FOR CONTRACT NO.: 07-224504

INFORMATION HANDOUT

MATERIALS INFORMATION

PORTION OF FOUNDATION REPORT

PORTION OF AERIALLY DEPOSITED LEAD STUDY

ROUTE: 07-LA-210-R57.1/R57.8

AESCO

17782 Georgetown Lane
Huntington Beach, California 92648
Tele: (714) 375 – 3830
Fax: (714) 375 – 3831

FOUNDATION REPORT
PROPOSED SOUNDWALL 85
STATE ROUTE 210
DUARTE, CA

AESCO Project No. 20081745-A6795

Prepared for:

Metropolitan Transportation Authority
c/o Tetra Tech, Inc.
16241 Laguna Canyon Road, Suite 200
Irvine, CA 92618

Attention: Ms. Hee-Young Oh

Prepared By:

AESCO
17782 Georgetown Lane
Huntington Beach, California 92648

Adam Chamaa, P.E., Manager

November 4, 2008
Revised April 16, 2009

Foundation Report

Table of Contents

SECTION	PAGE
1. Introduction	1
1.1 General Purpose	1
1.2 Scope of Work	1
1.3 Soundwall and Site Information	2
2. Site Conditions	3
2.1 Site Topography	3
2.2 Soil Investigation	3
2.2.1 Subsurface Conditions - Soundwall Alignment (Borings SB-1 through SB-14)	3
2.2.2 Subsurface Conditions - Duncannon Undercrossing (Borings BB-1, BB-2 and BB-7)	4
2.2.3 Subsurface Conditions - Highland Undercrossing (Borings BB-3 through BB-6)	4
2.3 Groundwater	4
2.4 General Geology	5
2.5 Preliminary Foundation Report	5
3. Subsurface Exploration	6
3.1 Field Procedures	6
3.2 Testing Sampling Equipments	6
3.3 Boreholes Locations	6
4. Laboratory Testing & Reporting	7
5. Liquefaction and Seismic Design Criteria	9
5.1 Liquefaction and Seismically Induced Settlement	9
6. Conclusions and Recommendations	10
6.1 Earthwork	10
6.2 Slope Stability	11
6.2.1 Slope Characterization	11
6.2.2 Slope Stability Evaluation	11
6.3 Retaining Wall and Soundwall Considerations	12
6.3.1 Earth Pressures for Retaining Walls	12
6.3.2 Static Earth Pressures	12
6.3.3 Surcharge Pressures	12
6.3.4 Backfill	12
6.3.5 Drainage Provision	13
6.4 Specifications for Installation of Cast In-Drilled Holes (CIDH)	13

6.5 Soil Properties for CIDH Selection 14
6.6 Recommendations for Soundwalls Foundation 15
6.7 Recommendations for Abutment Foundation (Shallow Foundation) 16
6.7.1 Allowable Bearing Pressure 16
6.7.2 Over-Excavation and Compaction 16
6.7.3 Sliding Resistance 17
6.7.4 Passive Resistance 17
6.7.5 Factors of Safety 17
6.7.6 Settlement 18
6.8 Corrosion 18
7. Construction Considerations 19
8. Limitations 20
8. Disclaimer 21

Appendices:

Quaternary Geologic Map of Azusa Quadrangle
Seismic Hazard Map
Historically Highest Groundwater Contours
Site Topography
Summary of Test Results on Log of Borings
Laboratory Test Results
Direct Shear Test Results
Sieve Analyses Tests
Corrosivity Test Results
Preliminary Foundation Report From Diaz Yourman
Log of Test Borings (LOTB)

1. INTRODUCTION

1.1. General Purpose

This report presents the Geotechnical Engineering Foundation Design Study and recommendations for the proposed new Soundwall No. 85 for the Metropolitan Transportation Authority (MTA). The project is located within the California Department of Transportation (Caltrans) right-of-way (ROW) along State Route 210 in Duarte, California. This study was performed pursuant to Tetra Tech, Inc., (TETRA) acceptance of AESCO's written proposal dated June 13, 2002.

The objective of the assessment was to evaluate the subsurface soil and rock properties, to explore the degree of variability and irregularity of subsurface conditions, the nature and type of structures and other relevant factors, to perform standard laboratory tests and to recommend foundation design criteria.

The assessment was undertaken in preparation for the construction of new Soundwall 85. The purpose of the soundwall is to reduce and mitigate the noise decibels generated by vehicles driving along the freeway to neighborhoods and residences located adjacent to portions of State Route 210.

1.2. Scope of Work

AESCO was contracted by Tetra Tech, Inc. to provide the following services:

- Conduct fourteen borings along the soundwall project alignment,
- Conduct seven borings adjacent to two bridge locations,
- Perform one trench exploration along the embankment,
- Perform geotechnical laboratory testing on soil samples,
- Perform corrosion testing on soil and water samples,
- Evaluate the engineering properties of the sub-soils,
- Use and comply with Caltrans' Foundation Report Manual, and Caltrans Standard Test Methods,
- Review the Preliminary Foundation Report, and

- Recommend soil parameters for design of the foundation of Soundwall 85.

1.3. Soundwall and Site Information

The proposed location of Soundwall No. 85 is along State Route 210 in the westbound direction from Buena Vista Street (K.P. 56.6) to Highland Avenue (K.P. 57.7) in the city of Duarte, California. The limits of Soundwall No. 85 will be between Stations 84 + 50 and 92 + 28. The length of the soundwall will be 778 meters with a proposed noise barrier height of 4.3 meters. It will be constructed at the edge of the shoulder. There are two existing bridge structures along the alignment, namely Duncannon Avenue Undercrossing and Highland Avenue Undercrossing.

