borings completed for the entire 1.8 miles length of Empire Interchange project.

Bore-hole geophysical measurements were completed within nearby Boring 05-47P-S including natural gamma, formation conductivity and resistivity, and primary compression – shear wave suspension log records. According to Mr. Dave Hughes (March 17, 2006) "engineering geologic interpretations for the increased conductivity measurements at approximately 131.2 to 137.8 feet depth, may be the result of brackish perched vadose groundwater but do not appear to be a saturated condition or groundwater zones.

Ground water was also not encountered during the 1957 field investigation for the nearby existing Southbound San Fernando Blvd UC (Br. No. 53-1215, As Built LOTB plan dated June 1961) down to approximate elevation +560 feet the maximum penetration depth of 63.3 feet obtained. Also no ground water was encountered on tape measured down to caving depth of 50 ft at elevation +568.7 ft within cone penetrometer boring B-1.

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 3 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 3, below.

Table 3 - Corrosion Test Summary for Empire Avenue UC Bridge No. 53-2920

Boring No	Depth Interval (ft)	pH*	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
04-7	0 to 20	8.6	3800	63	75
04-7	20 to 40	8.6	5500	9	45
04-7	40 to 60	8.0	4400	0	45
04-7	60 to 80	7.4	7400	33	45
04-7	80 to 100	8.3	7100	24	60
05-41	0 to 20	10	2400	210	60
05-41	20 to 36.4	8.7	3400	57	45
05-46A	70 to 95	8.6	13000	66	45
05-47	5.9 to 21	8.1	1900	55	60
05-47	5.9 to 41	8.5	5500	105	60

05-47	41 to 62	8.1	5700	ND	45
05-47	62 to 50	8.6	8200	33	60
05-47	50 to 101	8.0	10000	30	75
05-47	101 to 123	7.1	10000	12	45
05-47	123 to 125	7.9	5200	ND	60
05-7	129 to 130	8.2	3100	ND	75
05-7	136.5 to 139	7.6	3400	48	75
05-47	142.7 to 143	6.9	NT	ND	75
06-98	6.9 to 31.8	8.2	3000	6	105
06-98	31.8 to 53	8.5	4500	45	120
06-98	53 to 77	7.9	5100	6	90
06-99	3.9 to 29.8	7.6	4200	3	135
06-99	29.8 to 61.3	8.3	4000	36	105
06-99	61.3 to 100.7	8.5	6500	18	105
Caltrans Corrosion Guidelines		**≤5.5**	<1000	**≥2000**	**≥500**

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

For purpose of seismic evaluation, the soil profile is assigned soil type D based on recommendations in Caltrans Seismic Design Criteria (SDC, v 1.3). The recommended ARS curve was developed based on Figure B.7 of the Seismic Design Criteria by proportionality adjusting the values by a factor of 0.8/0.6 =1.33.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered to 96.5 ft depth below surface (elevation +532.1 feet). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

The proposed bridge can be supported on CIDH piles. The following recommendations are based on 1) Caltrans 2005/2006 soil borings test results, 2) Structure plans (plotted October 10, 2011) including design heights, bottom of footing elevations, footing dimensions, and CIDH diameter, 3) Wall (Abutment) layout (plotted October 10, 2011). Lateral loads were provided by Mr. Jorge Estrada via email dated January 24,, 2012.

Axial capacities of CIDH piles were evaluated based on the FHWA method using SHAFT version 4 computer program. Pile details and elevations are shown in Table 5 below.

Table 5 - Pile Data Table for Empire Avenue UC Bridge No. 53-2920

Support Location	Pile Type / Diameter	Design Loading (kips)	Nominal Resistance			Bottom of Footing Elev. (ft)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)
			Compression (kip)	Tension (kip)	Lateral (kips)			
Abutment 1 Left (A)	CIDH/ 24 inch	140	280	0	66	+606.75	+579.4 (1) +574.0 (2a) +576.0 (2b)	+574.0
Abutment 1 Right (B)	CIDH/ 24 inch	140	280	0	66	+609.25	+582.0 (1) +577.0 (2a) +578.0 (2b)	+577.0
Abutment 2 Left (C)	CIDH/ 24 inch	140	280	0	86	+604.75	+577.1 (1) +571.0 (2a) +574.0 (2b)	+571.0
Abutment 2 Left (D)	CIDH/ 24 inch	140	280	0	86	+606.00	+579.4 (1) +574.0 (2a) +576.0 (2b)	+574.0
Abutment 2 Right (E)	CIDH/ 24 inch	140	280	0	80	+607.50	+581.4 (1) +575.0 (2a) +579.0 (2b)	+575.0
Abutment 2 Right (F)	CIDH/ 24 inch	140	280	0	80	+609.00	+582.0 (1) +577.0 (2a) +578.0 (2b)	+577.0
Abutment 2 Right (G)	CIDH/ 24 inch	140	280	0	80	+610.75	+583.0 (1) +578.0 (2a) +580.0 (2b)	+578.0

Notes: Design Tip is controlled by the following demands:
(1) Nominal Resistance in Compression ($\leq$ 0.5 in vertical deflection at top of pile)
(2a) Nominal Lateral Resistance-Free Head Condition (specified 0.25 inch lateral deflection at top of pile)
(2b) Nominal Lateral Resistance-Fixed Head Condition (specified 0.25 inch lateral deflection at top of pile)
(3) Based on the General plan, it appears that finished grade ranges from approximately +615 to +617 feet elevation.

Lateral load capacities for the proposed CIDH piles were evaluated using the computer program LPILE PLUS 4.0. The evaluation was done based on both fixed and pin condition at the top of the piles for 0.25 inch pile head deflection.

Settlement

The settlement due to approximately 20 feet high approach fill is considered "immediate" and is expected to occur during construction. The magnitude of settlement during the construction is estimated to be 2 inches. The actual time to start of construction should be subject to review and monitoring data and approval by the resident engineer.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated in the footing excavation.

2. Moderate to heavy caving may be anticipated within sandy and gravelly soils during excavation of CIDH pile borings. Casing and/or slurry maybe required in CIDH pile borings.

3. The bottom of all excavations should be cleaned of loose debris before placing concrete.

4. Drilling during construction may be variable and sporadically hard (within gravel zones with scattered hard cobbles) down to anticipated pile tip elevations. According to information shown on the LOTB, normal auger drilling techniques should work at the site, however, OGDS-1 recommends a test hole be drilled at the site by the contractor to first verify the above assumptions.

5. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications. If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non expansive or soils having low expansion potential (EI less than 50) should be used in the Soil Expansion Exclusion Zone in bridge embankment and within 3 ft of the roadbed subgrade elevation.

6. Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1:V:1.5H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations.

7. A sound studio is located approximately 160 feet northwest from the proposed subject bridge.

For further information, please contact Akbar Mehrazar at 949-440-3415 or Shiva Karimi at 213-620-2146.

Prepared by: Date: 1/31/2012

A Mehrazar

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Transportation Engineering
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Supervised by: Date: 1/31/2012

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PCE (District 07) – Jan Rutenbergs (Electronic File)
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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

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To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: February 17, 2012

File: 07-LA-5-PM 30.07/30.38
07-1218W1
Empire Interchange
Burbank Western Channel (Cover)
Br. No. 53-3078

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Burbank Western Channel (Cover)

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 1, 2011 and Burbank Western Channel Cover General Plan, Profile, Abutment 1 and 2 Layout and Details Plan sheets (plotted December 16, 2011 and updated February 15, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Burbank Western Channel (Cover) as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Burbank Western Channel cover will be constructed along northbound I-5, south of San Fernando Boulevard between post miles 30.07 and 30.38. Burbank Western Channel cover will be a precast prestressed concrete slab girder supported by 30 inch Cast-In-Drilled Hole (CIDH) concrete piles to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed channel cover. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed channel cover site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed channel cover; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed channel cover.

This Foundation Report supersedes the previous Foundation Recommendations for Burbank Western Channel Box Culvert (based on updated metric plans) dated January 31, 2007 (Revised May 06, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Exiting Burbank Western Channel is a reinforced concrete open channel with side walls from 12.5 to 15.0 ft high and bottom slab approximately 30 ft wide. It is planned to cover the existing open channel with a 40 feet wide precast prestressed concrete slab girder supported by two abutments, Abutment 1 on southwest side and Abutment 2 on northeast side of the channel walls. Abutment 1 and 2 foundations will be 40.5 feet apart and will consist of 30 inch Cast-In-Drilled Hole (CIDH) concrete piles. CIDH piles will be cased from cut off elevation down to the bottom of the existing concrete channel slab elevation (approximately 15 ft below finished grade) to prevent load transfer from CIDH piles to the existing channel structure.

Existing Sound Wall 862 near proposed Abutment 1 will be removed and a new masonry block sound wall on Concrete Barrier Type 736 (Mod) will be constructed on top of Abutment 2 to accommodate the planned freeway widening.

Existing open channel is 1777.95± feet long measured along channel centerline. Channel cover will be 1644.61± ft long measured along channel centerline and start from channel LOL Station 149+82.22± to Station 165+17.050± (100 ft Rt. of Station 1587+26.77 to 162.78 ft Rt. of Station 1604+24.11, Route 5 Centerline. Top of channel cover concrete slab elevation varies from 600.40 to 618.88 ft. Elevations provided on current plans and recommendations are based on NAVD88 datum.

The location and geometric layout data for the “Channel Cover" is shown on the Burbank Western Channel (Cover) General Plan, Profile, Abutment 1 and 2 Layout and Details Plan sheets (plotted December 16, 2011 and updated February 15, 2012).

FIELD INVESTIGATION AND TESTING PROGRAM

The site-specific field exploration was performed between June 15, 2004 and November 9, 2005. The field investigation included one 8-inch hollow stem auger borings, and seven 4.5-inch and one 3.7-inch mud rotary borings. Standard Penetration Test (SPT) and undisturbed sampling were performed at the borings. Blow counts and SPT N values were continuously recorded at an interval of 5 feet during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

A Tri County Drilling Inc. and Caltrans operated drill rigs were used at the boring locations. Caltrans engineer/geologist performed the logging of the borings.

Location and elevation of borings were provided by the office of D7 Survey. Boring information, including exploration number, stationing, offset, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

Table 1 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-61	1582+33.9	110.9 Rt.	593.2	88.0	11/09/2005
05 – 5	1586+32.7	134 Rt.	596.9	20.0	6/16/2004
04 – 4	1589+19.3	125.9 Rt.	600.4	51.5	6/17/2004
04 – 3	1592+48.2	125.5 Rt.	603.8	51.5	6/16/2004
04 – 1	1595+73.6	127.7 Rt.	606.8	61.2	6/15/2004
05 – 6	1599+01.0	131.6 Rt.	610.3	66.5	7/19/2005
04 – 2	1601+59.4	155.5 Rt.	613.1	61.0	6/14/2004
05-30	1602+42.3	85.2 Rt.	624.8	36.5	6/16/2004
05 – 7	1606+41.7	137.3 Rt.	621.0	101.2	7/21/2005

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, moisture content, and plasticity index. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 2, below.

Table 2 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	19
Plasticity Index	CTM 204	6
Corrosion	CTM 417, 422, 643,532	19

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The existing Burbank Western Reinforced Concrete open channel site is bounded westerly by the Scott RD. Northbound on-ramp and off-ramp, easterly by Leland Way and by the Burbank Blvd. on the south. The existing structure is an open channel with an approximate depth of 16 ft below the Leland Way Street roadway surface. Above the ground is flat and consists of an open field. The residence properties are located along Leland Way.

The site consists of 1 to 5 ft of fill consisting of loose to medium dense silty sand. Underlying alluvium consists of loose to very dense silty sand/sandy silt with gravel/cobbles/ Schist rock fragments, and interbedded layers of soft to very stiff clay and sandy clay.

Groundwater

Groundwater was not encountered during the 2004/2005 field exploration to the maximum depth drilled, elevation +505.2 ft in Boring 05-61. Auger borings completed for the entire 1.9 miles length of Empire Interchange project also showed no groundwater was encountered.

SCOUR

There is no scour potential at the site as the existing channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 3 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 3, below.

Table 3 – Corrosion Test Summary for Burbank Channel Cover

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
04 – 1	5 – 16.5	2800	7.74	NA	NA
04 – 1	16.5 – 35.0	4100	7.81	NA	NA
04 – 1	35.0 – 55.0	6700	7.82	NA	NA
04 – 1	55.0 – 16.2	5000	7.45	NA	NA
05 – 5	15.1 –29.8	2200	8.5	30	24
05 – 6	0.0 – 20.0	6500	8.8	60	21
05 – 6	20.0– 35.1	5100	8.8	45	0
05 – 6	35.1 – 50.0	5000	8.9	45	6
05 – 6	50.0 – 64.9	5400	8.7	45	0
05 – 7	0.0 – 20.0	3800	8.6	75	63
05 – 7	20.0 – 40.1	5500	8.6	45	9
05 – 7	40.1 – 60.0	4400	8.0	45	0
05 – 7	60.0– 80.0	7400	7.4	45	33
05 – 7	80.0 – 100.0	7100	8.3	60	24
05-30	2.0-3.6	3900	8.7	15	186
05-30	22.0-23.9	4100	8.9	60	141
05-61	0.0-29.8	3000	7.9	45	45
05-61	29.8-60.0	3800	8.3	45	60
05-61	60.0-100.0	6200	8.5	45	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATIONS

The following seismicity information was provided by Dr. Mohammed Islam on September 16, 2005 and March 23, 2006. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered to the maximum depth drilled, elevation+505.2 ft in Boring 05-61. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

No known fault crosses or extends toward the project site. As stated above, the nearest known fault is located at a distance of about 0.4 miles east from the project site. Based on this information, the potential for ground rupture hazard at the site due to primary fault movement is considered low.

FOUNDATION RECOMMENDATIONS

The proposed channel cover can be supported on CIDH piles. The following recommendations are based on 1) Caltrans 2004/2005 soil borings test results, 2) Structure plans (plotted December 16, 2011 and updated February 15, 2012) including the top and bottom of channel cover elevations, and Abutment 1 and 2 proposed foundations (30" Dia CIDH pile with pile cap).

Axial capacities of CIDH piles were evaluated based on the FHWA method using SHAFT version 5.0 Computer Program. Pile details and elevations at Abutment 1 and 2 (support locations) are shown respectively in Tables 4 and 5 below.

Table 4 - Pile Data Table for Burbank Western Channel Cover Abutment 1 (Left)

Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Channel Cover CL STA (ft)	Bottom of Pile Cap Elevation (ft)	Pile Cut-off Elevation (ft)	Design Pile Tip Elevation (ft)	Specified Pile Tip Elevation (ft)
		Compression (kip)	Tension (kip)					
CIDH/ 30 inch	200	400	0	149+82.22	595.13	595.38	555.38	555.00
CIDH/ 30 inch	200	400	0	150+78.37	596.35	596.60	556.60	556.50
CIDH/ 30 inch	200	400	0	151+74.49	597.44	597.69	557.69	557.50
CIDH/ 30 inch	200	400	0	152+70.62	598.41	598.66	558.66	558.50
CIDH/ 30 inch	200	400	0	153+66.74	599.26	599.51	559.51	559.50

CIDH/ 30 inch	200	400	0	154+62.87	599.98	600.23	560.23	560.00
CIDH/ 30 inch	200	400	0	155+58.99	600.72	600.97	560.97	560.50
CIDH/ 30 inch	200	400	0	156+55.12	601.76	602.01	562.01	562.00
CIDH/ 30 inch	200	400	0	156+41.46	603.14	603.39	563.39	563.00
CIDH/ 30 inch	200	400	0	157+37.59	604.82	605.07	565.07	565.00
CIDH/ 30 inch	200	400	0	158+33.71	606.16	606.41	566.41	566.00
CIDH/ 30 inch	200	400	0	159+29.88	607.40	607.65	567.65	567.50
CIDH/ 30 inch	200	400	0	160+25.97	608.00	608.25	568.25	568.00
CIDH/ 30 inch	200	400	0	161+22.09	608.50	608.75	568.75	568.50
CIDH/ 30 inch	200	400	0	162+18.22	611.25	611.50	571.50	571.00
CIDH/ 30 inch	200	400	0	163+14.34	613.06	613.31	573.31	573.00
CIDH/ 30 inch	200	400	0	164+10.47	613.35	613.60	573.60	573.50
CIDH/ 30 inch	200	400	0	165+17.05	613.50	613.75	573.75	573.50

Table 5 - Pile Data Table for Burbank Western Channel Cover Abutment 2 (Right)

Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Channel Cover CL STA (ft)	Bottom of Pile Cap Elevation (ft)	Pile Cut-Off Elevation (ft)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression (kip)	Tension (kip)					
CIDH/ 30 inch	200	400	0	149+82.22	594.41	594.66	554.66	554.50
CIDH/ 30 inch	200	400	0	150+78.37	595.63	595.88	555.88	555.50
CIDH/ 30 inch	200	400	0	151+74.49	596.72	596.97	556.97	556.50
CIDH/ 30 inch	200	400	0	152+70.62	597.69	597.94	557.94	557.50
CIDH/ 30 inch	200	400	0	153+66.74	598.54	598.79	558.79	558.50
CIDH/ 30 inch	200	400	0	154+62.87	599.26	599.51	559.51	559.50
CIDH/ 30 inch	200	400	0	155+58.99	600.00	600.25	560.25	560.00
CIDH/ 30 inch	200	400	0	156+55.12	601.04	601.29	561.29	561.00
CIDH/ 30 inch	200	400	0	156+41.46	602.42	602.67	562.67	562.50

CIDH/ 30 inch	200	400	0	157+37.59	604.12	604.37	564.37	564.00
CIDH/ 30 inch	200	400	0	158+33.71	606.14	606.39	566.39	566.00
CIDH/ 30 inch	200	400	0	159+29.88	608.33	608.58	568.58	568.50
CIDH/ 30 inch	200	400	0	160+25.97	609.42	609.67	569.67	569.50
CIDH/ 30 inch	200	400	0	161+22.09	610.67	610.92	570.92	570.50
CIDH/ 30 inch	200	400	0	162+18.22	611.50	611.75	571.75	571.50
CIDH/ 30 inch	200	400	0	163+14.34	611.83	612.08	572.08	572.00
CIDH/ 30 inch	200	400	0	164+10.47	612.50	612.75	572.75	572.50
CIDH/ 30 inch	200	400	0	165+17.05	612.42	612.67	572.67	572.50

Notes: Design Tip is controlled by the following demands:
(1) Nominal Resistance in Compression ($\leq$ 0.5 in vertical deflection at top of pile)
(2) Nominal Resistance in Tension is assumed to be 0.
(3) Pile Tip Elevations for Lateral Loads will be evaluated by Structure Design.

Axial nominal resistance in compression, noted in the Table No.'s 4 and 5 above, is based on skin friction only within the alluvial soils. End bearing was not considered due to potentially caving soils near pile tip elevation.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated in the footing excavation.

2. Moderate to heavy caving may be anticipated within sandy and gravelly soils during excavation of CIDH pile borings. Casing and/or slurry maybe required in CIDH pile borings.

3. The bottom of all pile borings excavations should be cleaned of loose debris before placing concrete. Construction of CIDH piles should be completed the same day that pile borings are drilled.

4. Drilling during construction may be variable and sporadically hard (within gravel zones with scattered hard cobbles) down to anticipated pile tip elevations.

5. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications. If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non expansive or soils having low expansion potential (EI

less than 50) should be used in the Soil Expansion Exclusion Zone in bridge embankment and within 3 ft of the roadbed subgrade elevation.

6. Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1V:1H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations.

If you have any questions, please call Tatjana Halda at (213) 620-2347 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 2-17-12

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District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: April 27, 2012

File: 07-LA-5-PM 30.07/30.38
07-1218W1
Empire Interchange
Burbank Western Channel (Cover)
Br. No. 53-3078

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1 (OGDS1)
Branch D

Subject: Addendum to Foundation Report for Burbank Western Channel (Cover), dated February 17, 2012

INTRODUCTION

The Office of Geotechnical Design South 1 (OGDS1) prepared this addendum to the referenced Foundation Report to present the geotechnical recommendations for the proposed revised foundation pile lengths and pile casing based on the updated project plans (General Plan, Foundation Plan, Abutment 1 and 2 Layout, Abutment Details, Channel Cover Piles, Details, and Profile plan sheets) dated April 13, 2012. The piles proposed for a portion of the channel cover could exert significant lateral loads on the adjacent channel wall in addition to vertical loads. In order to mitigate lateral loading on the walls, a pile isolation system is proposed for this segment of the channel cover. The isolation system consists of an oversized cased hole installed around the piles to a depth of the channel bottom. The piles for the remaining segment of the channel cover will be cast in a cardboard casing extending down to the depth of the channel bottom, in order to mitigate vertical loading (downdrag) on the adjacent channel wall. The limits of the channel cover segment with and without isolated piles are described below.

Based on the information provided by the Office of Structure Design (OSD) and updated structure plans, OGDS1 understands that from Station 149+82.22 to Station 158+33.71 along the channel, there is the grade difference between the left side (the freeway grade) and the right side of the channel cover. The net lateral load due to grade difference on the channel cover in this segment was not considered significant enough to cause a distress to the channel wall structurally. Therefore, in this segment, OSD does not propose to use an isolation system with oversized cased holes around the piles. The piles in this segment would be cast to a 30-inch diameter in cardboard casing, extending down to the channel bottom elevation. From Station 158+33.71 to Station 165+17.05, grade difference between the left and the right sides of the cover increases from about 3 to about 10 feet. Since the lateral load transmitted from the piles to the channel wall could be significant, the piles in this segment will be cast in oversized holes (36 inches in diameter) that extend down to depths ranging from 12.5 feet to 15 feet (corresponding to the elevation of channel bottom).

The analysis was performed assuming that the piles would be constructed using a wet method and consequently would not have end resistances. In the case of piles, cast in cardboard casing in contact with the subsurface soils, no vertical stresses were assumed to be transmitted to the surrounding soils over the cased length. For the piles, cast in oversized holes, it was assumed that no vertical or lateral stresses would be transmitted to the surrounding soils. The geotechnical profile for the analysis was developed using the subsurface information obtained from Borings 05-61, 05-30, 05-7, 05-6, 05-5, 04-4, 04-3, 04-2, and 04-1, drilled for the project. Axial capacities of CIDH piles were evaluated using SHAFT version 5.0 Computer Program.

Revised pile tip elevations at Abutment 1 and 2 (support locations) are shown respectively in Tables 4A and 5A below. These recommendations supersede/supplement the section on foundation recommendations, presented in the referenced Foundation Report, dated February 17, 2012. The design Loads were provided by the Office of Structure Design (OSD).

Table 4A - Pile Data Table for Burbank Western Channel Cover Abutment 1 (Left)

Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Channel Cover CL STA (ft)	Bottom of Pile Cap Elevation (ft)	Pile Cut-off Elevation (ft)	Design Pile Tip Elevation (ft)	Specified Pile Tip Elevation (ft)
		Compression (kip)	Tension (kip)					
CIDH/ 30 inch	200	400	0	149+82.22	595.13	595.38	547.38	547.38
CIDH/ 30 inch	200	400	0	150+78.37	596.35	596.60	548.60	548.60
CIDH/ 30 inch	200	400	0	151+74.49	597.44	597.69	549.69	549.69
CIDH/ 30 inch	200	400	0	152+70.62	598.41	598.66	550.66	550.66
CIDH/ 30 inch	200	400	0	153+66.74	599.26	599.51	551.51	551.51
CIDH/ 30 inch	200	400	0	154+62.87	599.98	600.23	552.23	552.23
CIDH/ 30 inch	200	400	0	155+58.99	600.72	600.97	552.97	552.97
CIDH/ 30 inch	200	400	0	156+55.12	601.76	602.01	554.01	554.01
CIDH/ 30 inch	200	400	0	156+41.46	603.14	603.39	555.39	555.39
CIDH/ 30 inch	200	400	0	157+37.59	604.82	605.07	557.07	557.07
CIDH/ 30 inch	200	400	0	158+33.71	606.16	606.41	558.41	558.41
CIDH/ 30 inch	200	400	0	159+29.88	607.40	607.65	559.65	559.65
CIDH/ 30 inch	200	400	0	160+25.97	608.00	608.25	560.25	560.25
CIDH/ 30 inch	200	400	0	161+22.09	608.50	608.75	560.75	560.75
CIDH/ 30 inch	200	400	0	162+18.22	611.25	611.50	563.50	563.50

CIDH/ 30 inch	200	400	0	163+14.34	613.06	613.31	565.31	565.31
CIDH/ 30 inch	200	400	0	164+10.47	613.35	613.60	565.60	565.60
CIDH/ 30 inch	200	400	0	165+17.05	613.50	613.75	565.75	565.75

Table 5A - Pile Data Table for Burbank Western Channel Cover Abutment 2 (Right)

Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Channel Cover CL STA (ft)	Bottom of Pile Cap Elevation (ft)	Pile Cut-Off Elevation (ft)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)
		Compression (kip)	Tension (kip)					
CIDH/ 30 inch	200	400	0	149+82.22	594.41	594.66	546.66	546.66
CIDH/ 30 inch	200	400	0	150+78.37	595.63	595.88	547.88	547.88
CIDH/ 30 inch	200	400	0	151+74.49	596.72	596.97	548.97	548.97
CIDH/ 30 inch	200	400	0	152+70.62	597.69	597.94	549.94	549.94
CIDH/ 30 inch	200	400	0	153+66.74	598.54	598.79	550.79	550.79
CIDH/ 30 inch	200	400	0	154+62.87	599.26	599.51	551.51	551.51
CIDH/ 30 inch	200	400	0	155+58.99	600.00	600.25	552.25	552.25
CIDH/ 30 inch	200	400	0	156+55.12	601.04	601.29	553.29	553.29
CIDH/ 30 inch	200	400	0	156+41.46	602.42	602.67	554.67	554.67
CIDH/ 30 inch	200	400	0	157+37.59	604.12	604.37	556.37	556.37
CIDH/ 30 inch	200	400	0	158+33.71	606.14	606.39	558.39	558.39
CIDH/ 30 inch	200	400	0	159+29.88	608.33	608.58	560.58	560.58
CIDH/ 30 inch	200	400	0	160+25.97	609.42	609.67	561.67	561.67
CIDH/ 30 inch	200	400	0	161+22.09	610.67	610.92	562.92	562.92
CIDH/ 30 inch	200	400	0	162+18.22	611.50	611.75	563.75	563.75
CIDH/ 30 inch	200	400	0	163+14.34	611.83	612.08	564.08	564.08
CIDH/ 30 inch	200	400	0	164+10.47	612.50	612.75	564.75	564.75
CIDH/ 30 inch	200	400	0	165+17.05	612.42	612.67	564.67	564.67

Notes: Design Tip is controlled by the following:

1. Nominal Resistance in Compression.
2. Nominal Resistance in Tension is assumed to be 0.
3. Pile Lateral Resistance will be evaluated by the OSD.

Axial nominal resistance in compression, noted in the Table No.'s 4A and 5A above, is based on skin friction only. The end bearing was not considered due to potential use of drilling mud by the contractor to mitigate caving in subsurface soils.

CONSTRUCTION CONSIDERATIONS

1. Ground water is not anticipated at the proposed foundation depths.

2. The potential for caving exists in drilled holes for CIDH piles over their entire lengths. Due to the proximity of the piles to the channel wall, there is a potential for the caving to propagate all the way to the channel wall. The contractor should use an appropriate drilling method to minimize the caving.

3. The inside diameter of the cased oversized drilled hole within the top 12.5 to 15 feet of the pile should be large enough to avoid any contact between the pile and the casing. We recommend that the inside diameter of the oversized cased hole be at least 2 inches more than the diameter of the pile. The contractor should verify that no concrete or cement slurry enters the annular space between the pile and the outer casing during the casting of the pile. In addition, the contractor should take appropriate measures to keep the annular space free of any objects, down to its bottom, during and after the construction. The engineer should request that the contractor demonstrate the annular space is free of any objects down to its bottom. The casing installed for construction of the oversized hole should have a design life equal to the project design life.

4. Concrete placement for construction of the CIDH pile should be completed within the same day that excavation of the drilled hole has been completed.

5. Drilling during construction may be variable and sporadically hard (within gravel zones) down to anticipated pile tip elevations.

6. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications.

7. Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1:V:1H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations

If you have any questions, please call Tatjana Halda at (213) 620-2347 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 04-27-12

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District 07 Construction R.E. Pending File (Electronic File)
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State of California | Business, Transportation and Housing Agency

Memorandum

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To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: April 27, 2012

File: 07-LA-5-PM 30.07/30.38
07-1218W1
Overhead Sign Adjacent to Burbank Western Channel

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Recommendations for Overhead Sign Structure adjacent to Burbank Western Channel

Reference: Foundation Report for Burbank Western Channel (Cover), dated February 17, 2012.

INTRODUCTION

In response to the request by your office on April 5, 2012, the Office of Geotechnical Design South 1 (OGDS1), Branch D has prepared these foundation recommendations for the Overhead Sign Structure at Post Mile 30.2, proposed adjacent to Burbank Western Channel, Los Angeles County. The foundation recommendations provided in this memorandum were based on the subsurface explorations and testing conducted for the referenced report.

SITE DESCRIPTION

The proposed Overhead (OH) Sign No. 5B at Station 1595+40, (measured along centerline Interstate 5), will consist of a supporting post constructed on a square pedestal, which in turn is supported on a cast-in drilled-hole (CIDH) pile. The pedestal is 8 feet thick and has sides that are each 5 feet, 9 inches long. The CIDH pile is 5 feet in diameter and 22 feet in length. The pedestal and the pile will be constructed in accordance with Caltrans Standard Plans, S7.

The pile would be constructed approximately 7 feet away from the eastside channel wall. Between the sign structure and the channel wall, a row of 30-inch diameter CIDH piles will be installed parallel to the wall for the support of the proposed channel cover. The finished grade elevation at the location of the sign is 606 feet, approximately.

SITE SUBSURFACE CONDITIONS

Based on the subsurface conditions observed in the investigation for the referenced report, we are of the opinion that the site is underlain by artificial fill having a variable thickness. The fill consists of loose to medium dense silty sand. The alluvium underlying the fill, consists of loose to very dense silty sand/sandy silt with gravel/cobbles/ Schist rock fragments, and interbedded layers of soft to very stiff clay and sandy clay.

Groundwater

Groundwater was not encountered during the 2004/2005 field exploration at elevations above 505.2 feet, which is approximately 100 feet below the ground level at the site.

CORROSION

Soil samples were tested for corrosion potential indicated that subsurface soils are non-corrosive to metal and reinforced concrete.

FOUNDATION RECOMMENDATIONS

Based on the draft sign structure details and layout plan provided by the Office of Structure Design via an email on April 5, 2012 and the subsurface conditions described in the referenced report, OGDS1 believes that the foundation for overhead Sign No. 5B is adequate as proposed.

CONSTRUCTION CONSIDERATIONS

The following recommendations for CIDH pile construction should be incorporated in the plans and special provisions of the project.

1. The top several feet of the drilled hole is anticipated to be in the artificial fill placed during the channel wall construction. The fill and the underlying alluvium could consist predominantly of granular soils with gravel and cobbles.

2. The contractor shall clean out the bottom of the shaft prior to placing the cage and the concrete. Concrete placement for construction of the CIDH pile should be completed within the same day that excavation of the drilled hole has been completed.

3. Extensive caving could occur throughout its length of the drill hole. Caving could propagate to the channel cover pile and channel wall locations affecting the performance of those structures. Therefore, the contractor should take appropriate measures to mitigate caving of the hole. Any occurrence of a caving should be brought to the attention of the engineer immediately for taking appropriate remedial actions.

4. Groundwater is not expected during drilling or construction of CIDH piles.

For further information, contact Gamini Weeratunga at 949-440-3427 or Shiva Karimi at 213-620-2146.

Prepared by: Date: 04-27-12

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Reviewed by: Date: 04-27-12

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District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

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M e m o r a n d u m

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To: MR. MIKE POPE, CHIEF
Bridge Design Branch 18
Office of Structure Design

Date: January 31, 2012

File: 07-LA-5-PM 30.53/30.56
07-1218W1
I-5 Empire Ave. Interchange
Victory Place Separation (NEW)
Bridge No. 53C-2171

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Victory Place Separation (New), Bridge No. 53C-2171

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 1, 2011 and Wall General Plan and Foundation Plan (plotted November 1, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed new Victory Place Separation Bridge as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed bridge widening. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed bridge site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed bridge; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed bridge widening.

This Foundation Report supersedes the previous Foundation Recommendations for Victory Place Separation (based on updated metric plans) dated May 25, 2006 (Revised April 8, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the City of Burbank. The Empire interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metro link railroad tracks, and add high occupancy vehicle (HOV) lanes on Route 5 (one lane in each direction).

The proposed Victory Place Separation Bridge is a 149.79 feet long and 70.67 feet wide and consists of two span CIP prestressed box girder on multi-column bent and cantilever abutments. Proposed bridge is 43^0 55' 30" skewed and supports three traffic lanes, and shoulders of varying widths. Bridge abutment and bents will be supported on 70 ton 24 inch diameter Cast-in-Drilled-Hole (CIDH) piles. Begin and end bridge stations are at Sta. 35+58.43 and Sta. 37+08.24 ("V" Line), respectively, with corresponding elevations of +627.32 and +628.36 feet.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from January 11 to 31, 2006. The field investigation included drilling three 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 1.

Table 1 – Summary of Borings

Boring No.	"V" Line (Prop.) Stationing	Offset from "V" Line (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
06-77	33+95.31	28.56 Rt	623.5	106.5	1/11/2006
06-92	34+94.31	5.30 Lt.	626.0	100.2	1/25-27/2006
06-96	36+48.49	28.73 Rt.	628.0	115.4	1/31/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 2, below.

Table 2 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Corrosion	CTM 417, 422, 643	13
Atterberg Limits	CTM 204	2
Mechanical Analysis	CTM 201, 202, 203	2
Moisture Content	CTM 226	2

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Subsurface Conditions

The terrain is relatively flat and consists of an open field with no apparent vegetation, bounded easterly by Route 5, westerly by Empire Avenue, and southerly by Empire Center. There is no existing bridge at any other roadway structure at this location.

The upper one to three feet of the borings consisted of fill which is generally composed Asphaltic Concrete (AC) and base material, and loose to medium dense silty sand with gravel, and sand. The top of native material was logged at an elevation of about +615 to +623 ft in the borings. The native alluvium was generally composed of loose to very dense silty sands, and sands, with gravel lenses throughout. In addition, stiff to hard sandy lean clay layers and clayey sand layers were encountered during drilling. Density increases with depth, although the upper layers of alluvium may be less dense than the overlying fill. Below elevation +560 ft very dense well graded and poorly graded gravel were encountered. Rare scattered boulders (estimated $\leq$ 18 in length) were observed below approximate elevation +560 ft in Boring 06-96.

Groundwater

Groundwater was not encountered during the Caltrans 2006 field exploration. Perforated pipe was installed within boring 05-46A (approximately 600 feet to the northwest along southbound San Fernando Blvd.) and successive measurements taken during November 2005 and January 2006 revealed the boring to be dry to the bottom to 96.5 ft below surface (elevation +533 feet). Ground water was not encountered in any borings completed for the entire 2 miles length of Empire Interchange project.

SCOUR

There is no possibility of scour at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Corporation soils laboratory. Results presented in Table 3 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 3 - Corrosion Test Summary-Composite Sample

Boring No.	Sample Interval (ft)	pH	Minimum Resistivity (Ohm-Cm)	Sulfate Content (ppm)	Chloride Content (ppm)
06-77	0.0-25	7.8	5500	210	135
06-77	25-50	8.0	6900	18	150
06-77	50-75	8.2	5200	33	105
06-77	75-105	8.2	7100	12	105
06-92	10-30	8.0	6400	30	105
06-92	30-50	8.2	7700	18	105
06-92	50-70	8.2	10000	15	120
06-92	70-100	8.3	5600	36	120
06-96	6.9 -20.0	8.0	5000	45	120
06-96	20.0-40.0	8.0	3700	225	165
06-96	40.0-60.0	8.3	4100	540	315
06-96	60.0-80.0	8.1	3700	560	345
06-96	80.0-115.1	8.1	4900	ND	90
Caltrans Corrosion Guidelines		$\leq$5.5	<1000	$\geq$2000	$\geq$500

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMICITY

Faulting and Seismicity

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map or CSHM (CALTRANS, 1996) the Verdugo Fault (VDO), a reverse/oblique type fault is the nearest active seismic source from the site. Based on the CSHM, this fault is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w6.75. Based on the California Geological Survey (CGS, 2006) 2002 fault database, VDO is a reverse fault and capable of generating a maximum earthquake of Mw 6.9. Based on Weber et al (1980), this fault is located about 0.4 mile east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

For purpose of seismic evaluation, the soil profile is assigned soil type D based on recommendations in Caltrans Seismic Design Criteria (SDC, v 1.3). The recommended ARS curve was developed based on Figure B.7 of the Seismic Design Criteria by proportionality adjusting the values by a factor of 0.8/0.6 =1.33.

