FOR CONTRACT No.: 07-293904
PROJECT E-FIS No. 0712000254

INFORMATION HANDOUT

FINAL FOUNDATION REPORT FOR RARAMOUNT BLVD OVERCROSSING

MONITORING AND ANALYSIS REPORT OF BULK ASBESTOS ANALYSIS

ROUTE: LA-60, PM R7.8

M e m o r a n d u m

To : Mr. Howard Ng - Branch Chief
Office of Structure Design South 2

Attn. : Mr. Dawit Worku - Project Engineer

Date : Jan. 31, 2012
File No. : 07-LA-060
PM 7.9
Project ID 07-120002541
EA 07-293901
BR. No. 53-3076

From : **DEPARTMENT OF TRANSPORTATION**
DIVISOIN OF ENGINEERING SERVICE
METS-Geotechnical Service
Office of Geotechnical Design South-1

Subject : **Final Foundation Report for Paramount Blvd OC (Replace)**

The following is the Foundation Report (FR) for the replacement of Paramount Blvd. Overcrossing (New Bridge No. 53-3076) on Route 60, in the city of Montebello, Los Angeles County.

SCOPE OF WORK

The geotechnical scope of work for this project includes:

- Review As-Built plans and As-Built log of test borings (LOTBs)
- Perform subsurface exploration
- Perform laboratory tests on soil samples obtained during field investigation
- Evaluate site geology, subsurface, and groundwater conditions
- Perform site seismicity study
- Perform engineering analyses
- Provide geotechnical recommendations and construction considerations

PROJECT DESCRIPTION

The project is to replace the original bridge, which was severely damaged by fire from a double tanker under the bridge on December 14, 2011. The original Paramount Blvd Overcrossing (BR53-1910) was a four-span, 96-foot wide bridge. It was constructed in 1968, and had pre-cast (P/C) / pre-stressed (PS) reinforced concrete I girder superstructure with diaphragm abutments. The new bridge will be a two-span structure, with each span 125 feet long. The width of the new bridge will be increased to128 feet. Pre-cast and pre-stressed reinforced concrete Bulb Tee girders will be used for the new superstructure, which will be supported by 4-foot diameter columns at the bent, and cantilever seat type abutments at both ends of the bridge.

SUBSURFACE EXPLORATION AND TESTING PROGRAM

The subsurface exploration program consisted of three rotary wash borings. Standard Penetration Tests (SPT) were performed in compliance with ASTM D1586. The information on locations, investigation depths and equipment used for all borings is presented in Table 1 below.

Table 1 Summary of Subsurface Exploration

Exploratory Boring No.	Station	Offset (ft)	Approx. Top of Hole Elevation (ft)	Approx. Borehole Depth (ft)	Exploration Method	Equipment
R-11-001	186+21.28	9.83 R	352.5	180	Rotary Wash	CS-2000
R-11-002	186+59.53	202.82 R	382.8	71.5	Rotary Wash	CS-2000
R-11-003	186+24.94	170.73 L	369.7	71.5	Rotary Wash	CS-2000

LABORATORY TESTING

During subsurface exploration, samples retrieved from SPT sampler and bulk samples were collected to test for soil properties and corrosivity. The laboratory tests for geotechnical properties were performed at Caltrans' Geotechnical Laboratory in Sacramento. The soil corrosion tests were performed at the Caltrans' Material Laboratory in Sacramento.

SITE GEOLOGY AND SUBSURFACE CONDITIONS

Site Geology

The project site is within the Transverse Ranges geomorphic province of California. The Geologic Map of the El Monte and Baldwin Park Quadrangles, Los Angeles County, California, by Thomas W Dibblee, Jr. 1999 edited by Thomas W. Dibblee Scall 1:24000, indicates that the project site is covered by artificial fill (af) and nonmarine sandstone and conglomerate (Tfsc).

Subsurface Conditions

Based on information from As-Built Plans and recent subsurface exploration, at Abutment 1, there is a more than 30 feet thick backfill above the native soil. The backfill, with source most likely obtained from the area (Merced Hills) north of LA-60, is mostly medium dense to dense silty sand (SM). This backfill is underlain by native soil that is dense to very dense silty sand with gravel.

At Bent 2, there is a 20-foot thick backfill below the current highway grade. This backfill is also most likely originated from the hillside north of LA-60. The soil underneath the backfill is mostly medium dense to very dense silty sand with gravel.
Abutment 3 is on a cut slope that was part of Merced Hills. The soil in this area is mostly very dense silty sand with gravel.

The idealized soil profile and soil strength parameters for foundation design are presented in Table 2 of this memo.