The project limits are on fill material that varies from approximately 2.8 meters to 5.5 meters high. The existing route has four mixed-flow lanes and one high occupancy vehicle (HOV) lane in each direction. The width of the travel lanes ranges from 3.3 to 3.6 meters. The outside shoulder width is 3.0 meters and at the ramps is 2.4 meters. The side slopes are typically 1(V):2(H) or flatter toward the right-of-way, and are mostly covered with dense brush, shrubs and trees. In general, the most significant slopes are located at bridge abutments. Adjacent properties are generally fully developed and land uses consist of industrial and commercial developments with areas of residential development. North of the site, and northwest of the Highland Avenue and Central Avenue intersection, is Northview Intermediate School. West of the Northview Intermediate School and along Central Avenue is Duarte High School. Also, a Nissan dealership was observed northeast of Buena Vista Street and Central Avenue.

2. SITE CONDITIONS

2.1. Site Topography

Based on the available topographical maps from Tetra Tech, Caltrans and other sources, the site (location 34.13624 N, 117.97156 W) has topographic contours ranging between 152m to 158m on the northern side. The southern side of the site has lower topographic contours ranging from 140m to 146m. A topographic map of the site is presented in the Appendix. Surface water flow direction is generally perpendicular to topographic contours. Therefore, the flow direction is from northeast to southwest.

2.2. Site Investigation

AESCO performed a total of twenty-one (21) soil borings. The borings along the soundwall were advanced to a depth of 4 to 10 meters below the existing ground surface. The boring locations are as follows: along the soundwall alignment (Borings SB-1 through SB-14), Duncannon Undercrossing (Borings BB-1, BB-2, and BB-7) and Highland Undercrossing (Borings BB-3, BB-4, BB-5 and BB-6). Eight borings were performed at the top of the embankment and the remaining borings were along the embankment.

The subsoils were also explored by excavating a trench perpendicular to the freeway along the embankment. The trench was between 0.6 meters and 1 meter in depth. Photographs of the trench excavation are included in the Appendix.

2.2.1 Subsurface Conditions - Soundwall Alignment (Borings SB-1 through SB-14)

A total of fourteen borings were performed along the soundwall alignment and the freeway embankment. The upper 3 to 5 meters of the soil is fill that consists of medium to very dense very silty sand with gravel, cobbles and boulders. Loose to very loose slightly silty sand was encountered in borings SB-1 and SB-2 at the depth of 2.5m to 4m below existing ground surface. The fill is underlain by medium dense to very dense sand and gravel with cobbles (3

to 12 inches in diameter) and boulders (greater than 12 inches in diameter) per ASTM D2487 Standards.

Drilling operations were slow and difficult due to the presence of cobbles (3-12 inches in diameter) and boulders (greater than 12 inches in diameter).

2.2.2 Subsurface Conditions - Duncannon Undercrossing (Borings BB-1, BB-2 and BB-7)

The soil is generally dense to very dense sand and slightly silty sand with gravel and boulders, with the exception of the depth between 1.5m and 3m in Boring BB-2 and the upper 4m of Boring BB-1, which is dense to very dense gravel with boulders. Drilling operations were slow and difficult due to the presence of gravel, cobbles (3 to 12 inches in diameter) and boulders (greater than 12 inches in diameter).

2.2.3 Subsurface Conditions - Highland Undercrossing (Borings BB-3 through BB-6)

The soil consists of alternating layers of dense to very dense sand and gravel with cobbles and boulders. Drilling operations were difficult due to the large cobbles and boulders. Drilling operations were slow and difficult due to the presence of cobbles (3 to12 inches in diameter) and boulders (greater than 12 inches in diameter).

The soil details, profile and boring locations are shown on the Log of Test Borings (LOTB). The fill materials are shown on the LOTBs.

2.3. Groundwater

Groundwater observations were made from all available information sources, such as:

1. Log of Test Borings performed in the 1960's at the locations of Duncannon Avenue Undercrossing and Highland Avenue Undercrossing (document No. 70002427). No groundwater was observed at Duncannon Avenue Undercrossing where the maximum depth

of the boring drilled was 3m and at Highland Avenue Undercrossing, where the maximum depth of the boring drilled was 4.3m.

2. No groundwater was observed in any of the 14 borings performed along the soundwall alignment and the 7 borings performed at the two bridges. Maximum depth reached was 9.5m at boring BB-5 and BB-6. These borings were performed in November, 2002 and in September, 2003. The LOTB's are presented in the Appendix.

3. In reference to the Seismic Hazard Evaluation of the AZUSA 7.5 Minute Quadrangle, the historically highest ground water contours near the site ranged between 45 to 60m below the existing ground surface.

2.4. General Geology

The site is located in the southern part of the Azusa Quadrangle, in the northeastern area of the San Gabriel Valley. Streams draining from the San Gabriel Mountains have deposited alluvial fans in the San Gabriel Valley. The alluvium at the site is coarse granular. The largest stream in the mountains, the San Gabriel River, drains a watershed of over 200 square miles. The river enters the valley west of Azusa, and has deposited a large alluvial fan that extends to the south across the valley. A quaternary geologic map of the Azusa Quadrangle is presented in the Appendix.

2.5. Preliminary Foundation Report

A Preliminary Foundation Report (PFR) was performed by Diaz Yourman & Associates on December 20, 2001 and was reviewed by AESCO as part of this report. A copy of the PFR is in the Appendix.