Surface Ground Rupture

The site is not located within any Alquist Priolo Earthquake Fault Zone as defined by the California Department of Conservation (Special Publication 42, 1997). As stated above the nearest known fault is located at a distance of about 0.4 mile from the site. Based on this information, the potential for ground rupture hazard at the site due to primary fault movement is considered low.

Liquefaction

This site is not located in an area shown as potentially liquefiable on the Special Studies Zones Map of the Burbank Quadrangle. Since groundwater was not encountered and soils were generally dense, the potential for liquefaction at the site is considered low. The potential for other seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

FOUNDATION RECOMMENDATIONS

The proposed bridge can be supported on CIDH piles. The following recommendations are based on 1) Caltrans 2006 soil borings test results, 2) Structure plans (plotted November 1, 2011) including design heights, bottom of footing elevations, footing dimensions, and CIDH diameter.

Axial capacities of CIDH piles were evaluated based on the FHWA method using SHAFT version 5 Computer Program. Pile details and elevations are shown in Table 4 below.

Table 4 - Pile Data Table for Victory Place Separation, Bridge No. 53C-2171

Support Location	Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Bottom of Footing Elev. (ft)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)
			Compression (kip)	Tension (kip)			
Abutment 1	CIDH/ 24 inch	140	280	0	+603.0	+567.0 (1)	+567
Bent 2	CIDH/ 24 inch	140	280	0	+603.0	+570.0 (1)	+570
Abutment 3	CIDH/ 24 inch	140	280	0	+603.0	+570.0 (1)	+570

Notes: Design Tip is controlled by the following demands:
(1) Nominal Resistance in Compression ($\leq$ 0.5 in vertical deflection at top of pile)
(2) Nominal Resistance in Tension is assumed to be 0.
(3) Pile Tip Elevations for Lateral Loads will be provided by Structure Design.
(4) Based on the General plan, it appears that finished grade ranges from approximately +608 to +610 feet elevation.

Axial nominal resistance in compression, noted in the Table No. 4 above, is based on skin friction only within the alluvial soils. End bearing was not considered due to potentially caving soils near pile tip elevation.

Settlement

The settlement at approach fills is considered "immediate" and is expected to occur during construction. The magnitude of settlement during the construction is estimated to be 2 inches. The actual time to start construction should be subject to review and monitoring data and approval by the resident engineer.

SLOPE STABILITY ANALYSES

The global stability of the proposed abutment fill slopes was evaluated using the computer program XSTABLE version 5 under both static and pseudo-static conditions. Two critical cross sections at Abutment 1 and Abutment 3 were used to analyze the global stability. The result yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and pseudo static condition, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated in the footing excavation.

2. Moderate to heavy caving may be anticipated within sandy and gravelly soils during excavation of CIDH pile borings. Casing and/or slurry maybe required in CIDH pile borings.

3. The bottom of all excavations should be cleaned of loose debris before placing concrete.

4. Drilling during construction may be variable and sporadically hard (within gravel zones with scattered hard cobbles) down to anticipated pile tip elevations. Rare scattered boulders (estimated $\leq$ 18 in length) were observed below approximate elevation +560 ft during the 2006 boring program. Also, approximately 12 inch length boulders were found near footing grade elevation which would need to be excavated if encountered. Slightly larger CIDH piles (24 inch diameter) are recommended for potential excavation of possible cobbles/boulders.

5. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications. If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential.

6. Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1:V:1.5H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations.

For further information, please contact Akbar Mehrazar at 949-440-3415 or Shiva Karimi at 213-620-2146.

Prepared by: Date: 1/31/2012

A Mehrazar

Akbar Mehrazar
Transportation Engineering
Office of Geotechnical Design–South 1
Branch D

Supervised by: Date: 1/31/2012

P Shiva Karimi

Shiva Karimi, Ph.D., P.E.
Senior Transportation Engineer
Office of Geotechnical Design–South 1
Branch D

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Division of Engineering Services
Structure Design Services
Structure Hydraulics and Hydrology
Preliminary Hydraulic Report

Burbank Western Channel

Located along 5 Freeway, part of US Army Corps of Engineers Flood Control System in the County of Los Angeles

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JOB: Burbank Western Channel, EA 07-121821

LOCATION: Bridge No. 51-LACFCD 07-LA-5- PM 47.9 / 50.9

WRITTEN BY: Ronald McGaugh DATE: January 29, 2012

REVIEWED BY:

Ronald L McGaugh

General

The Office of Structure Design is proposing to close a concrete lined open channel by placing a cover on it and/or span a portion of the existing Flood Control District's (LACFCD) Burbank Western Channel, located in Los Angeles County. The proposed culvert will extend from approximately Scott Road to San Fernando Blvd.

This report makes reference to data and analysis found in (1) General plans and profiles submitted by the Office of Structure Design, (2) As-Built Plans dated August, 1958, (3) Information received from the Army Corp. of Engineers.

Drainage Basin

The Burbank Western Channel is part of an extensive flood control system built to protect the metropolitan area. The Burbank Western Channel drains a watershed of approximately 16 sq miles.

The climate surrounding the project is characterized as subtropical and dry, with warm summers and mildly cool winters. During the wet season, November to April, precipitation occurs in the form of localized cloudbursts and general heavy rains. Approximately 90% of the annual rainfall occurs during this period with an average annual precipitation from 305 to 406 millimeters, 12 to 16 inches. The area is characterized by high peak flows and short durations due to the highly developed areas.

Discharge

According the US Army Corp. of Engineers, Los Angeles District, the maximum capacity the channel was designed for within this project reach is 11,000 cfs.

Stage ,Velocity and Required Waterway

Existing conditions

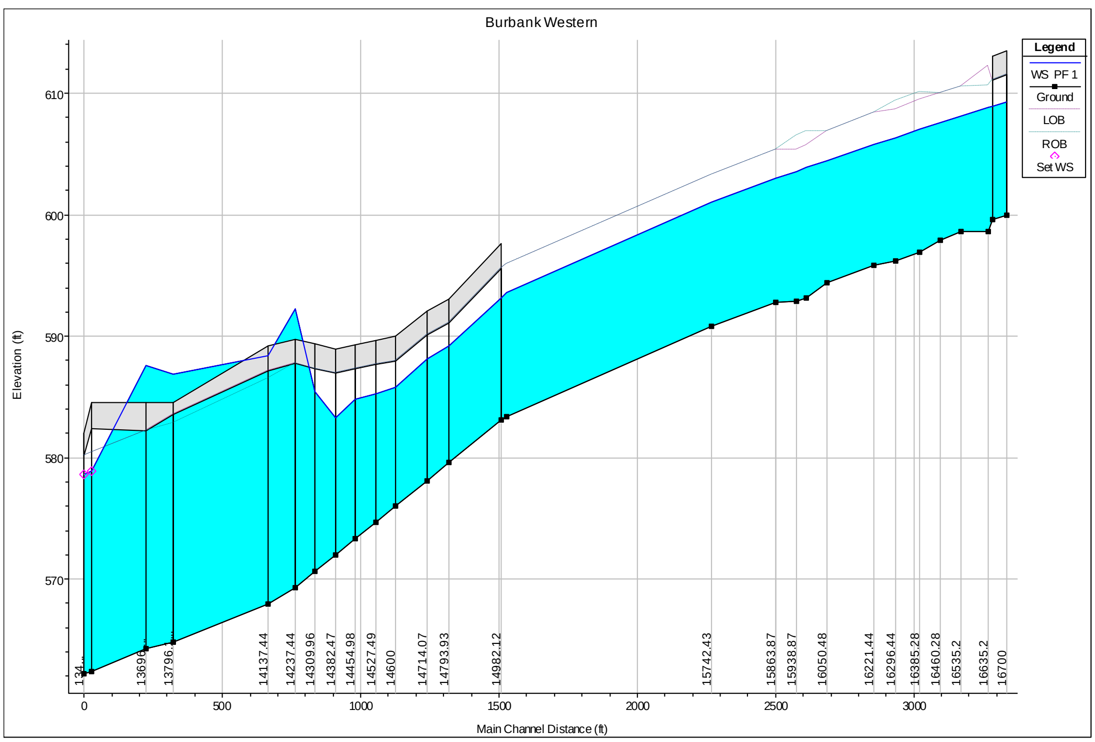
Table 1 shows the existing parameters according to the Pertinent Data Sheet submitted to this office by the Corps of Engineers. Using the following Parameters:

- Existing channel is a combination of open channel and closed box culvert
- Mannng's n value = 0.14
- Supercritical flow
- Upstream starting water surface elevation of 609.3 at sta 167+00
- Downstream starting water surface elevation of 578.6 at sta 134+74
- Cross section with "lids" provided for existing covered sections
- All information Input into Hec-Ras Model

Burbank Western Channel
Bridge # 51-LACFCD
07-LA-5- PM 47.9 / 50.9
EA 07-121821

TABLE 1 (and Profile plot)

River Sta	Q Total	Min Ch El	W.S. Elev	Vel Chnl	L. Freeboard	R. Freeboard	Invert Slope
	(cfs)	(ft)	(ft)	(ft/s)	(ft)	(ft)	
16700	10500.00	600.00	609.31	37.59	2.19	2.19	0.0086
16650	10500.00	599.57	608.96	37.28	2.11	2.11	0.0662
16635.2	10500.00	598.59	608.85	37.20	3.44	1.74	0.0001
16535.2	10500.00	598.58	608.13	36.65	2.45	2.45	0.0085
16460.28	10500.00	597.94	607.59	36.28	2.45	2.45	0.0132
16385.28	10500.00	596.95	607.03	35.94	2.42	3.10	0.0087
16296.44	10500.00	596.18	606.35	35.60	2.33	3.01	0.0040
16221.44	10500.00	595.88	605.79	35.32	2.59	2.59	0.0087
16050.48	10500.00	594.40	604.45	34.83	2.45	2.45	0.0161
15975.48	10500.00	593.19	603.87	34.63	1.82	2.96	0.0087
15938.87	10500.00	592.87	603.56	34.57	1.81	2.95	0.0009
15863.87	10500.00	592.80	602.97	34.41	2.33	2.33	0.0086
15742.43	10500.00	590.80	601.08	34.04	2.22	2.22	0.0099
15000	10500.00	583.42	593.57	34.50	2.35	2.35	0.0185
14982.12	10500.00	583.09	593.18	34.67	2.41	2.41	0.0186
14793.93	10500.00	579.59	589.22	36.34	1.87	1.87	0.0185
14714.07	11000.00	578.11	588.16	36.49	1.95	1.95	0.0186
14600	11000.00	575.99	585.81	37.35	2.18	2.18	0.0185
14527.49	11000.00	574.65	585.24	37.03	2.41	2.41	0.0186
14454.98	11000.00	573.30	584.85	36.55	2.45	2.45	0.0186
14382.47	11000.00	571.95	583.33	37.13	3.62	3.62	0.0185
14309.96	11000.00	570.61	585.48	33.41	1.88	1.88	0.0186
14237.44	11000.00	569.26	592.27	25.97	-4.51	-4.51	0.0128
14137.44	11000.00	567.98	588.37	29.18	-1.19	-1.19	0.0093
13796.19	11000.00	564.82	586.88	26.53	-3.36	-3.36	0.0058
13696.19	11000.00	564.24	587.59	26.07	-5.35	-5.35	0.0093
13499.67	11000.00	562.42	578.85	33.48	3.57	3.57	0.0093
13474	11000.00	562.18	578.60	33.50	1.58	1.58	



At station 149+82 downstream the Channel has a cover. There is an existing Hydraulic jump that occurs within the covered section near station 142+37.

Proposed Conditions

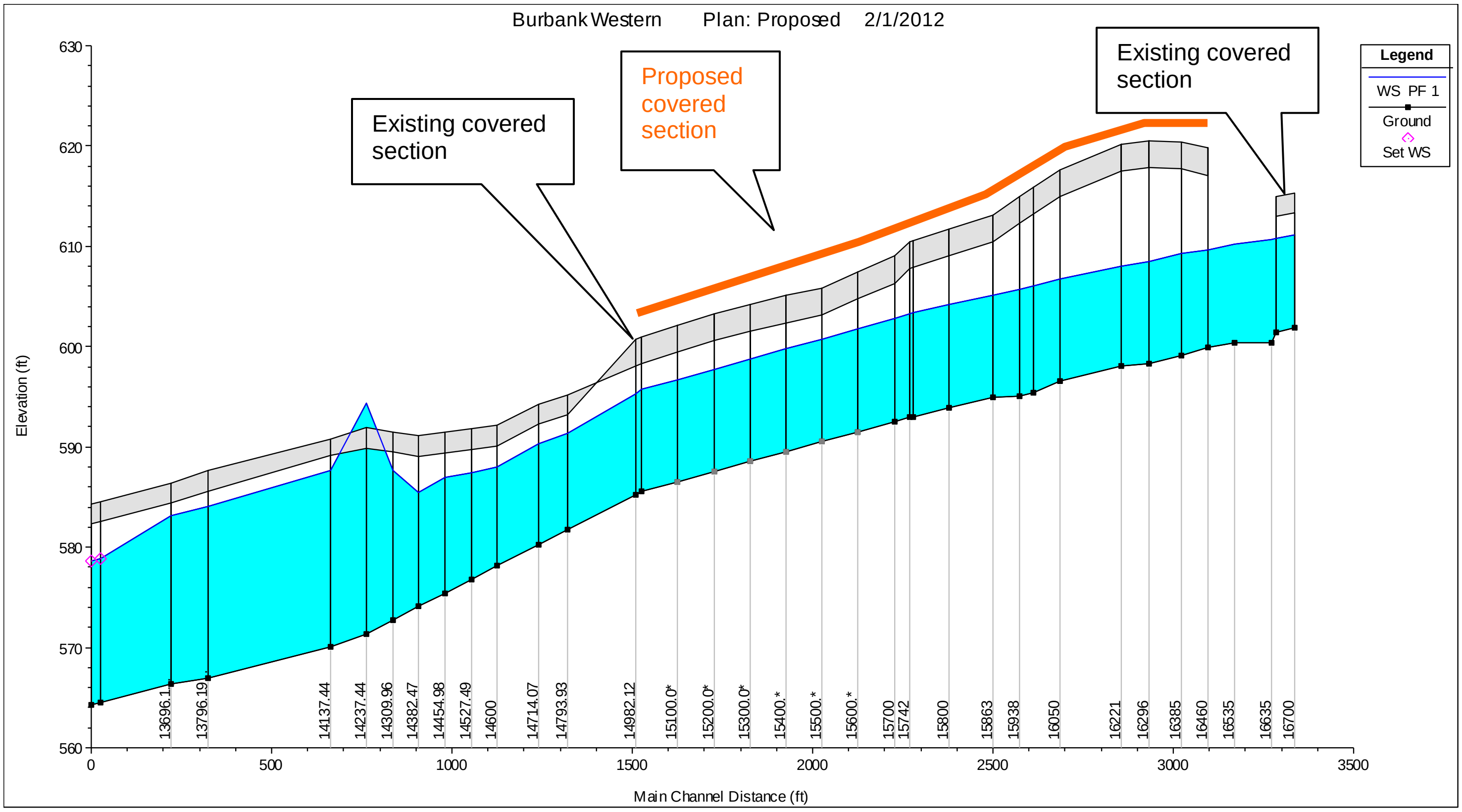
The open channel portions of the existing structure were surveyed by CALTRANS. The Survey was on a different Datum than the original Pertinent Data Sheets and The As-Built plans submitted to this office. New survey information, the As-Builts and the Pertinent Data Sheets, were used to generate a corrected channel geometry which was used for the Proposed Plan in the Hec- Ras model. More information about the survey can be found in the Appendix.
Table 2 shows the Proposed channel parameters using the following Parameters:

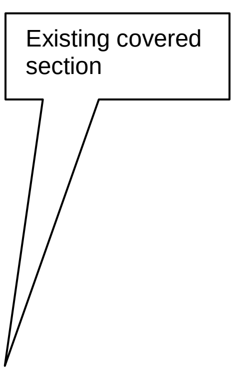
- Proposed channel has a cover for the limits of the project
- Mannng's n value = 0.14
- Supercritical flow
- Upstream starting water surface elevation of 609.3 at sta 167+00
- Downstream starting water surface elevation of 578.6 at sta 134+74
- Cross section with "lids" provided for existing covered sections
- Pressurization is expected downstream of sta 149+82 in the existing portion of the channel, our proposed work does not affect or change this condition

Table 2 (and Profile Plot)

River Sta	Q Total	Min Ch El	W.S. Elev	Vel Chnl	L. Freeboard	R. Freeboard	Invert Slope
	(cfs)	(ft)	(ft)	(ft/s)	(ft)	(ft)	
16700	10500.00	601.89	611.19	37.62	2.10	2.10	0.0086
16650	10500.00	601.46	610.84	37.31	2.12	2.12	0.0743
16635	10500.00	600.36	610.73	37.22	5.40	4.30	-0.0002
16535	10500.00	600.38	610.24	36.39	4.64	4.67	0.0059
16460	10500.00	599.94	609.69	36.04	4.32	4.78	0.0112
16385	10500.00	599.10	609.25	35.60	4.92	5.45	0.0086
16296	10500.00	598.34	608.51	35.37	4.77	5.39	0.0044
16221	10500.00	598.01	608.01	35.02	5.05	4.30	0.0085
16050	10500.00	596.56	606.70	34.54	4.89	2.42	0.0161
15975	10500.00	595.35	606.03	34.49	4.34	2.82	0.0089
15938	10500.00	595.02	605.71	34.46	4.48	2.96	0.0011
15863	10500.00	594.94	605.17	34.23	3.83	2.38	0.0087
15800	10500.00	593.89	604.17	34.06	4.20	2.86	0.0088
15746	10500.00	593.00	603.33	33.94	3.19	2.73	0.0087
15742	10500.00	592.92	603.26	33.93	2.13	1.88	0.0099
15700	10500.00	592.50	602.82	33.97	2.24	1.93	0.0099
15600.*	10500.00	591.51	601.80	34.05	2.23	1.71	0.0099
15500.*	10500.00	590.52	600.78	34.13	2.29	1.84	0.0100
15400.*	10500.00	589.52	599.77	34.20	2.40	2.14	0.0099
15300.0*	10500.00	588.53	598.75	34.27	2.38	2.32	0.0099
15200.0*	10500.00	587.54	597.74	34.33	2.37	2.36	0.0099
15100.0*	10500.00	586.55	596.73	34.38	2.37	2.26	0.0100
15000	10500.00	585.55	595.72	34.43	2.39	2.04	0.0187
14982.12	10500.00	585.22	595.33	34.62	2.39	2.39	0.0186

Burbank Western Channel
Bridge # 51-LACFCD
07-LA-5- PM 47.9 / 50.9
EA 07-121821





The Proposed project meets the minimum freeboard requirement of 1.5 ft. for the entire reach of the project for the left and right vertical walls of the project.

A new pumping plant is proposed to aid in additional drainage with a total outflow of 20 cfs to the channel. 20 cfs amounts to .001 % of the design flow so the pumping plant will have no impacts to the proposed design.

Supplemental Information requested by the Corps of Engineers are in Appendix 1 and 2 at the end of this report.

Streambed and Scour

Structure Hydraulics has no scour concerns for a concrete lined channel.

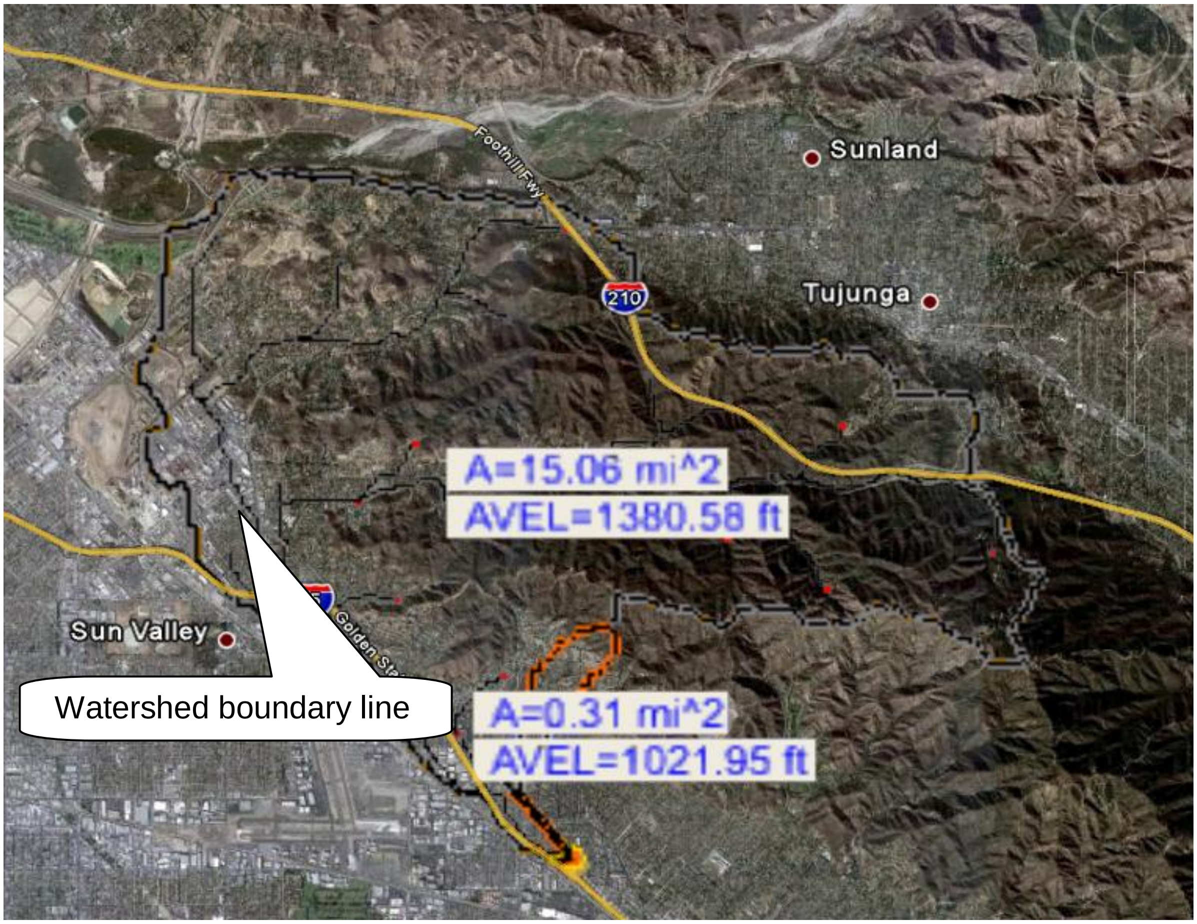
Drift

Structure Hydraulics has no drift concerns.

Appendix 1

Analysis of the upstream contributory water for degree of future watershed development and accompanying increase in discharges to the channel.

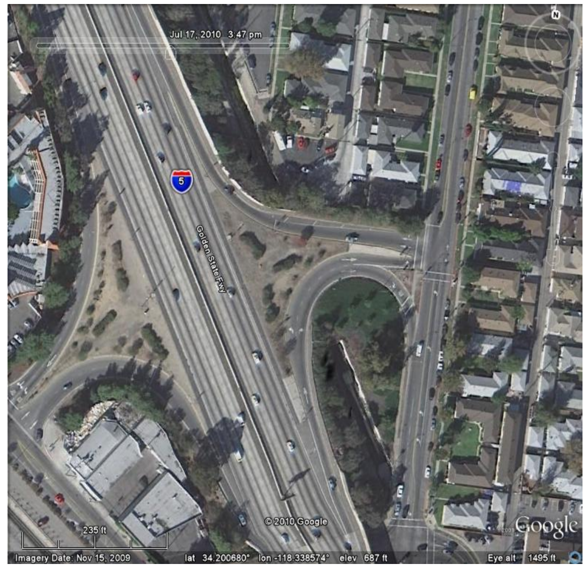
From a review of aerial photographs there has not been significant development in the watershed area since 1989. We know of no plans for any large developments within the watershed boundaries. We know of no changes to the watershed or any major expansion that would contribute any more to the discharge than is presently. We will use the Army Corps of Engineers developed Q of 10,500 at the yellow outlet indicated in the graphic below.



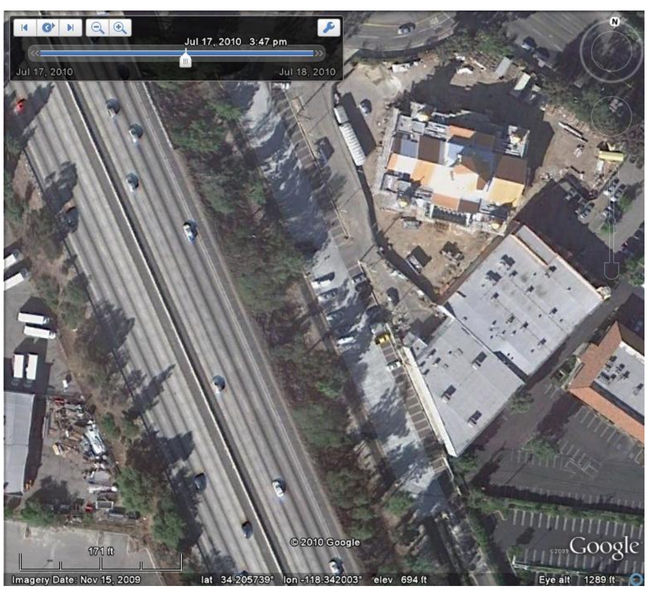
Identification of existing structures covering the channel upstream of the proposed overbuild and their distance from the site. This information should include, but not be limited to the type of overbuild and sectional views of the overbuild. The intent is to determine existing constraints that could prevent practical, future channel enlargement.

All photos below are referenced from the downstream starting point of the project at approximate station of 142+00

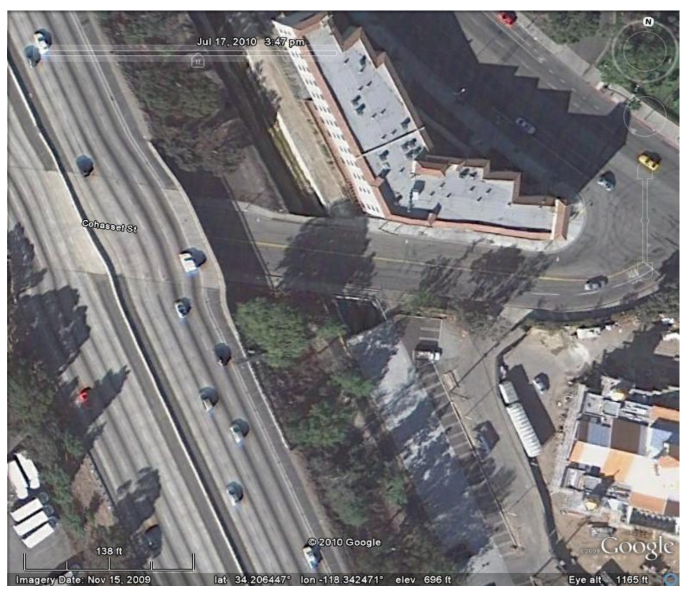
Winona Avenue and N Buena Vista Street
0.62 miles from station 142+00



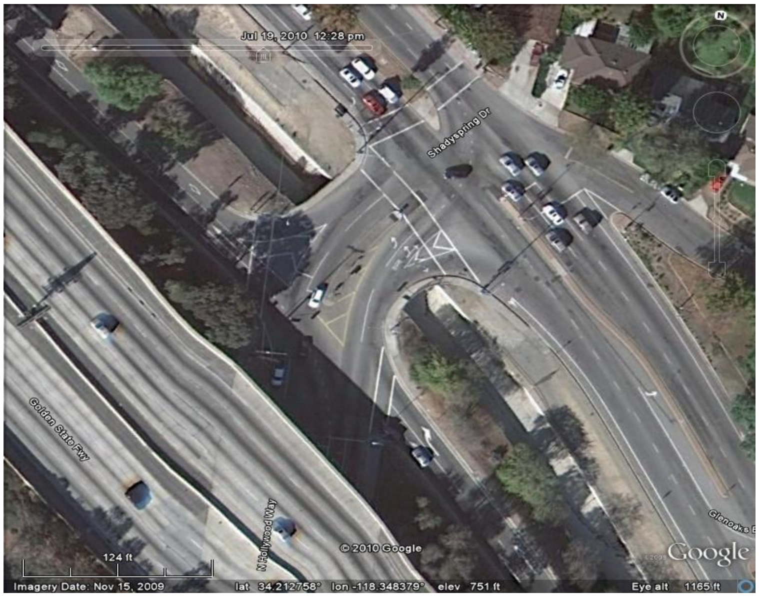
On and off ramps for the 5 Freeway near N Buena Vista Street
0.71 miles from station 142+00



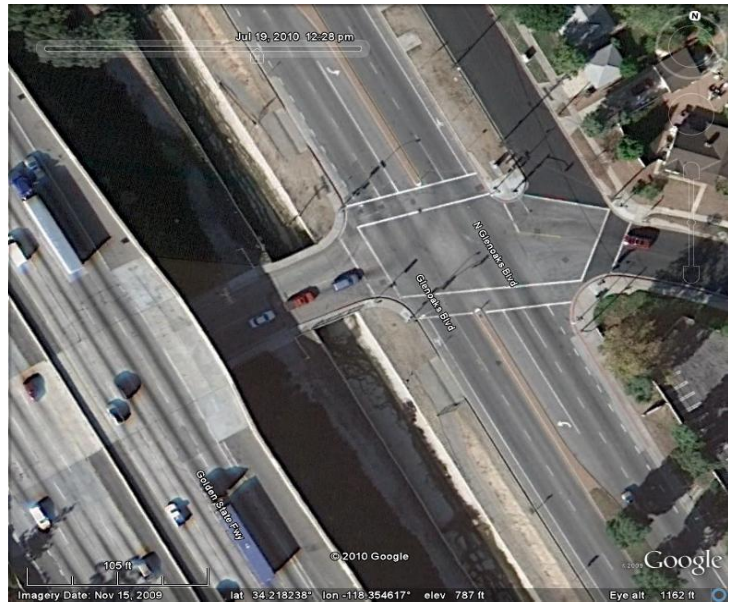
Existing covered section just south of Cohasset Street
0.71 miles from station 142+00



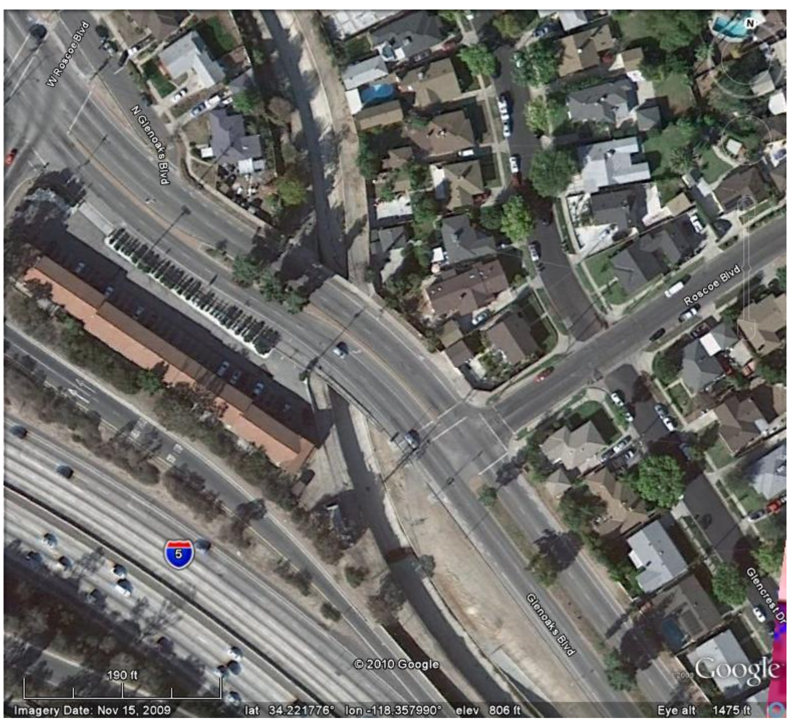
Cohasset Street and end of existing covered section--
1.17 miles from station 142+00



North Hollywood Blvd and Glenoaks Blvd
1.72 miles from station 142+00

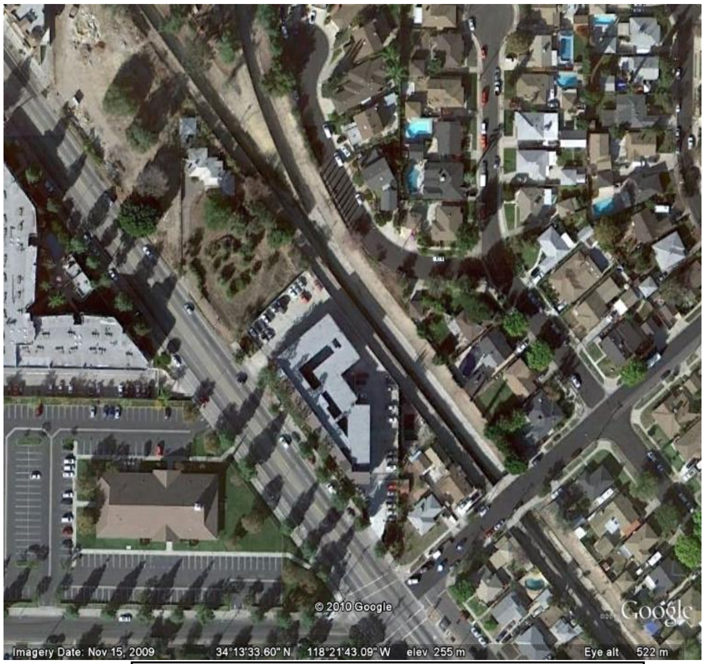


Lanark Street
2.24 miles from station 142+00



Glenoaks Blvd just north of Roscoe Blvd
2.56 miles from station 142+00

Nettleton Street
2.86 miles from station 142+00



Split just South of Vinedale Street
2.95 miles from station 142+00

Beginning of existing covered
section at station 142+00

Consideration of the potential for floating debris in the channel and its impacts

The watershed is in a highly urban area, with managed forest areas in the Verdugo Hills. This combination of areas produces a small amount of debris, however trash may be a problem. Our project will not adversely affect the potential for floating debris.

Residual overflow patterns if the discharges exceed the improved channel's capacity.

Exceeding the channel's capacity will cause local neighborhood flooding, local traffic inconveniences and other local nuisances until flows have receded.

Identify the discharge under which pressurization will occur

Pressurization is expected downstream of sta 149+82 in the existing portion of the channel, our proposed work does not affect or change this condition.

Provide a provision for adequate venting to prevent condition of pressurization.

