FOR CONTRACT NO.: 07-1219U**4**

INFORMATION HANDOUT

WATER QUALITY

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
LOS ANGELES REGION

PERMITS

UNITED STATES ARMY CORPS OF ENGINEERS
NON-REPORTING NATIONWIDE 404 PERMIT

AGREEMENTS

CALIFORNIA DEPARTMENT OF FISH AND GAME
NOTIFICATION NO.1600-2007-0379-R5

ENCROACHMENT PERMITS

COUNTY OF LOS ANGELES
DEPARTMENT OF PUBLIC WORKS
FLOOD CONTROL DISTRICT

MATERIALS INFORMATION

A. FINAL HYDRAULIC REPORT FOR THE TUJUNGA WASH CHANNEL BRIDGE (BR. NO. 53-1121 L/R), DATED JANUARY 20, 2005

B. FOUNDATION REPORT FOR 5/170 NEW NORTH BOUND CONNECTOR BRIDGE (BR. NO. 53-2976), DATED AUGUST 7, 2006

B1. ADDENDUM (NO. 1) FOUNDATION REPORT FOR NEW MIXED FLOW 5-170 CONNECTOR BRIDGE (BRIDGE NO. 53-2976, DATED JANUARY 23, 2008

C. FOUNDATION REPORT FOR 5/170 NEW HOV CONNECTOR BRIDGE (BR. NO. 53-2977), DATED AUGUST 7, 2006

C1. ADDENDUM (NO. 1) FOUNDATION REPORT FOR NEW MIXED FLOW 5-170 CONNECTOR BRIDGE (BRIDGE NO. 53-2977, DATED JANUARY 23, 2008

D. FOUNDATION REPORT FOR TUJUNGA WASH LEFT BRIDGE WIDENING (BR. NO. 53-1121L), DATED SEPTEMBER 8, 2006

D1. ADDENDUM FOUNDATION REPORT FOR TUJUNGA WASH LEFT BRIDGE WIDENING (BR. NO. 53-1121L), DATED JUNE 21, 2007

D2. SECOND ADDENDUM FOUNDATION REPORT FOR TUJUNGA WASH LEFT BRIDGE WIDENING (BR. NO. 53-1121L), DATED FEBRUARY 21, 2008

D3. THIRD ADDENDUM FOUNDATION REPORT FOR TUJUNGA WASH LEFT BRIDGE WIDENING (BR. NO. 53-1121L), DATED JULY 28, 2009

E. FOUNDATION REPORT FOR TUJUNGA WASH RIGHT BRIDGE WIDENING (BR. NO. 53-1121R) DATED AUGUST 10, 2006

E1. ADDENDUM MEMORANDUM FOR FOUNDATION REPORT FOR TUJUNGA WASH RIGHT BRIDGE WIDENING (BR. NO. 53-1121R), DATED SEPTEMBER 11, 2006

E2. SECOND ADDENDUM MEMORANDUM FOR FOUNDATION REPORT FOR TUJUNGA WASH RIGHT BRIDGE WIDENING (BR. NO. 53-1121R) DATED SEPTEMBER 24, 2007

E3. THIRD ADDENDUM FOUNDATION REPORT FOR TUJUNGA WASH RIGHT BRIDGE WIDENING (BR. NO. 53-1121R) DATED FEBRUARY 21, 2008

E4. FOURTH ADDENDUM FOUNDATION REPORT FOR TUJUNGA WASH RIGHT BRIDGE WIDENING (BR. NO. 53-1121R) DATED JULY 28, 2009

F. FOUNDATION REPORT FOR BRANFORD STREET UNDERCROSSING WIDENING (BR. NO. 53-1123), DATED AUGUST 16, 2006

F1. ADDENDUM FOR FOUNDATION REPORT FOR BRANFORD STREET UNDERCROSSING WIDENING (BR. NO. 53-1123) DATED AUGUST 14, 2007

G. FOUNDATION REPORT FOR OSBORNE STREET UNDERCROSSING WIDENING (BRIDGE NO. 53-1124), DATED AUGUST 16. 2006

G1. ADDENDUM FOR FOUNDATION REPORT FOR OSBORNE STREET UNDERCROSSING WIDENING (BRIDGE NO. 53-1124), DATED AUGUST 14. 2007

H. FOUNDATION REPORT FOR TERRA BELLA STREET UNDERCROSSING WIDENING (BR. NO. 53-1125), DATED SEPTEMBER 11, 2006

H1. ADDENDUM FOR FOUNDATION REPORT FOR TERRA BELLA STREET UNDERCROSSING WIDENING (BR. NO. 53-1125), DATED JUNE 21, 2007

I. FOUNDATION REPORT FOR ARLETA-SHELDON UNDERCROSSING WIDENING (BR. NO. 53-1763), DATED AUGUST 16, 2006

J. FOUNDATION REPORT FOR TONOPAH STREET PEDESTRIAN UNDERCROSSING EXTENSION (BR. NO. 53-692), DATED AUGUST 7, 2006

K. FOUNDATION RECOMMENDATIONS FOR RETAINING WALL NOS. 325, 326, 331, 332, 582, 588, 589, 602, SOUND WALL NOS. 581, 587, 593, 603, 611, TEN CULVERT EXTENSIONS ALONG SOUND WALL NOS. 581, 587, 593, 603, AND 611, DATED AUG. 10, 2006

K1. REVISED FOUNDATION RECOMMENDATIONS FOR RETAINING WALL NOS. 325, 326, 331, 332, 582, 588, 589, 602, SOUND WALL NOS. 581, 587, 593, 603, 611, TEN CULVERT EXTENSIONS ALONG SOUND WALL NOS. 581, 587, 593, 603, AND 611, DATED FEB. 22, 2007

K2. ADDENDUM TO FOUNDATION RECOMMENDATIONS FOR RETAINING WALL NOS. 325, 326, 331, 332, 582, 588, 589, 602, SOUND WALL NOS. 581, 587, 593, 603, 611, TEN CULVERT EXTENSIONS ALONG SOUND WALL NOS. 581, 587, 593, 603, AND 611-REVISION 2, DATED OCT. 15, 2007

K3. ADDENDUM TO FOUNDATION RECOMMENDATIONS FOR RETAINING WALL NOS. 325, 326, 331, 332, 582, 588, 589, 602, SOUND WALL NOS. 581, 587, 593, 603, 611, TEN CULVERT EXTENSIONS ALONG SOUND WALL NOS. 581, 587, 593, 603, AND 611-REVISION 3, DATED NOV. 30, 2007

K4. AMENDED FOUNDATION RECOMMENDATIONS FOR RETAINING WALL NOS. 325, 326, 331, 332, 582, 588, 589, 602, SOUND WALL NOS. 581, 587, 593, 603, 611, EIGHT CULVERT EXTENSIONS ALONG SOUND WALL NOS. 581, 587, 593, 603, AND 611-REVISION 4, DATED AUG. 4, 2008

L. FOUNDATION RECOMMENDATIONS FOR OVERHEAD SIGNS (28 SIGN STRUCTURES), DATED SEPT. 12, 2007

M. PORTIONS OF AERIALLY DEPOSITED LEAD SITE INVESTIGATION REPORT

N. OSHA UNDERGROUND CLASSIFICATION (NO. C088-037-08T)

O. BATTERY BACKUP SYSTEM DIAGRAMS

California Regional Water Quality Control Board

Los Angeles Region

121901

Recipient of the 2001 *Environmental Leadership Award* from Keep California Beautiful

Linda S. Adams
Agency Secretary

Arnold Schwarzenegge
Governor

320 W. 4th Street, Suite 200, Los Angeles, California 90013
Phone (213) 576-6600 FAX (213) 576-6640 - Internet Address: http://www.waterboards.ca.gov/losangeles

SECTION 401 WATER QUALITY CERTIFICATION APPLICATION FORM

Applications for Water Quality Certification shall be filed in accordance with Sections 3830 through 3869 of Title 23 of the California Code of Regulations. An initial deposit of **$500** must accompany all applications. Please include a check made out to the State Water Resources Control Board. The schedule of fees can be found at http://www.waterboards.ca.gov/losangeles/html/meetings/401wqc.html. **Failure to submit this fee deposit will make this application incomplete. Submit your completed application form to the address above, Attn: 401 Certification Staff.** Attach additional sheets as necessary.

1. APPLICANT/AGENT INFORMATION

a) Applicant: Paul Caron	b) Agent/Consultant*:
Main Contact: Bridget Cameron	Main Contact:
Address: 100 South Main Street, Los Angeles, CA 90012	Address:
Email: bridget_cameron@dot.ca.gov	Email:
Phone No. 213-897-9362	Phone No.
Fax No. 213-897-2593	Fax No.

*Complete only if applicable

2. PROJECT DESCRIPTION

a) Project Title:

I-5 High Occupancy Vehicle Lane Project EA: 121901

b) Purpose/Goal:

Alleviate congestion, encourage ride sharing, and reduce air pollution

c) Project Activities: (Attach additional sheets as necessary)

Please provide a detailed explanation of all project activities. Include information such as: avoidance and minimization measures for project impacts; alternatives analysis; project activity impacts to waterbodies and/or water quality; and implementation of Low Impact Development (LID) strategies.

Construct HOV lane in median of I-5, construct I-5/SR-170 interchange to provide mixed lane connector ramp, widen Tujunga Wash Bridge (No. 53-1121R and 53-1121L), use temporary falsework in concrete lined channel

d) Proposed Schedule (Start-up, duration, and completion dates):

Start-up: 5/1/09, duration: 18 months, completion: 1/1/11

3. FEDERAL LICENSES/PERMITS

a) Federal Agency(ies)/File Number(s): U.S. Army Corps of Engineers Representative: pending

U.S. Army Corps of Engineers X Other_____

File No.(s): pending

b) Permit Type(s) (please provide permit number(s):

Nationwide Permit No.(s): 33 Regional General Permit No.(s) _____

Individual Permit____ Other_____

c) Does the project require any Federal Application(s), Notification(s) or Correspondence?

Yes _____ (attach copy(ies)) No X (Attach detailed explanation) there are no listed species within or adjacent to the project study area

4. OTHER LICENSES/PERMITS/AGREEMENTS

a) Please list all other required regulatory approvals (submit final or draft copy if available):

Agency	Agency Representative	License/Permit/Agreement	Approval Date
DFG	Pending	1600 Streambed Alteration	Pending

b) Does the project require a Federal Energy Regulatory Commission (FERC) license or amendment to a FERC license?

No X Yes__ (Attach application copy)

5. CALIFORNIA ENVIRONMENTAL QUALITY ACT (CEQA)

Indicate CEQA Document (submit final or draft copy*) and Lead Agency:

Categorical Exemption__ Negative Declaration X_ Environmental Impact Report__

Has the document been certified/approved, or has a Notice of Exemption been filed? Yes

If yes, date of approval/filing: 12/19/00 If no, expected approval/filing date:

Lead Agency: Caltrans

*Note, ample time must be provided to the certifying agency to properly review a final copy of valid CEQA documentation before certification can occur.

6. PROJECT SITE DESCRIPTION (INCLUDES AREAS OUTSIDE OF U.S. WATERS)

a) Project Location (Attach map of suitable quality and detail):

City or Area: Los Angeles County: Los Angeles

b) Longitude/Latitude

[Information regarding submittal of longitude and latitude coordinates can be found at : http://www.swrcb.ca.gov/~rwqcb4/html/meetings/401wqc.html]

[A minimum of eight (8) coordinates – All project areas or zones must be delineated with enough waypoints to accurately depict polygons or polylines with at least two (2) points per line segment.]

(Decimal-Degrees) 34 13/58.55" N	(Decimal-Degrees) 118 24'36.93" W
(Decimal-Degrees)_____	(Decimal-Degrees)_____
(Decimal-Degrees)_____	(Decimal-Degrees)_____
(Decimal-Degrees)_____	(Decimal-Degrees)_____

Township/Range: 2N, 15W

c) Total Project Size:

2.42 Acres* 16,638 linear feet (if appropriate)

d) Area Type/Description (check as appropriate):

Urban X	Residential X	Recreation_____
Agriculture_____	Open Space_____	Wildlife Corridor_____
Migratory Pathway_____	Spawning Habitat_____	
Threatened/Endangered Species Habitat_____	Other_____	

*This information is required.

7. IMPACTED WATER BODIES

a) Name(s) of Receiving Water Body(ies)*:
Tujunga Wash

b) Indicate in ACRES and LINEAR FEET (where appropriate) the proposed **waters of the United States** to be impacted by <u>any discharge other than dredging</u>, and identify the impacts(s) as permanent and/or temporary for each water body type listed below:

Jurisdictional Wetland:	_____	permanent,	_____	temporary ACRES
	_____	permanent,	_____	temporary LINEAR FEET
Streambed (vegetated):	_____	permanent,	_____	temporary ACRES
	_____	permanent,	_____	temporary LINEAR FEET
Streambed (unvegetated):	_____	permanent,	~~0.44~~ .07	temporary ACRES
	_____	permanent,	418	temporary LINEAR FEET
Lake/Reservoir:	_____	permanent,	_____	temporary ACRES
	_____	permanent,	_____	temporary LINEAR FEET
Ocean/Estuary/Bay:	_____	permanent,	_____	temporary ACRES
	_____	permanent,	_____	temporary LINEAR FEET
Isolated waters:	_____	permanent,	_____	temporary ACRES
	_____	permanent,	_____	temporary LINEAR FEET

Please explain exactly how waters will be impacted by proposed project activities.
(Attach additional sheets as necessary)
Waters will be impacted by placement of falsework in channel to construct bridge widening. 84 cubic yards will be used for Tujunga Wash Bridge and 84 cubic yards will be used for connector ramps. Due to staging and geometric reasons, it can be expected that falsework for multiple bridges will not be in the channel at the same time. Only falsework for one bridge will be in channel at any given time.

c) Indicate in CUBIC YARDS the volume of <u>Dredged</u> material to be discharged in waters of the United States:

d) Indicate type(s) of material proposed to be discharged in waters of the United States:
Metal or wood beams will be used for falsework. 84 cubic yards will be used for Tujunga Wash bridge and 84 cubic yards will be used for connector ramps. (Refer to sample falsework construction plans for placement and construction details)

*All receiving water bodies must be identified in the *Water Quality Control Plan, Los Angeles Region* (Basin Plan). Any unnamed/unidentified waters must be extended to an identifiable tributary.

8. COMPENSATORY MITIGATION

a) Indicate in ACRES and LINEAR FEET (where appropriate) the total quantity of **waters of the United States** proposed to be Created, Restored and/or Enhanced for purposes of providing Compensatory Mitigation: None

Water Body Type	Created	Restored	Enhanced
Jurisdictional Wetland			
Streambed (vegetated)			
Streambed (unvegetated)			
Lake/Reservoir			
Ocean/Estuary/Bay			

Please describe mitigation activities proposed (Attach additional sheets as necessary).

No mitigation is proposed because the channel is concrete lined and no vegetation is present within the channel. Work will be conducted during the dry season so a water diversion will not be necessary. The only vegetation that will be cleared is landscaped areas adjacent to the existing freeway. Re-planting of existing landscaped plants at a ratio of 1:1 will be performed post construction (refer to planting plan provided for species and quantities). Removal of vegetation will be conducted outside of the bird nesting season to minimize impacts to nesting birds.

b) If contributing to a Mitigation or Conservation Bank, indicate the agency, dollar amount, acreage, and water body type (omit if not applicable):

Conservation Agency_____

$_____ for_____ acres of _____ (water body type)

How many acres of this qualify as waters of the United States?

c) Other Mitigation (omit if not applicable):

How many acres of this qualify as waters of the United States?

e) Location of Compensatory Mitigation Site(s) (Attach map of suitable quality and detail):

City or Area_____ County_____

Longitude/Latitude (Decimal-Degrees)_____
[A minimum of eight (8) coordinates]

9. OTHER ACTIONS/BEST MANAGEMENT PRACTICES (BMPs)

Briefly describe other actions/BMPs to be implemented to Avoid and/or Minimize impacts to waters of the United States, including SUSMPs/Low Impact Development (LID), habitat preservation, erosion control measures, project scheduling, flow diversions, etc.

Caltrans will use appropriate construction BMP's to prevent construction debris from entering the channel.

Removal of vegetation will occur outside the bird nesting season to minimize impacts to nesting birds.

Construction will occur during the dry season to minimize impacts to water quality.

10. PAST/FUTURE PROPOSALS BY THE APPLICANT

Briefly list/describe any projects carried out in the last 5 years or planned for implementation in the next 5 years that are in any way related to the proposed activity or may impact the same receiving body of water. Include estimated adverse impacts.

Additional sections of I-5 are in construction to create an HOV lane. Some portions will require a permit while others will not. Appropriate permits will be applied for at future dates.

Applicant's Signature
(Agent may not sign for Applicant)

Date

Should you have any questions regarding the water quality certification process, please contact Ms. Valerie Carrillo (213) 576-6759 or Mr. Dana Cole (213) 576-5733.

HOLD AT AN ANGLE TOWARD LIGHT TO VERIFY ARTIFICIAL WATERMARK ON FACE & BACK

401/121909

PHIL ANGELIDES, TREASURER

STATE OF CALIFORNIA

SACRAMENTO

VOID AFTER ONE YEAR

ACCOUNT - NUMBER - SERIAL

082- 201410

WARNING: THIS NUMBER BLEEDS THROUGH PINK TO THE BACK

90-1342

1211

082-201410

810709256861 8R31075

PAY TO THE ORDER OF

STATE WATER RESOURCES CTRL BRD
320 WEST FOURTH STREET STE 200
LOS ANGELES CA 90013

ISSUE DATE 09/25/07

CHECK AMOUNT $**3,536.00**

DEPARTMENT OF TRANSPORTATION

By Nancy Kataoka

MICR NUMBER APPEARS PINK ON THE REVERSE SIDE

⑈0082⑈ ⑆121113423⑆ 002014101 ⑈ ⑈0000353600⑈

proof of check cashed

THIS WARRANT MUST BE ENDORSED BY PAYEE EXACTLY AS DRAWN AND WHEN MADE PAYABLE TO MORE THAN ONE PAYEE, EACH PAYEE MUST ENDORSE. NOTE: VERIFY SECURITY FEATURES. ARTIFICIAL WATERMARK ON FACE AND [illegible] THRU PINK ON THE BACK.

Payees Signature Endorsement Area

PAY TO THE ORDER OF

BANK OF AMERICA
INTL JEWELRY CENTER
BRANCH 0310
For Immediate credit to the active
TRUST ACCOUNT of the
Treasurer of the State of California
AT BANK OF AMERICA
ENDORSER'S LINE
Credit given to proper account in this agency
Absence of endorsement guaranteed
STATE WATER RESOURCES
CONTROL BOARD
STATE OF CALIFORNIA
Sacramento, California

0000 03004

BANK OF AMERICA, N.A.
[illegible]

OCT 19

State Treasurer's Office
Received
10/22/2007 20103185 40

4580-10
1203041

TOTAL P.008

RWQCB PERMIT REQUIREMENTS

121841, 121821, 121901

Caltrans shall comply with all applicable provisions of sections 301 (Effluent Limitations), 302 (Water Quality Related Effluent Limitations), 303 (Water Quality Standards and Implementation Plans), 306 (National Standards of Performance), and 307 (Toxic and Pretreatment Effluent Standards).

Construction equipment will be staged in the Caltrans Right of Way adjacent to the freeway and away from watercourses.

All work done within the streambed will be performed during the dry season (Apr 1 – Nov 1) to avoid potential impacts to water quality.

Raw cement/concrete or washing thereof, asphalt, paint, oil, and/or petroleum products, or other substances, which could be hazardous to aquatic life, resulting from project activities, will be prevented from contaminating the soil and/or entering any drainage.

Maintenance and construction equipment shall be checked and maintained daily by a contractor so as to prevent leaks or other potential contamination problems.

At the end of the day when operations are complete, debris and/or trash shall be removed from the work area and properly disposed of by the contractor.

Equipment shall be washed prior to entering the streambed to prevent the spread of invasive species. This includes wheels, undercarriages, bumpers, and all parts of the vehicle. All washing must take place where rinse water is collected and disposed of in either a sanitary sewer or landfill.

Removal of vegetation will occur outside the bird-nesting season (Feb 15-Sep 1), if applicable.

The discharge shall not: a) degrade surface water communities and populations including vertebrate, invertebrate, and plant species, b) promote the breeding of mosquitoes, gnats, black flies, midges, or other pests; c) alter the color, create visual contrast with the natural appearance, nor cause aesthetically undesirable discoloration of the receiving waters; d) cause formation of sludge deposits; or e) adversely affect any designated beneficial uses.

Caltrans shall allow the Regional Board and its authorized representative entry to the premises, including all mitigation sites, to inspect and undertake any activity to determine compliance.

Caltrans shall not conduct any construction activities within waters of the State during a rainfall event. Caltrans shall maintain a five-day clear weather forecast before conducting any operations within waters of the State.

If rain is predicted after operations have begun, grading activities must cease immediately and the site must be stabilized to prevent impacts to water quality, and minimize erosion and runoff from the site.

121841 AND 121901

All surface waters, including ponded waters, shall be diverted away from areas undergoing grading, construction, excavation, vegetation removal, and/or any other activity, which may result in the discharge to the receiving water. If surface water diversions are anticipated, the Applicant shall develop and submit a Surface Water Diversion Plan to the Regional Water Quality Control Board. The plan shall include the proposed method and duration of diversion activities, structural configuration, construction materials, equipment, erosion and sediment controls, and a map or drawing indicating the locations of diversion and discharge points. Contingency measures shall be a part to any surface water diversions. If surface flows are present, then upstream and downstream monitoring for the following shall be implemented:

- PH
- Temperature
- Dissolved oxygen
- Turbidity
- Total suspended solids (TSS)
- Downstream TSS shall be maintained at ambient levels
- Where natural turbidity is between 0 and 50 NTU, increase shall not exceed 20%. Where natural turbidity is greater than 50 NTU, increases shall not exceed 10%

Analyses must be performed using EPA methods. These constituents shall be monitored for on a daily basis during the first week of diversion and/or dewatering activities and then on a weekly basis, thereafter, until in-stream work is complete.

Caltrans shall utilize the services of a qualified biologist during surface water diversion. The biologist shall have the authority to stop work, as necessary, if instructions are not followed.

121841 ONLY

Bridgework may not occur during swallow nesting season (Feb 15-Sep 1). If work must be done during the nesting season, a qualified biologist will be notified two weeks prior to construction to confirm the absence of nesting birds. If birds are present, work will cease until fledglings have left the nests. Exclusionary devices will be implemented if deemed necessary.

Jeff DeFevere/HQ/Caltrans/CAGov
05/05/2009 07:28 AM

To Branch 1
cc Rebecca Harnagel/HQ/Caltrans/CAGov@DOT
bcc
Subject Re: Did you know about the 401 permit waiver process? I didn't.

In case you were not aware of the permit waiver process ... see below.

----- Forwarded by Jeff DeFevere/HQ/Caltrans/CAGov on 05/05/2009 07:22 AM -----

Shawna Pampinella/HQ/Caltrans/CAGov
05/04/2009 03:13 PM

To Jeff DeFevere/HQ/Caltrans/CAGov@DOT
cc Paul D Caron/D07/Caltrans/CAGov@DOT, Richard Hill/HQ/Caltrans/CAGov@DOT
Subject Re: Fw: 07-121901 401 permit waiver process??

Jeff,
Hopefully this quick Q&A helps. Typically if the RWQCB does not respond to applicant within 30 days of receipt, the application is deemed complete. The board has an additional 30 days to issue or deny 401 cert. If there is inaction, the 401 cert has been waived. I know this has been the case for some projects in southern California and the Corps typically communicates with the RWQCB on this as well. If they do not get response they typically issue the 404. What you've been told by the bio's and planners falls in line with the process. As long as this is all documented and in the project files, there shouldn't be an issue.

http://www.swrcb.ca.gov/rwqcb6/water_issues/programs/clean_water_act_401/docs/401_Cert_action_TIMELINES_5-22-03.pdf

_____
Shawna Pampinella
Interagency Liaison
Office of Biological Studies and Technical Assistance
Phone: (916) 653-8056
Cell: (916) 201-4597
Shawna_Pampinella@dot.ca.gov

Jeff DeFevere/HQ/Caltrans/CAGov

Jeff DeFevere/HQ/Caltrans/CAGov
05/04/2009 02:23 PM

To Richard Hill/HQ/Caltrans/CAGov@DOT
cc Paul D Caron/D07/Caltrans/CAGov@DOT, Shawna Pampinella/HQ/Caltrans/CAGov@DOT
Subject Re: Fw: 07-121901 401 permit waiver process??

Thanks Richard. Just want to make sure that D7 doesn't have any problems in construction.

Jeff DeFevere
Office of PS&E: NR & District 7
DES-Office Engineer

(916) 227-6234
Richard Hill/HQ/Caltrans/CAGov

Richard Hill/HQ/Caltrans/CAGov
05/04/2009 02:16 PM

To Jeff DeFevere/HQ/Caltrans/CAGov@DOT
cc Shawna Pampinella/HQ/Caltrans/CAGov@DOT, Paul D Caron/D07/Caltrans/CAGov@DOT
Subject Re: Fw: 07-121901 401 permit waiver process??

Hi Jeff, I an not an expert on the current arcane mechanisms of the State and Regional Boards, nor of the ACOE. I need to refer you to Shawna Pampinella who may be able to assist through our interagency representatives. You need more than a general answer.

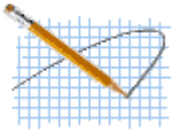
The guidance I have used in the past was from the FHWA web site: http://www.fhwa.dot.gov/cadiv/pre/404qas.htm (emphasis added below). It may explain some of the reason why the Board may not have responded.

> ***9. Section 401 Certification or Waiver***
>
> **Question**
> How should the preliminary agreement for the 401 certification or waiver be achieved for final NEPA documents? Appendix A of the MOU indicates a section 401 certification or waiver must be obtained from the state water quality management agency prior to approval of the NEPA document (MOU Appendix A, pages 2 and 6). However, state water quality management agencies will not usually issue a certification or waiver prior to receiving additional final project approval documents. In California, for example, the **Regional Water Quality Control Boards cannot grant a 401 certification or waiver until CEQA compliance is achieved**. (CEQA, the California Environmental Quality Act, is the California counterpart to NEPA, and for FHWA/Caltrans projects, CEQA and NEPA documents are processed jointly.)
>
> **Answer**
> State water quality management agencies did not participate in the development of the NEPA 404 MOU. As the MOU and Guidance Papers were being developed, it was assumed that the state water quality management agencies would respond to the section 404 permit process whether it occurred after NEPA approval (as was the case prior to the MOU) or during the development of the final NEPA document (as outlined in the MOU). During subsequent MOU implementation, it was realized that state water quality management agencies may need additional post-NEPA/CEQA documentation prior to granting a 401 certification or waiver, particularly in California.
>
> If a section 401 certification or waiver can not be obtained from the state water quality management agency during the development of the final NEPA document as outlined in Appendix A of the MOU, **then the final NEPA document will need to contain evidence of coordination** (documented telephone call or copy of letter or other record of communication) with the state water quality management agency. Ideally, this coordination would address any anticipated conditions or concerns that might arise during the 401 certification or waiver process.

Shawna will likely be able to direct you to a current answer that may be different.

Richard E. Hill, SEP
Aquatic Ecologist
Biological Studies Unit
PO Box 942874 MS27
Sacramento CA USA 94274-0001
916-653-8417 Office

916-653-7757 FAX
Jeff DeFevere/HQ/Caltrans/CAGov



Jeff DeFevere/HQ/Caltrans/CAGov

05/04/2009 01:52 PM

To Richard Hill/HQ/Caltrans/CAGov@DOT

cc

Subject Fw: 07-121901 401 permit waiver process??

Good afternoon,
District 7 Biologists and Environmental Planners do not have a 401 permit but state that it is not necessary as a result of the "401 waiver process."

I'm not familiar with this waiver process. Is this true?

- 401 waiver process:

-The waterboard has 60 days from the date of application submittal to respond or request additional information. Each time they request additional information the 60 day clock is reset. At the end of a 60 day period with no communication from the waterboard the 401 application legally becomes a valid 401 certification. This is a technicality and the ACOE does not normally accept as a valid 401.

-The permit was applied for in September 2007.

- The ACOE requires a valid 401 certification before issuing a 404 authorization. It is possible to have a 404 authorized without a complete 401 certification if proper documentation is forwarded to the ACOE. I sent the ACOE copies of the mailing labels and cashed checks, for the 401 application fees. The ACOE was unsuccessful in attempts to contact the waterboard and **as a result our 404 was issued in October 2008**.

- The waterboard failed to respond to both Caltrans and the ACOE. the 60 day waiting period had lapsed, **therefore the 401 application became a valid permit** in September 2008. The issuance of the 404 further substantiates the claim that our 401 application is a valid permit. (The ACOE very rarely issues 404 authorizations without a complete 401.)

Thanks,
Jeff DeFevere
Office of PS&E: NR & District 7
DES-Office Engineer
(916) 227-6234
----- Forwarded by Jeff DeFevere/HQ/Caltrans/CAGov on 05/04/2009 01:35 PM -----

Mumbie Fredson-Cole/D07/Caltrans/CAGov

05/04/2009 01:31 PM

To Keith Teraoka/D07/Caltrans/CAGov@DOT

cc Jeff DeFevere/HQ/Caltrans/CAGov@DOT, Khan A Hossain/D07/Caltrans/CAGov@DOT, Nelson Tran/HQ/Caltrans/CAGov@DOT

Subject Re: Fw: 07-121901 401 permit

Keith,

I concur.

Mumbie Fredson-Cole, P.E., PMP
Senior Transportation Engineer
Project Manager
Phone 213 897 9355
Fax 213 897 0500
Keith Teraoka/D07/Caltrans/CAGov

Keith Teraoka/D07/Caltrans/CAGov
05/04/2009 01:28 PM

To Khan A Hossain/D07/Caltrans/CAGov@DOT, Mumbie Fredson-Cole/D07/Caltrans/CAGov@DOT
cc Jeff DeFevere/HQ/Caltrans/CAGov@DOT, Nelson Tran/HQ/Caltrans/CAGov@DOT
Subject Fw: 07-121901 401 permit

Khan and Mumbie,
Looks like we'll be able to use the Permit Streamlining Act de facto 401 Permit with the PM's, Biological, and Environmental Planner's concurrence. If this is acceptable to you, please send your concurrence for inclusion into the job file (JF). The concurrence will be for the use of the permit application and utilization of the Permit Streamlining Act as our de facto permit.

Nelson Tran/HQ/Caltrans/CAGov

Nelson Tran/HQ/Caltrans/CAGov

Nelson Tran
916-227-6249
Keith Teraoka/D07/Caltrans/CAGov

Keith Teraoka/D07/Caltrans/CAGov
05/04/2009 11:23 AM

To Nelson Tran/HQ/Caltrans/CAGov@DOT
cc Khan A Hossain/D07/Caltrans/CAGov@DOT
Subject Fw: EA: 212901 401 permit

If this is the case, let's use the application and claim invocation of the Permit Streamlining Act, California Government Code Section 65920-65924.

In the special provision, we can list the permit and the above reference as authority to use the application in lieu of the permit:

"California Regional Water Quality Control Board Section 401 Permit invoked by California Government Code Sections 65920-65924 (Permit Streamlining Act)"

----- Forwarded by Keith Teraoka/D07/Caltrans/CAGov on 05/04/2009 11:14 AM -----

Khan A Hossain/D07/Caltrans/CAGov
05/04/2009 11:11 AM

To Keith Teraoka/D07/Caltrans/CAGov@DOT
cc Nelson Tran/HQ/Caltrans/CAGov@DOT

Subject Re: Fw: EA: 212901 401 permit

Hi Keith,

We have only the 401 Permit Application that is considered, as per Environmental Branch, as Permit. The application is attached.

[attachment "121901 RWQCB Permit.pdf" deleted by Keith Teraoka/D07/Caltrans/CAGov]

Please let me know if I can be of any help.

Khan, Office of Design D
7-0239
Keith Teraoka/D07/Caltrans/CAGov

Keith Teraoka/D07/Caltrans/CAGov
05/04/2009 10:36 AM

To Khan A Hossain/D07/Caltrans/CAGov@DOT
cc Nelson Tran/HQ/Caltrans/CAGov@DOT
Subject Re: Fw: EA: 212901 401 permit

Khan,
Do we have a copy of the 401 Permit (conforming to the Permit Streamlining Act) to use as an Informatio Handout? Is there a general permit or something specific. We need something to include for the info handout soon.
Thanks for your attention.
Keith

Khan A Hossain/D07/Caltrans/CAGov

Khan A Hossain/D07/Caltrans/CAGov
05/01/2009 08:12 AM

To Keith Teraoka/D07/Caltrans/CAGov@DOT
cc
Subject Fw: EA: 212901 401 permit

FYI please.
----- Forwarded by Khan A Hossain/D07/Caltrans/CAGov on 05/01/2009 08:11 AM -----

Paul D Caron/D07/Caltrans/CAGov
04/30/2009 08:33 AM

To Garrett Damrath/D07/Caltrans/CAGov@DOT
cc Eric Hanson/D07/Caltrans/CAGov@DOT, Khan A Hossain/D07/Caltrans/CAGov@DOT
Subject Re: EA: 212901 401 permit

I concur with Eric Hanson's statement on this issue. The 401 permit period is passed, per Code. The issuance of the 404 permit allows this project to move forward. thank you for your patience,

Paul Caron

Senior District Biologist
District 7 (L.A./Ventura Counties)
ph: 213 897-0610
fax: 213 897-0685

Garrett Damrath/D07/Caltrans/CAGov

Garrett Damrath/D07/Caltrans/CAGov

04/29/2009 12:44 PM

To Eric Hanson/D07/Caltrans/CAGov@DOT

cc Khan A Hossain/D07/Caltrans/CAGov@DOT, Paul D Caron/D07/Caltrans/CAGov@DOT

Subject Re: EA: 212901 401 permit

I concur with this statement.

Sincerely,

Garrett Damrath
Senior Environmental Planner
Division of Environmental Planning
Caltrans, District 7
213-897-9016

Eric Hanson/D07/Caltrans/CAGov

Eric Hanson/D07/Caltrans/CAGov

04/29/2009 12:25 PM

To Khan A Hossain/D07/Caltrans/CAGov@DOT

cc Paul D Caron/D07/Caltrans/CAGov@DOT, Garrett Damrath/D07/Caltrans/CAGov@DOT

Subject EA: 212901 401 permit

Kahn,
This e-mail is to follow up on our conversation earlier today.

Paul,
Please cc your concurrence with the information below to Khan.

- 401 waiver process:

-The waterboard has 60 days from the date of application submittal to respond or request additional information. Each time they request additional information the 60 day clock is reset. At the end of a 60 day period with no communication from the waterboard the 401 application legally becomes a valid 401 certification. This is a technicality and the ACOE does not normally accept as a valid 401.

-The permit was applied for in September 2007.

- The ACOE requires a valid 401 certification before issuing a 404 authorization. It is possible to have a 404 authorized without a complete 401 certification if proper documentation is forwarded to the ACOE.

I sent the ACOE copies of the mailing labels and cashed checks, for the 401 application fees. The ACOE was unsuccessful in attempts to contact the waterboard and as a result our 404 was issued in October 2008.

- The waterboard failed to respond to both Caltrans and the ACOE. the 60 day waiting period had lapsed, therefore the 401 application became a valid permit in September 2008. The issuance of the 404 further substantiates the claim that our 401 application is a valid permit. (The ACOE very rarely issues 404 authorizations without a complete 401.)

If you have any further questions please contact myself or my senior, Pul Caron (7-0610).

Thanks,
Eric

Environmental Planner/Biologist
Phone: (213) 897-9362

_____
California Department of Transportation
District 07- Division of Environmental Planning
100 South Main St, MS 16A - Los Angeles, Ca 90012-3712

Reference Number 7696 Job Address: N/A TONOPAH ST

CLASS "B" APPLICATION / PERMIT

ISSUED UNDER SECTIONS 62.105 TO 62.116 INCLUSIVE, LOS ANGELES MUNICIPAL CODE **(SEE REVERSE HEREOF)**

DISTRICT:	BD Number	BC Number
VALLEY	BD204802	

I HEREBY REQUEST PERMISSION TO CONSTRUCT THE IMPROVEMENTS DESCRIBED BELOW UNDER PRIVATE CONTRACT AND, IF PERMISSION IS GRANTED, I AGREE TO CONSTRUCT SAME IN ACCORDANCE WITH PLANS AND/OR SPECIFICATIONS APPROVED BY THE CITY ENGINEER. I ALSO ACKNOWLEDGE THAT I HAVE REVIEWED THE GENERAL INFORMATION PRESENTED ON THE REVERSE OF THIS APPLICATION, AND AGREE TO THE PROVISIONS STATED THEREON.

SHEET NO. 1 OF 2 SHEETS	DATE ISSUED: 07/17/2008	DATE EXPIRES: 07/17/2010

PROJECT: BD: EBPERGEN	PROGRAM: 0400	ENGR DIST.: E6201810	REIMB FLG: YES	DIR/INDIR: D

STREETS AND LIMITS TO BE IMPROVED
(SEE CONSTRUCTION TOTALS AT RIGHT)
CALTRAN STREET AND STORM DRAIN IMPROVEMENTS - ALL FEES WAVIED - ANOTHER GOVERNMENTAL AGENCY

ESTIMATED 2% SURCHARGE = **$0.00**
ESTIMATED 7% SURCHARGE = **$0.00**

INDICATE SCOPE

ENGR. PLAN CHECK DEPOSIT	Number of Trees	Wells	TREE PLANTING FEE	District Map Number	DIVISION INDEX	DRAINAGE MAP	COUNCIL DISTRICT	CONSTRUCTION ITEMS	ESTIMATED COST
$0.00	0	0	$0.00			TG 502-E7 TO C3	6		

SANDBLASTING FEE	DOT SIGN DEPOSIT	TRAFFIC DEPOSIT	PROPOSED R/W NO.	SPECIAL REFERENCES	GRADING	$0.00
$0.00	$0.00	$0.00				

CONST. - INSPECTION DEPOSIT $0.00	TOTAL FEE DEPOSIT $0.00	PAVING CURB / GUT. / SDWLK	$0.00
TYPE OF PROJECT General Improvement	TOTAL BOND DEPOSIT $0.00	SEWER	$0.00
IMPROVEMENT BOND NO.		STORM DRAIN	$0.00
LIABILITY INSURANCE NO.	LIAB. INSUR. EXPIR. DATE	STREET LIGHTS	$0.00
PRIVATE ENGINEER		TRAFFIC SIGNALS	$0.00
STREET ADDRESS		**SUBTOTAL**	$0.00
CITY / ZIP / PHONE NO.		+ % PCF	$0.00
OWNER-APPLICANT (SEE REVERSE) CALTRANS		CONSTRUCTION ESTIMATE	$0.00
STREET ADDRESS 100 S. MAIN ST, STE 100		+ % CIF	$0.00
CITY LOS ANGELES / STATE CA / ZIP 90012		BOND ESTIMATE	$0.00
(AREA CODE) PHONE NO. 213-897-1234	DATE SIGNED BY APPLICANT	REGISTER RECEIPT VALIDATION	
SIGNATURE (SEE REVERSE BEFORE SIGNING)	**Job Address: N/A TONOPAH ST**		
APPLICANT'S NAME (ADD TITLE IF OFFICER)			
DISTRICT/DIVISION OFFICE	ISSUED BY: DEBRA ENGLE	DIST. ENGR. M. KANTOR	DATE 8-15-08
TIME	DATE		

Permit Title:
ROUTE 5 HOV FROM TONOPAH TO VAN NUYS BLVD

1/2

GENERAL INFORMATION

This form shall be used for all Class "B" Permits. It is not necessary to repeat the STREETS AND LIMITS breakdown on **CONSTRUCTION** B-PERMITS that are preceded by checking only permits.

1. LIABILITY INSURANCE and performance bonds are only required for approval of construction B-Permits.

2. OWNER-APPLICANT must be the owner of the affected property, and must be the same as the performance bond principal. Contractors for governmental agencies under faithful performance bonds shall be entered as OWNER-APPLICANT. Applications issued to CORPORATIONS shall be executed by the President, Vice-President, Secretary or Asst. Secretary, and the CORPORATE SEAL shall be impressed.

3. SAFETY ORDERS—Sec. 6500 of the STATE LABOR CODE requires the APPLICANT to obtain a **STATE DIVISION OF INDUSTRIAL SAFETY (DIS) PERMIT** and a copy of **STATE INDUSTRIAL SAFETY ORDERS** before beginning work on any trench or excavation **FIVE FEET OR DEEPER** into which a person must descend. City personnel will not enter such excavations until the required permit and safety orders have been obtained and are displayed on the job site.*

4. PCF percentage factors apply to possible construction change orders. **CIF** percentage factors apply to cost increases through the life of the project. (See CONSTRUCTION ITEMS on the reverse side of this application.)

5. PLAN CHECKING DEPOSITS are subject to charges for blueprints, engineering, testing, traffic signal design, street lighting design, and administrative service. **CONSTRUCTION PERMIT DEPOSITS** may be charged against for blueprints, engineering, testing, surveying, inspection, administrative services, emergency light and barricade services, street tree planting and maintenance, street lighting burn tests, street name signs, traffic warning and regulatory signs, sandblasting and painting of traffic markings, erosion control, etc. Unused fees are refunded after closure of the permit. The applicant will be billed for unreimbursed charges (fee deficits).

6. DESIGN OFFICE will determine if applicant must dedicate a right of way to the City for construction purposes. The applicant shall open a right of way work order and deposit necessary fees with the REAL ESTATE DIVISION, BUREAU OF ENGINEERING.

7. NOT MORE THAN ONE TRACT OR PARCEL MAP SHALL BE ASSIGNED TO A SINGLE B-PERMIT.

☐ **THIS CONSTRUCTION B-PERMIT MUST BE ON THE JOB SITE AT ALL TIMES.**

☐ **THIS PERMIT VALID ONLY IF ACCOMPANIED BY A DIS PERMIT AND SAFETY ORDERS** (SEE NOTE 3 ABOVE). **DIS** PERMIT MAY BE OBTAINED FROM STATE DIVISION OF INDUSTRIAL SAFETY, 3460 WILSHIRE BOULEVARD, LOS ANGELES. **NOTE:** GOVERNMENTAL AGENCIES AND PUBLIC UTILITIES UNDER **PUC** CONTROL **ARE EXEMPT.** PRIVATE CONTRACTORS ACTING AS PERMITTEE FOR SUCH AGENCIES **ARE NOT EXEMPT.**

NOTE: CHARGES AGAINST THE PERMIT FEE DEPOSIT WILL BE COMPILED APPROXIMATELY 6 TO 8 WEEKS AFTER THE DATE FINAL INSPECTION IS COMPLETED. THE B-PERMIT WILL BE CLOSED AND THE IMPROVEMENT BOND EXONERATED AFTER ALL PERMIT FEE CHARGES HAVE BEEN ACCUMULATED AND PAID.

MULTIPLE PERMITTEE ENTRIES

2/2

CLASS "B" APPLICATION/PERMIT — ISSUED UNDER SECTIONS 62.105 TO 62.116 INCLUSIVE, LOS ANGELES MUNICIPAL CODE **(SEE REVERSE HEREOF)**

DIST: BD 204802
☐ CEN ☐ HAR
☐ WLA ☐ VAL

I HEREBY REQUEST PERMISSION TO CONSTRUCT THE IMPROVEMENTS DESCRIBED BELOW UNDER PRIVATE CONTRACT AND, IF PERMISSION IS GRANTED, I AGREE TO CONSTRUCT SAME IN ACCORDANCE WITH PLANS AND/OR SPECIFICATIONS APPROVED BY THE CITY ENGINEER. I ALSO ACKNOWLEDGE THAT I HAVE REVIEWED THE GENERAL INFORMATION PRESENTED ON THE REVERSE OF THIS APPLICATION, AND AGREE TO THE PROVISIONS STATED THEREON.

SHEET NO.	CHECKING NO. BD-	CONSTRUCTION NO. BC-
OF SHEETS	DATE ISSUED	DATE EXPIRES

PROJECT: PROGRAM: ENGR DIST: REIMB FLG: Y OR N DIR/INDR: D OR I

STREETS AND LIMITS TO BE IMPROVED
(SEE CONSTRUCTION TOTALS AT RIGHT)

INDICATE SCOPE — PAVING, CURB, GUTTER, WALK, SEWERS, DRAINS, CATCH BASINS, LIGHTS, TREES, ETC.

HOV Project on Rte 5

ENGR. PLAN CHECK DEPOSIT $	NO. TREES	WELLS	TREE PLANTING FEE $
SANDBLASTING FEE $	DOT SIGN DEPOSIT $	TRAFFIC DEPOSIT $	
CONSTRUCTION-INSPECTION DEPOSIT $		TOTAL FEE DEPOSIT $	
TYPE OF PROJECT		TOTAL BOND DEPOSIT*	
IMPROVEMENT BOND NO.		BONDING PURPOSE	
LIABILITY INSURANCE NO.		LIAB. INSUR. EXPIR. DATE	

DISTRICT MAP NO.	DIVISION INDEX	DRAINAGE MAP	COUNCIL DIST.
PROPOSED R/W NO.	SPECIAL REFERENCES		

CONSTRUCTION ITEMS	ESTIMATED COST
GRADING	$
PAVING, CURB/ GUT./SDWLK.	$
SEWER	$
STORM DRAIN	$
STREET LIGHTS	$
TRAFFIC SIGNALS	$
	$
+ % PCF	$
CONSTRUCTION ESTIMATE	$
+ % CIF	$
PLAN ESTIMATE*	$
BOND ESTIMATE	$

PRIVATE ENGINEER
CALTRANS
STREET ADDRESS
100 S. MAIN STREET, SUITE 100
CITY: LOS ANGELES ZIP CODE: 90012 TELEPHONE NO.: 213-897-[redacted]1234

OWNER-APPLICANT (SEE REVERSE)
CALTRANS (SAID Jallen 213897-1234)
STREET ADDRESS
100 SOUTH MAIN STREET, SUITE 100
CITY: LOS ANGELES STATE: CA ZIP CODE: 90012
(AREA CODE) TELEPHONE NO.: 213-897-5607 DATE SIGNED BY APPLICANT
SIGNATURE (SEE REVERSE SIDE BEFORE SIGNING)
APPLICANT'S NAME (TYPE OR PRINT, ADD TITLE IF OFFICER)
SULE ALABI

DISTRICT/DIVISION OFFICE ISSUED BY:
TIME DATE

APPROVED-ENGINEER IN CHARGE
DATE:
DIST. ENGR.

REGISTER RECEIPT VALIDATION

PERMIT TITLE (INCLUDE TRACT, PMLA, CPC, ZA OR PVT. ST. NUMBER, OR R3)

ENG. 3.659-A — (Rev. 7/87)
DEPARTMENT OF PUBLIC WORKS
City of Los Angeles

1/1

VIEW PLAN

Work Order Number:	**0**
B-Permit Number:	**BD204802**
Project Title:	**ROUTE 5 HOV FROM TONOPAH ST TO VAN NUYS BL**
Number of Sheets:	**12**
District Engineer Signed:	**2009-04-20 00:00:00**
INDEX Number:	**P-37176**

Date Received:	**04/07/2009**
From:	**VALLEY**
Work Type:	**STREET**
Work Name:	
New/Revised:	**NEW**
Contact:	**DYNL MIRANDA**
Phone 1:	**818-374-4600**
Phone 2:	

Last Update Time **04/23/2009 16:08:54**

PLAN REVIEW STATUS OF ALL DISCIPLINES IN ROW FORMAT

Plan Check ID	Plan Check Name	Sequence	Clearance	Plan Check Conditions	Approval Date	Last Update On
VALLEY	VALLEY DISTRICT/BUREAU OF ENGINEERING-DPW	1	YES		2009-04-20 15:08:50	2009-04-21 14:08:11
DOT	DEPARTMENT OF TRANSPORTATION	2	YES		2009-04-08 16:21:35	2009-04-20 15:09:09
TREE	STREET TREE/URBAN FORESTRY/BUREAU OF STREET SERVICES	3	YES		2009-04-16 10:55:06	2009-04-20 15:09:17
VAULT	VAULT INDEX/BUREAU OF ENGINEERING-DPW	4	YES	P-37176	2009-04-23 16:08:40	2009-04-23 16:08:40
MICROFILM	MICROFILM/REPROGRAPHICS/BUREAU OF ENGINEERING-DPW	5	YES		2009-04-23 16:08:54	2009-04-23 16:08:54

Agency	Approve By	Approve On
[1]VALLEY	D.M.	04/20/2009 03:44 PM
[2]VALLEY	D.M.	04/07/2009 08:59 AM
[2]DOT	F.E.	04/08/2009 04:57 PM

[2]TREE	O.M.	04/16/2009 11:11 AM

[1] Approval for sheet signed by the District Engineer
[2] Approval for unsigned sheet

Index Number	File
P-37176	BD204802002_20090407_082355.tif
P-37176	BD204802003_20090407_082407.tif
P-37176	BD204802004_20090407_082422.tif
P-37176	BD204802005_20090407_082434.tif
P-37176	BD204802006_20090407_082446.tif
P-37176	BD204802007_20090407_082458.tif
P-37176	BD204802008_20090407_082513.tif
P-37176	BD204802009_20090407_082525.tif
P-37176	BD204802010_20090407_082538.tif
P-37176	BD204802011_20090407_082550.tif
P-37176	BD204802012_20090407_082601.tif
P-37176	BD204802001_20090420_150752.tif

STATE OF CALIFORNIA

MINI-MEMO

To: Khan Hossein	Subject::07-LA-5,170 KP 58.5(Bridge No.53-1121L/R) Tujunga wash channel EA 07-121901	Date: December 23, 2008

MESSAGE: Attached is the conditional permit from the Los Angeles County Dept of Public Works (LACDPW) Flood Control District (LACFCD) for the work affecting Tujunga wash channel as part of the above referenced Contract.

Please be advised that the attached permit is for bidding purposes only. Please include instructions to the Resident Engineer that **the Contractor will have to pay an inspection fee in the sum of $ 25,000.00** to the County of Los Angeles Department of Public Works as indicated on page 1 of the permit. Upon receipt of payment the County will release the "Official" permit. In order to allocate funds for the Resident Engineer to reimburse the Contractor for the expense of the Permit - please include a "dummy" BEES item under Supplemental Funds in your BEES. If you are unsure how to establish the "dummy" BEES item for this purpose, please contact the District 07 Office Engineer for guidance.

Please insure that your Standard Special Provisions (SSP's) make reference to this permit and any requirements/restrictions on the Contractor it may have. The conditional Permit is enclosed for the Project files, the official permit (Field Copy) will be released to the contractor prior to construction and after fee payment.

Thank you

If you have any questions or comments please contact me at 7-6362

Return To:	Signed Mike Nouri	Address: Hydraulics 07-160	Phone: 7-6362

REPLY_____

Signed	Address	Date

Tract #:

Issued By:
Issued Date:

Permit #: **PCFL T200701397**

FOR BIDDING PURPOSES ONLY

Permit Office: 6

PC-MODIFIC
MODIFICATION OF FLOOD
CONTROL FACILITY

COUNTY OF LOS ANGELES-DPW
Department Of Public Works
Alhambra, CA 91803 - (626)458-3129

Flood Control District Permit

Individual's / Company Name	Address / City, State Zip	Work Phone	Home Phone
(APP) STATE OF CALIFORNIA, DOT MIKE NORI	100 S. MAIN ST., STE 100, MS13 LOS ANGELES, CA 90012	213-897-6362	
(CNT) STATE OF CALIFORNIA, DOT SAID JALLEO	100 S. MAIN ST., STE 100, MS12 LOS ANGELES, CA 90012	213 897-1234	

Emergency Contact

Location

Site Address:

Description: TUJUNGA WASH CHANNEL: I-5 FREEWAY AND ROUTE I-170, LOS ANGELES

Scope of Work

PERMIT PURPOSE:

********FOR BIDDING PURPOSES ONLY. NOT FOR CONSTRUCTION********

TO AUTHORIZE THE WORK AFFECTING THE SUBJECT STREAM IN ACCORDANCE WITH THE SUBMITTED PLANS, LOS ANGELES COUNTY FLOOD CONTROL DISTRICT DRAWING Nos. 11-F594.1-.20 (LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS DRAWING Nos. PF543907-926.)

WORK DESCRIPTION:
TO WIDEN TUJUNGA WASH CHANNEL NORTHBOUND & SOUTHBOUND BRIDGES AT I-5 FREEWAY, CONSTRUCT I-5 FREEWAY TO ROUTE 170 HOV CONNECTOR AND ROUTE 170 TO I-5 FREEWAY CONNECTOR. REMOVE EXISTING CONCRETE BARRIER, PORTION OF OVERHANG AND EXISTING WING WALLS. RETROFIT EXISTING COLUMNS. REMOVE TEMPORARILY TOP OF CHANNEL WALL FOR FALSEWORK BEAMS AND RECONSTRUCT AFTER FALSEWORK REMOVAL. REMOVE PERMANENTLY INTERFERING PORTION (6") OF CHANNEL WALL AND CONSTRUCT WALL EXTENSIONS. ALL WORK SHALL BE DONE PER SUBMITTED PLANS.

WORK WITHIN DISTRICT'S RIGHT OF WAY SHALL NOT COMMENCE UNTIL AN INSPECTION DEPOSIT OF $25,000 HAS BEEN PAID, CONTRACTOR HAS PROVIDED CONTACT INFORMATION AND LIABILITY INSURANCE, AND FALSEWORK PLANS HAVE BEEN APPROVED BY THE DISTRICT.

NO WORK IS ALLOWED IN OR AFFECTING THE CHANNEL BETWEEN OCTOBER 15 AND APRIL 15.

PERMITTEE MUST NOTIFY PERMIT OFFICE No. 2 (7:00 AM TO 3:30 PM) AT TELEPHONE (661) 222-2948 AT LEAST 24 HOURS BEFORE STARTING WORK UNDER THIS PERMIT. FAILURE TO SO NOTIFY THE PERMIT OFFICE IS CAUSE FOR REVOCATION OF PERMIT. SHOULD PERMITTEE FAIL TO TAKE ACTION WITHIN 180 DAYS FROM DATE OF ISSUANCE OF THIS PERMIT OR FAIL TO ACTIVELY AND DILIGENTLY EXERCISE THE PRIVILEGES OF THIS PERMIT, THE PERMIT BECOMES NULL AND VOID.

A COPY OF THIS PERMIT SHALL BE KEPT AT THE WORK SITE DURING ALL PERIODS OF OPERATION WITHIN THE DISTRICT'S RIGHT OF WAY AND SHALL BE SHOWN TO ANY DISTRICT REPRESENTATIVE OR LAW ENFORCEMENT OFFICER UPON DEMAND.

CC: Design (Chang, Zandieh)
Flood Maintenance (West)
Mapping and Property Management (Hernandez, Rothman)
Construction (Office, P.O. 2, Houmsi)

Permit Detail

FILE CODE NO. :	11.032
FLOOD FACILITY NAME :	TUJUNGA WASH CHANNEL
FLOOD STATION :	383+35
INSPECTION PCA :	TBD AFTER $25,000 PROVIDED BY CONTRACTOR
INSURANCE EXPIRE	TO BE PROVIDED BY CONTRACTOR
LOCATION 1:	I-5 FREEWAY AND ROUTE I-170, LOS ANGELES
THOMAS GUIDE :	502-E7

PCFL T200701397

REPORT: lapwrp028

Tract #:

Issued By:
Issued Date:

Permit #: **PCFL T200701397**

FOR BIDDING PURPOSES ONLY

Permit Office: 6

Comments

Fees	Fee Code	Acount Code	Amount
ACTUAL COST DEP FOR PLN CHK AND/OR INSP	PCACTPLD	B07_8371	$25,000.00
CONCEPT REVIEW NO FEE FLOOD PERMIT	PCCON99PLC	B07_8371	$0.00
PLN CK OF CAL TRANS PROJ.-FCD R/W-NO FEE	PCADOTPLCK	B07_8371	$0.00
		Total Fees:	$25,000.00

Is hereby permitted to complete scope of work on the public highways subject to provisions required by County of Los Angeles Highway Permit Ordinance (Division 1 of Title 16, Los Angeles County Code), the Municipal Code, and City Ordinance governing the area where this work is to be done, and the attachments hereon specified. Permit revocable at option of Public Works Director, in consideration of granting of this permit, it is agreed by the applicant that the County of Los Angeles and/or the city wherein the permit work is to be performed and any of their officers or employees thereof shall be saved harmless by the applicant from any liability or responsibility for any accident, loss, or damage to persons or property, happening occurring as the proximate result of any of the work undertaken under the terms of this application and the permit or permits which may be granted in response thereto, and that all of said liabilities are hereby assumed by the applicant, it is further agreed that if any part of this installation interferes with the future use of the highway by the general public, it must be removed or relocated, asdesignated by the Director of Public Works or Superintendent of Streets, at the expense of the permittee of his successor in interest. The permit is void if the permittee is not in compliance with Section 3800 of the Labor Code

Performance of the work of activity under this permit is tantamount to agreeing to the conditions of this permit, Copy of this permit shall be kept at work site during period of operation within District's/Road right of way and shall be shown to District's representative or any law enforcement officer upon demand.