3. SUBSURFACE EXPLORATION

3.1 Field Procedures

Field activities were conducted in accordance with ASTM, Soil & Rock Logging Classification Manual (Caltrans, 1996a) and Engineering Geology Field Manual (USBR) Standards.

Before commencing the field exploration work, site visits were initiated by AESCO's professional engineer on November 12, 2002, to locate desired borehole location points. In order to make sure that there is no existence of underground utilities within the location of the boreholes; Dig Alert was consulted to detect any underground utilities. The drilling process was commenced for the boring locations, where no underground utilities were encountered. Identifications of borings performed along the proposed soundwall location were prefixed with SB. Identifications of borings performed at the two bridges locations were prefixed with BB. Boring logs of the boreholes are presented in the Appendix.

3.2. Test Sampling Equipment

All boreholes were advanced by hollow stem auger and rotary drilling method, using a truck mounted CME 75 drilling rig, Rotary Failing 1500 Rig, and Hollow Stem Auger CME 85.

Borehole depth and sampling interval was based on the soil conditions and thickness of the fill materials. Samples were obtained by using Standard Penetration Test Method (SPT). Soil samples are collected from a split spoon sampler. Standard Penetration Tests are performed to evaluate the in-situ strength of the soil strata and (N-Value). The standard sampling interval for this project is 1.5 meters. Bulk samples were also collected between sampling intervals.

3.3 Boreholes Locations

Boreholes locations are shown in the attached Log of Test Borings (LOTBs).

4. LABORATORY TESTING & REPORTING

Acquired samples were selected for geotechnical laboratory testing and corrosion testing. For geotechnical laboratory testing, samples were chosen for individual testing methods such as: Direct Shear Test, Moisture Content Test, Sieve Analysis and Atterberg Limits Test. These tests were used to evaluate the engineering properties of the sub-soils, soil/rock classification, shear strength, and other design parameters.

Remolded samples for direct shear tests were compacted within the shear box (internal dimension 2 inch by 2 inch) in general accordance with the procedures for preparation of compacted specimens as described in the ASTM Standard D3080, Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. The boundaries between the compacted layers within the shear box did not coincide with the shear plane defined by the shear box halves. These remolded samples were compacted to 90 percent relative compaction of the maximum dry density at near the optimum moisture content. The maximum dry density and the optimum moisture content of the representative soil sample were determined following the ASTM Standard D1557. Direct shear tests were run under strain controlled condition.

The direct shear tests were performed on remolded, dense sandy soil samples obtained from the project site. In the case of dense soils, shear stress increases with shear displacement for a maximum or peak value and then decreases to an approximately constant value at large shear displacements.

Since the shear deformations of the soils along CIDH pile foundations will be relatively small for service state loads, stress-deformation behaviors for large shear strains have no significance for design purposes from the practical standpoint. For this reason, our direct shear tests were run up to the peak value, not continued for large shear displacements beyond the peak. Accordingly, apparently plastic behavior of stress-deformation curves at the peak shear stress as presented in our report should not be interpreted as plastic response beyond yielding. Yielding for dense sandy soils would occur beyond this peak value at a larger shear strain.

The test results are shown on the LOTB and in the Appendix. As requested by Caltrans, the stress-strain curves of the direct shear tests are also in the Appendix.

Corrosion testing included:

- **pH of soil using CTM 643 parts 2&3 method,**
- **Soil resistivity using CTM 643 part 4 method,**
- **Sulfate content using CTM 417 method, and**
- **Chloride content using CTM 422 method.**

With this report, a summary of laboratory results, graphical presentations, and summaries of the laboratory data are included. All test results are compiled in the appendices of this report and applied to geotechnical design considerations.

5. LIQUEFACTION AND SEISMIC DESIGN CRITERIA

5.1. Liquefaction and Seismically Induced Settlement

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The CGS has designated certain areas within southern California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is not located within a potential liquefaction hazard zone as designated by the CGS (1999). Materials encountered at the project site generally consist of medium dense to very dense granular material. Groundwater was not encountered within the borings which were drilled to a maximum depth of 9.5m beneath the ground surface. Historical high groundwater in the project vicinity is greater than 50m below the ground surface (CGS, 1997). Based on this, we conclude that the potential for liquefaction at the site is low. Other geologic hazards related to liquefaction, such as lateral spreading, are therefore also low.

The site is located in the United States Geological Survey Azusa Quadrangle. The CGS (CDMG, 2000-003) does not delineate this site as being within an Alquist-Priolo Earthquake Fault Zone. However, with the active faults in the region, the site could be subjected to future strong ground shaking that may result from earthquakes on local to distant sources.

6. CONCLUSIONS AND RECOMMENDATIONS

Based on our geologic reconnaissance and studies, it is our opinion that the construction of the proposed soundwall is feasible from a geotechnical standpoint. We recommend that the following geotechnical concerns, along with appropriate mitigation measures, be considered during the planning and design for the proposed construction.

6.1. Earthwork

Earthwork along the alignment should be performed in accordance with Caltrans Standard Specifications, July 1999 "Section 19".

Relative compaction of not less than 95 percent shall be obtained for a minimum depth of 0.2m below the grading plane for the width between the outer edges of shoulders, whether in excavation or embankment.

In addition, relative compaction of not less than 95 percent shall be obtained for a minimum depth of 0.8m below finished grade for the width of the traveled way plus one meter on each side thereof, whether in excavation or in embankment.