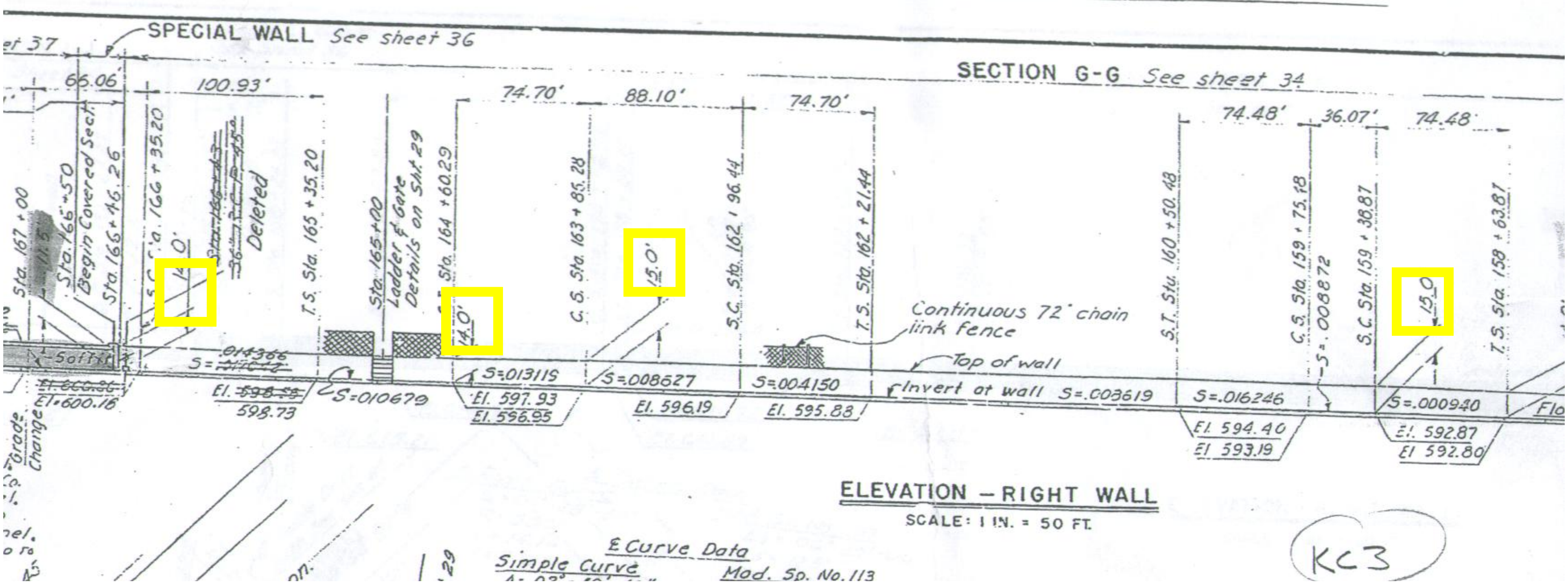
Manholes with bolted covers every 400 to 500 feet

APPENDIX 2

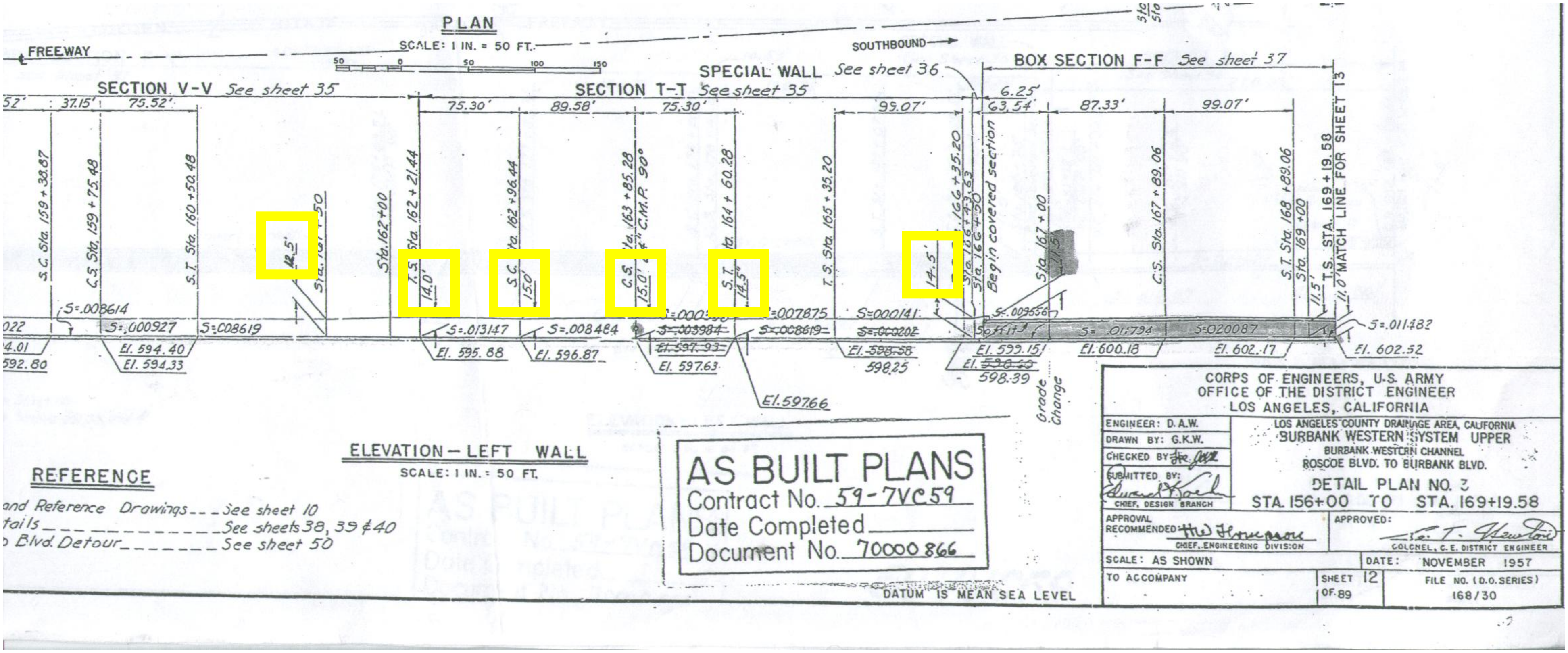
In response to the Memo from the Corps of Engineers CESPL- ED-DV dated January 4 2012 for the Burbank Western Channel

The Following Exhibits were taken from the AS Built Plans 59-7VC59 Document Number 70000 866

The geometry of the open channel potion of this project was surveyed by CALTRANS and matches well with the AS Built plans. This data does not match the PERTINET DATA SHEET (PDS) values. The AS BUILT and surveyed wall heights are greater than those in the PDS between stations 166+35 and 151+00. CALTRANS hec-ras model reflect the As-Built condition. The yellow highlights in the exhibits below show the wall heights used.



Only the open channel portion of this project was surveyed, for elevations within the covered sections of the channel the pertinet data sheet was used for the wall heights. Between stations 158+63 and 150+00 the wall height in the PDS was a general 12.5. These elevations matched well with the left wall height but was off as much as 0.5 foot for the right wall. The proposed Hec-ras run has the corrected wall heights.



There are three plans to run in the Hec-Ras model, an existing plan consisting of the information only from the PDS, the Modified Plan which is the surveyed open channel portion along with Modified PDS data to maintain slopes and account for elevation differences form the Caltrans Surveys to the PDS, and the proposed plan consisting of the proposed channel modifications using the modified plan data. Lids on the cross section have been provided in the appropriate locations to simulate a covered channel. The Proposed project does not affect any of the existing channel downstream of station 149+82 in the existing covered structure.

River Sta	Distance	left	right	C/L channel	Left	Right	Difference from	C/L
		channel	channel	Elevation	top of wall	top of wall	COE DaTA	Slope
		invert	invert	invert				
16700	50	601.8886	601.8886	601.888622	613.3886216	611.5087838	1.888621622	0.0086
16650	14.7	601.4586	601.4586	601.458622	612.9586216	612.9586216	1.888621622	0.0088
16635.3	99.52	**602.301**	**600.358**	**601.3295**	**615.033**	**616.133**	1.8895	0.0071
16535.78	75.44	**600.87**	**600.381**	**600.6255**	**614.9015**	**614.884**	2.0455	0.0085
16460.34	75.01	**600.027**	**599.94**	**599.9835**	**614.475**	**614.015**	2.4435	0.0075
16385.33	88.87	**599.103**	**599.736**	**599.4195**	**614.697**	**614.171**	2.1295	0.0091
16296.46	75.02	**598.341**	**598.881**	**598.611**	**613.901**	**613.279**	2.091	0.0079
16221.44	170.99	**598.03**	**598.009**	**598.0195**	**612.312**	**613.063**	2.1395	0.0085
16050.45	75.01	**596.56**	**596.574**	**596.567**	**609.102**	**611.594**	2.167	0.0092
15975.44	36.6	**595.349**	**596.41**	**595.8795**	**608.847**	**610.371**	2.1195	0.0090
15938.84	74.89	**595.023**	**596.076**	**595.5495**	**608.669**	**610.183**	2.1095	0.0081
15863.95	120.85	**594.944**	**594.944**	**594.944**	**607.548**	**608.998**	2.144	0.0087
15800**	101	**593.91**	**593.89**	**593.89**	**607.86**	**606.29**	2.144	0.0088
15700**	9.38	**593.04**	**593**	**593**	**606.92**	**605.23**		0.0064
15742.43EQ	42.43	**592.962**	**592.918**	**592.94**	**605.133**	**606.828**		0.0099
15700	100	**592.54**	**592.5**	**592.52**				0.0100
15600	100	**591.51**	**591.54**	**591.525**				0.0100
15500	100	**590.54**	**590.52**	**590.53**				0.0100
15400	100	**589.55**	**589.52**	**589.535**				0.0100
15300	100	**588.55**	**588.53**	**588.54**				0.0100
15200	100	**587.55**	**587.54**	**587.545**				0.0100
15100	100	**586.55**	**586.55**	**586.55**				0.0099
15000	17.88	**585.556**	**585.554**	**585.555**	**598.117**	**599.554**	2.135	0.0187
14982.12	188.19	585.22	585.22	585.22	597.72	597.72	2.13	0.0186
14793.93	79.86	581.72	581.72	581.72	593.22	593.22	2.13	0.0185
14714.07	114.07	580.24	580.24	580.24	592.24	592.24	2.13	0.0186
14600	72.51	578.12	578.12	578.12	590.12	590.12	2.13	0.0185
14527.49	72.51	576.78	576.78	576.78	589.78	589.78	2.13	0.0186
14454.98	72.51	575.43	575.43	575.43	589.43	589.43	2.13	0.0186
14382.47	72.51	574.08	574.08	574.08	589.08	589.08	2.13	0.0185
14309.96	72.52	572.74	572.74	572.74	589.49	589.49	2.13	0.0186
14237.44	100	571.39	571.39	571.39	589.89	589.89	2.13	0.0128
14137.44	341.25	570.81	570.11	570.46	589.31	588.61	2.13	0.0093
13796.19	100	567.65	566.95	567.3	585.65	584.95	2.13	0.0058
13696.19	196.52	566.37	566.37	566.37	584.37	584.37	2.13	0.0093
13499.67	25.67	564.55	564.55	564.55	582.55	582.55	2.13	0.0093

13474	7	564.31	564.31	564.31	582.31	582.31	2.13	0.0093
				564.245			2.13	

Bold indicates surveyed values given to us by profiles Submitted August 2011.

Other Data is an altered Corps data. The slopes and elevations of the invert for the Corps data was compared to the surveyed data and adjusted for use in this model. The difference in elevation from the surveyed data is shown in the COE Data Column.

Information from PDS and this survey data used for “composite” data set for a “new” channel geometery

Survey Verified by AS-Builts\

PDS used in “covered” beyond station 14982.12

Memorandum

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: January 16, 2012

File: 07-LA-5- PM 30.56/30.62
07-1218W1
Empire Interchange
Retaining Wall No. 25

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 25

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 7, 2011 and Wall General Plan and Structure Plans (plotted October 7, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 25 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 25 will be constructed along westbound Empire Avenue between post miles 30.56 and 30.62 of State Route 5 and next to Victory Place Separation Bridge No. 53C-2171. Retaining Wall No. 25 consists of two segments; 1) Caltrans Standard Type 1 retaining wall with Type 736A concrete barrier at the top and Type 60D concrete barrier at Empire Ave level, and 2) Caltrans Standard Type 7SW retaining wall with Type 736A concrete barrier at the top and Type 60D concrete barrier at Empire Ave level. Retaining Wall No. 25 will predominantly retain existing alluvial soils above the proposed Empire Avenue cut section within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;

c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.25 (based on updated metric plans) dated November 21, 2006 (Revised May 18, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 25 consists of two segments; 1) Caltrans Standard Type 1 retaining wall with Type 736A concrete barrier at the top and Type 60D concrete barrier at Empire Ave level, and 2) Caltrans Standard Type 7SW retaining wall with type 736A concrete barrier at the top and Type 60D concrete barrier at Empire Ave level.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 8 to 22 feet with a total length of 440.1 feet located from RW LOL Station 22+46.89 to Station 26+86.99 (approximate 48.72 ft Lt., Sta. 22+53.37 to 42.89 ft Lt., Sta. 27+04.56 Centerline Empire Ave. "E Line"). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 and 2 for Retaining Wall No. 25. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 22+46.89	STA 23+10.89	Type 1	8	5.25	617.00
STA 23+10.89	STA 24+06.89	Type 1	10	6.25	614.24
STA 24+06.89	STA 24+54.89	Type 7SW	12	9.25	611.22

STA 24+54.89	STA 25+02.89	Type 7SW	14	10.75	609.28
STA 25+02.89	STA 25+50.89	Type 7SW	18	13.75	607.40
STA 25+50.89	STA 25+98.89	Type 7SW	20	15.25	604.94
STA 25+50.89	STA 26+86.99	Type 7SW	22	16.75	602.46

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from January 31 through March 31, 2006. The field investigation included drilling three 4.5-inch outer diameter mud rotary borings and four electronic cone penetration tests (CPT). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Empire Ave (Prop.) Stationing	Offset from Empire Ave "E Line" (ft)	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elev. (ft)	Depth (ft)	Date Drilled
06-100 CPT	21+18.02	77.11 Rt.	1616+96.50	894.66 Lt.	625.2	58.9	3/31/2006
06-101 CPT	23+51.27	105.78 Rt.	1614+57.20	756.30 Lt.	624.5	59.6	3/31/2006
06-102 CPT	25+53.98	119.23 Rt.	1612+92.73	593.14 Lt.	622.5	62.2	3/31/2006
06-95 CPT	23+79.16	45.33 Lt.	1615+57.02	630.99 Lt.	624.0	60.7	1/30/2006
06-92	29+07.17	104.08 Rt.	1611+12.61	314.29 Lt.	626.0	100.2	1/25-27/2006
06-94	19+20.45	6.01 Rt.	1619+37.93	937.02 Lt.	629.1	78.1	1/30-31/2006
06-96	28+12.47	25.20 Lt.	1612+69.41	295.64 Lt.	628.0	115.4	1/31/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	2
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	12

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

Proposed Retaining Wall 25 is bounded to the north by the north curb of Empire Ave., to the south by the Burbank Empire Center (mall), east by Victory Place, and west near Maria Street. Existing conditions are predominantly level ground where the proposed walls will be located.

Based on OGDS1's 2006 foundation investigation, sediments at the proposed wall site consist of minor preexisting embankment fill and minor localized trench fill (approximately up to 7 feet thick, surrounding utilities beneath existing Empire Ave.) and fill (up to 2 feet thick, undifferentiated with road base material) beneath roadway pavement and base material. The above sporadic fills are underlain by Holocene alluvium (Qa unit of Dibblee, 1991a) and probable older undifferentiated Quaternary alluvial fan gravel derived from the Verdugo Mountains (Qf unit of Dibblee, 1991a) or older Pleistocene alluvium. Most recent deeper borings/penetration tests have likely terminated within older Pleistocene alluvium or fan gravel. The minor embankment fill was not sampled due to the restricted horizontal and vertical extent. Localized trench fill and undifferentiated fill beneath roadways are composed of estimated loose sand with sporadic gravel and silty sand. Partially underlying and exposed at surface was undifferentiated Holocene or older fan gravel/Pleistocene alluvium which can be separated into approximately four units. The upper alluvial unit is composed of predominantly medium dense to minor loose sand to silty sand with sporadic gravel and gravel lenses from elevations ranging from +624.8 and +621.9 ft down to elevations ranging from +614.7 and +610.1 ft. The underlying second alluvial unit, ranges between approximate elevations +614.7 and +610.0 ft down to approximate elevations ranging from +609.4 to +607.8 ft, consists predominantly of medium dense to dense (rare soft/very loose), sand and silty sand with sporadic gravel, and minor clayey sand. The underlying third alluvial unit, ranges between approximate elevations +609.4 to +607.8 ft down to approximate elevations +568.8 to +562.8 ft and generally consists of dense (minor loose/stiff to firm) sand to silty sand with sporadic gravel and cobbles and rare boulders (est. ≤12 in. length) interlensed with minor sandy silt, clayey sand, and rare sandy lean clay. The underlying lower alluvial unit ranges between approximate elevations +568.9 to +562.8 ft down to approximate elevation +512.6 ft, generally consists of very dense to dense/hard, silty sand to sand with gravel and cobbles, gravel/cobble lenses with sand matrix, and rare sandy lean clay. The deepest recent boring for the subject walls, Boring 06-96 (drilled January/February 2006, near the east end of the subject retaining walls) was drilled 115.4 ft below the surface to approximate elevation +512.6 ft. The LOTB should be consulted for more specific details.

Groundwater

Groundwater was not encountered during the recent field exploration. A perforated pipe was installed within nearby boring 05-46A (approximately 300 ft to the north and northeast of the walls along Southbound San Fernando Blvd.) and successive measurements taken during November 2005 and January 2006 revealed the boring to be dry to the bottom to 95.2 ft below surface (elevation +533.2 ft). All auger borings completed for the entire approximate 2 mile long Empire Interchange project also showed no groundwater was encountered.

Ground water was also not encountered during the 1957 field investigation for the nearby existing Southbound San Fernando Blvd UC (Br. No. 53-1215, As Built LOTB plan dated June 1961) down to approximate elevation +559 ft the maximum penetration depth 63.3 ft obtained. Also no ground water was encountered on tape measured down to caving depth of 50 ft at elevation +568.7 ft within cone penetrometer boring B-1.

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 25

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
06-92	3.3 to 26.3	6400	8.0	105	30
06-92	29.5 to 52.5	7700	8.2	105	18
06-92	55.8 to 72.2	10000	8.2	105	15
06-92	75.4 to 105.0	5600	8.3	120	36
06-94	3.3 to 42.6	7700	8.6	90	6

06-94	45.9 to 68.9	5500	8.3	75	ND
06-94	7.2 to 95.1	6700	7.1	105	45
06-96	3.3 to 16.4	5000	8.0	120	57
06-96	19.7 to 39.4	3700	8.0	165	225
06-96	42.6 to 59.0	4100	8.3	315	540
06-96	62.3 to 88.6	3700	8.1	345	560
06-96	91.8 to 111.5	4900	8.1	90	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM), the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 mile east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Ground water was not encountered to at least a measured depth below the surface from 95.2 ft, last measured January 09, 2006 within nearby boring 05-46A. Also, soils are predominantly medium dense to dense granular material and minor firm/stiff clays and clayey sands. Bottom foundation depths will be significantly above the measured dry bottom of boring depths.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

Caltrans Standard Type 1 (RW LOL STA 22+46.89 to STA 24+06.89) and Type 7SW (RW LOL STA 24+06.89 to STA 26+86.99) retaining walls are considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 wall and Type 7SW spread footings are recommended for retaining wall support as existing soils with minor remediation described below are adequate to support the wall. Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Existing poor quality soils beneath a portion of the wall footprint (from Retaining Wall LOL station 24+06.89 to 26+86.99) will be replaced with structure backfill compacted to 95% R.C. (relative compaction) per ASTM D1557 test method. Remedial treatment consists of overexcavating existing soils within the specified limits to 3 ft below footing grade and replacing these soils with structure backfill compacted to 95% R.C. up to footing grade. The horizontal limits of the structure backfill prism should extend from a line 1 ft beyond the retaining wall's toe and heel and then down and out at a 1.5:1 (H:V) slope to a depth equal to the bottom of the subexcavation. Refer to the Caltrans Standard Specifications (2006), section 19-5.03 for details.

Table 5A– Spread Footing Data for Retaining Wall No.25
Type 1 with Case I Loading (2 ft level surcharge)

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Over-Excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To						
STA 22+46.89	STA 23+10.89	Type 1 Case I	8	5.25	617.00	N/A	2.2
STA 23+10.89	STA 24+06.89	Type 1 Case I	10	6.25	614.24	N/A	2.5

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

Table 5B– Spread Footing Data for Retaining Wall No.25 (Type 7SW)

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Over-Excavation Elevation (ft)	Ultimate Bearing Capacity Required (ksf) (ksf)
From	To						
STA 24+06.89	STA 24+54.89	Type 7SW	12	9.25	611.22	608.22	5.50
STA 24+54.89	STA 25+02.89	Type 7SW	14	10.75	609.28	606.28	6.00

STA 25+02.89	STA 25+50.89	Type 7SW	18	13.75	607.40	604.40	5.25
STA 25+50.89	STA 25+98.89	Type 7SW	20	15.25	604.94	601.94	6.25
STA 25+50.89	STA 26+86.99	Type 7SW	22	16.75	602.46	599.46	6.00

A minimum toe cover of 2.0 feet is recommended over the spread footings for Type 7SW walls.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section at RW LOL Station 26+90 was used to analyze the global stability. Based on subsurface information collected via Caltrans field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The fill material behind the wall is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. Underlying alluvial material possesses similar soil parameters. The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	In situ Density (lbs/ft^3)	Friction Angle (Degree)	Cohesion (psf)
silty sand with gravel (fill)	3.0	130	36	0
sand with gravel and silt (alluvium)	>50	130	36	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1.5H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill placed at 95 percent Relative Compaction. Locally deeper overexcavations may be necessary and should be determined by the engineer. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 1/16/12

A Mehrazar

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Transportation Engineer
Office of Geotechnical Design – South 1
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Supervised by: Date: 1/16/12

Shiva Karimi

REGISTERED PROFESSIONAL ENGINEER
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PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: April 6, 2012

File: 07-LA-5- PM 30.56/30.62
07-1218W1
Empire Interchange
Retaining Wall No. 25

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Addendum to Foundation Report for Retaining Wall No. 25, dated January 16, 2012

Based on the request of the Office of Structure Design, Branch 18, and Wall General Plan and Structural Plan (plotted October 7, 2011), Office of Geotechnical Design South 1 (OGDS1) prepared this addendum to foundation report to address minimum unbounded length of tiedowns for subject wall.

Based on the subsurface information provided in the LOTBs and Foundation Report for Retaining Wall No. 25, OGDS1 recommends a minimum unbounded length of 15 feet for proposed tiedowns at Retaining Wall No. 25.

CONSTRUCTION CONSIDERATIONS

1. Difficult drilling for tiedown installation is anticipated due to possibility of caving soils and cobbles. If caving occurs in drilling, the contractor should take appropriate measures to stabilize the holes.

2. Drilled excavations for tiedown anchors should not be left open overnight. Excavation, anchor installation and initial grouting should be performed within the same work shift to minimize potential caving.

3. Groundwater is not expected to be encountered during the construction.

4. The contractor is responsible for determining the bonded length for tiedown anchors.

5. A minimum of 5% of the anchors should be tested for performance. However, the number of performance tests should not be less than 3. The proof tests should be performed on the remaining tiedowns.

6. The tiedowns should be locked off at 100% of the design load.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 04/06/12

A Mehrazar

Akbar Mehrazar
Transportation Engineer
Office of Geotechnical Design – South 1
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Supervised by: Date: 04/06/12

B Shiva Karimi

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REGISTERED PROFESSIONAL ENGINEER
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District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: January 16, 2012

File: 07-LA-5- PM 30.54/30.6
07-1218W1
Empire Interchange
Retaining Wall No. 26

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 26

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 4, 2011 and Wall General Plan and Structure Plans (plotted October 4, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 26 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 26 will be constructed along eastbound Empire Avenue between post miles 30.54 and 30.6 of State Route 5 and next to Victory Place Separation Bridge No. 53C-2171. Retaining Wall No. 26 consists of Caltrans Standard Type 7SW retaining wall with tiedown, with chain link railing Type 7 at the top and a sidewalk at Empire Ave level. Retaining Wall No. 26 will predominantly retain existing alluvial soils above the proposed Empire Avenue cut section within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.26 (based on updated metric plans) dated April 23, 2007 (Revised May 18, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 26 consists of Caltrans Standard Type 7SW retaining wall with tiedown, with chain link railing Type 7 at the top and a sidewalk at Empire Ave level. Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 8 to 22 feet with a total length of 562.0 feet located from RW LOL Station 22+39.99 to Station 28+01.98 (approximate 48.19 ft Rt., Sta. 22+34.66 to 50.72 ft Rt., Sta.27+88.81 Centerline Empire Ave. “E Line”). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 and 2 for Retaining Wall No. 26. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 22+39.99	STA 22+64.24	Type 7SW	8	7.5	618.15
STA 22+64.24	STA 23+12.24	Type 7SW	8	7.5	616.85
STA 23+12.24	STA 23+60.24	Type 7SW	10	8.0	615.40
STA 23+60.24	STA 24+08.24	Type 7SW	12	9.25	613.75
STA 24+08.24	STA 24+56.24	Type 7SW	12	9.25	611.90
STA 24+56.24	STA 25+04.24	Type 7SW	14	10.75	610.00
STA 25+04.24	STA 25+52.24	Type 7SW	16	12.25	608.20

STA 25+52.24	STA 26+00.24	Type 7SW	18	13.75	606.40
STA 26+00.24	STA 26+64.24	Type 7SW	20	15.25	603.85
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Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from January 31 through March 31, 2006. The field investigation included drilling three 4.5-inch outer diameter mud rotary borings and four electronic cone penetration tests (CPT). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Empire Ave (Prop.) Stationing	Offset from Empire Ave "E Line" (ft)	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elev. (ft)	Depth (ft)	Date Drilled
06-100 CPT	21+18.02	77.11 Rt.	1616+96.50	894.66 Lt.	625.2	58.9	3/31/2006
06-101 CPT	23+51.27	105.78 Rt.	1614+57.20	756.30 Lt.	624.5	59.6	3/31/2006
06-102 CPT	25+53.98	119.23 Rt.	1612+92.73	593.14 Lt.	622.5	62.2	3/31/2006
06-95 CPT	23+79.16	45.33 Lt.	1615+57.02	630.99 Lt.	624.0	60.7	1/30/2006
06-92	29+07.17	104.08 Rt.	1611+12.61	314.29 Lt.	626.0	100.2	1/25-27/2006
06-94	19+20.45	6.01 Rt.	1619+37.93	937.02 Lt.	629.1	78.1	1/30-31/2006
06-96	28+12.47	25.20 Lt.	1612+69.41	295.64 Lt.	628.0	115.4	1/31/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	2
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	12

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

Proposed Retaining Wall 26 is bounded to the north by the north curb of Empire Ave., to the south by the Burbank Empire Center (mall), east by Victory Place, and west near Maria Street. Existing conditions are predominantly level ground where the proposed walls will be located. Only minor existing embankment (small hill) estimated to be approximately 5.0 ft height near the east end of proposed Retaining Wall 26 is present (at the northern grassy area of the Burbank Empire Center). The top and toe of the small confined embankment ranges between approximate elevations from 628.1 to 623.2 ft, respectively. The small embankment slope is grass covered with minor shrubs and trees in the area.

Based on OGDS1's 2006 foundation investigation, sediments at the proposed wall site consist of minor preexisting embankment fill and minor localized trench fill (approximately up to 7 feet thick, surrounding utilities beneath existing Empire Ave.) and fill (up to 2 feet thick, undifferentiated with road base material) beneath roadway pavement and base material. The above sporadic fills are underlain by Holocene alluvium (Qa unit of Dibblee, 1991a) and probable older undifferentiated Quaternary alluvial fan gravel derived from the Verdugo Mountains (Qf unit of Dibblee, 1991a) or older Pleistocene alluvium. Most recent deeper borings/penetration tests have likely terminated within older Pleistocene alluvium or fan gravel. The minor embankment fill was not sampled due to the restricted horizontal and vertical extent. Localized trench fill and undifferentiated fill beneath roadways are composed of estimated loose, sand with sporadic gravel and silty sand. Partially underlying and exposed at surface was undifferentiated Holocene or older fan gravel/Pleistocene alluvium which can be separated into approximately four units. The upper alluvial unit is composed of predominantly medium dense to minor loose, sand to silty sand with sporadic gravel and gravel lenses from elevations ranging from +624.8 and +621.9 ft down to elevations ranging from +614.7 and +610.1 ft. The underlying second alluvial unit, ranges between approximate elevations +614.7 and +610.0 ft down to approximate elevations ranging from +609.4 to +607.8 ft, consists predominantly of medium dense to dense (rare soft/very loose), sand and silty sand with sporadic gravel, and minor clayey sand. The underlying third alluvial unit, ranges between approximate elevations +609.4 to +607.8 ft down to approximate elevations +568.8 to +562.8 ft and generally consists of dense (minor loose/stiff to firm) sand to silty sand with sporadic gravel and cobbles and rare boulders (est. $\leq$12 in. length) interlensed with minor sandy silt, clayey sand, and rare sandy lean clay. The underlying lower alluvial unit ranges between approximate elevations +568.9 to +562.8 ft down to approximate elevation +512.6 ft, generally consists of very dense to dense/hard, silty sand to sand with gravel and cobbles, gravel/cobble lenses with sand matrix, and rare sandy lean clay. The deepest recent boring for the subject walls, Boring 06-96 (drilled January/February 2006, near the east end of the subject retaining walls) was drilled 115.4 ft below the surface to approximate elevation +512.6 ft. The LOTB should be consulted for more specific details.

Groundwater

Groundwater was not encountered during the recent field exploration. A perforated pipe was installed within nearby boring 05-46A (approximately 300 ft to the north and northeast of the walls along Southbound San Fernando Blvd.) and successive measurements taken during November 2005 and January 2006 revealed the boring to be dry to the bottom to 95.2 ft below surface (elevation +533.2 ft). All auger borings completed for the entire 2 mile long Empire Interchange project also showed no groundwater was encountered.

Ground water was also not encountered during the 1957 field investigation for the nearby existing Southbound San Fernando Blvd UC (Br. No. 53-1215, As Built LOTB plan dated June 1961) down to approximate elevation +559 ft the maximum penetration depth 63.3 ft obtained. Also no ground water was encountered on tape measured down to caving depth of 50 ft at elevation +568.7 ft within cone penetrometer boring B-1.

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 26

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
06-92	3.3 to 26.3	6400	8.0	105	30
06-92	29.5 to 52.5	7700	8.2	105	18
06-92	55.8 to 72.2	10000	8.2	105	15
06-92	75.4 to 105.0	5600	8.3	120	36
06-94	3.3 to 42.6	7700	8.6	90	6
06-94	45.9 to 68.9	5500	8.3	75	ND
06-94	7.2 to 95.1	6700	7.1	105	45
06-96	3.3 to 16.4	5000	8.0	120	57
06-96	19.7 to 39.4	3700	8.0	165	225
06-96	42.6 to 59.0	4100	8.3	315	540
06-96	62.3 to 88.6	3700	8.1	345	560
06-96	91.8 to 111.5	4900	8.1	90	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 mile east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Ground water was not encountered to at least a measured depth below the surface from 95.2 ft, last measured January 09, 2006 within nearby boring 05-46A. Also, soils are predominantly medium dense to dense granular material and minor firm/stiff clays and clayey sands. Bottom foundation depths will be significantly above the measured dry bottom of boring depths.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

Caltrans Standard Type 7SW retaining wall with tiedown is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 7SW spread footings are recommended for retaining wall support as existing soils with minor remediation described below are adequate to support the wall. Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Existing poor quality soils beneath a portion of the wall footprint (from Retaining Wall LOL station 24+06.89 to 26+86.99) will be replaced with structure backfill compacted to 95% R.C. (relative compaction) per ASTM D1557 test method. Remedial treatment consists of overexcavating existing soils within the specified limits to 3 ft below footing grade and replacing these soils with structure backfill compacted to 95% R.C. up to footing grade. The horizontal limits of the structure backfill prism should extend from a line 1 ft beyond the retaining wall's toe and heel and then down and out at a 1.5:1 (H:V) slope to a depth equal to the bottom of the subexcavation. Refer to the Caltrans Standard Specifications (2006), section 19-5.03 for details.

Table 5– Spread Footing Data for Retaining Wall 26

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Over-Excavation Elevation (ft)	Ultimate Bearing Capacity Required (ksf)
From	To						
STA 22+39.99	STA 22+64.24	Type 7SW	8	7.5	618.15	N/A	3.75
STA 22+64.24	STA 23+12.24	Type 7SW	8	7.5	616.85	N/A	3.75
STA 23+12.24	STA 23+60.24	Type 7SW	10	8.0	615.40	N/A	4.25
STA 23+60.24	STA 24+08.24	Type 7SW	12	9.25	613.75	610.75	5.50
STA 24+08.24	STA 24+56.24	Type 7SW	12	9.25	611.90	608.90	5.50
STA 24+56.24	STA 25+04.24	Type 7SW	14	10.75	610.00	607.00	6.00
STA 25+04.24	STA 25+52.24	Type 7SW	16	12.25	608.20	605.20	5.75

STA 25+52.24	STA 26+00.24	Type 7SW	18	13.25	606.40	603.40	5.25
STA 26+00.24	STA 26+64.24	Type 7SW	20	15.25	603.85	600.85	6.25
STA 26+64.24	STA 27+28.24	Type 7SW	22	16.75	602.15	599.15	6.00
STA 27+28.24	STA 28+01.98	Type 7SW	22	16.75	601.10	598.10	6.00

A minimum toe cover of 2.0 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section at approximate RW LOL Station 28+00 was used to analyze the global stability. Based on subsurface information collected via Caltrans field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The retained fill material is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. Underlying alluvial material possesses similar soil parameters. The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	In situ Density (lbs/ft^3)	Friction Angle (Degree)	Cohesion (psf)
silty sand with gravel (fill)	3.0	130	36	0
sand with gravel and silt (alluvium)	>50	130	36	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1.5H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill placed at 95 percent Relative Compaction. Locally deeper overexcavations may be necessary and should be determined by the engineer. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 1/16/12

A Mehrazar

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Supervised by: Date: 1/16/12

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cc: GS Corporate – Shira Rajandra (Electronic File)
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PCE (District 07) – Jan Rutenbergs (Electronic File)
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District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: April 3, 2012

File: 07-LA-5- PM 30.54/30.6
07-1218W1
Empire Interchange
Retaining Wall No. 26

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Addendum to Foundation Report for Retaining Wall No. 26, dated January 16, 2012

Based on the request of the Office of Structure Design, Branch 18, and Wall General Plan and Structural Plan (plotted October 4, 2011), Office of Geotechnical Design South 1 (OGDS1) prepared this addendum to foundation report to address minimum unbounded length of tiedowns for subject wall.

Based on the subsurface information provided in the LOTBs and Foundation Report for Retaining Wall No. 26, OGDS1 recommends a minimum unbounded length of 15 feet for proposed tiedowns at Retaining Wall No. 26.

CONSTRUCTION CONSIDERATIONS

1. Difficult drilling for tiedown installation is anticipated due to possibility of caving soils and cobbles. If caving occurs in drilling, the contractor should take appropriate measures to stabilize the holes.

2. Drilled excavations for tiedown anchors should not be left open overnight. Excavation, anchor installation and initial grouting should be performed within the same work shift to minimize potential caving.

3. Groundwater is not expected to be encountered during the construction.

4. The contractor is responsible for determining the bonded length for tiedown anchors.

5. A minimum of 5% of the anchors should be tested for performance. However, the number of performance tests should not be less than 3. The proof tests should be performed on the remaining tiedowns.

6. The tiedowns should be locked off at 100% of the design load.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 04/03/12

A Mehrazar

Akbar Mehrazar
Transportation Engineer
Office of Geotechnical Design – South 1
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Supervised by: Date: 04/03/12

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District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: February 22, 2012

File: 07-LA-5- PM 30.09/30.32
07-1218W1
Empire Interchange
Retaining Wall No.1595

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1595

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 16, 2011 and Wall General Plan and Structure Plans (dated February 16, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1595 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1595 will be constructed along southbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.09 and 30.32 and will retain proposed railroad track above freeway level. Retaining Wall No. 1595 is a Type 1RR (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the Route 5 level to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.487A. (based on updated metric plans) dated April 2, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1595 will be constructed along the western edge of southbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.09 and 30.32 and will retain proposed railroad track above freeway level. Retaining Wall No. 1595 is a Type 1RR, Case 1 (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the Route 5 level. Based on the information provided by Office of Structure Design, the minimum horizontal distance from top of Retaining Wall 1595 to the nearest railroad track centerline (retained by RW1595) is 13.0 feet.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 14 feet with an approximate length of 1136.6 feet located from RW LOL Station 589+03.43 to Station 600+40.04 (108.11 ft Lt. of Sta. 1589+03.43 to 112.83 ft Lt. of Sta. 1600+38.70 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 5 for Retaining Wall No. 1595. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 589+03.43	STA 589+28.19	Type 1RR	6	9.0	597.0
STA 589+28.19	STA 589+52.19	Type 1RR	8	11.0	596.75
STA 589+52.19	STA 590+96.19	Type 1RR	10	12.5	596.5

STA 590+96.19	STA 592+40.19	Type 1RR	12	13.5	596.5
STA 592+40.19	STA 593+36.19	Type 1RR	14	15.0	596.5
STA 593+36.19	STA 594+80.19	Type 1RR	14	15.0	598.0
STA 594+80.19	STA 595+76.19	Type 1RR	14	15.0	600.0
STA 595+76.19	STA 596+72.19	Type 1RR	14	15.0	602.0
STA 596+72.19	STA 597+68.19	Type 1RR	14	15.0	604.0
STA 597+68.19	STA 598+16.19	Type 1RR	10	12.5	607.0
STA 598+16.19	STA 598+64.19	Type 1RR	10	12.5	609.0
STA 598+64.19	STA 599+60.19	Type 1RR	8	11.0	611.25
STA 599+60.19	STA 600+08.19	Type 1RR	8	11.0	613.25
STA 600+08.19	STA 600+40.04	Type 1RR	6	9.0	615.5

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 10, 2005 through January 19, 2006. The field investigation included drilling six 8-inch outer diameter hollow-stem auger borings and two Cone Penetration Tests (CPTs). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-22	1601+37.85	59.62 Lt.	623.9	61.5	8/10-11/2005
05-23	1593+01+88	63.86 Lt.	603.1	65.9	8/11-12/2005
05-24	1585+38.71	58.61 Lt.	601.2	66.5	8/11-12/2005
05-53CPT	1596+90.11	60.87 Lt.	609.6	60.0	10/13/2005
05-54CPT	1589+78.20	60.98 Lt.	600.7	63.2	10/13/2006
06-72	1591+45.29	194.62 Lt.	597.4	60.9	1/19/2006
06-73	1594+96.96	186.38 Lt.	603.7	61.5	1/9/2006
06-74	1599+93.44	189.00 Lt.	612.0	62.0	1/10/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	4
Moisture Content	CTM 212, 226	4
Corrosion	CTM 417, 422, 643,532	11

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately from 3 to 12 feet of fill generally composed of loose to dense silty sand with varying amounts of gravel. Underlying alluvium is typically composed of loose to very dense silty sands with gravel, poorly and well graded sands with varying amounts of gravel and silt, medium stiff to very stiff sandy clays and loose to medium dense sandy silts.