INSPECTION REQUIRED

CALL PERMIT OFFICER 24 HOURS BEFORE STARTING WORK UNDER THIS PERMIT. FAILURE TO DO SO IS CAUSE FOR REVOCATION OF THIS PERMIT. THIS PERMIT IS VOID IF WORK NOT STARTED IN 60 DAYS (FOR ROAD PERMIT) OR 180 DAYS (FOR FLOOD PERMIT) FROM THE DATE OF THE ISSUANCE.

PERMIT OFFICE NO. PCHQ
PUBLIC WORKS CONSTRUCTION
900 S. Fremont Ave.
Los Angeles County, CA 91803
PHONE NO. 626-458-3129
FAX NO. 626-576-7739

PCFL T200701397

REPORT: lapwrp028

Conditions of Approval
By Permit

Run Date: Tuesday December 23, 2008 7:4

Permit: PCFL - T200701397

The following Conditions of Approval are required to complete the permit:

Condition of Approval	Entered	By	Completed	By
GENERAL FLOOD PROVISION NO. 1	22-DEC-08	HHOUMSI		
Use of District's right of way for the construction or activity authorized under this permit is tantamount to agreeing to the conditions herein.(G1)				
GENERAL PROVISION NO.2	22-DEC-08	HHOUMSI		
Permittee shall be responsible for notifying his contractor and all subcontractors of the provisions of this permit. No work will be started until a copy of this permit is given to the contractor and each of his subcontractors. Further, the copy will be left at the site of the work being done by each contractor.(G2)				
GENERAL PROVISION NO.3	22-DEC-08	HHOUMSI		
Permittee is notified that in accordance with the STATE OF CALIFORNIA CONSTRUCTION SAFETY ORDERS, Section 1503, the permittee or his contractor may be required to acquire a permit from CAL/OSHA if the work authorized herein more than 5 feet deep. The inspection provided by the District can in no way be construed as a safety inspection.(G3)				
GENERAL PROVISION NO. 4	22-DEC-08	HHOUMSI		
Unless otherwise indicated in this permit, all work authorized by this permit shall conform to the latest edition of the Standard Specifications for Public Work Construction, as amended, and published by Building News, Inc., 3055 Overland Avenue, Los Angeles, CA 90034 and the latest edition of the Los Angeles County Department of Public Works "Additions and Amendments to the Standard Specifications for Public Works Construction".(G4)				
GENERAL PROVISION NO.5	22-DEC-08	HHOUMSI		
This permit is subject to such further conditions as the Director or his representative may issue during the period of this use. When possible, such additional conditions shall be promptly delivered in writing to the address shown on page one of this permit. Conditions delivered orally of necessity shall be promptly confirmed in writing.(G5)				
GENERAL PROVISION NO.8	22-DEC-08	HHOUMSI		
Issuance of this permit shall not be construed as an obligation on the part of this District for the operation and maintenance of the proposed facilities.(G8)				
GNERAL PROVISION NO. 13	22-DEC-08	HHOUMSI		
The District reserves the right to order the removal of all equipment if District's activities so require. The District assumes no responsibility for any loss to permittee's equipment or personnel's.(G13)				
GENERAL PROVISION NO. 14	22-DEC-08	HHOUMSI		
Upon completion of work authorized under this permit, permittee shall restore the area to the satisfaction of the District's representative.(G14)				
GENERAL PROVISION NO.16	22-DEC-08	HHOUMSI		
Permittee shall take the necessary precautionary measures to prevent dust or other nuisances which might be created by reason of his activities.(G16)				
GENERAL PROVISION NO. 17	22-DEC-08	HHOUMSI		
Permittee shall keep District right of way clear of obstructions for through access at all times and shall not interfere with the activities of the District's employees or the District's contractors.(G17)				
GENERAL PROVISION NO. 18	22-DEC-08	HHOUMSI		
Permittee shall not use District's right of way for the temporary or permanent storage of excavated materials, rock, sand, cement, or other material or any equipment, except as specifically noted.(G18)				
GENERAL PROVISION NO. 19	22-DEC-08	HHOUMSI		
This permit shall not be construed as a permanent right for these operations.(G19)				
GENERAL PROVISION NO. 24	22-DEC-08	HHOUMSI		
During the period of operations conducted under the permit. Permittee shall maintain in effect an insurance policy (minimum limit $ONE million) naming the Los Angeles County Flood Control DistrictLos Angeles County Department of Public Works and/or U.S. Army Corps of Engineers as co-insured with respect to these operations. A copy of this policy shall be submitted to the District for inclusion in the District file copy of this permit. Expiration or cancellation of the insurance policy shall constitute revocation of this permit.(G24)				
GENERAL PROVISION NO. 47	22-DEC-08	HHOUMSI		
Permittee shall be prepared to remove all material or equipment upon short notice when required for operation and maintenance. Permittee's use of heavy equipment within the right of way is specifically prohibited.(G47)				
PROVISION OVERBUILT NO. 01	22-DEC-08	HHOUMSI		
The inspection fee deposited with the District is the estimated cost to inspect the work authorized under this permit. Should the actual cost be more than the amount deposited, permittee shall submit the difference to the District upon receipt of a written request. In no case will the fee for the actual cost inspection be less than $600. Actual cost will include cost to the District for inspector's time, if required; interim and/or actual cost inspection; and the connection fees to District's facilities, where applicable. (O1)				

Conditions of Approval
By Permit

Run Date: Tuesday December 23, 2008 7:4

Permit: PCFL - T200701397

The following Conditions of Approval are required to complete the permit:

Condition of Approval	Entered	By	Completed	By
PROVISION OVERBUILT NO. 02	22-DEC-08	HHOUMSI		
Permittee shall submit in writing the name and telephone number of individual(s) authorized to request interim andlor inspections. Should permittee fail to provide name. it is understood that permittee's contractor has the authority to request inspections. Cost for said inspections will be taken from the amount deposited for actual cost inspection as set forth in the paragraph above.(O2)				
PROVISION OVERBUILT NO. 03	22-DEC-08	HHOUMSI		
Issuance of this permit shall not be construed as an obligation on the part of this District for the operation and maintenance of the proposed facility.(O3)				
PROVISION POLUTION NO. 02	22-DEC-08	HHOUMSI		
Permittee shall be responsible for the selection and implementation of Best Management Practices (BMP's) for construction activities. If the Director or authorized representative determines that additional BMP's or corrective steps for existing ones are necessary. permittee shall immediately comply with the requests. (P2)				
PROVISION WORK IN CHANNEL NO.3	22-DEC-08	HHOUMSI		
Plans and calculations of any falsework or cofferdam to be placed within the channel waterway area must be submitted to this District for review and approval at least 30 days prior to installation.(W3)				
PROVISION WORK IN CHANNEL NO.7	22-DEC-08	HHOUMSI		
Permittee shall not allow any primary fallout of sandblasting material or paint residue to fall onto the flowing water under any conditions. Any material accidentally deposited in this area shall be removed immediately by the permittee prior to any further painting or sandblasting operations.(W7)				

COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS

Date: 12/23/2008
Permit No: PCFL T200701397

STANDARD FLOOD CONTROL PERMIT PROVISIONS

A. This permit is valid only for the purpose specified herein. No change of purpose as outlined in application or drawings submitted with application is permitted except upon written permission of the Chief Engineer or his representative.

B. Activities and uses authorized under this permit are subject to any instructions of the Chief Engineer or his representative. **ALL INSTRUCTIONS MUST BE STRICTLY OBSERVED.**

C. Permittee shall assume entire responsibility for all activities and uses under this permit and shall save the District and Los Angeles County free and harmless from any and all expense, cost, or liability in connection with or resulting from the exercise of this permit including, but not limited to, property damage, personal injury, and wrongful death.

D. Any damage caused to Flood Control structures by reason of exercise of this permit shall be repaired, at the permittee's sole expense, to the satisfaction of the District. Should the permittee neglect to promptly make repairs, the District may perform such work or have others perform the work, and the permittee agrees to reimburse the District for all costs of the work so performed upon receipt of a statement thereof.

E. Any structure or portions thereof or plantings placed on District rights of way or which affect District structures must be removed, revised, and/or relocated by permittee without cost to the District, or any other public agency the District shall so designate, should future activities or policy so require.

F. This permit is valid only to the extent of District jurisdiction. Acquisition of permits required by other affected agencies and consent of underlying fee owner(s) of District easement lands are the responsibility of the permittee. **NOTHING CONTAINED IN THIS PERMIT SHALL BE CONSTRUED AS A RELINQUISHMENT OF ANY RIGHTS NOW HELD BY THE DISTRICT.**

G. This permit is subject to all prior unexpired permits, agreements, easements, privileges, or other rights, whether recorded or unrecorded, in the area specified by this permit. Permittee shall make his own arrangements with holders of such prior rights.

H. Unless otherwise specified herein, this permit may be revoked or canceled at any time by the Chief Engineer or his representative when required for District purposes.

I. Upon written notice of cancellation or revocation of this permit for any cause whatsoever, permittee shall restore District right of way and structures to their condition prior to the issuance of the permit and then shall vacate District property. Should permittee neglect to restore the premises or structures to a condition satisfactory to the Chief Engineer or his representative, the District may perform such work or have others perform the work, and the permittee agrees to reimburse the District for all costs of the work so performed upon receipt of a statement thereof.

J. In the event of a District employee work stoppage, the Chief Engineer or his representative reserves the right to suspend all activity authorized under this permit which requires inspection by the District. Activity authorized by the permit shall not resume until District approval to do so is given.

K. Unless otherwise specifically provided, all costs incurred by permittee as a result of the conditions of the permit or exercise by District of any right, authority, or reservation contained therein shall be the sole responsibility of and shall be borne entirely by the permittee.

State of California - The Resources Agency

ARNOLD SCHWARZENEGGER, *Governor*

DEPARTMENT OF FISH AND GAME
South Coast Region
Jamie Jackson
P.O. Box 92890
Pasadena, CA 91109
(626) 296-3430

November 09, 2007

Ms. Bridget Cameron
100 South Main Street MS-16A
Los Angeles, California 90012

Streambed Alteration Agreement Number # 1600-2007-0379-R5
I-5 High Occupancy Vehicle Lane Project EA: 421001

Dear Bridget Cameron:

On October 12, 2007 the Department of Fish and Game received your Notification of Lake or Streambed Alteration. On November 09, 2007 the Department determined that your notification is complete. By law, the Department is required to submit a draft Lake or Streambed Alteration Agreement to you within 60 calendar days from the date the notification is complete, if the Department determines that an agreement is required for the project. Hence, the Department has until January 13, 2008 to issue you a draft agreement or inform you that an agreement is not required.

As explained in the notification package you received, the Department must comply with the California Environmental Quality Act ("CEQA") (Public Resources Code section 21000 *et seq.*) before it may issue a final agreement. The Department will issue a final agreement after it receives from you the signed draft agreement. If the project described in your notification is not exempt from CEQA, the lead agency must prepare an environmental document for the project. If you represent a public agency, that agency is the lead agency for the project.

If the Department does not issue you a draft agreement or inform you that an agreement is not required by January 13, 2008 you may complete the project without an agreement. If that occurs, however, the project must be the same one and conducted in the same manner as described in the notification, which would include implementing all measures to protect fish and wildlife resources identified in the notification. (Fish and Game Code section 1602(a)(4)(D).) If your project differs from the one described in the notification, you may be in violation of Fish and Game Code section 1602. Also, even though you would be entitled to complete the project without an agreement, you would still be responsible for complying with all other applicable local, state, and federal laws, including, for example, the state and federal Endangered Species Acts and Fish and Game Code sections 5650 (water pollution) and 5901 (fish passage).

If you have any questions regarding this matter, please contact me at (626) 296-3430.

Sincerely,

Jamie Jackson
Environmental Scientist

Flex your POWER

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 532711
LOS ANGELES, CALIFORNIA 90053-2325

October 1, 2008

REPLY TO
ATTENTION OF:

Office of the Chief
Regulatory Division

DEPARTMENT OF THE ARMY NATIONWIDE PERMIT AUTHORIZATION

Paul Caron
California Department of Transportation
100 South Main Street
Los Angeles, CA 90012

Dear Mr. Caron:

I am writing in reply to your application (File No. SPL-2008-00031-VEN) dated September 27, 2007, for a Department of the Army Permit to discharge fill and temporarily impact 0.07 acre of non-wetland waters of the U.S., in association with the widening of Tujunga Wash Bridge (No. 53-1121R and 53-1121L) for the Route 5 High Occupancy Vehicle (HOV) Lane Project EA: 121901. The proposed work would take place in the city of Los Angeles, Los Angeles County, California.

Based on the information you have provided, the Corps of Engineers has determined that your proposed activity complies with the enclosed terms and conditions of Nationwide Permit No. 14, as described in enclosure 1.

Specifically, you are authorized to use temporary falsework constructed of wooden or metal beams within the Tujunga Wash concrete lined channel, as shown on the attached drawings.

Furthermore, you must comply with the following non-discretionary Special Conditions:

Special Conditions:

1. No debris, soil, silt, sand, rubbish, cement or concrete washings thereof, oil or petroleum products or washings thereof, shall be allowed to enter into or placed where

it may be washed by rainfall or runoff into the waterway. When project operations are completed, any and all excess construction materials, debris, and or other associated excess project materials shall be removed to an appropriate off-site location outside of any waters of the U.S. At no time shall this material be sidecast into the waterway.

2. Staging, storage, fueling, and maintenance of equipment and materials shall be located outside of waters of the U.S.

This letter of verification is valid through October 1, 2010. All nationwide permits expire on March 18, 2012. It is incumbent upon you to remain informed of changes to the nationwide permits. If the Corps of Engineers modifies, reissues, or revokes any nationwide permit at an earlier date, we will issue a public notice announcing the changes.

A nationwide permit does not grant any property rights or exclusive privileges. Also, it does not authorize any injury to the property or rights of others or authorize interference with any existing or proposed Federal project. Furthermore, it does not obviate the need to obtain other Federal, state, or local authorizations required by law.

Attached to this verification that your project is authorized by Nation Wide Permit No. 14 is an approved jurisdictional determination (JD). If you are not in agreement with that approved JD, you can make an administrative appeal under 33 CFR 331.

Thank you for participating in our regulatory program. If you have any questions, please contact Valisa Nez of my staff at 213.452.3419 or via e-mail at Valisa.E.Nez@usace.army.mil.

Please be advised that you can now comment on your experience with Regulatory Division by accessing the Corps web-based customer survey form at: http://per2.nwp.usace.army.mil/survey.html.

Sincerely,

Stephanie J. Hall

Stephanie J. Hall
Senior Project Manger
Regulatory Division

Enclosure

LOS ANGELES DISTRICT
U.S. ARMY CORPS OF ENGINEERS

CERTIFICATION OF COMPLIANCE WITH
DEPARTMENT OF THE ARMY NATIONWIDE PERMIT

Permit Number: *SPL-2008-00031-VEN*

Name of Permittee: Paul Caron, California Department of Transportation

Date of Issuance: *October 1, 2008*

Upon completion of the activity authorized by this permit and any mitigation required by the permit, sign this certification and return it to the following address:

U.S Army Corps of Engineers
Regulatory Division
ATTN: CESPL-RG-SPL-2008-00031-VEN
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P.O. BOX 532711
LOS ANGELES, CALIFORNIA 90053-2325

Please note that your permitted activity is subject to a compliance inspection by an Army Corps of Engineers representative. If you fail to comply with this nationwide permit you may be subject to permit suspension, modification, or revocation procedures as contained in 33 CFR 330.5 or enforcement procedures such as those contained in 33 CFR 326.4 and 326.5.

I hereby certify that the work authorized by the above referenced permit has been completed in accordance with the terms and conditions of the said permit, and required mitigation was completed in accordance with the permit condition(s).

_____ _____

Signature of Permittee Date

FALSEWORK

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET NO	TOTAL SHEETS
07	LA	5/170			

REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER MARK J. OKIMURA 62908 Exp. 6-30-06 CIVIL STATE OF CALIFORNIA

The State of California or its officers or agents shall not be responsible for the accuracy or completeness of electronic copies of this plan sheet.

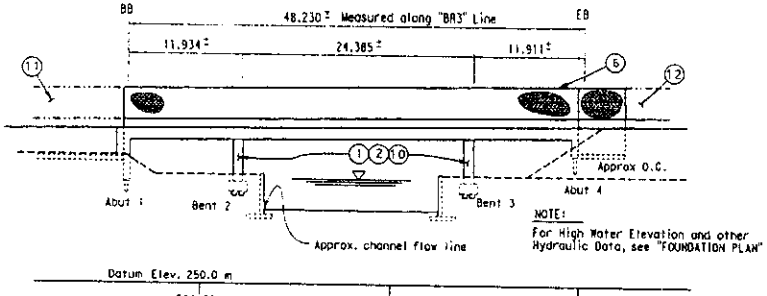
Caltrans web site: http://www.dot.ca.gov

NOTE:

For 'Index to Plans', 'Standard Plans', 'General Notes', 'Pile Data Table', and 'Camber Diagram', see "INDEX TO PLANS" sheet

Legend:

- Indicates Existing Structure
- Indicates Concrete Removal
- Indicates Closure Pour

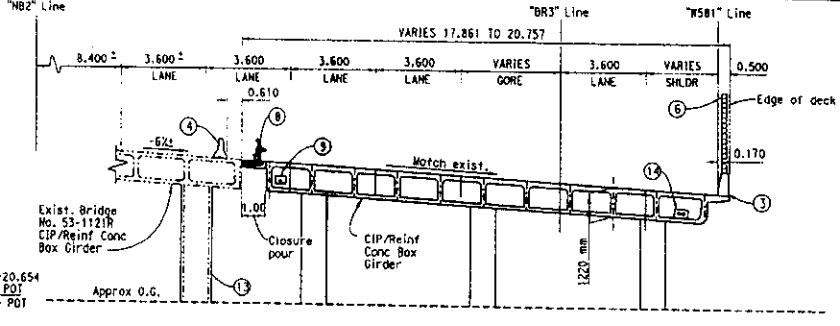


NOTE:

For High Water Elevation and other Hydraulic Data, see "FOUNDATION PLAN"

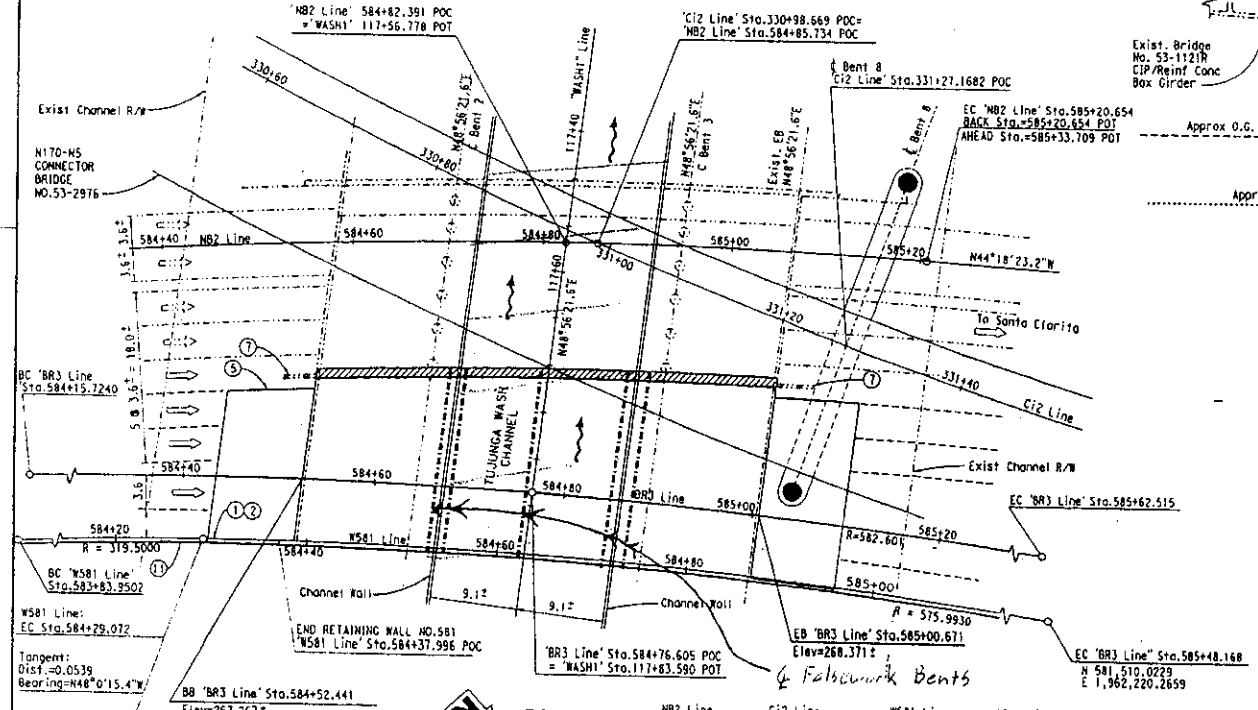
ELEVATION

1:250



TYPICAL SECTION

Scale = 1:100



PLAN

1:250

NB2 Line	Ci2 Line	W581 Line	W581 Line	BR3 Line
R = 609.6000	R = 500.0000	R = 319.5000	R = 575.9930	R = 582.6010
D = 33°38'33.2"	D = 64°48'14.2"	D = 8°5'29.9"	D = 11°50'29.2"	D = 14°26'10.1"
T = 184.2961	T = 317.3337	T = 22.5984	T = 59.7337	T = 73.7863
L = 357.9408	L = 565.5210	L = 45.1217	L = 119.0410	L = 146.7910

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

Notes:

1. Point "TUJUNGA CHANNEL BRIDGE"
2. Point "53-1121R"
3. Concrete Barrier Type 736 (Modified)
4. Temporary Barrier Railing Type "K", see "ROAD PLANS"
5. Structure Approach Type N(9D)
6. Masonry Block Soundwall on Concrete Barrier Type 736 (Mod.), h=4.27 m
7. Remove Exist. Wingwalls to 1.0 m below finished ground level
8. Remove existing Concrete Barrier Type 1 and portion of existing Overhang. Existing Transverse Deck Reinforcement to be undamaged. Salvage Metal Railing Posts, Tubing, and Hardware. Existing 75 mm pipe strapped to outside of existing East side barrier rail to be relocated inside new box girder.
9. Utility Opening, 450 mm x 300 mm, to accommodate relocated 75 mm pipe
10. Point Bent Number
11. Soundwall on Retaining Wall No. 581, see "RETAINING WALL NO. 581" details
12. Soundwall No. 581 on Trench Footing
13. Column casing at existing Bents 2 & 3 inner left, inner right, and outer right columns.
14. Utility opening, 450 mm x 300 mm to accommodate 2-103 mm ø conduits

Caltrans Metric

DESIGN ENGINEER

DESIGN	A. NEDWICK	CHECKED	M. OKIMURA
DETAILS	LC/DWR	CHECKED	D.TEXLER
QUANTITIES		CHECKED	M. OKIMURA

LOAD FACTOR DESIGN — LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD — LAYOUT — SPECIFICATIONS — CHECKED M. OKIMURA — PLANS AND SPECS COORDINATOR

ALL DIMENSIONS ARE IN METERS UNLESS OTHERWISE SHOWN

ORIGINAL SCALE IN MILLIMETERS FOR MEASURED PLANS

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

DIVISION OF ENGINEERING SERVICES
STRUCTURE DESIGN
DESIGN BRANCH 11

CU 07274
EA 121901

BRIDGE NO. 53-1121R
58.49

TUJUNGA WASH CHANNEL RIGHT BRIDGE (WIDEN)

GENERAL PLAN

DESIGNED PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 1 OF

FILE => 30100_dwr.dgn

USERNAME => mkimura DATE PLOTTED => 13-SEP-2007 TIME PLOTTED => 11:44

"Nez, Valisa E SPL" <Valisa.E.Nez@usace.army.mil>
09/22/2008 11:29 AM

To "Eric Hanson" <eric_hanson@dot.ca.gov>
cc
bcc
Subject Corps File: 2008-00031-VEN

History: This message has been forwarded.

Eric,

I am writing in regards to the Nationwide Permit application for the I-5 HOV Lane Project EA: 121901 (File: 2008-00031-VEN) for the widening of the Tujunga Wash bridge. The Corps has contacted the RWQCB regarding the status and has received no response, since the RWQCB has had the application for greater than 60 days and provided no response to the applicant or the Corps, the Corps would waive the 401 certification. However, we are not issuing nationwide permits until the end of the fiscal year, therefore I am able to issue the permit on October 1st or shortly thereafter. Thanks,

Valisa E. Nez
Project Manager
U.S. Army Corps of Engineers
Regulatory Division, LA District
Phone: (213) 452 – 3419
Fax: (213) 452 - 4196

Memorandum

To: Ramin Rashedi, Chief
Bridge Design Branch 11
Att: Mark Okimura

Date: January 20, 2005

File: 07-LA-5, 170-KP58.5
EA 07-121901
Tujunga Wash Channel
Bridge No. 53-1121L/R

From: **Department of Transportation**
Hydraulic Engineering Branch

Subject: Final Hydraulic Report for the Tujunga Wash Channel Bridge.

Attached for your records is the Final Hydraulic Report for the above referenced project. If you have any questions, please contact me at 227-0442.

Sincerely,

Neal Alie, P.E.
Hydrology/Hydraulics Engineer

cc: Steve Jaques

DIVISION OF STRUCTURES
Final Hydraulic Report

Tujunga Wash Channel Bridge

Located in Los Angeles County

JOB:
Bridge No. 53-1121L/R Bridge Widening

LOCATION:
07-LA-5-KP 58.5

DATE:
January 20, 2005

WRITTEN BY:
Neal Alie

REVIEWED BY:
Steve Jaques

General

The Office of Structure Design is proposing to widen the existing left and right structures of Tujunga Wash Channel on Interstate 5 in Los Angeles County. The proposed widening of approximately 7.0 meters on the left and 6.0 meters on the right of 53-1121L will be a CIP/PS Concrete Box Girder with a structural section of 1.067 meters. The proposed widening of approximately 21.0 meters on 53-1121R will be a CIP/RC Box Girder with a structural section of 1.220 meters.

The existing left structure (53-1121L), built in 1963, is a simple span RC Box Girder (7 cell) structure with RC open end diaphragm abutments supported on concrete piles. This bridge has a total length of 24.4 meters and a width of 17.3 meters.

The existing right structure (53-1121R), built in 1963 and widened in 1987, is three span continuous RC Box Girder (8 cell) structure with RC four column piers and RC open end diaphragm abutments all supported on concrete piles. This structure has a total length of 48.2 meters and a width of 21.1 meters.

This report makes reference to data and analysis found in (1) General plans and profiles submitted by the Office of Structure Design, (2) Structure Maintenance Records and (3) As-Built Plans dated June, 1959, (4) Information received from the Army Corp. of Engineers.

All elevations indicated in this report are referenced to the (As Built Plans dated June, 1959) datum.

Drainage Basin

The proposed Tujunga Wash project is located approximately 3.5 kilometers downstream from the Hansen Dam Spillway. The Tujunga Wash Channel is part of an extensive flood control system built to protect the metropolitan area. The Tujunga Wash Channel drains a watershed of approximately 391 square kilometers. The channel flows west of the San Gabriel Mountains and turns south along the west of the Verdugo Mountains and merges into the Los Angeles River from the north near the intersection of Colfax Ave. and Ventura Blvd.

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. The area is characterized by high peak flows and short durations due to the highly developed areas.

In the project reach, Tujunga Wash is a concrete lined trapezoidal channel with a width of 18.3 meters and vertical side slopes ranging from 2.4 to 3.4 meters high, with a channel slope of 0.009.

Discharge

According to Mr. Van Crisostomo with the US Army Corp. of Engineers, Los Angeles District, and using the " Discharge Frequency Curves for Hansen Dam" he provided the 50-year discharge (424.75 cubic meters per second) and the 100-year discharge (538.02 cubic meters per second). The 500-year discharge is 707.92 cubic meters per second.

Stage and Velocity

The Hydraulic Program (Brease) was used to perform a one-dimensional hydraulic analysis to calculate the water surface elevation and velocity for the left and right structures of the Tujunga Wash Channel.

The average velocity and the stage for the 50-year and 100-year discharges at the upstream face of the structures are given below. The results are based on a roughness coefficient of 0.014 and an average gradient of 0.009.

	53-1121L	**53-1121R**
50-Year Discharge	424.75 cms	424.75 cms
Velocity (50-year)	10.09 mps	10.25 mps
Water Surface Elev.	260.51 m	261.38 m
Available Freeboard to top of Channel	**1.08 m**	**0.92 m**
100-year Discharge	538.02 cms	538.02 cms
Velocity (100-year)	10.95 mps	11.12 mps
Water Surface Elev.	260.89 m	261.75
Available Freeboard to top of Channel	**0.70 m**	**0.56 m**

There is adequate freeboard for both the 50-year and 100-year storm events for the left and right structures of Tujunga Wash Channel.

Required Waterway

The required waterway of 49.13 square meters for the left structure and 48.38 square meters for the right structure, below the 100-year water surface elevation is adequate for the flow.

Streambed and Scour

In the project reach, Tujunga Wash is a concrete lined trapezoidal channel with a width of 18.3 meters and vertical side slopes ranging from 2.4 to 3.4 meters high, with a channel slope of 0.009. Structure Hydraulics has no scour concerns.

In July 2000 both the left and right bridge's scour potential was assessed in accordance with FHWA Technical Advisory T5140.23, "Evaluating Scour at Bridges", and within current Caltrans guidelines. The NBI item 113, " Vulnerability to Scour", was changed, from 6 to 8, concrete lined channel.

Drift

According to the Bridge Maintenance Records there are no indications of drift problems. However, a moderate amount of drift can be expected during a flood event.

Bank Protection

Bank protection will not be needed for the Tujunga Wash Channel because it is a concrete lined channel.

HYDROLOGIC SUMMARY FOR TUJUNGA WASH CHANNEL (LEFT)

HYDROLOGIC SUMMARY			
Drainage Area: 391.1 sq km			
	Design Flood	**Base Flood**	**Overtopping Flood**
Frequency	50-yr	100-yr	500-yr
Discharge	424.75 cms	538.02 cms	707.92 cms
Water Surface Elevation at Bridge	260.51 m	260.89 m	261.41 m
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.			

HYDROLOGIC SUMMARY FOR TUJUNGA WASH CHANNE, (RIGHT)

HYDROLOGIC SUMMARY			
Drainage Area: 391.1 sq km			
	Design Flood	**Base Flood**	**Overtopping Flood**
Frequency	50-yr	100-yr	500-yr
Discharge	424.75 cms	538.02 cms	707.92 cms
Water Surface Elevation at Bridge	261.38 m	261.75 m	262.43 m
Flood plain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.			

This report has been prepared under my direction as the professional engineer in responsible charge of the work, in accordance with the provisions of the professional Engineers Act of the State of California.

REGISTERED CIVIL ENGINEER (SIGNATURE) Neal Ali

REGISTRATION NUMBER: C056398 DATE: 1/20/05

REGISTERED PROFESSIONAL ENGINEER
NEAL ALI
No. C056398
Exp. 6/05
CIVIL
STATE OF CALIFORNIA

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

N170-N5 Conn.
Structure Name

07 - LA - 05/170 - 58.0-63.4
District County Route Post Km

07-121901 53-2976
E.A. Number Structure Number

Foundation Report By: S. Sukiasian
Dated: 8/7/06; 1/23/08

Reviewed By: M. Okimura (OSD) R. Price (GS)

General Plan Dated: 8/11/07
Foundation Plan Dated: 10/12/07 Var.

☐ No changes. ☒ The following changes are necessary.

Abutment Pile Data is missing from Plans.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☒ Yes ☐ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: January 23, 2008

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-2976
(5-170 Mixed Flow Connector)

Attention: Mr. Mark Okimura

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

Subject: Addendum (No. 1) Foundation Report for New Mixed Flow 5-170 Connector Bridge (Bridge No. 53-2976).

Per your e-mail request dated December 10, 2007, we are providing an Addendum Memorandum for the 5-170 Northbound (Mixed Flow) Connector Bridge Foundation Report dated August 7, 2006. This Addendum provides in Memorandum form the recommendations given in the e-mail response from our Office to Structure Design, dated December 24, 2007. This memorandum, like the e-mail response, summarizes changes made to the foundation data for the Mixed Flow 5-170 Bridge Connector Project.

Based on the latest pile data provided by the Bridge Design Branch 11 on December 10, 2007, the new foundation data is summarized below.

- In regard to Section 8.1 (Pile Recommendations) page 8 of the August 7, 2006 Foundation Report, pile capacities remain based on the inclusion steel casings of 15 meter (50 foot) lengths for all new bridge bents except Bents 2 and 11. The piles capacities for Bents 2 and 11 are based on steel casing lengths of 15.64 and 16.42 meters long, respectively. The longer casing lengths were used in the pile calculations for the purpose of covering the construction joint, which will be located at this depth.

- Based on the revised top of pile elevations at Abutments 1 and 12, the corresponding pile tip elevations have been revised and are shown in the Table below. Please note that a 15.2-meter long pile length is maintained.

Abutment	New Top of Pile Elevation, m	New Pile Tip Elevation, m
1	265.42	250.22
12	261.62	246.42

- Please note that pre-drilling or jetting is not allowed at the abutment locations

A separate Addendum Memorandum regarding changes for the HOV Bridge Connector (Bridge No. 53-2977) is also being prepared.

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 1/23/08

SAM SUKIASIAN, G.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
Branch B

REGISTERED PROFESSIONAL ENGINEER
SAMUEL G. SUKIASIAN
No. GE2681
Exp. 6-30-08
GEOTECHNICAL
STATE OF CALIFORNIA

cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Office of Bridge Design South 1 Branch 11

Date: August 7, 2006

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-2976
(New 5/170 Connector)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for 5/170 New Northbound Connector Bridge (Bridge No. 53-2976).

This Memorandum provides foundation recommendations for the new 5/170 Northbound Connector Bridge in Los Angeles County (Bridge No. 53-2976). The recommendations in this memo are based on borings drilled for this project. Originally, the borings were drilled at or near fifteen proposed bent and abutment locations (based on the January 31, 2005 Plans) by our office or drilled by a consultant under our supervision. On February 14, 2006, the Office of Bridge Design South 1 submitted new abutment and bent locations based on longer spans. The foundation recommendations in this report are based on the latest abutment and bent locations and their associated structural loads. The Office of Bridge Design South 1 provided the latest plans dated February 14, 2006.

1.0 PROJECT DESCRIPTION

1.1 Proposed Structure

According to the 5/170 Northbound Connector latest layout data dated February 14, 2006, the bridge will be a Cast-in-Place/Prestressed (CIP/PS) Box Girder eleven span structure with open-end seated abutments. The single column bents and the outrigger bent structure at bent designation 7 will be supported on Cast-in-drilled-hole (CIDH) piles. The abutments will be founded on steel H-piles. According to the latest information provided by Structure Design, the connector will be 717.766 meters (2,354.9 feet) long and 17.70 meters (58.07 feet) wide. The design loads nominal loads, pile diameters/widths and recommended pile tip elevations are shown in Section 8.2, Table 4 of this report.

2.0 FIELD EXPLORATION PROGRAM

Eleven borings were drilled for this investigation. The borings were originally drilled at or very near eleven of the original 15 bent and abutment locations. With the latest change of the abutment and bent locations, four of the eleven drilled borings are now located between the new proposed abutment and bent locations. Due to inaccessibility, 3 bent locations were not drilled. These bent locations, located in present residential areas adjacent to the existing I-5 Freeway, should be drilled as soon as the area is cleared for construction of the proposed connector. The eleven borings drilled for this report were completed between June 7 and November 15, 2005. The borings were drilled to a depth of 35 to 61 meters. The borings were advanced utilizing the mud rotary and diamond coring methods with CME-85, 95 and CS-2000 drill rigs. The location of the borings is summarized in the Table No. 1 below. The boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's are presently being prepared by the Office of Geotechnical Support and will be submitted to your Office.

Table No. 1 – Summary of Boring Locations

Boring	Location	Station [1]	Offset, m	Elevation[2], m
76-B1	Near Abutment 1	583+15.57	291.44L	265.90
76-B2	Between Abutment 1 & Bent 2	583+29.85	241.83L	267.34
76-B3	Between Bent 2 & Bent 3	583+44.07	185.81L	268.44
76-B4	Between Bent 3 & Bent 4	583+49.90	123.33L	261.02
76-B5	Near Bent 4	583+73.56	73.74L	261.60
76-B6	Bent 5	584+05.79	32.49L	262.35
76-B7	Bent 6	584+66.42	5.58R	282.88
76-B7L	Outrigger Bent 7L	585+26.17	25.18R	269.26
76-B7R	Outrigger Bent 7R	585+11.64	60.07R	264.09
76-B9	Bent 8	TBD	TBD	TBD
76-B10	Bent 9	TBD	TBD	TBD
76-B11	Bent 10	TBD	TBD	TBD
76-B12	Bent 11	588+06.35	35.91R	264.84
76-B13	Abutment 12	588+49.10	38.61R	265.18

Note: 1. Stationing according to I-5 Centerline.
2. Elevations are Above Mean Sea Level (MSL).
3. TBD: To be drilled at a later date.

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on January 19, 2006.

Soil samples in the top 15 to 18 meters (50 to 60 feet) of the borings were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). Below 18 meters, where gravels and cobbles were typically encountered, the rig was outfitted with a 94 mm HQ-Wireline core barrel with an impregnated diamond bit. Core samples were typically retrieved by advancing the core barrel 1.5 meters at a time. Core sampling was performed continuously from the depth of drive

sampler refusal to the completion of the boring. At the completion of holes certain borings were fitted with slotted 1.5-inch PVC pipe casing with sand backfill in order to accurately measure groundwater levels (see Section 4.3.1 for groundwater information).

3.0 LABORATORY TESTING

Corrosivity testing was conducted by the geotechnical consultant, under Caltrans direction, on various selected SPT and bulk samples from the borings and summarized in their Final Geotechnical Data Report, (URS Corp. dated December 15, 2005). The corrosivity test results from that report is summarized in this memo under Section 7.0. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643. The Table below summarizes the Corrosivity Test and CTM designation.

Table No. 2 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

4.0 GEOLOGY

4.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 5.0, Seismicity, of this report.

4.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. The Tujunga Wash crosses under the proposed bridge structure and it has been confined in a concrete lined open top rectangular channel. To the southeast and southwest of the proposed bridge the County of Los Angeles maintains spreading grounds on the order of several acres in size that are used to re-charge groundwater. The southbound 5 Freeway is near pre-freeway grade and the northbound 5

Freeway is in fill section in the vicinity of the proposed bridge. Fill ranges in thickness up to approximately 10 meters. 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 1.7 kilometers north of the proposed project (Please see also Section 5.0, Seismicity).

4.3 Subsurface Conditions

Subsurface conditions at the proposed abutment and bent locations were determined based on the eleven borings drilled for this project. In general, the top 9 meters of alluvium (about elevation +265 to +256) was composed of medium dense to dense sands with silt and fine to coarse gravel. This upper layer was underlain by 6 meters (elevation +256 to +250 meters) of dense sands with coarser gravels with few cobbles and scattered seams of stiff clayey silt. Below about +250 meters elevation, typically very dense sands with abundant coarse gravels and cobbles were encountered. Scattered boulders should also be expected below the +250 meter elevation level. Embankment fills were typically composed of dense sands with fine to coarse gravels.

Log of Test Borings (LOTB's) graphically show the results of the soil borings drilled for this project. The LOTB's will be provided at a later date.

4.3.1 Groundwater

Groundwater was not encountered during this investigation to the maximum depth explored (elevation +203.3). Groundwater observation wells were installed during June to November 2005 in boreholes 76-B4, 76-B5R 76-B6 and 76-B14. An additional deeper well, boring 77-B4, was installed for the adjacent proposed bridge 53-2977 during the same period. This well was installed approximately 64 meters southwest of boring 76-B7 for the proposed bridge 53-2976. An attempt to measure the groundwater elevation in these five well locations was undertaken on February 10, 2006. Wells 76-B4, 76-B5R and 76-B14 were measured and found to have no groundwater to the maximum depth of the casings of elevation +218.9 meters, +218.0 meters and +229.1 meters respectively. Well 76-B6 was left as an open hole with no casing for monitoring purposes and was found to have experienced caving and was blocked at an elevation of +253.5 meters and no groundwater was measured. Well 77-B-4 was measured and found to have no groundwater to the maximum depth of the casing of elevation +203.3 meters. Soil borings from Log of Test Borings drilled within the general area of the proposed HOV Connector Bridge in 1961 did not encounter groundwater to an elevation of +243 meters (this was the level of the deepest penetration of those borings found).

Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from +160 to +175 meters (100 to 115 meters depth below ground surface). The area nearby the proposed new connectors contains surface basins for recharge of surface waters to the groundwater. Historically in the past several decades during times of recharge operations water elevations nearby the proposed project have been measured at an elevation of approximately +244 meters (40 meters depth below ground surface). For this project the highest anticipated groundwater should be estimated at elevation +244 meters.

5.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated April 30, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table below summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 3 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.7	NE	0.7g
Northridge Hills, SSN	Reverse-Oblique	7.5	5.4	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.4	NE	0.6g

5.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 seconds. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

5.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 5.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 100 feet.

6.0 SCOUR EVALUATION

The Tujunga Wash passes under Bridge 53-2976 between proposed bent 6 and outrigger bent 7R and 7L. The wash is a concrete lined rectangular open top channel. The proposed columns will be placed outside the channel and thus will not be affected by scour. Because of this and the absence of a natural channel running underneath the bridge, the potential for scour is considered low to remote.

7.0 CORROSION EVALUATION

A consultant under the supervision of our office performed corrosion testing. The corrosion test results for the 53-2976 Connector Bridge are taken from their Final Geotechnical Data Report, (URS Corp. dated December 15, 2005), and summarized here. As shown in Table No. 4 below, composite soil samples were taken within the top 50 feet of the soil borings at 76-B1 and 76-B7 and were tested for corrosion potential. The test results summarized in Table No. 3 indicate that the soils expected to come in contact with the CIDH pile foundations is non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
76-B1	1.5 – 15.7	Abutment 1	8.7	11,000	ND	60
76-B7	1.5 – 15.7	Bent 7	8.7	11,000	ND	60

Note: For corrosion definitions refer to "Memo to Designers" 3-1.
ND: Note Detected.
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.

8.0 FOUNDATION RECOMMENDATIONS

The proposed abutments and bents for the new 5/170 Northbound Connector Bridge may be supported on driven H-piles for the abutments and Cast-in-Drilled-Holes (CIDH) piles for the bents. Section 8.1 provides detailed recommendations on the CIDH piles and steel H-piles for the abutment and bent locations and pile diameters and/or widths and specified tip elevations.

8.1 Pile Recommendations

3657 and 2740 mm CIDH Diameter piles are proposed for Bents 2 through 11, as shown in the pile data table below. HP360x132 Steel H-piles are proposed for Abutments 1 and 12. Preliminary Pile cutoff elevations and specified Pile Tip Elevations (SPTE) are listed in Table No. 5 below. The Office of Bridge Design Branch 11 provided the latest pile diameters and compression loads on February 14, 2006. The latest Pile cutoff elevations have been provided by Bridge Design and summarized below. In Table 5 below, the ultimate bearing capacity of the piles will equal or exceed the required nominal resistance in compression.

Table No. 5 - Pile Data Table – N170/N5 Connector (New Bridge No. 53-2976)

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
			COMPRESSION (kN)	TENSION			
Abut 1	HP360x132	900		Not provided	+265.9	+250.7	15.2[1]
Bent 2	3657 mm CIDH	NA	50100	"	+257.2	+229.8	27.4[1]
Bent 3	3657 mm CIDH	NA	48400	"	+259.2	+225.7	33.5[1]
Bent 4	3657 mm CIDH	NA	50100	"	+260.8	+224.2	36.6[1]
Bent 5	3657 mm CIDH	NA	52400	"	+263.1	+226.5	36.6
Bent 6	3657 mm CIDH	NA	48100	"	+263.9	+227.4	36.6
Bent 7L	2740 mm CIDH	NA	21300	"	+263.7	+242.4	21.4
Bent 7R	2740 mm CIDH	NA	35600	"	+266.6	+230.0	36.6
Bent 8	3657 mm CIDH	NA	50450	"	+263.7	+230.2	33.5[1]
Bent 9	3657 mm CIDH	NA	49100	"	+261.9	+231.4	30.4[1]
Bent 10	3657 mm CIDH	NA	48450	"	+258.8	+225.3	33.5[1]
Bent 11	3657 mm CIDH	NA	50100	"	+255.7	+222.2	33.5[1]
Abut 12	HP360x132	900		"	+263.0	+247.8	15.2

Note: 1. Pile lengths on Bents 9, 10, & 11 preliminary only. These pile lengths need to be verified by soil boring information at the Bent locations when access to those locations is available.

The pile tip elevations are based on skin friction of axial compressive loads only. However, lateral loading should be considered as well to verify or if necessary, adjust the pile lengths (see Section 8.1.1 for a brief discussion on lateral pile capacities).

Based on observations of the drilled soil boring borehole conditions after completion of drilling, caving within the upper 15 meters is considered likely. Therefore, the drilling contractor should be prepared for borehole caving in the upper 15 meters. A method to stabilize the hole, such as a 15-meter (50-foot) long steel casing, should be utilized. The Pile recommendations provided in Table 5 above are based on subsurface conditions with casing in the drilled hole.

Bents 2, 7L, 10 and 11 are proposed to have isolation casing lengths of 4.6 to 6.4 meters. Therefore, the casing lengths used in calculating the skin friction capacity was the remaining casing below the pile cutoff elevation (4.6 to 6.4 isolation casing length subtracted from the total casing length, 15.2 meters).

Finally, the pile data for Bents 8, 9, 10 and 11, are based on preliminary subsurface information only. Due to access constraints briefly discussed in Section 2.0 of this memo, the borings for the bents mentioned were not drilled. Therefore, our office must be notified at least two weeks in advance of the date when the bents location sites are cleared for construction. Provisions must be made in the construction schedule to allow four weeks for drilling at these bent locations.

Steel HP360x132 piles must be driven at the abutment locations to the pile tip elevations specified in Table 4. Driving H-piles to 15.2 meters below the pile cutoff elevation should be achievable at the abutments. Therefore, pre-drilling for H-piles is not recommended.

8.1.1 Lateral Pile Analysis

P-y Curves based on soil borings drilled for the abutment and bent locations have been provided to the Office of Bridge Design for use in determining anticipated deflections and moment magnitudes of the proposed pile foundations. The modulus of elasticity of concrete is recommended to be based on a unit weight of 23.55 kPa (150 pcf) and a 28-day compressive strength of 25920 kPa. The provided p-y curves were generated utilizing the computer program LPILE ^{plus} (version 4M).

9.0 CONSTRUCTION CONSIDERATIONS

CIDH Piles

1. Although, based on available records the highest anticipated groundwater elevation is estimated at +244 meters, groundwater was not encountered to an elevation of +203 meters during our field investigation. Therefore, groundwater is considered unlikely to be encountered during construction. Nevertheless, the contractor should be prepared for the unlikely event of handling groundwater in the deeper boreholes. The slurry displacement method based on the Wet Method of CIDH construction would be recommended.

2. Concrete cover and mix design requirements for CIDH piles shall be consistent with Section 8.22 of the Bridge Design Specifications and Section 49-4.01 of the July 2002 Standard Specifications.

3. Adjacent bridge columns must be monitored during CIDH installation. Damage may occur to these columns and piles if excessive caving occurs due to CIDH pile drilling.

4. Drilled holes must conform to Section 49-4.03 of the July 2002 Standard Specifications.

5. As discussed in Section 8.1, borehole-caving potential is likely in the upper 15 meters. Drilled CIDH holes should be inspected for caving after completion of drilling. Methods to keep the borehole open, such as casing should be employed. Plans for the chosen method should be submitted to the Resident Engineer for review.

6. As discussed in Section 4.3 of this report, large cobbles and possibly boulders should be expected to be encountered starting at elevation +250 meters MSL. Some cobbles and the occasional boulder may be encountered above this elevation as well. Therefore, the drilling contractor should have the correct drilling equipment to effectively penetrate these materials.

Driven H-Piles

1. The contractor should anticipate the possibility of hard and erratic driving of steel H piles due to the presence of scattered cobbles. The contractor should anticipate field cutting and splicing of all H piles. Refer to the LOTB sheets for details.

2. Hard driving tips shall be required and installed on all driven steel H piles to ensure pile integrity and limit damage to piles during hard driving.

3. Piles should be driven at least to the specified tip elevations. Any pile that achieves refusal within 4 feet (1.2 meters) of the specified pile tip elevations may be accepted at the Resident Engineer's discretion. This procedure should prevent damage to the piles. Refusal for the piles shall be defined as 3.5 times the design load as shown on the contract plans (per FHWA recommendations for the Gates formula, Section 16.3, Publication No. FHWA HI 97-014 Revised November 1998).

4. If the specified tip elevation is reached without achieving the design (service) load, pile driving should continue until bearing is attained. In this case, it may be prudent to allow the pile to "set up" for a minimum period of 24 hours before continuing the driving.

5. The selected pile-driving hammer should be able to deliver sufficient energy to drive the piles at a penetration rate of 3 mm per blow at the required bearing value. Predrilling is not allowed.

6. Noise and vibration from the pile driving operation should be studied prior to construction due to close proximity of adjacent structures lying outside the state right of way.

7. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents.

10.0 REFERENCES

California Department of Transportation, Corrosion Technology Branch, Corrosion Guidelines, updated to July 2002.

California Geologic Survey, Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

Dibblee, T.W., Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, 1991.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

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Mualchin, L., Caltrans California Seismic Hazard Map 1996, California Department of Transportation, 1996.

URS, Corporation, Final Geotechnical Data Report, Interstate 5-State Route 170 New Connector, Los Angeles County, California, For Caltrans (Contract No. 59A0283), December 15, 2005.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date:

Supervised by: Date: 8-7-08

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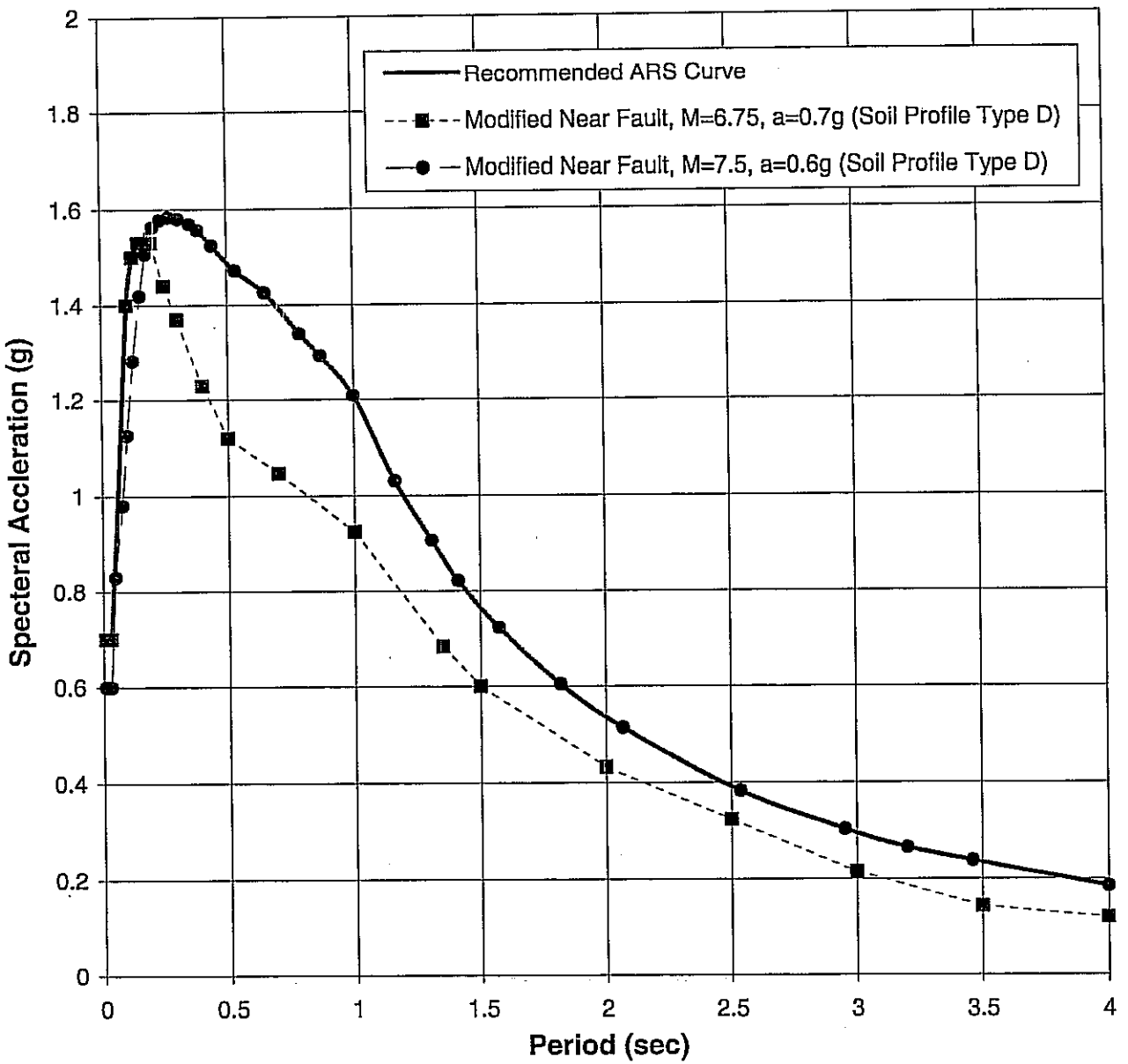
STATE OF CALIFORNIA

cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve r SR-170N/I-5N Connector
Bridge #53-2976
EA-07-121901



FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

5-170 HOV Conn
Structure Name

07 - LA - 05/170 - 5.0 - 13.
District County Route Post Km

07-121901 53-2977
E.A. Number Structure Number

Foundation Report By: S Sukiasian
Dated: 8/7/08 ; 1/23/08

Reviewed By: M Okimura (OSD) R. Price (GS)

General Plan Dated: 1/2/08
Foundation Plan Dated: 4/18/07 - unr.

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☒ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: January 23, 2008

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-2977
(5-170 HOV Connector)

Attention: Mr. Mark Okimura

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

Subject: Addendum (No. 1) Foundation Report for New HOV 5-170 Connector Bridge (Bridge No. 53-2977).

Per your e-mail request dated December 10, 2007, we are providing an Addendum Memorandum for the 5-170 HOV Connector Bridge Foundation Report dated August 7, 2006. This Addendum provides in Memorandum form the recommendations given in the e-mail response from our Office to Structure Design, dated December 24, 2007. This memorandum, like the e-mail response, summarizes changes made to the foundation data for the HOV 5-170 Bridge Connector Project.

Based on the latest pile and casing data provided by the Bridge Design Branch 11 on December 10, 2007, the new foundation data is summarized below.

- Based on revised pile cutoff elevations provided by Structure Design on December 10, 2007, the pile tip elevations provided in Section 8.1 (Pile Recommendations) Table 4, page 7 of the August 7, 2006 Report have been revised (and should be replaced) per the data provided in the table below.

Bent	New Cutoff Elevations, m	New Pile Tip Elevations, m
2	260.0	232.6
3	260.4	236.0
4	260.7	239.3
5L	259.6	235.2
5R	261.2	236.8
6	260.1	232.7
7	258.7	231.3

- Please note that pre-drilling or jetting is not allowed at the abutment locations.

A separate Addendum Memorandum regarding changes for the Mixed Flow Bridge Connector (Bridge No. 53-2976) is also being prepared.

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 1/23/08

REGISTERED PROFESSIONAL ENGINEER
SAMUEL G. SUKIASIAN
No. GE2661
Exp. 6-30-08
GEOTECHNICAL
STATE OF CALIFORNIA

SAM SUKIASIAN, G.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
Branch B

cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Office of Bridge Design South 1 Branch 11

Date: August 7, 2006

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-2977
(New 5/170 HOV Connector)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for 5/170 New HOV Connector Bridge (Bridge No. 53-2977).

This Memorandum provides foundation recommendations for the new 5/170 HOV Connector Bridge in Los Angeles County (Bridge No. 53-2977). The HOV proposed connector will connect the 170 and I-5 Freeways and will be built to the west of the proposed N170 to N5 Connector (Bridge 53-2976). This Office is currently preparing a separate Foundation Report for that connector as well. The recommendations in this memo are based on borings drilled for this project. The borings drilled at or near the bent and abutment locations were drilled by our office or drilled by a consultant under our supervision. The foundation recommendations in this report are also based on structural loads provided by your office on January 31, 2005.

1.0 PROJECT DESCRIPTION

1.1 Proposed Structure

According to the April 30, 2003 Preliminary Foundation Report and the 5/170 HOV Connector General Plans, dated November 22, 2004, the bridge will be a Cast-in-Place/Prestressed (CIP/PS) Box Girder seven span structure with open-end seated abutments. The single column bents and the outrigger bent structure at bent designation 5 will be supported on Cast-in-drilled-hole (CIDH) piles. The abutments will be founded on steel H-piles. According to the plans, the HOV connector will be 251.0 meters (823.5 feet) long and 17.67 meters (58 feet) wide. The design loads, nominal loads, pile diameters/widths and recommended pile tip elevations are shown in Section 8.2, Table 4 of this report.