Except for the outer 1.5m measured horizontally from the embankment side slopes, the full width of embankment within 50m of bridge abutments shall be compacted to a relative compaction of not less than 95 percent. The 50m limit of 95 percent compaction will be measured horizontally from the bridge abutment and either parallel or concentric with the roadway centerline. In addition, a relative compaction of not less than 95 percent shall be obtained for embankment under retaining wall footings without pile foundations within the limits established by inclined planes sloping 1:1.5 (vertical: horizontal) out and down from lines 0.3m outside the bottom edges of the footing. Relative compaction of not less than 90 percent shall be obtained in all material in embankment, except as specified herein to be 95 percent.

The fill material should be tested and monitored by a representative from the geotechnical firm.

6.2. Slope Stability

6.2.1. Slope Characterization

As described above, portions of the proposed soundwalls will be located near the crest of or on slopes that descend away from the proposed soundwalls.

6.2.2. Slope Stability Evaluation

Based on our site reconnaissance and geotechnical study, the existing slopes were generally in fair to good condition. Some minor ground surface erosion and failures were observed at some of the existing freeway embankments.

Cut slopes are not anticipated in this project. Particular care should be taken during grading to perform geologic mapping and observation of the cut slopes. The slopes may need to be modified to provide adequate stability. Embankment construction shall be as per Section 19–6 of Department of Transportation Standard Specifications, July 1999.

Where new fill slopes are to be constructed against the outside of existing slopes, it is recommended that a minimum 0.91m deep keyway be excavated at the base of the existing toe of slope, equipment width at the base of the keyway, with the fill then placed in thin horizontal lifts and compacted. Proper benching into the existing slope is recommended as the fill is brought up.

All surface water runoff should be directed to suitable outlets to reduce the likelihood of surface erosion and saturation of the slopes. Any seepage observed from the current slopes should also be collected with a subsurface drainage system and diverted away from the slope area. Further, the slopes should be planted with vegetation as soon as feasible after the

completion of grading to reduce the amount of erosion on the slope face. Abutment slopes beneath the undercrossings should be paved.

6.3. Retaining Wall and Soundwall Considerations

The proposed soundwall and the retaining barrier should be supported on Caltrans Standard Cast-in-Drilled-Hole (CIDH) pile foundations. The CIDH piles will be installed in granular alluvium.

6.3.1. Earth Pressures for Retaining Walls

Recommendations for retaining wall design that are presented herein generally follow Standard Plan No. B3-1 (Caltrans Standard Plans May 2006).

6.3.2. Static Earth Pressures

In accordance with Standard Plan No. B3-1 (Caltrans Standard Plans May 2006), an equivalent fluid weight of 5.6 kN/m^3 (about 36 pounds per cubic foot [pcf]) may be used for active static conditions for level backfill conditions behind the wall. That value implies an active earth pressure coefficient of about 0.22 for a total unit weight of 19 kN/m^3.

The equivalent fluid weights should be applied to a vertical plane passing through the back part of the heel. The height of the vertical plane should extend from the point where the vertical plane intersects the ground surface down to the elevation of the lowest retaining wall foundation element (e.g., bottom of shear key or passive pressure resisting element).

6.3.3. Surcharge Pressures

Surcharge loads induce additional pressures on earth retaining structures. Surcharge pressures should be included in retaining wall design in accordance with Standard Plan No. B3-1 (Caltrans Standard Plans May 2006).

6.3.4. Backfill

Backfill behind retaining walls should be selected, placed, and compacted in general conformance with Section 19-3.06 of the Caltrans (1999a) Standard Specifications, with the exception of the following qualification:

The fines content (percentage of soil passing the No. 200 sieve) of the backfill should be no more than 35 percent by dry soil weight.

6.3.5. Drainage Provision

We note that provisions for drainage per Caltrans (1995a) Standard Details are recommended to help preclude the buildup of hydrostatic pressure behind the retaining wall. Bridge Details 3-1 and 3-5, Standard Plan B0-3, pg. 131, and the Design and Drainage Detail, Standard Plan B3-8, pg. 148 of the Caltrans (1995b) Standard Plans should be referenced for design details. Filter fabric should be placed between permeable drainage materials and backfill materials.

6.4 Specifications for Installation of Cast In-Drilled Holes (CIDH)

Construction of CIDH piles should be in general accordance with Section 49-4.01 of the Caltrans (1999a) Standard Specifications and Chapter 6, Cast-In-Drilled-Hole Piles in Caltrans Foundation Manual (July 1997).

The excavation of the CIDH may be done "in the dry" using an auger to remove the soil from the hole. If the soil conditions are such that an augered open hole cannot be maintained, the contractor shall use whatever measures may be necessary. These measures may consist of, but not necessarily limited to, casing the excavation, wet boring with drilling mud, pumping, shoring, hand excavation, or any other measures that may be required to achieve the desired construction. The drilling operation will encounter cobbles and large boulders (per ASTM D2487 Standards), as indicated in the soil borings, which may require special excavation method. The contractor should be prepared to excavate numerous, large boulders (greater than 12 inches in diameter). The cost for any of these measures shall be included in the base bid for the project. No excavations for slush pits shall be made in the ground surface if the wet boring process is used. A portable mud pit shall be used.