Groundwater

Groundwater was not encountered in borings drilled for this study to maximum depths of 66.5 feet (elevation +534.7). In the vicinity, DWR wells (01N14W03F03S and 01N14W03F06S) located near Buena Vista Street/Winona Avenue intersection show groundwater measurements below the surface vary from 211.8 to 167.5 ft depth corresponding to approximate elevations +471.2 to historically high +515.5 ft NAVD 88. No dates were provided but the wells had 35 to 14 measurements taken.

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1595

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-22	4.9 - 9.8	6600	9.4	75	12
05-22	9.8-15.0	1200	8.6	30	3
05-22	15.0-36.4	9000	8.7	117	45
05-22	36.4-35.1	7000	8.9	45	36
05-22	40.0-61.3	6000	8.8	30	138
05-23	9.9- 15.1	4800	8.2	75	12
05-23	30.0-35.3	2700	8.3	90	5
05-24	5.0-9.8	1000	7.9	75	123
05-24	15.1-20.1	2200	9.3	30	0
05-24	25.0-46.9	5100	8.8	45	15
05-24	50.2-67.0	5100	8.5	60	18
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in borings drilled for this study to maximum depths of 66.5 feet (elevation +534.7). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1RR (AREMA, 2005) retaining wall is considered the best solution for retaining soils for proposed railroad track above I-5 Freeway at this location. A Type 1RR wall (Case I) on spread footings can be used for the proposed construction provided that the recommendations presented in Construction Considerations section of this report are followed.

Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1595

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Over Excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 589+03.43	STA 589+28.19	6	9.0	597.0	591.0	1.97
STA 589+28.19	STA 589+52.19	8	11.0	596.75	590.75	2.18
STA 589+52.19	STA 590+96.19	10	12.5	596.5	590.5	2.60
STA 590+96.19	STA 592+40.19	12	13.5	596.5	590.5	3.13
STA 592+40.19	STA 593+36.19	14	15.0	596.5	590.5	3.31
STA 593+36.19	STA 594+80.19	14	15.0	598.0	592.0	3.31
STA 594+80.19	STA 595+76.19	14	15.0	600.0	594.0	3.31
STA 595+76.19	STA 596+72.19	14	15.0	602.0	596.0	3.31
STA 596+72.19	STA 597+68.19	14	15.0	604.0	598.0	3.31
STA 597+68.19	STA 598+16.19	10	12.5	607.0	601.0	2.60
STA 598+16.19	STA 598+64.19	10	12.5	609.0	603.0	2.60

STA 598+64.19	STA 599+60.19	8	11.0	611.25	605.25	2.18
STA 599+60.19	STA 600+08.19	8	11.0	613.25	607.25	2.18
STA 600+08.19	STA 600+40.04	6	9.0	615.5	609.5	1.97

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 2.0 feet is recommended over the spread footings.

Settlement

The settlement is anticipated to be about 1 inch for walls founded on compacted soil as described in Table 5. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program PCSTABLm2/STED under both static and pseudo-static conditions. The soil profile and the strength parameters used in performing the stability analysis as developed from the subsurface investigation, are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. For the analysis, it was assumed that the wall is founded on shallow footings.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Fill	6 to 20	36	130	0
Alluvium	>50	34	130	0

The stability analysis was performed for 2 cross sections with wall heights 10 and 14 (the maximum height of the proposed wall) to evaluate the global stability under static and design seismic conditions, respectively. Based on the information provided by the Office of Structure Design, the railroad loading was considered to be a 1200 psf load acting on a 14-foot wide strip located along the railroad track. The stability under the design seismic conditions was evaluated using a pseudostatic analysis with a horizontal acceleration of 0.15g.

The results of the stability analysis are summarized in Table 7 below.

Table 7 – Summary of Slope Stability Analysis

Cross Section (Station)	Wall Ht (feet)	Distance to the CL of Track (feet)*	FOS	
			Static	Pseudostatic
590+00	10	13	2.0	1.6
597+00	14	13	2.1	1.7

Note: * - Distances to the track were provided by the Office of Structure Design.

The results of the stability analysis indicate that the wall segments will have FOS greater than 1.5 and 1.1 under static and design seismic conditions, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction per ASTM 1557 test method.

7. A minimum over-excavation of 6 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. However, the compaction standard used should be the ASTM 1557 test method.

If you have any questions, please call Akbar Mehrazar at (949) 440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by:	Date: 02/22/2012	Supervised by:	Date: 02/22/2012

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cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

M e m o r a n d u m

Flex your power!
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To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: January 26, 2012

File: 07-LA-5- PM 30.32/30.44
07-1218W1
Empire Interchange
Retaining Wall No.1601

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1601

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated January 25, 2012 and Wall General Plan and Structure Plans (plotted January 20, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1601 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1601 will be constructed along southbound I-5, south of Empire Avenue Undercrossing, Bridge No. 53-2920 between post miles 30.32 and 30.44 and will retain freeway above the proposed railroad track level. Retaining Wall No. 1601 is a Caltrans Standard Type 1 retaining wall with Type 736A concrete barrier on top, and will be constructed to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1601 will be constructed along southbound I-5 and will consist of a Standard Type 1 retaining wall with a concrete barrier Type 736A at freeway level. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I plus 2 foot level surcharge) to accommodate the freeway widening.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 4 to 8 feet with an approximate length of 485.9 feet located from RW LOL Station 600+40.0 to Station 605+25.93 (approximately 108.11 ft Lt of Sta. 1600+37.05 to 138.36 Lt of Sta. 1605+18.48 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 3 for Retaining Wall No. 1601. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 600+40.0	STA 601+34.37	Type 1	6	4.25	616.17
STA 601+34.37	STA 601+82.37	Type 1	6	4.25	617.67
STA 601+82.37	STA 603+74.37	Type 1	8	5.25	617.67
STA 603+74.37	STA 604+22.37	Type 1	8	5.25	618.17
STA 604+22.37	STA 604+70.37	Type 1	6	4.25	619.17
STA 604+70.37	STA 604+94.37	Type 1	6	4.25	620.17
STA 604+94.37	STA 605+25.93	Type 1	4	3.25	620.17

Caltrans 2004 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2004 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 10, 2005 through January 01, 2006. The field investigation included drilling two 8-inch outer diameter hollow-stem auger and one 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
06-74	1599+93.44	189.00 Lt.	612.0	62.0	1/10/2006
05-22	1601+37.85	59.62 Lt.	623.9	61.5	8/10-11/2005
05-21A	1605+68.6	63.7 Lt	636.4	61.5	8/10/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	1
Moisture Content	CTM 212, 226	1
Corrosion	CTM 417, 422, 643,532	5

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately 5 to 15 feet of fill generally composed of loose to dense silty sands with coarse gravel. Underlying alluvium is composed of loose to very dense silty sands, sandy silts with gravel and cobbles, and stiff to very stiff sandy lean clay with minor gravel with sand interbeds. Existing ground is relatively flat.

Groundwater

Groundwater was not encountered in auger Boring Nos. 06-74 and 05-22 drilled for this study to maximum depths of 62.0 feet (dry down to at least elevation +550 ft).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Corporation soils laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1601

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-22	4.9 - 9.8	6600	9.4	75	12
05-22	9.8-15.0	1200	8.6	30	3
05-22	15.0-36.4	9000	8.7	117	45
05-22	36.4-35.1	7000	8.9	45	36
05-22	40.0-61.3	6000	8.8	30	138
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable
Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 62 feet below the surface (elevations +550 ft). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 spread footings are recommended for retaining wall support as existing soils are adequate to support the wall (Case I: 2 ft level surcharge). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1601

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Over excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 600+40.0	STA 601+34.37	6	4.25	616.17	613.17	1.9
STA 601+34.37	STA 601+82.37	6	4.25	617.67	614.67	1.9
STA 601+82.37	STA 603+74.37	8	5.25	617.67	614.67	2.2
STA 603+74.37	STA 604+22.37	8	5.25	618.17	615.17	2.2
STA 604+22.37	STA 604+70.37	6	4.25	619.17	616.17	1.9
STA 604+70.37	STA 604+94.37	6	4.25	620.17	617.17	1.9
STA 604+94.37	STA 605+25.93	4	3.25	620.17	617.17	1.7

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 01/26/12

A Mehrazar

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cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: January 23, 2011

File: 07-LA-5- PM 30.33/30.44
07-1218W1
I-5 Empire Interchange
Retaining Wall No.1604

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1604

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 12, 2011 and Wall General Plan and Structure Plans (plotted January 4, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1604 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1604 will be constructed along northbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.33 and 30.44. Retaining Wall No. 1604 is a Caltrans standard Masonry Block Sound Wall on Type 736A (MOD) concrete barrier at freeway level supported by Type 1SWB retaining wall with Type 60 concrete barrier at Scott Road off-ramp level. Retaining Wall No. 1604 will be located near base of the existing I-5 slope and west of the proposed Scott Road northbound off-ramp to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;

c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1604 will be located on the north side of northbound Route 5 and west of the proposed Scott Road NB off-ramp. The structure will consist of a masonry block wall with a Type 736A concrete barrier along the top, at roadway level supported by Type 1SWB retaining wall with Type 60 concrete barrier at Scott Rd. off-ramp level. The proposed wall is located near the base of the existing I-5 embankment slope and will retain proposed embankment fill to accommodate the off-ramp improvements.

Several utility lines including a 6-inch diameter abandoned water line, 24-inch Corrugated Metal Pipe (CMP), and 14-ft diameter sanitary sewer line are located near/under proposed wall foundation. Existing Burbank Western Flood Control Channel (LACFCD) is located east of the proposed Retaining Wall No. 1604. Applied load from proposed Retaining Wall No. 1604 to the channel structure and utility lines will be evaluated in an addendum upon receiving detailed plans from Structure Design.

In addition, slope stability analysis yields a factor of safety less than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively, from RW LOL Sta. 601+00 (begin wall) to approximate RW LOL Sta. 603+00. OGDS1 recommends that alternative design such as using pile foundation be considered for the above mentioned segment or entire wall.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 24 feet with an approximate length of 569.25 feet located from RW LOL Station 601+00 to Station 606+69.22 (approximate 97.28 ft. Rt. of Sta. 1600+99.42 to 97.02 ft Rt. of Sta. 1606+73.26 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 4 for Retaining Wall No. 1604. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Design Height (ft)	Max. Sound Wall Design Height (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)
From	To				
STA 601+00.00	STA 601+72.00	6	16	7.75	616.75
STA 601+72.00	STA 602+44.00	8	16	8.00	616.75
STA 602+44.00	STA 603+16.00	10	16	8.75	616.75
STA 603+16.00	STA 603+72.00	12	16	9.75	616.75
STA 603+72.00	STA 604+44.00	14	16	10.75	616.75
STA 604+44.00	STA 605+00.00	16	16	12.00	615.25
STA 605+00.00	STA 605+40.00	18	16	13.00	615.25
STA 605+40.00	STA 605+80.00	20	16	14.25	614.25
STA 605+80.00	STA 606+28.00	22	16	15.25	612.75
STA 606+28.00	STA 606+69.25	24	16	16.50	611.25

The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies for Retaining Wall No 490 were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from June 16, 2004 to August 17, 2005. The field investigation included drilling two 8-inch outer diameter hollow-stem auger and three 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5-foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used for all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-6	1599+01.17	131.57 Rt.	610.3	66.5	7/19/05
04-2	1601+59.37	155.48 Rt.	613.1	61.0	6/16/04
05-30	1602+42.30	85.18 Rt.	624.8	36.5	8/16/05
05-34	1605+91.54	60.43 Rt.	635.4	36.5	08/16-17/05
05-7	1606+41.69	137.26 Rt.	621.0	101.2	07/20-21/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	3
Moisture Content	CTM 212, 226	3
Corrosion	CTM 417, 422, 643,532	12

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The current location of proposed Wall 1604 is an embankment. The existing embankment has an approximate slope of 1V:2H and is vegetated.

The boring logs were used to develop a continuous soil profile with depth for the wall location. The upper 1 to 15 feet of the borings were logged as fill (elevation +605 to +620 ft). The fill was generally composed of from loose to dense silty sand with gravel. The top of native material was logged at an elevation of about +605 to +620 feet in the borings. The native alluvium was composed of from loose to very dense silty sand/sandy silt, and sand, and stiff sandy lean clay, with gravel lenses and cobbles throughout.

Groundwater

Groundwater was not encountered in exploratory borings drilled for this study to maximum depth of 101.2 feet (elevation +519.8 ft).

SCOUR

There is no scour potential at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1604

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-6	0-20	6500	8.8	60	21
05-6	20-35	5100	8.8	45	0
05-6	35-50	5000	8.9	45	6
05-6	50-65	5400	8.7	45	0
05-7	0-20	3800	8.6	75	63
05-7	20-40	5500	8.6	45	9
05-7	40-60	4400	8.0	45	0
05-7	60-80	7400	7.4	45	33
05-7	80-100	7100	8.3	60	24
05-30	2-4	3900	8.7	15	186
05-30	22-24	4100	8.9	60	141
05-34	5-16.5	4400	9.2	30	30
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w=6.75, is the nearest seismic source from the project site. Based on Weber (1980), this reverse type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered to at least a depth of 101.2 feet (dry down to at least elevation +519.8 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A sound wall on Type 1SWB retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1SWB wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall with some earthwork. Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacity were calculated for subsurface soils at the project site. The results are summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1604

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. of Sub-excavation Elevation (ft)	Ultimate Bearing Capacity Required (k/sf)
From	To					
STA 601+00.00	STA 601+72.00	6	7.75	616.75	NA	NA
STA 601+72.00	STA 602+44.00	8	8.00	616.75	NA	NA
STA 602+44.00	STA 603+16.00	10	8.75	616.75	NA	NA
STA 603+16.00	STA 603+72.00	12	9.75	616.75	613.75	6.6
STA 603+72.00	STA 604+44.00	14	10.75	616.75	613.75	7.3
STA 604+44.00	STA 605+00.00	16	12.00	615.25	612.25	8.1
STA 605+00.00	STA 605+40.00	18	13.00	615.25	612.25	9.0
STA 605+40.00	STA 605+80.00	20	14.25	614.25	611.25	9.9
STA 605+80.00	STA 606+28.00	22	15.25	612.75	609.75	11.3
STA 606+28.00	STA 606+69.25	24	16.50	611.25	608.25	12.2

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the new fill embankment slope was evaluated using the computer program XSTABLE version 5 under both static and pseudo-static conditions. Critical cross sections were analyzed for the global stability. Based on subsurface information collected via our field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. From approximate RW LOL Sta. 603+00 to Sta. 606+69.25 (end wall), the result yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Structural Backfill	32	120	0
Native Soil	32	120	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

3. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May, 2006). Imported materials used to construct the new fill embankment should be tested during grading to assess their expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft. of the subgrade elevation.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. Complete removal and re-compaction of compressible loose materials below spread footing are required prior to fill placement in order to expose firm and unyielding ground. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Akbar Mehrazar at (949) 440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 01/23/12

A Mehrazar

Akbar Mehrazar
Engineering Geologist
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Supervised by: Date: 01/23/12

Shiva Karimi

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District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: April 20, 2012

File: 07-LA-5-PM 30.33/30.44
07-1218W1
I-5 Empire Interchange
Retaining Wall No.1604

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1 (OGDS1)
Branch D

Subject: Addendum to Foundation Report for Retaining Wall 1604, dated January 23, 2012

The Office of Geotechnical South 1 (OGDS1) prepared this addendum to the referenced Foundation Report to present the geotechnical recommendations for the proposed revised foundation types. The Wall 1604 alignment traverses along an alignment that deviates gradually from the channel alignment. The original Foundation Report was prepared to provide recommendations for shallow footings. However, due to the potential impact on the channel wall due to footing pressure, the Office of Structure Design, Branch 18 (OSD) in communication with OGDS1 proposed using 24-inch diameter cast-in-drilled-hole (CIDH) piles to support the retaining wall segment within a 1V:1H projection from the edge of the channel bottom. The General Plan and Structural Details (dated 4-19-2012) provided by OSD indicate that the wall segments from Station 601+00 to Station 602+44 (RW LOL) will be founded on piles, and from Station 602+44 to Station 606+69.25 (RW LOL) will be supported on shallow footings. Foundation recommendations provided in this addendum address wall segment founded on piles, and are supplemental to the recommendations presented in the referenced report.

The proposed piles are located at distances that are greater than 9 feet (more than 3 times the pile diameter) from the channel wall. Therefore, it was assumed that the piles would not exert either lateral or vertical stresses on the adjacent flood control channel wall. Consequently, OGDS1 does not recommend using any special construction methods to isolate piles from the surrounding subsurface materials.

The analysis was performed assuming that the piles would be constructed using a wet method and therefore would not have end resistances. The computer programs SHAFT, Version 5.0 and LPILE, Version 5.0 were used for the vertical and lateral analyses, respectively. The geotechnical profile for the analysis was developed using the subsurface information obtained from Borings 04-2 and 05-30, drilled for the project.

Pile Data Table for Retaining Wall 1604

Wall Segment	Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Lateral Load (Kips)	Pile Cut-off Elevation (ft)	Design Pile Tip Elevation (ft)	Specified Pile Tip Elevation (ft)
			Compression (kip)	Tension (kip)				
6-ft High Wall	CIDH/ 24- inches	69	138	32	11.0	616.75	579.75(a) 586.75 (b) 590.75©	579
8-ft High Wall	CIDH/ 24 inches	84	168	42	15.5	616.75	575.75(a) 581.75(b) 585©	575

Notes: Design Tip is controlled by the following demands:
(a) Nominal Resistance in Compression (b) Nominal Resistance in Tension. (c) Lateral Loading..

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated in the footing excavation.

2. The potential for caving exists in drilled holes for CIDH piles over their entire lengths. Due to the proximity of the piles to the channel wall, there is a potential for the caving to propagate all the way to the channel wall. The contractor should use an appropriate drilling method to minimize the caving.

3. Concrete placement for construction of the CIDH pile should be completed within the same day that excavation of the drilled hole has been completed.

4. Drilling during construction may be variable and sporadically hard (within gravel zones) down to anticipated pile tip elevations.

5. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications.

6 Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1:V:1H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations

If you have any questions, please call Gamini Weeratunga at (949) 440-3427 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 04-20-12

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Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: December 20, 2011

File: 07-LA-5- PM 30.35/30.48
07-1218W1
Empire Interchange
Retaining Wall No.1605

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1605

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 9, 2011 and Wall General Plan and Structure Plans (plotted November 1, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for the proposed Retaining Wall No. 1605 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1605 will be constructed along southbound I-5, south of Empire Avenue Undercrossing (Br. 53-2920) between post miles 30.35 and 30.48. Retaining Wall No. 1605 is a Caltrans Standard Type 1 retaining wall and will be constructed to retain I-5 southbound above the proposed Empire Avenue southbound on-ramp within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No. 485 (based on updated metric plans) dated September 28, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Route 5 (one lane in each direction).

Retaining Wall No. 1605 will be located on the southbound (SB) side of Route 5 at the Empire Avenue S/B On-Ramp. The structure will consist of a Type 1 wall with a type 736A concrete barrier along the top, at roadway level. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I: plus 2 foot level surcharge) to accommodate the freeway widening.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 30 feet with an approximate length of 647.33 feet located from RW LOL Station 602+00.77 to Station 608+48.12 (approximate 91.06 ft Lt of Sta. 1601+98.46 to 96.97 ft Lt of Sta. 1608+01.84 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1, 2 and 3 for Retaining Wall No. 1605. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

Approximate RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)
From	To			
STA 602+00.77	STA 602+68.69	6	4.25	620.75
STA 602+68.69	STA 603+34.19	8	5.25	620.75
STA 603+34.19	STA 604+04.60	10	6.25	620.75
STA 604+04.60	STA 604+68.12	12	7.25	620.75
STA 604+68.12	STA 605+64.12	16	9.0	618.58
STA 605+64.12	STA 606+60.12	20	11.0	616.50
STA 606+60.12	STA 607+32.12	24	13.25	614.17
STA 607+32.12	STA 608+04.12	26	14.25	611.00
STA 608+04.12	STA 608+48.12	30	16.75	606.75

Caltrans 2004 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2004 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 10, 2005 to January 11, 2006. The field investigation included drilling three 8-inch outer diameter hollow-stem auger and two 4.5-inch mud rotary borings. Standard Penetration Tests (SPT's) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans Drilling Services and URS Corp Drilling operated drill rigs were used at boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
5-22	1601+37.85	59.62 Lt.	623.9	61.5	8/10-11/2005
6-75	1603+90.06	211.01 Lt.	615.5	61.5	1/10/2006
5-21A	1605+68.60	63.74 Lt.	636.4	61.5	8/10/2005
6-76	1607+84.01	248.27 Lt.	619.8	70.8	1/11/2006
5-47 P-S	1609+02.93	138.18 Lt.	624.0	181.0	9/27-30/2005

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	4
Moisture Content	CTM 212, 226	3
Corrosion	CTM 417, 422, 643,532	17

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Route 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

Proposed Retaining Wall No. 1605 is bounded to the northwest by Southbound San Fernando Blvd. (future Empire Ave.), northeast by I-5, to the southwest by SCRRA Railroad, and will begin across from Empire Center Drive. Existing I-5 embankment ranges from approximately 20.6 ft near Southbound San Fernando Blvd. (future Empire Ave.) to 9.8 ft near the south end of the proposed wall. Existing embankment side slopes have a 2(H):1(V) gradient. Embankment slopes are partially shrub and leaf covered with trees at the base of the slope.

Based on OGDS1's 2005/2006 foundation investigation, sediments at the proposed wall site consist of preexisting embankment fill (approximately 7 to 17 ft thick) underlain by alluvium. Embankment fill consists predominantly of dense to medium dense, silty sand with sporadic gravel interlayered with sand. Underlying alluvium can be separated into approximately two units. The upper alluvial unit is composed of predominantly very loose to loose and minor medium dense, sand to silty sand with sporadic gravel and minor scattered cobbles (up to 5 inch diameter) interbedded with lenses of coarse gravel with scattered cobbles from elevations ranging from 623 and 611 ft down to elevations ranging from 609 and 597 ft. The underlying second alluvial unit, ranges between approximate elevations 609 and 597 ft down to approximate elevation of 443 and consists predominantly of medium dense to very dense silty sand and sand with sporadic gravel, sandy silt to silt, minor clayey sand and stiff sandy lean clay, gravel/cobble lenses.

Groundwater

Groundwater was not encountered during the 2005/2006 field exploration to elevation +443 ft (181 ft depth).

Ground water was also not encountered during the 1957 field investigation for the nearby existing southbound San Fernando Blvd UC (Br. No. 53-1215, As Built LOTB plan dated June 1961) down to approximate elevation +559 ft the maximum penetration depth (63.3 ft) obtained. Also no ground water was encountered on tape measured down to caving depth of 50 ft at elevation +568.7 ft within cone penetrometer boring B-1.

SCOUR

There is no scour potential at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1605

Boring No./Sample No.	Depth Intervals (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-22	4.9 to 11.5	6600	9.4	75	12
05-22	11.5 to 15.1	1200	8.6	30	3
05-22	15.1 to 31.5, 35.1 to 36.4	9000	8.7	117	45
05-22	31.5 to 35.1	7000	8.9	45	36
05-22	40.0 to 61.3	6000	8.8	30	138
06-75	0-61.5	10200	8.4	90	ND
05-47	5.9 to 21.0	1900	8.1	60	66
05-47	21.0 to 41.0	5500	8.5	60	105
05-47	41.0 to 62.7	5700	8.1	45	ND
05-47	62.7 to 85.0	8200	8.6	60	33
05-47	85.0 to 101.0	10000	8.0	75	30
05-47	101.0 to 110.9	9000	7.8	60	ND
05-47	110.9 to 123.0	10000	7.1	45	12
05-47	123.0 to 125.0	5200	7.9	60	ND
05-47	128.9 to 129.9	3100	8.2	75	ND
05-47	136.5 to 139.1	3400	7.6	75	48
05-47	142.7 to 143.0	Not tested	6.9	75	ND
Corrosion Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

Seismicity

Proposed Retaining Wall No. 1605 is located approximately 0.6 miles southwest of the active Verdugo fault (VDO), a reverse/oblique fault, which has a maximum credible earthquake moment magnitude (Mw) of 6.75 based on the Caltrans California Seismic Hazard Map. According to Dr. Mohammed Islam (Caltrans OGDS1 Senior Seismic Specialist, E-mail correspondences received March 23, 2006 and September 16, 2005) peak horizontal bedrock acceleration at the site is estimated to be about 0.8g based on Sadigh et al (1997) attenuation relationships and the corresponding Peak Ground Acceleration (design PGA to be used for the retaining walls) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Ground water was not encountered to at least a measured depth of 95.2 ft (last measured January 09, 2006 within nearby boring 05-46A). Also, soils are dominantly medium dense to dense granular material and stiff to hard clays and silts. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall (Case I: Level plus 2 feet surcharge) with some earthwork. Based on results of laboratory testing and average corrected SPT “N” values obtained from the field investigation, ultimate bearing capacity were calculated for subsurface soils at the project site. The results are summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1605

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. Of Sub-excavation Elev. (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 602+00.77	STA 602+68.69	6	4.25	620.75	N/A	1.9
STA 602+68.69	STA 603+34.19	8	5.25	620.75	N/A	2.2
STA 603+34.19	STA 604+04.60	10	6.25	620.75	617.75	2.5
STA 604+04.60	STA 604+68.12	12	7.25	620.75	617.75	2.8
STA 604+68.12	STA 605+64.12	16	9.0	618.58	615.58	3.5
STA 605+64.12	STA 606+60.12	20	11.0	616.50	613.50	4.3
STA 606+60.12	STA 607+32.12	24	13.25	614.17	611.17	4.9
STA 607+32.12	STA 608+04.12	26	14.25	611.00	608.00	5.3
STA 608+04.12	STA 608+48.12	30	16.75	606.75	603.75	6.3

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

Settlement

Differential settlement of the foundations will be acceptable and within tolerance (1V:500H for CIP concrete retaining walls) after the required remedial treatment (subexcavation and replacement) of a portion of the existing material beneath the wall spread footing footprint.

SLOPE STABILITY

The global stability of the new fill and existing embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section at approximate Wall LOL Station 608+26 (RW LOL Sta. 90+20 Metric units) was used to analyze the global stability. Based on subsurface information from the recent field investigation, the soil profile with corresponding strength parameters used in performing the stability analysis are given in Table No. 6 below. The proposed fill material is assumed to have a friction angle of 32 degrees and unit weight of 120 pcf, based on material compacted to at least 90 percent relative compaction. The slope stability analysis yielded a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static (global) stability and seismic condition, respectively.

Table 6 - Soil Parameters for Slope Stability Analysis

Idealized Soil Type	Approximate Top of Layer Elevation (ft)	Approx. Thickness (ft)	Approx. Unit Weight (pcf)	Average Friction Angle (degrees)	Cohesion (psf)
silty sand with gravel (fill)	639 to 611	7 to 17	120	32	0
silty sand, sand, and gravel (alluvium)	620 to 597	60	120	32	0

CONSTRUCTION CONSIDERATIONS

1. Foundation excavations should be cleaned of loose debris.

2. Should any large rock fragments, rebar, or other debris be found at the bottom of footing elevations, the contractor should be prepared to remove and replace them with either granular material compacted to 95% R.C. or lean concrete.

3. A minimum soil cover of 2 ft is required over the retaining wall footings.

4. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft of the roadbed subgrade elevation.

5. Free water shall not be allowed to stand in any excavations. If excavations become flooded, at least the bottom 6 inch of soils shall be removed and replaced or recompacted per Caltrans specifications.

6. Based on soil types encountered during the recent investigation, OGDS1 recommends a slope ratio of 1V:1H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations for the proposed spread footing.

7. Complete removal and re-compaction of compressible loose materials below spread footing are required prior to fill placement in order to expose firm and unyielding ground. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Akbar Mehrazar (949) 440-2315 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 12/20/11

A Mehrazar

Akbar Mehrazar
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervised by: Date: 12/20/11

P Shiva Karimi

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Shiva Karimi, Ph.D, G.E.
Branch Chief
Office of Geotechnical Design – South 1
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cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbie Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: December 20, 2011

File: 07-LA-5- PM 30.39/30.41
07-1218W1
I-5 Empire Interchange
Retaining Wall No.1606

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1606

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 5, 2011 and Wall General Plan and Structure Plans (plotted December 5, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1606 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1606 will be constructed along northbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.39 and 30.41. Retaining Wall No. 1606 is a Caltrans Standard Type 5 retaining wall and will be located near the base of the proposed Scott Road northbound Off-Ramp slope to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1606 will be located on the north side of northbound Route 5 at the Scott Road NB Off-Ramp. The structure will consist of a Type 5 wall with a Type 736A concrete barrier along the top, at roadway level. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I: plus 2 foot level surcharge) to accommodate the off-ramp widening. Existing LACFCD Channel ends at the beginning of the proposed Retaining Wall No. 1606 with bottom of wall footing elevation slightly below bottom of channel cover elevation. Applied load from proposed Retaining Wall No. 1606 to the channel structure will be evaluated in an addendum upon receiving details plans from Structure Design.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 8 feet with an approximate length of 116.54 feet located from RW LOL Station 604+23.46 to Station 605+40.00 (approximate 158.23 ft. Rt. of Sta. 1604+24.11 to 158.23 ft Rt. of Sta. 1605+40.00 Route 5 Centerline). The location and geometric layout data for the wall are shown on the General Plan and Structure Plan for Retaining Wall No. 1606. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)
From	To			
STA 604+23.46	STA 604+60.00	8	6.5	610.00
STA 604+60.00	STA 605+40.00	6	5.0	610.17

Caltrans 2006 Standard Plans and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from May 15, 2004 to August 17, 2005. The field investigation included drilling two 8-inch outer diameter hollow-stem auger and two 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5-foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used for all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
04-2	1601+59.4	155.5 Rt.	613.1	61.0	6/15/04
05-30	1602+42.3	85.2 Rt.	624.8	36.5	8/16/05
05-34	1605+91.5	60.4 Rt.	635.4	36.5	08/16-17/05
05-7	1606+41.7	137.3 Rt.	621.0	101.2	07/20-21/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	2
Moisture Content	CTM 212, 226	2
Corrosion	CTM 417, 422, 643,532	8

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The current location of proposed Wall 1606 is an embankment. The embankment has an approximate slope of 1V:2H or flatter.

The boring logs were used to develop a continuous soil profile with depth for the wall location. The upper 15 feet of the borings were logged as fill. The fill was generally composed of from loose to dense silty sand with gravel. The top of native material was logged at an elevation of about 606 to 621 feet in the borings. The native alluvium was generally composed of from loose to very dense silty sand and sand, and sandy silt, with gravel lenses throughout.

Groundwater

Groundwater was not encountered in borings drilled for this study to maximum depths of 101.2 feet (dry down to at least elevation +519.8 ft.).

SCOUR

There is no scour potential at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1606

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-30	2.0-3.6	3900	8.7	15	186
05-30	22.0-24.0	4100	8.9	60	141
05-7	0-20	3800	8.6	75	63
05-7	20-40	5500	8.6	45	9
05-7	40-60	4400	8.0	45	0
05-7	60-80	7400	7.4	45	33
05-7	80-100	7100	8.3	60	24
05-34	5-16.5	4400	9.2	30	30
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w=6.75, is the nearest seismic source from the project site. Based on Weber (1980), this reverse type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 101.2 feet (dry down to at least elevation +519.8 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 5 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 5 wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall with some earthwork (Case I: 2 ft level surcharge). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1606

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bottom of Sub-excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 604+23.46	STA 604+60.00	8	6.5	610.00	607.00	2.5
STA 604+60.00	STA 605+40.00	6	5.0	610.17	607.17	2.2

Notes: 1) Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum cover of 1.5 feet (above top of spread footing elevation) is recommended in the back of Type 5 wall.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

3. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May, 2006). Imported materials used to construct the new fill embankment should be tested during grading to assess their expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft. of the subgrade elevation.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. Complete removal and re-compaction of compressible loose materials below spread footing are required prior to fill placement in order to expose firm and unyielding ground. A minimum over-excavation of 3 ft should be performed within this area to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Akbar Mehrazar at (949) 440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 12/20/11

A Mehrazar

Akbar Mehrazar
Engineering Geologist
Office of Geotechnical Design–South 1
Branch D

Supervised by: Date: 12/20/11

P Shiva Karimi

Shiva Karimi, Ph.D., G.E., Chief
Transportation Engineer
Office of Geotechnical Design–South 1
Branch D

REGISTERED PROFESSIONAL ENGINEER
SHIVA KARIMI
No. GE2651
Exp. 6/30/12
GEOTECHNICAL
STATE OF CALIFORNIA

cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: April 20, 2012

File: 07-LA-5-PM 30.39/30.41
07-1218W1
I-5 Empire Interchange
Retaining Wall No.1606

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design South 1 (OGDS1)
Branch D

Subject: Addendum to Foundation Report for Retaining Wall 1606, dated December 20, 2011

The Office of Geotechnical South 1 (OGDS1) prepared this addendum to the referenced Foundation Report to present the geotechnical recommendations for the proposed revised foundation types. The recommendations presented in the Foundation Report were for shallow footings, proposed originally for the Wall 1606. Since, shallow footings could exert lateral and vertical loads on the adjacent flood control channel wall, Office of Structure Design, Branch 18 (OSD) in communication with OGDS1 proposed using 24-inch diameter cast-in-drilled-hole (CIDH) pile foundations to mitigate the impact on the channel wall. Accordingly, OGDS1 prepared these recommendations for CIDH piles based on General Plan (dated 4-19-201) and the design loads provided by the OSD on March 3, 2012 and March 27, 2012 through email communications. These recommendations supersede the section on foundation recommendations, presented in the referenced report.

The analysis was conducted for 24-inch diameter CIDH piles installed in a manner that would mitigate the transmission of loading from the retaining wall on to the channel wall. We understand that in order to mitigate the loading on the channel wall, the proposed CIDH piles would be installed inside oversized cased holes drilled down to 12 feet, which is approximately the depth to the channel bottom. Therefore, down to a depth of 12 feet, the hole would be drilled to a size larger than 24 inches and provided with a permanent casing. Below 12 feet (or the bottom of the oversized hole) the hole would be drilled to a 24 inches in diameter. The pile within the oversized hole would be formed or cast to have a diameter of 24 inches using a second casing and to have a gap between the pile and the cased oversized hole. The piles installed in this manner would not transmit either lateral or vertical stresses to the surrounding ground in the top 12 feet. Consequently, the piles would not have any lateral and vertical geotechnical capacities in the top 12 feet.

The analysis was performed assuming that the piles would be constructed using a wet method and consequently would not have end resistances. The computer programs SHAFT, Version 5.0 and LPILE, Version 5.0 were used for the vertical and lateral analyses, respectively. The geotechnical profile for the analysis was developed using the subsurface information obtained from Borings 05-30, 05-34 and 05-07, drilled for the project.

Pile Data Table for Retaining Wall 1606

RW LOL Station		Wall Design Height (H) (ft)	Pile Type / Diameter	Design Loading (kips)	Nominal Resistance		Lateral Load (Kips)	Pile Cut-off Elev. (ft)	Design Pile Tip Elev. (ft)	Specified Pile Tip Elev. (ft)
From	To				Compression (kip)	Tension (kip)				
STA 604+23.46	STA 604+87.46	6	CIDH/ 24 inches	68	136	32	10.0	612.0	575(a) 588(b) 586(c)	575
STA 604+87.46	STA 605+35.46	4	CIDH/ 24- inches	43	86	0	6.0	612.0	580(a) 587(c)	580

Notes: Design Tip is controlled by the following demands:
(a) Nominal resistance in compression (b) Nominal resistance in tension. (c) Lateral loading for a 1 inch deflection at the top.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated in the footing excavation.

2. The potential for caving exists in drilled holes for CIDH piles over their entire lengths. Due to the proximity of the piles to the channel wall, there is a potential for the caving to propagate all the way to the channel wall. The contractor should use an appropriate drilling method to minimize the caving.

3. The inside diameter of the cased oversized drilled hole within the top 12 feet of the pile should be large enough to avoid any contact between the pile and the casing. We recommend that the inside diameter of the oversized cased hole be at least 2 inches more than the diameter of the pile. The contractor should verify that no concrete or cement slurry enters the annular space between the pile and the outer casing during the casting of the pile. In addition, the contractor should take appropriate measures to keep the annular space free of any objects, down to its bottom, during and after the construction. The engineer should request that the contractor demonstrate the annular space is free of any objects down to its bottom. The casing installed for construction of the oversized hole should have a design life equal to the project design life.

4. Concrete placement for construction of the CIDH pile should be completed within the same day that excavation of the drilled hole has been completed.

5. Drilling during construction may be variable and sporadically hard (within gravel zones) down to anticipated pile tip elevations.

6. All earthwork is expected to be carried out by conventional equipment. Fill Placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19 of the Caltrans Standard Specifications.