Groundwater

Rotary wash method was used for the subsurface exploration with no groundwater monitoring device installed for the project. However, according to the records on GeoTracker (California Regional Water Quality Control Board), several monitoring wells were installed at Chevron Gas Station about 500 feet north of the proposed Bent 2 of the new bridge for long-term monitoring of groundwater quality. The highest groundwater table recorded in these monitoring wells was Elev. 309 feet, or 34 feet below ground surface (bgs) since the date of well installation (June, 2003), with general hydraulic gradient trends toward the west.

In the subsurface exploration for the original bridge in 1965, the borehole near the current proposed Bent 2, which was drilled down to Elev. 297 feet Mean Sea Level (MSL, NGVD29), did not encounter groundwater. However, groundwater was encountered at Elev. 315 feet (NGVD29) during CIDH pile installation at the same area in June 1967. Therefore, for the design of the new bridge the groundwater table is conservatively assumed to be at 317 feet above MSL, the recorded highest groundwater table during construction for the original bridge, after vertical datum conversion to NAVD88.

Table 2 Idealized Soil Profile and Strength Parameters

Approximate Elevation (ft)	Predominant Soil Type, USCS	Average Blowcount, N_{60}	Total Unit Weight (pcf)	Apparent Friction Angle (degree)	Undrained Shear Strength (psf)
Abutment 1					
+379' to +361'	SM	15	120	32	N/A
+361' to +356'	SM	29	120	34	N/A
+356' to +343'	SM	50	125	37	N/A
+343' to +312'	SM	>100	130	40	N/A
Bent 2					
+352' to +334'	SM	21	120	34	N/A
+334' to +329'	SC-SM	7	115	28	N/A
+329' to +319'	SM-SC	20	120	30	N/A
+319' to +309'	SM	42	125	34	N/A
+309' to +171'	SM	>100	130	40	N/A
Abutment 3					
+370' to +337'	SM	70	125	38	N/A
+337' to +299'	SM	>100	130	40	N/A

CORROSION EVALUATION

The bulk samples obtained from soil borings were shipped to Caltrans' Materials Laboratory, and tested for pH value and minimum electrical resistivity. Based on Caltrans Corrosion Guidelines (Version 1.0, September 2003), the test results indicate the soils are non-corrosive to structure foundation.

Table 3 Summary of Corrosion Test Results

Location	Depth of Sample (ft)	pH	Soluble Sulfates	Soluble Chlorides	Minimum Resistivity
R-11-001	Composite	6.56	N/A	N/A	7689 ohm-cm
R-11-002	Composite	6.66	N/A	N/A	10507 ohm-cm
R-11-003	Composite	6.75	N/A	N/A	9300 ohm-cm

Note: 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. 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.

SEISMIC RECOMMENDATIONS

Ground Motion

A seismic hazard analysis was performed to develop the design ground motion parameters, including the Acceleration Response Spectrum (ARS) for bridge design. This analysis was performed in accordance with the Caltrans' 2009 design ground motion evaluation procedure. This procedure is documented in detail in Appendix B of the Seismic Design Criteria (SDC) and the Geotechnical Services Design Manual (2009), Version 1.0. The "*Caltrans ARS Online"* web tool was used to evaluate deterministic, probabilistic (Return Period of 1000 years) and the design ground motion as defined in the SDC.

The average small strain shear wave velocity $(Vs)_{30}$ for the upper 30 meters (100 feet) of the profile is required to determine the design ground motion. The soil deposits at the job site consist mainly of silty sands with gravels. The shear wave velocity $(Vs)_{30}$ was estimated by using correlations between shear wave velocity and SPT N value. In the upper 30 meters, the job site is underlain by soil deposits with an average shear wave velocity $(Vs)_{30}$ of 300 m/s to 400 m/s. For the seismic hazard analysis, an average $(Vs)_{30}$ of 300 m/sec is assumed.

The recommended ARS curve developed based on the above analysis is shown in Figure 1 of Attachment. The probabilistic ARS curve, instead of deterministic curve from one of the nearby faults controls the design ground motion at this site. Nevertheless, the information of the nearby faults which may cause major future events is still presented in Table 4 below:

Table 4 Summary of the Adjacent Faults

Fault Name	Fault Type	Max. Earthquake Moment Magnitude	Distance to Site
Upper Elysian Park Blind Thrust	Strike Slip	7.8	4.7 km
Elsinore Fault Zone (Whittier Section)	Strike Slip	7.3	5.0 km
Puente Hills (LA)	Reverse	7.3	11.8 km

Note: Information is based on Caltrans 2007 Fault Database.