The concrete fill shall have a minimum of 28 day ultimate compressive strength of 28 MPa (4,000 psi) and maximum slump of 100mm (4 inches). The excavation shall be filled with concrete immediately after completion. The holes advanced with an auger shall first be inspected to insure that no loose material is in the bottom and the wall of the excavation is acceptable. Holes excavated using a wet drilling process shall have the concrete installed with a tremie pipe which shall be kept

below the surface of the concrete at all times during placement. No concrete shall be dropped through free water or height more than 1m. If CIDH piles spacing is less than 3 diameters, ABC installation shall be performed to prevent caving or damage of adjacent installed CIDH piles. The term "ABC installation" refers to drilling holes alternately. If CIDH piles spacing is less than 3 diameters center-to-center, holes should be drilled and concrete placed alternately to prevent caving or damage of adjacent installed CIDH piles. No drilling shall be performed adjacent to open holes or CIDH piles with concrete less than 72 hours old. In other words, the first caisson "A" should be drilled, then "C" should be drilled, concrete should then be placed in "A" and "C", wait 72 hours, then repeat with "B", etc.

The CIDH piles shall be installed to within 2 inches of the design locations. Piles at utility or obstruction may require adjustment with the resident engineer approval. If any pile is more than 2 inches from the design location, the entire installation shall be surveyed by a licensed.

A subsoil investigation has been made on the site and boring logs are included in this report. The information is correct only for each boring. The soil conditions can vary between boreholes.

6.5. Soil Properties for CIDH Selection

The following soil parameters can be used for selecting the CIDH based on CalTrans Standard Plans.

1. Soil classification: Sand and gravel above water table,
2. Relative density 77% for the upper 10' and 88% for soil below,
3. An average angle of internal friction of 35 degrees
4. Soil unit weight equals 19.6 kN/m^3.

Relative densities were determined from the following empirical correlations between field SPT N-values and relative density for sandy soils.

Correlation of SPT N-Value and Relative Density of Sand

N-Value, Blows per Foot	Sand Density	Relative Density (D_r)
0 to 4	Very loose condition	0 to 15%
4 to 10	Loose condition	15 to 35%
10 to 30	Medium dense condition	35 to 65%
30 to 50	Dense condition	65 to 85%
Over 50	Very dense condition	85 to 100%

Source: Lambe, T.W., and Whitman, R.V., 1969, Soil Mechanics, John Wiley & Sons, New York, 555 pp.
Source: Lambe, T.W., and Whitman, R.V., 1969, Soil Mechanics, John Wiley & Sons, New York, 555 pp.

Onsite soils as encountered during our field exploration were primarily sand mixed with gravel, cobbles, and variable amounts of silts. Based on SPT N-value, these soils were generally dense within the upper 10 feet and became very dense at depths greater than 10 feet.

6.6. Recommendations for Soundwalls Foundation

AESCO recommends the proposed soundwall to be supported by a combination of reinforced concrete barrier and Caltrans Standard Cast-in-Drilled Hole (CIDH) pile. The design of the CIDH shall be based on the Standard Plans and using an angle of internal friction of 35 degrees. This specification is in compliance with the CalTrans minimum Standards. In general, spacing between the piles should not be less than two pile diameters. If spacing is closer than 2 diameters, efficiency of pile groups shall be considered as the capacity of the pile will be reduced.

The CIDH piles derive their axial capacity through side friction and tip bearing. Based on the soil conditions, the design pile loads, for a factor of safety of 3.0, are shown in the table below:

Design Load for CIDH Piles for Soundwall 85				
Diameter (mm)	Depth (m)	Downward Load (kN)	Uplift (kN)	Lateral Load (kN)
406	4.877	230	65	98

Based on an allowable lateral displacement of 0.64 cm (1/4 in.), the allowable lateral load of 98 kN (22 kips) can be used. The lateral load calculations are in the Appendix.

6.7. Recommendations for Abutment Foundation (Shallow Foundation)

Where soundwalls are located on existing bridge structures, it is our opinion that the soundwall loads will be small compared to those of the existing structure and based on the existing soil conditions. Therefore, existing foundations should be capable of supporting the new weight of the soundwalls without foundations modification. However, adding soundwalls may require widening and retrofitting of the existing structure for conformance to current design standards. The structural engineer should verify the actual soundwall loading and required retrofitting.

6.7.1 Allowable Bearing Pressure

Footings may be sized using an allowable bearing pressure of 430 kPa for Highland and 382 kPa for Duncannon and an ultimate bearing pressure of 860 kPa and 764 KPa, respectively. These design pressures can be utilized for a minimum embedded depth of 1.52m into the natural soils or into the dense compacted fill materials. AESCO recommends footings be a minimum of 0.6m wide.

6.7.2 Over-Excavation and Compaction

Over-excavation for abutment footings should extend at least 0.6 meter below footing bottoms. However, if the footing embedment is greater than 3 feet below existing grade and at the same elevation as the existing footings, overexcavation and recompaction may be eliminated and only the bottom of the footing should be compacted. The horizontal distance between the footing edge and the toe of the temporary cut for the footing over-excavation

should be no closer than that determined by using a 1:2 projection down from the footing edge to the level of the over-excavation to mitigate the potential for the existing footing load distribution within the bearing soils to cause either a slope problem or a footing settlement problem. For example, for a 0.6-meter-deep over-excavation below the base of a footing, the horizontal distance between the footing edge and the toe of the temporary cut should not be less than 0.6 meter. However, any temporary excavations shall also be evaluated for slope stability of the excavation slopes themselves since some of the near-surface soils may not be stable, even if the above criteria are satisfied.