7. Based on soil types encountered during the recent investigation, OGDS-1 recommends a slope ratio of 1:V:1H or flatter for the temporary back cut slope and excavations for construction. If there are constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate steeper excavations

If you have any questions, please call Gamini Weeratunga at (949) 440-3427 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 04-20-12

REGISTERED PROFESSIONAL ENGINEER GAMINI C. WEERATUNGA No. 2403 Exp. 9/30/12 GEOTECHNICAL STATE OF CALIFORNIA

Gamini Weeratunga, G.E.
Transportation Engineer (civil)
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Reviewed by: Date: 04-20-12

P Shiva Karimi

REGISTERED PROFESSIONAL ENGINEER SHIVA KARIMI No. GE2651 Exp. 6/30/12 GEOTECHNICAL STATE OF CALIFORNIA

Shiva Karimi, Ph.D, G.E.
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Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
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To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: February 22, 2012

File: 07-LA-5- PM 30.4/30.5
07-1218W1
Empire Interchange
Retaining Wall No.1607

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1607

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 16, 2011 and Wall General Plan and Structure Plans (dated February 10, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1607 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1607 will be constructed along southbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.4 and 30.5 and will retain proposed railroad track above freeway level. Retaining Wall No. 1607 is a Type 1RR (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the Empire Avenue southbound on-ramp level to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.487B. (based on updated metric plans) dated April 2, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1607 will be constructed along an alignment approximately parallel to and west of southbound I-5, south of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.4 and 30.5 and will retain proposed railroad track above freeway level. Retaining Wall No. 1607 is a Type 1RR (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the Empire Avenue southbound on-ramp level. Based on the information provided by Office of Structure Design, the minimum horizontal distance from top of Retaining Wall 1607 to the nearest railroad track centerline (retained by RW1607) is 20.0 feet. A 2H:1V slope connects top of the RW1607 to the railroad track platform.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 26 feet with an approximate length of 494.54 feet located from RW LOL Station 605+25.93 to Station 610+20.47. The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 3 for Retaining Wall No. 1607. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 605+25.93	STA 605+56.47	Type 1RR	6	15.00	619.50
STA 605+56.47	STA 605+88.47	Type 1RR	6	15.00	618.50
STA 605+88.47	STA 606+36.47	Type 1RR	8	16.50	616.75

STA 606+36.47	STA 606+84.47	Type 1RR	10	18.00	614.50
STA 606+84.47	STA 607+32.47	Type 1RR	14	20.50	612.25
STA 607+32.47	STA 607+80.47	Type 1RR	16	22.00	610.00
STA 607+80.47	STA 608+28.47	Type 1RR	18	23.50	608.25
STA 608+28.47	STA 608+76.47	Type 1RR	20	24.50	606.50
STA 608+76.47	STA 609+72.47	Type 1RR	24	26.50	604.00
STA 609+72.47	STA 610+20.47	Type 1RR	26	28.00	603.75

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 10, 2005 through January 25, 2006. The field investigation included drilling five 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-21A	1605+68.6	63.7 Lt.	636.4	61.5	8/10/2005
05-47(P-S)	1609+02.9	138.2 Lt.	624.0	181.0	9/30/2005
06-76	1607+84.0	248.3 Lt.	619.8	70.8	1/11/2006
06-77	1610+17.4	270.8 Lt.	623.5	106.5	1/11/2006
6-92	1611+12.6	314.3 Lt.	626.3	100.2	1/25/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	3
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	8

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately from 0 to 18 feet of fill generally composed of loose to dense silty sand with gravel and sand interlayers. Underlying alluvium is composed of loose to very dense silty sand/sandy silt with gravel/cobbles. It also includes medium stiff to hard sandy lean clay.

Groundwater

Groundwater was not encountered in borings drilled for this study at elevations above +575 ft (bottom elevation of boring 05-21A).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1607

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
06-77	0-25	5500	7.8	135	210
06-77	25-50	6900	8.0	150	18
06-77	50-75	5200	8.2	105	33
06-77	75-105	7100	8.2	105	12
06-92	9.8-30	6400	8.0	105	30
06-92	30-50	7700	8.2	105	18
06-92	50-70	10000	8.2	120	15
06-92	70-100	5600	8.3	120	36
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in borings drilled for this study to maximum depths of 60.0 feet (elevation +575 ft). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1RR (AREMA, 2005) retaining wall is considered the best solution for retaining soils for proposed railroad track above I-5 Freeway at this location. Standard Type 1RR wall (Case II) spread footings can be used to support the wall, provided that the recommendations presented in Construction Considerations section of this report are followed.

Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1607

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bottom of Sub-excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 605+25.93	STA 605+56.47	6	15.00	619.50	613.50	1.88
STA 605+56.47	STA 605+88.47	6	15.00	618.50	612.50	1.88
STA 605+88.47	STA 606+36.47	8	16.50	616.75	610.75	2.45
STA 606+36.47	STA 606+84.47	10	18.00	614.50	608.50	2.71
STA 606+84.47	STA 607+32.47	14	20.50	612.25	606.25	3.23
STA 607+32.47	STA 607+80.47	16	22.00	610.00	604.00	3.92
STA 607+80.47	STA 608+28.47	18	23.50	608.25	602.25	4.47
STA 608+28.47	STA 608+76.47	20	24.50	606.50	600.50	4.43
STA 608+76.47	STA 609+72.47	24	26.50	604.00	598.00	5.63
STA 609+72.47	STA 610+20.47	26	28.00	603.75	597.75	5.67

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 2.0 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program PCSTABLm2/STED under both static and pseudo-static conditions. The soil profile and the strength parameters used in performing the stability analysis as developed from the subsurface investigation, are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. For the analysis, it was assumed that the wall is founded on shallow footings.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Fill	20	36	130	0
Alluvium	>50	33	130	0

The stability analysis was performed for 3 cross sections with wall heights 14, 24 and 26 (the maximum height of the proposed wall) to evaluate the global stability under static and design seismic conditions, respectively. Based on the information provided by the Office of Structure Design, the railroad loading was considered to be a 1200 psf load acting on a 14-foot wide strip located along the railroad track. The stability under the design seismic conditions was evaluated using a pseudostatic analysis with a horizontal acceleration of 0.15g.

The results of the stability analysis are summarized in Table 7 below.

Table 7 – Summary of Slope Stability Analysis

Cross Section (Station)	Wall Ht (feet)	Distance to the CL of Track (feet)*	FOS	
			Static	Pseudostatic
607+00	14	20	2.0	1.6
609+00	24	20	2.0	1.6
610+00	26	20	2.1	1.7

Note: * - Distances to the track were provided by the Office of Structure Design.

The results of the stability analysis indicate that the wall segments will have FOS greater than 1.5 and 1.1 under static and design seismic conditions, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1 H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction per ASTM 1557 test method.

7. A minimum over-excavation of 6 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. However, the compaction standard used should be the ASTM 1557 test method.

If you have any questions, please call Akbar Mehrazar at (949) 440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 02/22/2012

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cc: GS Corporate – Shira Rajendra (Electronic File)
Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: December 28, 2011

File: 07-LA-5- PM 30.47/30.74
07-1218W1
Empire Interchange
Retaining Wall No.1610

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1610

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 1, 2011 and Wall General Plan and Structure Plans (plotted November 28 through 30, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for the proposed Retaining Wall No. 1610 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1610 will be constructed along northbound I-5, north of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.47 and 30.74. Retaining Wall No. 1610 consists of two segments; 1) Masonry Block Soundwall on Retaining Wall (Type 1SWB) (Station 608+98.59 to Station 618+52.17 RW/SW LOL), and 2) Sound Wall Masonry Block on Type 736S/SV (MOD) concrete barrier (Station 618+52.17 to Station 622+45.67 RW/SW LOL). Retaining Wall 1610 will be located along northbound I-5 east shoulder and northbound San Fernando Boulevard on-ramp to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;

c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall;

This Foundation Report supersedes the previous Foundation Recommendations for Soundwall/ Retaining Wall No.492 (based on updated metric plans) dated June 27, 2007 (Revised March 2, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1610 will be constructed along northbound I-5 east shoulder and will consist of two segments; 1) A Masonry Block Soundwall on Retaining Wall (Type 1SWB). , and 2) A Sound Wall Masonry Block on Type 736S/SV (MOD) concrete barrier supported by 16 inch diameter CIDH piles (Case II/I: Standard Plan B15-6). Existing Burbank Western Channel (LACFCD) is located parallel and near east side of the proposed Retaining Wall No. 1610 with bottom of wall footing elevation above channel cover elevation. Applied load from proposed Retaining Wall No. 1610 to the channel structure will be evaluated in an addendum upon receiving detailed plans from Structure Design.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The segment 1 wall height varies from 8 to 26 feet with a total length of 953.58 feet located from RW/SW LOL 608+98.59 to Station 618+52.17 (97.07 ft Rt. Sta. 1608+98.59 to 86.75 ft Rt. Sta. 1618+44.55 Route 5 centerline). The segment 2 wall height is from 14 feet with a total length of 393.5 feet located from Station 618+52.17 to Station 622+45.67 RW/SW LOL (86.75 ft Rt. Sta. 1618+44.55 to 85.42 Rt. Station 1622+32.99 Route 5 centerline).

The location and geometric layout data for the wall is shown on the General Plan, Structure Plan Nos. 1 through 6 for Retaining Wall No. 1610. Additional soundwall details are shown in Table 1, below.

Table 1A –Summary of wall Information (Segment 1)

SW/RW LOL Station (ft)		Wall Type & Concrete Barrier	Sound Wall Height (ft)	Retaining Wall Height (ft)	Bottom of Footing Elevation (ft)	Type of Foundation
From	To					
608+98.59	609+40.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	26	614.75	Spread Footing
609+40.17	610+36.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	26	615.75	Spread Footing
610+36.17	611+32.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	26	616.75	Spread Footing
611+32.17	612+28.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	26	617.58	Spread Footing
612+28.17	613+24.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	26	618.58	Spread Footing
613+24.17	614+20.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	24	621.00	Spread Footing
614+20.17	615+16.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	24	622.08	Spread Footing
615+16.17	616+12.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	22	623.33	Spread Footing
616+12.17	616+60.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	20	625.83	Spread Footing
616+60.17	617+32.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	16	629.17	Spread Footing
617+32.17	618+04.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	14	632.50	Spread Footing
618+04.17	618+52.17	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	8	636.50	Spread Footing

Table 1B –Summary of wall Information (Segment 2)

SW LOL Station (ft)		Wall Type & Concrete Barrier	Wall Design Height (ft)	He (ft)	Bottom of Concrete Barrier Elev. (ft)	Type of Foundation
From	To					
618+52.17	618+76.17	Masonry Block on MOD 736 SV Concrete Barrier (Case II)	14	4	645.74	16 inch dia. CIDH Piles
618+76.17	619+00.17	Masonry Block on MOD 736 SV Concrete Barrier (Case II)	14	2	645.56	16 inch dia. CIDH Piles

619+00.17	622+45.67	Masonry Block on MOD 736 S Concrete Barrier (Case I)	14	N/A	645.43 to 645.35	16 inch dia. CIDH Piles

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Structure Details No.1 through 6. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 15, 2005 to February 7, 2006. The field investigation included drilling five 8-inch outer diameter hollow-stem auger and two 4.5-inch outer diameter mud rotary borings, and one Cone Penetration Test (CPT). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans Drilling Services and Prosonic/Tri County Drilling operated drill rig were used at boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
06-99	1608+07.78	126.9 Rt.	621.0	100.7	2/7-8/2006
05-33	1610+17.55	60.5 Rt.	641.1	36.5	8/16-17/2005
05-41	1612+23.20	64.0 Lt.	641.5	36.5	8/22-23/2005
05-32	1615+59.93	81.0 Rt.	611.3	36.5	8/16-17/2005
05-31	1618+14.32	74.3 Rt.	644.3	61.5	8/16/2005
05-56 CPT	1620+50.72	72.0 Rt.	644.6	60.0	10/19/2005
05-10	1621+72.20	213.0 Rt.	634.8	75.2	7/26-27/2005
05-25	1623+95.88	93.9 Rt.	646.5	61.5	8/15/2005

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Corrosion	CTM 417, 422, 643,532	6

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site is generally composed of a top fill layer, from 1 to 21 feet thick, underlain with alluvium to the maximum 100.7 feet depth drilled. Top fill materials consist of loose to very dense silty sands/sandy silts and sands with gravel and cobbles. Underlaying alluvium consist of loose to very dense silty sands/sandy silts and sand with fine to coarse gravel, and stiff to very stiff sand clay.

Groundwater

Groundwater was not encountered in all auger borings drilled for this study to maximum depths of 36.5 feet (elevation +574.8 ft.).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1610

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)

06-99	4 to 30	5100	8.3	120	45
06-99	30 to 47	4200	7.6	135	3
06-99	47 to 61	4000	8.3	105	36
06-99	61 to 100	6500	8.5	105	18
05-41	0-20	2400	10.0	60	210
05-41	20-36.4	3400	8.7	45	57
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in all auger borings drilled for this study to maximum depths of 36.5 feet (elevation +574.8 ft.) and to at least a depth of 100.7 feet below the surface (lowest elevs. +520.3 ft) in mud rotary Boring 06-99. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

The following recommendations are based on 1) Updated Retaining Wall No. 1610 General Plan, Structure Plan Nos. 1 through 6, and Structure Details No.1 (plotted November 28 through 30, 2011) provided by Mr. Jorge Estrada of Office of Structure Design, Branch 18, and 2) Results of laboratory testing and field investigation by OGDS1 and URS consultants.

Table 5A - Retaining Wall No. 1610 Spread Footing Data (Segment 1)

SW/RW LOL Station (ft)		Retaining Wall Design Height H (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)	Bottom of Sub-Excavation Elev. (ft)	Ultimate Bearing Capacity Stem With Haunch (ksf)
From	To					
608+98.59	609+40.17	26	18.5	614.75	611.75	12.5
609+40.17	610+36.17	26	18.5	615.75	612.75	12.5
610+36.17	611+32.17	26	18.5	616.75	613.75	12.5
611+32.17	612+28.17	26	18.5	617.58	614.58	12.5
612+28.17	613+24.17	26	18.5	618.58	615.58	12.5
613+24.17	614+20.17	24	16.5	621.00	618.00	12.2
614+20.17	615+16.17	24	16.5	622.08	619.08	12.2
615+16.17	616+12.17	22	15.25	623.33	620.33	11.3
616+12.17	616+60.17	20	14.25	625.83	622.83	9.9
616+60.17	617+32.17	16	12.0	629.17	626.17	8.1
617+32.17	618+04.17	14	10.75	632.50	629.50	7.3
618+04.17	618+52.17	8	8.0	636.50	633.50	5.5

Table 5B–Retaining Wall No.1610 Data (Segment 2)

SW LOL Station (ft)		Standard Plan Sheet No. /Case No.	Wall Design Height (ft)	He (ft)	Bottom of Concrete Barrier Elev. (ft)	Wall Type/Foundation
From	To					
618+52.17	618+76.17	B15-6/ Case II (ϕ=32^o)	14	4	645.74	Masonry Block On Type 736 SV Barrier on CIDH Piles
618+76.17	619+00.17	B15-6/ Case II (ϕ=32^o)	14	2	645.56	Masonry Block On Type 736 SV Barrier on CIDH Piles
619+00.17	622+45.67	B15-6/ Case I (ϕ=32^o)	14	N/A	645.43 to 645.35	Masonry Block On Type 736 S Barrier on CIDH Piles

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section was used to analyze the global stability. Based on subsurface information collected via Caltrans field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 32 degrees and a minimum in situ density of 125 pcf, based on the material compacted to at least 90 percent relative compaction. Underlying alluvial material possesses similar soil parameters. The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Fill	0-20	32	125	0
Alluvium	80	32	125	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft of the roadbed subgrade elevation.

3. A minimum over-excavation of 3 feet should be performed under bottom of footing elevations (as shown in Table 5 above) to receive fill compacted to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture conditioned, and recompacted in place prior to fill placement.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. No ground water is anticipated at the CIDH boring excavations.

8. Caving is anticipated during CIDH boring excavations. Prior to placement of concrete, the interior surface of the shaft including the bottom should be cleaned of residue from drilling operations.

9. The drilling of the CIDH piles, the placement of the rebar cage, and concrete pour shall be completed at the same day.

If you have any questions, please call Akbar Mehrazar (949) 440-3415 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 12/28/11

A Mehrazar

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Supervised by: Date: 12/28/11

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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: February 24, 2012

File: 07-LA-5- PM 30.54/30.69
07-1218W1
Empire Interchange
Retaining Wall No.1613

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1613

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 16, 2011 and Wall General Plan and Structure Plans (dated February 22, 2012), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1613 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1613 will be constructed on an alignment that runs approximately parallel to and west of southbound I-5, north of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.54 and 30.69 and will retain proposed railroad track above freeway level. Except for a length of 40 feet at its southern end, the Retaining Wall No. 1613 is a Type 1RR (AREMA, 2005) wall with cable railing at the top and concrete barrier Type 60D at the Empire Avenue southbound off-ramp level to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California. A 40 feet length of the wall at its southern end consists of Type 1 wall.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;

c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall Nos.501A and 491 (based on updated metric plans).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1613 will be constructed on an alignment that runs approximately parallel to and west of southbound I-5, north of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.54 and 30.69 and will retain proposed railroad track above freeway level. Except for a length of 40 feet at its southern end, the Retaining Wall No. 1613 is a Type 1RR (AREMA, 2005) wall with cable railing at the top and concrete barrier Type 60D at the Empire Avenue southbound off-ramp level. A 40 feet length of the wall at its southern end consists of Type 1 wall. Based on the information provided by Office of Structure Design, the minimum horizontal distance from top of Retaining Wall 1613 to the railroad track centerline (retained by RW1613) is 20.5 feet. A 2H:1V slope connects top of the RW1613 to the railroad track platform.

A pump station structure is located below bottom of Retaining Wall No. 1613 foundation, near RWLOL Station 617+00. Foundation recommendations for the wall foundation near this pump station will be provided upon receiving detailed plans from Office of Structure Design.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 30 to 8 feet with an approximate length of 730.4 feet located from RW LOL Station 612+35.56 to Station 619+65.95 (approximate 189.24 ft Lt of Sta. 1612+35.41 to 226.16 ft Lt. of Sta. 1619+87.26 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 4 for Retaining Wall No. 1613. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 612+35.56	STA 612+75.81	Type 1 Case III	26	14.25	603.67
STA 612+75.81	STA 613+07.81	Type 1RR Case II	28	29	602.67
STA 613+07.81	STA 614+03.81	Type 1RR Case II	28	29	603.67
STA 614+03.81	STA 614+75.81	Type 1RR Case II	30	30	603.42
STA 614+75.81	STA 615+23.81	Type 1RR Case II	28	29	605.67
STA 615+23.81	STA 615+71.81	Type 1RR Case II	28	29	607.67
STA 615+71.81	STA 616+19.81	Type 1RR Case II	28	29	609.67
STA 616+19.81	STA 616+67.81	Type 1RR Case II	26	28	611.92
STA 616+67.81	STA 617+15.81	Type 1RR Case II	26	28	613.92
STA 617+15.81	STA 617+63.81	Type 1RR Case II	24	26.5	616.67
STA 617+63.81	STA 618+11.81	Type 1RR Case II	22	25.5	619.42
STA 618+11.81	STA 618+59.81	Type 1RR Case II	20	24.5	622.17
STA 618+59.81	STA 619+07.81	Type 1RR Case II	18	23.5	624.92
STA 619+07.81	STA 619+31.81	Type 1RR Case II	16	22	627.17
STA 619+31.81	STA 619+43.81	Type 1RR Case II	16	22	628.42
STA 619+43.81	STA 619+55.81	Type 1RR Case II	12	19	632.67
STA 619+55.81	STA 619+65.95	Type 1RR Case II	8	16.5	636.67

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 4, 2005 through February 3, 2006. The field investigation included drilling five 8-inch outer diameter hollow-stem auger and four 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-20	1615+69.70	64.7 Lt.	641.9	51.5	8/4-5/2005
05-41	1612+23.20	64.0 Lt.	641.5	36.5	8/22-23/2005
05-45	1617+96.02	199.3 Lt.	632.0	51.5	8/24-25/2005
05-46	1614+45.89	194.7 Lt.	628.6	71.5	8/25-29/2005
05-46A	1614+49.44	196.3 Lt.	628.5	96.5	11/9-10/2005
06-78	1617+05.95	323.5 Lt.	632.3	62.0	1/12/2006
06-79	1618+76.47	327.52 Lt.	635.3	61.5	1/12/2006
06-96	1612+69.41	295.64 Lt.	628.0	115.4	1/31/2006
06-98	1611+26.82	123.20 Lt.	624.6	100.3	2/1-3/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	1
Moisture Content	CTM 212, 226	1
Corrosion	CTM 417, 422, 643,532	17

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site is underlain by 0 to 15 feet of fill (down to an approximate elevation of +620 ft) generally composed of loose to dense silty sand/sandy silt with gravel. Underlying alluvium is composed of loose to very dense silty sand/sandy silt with gravel/cobbles, sand, and gravel. It also includes layers of firm to hard lean clay, clayey sand/sandy clay from approximate elevation of +560 to +600 ft.

Groundwater

Groundwater was not encountered in borings drilled for this study at elevations above +534 feet (the bottom elevation of Boring 05-46A).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1613

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-20	10.1	No Test	8.3	75	222
05-20	15.0-31.5	1700	8.4	45	140
05-20	35.1-51.5	8000	8.4	30	0
05-41	0-20.0	2400	10.0	60	210
05-41	20.0-36.4	3400	8.7	45	57
5-46A	70.0-95.0	13000	8.6	45	66
06-79	0.0-29.8	9400	8.2	120	39
06-79	29.8-60.0	8000	8.3	105	15
06-96	6.9-20.0	5000	8.0	120	57
06-96	20.0-40.0	3700	8.0	165	225
06-96	40.0-60.0	4100	8.3	315	540
06-96	60.0-80.0	3700	8.1	345	560
06-96	80.0-115.1	4900	8.1	90	ND
06-98	6.9-31.8	3000	8.2	105	6
06-98	31.8-53.1	4600	8.5	120	45
06-98	53.1-77.0	5100	7.9	90	6
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in borings drilled for this study to maximum depths of 96.5 feet (elevation +534). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1RR (AREMA, 2005) retaining wall is considered the best solution for retaining soils for proposed railroad track above I-5 Freeway at this location. Type 1RR wall (with a Caltrans Standard Type 1 wall at the southern end) on spread footings can be used for the proposed construction provided that the recommendations presented in Construction Considerations section of this report are followed.

Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No. 1613

RW LOL Station (ft)		Wall Type	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bottom of Sub-excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD¹ (q_{all}) (ksf)
From	To						
STA 612+35.56	STA 612+75.81	Type 1 Case III	26	14.25	603.67	597.67	6.5
STA 612+75.81	STA 613+07.81	Type 1RR Case II	28	29	602.67	596.67	6.65
STA 613+07.81	STA 614+03.81	Type 1RR Case II	28	29	603.67	597.67	6.65
STA 614+03.81	STA 614+75.81	Type 1RR Case II	30	30	603.42	597.42	6.81
STA 614+75.81	STA 615+23.81	Type 1RR Case II	28	29	605.67	599.67	6.65
STA 615+23.81	STA 615+71.81	Type 1RR Case II	28	29	607.67	601.67	6.65
STA 615+71.81	STA 616+19.81	Type 1RR Case II	28	29	609.67	603.67	6.65
STA 616+19.81	STA 616+67.81	Type 1RR Case II	26	28	611.92	605.92	5.67
STA 616+67.81	STA 617+15.81	Type 1RR Case II	26	28	613.92	607.92	5.67
STA 617+15.81	STA 617+63.81	Type 1RR Case II	24	26.5	616.67	610.67	5.63
STA 617+63.81	STA 618+11.81	Type 1RR Case II	22	25.5	619.42	613.42	4.60
STA 618+11.81	STA 618+59.81	Type 1RR Case II	20	24.5	622.17	616.17	4.43
STA 618+59.81	STA 619+07.81	Type 1RR Case II	18	23.5	624.92	618.92	4.47
STA 619+07.81	STA 619+31.81	Type 1RR Case II	16	22	627.17	621.17	3.92
STA 619+31.81	STA 619+43.81	Type 1RR Case II	16	22	628.42	622.42	3.92
STA 619+43.81	STA 619+55.81	Type 1RR Case II	12	19	632.67	626.67	3.05
STA 619+55.81	STA 619+65.95	Type 1RR Case II	8	16.5	636.67	630.67	2.45

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 2.0 feet is recommended over the spread footings.

Settlement

The settlement is anticipated to be about 1 inch for walls founded on compacted soil as described in Table 5. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program PCSTABLm2/STED under both static and pseudo-static conditions. The soil profile and the strength parameters used in performing the stability analysis as developed from the subsurface investigation, are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. For the analysis, it was assumed that the wall is founded on shallow footings.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	Friction Angle (Degree)	In situ Density (lbs/ft³)	Cohesion (psf)
Fill	6 to 33	36	130	0
Alluvium	>50	34	130	0

The stability analysis was performed for 3 cross sections with wall heights 12, 20 and 26 (the maximum height of the proposed wall) to evaluate the global stability under static and design seismic conditions, respectively. Based on the information provided by the Office of Structure Design, the railroad loading was considered to be a 1200 psf load acting on a 14-foot wide strip located along the railroad track. The stability under the design seismic conditions was evaluated using a pseudostatic analysis with a horizontal acceleration of 0.15g.

The results of the stability analysis are summarized in Table 7 below.

Table 7 – Summary of Slope Stability Analysis

Cross Section (Station)	Wall Ht (feet)	Distance to the CL of Track (feet)*	FOS	
			Static	Pseudostatic
619+50	12	20.5	2.2	1.8
618+50	20	20.5	2.3	1.8
614+50	28	22.5	2.2	1.8

Note: * - Distances to the track were provided by the Office of Structure Design.

The results of the stability analysis indicate that the wall segments will have FOS greater than 1.5 and 1.1 under static and design seismic conditions, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. A minimum over-excavation of 6 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. However, the compaction standard used should be the ASTM 1557 test method.

If you have any questions, please call Akbar Mehrazar at (949)440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 02/24/2012

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Supervised by: Date: 02/24/2012

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District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: March 28, 2012

File: 07-LA-5- PM 30.54/30.69
07-1218W1
Empire Interchange,
Pump Station Near
Retaining Wall No.1613

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Plan Review for Pump Station Near Retaining Wall No. 1613

Reference: Foundation Report for Retaining Wall 1613, prepared by OGDS1, dated February 24, 2012

Based on the request from the Office of Structure Design (OSD), Branch 18, dated March 3, 2012, Office of Geotechnical Design South 1 (OGDS1) has reviewed the plan sheet titled "Empire Avenue ST-3, General Structural Notes 3" (drafted 2/15/2012) for proposed pump station near proposed Retaining Wall No. 1613 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Based on the information received from OSD via email dated 3/22/12, the subject pump station is located from Station 616+73.06 to Station 617+26.03 (RW 1613 LOL). Pump station structure is approximately 48 feet long and 26 feet wide with bottom of footing elevation at 582 to 587 feet and top of slab elevation at 610 feet. The distance between edge of RW 1613 footing and pump station varies and is 4 feet minimum. Bottom of footing elevation of RW 1613 adjacent to pump station is 613.92 to 616.67 feet.

OGDS1 reviewed the subject plan sheet ST-3 from geotechnical point of view only and has the following comments:

1. Based on the plan sheet ST-3 Notes, OGDS1 understands that toe pressure from adjacent Retaining Wall (RW 1613) is considered in the pump station design.
2. OGDS1 recommends that seismic design loads applied from RW 1613 to the pump station be considered for design of pump station structure (if not already included).
3. OGDS1 recommends that construction of pump station starts/completes prior to construction of RW 1613.

If you have any questions, please call Akbar Mehrazar at (949)440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 03/28/2012

A Mehrazar

Akbar Mehrazar
Transportation Engineer
Office of Geotechnical Design – South 1
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Supervised by: Date: 03/28/2012

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State of California **Business, Transportation and Housing Agency**

M e m o r a n d u m

Flex your power!
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To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: November 4, 2011

File: 07-LA-5- PM 30.52/30.65
07-1218W1
Empire Interchange
Retaining Wall No.1615

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1615

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 28, 2011 and Wall General Plan and Structure Plans (plotted October 28, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for the proposed Retaining Wall No. 1615 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1615 will be constructed along southbound I-5, north of Empire Avenue Undercrossing (Br. 53-2920) between post miles 30.52 and 30.65. Retaining Wall No. 1615 is a Caltrans Standard Type 1 retaining wall and will be constructed to retain I-5 southbound above the proposed Empire Avenue Southbound Off-ramp within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.493 (based on updated metric plans) dated February 26, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1615 will be located on the southbound side of Route 5 between a descending slope from freeway shoulder and the proposed Empire Avenue SB off-ramp. The structure will consist of a Type 1 wall (Case II, 2H:1V loading) with a type 60D concrete barrier at Empire Avenue SB off-ramp roadway level.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 28 feet with an approximate length of 696.42 feet located from RW LOL Station 611+42.06 to Station 618+38.48 (approximate 114.58 ft Lt of Sta. 1611+15.16 to 130.31 ft Lt of Sta. 1618+21.57 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 ,2 and 3 for Retaining Wall No. 1615. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

Approximate RW LOL Station (ft)		**Wall Type & Concrete Barrier**	**Wall Design Height (ft)**	**Footing Width (ft)**	**Bottom of Footing Elev. (ft)**
From	**To**				
STA 611+42.06	STA 612+12.06	Type 1 Concrete Barrier Type 60D	28	15.25	605.00
STA 612+12.06	STA 612+84.06	Type 1 Type 1 Concrete Barrier Type 60D	26	14.25	606.08
STA 612+84.06	STA 613+56.06	Type 1 Concrete Barrier Type 60D	24	13.25	607.17
STA 613+56.06	STA 614+20.06	Type 1 Concrete Barrier Type 60D	20	11.00	608.17
STA 614+20.06	STA 614+92.06	Type 1 Concrete Barrier Type 60D	20	11.00	609.17

STA 614+92.06	STA 615+48.06	Type 1 Concrete Barrier Type 60D	16	9.00	610.58
STA 615+48.06	STA 616+04.06	Type 1 Concrete Barrier Type 60D	14	8.00	612.17
STA 616+04.06	STA 616+60.06	Type 1 Concrete Barrier Type 60D	12	7.25	614.17
STA 616+60.06	STA 617+16.06	Type 1 Concrete Barrier Type 60D	10	6.25	616.25
STA 617+16.06	STA 617+56.06	Type 1 Concrete Barrier Type 60D	8	5.25	618.75
STA 617+56.06	STA 618+38.48	Type 1 Concrete Barrier Type 60D	6	4.25	620.67

Caltrans 2004 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2004 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 5, 2005 to February 8, 2006. The field investigation included drilling seven 8-inch outer diameter hollow-stem auger borings. Standard Penetration Tests (SPT's) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans Drilling Services and Prosonic/Tri County Drilling operated drill rigs were used at boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
06-98	1611+26.82	123.2 Lt.	624.6	100.3	02/08/06
05-41	1612+23.2	64.0 Lt.	641.5	36.5	08/23/05
05-46	1614+45.89	194.7 Lt	628.6	71.5	08/25-26/05
05-46A	1614+49.44	196.3 Lt	628.5	96.5	11/9/05
05-20	1615+69.7	64.6 Lt.	641.8	51.5	08/5/05
05-45	1617+96.02	199.3 Lt.	632.0	51.5	08/24-25/05
05-51	1619+10.3	55.9. Lt.	642.5	61.5	11/3/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	-
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	10

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately 3-10 feet of embankment fill generally composed of from loose to very dense silty sand with gravel, and coarse gravel with sporadic cobbles and sand interlayers. Underlying alluvium is generally composed of from loose to very dense silty sand, silty sand with gravel and cobbles, and sand with silt, clayey sand and minor gravel with sand interbeds.

Groundwater

Groundwater was not encountered in borings drilled for this study to maximum depth of 100.3 feet (dry down to at least elevation +524.3 ft). In the vicinity, DWR wells (01N14W03F03S and 01N14W03F06S) located near Buena Vista Street/Winona Avenue intersection show groundwater measurements below the surface vary from 211.8 to 167.5 ft depth corresponding to approximate elevations +471.2 to historically high +515.5 ft NAVD 88. No dates were provided but the wells had 35 to 14 measurements taken.

SCOUR

There is no scour potential at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1615

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
06-98	6.9-32	3000	8.2	105	6
06-98	32-53	4600	8.5	120	45
06-98	53-77	5100	7.9	90	6
05-41	0-20.0	2400	10	60	210
05-41	20.0-36.4	3400	8.7	45	57
05-20	10.2	NA	8.3	75	222
05-20	15.0-31.5	1700	8.4	45	140
05-20	35.1-51.5	8000	8.4	30	0
05-51	0.0-29.8	8000	9.6	60	ND
05-51	29.8-60.0	13000	8.2	45	36
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (borings were dry) to at least a depth of 100.3 feet below the surface (lowest elev. +524.3 ft) in Boring No. 06-98. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall with some earthwork (Case II: 2:1 slope). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1615

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bot. Of Sub-excavation Elev. (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
STA 611+42.06	STA 612+12.06	28	15.25	605.00	N/A	7.1
STA 612+12.06	STA 612+84.06	26	14.25	606.08	N/A	6.5
STA 612+84.06	STA 613+56.06	24	13.25	607.17	604.17	6.0
STA 613+56.06	STA 614+20.06	20	11.00	608.17	605.17	4.7
STA 614+20.06	STA 614+92.06	20	11.00	609.17	606.17	4.7
STA 614+92.06	STA 615+48.06	16	9.00	610.58	607.58	3.7
STA 615+48.06	STA 616+04.06	14	8.00	612.17	609.17	3.3
STA 616+04.06	STA 616+60.06	12	7.25	614.17	611.17	2.7
STA 616+60.06	STA 617+16.06	10	6.25	616.25	613.25	2.3
STA 617+16.06	STA 617+56.06	8	5.25	618.75	615.75	2.0
STA 617+56.06	STA 618+38.48	6	4.25	620.67	617.67	1.5

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section at approximate RW LOL Station 611+42 (RW Station 91+17 Metric unit) was used to analyze the global stability. The fill material is assumed to have a minimum friction angle of 32 degrees and a minimum in situ density of 120 pcf, based on the material compacted to at least 90 percent relative compaction. Underlying alluvial material possesses similar soil parameters. For the purpose of slope stability analysis, groundwater was not encountered down to at least elevation +524.3 ft (100.3 ft depth). The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. Complete removal and re-compaction of compressible loose materials below spread footing form RW LOL STA 612+84.06 to STA 618+38.48 are required prior to fill

Retaining Wall No. 1615
07-1218W1

placement in order to expose firm and unyielding ground. A minimum over-excavation of 3 ft should be performed within this area to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

If you have any questions, please call Kevin Lai (213) 620-2344 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 10/27/11

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Transportation Engineer
Office of Geotechnical Design – South 1
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Supervised by: Date: 10/27/11

Shiva Karimi

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Memorandum

To: MR.MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attn: Mr. Jorge Estrada

Date: January 6, 2012

File: 07-LA-05-PM 30.71/31.23
EA: 07-1218W1
I-5 Empire Interchange
Retaining Wall 1630

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Recommendations for Retaining Wall 1630

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 28, 2011 and Wall General Plan and Structure Plans (plotted December 16, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1630 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1630 will be constructed along northbound I-5 at the San Fernando Blvd. On-Ramp. Approximately half of the wall will be a masonry soundwall on a Type 736SV concrete barrier, and the other half will be a masonry soundwall on a Type 736A (MOD) concrete barrier on top of a Type 1SWB retaining wall. The wall will be located near the base of the existing freeway slope to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No. 500 (based on updated metric plans) dated December 21, 2006 (Revised March 12, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the City of Burbank. The Empire interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metrolink railroad tracks, and add high occupancy vehicle (HOV) lanes on Route 5 (one lane in each direction).

Retaining Wall No. 1630 will be located on the northbound side of Route 5 at the San Fernando Blvd. On-Ramp from RW LOL Station 621+23.32 to 648+51.69 (129.82 ft Rt., Station 1621+33.57 to 95.66 ft, Station 1648+77.87 of Route 5 Centerline). Approximately half of the wall will be a masonry soundwall on a Type 736SV concrete barrier, and the other half will be a masonry soundwall on a Type 736A (MOD) concrete barrier on top of a Type 1SWB retaining wall. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I plus 2 foot level surcharge) to accommodate the freeway widening.

Elevations provided on current plans and recommendations are based on the NAVD88 datum.