The Peak Ground Acceleration (PGA) at the job site is expected to be 0.68g. The spectrum acceleration is based on USGS Interactive Deaggregations, considering adjustment for near fault effect and basin effect.

Liquefaction Hazard

The soils at the bridge site and below the historical high groundwater table are very dense silty sand with gravel. Therefore, liquefaction potential is negligible. The potential of lateral spreading is also negligible, as it is triggered by liquefaction and affected by other factors such as ground geometry.

Ground Rupture

No major fault traverses the job site according to the Fault Activity Map of California (2010). Although it was reported that a short fault (East Montebello Hills fault) with low activity crosses the highway near the bridge, the distance from the fault to the job site is about one mile. Therefore, the potential for ground rupture at this site is negligible.

AS-BUILT FOUNDATION DATA

The original Paramount Blvd OC was a four-span bridge with three four-column bents and two diaphragm abutments with footing.

Spread footings were used for the foundations of both abutments. Bent 4 of the original bridge was supported by spread footing on dense to very dense sandy native material. Bents 2 and 3 of the original bridge were supported by CIDH piles with pile caps. Bent 2 of the new bridge will be in line with Bent 3 of the original bridge, and Abutments 1 and 3 of the new bridge will be constructed at original Abutments 1 and 5. The foundation information for original bridge is presented in Table 5 below:

Table 5 Original Bridge Foundation Data from As-Built Plans

Support Location	Shallow Foundation Type	Approx. Foundation Dimension	Quantity	Allowable Bearing Capacity	Bottom of Foundation Elevation*, feet
Abutment 1	Strip Footing	3.0 ft (width)	1	2.0 tsf	362 to 364
Bent 2	CIDH Piles	16 in (diameter)	11,11,9,9	45 tons	330
Bent 3	CIDH Piles	16 in (diameter)	14,14,10,10	45 tons	310
Bent 4	Spread Footings	14'X12', 14'X12', 12'X12', 12'X12'	4	3.0 tsf	342 to 342.5
Abutment 5	Strip Footing	3.0 in (width)	1	2.5 tsf	354 to 356

Note: * Approximately 2.2 feet need to be added to the elevation on as-built plans (NGVD29 datum) to convert to current vertical datum (NAVD 88).

Based on as-built LOTBs from subsurface exploration performed on October 1965 for the original bridge, native materials, which was identified as compact to very dense silty sand, was encountered from Elev. 333 feet to Elev. 297 feet at Bent 3 of the original bridge, near the centerline of the highway. The existing highway grade is about 20 feet above the native grade.

GEOTECHNICAL RECOMMENDATIONS

Foundation Type Selection

Due to the high seismicity of the Site, deep foundation is needed at the bridge abutments. Deep foundation is also recommended to support the four-foot diameter columns at Bent 2. Cast-in-drilled-hole (CIDH) piles were considered at first for their relatively low installation noise and vibration during construction. However, due to the possible presence of groundwater in drilled holes during pile installation, limited overhead clearance to high voltage power lines, site constraint for the foundation size, and tight construction schedule, driven piles are recommended for the new bridge. Steel H pile is recommended for its ease of splicing, better drivability, and relatively fast installation. At the abutments, the piles at the front row of the pile group will be battered to resist lateral earth pressure under service condition of high cantilever abutment walls, which are needed to facilitate future highway widening.

Foundation Data Provided by Structural Designers

Based on the Department policy, the bent foundations are designed using Load and Resistance Factor Design (LRFD), and the abutment foundations are designed using Work Stress Design

(WSD). The foundation design data and load data were provided by the structural designers and presented in the following tables:

Table 6.1 Deep Foundation Data

Support No.	Design Method	Pile Type	Finish Grade Information		BOF Elevation (ft)	Pile Cap Size (ft)		Number of Piles per Column	Permissible Settlement Under Service Load (in)
			Current	Future		B	L		
Abut 1	WSD	H-Pile	Beg 367 End 370	Beg 361 End 368	Step1 = 356, Step 2 = 359.5 Step 3 = 363	132	14	52	1
Bent 2	LRFD	H-Pile	352	352	345 346	14	14	16	1
Abut 3	WSD	H-Pile	Beg 364 End 367	Beg 358 End 356	Step 1 = 352.5 Step 2 = 351	132	14	52	1

Table 6.2 LRFD Service Limit States I Load Data

Support No.	Total Vertical Load (kip)		Permanent Loads Per Supprt (kip)
	Per Support	Maximum Per Pile	
Abut 1	7500	190	6500
Bent 2	1965	145	1475
Abut 3	7500	190	6500

Table 6.3 LRFD Strength and Extreme Limit States Load Data

Support No.	Strength Limit State (Controlling Group, Kip)				Extreme Limit State (Controlling Group, kip)			
	Compreesion		Tension		Compression		Tension	
	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	2780	210			1800	320		-140
Abut 3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Axial Capacity

The abutments and bent foundation design recommendations for axial capacities are presented in Tables 7.1 and 7.2. The Pile Data Table to be included in contract plans is presented in Table 7.3. The driven HP14X89 piles are designed using pile design/analysis computer program DRIVEN 1.2 (FHWA), where Nordlund's method was used for pile side resistance within cohesionless soils. The end bearing is calculated based on cross sectional area of the steel only, disregarding the soil-plug in piles, while the side friction is calculated based on "box" perimeter of the H piles.