The bottom of the over-excavation should be scarified to a depth of 0.3 meter, moisture conditioned to within 2 percent of optimum, and compacted to a relative compaction of at least 95 percent. Over-excavated materials may be used as compacted fill after the removal of any particle size greater than 6 inches. Fill should be placed in horizontal lifts no thicker than 0.2 meter (loose thickness), moisture-conditioned to within 2 percent of optimum, and compacted to a relative compaction of at least 95 percent.

6.7.3 Sliding Resistance

Ultimate sliding resistance generated through a soil/concrete interface can be computed by multiplying the total dead weight structural loads by a coefficient of 0.4.

6.7.4 Passive Resistance

Passive resistance developed from lateral bearing of footings or shear keys bearing against compacted backfill or undisturbed native soils may be estimated using an equivalent fluid weight of 86 kN/m^3. Passive resistance should be neglected for the upper 0.3 meter of soil in front of the wall.

6.7.5 Factors of Safety

Sliding resistance and passive pressure may be used together without reduction when used with the following safety factors. Minimum factors of safety of 1.5 and 2.0 respectively, are recommended for foundation overturning and sliding where sliding resistance and passive

pressure are used together. The safety factor for sliding can be reduced to 1.5 if passive pressure is neglected.

6.7.6 Settlement

The settlement is anticipated to be less than ½ inch for static conditions for shallow foundation with design loads described above. Differential settlement between new and old footings should be less than ¼ inch due to the type of foundations encountered and the blow counts in the borings.

6.8 Corrosion

Corrosivity testing was performed on 36 samples from the borings. Test results are presented in the Appendix. As indicated on the test reports, the corrosion tests were performed in accordance with Caltrans Test Methods.

All resistivity values were greater than 2000 ohm-cm. The maximum sulfate content obtained was 0.008% from boring SB-8 at a depth of 0.304m. The maximum chloride content obtained was 0.016% from boring BB-7 at a depth of 3.34m-3.64m. The minimum pH value obtained was 7.5 from boring SB-7 at a depth of 1.52m-1.82m. According to the Draft Corrosion Guidelines (Caltrans, 1999c), a corrosive area is defined as one where "...the soil and/or water contains more than 500 ppm of chlorides, more than 2,000 ppm of sulfates, has a minimum resistivity of less than 1,000 ohm-cm, or has a pH less than 5.5." Therefore, the test results do not suggest a corrosive environment for metal or one that is potentially deleterious to concrete. Resistivity test results are presented in the Appendix. However, if fill materials are required for site work and grading purposes, all fill material shall be evaluated for corrosivity prior to use.

7. CONSTRUCTION CONSIDERATIONS

Difficult drilling will be encountered when drilling the CIDH due to the presence of cobbles (3 to 12 inches in diameter) and boulders (over 12 inches in diameter). To ensure proper implementation, the geotechnical engineer should review the method proposed by the contractor to drill the CIDH. Jetting should not be allowed without written approval of the Resident Engineer. Contruction of pile foundations shall be in general accordance with Section 49 of the Caltrans *Standard Specifications* (Caltrans, 1999).

Existing infrastructure that is located within a 1:2 (vertical:horizontal) line projected from the bottom edge of temporary slopes should be monitored during construction. The contractor should note that materials encountered can vary throughout the site. Excavations, temporary slopes and temporary support systems should be regularly monitored by a geotechnical engineer. Foundation installation and grading should also be inspected by the geotechnical engineer. The contractor should be prepared to use special methods to break up and remove cobbles and boulders during drilling.

8. LIMITATIONS

It must be recognized that conclusions reached in this report are based on conditions, which exist at the boring location and are assumed to exist over the entire site. In any subsoil investigation, it is necessary to assume that the subsoil conditions between boring(s) do not change significantly. The number of the borings, locations, and spacing are chosen in such a manner as to decrease the possibility of undiscovered anomalies, while considering the nature of loading, size, existing structures, and cost of the project. Note that the boring(s) were placed as close to the location of the proposed structure(s) as possible. Consequently, careful observations must be made during construction to detect significant deviations of actual conditions throughout the construction area from those inferred from the exploratory borings.

In the event that significant changes in design loads or structural characteristics are made, AESCO should be retained to review our original design recommendations and their applicability to the revised design plans. In this way, any required supplemental recommendations can be made in a timely manner.

Should any unusual conditions be encountered during construction, this office should be notified immediately so that further investigations and supplemental recommendations can be made. Geotechnical observations and testing should be provided on a continuous basis during grading, excavation, and installation of the foundations. If parties other than AESCO are engaged to provide geotechnical services during construction they will be required to assume the full responsibility for the geotechnical phase of the project by adhering to the recommendations of this report.

AESCO

17782 Georgetown Lane
Huntington Beach, California 92647
Tele: (714) 375 – 3830
Fax: (714) 375 – 3831

November 4, 2008
Revised April 16, 2009

Metropolitan Transportation Authority
c/o Tetra Tech, Inc.
16241 Laguna Canyon Road, Suite 200
Irvine, CA 92618

Attention: **Ms. Hee-Young Oh, P.E.**

Subject: **Soundwall 85**
Foundation Report
State Route 210
Duarte, CA
Subcontract Number 2817.0001.00
MTA Coantract No. PS-4240-1003
AESCO Project No. 20081745-A6795

Dear Ms. Oh:

AESCO Technologies, Incorporated (AESCO) is pleased to provide you six (6) copies of the geotechnical report for the above-referenced project. This report has been revised based on input from CalTrans and the structural engineer, Mr. Vinh Trinh with WKE-Inc.