The type 1 SWB retaining wall height ranges from 6 to 14 feet with an approximate length of 1280 feet located from RW LOL Station 635+71.70 to Station 648+51.69. The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 11 for Retaining Wall No. 1630. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

Wall Type	RW LOL Station (ft)		Wall Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
	From	To			
736 SV	621+23.52	635+71.70	NA	NA	NA
Type 1 SWB	635+71.70	636+67.70	6	7.75	650.25
Type 1 SWB	636+67.70	637+39.70	8	8.0	650.25
Type 1 SWB	637+39.70	638+11.70	10	8.75	650.25
Type 1 SWB	638+11.70	638+59.70	12	9.75	650.25
Type 1 SWB	638+59.70	639+31.70	12	9.75	652.25
Type 1 SWB	639+31.70	640+03.70	12	9.75	654.25

Type 1 SWB	640+03.70	640+51.70	14	10.75	654.25
Type 1 SWB	640+51.70	641+47.70	14	10.75	656.25
Type 1 SWB	641+47.70	641+95.70	14	10.75	657.50
Type 1 SWB	641+95.70	642+43.70	14	10.75	658.75
Type 1 SWB	642+43.70	642+91.70	14	10.75	660.25
Type 1 SWB	642+91.70	643+63.70	14	10.75	662.25
Type 1 SWB	643+63.70	644+35.70	14	10.75	664.17
Type 1 SWB	644+35.70	645+07.70	14	10.75	666.33
Type 1 SWB	645+07.70	645+79.70	14	10.75	669.33
Type 1 SWB	645+79.70	646+27.70	12	9.75	672.25
Type 1 SWB	646+27.70	646+75.70	12	9.75	674.17
Type 1 SWB	646+75.70	647+23.70	10	8.75	676.17
Type 1 SWB	647+23.70	647+71.70	8	8.0	680.17
Type 1 SWB	647+71.70	648+51.69	6	7.75	682.25

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

A site-specific field exploration was performed from June 15 to November 19, 2005. The field investigation included drilling six 4.5-inch diameter mud rotary borings, six 8-inch diameter hollow-stem auger borings and two Cone Penetration Tests. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5-foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

A Caltrans and Tri-County operated drill rigs were used at all boring locations. A Caltrans engineer or a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-56 CPT	1620+50.7	71.9 Rt.	644.6	60.0	10/19/05
05-10	1621+72.2	213 Rt.	634.8	75.2	7/27/05
05-25	1623+95.9	93.9 Rt.	646.5	61.5	8/15/05
05-26	1627+94.8	67.7Rt.	648.9	31.5	6/17/04
05-50 CPT	1628+91.6	63.7 Lt.	649.9	61.0	10/12/05
05-11	1631+34.3	110.8 Rt.	649.9	66.5	6/15/04
05-27	1634+57.78	64.536 RT	655.2	31.5	8/15/05

05-37	1637+73.00	60.140 RT	660.9	36.5	8/18/05
05-12	1639+73.54	106.275 RT	656.8	71.5	7/28/05
05-28	1641+12.61	61.425 RT	671.2	36.5	8/16/05
05-9	1643+66.93	124.49' RT	661.3	70.2	7/26/05
05-29	1644+50.19	65.889 RT	680.3	36.5	8/16/05
05-8	1647+20.30	150.342 RT	664.2	66.5	7/25/05
05-36	1647+83.44	84.261 RT	688.9	36.5	8/17/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Corrosion	CTM 417, 422, 643,532	13
Sieve Analysis	CTM 202	1
Moisture Content	CTM 226	1

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Surface Conditions

The current location of proposed Wall 1630 is an embankment. The embankment has an approximate slope of 1V:2H and is vegetated.

The boring logs were used to develop a continuous soil profile with depth for the wall location. The upper 3-22 feet of the borings were logged as fill. The fill was generally composed of loose to medium dense silty sand with gravel. The top of native material was logged at an elevation of about 649-667 feet in the borings. The native alluvium was generally composed of loose to dense silt, silty sand, and sand, with gravel lenses throughout.

Groundwater

During the previous investigation in 1957 for nearby bridge 53-1110, Buena Vista/Winona UC (As Built LOTB plan dated September 1961), ground water was not encountered down to an approximate elevation of +592 ft, the maximum depth obtained. In addition, no groundwater was encountered on tape measured down to the caving depth of 68.2 ft at elevation +599.7 ft within cone penetrometer hole B-6.

Of the recent borings, the lowest elevation drilled to was approximately 559.6 feet above mean sea level at boring 05-10. No groundwater was encountered.

CORROSION

The results of the laboratory tests determined that the soils at this site are not corrosive to metal and reinforced concrete. Corrosion resistant design and construction materials are not necessary. Laboratory test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1630

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-11	0.0-25	7800	8.0	75	63
05-11	25-50	8900	7.9	45	24
05-11	50-65	12000	8.1	45	15
05-37	10-15	4900	11.0	45	60
05-12	0.0-25	7600	6.7	60	21
05-12	25-50	6600	7.1	60	21
05-12	50-71.5	9600	7.4	45	27
05-9	0.0-25	4850	8.6	60	9
05-9	25-50	2800	8.3	60	0
05-9	50-70	6000	8.4	60	15
05-8	0.0-25	1500	8.1	75	81
05-8	25-50	9700	8.4	45	12
05-8	50-65	6600	8.2	45	0
Corrosive Guidelines		**<1000**	**≤5.5**	**≥500**	**≥2000**

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans 1996 Seismic Hazard Map (CSHM), the Verdugo Fault (VDO) is the nearest seismic source to the project site. The fault is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w=6.75. Based on Weber (1980), this reverse type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered to at least a depth of 75.2 feet (elevation +559.6 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A sound wall on Type 1SWB retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1SWB wall spread footings are recommended for retaining wall support (Case I: 2 ft level surcharge) as existing soils are adequate to support the wall with some earthwork. Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacity were calculated for subsurface soils at the project site. The results are summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No. 1630

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bottom of Subexcavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To					
635+71.70	636+67.70	6	7.75	650.25	647.25	4.6
636+67.70	637+39.70	8	8.0	650.25	647.25	5.5
637+39.70	638+11.70	10	8.75	650.25	647.25	6.1
638+11.70	638+59.70	12	9.75	650.25	647.25	6.6
638+59.70	639+31.70	12	9.75	652.25	649.25	6.6
639+31.70	640+03.70	12	9.75	654.25	651.25	6.6
640+03.70	640+51.70	14	10.75	654.25	651.25	7.3
640+51.70	641+47.70	14	10.75	656.25	653.25	7.3
641+47.70	641+95.70	14	10.75	657.50	NA	7.3
641+95.70	642+43.70	14	10.75	658.75	NA	7.3
642+43.70	642+91.70	14	10.75	660.25	NA	7.3
642+91.70	643+63.70	14	10.75	662.25	NA	7.3
643+63.70	644+35.70	14	10.75	664.17	NA	7.3
644+35.70	645+07.70	14	10.75	666.33	NA	7.3
645+07.70	645+79.70	14	10.75	669.33	NA	7.3

645+79.70	646+27.70	12	9.75	672.25	NA	6.6
646+27.70	646+75.70	12	9.75	674.17	NA	6.6
646+75.70	647+23.70	10	8.75	676.17	NA	6.1
647+23.70	647+71.70	8	8.0	680.17	NA	5.5
647+71.70	648+51.69	6	7.75	682.25	NA	4.6

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the new fill embankment slope was evaluated using the computer program Xstabl version 5 under both static and pseudo-static conditions. One critical cross section at the maximum wall height was used to analyze the global stability. Based on subsurface information collected via our field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The result yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively. This analysis assumes that three feet of material below the footing elevations is removed and recompacted as structural backfill.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	**Friction Angle (Degree)**	**In situ Density (lbs/ft³)**	**Cohesion (psf)**
Structural Backfill	34	120	0
Native Soil	32	115	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

3. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May, 2006). Imported

materials used to construct the new fill embankment should be tested during grading to assess their expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft. of the subgrade elevation.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. Complete removal and re-compaction of compressible loose materials below spread footing are required prior to fill placement in order to expose firm and unyielding ground. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill to 95 percent Relative Compaction. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details. Over-excavated area should be cleaned of any loose soils and debris before receiving fill.

For further information, please contact Kristopher Barker at 213-620-2334 or Shiva Karimi at 213-620-2146.

Prepared by: Date: 01/06/12

Kristopher Barker

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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attention: Mr. Jorge Estrada

Date: February 24, 2012

File: 07-LA-5- PM 30.91/31.17
07-1218W1
Empire Interchange
Retaining Wall No.1635

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1635

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated December 16, 2011 and Wall General Plan and Structure Plans (plotted December 16, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1613 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1635 will be constructed along the western edge of southbound I-5, north of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.91 and 31.17 and will retain proposed railroad track above freeway level. Retaining Wall No. 1635 is a Type 1RR (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the freeway level to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall Nos.501A and 491 (based on updated metric plans).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1635 will be constructed along the western edge of southbound I-5, north of Empire Avenue Undercrossing Bridge No. 53-2920 between post miles 30.91 and 31.17 and will retain proposed railroad track above freeway level. Retaining Wall No. 1635 is a Type 1RR (AREMA, 2005) retaining wall with cable railing at the top and concrete barrier Type 60D at the freeway level.

Based on the information provided by Office of Structure Design, the minimum horizontal distance from top of Retaining Wall 1635 to the railroad track centerline (retained by RW1635) is 15.5 feet. The first segment of the wall is connected from top of the wall with an ascending 2H:1V slope to the railroad track platform (Loading Case II), and the second segment of the wall is level backfill (Loading Case 1).

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 6 to 16 feet with a total length of 1371.21 feet located from RW LOL Station 630+00.00 to Station 643+71.21 (approximate 141.30 ft Lt of Sta. 1630+42.23 to 93.52 ft Lt. of Sta. 1643+97.48 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 6 for Retaining Wall No. 1635. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type/Loading Case	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To				
STA 630+00.00	STA 630+96.00	Type 1RR (case II)	6	15.00	655.00
STA 630+96.00	STA 631+40.50	Type 1RR (Case I)	12	13.5	653.00
STA 631+40.50	STA 632+36.50	Type 1RR (Case I)	14	15.0	652.00
STA 632+36.50	STA 633+32.50	Type 1RR (Case I)	16	16.5	650.75
STA 633+32.50	STA 634+28.50	Type 1RR (Case I)	16	16.5	652.25
STA 634+28.50	STA 635+24.50	Type 1RR (Case I)	16	16.5	653.25
STA 635+24.50	STA 635+72.50	Type 1RR (Case I)	16	16.5	654.25
STA 635+72.50	STA 636+68.50	Type 1RR (Case I)	16	16.5	655.50
STA 636+68.50	STA 637+16.50	Type 1RR (Case I)	16	16.5	656.75
STA 637+16.50	STA 637+64.50	Type 1RR (Case I)	16	16.5	658.25
STA 637+64.50	STA 638+60.50	Type 1RR (Case I)	16	16.5	659.75
STA 638+60.50	STA 639+08.50	Type 1RR (Case I)	14	15.0	661.50
STA 639+08.50	STA 639+56.50	Type 1RR (Case I)	14	15.0	662.50
STA 639+56.50	STA 640+04.50	Type 1RR (Case I)	14	15.0	663.50
STA 640+04.50	STA 640+52.50	Type 1RR (Case I)	14	15.0	664.50
STA 640+52.50	STA 641+00.50	Type 1RR (Case I)	14	15.0	665.50
STA 641+00.50	STA 641+48.50	Type 1RR (Case I)	12	13.5	667.00
STA 641+48.50	STA 641+96.50	Type 1RR (Case I)	12	13.5	669.00
STA 641+96.50	STA 642+44.50	Type 1RR (Case I)	10	12.5	671.00
STA 642+44.50	STA 642+92.50	Type 1RR (Case I)	8	11.0	673.75
STA 642+92.50	STA 643+40.50	Type 1RR (Case I)	6	9.0	676.50
STA 643+40.50	STA 643+71.21	Type 1RR (Case I)	6	9.0	679.00

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 2, 2005 through February 1, 2006. The field investigation included drilling two 8-inch outer diameter hollow-stem auger and seven 4.5-inch and three Cone Penetration Tests (CPTs). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-15	1643+82.40	58.4 Lt.	682.2	71.5	8/2/2005
05-16	1638+34.60	67.6 Lt.	664.7	71.5	8/2/2005
05-17	1632+34.30	93.9 Lt.	655.1	71.5	8/4/2005
05-48CPT	1641+65.80	59.9 Lt.	673.0	65.1	10/13/2005
05-49 CPT	1634+24.40	76.0 Lt.	657.3	34.1	10/12/2005
05-50CPT	1628+91.60	63.7 Lt.	649.9	61.0	10/12/2005
05-59	1645+71.00	58.6 Lt.	688.4	71.5	11/2/2005
06-85	1633+00.00	302.8 Lt.	635.6	61.5	1/20/2006
06-86	1630+72.40	249.5 Lt.	646.1	52.0	1/21/2006
06-87	1636+23.50	193.6 Lt.	650.6	66.5	1/21/2006
06-90	1640+11.70	189.0 Lt.	655.6	66.5	1/25/2006
06-97	1643+53.10	208.4 Lt.	660.6	68.3	2/1/2006

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	1
Moisture Content	CTM 212, 226	1
Corrosion	CTM 417, 422, 643,532	12

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately from 1 to 28 feet of fill (down to approximate elevations of +645 to +660 ft) generally composed of loose to dense silty sands and sandy silts with gravel. In general, thicker fills were encountered in the northern portion of the site. The fill as encountered in the borings drilled at locations close to the wall alignment is typically dense at depths greater than 5 feet below ground surface.

Underlying alluvium is composed of loose to very dense silty sands and sandy silts with gravel/cobbles, sand, and gravel to the maximum depth explored.

Groundwater

Groundwater was not encountered in this study down to an elevation of +574.1 (depth of 61.5 feet in boring 06-85).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Corporation's soils laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1635

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-16	0.0-24.9	3500	7.9	60	0
05-16	29.8-49.9	5100	7.5	60	33
05-16	55.1-70.5	7100	6.7	60	6

05-17	1.6	-	9.0	45	72
05-17	9.8-21.6	7700	8.7	60	3
05-17	29.8-46.6	4900	8.3	45	234
05-17	49.9-71.5	7600	8.7	60	3
05-59	0.0-20.0	2100	8.2	120	66
05-59	20.0-49.9	9000	8.7	45	117
05-59	49.9-69.9	15000	8.6	45	ND
06-86	3.3-33	8500	8.7	75	3
06-86	36.1-62.3	7200	8.5	75	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM), the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 mile east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in borings drilled for this study to maximum depth of 61.5 feet (elevation +574.1) in boring 06-86. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1RR (AREMA, 2005) retaining wall is considered the best solution for retaining soils for proposed railroad track above I-5 freeway at this location. Type 1RR wall (Loading Case I and Case II) spread footings are recommended for retaining wall support as existing soils are adequate to support the wall. Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1635

RW LOL Station (ft)		Wall Type/Loading Case	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Bottom of Sub-excavation Elevation (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To						
STA 630+00.00	STA 630+96.00	Type 1RR (case II)	6	15.0	655.00	652.00	1.88
STA 630+96.00	STA 631+40.50	Type 1RR (Case I)	12	13.5	653.00	650.00	3.13
STA 631+40.50	STA 632+36.50	Type 1RR (Case I)	14	15.0	652.00	649.00	3.31
STA 632+36.50	STA 633+32.50	Type 1RR (Case I)	16	16.5	650.75	647.75	3.84
STA 633+32.50	STA 634+28.50	Type 1RR (Case I)	16	16.5	652.25	649.25	3.84
STA 634+28.50	STA 635+24.50	Type 1RR (Case I)	16	16.5	653.25	650.25	3.84
STA 635+24.50	STA 635+72.50	Type 1RR (Case I)	16	16.5	654.25	651.25	3.84
STA 635+72.50	STA 636+68.50	Type 1RR (Case I)	16	16.5	655.50	652.50	3.84
STA 636+68.50	STA 637+16.50	Type 1RR (Case I)	16	16.5	656.75	653.75	3.84
STA 637+16.50	STA 637+64.50	Type 1RR (Case I)	16	16.5	658.25	655.25	3.84
STA 637+64.50	STA 638+60.50	Type 1RR (Case I)	16	16.5	659.75	656.75	3.84
STA 638+60.50	STA 639+08.50	Type 1RR (Case I)	14	15.0	661.50	658.50	3.31
STA 639+08.50	STA 639+56.50	Type 1RR (Case I)	14	15.0	662.50	659.50	3.31
STA 639+56.50	STA 640+04.50	Type 1RR (Case I)	14	15.0	663.50	660.50	3.31
STA 640+04.50	STA 640+52.50	Type 1RR (Case I)	14	15.0	664.50	661.50	3.31
STA 640+52.50	STA 641+00.50	Type 1RR (Case I)	14	15.0	665.50	662.50	3.31
STA 641+00.50	STA 641+48.50	Type 1RR (Case I)	12	13.5	667.00	664.00	3.13

STA 641+48.50	STA 641+96.50	Type 1RR (Case I)	12	13.5	669.00	666.00	3.13
STA 641+96.50	STA 642+44.50	Type 1RR (Case I)	10	12.5	671.00	668.00	2.60
STA 642+44.50	STA 642+92.50	Type 1RR (Case I)	8	11.0	673.75	670.75	2.18
STA 642+92.50	STA 643+40.50	Type 1RR (Case I)	6	9.0	676.50	673.50	1.97
STA 643+40.50	STA 643+71.21	Type 1RR (Case I)	6	9.0	679.00	676.00	1.97

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 2.0 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program PCSTABLm2/STED under both static and pseudo-static conditions. The soil profile and the strength parameters used in performing the stability analysis as developed from the subsurface investigation, are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 36 degrees and a minimum in situ density of 130 pcf, based on the material compacted to at least 90 percent relative compaction. For the analysis, it was assumed that the wall is founded on shallow footings.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	**Thickness (ft)**	**Friction Angle (Degree)**	**In situ Density (lbs/ft³)**	**Cohesion (psf)**
Fill	3 to 20	36	130	0
Alluvium	>50	36	130	0

The stability analysis was performed for 2 cross sections with wall heights of 10 and 16 (the maximum height of the proposed wall) to evaluate the global stability under static and design seismic conditions, respectively. Based on the information provided by the Office of Structure Design, the railroad loading was considered to be a 1200 psf acting on a 14-foot wide strip located along the railroad track. The stability under the design seismic conditions was evaluated using a pseudostatic analysis with a horizontal acceleration of 0.15g.

The results of the stability analysis are summarized in Table 7 below.

Table 7 – Summary of Slope Stability Analysis

Cross Section (Station)	Wall Ht (feet)	Distance to the CL of Track (feet)*	FOS	
			Static	Pseudostatic
642+00	10	15.5	2.2	1.8
636+00	16	15.5	2.2	1.8

Note: * - Distances to the track were provided by the Office of Structure Design.

The results of the stability analysis indicate that the wall segments will have FOS greater than 1.5 and 1.1 under static and design seismic conditions, respectively.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. A minimum over-excavation of 3 ft should be performed within the area shown in Table 5 of this report to receive compacted fill placed at 95 percent minimum relative compaction per ASTM D1557 test method. Locally deeper overexcavations may be necessary and should be determined in the field by the engineer. The over-excavation bottom should be scarified, moisture-conditioned, and re-compacted in place prior to fill placement. Refer to Caltrans Standard specifications (May 2006), Section 19-5.03 for details.

If you have any questions, please call Akbar Mehrazar at (949)440-3415 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 02/24/2012

A Mehrazar

Akbar Mehrazar
Transportation Engineer
Office of Geotechnical Design – South 1
Branch D

Supervised by: Date: 02/24/2012

Shiva Karimi

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District 07 Materials Engineer – Kristen Stahl (Electronic File)
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District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: October 31, 2011

File: 07-LA-5- PM 31.32/31.38
07-1218W1
I-5 Empire Interchange
Retaining Wall No.1655

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1655

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 4, 2011 and Wall General Plan and Structure Plans (plotted October 27, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1655 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1655 will be constructed along northbound I-5, north of Buena Vista Street between post miles 31.32 and 31.38. Retaining Wall No. 1655 is a Caltrans Standard Type 1 retaining wall and will be located near the base of the existing freeway slope to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.505 (based on updated metric plans) dated February 15, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1655 will be located on the southbound side of Route 5 at the Buena Vista St S/B On-Ramp. The structure will consist of a Type 1 wall with a type 736A concrete barrier along the top, at roadway level. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I plus 2 foot level surcharge) to accommodate the freeway widening.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 26 to 8 feet with an approximate length of 338.6 feet located from RW LOL Station 653+53.40 to Station 656+91.97 (approximate 96.78 ft. Lt. of Sta. 1653+49.29 to 170.06 ft. Lt. of Sta. 1656+66.49 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 and 2 for Retaining Wall No. 1655 plus additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)
From	To			
STA 653+53.40	STA 654+25.00	26	14.25	670.08
STA 654+25.00	STA 655+00.00	24	13.25	671.75
STA 655+00.00	STA 655+50.00	20	11.0	673.75
STA 655+50.00	STA 655+75.00	16	9.0	677.67
STA 655+75.00	STA 656+00.00	14	8.0	677.67
STA 656+00.00	STA 656+50.00	12	7.25	677.83
STA 656+50.00	STA 656+75.00	10	6.25	677.83
STA 656+75.00	STA 656+91.97	8	5.25	677.83

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from July 14 to August 18, 2005. The field investigation included drilling two 8-inch outer diameter hollow-stem auger and two 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5-foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rigs were used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-4	1656+59.14	127.6 Lt.	686.9	76.5	7/14-15/05
05-13	1655+22.56	46.3 Lt.	695.5	76.5	8/1/05
04-5	1652+25.30	63.8 Lt.	673.4	100.0	6/23/04
05-38	1653+36.50	111.1 Rt.	692.3	51.5	8/18/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	4
Moisture Content	CTM 212, 226	4
Corrosion	CTM 417, 422, 643,532	12

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The current location of proposed Wall 1655 is an embankment. The embankment has an approximate slope of 1V:2H and is vegetated.

The boring logs were used to develop a continuous soil profile with depth for the wall location. The upper 1-18 feet of the borings were logged as fill. The fill was generally composed of from loose to dense silty sand, and gravel with sand. The top of native material was logged at an elevation of about 677-679 feet in the borings. The native alluvium was generally composed of from medium dense to dense sandy silt and silty sand, with gravel lenses throughout. There were also lean clayey sand lenses in Borings 05-13 and 04-5.

Groundwater

Groundwater was not encountered in auger Boring 05-13 drilled for this study to maximum depth of 76.5 feet (dry down to at least elevation +619.0 ft.). During the previous investigation in 1957 for nearby bridge 53-1110, Buena Vista/Winona UC (As Built LOTB plan dated September 1961), ground water was not encountered down to an approximate elevation of 592 ft, the maximum depth obtained. In addition, no ground water was encountered on tape measured down to the caving depth of 68.2 ft at elevation 599.7 ft within cone penetrometer hole B-6.

SCOUR

There is no potential scour at the site.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1655

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-4	0-1.5	5400	6.7	165	0
05-4	1.5-3.0	6700	7.6	60	240
05-4	3.0-9.1	7100	8.0	45	159
05-4	10.7-15.2	1100	8.5	60	21
05-4	16.8-23.3	6100	8.7	45	0
05-13	9.9-15.2	7400	8.3	75	12
05-13	35.3-40.3	9500	8.3	120	7
04-5	5.0-26.5	7500	7.58	ND	ND
04-5	26.5-41.5	4000	7.58	ND	ND
04-5	41.5-61.5	8900	7.58	ND	ND
04-5	61.5-81.5	2400	7.39	ND	ND
04-5	85.0-92.5	9100	7.63	ND	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w=6.75, is the nearest seismic source from the project site. Based on Weber (1980), this reverse type fault is located about 0.6 km east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 76.5 feet (dry down to at least elevation +619.0 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall (Case I: 2 ft level surcharge). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1655

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To				
STA 653+53.40	STA 654+25.00	26	14.25	670.08	5.3
STA 654+25.00	STA 655+00.00	24	13.25	671.75	4.9
STA 655+00.00	STA 655+50.00	20	11.0	673.75	4.3
STA 655+50.00	STA 655+75.00	16	9.0	677.67	3.5
STA 655+75.00	STA 656+00.00	14	8.0	677.67	3.3
STA 656+00.00	STA 656+50.00	12	7.25	677.83	2.8
STA 656+50.00	STA 656+75.00	10	6.25	677.83	2.5
STA 656+75.00	STA 656+91.97	8	5.25	677.83	2.2

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the new fill embankment slope was evaluated using the computer program Xstabl version 5 under both static and pseudo-static conditions. One critical cross section at the maximum wall height was used to analyze the global stability. Based on subsurface information collected via our field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The result yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Structural Backfill	34	120	0
Existing fill/Alluvium	32	115	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

3. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May, 2006). Imported materials used to construct the new fill embankment should be tested during grading to assess their expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft. of the subgrade elevation.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

If you have any questions, please call Kristopher Barker at (213) 620-2334 or Shiva Karimi at (213) 620-2146.

Prepared by: Date: 10/31/11

Kristopher Barker

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Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: December 20, 2011

File: 07-LA-5- PM 31.42/31.65
07-1218W1
Empire Interchange
Retaining Wall No.1662

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1662

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 23, 2011 and Wall General Plan and Structure Plans (plotted November 1 through 22, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for the proposed Retaining Wall No. 1662 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1662 will be constructed along northbound I-5, north of Buena Vista-Winona Undercrossing Bridge No. 53-1110 between post miles 31.42 and 31.65. Retaining Wall No. 1662 consist of two segments; 1) Sound Wall Masonry Block on Type 736S/SV (MOD) concrete barrier (Station 659+25.00 to Station 661+42.83 RW/SW LOL), and 2) Masonry Block Soundwall on Retaining Wall (Type 1SWB) (Station 661+42.83 to Station 668+16.42 RW/SW LOL). Retaining Wall 1662 will be located along Buena Vista NB On-Ramp to NB I-5, and northbound I-5 east shoulder to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;

c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall;
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Soundwall Retaining Wall No.506 (based on updated metric plans) dated October 22, 2007 (Revised April 2, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1662 will be constructed along Buena Vista NB On-Ramp to NB I-5, and northbound I-5 east shoulder and will consist of two segments; 1) A Sound Wall Masonry Block on Type 736S/SV (MOD) concrete barrier supported by 16 inch diameter CIDH piles (Case II: Standard Plan B15-6), and 2) A Masonry Block Soundwall on Retaining Wall (Type 1SWB). The beginning of the proposed Retaining Wall 1662 will start at the end of an existing soundwall. Existing 24 inch diameter C.I. Wtc class with 6 inch concrete encasement will be located at the footing elevation of the proposed wall at approximate Station 666+15 to Station 666+20 RW/SW LOL.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The segment 1 wall height is 14 feet with a total length of 217. 83 feet located from RW/SW LOL 659+25.00 to Station 661+42.83 (24.06 ft Rt. Sta. 659+25.00 to 22.0 ft Rt. Sta. 1661+44 " BVNBON1"). The segment 2 wall height is from 8 to 16 feet with a total length of 673.58 feet located from Station 661+42.83 to Station 668+16.42 RW/SW LOL (22.0 ft Rt. Sta. 1661+44 " BVNBON1" to 93.72 ft Rt. Station 1668+09.53 "NBRTE5").

The location and geometric layout data for the wall is shown on the General Plan, Structure Plan Nos. 1 through 6, Miscellaneous Details, and Structure Details Nos.1 and 2 for Retaining Wall No. 1662. Additional soundwall details are shown in Table 1, below.

Table 1A –Summary of wall Information (Segment 1)

SW LOL Station (ft)		Wall Type & Concrete Barrier	Wall Design Height (ft)	He (ft)	Approx. Bottom of Concrete Barrier Elev. (ft)	Type of Foundation
From	To					
STA 659+25.00	STA 661+42.83	Masonry Block on MOD 736 SV Concrete Barrier (Case II)	14	4	694.05 to 698.29	16 inch dia. CIDH Piles

Table 1B –Summary of wall Information (Segment 2)

SW/RW LOL Station (ft)		Wall Type & Concrete Barrier	Sound Wall Design Height (ft)	Retaining Wall Design Height (ft)	Approx. Bot. of Footing Elev. (ft)	Type of Foundation
From	To					
STA 661+42.83	STA 661+90.83	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	8	691.08	Spread Footing
STA 661+90.83	STA 662+38.83	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	10	689.08	Spread Footing
STA 662+38.83	STA 663+34.83	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	688.50	Spread Footing
STA 663+34.83	STA 664+30.83	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	689.83	Spread Footing
STA 664+30.83	STA 665+26.83	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	691.17	Spread Footing
STA 665+26.83	STA 665+88.12	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	692.75	Spread Footing
STA 665+88.12	STA 666+62.26	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	16	690.73	Spread Footing
STA 666+62.26	STA 667+22.09	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	694.92	Spread Footing
STA 667+22.09	STA 667+70.77	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	695.83	Spread Footing
STA 667+70.77	STA 668+16.42	Masonry Block on Concrete Barrier Type 736A (MOD) on Retaining Wall (Type 1SWB)	14	12	696.58	Spread Footing

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Structure Details No.1 and 2. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from July 15, 2005 to November 08, 2005. The field investigation included drilling four 8-inch outer diameter hollow-stem auger borings and one Cone Penetration Test (CPT). Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans Drilling Services and Prosonic/Tri County Drilling operated drill rig models were used at boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The locations and elevations of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-35	1658+05.85	118.98 Rt.	691.9	71.5	8/17/05
05-3	1660+21.10	74.8 Lt.	696.5	76.5	7/15/05
05-57 (CPT)	1661+18.50	89.82 Rt.	698.7	59.1	10/20/05
05-40	1664+65.70	73.69 Rt.	703.9	81.5	8/18-19/05
05-62	1667+80.40	67.02 Rt.	709.2	81.5	11/7-8/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Corporation's soils laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	3
Moisture Content	CTM 212, 226	3
Corrosion	CTM 417, 422, 643,532	10

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site is generally composed of a top fill layer, from 1 to 21 feet thick, underlain with alluvium to the maximum 81.5 feet depth drilled. Top fill materials consist of from medium dense to very dense silty sand, and sand with gravel and up to 2 inch rock pieces. Underlying alluvium consist of from medium dense to very dense silty sand and sand with fine to coarse gravel.

Groundwater

Groundwater was not encountered in all auger borings drilled for this study to maximum depth of 81.5 feet (dry down to at least elevation +622.4 ft.).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1662

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-35	2 to 3.6	3800	7.5	45	39
05-35	30 to 47	9900	8.0	30	27
05-35	32 to 34	8000	7.2	45	18
05-35	50	5700	8.6	45	30
05-40	10 to 15	5100	9.0	60	33
05-40	25 to 47	20000	8.9	45	9.0
05-40	55 to 77	1700	8.9	45	3.0
05-62	0 to 25	4500	9.0	45	36
05-62	25 to 50	4500	9.1	45	12
05-62	50 to 90	11000	9.0	45	ND
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 81.5 feet below the surface (lowest elevs. +622.4 ft) in Boring 05-40. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

The following recommendations are based on 1) Updated Retaining Wall No. 1662 General Plan, Structure Plan Nos. 1 through 6 (plotted November 1 through 22, 2011) provided by Mr. Jorge Estrada of Office of Structure Design, Branch 18, and 2) Results of laboratory testing and field investigation completed from July 15, 2005 to November 08, 2005, by OGDS1 and URS consultants.

Table 5A–Retaining Wall No.1662 Data (Segment 1)

SW LOL Station (ft)		Standard Plan Sheet No. /Case No.	Wall Design Height (ft)	He (ft)	Bottom of Concrete Barrier Elev. (ft)	Wall Type/Foundation
From	To					
STA 659+25.00	STA 661+42.83	B15-6/ Case II	14	4	694.05 to 698.29	Masonry Block On Type 736 SV Barrier on CIDH Piles

Table 5B - Retaining Wall No. 1662 Spread Footing Data (Segment 2)

SW/RW LOL Station (ft)		Retaining Wall Design Height (ft)	Footing Width (ft)	Bottom of Footing Elev. (ft)	Ultimate Bearing Capacity Stem with Haunch (ksf)
From	To				
STA 661+42.83	STA 661+90.83	8	8.0	691.08	5.5
STA 661+42.83	STA 662+38.83	10	8.75	689.08	6.1
STA 662+38.83	STA 663+34.83	12	9.75	688.50	6.6
STA 663+34.83	STA 664+30.83	12	9.75	689.83	6.6
STA 664+30.83	STA 665+26.83	12	9.75	691.17	6.6
STA 665+26.83	STA 665+88.12	12	9.75	692.75	6.6
STA 665+88.12	STA 666+62.26	16	12.0	690.73	8.1
STA 666+62.26	STA 667+22.09	12	9.75	694.92	6.6
STA 667+22.09	STA 667+70.77	12	9.75	695.83	6.6
STA 667+70.77	STA 668+16.42	12	9.75	696.58	6.6

Minimum toe cover of 1.5 foot is recommended over the spread footings.

Existing 24 inch diameter C.I. Wtc class with 6 inch concrete encasement is located at the footing elevation of the proposed wall at approximate Station 666+15 to Station 666+20 RW/SW LOL. Based on our communications with Mr. Jorge Estrada, OGDS1 understands that existing 24 inch C.I. Wtc class will be above the proposed footing influence zone (limits established by inclined planes sloping 1H:1V out and down from bottom edges of the spread footing). OGDS1 also understands that wall section directly above the concrete encasement will be supported structurally (with additional reinforcement) and grade beam will not be used.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section was used to analyze the global stability. Based on subsurface information collected via Caltrans field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 32 degrees and a minimum in situ density of 125 pcf, based on the material compacted to at least 90 percent relative compaction. The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	**Thickness (ft)**	**Friction Angle (Degree)**	**In situ Density (lbs/ft³)**	**Cohesion (psf)**
Fill	6	32	125	0
Alluvium	>50	32	125	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May 2006). If imported materials are used to construct the new fill embankment, the material should be tested during grading to assess expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft of the roadbed subgrade elevation.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

7. No ground water is anticipated at the CIDH boring excavations.

8. Caving is anticipated during CIDH boring excavations. Prior to placement of concrete, the interior surface of the shaft including the bottom should be cleaned of residue from drilling operations.

9. The drilling of the CIDH piles, the placement of the rebar cage, and concrete pour shall be completed at the same day.

If you have any questions, please call Akbar Mehrazar (949) 440-2315 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 12/20/11

A Mehrazar

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Transportation Engineer
Office of Geotechnical Design – South 1
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PCE (District 07) – Jan Rutenbergs (Electronic File)
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District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbie Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

To: MR.MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Attn: Mr. Jorge Estrada

Date: November 30, 2011

File: 07-LA-05-PM 31.43/31.6
EA: 07-1218W1
I-5 Empire Interchange
Retaining Wall 1665

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Recommendations for Retaining Wall 1665

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated November 28, 2011 and Wall General Plan and Structure Plans (plotted November 28, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1665 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1665 will be constructed along southbound I-5 and north of Buena Vista Street southbound off-ramp. Retaining Wall No. 1655 is a Caltrans Standard Type 1 retaining wall and will be located near the base of the existing freeway slope to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.507 (based on updated metric plans) dated December 21, 2006 (Revised March 12, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the City of Burbank. The Empire interchange project will extend and widen Empire Avenue beneath Route 5, realign and elevate the SCCRA/Metrolink railroad tracks, and add high occupancy vehicle (HOV) lanes on Route 5 (one lane in each direction).

Retaining Wall No. 1665 will be located on the southbound side of Route 5 at the Buena Vista St. SB Off-Ramp. The structure will consist of a Type 1 wall with a type 736A concrete barrier along the top, at roadway level. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I plus 2 foot level surcharge) to accommodate the freeway widening.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 16 to 4 feet with an approximate length of 1206.25 feet located from RW LOL Station 658+20.99 to Station 670+27.27 (approximate 179.01 ft. Lt. of Sta. 1658+48.05 to 95.82 ft. Lt. of Sta. 1670+38.19 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 through 5 for Retaining Wall No. 1665. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	To			
658+20.99	658+51.24	4	3.25	678.42
658+51.24	658+99.24	8	5.25	678.42
658+99.24	659+71.24	10	6.25	678.42
659+71.24	660+19.24	12	7.25	679.42
660+19.24	660+67.24	14	8.0	679.42
660+67.24	661+63.24	16	9.0	680.25
661+63.24	662+11.24	16	9.0	681.33
662+11.24	663+07.24	16	9.0	682.42
663+07.24	663+55.24	16	9.0	683.50

663+55.24	664+03.24	16	9.0	685.08
664+03.24	664+51.24	16	9.0	686.50
664+51.24	665+47.24	16	9.0	687.50
665+47.24	666+43.24	16	9.0	689.50
666+43.24	667+39.24	16	9.0	689.50
667+39.24	668+35.24	14	8.0	693.08
668+35.24	669+31.24	14	8.0	694.33
669+31.24	670+27.27	16	9.0	695.33

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from July 14, 2005 through January 1, 2006. The field investigation included drilling three 8-inch outer diameter hollow-stem auger and three 4.5-onch diameter mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5-foot intervals during drilling. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans and Prosonic drill rigs were used at all boring locations. A Caltrans engineer or a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
06-65	1656+72.9	422.7 Lt.	681.8	72.0	1/4/06
05-4	1656+59.1	127.6 Lt.	686.9	76.5	7/15/05
05-3	1660+21.12	-74.840 LT	696.5	76.5	7/15/05
05-2	1664+17.28	-59.053 LT	703.4	71.5	7/14/05
05-58	1667+48.13	-62.110 LT	708.5	70.3	11/1/05
05-1	1670+02.35	-66.724 LT	713.0	76.5	7/14/05

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Corrosion	CTM 417, 422, 643,532	11

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Surface Conditions

The current location of proposed Wall 1665 is an embankment. The embankment has an approximate slope of 1V:2H and is vegetated.