Table 7.1 Abutment Foundation Design Recommendations

Support Location		Pile Type	Cut-Off Elevation (ft)	LRFD Service-I Limit State Load per Support (kip)		LRFD Service I Limit State Load per Pile in Compression (kip)	Required Nominal Resistance (kip)	Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kip)
				Total	Permanent					
Abut 1	Step 1	HP 14X89	356.42	7500	6500	190	380	319 (a) 336 (d)	319	380
	Steps 2, and 3		359.94, 363.42					322 (a) 340 (d)	322	
Abut 3		HP 14X89	352.92, 351.42	7500	6500	190	380	314 (a) 331 (d)	314	380

Notes:
1. *Design Tip Elevations for Abutments are controlled by: (a) Compression, and (d) Lateral load.*
2. *The specified tip elevation shall not be raised above design tip elevations for lateral.*

Table 7.2 Bent Foundation Design Recommendations

Support Location	Pile Type	Cut-off Elevation (ft)	Service-I Limit State Load per Support (kip)	Total Permissible Support Settlement (inch)	Required Factored Nominal Resistance (kip)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
					Strength Limit		Extreme Event				
					Compression (ψ = 0.7)	Tension (ψ= 0.7)	Compression (ψ= 1)	Tension (ψ= 1)			
Bent 2	HP14X89	345.42, 346.42	1965	1	210	N/A	320	140	304 (a-I) 303 (a-II) 308 (b-II) 323 (d)	303	320

Notes:
1. *Design Tip Elevations for Bent 2 are controlled by: (a-I) Compression (Strength Limit State), (b-I) Tension (Strength Limit State), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), and (d) Lateral load, respectively.*
2. *The specified tip elevation shall not be raised above the design tip elevations for tension and lateral.*

Table 7.3

Support Location		Pile Type	Nominal Resistance (kip)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kip)
			Compression	Tension			
Abut 1	Step 1	HP14X89	380	0	319 (a) 336 (d)	319	380
	Step 2, 3				322 (a) 340 (d)	322	380
Bent 2		HP14X89	320	140	303 (a) 308 (b) 323 (d)	303	320
Abut 3		HP14X89	380	0	314 (a) 331 (d)	314	380

Notes:

1. *Design Tip Elevations are controlled by: (a) Compression, (b) Tension, and (d) Lateral load.*
2. *The specified tip elevation shall not be raised above the design tip elevations for tension load, and lateral load.*

Lateral Capacity

Lateral capacity of the driven piles was analyzed using LPILE PLUS 5.0 program (Ensoft Inc.). The boundary condition at the pile top is assumed to be hinged (pinned) reflecting the pile-to-cap structural connection.

For the lateral capacity of vertical piles under service condition at both abutments, piles at Abutment 1 were analyzed due to relatively lower soil resistance at Abutment 1 than that at Abutment 3. The results of lateral analysis at strong axis direction of H piles are presented in Table 8.1. The calculated maximum bending moments and locations of the maximum bending moment under each lateral load and deflection are also presented in the table.

Table 8.1 Summary of Lateral Analysis at Abutments (Strong Axis)

Load vs Pile Top Deflection of HP14X89 Steel Driven Piles at Abutments (BR 53-3076)			
Pile Top Deflection (in)	Assuming Pile/Cap Hinge Connection		
	Lateral Load on Pile Top (kip)	Maximum Bending Moment (kip-ft)	Depth to Maximum Moment from Pile Top (ft)
0.25	17	54	6.5
0.50	29	96	6.8
1.00	44	102	7.2
2.00	61	293	8.3
Notes:			
1) Group Reduction Factor (P-Multiplier) is assumed to be 0.9;			
2) Axial Load is assumed to be 190 kips on each pile;			
3) Lateral analysis conducted for Longitudinal direction (Strong axis of HP shape) with moment of inertia of 904 in^4.			