AESCO will be happy to assist you further on this project by furnishing any Construction Materials Testing and Inspection Services you may require during the construction phase of the project. We are a full service-testing laboratory and inspection service and can supply the full range of testing and inspection services such as soils, concrete, asphalt, steel, welding, etc. that may be necessary for construction of this project.

Please do not hesitate to contact us if you have any questions or if we may be of any additional assistance. We look forward to assisting you during the construction of the proposed facility.

Sincerely,
AESCO, Inc.

Debra L. Perez
Project Manager

Adam Chamaa, MSCE, P.E., G.E.
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
ADAM CHAMAA
GE 2784
Exp. 12/31/09
GEOTECHNICAL
STATE OF CALIFORNIA

8. DISCLAIMER

This report has been prepared and compiled by the person below. All correspondence shall be directed to the following office:

Adam Chamaa, P.E., G.E., Project Engineer
AESCO Technologies, Inc.
17782 Georgetown Lane
Huntington Beach, CA 92647
(714)375-3830
(714)375-3831 fax

DIAZ•YOURMAN

& ASSOCIATES

Geotechnical Services

A Report Prepared for:

Tetra Tech, Inc.
Infrastructure Services Group
16241 Laguna Canyon Road, Suite 200
Irvine, CA 92618

AERIALLY DEPOSITED LEAD STUDY
NBSSR, PACKAGE 3
STATE ROUTE 210
EA 22450K and 23140K
LOS ANGELES COUNTY, CALIFORNIA

Project No. 245-02.3

by

Gary J. Halbert
Certified Engineering Geologist 1318

Allen M. Yourman Jr.
Geotechnical Engineer 925

Diaz•Yourman & Associates
17421 Irvine Boulevard
Tustin, CA 92780
(714) 838-8565

REGISTERED PROFESSIONAL ENGINEER
ALLEN MILES YOURMAN, JR.
925
EXP 12/31/04
GEOTECHNICAL
STATE OF CALIFORNIA

February 25, 2002

TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 PROJECT BACKGROUND AND DESCRIPTION 3

2.1 ADL WITHIN CALTRANS ROW 3
2.2 SITE DESCRIPTION 3
2.3 GEOLOGY AND GROUNDWATER 4

3.0 FIELD INVESTIGATION 5

3.1 PERMITS AND PLANNING 5
3.2 SOIL SAMPLING 5
3.3 LABORATORY ANALYSES 6

4.0 DATA EVALUATION 8

4.1 CRITERIA 8
4.2 DATA ANALYSES 8

5.0 CONCLUSIONS AND RECOMMENDATIONS 10

6.0 LIMITATIONS 11

7.0 BIBLIOGRAPHY 12

APPENDIX A - LABORATORY ANALYSES REPORTS A-1

APPENDIX B - STATISTICAL CALCULATIONS B-1

LIST OF TABLES

Table 1 - NOISE BARRIER LOCATIONS 4
Table 2 - LABORATORY LEAD TEST RESULTS 7
Table 3 - STATISTICAL ANALYSES 9

LIST OF FIGURES

Figure 1 - VICINITY MAP 1

PLATES

Sheets 1 through 10 ..... ---

1.0 INTRODUCTION

This report presents the results of a study of aerially deposited lead (ADL) performed by Diaz•Yourman & Associates (DYA) for a proposed Noise Barrier Scope Summary Report (NBSSR) along the Foothill Freeway, State Route (SR) 210 in the eastern San Gabriel Valley of Los Angeles County, California. Tetra Tech, Inc. authorized DYA to perform this work on September 25, 2001.

The project consisted of evaluating the site for ADL for the proposed Los Angeles County Metropolitan Transportation Authority (MTA) noise barriers (sound walls) located within the California Department of Transportation (Caltrans) right-of-way (ROW) on SR 210 between Buena Vista Street and Glendora Avenue in Azusa and Duarte as shown on the Vicinity Map, Figure 1. The site features of the proposed noise barrier locations are shown on the Site Plans, Plates 1 through 10.