The boring logs were used to develop a continuous soil profile with depth for the wall location. The upper 1 to 17 feet of the borings were logged as fill. The fill was generally composed of loose to very dense silty sand with gravel. The top of native material was logged at an elevation of about 680 to 696 feet in the borings. The native alluvium was generally composed of loose to very dense silt with gravel, silty sand, sand, and gravel with sand. There were gravel lenses throughout the native material.

Groundwater

During the previous investigation in 1957 for nearby bridge 53-1110, Buena Vista/Winona UC (As Built LOTB plan dated September 1961), ground water was not encountered down to an approximate elevation of +592 ft, the maximum depth obtained. In addition, no groundwater was encountered on tape measured down to the caving depth of 68.2 ft at elevation +599.7 ft within cone penetrometer hole B-6.

Of the recent borings, the lowest elevation drilled to was approximately +610.4 feet at boring 05-4. No groundwater was encountered.

CORROSION

The results of the laboratory tests determined that the soils at this site are not corrosive to metal and reinforced concrete. Corrosion resistant design and construction materials are not necessary. Laboratory test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1665

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-4	5.0	5400	6.7	165	0
05-4	5.0-10.0	6700	7.6	60	240
05-4	10.0-30.0	7100	8.0	45	159
05-4	35.0-50.0	1100	8.5	60	21
05-4	55.4-76.9	6100	8.7	45	0
05-1	0-20	2300	8.1	60	27
05-1	5-10	8650	8.5	45	0
05-1	10-30	1400	8.5	45	0
05-58	35-50	2900	9.7	60	102
05-58	55-76.5	11,000	9.8	60	0
05-58	10-15	11,000	8.6	60	0
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The project site is located in a seismically highly active region of Southern California. Based on the Caltrans 1996 Seismic Hazard Map (CSHM) the Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude M_w=6.75, is the nearest seismic source from the project site. Based on Weber (1980), this reverse type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 76.5 feet (dry down to at least elevation +620 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 wall spread footings are recommended for retaining wall support as existing soils are adequate to support the wall (Case I: 2 ft level surcharge). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1665

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To				
658+20.99	658+51.24	4	3.25	678.42	1.7
658+51.24	658+99.24	8	5.25	678.42	2.2
658+99.24	659+71.24	10	6.25	678.42	2.5
659+71.24	660+19.24	12	7.25	679.42	2.8
660+19.24	660+67.24	14	8.0	679.42	3.3
660+67.24	661+63.24	16	9.0	680.25	3.5
661+63.24	662+11.24	16	9.0	681.33	3.5
662+11.24	663+07.24	16	9.0	682.42	3.5
663+07.24	663+55.24	16	9.0	683.50	3.5
663+55.24	664+03.24	16	9.0	685.08	3.5
664+03.24	664+51.24	16	9.0	686.50	3.5
664+51.24	665+47.24	16	9.0	687.50	3.5
665+47.24	666+43.24	16	9.0	689.50	3.5
666+43.24	667+39.24	16	9.0	689.50	3.5
667+39.24	668+35.24	14	8.0	693.08	3.3
668+35.24	669+31.24	14	8.0	694.33	3.3
669+31.24	670+27.27	16	9.0	695.33	3.5

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section at the maximum wall height was used to analyze the global stability. Based on subsurface information collected via our field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The result yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Structural Backfill	34	120	0
Native Soil	32	115	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

3. All earthwork is expected to be carried out by conventional equipment. Fill placed on sloping ground shall be properly keyed and benched into existing ground and placed as specified in Section 19-6 of the Caltrans Standard Specifications (May, 2006). Imported materials used to construct the new fill embankment should be tested during grading to assess their expansion potential. Only non-expansive soils or soils having a low expansion potential (EI: Expansion Index <50) should be used for new fill placed within 3 ft. of the subgrade elevation.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

For further information, please contact Kristopher Barker at 213-620-2334 or Shiva Karimi at 213-620-2146.

Prepared by: Date: 11/30/11

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PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

Memorandum

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: October 27, 2011

File: 07-LA-5- PM 31.60/31.66
07-1218W1
Empire Interchange
Retaining Wall No.1670

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Retaining Wall No. 1670

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 4, 2011 and Wall General Plan and Structure Plans (plotted October 12, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for proposed Retaining Wall No. 1670 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Retaining Wall No. 1670 will be constructed along northbound I-5, north of Buena Vista Street between post miles 31.60 and 31.66. Retaining Wall No. 1670 is a Caltrans Standard Type 1 retaining wall and will be located near the base of the existing freeway slope to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluations and recommendations for foundation design and construction of the proposed retaining wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed retaining wall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);
d. Field investigation and laboratory testing;

e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall; and
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Retaining Wall No.508 (based on updated metric plans) dated April 2, 2009.

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Retaining Wall No. 1670 will be constructed right of Centerline I-5 (northbound) and will consist of a Type 1 wall predominantly with concrete barrier at roadway level, and chain link railing on top of the wall. Both sloping ends of the wall will include the chain link railing on top of the wall without the concrete barrier. The proposed wall is located near the base of the existing embankment slope and will retain proposed embankment fill (Case I plus 2 foot level surcharge) to accommodate the freeway widening. The proposed wall will also be located parallel to the existing Burbank Western Channel with minimum horizontal distance between the edge of proposed retaining wall footing to the channel wall footing estimated at 14.3 ft.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The retaining wall height ranges from 22 to 12 feet with an approximate length of 298.1 feet located from RW LOL Station 667+73.76 to Station 670+71.84 (approximate 129.5 ft Rt of Sta. 1667+73.76 to Sta. 1670+71.84 Route 5 Centerline). The location and geometric layout data for the wall is shown on the General Plan and Structure Plan Nos. 1 and 2 for Retaining Wall No. 1670. Additional wall and footing details are shown in Table 1, below.

Table 1 –Summary of Retaining Wall Information

RW LOL Station (ft)		Wall Type & Concrete Barrier	Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)
From	**To**				
STA 667+73.76	STA 667+91.84	Type 1	12	7.25	685.16
STA 667+91.84	STA 668+09.92	Type 1	20	11.00	684.16
STA 668+09.92	STA 668+42.00	Type 1 and 736A Conc. Barrier	22	12.08	684.42

STA 668+42.00	STA 669+06.10	Type 1 and 736A Conc. Barrier	22	12.08	685.42
STA 669+06.10	STA 670+02.00	Type 1 and 736A Conc. Barrier	20	11.00	686.75
STA 670+02.00	STA 670+38.26	Type 1 and 736A Conc. Barrier	20	11.00	687.92
STA 670+38.26	STA 670+55.13	Type 1	20	11.00	687.92
STA 670+55.13	STA 670+71.84	Type 1	12	7.25	689.50

Caltrans 2004 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Tables 1 and 5. The 2004 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from August 18 through November 08, 2005. The field investigation included drilling two 8-inch outer diameter hollow-stem auger borings. Standard Penetration Tests (SPT's) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

URS and Prosonic/Tri County Drilling operated drill rig models CME 85 and CME 75 with 8-inch hollow stem augers used at all boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-62	1667+80.4	67.0 Rt.	709.2	81.5	11/7-8/2005
05-39	1671+13.6	64.2 Rt.	714.6	81.5	08/18/2005

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Test Performed
Mechanical Analysis	CTM 201, 202, 203	1
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	6

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site consists of approximately 22 feet of embankment fill generally composed of from dense to very dense silty sand with gravel and sporadic cobbles and sand interlayers. Underlying alluvium is composed of from dense to very dense silty sand, silty sand with gravel, and sand with silt, clayey sand and minor gravel with sand interbeds. Existing embankment side slopes are 1V:2H.

Groundwater

Groundwater was not encountered in auger Boring Nos. 05-39 and 05-62 drilled for this study to maximum depths of 81.5 feet (dry down to at least elevation +633.1 to +627.7 ft.). In the vicinity, DWR wells (01N14W03F03S and 01N14W03F06S) located near Buena Vista Street/Winona Avenue intersection show groundwater measurements below the surface vary from 211.8 to 167.5 ft depth corresponding to approximate elevations +471.2 to historically high +515.5 ft NAVD 88. No dates were provided but the wells had 35 to 14 measurements taken.

SCOUR

There is no potential scour at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Retaining Wall No. 1670

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05 - 62	0 – 25.0	4500	9.0	45	36
05 - 62	25.0 – 50.0	4500	9.1	45	12
05 - 62	50.0 – 90.0	11000	9.0	45	ND
05 – 39	3.0 – 4.6	5300	9.1	18	45
05 – 39	30.0 – 56.5	3800	9.0	30	12
05 – 39	60.0 – 81.5	19000	9.3	45	3
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATIONS

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered (auger borings were dry) to at least a depth of 81.5 feet below the surface (elevations +6331.1 and +627.7 ft) in Boring Nos. 05-39 and 05-62. The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

A Type 1 retaining wall is considered the best solution for retaining soils for I-5 Freeway widening at this location. Standard Type 1 spread footings are recommended for retaining wall support as existing soils are adequate to support the wall (Case I: 2 ft level surcharge). Based on results of laboratory testing and average corrected SPT "N" values obtained from the field investigation, ultimate bearing capacities were calculated with allowable bearing capacities for subsurface soils at the project site summarized in Table 5, below.

Table 5– Spread Footing Data for Retaining Wall No.1670

RW LOL Station (ft)		Wall Design Height (ft)	Footing Width (ft)	Bot. of Footing Elev. (ft)	Gross Allowable Soil Bearing Pressure ASD[1] (q_{all}) (ksf)
From	To				
STA 667+73.76	STA 667+91.84	12	7.25	685.16	2.8
STA 667+91.84	STA 668+09.92	20	11.00	684.16	4.3
STA 668+09.92	STA 668+42.00	22	12.08	684.42	4.6
STA 668+42.00	STA 669+06.10	22	12.08	685.42	4.6
STA 669+06.10	STA 670+02.00	20	11.00	686.75	4.3
STA 670+02.00	STA 670+38.26	20	11.00	687.92	4.3
STA 670+38.26	STA 670+55.13	20	11.00	687.92	4.3
STA 670+55.13	STA 670+71.84	12	7.25	689.50	2.8

Notes: Allowable Stress Design, (ASD). The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Gross Allowable Soil Bearing Pressure, (q_{all}). The Ultimate Soil bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Gross Allowable Soil Bearing Pressure, (q_{all}).

A minimum toe cover of 1.5 feet is recommended over the spread footings.

The proposed retaining wall spread footing will be a minimum horizontal distance of at least 14 feet from the existing concrete lined Burbank Western Channel. With this minimum horizontal distance from the channel and at current bottom of footing elevations for the proposed wall, no additional load will be imposed by the added retained soil and wall on the existing channel wall or footings.

Settlement

The anticipated settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement. The settlement period will be short term and will be essentially completed during construction.

SLOPE STABILITY

The global stability of the proposed new fill embankment slope was evaluated using the computer program XSTABL version 5 under both static and pseudo-static conditions. One critical cross section (RW LOL Station 668+42 to Station 669+06) was used to analyze the global stability. Based on subsurface information collected via Caltrans field investigation, the soil profile and corresponding strength parameters used in performing the stability analysis are presented in Table 6, below. The fill material is assumed to have a minimum friction angle of 32 degrees and a minimum in situ density of 125 pcf, based on the material compacted to at least 90 percent relative compaction. Underlying alluvial material possesses similar soil parameters. For the purpose of slope stability analysis, groundwater was not encountered down to at least elevation +627.7 ft (81.5 ft depth). The stability analysis yields a factor of safety greater than the minimum acceptable values of 1.5 and 1.1 for static and seismic condition, respectively.

Table 6 – Idealized Soil Parameters for Slope Stability Analysis

Materials (Soil)	Thickness (ft)	Friction Angle (Degree)	In situ Density (lbs/ft^3)	Cohesion (psf)
Fill	22	32	125	0
Alluvium	59.5	32	125	0

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the footing excavation depths.

2. All earthwork is expected to be carried out by conventional equipment.

3. Free water shall not be allowed to stand in any excavations. If excavations become flooded, a minimum of 6 inches of soil below footing grade shall be removed and replaced or recompacted per Caltrans specifications.

4. Based on the soil types encountered during Caltrans investigation, a slope ratio of 1V: 1H or flatter for the temporary back cut slope can be considered for construction. If there are additional space constraints due to construction or traffic concerns, temporary shoring may be utilized to accommodate a steeper slope for the excavation of the proposed footing.

5. The on-site soils are considered suitable for being used as structure backfill. Backfill material should be cleaned of any debris.

6. Quality control should be practiced to ensure that bottom of the footing excavation is level and clear of any loose debris. Should any large detached rock fragment or foreign object be found at the bottom of the footing elevations, the contractor should be prepared to remove and replace them with granular material at 95% Relative Compaction or lean concrete.

If you have any questions, please call Kevin Lai (213) 620-2344 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 10/27/11

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Branch D

Supervised by: Date: 10/27/11

P Shiva Karimi

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Structure Construction R.E. Pending (Electronic File to: RE_Pending_file@dot.ca.gov)
PCE (District 07) – Jan Rutenbergs (Electronic File)
DES Office Engineer, Office of PS&E – (Electronic File)
District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbic Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

M e m o r a n d u m

Flex your power!
Be energy efficient!

To: MR. MIKE POPE, CHIEF
Design Branch 18
Office of Structure Design

Date: November 4, 2011

File: 07-LA-5- PM 30.00/30.07
07-1218W1
Empire Interchange
Soundwall No.1584

Attention: Mr. Jorge Estrada

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
Geotechnical Services
Office of Geotechnical Design – South 1
Branch D

Subject: Foundation Report for Soundwall No. 1584

INTRODUCTION

Based on the request from the Office of Structure Design, Branch 18, dated October 5, 2011 and Wall General Plan and Structure Plans (plotted November 02, 2011), a Foundation Report was prepared by the Office of Geotechnical Design South 1 (OGDS1) for the proposed Soundwall No. 1584 as part of the Interstate 5 (I-5) Improvement and Bridge Widening project.

Soundwall No. 1584 will be constructed along northbound I-5, north of Burbank Boulevard between post miles 30.00 and 30.07 and over/adjacent proposed Burbank Western Channel cover slab. Soundwall No. 1584 is a Caltrans standard Sound Wall Masonry Block on Type 736SV / SV (MOD) concrete barrier and will be located between northbound I-5 east shoulder and Leland Avenue to accommodate the planned freeway widening within the City of Burbank, Los Angeles County, California.

SCOPE OF WORK

The purpose of OGDS1's geotechnical investigation was to evaluate site soil conditions and to provide seismic evaluation and recommendations for foundation design and construction of the proposed sound wall. The scope of work for the current study included performing the following tasks:

a. Review of the pertinent literature and current plans;
b. Field reconnaissance by an engineer to observe the existing conditions at the proposed soundwall site;
c. Project coordination with Structures Design and D07 Design, Underground Service Alert, Caltrans Maintenance and Drilling Services, City of Burbank, Traffic Control Contractor, and Laboratory Contractor (URS);

d. Field investigation and laboratory testing;
e. Interpretation of subsurface soils and groundwater conditions at the site of the proposed wall;
f. Engineering analyses and preparation of this report to present geotechnical recommendations for foundation design of the proposed wall.

This Foundation Report supersedes the previous Foundation Recommendations for Soundwall No.484 (based on updated metric plans) dated January 24, 2017 (Revised February 26, 2009).

PROJECT DESCRIPTION

This project is part of planned improvements to Route 5 in the city of Burbank. The Empire Interchange project will extend and widen Empire Avenue beneath Rte 5, realign and elevate the SCCRA/Metro-link Railroad tracks, and add HOV (high occupancy vehicle) lanes on Rte. 5 (one lane in each direction).

Soundwall No. 1584 will be constructed east of northbound I-5 and will consist of two segments; 1) A masonry block soundwall on Type 736SV (MOD) concrete barrier supported by structural connection to the top of the existing Burbank Western Channel concrete slab, 2) A masonry block soundwall on Type 736SV concrete barrier supported by 16 inch diameter CIDH piles (Case II: Standard Plan B15-6). Existing Soundwall No. 862 located on west side of proposed soundwall will be removed to accommodate the freeway widening. Existing Scott Road Drainage structure (rectangular concrete conduit) is located adjacent to the existing Burbank Channel Culvert near Station 1584+60 Route 5 Centerline.

Elevations provided on current plans and recommendations are based on NAVD88 datum.

The soundwall height is 16 feet with a total length of 359.08 feet located from SW LOL Station 583+64.46 to Station 587+23.54 (108.85 Rt. of Sta. 1583+65.36 to 123.0 ft Rt. Of Sta. 1587+26.77 Route 5 Center line). The location and geometric layout data for the wall is shown on the General Plan, Structure Plan Nos. 1 and 2, and Structure Details No.1 for Soundwall No. 1584. Additional soundwall details are shown in Table 1, below.

Table 1 –Summary of Soundwall Information

SW LOL Station (ft)		**Wall Type & Concrete Barrier**	**Wall Design Height (ft)**	**He (ft)**	**Approx. Bottom of Concrete Barrier Elev. (ft)**	**Type of Foundation**
From	**To**					
STA 583+64.46	STA 586+14.89	Masonry block on 736 SV (MOD) Concrete Barrier	16	3	593.67 (FG of Exist. Culvert)	Concrete Slab of Existing Culvert
STA 586+14.89	STA 587+26.77	Masonry Block on 736 SV Concrete Barrier (Case II)	16	4	595.68 to 596.75	16 inch dia. CIDH Piles

Caltrans 2006 Standard Plans (metric but converted to English units) and current Structure Plans were utilized for data in Structure Details No.1. The 2006 Standard Plans are considered applicable to current foundation recommendations as the earlier studies were completed under these standards.

FIELD INVESTIGATION AND TESTING PROGRAM

Site-specific field exploration was performed from June 17, 2004 to November 09, 2005. The field investigation included drilling one 8-inch outer diameter hollow-stem auger and two 4.5-inch mud rotary borings. Standard Penetration Tests (SPTs) were performed within the borings. Blow counts (SPT N-values) were generally recorded at 5 foot intervals during drilling. The SPT's were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch I.D. sampler with a 140 lb hammer dropped 30 inches.

Caltrans Drilling Services and Prosonic/Tri County Drilling operated drill rigs were used at boring locations. Caltrans engineers and a URS engineer performed the logging of the borings.

The location and elevation of all borings were provided by D07 Surveys. Boring number, offset and stationing, ground surface elevation, boring depth, and date drilled are summarized in Table 2.

Table 2 – Summary of Borings

Boring No.	C/L Rte 5 (Prop.) Stationing	Offset from I-5 (ft)	Top of Boring Elevation (ft)	Depth (ft)	Date Drilled
05-61	1582+33.94	110.9 Rt.	593.2	88.0	11/8-9/05
05-5	1586+32.67	134.0 Rt.	596.9	25.5	7/18/05
04-4	1589+19.19	125.9 Rt.	600.4	51.5	06/17/04

LABORATORY TESTING PROGRAM

Selected soil samples were sent to URS Company's Soils Laboratory in Santa Ana, California for laboratory testing. Soil samples were tested for corrosivity, mechanical analysis, and moisture content. Laboratory tests were performed in accordance with ASTM standard procedures and California Test Methods. A laboratory test summary is shown in Table 3, below.

Table 3 – Summary of Laboratory Testing

Test	Standard	No. of Tests Performed
Mechanical Analysis	CTM 201, 202, 203	-
Moisture Content	CTM 212, 226	-
Corrosion	CTM 417, 422, 643,532	4

SITE GEOLOGYAND SUBSURFACE CONDITIONS

Regional Geology

The Rte. 5 – Burbank project is located in the Transverse Range Province in the northwestern block of the Los Angeles Basin, which includes the San Fernando Valley. The northwestern block site is bounded on the south by the Santa Monica and Raymond Hill faults, on the east and northeast by the San Gabriel Mountains, and on the west and north by the ranges included in the Ventura Basin portion of the transverse ranges. Burbank is further bounded by the Verdugo Mountains to the Northeast. A thick Cenozoic sedimentary section underlies the San Fernando Valley (synform).

Site Description and Subsurface Conditions

The site is generally composed of 1 to 10 feet of fill underlain by alluvium consisting of loose to medium dense silty/clayey sand, and sand with silt and gravel, and from stiff to hard sandy clay, clayey silt, and silt. During drilling for Boring 05-5, red brick and clay fragments of a city sewer line were encountered at approximate depth of 18.5 to 20 ft. (approx. elevations +576 to +578 ft).

Groundwater

Groundwater was not encountered in all auger borings drilled for this study to maximum depths of 88.0 feet (dry down to at least elevation +505.2 ft.).

SCOUR

There is no scour potential at the site as the nearby channel is concrete-lined.

CORROSION

Soil samples were tested for corrosion potential at URS Soils Laboratory. Results presented in Table 4 show that subsurface soils are non-corrosive to metal and reinforced concrete. Corrosion test results are presented in Table 4, below.

Table 4 – Corrosion Test Summary for Soundwall No. 1584

Boring No.	Sample Depth (ft)	Minimum Resistivity (ohm – cm)	PH	Chloride Content (PPM)	Sulfate Content (PPM)
05-61	0-30	3000	7.9	45	45
05-61	30-60	3800	8.3	60	45
05-61	60-88	6200	8.5	ND	45
05-5	15-30	2200	8.5	30	24
Corrosive Guidelines		<1000	≤5.5	≥500	≥2000

ND=Not detectable

Note: It is the practice of Caltrans Corrosion Technology Section (with the exception of MSE Walls) if the minimum resistivity of the sample is greater than 1000 ohm-cm and the pH is greater than 5.5, the sample is considered to be non-corrosive. Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm; or the PH is 5.5 or less.

SEISMIC EVALUATION

The following seismicity information was provided by Dr. Mohammed Islam on March 23, 2006 and September 16, 2005. The project site is located in a seismically highly active region of Southern California. Based on the Caltrans' 1996 Seismic Hazard Map (CSHM) the active Verdugo Fault (VDO), which is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude Mw = 6.75, is the nearest and controlling seismic source for the project site. Based on Weber (1980), this reverse/oblique type fault is located about 0.4 miles east of the project site. The median or design Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.8g based on the Sadigh et al (1997) attenuation relationships. The corresponding Peak Ground Acceleration (PGA) at the site is estimated to be about 0.7g.

LIQUEFACTION POTENTIAL

Liquefaction potential is considered low at the site. Groundwater was not encountered in all auger borings drilled for this study to maximum depths of 88.0 feet (dry down to at least elevation +505.2 ft.). The potential for other secondary seismic hazards including significant seismically induced settlement and lateral spreading are also considered low.

SURFACE FAULT RUPTURE HAZARD EVALUATION

The project site is not located within any CGS designated Earthquake Fault Zone (EFZ) or directly underlain by any active fault considered for wall design. The possibility of surface fault rupture hazard at the wall site is considered low.

FOUNDATION RECOMMENDATIONS

The following recommendations are based on 1) Updated Soundwall No. 1584 General Plan, Structure Plan Nos. 1 and 2, and Structure Details No.1 (plotted November 2, 2011) provided by Mr. Jorge Estrada of Office of Structure Design, Branch 18, and 2) Results of laboratory testing and field investigation completed from June 17, 2004 to November 09, 2005, by OGDS1 and URS consultants.

Table 5– Summary Data for Soundwall No.1584

RW LOL Station (ft)		Standard Plan Sheet No. /Case No.	He (ft)	Approx. Bottom of Concrete Barrier Elev. (ft)	Wall Type/Foundation
From	To				
STA	STA	B15-6/	3	593.67 (FG of	Masonry Block On Type 736 SV

583+64.46	586+14.89	N/A		Exist. Culvert)	(MOD) Barrier (structurally supported in top concrete slab of existing culvert)
STA 586+14.89	STA 587+26.77	B15-6/ Case II	4	595.68 to 596.75	Masonry Block On Type 736 SV Barrier on CIDH Piles

The proposed soundwall CIDH piles will be a minimum horizontal distance of at least 19 feet (+/- varies) from the existing concrete lined Burbank Western Channel. With this minimum horizontal distance from the channel and at current bottom of barrier elevations for the proposed Soundwall No. 1584, no major additional load will be imposed by the CIDH piles on the existing channel wall or footings.

Settlement

The anticipated pile settlement is less than 1 inch for both total and differential settlement, which satisfies acceptable tolerance criteria for settlement.

CONSTRUCTION CONSIDERATIONS

1. No ground water is anticipated at the CIDH boring excavations.

2. Prior to placement of concrete, the interior surface of the shaft including the bottom should be cleaned of residue from drilling operations.

3. The drilling of the CIDH piles, the placement of the rebar cage, and concrete pour shall be completed in a relatively continuous operation.

If you have any questions, please call Kevin Lai (213) 620-2344 or Shiva Karimi (213) 620-2146.

Prepared by: Date: 11/4/11

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District 07 Materials Engineer – Kristen Stahl (Electronic File)
District 07 Project Manager – Mumbie Fredson-Cole (Electronic File)
District 07 Construction R.E. Pending File (Electronic File)
District 07 Environmental Planning – Garrett Damrath (Electronic File)
District 07 Design - Charles Ton (Electronic File)

GEOTECHNICAL EXPLORATION FOR THE EMPIRE AVENUE STORM WATER PUMP STATION, NORTHWEST OF EMPIRE AVENUE AND INTERSTATE 5, BURBANK, CALIFORNIA

Prepared for:

TETRA TECH, INC.
17885 Von Karman Avenue, Suite 500
Irvine, California 92614

Project No. 603239-001

November 21, 2011

Leighton Consulting, Inc.
A LEIGHTON GROUP COMPANY

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A LEIGHTON GROUP COMPANY

November 21, 2011

Project No. 603239-001

To: Tetra Tech, Inc.
17885 Von Karman Avenue, Suite 500
Irvine, California 92614

Attention: Ms. Erica Jenkins

Subject: Geotechnical Exploration for the Empire Avenue Storm Water Pump Station, Northwest of Empire Avenue and Interstate 5, Burbank, California

In accordance with your request, Leighton Consulting, Inc. has performed a geotechnical exploration for the proposed Empire Avenue storm water pump station to be located to the northwest of Empire Avenue and Interstate 5 (I-5) in Burbank, California. This exploration was performed based on the preliminary plan and profile provided by Tetra Tech. We understand that the pump station structure will be approximately 43 feet below the existing grade. Based on our field exploration, the onsite soil consists primarily of loose to medium dense sand and silty sand. Intermittent layers of dense gravel were encountered from 30 feet to the maximum explored depth of 52 feet. Groundwater was not encountered in our boring and is expected to be below the pump station invert.

This report presents the results of our field exploration, laboratory testing, and geotechnical analyses, and provides our conclusions and recommendations for the proposed project.

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We appreciate the opportunity to work with you on this project. If you have any questions, or if we can be of further service, please call us at your convenience.

Respectfully submitted,

LEIGHTON CONSULTING, INC.

REGISTERED PROFESSIONAL ENGINEER
DJAN CHANDRA
No. 2376
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STATE OF CALIFORNIA

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DJC/gv

Distribution: (4) Addressee

TABLE OF CONTENTS

Leighton

1.0 INTRODUCTION

1.1 Site Location and Proposed Project

The proposed storm water pump station will be located in an unpaved area to the northwest of Empire Avenue and I-5. The site is an elongated land bounded to the east by I-5 off-ramp and I-5 mainline and to the west by an existing railroad tracks and Victory Place. The approximate location of the site is shown on Figure 1 - *Site Location Map*.

The proposed project includes a below-ground concrete structure with a footprint of approximately 25 feet by 40 feet that extends 43 feet below the existing grade. We understand that the finished grade in the area will be lowered to approximately 10 feet below the existing grade, which makes the bottom of the structure about 33 feet below the finished grade. A concrete pad for generator, transformer, and other equipment will also be constructed as part of the project.

We understand that the existing railroad tracks will be alleviated in the future. The new railroad embankment will be supported on a Caltrans standard retaining wall. In the area near the pump station, the retaining wall will be 26 and 28 feet high and footing for the retaining wall will be located approximately 5 feet horizontally from the pump station footprint.

1.2 Purpose and Scope of Exploration

The purpose of our geotechnical exploration was to evaluate the subsurface soil and groundwater characteristics at the project site and to provide geotechnical parameters for design and installation of the pump station. The scope of this exploration included the following tasks:

- *Background Review* – A background review was performed of readily available, relevant geotechnical and geological literature pertinent to the site. References used in preparation of this report are listed in Section 4.0.

- *Pre-Field Exploration Activities* – Boring locations were marked and Underground Service Alert (USA) was notified to locate and mark existing underground utilities prior to our subsurface exploration. Our boring was located within the property of City of Burbank. It was determined by the city that no permit was required.

- *Field Exploration* – We advanced one hollow-stem auger boring to a depth of 52 feet below existing grade on September 19, 2011. The boring was geotechnically logged and sampled using Standard Penetration Test (SPT) and California Ring samplers at selected intervals. The SPT and Ring samplers were driven into the soil with a 140-pound hammer, free falling 30 inches. The number of blows was noted for every 6 inches of sampler penetration. Relatively undisturbed samples were collected from the boring using the Ring sampler. The sampling procedures generally followed ASTM Test Method D 1586 and D 3550 for SPT and split-barrel sampling of soil. In addition to driven samples, representative bulk soil samples were also collected from the boring. Each soil sample collected was described in general conformance with the Unified Soil Classification System (USCS). The samples were sealed, packaged, and transported to our soil laboratory. The soil descriptions and depths are noted on the Log of Test Boring included in Appendix A.

- *Laboratory Tests* – Laboratory tests were performed on selected soil samples obtained during our field investigation. The laboratory testing program was designed to evaluate the physical and engineering characteristics of the onsite soil. Tests performed during this investigation include:

  - Moisture content and dry density (ASTM D 2216 and ASTM D 2937);
  - Sieve Analysis (ASTM D 422);
  - Direct Shear (ASTM D 3080); and
  - Corrosivity Suite – pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

  Results of the laboratory tests are presented in Appendix B.

- *Engineering Analysis* - The data obtained from our background review, field exploration, and laboratory testing program were evaluated and analyzed to develop the recommendations presented in this report for the proposed project.

- *Report Preparation* - The results of the exploration are summarized in this report presenting our findings and recommendations.

Project: 603239-001	Eng/Geol: DJC
Scale: 1 " = 500 '	Date: November, 2011

Base Map: ESRI Resource Center, 2010
Thematic Info: Leighton
Author: (btran)

SITE LOCATION MAP

EMPIRE AVENUE STORM WATER PUMP STATION
CITY OF BURBANK, CALIFORNIA

Figure 1

Leighton

Map Saved as V:\Drafting\603239\001\OF_2011-10-03\GIS figures\Figure-1.mxd on 11/10/2011 12:55:09 PM

2.0 FINDINGS

2.1 Subsurface Soil Conditions

The subsurface conditions subsequently described in this section of the report have been summarized for ease of interpretation. Detailed descriptions of the materials encountered in the test boring and the approximate stratigraphy are presented on the boring log in Appendix A.

The soil encountered in our boring consists mainly of sand and silty sand. The upper 20 feet of the soil is generally fine-grained sand that is loose and yellowish to olive brown in color. Below 30 feet to the maximum explored depth of 52 feet, the sand increases in consistency to medium dense and dense with depth. Intermittent layers of gravel were encountered below 30 feet.

2.2.1 Expansion Potential

The onsite soil is relatively granular. Based on our field observation, the onsite soil is expected to have a low expansion potential.

2.2.2 Corrosivity

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. As adopted by the California Building Code, specific guidelines have been established for concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The concentration of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures such as steel pipes or piles, is 500 ppm or greater as determined by California Test 532.

For screening purposes, a representative sample of the soil was tested for corrosivity potential. The test results indicate the tested soil does not pose a significant potential for sulfate attack on structural concrete (soluble sulfate concentration of less than 42 ppm). The tests indicated a chloride content of 31, pH values of 5.9, and minimum resistivity of 22,800 ohm-cm. Based on these test results, the soil is not considered corrosive to buried ferrous metal in direct contact with the soil.

2.3 Groundwater

Groundwater was not encountered in our boring to the depth of 52 feet below the existing grade during our field exploration. Our review of the seismic hazard zone report for the Burbank Quadrangle (CGS, 1998) indicates that the historically high groundwater table in the area is on the order of 40 feet below the existing grade. Groundwater is not expected to have an adverse impact on the proposed project. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff.

2.4 Faulting and Seismicity

Our review of available in-house literature indicates that there are no known active or potentially active faults traversing the site and the site is not located within a State of California designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). The principal seismic hazard that could affect the site is ground shaking resulting from an earthquake occurring along several major active or potentially active faults in southern California. According to the available fault database by United States Geological Survey and Caltrans, the closest active faults that could affect the site are the Verdugo, Sierra Madre, Hollywood, Elysian park, and Santa Monica faults located approximately 1.1, 5.3, 5.7, 5.8, and 6.3 miles, respectively, from the site.

The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the source, and the site response characteristics. Peak Horizontal Ground Accelerations (PHGA) is generally used to evaluate the intensity of ground motion. A probabilistic seismic hazard analysis was performed using the deaggregation program developed by the United States Geological Survey (USGS) National Seismic Hazard Mapping Program (2011). The analysis was conducted for a 2 percent probability of exceedance in 50 years (average return period of 2,475 years). The results of the probabilistic seismic hazard analysis indicate the modal seismic event is Moment Magnitude (M_W) 6.6 at a distance of 7 kilometers and a PHGA of 0.85g.

2.5 Secondary Seismic Hazards

Secondary seismic hazards in the region could include soil liquefaction and associated surface manifestations, earthquake-induced settlement, landsliding, seiches, and tsunamis. The potential for seismic hazards at the site is discussed below.

Liquefaction Potential - Liquefaction is a seismic phenomenon in which loose, saturated soils, generally fine-grained sands and silts, behave similarly to a fluid when subjected to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density, fine-grained, non-cohesive sandy and silty soils; and 3) high-intensity ground motion. Effects of liquefaction on level ground can include sand boils, settlement, and bearing capacity failures below structural foundations. Lateral spreading can also occur in areas of sloping ground. The site is located at the edge of the liquefaction zone on the State of California *Seismic Hazard Zones Map for the Burbank Quadrangles* (CGS, 1999). However, our subsurface exploration did not reveal the presence of a shallow groundwater table at the subject site. The historically high groundwater table at the site is approximately 40 feet below the existing grade and the soil below 40 feet is medium dense to very dense. Therefore, the potential for liquefaction occurrence and related effects at the subject site is considered low.

Seismically Induced Settlement - During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement. Settlement below bottom of the underground structure at 43 feet below the existing grade is expected to be negligible. Settlement at the proposed finished grade is estimated to be on the order of 2 inches.

Seismically-Induced Landslides - Based on the relatively flat topography of the site, the potential for seismically-induced landsliding is considered low.

Seiches and Tsunamis - Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland location of the site, seiche and tsunami risks at the site are considered negligible.

3.0 RECOMMENDATIONS

Presented below are the geotechnical recommendations for design and construction of the project. The recommendations are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction as well as proper field observation and testing during construction. The recommendations are considered minimum and may be superseded by more restrictive requirements of the architect, structural engineer, building code, or governing agencies.

3.1 Site Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional.

Site Preparation – Prior to construction, the site should be cleared of existing improvements and debris. Existing utility and irrigation lines should also be removed if they interfere with the proposed construction. Cavities resulting from removal of the existing underground structures and lines should be excavated to expose competent material before being properly backfilled and compacted.

Overexcavation and Recompaction – Foundation for the proposed underground structure at 43 feet below existing grade is expected to be supported on medium dense sand. If the soil is disturbed during excavation, it should be removed and recompacted. Footings for at-grade structures should be underlain by compacted fill. The compacted fill should extend a minimum 3 feet below bottom of the footings and a minimum 3 feet beyond outside edges of the footings. Other local conditions may be encountered which may require additional removals and recompaction. The exact extent of removals can best be determined during grading by the geotechnical engineer when direct observation and evaluation of materials are possible.

Fill Placement and Compaction – The onsite soil is suitable for use as compacted structural fill, provided it is free of debris and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be accepted by the geotechnical engineer. All fill soil should be placed in thin, loose lifts, moisture-conditioned, as necessary, to near optimum moisture content, and compacted to a minimum 90 percent relative compaction as determined by ASTM Test Method D 1557.

3.2 Seismic Design Parameters

Seismic design parameters are provided based upon the 2010 California Building Code (CBC). The following design parameters may be considered for seismic analysis of the structures proposed within the site.