Lateral analysis was also performed for the piles at Bent 2 in transverse direction, to which the weak axis of H piles are oriented. The ultimate pile lateral capacity, which is 70 kips at transverse direction at the bent, is defined as the maximum lateral shear on pile top, under which, the moment capacity (plastic moment) will be reached (or the second hinge introduced) on 14X89 steel H piles. The results of lateral analysis for the piles at Bent 2 are presented in Table 8.2 below.

Table 8.2 Summary of Lateral Analysis for Bent 2 (Weak Axis)

Load vs Pile Top Deflection of HP14X89 Steel Driven Piles at Bent 2 (BR 53-3076)			
Pile Top Deflection (in)	Assuming Pile/Cap Hinge Connection		
	Lateral Load on Pile Top (lb)	Maximum Bending Moment (lb-ft)	Depth to Maximum Moment from Pile Top (ft)
0.25	19	44	4.1
0.50	33	80	4.5
1.00	51	145	4.9
1.50	62	202	5.0
2.00*	70	254	5.4

Notes:

1) Group Reduction Factor (P-Multiplier) is assumed to be 0.55;

2) Axial Load is assumed to be 300 kips on each pile;

3) Lateral controlled pile tip is 22 ft below pile top under hinge condition.

4) Lateral analysis conducted for transverse direction (weak axis of HP shape) with moment of inertia of 326 in^3.

***** Pile moment capacity (254 kips-ft) reached at top deflection of 2.0 in. with corresponding shear of 70 kips on pile top.

The lateral controlled pile tip elevations and lengths at all structure support locations are also calculated. The lateral controlled pile length is defined as a critical pile length, beyond which, the increase of the pile length will not reduce pile deflection under the design lateral load. The above critical pile length under maximum lateral load on pile top is the lateral controlled pile length. Its corresponding tip elevation is lateral load controlled pile tip elevation. The lateral load controlled pile tip elevations, which are based on the results of lateral analyses in transverse direction, are presented in foundation design recommendations and pile data table, Tables 7.1 to 7.3, in previous section.

ABUTMENT RETAINING WALLS (WINGWALLS)

Standard Type 1 walls (Standard Plans 2006) with spread footings can be used for all abutment wingwalls. The new retaining wall type of abutment wingwalls will be constructed adjacent to the existing cantilever wingwalls. The new abutment retaining wall footings will be constructed below the existing grade, on native soils at Abutment 3 and on existing backfill at Abutment 1. A minimum factor of safety of 3.0 can be achieved for the bearing capacity of wingwall footings with design wall height up to 16 feet.

CONSTRUCTION CONSIDERATIONS

1) Groundwater will not be encountered during footing excavation for the structural foundations according to the available groundwater information.

2) Piles should be driven at least to the specified tip elevation with the driving resistance value verified by the acceptance criteria specified in Section 49-1.08 of Standard Specifications (Caltrans, May 2006).

3) If the required nominal driving resistance cannot be obtained at 6 inches above the specified tip elevation, pile driving should be stopped. A minimum of 24-hour set-up period should be required before resuming pile driving to the specified tip elevation.

4) Should the nominal driving resistance still cannot be attained during re-strike, an extension of the pile length will be needed to meet the bearing requirement. In this case, it is prudent to furnish steel lugs on pile to improve driving resistance, as specified in Bridge Construction Memo 130-5.0, for the remaining piles at both abutments.

5) Based on available soil boring data, driving steel H piles is not anticipated to be difficult with appropriate choice of equipment. To ensure proper execution of construction, the Contractor should provide driving system submittal to Geotechnical Services and allow the Engineer 15 working days for equipment submittal review prior to pile installation.

If you have any questions, please contact Haitao Liu at (916) 227-0992, or Sungro Cho at (916) 227-5398.

Report by:

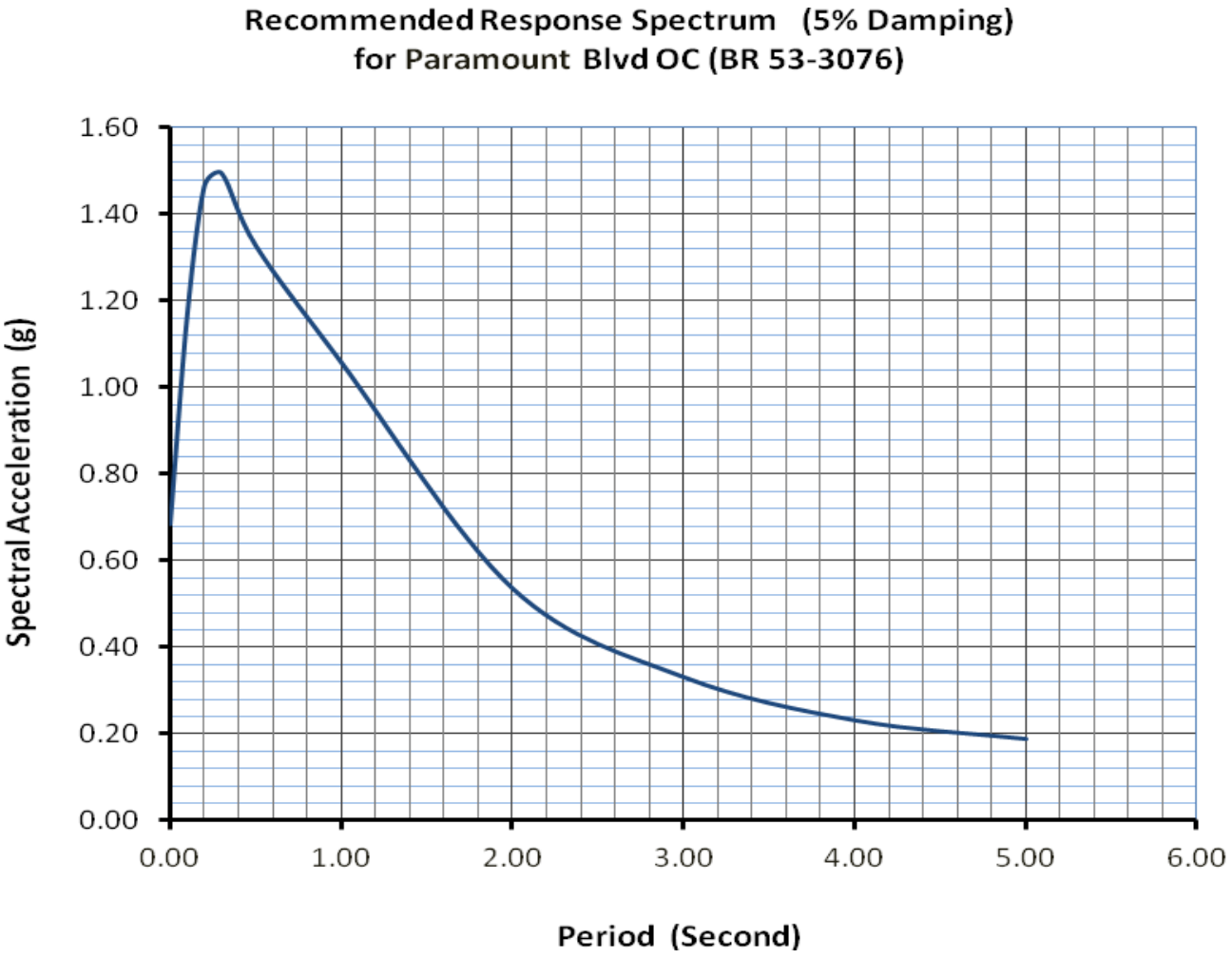
Date: 1/31/2012

Haitao Liu, P.E.
Transportation Engineer - Civil
Branch A

Sungro Cho, Ph.D., P.E
Transportation Engineer - Civil
Branch A

cc: Zareh Shahbazian, D07 Project Manager
GS Corporate
Structure Construction R.E. Pending Files
DES Office Engineer, Office of PS&E
Kirsten Stahl, D07 Material Engineer

ATTACHMENT



Period (Second)	0	0.1	0.2	0.3	0.5	1	2	3	4	5
Spectrum Acceleration (g)	0.68	1.17	1.47	1.50	1.33	1.06	0.54	0.33	0.23	0.19

Figure 1 Recommended Acceleration Response Spectra (ARS) Curve, BR 53-3076

SOUTH COAST AIR QUALITY MANAGEMENT DISTRICT
21865 Copley Dr., Diamond Bar, CA 91765-4182

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

TO: Edwin Pupka
Senior Enforcement Manager
Engineering and Compliance

LABORATORY NO: 1135005

DATE RECEIVED: 12-16-11

REFERENCE NO: ASB-62-66

REQUESTED BY: C. Ravenstein

SOURCE I.D.#: None listed

SAMPLE SOURCE:

Paramount Overpass @ 60 Fwy (Notif. 292069)
Paramount Overpass @ 60 Fwy, Montebello

See attached report.

Date Approved: 12/21/11

Approved by: Rudy Eden

Rudy Eden, Sr. Manager
Laboratory

South Coast
AIR QUALITY MANAGEMENT DISTRICT
21865 Copley Drive, Diamond Bar, CA 91765-4178

NVLAP LAB CODE 101567-0

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

TO: Edwin Pupka
Senior Enforcement Manager
Engineering and Compliance

LABORATORY NO: 1135005-01

DATE RECEIVED: 12-16-11

REFERENCE NO: ASB-62-66

REQUESTED BY: C. Ravenstein

SOURCE I.D.#: None listed

SAMPLE DESCRIPTION:
Sample #1: One 8 oz. plastic container containing an approximately four inch square piece of grey transite from the Paramount bridge over the 60 Freeway in Montebello.