2.0 FIELD EXPLORATION PROGRAM

Six borings were drilled for this investigation. Each boring was drilled at or near six of the 9 bent and abutment locations. Due to inaccessibility, 3 of the total 9 bent/abutment locations were not drilled. These remaining locations, located in the present I-5 Freeway traveled way, should be drilled as soon as the area is made accessible for construction of the proposed connector. The six borings drilled for this report were completed between May 24 and November 15, 2005. The borings were drilled to a depth of 37 to 51 meters. The borings were advanced utilizing the mud rotary and diamond- coring methods with CME-75, B-47 and CS-2000 drill rigs. The location of the borings is summarized in the Table No. 1 below. The boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's are presently being prepared by the Office of Geotechnical Support and will be submitted to your office.

Table No. 1 – Summary of Boring Locations

Boring	Location	Station [1]	Offset, m	Elevation[2], m
77-B1	Abutment 1	584+04.05	91.82L	270.50
77-B2	Bent 2	584+31.20	76.88L	261.50
77-B3	Bent 3	584+57.39	45.58L	262.12
77-B4	Bent 4	584+78.15	27.37L	263.73
77-B5L	Outrigger Bent 5L	585+26.35	21.44L	264.43
77-B5R	Outrigger Bent 5R	585+08.22	8.22R	262.11
77-B6	Bent 6	TBD	TBD	TBD
77-B7	Bent 7	TBD	TBD	TBD
77-B8	Abutment 8	TBD	TBD	TBD

Note: 1. Stationing according to I-5 Centerline.
2. Elevations are Above Mean Sea Level (MSL).
3. TBD: To be drilled at a later date.

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on January 19, 2006.

Soil samples in the top 15 to 18 meters (50 to 60 feet) of the borings were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). Below 18 meters, where gravels and cobbles were typically encountered, the rig was outfitted with a 94 mm HQ-Wireline core barrel with an impregnated diamond bit. Core samples were typically retrieved by advancing the core barrel 1.5 meters at a time. Core sampling was performed continuously from the depth of drive sampler refusal to the completion of the boring. At the completion of holes certain borings were fitted with slotted 4-inch PVC pipe casing with sand backfill in order to accurately measure groundwater levels (see Section 4.3.1 for groundwater information).

3.0 LABORATORY TESTING

Corrosivity testing was conducted by the geotechnical consultant, under Caltrans direction, on various selected SPT and bulk samples from the borings for the N5-170 and 5-170 HOV Bridges (53-2876 and 53-2977). The results from the two bridges are summarized in the consultants Final Geotechnical Data Report (URS Corp. dated December 15, 2005). The corrosivity test results for Bridge 53-2976 from the consultant report are summarized in the Foundation Recommendations Memo for that bridge. An additional corrosivity test result taken from a sample drilled for this bridge is given in Section 7.0. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643. The Table below summarizes the Corrosivity Test and CTM designation.

Table No. 2 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

4.0 GEOLOGY

4.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 5.0, Seismicity, of this report.

4.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. The Tujunga Wash crosses under the proposed bridge structure and it has been confined in a concrete lined open top rectangular channel. To the southeast and southwest of the proposed bridge the County of Los Angeles maintains spreading grounds on the order of several acres in size that are used to re-charge groundwater. The southbound 5 Freeway is near pre-freeway grade and the northbound 5

Freeway is in fill section in the vicinity of the proposed bridge. Fill ranges in thickness up to approximately 10 meters. 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 1.7 kilometers north of the proposed project (Please see also Section 5.0, Seismicity).

4.3 Subsurface Conditions

Subsurface conditions at the proposed abutment and bent locations were determined based on the six borings drilled for this project. Due to their proximity to each other, conditions for the HOV bridge are similar to the conditions described for the N5-170 Bridge. As described in the subsurface conditions section for the N5-170 Bridge, generally, the top 9 meters of alluvium (about elevation +265 to 256) was composed of medium dense to dense sands with silt and fine to coarse gravel. This upper layer was underlain by 6 meters (elevation +256 to +250 meters) of dense sands with coarser gravels with few cobbles and scattered seams of stiff clayey silt. Below about +250 meters elevation, typically very dense sands with abundant coarse gravels and cobbles were encountered. Scattered boulders should also be expected below the +250 meter elevation level. Embankment fills were typically composed of dense sands with fine to coarse gravels.

Log of Test Borings (LOTB's) graphically show the results of the soil borings drilled for this project. The LOTB's will be provided at a later date.

4.3.1 Groundwater

Groundwater was not encountered during this investigation to the maximum depth explored (elevation +203.3 meters). Groundwater observation wells were installed during June to November 2005 in boreholes 77-B3, 77-B4 and 77-B5R. An attempt to measure the groundwater elevation in these three well locations was undertaken on February 10, 2006. Well 77-B-4 was measured and found to have no groundwater to the maximum depth of the casing at elevation +203.3 meters. The other two well locations 77-B3 and 77-B5R were not measured due to inaccessibility resulting from mulch stockpiles being placed over the wells inadvertently by a landscape contractor. Soil borings from Log of Test Borings drilled within the general area of the proposed HOV Connector Bridge in 1961 did not encounter groundwater to an elevation of +243 meters (this was the level of the deepest penetration of those borings found).

Several wells are maintained by the LACDPW nearby the vicinity of this project, with a measured elevation of groundwater ranging from 160 to 175 meters (100 to 115 meters depth below ground surface). The area nearby the proposed new connectors contains surface basins for recharge of surface waters to the groundwater. Historically in the past several decades during times of recharge operations water elevations nearby the proposed project have been measured at an elevation of approximately 244 meters (40 meters depth below ground surface). For this project the highest anticipated groundwater should be estimated at elevation 244 meters.

5.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated February 24, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table below summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration is based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 3 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.7	NE	0.7g
Northridge Hills, SSN	Reverse-Oblique	7.5	5.4	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.4	NE	0.6g

5.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 second. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

5.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 5.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 100 feet.

6.0 SCOUR EVALUATION

The Tujunga Wash passes under Bridge 53-2977 between proposed bent 4 and outrigger bent 5R and 5L. The wash is a concrete lined rectangular open top channel. The proposed columns will be placed outside the channel and thus will not be affected by scour. Because of this and the absence of a natural channel running underneath the bridge, the potential for scour is considered low to remote.

7.0 CORROSION EVALUATION

A consultant under the supervision of our office performed corrosion testing. The corrosion test result for the 53-2977 Connector HOV Bridge is taken from their Final Geotechnical Data Report, (URS Corp. dated December 15, 2005) and presented here. A composite soil sample was taken within the top 50 feet of soil boring 77-B5L and was tested for corrosion potential. The test results revealed a minimum resistivity of 4,500 ohm-cm, a pH of 8.0, a soluble sulfate concentration of 21 ppm and a chloride concentration of 60 ppm. Based on these test results and considering the consistency of the soils encountered for both bridges and similar corrosivity results from the 53-2976 bridge the soils that are expected to come in contact with buried concrete should be considered potentially non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

8.0 FOUNDATION RECOMMENDATIONS

The proposed abutments and bents for the new 5/170 HOV Connector Bridge may be supported on driven H-piles for the abutments and Cast-in-Drilled-Holes (CIDH) piles for the bents. Section 8.1 provides detailed recommendations on the CIDH piles and steel H-piles for the abutment and bent locations. Pile diameters and/or widths and specified tip elevations may be found in Section 8.1.

8.1 Pile Recommendations

2700 and 3600 mm Diameter CIDH piles are proposed for Bents 2 through 7, as shown in the pile data table below. HP360x132 Steel H-piles are proposed for Abutments 1 and 8. Pile cutoff elevations and specified Pile Tip Elevations (SPTE) are listed in Table No. 4 below. Pile diameters and compression loads were provided by the Office of Bridge Design on January 31, 2005. In Table 4, the ultimate bearing capacity of the piles will equal or exceed the required nominal resistance in compression.

Table No. 4 - Pile Data Table – 5/170 HOV Connector (New Bridge No. 53-2977)

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
			COMPRESSION (kN)	TENSION			
Abut 1	HP360x132	900		Not Provided	+268.9	+253.7	15.2
Bent 2	3600 mm CIDH	NA	23000	"	+261.0	+233.6	27.4
Bent 3	3600 mm CIDH	NA	25200	"	+262.0	+237.6	24.4
Bent 4	3600 mm CIDH	NA	28100	"	+262.5	+244.2	21.4
Bent 5L	2700 mm CIDH	NA	20400	"	+263.0	+238.6	24.4
Bent 5R	2700 mm CIDH	NA	24100	"	+263.0	+238.7	27.4
Bent 6	3600 mm CIDH	NA	28200	"	+263.0	+235.6	27.4[1]
Bent 7	3600 mm CIDH	NA	26100	"	+264.0	+236.6	27.4[1]
Abut 8	HP360x132	900		"	+263.0	+247.8	15.2[1]

Note: 1. Pile lengths on Bents 6, 7, & Abutment 8 are preliminary only. These pile lengths need to be verified by soil boring information at the Bent and Abutment locations when access to those locations is available.

The pile tip elevations for CIDH piles are based on the skin friction of axial compressive loads only. Driven H-pile tip elevations are based on end bearing and skin friction. However, lateral loading should be considered as well to verify or if necessary, adjust the pile lengths (see Section 8.1.1 for a brief discussion on lateral pile capacities).

Based on observations of the drilled soil boring borehole conditions after completion of drilling, caving within the upper 15 meters is considered likely. Therefore, the drilling contractor should be prepared for borehole caving in the upper 15 meters. A method to stabilize the hole, such as a 15-meter (50-foot) long steel casing, should be utilized. The Pile recommendations provided in Table 4 above are based on subsurface conditions with casing in the drilled hole.

These are perm. casings ?

In addition, the pile data for Bents 6, 7, and Abutment 8 are based on preliminary subsurface information only. Due to access constraints briefly discussed in Section 2.0 of this memo, the borings for the bents mentioned were not drilled. Therefore, our office must be notified at least two weeks in advance of the date when the bents location sites are cleared for construction. Provisions must be made in the construction schedule to allow two weeks for drilling at the bent and abutment locations.

Steel HP360x132 piles must be driven at the abutment locations to the pile tip elevations specified in Table 4. Driving H-piles to 15.2 meters below the pile cutoff elevation should be achievable at the abutments. Therefore, pre-drilling for H-piles is not recommended.

8.1.1 Lateral Pile Analysis

P-y Curves based on soil borings drilled for the abutment and bent locations have been provided to the Office of Bridge Design for use in determining anticipated deflections and moment magnitudes of the proposed pile foundations. The modulus of elasticity of concrete is recommended to be based on a unit weight of 23.55 kPa (150 pcf) and a 28-day compressive strength of 25920 kPa. The provided p-y curves were generated utilizing the computer program LPILE ^{plus} (version 4M).

9.0 CONSTRUCTION CONSIDERATIONS

CIDH Piles

1. Although, based on available records the highest anticipated groundwater elevation is estimated at +244 meters, groundwater was not encountered to an elevation of +203 meters during our field investigation. Therefore, groundwater is considered unlikely to be encountered during construction. Nevertheless, the contractor should be prepared for the unlikely event of handling groundwater in the deeper boreholes. The slurry displacement method based n the Wet Method of CIDH construction would be recommended.

2. Concrete cover and mix design requirements for CIDH piles shall be consistent with Section 8.22 of the Bridge Design Specifications and Section 49-4.01 of the July 2002 Standard Specifications.

3. Adjacent bridge columns must be monitored during CIDH installation. Damage may occur to these columns and piles if excessive caving occurs due to CIDH pile drilling.

4. Drilled holes must conform to Section 49-4.03 of the July 2002 Standard Specifications.

5. As discussed in Section 8.2, borehole-caving potential is likely in the upper 15 meters. Drilled CIDH holes should be inspected for caving after completion of drilling. Methods to keep the borehole open, such as the use of casing should be employed. Plans for the chosen method should be submitted to the Resident Engineer for review.

6. As discussed in Section 4.3 of this report, large cobbles and possibly boulders should be expected to be encountered starting at elevation +250 meters MSL. Some cobbles and the occasional boulder may be encountered above this elevation as well. Therefore, the drilling contractor should have the correct drilling equipment to effectively penetrate these materials.

Driven H-Piles

1. The contractor should anticipate the possibility of hard and erratic driving of steel H piles due to the presence of scattered cobbles. The contractor should anticipate field cutting and splicing of all H piles. Refer to the LOTB sheets for details.

2. Hard driving tips shall be required and installed on all driven steel H piles to ensure pile integrity and limit damage to piles during hard driving.

3. Piles should be driven at least to the specified tip elevations. Any pile that achieves refusal within 4 feet (1.2 meters) of the specified pile tip elevations may be accepted at the Resident Engineer's discretion. This procedure should prevent damage to the piles. Refusal for the piles shall be defined as 3.5 times the design load as shown on the contract plans (per FHWA recommendations for the Gates formula, Section 16.3, Publication No. FHWA HI 97-014 Revised November 1998).

4. If the specified tip elevation is reached without achieving the design (service) load, pile driving should continue until bearing is attained. In this case, it may be prudent to allow the pile to "set up" for a minimum period of 24 hours before continuing the driving.

5. The selected pile-driving hammer should be able to deliver sufficient energy to drive the piles at a penetration rate of 3 mm per blow at the required bearing value. Predrilling is not allowed.

6. Noise and vibration from the pile driving operation should be studied prior to construction due to close proximity of adjacent structures lying outside the state right of way.

7. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 to confirm that the general intent of the recommendations contained in this report has been incorporated into the final construction documents.

10.0 REFERENCES

California Geologic Survey, Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

California Department of Transportation, Corrosion Technology Branch, Corrosion Guidelines, updated to July 2002.

Dibblee, T.W., Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, 1991.

Los Angeles County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

Mualchin, L., A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996, California Department of Transportation, July 1996.

Mualchin, L., Caltrans California Seismic Hazard Map 1996, California Department of Transportation, 1996.

URS, Corporation, Final Geotechnical Data Report, Interstate 5-State Route 170 New Connector, Los Angeles County, California, For Caltrans (Contract No. 59A0283), December 15, 2005.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date: Supervised by: Date: 8-7-06

8-7-06

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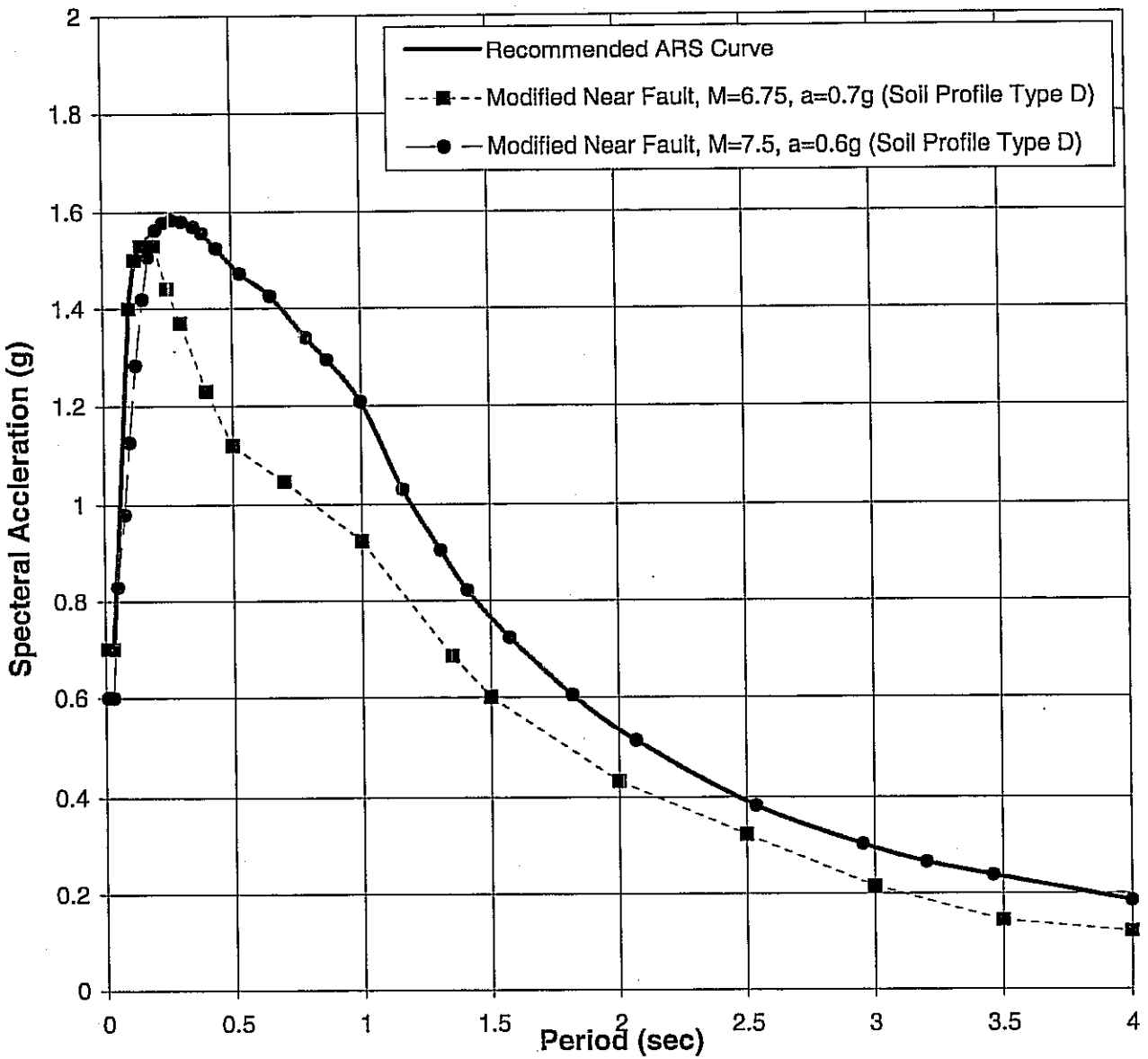
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve ; I-5/SR-170 HOV Connector
Bridge #53-2977
EA-07-121901



FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Tujunga Wash Channel Left Br.
Structure Name

07 - LA - 05/170 - 58.49
District County Route Post Km

07-121901 53 - 1121L
E.A. Number Structure Number

Foundation Report By: S. Sukiasian Dated: 9/8/06; 6/21/07

Reviewed By: M. Okimura (OSD) R. Price (GS)

General Plan Dated: 11/13/07 Foundation Plan Dated: 11/9/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
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- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: February 21, 2008

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121L
(Tujunga Wash Left Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: 2nd Addendum Foundation Report for Tujunga Wash Left Bridge Widening (Bridge No. 53-1121L).

In response to your e-mail dated 2/6/08 concerning the Tujunga Wash Bridge Widening and the adjacent Tujunga Channel Wash (in reference to the County of Los Angeles Permit issues - permit # 200701397) we have prepared an Addendum Memorandum for the Tujunga Wash Left Bridge Widening Foundation Report dated September 8, 2006. This memorandum summarizes changes made to the foundation data for the Tujunga Wash Left Bridge Undercrossing Widen Project.

The data summarized in the tables below shows pile diameters changed from 400 mm previously to 600 mm Cast-in-Drilled-Hole (CIDH) piles. Also shown are revised pile tips and permanent smooth steel casing lengths used to minimize drag or stress from pile friction on the adjacent Tujunga Wash Channel Spread Footing and the influence zone beneath the footing.

The new foundation data summarized in the following tables 2, 7, and 7a, will replace Table No's. 2 and 7, in the June 21, 2007 1st Addendum Memorandum and the September 8, 2006 Foundation Report.

Table No. 2 – Required Pile Data

Support Location	Pile Type (m)	Design Loading (kN)	Required Nominal Resistance		
			Compression (kN)	Tension (kN)	Pile Cutoff Elevation (m)
Abutment 1 Left Side	600-mm CIDH	900	1800	0	260.1
Abutment 2 Left Side	600-mm CIDH	900	1800	0	260.1
Abutment 1 Right Side	600-mm CIDH	900	1800	0	259.3
Abutment 2 Right Side	600-mm CIDH	900	1800	0	259.5

Note: Elevations based on 1988's NAVD Datum.

Table No. 7 - Pile Data Table – Tujunga Channel Left Bridge (Bridge No. 53-1121L)

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
			COMPRESSION (kN)	TENSION			
Abut 1 Left Side Widen	600 mm CIDH	900	1800	0	+260.1	+244.9	15.2[1]
Abut 2 Left Side Widen	600 mm CIDH	900	1800	0	+260.1	+244.9	15.2[1]
Abut 1 Right Side Widen	600 mm CIDH	900	1800	0	+259.3	+244.1	15.2[1]
Abut 2 Right Side Widen	600 mm CIDH	900	1800	0	+259.5	+244.3	15.2[1]

Note: 1. Pile lengths based on axial capacity.

Table No. 7a – Steel Casing Data

LOCATION	DEPTH OF CASING (LENGTH) FROM CUTOFF ELEVATION (m)
Abut 1 Left Side Widen	7.3
Abut 2 Left Side Widen	7.3
Abut 1 Right Side Widen	6.1
Abut 2 Right Side Widen	6.1

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 2/21/08

REGISTERED PROFESSIONAL ENGINEER
SAMUEL G. SUKIASIAN
No. GE2681
Exp. 6-30-08
GEOTECHNICAL
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SAM SUKIASIAN, G.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
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cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: June 21, 2007

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121L
(Tujunga Wash Left Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: Addendum Foundation Report for Tujunga Wash Left Bridge Widening (Bridge No. 53-1121L).

Per your e-mail request dated June 11, 2007, we are providing an Addendum Memo for the Tujunga Wash Left Bridge Widening Foundation Report dated September 8, 2006. This memorandum summarizes changes made to the foundation data for the Tujunga Wash Left Bridge Undercrossing Widen Project.

Please note that the revised data includes new pile cutoff elevations for Abutment 1 (left and right side) and Abutment 2 (left side). Based on the data provided by the Bridge Design Branch 11, the new foundation data is summarized in the following tables, which will replace Table No's. 2 and 7 respectively in the September 8, 2006 Report.

Table No. 2 – Required Pile Data

Support Location	Pile Type (m)	Design Loading (kN)	Required Nominal Resistance		
			Compression (kN)	Tension (kN)	Pile Cutoff Elevation (m)
Abutment 1 Left Side	400-mm CIDH	375	750	0	260.1
Abutment 2 Left Side	400-mm CIDH	375	750	0	260.1
Abutment 1 Right Side	400-mm CIDH	375	750	0	259.3
Abutment 2 Right Side	400-mm CIDH	375	750	0	259.5

Note: Elevations based on 1988's NAVD Datum.

Table No. 7 - Pile Data Table – Tujunga Channel Left Bridge (Bridge No. 53-1121L)

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
			COMPRESSION (kN)	TENSION			
Abut 1 Left Side Widen	400 mm CIDH	375	750	0	+260.1	+250.8	9.3[1]
Abut 2 Left Side Widen	400 mm CIDH	375	750	0	+260.1	+250.8	9.3[1]
Abut 1 Right Side Widen	400 mm CIDH	375	750	0	+259.3	+250.2	9.1[1]
Abut 2 Right Side Widen	400 mm CIDH	375	750	0	+259.5	+250.4	9.1[1]

Note: 1. Pile lengths based on axial capacity.

If you have any questions, please call Sam Sukiasian at (213) 620-2135, or Chi-Tseng "Ted" Liu at (213) 620-2136.

Prepared by: Date: 6-21/07

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Supervised by: Date: 06/21/2007

Deepa Wathugala
for CHI-TSENG "TED" LIU, Ph.D., G.E.,
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cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

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STATE OF CALIFORNIA

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: September 8, 2006

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121L
(Tujunga Wash Left Bridge Widen)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for Tujunga Wash Left Bridge Widening (Bridge No. 53-1121L).

This Memorandum provides foundation recommendations for the Tujunga Wash Left Bridge widening in Los Angeles County (Bridge No. 53-1121L). The recommendations in this memo are based on three borings drilled for this project. The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on three borings drilled for this project. The foundation recommendations in this report are based on the latest abutment and bent locations and their associated structural loads. The latest plans were provided by your office on October 25, 2005. Finally, this memorandum is preceded by and associated with the Tujunga Wash Left Bridge Preliminary Foundation and Seismic Design Reports, dated April 28th and February 20th, 2003, respectively.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, by prepared by the Office of Geotechnical Design South 1 – Branch C, dated April 28, 2003, the existing bridge was built in 1963 as a 24.4 meter long and 17.3 meter wide single-span structure with reinforced concrete open-ended diaphragm abutments. The existing abutments are supported on 406-mm diameter Cast-in-Drilled-Hole (CIDH) concrete piles. The Tujunga Wash Channel flowing underneath the bridge is rectangular in shape and concrete-lined. Abutment end slopes are graded at 1V: 1.5H to 1V: 2.0H (Vertical to Horizontal). The Table below summarizes the existing foundation data.

Table No. 1 – Existing Foundation Summary

Support Location	Pile Type	Design Loading	Elevations (m)		
			Bottom of Pile Cap	Specified Pile Tip	Average Pile Tip
Abutment 1	406-mm CIDH	400 kN	+260.52	+253.05	+252.62
Abutment 2	406-mm CIDH	400 kN	+260.52	+253.05	+252.62

Note: Elevations based on 1929's NGVD and converted to metric units. However, they have not been updated to the current 1988's NAVD Datum.

2.2 Proposed Widening

The proposed widening will involve an increase of 7.05 meters on the left side widening (southwest side of the bridge) and 5.158 meters on the right side widening (northeast side of the bridge). The abutments will be open-ended diaphragm type supports founded on 400-mm CIDH piles. Abutment end slopes will be graded roughly to match the adjacent existing slope grade.

The Table summarizes proposed pile data provided by the Office of Structure Design Branch 11 to our office on October 25, 2005. The Table summarizes design loads nominal loads, and pile tip and cutoff elevations for the abutments.

Table No. 2 – Required Pile Data

Support Location	Pile Type (m)	Design Loading (kN)	Required Nominal Resistance		
			Compression (kN)	Tension (kN)	Pile Cutoff Elevation (m)
Abutment 1 Left Side	400-mm CIDH	375	750	0	260.7
Abutment 2 Left Side	400-mm CIDH	375	750	0	260.9
Abutment 1 Right Side	400-mm CIDH	375	750	0	259.4
Abutment 2 Right Side	400-mm CIDH	375	750	0	259.5

Note: Elevations based on 1988's NAVD Datum.

3.0 FIELD EXPLORATION PROGRAM

Three borings were drilled for this investigation. The borings were drilled at the proposed abutment 1 and 2 left side locations and the proposed abutment 1 right side location. The abutment 1 left side location could not be drilled due to access difficulty. The borings were drilled between July 16th and July 30th, 2003. The borings were drilled to a depth of up to 26 meters (elevation 237.2 meters). The borings were advanced utilizing the mud rotary method with CME-85 drill rig. The location of the borings is summarized in the Table No. 3 below. The boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's

are presently being prepared by the Office of Geotechnical Support and will be submitted to the Office of Structure Design.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset, m	Elevation[2], m
B-1	Abutment 1 - right side	584+79.021	5.31R	262.53
B-2	Abutment 2 - left side	585+25.162	25.00L	264.47
B-3	Abutment 1 – right side	584+77.741	17.80L	263.23

Note: 1. Stationing according to CL5A2 Line.
2. Elevations are Above Mean Sea Level (MSL) (Based on 1988 NAVD Datum).

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on February 10, 2004.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). At the completion of the borings the holes were backfilled with bentonite cement chips.

3.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT, and bulk samples from the borings. Laboratory testing included mechanical analysis and corrosivity. Samples submitted for testing were analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). Grain size curves developed from the mechanical analysis results are shown in the Appendix. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643.

Table No. 4 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 201, 202, 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

4.0 GEOLOGY

4.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 5.0, Seismicity, of this report.

4.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. The Tujunga Wash crosses under the proposed bridge structure and it has been confined in a concrete lined open top rectangular channel. To the southeast and southwest of the proposed bridge the County of Los Angeles maintains spreading grounds on the order of several acres in size that are used to re-charge groundwater. The southbound 5 Freeway is near pre-freeway grade and the northbound 5 Freeway is in fill section in the vicinity of the proposed bridge. Fill ranges in thickness up to approximately 10 meters. 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 1.7 kilometers north of the proposed project (Please see also Section 5.0, Seismicity).

4.3 Subsurface Conditions

Subsurface conditions at the proposed abutment locations were determined based on the three borings drilled for this project. 2.4 to 6.4-meter thick loose to medium dense sand with gravel fills were encountered below the surface (elevation 263 to 254 meters MSL) on the left side of the widening. The fill was underlain by native material, 5.5 to 10.7 meters thick, which was composed of native medium dense sand. Below the medium dense sands, at an elevation of about +250 to +252 meters MSL, dense to very dense sands with gravel and some cobbles were encountered.

On the right side of the widening, 6.7 meters of medium dense sand fill was encountered at the proposed Abutment 1 location (elevation +262.5 to 255.8 meters MSL). The fill was underlain by about 2.5 meters of dense sand and a 1-meter layer of loose silty sand. At an elevation of +252.6 meters MSL, dense to very dense sand with gravel was encountered with increasing cobbles and

boulders at elevation +245 meters MSL. The original LOTB for the Tujunga Wash Bridge, dated March 27, 1961, was reviewed to approximately determine the subsurface conditions at the proposed right side Abutment 2 location. The nearest boring, B-11, revealed medium dense medium to coarse sand underlain by very dense sands with gravels and scattered cobbles at an elevation of +253 meters MSL.

4.3.1 Groundwater

Groundwater was not encountered in any of the borings. The deepest boring was B-1, which was drilled to an elevation of approximately +238 meters did not reveal groundwater. Groundwater was not encountered in any of the borings drilled for the original construction of the bridge (fieldwork dated December 1958) as well. Historical groundwater information is limited, however, the Los Angeles Department of Public works well No. 4896A, located approximately 120 meters to the east, had a reading on October 26, 1983 that was at an elevation of 186.3 meters above mean sea level (MSL) or 76.4 meters below ground surface.

Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from 160 to 175 meters (100 to 115 meters depth below ground surface). The area nearby the proposed new connectors contains surface basins for recharge of surface waters to the groundwater. Historically in the past several decades during times of recharge operations water elevations nearby the proposed project have been measured at an elevation of approximately 244 meters (40 meters depth below ground surface). For this project the highest anticipated groundwater should be estimated at elevation +244 meters.

5.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated April 30, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table below summarizes the Moment Magnitude (Mw) of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.7	NE	0.7g
Northridge Hills, SSN	Reverse-Oblique	7.5	5.4	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.4	NE	0.6g

5.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 seconds. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

5.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 5.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 60 feet (see Section 4.3.1).

6.0 SCOUR EVALUATION

The Tujunga Wash passing under Tujunga Wash Bridge 53-1121L is a concrete-lined rectangular open top channel. The proposed columns will be placed outside the channel and thus will not be affected by scour. Because of this and the absence of a natural channel running underneath the bridge, the potential for scour is considered low to remote.

7.0 CORROSION EVALUATION

Soil samples taken at 3 to 4.5 meter depth at the subject bridge site was tested in the laboratory for corrosion potential. The test result is given in Table No. 6 and indicates that the soil at the site is non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1	1.5 – 3.0 (5-10 ft)	Abutment 1 – Right side	6.98	23000	N/A	N/A
B-2	1.5 – 3.0 (5-10 ft)	Abutment 1 – Right side	7.43	12000	N/A	N/A
B-2	3.0 – 4.6 (10-15 ft)	Abutment 2 – Left side	8.32	9000	N/A	N/A

Note: For corrosion definitions refer to "Memo to Designers" 3-1.

* 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.

Based on the test result and the consistency of the soils at the site, the site is generally not considered corrosive to buried metal and concrete.

8.0 FOUNDATION RECOMMENDATIONS

The proposed abutments for the Tujunga Wash Left Bridge Widening may be supported on Cast-in-Drilled-Holes (CIDH) piles. Section 8.1 provides detailed recommendations on the CIDH piles for the abutment locations. Pile diameters and specified tip elevations may be found in Table No. 7 for the left and right side widening of the Tujunga Wash Left Bridge.

8.1 Pile Recommendations

400-mm CIDH piles are proposed for Abutments 1 and 2, as shown in the pile data table below. Pile cutoff elevations and specified Pile Tip Elevations (SPTE) are listed in Table No. 7 below. The Office of Bridge Design provided the latest compression loads on October 25, 2005. In Table 7 below, the ultimate bearing capacity of the piles will equal or exceed the required nominal resistance in compression.

Table No. 7 - Pile Data Table – Tujunga Channel Left Bridge (Bridge No. 53-1121L)

LOCATION	PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
			COMPRESSION (kN)	TENSION			
Abut 1 Left Side Widen	400 mm CIDH	375	750	0	+259.9	+250.8	9.1[1]
Abut 2 Left Side Widen	400 mm CIDH	375	750	0	+259.9	+250.8	9.1[1]
Abut 1 Right Side Widen	400 mm CIDH	375	750	0	+259.3	+250.2	9.1[1]
Abut 2 Right Side Widen	400 mm CIDH	375	750	0	+259.5	+250.4	9.1[1]

Note: 1. Pile lengths based on axial capacity.

The pile tip elevations are based on skin friction of axial compressive loads only. However, lateral loading should be considered as well to verify or if necessary, adjust the pile lengths (see Section 8.1.1 for a brief discussion on lateral pile capacities). The pile lengths given in the table above should be considered the minimum lengths allowed. These lengths may be increased depending on the results of the lateral pile capacity analysis.

Based on observations during our field investigation on borehole conditions after drilling, caving within the upper 15 meters is likely. Therefore, the drilling contractor should be prepared for borehole caving in the upper 15 meters.

8.1.1 Lateral Pile Analysis

P-y Curves based on soil borings drilled for the abutment and bent locations have been provided to the Office of Bridge Design for use in determining anticipated deflections and moment magnitudes of the proposed pile foundations. The modulus of elasticity of concrete is recommended to be based on a unit weight of 23.55 kPa (150 pcf) and a 28-day compressive strength of 25,920 kPa. The provided p-y curves were generated utilizing the computer program LPILE ^{plus} (version 4M).

9.0 CONSTRUCTION CONSIDERATIONS

<u>CIDH Piles</u>

1. Concrete cover and mix design requirements for CIDH piles shall be consistent with Section 8.22 of the Bridge Design Specifications and Section 49-4.01 of the July 2002 Standard Specifications.

2. Adjacent bridge columns must be monitored during CIDH installation. Damage may occur to these columns and piles if excessive caving occurs due to CIDH pile drilling.

3. Drilled holes must conform to Section 49-4.03 of the July 2002 Standard Specifications.

4. As discussed in Section 8.1, borehole-caving potential is likely in the upper 15 meters. Drilled CIDH holes should be inspected for caving after completion of drilling. Permanent casing is not allowed. Other methods to keep the borehole open, such as temporary casing or using a 4-sack slurry mix should be employed. Plans for the chosen method should be submitted to the Resident Engineer for review.

5. As discussed in Section 4.3 of this report, cobbles should be expected starting at elevation +253 meters MSL. Therefore, the drilling contractor should have the correct drilling equipment to effectively penetrate these materials to reach the required pile tip.

6. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents.

10.0 REFERENCES

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Foundation Report for the Tujunga Channel Left Bridge Widening", Tujunga Wash Channel Left Bridge (53-1121L), EA 07-121901, April 28, 2003.

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Seismic Design Recommendations for the Tujunga Channel Left Bridge Widening", Tujunga Wash Channel Left Bridge (53-1121L), EA 07-121901, February 20, 2003.

California Department of Transportation, Corrosion Technology Branch, "Corrosion Guidelines", updated to July 2002.

California Geological Survey (formerly California Division of Mines and Geology), State of California Special Studies Zones, map for San Fernando Quadrangle, 1979.

Dibblee, T.W., Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, 1991.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

URS, Corporation, Final Geotechnical Data Report, Interstate 5-State Route 170 New Connector, Los Angeles County, California, For Caltrans (Contract No. 59A0283), December 15, 2005.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date: 9-11-06

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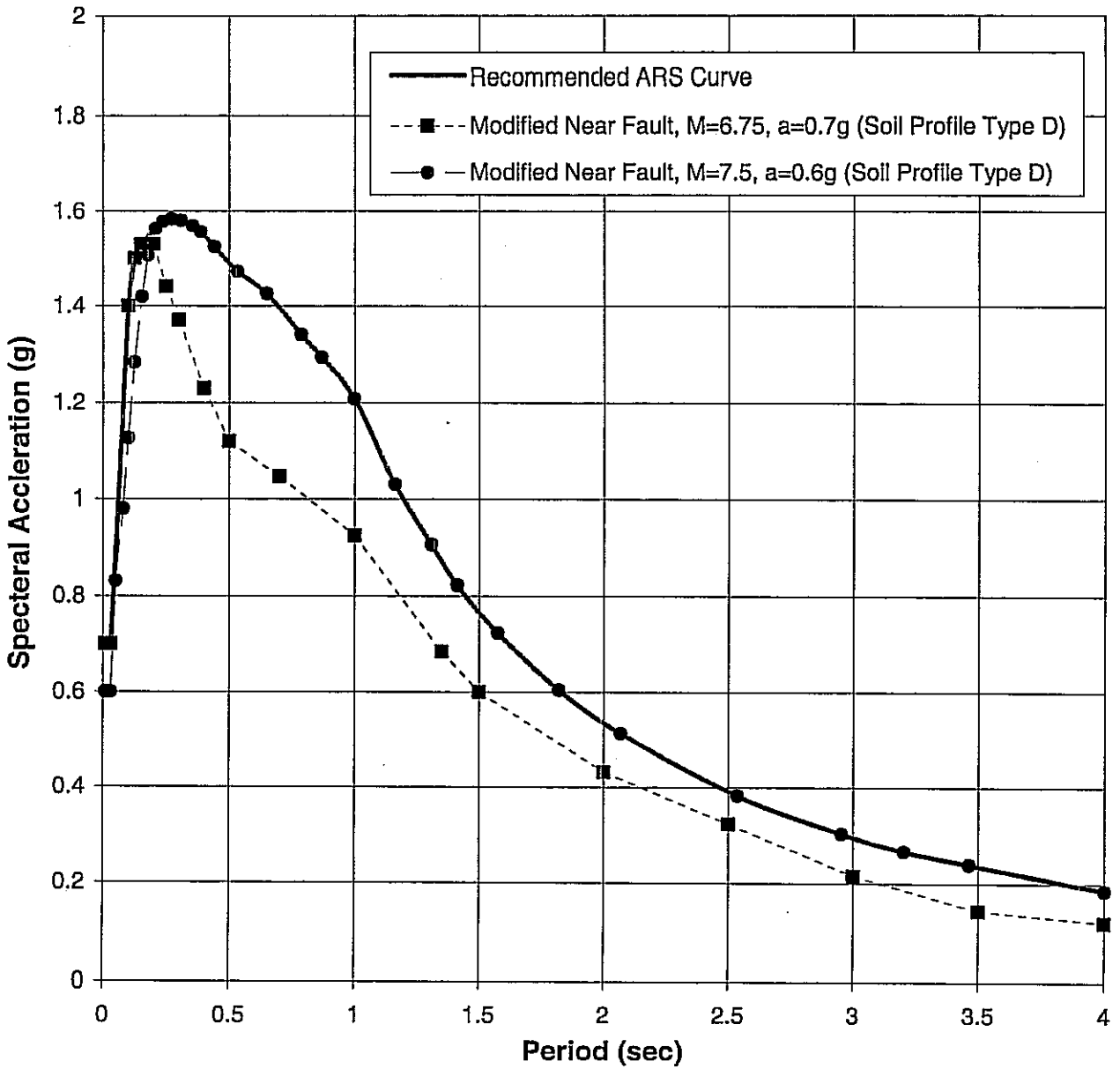
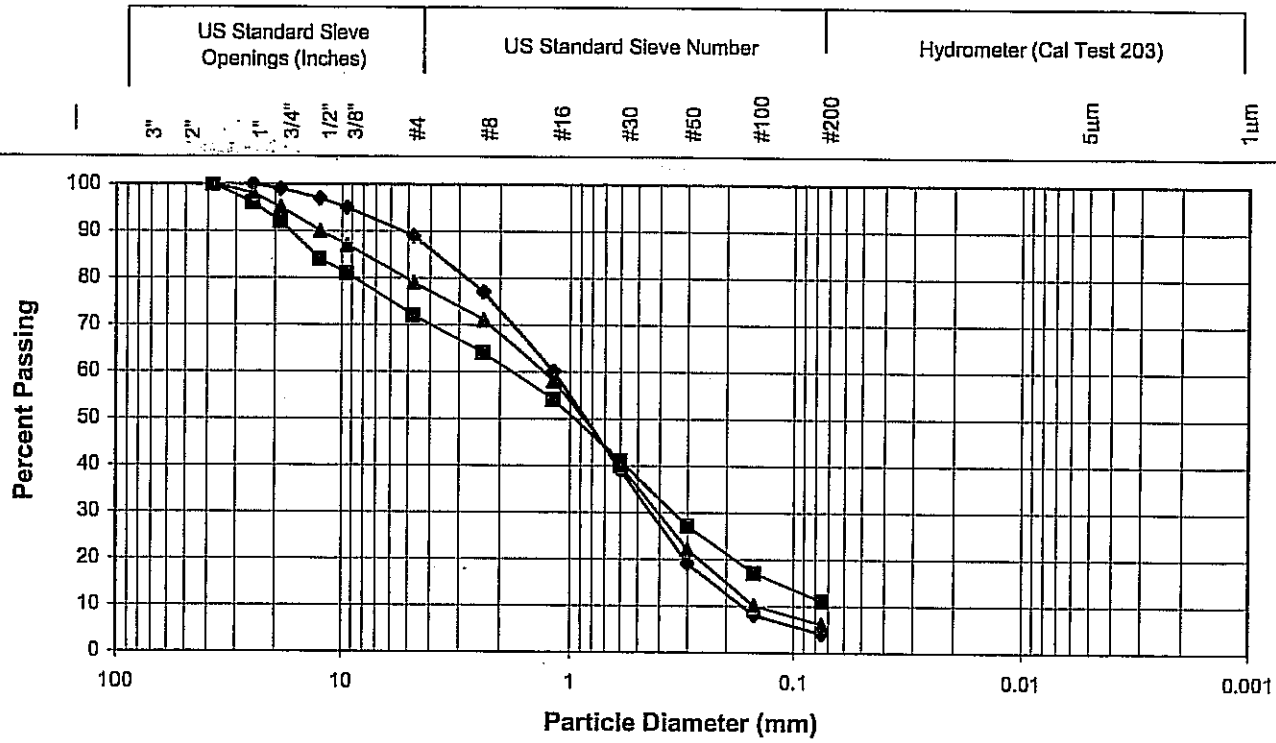
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve for Tujunga Wash Left Bridge
Bridge #53-1121L
EA-07-121901



Appendix – Gradation Test Results

Gradation Analysis Test Results



Sample ID: B-1@1.5m, B-2@1.5m, B-2@4.6m

COBBLES	GRAVELS		SANDS			SILT	CLAY
	Coarse	Fine	Crse.	Medium	Fine		

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	Tujunga Wash Left Bridge Widen
EA:	07-121901
Dist-Co-Rte-KP:	07-LA-5-KP 58.0/63.4

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: July 28, 2009

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121L
(Tujunga Wash Left Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: 3rd Addendum Foundation Report for Tujunga Wash Left Bridge Widening (Bridge No. 53-1121L).

In response to your e-mail dated 7/22/09 concerning the Tujunga Wash Bridge Widening and the adjacent Tujunga Channel Wash (in reference to the County of Los Angeles Permit issues - permit # 200701397) we have prepared an Addendum Memorandum for the Tujunga Wash Left Bridge Widening Foundation Report dated September 8, 2006. This revised memorandum summarizes changes made to the foundation data for the Tujunga Wash Left Bridge Undercrossing Widen Project. This 3rd Addendum follows the 2nd Addendum dated February 21, 2008. This revised Addendum includes additional information on the installation of the permanent casings.

Table No. 7a given in the Addendum No. 2 Report is repeated here again.

Table No. 7a – Steel Casing Data

LOCATION	DEPTH OF CASING (LENGTH) FROM CUTOFF ELEVATION (m)
Abut 1 Left Side Widen	7.3
Abut 2 Left Side Widen	7.3
Abut 1 Right Side Widen	6.1
Abut 2 Right Side Widen	6.1

To minimize the damage potential to the existing channel walls casings should be installed by first predrilling a hole 150-mm (6-inches) smaller in diameter than the casing diameter. The casing, which should be a minimum 205-mm (8-inches) larger than the CIDH diameter should then be driven to the required depth given in Table 7a by impact hammer. The channels walls should also be monitored during pile construction.

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 7/28/09

SAM SUKIASIAN, G.E.
Senior Transportation Engineer
Office of Geotechnical Design – South 1
Branch B

REGISTERED PROFESSIONAL ENGINEER
SAMUEL G. SUKIASIAN
No. GE2681
Exp. 6-30-10
GEOTECHNICAL
STATE OF CALIFORNIA

cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Mingxia W Pan – HQ-Specifications

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Tujunga Wash Channel Right Br.
Structure Name

07 – LA – 05/170 – 58.49
District County Route Post Km

07-121901 53-1121R
E.A. Number Structure Number

Foundation Report By: S. Sukiasian

Dated: 8/10/06; 9/7/06; 9/24/07

Reviewed By: M. Okimura (OSD) R. Price (GS)

General Plan Dated: 7/13/07

Foundation Plan Dated: 11/13/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: February 21, 2008

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121R
(Tujunga Wash Right Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: 3rd Addendum Foundation Report for Tujunga Wash Right Bridge Widening (Bridge No. 53-1121R).

In response to your e-mail dated 2/6/08 concerning the Tujunga Wash Bridge Widening and the adjacent Tujunga Channel Wash (in reference to the County of Los Angeles Permit issues - permit # 200701397) we have prepared a 3rd Addendum Memorandum for the Tujunga Wash Right Bridge Widening Foundation Report originally dated August 10, 2006. This 3rd Addendum Memorandum summarizes the latest changes made to the foundation data for the Tujunga Wash Right Bridge Undercrossing Widen Project.

The use of permanent smooth steel casing lengths are recommended at Bents 2 and 3 to minimize drag or stress from pile friction on the adjacent Tujunga Wash Channel Spread Footings and the influence zone beneath the footings. The lengths of the casing are given in Table 7a below. <u>In addition, 600-mm Cast-in-drilled-hole (CIDH) 13.7-meter long piles should be used at Bents 2 and 3, as provided in the latest Pile Data Table provided below. This latest Pile Data Table replaces all the previous Pile Data Tables for the Tujunga Wash Right Bridge Widening Project: Table No. 7 in Addendum Memorandums 1 and 2 and the Pile Data Table No. 7 in the August 10, 2006 Foundation Report.</u>

Pile Data Table No. 7 – Tujunga Channel Right Bridge (Bridge No. 53-1121R)

LOCATION		PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
				COMPRESSION (kN)	TENSION			
Abut 1	Step 1	400 mm CIDH	400	800	0	+265.0	+255.9	9.1[1]
	Step 2	400 mm CIDH	400	800	0	+264.5	+255.4	9.1[1]
	Step 3	400 mm CIDH	400	800	0	+263.2	+254.1	9.1[1]
	Step 4	400 mm CIDH	400	800	0	+262.0	+252.9	9.1[1]
Bent 2		600 mm CIDH	N/a	1800	0	+260.1	+246.4	13.7[1]
Bent 3		600 mm CIDH	N/a	1800	0	+260.1	+246.4	13.7[1]
Abut 4	Step 1	400 mm CIDH	400	800	0	+265.6	+256.5	9.1[1]
	Step 2	400 mm CIDH	400	800	0	+264.7	+255.6	9.1[1]

Note: 1. Pile lengths based on axial capacity.
2. Elevations based on 1988's NAVD Datum.

The table below shows recommended lengths of smooth steel casing for Bents 2 and 3.

Table No. 7a – Steel Casing Data

LOCATION	DEPTH OF CASING (LENGTH) FROM CUTOFF ELEVATION (m)
Bent 2	6.1
Bent 3	5.5

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 2/21/08

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Office of Geotechnical Design – South 1
Branch B

cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

If you have any questions, please call Sam Sukiasian at (213) 620-2135, or Chi-Tseng "Ted" Liu at (213) 620-2136.

Prepared by: Date: 9-24-07 Supervised by: Date: 9/24/07

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SAM SUKIASIAN, G.E.
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CHI-TSENG "TED" LIU, Ph.D., G.E.,
CHIEF, Branch C
Office of Geotechnical Design – South 1

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STATE OF CALIFORNIA

cc: OGDS1 – District 7 OGDS1 File (2)
GS File- Sacramento;
RE Pending File (District 7 Project Engineer)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: August 10, 2006

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121R
(Tujunga Wash Right Bridge Widen)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for Tujunga Wash Right Bridge Widening (Bridge No. 53-1121R).

This Memorandum provides foundation recommendations for the Tujunga Wash Right Bridge widening in Los Angeles County (Bridge No. 53-1121R). The recommendations in this memo are based on two borings drilled for this project. The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on two borings drilled for this project. The foundation recommendations in this report are based on the latest abutment and bent locations and their associated structural loads. The latest plans were provided by your office on October 25, 2005. Finally, this memorandum is preceded by and associated with the Tujunga Wash Right Bridge Preliminary Foundation and Seismic Design Reports, dated April 25th and February 21st, 2003, respectively.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, by prepared by the Office of Geotechnical Design South 1 – Branch C, dated April 25, 2003, the existing bridge was built in 1963 as a 48.2 meter long three-span structure with reinforced concrete open-ended diaphragm abutments. In 1987 the bridge was widened by 3.7 meters on the West Side making a present bridge width of 21.1 meters. The existing abutments and bents are supported on 406-mm diameter Cast-in-Drilled-Hole (CIDH) concrete piles. The Tujunga Wash Channel flowing underneath the bridge is rectangular in shape and concrete-lined. Abutment end slopes are graded at 1V: 1.5 to 2.0H (Vertical to Horizontal). The Table below summarizes the existing foundation data.

Table No. 1 – Existing Foundation Summary

Support Location	Pile Type	Design Loading	Elevations (m)		
			Bottom of Pile Cap	Specified Pile Tip	Average Pile Tip
Abutment 1	406-mm CIDH	400 kN	+265.24	+254.57	+252.65
Bent 2	406-mm CIDH	625 kN	+260.06	+252.13	+252.44
Bent 3	406-mm CIDH	625 kN	+260.06	+253.13	+252.84
Abutment 4	406-mm CIDH	400 kN	+260.24	+253.57	+253.02

Note: Elevations based on1929's NGVD and converted to metric units. However, they have not been updated to the current 1988's NAVD Datum.

2.2 Proposed Widening

The proposed widening will involve an increase of 21.3 to 24.3 meters on the right side widening (eastside of the bridge). The abutments will be open-ended diaphragm type supports founded on 600-mm CIDH piles. Abutment end slopes will be graded to match the existing adjacent slopes.

The Table summarizes proposed pile data provided by the Office of Structure Design Branch 11 to our office on October 25, 2005. The Table summarizes design loads nominal loads, and pile tip and cutoff elevations for the abutments.

Table No. 2 – Required Pile Data

Support Location		Pile Type (m)	Design Loading (kN)	Required Nominal Resistance		
				Compression (kN)	Tension (kN)	Pile Cutoff Elevation (m)
Abutment 1	Step 1	600 mm CIDH	400	800	0	+265.2
	Step 2	600 mm CIDH	400	800	0	+264.1
	Step 3	600 mm CIDH	400	800	0	+263.0
	Step 4	600 mm CIDH	400	800	0	+262.3
Bent 2		600 mm CIDH	n/a	1250	0	+260.1
Bent 3		600 mm CIDH	n/a	1250	0	+260.1
Abutment 4	Step 1	600 mm CIDH	400	800	0	+265.6
	Step 2	600 mm CIDH	400	800	0	+264.7

Note: Elevations based on 1988's NAVD Datum.

3.0 FIELD EXPLORATION PROGRAM

Two borings were drilled for this investigation. The borings were drilled at the proposed bent 2 and 3 locations. The borings were drilled between July 10th and July 14th, 2003. The borings were drilled to a depth of up to 24.5 meters (elevation +237.9 meters). The borings were advanced utilizing the hollow stem auger method with CME-85 drill rig. The location of the borings is summarized in the Table No. 3 below. The boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's are presently being prepared by the Office of Geotechnical Support and will be submitted to the Office of Structure Design.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset, m	Elevation[2], m
B-1	Proposed Bent 2	584+70.248	57.68R	262.28
B-2	Proposed Bent 3	584+97.924	54.22R	262.43

Note: 1. Stationing according to CL5A2 Line.
2. Elevations are Above Mean Sea Level (MSL). 1988 NAVD Datum.

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on February 10, 2004.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). At the completion of the borings the holes were backfilled with bentonite cement chips.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT, and bulk samples from the borings. Laboratory testing included mechanical analysis and corrosivity. Samples submitted for testing were analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). Grain size curves developed from the mechanical analysis results are shown in the Appendix. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643.

Table No. 4 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 201, 202, 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 6.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. The Tujunga Wash crosses under the proposed bridge structure and it has been confined in a concrete lined open top rectangular channel. To the southeast and southwest of the proposed bridge the County of Los Angeles maintains spreading grounds on the order of several acres in size that are used to re-charge groundwater. The southbound 5 Freeway is near pre-freeway grade and the northbound 5 Freeway is in fill section in the vicinity of the proposed bridge. Fill ranges in thickness up to approximately 10 meters. 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 1.6 kilometers north of the proposed project (Please see also Section 6.0, Seismicity).

5.3 Subsurface Conditions

Subsurface conditions at the proposed abutment and bent locations were determined based on the two borings drilled for this project. 4-meter thick medium dense sand with occasional cobble fills was encountered below the surface (elevation +262 to +258 meters MSL) at the proposed bent locations. The fill was underlain by native material, 6 to 7.6 meters thick, which was composed of native medium dense fine to coarse sand. Below the medium dense sands, at an elevation of about +250 to +252 meters MSL, dense to very dense sands with gravel and some cobbles were encountered.

Based on Log of Test Borings (LOTB's) for the Tujunga Wash Right Channel Bridge Widening, dated January 27, 1986, subsurface conditions for the proposed abutment locations are most likely similar to the soil borings drilled at the proposed bent locations. Boring locations drilled within the existing abutment locations, near the proposed abutments, generally showed that medium dense sand was encountered between elevation +260 to +250 meters MSL. Dense sands with gravels and cobbles were encountered below an elevation of +250 meters MSL.

5.3.1 Groundwater

Groundwater was not encountered in any of the borings. The deepest boring was B-1, which was drilled to an elevation of approximately +237.8 meters did not reveal groundwater. Groundwater was not encountered in any of the borings drilled for the original bridge widening construction (fieldwork dated December 1986) as well. Historical groundwater information is limited, however, the Los Angeles Department of Public works well No. 4896A, located approximately 60 meters to the southeast, had a reading on October 26, 1983 that was at an elevation of 186.3 meters above mean sea level (MSL) or 76.4 meters below ground surface.

Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from 160 to 175 meters (100 to 115 meters depth below ground surface). The area nearby the proposed new connectors contains surface basins for recharge of surface waters to the groundwater. Historically in the past several decades during times of recharge operations water elevations nearby the proposed project have been measured at an elevation of approximately 244 meters (40 meters depth below ground surface). For this project the highest anticipated groundwater should be estimated at elevation +244 meters.

6.0 SEISMICITY

As noted in the Preliminary Seismic Design Recommendations, dated February 21, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table below summarizes the Moment Magnitude (Mw) of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.6	NE	0.7g
Northridge Hills, SSN	Reverse-Oblique	7.5	5.4	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.4	NE	0.6g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 seconds. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 6.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 70 feet.

7.0 SCOUR EVALUATION

The Tujunga Wash passing under Tujunga Wash Bridge 53-1121R is a concrete-lined rectangular open top channel. The proposed columns will be placed outside the channel and thus will not be affected by scour. Because of this and the absence of a natural channel running underneath the bridge, the potential for scour is considered low to remote.

8.0 CORROSION EVALUATION

Soil samples taken at 3 to 4.5 meter depth at the subject bridge site was tested in the laboratory for corrosion potential. The test result is given in Table No. 6 and indicates that the soil at the site is non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1	1.5 – 3.0 (5-10 ft)	Proposed Bent 2	7.22	12000	N/A	N/A
B-2	4.6 – 6.1 (15-20 ft)	Proposed Bent 3	7.88	18000	N/A	N/A

Note: For corrosion definitions refer to "Memo to Designers" 3-1.

* 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.

Based on the test result and the consistency of the soils at the site, the site is generally not considered corrosive to buried metal and concrete.

9.0 FOUNDATION RECOMMENDATIONS

The proposed abutments for the Tujunga Wash Right Bridge Widening may be supported on Cast-in-Drilled-Holes (CIDH) piles. Section 9.1 provides detailed recommendations on the CIDH piles for the abutment locations. Pile diameters and specified tip elevations may be found in Table No. 7 for the widening of the Tujunga Wash Right Bridge.