Table 1 – 2010 CBC Seismic Design Parameters

Categorization/Coefficient	Design Value
Site Class	D
Short Period (0.2 second) Site Coefficient, F_a	1.0
Long Period (1 second) Site Coefficient, F_v	1.5
Design (5% damped) spectral response acceleration parameter at a period of 0.2 second, S_{DS}	1.489g
Design (5% damped) spectral response acceleration parameter at a period of 1 second, S_{D1}	0.758g

3.3 Foundation Recommendations

The bottom of the pump station at 43 feet below the existing grade can be designed as a mat foundation with an allowable bearing pressure of 6,000 pounds per square foot (psf) and a modulus of subgrade reaction of 150 pounds per cubic inch (pci). For at-grade structures, conventional shallow foundations may be used. Footings for at-grade structures should have a minimum embedment depth of 18 inches and a minimum width of 12 inches. An allowable bearing pressure of 2,000 psf may be used based on the minimum embedment depth and width. The allowable bearing value may be increased by 300 psf per foot increase in depth or width to a maximum allowable bearing pressure of 3,500 psf. The allowable bearing pressures are for total dead load and sustained live loads, and may be increased by one third when considering loads of short duration, such as those imposed by wind and seismic forces. Total static settlement is estimated to be on the order of 1 inch with a differential settlement of ½ inch over 30 feet.

Lateral loads may be resisted by friction between the footings and the supporting subgrade, and passive resistance of properly compacted backfill and/or undisturbed native soils, in combination. A friction coefficient of 0.35 may be used at the soil-concrete interface for calculating the sliding resistance. A passive pressure based on an equivalent fluid pressure of 380 pounds per cubic foot (pcf) may be used for calculating the lateral passive resistance. The lateral passive resistance can be taken into account only if it is ensured that the soil against embedded structures will remain intact with time.

3.4 Retaining Walls

Below-grade walls for the pump station should be designed using the lateral earth pressures provided in Table 2 below. These values do not contain an appreciable factor of safety, so the civil and/or structural engineer should apply the applicable factors of safety and/or load factors during design.

Table 2 – Equivalent Fluid Pressure

Condition	Level Backfill
Active	38 pcf
At-Rest	58 pcf
Passive	380 pcf (Maximum of 4,500 psf)

A soil unit weight of 120 pounds per cubic foot (pcf) may be assumed for calculating the actual weight of the soil over the wall footing. Cantilever walls that are designed to yield at least 0.001H, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.35 may be used at the concrete and soil interface.

In addition to the above lateral forces due to retained earth, surcharge due to adjacent improvements and traffic should be considered in the design of the pump station walls. We understand that footing for the proposed retaining wall for the railroad embankment will be located approximately 5 feet from the western wall of the pump station. The retaining wall will be 26 and 28 feet high with a toe pressure of 5.87 and 7.74 ksf, respectively. The resulting lateral stress on the pump station wall is presented on Figure 2. The retaining wall will also impose a downward frictional load on the pump station wall. This drag load is estimated to be on the order of 240 psf.

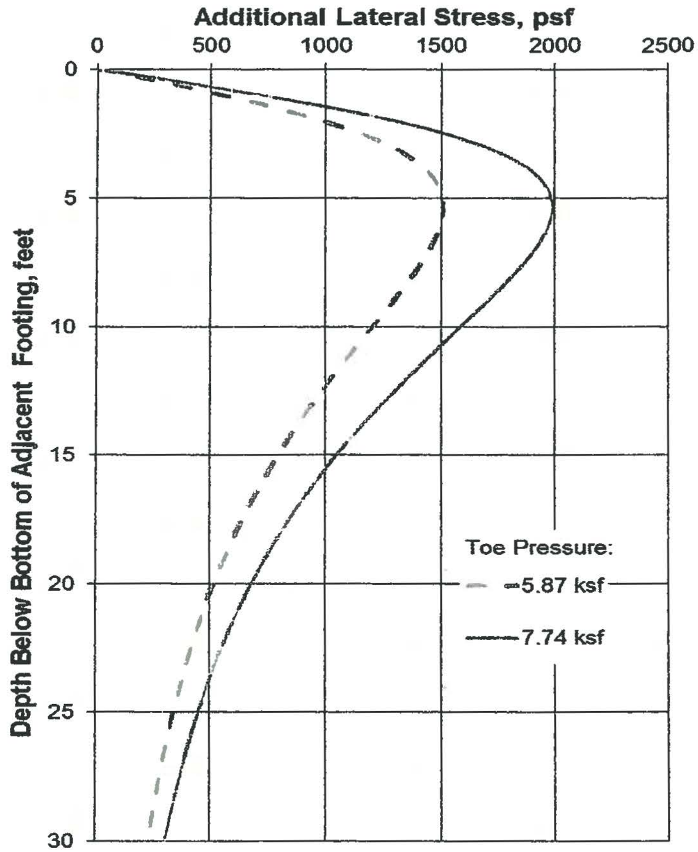

Figure 2 – Lateral Stress on Pump Station Wall from Adjacent Retaining Wall

3.5 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil are expected to have negligible exposure to water-soluble sulfates in the soil. Common Type II cement may be used for concrete construction onsite and the concrete should be designed in accordance with CBC requirements.

Based on our laboratory testing, the onsite soil is not considered corrosive to ferrous metals and reinforcing steel. No special measures are considered necessary.

3.6 Temporary Excavations

All temporary excavations, including utility trenches and retaining wall excavations, should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination between the competent person and the geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.7 Temporary Shoring

Excavation for the below-grade walls may be supported by several methods, including conventional soldier piles, sheet piles or tiebacks, to name a few. The choice should be left to the contractor's judgment since economic considerations and/or the individual contractor's construction experience may determine which method is more economical and/or appropriate. Support of all adjacent existing structures without distress is the contractor's responsibility. These shoring systems adjacent to existing structures should be designed by a California licensed civil or structural engineer following the guidelines in Caltrans *Trenching and Shoring Manual*. The contractor should forward their plans for the support system to Leighton Consulting for pre-construction review. In addition, it should be the contractor's responsibility to undertake a pre-construction survey with benchmarks and photographs of the adjacent structure(s).

The contractor should be aware of the granular nature of the soils, being careful to guard against potential for sloughing and caving of excavation sides. This is for both human safety and safety of the improvements being shored.

The contractor and shoring designer should perform additional geotechnical studies as necessary to refine the means and methods of shoring construction. As preliminary design guidelines, we present the following geotechnical parameters for shoring design, based on the assumption that grade behind and in front of the shoring will be relatively level (e.g. not for shoring at the toe or top of a cut slope):

- *Design Lateral Earth Pressures* – Unrestrained (cantilever) shoring can be designed to resist an equivalent fluid pressure of 38 pounds-per-cubic-foot (pcf), for shoring no more than approximately 15 feet in height. For braced shoring (restrained from movement at the top) a uniform pressure of 25H psf may be used, where H is the shoring height in feet.

- *Surcharges* – For cantilever shoring, one-third of any uniform vertical surcharge load should be applied in design as a uniform horizontal pressure. Restrained shoring should be designed to resist an additional uniform horizontal pressure equivalent to one-half of any vertical surcharge loads applied at the surface. Surcharges need not be included in design if the surcharge is setback behind the shoring a horizontal distance greater than the height of the shoring.

- *Soldier Piles* – Soldier piles may be assumed to have a passive resistance below the lowest adjacent excavation (bottom of footings) of 700 psf per foot of embedment. The soldier pile should be encased in concrete in firm contact with the soil. This passive pressure should not exceed 7,500 psf, and is based on the assumption that soldier piles will be spaced at least three diameters on center. Soldier pile installation can be problematic in the area, where dry sands can flow around soldier piles if lagging is not rapidly and completely installed between soldier piles, and backpacked with Controlled Low Strength Material (CLSM). Although somewhat uncommon in southern California, sheet piles may be a more appropriate shoring system than solider piles at this site.

- *Sheet Piles* – Sheet piles may be assumed to have a passive resistance below the lowest adjacent excavation (bottom of footings) of 380 psf per foot of embedment of the soldier pile driven/vibrated in firm contact with the native soils. This passive pressure should not exceed 4,000 psf.

- *Tieback Anchors* – Actual anchor capacity should be verified by testing during installation. Each anchor should be proof-loaded to 150 percent of the design load for 15 minutes. The anchor rod (including rod stretching) should not move more than 6 inches axially during application of the test load from 0 to 150 percent of the design load. At 150 percent design load, anchor movement shall not exceed 0.1 inch over 15-minute period. Excavation below the tiebacks can only proceed after all tiebacks have met the proof-load requirement. The tiebacks after testing should be locked off at design loads. A minimum of 10 percent of the tiebacks should be tested to 200 percent of the design load for 30 minutes. Additionally, at least one anchor should be tested up to 200 percent of the design load for 24 hours. Anchor movement shall not exceed 0.25 inch during the testing period, measured after the 200 percent test load is applied. Testing jack calibration must be provided to Leighton Consulting and must be no more than one month old.

As preliminary design guidelines, skin friction on tieback anchors may be assumed to be $50H_a$ psf, where H_a is the average depth in feet of the tieback anchor portion (excluding the active wedge), not to exceed 1,100 psf. Skin friction values can be increased for pressure-injected (grouted) anchors.

- <u>*Monitoring*</u> – Soldier piles should be monitored weekly for line and grade, surveyed by a California licensed Professional Land Surveyor (PLS). Survey results must be sent to Leighton Consulting, weekly, preferably by e-mail. If total horizontal deflection inward (towards the excavation) exceeds one inch, then excavation adjacent to excessively deflecting soldier/sheet pile(s) should be halted immediately, and the shoring design at that location should be reevaluated by the shoring designer, owner and Leighton Consulting. Any movement more than one inch will require remedial shoring at the location of excessive deflection, to prevent additional movement prior to further construction in that area.

Our boring was drilled with a hollow-stem auger. Therefore, we do not have empirical information regarding the potential for caving in drilled holes (e.g. soldier piles and/or tiebacks). The contractor may therefore choose to evaluate the potential for difficult drilling conditions and caving of soldier pile and tieback shafts by drilling pilot holes with the intended production drilling equipment. We expect the granular soils at this site are prone to caving.

3.8 Trench Backfill

Utility trenches can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified in-place by jetting. The native backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction.

3.9 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our recommendations are based on information available at the time the report was prepared and may change as plans are developed. Additional geotechnical investigation and analysis may be required based on final development plans. Leighton Consulting should review the foundation, grading and shoring plans, when they become available.

The nature of many sites is such that differing soil or geologic conditions can be present within small distances. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if Leighton has the opportunity to observe the subsurface conditions during grading and construction in order to confirm that our preliminary data are representative for the site. Geotechnical observation and testing should be conducted during excavation and all phases of grading operations, including the following stages:

- Upon completion of site clearing;
- During subgrade overexcavation and recompaction;
- During fill placement;
- During shoring system installation;
- After building footing excavations and prior to placement of concrete;
- During backfilling of utility trenches; and
- When any unusual or unexpected geotechnical conditions are encountered.

4.0 REFERENCES

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DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
7	LA	I-5	----		

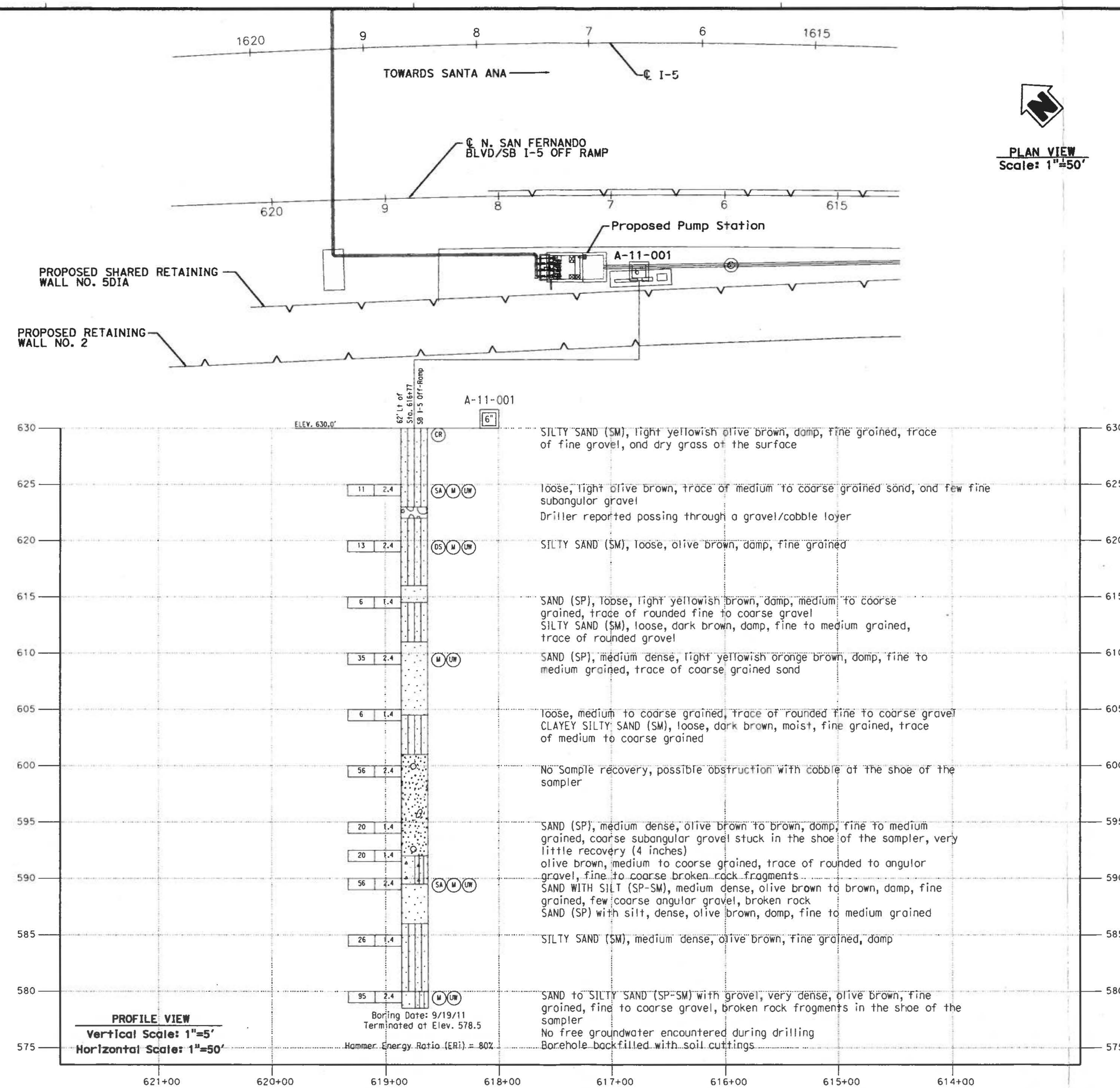
REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER DJAN CHANDRA No. 2376 Exp. 6/30/13 GEOTECHNICAL STATE OF CALIFORNIA

CITY OF BURBANK
150 NORTH THIRD STREET
BURBANK, CA 91502

LEIGHTON CONSULTING INC.
17781 COWAN
IRVINE, CA 92614



P/drafting/603239/001/of_2011-10-17

ENGINEERING SERVICES		GEOTECHNICAL SERVICES
FUNCTIONAL SUPERVISOR	DRAWN BY: BUU TRAN	FIELD INVESTIGATION BY:
NAME:	CHECKED BY: DJAN CHANDRA	Sreekar Pulijala

OGS CIVIL LOG OF TEST BORINGS SHEET

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN
DESIGN BRANCH

CU --
EA --

BRIDGE NO. ---
POST MILES ---

EMPIRE AVENUE STORM WATER PUMP STATION

LOG OF TEST BORINGS 1 OF 3

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES | SHEET | OF

REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
7	LA	I-5	--		

REGISTERED PROFESSIONAL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL GEOTECHNICAL ENGINEER DJAN CHANDRA No. 2376 Exp. 6/30/13 STATE OF CALIFORNIA

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CITY OF BURBANK
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IRVINE, CA 92614

CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

CONSISTENCY OF COHESIVE SOILS

Description	Unconfined Compressive Strength (tsf)	Pocket Penetrometer Measurement (tsf)	Torvane Measurement (tsf)	Field Approximation
Very Soft	< 0.25	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	0.25 to 0.50	0.25 to 0.50	0.12 to 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 to 1.0	0.50 to 1.0	0.25 to 0.50	Penetrated several inches by thumb with moderate effort
Stiff	1 to 2	1 to 2	0.50 to 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2 to 4	2 to 4	1.0 to 2.0	Readily indented by thumbnail
Hard	> 4.0	> 4.0	> 2.0	Indented by thumbnail with difficulty

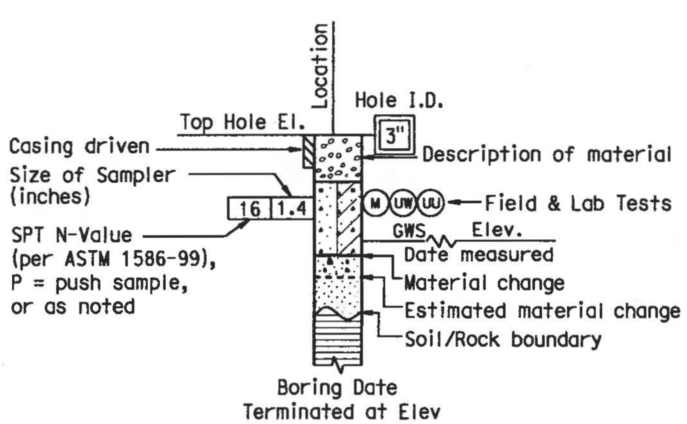
BOREHOLE IDENTIFICATION

Symbol	Hole Type	Description
Size	A	Auger Boring
Size	R P	Rotary drilled boring Rotary percussion boring (air)
Size	R	Rotary drilled diamond core
Size	HD HA	Hand driven (1-inch soil tube) Hand Auger
●	D	Dynamic Cone Penetration Boring
▲	CPT	Cone Penetration Test (ASTM D 5778-95)
⬚	O	Other

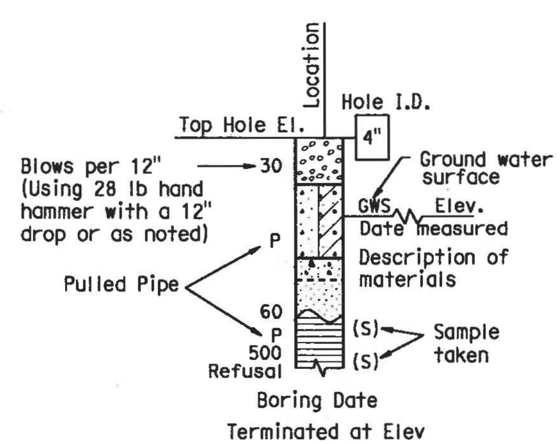
Note: Size in inches.

PLASTICITY OF FINE-GRAINED SOILS

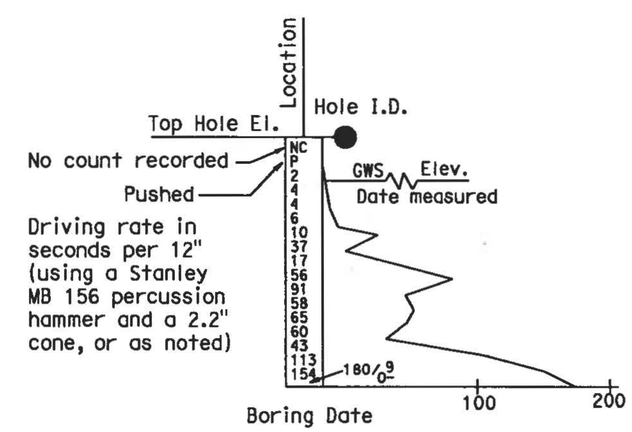
Description	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.



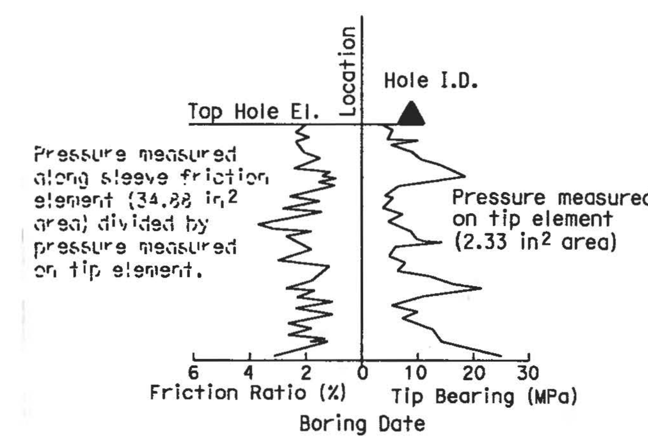
DRY BORING



HAND BORING



DYNAMIC CONE PENETRATION BORING



CONE PENETRATION TEST (CPT) SOUNDING

ENGINEERING SERVICES	GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN	BRIDGE NO. --	EMPIRE AVENUE STORM WATER PUMP STATION
PREPARED BY	BUU TRAN		DESIGN BRANCH	POST MILE --	LOG OF TEST BORINGS 2 OF 3
CHECKED BY	DJAN CHANDRA				

GS LOTB SOIL LEGEND

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU ---
EA ---

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES

SHEET OF

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET No	TOTAL SHEETS
7	LA	I-5	--		

REGISTERED PROFESSIONAL ENGINEER

REGISTERED PROFESSIONAL ENGINEER DJAN CHANDRA No. 2376 Exp. 6/30/13 GEOTECHNICAL STATE OF CALIFORNIA

PLANS APPROVAL DATE

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REFERENCE: CALTRANS SOIL & ROCK LOGGING, CLASSIFICATION, AND PRESENTATION MANUAL (JUNE 2007)

GROUP SYMBOLS AND NAMES

Graphic/Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic/Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY ORGANIC elastic SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTING

- C Consolidation (ASTM D 2435)
- CL Collapse Potential (ASTM D 5333)
- CP Compaction Curve (CTM 216)
- CR Corrosivity Testing (CTM 643, CTM 422, CTM 417)
- CU Consolidated Undrained Triaxial (ASTM D 4767)
- DS Direct Shear (ASTM D 3080)
- EI Expansion Index (ASTM D 4829)
- M Moisture Content (ASTM D 2216)
- OC Organic Content-% (ASTM D 2974)
- P Permeability (CTM 220)
- PA Particle Size Analysis (ASTM D 422)
- PI Plasticity Index (AASHTO T 90) Liquid Limit (AASHTO T 89)
- PL Point Load Index (ASTM D 5731)
- PM Pressure Meter
- R R-Value (CTM 301)
- SE Sand Equivalent (CTM 217)
- SG Specific Gravity (AASHTO T 100)
- SL Shrinkage Limit (ASTM D 427)
- SW Swell Potential (ASTM D 4546)
- UC Unconfined Compression-Soil (ASTM D 2166) Unconfined Compression-Rock (ASTM D 2938)
- UU Unconsolidated Undrained Triaxial (ASTM D 2850)
- UW Unit Weight (ASTM D 4767)
- VS Vane Shear (AASHTO T 223)

APPARENT DENSITY OF COHESIONLESS SOILS

Description	SPT N_{60} (Blows / 12 inches)
Very loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

MOISTURE

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

PERCENT OR PROPORTION OF SOILS

Description	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Description		Size
Boulder		> 12"
Cobble		3" to 12"
Gravel	Coarse	3/4" to 3"
	Fine	No. 4 to 3/4"
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40

ENGINEERING SERVICES	GEOTECHNICAL SERVICES	STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION	DIVISION OF ENGINEERING SERVICES STRUCTURE DESIGN	BRIDGE NO. --	EMPIRE AVENUE STORM WATER PUMP STATION
PREPARED BY	BUU TRAN		DESIGN BRANCH	POST MILE --	LOG OF TEST BORINGS 3 OF 3
CHECKED BY	DJAN CHANDRA				

OS LOTB SOIL LEGEND

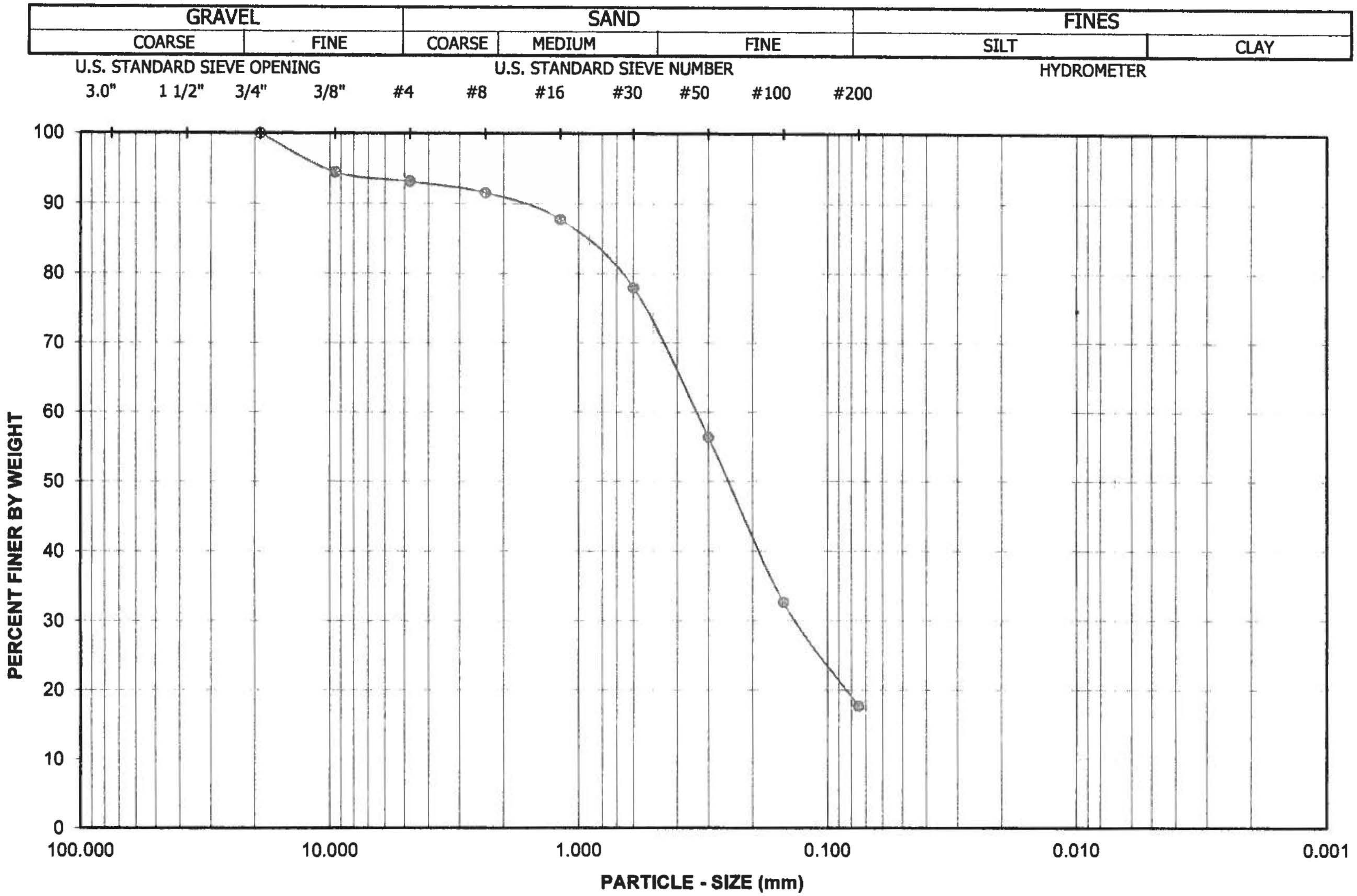
ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU --
EA --

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES 10/05 2/10

SHEET OF



Project Name: Burbank Storm Water Pump Station

Project No.: 603239-001

Exploration No.: A-11-001 Sample No.: R-1

Depth (feet): 5.0 Soil Type : SM

Soil Identification: Brown silty sand (SM)

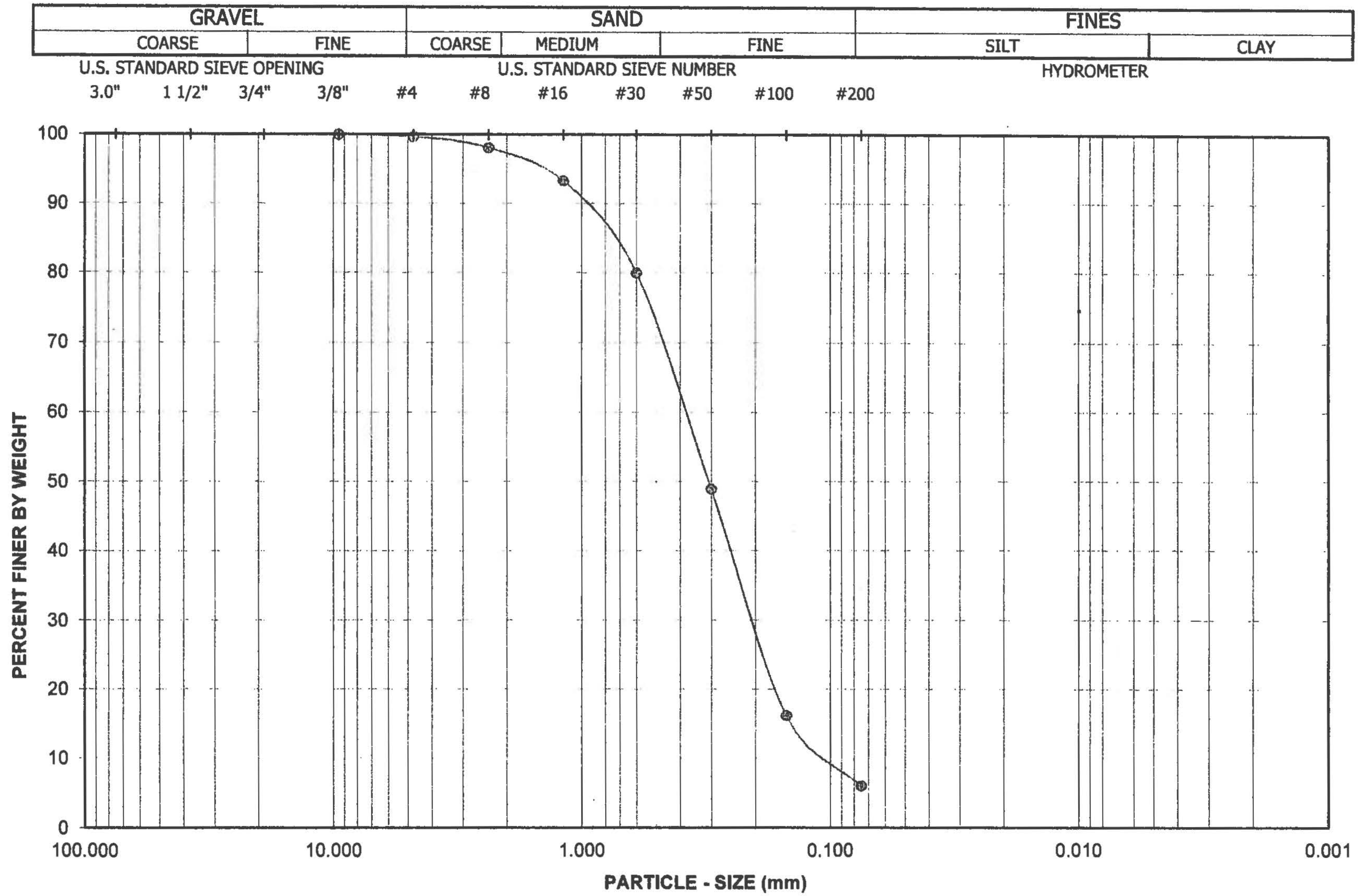
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 7 : 75 : 18

Nov-11

SA / B-1 R-1 @ 5



Project Name: Burbank Storm Water Pump Station

Project No.: 603239-001

Exploration No.: A-11-001 Sample No.: R-4

Depth (feet): 40.0 Soil Type : SP-SM

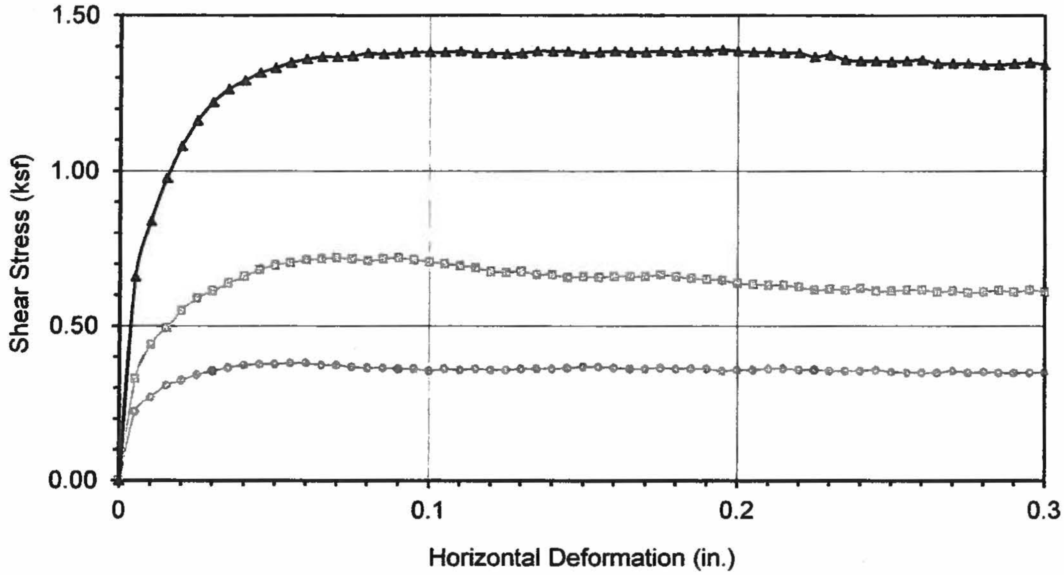
Soil Identification: Gray poorly-graded sand with silt (SP-SM)

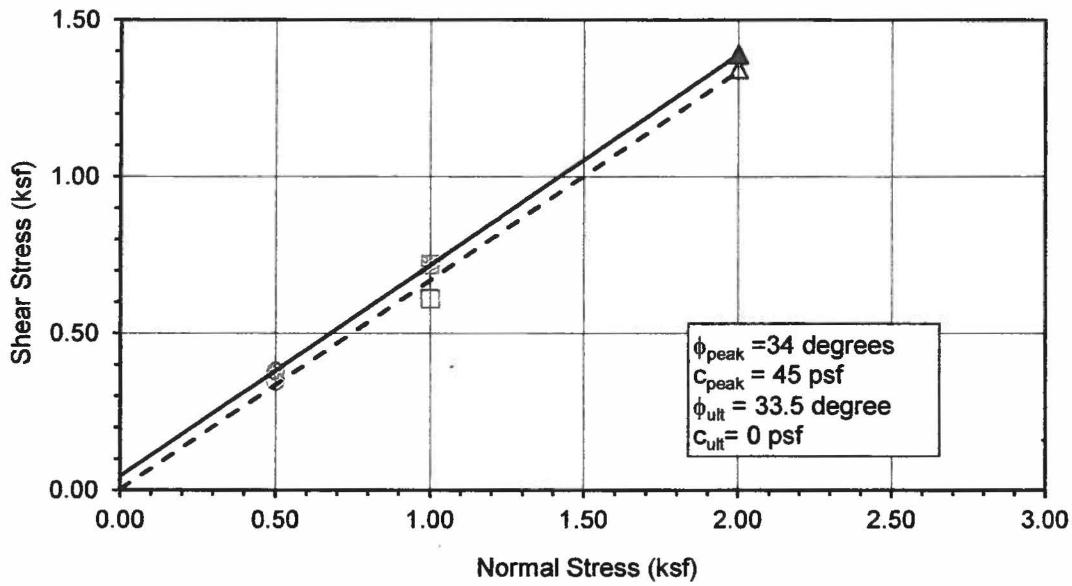
Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

GR:SA:FI : (%) 0 : 94 : 6

Nov-11





Boring No.	**A-11-001**
Sample No.	**R-2**
Depth (ft)	**10**
Sample Type: Drive	
Soil Identification: Light brown silty sand (SM)	

Normal Stress (kip/ft²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft²)	0.380	0.720	▲ 1.390
Shear Stress @ End of Test (ksf)	○ 0.349	□ 0.610	△ 1.342
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	3.22	3.22	3.22
Dry Density (pcf)	97.1	97.8	101.7
Saturation (%)	11.8	12.0	13.2
Soil Height Before Shearing (in.)	0.9932	0.9921	0.9863
Final Moisture Content (%)	26.1	24.1	22.5

Leighton

DIRECT SHEAR TEST RESULTS
Consolidated Undrained

Project No.: 603239-001

Burbank Storm Water Pump Station

09-11

Leighton

SOIL RESISTIVITY TEST

DOT CA TEST 532 / 643

Project Name:	Burbank Storm Water Pump Station	Tested By :	V. Juliano Date: 09/23/11
Project No. :	603239-001	Data Input By:	J. Ward Date: 09/28/11
Boring No.:	A-11-001	Depth (ft.) :	0-5
Sample No. :	Bag-1		

Soil Identification:* Light brown (SP)

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	10	10.92	45000	45000
2	20	18.84	29000	29000
3	30	26.76	23000	23000
4	40	34.69	24000	24000
5				

Moisture Content (%) (MCi)	2.99
Wet Wt. of Soil + Cont. (g)	129.12
Dry Wt. of Soil + Cont. (g)	126.51
Wt. of Container (g)	39.35
Container No.	
Initial Soil Wt. (g) (Wt)	130.00
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 532 / 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 532 / 643	
22800	**28.5**	**42**	**31**	**5.93**	**22.3**