SAMPLE SOURCE:
Paramount Overpass @ 60 Fwy (Notif. 292069)
Paramount Overpass @ 60 Fwy, Montebello

Percent Asbestos by Polarized Light Microscopy in accordance with the Test Method: EPA/600/R-93/116, *Method for the Determination of Asbestos in Bulk Materials* (Section 2.1, 2.2 and 2.3), July 1993.

Total Asbestos Present: 10%

Sandra Essner, Approved Signatory

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

LABORATORY NO: 1135005-01 **Sample Description:** Grey transite fragment with protruding fibers			
Layer	1		
Layer Description:	Grey transite fragment with protruding fibers		
Percent of Sample	100		
Fibrous Non-Asbestos Materials Present	None Detected		
Non-Fibrous Materials Present	90% Crystalline material & residue		
Asbestos Type and Percent Found	10% Chrysotile		
Total Percent Asbestos Found	10		
Comments	Calibrated visual estimate		

Sample Summary

Layer	% of Total Sample	% Asbestos in Layer
1	100	10

Total Asbestos in Sample: 10%

Analyst: _____
Robert Yi, A. Q. Chemist

South Coast
AIR QUALITY MANAGEMENT DISTRICT
21865 Copley Drive, Diamond Bar, CA 91765-4178

NVLAP LAB CODE 101567-0

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

TO: Edwin Pupka
Senior Enforcement Manager
Engineering and Compliance

LABORATORY NO: 1135005-02

DATE RECEIVED: 12-16-11

REFERENCE NO: ASB-62-67

REQUESTED BY: C. Ravenstein

SOURCE I.D.#: None listed

SAMPLE DESCRIPTION:
Sample #2: One 8 oz. plastic container containing an approximately three inch square piece of grey transite from the Paramount bridge over the 60 Freeway in Montebello.

SAMPLE SOURCE:
Paramount Overpass @ 60 Fwy (Notif. 292069)
Paramount Overpass @ 60 Fwy, Montebello

Percent Asbestos by Polarized Light Microscopy in accordance with the Test Method: EPA/600/R-93/116, *Method for the Determination of Asbestos in Bulk Materials* (Section 2.1, 2.2 and 2.3), July 1993.

Total Asbestos Present: 10%

Sandra Essner

Sandra Essner, Approved Signatory

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

LABORATORY NO: 1135005-02 **Sample Description:** Grey transite fragment with protruding fibers			
Layer	1		
Layer Description:	Grey transite fragment with protruding fibers		
Percent of Sample	100		
Fibrous Non-Asbestos Materials Present	None Detected		
Non-Fibrous Materials Present	90% Crystalline material & residue		
Asbestos Type and Percent Found	10% Chrysotile		
Total Percent Asbestos Found	10		
Comments	Calibrated visual estimate		

Sample Summary

Layer	% of Total Sample	% Asbestos in Layer
1	100	10

Total Asbestos in Sample: 10%

Analyst: _____
Robert Yi, A. Q. Chemist

South Coast
AIR QUALITY MANAGEMENT DISTRICT
21865 Copley Drive, Diamond Bar, CA 91765-4178

NVLAP LAB CODE 101567-0

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

TO: Edwin Pupka
Senior Enforcement Manager
Engineering and Compliance

LABORATORY NO: 1135005-03

DATE RECEIVED: 12-16-11

REFERENCE NO: ASB-62-68

REQUESTED BY: C. Ravenstein

SOURCE I.D.#: None listed

SAMPLE DESCRIPTION:
Sample #3: One 4 oz. plastic container containing an approximately two inch square piece of brown-black material from the Paramount bridge over the 60 Freeway in Montebello.

SAMPLE SOURCE:
Paramount Overpass @ 60 Fwy (Notif. 292069)
Paramount Overpass @ 60 Fwy, Montebello

Percent Asbestos by Polarized Light Microscopy in accordance with the Test Method: EPA/600/R-93/116, *Method for the Determination of Asbestos in Bulk Materials* (Section 2.1, 2.2 and 2.3), July 1993.