9.1 Pile Recommendations

600-mm CIDH piles are proposed for the Abutments and Bents, as shown in the pile data table below. Pile cutoff elevations and specified Pile Tip Elevations (SPTE) are listed in Table No. 7 below. The Office of Bridge Design provided the latest pile diameters and compression loads on October 25, 2005. In Table 7 below, the ultimate bearing capacity of the piles will equal or exceed the required nominal resistance in compression.

Table No. 7 - Pile Data Table – Tujunga Channel Right Bridge (Bridge No. 53-1121R)

LOCATION		PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
				COMPRESSION (kN)	TENSION			
Abut 1	Step 1	600 mm CIDH	400	800	0	+265.2	+253.0	12.2[1]
	Step 2	600 mm CIDH	400	800	0	+264.1	+251.9	12.2[1]
	Step 3	600 mm CIDH	400	800	0	+263.0	+250.8	12.2[1]
	Step 4	600 mm CIDH	400	800	0	+262.3	+250.1	12.2[1]
Bent 2		600 mm CIDH	N/a	1250	0	+260.1	+247.9	12.2[1]
Bent 3		600 mm CIDH	N/a	1250	0	+260.1	+247.9	12.2[1]
Abut 4	Step 1	600 mm CIDH	400	800	0	+265.6	+253.4	12.2[1]
	Step 2	600 mm CIDH	400	800	0	+264.7	+252.5	12.2[1]

Note: 1. Pile lengths based on axial capacity.
2. Elevations based on 1988's NAVD Datum.

The pile tip elevations are based on skin friction of axial compressive loads only. However, lateral loading should be considered as well to verify or if necessary, adjust the pile lengths (see Section 9.2.1 for a brief discussion on lateral pile capacities). The pile lengths given in the table above should be considered the minimum lengths allowed. These lengths may be increased depending on the results of the lateral pile capacity analysis.

Based on observations during our field investigation on borehole conditions after drilling, the soil borings caved in the bottom 14 meters of the borehole (elevation 251 to 237 MSL). Therefore, the drilling contractor should be prepared for borehole caving below this level.

9.2.1 Lateral Pile Analysis

P-y Curves based on soil borings drilled for the abutment and bent locations have been provided to the Office of Bridge Design for use in determining anticipated deflections and moment magnitudes of the proposed pile foundations. The modulus of elasticity of concrete is recommended to be based on a unit weight of 23.55 kPa (150 pcf) and a 28-day compressive strength of 25920 kPa. The provided p-y curves were generated utilizing the computer program LPILE ^{plus} (version 4M).

10.0 CONSTRUCTION CONSIDERATIONS

CIDH Piles

1. Although, based on available records the highest anticipated groundwater elevation is estimated at +244 meters, groundwater was not encountered to an elevation of +238 meters during our field investigation. Therefore, groundwater is considered unlikely to be encountered during construction. Nevertheless, the contractor should be prepared for the unlikely event of

handling groundwater in the deeper boreholes. The slurry displacement method based on the Wet Method of CIDH construction would be recommended.

2. Concrete cover and mix design requirements for CIDH piles shall be consistent with Section 8.22 of the Bridge Design Specifications and Section 49-4.01 of the July 2002 Standard Specifications.

3. Adjacent bridge columns must be monitored during CIDH installation. Damage may occur to these columns and piles if excessive caving occurs due to CIDH pile drilling.

4. Drilled holes must conform to Section 49-4.03 of the July 2002 Standard Specifications.

5. As discussed in Section 9.1, borehole-caving potential is likely in the upper 15 meters. Drilled CIDH holes should be inspected for caving after completion of drilling. Permanent casing is not allowed. Other methods to keep the borehole open, such as temporary casing or using a 4-sack slurry mix should be employed. Plans for the chosen method should be submitted to the Resident Engineer for review.

6. As discussed in Section 5.3 of this report, cobbles should be expected starting at elevation +253 meters MSL. Therefore, the drilling contractor should have the correct drilling equipment to effectively penetrate these materials to reach the required pile tip.

7. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents.

11.0 REFERENCES

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Foundation Report for the Tujunga Channel Right Bridge Widening", Tujunga Wash Channel Right Bridge (53-1121R), EA 07-121901, April 25, 2003.

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If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date: 8-10-06

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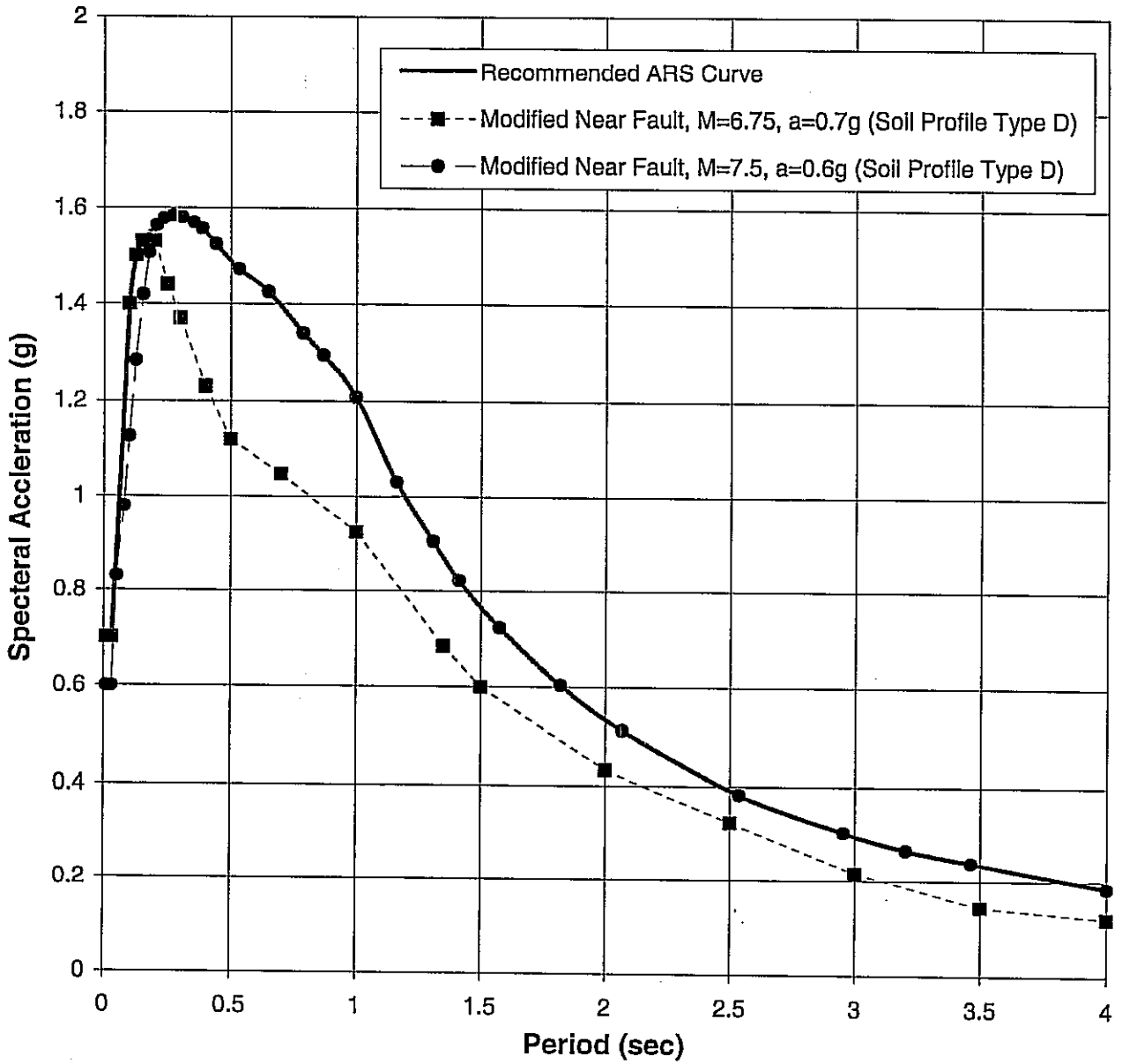
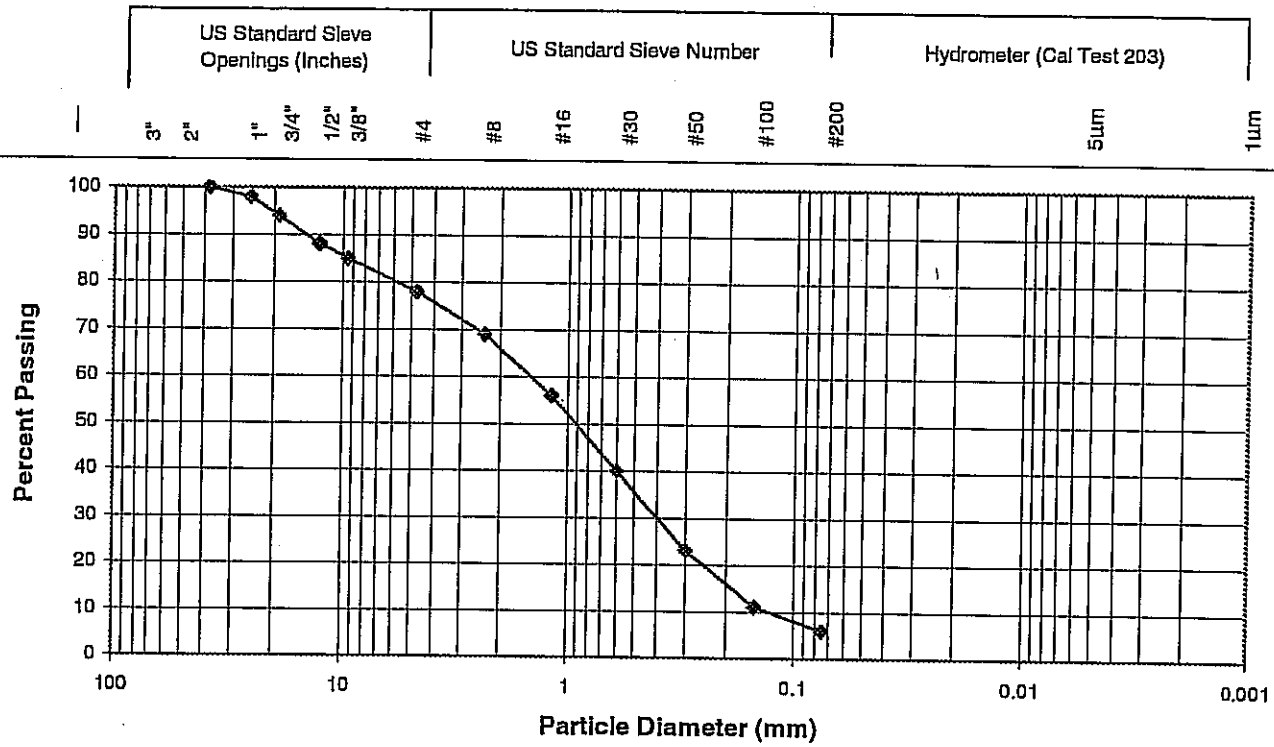
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve fo... ujunga Wash Right Bridge
Bridge #53-1121R
EA-07-121901



Appendix – Grain Size Analysis Results

Gradation Analysis Test Results



Sample ID:	B-1@1.5m			

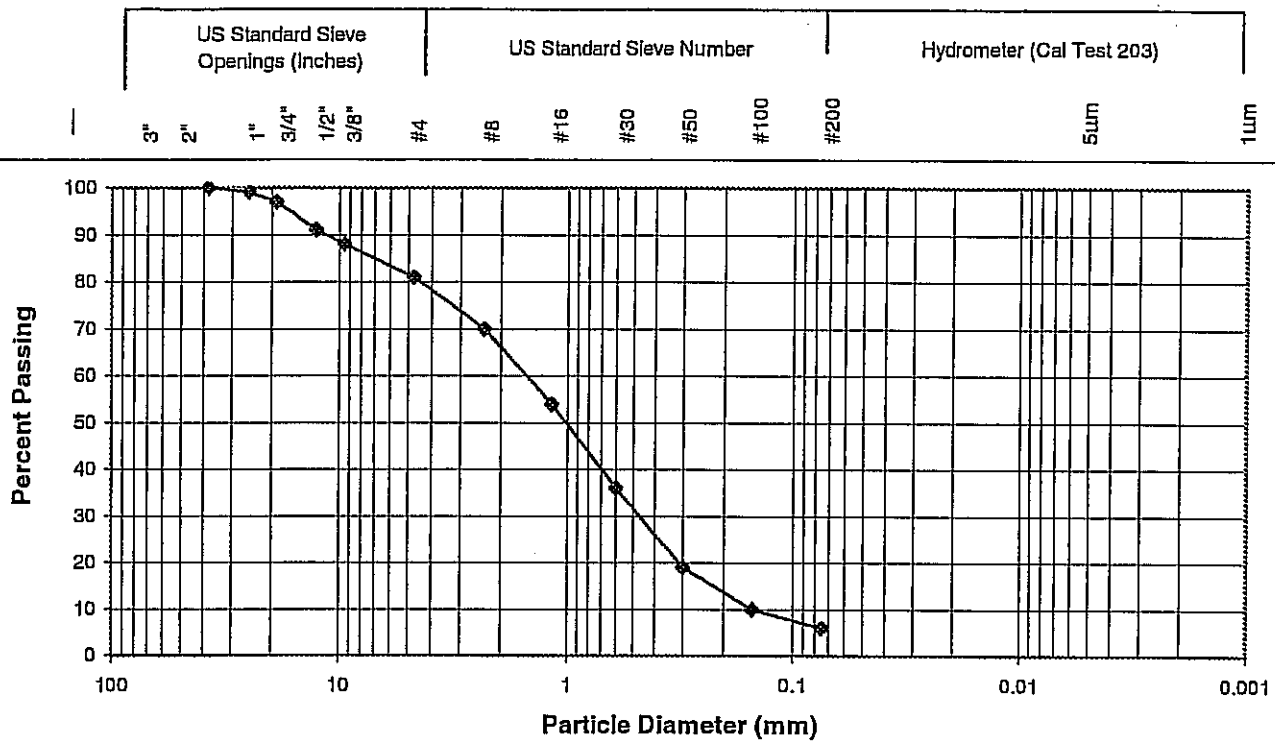
COBBLES	GRAVELS		SANDS			SILT	CLAY
	Coarse	Fine	Crse.	Medium	Fine		

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	Tujunga Bridge Right - Widen
EA:	07-121901
Dist-Co-Rte-KP:	07-LA-5-KP 58.0/63.4

Gradation Analysis Test Results



Sample ID:	B-2@4.6m			

COBBLES	GRAVELS		SANDS			SILT	CLAY
	Coarse	Fine	Crse.	Medium	Fine		

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	Tujunga Bridge Right - Widen
EA:	07-121901
Dist-Co-Rte-KP:	07-LA-5-KP 58.0/63.4

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: September 11, 2006

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121R
(Tujunga Wash Right Bridge Widen)

Attention: Mr. Jose Higareda

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

Subject: Addendum Memorandum for Foundation Report for Tujunga Wash Right Bridge Widening (Bridge No. 53-1121R).

This Addendum Memorandum provides revised pile recommendations that should replace the pile recommendations in the August 10, 2006 Tujunga Wash Right Bridge Foundation Report. The pile data table shown below should replace the Pile Data Table No. 7 in the August 10 Report.

Pile Data Table – Tujunga Channel Right Bridge (Bridge No. 53-1121R)

LOCATION		PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
				COMPRESSION (kN)	TENSION			
Abut 1	Step 1	400 mm CIDH	400	800	0	+265.2	+256.1	9.1[1]
	Step 2	400 mm CIDH	400	800	0	+264.1	+255.0	9.1[1]
	Step 3	400 mm CIDH	400	800	0	+263.0	+253.9	9.1[1]
	Step 4	400 mm CIDH	400	800	0	+262.3	+253.2	9.1[1]
Bent 2		400 mm CIDH	N/a	1250	0	+260.1	+251.0	9.1[1]
Bent 3		400 mm CIDH	N/a	1250	0	+260.1	+251.0	9.1[1]
Abut 4	Step 1	400 mm CIDH	400	800	0	+265.6	+256.5	9.1[1]
	Step 2	400 mm CIDH	400	800	0	+264.7	+255.6	9.1[1]

Note: 1. Pile lengths based on axial capacity.
2. Elevations based on 1988's NAVD Datum.

The new table shows recommendations for 400-mm diameter Cast-in-Drilled-Hole (CIDH) piles being recommended in place of the previously recommended 600-mm piles. As required in the Memo to Bridge Designers, Section 3-1, 400-mm piles may only be used where groundwater or perched water is not encountered. The recommended pile tip elevations shown in the Table are above the highest design groundwater elevation (+244 elevation, per the August 10 Report) and therefore the use of 400-mm diameter piles can be implemented. Perched water is also not expected at this project site.

The table given above is for axial loading only. Lateral loading should be considered as well to verify the pile lengths given herein.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date: 9-11-06

Supervised by: Date: 9-11-06

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cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: September 24, 2007

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121R
(Tujunga Wash Right Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: 2nd Addendum Memorandum for Foundation Report for Tujunga Wash Right Bridge Widening (Bridge No. 53-1121R).

This 2nd Addendum Memorandum provides revised pile cutoff elevations and corresponding revised tip elevations per your e-mail request dated August 23, 2007. The Pile Data table below should replace the previous Pile Data Table given in the September 11, 2006 Addendum Memorandum and the original Pile Data Table (Table No. 7) given in the August 10, 2006 Foundation Report.

Pile Data Table – Tujunga Channel Right Bridge (Bridge No. 53-1121R)

LOCATION		PILE TYPE	DESIGN LOADING (kN)	NOMINAL RESISTANCE		APPROXIMATE CUTOFF ELEVATION (m)	DESIGN TIP ELEVATION (m)	PILE LENGTH (m)
				COMPRESSION (kN)	TENSION			
Abut 1	Step 1	400 mm CIDH	400	800	0	+265.0	+255.9	9.1 [1]
	Step 2	400 mm CIDH	400	800	0	+264.5	+255.0	9.1 [1]
	Step 3	400 mm CIDH	400	800	0	+263.2	+254.1	9.1 [1]
	Step 4	400 mm CIDH	400	800	0	+262.0	+252.9	9.1 [1]
Bent 2		400 mm CIDH	N/a	1250	0	+260.1	+251.0	9.1 [1]
Bent 3		400 mm CIDH	N/a	1250	0	+260.1	+251.0	9.1 [1]
Abut 4	Step 1	400 mm CIDH	400	800	0	+265.6	+256.5	9.1 [1]
	Step 2	400 mm CIDH	400	800	0	+264.7	+255.6	9.1 [1]

Note: 1. Pile lengths based on axial capacity.
2. Elevations based on 1988's NAVD Datum.

If you have any questions, please call Sam Sukiasian at (213) 620-2135, or Chi-Tseng "Ted" Liu at (213) 620-2136.

Prepared by: Date: 9-24-07 Supervised by: Date: 9/24/07

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Office of Geotechnical Design – South 1

REGISTERED PROFESSIONAL ENGINEER
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STATE OF CALIFORNIA

cc: OGDS1 – District 7 OGDS1 File (2)
GS File- Sacramento;
RE Pending File (District 7 Project Engineer)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI
Structure Design
Office of Bridge Design South 1

Date: July 28, 2009

File: 07-LA-5-KP 58.0/63.4
07-121901
Bridge No. 53-1121R
(Tujunga Wash Right Bridge Widen)

Attention: Mr. Mark Okimura

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

Subject: 4th Addendum Foundation Report for Tujunga Wash Right Bridge Widening (Bridge No. 53-1121R).

In response to your e-mail dated 7/22/09 concerning the Tujunga Wash Bridge Widening and the adjacent Tujunga Channel Wash (in reference to the County of Los Angeles Permit issues - permit # 200701397) we have prepared an Addendum Memorandum for the Tujunga Wash Right Bridge Widening Foundation Report originally dated August 10, 2006. This 4th Addendum Memorandum summarizes the latest changes made to the foundation data for the Tujunga Wash Right Bridge Undercrossing Widen Project. This Addendum also follows the 3rd Addendum dated February 21, 2008. It includes additional information on the installation of the permanent casings.

Table No. 7a given in the Addendum No. 3 Report is repeated here again

Table No. 7a – Steel Casing Data

LOCATION	DEPTH OF CASING (LENGTH) FROM CUTOFF ELEVATION (m)
Bent 2	6.1
Bent 3	5.5

To minimize the damage potential to the existing channel walls casings should be installed by first predrilling a hole 150-mm (6-inches) smaller in diameter than the casing diameter. The casing, which should be a minimum 205-mm (8-inches) larger than the CIDH diameter should then be driven to the required depth given in Table 7a by impact hammer. The channels walls should also be monitored during pile construction.

If you have any questions, please call Sam Sukiasian at (213) 620-2135.

Prepared by: Date: 7/28/09

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STATE OF CALIFORNIA

cc: OGDS1 – LOS ANGELES File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Mingxia W Pan – HQ-Specifications

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Bramford St UC
Structure Name

07 – LA – 05/170 – 19.3
District County Route Post Km

07-121901 53-1123
E.A. Number Structure Number

Foundation Report By: D. Wathugala
Dated: 8/16/06 ; 8/14/07

Reviewed By: M. Okimura (OSD) R. Price (GS)

General Plan Dated: 12/7/07
Foundation Plan Dated: 12/10/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

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To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Attn: Mr. Mark Okimura

Date: August 14, 2007

File: 07-LA-5, KP-59.31
EA: 07-121901
Bridge No. 53-1123
(Branford Street U.C. Widening)

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

Subject: Addendum Foundation Report for Branford Street Undercrossing Widening (Bridge No. 53-1123)

Per your email request dated June 11, 2007, we are providing an Addendum Memo for the Foundation Report for Branford Street Undercrossing Widening (Bridge No. 53-1123), dated August 16, 2006. This memorandum summarizes changes made to the foundation data for the Branford Street Undercrossing Widen project.

Based on the data provided by Bridge Design Branch 11 on June 11, 2007, the Tables 2(a), 2(b), 7 and 8 of August 16, 2006 report will be replaced by the Tables 2(a), 2(b), 7 and 8 of this Addendum report.

Table No. 2 (a) – Spread Footing Data for Left Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	269.7 (Step 1)	3354	610	220	n/a	n/a
		268.2 (Step 2)	3660	610	250	n/a	n/a
Pier 2	LFD	268.8	1524	610	n/a	820	I-VI
Pier 3	LFD	268.8	1524	610	n/a	820	I-VI
Abut 4	WSD	272.6 (Step 1)	2440	610	150	n/a	n/a
		271.3 (Step 2)	2440	610	200	n/a	n/a

Table No. 2 (b) – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	272.0 (Step 1)	2440	610	150	n/a	n/a
		270.6 (Step 2)	2440	610	200	n/a	n/a
		269.4 (Step 3)	3050	610	200	n/a	n/a
		268.2 (Step 4)	3660	610	250	n/a	n/a
Pier 2	LFD	268.8	1524	610	n/a	820	I-VI
Pier 3	LFD	268.8	1524	610	n/a	820	I-VI
Abut 4	WSD	272.6 (Step 1)	2440	610	150	n/a	n/a
		271.4 (Step 2)	2440	610	200	n/a	n/a
		270.2 (Step 3)	3050	610	200	n/a	n/a
		269.0 (Step 4)	3660	610	250	n/a	n/a

Note:
(1) Required allowable bearing capacity, q_{all}= unfactored q_{max}
(2) Required nominal bearing resistance, q_n= factored q_{max}/ϕ; Groups I-VI Loading, $\phi = 0.5$ and Group VII, $\phi = 1.0$
(3) Elevations based on 1988's NAVD datum.

Table No. 7 – Spread Footing Data for Left Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	269.7 (Step 1)	3354	220	n/a
		268.2 (Step 2)	3660	250	n/a
Pier 2	LFD	268.8	1524	n/a	820
Pier 3	LFD	268.8	1524	n/a	820
Abut 4	WSD	272.6 (Step 1)	2440	150	n/a
		271.3 (Step 2)	2440	200	n/a

Table No. 8 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	272.0 (Step 1)	2440	150	n/a
		270.6 (Step 2)	2440	200	n/a
		269.4 (Step 3)	3050	200	n/a
		268.2 (Step 4)	3660	250	n/a
Pier 2	LFD	268.8	1524	n/a	820
Pier 3	LFD	268.8	1524	n/a	820
Abut 4	WSD	272.6 (Step 1)	2440	150	n/a
		271.4 (Step 2)	2440	200	n/a
		270.2 (Step 3)	3050	200	n/a
		269.0 (Step 4)	3660	250	n/a

Notes:

1) Working Stress Design, (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Soil Bearing Capacity, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Allowable Soil Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Soil Bearing Resistance (q_n). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Nominal Soil Bearing Resistance (q_n).

3) All footings are to be constructed at or below the elevations listed above in Table No. 1.

4) Piers 2 and 3 footings to be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5) Abutment 1 and 4 footings to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 2002 Standard Specifications.

6) Elevations based on 1988's NAVD datum.

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: 8/14/2007

Deepa Wathugala, Ph.D., G.E.
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Supervised by: Date: 8/14/07

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cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

State of California.

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: August 16, 2006

File: 07-LA-5, KP-59.31
EA: 07-121901
Bridge No. 53-1123
(Branford Street U.C. Widening)

Attn: Mr. Jose Higareda

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

Subject: Foundation Report for Branford Street Undercrossing Widening (Bridge No. 53-1123)

1.0 INTRODUCTION

This Memorandum provides foundation recommendations for the Branford Street Undercrossing Widening in Los Angeles County (Bridge No. 53-1125). The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on the three borings drilled for this project.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, the existing bridge was originally built in 1963 in Los Angeles. The bridge was widened by 2.7 m on the right side of the bridge in 1987 and by 7.4 m on the left side of the bridge in 1993. The bridge is 44.8 m long and 55.7 m wide. This bridge is a three-span structure of continuous PC box girder and PC/PS I girders at the right widening, and PC/PS I girders and rectangular girder at the left widening with reinforced concrete solid bents and open-end diaphragm abutments. The solid bents and abutments are supported on spread continuous footings. Table No. 1 is a summary of as-built foundation information. The original and left widening are enclosed by curtain walls and open on the east widening. Masonry block sound walls are constructed atop of the easterly and westerly railings. Abutment end slopes were graded roughly at 1V: 1.5H to 2.0H (Vertical: Horizontal).

Table No. 1 - Existing Foundation Information

Location	Foundation Type (Design Load)	Approx. Footing Bottom Elevation (m)
Abutment 1	Continuous footing (190 kPa)	+271.5
Pier 2	Continuous footing (240 kPa)	+267.7
Pier 3	Continuous footing (240 kPa)	+267.7
Abutment 4	Continuous footing (190 kPa)	+272.1

Notes: Elevations were based on 1929's NGVD and converted into metric units. However, they have not been updated to the current 1988's NAVD datum

2.2 Proposed Widening

The proposed construction will include widening the northbound and southbound directions within the bridge structure. The widening will consist of reinforced concrete box girders. The bents and abutments will be supported on spread continuous footings. The widening will include a sound wall on concrete barrier located on top of the bridge deck on the northbound and southbound side.

The information on spread footing received from the Office of Bridge Design is given in Table 2 (a) and Table 2 (b) for Left Side Widening and Right Side Widening respectively.

Table No. 2 (a) – Spread Footing Data for Left Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	269.7 (Step 1)	3354	610	220	n/a	n/a
		268.2 (Step 2)	3660	610	250	n/a	n/a
Pier 2	LFD	268.8	1524	610	n/a	820	I-VI
Pier 3	LFD	268.8	1524	610	n/a	820	I-VI
Abut 4	WSD	272.6 (Step 1)	2140	610	200	n/a	n/a
		271.3 (Step 2)	2440	610	200	n/a	n/a

Table No. 2 (b) – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	272.0 (Step 1)	1830	610	200	n/a	n/a
		270.6 (Step 2)	2440	610	200	n/a	n/a
		269.4 (Step 3)	3050	610	200	n/a	n/a
		268.2 (Step 4)	3660	610	250	n/a	n/a
Pier 2	LFD	268.8	1524	610	n/a	820	I-VI
Pier 3	LFD	268.8	1524	610	n/a	820	I-VI
Abut 4	WSD	272.6 (Step 1)	1830	610	200	n/a	n/a
		271.4 (Step 2)	2440	610	200	n/a	n/a
		270.2 (Step 3)	3050	610	200	n/a	n/a
		269.0 (Step 4)	3660	610	250	n/a	n/a

Note:
(1) Required allowable bearing capacity, q_{all}= unfactored q_{max}
(2) Required nominal bearing resistance, q_n= factored q_{max}/ϕ; Groups I-VI Loading, $\phi = 0.5$ and Group VII, $\phi = 1.0$
(3) Elevations based on 1988's NAVD datum.

3.0 FIELD EXPLORATION PROGRAM

Three borings were drilled for this investigation between July 17 and 23, 2003. The borings were advanced utilizing the hollow-stem auger method with a CEM-85 drill rig with a automatic trip hammer. These borings were drilled to an approximate depth of 18 meters. The location of the borings is summarized in the Table No. 3 below.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset	Elevation[2], m
B-1	Pier 2/Abutment 1 N/B	593+20.015	38.38R	269.71
B-2	Pier 3/Abutment 4 N/B	593+57.394	39.61R	270.38
B-3	Pier 2/Abutment 1 S/B	593+20.937	37.11L	269.45

Notes: 1. Reference line is Ciil Line.
2. Elevations based on 1988's NAVD datum.

The survey results were recorded and provided by District 7 Surveys on February 10, 2004.

An engineering geologist from Geotechnical Design South-1 Branch C logged and sampled the borings using a Standard Penetration Test (SPT) sampler at 1.52-meter (5-foot) intervals. The SPT samples were driven using a 63.6-kilogram hammer falling freely for 762 mm for a total penetration of 457 mm. In addition, representative bulk samples were collected from borings. Following drilling, sampling, and logging, the borings were backfilled with bentonite chips.

LOTB's (Log of Test Borings) will be prepared by Contracts, Graphics & Records of Office of Geotechnical Support and will be submitted to the Office of Structure Design.

4.0 LABORATORY TESTING

Laboratory testing was performed on a bulk sample from the borings. Laboratory testing included corrosion test. Sample submitted for testing was analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). A summary of corrosion test results is presented in Section 8.0.

Table No. 4 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 6.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. Fill ranges in thickness up to approximately 10 meters. 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 1.7 kilometers north of the proposed project (Please see also Section 6.0, Seismicity).

5.3 Subsurface Conditions

Subsurface soil conditions at the proposed abutment and bent locations were determined based on the three borings drilled for this project, and the as-built LOTB. The subject area generally consists of artificial fill up to an approximate elevation of +269 m. This artificial material is composed of well graded medium dense to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, the top 12 meters of native alluvium (about elevation +269 to +257) is composed of medium dense to dense sand with fine to coarse gravel. This upper layer was underlain by 6 meters (elevation +257 to +251 meters) of dense to very dense gravelly sand with cobbles. Native soil is occasionally interbedded with silt, sandy silt and silty sand.

5.3.1 Groundwater

Groundwater was not encountered in any of the borings. The deepest boring was B-3, which was drilled to an elevation of approximately +251 meters. 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 well number 4885, located approximately 0.5 km northeast of the project site, had a maximum reading from 1994 to 2001 as an elevation of +170 meters above mean sea level (MSL).

6.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated April 30, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as

stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table No. 5 summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance (km)	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.5	NE	0.7g
Simi/Santa Rosa-Northridge Hills, SSN	Reverse-Oblique	7.5	6.6	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.0	NE	0.6g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 second. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1, the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated, fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Due to the dense nature of the granular soils beneath the proposed bent and abutment locations and the absence of groundwater within the zone of influence, the potential for the occurrence of liquefaction at the site is considered low to remote.

7.0 SCOUR EVALUATION

Due to the absence of a channel running under the bridge and concrete and asphalt covering the bottom of the bridge, the potential for scour is considered low to remote.

8.0 CORROSION EVALUATION

A soil sample from Boring No. B-2 was tested at the District 7 Transportation Laboratory in Los Angeles for corrosion potential. The test results summarized in Table No. 6 indicate that the soil at the site is generally non-corrosive to metal and reinforced concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-2	1.5 – 3.0	Bent 3/Abutment 4 N/B	6.44	14000	N/A	N/A

Note: For corrosion definitions refer to "Memo to Designers" 3-1.
* 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.

9.0 FOUNDATION RECOMMENDATIONS

The proposed abutment and bent locations may be supported on continuous spread footings. Recommended bearing capacities and estimated settlements are given in the following sections. Spread footing sizes are summarized in Table 7 and Table 8.

9.1 Bearing Capacity

Ultimate bearing capacity values were calculated using Terzaghi's equation with a friction angle and cohesive values developed from the soil profiles for the northbound and southbound abutment and bent locations. The Table below summarizes the minimum spread footing dimensions and the corresponding Allowable Soil Bearing Capacity and Nominal Soil Bearing Resistance for the proposed abutment and bent locations.

Table No. 7 – Spread Footing Data for Left Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	269.7 (Step 1)	3354	220	n/a
		268.2 (Step 2)	3660	250	n/a
Pier 2	LFD	268.8	1524	n/a	820
Pier 3	LFD	268.8	1524	n/a	820
Abut 4	WSD	272.6 (Step 1)	2440	150	n/a
		271.3 (Step 2)	2440	200	n/a

Table No. 8 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	272.0 (Step 1)	2440	150	n/a
		270.6 (Step 2)	2440	200	n/a
		269.4 (Step 3)	3050	200	n/a
		268.2 (Step 4)	3660	250	n/a
Pier 2	LFD	268.8	1524	n/a	820
Pier 3	LFD	268.8	1524	n/a	820
Abut 4	WSD	272.6 (Step 1)	2440	150	n/a
		271.4 (Step 2)	2440	200	n/a
		270.2 (Step 3)	3050	200	n/a
		269.0 (Step 4)	3660	250	n/a

Notes:

1) Working Stress Design, (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Soil Bearing Capacity, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Allowable Soil Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Soil Bearing Resistance (q_n). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Nominal Soil Bearing Resistance (q_n).

3) All footings are to be constructed at or below the elevations listed above in Table No. 1.

4) Piers 2 and 3 footings to be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5) Abutment 1 and 4 footings to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 2002 Standard Specifications.

6) Elevations based on 1988's NAVD datum.

9.2 Anticipated Settlement of Spread Footings

Total settlements were calculated for the proposed abutment and pier footings. Settlement was based on allowable bearing capacities at the abutments and unfactored loads at the piers. The settlement parameters were estimated from generalized soil profiles for each proposed abutment and bent. Table No. 9 and Table No.10 give the approximate total settlements estimated. Due to the granular nature of the underlying granular soils, the anticipated settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

Table No. 9 – Approximate Immediate Total Settlements - Left Side Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Abut 1	-	250 kPa	30
Pier 2	350 kPa	-	20
Pier 3	350 kPa	-	20
Abut 4		200 kPa	15

Table No. 10 – Approximate Immediate Total Settlements - Right Side Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Abut 1	-	250 kPa	35
Pier 2	300 kPa	-	25
Pier 3	300 kPa	-	30
Abut 4		250 kPa	30

The existing structure has been in service for over 40 years, since 1963. Therefore, we anticipate that it will experience no more settlements. To alleviate stressing the existing structure and its foundations by the potential vertical settlements of proposed widening, the new foundations and structure should be structurally tied to the existing structure in such a way as to allow vertical displacement between them without lateral separation.

9.3 Lateral Active Earth Pressure

If abutment walls are free to move laterally at the top, a static active lateral earth pressure of 5.7 kPa per meter of depth is recommended. This active lateral earth pressure was calculated using an active earth pressure coefficient of 0.3 and a soil unit weight of 19 kN/m^3.

10.0 CONSTRUCTION CONSIDERATIONS

1. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 or a Sand Equivalent of 20 or more should be used within the approach embankment, in accordance with standard Caltrans requirements.

2. Coarse gravels and cobbles should be expected during excavation of the footing locations and keyways. This material may be removed using conventional excavators and scraper and ripper equipment.

3. On-site material may be used as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material (greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

4. The new embankment fill should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting).

The recommendations contained in this report are based on specific project information that has been provided by the Office of Structure Design. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

11.0 REFERENCES AND CORRESPONDENCES

California Geologic Survey, Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

California Department of Transportation, Corrosion Technology Branch, Corrosion Guidelines, July 2002.

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Caltrans California Seismic Hazard Map 1996, California Department of Transportation, L. Mualchin, 1996.

Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, T.W. Dibblee, 1991.

"Principles of Foundations Engineering", Braja M. Das, Fourth Edition, 1999.

"Foundations Design, Principles and Practices", Donald P. Coduto, Second Edition, 2001.

"Foundations and Earth Structures", Design Manual 7.02, NAVFAC, Naval Facilities Engineering Command, 1986.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

"As Built" Log of Test Borings, Branford Street Undercrossing, March 7, 1961.

"As Built" General plan, Foundation plan, Abutment and Bent Details plans for Branford Street Undercrossing, 1961.

"As Built" General plan, Foundation plan, Abutment and Bent Details plans for Branford Street Undercrossing Widening, 1986.

"As Built" General plan, Foundation plan, Abutment and Bent Details plans for Branford Street Undercrossing Widening, 1992.

Foundation Recommendations for Branford Street Undercrossing, dated January 8, 1959.

Foundation Recommendations for Branford Street Undercrossing Widening, dated may 31, 1985.

Foundation Recommendations for Branford Street Undercrossing Widening, dated August 26, 1991.

Preliminary Seismic Design Recommendations for Branford Street Undercrossing Widening, dated February 26, 2003.

Preliminary Foundation Report for Branford Street Undercrossing Widening, dated April 23, 2003.

If you have any questions or comments, please call Deepa Wathugala at (562) 465-0152, or Ted Liu at or (562) 864-0805.

Prepared by: Date:

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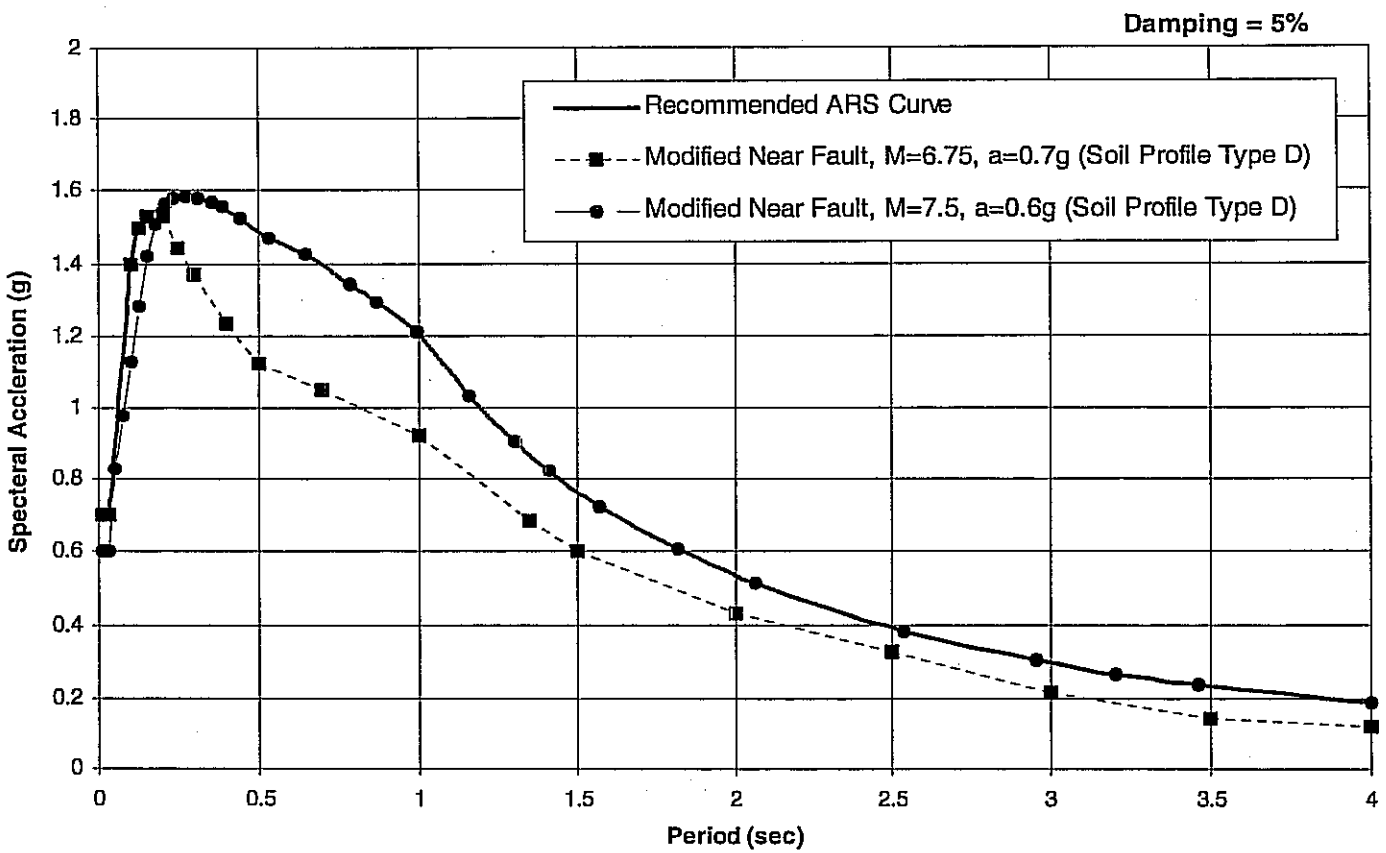
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve for Branford Street UC
Bridge #53-1123
EA: 07-121901



FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Osborne St. UC
Structure Name

07 - LA - 05/170 - 59.3
District County Route Post Km

07-121901 53 - 1124
E.A. Number Structure Number

Foundation Report By: D. Wathugala Dated: 8/14/06 ; 8/14/07

Reviewed By: M Okimura (OSD) R. Price (GS)

General Plan Dated: 12/3/07 Foundation Plan Dated: 12/4/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: August 14, 2007

File: 07-LA-5, KP-60.20
EA: 07-121901
Bridge No. 53-1124
(Osborne Street U.C. Widening)

Attn: Mr. Mark Okimura

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

Subject: Addendum Foundation Report for Osborne Street Undercrossing Widening (Bridge No. 1124)

Per your email request dated June 11, 2007, we are providing an Addendum Memo for the Foundation Report for Osborne Street Undercrossing Widening (Bridge No. 1124) dated August 16, 2006. This memorandum summarizes changes made to the foundation data for Osborne Street Undercrossing Widen project.

Based on the data provided by Bridge Design Branch 11 on June 11, 2007, the Tables 2, 7 and 8 of August 16, 2006 report will be replaced by the Tables 2, 7 and 8 of this Addendum report.

Table No. 2 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	281.5	2135	610	125	n/a	n/a
Bent 2	LFD	277.3	1830	610	n/a	810	I-VI
Bent 3	LFD	277.3	3050 x 3050	760	n/a	800	I-VI
Bent 4	LFD	277.3	1830	610	n/a	810	I-VI
Abut 5	WSD	281.5 (Step 1)	2135	610	125	n/a	n/a
		280.3 (Step 2)	2745	610	150	n/a	n/a
		279.1 (Step 3)	3200	610	160	n/a	n/a
		278.0 (Step 4)	3660	610	170	n/a	n/a

Note:

(1) Required allowable bearing capacity, q_{all}= unfactored q_{max}

(2) Required nominal bearing resistance, q_n= factored q_{max}/ϕ; Groups I-VI Loading, $\phi = 0.5$ and Group VII, $\phi = 1.0$

(3) Elevations based on 1988's NAVD datum.

Table No. 7 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	281.5	2135	125	n/a
Bent 2	LFD	277.3	1830	n/a	810
Bent 3	LFD	277.3	3050 x 3050	n/a	800
Bent 4	LFD	277.3	1830	n/a	810
Abut 5	WSD	281.5 (Step 1)	2135	125	n/a
		280.3 (Step 2)	2745	150	n/a
		279.1 (Step 3)	3200	160	n/a
		278.0 (Step 4)	3660	170	n/a

Notes:

1) Working Stress Design, (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Soil Bearing Capacity, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Allowable Soil Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Soil Bearing Resistance (q_n). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Nominal Soil Bearing Resistance (q_n).

3) All footings are to be constructed at or below the elevations listed above in Table No. 1.

4) Bents 2, 3 and 4 footings to be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5)Abutment 1 and 5 footings to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 2002 Standard Specifications.

6) Elevations based on 1988's NAVD datum.

Table No. 8 – Approximate Immediate Total Settlements - Right Side Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Abut 1	-	125 kPa	10
Bent 2	220 kPa	-	15
Bent 3	240 kPa	-	15
Bent 4	220 kPa		15
Abut 5		170 kPa	10

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: 8/14/2007

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STATE OF CALIFORNIA

cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: August 16, 2006

File: 07-LA-5, KP-60.20
EA: 07-121901
Bridge No. 53-1124
(Osborne Street U.C. Widening)

Attn: Mr. Jose Higareda

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

Subject: Foundation Report for Osborne Street Undercrossing Widening (Bridge No. 1124)

1.0 INTRODUCTION

This Memorandum provides foundation recommendations for the Osborne Street Undercrossing Widening in Los Angeles County (Bridge No. 53-1124). The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on the three borings drilled for this project.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, the existing bridge was originally built in 1963 in Los Angeles. The bridge was widened by 11.0 m on the left side of the bridge in 1994. The bridge is 51.2 m long and 56.6 m wide. This bridge is a four-span structure of continuous RC box girder, widened at the west side with CIP/PS concrete box girder with RC solid bents and eight column bent and RC open-end diaphragm abutments. The bents are supported on spread square footings and spread continuous footings. The abutments are supported on spread continuous footings. The end spans are enclosed by curtain walls. Table 1 is a summary of as-built foundation information. Abutment end slopes were graded roughly at 1V: 1.5H to 2.0H (Vertical: Horizontal).

Table 1. As-built Spread Foundation Information

Location	Foundation Type	Approx. Ground Surface Elevation (m)	Approx. Footing Bottom Elevation (m)
Abutment 1	Continuous footing	+283.8	+280.95
Bent 2	Continuous footing	+278.0	+276.83
Bent 3	Square footing	+278.0	+276.22
Bent 4	Continuous footing	+278.0	+276.83
Abutment 5	Continuous footing	+284.2	+280.95

Notes: Elevations were based on 1929's NGVD and converted into metric units. However, they have not been updated to the current 1988's NAVD datum.

2.2 Proposed Widening

The proposed construction will include widening the northbound direction within the bridge structure. The widening will consist of CIP/RC box girders at spans 1 and 4, and PC/PS girders with concrete slab at spans 2 and 3. The bents 2 and 4 will be supported on spread continuous footings. The bent 3 will be supported on spread square footings. The abutments will be supported on spread continuous footings.

The information on spread footing received from the Office of Bridge Design is given in Table 2.

Table No. 2 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	281.5	2135	610	125	n/a	n/a
Bent 2	LFD	277.3	2440	610	n/a	670	I-VI
Bent 3	LFD	277.3	3050 x 3050			800	
Bent 4	LFD	277.3	2440	610	n/a	670	I-VI
Abut 5	WSD	281.5 (Step 1)	2135	610	125	n/a	n/a
		280.3 (Step 2)	2440	610	150	n/a	n/a
		279.1 (Step 3)	3050	610	145	n/a	n/a
		278.0 (Step 4)	3660	610	170	n/a	n/a

Note:
(1) Required allowable bearing capacity, q_{all}= unfactored q_{max}
(2) Required nominal bearing resistance, q_n= factored q_{max}/ϕ; Groups I-VI Loading, $\phi = 0.5$ and Group VII, $\phi = 1.0$
(3) Elevations based on 1988's NAVD datum.

3.0 FIELD EXPLORATION PROGRAM

Three borings were drilled for this investigation between July 17 and 23, 2003. The borings were advanced utilizing hollow-stem auger method with a CEM-85 drill rig with a automatic trip hammer. These borings were drilled to an approximate depth of 18 meters. The location of the borings is summarized in the Table No. 3 below.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset	Elevation[2], m
B-1	Bent 2/Abutment 1	601+97.292	38.64R	279.33
B-2	Bent 4/Abutment 5	602+37.605	38.89R	279.49
B-3	Bent 3	602+16.449	25.73R	279.23

Notes: 1. Reference line is Ciil Line.
2. Elevations based on 1988's NAVD datum.

The survey results were recorded and provided by District 7 Surveys on February 10, 2004.

An engineering geologist from Geotechnical Design South-1 Branch C logged and sampled the borings using a Standard Penetration Test (SPT) sampler at 1.52 meter (5 foot) intervals. The SPT samples were driven using a 63.6-kilogram hammer falling freely for 762 mm for a total penetration of 457 mm. In addition, representative bulk samples were collected from borings. Following drilling, sampling, and logging, the borings were backfilled with bentonite chips.

LOTB's (Log of Test Borings) are being prepared by Contracts, Graphics & Records of Office of Geotechnical Support and will be submitted to the Office of Structure Design.

4.0 LABORATORY TESTING

Laboratory testing was performed on a bulk sample from the borings. Laboratory testing included corrosion test. Sample submitted for testing was analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). A summary of corrosion test results is presented in Section 8.0.

Table No. 4 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 6.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. Fill ranges in thickness up to approximately 10 meters. 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 1.7 kilometers north of the proposed project (Please see also Section 7.0, Seismicity).

5.3 Subsurface Conditions

Subsurface soil conditions at the proposed abutment and bent locations were determined based on the three borings drilled for this project. The subject area generally consists of artificial fill up to an approximate elevation of +279 m. This artificial material is composed of well graded medium dense to dense, fine to coarse sand with occasional

gravel and cobbles. Below the fill material, the top 12 meters of native alluvium (about elevation +279 to +267) is composed of medium dense to dense sands and gravelly sands with cobbles. This upper layer was underlain by 6 meters (elevation +267 to +261 meters) of dense to very dense gravelly sands with cobbles. Native soil is occasionally interbedded with silty sand.

5.3.1 Groundwater

Groundwater was not encountered in any of the borings. The deepest boring was B-3, which was drilled to an elevation of approximately +261 meters. 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 well number 4885, located approximately 1.0 km southeast of the project site, had a maximum elevation reading from 1994 to 2001 of +170 meters above mean sea level (MSL).

6.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated April 30, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table No. 5 summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.2	NE	0.8g
Simi/Santa Rosa-Northridge Hills, SSN	Reverse-Oblique	7.5	6.0	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	5.5	NE	0.7g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.8g (VDO Fault) and Mw of 7.5 and PBA of 0.7g (SSD Fault). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 second. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated, fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Due to the dense nature of the granular soils beneath the proposed bent and abutment locations and the absence of groundwater within the zone of influence, the potential for the occurrence of liquefaction at the site is considered low to remote.

7.0 SCOUR EVALUATION

Due to the absence of a channel running under the bridge and concrete and asphalt covering the bottom of the bridge, the potential for scour is considered low to remote.

8.0 CORROSION EVALUATION

A soil sample from Boring No. B-2 was tested at the District 7 Transportation Laboratory in Los Angeles for corrosion potential. The test results summarized in Table No. 6 indicate that the soil at the site is generally non-corrosive to metal and reinforced concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-2	1.5 – 3.0	Bent 5	7.28	6400	N/A	N/A

Note: For corrosion definitions refer to "Memo to Designers" 3-1.

* 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.

9.0 FOUNDATION RECOMMENDATIONS

The proposed abutment and bent locations may be supported on spread footings. Recommended bearing capacities and estimated settlements are given in the following sections. Spread footing sizes are summarized in Table 7.

9.1 Bearing Capacity

Ultimate bearing capacity values were calculated using Terzaghi's equation with a friction angle and cohesive values developed from the soil profiles for the northbound and southbound abutment and bent locations. The Table below summarizes the minimum spread footing dimensions and the corresponding Allowable Soil Bearing Capacity and Nominal Soil Bearing Resistance for the proposed abutment and bent locations.

Table No. 7 – Spread Footing Data for Right Side Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	281.5	2135	125	n/a
Bent 2	LFD	277.3	2440	n/a	670
Bent 3	LFD	277.3	3050 x 3050	n/a	800
Bent 4	LFD	277.3	2440	n/a	670
Abut 5	WSD	281.5 (Step 1)	2135	125	n/a
		280.3 (Step 2)	2440	150	n/a
		279.1 (Step 3)	3050	145	n/a
		278.0 (Step 4)	3660	170	n/a

Notes:

1) Working Stress Design, (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Soil Bearing Capacity, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Allowable Soil Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Soil Bearing Resistance (q_n). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Nominal Soil Bearing Resistance (q_n).

3) All footings are to be constructed at or below the elevations listed above in Table No. 1.

4) Bents 2, 3 and 4 footings to be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5)Abutment 1 and 5 footings to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 2002 Standard Specifications.

6) Elevations based on 1988's NAVD datum.

9.2 Anticipated Settlement of Spread Footings

Total settlements were calculated for the proposed abutment and pier footings. Settlement was based on allowable bearing capacities at the abutments and unfactored loads at the bents. The settlement parameters were estimated from generalized soil profiles for each proposed abutment and bent. Table No. 8 gives the approximate total settlements estimated. Due to the granular nature of the underlying granular soils, the anticipated

settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

Table No. 8 – Approximate Immediate Total Settlements - Right Side Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Abut 1	-	125 kPa	10
Bent 2	215 kPa	-	15
Bent 3	240 kPa	-	15
Bent 4	215 kPa		15
Abut 5		170 kPa	10

The existing structure has been in service for over 40 years, since 1963. Therefore, we anticipate that it will experience no more settlements. To alleviate stressing the existing structure and its foundations by the potential vertical settlements of proposed widening, the new foundations and structure should be structurally tied to the existing structure in such a way as to allow vertical displacement between them without lateral separation.

9.3 Lateral Active Earth Pressure

If abutment walls are free to move laterally at the top, a static active lateral earth pressure of 5.7 kPa per meter of depth is recommended. This active lateral earth pressure was calculated using an active earth pressure coefficient of 0.3 and a soil unit weight of 19 kN/m^3.

10.0 CONSTRUCTION CONSIDERATIONS

1. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 or a Sand Equivalent of 20 or more should be used within the approach embankment, in accordance with standard Caltrans requirements.

2. Coarse gravel and cobbles should be expected during excavation of the footing locations and keyways. This material may be removed using conventional excavators and scraper and ripper equipment.

3. On-site material may be used as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material

(greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

4. The new embankment fill should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting).

5. As stated in the Foundation Recommendations, dated August 26, 1991, during the original 1962 construction, undesirable soils, septic tanks and cesspools had been present in the area. Close examination of the bent footing soils is recommended before constructing the footings.

The recommendations contained in this report are based on specific project information that has been provided by the Office of Structure Design. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

11.0 REFERENCES AND CORRESPONDENCES

California Geologic Survey, Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

California Department of Transportation, Corrosion Technology Branch, Corrosion Guidelines, July 2002.

California Geological Survey Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

Caltrans California Seismic Hazard Map 1996, California Department of Transportation, L. Mualchin, 1996.

Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, T.W. Dibblee, 1991.

"Principles of Foundations Engineering", Braja M. Das, Fourth Edition, 1999.

"Foundations Design, Principles and Practices", Donald P. Coduto, Second Edition, 2001.

"Foundations and Earth Structures", Design Manual 7.02, NAVFAC, Naval Facilities Engineering Command, 1986.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

Foundation Recommendations for Osborne Street UC, dated January 8, 1959.

Supplementary Foundation Review for Osborne Street UC, dated March 12, 1959.

Foundation Recommendations for Osborne Street UC Widening, dated August 26, 1991.

Preliminary Seismic Design Recommendations for Osborne Street UC Widening, dated February 27, 2003.

Preliminary Foundation Report for Osborne Street UC Widening, dated April 29, 2003.

If you have any questions or comments, please call Deepa Wathugala at (562) 465-0152, or Ted Liu at or (562) 864-0805.

Prepared by: Date:

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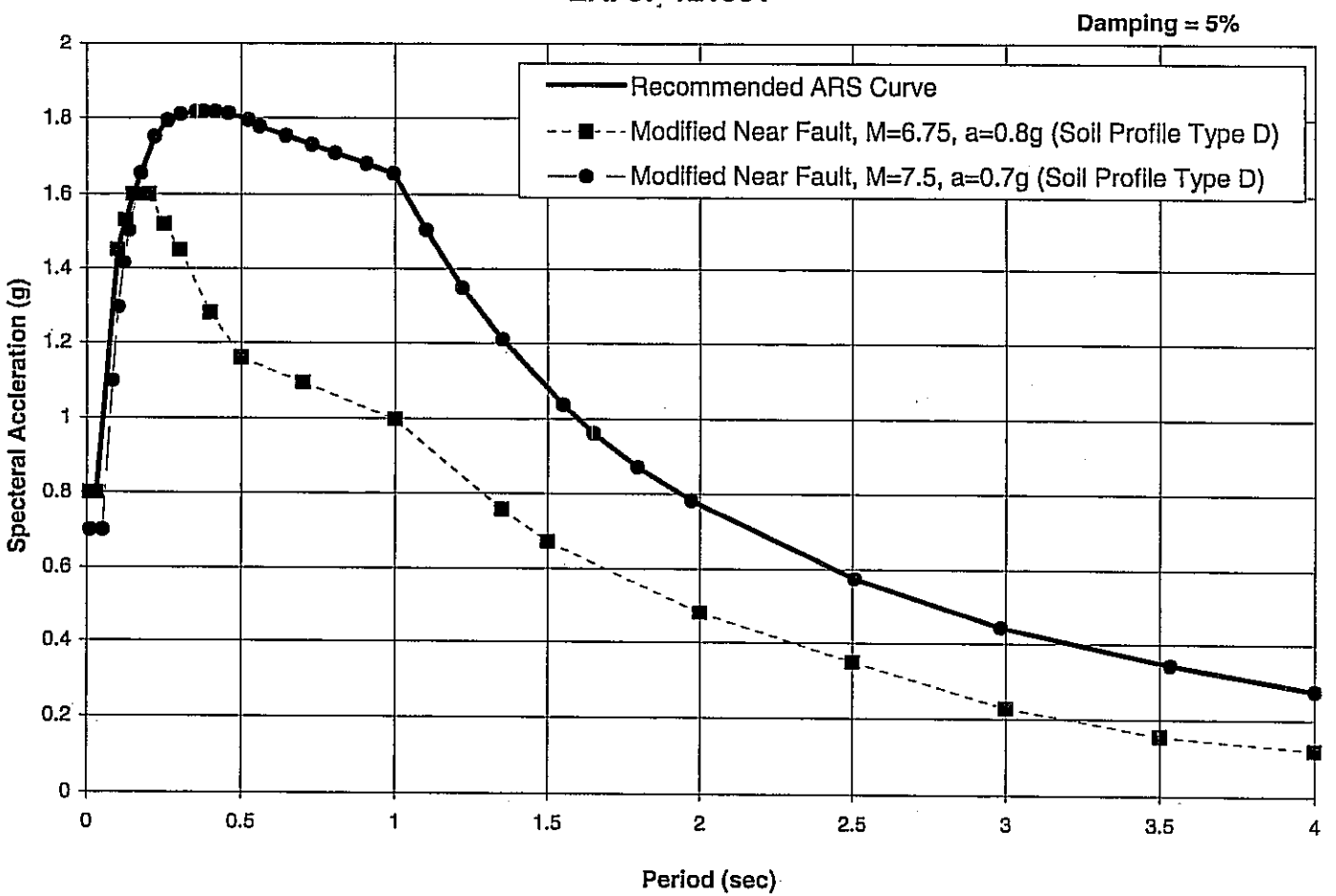
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve for Osborne Street UC
Bridge #53-1124
EA: 07-121901



FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Terra Bella St UC
Structure Name

07 - LA - 05/170 - 67.2
District County Route Post Km

07-121901 53-1125
E.A. Number Structure Number

Foundation Report By: S. Sukiasian

Dated: 9/11/06; 6/21/07

Reviewed By: M Okimura (OSD) R. Price (GS)

General Plan Dated: 1/9/08

Foundation Plan Dated: 1/10/08

[X] No changes. [ ] The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles [ ] Yes [X] No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design — Branch No. 11 — Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: June 21, 2007

File: 07-LA-5-KP 61.15
07-121901
Bridge No. 53-1125
(Terra Bella St. UC Widen)

Attention: Mr. Mark Okimura

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

Subject: Addendum Foundation Report for Terra Bella Street Undercrossing Widening (Bridge No. 53-1125).

1.0 INTRODUCTION

Per your e-mail request dated June 11, 2007, we are providing an Addendum Memo for the Terra Bella Undercrossing Widening Foundation Report dated September 11, 2006. This memorandum summarizes changes made to the foundation data ~~for the Terra Bella Undercrossing Widen Project.~~

Please note that the revised data includes new minimum footing widths for steps 2 through 4 of Abutment 1 and new bottom of footing elevations of Abutment 1-Step 3, Pier 2 and Pier 3. Based on the data provided by the Bridge Design Branch 11, the new foundation data is summarized in the following tables, which will replace Table No's. 2 and 7 respectively in the September 11, 2006 Report.

Table No. 2. Required Spread Footing Data

Support Location		Minimum Footing Width (m)	Bottom of Footing Elevation (m)	Required Bearing (kPa)		Controlling Load Group (4)
				WSD Method (2)	LFD Method (3)	
				Q_{all}	Q_n	
Abut. 1	Step 1	1830	288.4 (6)	185	N/A	N/A
	Step 2	2440 (8)	287.5	185	N/A	N/A
	Step 3	3050 (8)	286.3 (7)	200	N/A	N/A
	Step 4	3965 (8)	284.78 (6)	230	N/A	N/A
Pier 2		1525	284.3 (1) (7)	N/A	880	I-VI
Pier 3		1525	284.3 (1) (7)	N/A	890	I-VI
Abut. 4		1830	288.521	190	N/A.	N/A

Note:
(1) Preferred-footing elevations with obtainable bearing pressure.
(2) Required allowable bearing capacity, q_{all} = unfactored q_{max}
(3) Required nominal bearing resistance, q_n = factored q_{max}/ϕ
(4) Group I-VI Loading, $\phi = 0.5$
(5) Elevations based on 1988's NAVD Datum.
(6) Revised Footing Elevations 9-11-06.
(7) Revised Footing Elevations 6-11-07.
(8) Revised Footing Widths 6-11-07.

Table No. 7. Recommended Spread Footing Data

Support Location		Minimum Footing Width (m)	Bottom of Footing Elevation (3) (m)	Recommended Soil Bearing Pressures	
				WSD Method (1)	LFD Method (2)
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut. 1	Step 1	1830	288.4	185	N/A
	Step 2	2440	287.5	185	N/A
	Step 3	3050	286.3	200	N/A
	Step 4	3965	284.78	230	N/A
Pier 2		1525	284.3	N/A	880
Pier 3		1525	284.3	N/A	890
Abut. 4		1830	288.521	190	N/A

Notes: 1) Working Stress Design (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended 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}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Bearing Resistance (q_n). q_n = factored q_{max}/ϕ.

3) All footings are to be constructed at or below the elevations listed above in Table No. 4.

4) Footings for Piers 2 and 3 will be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5) Abutment 1 and 4 footings are to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 1999 Standard Specifications.

If you have any questions, please call Sam Sukiasian at (213) 620-2135 or Chi-Tseng "Ted" Liu at (213) 620-2136.

Prepared by: Date:

Supervised by: Date: 6/21/07

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CHI-TSENG "TED" LIU, Ph.D., G.E.,
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Office of Geotechnical Design – South 1

cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

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State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: September 11, 2006

File: 07-LA-5-KP 61.15
07-121901
Bridge No. 53-1125
(Terra Bella St. UC Widen)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for Terra Bella Street Undercrossing Widening (Bridge No. 53-1125).

1.0 INTRODUCTION

This Memorandum provides foundation recommendations for the Terra Bella Street Undercrossing Widening in Los Angeles County (Bridge No. 53-1125). The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on two borings drilled for this project. The foundation recommendations in this report are based on the latest abutment and bent locations and their associated structural loads. The footing loads and plans were provided by your office on October 25, 2005. Finally, this memorandum is preceded by and associated with the Terra Bella Bridge Widening Preliminary Foundation and Seismic Design Reports, dated May 19th and February 28th, 2003, respectively.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, by prepared by the Office of Geotechnical Design South 1 – Branch C, dated May 19, 2003, the existing bridge was built in 1963 as a 45.1 meter long and 57.2 meter wide three-span structure with reinforced concrete open-ended diaphragm abutments. In 1994 the bridge was widened on its Westside by 7.4 meters. The existing bents and abutments are founded on continuos spread footings. Abutment end slopes are graded at 1V: 1.5 to 2.0H (Vertical to Horizontal). The Table below summarizes the existing bottom footing elevations of the bents and abutments.

Table No. 1 – Existing Foundation Summary

Existing Location	Foundation Type (Design Load)	Footing Bottom Elevation (m)
Abutment 1	Continuous Footing (190 kPa)	+288.1
Bent 2	Continuous Footing (240 kPa)	+282.0
Bent 3	Continuous Footing (240 kPa)	+282.0
Abutment 4	Continuous Footing (190 kPa)	+288.4

Note: Elevations based on 1929's NGVD and converted to metric units. However, they have not been updated to the current 1988's NAVD Datum.

2.2 Proposed Widening

The proposed widening will involve an increase of 12.5 to 15.5 meters on the Eastside of the bridge. The abutments will be seat type supports. The bents and abutments will be supported on spread footings.

The Table summarizes proposed footing data provided by the Office of Structure Design Branch 11 to our office on October 25, 2005. The Table summarizes design loads nominal loads, and spread-footing widths and elevations.

Table No. 2. Required Spread Footing Data

Support Location		Minimum Footing Width (m)	Bottom of Footing Elevation (m)	Required Bearing (kPa)		Controlling Load Group (4)
				WSD Method (2) Q_{all}	LFD Method (3) Q_n	
Abut. 1	Step 1	1830	288.4 (6)	185	N/A	N/A
	Step 2	2140	287.5	185	N/A	N/A
	Step 3	2740	286.2	200	N/A	N/A
	Step 4	3350	284.78 (6)	230	N/A	N/A
Pier 2		1525	285.66 (1)	N/A	880	I-VI
Pier 3		1525	284.09 (1)	N/A	890	I-VI
Abut. 4		1830	288.521	190	N/A	N/A

Note: (1) Preferred-footing elevations with obtainable bearing pressure.
(2) Required allowable bearing capacity, q_{all} = unfactored q_{max}
(3) Required nominal bearing resistance, q_n = factored q_{max}/ϕ
(4) Group I-VI Loading, $\phi = 0.5$
(5) Elevations based on 1988's NAVD Datum.
(6) Revised Footing Elevations 9-11-06.

3.0 FIELD EXPLORATION PROGRAM

Two borings were drilled for this investigation. The borings were drilled at the proposed Pier 2 and Pier 3 widen locations approximately in-line with the existing Piers 2 and 3 on the eastside of the bridge. The borings drilled for this report were completed on June 29th, 2003. The borings were drilled to a depth of up to 18.4 meters. The borings were advanced utilizing the mud rotary method with CME-85 drill rig. The location of the borings is summarized in the Table No. 3 below. The boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's are presently being prepared by the Office of Geotechnical Support and will be submitted to the Office of Structure Design.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset, m	Elevation[2], m
B-1	Proposed Pier 2	610+80.363	41.40R	285.95
B-2	Proposed Pier 3	611+17.405	62.95R	286.67

Note: 1. Stationing according to CL5A2 Line.
2. Elevations are Above Mean Sea Level (MSL) and based on 1988's NAVD Datum.

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on February 10, 2004.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). At the completion of the borings the holes were backfilled with bentonite cement chips.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT, and bulk samples from the borings. Laboratory testing included mechanical analysis and corrosivity. Samples submitted for testing were analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). Grain size curves developed from the mechanical analysis results are shown in the Appendix. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643.

Table No. 4 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 201, 202, 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 6.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project.

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 1.0 kilometer north of the proposed project (Please see also Section 6.0, Seismicity).

5.3 Subsurface Conditions

Subsurface conditions at the proposed abutment and pier locations were determined based on the two borings drilled for this project. 4 meters of medium dense silty to gravelly sand fill was encountered at boring B-1. No fill was encountered at the B-2 location. The native material consists of medium dense sand with gravel and some cobbles in the upper 3 meters. This is underlain by dense to very dense sand to silty sand with increasing gravels and cobbles to the full depth explored (18.4 meters). The elevation of the dense to very dense material encountered was at +279 meters MSL at B-1, near Pier 2 and +283 meters MSL at B-2, near Pier 3.

5.3.1 Groundwater

Groundwater was not encountered in any of the borings. The deepest boring was B-1, which was drilled to an elevation of approximately +267.55 meters. Historical groundwater information is limited, however, the Los Angeles Department of Public works well No. 4865, located approximately 650 meters to the southwest, had a reading on April 16th 1984 that was at an elevation of 184.7 meters above mean sea level (MSL) or 92.2 meters below the ground surface.

Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from 160 to 175 meters (100 to 115 meters depth below ground surface). Historically in the past several decades water elevations nearby the proposed project have been measured at an elevation of approximately 244 meters (40 meters depth below ground surface). For this project the highest anticipated groundwater should be estimated at elevation +244 meters.

6.0 SEISMICITY

As noted in the Preliminary Seismic Design Foundation Recommendations, dated February 28, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Semi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table below summarizes the Moment Magnitude (Mw) of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.0	NE	0.8g
Northridge Hills, SSN	Reverse-Oblique	7.5	3.7	NW	0.7g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	5.1	NE	0.7g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.8g (VDO Fault) and Mw of 7.5 and PBA of 0.7g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 seconds. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 6.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 100 feet (see Section 5.3.1).

7.0 SCOUR EVALUATION

Due to the absence of a channel running under the bridge and concrete and asphalt covering the road below of the bridge, the potential for scour is considered low to remote.

8.0 CORROSION EVALUATION

Soil samples taken at 3 to 4.5 meter depth at the subject bridge site was tested in the laboratory for corrosion potential. The test result is given in Table No. 6 and indicates that the soil at the site is non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-2	3.0 – 4.6 (10-15 ft)	Pier 3 EB	8.32	9000	NA	NA

Note: For corrosion definitions refer to "Memo to Designers" 3-1.
* 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.

Based on the test result and the consistency of the soils at the site, the site is generally not considered corrosive to buried metal and concrete.

9.0 FOUNDATION RECOMMENDATIONS

The proposed abutments and bents for the new Terra Bella Bridge widening may be supported on spread footings. Section 9.1 provides detailed recommendations on the spread footings for the abutment and bent locations. Lateral resistance and potential settlement of the abutments and piers is discussed in Sections 9.2 and 9.3, respectively.

9.1 Bearing Capacity

Allowable bearing capacity was calculated using Terzaghi's equation with a friction angle and cohesive values developed from the soil profiles based on the LOTB. Table 7 summarizes the minimum spread footing dimensions and the corresponding gross allowable and ultimate bearing capacity for the proposed abutment and pier locations.

Table No. 7. Recommended Spread Footing Data

Support Location		Minimum Footing Width (m)	Bottom of Footing Elevation (3) (m)	Recommended Soil Bearing Pressures	
				WSD Method (1)	LFD Method (2)
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut. 1	Step 1	1830	288.4	185	N/A
	Step 2	2140	287.5	185	N/A
	Step 3	2740	286.2	200	N/A
	Step 4	3350	284.78	230	N/A
Pier 2		1525	285.66	N/A	880
Pier 3		1525	284.09	N/A	890
Abut. 4		1830	288.521	190	N/A

Notes: 1) Working Stress Design (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended 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}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Bearing Resistance (q_n). q_n = factored q_{max}/ϕ.

3) All footings are to be constructed at or below the elevations listed above in Table No. 4.

4) Footings for Piers 2 and 3 will be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5) Abutment 1 and 4 footings are to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 1999 Standard Specifications.

9.2 Anticipated Settlement of Spread Footings

Total and differential settlements were calculated for the proposed abutment and pier footings. Settlement was based on allowable bearing capacities at the abutments and unfactored loads at the piers. The settlement parameters were estimated from generalized soil profiles for each proposed abutment and pier. Table No. 8 summarizes the estimated immediate total settlements for each abutment and pier location. Due to the granular nature of the underlying granular soils, the anticipated settlements will occur shortly upon the application of loads. As is shown in the Table No. 8, long-term total and differential settlements are expected to be negligible.

Table No. 8 – Summary of Anticipated Settlement

Location	Footing Load – Allowable Bearing Capacity (kPa)	Footing Load – Unfactored (kPa)	Anticipated Settlement (mm)
Abut. 1 Step 1-4	230(1)		15
Pier 2		880	20
Pier 3		890	15
Abut 4	190		10

Note: (1) Maximum load for Steps 1 through 5

The existing structure has been in service for over 40 years, since early 1960's. Therefore, we anticipate that it will experience no more settlements. To alleviate stressing the existing structure and its foundations by the potential vertical settlements of proposed widening, the new foundations and structure should be structurally tied to the existing structure in such a way as to allow vertical displacement between them without lateral separation.

9.3 Lateral Active Earth Pressure

If abutment walls are free to move laterally at the top, an active lateral earth pressure of 5.7 kPa per meter of depth is recommended. This active lateral earth pressure was calculated using an active earth pressure coefficient of 0.3 and a soil unit weight of 18.8 kN/m^3.

10.0 CONSTRUCTION CONSIDERATIONS

1. Coarse gravels and cobbles should be expected during excavation of the footing locations and keyways. This material may be removed using conventional excavators and scraper and ripper equipment.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the July 2002 Caltrans Standard Specifications. Soils should be predominantly Soils with an Expansion Index of less than 50 and/or a Sand Equivalent of 20 or more. These soils should be used within the approach embankment, in accordance with standard Caltrans requirements.

3. On-site material is suitable for use as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material (greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

4. The new embankment fill should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting) of the July 2002 Standard Specifications.

5. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 Branch C to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents.

11.0 REFERENCES

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Foundation Report for the Terra Bella Street Undercrossing Widening", Terra Bella Street Undercrossing (53-1125), EA 07-121901, May 19, 2003.

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Seismic Design Recommendations for the Terra Bella Street Undercrossing Widening", Terra Bella Street Undercrossing (53-1125), EA 07-121901, February 28, 2003.

California Department of Transportation, Corrosion Technology Branch, "Corrosion Guidelines", updated to July 2002.

California Geological Survey (formerly California Division of Mines and Geology), State of California Special Studies Zones, map for San Fernando Quadrangle, 1979.

Dibblee, T.W., Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, 1991.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date: 9-11-06

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cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

REGISTERED PROFESSIONAL ENGINEER
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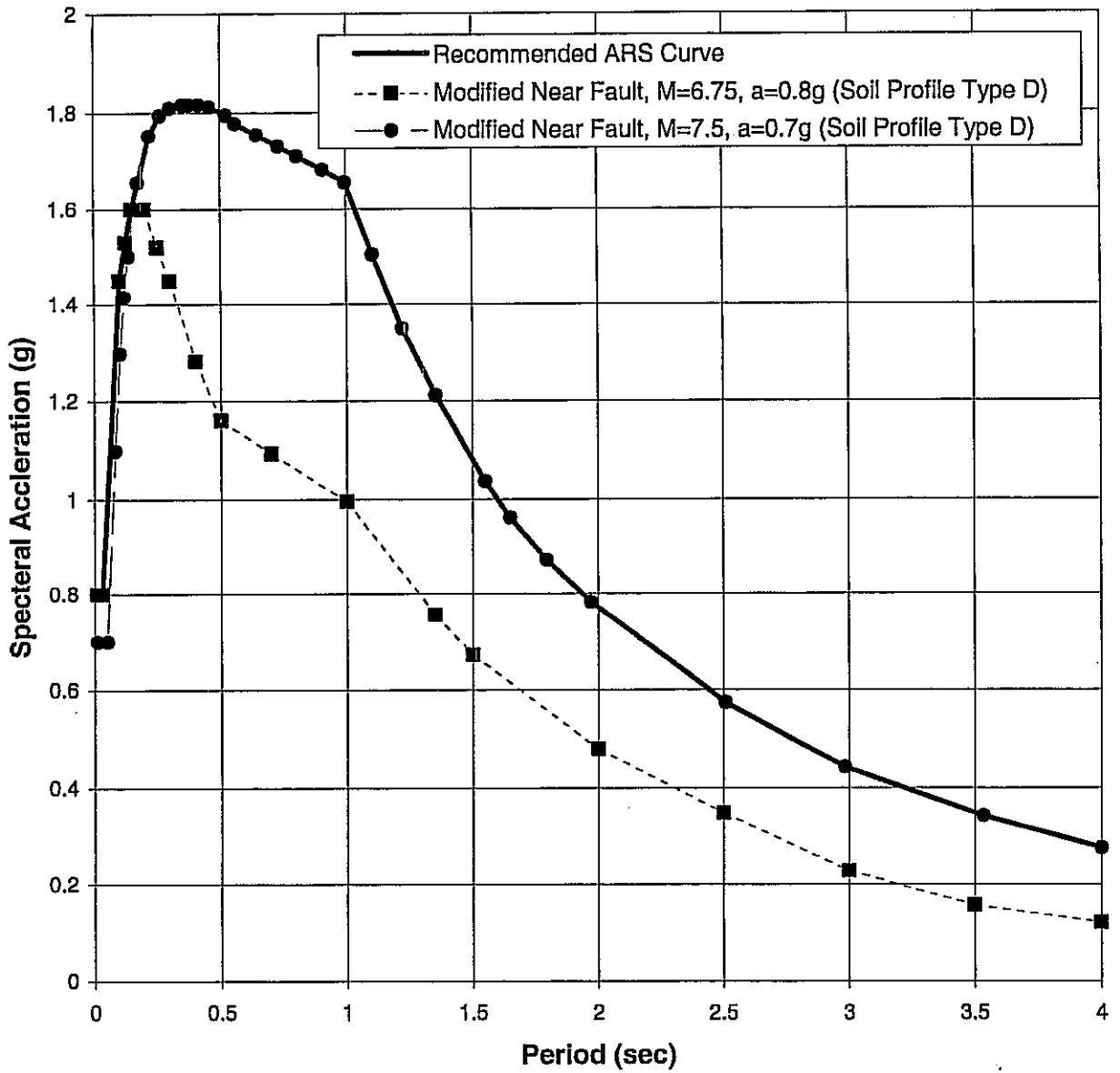
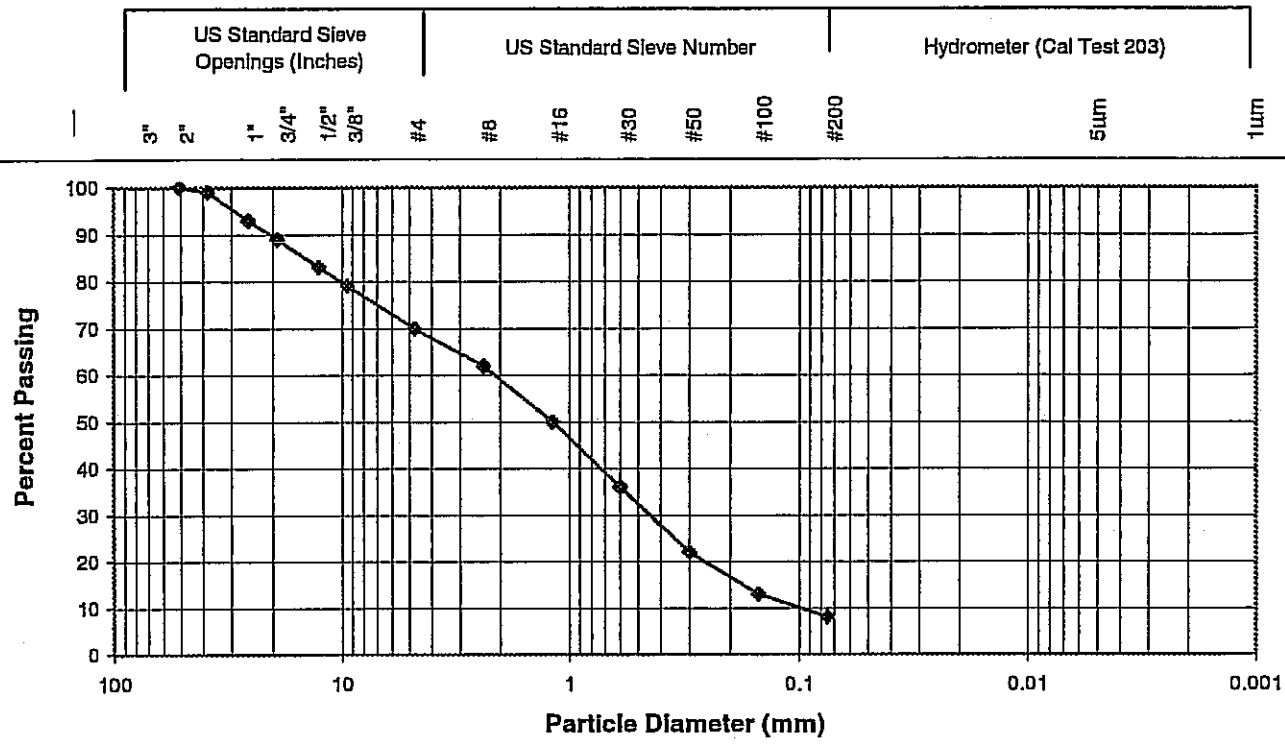
STATE OF CALIFORNIA

Figure 1. ARS Curve for Terra Bella Street UC
Bridge #53-1125
EA-07-121901



Appendix – Gradation Test Results

Gradation Analysis Test Results



Sample ID: B-2@3m

COBBLES	GRAVELS		SANDS			SILT	CLAY
	Coarse	Fine	Crse.	Medium	Fine		

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	Terra Bella St. UC Widen
EA:	07-121901
Dist-Co-Rte-KP:	07-LA-5-KP 58.0/63.4

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Arleta-Sheldon UC
Structure Name

07 - LA - ~~05~~/170 - 32.3
District County Route Post Km

07-121901 53-1763
E.A. Number Structure Number

Foundation Report By: D Wathugala Dated: 8/16/06

Reviewed By: M Okimura (OSD) R. Price (GS)

General Plan Dated: 11/27/07 Foundation Plan Dated: 11/27/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- ✓ Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design Branch No. Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: August 16, 2006

File: 07-LA-5, KP-R 28.97/R32.99
EA: 07-121901
Bridge No. 53-1763
(Arleta-Sheldon U.C. Widening)

Attn: Mr. Jose Higareda

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

Subject: Foundation Report for Arleta-Sheldon Undercrossing Widening (Bridge No. 1763)

1.0 INTRODUCTION

This Memorandum provides foundation recommendations for the Arleta-Sheldon Undercrossing Widening in Los Angeles County (Bridge No. 53-1763). The widening of the bridge is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on the one boring drilled for this project and the review of the as-built Log of Test Boring (LOTB) dated April 17,1967.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation Report, the existing Arleta-Sheldon Undercrossing was built in 1968 in Los Angeles. This bridge is a six- span (southbound) and eight-span (northbound) structure of reinforced concrete box girder with open-end diaphragm abutments and solid piers and 2, 3 and 6 column bents. The abutments and all bents are supported on spread footings. Table No. 1 is a summary of as-built foundation information. The end spans are enclosed by PCC curtain walls. The bridge is 132.6 m (435 feet) and 147.0 m (482 feet) long in the southbound and northbound direction, respectively, and 36.1 m (118.3 feet) wide. Abutment end slopes were graded roughly at 1V: 1.5H to 2.0H (Vertical: Horizontal).

Table 1A - As-built Spread Foundation Information (Southbound, Left Bridge)

Location	Foundation Type (Design Load)	Footing Bottom Elevation (m)
Abut 1L	Continuous footing (190 kPa)	+259.0
Bent 2L	Continuous footing (290 kPa)	+253.66
Bent 3L	Square footing (360 kPa)	+253.66
Bent 4L	Square footing (360 kPa)	+253.66
Bent 5L	Square footing (360 kPa)	+253.66
Bent 6L	Continuous footing (290 kPa)	+253.66
Abut 7L	Continuous footing (190 kPa)	+258.69

Table 1B - As-built Spread Foundation Information (Northbound, Right Bridge)

Location	Foundation Type (Design Load)	Footing Bottom Elevation (m)
Abut 1R	Continuous footing (190 kPa)	+ 257.16
Pier 2R	Continuous footing (290 kPa)	+253.66
Bent 3R	Square footing (290 kPa)	+253.66
Bent 4R	Square footing (290 kPa)	+253.66
Bent 5R	Square footing (290 kPa)	+253.66
Bent 6R	Square footing (290 kPa)	+253.66
Bent 7R	Square footing (290 kPa)	+253.66
Pier 8R	Continuous footing (290 kPa)	+253.66
Abut 9R	Continuous footing (190 kPa)	+259.45

Notes: Elevations were based on 1929's NGVD and converted into metric units. However, they have not been updated to the current 1988's NAVD datum.

2.2 Proposed Widening

The proposed construction will include the widening the median (gap closure) and the widening of the northbound (right side) within the bridge structure.

The center widening will consist of CIP/RC box girders supported on Bents 5, 6, 7, Pier 8 and Abutment 9. The Bent 5, 6, and 7 will be supported on spread square footings. The Pier 8 and Abutment 9 will be supported on spread continuous footings.

The right side widening will consist of CIP/RC box girders at spans 1, 4, 5 and 8, and PC/PS girders with concrete slab at spans 2, 3, 6 and 7. The bents 3, 4, 5, 6 and 7 will be

supported on spread square footings. The Pier 2, Pier 8 and the abutments will be supported on spread continuous footings.

The information on spread footing received from the Office of Bridge Design is given in Table 2A and Table 2B.

Table No. 2A – Spread Footing Data – Right Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Abut 1	WSD	257.7	1500	450	135	n/a	n/a
Pier 2	LFD	254.2	2600	600	n/a	690	VII
Bent 3	LFD	253.8	4600 X 4600	900	n/a	865	VII
Bent 4	LFD	254.2	5350 X 5350	1070	n/a	855	VII
Bent 5	LFD	254.2	5350 X 5350	1070	n/a	815	VII
Bent 6	LFD	254.2	5350 X 5350	1070	n/a	750	VII
Bent 7	LFD	254.2	4600 X 4600	900	n/a	705	VII
Pier 8	LFD	254.2	2600	600	n/a	705	VII
Abut 9	WSD	260.0	1500	450	130	n/a	n/a

Table No. 2B – Spread Footing Data – Center Widen

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Footing Thickness (mm)	Required Bearing (kPa)		Controlling Load Group
					q_{all} (1)	q_n (2)	
Bent 5	LFD	254.2	4000 X 4000	900	n/a	560	VII
Bent 6	LFD	254.2	4600 X 4600	900	n/a	785	VII
Bent 7	LFD	254.2	4600 X 4600	900	n/a	660	VII
Pier 8	LFD	254.2	2600	600	n/a	720	I-VI
Abut 9 – Left Half	WSD	259.2	1500	450	110	n/a	n/a
Abut 9 – Right Half	WSD	260.0	1500	450	110	n/a	n/a

Note: (1) Required allowable bearing capacity, q_{all}= unfactored q_{max}
(2) Required nominal bearing resistance, q_n= factored q_{max}/ϕ; Groups I-VI Loading, $\phi = 0.5$ and Group VII, $\phi = 1.0$
(3) Elevations based on 1988's NAVD datum.

3.0 FIELD EXPLORATION PROGRAM

One boring was drilled for this investigation on July 22, 2003. The boring was advanced utilizing hollow-stem auger method with a CEM-85 drill rig with a automatic trip hammer. These borings were drilled to an approximate depth of 18 meters. The location of the boring is summarized in Table No. 3 below.

Table No. 3 – Summary of Boring Locations

Boring	Location	Station [1]	Offset	Elevation[2], m
B-1	Bent 7	324+58.468	5.41L	256.30

Notes: 1. Reference line is Ciil Line.
2. Elevations based on 1988's NAVD datum.

The survey results were recorded and provided by District 7 Surveys on February 10, 2004.

An engineering geologist from Geotechnical Design South-1 Branch C logged and sampled the borings using a Standard Penetration Test (SPT) sampler at 1.52-meter (5-foot) intervals. The SPT samples were driven using a 63.6-kilogram hammer falling freely for 762 mm for a total penetration of 457 mm. In addition, representative bulk samples were collected from borings. Following drilling, sampling, and logging, the borings were backfilled with bentonite chips.

LOTB's (Log of Test Borings) are being prepared by Contracts, Graphics & Records of Office of Geotechnical Support and will be submitted to the Office of Structure Design.

4.0 LABORATORY TESTING

Laboratory testing was performed on a bulk sample from the borings. Laboratory testing included corrosion test. Sample submitted for testing was analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). A summary of corrosion test results is presented in Section 8.0.

Table No. 4 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 6.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. Fill ranges in thickness up to approximately 10 meters. 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 2.2 kilometers north of the proposed project (Please see also Section 6.0, Seismicity).

5.3 Subsurface Conditions

Subsurface soil conditions at the proposed abutment and bent locations were determined based on the boring drilled for this project project and the review of the as-built Log of Test Boring (LOTB) dated 4/17/67. The top 16 meters of native alluvium (approximate elevation +256 to +250) is composed of medium dense to very dense sands and gravelly sands with cobbles. This upper layer was underlain by dense to very dense sand with increasing gravel and cobbles to the elevation explored (approximately +245). Native soil is occasionally interbedded with silty sand.

5.3.1 Groundwater

Groundwater was not encountered in the boring, which was drilled to an elevation of approximately +235 meters. 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 well number 4897G, located approximately 0.1 km west of the project site, had a maximum reading from 1993 to 2002 as an elevation of +211 meters above mean sea level (MSL).

6.0 SEISMICITY

As noted in the Preliminary Foundation Recommendations, dated May 21, 2003, the bridge site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed bridge site. However, as stated in the Preliminary Report, the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are two nearby causative faults that should also be considered in the design of the proposed bridge.

The Table No. 5 summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration is based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	2.2	NE	0.7g
Simi/Santa Rosa-Northridge Hills, SSN	Reverse-Oblique	7.5	7.2	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	7.0	NE	0.6g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the Mw of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSD Fault or SSN Fault). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater

than 1.0 second. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1, the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated, fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Due to the dense nature of the granular soils beneath the proposed bent and abutment locations and the absence of groundwater within the zone of influence, the potential for the occurrence of liquefaction at the site is considered low to remote.

7.0 SCOUR EVALUATION

Due to the absence of a channel running under the bridge and concrete and asphalt covering the bottom of the bridge, the potential for scour is considered low to remote.

8.0 CORROSION EVALUATION

A soil sample from Boring No. B-1 was tested at the District 7 Transportation Laboratory in Los Angeles for corrosion potential. The test results summarized in Table No. 6 indicate that the soil at the site is generally non-corrosive to metal and reinforced concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 6 - Corrosion Test Results

Boring	Sample Depth (m)	Support Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1	1.5–3.0	Bent 7	7.76	10000	N/A	N/A

Note: For corrosion definitions refer to "Memo to Designers" 3-1.

* 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.

9.0 FOUNDATION RECOMMENDATIONS

The proposed abutment and bent locations may be supported on spread footings. Recommended bearing capacities and estimated settlements are given in the following sections. Spread footing sizes are summarized in Table 7.

9.1 Bearing Capacity

Ultimate bearing capacity values were calculated using Terzaghi's equation with a friction angle and cohesive values developed from the soil profiles for the northbound and southbound abutment and bent/pier locations. The Tables 7 and 8 below summarize the minimum spread footing dimensions and the corresponding Allowable Soil Bearing Capacity and Nominal Soil Bearing Resistance for the proposed abutment and bent locations.

Table No. 7 – Spread Footing Data – Right Widening

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Abut 1	WSD	257.7	1500	135	n/a
Pier 2	LFD	254.2	2600	n/a	690
Bent 3	LFD	253.8	4600 X 4600	n/a	865
Bent 4	LFD	254.2	5350 X 5350	n/a	855
Bent 5	LFD	254.2	5350 X 5350	n/a	815
Bent 6	LFD	254.2	5350 X 5350	n/a	750
Bent 7	LFD	254.2	4600 X 4600	n/a	705
Pier 8	LFD	254.2	2600	n/a	705
Abut 9	WSD	260.0	1500	130	n/a

Table No. 8 – Spread Footing Data – Center Widening

Location	Design Method	BOF Elevation (m)	Minimum Footing Width (mm)	Recommended Bearing Limits (kPa)	
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Resistance (q_n)
Bent 5	LFD	254.2	4000 X 4000	n/a	560
Bent 6	LFD	254.2	4600 X 4600	n/a	785
Bent 7	LFD	254.2	4600 X 4600	n/a	660
Pier 8	LFD	254.2	2600	n/a	720
Abut 9 (Left Half)	WSD	259.2	1500	110	n/a
Abut 9 (Right Half)	WSD	260.0	1500	110	n/a

Notes: 1) Working Stress Design, (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended Allowable Soil Bearing Capacity, (q_{all}). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed 3 times the recommended Allowable Soil Bearing Capacity, (q_{all}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Soil Bearing Resistance (q_n). The Ultimate Soil Bearing Capacity, (q_{ult}), will equal or exceed the recommended Nominal Soil Bearing Resistance (q_n).

3) All footings are to be constructed at or below the elevations listed above in Table No. 1.

4) Bents 2, 3, 4, 5, 6, 7 and 8 footings to be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

5) Abutment 1 and 9 footings to be placed on compacted fill. Fill shall be compacted to 95% Relative Compaction per Section 19-5.03 of the 2002 Standard Specifications.

6) Elevations based on 1988's NAVD datum.

9.2 Anticipated Settlement of Spread Footings

Total settlements were calculated for the proposed abutment, pier and bent footings. Settlement was based on allowable bearing capacities at the abutments and unfactored loads at the piers and bents. The settlement parameters were estimated from generalized soil profiles for each proposed abutment and bent. Table No. 9 and Table No.10 give the approximate total settlements estimated. Due to the granular nature of the underlying granular soils, the anticipated settlements will occur shortly upon the application of loads. The long-term total and differential settlements are expected to be negligible.

Table No. 9 – Approximate Immediate Total Settlements - Right Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Abut 1	-	110 kPa	10
Pier 2	200 kPa	-	20
Bent 3	175 kPa	-	20
Bent 4	175 kPa	-	20
Bent 5	175 kPa	-	20
Bent 6	175 kPa	-	20
Bent 7	175 kPa	-	20
Pier 8	200 kPa	-	20
Abut 9		110 kPa	10

Table No. 10 – Approximate Immediate Total Settlements - Center Widen

Location	Bearing Pressure due to Unfactored Loads Considered for Settlement Computation	Allowable Bearing Capacity Considered for Settlement Computation	Settlement (mm)
Bent 5	160 kPa	-	20
Bent 6	160 kPa	-	20
Bent 7	210 kPa	-	25
Pier 8	240 kPa	-	20
Abut 9 (Left Half)	-	110 kPa	10
Abut 9 (Right Half)	-	110 kPa	10

The existing structure has been in service for over 35 years, since 1968. Therefore, we anticipate that it will experience no more settlements. To alleviate stressing the existing structure and its foundations by the potential vertical settlements of proposed widening, the new foundations and structure should be structurally tied to the existing structure in such a way as to allow vertical displacement between them without lateral separation.

9.3 Lateral Active Earth Pressure

If abutment walls are free to move laterally at the top, a static active lateral earth pressure of 5.7 kPa per meter of depth is recommended. This active lateral earth pressure was calculated using an active earth pressure coefficient of 0.3 and a soil unit weight of 19 kN/m^3.

10.0 CONSTRUCTION CONSIDERATIONS

1. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 or a Sand Equivalent of 20 or more should be used within the approach embankment, in accordance with standard Caltrans requirements.

2. Coarse gravel and cobbles should be expected during excavation of the footing locations and keyways. This material may be removed using conventional excavators and scraper and ripper equipment.

3. On-site material may be used as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material (greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

4. The new embankment fill should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting).

5. As stated in the Foundation Recommendations, dated August 26, 1991, during the original 1962 construction, undesirable soils, septic tanks and cesspools had been present in the area. Close examination of the bent footing soils is recommended before constructing the footings.

The recommendations contained in this report are based on specific project information that has been provided by the Office of Structure Design. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

11.0 REFERENCES AND CORRESPONDENCES

California Geologic Survey, Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

California Department of Transportation, Corrosion Technology Branch, Corrosion Guidelines, July 2002.

California Geological Survey Alquist-Priolo Earthquake Fault Zones of California, Southern Region, San Fernando Quadrangle, 2000.

Caltrans California Seismic Hazard Map 1996, California Department of Transportation, L. Mualchin, 1996.

Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, T.W. Dibblee, 1991.

"Principles of Foundations Engineering", Braja M. Das, Fourth Edition, 1999.

"Foundations Design, Principles and Practices", Donald P. Coduto, Second Edition, 2001.

"Foundations and Earth Structures", Design Manual 7.02, NAVFAC, Naval Facilities Engineering Command, 1986.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

Preliminary Seismic Design Recommendations for Arleta-Sheldon Undercrossing Widening, dated February 19, 2003.

Preliminary Foundation Report for Arleta-Sheldon Undercrossing Widening, dated May 21, 2003.

If you have any questions or comments, please call Deepa Wathugala at (562) 465-0152, or Ted Liu at or (562) 864-0805.

Prepared by: Date:

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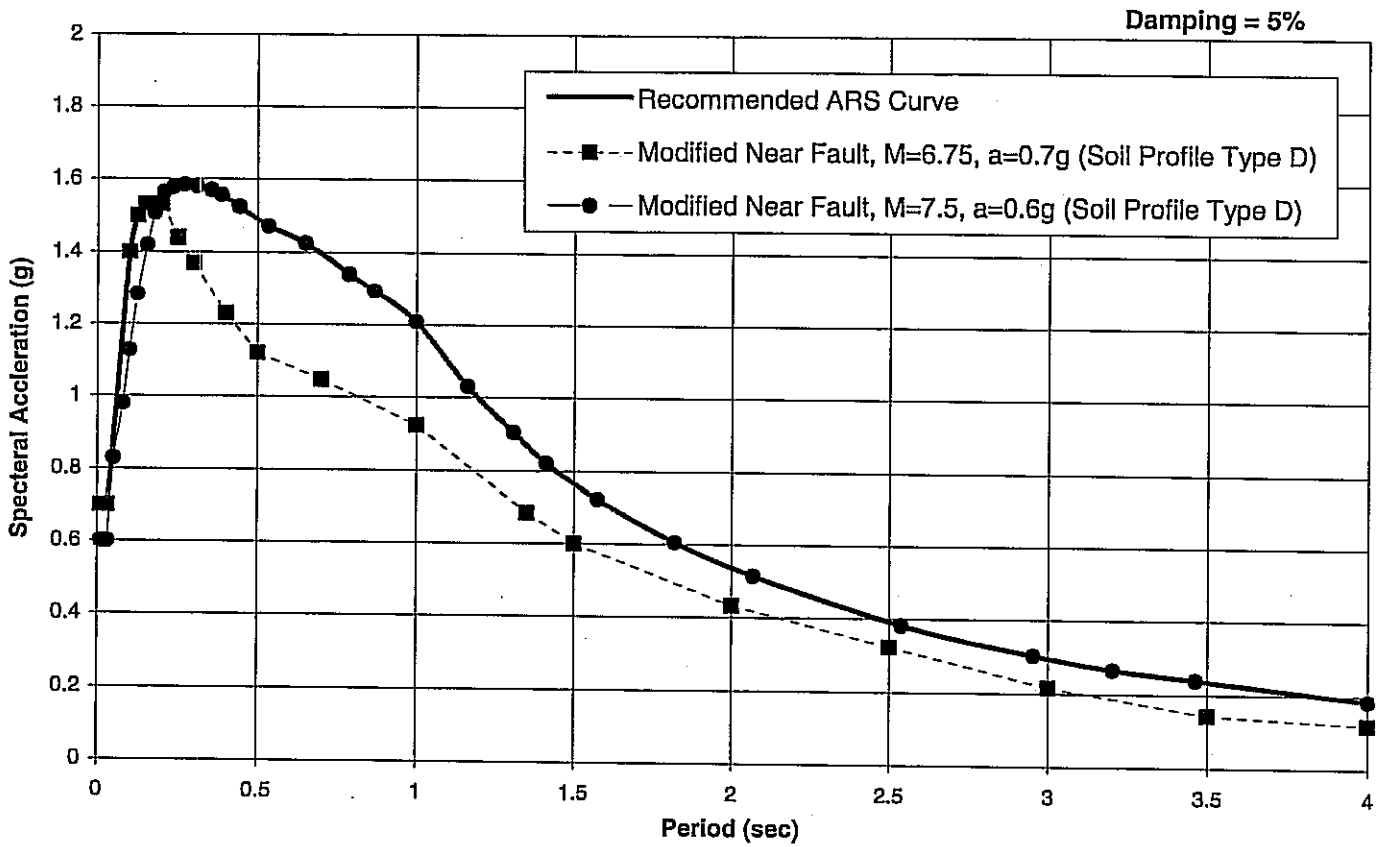
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve for Arleta-Sheldon UC
Bridge #53-1763
EA: 07-121901



FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: Structure Design
1. Preliminary Report
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services
1. GS (Sacramento)
2. GS

District Project Development
District Project Engineer

Date: 1/28/08

Tonopah St. PUC
Structure Name

07 - LA - 05/170-58.6
District County Route Post Km

07-121901 53-1692
E.A. Number Structure Number

Foundation Report By: S. Sukiasian

Dated: 8/7/06

Reviewed By: M. Okimura (OSD) R. Price (GS)

General Plan Dated: 12/6/07

Foundation Plan Dated: 7/16/07

☒ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

- ✓ Pile Types and Design Loads
- ✓ Pile Lengths
- ✓ Predrilling
- ✓ Pile Load Test
- Substitution of H Piles For Concrete Piles ☐ Yes ☒ No
- ✓ Footing Elevations, Design Loads, and Locations
- ✓ Seismic Data
- ✓ Location of Adjacent Structures and Utilities
- ✓ Stability of Cuts or Fills
- ✓ Fill Time Delay
- ✓ Effect of Fills on Abutments and Bents
- ✓ LOTB's
- ✓ Fill Surcharge
- ✓ Approach Paving Slabs
- ✓ Scour
- ✓ Ground Water
- ✓ Tremie Seals/Type D Excavation

Office of Structure Design

11
Branch No.

Geotechnical Services

Rev. 10/02

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design South 1
Bridge Design Branch 11

Date: August 7, 2006

File: 07-LA-5-KP 63.4
07-121901
Bridge No. 53-1692
(Tonopah St. PUC Extension)

Attention: Mr. Jose Higareda

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

Subject: Foundation Report for Tonopah Street Pedestrian Undercrossing Extension (Bridge No. 53-1692).

1.0 INTRODUCTION

This Memorandum provides foundation recommendations for the Tonopah Street Pedestrian Undercrossing (PUC) Extension in Los Angeles County (Bridge No. 53-1692). The extension of the PUC is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on one boring drilled for this project. The foundation recommendations in this report are based on the latest PUC extension dimensions and the associated structural loads. The latest plans were provided by your office on October 25, 2005. Finally, this memorandum is preceded by and associated with the Tonopah PUC Extension Preliminary Foundation and Seismic Design Report, dated May 29th.

2.0 PROJECT DESCRIPTION

2.1 Existing Structure

As is stated in the Preliminary Foundation and Seismic Report, dated May 29, 2003, the existing PUC was built in 1963 and is 135.6-meters long between the headwalls from the northeast end to the southwest end of Tonopah Street. The PUC is a 2.4 by 2.4 meter reinforced concrete rigid frame box structure under 2.6 to 9.1 meters of freeway embankment fill. The existing PUC is founded on a rigid concrete slab.

2.2 Proposed Extension

The proposed widening will involve extending the PUC 21.4 meters on the northeast end of Tonopah Street. The extension will be composed of a 2.4 by 2.4-meter reinforced concrete box with an invert elevation of about 262.8 meters. A maximum height of fill of 4.1 meters will be placed on top of the proposed PUC extension to accommodate the widening of the I-5 freeway. A soundwall (Number 581) will be placed at the end of the PUC. Foundation recommendations for Soundwall No. 581 will be presented in a separate Geotechnical Design Report (GDR). The PUC box extension will be founded on a rigid concrete slab.

The table below summarizes proposed PUC extension foundation data provided by the Office of Structure Design Branch 11 to our office on October 25, 2005.

Table No. 1. Required Foundation Data

Support Location	Design Method	Bottom of Slab Elevation (m)	Minimum Culvert Width (m)	Slab Thickness (m)	Required Bearing (kPa)	
					Q_{all} (1)	Q_n (2)
Abutment 1	LFD	262.6	3.020	230	N/A	292

Note: (1) Required allowable bearing capacity, q_{all} = unfactored q_{max}
(2) Required nominal bearing resistance, q_n = factored q_{max}/ϕ
(3) Elevation based on 1988's NAVD Datum

3.0 FIELD EXPLORATION PROGRAM

One boring was drilled for this investigation. The boring was drilled near the northeast end of the existing PUC entrance. This is the location of the greatest amount of the fill that is proposed to be placed over the PUC extension. The boring drilled for this report was completed on July 8th, 2003 and was drilled to a depth of 15.4 meters. The boring was advanced utilizing the mud rotary method with CME-85 drill rig. The location of the boring is summarized in the Table No. 2 below. The boring location will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTB's are presently being prepared by the Office of Geotechnical Support and will be submitted to the Office of Structure Design.

Table No. 2 – Summary of Boring Location

Boring	Location	Station [1]	Offset, m	Elevation[2], m
B-1	Proposed PUC connection to the existing PUC	585+25.162	65.09R	262.59

Note: 1. Stationing according to CL5A2 Line.
2. Elevations are Above Mean Sea Level (MSL) (1988's NAVD Datum).

Stations, offsets, and elevations of the borings were surveyed by a District 7 Surveys Crew and provided on February 10, 2004.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler at typically 1.52-meter (5-foot) intervals. The SPT samples were driven using a 623 N (140-pound) hammer falling freely for 760-mm (30-inches) for a total penetration of 460-mm (18-inches). At the completion of the borings the holes were backfilled with bentonite cement chips.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT, and bulk samples from the boring. Laboratory testing included mechanical analysis and corrosivity. Samples submitted for testing were analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below). Grain size curves developed from the mechanical analysis results are shown in the Appendix. Corrosivity testing was performed in accordance with Caltrans Test Methods (CTM) 417, 422, and 643.

Table No. 3 – Laboratory Test Methods

Test	Standard
Mechanical Analysis of Soils	CTM 201, 202, 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY

5.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 7.0, Seismicity, of this report.

5.2 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 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 with occasional boulders. Depth to bedrock or bedrock like

The Table below summarizes the Moment Magnitude, M_w, of the Maximum Credible Earthquake (MCE), type of faulting, distance, and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 4 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.6	NE	0.7g
Northridge Hills, SSN	Reverse-Oblique	7.5	5.3	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	6.3	NE	0.6g

6.1 Acceleration Response Spectra Curve

Based on the subsurface soils encountered during the field investigation, the soil profile at the site may be classified as Type D as defined in the Caltrans Seismic Design Criteria (SDC 2001). ARS curves presented in this memo have been generated by means of extrapolation to accurately reflect the M_w of 6.75 and PBA of 0.7g (VDO Fault) and Mw of 7.5 and PBA of 0.6g (SSN and SSD Faults). In addition, due to the proximity of the site to the fault, these ARS curves have been further modified for near fault effects, as discussed in the 2001 SDC. The modifications are such that there is no increase in spectral acceleration for periods less than 0.5 seconds and a 20 percent increase for periods greater than 1.0 seconds. In addition, a linear interpolation was used between 0.5 and 1.0 seconds. On Figure 1 the recommended ARS curve, which is a combination of both modified ARS curves, is represented by a solid black line.

6.2 Liquefaction Evaluation

Liquefaction is a phenomenon in which loose, saturated fine-grained, granular soils behave like a liquid while being subjected to high-intensity ground shaking. Liquefaction occurs when shallow ground water, low-density, fine, sandy soils and high-intensity ground motion exist in a site.

Saturated, loose to medium dense, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential. Using the seismic parameters discussed in Section 6.0 of this memo, liquefaction is unlikely to occur due to the dense to very dense consistency of the granular materials encountered in the soil borings and the absence of groundwater within the top 100 feet.

7.0 CORROSION EVALUATION

A soil sample taken at a 1.5 to 3 meter depth at the boring location drilled for this project was tested in the laboratory for corrosion potential. The test result is given in Table No. 5 and indicates that the soil at the site is non-corrosive to metal and concrete (Corrosion Guidelines, 1996). Caltrans currently defines a corrosive area as an area where the soil and/or water contains more than 500 PPM of chlorides, more than 2000 PPM of sulfates, and a minimum resistivity of less than 1000 ohm-centimeters or has a pH of 5.5 or less.

Table No. 5 - Corrosion Test Results

Boring	Sample Depth (m)	Location	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-1	1.5 – 3.0 (5-10 ft)	Proposed PUC connection to the existing PUC	8.51	16,000	NA	NA

Note: For corrosion definitions refer to "Memo to Designers" 3-1.
* 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.

Based on the test result and the consistency of the soils at the site, the site is generally not considered corrosive to buried metal and concrete.

8.0 FOUNDATION RECOMMENDATIONS

The proposed Tonopah Street PUC extension may be supported on a rigid reinforced concrete slab. Section 8.1 provides recommendations for net allowable bearing capacity for the slab. Sections 8.2 and 8.3 provide discussions on expected potential settlement and at-rest lateral and overburden pressure for Sections 8.2 and 8.3, respectively.

8.1 Bearing Capacity

The recommended bearing capacity was calculated using Terzaghi's equation with a friction angle and cohesive values developed from the soil profile based on the soil boring drilled for this project. Table 6 summarizes the recommended minimum culvert (bottom slab) width for the PUC and the corresponding recommended nominal bearing capacity.

Table No. 6. Recommended Spread Footing Data

Support Location	Bottom of Slab Elevation (3) (m)	Minimum Culvert Width (m)	Slab Thickness (m)	Recommended Soil Bearing Pressures	
				WSD Method (1)	LFD Method (2)
				Allowable Soil Bearing Capacity (q_{all})	Nominal Soil Bearing Capacity (q_n)
Abutment 1	262.6	3.020	230	N/A	292

Notes: 1) Working Stress Design (WSD): The Maximum Contact Pressure, (q_{max}), is not to exceed the recommended 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}).

2) Load Factor Design, (LFD): The Maximum Contact Pressure, (q_{max}), divided by the Strength Reduction Factor, (ϕ), is not to exceed the recommended Nominal Bearing Resistance (q_n). q_n = factored q_{max}/ϕ.

3) The slab foundation is to be constructed at or below the elevation listed above in Table No. 6.

4) The slab foundation will be placed on existing native soil. Exposed soils at the bottom of the footing excavation shall be compacted to 95% Relative Compaction to a minimum depth of 150 mm (0.5 ft) below the bottom of footing elevation.

8.2 Anticipated Settlement

Total and differential settlements were calculated for the proposed PUC extension. The settlement parameters were estimated from the generalized soil profile developed from the soil boring drilled for this project. The estimate for total and differential settlement is based on the recommended nominal bearing capacity of the underlying soils discussed in Section 8.1. Based on this, the maximum immediate total settlement at the invert is expected to be less than 25-mm (less than 1-inch) at the location of boring B-1, where a maximum overburden fill placement of 4.1 meters is estimated. The differential settlement is anticipated to be about 12 mm across the length of the PUC Extension (about 21.4 meters). Due to the granular nature of the underlying granular soils, the anticipated settlements will occur immediately upon the application of loads. The long-term total and differential settlements are expected to be negligible.

The existing structure has been in service for over 40 years, since early 1960's. Therefore, we anticipate that it will experience no more settlements. To alleviate stressing the existing structure and its foundations by the potential vertical settlements of proposed widening, the new foundations and structure should be structurally tied to the existing structure in such a way as to allow vertical displacement between them without lateral separation.

8.3 Vertical and Lateral Earth Pressure

Vertical earth pressure resulting from overburden stress varies along the length of the PUC extension. The location of maximum overburden pressure (4.1-meter height of fill) is located at the end of the existing PUC. The vertical stress on the top of the PUC due to the overburden pressure at this location is 96 kPa. Traffic traveling over this section will produce an additional load of 6 kPa. At about 13.5 meters northeast of the existing PUC entrance, along the PUC extension, the Branford Street Offramp goes passes over the proposed PUC. Here the fill height is about 2.1 meters high with an overburden stress on the top of the PUC of 41 kPa with an additional traffic load of 6 kPa. The overburden pressures are based on a unit weight of 20 kN/m^3.

For lateral earth pressures acting on the sides of the PUC walls, a rigid condition exists. Therefore, the at-rest condition is appropriate for design use. An at-rest horizontal earth pressure of 12 kPa per meter of depth is recommended. A uniform horizontal pressure due to the surcharge from overburden and traffic loads should also be added. These are 12 kPa per meter of depth for the overburden pressure and 6 kPa for the traffic loads. The at-rest and overburden pressures are based on an at-rest earth pressure coefficient of 0.6 and a unit weight of 20 kN/m^3.

9.0 CONSTRUCTION CONSIDERATIONS

1. Coarse gravels and some cobbles should be expected during excavation of the PUC slab foundation. This material may be removed using conventional excavators and scraper and ripper equipment.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the July 2002 Caltrans Standard Specifications. Soils should be predominantly granular with an Expansion Index of less than 50 and/or a Sand Equivalent of 20 or more. These soils should be used within the approach embankment, in accordance with standard Caltrans requirements.

3. On-site material is suitable for use as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material (greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

4. The new embankment fill surrounding the PUC should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting) of the July 2002 Standard Specifications.

5. The roof slab and walls of the PUC should be waterproofed to prevent leakage. Waterproofing the PUC roof and walls should be followed in accordance with Section 54 of the July 2002 Standard Specifications.

6. Recommendations contained in this report are based on specific project information regarding design loads, structure type and support locations that have been provided by the Office of Structure Design. The final construction plans and specifications should be submitted to the Office of Geotechnical Design South 1 Branch C to confirm that the general intent of the recommendations contained in this report have been incorporated into the final construction documents.

10.0 REFERENCES

California Department of Transportation, Office of Geotechnical Design South 1 Branch C, "Preliminary Foundation and Seismic Report for the Tonopah Street Pedestrian Undercrossing (PUC) Extension", Tonopah Street PUC (53-1692), EA 07-121901, May 29, 2003.