Total Asbestos Present: **None Detected**

Sandra Essner
Sandra Essner, Approved Signatory

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

LABORATORY NO: 1135005-03 **Sample Description:** Black and brown fibrous material with tar			
Layer	1		
Layer Description:	Black and brown fibrous material with tar		
Percent of Sample	100		
Fibrous Non-Asbestos Materials Present	43% Natural fibers 3% Mineral wool		
Non-Fibrous Materials Present	43% Tar 4% Acid/water soluble 7% Crystalline material & residue		
Asbestos Type and Percent Found	None Detected		
Total Percent Asbestos Found	None Detected		
Comments	Gravimetric analysis		

Sample Summary

Layer	% of Total Sample	% Asbestos in Layer
1	100	None Detected

Total Asbestos in Sample: None Detected

Analyst: _____
Robert Yi, A. Q. Chemist

South Coast
AIR QUALITY MANAGEMENT DISTRICT
21865 Copley Drive, Diamond Bar, CA 91765-4178

NVLAP LAB CODE 101567-0

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

TO: Edwin Pupka
Senior Enforcement Manager
Engineering and Compliance

LABORATORY NO: 1135005-04

DATE RECEIVED: 12-16-11

REFERENCE NO: ASB-62-69

REQUESTED BY: C. Ravenstein

SOURCE I.D.#: None listed

SAMPLE DESCRIPTION:
Sample #4: One 4 oz. plastic container containing an approximately two inch square piece of brown-black material from the Paramount bridge over the 60 Freeway in Montebello.

SAMPLE SOURCE:
Paramount Overpass @ 60 Fwy (Notif. 292069)
Paramount Overpass @ 60 Fwy, Montebello

Percent Asbestos by Polarized Light Microscopy in accordance with the Test Method: EPA/600/R-93/116, *Method for the Determination of Asbestos in Bulk Materials* (Section 2.1, 2.2 and 2.3), July 1993.

Total Asbestos Present: **None Detected**

Sandra Essner
Sandra Essner, Approved Signatory

MONITORING AND ANALYSIS
REPORT OF BULK ASBESTOS ANALYSIS

LABORATORY NO: 1135005-04 **Sample Description:** Black and brown fibrous material with tar			
Layer	1		
Layer Description:	Black and brown fibrous material with tar		
Percent of Sample	100		
Fibrous Non-Asbestos Materials Present	43% Natural fibers 3% Mineral wool		
Non-Fibrous Materials Present	43% Tar 4% Acid/water soluble 7% Crystalline material & residue		
Asbestos Type and Percent Found	None Detected		
Total Percent Asbestos Found	None Detected		
Comments	Gravimetric analysis		

Sample Summary

Layer	% of Total Sample	% Asbestos in Layer
1	100	None Detected

Total Asbestos in Sample: None Detected

Analyst: _____
Robert Yi, A. Q. Chemist

SOUTH COAST AIR QUALITY MANAGEMENT DISTRICT
SAMPLE ANALYSIS REQUEST

☒ DISTRICT INFORMATION
☐ INVOICE SOURCE
LABORATORY NO. 135005

TO: SCAQMD LAB: ☒ OTHER: ☐

SOURCE NAME: Paramount Overpass @ 60 Fwy (Notif. 292069) I.D. No. None

Source Address: Paramount Overpass @ 60 Fwy City: Montebello

Mailing Address: City: Zip:

Contact Person: Dan Beck Title: C.A.C. Tel: (562) 307-0744

Analysis Requested by: Christopher Ravenstein, x2486 Date: 12/16/11

Approved by: Rich Tambara Office: Toxics & Waste Managment Budget #: 50-375

REASON REQUESTED: Court/Hearing Board ☐ Permit Pending ☐ Hazardous/Toxic Spill ☐

Suspected Violation Rule(s) 1403 Other ☐

Sample Collected by: Christopher Ravenstein Date: 12/16/11 Time: ~01015 hours

Specify the description and location where the sample was collected:

Sample #1: One 8 oz. plastic container containing an approximately four inch square piece of grey transite from the Paramount bridge over the 60 Freeway in Montebello.

Sample #2: One 8 oz. plastic container containing an approximately three inch square piece of grey transite from the Paramount bridge over the 60 Freeway in Montebello.

Sample #3: One 4 oz. plastic container containing an approximately two inch square piece of brown-black material from the Paramount bridge over the 60 Freeway in Montebello.

Sample #4: One 4 oz. plastic container containing an approximately two inch square piece of brown-black material from the Paramount bridge over the 60 Freeway in Montebello.

Analysis Requested: Percent asbestos analysis per Rule 1403 of material in 125 ml plastic container

Relinquished by	Received by	Firm/Agency	Date	Time
		SCAQMD Lab	12/16/11	11:00

Remarks: Please send results to Ed Pupka, Manager Toxics & Waste Management; Enforcement action Pending. Notification #292069