California Department of Transportation, Corrosion Technology Branch, "Corrosion Guidelines", updated to July 2002.

California Geological Survey (formerly California Division of Mines and Geology), State of California Special Studies Zones, map for San Fernando Quadrangle, 1979.

Dibblee, T.W., Geologic Map of the San Fernando and Van Nuys (North ½) Quadrangles, 1991.

La County Department of Public Works, Well Measurement Data Website, www.ladpw.org/wrd/wellinfo/, 2006.

A Technical Report to Accompany the Caltrans California Seismic Hazard Map 1996 (Based on Maximum Credible Earthquakes), 1996, by L. Mualchin.

If you have any questions, please call Sam Sukiasian at (562) 864-1565, Christopher Harris at (562) 864-5610, or Chi-Tseng "Ted" Liu at (562) 864-0805.

Prepared by: Date:

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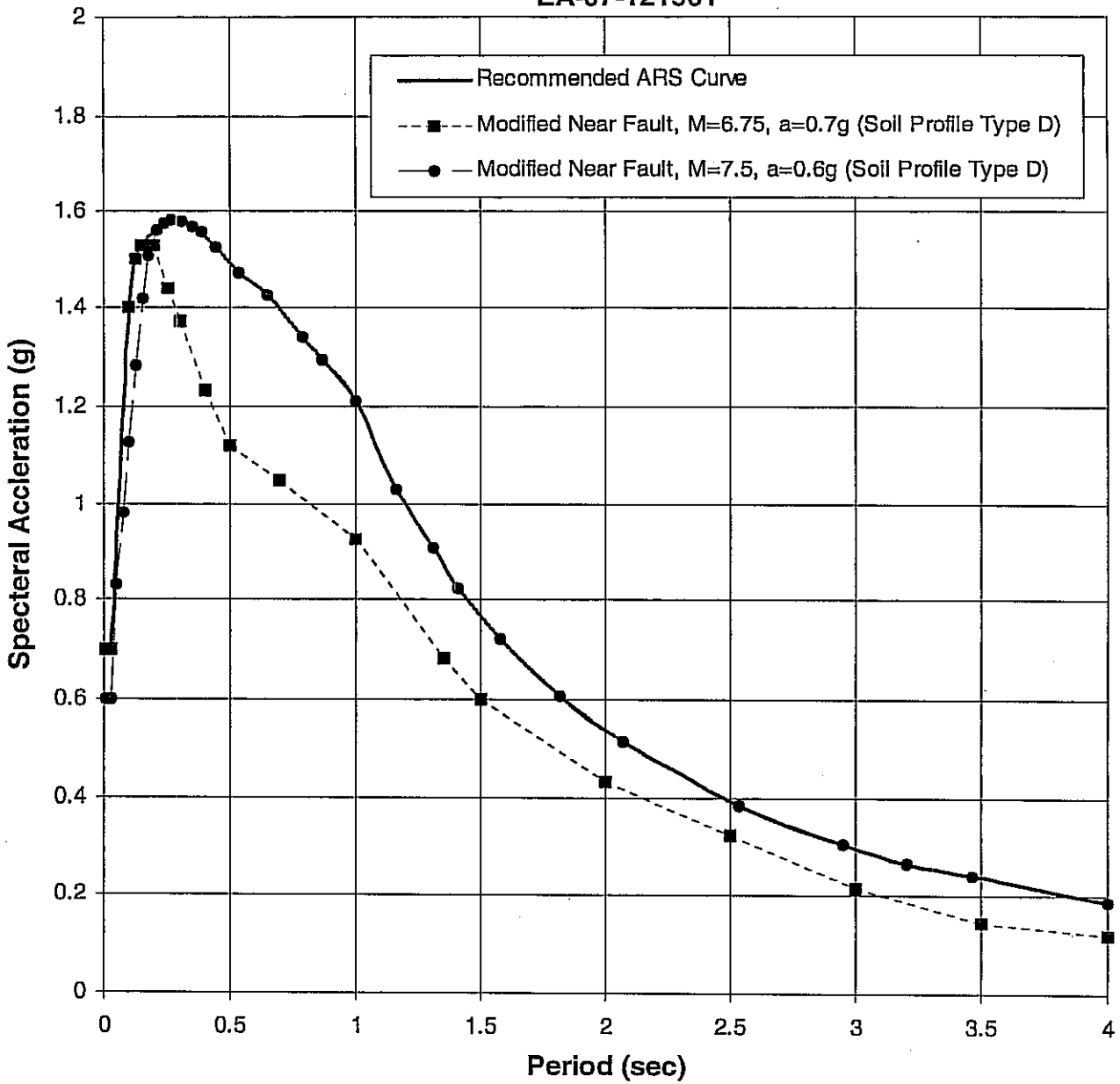
STATE OF CALIFORNIA

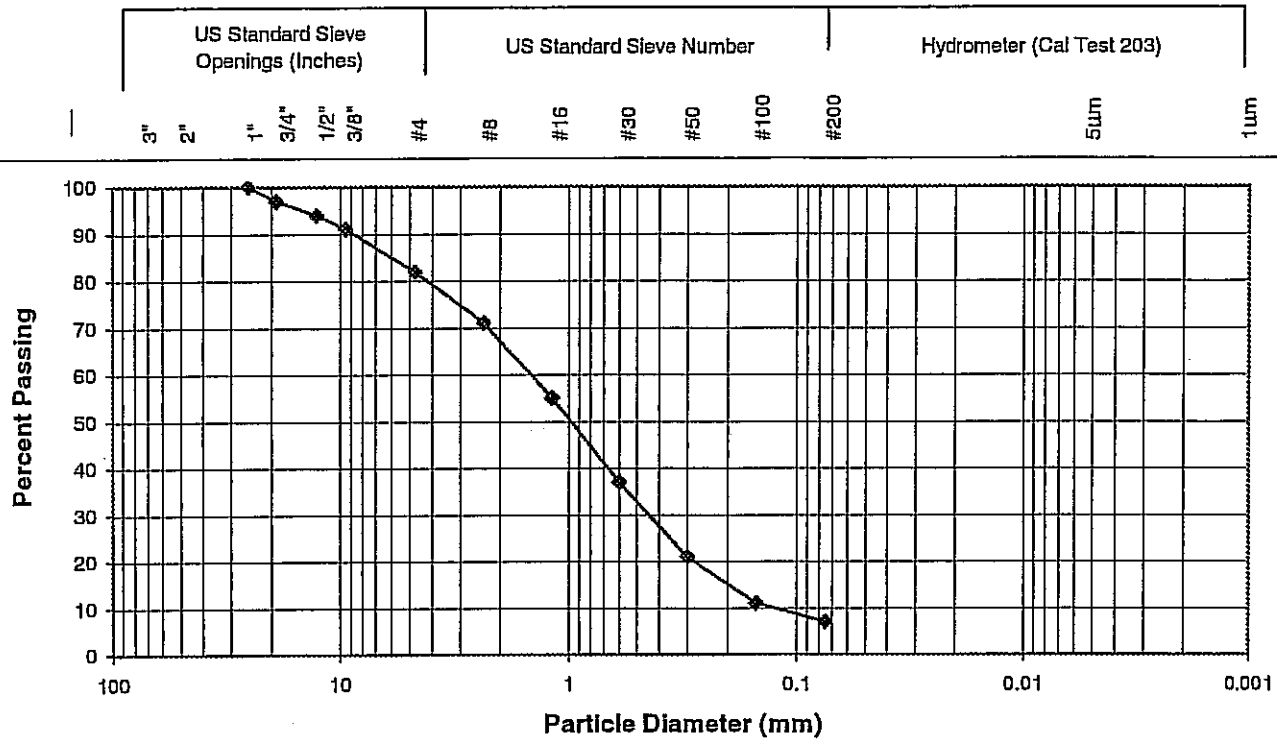
cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)

Figure 1. ARS Curve for Tonopah Street PUC
Bridge #53-1692
EA-07-121901



Appendix – Grain Size Analysis Results

Gradation Analysis Test Results



Sample ID:	B-1@1.5m			

COBBLES	GRAVELS		SANDS			SILT	CLAY
	Coarse	Fine	Crse.	Medium	Fine		

Caltrans

Engineering Services
Division of Geotechnical Services
Office of Geotechnical
Design - South 1

Project:	Tonopah PUC Extension
EA:	07-121901
Dist-Co-Rte-KP:	07-LA-5-KP 58.0/63.4

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design – South 1, Branch 11

Attn: Mr. Mark Okimura

Date: August 4, 2008
File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602
Sound Wall Nos. 581,587, 593, 603, 611
Eight Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611

Revision 4

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

Subject: Amended Foundation Report for Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Eight Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611 – REVISION 4

Per your email requests dated 11/28/2005 and 4/18/06, the Office of Geotechnical Design– South 1 (OGDS-1) has submitted the Foundation Report to provide the geotechnical recommendations for the construction of the subject retaining walls, sound walls and culvert extensions along Sound Wall Nos. 581,587, 593, 603 and 611 on 08/10/2006.

On 02/02/2007, our office received an email from Li Zhou of Special Design, with updated information on culverts, requesting revised recommendations on culverts. On 02/22/2007, OGDS-1 has submitted the Revised Foundation Recommendations (Revision 1) for the culverts.

In your email dated 08/23/2007, you have updated information on culverts, retaining walls and sound walls, and requested revised foundation recommendations. An addendum memo dated 10/15/2007 was provided to you with the revised foundation recommendations (Revision 2) that shall have the precedence over the recommendations given in the reports mentioned above.

On 12/20/2007, to update our recommendations on over-excavation and re-compaction of soil at the project site, OGDS-1 has submitted the Revised Foundation Recommendations (Revision 3).

In your email dated 06/06/2008, you have sent the following revised data for Retaining Walls 325, 603 and 611.

1. RW 325 will be changed from a Type 1A to a Type 5 per standard plan B3-7.
2. RW603 will be changed from Type 1SWB to Type 5SWB from sta 603+01.2 to sta 603+66.9.
3. RW611 will be changed from standard plan soundwall to Type 1SWB from sta 610+56.7 to sta 611+37.3.

In your email dated 7/24/2008, you have sent the revised data for Retaining Walls 593 to account for the fact that there are four swimming pools near the Sound Wall No. 593. SW593 will be changed from Type 5SWB to Type 5SWBP from sta 595+65.47 to sta 596+74.97 and from sta 598+57.50 to sta 598+86.70. Your office provided us the pile cutoff elevations and required nominal resistances for the 400-mm CIDH piles that will be used for Type 5SWBP wall.

OGDS-1 has prepared this memo (Revision 4) to revise Foundation Recommendations to update our recommendations.

The changes in your email dated 06/06/2008 are feasible from a geotechnical standpoint.

Table Nos. 1 and 2 in the Foundation Report (Revision 2) dated 10/15/2007 shall be replaced by the Table Nos. 1 and 2 given below.

The axial pile capacity evaluation for the proposed CIDH piles was performed using SHAFT for Windows, V5.0 by ENSOFT Inc. The tip elevations based on the axial capacity are given in the Table No. 3. It is our understanding that your office has already evaluated lateral capacity and determined that axial capacity governs the pile tip elevation.

Table No. 1- Retaining Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	Referred Borings and CPT
325	RW (Type 5) (Load Case II)	325+14.73	327+08.02	1.2 to 1.8	B-18F, B-36, B-37
326	RW (Type 1)	325+81.79	327+68.81	1.8 to 3.6	B-36, B-37
331	RW (Type 1)	331+92.06	588+12.00	2.4 to 6.7	B-34, B-35
332	RW (Type 1)	331+92.06	588+08.00	2.4 to 5.5	B-34, B-35
582	RW (Type 1)	581+90.00	584+10.00	1.8 to 4.2	B-1
588	RW (Type 1)	588+50.74	592+99.04	3.6 to 7.3	B-6, B-7 ALT, B-8, C-5, C-7
589	RW (Type 1)	588+50.74	589+56.00	2.4 to 6.1	B-6, C-5
602	RW (Type 1)	602+43.43	603+79.83	1.8 to 6.1	B-13

Table No. 2- Sound Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	SW Height (m)	Referred Borings and CPT
581	SW on Pile Cap (Std Plan B15-3 (Case 1))	580+45.49	580+99.70	-	4.877	B-2 ALT
	SW/RW (Type 1SW)	580+99.70	581+82.50	1.8 to 4.2	4.877	B-2 ALT
	SW/RW (Type 1SWB)	581+82.50	584+38.21	1.8 to 4.8	4.369	B-2 ALT, B-3, B-4
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	584+92.53	585+80.00	He = 0.6	4.369	B-4
	SW/RW (Type 1SWB)	585+80.00	586+74.88	2.4 to 5.5	4.369	B-4, B-5
	SW/RW (Type 5SWB)	586+74.88	587+04.07	4.2	4.369	B-5
	SW/RW (Type 1SWB)	587+04.07	587+74.08	3.6	4.369	B-5
587	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	587+00.00	588+38.00	He = 0.3 to 1.2	4.369	B-32
	SW/RW (Type 1SWB)	588+38.00	591+94.48	1.8 to 6.7	4.369	B-31, B-32
	SW/RW (Type 7SWB)	591+94.48	593+11.28	6.7	4.369	B-30, B-31
	SW/RW (Type 1SWB)	593+62.10	597+41.59	3.6 to 4.2	4.369	B-29, B-6A
593	SW/RW (Type 1SWB)	593+00.00	595+36.27	5.5 to 6.7	4.369	B-8, B-9, C-7
	SW/RW (Type 5SWB)	595+36.27	595+65.47	4.8	4.369	B-9
	SW/RW (Type 5SWBP)	595+65.47	596+74.97	4.2 to 4.8	4.369	B-9, B-10, C-9
	SW/RW (Type 5SWB)	596+74.97	598+57.50	4.2	4.369	B-10, B-11, C-10
	SW/RW (Type 5SWBP)	598+57.50	598+86.70	4.2	4.369	B-11
	SW/RW (Type 5SWB)	598+86.70	602+11.15	1.8 to 4.8	4.369	B-11, B-12, C-11
603	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	602+61.80	603+01.20	He = 0.6 to 1.2	4.369	B-CMS, B-13
	SW/RW (Type 5SWB)	603+01.20	606+61.90	3.0 to 5.5	4.369	B-13, B-14, B-15
	SW/RW (Type 1SWB)	606+61.90	610+73.24	4.8 to 6.7	4.369	B-15, B-16
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	611+23.84	611+30.61	He = 0.3	4.369	B-10C
611	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	610+32.25	610+56.70	He =0.6 to 1.2	3.759	B-17
	SW/RW (Type 1SWB)	610+56.70	611+37.31	1.8 to 3.6	3.759	B-17
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	611+37.31	611+80.00	He =0.9 to1.2	3.759	B-17
	SW/RW (Type 1SWB)	611+80.00	615+49.48	3.0 to 6.1	4.369	B-17, B-18, B-19, B-20
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	615+49.48	617+41.02	He = 0.6 to 1.2	4.369	B-20
	SW/RW (Type 1SWB)	617+41.02	618+44.52	2.4 to 3.0	4.369	B-20

Table No. 3- Pile Data for Sound Wall 593

Station	Required Nominal Resistance (kN)	Pile Cutoff Elevation (m)	Recommended Pile Tip Elevation Based on Axial Capacity (m)
595+65.47 to 595+72.77	641	271.925	262.125
595+72.77 to 596+09.27	641	272.225	262.425
596+09.27 to 596+67.67	605	272.605	263.305
596+67.67 to 596+74.97	605	273.205	263.905
598+57.5 to 598+86.7	605	274.655	265.355

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: 08/05/2008

Deepa Wathugala, Ph.D., G.E.
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Supervised by: Date: 8/5/08

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Exp. 6-30-2010
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STATE OF CALIFORNIA

cc: OGDS1 – Los Angeles File
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Ruth Fernandez, Specification Branch Chief (MS 9-2/2H)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design – South 1, Branch 11

Attn: Mr. Mark Okimura

Date: December 20, 2007
File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602
Sound Wall Nos. 581,587, 593, 603, 611
Eight Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611

Addendum

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

Subject: Addendum Foundation Report for Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Eight Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611 – REVISION 3

Per your email requests dated 11/28/2005 and 4/18/06, the Office of Geotechnical Design– South 1 (OGDS-1) has submitted the Foundation Report to provide the geotechnical recommendations for the construction of the subject retaining walls, sound walls and culvert extensions along Sound Wall Nos. 581,587, 593, 603 and 611 on 08/10/2006. On 02/02/2007, our office received an email from Li Zhou of Special Design, with updated information on culverts, requesting revised recommendations on culverts. On 02/22/2007, OGDS-1 has submitted the Revised Foundation Recommendations for the culverts.

On your email dated 08/23/2007, you have updated information on culverts, retaining walls and sound walls, and requested revised foundation recommendations. An addendum memo dated 10/15/2007 was provided to you with the revised foundation recommendations that shall have the precedence over the recommendations given in the reports mentioned above.

This addendum was prepared to update our recommendations on over-excavation and re-compaction of soil at the project site. The updated over-excavation and re-compaction limits are as given in Table No. 1 below. We recommend to over-excavate 1.0 meter of poor quality soils below the bottom of footing and to replace with structure backfill at 95% relative compaction (RC). The horizontal limits of 95% RC structure backfill shall comply with Section 19-5-03 of July 1999 Standard Specifications, except limits may be reduced to the face of temporary shoring if required for construction.

Table No. 1

Wall #	Limits
RW 582	(No over-excavation is needed).
RW 588	STA 588+75 to STA 589+10
SW 587	STA 587+38 to STA 591+94 and STA 593+62 to STA 596+00
SW 593	STA 594+50 to STA 597+05
SW 603	STA 606+45 to STA 607+60
SW 611	(No over-excavation is needed).

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: 12/20/2007

Deepa Wathugala, Ph.D., G.E.
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Supervised by: Date: 12/20/07

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Branch C

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CHI-TSENG TED LIU
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Exp. 6-30-08
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STATE OF CALIFORNIA

cc: OGDS1 – Los Angeles File
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Ruth Fernandez, Specification Branch Chief (MS 9-2/2H)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design – South 1, Branch 11

Attn: Mr. Mark Okimura

Date: October 15, 2007
File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602
Sound Wall Nos. 581,587, 593, 603, 611
Eight Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611

Addendum

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

Subject: Addendum Foundation Report for Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Ten Culvert Extensions along Sound Wall Nos. 581, 593, 603 and 611 – REVISION 2

Per your email requests dated 11/28/2005 and 4/18/06, the Office of Geotechnical Design – South 1 (OGDS-1) has submitted the Foundation Report to provide the geotechnical recommendations for the construction of the subject retaining walls, sound walls and culvert extensions along Sound Wall Nos. 581,587, 593, 603 and 611 on 08/10/2006. On 02/02/2007, our office received an email from Li Zhou of Special Design, with updated information on culverts, requesting revised recommendations on culverts. On 02/22/2007, OGDS-1 has submitted the Revised Foundation Recommendations for the culverts.

On your email dated 08/23/2007, you have updated information on culverts, retaining walls and sound walls, and requested revised foundation recommendations. This Addendum Memo will provide the revised foundation recommendations and these recommendations shall have the precedence over the recommendations given in the reports mentioned above.

Tables 1, 2, 3 and 4 in the Foundation Report dated 08/10/2006 shall be replaced by the Tables 1, 2, 3 and 4 given below.

Table No. 1- Retaining Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	Referred Borings and CPT
325	RW (Type 1A) (Load Case II)	325+14.35	327+13.65	1.2 to 1.8	B-18F, B-36, B-37
326	RW (Type 1)	325+81.79	327+68.81	1.8 to 3.6	B-36, B-37
331	RW (Type 1)	331+92.06	588+12.00	2.4 to 6.7	B-34, B-35
332	RW (Type 1)	331+92.06	588+08.00	2.4 to 5.5	B-34, B-35
582	RW (Type 1)	581+90.00	584+10.00	1.8 to 4.2	B-1
588	RW (Type 1)	588+50.74	592+99.04	3.6 to 7.3	B-6, B-7 ALT, B-8, C-5, C-7
589	RW (Type 1)	588+50.74	589+56.00	2.4 to 6.1	B-6, C-5
602	RW (Type 1)	602+43.43	603+79.83	1.8 to 6.1	B-13

Note: 1.RW: Retaining Wall

Table No. 2- Sound Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	SW Height (m)	Referred Borings and CPT
581	SW on Pile Cap (Std Plan B15-3 (Case 1))	580+45.49	580+99.70	-	4.877	B-2 ALT
	SW/RW (Type 1SW)	580+99.70	581+82.50	1.8 to 4.2	4.877	B-2 ALT
	SW/RW (Type 1SWB)	581+82.50	584+38.21	1.8 to 4.8	4.369	B-2 ALT, B-3, B-4
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	584+92.53	585+80.00	He = 0.6	4.369	B-4
	SW/RW (Type 1SWB)	585+80.00	586+74.88	2.4 to 5.5	4.369	B-4, B-5
	SW/RW (Type 5SWB)	586+74.88	587+04.07	4.2	4.369	B-5
	SW/RW (Type 1SWB)	587+04.07	587+74.08	3.6	4.369	B-5
587	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	587+00.00	588+38.00	He = 0.3 to 1.2	4.369	B-32
	SW/RW (Type 1SWB)	588+38.00	591+94.48	1.8 to 6.7	4.369	B-31, B-32
	SW/RW (Type 7SWB)	591+94.48	593+11.28	6.7	4.369	B-30, B-31
	SW/RW (Type 1SWB)	593+62.10	597+41.59	3.6 to 4.2	4.369	B-29, B-6A
593	SW/RW (Type 1SWB)	593+00.00	595+36.27	5.5 to 6.7	4.369	B-8, B-9
	SW/RW (Type 5SWB)	595+36.27	602+11.15	1.8 to 4.8	4.369	B-10, B-11, B-12, C-7, C-9, C-10, C-11, B-CMS
603	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	602+61.80	603+01.20	He = 0.6 to 1.2	4.369	B-CMS, B-13
	SW/RW (Type 1SWB)	603+01.20	603+66.90	3.0 to 4.2	4.369	B-13
	SW/RW (Type 5SWB)	603+66.90	606+61.90	4.2 to 5.5	4.369	B-13, B-14, B-15
	SW/RW (Type 1SWB)	606+61.90	610+73.24	4.8 to 6.7	4.369	B-15, B-16
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	611+23.84	611+30.61	He = 0.3	4.369	B-10C
611	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	610+32.25	611+80.00	He = 0.9	3.759	B-17
	SW/RW (Type 1SWB)	611+80.00	615+49.48	3.0 to 6.1	4.369	B-17, B-18, B-19, B-20
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	615+49.48	617+41.02	He = 1.2	4.369	B-20
	SW/RW (Type 1SWB)	617+41.02	618+44.52	2.4 to 3.0	4.369	B-20

Note: SW: Sound Wall; SW/RW: Sound Wall on Retaining Wall

Table No. 3 - Culvert Data

Locations	Ret/Sound Wall No.	Intersection Station	Elevation of Bottom of Footing	Proposed Type of Footing	Width of Footing W (mm)	Depth of Footing F (mm)
Location 1	581	587+53.861	258.667	Spread Ftg	4267	838
Location 4	593	597+73.113	272.476	Spread Ftg	4267	838
Location 5	603	604+73.237	279.038	Spread Ftg	5181	838
Location 6	603	605+54.553	279.848	Spread Ftg	5181	838
Location 7	603	606+55.335	281.51	Spread Ftg	2740	400
Location 8	603	606+84.713	280.286	Spread Ftg	4267	838
Location 9	603	608+63.590	281.256	Spread Ftg	4876	838
Location 10	611	612+26.672	284.45	Spread Ftg	5000	700

Table No. 4 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (m)	Reference Line	Surface Elevation[2] (m)	Total Depth (m)	Groundwater Elevation[3] (m)
B-1	1/27/03	584+02	3.03R	CL 5A2 Line	262.68	10.7	Not encountered.
B-2 ALT	1/30/03	581+18	24.75R		259.09	9.1	
B-3	1/30/03	583+19	30.79R		264.70	13.7	
B-4	2/4/03	585+29	45.74R		268.83	13.8	
B-5	2/4/03	587+32	39.94R		265.49	10.5	
B-6	2/5/03	591+02	40.54R		268.66	10.7	
B-7 ALT	2/3/03	588+93	34.72R		265.82	10.8	
B-8	1/28/03	592+95	22.34R		275.11	12.1	
B-9	1/28/03	594+91	23.00R		276.96	15.0	
B-10	1/29/03	596+90	23.19R		277.58	13.6	
B-11	1/29/03	598+89	25.35R		279.57	15.1	
B-12	2/5/03	600+77	72.92R		281.00	11.0	
B-13	2/24/03	603+54	36.80R		283.02	10.0	
B-14	3/4/03	604+48	26.71R		285.35	10.8	
B-15	2/18/03	607+45	22.22R		287.89	11.9	
B-16	2/25/03	609+45	21.96R		290.44	15.9	
B-17	2/25/03	611+28	99.66R		286.60	9.6	
B-18	3/6/03	613+07	25.99R		292.46	13.5	
B-19	3/6/03	614+09	25.68R		292.69	12.2	
B-20	2/26/03	616+05	38.31R		292.96	12.5	
B-29	1/23/03	594+69	28.33L		276.79	14.8	
B-30	1/23/03	592+80	25.86L		274.97	15.5	
B-31	2/6/03	590+82	25.96L		270.48	14.0	
B-32	2/6/03	588+77	26.07L		265.75	10.8	
B-33	1/21/03	587+22	48.00L		265.16	13.7	
B-34	1/22/03	587+79	5.41R		254.85	10.8	
B-35	1/22/03	585+70	10.86R		269.42	16.9	
B-6A	6/25/07	597+19	28.16L		277.55	15.7	
B-10C	7/19/07	611+79	21.94R		292.25	15.7	
B-CMS	7/5/07	601+06	35.02R		282.84	15.7	
B-36	11/17/05	327+19	9.00R	SB1 Line	262.40	15.7	
B-37	11/17/05	325+80	9.40L		262.57	18.8	
B-18F	6/28/07	325+32	36.50R		263.60	15.7	

The "**Section 7.0**" in the Foundation Report dated 08/10/2006, shall be replaced by Section 7.0 given below.

7.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS FOR RETAINING WALLS AND SOUNDWALLS

7.1 Sound Walls and Retaining Walls - General

This project consists of retaining walls, sound walls mounted on retaining walls, and sound walls founded directly on cast-in-drilled-hole (CIDH) piles. The standard design details for the Type1 Retaining walls are given in the Caltrans Standard Plans (July 2004) sheet B3-1. The standard design details for sound walls founded on pile cap are given in the Caltrans Standard Plans (July 2004) sheet B15-3 to B-15-5. The standard design details for sound walls mounted on retaining walls are given in the Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), XS 14-350 (July 2003), XS 14-390-1. The sound wall height is typically 4.369 meters. Design height of the retaining walls varies between 1.8 to 7.3 meters.

Soil parameters used in the analysis were derived from generalized soil profiles developed for each sound wall and/or retaining wall location. Blow counts were used to develop soil parameters for each sound wall and/or retaining wall section. These soil parameters were used in the bearing capacity analysis, settlement analysis and global stability analysis.

7.1.1 Bearing Capacity

7.1.1.1 Retaining Walls

Allowable bearing capacity was calculated using Terzaghi's equation for each retaining wall section. 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.

7.1.1.2 Sound wall on Retaining Wall

The ultimate bearing capacity was calculated using Terzaghi's equation for each sound wall section. The ultimate bearing capacity obtained was compared against the required ultimate bearing capacity given in the Caltrans Bridge Standard Details sheets.

7.1.1.3 Sound Walls on CIDH Piles

The average friction angle for the subsurface soil was determined so that depth of CIDH pile can be obtained directly form the Tables given in the in the Caltrans Standard Plans (July 2004) sheet B15-5.

7.1.2 Settlement of Spread Footings

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

7.1.3 Slope Stability

For the walls on spread footings where there is a slope in front of the footing, 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 method for circular slip surfaces.

Analyses indicate that these walls meet the required minimum factors of safety, 1.5 for static condition and 1.1 for pseudo-static condition. It was assumed that when the loose, disturbed soil within the areas to receive retaining walls are encountered during the construction, it would be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.

7.2 Retaining Wall 325

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1A, Loading Case II condition) from a geotechnical standpoint. The bottom of spread footings will be founded on the existing competent soils or properly compacted fill.

7.3 Retaining Wall 326

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I and II conditions) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.4.1 Retaining Wall 331

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.5 Retaining Wall 332

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.6 Retaining Wall 582

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I and II conditions) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.7 Retaining Wall 588

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill. **LOTB shows cobbles and boulders at the bottom of footing elevation. If cobbles or boulders are is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.8 Retaining Wall 589

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.9 Retaining Wall 602

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.10 Sound Wall 581

The portion of the wall from STA 580+45.49 to STA 580+99.70, are supported on pile cap as per Caltrans Standard Plans (July 2004) sheet B15-3 to B15-5 (Case 1). As per the Data Table for Case 1, on sheet B15-5, with friction angle = 30 degrees (minimum), the length of the CIDH pile is 2.6 and the spacing of the piles is 3.5 m.

The portion of the wall from STA 580+99.70 to STA 581+82.50 (Wall Type 1SW), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-210 (July 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 581+82.50 to STA 584+38.21 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 584+92.53 to STA 585+80.00, are supported on Type 736 barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) and He = 600 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 1.55 m.

The portion of the wall from STA 585+80.00 to STA 586+74.88 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 586+74.88 to STA 587+04.07 (Wall Type 5SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-350 (July 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 587+04.07 to STA 587+74.08 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.11 Sound Wall 587

The portion of the wall from STA 587+00.0 to STA 588+38.000 are supported on Type 736 barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as

follows: for He = 300 mm, L= 4.9 m and S=1.9 m; for He = 600 mm, L= 4.9 m and S=1.55 m; for He = 900 mm, L= 4.9 m and S=1.25 m; for He = 1200 mm, L= 4.9 m and S=0.9 m.

The portion of the wall from STA 588+38.00 to STA 591+94.48 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

The portion of the wall from STA 591+94.48 to STA 593+11.28 is a Retaining Wall Type 7SWB (L-shaped wall) supported on spread footing with vertical tie downs as per Caltrans Bridge Standard Details sheets XS 14-390-1 and XS 14-390-2 (July 2003), and is feasible from a geotechnical standpoint. The bonded length of the vertical tiedown ground anchors should be placed below the potential failure plane projected from the bottom of the wall footing heel at a 61 degree angle downward from horizontal. A minimum of 6.0-m unbonded length is recommended for the tiedown anchors. **Difficult drilling should be anticipated for tie downs due to gravels and cobbles at the site.**

The portion of the wall from STA 593+62.10 to STA 597+41.59 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.12 Sound Wall 593

The portion of the wall from STA 593+00.00 to STA 595+36.27 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 595+36.27 to STA 602+11.15 (Wall Type 5SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-350 (July 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The soil at the bottom of footing elevation from STA 593+50 to STA 596+50 may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. As per LOTB, cobbles or boulders may be encountered at the bottom of the footing elevation. If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.

7.13 Sound Wall 603

The portion of the wall from STA 602+61.80 to STA 603+01.20 is supported on Type 736 barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum), for He = 600 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 1.55 m; for He = 900 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 1.25 m; for He = 1200 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 0.95 m.

The portion of the wall from STA 611+61.80 to STA 611+30.61 is supported on Type 736 barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum), for He = 300 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 1.9 m.

The portion of the wall from STA 603+01.20 to STA 603+66.90 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint.

The portion of the wall from STA 603+66.90 to STA 606+61.90 (Wall Type 5SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-350 (July 2003), is feasible from a geotechnical standpoint.

The portion of the wall from STA 606+61.90 to STA 610+73.24 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint.

The soil at the bottom of footing elevation of this sound wall may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. As per LOTB, cobbles or boulders may be encountered at the bottom of the footing elevation. If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.

7.14 Sound Wall 611

The portion of the wall from STA 610+32.25 to STA 611=80.00 is supported on Type 736 barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as follows: for He = 900 mm, L= 4.9 m and S=1.25 m.

The portion of the wall from STA 611+80.00 to STA 615+49.48 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

The portion of the wall from STA 615+49.48 to STA 617+41.02 are supported on barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as follows: or He = 1200 mm, L= 4.9 m and S=0.9 m.

The portion of the wall from STA 617+41.02 to STA 618+44.52 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

Table 5 in the **Section 8.0** of the Foundation Report dated 08/10/2006 and Table 1 of the Revised Foundation Report dated 02/22/2007, shall be replaced by the Tables 5 given below.

Table No. 5 – Spread Footing Data for Culverts

Culvert Location	BOF Elevation (m)	Footing Width (mm)	Referred Borings and CPT	Maximum Allowable Soil Bearing Capacity (kPa)	Allowable Soil Bearing Pressure (kPa) and Estimated Settlement (mm)
1	258.782	4267	B-5	355	355 (40 mm)
4	272.476	3650	B-10	325	290 (40 mm)
5	278.764	5334	B-14	410	265 (40 mm)
6	278.831	5182	B-14	400	265 (40 mm)
7	281.371	2740	B-15	265	265(25 mm)
8	280.362	4267	B-15	355	340 (40 mm)
9	281.088	4267	B-16	355	340 (40 mm)
10	284.200	5000	B-17 & B-18	325	260 (40 mm)

The recommendations contained in this report are based on specific project information that has been provided by the Office of Bridge Design – South 1, Branch 11 and Office of Design and Technical Services. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

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: 10/15/2007

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cc: OGDS1 – Los Angeles File
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Ruth Fernandez, Specification Branch Chief (MS 9-2/2H)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design – South 1, Branch 11

Attn: Mr. Mark Okimura

Date: February 22, 2007
File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602
Sound Wall Nos. 581,587, 593, 603, 611
Ten Culvert Extensions along Sound Wall Nos. 581,587, 593, 603 and 611

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

Subject: Revised Foundation Recommendations for Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Ten Culvert Extensions along Sound Wall Nos. 581,587, 593, 603 and 611

Per your email requests dated 11/28/2005 and 4/18/06, the Office of Geotechnical Design – South 1 has submitted the Foundation Report to provide the geotechnical recommendations for the construction of the subject retaining walls, sound walls and ten culvert extensions along Sound Wall Nos. 581,587, 593, 603 and 611 on 08/10/2006. On 02/02/2007, our office received an email from Li Zhou of Special Design, with updated information on culverts, requesting revised recommendations on culverts.

This report provides the revised recommendations for the culverts.

In our report dated 08/10/2006, the allowable soil bearing capacities for spread footings for culverts, given in Table No, 5, was limited to values such that the total immediate settlements will not exceed 12 mm. As per email dated 02/06/2007 from Li Zhou, there is no special restriction for settlements of culverts.

The revised geotechnical recommendations for the culverts are as follows.

The soil at the bottom of footing elevation of the culverts may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of the Foundation Report dated 08/10/2006. If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of that report. We recommend that a geotechnical engineer or a geologist from our office shall inspect the over-excavation and compaction of the foundation soil. For this purpose, our office shall be notified two weeks in advance before the over-excavation begins.

Maximum allowable bearing capacity in Table No.1 was calculated using Terzaghi's equation with soil parameters developed from the soil profiles for the culvert locations. A factor of safety of 3 was used. Due to the granular nature of the underlying soils, the settlements will occur shortly upon the application of loads, and will be completed during the first year of construction.

Table No. 1 – Spread Footing Data for Culverts

Culvert Location	BOF Elevation (m)	Footing Width (mm)	Referred Borings and CPT	Maximum Allowable Soil Bearing Capacity (kPa)	Allowable Soil Bearing Pressure (kPa) and Estimated Settlement (mm)
1	258.667	4267	B-5	355	355 (40 mm)
2	262.56	3300	B-31 & B-32	290	290 (35 mm)
3	265.185	3650	B-31	290	275 (40 mm)
4	272.476	3650	B-10	325	290 (40 mm)
5	279.038	5334	B-14	410	265 (40 mm)
6	279.848	5182	B-14	400	265 (40 mm)
7	281.51	2740	B-15	265	265(25 mm)
8	280.286	4267	B-15	355	340 (40 mm)
9	281.256	4267	B-16	355	340 (40 mm)
10	284.45	5000	B-17 & B-18	325	260 (40 mm)

The recommendations contained in this report are based on specific project information that has been provided by the Office of Design and Technical Services. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions or comments, please call Deepa Wathugala at (562) 465-0152, or Ted Liu at or (562) 864-0805.

Prepared by: Date: 02/22/2007

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No. 2669
Exp. 6-30-08
GEOTECHNICAL
STATE OF CALIFORNIA

cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
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RE Pending File (District 7 Project Engineer)
Ruth Fernandez, Specification Branch Chief (MS 9-2/2H)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
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To: MR. RAMIN RASHEDI, Chief
Structure Design
Office of Bridge Design – South 1, Branch 11

Attn: Mr. Mark Okimura

Date: August 10, 2006
File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602
Sound Wall Nos. 581,587, 593, 603, 611
Ten Culvert Extensions along Sound Wall Nos. 581,587, 593, 603 and 611

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

Subject: Foundation Recommendations for Retaining Wall Nos. 325, 326, 331, 332,582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Ten Culvert Extensions along Sound Wall Nos. 581,587, 593, 603 and 611

1.0 INTRODUCTION

Per your email request dated 11/28/2005 and the email request dated4/18/06, the Office of Geotechnical Design – South 1 has prepared this Memorandum to provide the geotechnical recommendations for the construction of the subject retaining walls, sound walls and ten culvert extensions along Sound Wall Nos. 581,587, 593, 603 and 611. The construction of the walls and culverts is needed for the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4). The recommendations in this memo are based on the borings drilled for this project. Table No. 1 and Table No. 2 show the information on the proposed retaining walls and sound walls respectively. Table No. 3 shows the information on the culverts. The information in tables 1 and 2 was provided to our office by the Office of Bridge Design-South 1, Branch 11. The information in table 3 was provided to our office by the Office of Design and Technical Services.

Table No. 1- Retaining Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	Referred Borings and CPT
325	RW (Type 1)	325+14.21	325+80.00	2.4 to 3.0	B-36, B-37
326	RW (Type 1)	325+73.35	327+59.28	1.2 to 3.0	B-36, B-37
331	RW (Type 1)	331+92.06	588+50.00	3.0 to 6.1	B-34, B-35
332	RW (Type 1)	331+92.06	588+60.00	1.8 to 5.5	B-34, B-35
582	RW (Type 1)	581+90.00	584+10.00	1.8 to 4.2	B-1
588	RW (Type 1)	588+50.74	592+99.74	3.6 to 7.3	B-6, B-7 ALT, B-8, C-5, C-7
589	RW (Type 1)	588+50.74	589+67.54	2.4 to 6.1	B-6, C-5
602	RW (Type 1)	602+43.33	603+79.89	1.8 to 6.1	B-13

Note: 1.RW: Retaining Wall

Table No. 2- Sound Wall Data

Wall No.	Structure Type	Begin Station	End Station	RW Height (m)	SW Height (m)	Referred Borings and CPT
581	SW on Pile Cap (Std Plan B15-3 (Case 1))	580+45.49	581+04.71	-	4.877	B-2 ALT
	SW/RW (Type 1SW)	581+04.71	581+80.00	1.8 to 4.2	4.877	B-2 ALT
	SW/RW (Type 1SWB)	581+80.00	584+38.21	1.8 to 4.8	4.364 to 4.978	B-2 ALT, B-3, B-4
	SW on Type 736 Barrier (Std Plan B15-6 (Case 2))	584+87.46	586+20.00	-	4.369	B-4
	SW/RW (Type 1SWB)	586+20.00	586+90.00	2.4 to 3.6	4.369	B-4, B-5
	SW/RW (Type 5SWB)	586+90.00	587+20.00	3.6	4.369	B-5
	SW/RW (Type 1SWB)	587+20.00	587+74.08	4.2	4.369	B-5
587	SW on Type 736 Barrier (Std Plans B15-6)	587+00.00	588+44.08	-	4.369	B-32
	SW/RW (Type 1SWB)	588+44.08	591+94.48	2.4 to 6.7	4.369	B-31, B-32
	SW/RW (Type 7SWB)	591+94.48	593+11.28	6.7 to 7.3	4.369	B-30, B-31
	SW/RW (Type 1SWB)	593+62.10	594+89.14	3.6 to 4.2	4.369	B-29
593	SW/RW (Type 1SWB)	593+00.00	602+03.73	1.8 to 6.1	4.369	B-8, B-9. B-10, B-11, B-12, C-7, C-9, C-10, C-11
603	SW/RW (Type 1SWB)	602+79.30	603+66.90	1.2 to 3.6	4.369	B-13
	SW/RW (Type 5SWB)	603+66.90	606+55.20	4.2 to 5.5	4.369	B-13, B-14, B-15
	SW/RW (Type 1SWB)	606+55.20	610+71.32	4.8 to 6.7	4.369	B-15, B-16
611	SW on Type 736 Barrier (Std Plan B15-6)	610+51.20	610+78.65	-	4.369	B-17
	SW/RW (Type 1SWB)	610+78.65	615+49.48	2.4 to 6.1	4.369	B-17, B-18, B-19, B-20
	SW on Type 736 Barrier (Std Plan B15-6)	615+49.48	617+50.00	-	4.369	B-20
	SW/RW (Type 1SWB)	617+50.00	618+44.52	2.4 to 3.0	4.369	B-20

Note: SW: Sound Wall; SW/RW: Sound Wall on Retaining Wall

Table No. 3 - Culvert Data

Locations	Ret/Sound Wall No.	Intersection Station	Elevation of Bottom of Footing	Proposed Type of Footing	Width of Footing W (mm)	Depth of Footing F (mm)
Location 1	581	587+53.861	258.667	Spread Ftg	4267	838
Location 2	587	589+71.4	262.56	Spread Ftg	3950	450
Location 3	587	591+50.8	265.185	Spread Ftg	3950	450
Location 4	593	597+73.113	272.476	Spread Ftg	4267	838
Location 5	603	604+73.237	279.038	Spread Ftg	5181	838
Location 6	603	605+54.553	279.848	Spread Ftg	5181	838
Location 7	603	606+55.335	281.51	Spread Ftg	2740	400
Location 8	603	606+84.713	280.286	Spread Ftg	4267	838
Location 9	603	608+63.590	281.256	Spread Ftg	4876	838
Location 10	611	612+26.672	284.45	Spread Ftg	5000	700

2.0 FIELD EXPLORATION PROGRAM

Site-specific field exploration was performed between December 17, 2002 and November 17, 2005. The field investigation included 36 hollow stem auger borings and 16 Cone Penetrometer Tests (CPTs). Samples were obtained from the borings. Blow counts and SPT N values were continuously recorded at an interval of 1.5 meters (5 feet) during drilling. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 35-mm I.D. sampler with a 63.5-kg hammer dropped 762-mm. Caltrans drill rig model CME 85 was used. The CPT consisted of pushing an instrumented cone-tipped probe into the ground while simultaneously recording the resistance to penetration of the cone tip and friction along the sleeve. The CPT is performed in accordance with the ASTM specification (ASTM D5778-95) using a 10-centimeter (cm) electric cone penetrometer.

A summary of exploratory borings and CPT's are presented in Tables 4 and 5, respectively. Surface elevations, stations, and offsets of the Borings and CPT's conducted during the investigation 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.

Table No. 4 – Summary of Borings

Boring No.	Date Drilled	Station	Offset (m)	Reference Line	Surface Elevation[2] (m)	Total Depth (m)	Groundwater Elevation[3] (m)
B-1	1/27/03	584+02	3.03R	CL 5A2 Line	262.68	10.7	Not encountered.
B-2 ALT	1/30/03	581+18	24.75R		259.09	9.1	
B-3	1/30/03	583+19	30.79R		264.70	13.7	
B-4	2/4/03	585+29	45.74R		268.83	13.8	
B-5	2/4/03	587+32	39.94R		265.49	10.5	
B-6	2/5/03	591+02	40.54R		268.66	10.7	
B-7 ALT	2/3/03	588+93	34.72R		265.82	10.8	
B-8	1/28/03	592+95	22.34R		275.11	12.1	
B-9	1/28/03	594+91	23.00R		276.96	15.0	
B-10	1/29/03	596+90	23.19R		277.58	13.6	
B-11	1/29/03	598+89	25.35R		279.57	15.1	
B-12	2/5/03	600+77	72.92R		281.00	11.0	
B-13	2/24/03	603+54	36.80R		283.02	10.0	
B-14	3/4/03	604+48	26.71R		285.35	10.8	
B-15	2/18/03	607+45	22.22R		287.89	11.9	
B-16	2/25/03	609+45	21.96R		290.44	15.9	
B-17	2/25/03	611+28	99.66R		286.60	9.6	
B-18	3/6/03	613+07	25.99R		292.46	13.5	
B-19	3/6/03	614+09	25.68R		292.69	12.2	
B-20	2/26/03	616+05	38.31R		292.96	12.5	
B-29	1/23/03	594+69	28.33L		276.79	14.8	
B-30	1/23/03	592+80	25.86L		274.97	15.5	
B-31	2/6/03	590+82	25.96L		270.48	14.0	
B-32	2/6/03	588+77	26.07L		265.75	10.8	
B-33	1/21/03	587+22	48.00L		265.16	13.7	
B-34	1/22/03	587+79	5.41R		254.85	10.8	
B-35	1/22/03	585+70	10.86R		269.42	16.9	
B-36	11/17/05	583+52	289.27L		262.40	15.7	
B-37	11/17/05	583+33	426.44L		262.57	18.8	

Table No. 5 – Summary of Cone Penetration Test Sounding

CPT No.	Date Drilled	Station	Offset (m)	Reference Line	Surface Elevation[2] (m)	Total Depth (m)	Groundwater Elevation[3] (m)
C-5	12/18/02	587+98	35.92R	CL 5A2 Line	264.77	1.4	N/A
C-7	12/18/02	591+96	23.63R		273.33	1.3	
C-9	12/18/02	595+92	23.67R		277.29	1.3	
C-10	12/18/02	597.90	23.64R		278.22	0.5	
C-11	12/18/02	599+89	37.77R		280.94	1.0	

3.0 LABORATORY TESTING

Laboratory testing was performed on a bulk sample from the borings. Laboratory testing included corrosion test. Sample submitted for testing was analyzed at a Department of Transportation laboratory. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table 3 below). A summary of corrosion test results is presented in Section 8.0.

Table 3 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

4.0 GEOLOGY

4.1 Regional Geology

The subject site is located within the Transverse Range Geomorphic Province. The Transverse Ranges are characterized by east-west trending mountain ranges and valleys. The site is located within the east-west trending San Fernando Valley, which is comprised of Holocene alluvial deposits. The San Fernando Valley is bounded by the Santa Monica Mountains on the south and bounded by the San Gabriel Mountains and the Santa Susanna Mountains on the north. Several east-west trending reverse faults are associated with compression of the northern portion of the San Fernando Valley. These faults are discussed in Section 7.0, Seismicity, of this report.

4.2 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 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 with occasional boulders. Depth to bedrock or bedrock like material should be estimated at greater than 120 meters for this project. The proposed soundwalls along the northbound and southbound Interstate 5 Freeway will be founded either on fill section or partially on fill and the underlyinjg alluvium or may be founded entirely on the alluvium that underlies the fill. Fill ranges in thickness up to approximately 8 meters. 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 1.2 and 2.0 kilometers north of the proposed project respectively. Distances are measured to Osborne Street at Interstate 5 that is roughly the middle of this project.

4.3 Subsurface Conditions

Subsurface soil conditions at the proposed abutment and bent locations were determined based on the 36 hollow stem auger borings and 16 Cone Penetrometer Tests (CPTs) performed for this project. The subject area generally consists of artificial fill composed of well graded loose to dense, fine to coarse sand with occasional gravel and cobbles. Below the fill material, native alluvium is composed of medium dense to very dense sands and gravelly sands with cobbles. Native soil is occasionally interbedded with silty sand.

4.3.1 Groundwater

Groundwater was not encountered in any of the borings for this investigation to the maximim depth explored. In addition a deeper boring, 77-B3 for proposed Bridge 53-2977, which was drilled in May 2005 to an elevation of approximately +201 meters did not encounter groundwater. Boring 77-B3 is located at the south end of this project in the vincinity of the 5/170 interchange. Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from 160 to 175 meters (100 to 115 meters depth below ground surface). Well number 4885, located approximately 0.3 km north of the freeway at the corner of Branford Street and Amboy Ave., had a maximum reading from 1994 to 2001 as an elevation of +170 meters above mean sea level (MSL). Historically in the past several decades the highest groundwater nearby the proposed project has been measured at an elevation of approximately 244 meters. For this project the highest anticipated groundwater should be estimated at elevation +244 meters. 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 .

5.0 SEISMICITY

The project site is not located within any established Alquist-Priolo Earthquake Fault Zone. Based on the Caltrans Seismic Hazard Map, the Verdugo Fault (VDO, Reverse-Oblique Fault) is the nearest active seismic source from the proposed project site. Two other faults the Simi-Santa Rosa Northridge Hills Fault (SSN, Reverse-Oblique Fault) and the San Fernando-Sierra Madre-Duarte (SSD, Reverse-Thrust Fault) are nearby causative faults that should also be considered in the design of the proposed soundwall project.

The Table No. 5 summarizes the Moment Magnitude of the Maximum Credible Earthquake (MCE), type of faulting, distance (distance is measured to Osborne Street at Interstate 5 roughly the middle of this project), and Peak Bedrock Acceleration of the three Faults mentioned above. The Peak Bedrock Acceleration are based on the Attenuation relationships by Sadigh et al, 1997.

Table No. 5 - Summary of Seismic Parameters

Fault	Type of Faulting	Mw	Distance, km	Direction	PBA
Verdugo Hills, VDO	Reverse-Oblique	6.75	1.2	NE	0.8g
Simi/Santa Rosa-Northridge Hills, SSN	Reverse-Oblique	7.5	6.0	NW	0.6g
San Fernando-Sierra Madre-Duarte, SSD	Reverse-Thrust	7.5	5.5	NE	0.7g

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 deep Ground Water elevation, the liquefaction potential is low at this site.

6.0 CORROSION EVALUATION

Caltrans considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples (Corrosion Guidelines, 2003): 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.

Samples retrieved from the exploratory borings were combined to make composite samples of earth material from various depths. The District 7 and Sacramento Material Laboratories tested samples for borings conducted in-house, and URS Consultants tested samples from the borings they conducted for corrosivity potential. The test results are summarized in Table No. 4. Based on the results of the corrosion analysis, the site is non-corrosive.

Table No. 4 - Corrosion Test Results

Boring	Sample Depth (m)	pH	Minimum Resistivity* (ohm-cm)	Sulfate Content (PPM)	Chloride Content (PPM)
B-17	1.2 - 2.7	8.3	7,700	N/A	N/A
B-18	7.6 - 9.1	8.0	900	1000	15
B-36	1.5 - 5.0	8.2	20,000	45	60
B-36	7.6 - 9.1	7.2	21,000	240	90
B-37	1.5 - 5.0	8.5	10,000	240	90

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.

7.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS FOR RETAINING WALLS AND SOUNDWALLS

7.1 Sound Walls and Retaining Walls - General

This project consists of retaining walls, sound walls mounted on retaining walls, and sound walls founded directly on cast-in-drilled-hole (CIDH) piles. The standard design details for the Type1 Retaining walls are given in the Caltrans Standard Plans (July 2004) sheet B3-1. The standard design details for sound walls founded on pile cap are given in the Caltrans Standard Plans (July 2004) sheet B15-3 to B-15-5. The standard design details for sound walls mounted on retaining walls are given in the Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), XS 14-350 (July 2003), XS 14-390-1. The sound wall height is typically 4.369 meters. Design height of the retaining walls varies between 1.8 to 7.3 meters.

Soil parameters used in the analysis were derived from generalized soil profiles developed for each sound wall and/or retaining wall location. Blow counts were used to develop soil parameters for each sound wall and/or retaining wall section. These soil parameters were used in the bearing capacity analysis, settlement analysis and global stability analysis.

7.1.1 Bearing Capacity

7.1.1.1 Retaining Walls

Allowable bearing capacity was calculated using Terzaghi's equation for each retaining wall section. 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.

7.1.1.2 Sound wall on Retaining Wall

The ultimate bearing capacity was calculated using Terzaghi's equation for each sound wall section. The ultimate bearing capacity obtained was compared against the required ultimate bearing capacity given in the Caltrans Bridge Standard Details sheets.

7.1.1.3 Sound Walls on CIDH Piles

The average friction angle for the subsurface soil was determined so that depth of CIDH pile can be obtained directly form the Tables given in the in the Caltrans Standard Plans (July 2004) sheet B15-5.

7.1.2 Settlement of Spread Footings

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

7.1.3 Slope Stability

For the walls on spread footings where there is a slope in front of the footing, 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 method for circular slip surfaces.

Analyses indicate that these walls meet the required minimum factors of safety, 1.5 for static condition and 1.1 for pseudo-static condition. It was assumed that when the loose, disturbed soil within the areas to receive retaining walls are encountered during the construction, it would be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.

7.2 Retaining Wall 325

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings will be founded on the existing competent soils or properly compacted fill.

7.3 Retaining Wall 326

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I and II conditions) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.4.1 Retaining Wall 331

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.5 Retaining Wall 332

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.6 Retaining Wall 582

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I and II conditions) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.7 Retaining Wall 588

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill. **LOTB shows cobbles and boulders at the bottom of footing elevation. If cobbles or boulders are is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.8 Retaining Wall 589

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.9 Retaining Wall 602

The subsurface condition along the wall alignment is considered suitable for the proposed Caltrans standard retaining wall, supported on spread footing (Standard Plans (July 2004) sheet B3-1, Type 1, Loading Case I condition) from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.10 Sound Wall 581

The portion of the wall from STA 580+45.494 to STA 581+04.707, are supported on pile cap as per Caltrans Standard Plans (July 2004) sheet B15-3 to B15-5 (Case 1). As per the Data Table for Case 1, on sheet B15-5, with friction angle = 30 degrees (minimum), the length of the CIDH pile is 2.6 and the spacing of the piles is 3.5 m.

The portion of the wall from STA 581+04.707 to STA 581+80.000 (Wall Type 1SW), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-210 (July 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 581+80.000 to STA 584+38.213 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 584+38.213 to STA 586+20.000, are supported on barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) and He = 600 mm, the length of the CIDH pile is 4.9 and the spacing of the piles is 1.55 m.

The portion of the wall from STA 586+20.000 to STA 586+90.000 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 586+90.000 to STA 587+20.000 (Wall Type 5SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-350 (July 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

The portion of the wall from STA 587+20.000 to STA 587+74.085 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. The bottom of spread footings shall be founded on the existing competent soils or properly compacted fill.

7.11 Sound Wall 587

The portion of the wall from STA 587+00.000 to STA 588+44.084 are supported on barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as follows: for He = 300 mm, L= 4.9 m and S=1.9 m; for He = 600 mm, L= 4.9 m and S=1.55 m; for He = 900 mm, L= 4.9 m and S=1.25 m; for He = 1200 mm, L= 4.9 m and S=0.9 m.

The portion of the wall from STA 588+44.084 to STA 591+94.484 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

The portion of the wall from STA 591+94.484 to STA 593+11.284 is a Retaining Wall Type 7SWB (L-shaped wall) supported on spread footing with vertical tie downs as per Caltrans Bridge Standard Details sheets XS 14-390-1 and XS 14-390-2 (July 2003), and is feasible from a geotechnical standpoint. The bonded length of the vertical tiedown ground anchors should be placed below the potential failure plane projected from the bottom of the wall footing heel at a 61 degree angle downward from horizontal. A minimum of 6.0-m unbonded length is recommended for the tiedown anchors. **Difficult drilling should be anticipated for tie downs due to gravels and cobbles at the site.**

The portion of the wall from STA 593+62.101 to STA 594+89.136 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.12 Sound Wall 593

The portion of the wall from STA 593+00.000 to STA 602+03.735 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation from STA 593+50 to STA 596+50 may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. As per LOTB, cobbles or boulders may be encountered at the bottom of the footing elevation. If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

7.13 Sound Wall 603

The portion of the wall from STA 602+79.296 to STA 603+66.896 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint.

The portion of the wall from STA 603+66.896 to STA 606+55.198 (Wall Type 5SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-350 (July 2003), is feasible from a geotechnical standpoint.

The portion of the wall from STA 606+55.198 to STA 606+71.316 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheets XS 14-220 (February 2003), is feasible from a geotechnical standpoint.

The soil at the bottom of footing elevation of this sound wall may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. . As per LOTB, cobbles or boulders may be encountered at the bottom of the footing elevation. If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.

7.14 Sound Wall 611

The portion of the wall from STA 610+51.204 to STA 610+78.654 are supported on barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as follows: for He = 900 mm, L= 4.9 m and S=1.25 m.

The portion of the wall from STA 610+78.654 to STA 615+49.481 (Wall Type 1SWB),

supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

The portion of the wall from STA 615+49.481 to STA 617+50.000 are supported on barrier on the cast-in-drilled-hole piles as per Caltrans Standard Plans (July 2004) sheet B15-6 to B15-8 (Case 2). As per the Data Table for Case 2, on sheet B15-8, with friction angle = 30 degrees (minimum) the length of the CIDH pile (L) and the spacing of the piles (S) are as follows: or He = 1200 mm, L= 4.9 m and S=0.9 m.

The portion of the wall from STA 617+50.000 to STA 618+44.525 (Wall Type 1SWB), supported on spread footing as per Caltrans Bridge Standard Details sheet XS 14-220 (February 2003), is feasible from a geotechnical standpoint. **The soil at the bottom of footing elevation may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report.**

8.0 FOUNDATION ANALYSIS AND RECOMMENDATIONS FOR CULVERTS

8.1 Bearing Capacity

Soil parameters used in the analysis were derived from generalized soil profiles developed for each culvert location. Blow counts were used to develop soil parameters. These soil parameters were used in the bearing capacity analysis and settlement calculations.

Allowable bearing capacity was calculated using Terzaghi's equation with soil parameters developed from the soil profiles for the culvert locations. A factor of safety of 3 was used. The Table No. 5 below summarizes the spread footing dimensions and the corresponding Allowable Soil Bearing Capacity for the proposed culvert locations. The Allowable Soil Bearing Capacities were limited to the values such that the total immediate settlements will not exceed 12 mm (due to the granular nature of the underlying granular soils, the settlements will occur shortly upon the application of loads).

The soil at the bottom of footing elevation of the culverts may consist of loose fine sand. If loose soil is encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. . If cobbles or boulders are encountered at the bottom of the footing elevation, soil in this area should be over-excavated for 1.5-m and replaced with the compacted fill as described in the Section 9.0 of this report. We recommend that a geotechnical engineer or a geologist from our office shall inspect the over-excavation and compaction of the foundation soil. For this purpose, our office shall be notified two weeks in advance before the over-excavation begins.

Table No. 5 – Spread Footing Data for Culverts

Location	BOF Elevation (m)	Footing Width (mm)	Referred Borings and CPT	Allowable Soil Bearing Capacity (kPa)
Location 1	258.667	4267	B-5	170
Location 2	262.56	3950	B-31 & B-32	120
Location 3	265.185	3950	B-31	130
Location 4	272.476	4267	B-10	145
Location 5	279.038	5181	B-14	140
Location 6	279.848	5181	B-14	140
Location 7	281.51	2740	B-15	175
Location 8	280.286	4267	B-15	170
Location 9	281.256	4876	B-16	160
Location 10	284.45	5000	B-17 & B-18	150

8.2 Settlement of Spread Footings

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

9.0 CONSTRUCTION CONSIDERATIONS

1. The proposed retaining walls and sound walls on retaining walls with spread footings should be founded on the existing competent soils or properly compacted fill. Existing loose, disturbed, dry soils within the areas to receive retaining walls should be over-excavated for 1.5 m and replaced with compacted fill. The compacted fill beneath the retaining wall footing should be granular in nature, have a Sand Equivalent value of 20 as determined by California Test Method 217, and have lees than 50% of material passing No.200 sieve size. The compacted fill beneath the retaining wall footing should be placed in horizontal loose layers of approximately 0.2 m thick, and compacted to at least 95% relative compaction.

2. Earthwork should be performed in accordance with Sections 6 and 19 of the latest Caltrans Standard Specifications. Soils with an Expansion Index of less than 50 or a Sand

Equivalent of 20 or more should be used within the approach embankment, in accordance with standard Caltrans requirements.

3. Coarse gravel and cobbles should be expected during excavation of the footing locations and keyways. This material may be removed using conventional excavators and scraper and ripper equipment.

4. On-site material may be used as replacement material. However, large cobbles and possibly boulders may be encountered during excavation. Oversized material (greater than 0.2 meters in the widest dimension) should be excluded from the replacement fill material.

5. The new embankment fill should be benched into the existing slope at minimum 1.2-meter wide cuts. Fill should be placed in conformance with Sections 19-6.01 (Placing) and 19-6.02 (Compacting).

The recommendations contained in this report are based on specific project information that has been provided by the Office of Bridge Design-South 1, Branch 11 and the Office of Design and Technical Services. If any conceptual changes are made during final project design, the office of Geotechnical Design South-1 should review those changes to determine if these foundation recommendations are still applicable.

If you have any questions or comments, please call Deepa Wathugala at (562) 465-0152, or Ted Liu at or (562) 864-0805.

Prepared by: Date: 8/10/06

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cc: OGDS1 – NORWALK File (2)
OGDS1 – SAC File (MS-5)
GS – SAC File (MS-5)
RE Pending File (District 7 Project Engineer)
Ruth Fernandez, Specification Branch Chief (MS 9-2/2H)

State of California

Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. KHAN A. HOSSAIN
Senior Transportation Engineer
D07 Project Development
Office of Design - Branch D

Attn: Mr. Sule K. Alabi

Date: September 12, 2007

File: 07-LA-5-KP 58.0/63.4
EA: 07-121901

OH Sign Nos. 1A, 2D, 2F, 3B, 4A, 5A, 5C, 6A, 6B, 6D, 7A, 8B, 9A, 9B, 10C, 11B, 12A, 13A, 13C, 14B, 15A, 15B, 16A, 17B, 18A, 18D, 18F and 19A

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

Subject: Foundation Recommendations for Overhead Signs (28 Sign Structures)

1.0 INTRODUCTION

The Office of Geotechnical Design South 1 (OGDS-1), Branch C has conducted a foundation investigation pursuant to the email request by your office on October 31, 2006 for a foundation investigation and recommendations for the proposed Overhead Signs (OH signs) supported on Cast in Drilled Hole (CIDH) pile foundations.

2.0 SITE DESCRIPTION

The proposed OH signs are located on the northbound and southbound I-5 and on the northbound and southbound SR-170 in the City of Los Angeles, in Los Angeles County. The proposed construction of the OH signs is a part of the project that includes the proposed addition of HOV Lanes in the median of the I-5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4).

3.0 FIELD EXPLORATION PROGRAM

A site-specific field exploration was performed between June 24, 2007 and July 26, 2007 by the Consultants, URS Corp. The field investigation included 22 hollow stem auger borings and two bucket auger holes. Blow counts and SPT N values were continuously recorded at an interval of 1.5 meters (5 feet) during drilling of the 23 hollow stem auger

borings. The SPT was performed in accordance with ASTM Test Method D1584-84 using a standard 35-mm I.D. sampler with a 63.5-kg hammer dropped 762-mm. Potential for caving was observed in the bucket auger holes.

A summary of exploratory borings is presented in Tables 1. 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 Consultants, URS Corp. and will be submitted to your office to be included in the project plans.

Table 1 - Summary of Boring Information

Boring No.	Date Drilled	Station	Offset (m)	Reference Line	Surface Elevation (m)	Total Depth (m)	Groundwater Elevation (m)
1A	07/02/07	579+97.986	30.61 RT	CL5	255.648	14.6	Not encountered
3BP1	06/24/07	586+82.775	30.21 LT	SBETW2	264.023	15.7	
3BP2	07/03/07	586+76.325	9.17 RT	SBETW2	266.116	15.7	
3BP2BKT	07/16/07	586+77.351	10.68 RT	SBETW2	266.291	10.7	
4A	07/11/07	590+02.116	2.51 LT	CL5	268.664	15.7	
5A	06/26/07	593+79.981	26.23 LT	CL5A2	276.287	15.7	
5C	07/06/07	593+99.450	37.90 RT	CL5	271.156	15.7	
6A	06/25/07	597+18.720	28.16 LT	CL5A2	277.550	15.7	
6B	07/10/07	597+56.972	23.14 RT	CL5	277.943	16.2	
9A	07/06/07	607+99.610	35.48 RT	CL5	282.992	16.2	
9B	07/20/07	607+97.032	29.48 LT	CL5	288.146	15.7	
10C	07/19/07	611+78.912	21.94 RT	CL5	292.250	15.7	
11B	07/17/07	614+58.026	26.71 RT	CL5	292.712	16.5	
12A	07/18/07	616+99.974	45.05 RT	CL5	293.295	15.7	
13A	07/23/07	620+99.691	30.31 RT	CL5	295.255	15.7	
13C	07/25/07	621+96.311	25.67 RT	CL5	295.597	15.7	
14B	07/24/07	625+01.631	30.42 RT	CL5	296.049	15.7	
15	07/16/07	627+23.546	23.72 RT	CL5	299.687	15.7	
15BKT	07/16/07	627+10.439	25.67 RT	CL5	299.499	10.7	
17B	07/26/07	634+33.339	19.98 LT	CL5	300.737	15.7	
18A	07/12/07	323+47.611	18.77 LT	NB2	262.692	15.7	
18D	07/09/07	322+81.097	16.77 RT	NB2	260.091	15.7	
18F	06/28/07	325+31.211	13.02 RT	CI 2 Line	263.601	15.7	
19A	06/27/07	328+83.064	7.68 LT	SB1 Line	261.470	15.7	

Subsurface soil conditions at the proposed OH signs locations were determined based on the 22 hollow stem auger borings given in Table 1 and four additional hollow stem auger borings (B3 and B11 performed for retaining walls and sound walls, 77B5-BL for the HOV Connectors, and B-CMS for the CMS sign) drilled for the same project from year 2003 through 2007. The subject area generally consists of artificial fill composed of poorly graded loose to dense, fine to coarse sand with scattered gravel and cobbles. Below the fill material, native alluvium is composed of loose to very dense sands and gravelly sands with cobbles. Native soil is occasionally interbedded with silty sand.

Groundwater

Groundwater was not encountered in any of the borings for this investigation to the maximum depth explored (16.5 m).

In addition a deeper boring, 77-B3 for proposed Bridge 53-2977, which was drilled in May 2005 to an elevation of approximately +201 meters did not encounter groundwater. Boring 77-B3 is located at the south end of this project in the vicinity of the 5/170 interchange. Several wells are maintained by the LACDPW near the vicinity of this project, with a measured elevation of groundwater ranging from +160 to +175 meters (100 to 115 meters depth below ground surface). Well number 4885, located approximately 0.3 km north of the freeway at the corner of Branford Street and Amboy Ave., had a maximum reading from 1994 to 2001 as an elevation of +170 meters above mean sea level (MSL). Historically in the past several decades the highest groundwater nearby the proposed project has been measured at an elevation of approximately +244 meters. For this project the highest anticipated groundwater should be estimated at elevation +244 meters. 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 .

4.0 LABORATORY TESTING

Corrosion tests were performed on bulk samples from the borings by URS Corp. Testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table 2 below). A summary of corrosion test results is presented in Section 6.0.

Table 2 – Laboratory Test Methods

Test	Standard
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride content	CTM 422
Corrosion – Sulfate content	CTM 417

5.0 GEOLOGY AND SEISMICITY

Geology and Seismicity of the location of the proposed OH Signs is the same as those for the proposed for sound walls at SR 5 from I-5/SR-170 Interchange (KP 58.0) to the I-5/SR-118 Interchange (KP 63.4) (same EA: 07-121901). Please refer to the report on "Foundation Recommendations for Sound Wall Nos. 585, 617, 618, 620-A, 620-B, 621-A and 621-B", prepared by our office, and submitted on December 11, 2006 to the D-07 Project Development, Office of Design - Branch D.

6.0 CORROSION EVALUATION

Caltrans considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples (Corrosion Guidelines, 2003): 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.

URS Consultants tested the composite soil samples from the borings for corrosivity potential. The test results are summarized in Table No. 3. Based on the results of the corrosion analysis, the site is non-corrosive at the all the boring locations except at B-9B. Corrosion resistant design and construction materials are advised at the location of OH Sign 9B.

Table No. 3 - Corrosion Test Results

Boring	Sample Depth (m)	pH	Minimum Resistivity (ohm-cm)	Sulfate Content* (PPM)	Chloride Content (PPM)
1A	0 ~ 25	7.8	16,000	60	60
3BP-1	0 ~ 25	8.0	24,000	60	75
3BP-2	0 ~ 25	8.3	15,600	ND	75
4A	0 ~ 25	8.9	14,000	ND	90
5A	0 ~ 25	8.9	6,000	96	75
5C	0 ~ 25	8.8	12,000	36	75
6A	0 ~ 25	9.0	6,000	66	45
6B	0 ~ 25	7.6	5,700	51	45
9A	0 ~ 25	7.4	9,000	51	60
9B	0 ~ 25	7.6	675	2496	45
10C	0 ~ 25	7.0	815	936	60
11B	0 ~ 25	7.8	830	1617	60
12A	0 ~ 25	8.1	1,400	387	45
13A	0 ~ 25	6.5	755	1110	60
13C	0 ~ 25	7.1	1,000	885	60
14B	0 ~ 25	7.7	1,350	468	45
15	0 ~ 25	7.9	620	1080	45
17B	0 ~ 25	6.5	2,350	219	75
18A	0 ~ 25	9.0	9,000	207	105
18D	0 ~ 25	9.0	11,000	135	90
18F	0 ~ 25	8.9	9,000	ND	75
19A	0 ~ 25	8.8	19,000	ND	60

* ND = Not Detected

7.0 FOUNDATION RECOMMENDATIONS

The proposed CIDH piles for support of the subject OH signs are 1524-mm in diameter. The axial pile capacity evaluation for the proposed CIDH piles was performed using SHAFT for Windows, V5.0 by ENSOFT Inc. The lateral load-deformation response of single pile was analyzed utilizing the LPILE plus for Windows, V5.0 by ENSOFT Inc. The depth of sign foundation was computed based on the boundary conditions shown in Table 4. Recommended pile depths are given in Table 5.

Table 4 – Unfactored Loadings

Sign Post No.	Station	Height above Ground Level/ Height of Center Barrier (m)	Height above BOF of Wall (m)	Bending Moment at Pile Head (kN-m)	Shear Force at Pile Head (kN)	Axial Load (kN)
1A	580+00	N/A	N/A	550	60	94
2D	585+50	N/A	N/A	572	60	96
2F	583+50	N/A	4.43	623	63	57
3B- POST 1 (SHOULDER)	586+84	N/A	N/A	1289	108	144
3B- POST 2 (MEDIAN)	586+84	0	N/A	1289	108	144
4A	590+05	1.57	N/A	432	58	85
5A- POST 1 (SHOULDER)	593+80	N/A	5.29	785	105	137
5A- POST 2 (MEDIAN)	593+80	1.57	N/A	786	105	137
5C	594+00	N/A	7.54	503	51	52
6A	597+20	N/A	5.05	623	63	57
6B	597+45	N/A	5.03	623	63	57
6D	599+15	1.57	N/A	307	44	70
7A	601+25	1.57	N/A	191	31	51
8B (SHOULDER)	605+25	N/A	N/A	954	103	146
8B (MEDIAN)	605+25	1.57	N/A	954	103	146
9A	608+03	N/A	6.94	623	63	57
9B	608+00	N/A	N/A	492	49	88
10C	612+03	N/A	7.18	507	54	50
11B	614+60	N/A	4.89	623	63	57
12A	617+00	0.93	N/A	623	63	57
13A- POST 1 (SHOULDER)	621+00	N/A	N/A	785	105	137
13A- POST 2 (MEDIAN)	621+00	1.57	N/A	785	105	137
13C	622+00	N/A	N/A	432	48	85
14B- POST 1 (SHOULDER)	625+02	N/A	N/A	785	105	137
14B- POST 2(MEDIAN)	625+02	1.57	N/A	785	105	137
15A	627+20	N/A	N/A	518	58	92
15B- POST 1 (SHOULDER)	627+10	N/A	N/A	599	64	73
15B- POST 2 (MEDIAN)	627+10	0	N/A	599	64	73
16A	632+25	1.57	N/A	220	34	61
17B	634+25	N/A	N/A	637	67	108
18A	323+51	N/A	N/A	423	51	83
18D- POST 1 (SHOULDER)	322+80	N/A	N/A	562	61	70
18D- POST 2 (MEDIAN)	322+80	1.31	N/A	439	60	66
18F	325+23	N/A	N/A	408	49	82
19A	328+80	N/A	N/A	525	55	93

Table 5 – Recommended Pile Depths

Sign Post No.	Station	Height above Ground Level/ Height of Center Barrier (m)	Height above Bottom of Wall Footing (m)	Pile Depth (Length from pile head to pile tip) (m)
1A	580+00	N/A	N/A	7.0
2D	585+50	N/A	N/A	7.0
2F	583+50	N/A	4.43	11.5
3B- POST 1 (SHOULDER)	586+84	N/A	N/A	8.5
3B- POST 2 (MEDIAN)	586+84	0	N/A	8.5
4A	590+05	1.57	N/A	9.0
5A- POST 1 (SHOULDER)	593+80	N/A	5.29	13.5
5A- POST 2 (MEDIAN)	593+80	1.57	N/A	11.0
5C	594+00	N/A	7.54	19.0
6A	597+20	N/A	5.05	13.0
6B	597+45	N/A	5.03	13.0
6D	599+15	1.57	N/A	8.0
7A	601+25	1.57	N/A	7.0
8B (SHOULDER)	605+25	N/A	N/A	8.5
8B (MEDIAN)	605+25	1.57	N/A	10.5
9A	608+03	N/A	6.94	17.0
9B	608+00	N/A	N/A	7.5
10C	612+03	N/A	7.18	17.0
11B	614+60	N/A	4.89	12.5
12A	617+00	0.93	N/A	8.5
13A- POST 1 (SHOULDER)	621+00	N/A	N/A	8.0
13A- POST 2 (MEDIAN)	621+00	1.57	N/A	9.0
13C	622+00	N/A	N/A	7.0
14B- POST 1 (SHOULDER)	625+02	N/A	N/A	8.0
14B- POST 2 (MEDIAN)	625+02	1.57	N/A	9.5
15A	627+20	N/A	N/A	7.0
15B- POST 1 (SHOULDER)	627+10	N/A	N/A	7.0
15B- POST 2 (MEDIAN)	627+10	0	N/A	7.0
16A	632+25	1.57	N/A	8.0
17B	634+25	N/A	N/A	7.0
18A	323+51	N/A	N/A	8.0
18D- POST 1 (SHOULDER)	322+80	N/A	N/A	8.0
18D- POST 2 (MEDIAN)	322+80	1.31	N/A	8.0
18F	325+23	N/A	N/A	9.0
19A	328+80	N/A	N/A	9.0

A maximum bending moments and maximum shear forces computed are presented in Table 6 below.

Table 6 – Maximum Bending Moments and Maximum Shear Forces

Sign Post No.	Max. BM (kN m)	Depth of Max BM below the pile head (m)	Max. Shear (kN)	Depth of Max Shear below the pile head (m)	Maximum lateral pile head deflection (mm)
1A	620	1.7	180	4.8	2
2D	634	1.5	202	4.9	4
2F	900	4.9	210	8.7	6
3B- POST 1 (SHOULDER)	1405	1.5	299	5.3	4
3B- POST 2 (MEDIAN)	1407	1.7	316	5.3	5
4A	588	3.3	165	7.0	6
5A- POST 1 (SHOULDER)	1350	5.8	258	9.8	8
5A- POST 2 (MEDIAN)	1077	3.5	211	7.7	6
5C	886	7.9	149	11.8	6
6A	935	5.4	183	9.0	5
6B	933	5.5	193	9.3	5
6D	415	2.9	117	5.8	4
7A	264	2.7	88	5.3	4
8B (SHOULDER)	1102	2.2	270	6.6	7
8B (MEDIAN)	1227	3.2	255	7.4	7
9A	1056	7.2	195	10.8	6
9B	554	1.9	158	5.4	4
10C	898	7.8	163	11.3	6
11B	935	5.3	202	8.5	4
12A	740	2.4	185	6.	4
13A- POST 1 (SHOULDER)	908	1.7	202	4.7	3
13A- POST 2 (MEDIAN)	1038	2.8	243	7.0	6
13C	486	1.6	141	4.8	2
14B- POST 1 (SHOULDER)	947	2.3	270	6.0	6
14B- POST 2 (MEDIAN)	1055	3.2	263	7.2	6
15A	581	1.6	169	4.9	3
15B- POST 1 (SHOULDER)	675	1.7	186	5.2	3
15B- POST 2 (MEDIAN)	669	1.6	194	4.8	4
16A	304	2.9	98	5.9	3
17B	720	1.8	240	5.2	4
18A	490	1.9	125	5.7	3
18D- POST 1 (SHOULDER)	641	1.9	165	5.9	3
18D- POST 2 (MEDIAN)	581	2.9	174	6.1	4
18F	483	2.3	112	6.7	3
19A	606	2.2	137	6.5	4

8.0 CONSTRUCTION CONSIDERATIONS

Regarding the installation and construction of CIDH piles, following recommendations should be incorporated in the Special Provisions of the project.

- The contractor shall be required to clean out the bottom of the shaft prior to placing the cage and the concrete.
- Concrete placement for construction of the CIDH piles shall be completed within the same day that excavation of the drilled hole has been completed.
- Some caving should be anticipated during excavation of the pile boring and during CIDH pile construction due to the presence of scattered gravel and cobbles. It will be necessary for the Contractor to utilize a stabilizing method, such as temporary casing, to keep the holes open during construction.
- Corrosion resistant design and construction materials are advised at the location of OH Sign 9B.

9.0 REFERENCES

I. "Foundation Recommendations for Sound Wall Nos. 585, 617, 618, 620-A, 620-B, 621-A and 621-B", dated December 11, 2006, prepared by OGDS-1, and submitted to the D-07 Project Development, Office of Design - Branch D.

II. "Foundation Recommendations for Retaining Wall Nos. 325, 326, 331, 332, 582, 588, 589, 602, Sound Wall Nos. 581,587, 593, 603, 611 and Ten Culvert Extensions along Sound Wall Nos. 581,587, 593, 603 and 611", dated August 10, 2006, prepared by OGDS-1, and submitted to the Office of Bridge Design South 1 – Branch 11.

III. "Final Geotechnical Data Report: Interstate 5- State Route 170 new Connector, Los Angeles County, California", dated December 15, 2005, prepared by URS, and submitted to the OGDS-1- Branch C, California Department of Transportation.

IV. "Foundation Recommendation for CMS Sign" dated August 27, 2007, prepared by OGDS-1, and submitted to the Office of Intelligent Transportation Systems.

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: 9/12/2007

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OGDS1- Sac. File
GS- Sac. File

AERIALLY DEPOSITED LEAD INVESTIGATION REPORT

ROUTE 5 KP 57.6/63.0
LOS ANGELES COUNTY, CALIFORNIA
CONTRACT NO. 43A0078
TASK ORDER NO. 07-121901-3Q

PREPARED FOR

CALIFORNIA DEPARTMENT
OF TRANSPORTATION
DISTRICT 7
ENVIRONMENTAL ENGINEERING
LOS ANGELES COUNTY, CALIFORNIA

PREPARED BY

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GEOCON PROJECT NO. 09100-06-78

JUNE 29, 2004

Project No. 09100-06-78
Task Order No. 07-121901-3Q
June 29, 2004

HAND DELIVERED

Mr. Jack Liu
California Department of Transportation, District 7
Environmental Engineering
801 S. Grand, Suite 1600
Los Angeles, California 90017

Subject: AERIALLY DEPOSITED LEAD INVESTIGATION
ROUTE 5 KP 57.6/63.0
LOS ANGELES COUNTY, CALIFORNIA
CONTRACT NO. 43A0078
TASK ORDER NO. 07-121901-3Q

Dear Mr. Liu:

In accordance with Caltrans Contract No. 43A0078 and Task Order No. 07-121901-3Q dated April 30, 2003, Geocon Consultants, Inc. has performed an aerially deposited lead (ADL) investigation of the soil in the unpaved areas along the north and southbound shoulders of Route 5 from Route 170 to Route 118, Los Angeles County, California. The accompanying report summarizes the services performed, including the advancement of hand-auger borings, limited soil sampling, laboratory analyses, statistical analyses, and Geographical Information Systems (GIS) Surveying. Please call us if you have any questions.

Sincerely,

GEOCON CONSULTANTS, INC.

Michael P. Conkle, RG
Project Geologist

Ronald J. Kofron, CEG 1527
Manager, Environmental Services

MPC:RJK:kor

(5) Addressee

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I. EXECUTIVE SUMMARY

Geocon Consultants, Inc. (Geocon) has performed an aerially deposited lead (ADL) investigation of the soil in the unpaved areas along the north and southbound shoulders of Route 5 from Route 170 to Route 118, Los Angeles County, California. The California Department of Transportation (Caltrans) proposes to excavate soil at the site as part of the widening of this portion of Route 5 to accommodate the additional HOV lanes, and new soundwalls.

The investigation was performed to evaluate the presence of lead resulting from the historical combustion of leaded fuels from freeway traffic. Data from the investigation was used to evaluate the potential reuse or disposal considerations for soil excavated at the site, and to inform Caltrans of potential health and safety issues concerning the presence of lead in soil for workers at the site during construction activities.

Soil samples collected from the site were subsequently analyzed for total lead. Selected samples were analyzed for soluble lead using the Waste Extraction Test method using citric acid (WET-Citric) as the extractant, soluble lead using a modified WET method using deionized water (WET-DI) as the extractant, soluble lead using the Toxicity Characteristic Leaching Procedure (TCLP), and/or pH.

Laboratory analytical results and statistical analysis using one-sided 90 percent upper confidence limits (UCLs) were compared to the guidelines of the Department of Toxic Substances Control (DTSC) Lead Variance issued to Caltrans, Assembly Bill 414, and the DTSC variance modification letter dated December 13, 2002, which modifies the Variance, to develop recommendations for reuse of soil from the site. Offsite disposal conclusions were based upon comparison of the total lead 95 percent UCLs to the California Health and Safety Code (HSC) threshold of 350 milligrams per kilogram (mg/kg), predicted WET-Citric results to the CCR Title 22 soluble lead threshold of 5.0 milligrams per liter (mg/l), and TCLP 95% UCLs to the RCRA threshold of 5.0 mg/l.

Group 1

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 1 is to be disposed of separately, it would likely be classified as a California Class I

hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters were treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 17% (8 out of 46) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 2

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 0.9 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 0.9 meters of soil excavated from Group 2 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 2 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 0.9 meters would likely be classified as non-hazardous. No sample deeper than 0.9 meters was collected. If the entire soil column to a depth of 0.9 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 9% (3 out of 34) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.15 m.

Group 3

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil excavated from Group 3 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the

DTSC Variance. The 90% UCL for WET-DI, based on soil samples with total lead concentrations between 50 and 1,000 mg/kg, was calculated to be 0.91 mg/l. The 90% UCL for WET-DI should be considered a conservative number because only 32% (19 of 58) of the soil samples were analyzed for WET-DI. Samples analyzed for WET-DI were collected at depths up to 0.9 meters.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.6 meters of soil from Group 3 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 19% (11 out of 58) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.6 m.

Group 4

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil excavated from Group 4 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 4 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 40% (21 out of 52) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 5

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 21% (13 out of 62) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 1.5 m. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 6

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 57% (16 out of 28) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 7

Based upon the 90 and 95% UCLs for total lead and predicted WET-Citric results, any portion of the upper 1.5 m of soil excavated may be handled as a non-hazardous material and is suitable for reuse or disposal without restriction. Because there were no samples collected from Group 7 that exhibited a total lead concentration greater than 100 mg/kg, TCLP tests were not performed on any of the samples collected from this Group. Based on the total lead concentrations reported for Group 7, it is unlikely that the soil would be classified as a RCRA hazardous waste.

Group 8

Based upon the 90% UCLs for total lead and WET-DI, if the entire soil column up to 1.5 m beneath the surface were treated as a single unit it may be reused or disposed of without restriction.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 8 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 0.9 m or deeper is treated as a single unit, it would likely be classified as a non-hazardous material and can be re-used or disposed of without restriction. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 7% (5 out of 74) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 9

Based upon the 90% UCLs for total lead and WET-DI, if the entire soil column up to 1.5 m beneath the surface were treated as a single unit it may be reused or disposed of without restriction.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 9 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 0.6 m or deeper meters is treated as a single unit, it would likely be classified as a non-hazardous material and can be reused or disposed of without restriction. Because the average value for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The average, rather than the 95% UCL, for TCLP was used for this data set because the size of the data set was too small for an accurate evaluation of the 95% UCL. The average value should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 2% (2 out of 81) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 10

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance.

The 90% UCL for WET-DI, based on soil samples with total lead concentrations between 50 and 1,000 mg/kg, was calculated to be 0.51 mg/l. The 90% UCL for WET-DI should be considered a conservative number because only 19% (6 of 32) of the soil samples were analyzed for WET-DI. Samples analyzed for WET-DI were collected at depths up to 0.3 meters.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.3 meters of soil from Group 10 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 13% (4 out of 32) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 11

Based on the TCLP 95% UCL data set, if any portion of the upper 0.6 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 30% (6 out of 20) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.6 m. Because no samples were analyzed by the TCLP from depths greater than 0.6 m it is likely that excavation of the entire soil column to a depth of 1.5 m would change the RCRA classification of the soil. Exsitu characterization of the soil may result in a non-RCRA classification.

Based upon the 90% UCLs for total lead and WET-DI soil excavated from depths of 0.6 to 1.5 m would likely be suitable for reuse and disposal without restriction

Group 12

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil excavated from Group 12 is to be reused on-site, it may be used as fill provided that the

soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 12 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 48% (14 out of 29) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 13

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil excavated from Group 13 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if any portion of the upper 1.5 meters of soil from Group 13 is excavated for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 39% (9 out of 23) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 14

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. If any portion of the upper 1.5 meters of soil excavated from Group 14 is to be reused on-site, it may be used as fill provided that the

soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 14 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 19% (8 out of 43) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Bridge No. 53-1123 Widening

Based on the non-statistical evaluation of the results from borings 615-063 and 615-064, collected beneath Bridge No. 53-1123, it appears that soils excavated to a depth of 1.5 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance results.

Based on the reported total lead and WET-Citric results, if the upper 1.5 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Because the reported TCLP results were all less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. Based on the limited results presented in this report, further evaluation with respect to lead at this location appears warranted.

Bridge No. 53-1124 Widening

Based on the non-statistical evaluation of the results from borings 615-087 and 615-088, collected beneath Bridge No. 53-1124, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum

water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance results.

Based on the reported total lead and WET-Citric results, if the upper 0.9 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Because the reported TCLP results were all less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. Based on the limited results presented in this report, further evaluation with respect to lead at this location appears warranted.

Bridge No. 53-1125 Widening

Based on the non-statistical evaluation of the results from borings 615-103 and 615-104, collected beneath Bridge No. 53-1125, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance.

Based on the reported total lead and WET-Citric results, if the upper 0.9 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Since none of the samples collected at this location exceeded 100 mg/kg, and therefore no TCLP analyses were performed, it is unlikely that the soil would be classified as a RCRA hazardous waste. Based on the limited results presented in this report, further evaluation with respect to lead at this location appears warranted.

Caltrans should notify the contractors performing the construction activities that hazardous concentrations of lead may be present in onsite soil and that appropriate health and safety measures should be taken to minimize the exposure to lead.

AERIALLY DEPOSITED LEAD INVESTIGATION REPORT

1. INTRODUCTION

1.1 Project Description and Objectives

Geocon has performed an ADL investigation of the exposed soil in the unpaved areas along the east and westbound shoulders of Route 5 from Route 170 to Route 118, Los Angeles County, California (Figure 1). Caltrans proposes to excavate soil at the site to accommodate the additional HOV lanes and new soundwalls.

The objective of the ADL investigation was to evaluate soil at the site for the presence of lead resulting from the historical combustion of leaded fuels from freeway traffic. The information obtained from the limited soil sampling and laboratory testing was used to determine the method of reuse or disposal of soil excavated during the proposed construction activities at the site. The data was also used to inform Caltrans of potential health and safety issues for workers at the site.

1.2 Scope of Work

Geocon performed the following tasks:

1.2.1 Pre-field Activities

- Attended a Task Order meeting on February 6, 2004, to discuss issues such as field methods, boring locations, health and safety measures, and the completion schedule.
- Prepared a Health and Safety Plan (H&SP) dated February 17, 2004, for the proposed activities. The Health and Safety Plan included guidelines for the use of personal protective equipment for Geocon employees during the field activities. The H&SP specifies the safety procedures for work to be performed at the site, chemical hazard information, site safety officers, and medical emergency locations. The H&SP was prepared as required by Contract 43A0078 in general accordance with 29 CFR 1910.120 and CCR Title 8.
- Contacted Underground Service Alert (USA) to notify utility companies of the field activities. The USA ticket numbers are A500502 and A500493.

1.2.2 Limited Soil Sampling

A 7.62-centimeter diameter hand auger was used to collect 632 soil samples from 159 boring locations from the site between February 19 and February 25, 2004. Boring locations were provided by Caltrans as specified on the figures furnished to Geocon by the Caltrans Contract Manager at the time of the Task Order meeting and by discussions with the Caltrans Contract Manager, for the evaluation of the

subsurface condition at the site. Borings locations were divided into 14 groups and three non-statistical groups based on geographical location as follows:

Statistical Analysis Groups

Group 1 (Wall # 581, Layouts L-1 and L-2)
Boring Numbers: 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, and 15 (11 Borings)

Group 2 (Wall # 582, Layouts L-2, and L-3)
Boring Numbers: 16, 17, 18, 19, 20, 38, 39, 40, and 41 (9 Borings)

Group 3 (Wall # 585 and 587, Layouts L-3, L-4, and L-5)
Boring Numbers: 43, 44, 45, 46, 47, 48, 56, 57, 58, 59, 60, 68, 69, 70 and 71 (15 Borings)

Group 4 (Wall # 588 and 589, Layouts L-3, L-4, and L-5)
Boring Numbers: 33, 34, 35, 36, 37, 49, 50, 51, 52, 53, 54, 55, and 62 (13 Borings)

Group 5 (Wall # 593 Layouts L-5, L-6, and L-7)
Boring Numbers: 65, 66, 67, 72, 73, 74, 75, 76, 77, 78, 79, 80, 81, 82, and 86 (15 Borings)

Group 6 (Wall # 603 Station 602 to 606.5 Layouts L-7 and L-8)
Boring Numbers: 89, 90, 91, 92, 93, 94, 95, and 96 (8 Borings)

Group 7 (Wall # 603 Station 606.5 to 610.5 Layouts L-9 and L-10)
Boring Numbers: 97, 98, 99, 100, 101, and 102 (6 Borings)

Group 8 (Wall # 611 and 612 Layouts L-10, L-11, L-12, and L-13)
Boring Numbers: 105, 106, 107, 108, 109, 110, 111, 112, 113, 114, 115, 116, 117, 118, 119, 120, 121, 122, 127, and 128 (20 Borings)

Group 9 (Layouts L-13, L-14, L-15, and L-16)
Boring Numbers: 129, 130, 131, 132, 133, 134, 139, 140, 141, 142, 143, 144, 145, 146, 147, 148, 149, 150, and 151 (19 Borings)

Group 10 (Layouts L-12 and L-13)
Boring Numbers: 123, 124, 125, 125, 135, 136, 137, and 138 (8 Borings)

Group 11 (Layouts L-1 and L-2)
Boring Numbers: 1, 2, 3, 4, and 21 (5 Borings)

Group 12 (Layout L-2)
Boring Numbers: 22, 23, 24, 25, 26, 27, and 42 (7 Borings)

Group 13 (Route 170, Layouts L-2 and L-19)
Boring Numbers: 30, 31, 32, 162, 163, and 164 (6 Borings)

Group 14 (Route 170 HOV and Future Route 170, Layout L-2, L-18, and L-19)
Boring Numbers: 28, 29, 153, 154, 155, 156, 157, 158, 159, 160, and 161 (11 Borings)

Non-Statistical Analysis Groups

Bridge No. 53-1123 Widening (Layout L-5)
Boring Numbers. 63 and 64 (2 Borings)

Bridge No. 53-1124 Widening (Layout L-7)
Boring Numbers. 87 and 88 (2 Borings)

Bridge No. 53-1125 Widening (Layout L-10)
Boring Numbers. 103 and 104 (2 Borings)

Borings were advanced to a maximum depth of 1.5 meters below the ground surface, and soil samples were collected at 0.15, 0.3, 0.6, 0.9, and 1.5 meters below the ground surface. The approximate boring locations are shown on the Boring Location Maps (L-1 to L-19). The borings were subsequently backfilled with the soil cuttings generated.

1.2.3 Laboratory Analyses

Geocon submitted the soil and water samples under chain of custody procedures to Advanced Technology Laboratories (ATL), a California Department of Health Services (CDOHS)-certified analytical laboratory. All soil samples were analyzed for total lead following United States Environmental Protection Agency (EPA) Test Method 6010B. Soil samples exhibiting total lead concentrations greater than or equal to 50 milligrams per kilograms (mg/kg) were analyzed for soluble lead following EPA Test Method 7420 using the WET-Citric method. Samples exhibiting WET-Citric concentrations greater than or equal to 5.0 milligrams per liter (mg/l) were analyzed for soluble lead following EPA Test Method 7420 using the WET-DI method. Samples exhibiting total lead concentrations greater than or equal to 100 mg/kg were analyzed for soluble lead following EPA Test Method 1311 using the TCLP. In addition, ten percent of the soil samples were analyzed for pH following EPA Test Method 9045.

A total of sixty-four equipment blanks were analyzed for total lead using EPA Test Method 6010B.

1.2.4 GPS Surveying

Each boring location was recorded using a Global Positioning System (GPS) receiver. Data was recorded using the Axis III™ receiver system, using State Plane 83 coordinates, with the IMAP™ software package. Boring location coordinates, in latitude and longitude, are provided in Table I.

Each boring location, sample, and analytical test result was entered into a Microsoft 2000 Access database in a format provided by Caltrans. Boreholes were given a unique three digit ID assigned by Caltrans followed by a dash and then sequential numbering beginning with "001." The unique three digit ID for this EA is 615. The Microsoft 2000 Access database will be provided to Caltrans in an electronic submittal.

1.2.5 Report Preparation

This report was prepared as outlined in Contract No. 43A0078 and Task Order No. 07-121901-3Q summarizing the results of the aerially deposited lead investigation activities requested by Caltrans.

1.3 Previous Site Investigations

Geocon has not performed a previous investigation at the site. In addition, Caltrans has not notified Geocon of previous investigations performed at the site.

2. BACKGROUND

2.1 Aerially Deposited Lead in Soil

Testing by Caltrans throughout the State has shown that aerially deposited lead exists in soil along major freeway routes resulting from automobile exhaust containing lead from the combustion of leaded gasoline. Elevated lead concentrations are generally found within 9.1 meters of the edge of pavement and within the top 0.15 meters of soil. Elevated lead concentrations can also be present as deep as 0.60 to 0.90 meters below the surface. The concentration and distribution of aerially deposited lead in soil is dependent on many variables, but in general, traffic volume and age of a highway are the primary factors.

2.2 Hazardous Waste Classification Criteria

Regulatory criteria to classify a waste as "California hazardous" for handling and disposal purposes are contained in the CCR, Title 22, Division 4.5, Chapter 11, Article 3, §66261.24. Criteria to classify a waste as "Resource, Conservation and Recovery Act (RCRA) hazardous" are contained in Chapter 40 of the Code of Federal Regulations (40 CFR), §261.

For a waste containing metals, the waste is classified as "California hazardous" when: (1) the total metal content exceeds the Total Threshold Limit Concentration (TTLC); or (2) the soluble metal content exceeds the Soluble Threshold Limit Concentration (STLC) based on a Waste Extraction Test

(WET) analysis. A material is classified as "RCRA hazardous" when the soluble metal content exceeds the Federal Regulatory Level based on Toxicity Characteristic Leaching Procedure (TCLP) testing.

The above regulatory criteria are based on toxicity. Wastes may also be classified as hazardous based on other criteria including ignitability, toxicity, corrosivity, and reactivity. However, for the purposes of ADL investigations, toxicity and corrosivity (e.g., chemical concentrations and soil pH values, respectively) are the primary factors considered for waste classification. Waste that is classified as either "California hazardous" or "RCRA hazardous" requires management as a hazardous waste and disposal at an approved disposal facility.

According to §25157.8 of the HSC, after January 1, 1999, no person shall dispose of waste that contains total lead in excess of 350 mg/kg to land other than a Class I hazardous waste disposal facility.

2.3 DTSC Variance

The DTSC issued a variance to selected Caltrans Districts on September 22, 2000, to provide guidance for the disposition of soil containing ADL within Caltrans projects. The California State Assembly passed Assembly Bill (AB) 414, dated October 14, 2001, which allows Caltrans to reuse lead-impacted soil within their right-of-way provided that total lead concentrations do not exceed 1,496 mg/kg. The DTSC further modified the variance in a letter dated December 13, 2002, allowing lead-impacted soil to be reused on-site provided that total lead concentrations do not exceed 3,397 mg/kg. Review of the original and revised variance and AB 414 regarding Caltrans' reuse and management of ADL-impacted soil as fill material for construction and maintenance operations indicates the following conditions.

2.3.1 Condition 1 – Cover with Non-Hazardous Soil

Soil exhibiting soluble lead concentrations less than or equal to 0.5 mg/l (WET-DI) and total lead concentrations of 1,411 mg/kg or less may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil. Contaminated soil with a pH less than 5.0 shall only be used as fill material under the paved portion of the roadway as described in Condition 3 below.

2.3.2 Condition 2 – Cover with Pavement Structure

Soil exhibiting soluble lead concentrations greater than 0.5 mg/l and less than 50 mg/l (WET-DI) and/or total lead concentrations more than 1,411 mg/kg but less than 3,397 mg/kg may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans. Contaminated soil with a pH less than 5.0 shall only be used as fill material under the paved portion of the roadway as described in Condition 3 below.

2.3.3 Condition 3

Contaminated soil with a pH less than 5.0 may be used as fill material only under the paved portion of the roadway. Condition 3 prevails under either Condition 1 or 2.

2.4 Criteria for Disposal of Soil Not Intended for Reuse Onsite

If the excavated soil is not intended to be reused within the Caltrans right-of-way, then hazardous waste determination of the soil is based on total and soluble lead concentrations using the lead TTLC and STLC contained in Title 22 of the *CCR* Article 3, §66261.24. When the total lead concentration is greater than ten times the lead STLC, regulatory agencies typically require the WET using citric acid. It is the result from the WET that is compared to the STLC. The TTLC for lead is 1,000 mg/kg and the STLC for lead using acid extract is 5.0 mg/l. However, as previously indicated, disposal of waste that contains total lead in excess of 350 mg/kg to land other than a Class I hazardous waste disposal facility (or other designated facility meeting all the criteria in HSC 25157.8(3)(b)) is prohibited.

3. INVESTIGATIVE METHODS

3.1 Field Methods

3.1.1 Soil Sampling

Soil sampling and handling methods used by Geocon to complete this Task Order are outlined in the Geocon Standard Operating Procedure (SOP) *Modified SOP No. 11 - Hand-Augering and Soil Sample Collection/Handling Procedures*, presented as Appendix A.

3.1.2 Equipment Blank Sampling

One equipment blank sample was collected per chain-of-custody (every ten soil samples) to verify proper cleaning of the sampling equipment. The equipment blank sample was obtained by passing distilled water over the decontaminated sampling equipment and into laboratory-provided containers.

3.2 Deviations from Work Plan

A work plan was not prepared for this TO; however, Geocon performed the scope of work as described in the TO with the following exceptions:

- Borings 615-001, 615-005, 615-006, 615-008, 615-022, 615-031, 615-052, 615-056, 615-057, 615-059, 615-063, 615-064, 615-072, 615-073, 615-074, 615-075, 615-077, 615-089, 615-094, 615-101, 615-102, 615-114, 615-117, 615-118, 615-124, 615-128, 615-132, 615-141, 615-144, 615-146, 615-148, and 615-160 were the only borings completed to the planned total depth of 1.5 meters. The other borings were not completed to the proposed depth because of refusal due to the presence of large rocks.

- Borings 615-061, 615-083, 615-084, 615-085 and 615-152 were not installed due to paving in the proposed locations.

4. INVESTIGATIVE RESULTS AND FIELD OBSERVATIONS

4.1 Site Geology and Hydrology

The soil conditions encountered consisted generally of loose to moderately dense, dry, brown to dark-brown, clayey sand with coarse gravel. Groundwater was not encountered in the hand-auger borings.

4.2 Analytical Laboratory Results

A summary of the results of the laboratory analyses for total lead, WET-Citric, WET-DI, TCLP, and pH is presented in Table I. Reproductions of the laboratory reports and chain-of-custody documentation are presented as Appendix B. All analyses were processed using laboratory seven-business-day turn-around times. The sixty-four equipment blank water samples were analyzed for total lead. All concentrations were below the detection limit of 0.005 mg/l. Soil sample analytical results are summarized as follows (see Section 1.2.3 for analytical methods used).

4.2.1 Lead and pH Results – Group 1

Soil sample analytical results for Group 1 are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Forty-six soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 990 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Twenty-one soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 5.1 mg/l to 84 mg/l;

- **WET-DI –** Twenty-one samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.68 mg/l;

- **TCLP** – Eight soil samples exhibiting the highest total lead concentrations were analyzed by the TCLP method. The TCLP concentrations ranged from below the laboratory reporting limit of 0.25 mg/l to 3.2 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Four soil samples were tested for pH. Values ranged from 7.19 to 8.01, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.2 Lead and pH Results – Group 2

Soil sample analytical results for Group 2 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Thirty-four soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 350 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Ten soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 3.5 mg/l to 25 mg/l;

- **WET-DI –** Eight samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.52 mg/l;

- **TCLP** – Three soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 1.1 mg/l to 2.1 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Three soil samples were tested for pH. Values ranged from 7.31 to 8.74, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.3 Lead and pH Results – Group 3

Soil sample analytical results for Group 3 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Fifty-eight soil samples were analyzed for total lead. Concentrations ranged from below the laboratory reporting limit of 5.0 mg/kg to 700 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –**Twenty-six soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 3.6 mg/l to 99 mg/l;

- **WET-DI –** Twenty-four samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 2.1 mg/l;

- **TCLP** – Eleven soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from below the laboratory reporting limit of 0.25 mg/l to 1.9 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Seven soil samples were tested for pH. Values ranged from 7.43 to 8.99, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.4 Lead and pH Results – Group 4

Soil sample analytical results for Group 4 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Fifty-two soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 650 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Thirty-five soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 2.6 mg/l to 80 mg/l;

- **WET-DI –** Thirty-one samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.91 mg/l;

- **TCLP** – Twenty-four soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from below the laboratory reporting limit of 0.25 mg/l to 2.9 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Six soil samples were tested for pH. Values ranged from 7.07 to 8.03, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.5 Lead and pH Results – Group 5

Soil sample analytical results for Group 5 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Sixty-two soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 900 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Thirty-one soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 4.5 mg/l to 110 mg/l;

- **WET-DI –** Twenty-six samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 5.9 mg/l;

- **TCLP** – Thirteen soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.58 mg/l to 12 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Five soil samples were tested for pH. Values ranged from 7.04 to 7.61, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.6 Lead and pH Results – Group 6

Soil sample analytical results for Group 6 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Twenty-seven soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 3,400 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Nineteen soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 0.26 mg/l to 140 mg/l;

- **WET-DI –** Thirteen samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from 0.27 mg/l to 8.4 mg/l;

- **TCLP** – Eighteen soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations (including re-analysis) ranged from 0.44 mg/l 22 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Six soil samples were tested for pH. Values ranged from 7.19 to 8.54, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.7 Lead and pH Results – Group 7

Soil sample analytical results for Group 7 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Twenty-five soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 52 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** One soil sample exhibited a total lead concentration greater than 50 mg/kg, and was analyzed using the WET-Citric method. The WET-Citric concentrations was 4.1 mg/l;

- **WET-DI –** No samples were analyzed using the WET-DI method;

- **TCLP** – No soil samples were analyzed by the TCLP method; and

- **pH –** Two soil samples were tested for pH. The reported values were 7.46 and 8.41, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.8 Lead and pH Results – Group 8

Soil sample analytical results for Group 8 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Seventy-four soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 410 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Ten soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations (including re-analysis) ranged from 5.3 mg/l to 30 mg/l;

- **WET-DI –** Nine samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.69 mg/l;

- **TCLP** – Five soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from less than the laboratory reporting limit of 0.25 mg/l to 1.4 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Eight soil samples were tested for pH. Values ranged from 7.32 to 8.61, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.9 Lead and pH Results – Group 9

Soil sample analytical results for Group 9 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Eighty-one soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 460 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Seven soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 4.4 mg/l to 28 mg/l;

- **WET-DI –** Six samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations (including re-analysis) ranged from below the laboratory reporting limit of 0.25 mg/l to 0.71;

- **TCLP** – Two soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations were 0.63 mg/l and 8.7 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Seven soil samples were tested for pH. Values ranged from 7.81 to 8.90, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.10 Lead and pH Results – Group 10

Soil sample analytical results for Group 10 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Thirty-two soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 260 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Ten soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 3.6 mg/l to 36 mg/l;

- **WET-DI –** Six samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.60 mg/l;

- **TCLP** – Four soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.60 mg/l to 3.7 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Four soil samples were tested for pH. Values ranged from 8.42 to 8.68, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.11 Lead and pH Results – Group 11

Soil sample analytical results for Group 11 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Twenty soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 1,600 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Ten soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 4.6 mg/l to 64 mg/l;

- **WET-DI –** Ten samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 2.4 mg/l;

- **TCLP** – Six soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.77 mg/l to 7.6 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Three soil samples were tested for pH. Values ranged from 8.16 to 8.42, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.12 Lead and pH Results – Group 12

Soil sample analytical results for Group 12 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Twenty-nine soil samples were analyzed for total lead. Concentrations ranged from 7.7 mg/kg to 870 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Twenty-two soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 3.8 mg/l to 64 mg/l;

- **WET-DI –** Twenty samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.81 mg/l;

- **TCLP** – Fourteen soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.63 mg/l to 4.0 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Three soil samples were tested for pH. Values ranged from 7.85 to 9.02, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.13 Lead and pH Results – Group 13

Soil sample analytical results for Group 13 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Twenty-three soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 510 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Twenty soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-Citric concentrations ranged from 4.5 mg/l to 74 mg/l;

- **WET-DI –** Fifteen samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.79 mg/l;

- **TCLP** – Nine soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.70 mg/l to 7.1 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Two soil samples were tested for pH. The reported values were 8.45 and 8.51, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.14 Lead and pH Results – Group 14

Soil sample analytical results for Group 14 lanes are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Forty-three soil samples were analyzed for total lead. Concentrations ranged from below the reporting limit of 5.0 mg/kg to 270 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Fourteen soil samples exhibited a total lead concentration greater than 50 mg/kg, and were analyzed using the WET-Citric method. The WET-DI concentrations in the samples ranged from 1.5 mg/l to 29 mg/l;

- **WET-DI –** Eleven samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 0.47 mg/l;

- **TCLP** – Eight soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.57 mg/l to 2.6 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Six soil samples were tested for pH. Values ranged from 7.46 to 9.18, which are above the minimum pH of 5.0 described in the DTSC variance.

4.2.15 Lead and pH Results – Bridge 53-1123

Soil sample analytical results for Bridge 53-1123 are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Ten soil samples were analyzed for total lead. Concentrations ranged from 50 mg/kg to 400 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Ten soil samples exhibited a total lead concentration greater than or equal to 50 mg/kg, and were analyzed using the WET-Citric method. The WET-DI concentrations in the samples ranged from 4.8 mg/l to 54 mg/l;

- **WET-DI –** Nine samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples ranged from below the laboratory reporting limit of 0.25 mg/l to 1.3 mg/l;

- **TCLP** – Seven soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from 0.85 mg/l to 3.1 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** One soil sample was tested for pH. The value was 7.75, which is above the minimum pH of 5.0 described in the DTSC variance.

4.2.16 Lead and pH Results – Bridge 53-1124

Soil sample analytical results for Bridge 53-1124 are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Nine soil samples were analyzed for total lead. Concentrations ranged from 5.4 mg/kg to 1,300 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Three soil samples exhibited a total lead concentration greater than or equal to 50 mg/kg, and were analyzed using the WET-Citric method. The WET-DI concentrations in the samples ranged from 2.2 mg/l to 32 mg/l;

- **WET-DI –** Two samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples were 0.31 mg/l and 2.4 mg/l;

- **TCLP** – Three soil samples exhibiting the highest total lead concentrations and were analyzed by the TCLP method. The TCLP concentrations ranged from below the laboratory reporting limit of 0.25 mg/l to 4.4 mg/l. The TCLP threshold for RCRA waste for lead is 5.0 mg/l; and

- **pH –** Two soil samples were tested for pH. The values were 7.20 and 7.26, which is above the minimum pH of 5.0 described in the DTSC variance.

4.2.17 Lead and pH Results – Bridge 53-1125

Soil sample analytical results for Bridge 53-1125 are summarized as follows (see Section 1.2.3 for analytical methods used):

- **Total Lead –** Seven soil samples were analyzed for total lead. Concentrations ranged from below the laboratory reporting limit of 5.0 mg/kg to 96 mg/kg. HSC specifies 350 mg/kg as the total lead threshold;

- **WET-Citric –** Two soil samples exhibited a total lead concentration greater than or equal to 50 mg/kg, and were analyzed using the WET-Citric method. The WET-DI concentrations in the samples were 9.0 mg/l and 9.1 mg/l;

- **WET-DI –** Two samples exhibited a WET-Citric concentration greater than 5.0 mg/l and were analyzed using the WET-DI method. The WET-DI concentrations in the samples were below the laboratory reporting limit of 0.25 mg/l;

- **TCLP** – No soil samples were analyzed by the TCLP method; and

- **pH –** One soil sample was tested for pH. The value was 7.46, which is above the minimum pH of 5.0 described in the DTSC variance.

4.3 Data Validation

Geocon and ATL use QA/QC measures to minimize and control errors associated with field and laboratory methods. Field QA/QC measures consist of cleaning sampling equipment between each use with a detergent solution followed by successive rinses in tap and deionized water. Geocon considers the field investigation free from potential cross-contamination resulting from inadequate equipment decontamination.

Laboratory QA/QC measures include the use of matrix spikes, duplicates and method blanks, in addition to calculation of percent recovery and relative percentage difference (RPD). A review of the laboratory QA/QC results indicates satisfactory data reporting.

5. DATA EVALUATION

5.1 Lead Distribution Analysis

The results of the analytical testing indicate that 32 soil samples were above the California disposal threshold of 350 mg/kg total lead content.

5.2 Statistical Evaluation Methods

The analytical laboratory results were evaluated statistically to examine the appropriate method of reuse or off-site disposal of the soils. Prior to performing the following calculations, analytical results reported as below the detection limit were assigned a value of one-half the detection limit. In instances where samples were reanalyzed the average results were used in the calculations. Statistical methods were applied to the lead data set collected adjacent to the site to evaluate: 1) if an acceptable correlation between total and soluble lead concentrations exists that would allow the prediction of soluble lead concentrations based on calculated UCLs; 2) the total lead data population distribution; and 3) the one-sided upper-confidence limits (UCLs) on the true means of the total lead concentrations for different soil mixing scenarios.

Statistical methods were also applied to the WET-DI and TCLP lead data sets to evaluate the one-sided UCLs on the true means of the soluble lead concentrations using these analytical methods.

The total lead and WET-Citric data was divided into the following subsets for evaluation of population distribution and correlation and regression analysis:

- Group 1, and Groups 4 through Group 9 (Northbound);
- Group 2, Group 3, and Groups 10 through 14 (Southbound).
- Borings 615-063, 615-064, 615-087, 615-088, 615-103, and 615-104 were conducted for bridge widening projects and at the request of Caltrans were not included in the statistical analysis.

5.3 Data Correlation

A test for data correlation is used to verify the quality of the equation used to predict soluble lead concentrations. There should be a correlation coefficient (“r”) of 0.8 or greater between total and soluble lead (WET-citric) analytical results. The correlation coefficients for were 0.9150 and 0.9132 for the northbound and southbound groups, respectively.

5.4 Regression Analysis

A linear regression analysis is necessary to create a soluble lead prediction model for use with the 90-percent and 95-percent UCLs. The model is created by plotting the total lead and soluble lead (WET-Citric) paired data points on a scatter plot chart. A linear regression line is then added to the chart using the equation:

$$y = mx + b$$

where:

y = *WET Citric result, mg/l*

x = *total lead result, mg/kg*

b = *the y-intercept*

$$m = Slope = \frac{r \times s_t}{s_s}$$

where:

r = *correlation coefficient*

s_t = *standard deviation of the total lead results*

s_s = *standard deviation of the soluble lead results*

The linear equation corresponding to the regression line is then used to predict a soluble lead concentration for the statistical total lead UCLs. The integrity of the equation is directly related to the correlation coefficient described in Section 5.3.

5.5 Population Distribution

A test for population distribution is necessary to apply the appropriate methods when examining the UCLs on the true total lead means. When evaluating the distribution of total lead concentrations, all total lead data from each area were treated as one data set. In accordance with *Chapter Nine, SW-846, 3rd Edition, U.S. Environmental Protection Agency, 1986,* (*Chapter Nine, SW-846),* distribution was evaluated by comparing the mean versus the variance of the total lead data sets. If the mean was greater than the variance, the data set was assumed to be normally distributed and transformation was not performed. If the mean was less than the variance, the data set was transformed using an arcsine conversion. If the mean was approximately equal to the variance, the data set was transformed using a square-root conversion.

5.6 Calculating the Upper Confidence Limits for the True Mean

Statistical confidence limits are the classical tool for addressing uncertainties of a distribution mean. The UCLs of the true mean concentration are used as the mean concentrations because it is not

possible to know the true mean. The UCLs therefore account for uncertainties due to limited sampling data. As more data are available for a given site, uncertainty decreases and the UCLs move closer to the true mean.

A 90 percent UCL is desired if the soil is to be reused on-site and a 95 percent UCL is desired if the soil is to be disposed of offsite or relinquished to a contractor as described in Task Order No. 07-121901-3Q. The maximum 90 percent UCL allowed for reuse of on-site soil is 3,397 mg/kg and the maximum 95 percent UCL allowed for disposal is 350 mg/kg. The one-sided 90- and 95 percent UCLs of the true mean are defined as the values that, when calculated repeatedly for randomly drawn subsets of site data, equal or exceed the true mean 90 and 95 percent of the time, respectively. The following statistical equation (from *Chapter Nine, SW-846)* was used to calculate the UCLs:

Where:

$\bar{x}$ = sample mean
t_p = student's t for a one-tailed confidence interval and a probability of p
S = standard deviation
n = number of samples

For the purpose of this investigation, the samples were assumed to be collected using systematic random sampling. *Chapter Nine of SW-846* indicates that a statistical transformation should be used if the data set is not normally distributed, and statistical evaluations should be performed on the transformed scale. Based on calculation of the mean and variance of the data sets and visual interpretation of the data, the data sets should be transformed.

The mean was less than the variance for the non-transformed data indicating that the data set was not normally distributed and transformation was necessary. The raw data was transformed using the arcsine transformation. The arcsine transformation was accomplished by dividing each total lead result by the maximum concentration (this results in a data set of all numbers falling between 0 and 1), calculating the arcsine of the quotient ($y_i = \text{arcsine}(x_i/x_{max})$), performing the statistical calculations on the transformed data, and finally re-converting the result to real numbers ($z_i = x_{max}\sin y_i$).

In order to evaluate different soil excavation scenarios, different UCLs were calculated. Data was divided into the following data sets:

- Total lead concentrations for soil samples collected from 0 to 0.15 meter (Data Set A);
- Total lead concentrations for soil samples collected from 0.15 to 0.3 meter (Data Set B);
- Total lead concentrations for soil samples collected from 0.45 to 0.6 meter (Data Set C);

- Total lead concentrations for soil samples collected from 0.75 to 0.9 meter (Data Set D); and
- Total lead concentrations for soil samples collected from 1.35 to 1.5 meters (Data Set E).

Using the data sets above, the following UCLs for the true means were calculated:

- UCL for the top 0.15 meter of soil (Data Set A) and the UCL for the underlying soil (Data Sets B, C, D and E);
- UCL for the top 0.3 meter of soil (Data Sets A and B) and the UCL for the underlying soil (Data Sets C, D, and E);
- UCL for the top 0.6 meter of soil (Data Sets A, B, and C) and the UCL for the underlying soil (Data Sets D and E);
- UCL for the top 0.9 meter of soil (Data Sets A, B, and C) and the UCL for the underlying soil (Data Set E); and
- UCL for the entire 1.5 meters soil column (Data Sets A, B, C, D and E).

Group 2 was only divided into four data sets since no 1.5-meter samples were collected from that Group. For reference, tables summarizing the results of the 90 percent and 95 percent UCLs and predicted soluble lead concentrations are presented below along with reuse and disposal conditions. Additionally, soil excavation and mixing scenarios can be found on the block diagrams in Appendix C. Where all statistical results were reported as less than the laboratory detection limit, the UCL was also reported as “less than” the detection limit.

Lead Analysis and Soil Management Summary – Group 1

Route 5 HOV Project – Statistical Analysis Group 1						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	383.55	437.75	40.79	46.73	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	69.13	73.15	6.33	6.77	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	231.11	257.33	24.09	26.96	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	62.41	67.07	5.60	6.11	YES (Y)	Class I Landfill (Z-2)
0 to 0.60 meters	171.69	189.09	17.57	19.48	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	52.55	58.48	4.52	5.17	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	138.36	151.76	13.92	15.39	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	46.00	43.11	3.80	3.48	YES (X)	Non-Hazardous (X)
0 to 1.5 meters	131.44	143.96	13.16	14.53	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				2.26		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			0.24		Condition 1	

Lead Analysis and Soil Management Summary – Group 2

Route 5 HOV Project – Statistical Analysis Group 2						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	219.09	239.21	21.47	23.45	YES (Y)	Class I Landfill (Z-2)
0.15 to 0.9 meters	38.70	91.01	3.68	3.92	YES (X)	Non-Hazardous (X)
0 to 0.30 meters	133.12	145.05	12.99	14.17	YES (Y)	Class I Landfill (Z-2)
0.31 to 0.9 meters	41.00	44.54	3.91	4.26	YES (X)	Non-Hazardous (X)
0 to 0.60 meters	97.80	106.15	9.51	10.33	YES (Y)	Class I Landfill (Z-2)
0.61 to 0.9 meters	59.32	66.18	5.71	6.39	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	84.29	91.01	8.18	8.84	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.15 meters				1.90		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.6 meters			0.31		Condition 1	

UCL Lead Analysis and Soil Management Summary – Group 3

Route 5 HOV Project – Statistical Analysis Group 3						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	254.17	282.15	24.93	27.69	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	56.07	59.48	5.39	5.73	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	156.53	170.75	15.30	16.70	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	50.83	55.05	4.88	5.29	YES (Y)	Class I Landfill (Z-2)
0 to 0.60 meters	120.47	130.10	11.74	12.69	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	32.25	36.14	3.05	3.43	YES (X)	Non-Hazardous (X)
0 to 0.90 meters	102.57	110.54	9.98	10.76	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	<5.0	<5.0	<0.25	<0.25	YES (X)	Non-Hazardous (X)
0 to 1.5 meters	97.54	105.13	9.48	10.23	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.6 meters				1.46		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			0.91		Condition 2	

Lead Analysis and Soil Management Summary – Group 4

Route 5 HOV Project – Statistical Analysis Group 4						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	314.93	341.53	33.27	36.19	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	115.27	121.23	11.39	12.04	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	237.76	251.63	24.81	26.33	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	78.05	84.03	7.31	7.97	YES (Y)	Class I Landfill (Z-2)
0 to 0.60 meters	185.12	195.29	19.05	20.16	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	60.92	65.75	5.43	5.96	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	152.29	160.50	15.45	16.35	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	19	19	0.84	0.84	YES (X)	Non-Hazardous (X)
0 to 1.5 meters	149.88	157.96	11.39	16.07	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				1.85		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			0.37		Condition 1	

Lead Analysis and Soil Management Summary – Group 5

Route 5 HOV Project – Statistical Analysis Group 5						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	351.87	387.59	37.32	41.24	NO	Class I Landfill (Z-2)
0.15 to 1.5 meters	103.78	113.08	10.13	11.15	NO	Class I Landfill (Z-2)
0 to 0.30 meters	243.12	264.00	25.40	27.69	NO	Class I Landfill (Z-2)
0.31 to 1.5 meters	64.64	68.44	5.84	6.26	NO	Class I Landfill (Z-2)
0 to 0.60 meters	179.20	193.50	18.40	19.96	NO	Class I Landfill (Z-2)
0.61 to 1.5 meters	69.78	74.72	6.40	6.95	NO	Class I Landfill (Z-2)
0 to 0.90 meters	146.68	157.73	14.83	16.04	NO	Class I Landfill (Z-2)
0.91 to 1.5 meters	130.08	141.13	13.01	14.22	NO	Class I Landfill (Z-2)
0 to 1.5 meters	140.68	150.88	14.18	15.29	NO	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 1.5 meters				6.33		Potential RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 1.5 meters			1.56		Condition 2	a

Lead Analysis and Soil Management Summary – Group 6

Route 5 HOV Project – Statistical Analysis Group 6						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	2,356.56	2,591.35	257.04	282.77	NO	Class I Landfill (Z-2)
0.15 to 1.5 meters	395.60	433.01	42.11	46.21	NO	Class I Landfill (Z-2)
0 to 0.30 meters	1,539.63	1,689.44	167.50	183.92	NO	Class I Landfill (Z-2)
0.31 to 1.5 meters	223.31	246.40	23.23	25.76	NO	Class I Landfill (Z-2)
0 to 0.60 meters	1,130.81	1,239.07	122.69	134.56	NO	Class I Landfill (Z-2)
0.61 to 1.5 meters	185.06	212.71	19.04	22.07	NO	Class I Landfill (Z-2)
0 to 0.90 meters	979.72	1,073.05	106.13	116.36	NO	Class I Landfill (Z-2)
0.91 to 1.5 meters	1.26	<5.0	<0.25	<0.25	NO	Class I Landfill (Z-2)
0 to 1.5 meters	914.22	1,001.66	98.95	108.54	NO	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				8.03		Potential RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			3.52		Condition 2	

Lead Analysis and Soil Management Summary – Group 7

Route 5 HOV Project – Statistical Analysis Group 7						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	31.43	34.87	2.20	2.58	NO (X)	Non-Hazardous (X)
0.15 to 1.5 meters	16.20	17.83	0.53	0.71	NO (X)	Non-Hazardous (X)
0 to 0.30 meters	25.82	28.45	1.59	1.87	NO (X)	Non-Hazardous (X)
0.31 to 1.5 meters	13.16	14.41	0.20	0.34	NO (X)	Non-Hazardous (X)
0 to 0.60 meters	21.59	23.39	1.12	1.32	NO (X)	Non-Hazardous (X)
0.61 to 1.5 meters	6.10	6.59	<0.25	<0.25	NO (X)	Non-Hazardous (X)
0 to 0.90 meters	17.96	19.43	0.73	0.89	NO (X)	Non-Hazardous (X)
0.91 to 1.5 meters	2.25	<5.0	<0.25	<0.25	NO (X)	Non-Hazardous (X)
0 to 1.5 meters	16.93	18.29	0.61	0.76	NO (X)	Non-Hazardous (X)
TCLP 95% UCL (mg/l) 0 to 1.5 meters				Not Available		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 1.5 meters			Not Available		Does Not Apply	

Lead Analysis and Soil Management Summary – Group 8

Route 5 HOV Project – Statistical Analysis Group 8						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	133.67	147.00	13.41	14.87	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	21.5	22.82	1.11	1.26	YES (X)	Non-Hazardous (X)
0 to 0.30 meters	77.75	84.67	7.28	8.04	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	19.92	21.34	0.94	1.10	YES (X)	Non-Hazardous (X)
0 to 0.60 meters	59.32	64.14	5.26	5.79	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	16.02	17.54	0.51	0.68	NO (X)	Non-Hazardous (X)
0 to 0.90 meters	51.25	55.27	4.37	4.81	NO (X)	Non-Hazardous (X)
0.91 to 1.5 meters	5.38	5.70	<0.25	<0.25	NO (X)	Non-Hazardous (X)
0 to 1.5 meters	48.71	52.52	4.09	4.51	NO (X)	Non-Hazardous (X)
TCLP 95% UCL (mg/l) 0 to 0.3 meters				1.31		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.6 meters			0.41		Condition 1	

Lead Analysis and Soil Management Summary – Group 9

Route 5 HOV Project – Statistical Analysis Group 8						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	111.23	125.64	10.95	12.53	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	18.39	19.71	0.77	0.92	YES (X)	Non-Hazardous (X)
0 to 0.30 meters	66.02	73.16	5.99	6.77	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	14.39	15.57	0.33	0.46	NO (X)	Non-Hazardous (X)
0 to 0.60 meters	47.42	52.18	3.95	4.48	NO (X)	Non-Hazardous (X)
0.61 to 1.5 meters	16.53	18.39	0.57	0.77	NO (X)	Non-Hazardous (X)
0 to 0.90 meters	38.10	41.68	2.93	3.32	NO (X)	Non-Hazardous (X)
0.91 to 1.5 meters	6.52	7.14	<0.25	<0.25	NO (X)	Non-Hazardous (X)
0 to 1.5 meters	35.99	39.35	2.70	3.07	NO (X)	Non-Hazardous (X)
Average* TCLP (mg/l) 0 to 0.3 meters				4.67		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			0.64		Condition 2	

Note: * Average TCLP values were used because the size of the data set (2 samples) does not allow for an accurate evaluation of the 95% UCL.

Lead Analysis and Soil Management Summary – Group 10

Route 5 HOV Project – Statistical Analysis Group 10						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	169.18	185.27	16.55	18.13	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	60.96	66.78	5.88	6.45	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	122.28	132.43	11.92	12.92	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	41.28	44.67	3.94	4.27	YES (X)	Non-Hazardous (X)
0 to 0.60 meters	97.98	97.98	8.80	9.53	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	61.55	67.85	5.94	6.56	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	80.71	86.65	7.82	8.41	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	<5.0	<5.0	<0.25	<0.25	YES (X)	Non-Hazardous (X)
0 to 1.5 meters	78.38	84.17	7.59	8.17	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.3 meters				3.62		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.3 meters			0.51		Condition 2	

Lead Analysis and Soil Management Summary – Group 11

Route 5 HOV Project – Statistical Analysis Group 11						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	579.44	617.25	57.00	60.73	NO	Class I Landfill (Z-2)
0.15 to 1.5 meters	460.14	532.32	45.24	52.35	NO	Class I Landfill (Z-2)
0 to 0.30 meters	808.64	911.41	79.60	89.73	NO	Class I Landfill (Z-2)
0.31 to 1.5 meters	83.44	93.20	8.09	9.06	NO	Class I Landfill (Z-2)
0 to 0.60 meters	570.54	640.74	56.12	63.04	NO	Class I Landfill (Z-2)
0.61 to 1.5 meters	36.51	39.57	3.47	3.77	NO	Non-Hazardous (X)
0 to 0.90 meters	459.55	515.40	45.18	50.68	NO	Class I Landfill (Z-2)
0.91 to 1.5 meters	<5.0	<5.0	<0.25	<0.25	NO	Non-Hazardous (X)
0 to 1.5 meters	437.46	490.61	43.00	48.24	NO	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.6 meters				6.23		Potential RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.6 meters			1.53		Condition 2	

Lead Analysis and Soil Management Summary – Group 12

Route 5 HOV Project – Statistical Analysis Group 12						
Layer (s)	**TTLC**		**STLC**		**SOIL HANDLING**	
	90% UCL (mg/kg)	**95% UCL (mg/kg)**	**90% UCL (mg/kg)**	**95% UCL (mg/kg)**	**Invoke Variance (Soil Type)**	**Surplus Soil (Soil Type)**
0.0 to 0.15 meters	549.91	615.48	54.09	60.55	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	183.57	196.47	17.97	19.24	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	345.94	380.86	33.98	37.42	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	208.47	226.80	20.42	22.23	YES (Y)	Class I Landfill (Z-2)
0 to 0.60 meters	291.45	315.63	28.60	30.99	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	150.47	163.34	14.70	15.97	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	245.87	264.29	24.11	25.92	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	14	14	1.25	1.25	NO	Non-Hazardous (X)
0 to 1.5 meters	238.44	256.35	23.38	25.14	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				2.48		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 0.9 meters			0.35		Condition 1	

Lead Analysis and Soil Management Summary – Group 13

Route 5 HOV Project – Statistical Analysis Group 13						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	418.47	453.48	41.13	44.58	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	144.32	155.74	14.10	15.22	YES (Y)	Class I Landfill (Z-2)
0 to 0.30 meters	266.83	292.56	26.17	28.71	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	183.58	200.11	17.97	19.60	YES (Y)	Class I Landfill (Z-2)
0 to 0.60 meters	226.96	245.04	22.24	24.03	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	157.37	171.88	15.38	16.81	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	203.72	218.85	19.95	21.44	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	76.5	76.5	7.41	7.41	YES (Y)	Class I Landfill (Z-2)
0 to 1.5 meters	198.46	212.99	19.43	20.87	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				4.48		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 1.5 meters			0.42		Condition 1	

Lead Analysis and Soil Management Summary – Group 14

Route 5 HOV Project – Statistical Analysis Group 14						
Layer (s)	TTLC		STLC		SOIL HANDLING	
	90% UCL (mg/kg)	95% UCL (mg/kg)	90% UCL (mg/kg)	95% UCL (mg/kg)	Invoke Variance (Soil Type)	Surplus Soil (Soil Type)
0.0 to 0.15 meters	204.98	218.63	20.08	21.42	YES (Y)	Class I Landfill (Z-2)
0.15 to 1.5 meters	40.82	43.91	3.89	4.19	YES (X)	Non-Hazardous (X)
0 to 0.30 meters	128.81	138.73	15.57	13.54	YES (Y)	Class I Landfill (Z-2)
0.31 to 1.5 meters	46.81	51.46	4.48	4.94	YES (X)	Non-Hazardous (X)
0 to 0.60 meters	96.68	103.92	9.40	10.11	YES (Y)	Class I Landfill (Z-2)
0.61 to 1.5 meters	65.03	73.89	6.28	7.15	YES (Y)	Class I Landfill (Z-2)
0 to 0.90 meters	82.16	88.12	7.97	8.55	YES (Y)	Class I Landfill (Z-2)
0.91 to 1.5 meters	71	71	6.87	6.87	YES (Y)	Class I Landfill (Z-2)
0 to 1.5 meters	81.89	87.71	7.94	8.51	YES (Y)	Class I Landfill (Z-2)
TCLP 95% UCL (mg/l) 0 to 0.9 meters				2.17		NON-RCRA Waste
DI-WET 90% UCL (mg/l) 0 to 1.5 meters			0.32		Condition 1	

6. CONCLUSIONS

A review of the data set histogram indicated that the total lead data set is not normally distributed. In addition, the mean is less than the variance for the data set; therefore, an arsine transformation was applied. A histogram, regression analysis chart of total lead vs. soluble lead, and 90 percent and 95 percent arcsine UCLs with corresponding soil excavation scenarios shown in block diagrams are presented as Appendix C, respectively. The correlation factors "r" were equal to or above the minimum requirement of 0.8. For reference, the regression analysis charts contain the equation for the regression lines and the "r" values. Reuse conclusions were based upon comparison of the referenced 90 percent UCLs and WET-DI 90% UCLs for each group to the DTSC Variance, AB 414, and the DTSC Variance modification letter dated December 13, 2002. Conclusions for surplus material and material relinquished to the contractor was based upon comparison of the total lead 95 percent transformed UCLs to the HSC disposal limit of 350 mg/kg and predicted WET-Citric results to the CCR Title 22 soluble lead threshold of 5.0 mg/l, and TCLP 95% UCLs to the RCRA threshold of 5.0 mg/l.

Group 1

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 1 is to be disposed of separately, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 17% (8 out of 46) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 2

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 0.9 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for

total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 2 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 0.9 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 0.9 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 9% (3 out of 34) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.15 m.

Group 3

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.6 meters of soil from Group 3 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 19% (11 out of 58) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.6 m.

Group 4

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 4 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA

hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 40% (21 out of 52) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 5

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 21% (13 out of 62) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 1.5 m. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 6

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 57% (16 out of 28) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 7

Based upon the 90 and 95% UCLs for total lead and predicted WET-Citric results, any portion of the upper 1.5 m of soil excavated may be handled as a non-hazardous material and is suitable for reuse or disposal without restriction. Because there were no samples collected from Group 7 that exhibited a total lead concentration greater than 100 mg/kg, TCLP tests were not performed on any of the samples collected from this Group. Based on the total lead concentrations reported for Group 7, it is unlikely that the soil would be classified as a RCRA hazardous waste.

Group 8

Based upon the 90% and 95% UCLs for total lead and WET-DI, if the entire soil column up to 0.9 m or deeper beneath the surface is treated as a single unit it may be handled as a non-hazardous material and is suitable for reuse or disposal without restriction. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 8 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 7% (5 out of 74) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 9

Based upon the 90% and 95% UCLs for total lead and WET-DI, the soil column from 0.6 m or deeper beneath the surface is treated as a single unit it may be handled as a non-hazardous material and is suitable for reuse or disposal without restriction. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 9 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a non-hazardous material and can be re-used or disposed of without restriction. Because the average value for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The average, rather than the 95% UCL, for TCLP was used for this data set because the size of the data set was too small for an accurate evaluation of the 95% UCL. The average value should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 2% (2 out of 81) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 10

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.3 meters of soil from Group 10 is excavated

separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified as a California Class I hazardous material with respect to soluble lead. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 13% (4 out of 32) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.3 m.

Group 11

Based on the TCLP 95% UCL data set, if any portion of the upper 0.6 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 30% (6 out of 20) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.6 m. Because no samples were analyzed by the TCLP from depths greater than 0.6 m it is likely that excavation of the entire soil column to a depth of 1.5 m would change the RCRA classification of the soil. Exsitu characterization of the soil may result in a non-RCRA classification.

Based upon the 90% and 95% UCLs for total lead and WET-DI, if the soil column from 0.6 m or deeper beneath the surface is excavated separately it may likely be handled as a non-hazardous material and is suitable for reuse or disposal without restriction

Group 12

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.9 meters of soil from Group 12 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a

RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 48% (14 out of 29) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 13

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if any portion of the upper 1.5 meters of soil from Group 13 is excavated for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 39% (9 out of 23) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Group 14

Based upon the 90% UCLs for total lead and WET-DI, all soils from up to 1.5 m beneath the surface would likely be suitable for reuse according to the DTSC Variance. Based upon the 95% UCLs for total lead and predicted WET-Citric results, if the upper 0.15 meters of soil from Group 14 is excavated separately for disposal, it would likely be classified as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters would likely be classified as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it would likely be classified California Class I hazardous material with respect to soluble lead content. Because the 95% UCL for TCLP was less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples from the group with total lead concentrations exceeding 100 mg/kg. Total lead concentrations exceeding 100 mg/kg were only encountered in 19% (8 out of 43) of the samples collected from this Group. Samples with total lead concentrations exceeding 100 mg/kg were encountered in soils at depths up to 0.9 m.

Bridge No. 53-1123 Widening

Based on the non-statistical evaluation of the results from borings 615-063 and 615-064, collected beneath Bridge No. 53-1123, it appears that soils excavated to a depth of 1.5 m would be suitable for reuse according to the DTSC Variance. Based on the reported total lead and WET-Citric results, if the upper 1.5 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Because the reported TCLP results were all less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste.

Bridge No. 53-1124 Widening

Based on the non-statistical evaluation of the results from borings 615-087 and 615-088, collected beneath Bridge No. 53-1124, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported total lead and WET-Citric results, if the upper 0.9 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Because the reported TCLP results were all less than 5.0 mg/l, it is unlikely that the soil would be classified as a RCRA hazardous waste.

Bridge No. 53-1125 Widening

Based on the non-statistical evaluation of the results from borings 615-103 and 615-104, collected beneath Bridge No. 53-1125, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported total lead and WET-Citric results, if the upper 0.9 meters of soil from this Bridge is excavated for disposal, it would potentially be classified as a California Class I hazardous material with respect to soluble lead content. Since none of the samples collected at this location exceeded 100 mg/kg, and therefore no TCLP analyses were performed, it is unlikely that the soil would be classified as a RCRA hazardous waste.

7. RECOMMENDATIONS

Group 1

Geocon recommends that if the upper 1.5 meters of soil excavated from Group 1 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If the upper 0.9 meters of soil from Group 1 is excavated

separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated as a California Class I hazardous material with respect to soluble lead.

Group 2

If the upper 0.9 meters of soil excavated from Group 2 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If the upper 0.15 meters of soil from Group 2 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 0.9 meters should be treated as non-hazardous. If the entire soil column to a depth of 0.9 meters is treated as a single unit, it should be treated as a California Class I hazardous material with respect to soluble lead.

Group 3

If the upper 1.5 meters of soil excavated from Group 3 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance. The 90% UCL for WET-DI, based on soil samples with total lead concentrations between 50 and 1,000 mg/kg, was calculated to be 0.91 mg/l. The 90% UCL for WET-DI should be considered a conservative number because only 19 of 58 (32%) of the soil samples were analyzed for WET-DI. Samples analyzed for WET-DI were collected at depths up to 0.9 meters. If the upper 0.6 meters of soil from Group 3 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated as a California Class I hazardous material with respect to soluble lead.

Group 4

If the upper 1.5 meters of soil excavated from Group 4 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If the upper 0.9 meters of soil from Group 4 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content.

Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated as a California Class I hazardous material with respect to soluble lead.

Group 5

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples with the highest total lead concentrations from the group. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 6

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples with the highest total lead concentrations from the group. Exsitu characterization of the soil may result in a non-RCRA classification.

Group 7

If any portion of the soil column from Group 7 is to be excavated it may be re-used or disposed of without restriction.

Group 8

If the upper 0.9 m or deeper of soil is excavated seperatly from Group 8 it may be re-used or disposed of without restriction. If the upper 0.15 meters of soil from Group 8 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous.

Group 9

If the upper 0.6 m or deeper of soil is excavated separately from Group 9 it may be re-used or disposed of without restriction. If the upper 0.15 meters of soil from Group 9 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead

content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated as a non-hazardous material and can be re-used or disposed of without restriction.

Group 10

If the upper 1.5 meters of soil excavated from Group 10 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance. The 90% UCL for WET-DI, based on soil samples with total lead concentrations between 50 and 1,000 mg/kg, was calculated to be 0.51 mg/l. The 90% UCL for WET-DI should be considered a conservative number because only 19% (6 of 32) of the soil samples were analyzed for WET-DI. Samples analyzed for WET-DI were collected at depths up to 0.3 meters. If the upper 0.3 meters of soil from Group 10 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated as a California Class I hazardous material with respect to soluble lead.

Group 11

Based on the TCLP 95% UCL data set, if any portion of the upper 1.5 m of soil is excavated, it would not be suitable for reuse according to the DTSC variance and would likely be classified as both a California and RCRA hazardous material. The TCLP 95% UCL should be considered a conservative value because it is based upon soil samples with the highest total lead concentrations from the group. Exsitu characterization of the soil may result in a non-RCRA classification. If the soil column between 0.6 and 1.5 m is excavated separately it may likely be suitable for reuse and disposal without restriction.

Group 12

If the upper 1.5 meters of soil excavated from Group 12 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If the upper 0.9 meters of soil from Group 12 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column

to a depth of 1.5 meters is treated as a single unit, it should be treated California Class I hazardous material with respect to soluble lead content.

Group 13

If the upper 1.5 meters of soil excavated from Group 13 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If any portion of the upper 1.5 meters of soil from Group 13 is excavated for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content.

Group 14

If the upper 1.5 meters of soil excavated from Group 14 is to be reused on-site, it may be used as fill provided that the soil containing ADL is placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. If the upper 0.15 meters of soil from Group 14 is excavated separately for disposal, it should be treated as a California Class I hazardous material with respect to soluble lead content. Underlying soil to a depth of 1.5 meters should be treated as non-hazardous. If the entire soil column to a depth of 1.5 meters is treated as a single unit, it should be treated California Class I hazardous material with respect to soluble lead content.

Bridge No. 53-1123 Widening

Based on the non-statistical evaluation of the results from borings 615-063 and 615-064, collected beneath Bridge No. 53-1123, it appears that soils excavated to a depth of 1.5 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance results. If the upper 1.5 meters of soil from this Bridge is excavated for disposal, it should be handled as a potentially California Class I hazardous material with respect to soluble lead content. Based on the limited results presented in this report further evaluation with respect to lead at this location appears warranted.

Bridge No. 53-1124 Widening

Based on the non-statistical evaluation of the results from borings 615-087 and 615-088, collected beneath Bridge No. 53-1124, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum water table elevation and protected from infiltration by a pavement structure maintained by Caltrans in accordance with the DTSC Variance results. If the upper 0.9 meters of soil from this Bridge is excavated for disposal, it should be handled as a potentially California Class I hazardous material with respect to soluble lead content. Based on the limited results presented in this report further evaluation with respect to lead at this location appears warranted.

Bridge No. 53-1125 Widening

Based on the non-statistical evaluation of the results from borings 615-103 and 615-104, collected beneath Bridge No. 53-1125, it appears that soils excavated to a depth of 0.9 m would be suitable for reuse according to the DTSC Variance. Based on the reported WET-DI concentrations it appears that the soil could potentially be reused on-site if placed a minimum of 1.5 meters above the maximum water table elevation and covered with at least 0.3 meters of non-hazardous soil in accordance with the DTSC Variance. Based on the reported total lead and WET-Citric results, if the upper 0.9 meters of soil from this Bridge is excavated for disposal, it should be handled as a potentially California Class I hazardous material with respect to soluble lead content. Based on the limited results presented in this report further evaluation with respect to lead at this location appears warranted.

Caltrans should notify the contractors performing the construction activities that hazardous concentrations of lead may be present in onsite soil and that appropriate health and safety measures should be taken to minimize the exposure to lead.

8. REPORT LIMITATIONS

This report has been prepared exclusively for Caltrans. The information obtained is only relevant as of the date of the latest site visit. The information contained herein is only valid as of the date of the report and will require an update to reflect additional information obtained.

The conclusions and recommendations presented herein are based on a limited number of samples collected from in-place soil and from widely spaced locations according to Caltrans-prescribed protocol. The purpose of these sampling and characterization activities was to reasonably predict the character of soil to be disturbed for planned construction activities within the described limits of the

Caltrans right-of-way. The disposition and handling of the soil are governed by the California regulations cited above. Characterization of the soil in the study areas for Federal waste criteria was beyond the scope of work in this task order.

A relatively small number of the samples were analyzed using the TCLP used to classify Federal waste. It is possible that soil disturbed, excavated and stockpiled could exceed Federal standards for hazardous waste and may require handling as a RCRA waste.

The Client should recognize that this report is not a comprehensive site characterization and should not be construed as such. The appropriate regulatory agency may require additional investigations. The findings and conclusions as presented in this report are predicated on the results of the limited soil sampling and laboratory analyses performed. In addition, the information obtained is not intended to address potential impacts related to sources other than those specified herein.

Therefore, the report should only be deemed conclusive with respect to the information obtained. No guarantee or warranty of the results of the report is implied within the intent of this report or any subsequent reports, correspondence, or consultation, either express or implied. Geocon strived to perform the services summarized herein in accordance with the local standard of care in the geographic region at the time the services were rendered.

STATE OF CALIFORNIA
DEPARTMENT OF INDUSTRIAL RELATIONS
DIVISION OF OCCUPATIONAL SAFETY
AND HEALTH ADMINISTRATION
MINING AND TUNNELING UNIT
6150 VAN NUYS BOULEVARD, SUITE 310
VAN NUYS, CA 91401-3333
(818) 901-5420 FAX (818) 901-5579

ARNOLD SCHWARZENEGGER, *Governor*

May 2, 2008

Department of Transportation
100 South Main Street
Los Angeles, CA 90012

Attention: Mr.Kahan A. Hossain, P.E.

Subject: Underground Classification Number: C088-037-08T
Route 170/5 HOV Project

Dear: Mr. Hossain:

The information provided to this office regarding the above project has been reviewed. On the basis of this analysis, Underground Classification of "Potentially Gassy" has been assigned to the shafts(s) identified in your submittal. Please provide a true and accurate copy of the Classification to the Boring/Excavation/Construction Contractor and insure that a copy of the Classification is posted at the job site.

Kindly insure that the Sub-Contractor notify this office to schedule the mandated Pre-job Safety Conference with the Division prior to commencing any activity associated with the project.

Also, be advised that, whenever an employee enters any bore or shaft being constructed under 30-inches in diameter, the Mining and Tunneling Unit then has immediate jurisdiction over that job. Please contact us prior to entering such spaces.

If you have any questions on these subjects, please contact this office at your earliest convenience.

Sincerely,

Tony Serpas

Tony Serpas
Senior Engineer

c: File

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

C088-037-08T

Van Nuys Office R5D2

Underground Classification

Route 170/5 HOV Project - Cal Trans
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of Department of Transportation, 100 South Main St, Los Angeles, CA 90012
(MAILING ADDRESS)

at At the New Interchange of SR 170 and I - 5
(LOCATION)

has been classified as ***POTENTIALLY GASSY***
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

The Division shall be notified if sufficient quantities of flammable gas or vapors have been encountered underground. Classifications are based on the California Labor Code Part 9, Tunnel Safety Orders and Mine Safety Orders.

Seven Bents cast-in-drilled-holes (CIDH) ranging from 2,700mm to 3,600 mm in diameter, and between 15m to 36m in depth, to be excavated at locations to be field determined within the new Interchange location for the SR 170 and I-5 freeways, Los Angeles County.

The bents are to be designated and located by CalTrans drawings as:

Bent - 2	Bent - 5R
Bent - 3	Bent - 6
Bent - 4	Bent - 7
Bent - 5L	Bent - 8

Date May 2, 2008

Tony Serpas
(Senior Engineer)

State of California

Department of Industrial Relations

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH
MINING AND TUNNELING UNIT

C034-037-09T (Revised)

Van Nuys Office R5D2

Underground Classification

Route 170/5 HOV Project - Cal Trans
(NAME OF TUNNEL OR MINE AND COMPANY NAME)

of **Department of Transportation, 100 South Main St, Los Angeles, CA 90012**
(MAILING ADDRESS)

at **At the New Interchange of SR 170 and I - 5**
(LOCATION)

has been classified as *****POTENTIALLY GASSY*****
(CLASSIFICATION)

as required by the California Labor Code Section 7955.

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The bents are to be designated and located by CalTrans drawings as:

Bent - 2	Bent - 7
Bent - 3	Bent - 8
Bent - 4	Bent - 9
Bent - 5	Bent - 10
Bent - 6	Bent - 11

Date September 12, 2008

Tony Serpas
(Senior Engineer)

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS

REGISTERED ELECT ENGINEER Theresa A. Gabriel DATE 12-06-07

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. ET5129 Exp. 6-30-08 ELECT STATE OF CALIFORNIA

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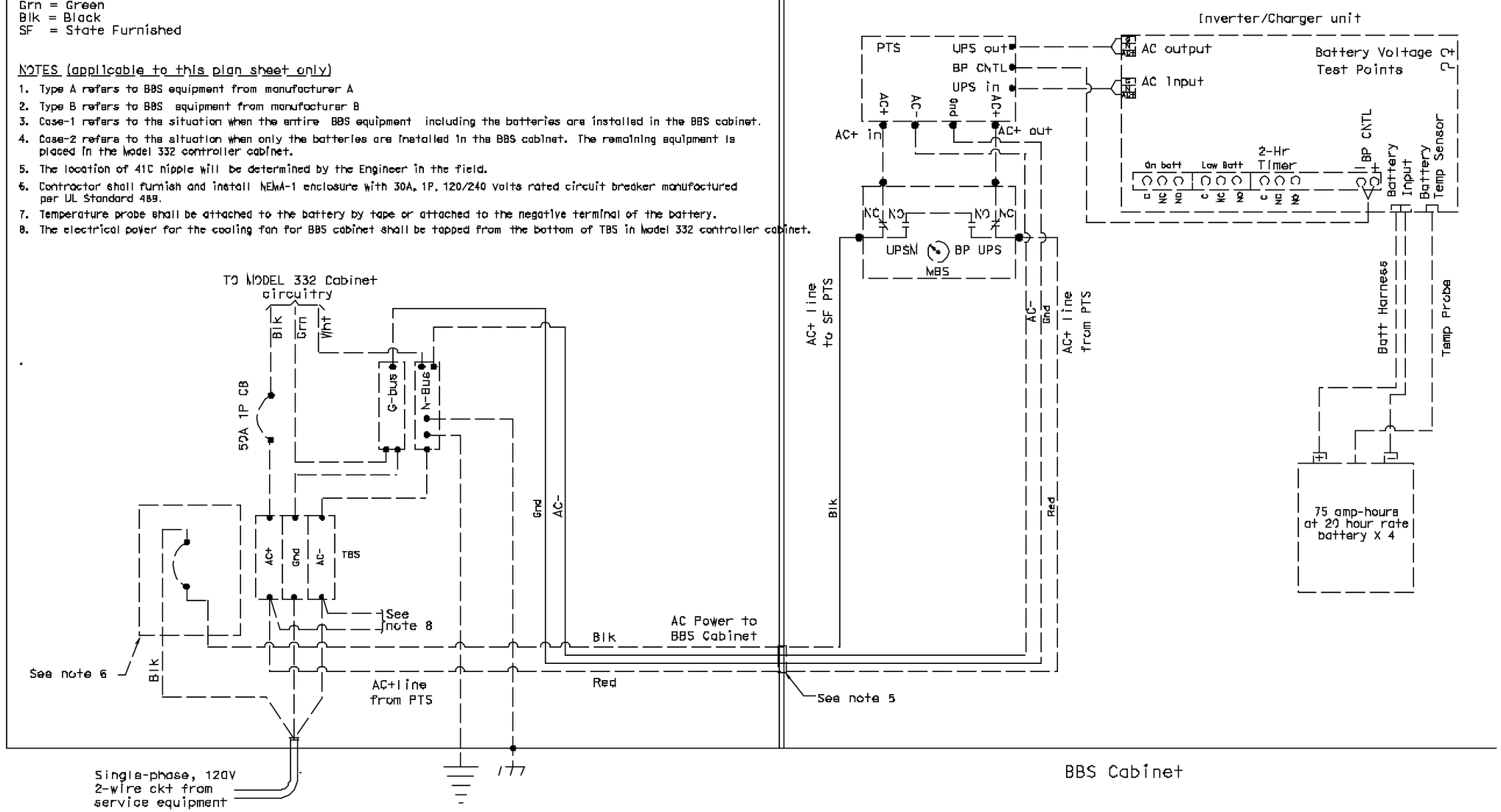
Caltrans Metric

LEGEND APPLICABLE TO THIS SHEET

BBS = Battery backup system
PTS = Power transfer switch
PTR = Power transfer relay
UPS = Uninterruptible power supply
UPSC= Uninterruptible power supply controller
UPSM= UPS Mode
BP = Bypass
BPC = Bypass control
MBPS= Manual bypass switch
MBJ = Main bonding jumper
AC+ = Ungrounded conductor
AC- = Grounded conductor
EG = Equipment ground
C = Common
Grn = Green
Blk = Black
SF = State Furnished

NOTES (applicable to this plan sheet only)

1. Type A refers to BBS equipment from manufacturer A
2. Type B refers to BBS equipment from manufacturer B
3. Case-1 refers to the situation when the entire BBS equipment including the batteries are installed in the BBS cabinet.
4. Case-2 refers to the situation when only the batteries are installed in the BBS cabinet. The remaining equipment is placed in the Model 332 controller cabinet.
5. The location of 41C nipple will be determined by the Engineer in the field.
6. Contractor shall furnish and install NEMA-1 enclosure with 30A, 1P, 120/240 volts rated circuit breaker manufactured per UL Standard 489.
7. Temperature probe shall be attached to the battery by tape or attached to the negative terminal of the battery.
8. The electrical power for the cooling fan for BBS cabinet shall be tapped from the bottom of TBS in Model 332 controller cabinet.



ELECTRICAL SYSTEMS

(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-1)

NO SCALE

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

FUNCTIONAL SUPERVISOR | CALCULATED-DESIGNED BY | CHECKED BY | REVISED BY | DATE REVISED

BBS 1250FSM metric 12-06-07.dgn 6/20/2008 8:32:51 AM

PLOT	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-06-07
REGISTERED ELECT ENGINEER DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-08 ELECT STATE OF CALIFORNIA

PLANS APPROVAL DATE

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Grn = Green
Blk = Black
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NOTES (applicable to this plan sheet only)

1. Type A refers to BBS equipment from manufacturer A
2. Type B refers to BBS equipment from manufacturer B
3. Case-1 refers to the situation when the entire BBS equipment including the batteries are installed in the BBS cabinet.
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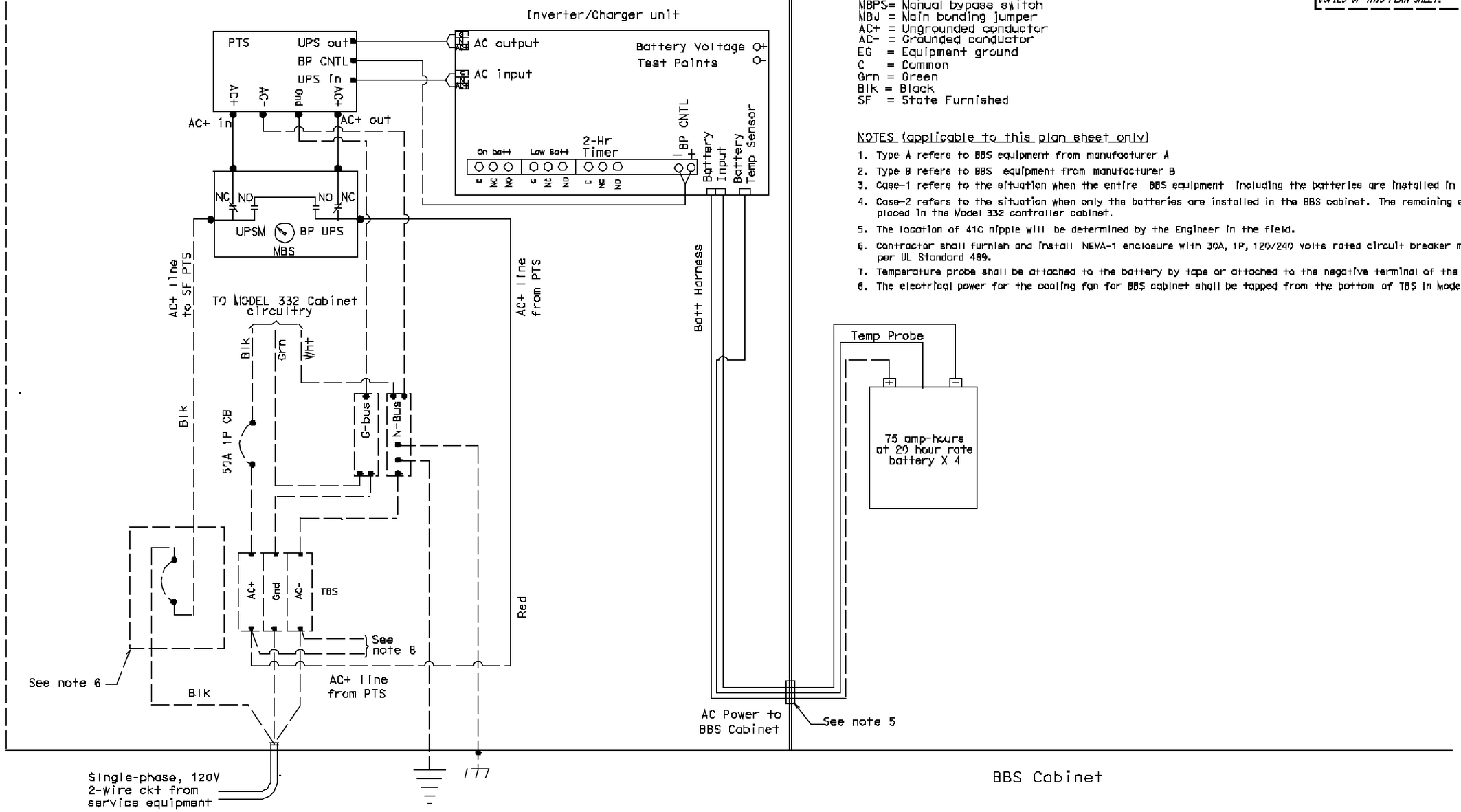

ELECTRICAL SYSTEMS

(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-2)

NO SCALE

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

FUNCTIONAL SUPERVISOR

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DATE REVISED

BBS 1250FSP metric 12-06-07.dgn 6/20/2008 8:34:03 AM

DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-06-07
REGISTERED ELECT ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-08 ELECT STATE OF CALIFORNIA

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Caltrans Metric

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NOTES (applicable to this plan sheet only)

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2. Type B refers to BBS equipment from manufacturer B
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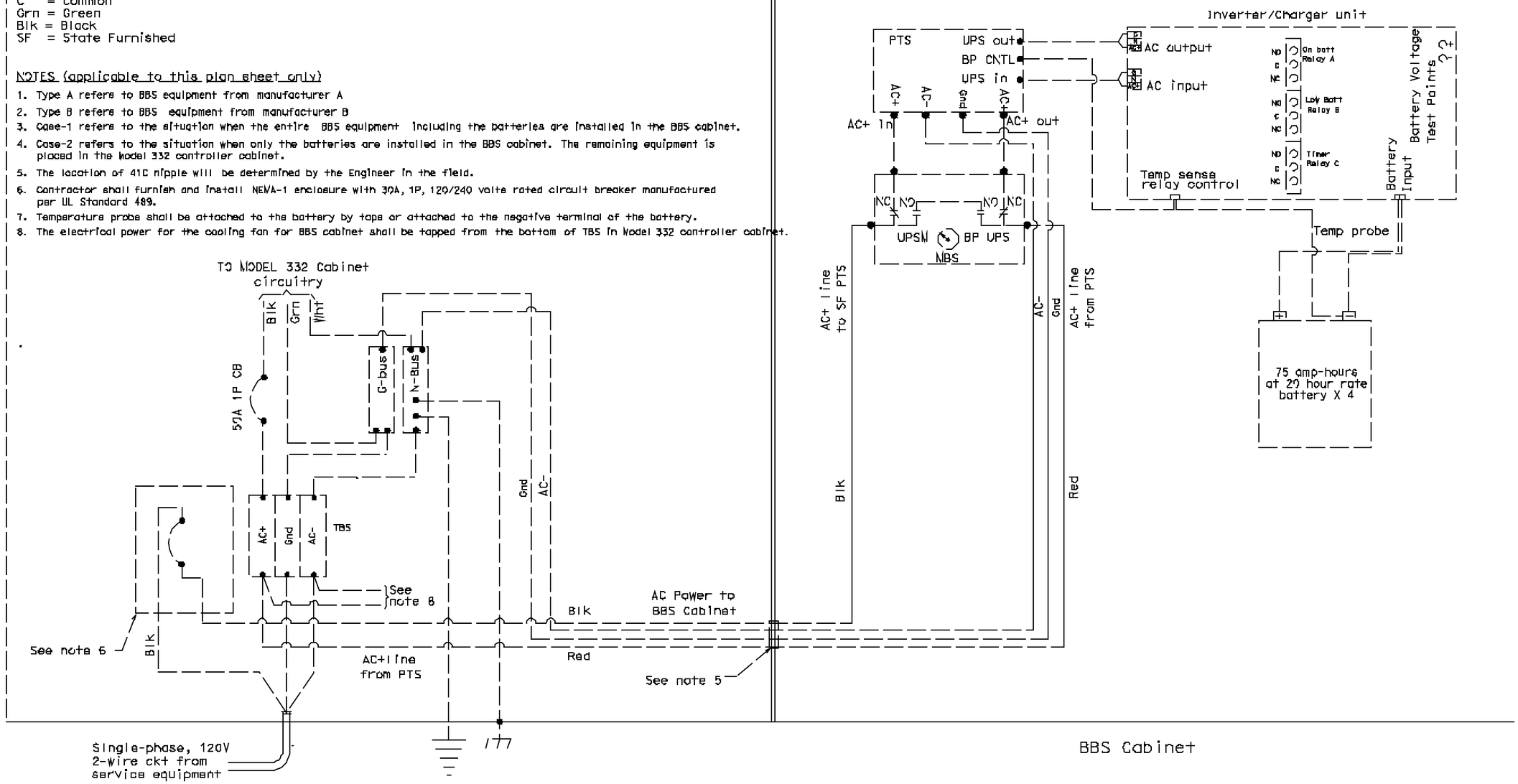
8. The electrical power for the cooling fan for BBS cabinet shall be tapped from the bottom of TBS in Model 332 controller cabinet.



ELECTRICAL SYSTEMS
(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-1)

NO SCALE

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DIST	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-06-07
REGISTERED ELECT ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-08 ELECT STATE OF CALIFORNIA

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LEGEND APPLICABLE TO THIS SHEET

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AC- = Grounded conductor
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Blk = Black
SF = State Furnished

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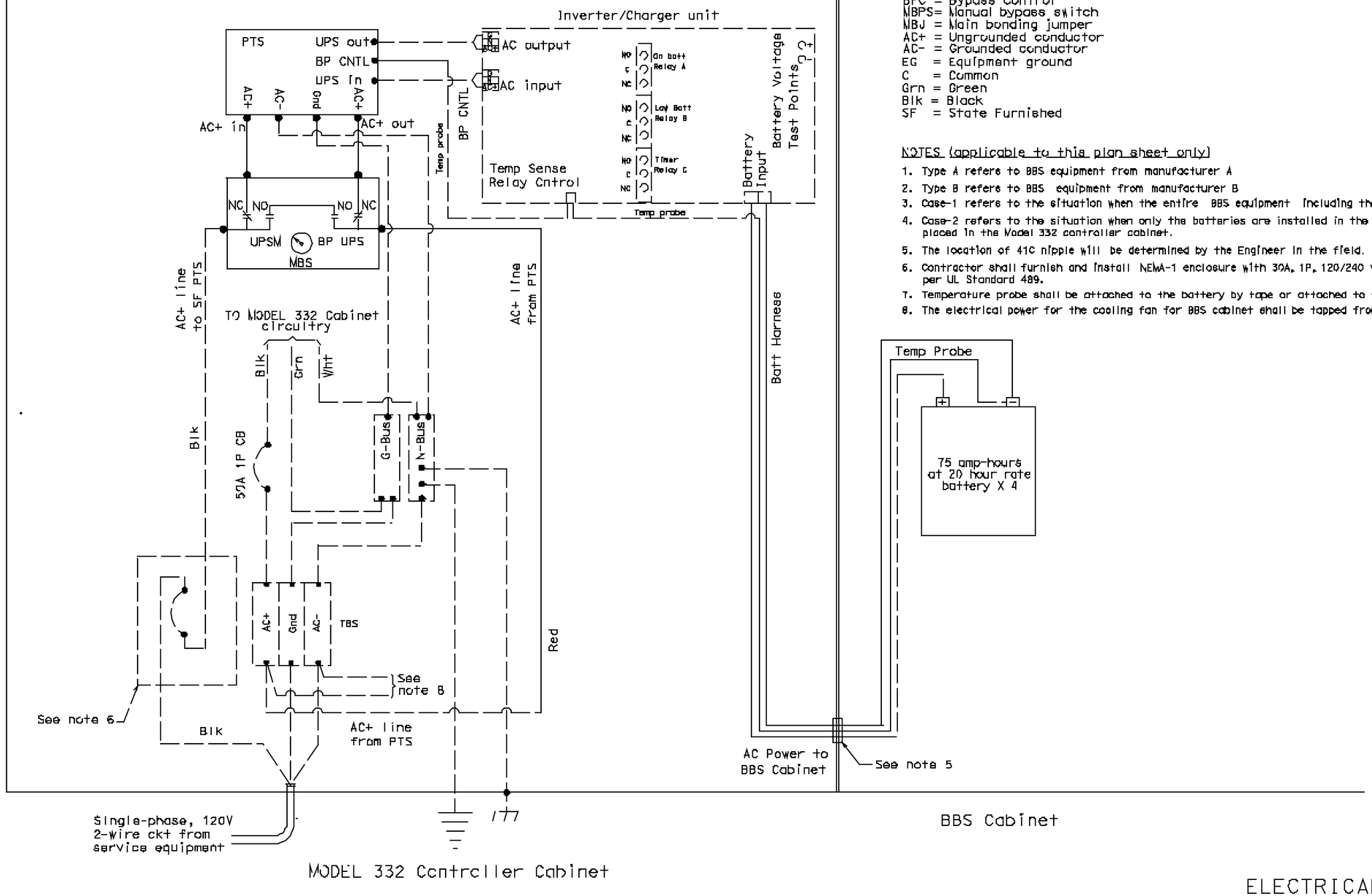

ELECTRICAL SYSTEMS
(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-2)

NO SCALE

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Dist	COUNTY	ROUTE	KILOMETER POST TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-06-07
REGISTERED ELECT ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-08 ELECT STATE OF CALIFORNIA

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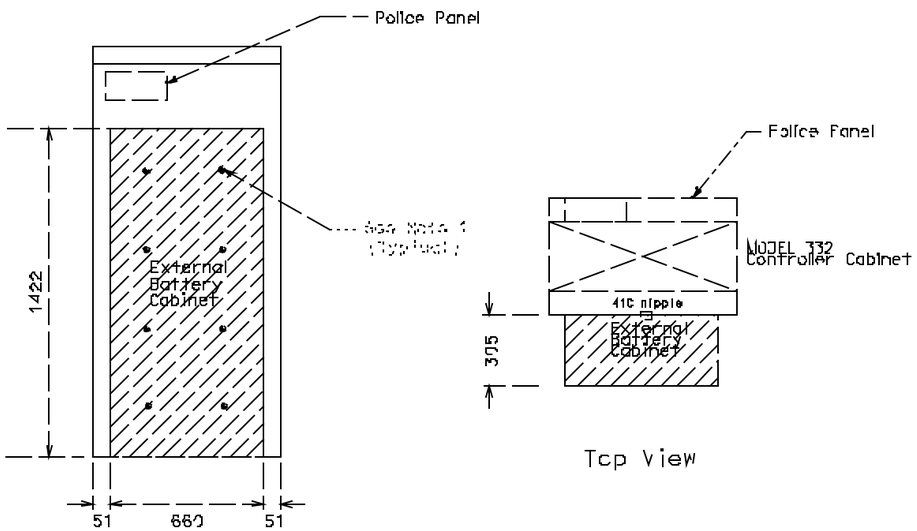
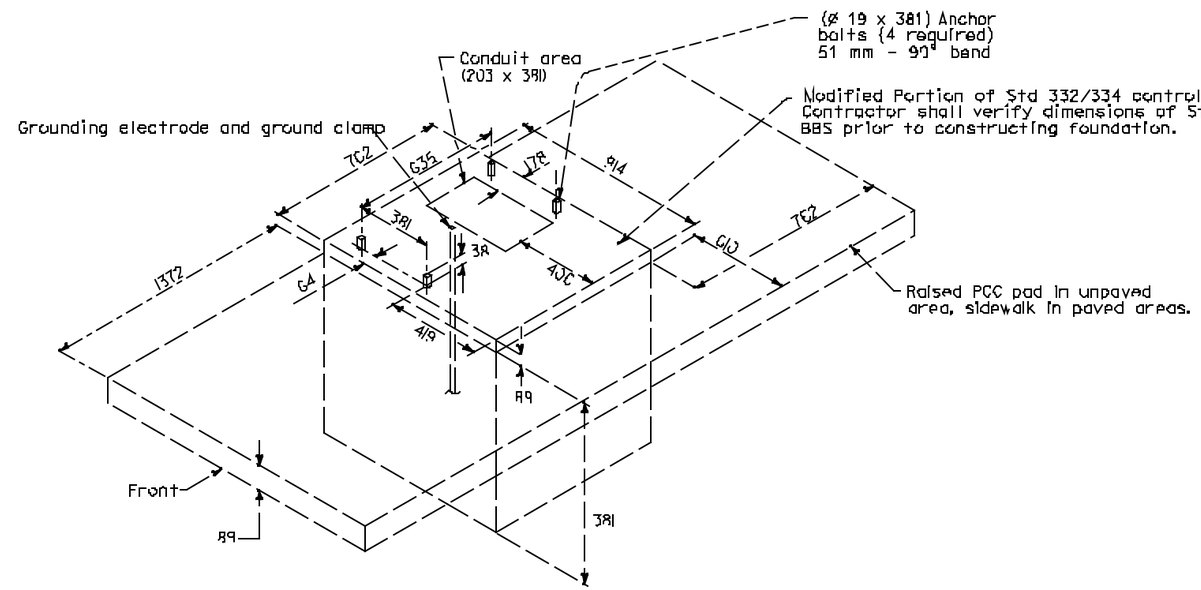


EXTERNAL BATTERY CABINET
FOR MODEL 332 CONTROLLER CABINETS

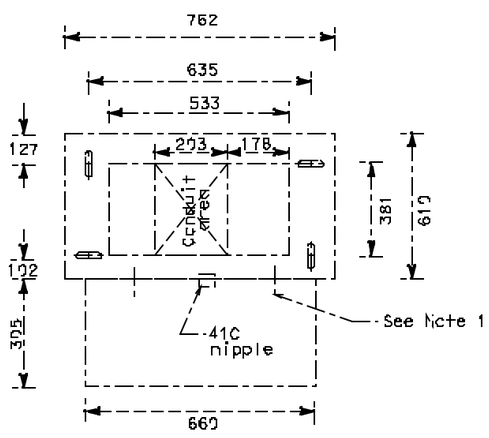
Notes (applicable to this plan sheet only)

1. The BBS cabinet shall be mounted to the 332 Cabinet with eight 18-8 Stainless Steel 9.5 mm-16 X 25.4 mm bolts, two 18-8 Stainless Steel 25.4 mm OD round flat washers, and one Stainless Steel 9.5 mm-16 Screw size K-lock Hex Nut. The location of the bolts shall be determined by the Engineer.



MODIFIED FOUNDATION DETAILS FOR BATTERY BACKUP SYSTEM

(See ES-3C of the Standard Plans for Standard Foundation Detail for Model 332 and 334 Controller Cabinets)



BASE PLAN FOR BBS PLUS MODEL 332 CONTROLLER CABINETS

ELECTRICAL SYSTEMS
(BBS FOUNDATION DETAILS)

NO SCALE

ALL DIMENSIONS ARE IN
MILLIMETERS UNLESS OTHERWISE SHOWN

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

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10-31-07 TIME PLOTTED => #TIME