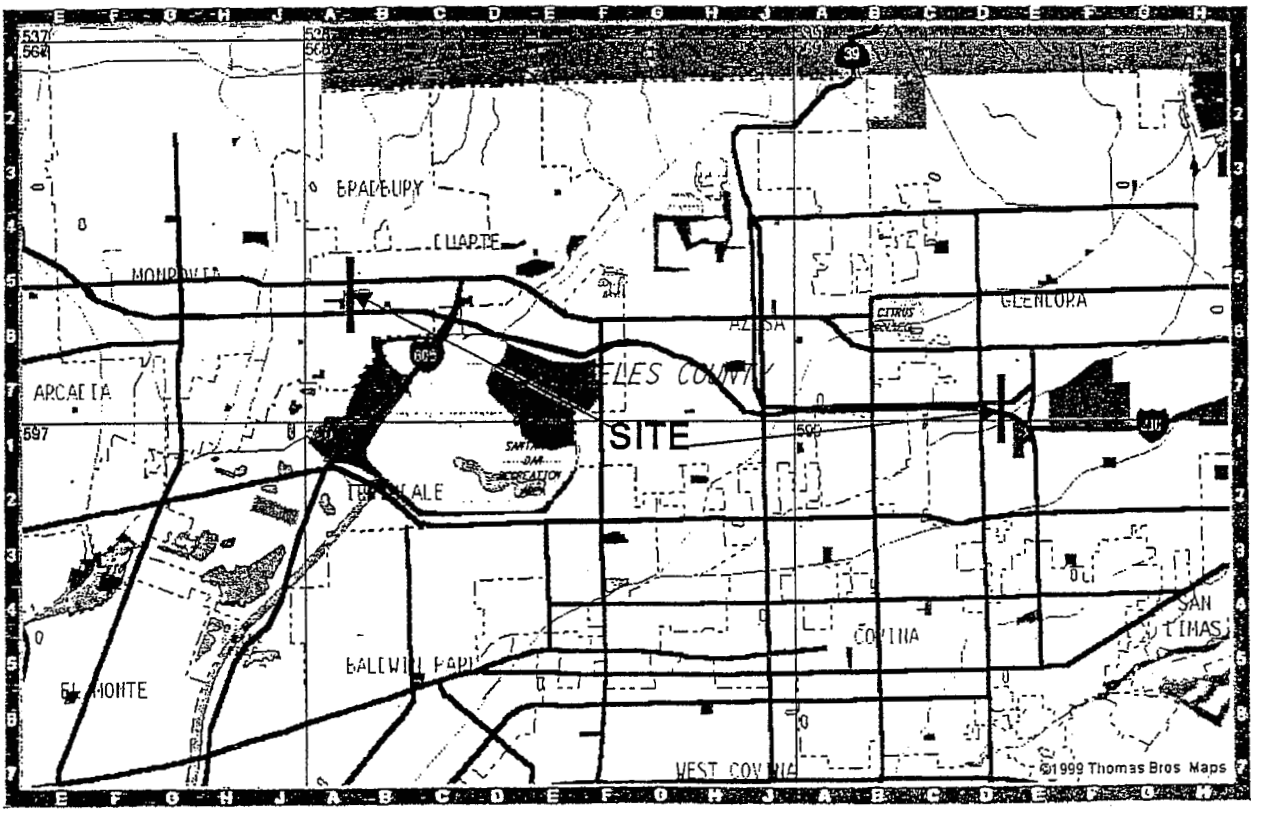

Figure 1 - VICINITY MAP

The purpose of DYA's services was to provide data for estimating ADL in soil to be excavated for sound barrier foundations. The scope of our services consisted of

- reviewing existing data,
- obtaining an encroachment permit from Caltrans,
- preparing a work plan reviewed by Caltrans,
- preparing a site worker health and safety plan reviewed by a Certified Industrial Hygienist,
- conducting field soil sampling,
- performing laboratory analyses,
- performing engineering analyses to develop preliminary conclusions regarding ADL in soil, and
- preparing this report.

2.0 PROJECT BACKGROUND AND DESCRIPTION

2.1 ADL WITHIN CALTRANS ROW

In soils adjacent to highways, lead has accumulated above natural levels primarily due to historic use of lead antiknock compounds in gasoline. These previous studies show that lead concentrations in near-highway surface soil were commonly highest within the upper 600 millimeters (mm) and decrease with depth (Caltrans 1998). When soil with elevated levels is excavated, the soil becomes a regulated waste. The U.S. Environmental Protection Agency (EPA) and California Department of Toxic Substances (DTSC) have established limit concentrations of lead in waste soil based on specified testing methods that trigger classification of waste soil as a special waste or hazardous waste, with guidelines for appropriate disposition (EPA 2001).

The DTSC has issued variances to Caltrans governing the reuse of near-highway surface soil with elevated lead concentrations as construction fill in new or redeveloped/maintained highways. A recent variance, issued to each individual Caltrans district, was dated September 22, 2000. Based on this DTSC variance, Caltrans provided ADL testing guidelines dated March 16, 2001. Determining the disposition of soil excavated for a project should be based on results of the specific ADL testing, data analysis, and potential reuse of excavated soil within the project corridor. If the soil cannot be reused in the corridor, project planning should include allowances for managing of lead-impacted waste soil as a regulated waste, usually by disposal at a landfill accepting hazardous or regulated wastes. Excavated soil can be stored on-site temporarily under DTSC variance conditions but should not remain in the Caltrans ROW permanently without permission from the Regional Water Quality Control Board.

Typically, noise barriers are supported by spread footing foundations bottomed 500 mm below surface or cast-in-drilled-hole (CIDH) piles excavated to a depth of 3 meters (m). The proposed noise barriers will likely be supported on shallow spread footings foundations (DYA 2001).

2.2 SITE DESCRIPTION

Based on the topography shown on the site plans, shallow soils at the proposed noise barrier locations appear to be within graded fill areas near the edges of SR 210 or adjacent to nearby streets. Noise barrier segment locations and general surface conditions are summarized in Table 1.

Table 1 - NOISE BARRIER LOCATIONS

ROUTE Station	AREA	DIRECTION[1]	LENGTH (m)	SURFACE CONDITIONS
210 Sta-830	Buena Vista Off-Ramp	WB	160	Retaining wall
210 Sta-850	Buena Vista Off-Ramp/ Santo Domingo Ave.	WB	1,000	Top of slope, shoulder
210 Sta-1680	Citrus Ave. On-Ramp/ Galanto Ave. Ped. Bridge	EB	480	Bottom of slope 20 m from lanes, adjacent to Alford St.
210 Sta-1730	Galanto Ave. Ped. Bridge/ Barranca Ave.	WB	300	Top of slope, 6 m wide, adjacent to Base Line Rd.
210 Sta-1740	Galanto Ave. Ped. Bridge/ Barranca Ave.	EB	300	Bottom of slope 20 m from lanes, adjacent to Alford St.
210 Sta-1770	Barranca Ave./ Grand Ave. On Ramp	WB	630	Bottom of slope 20 to 30 m from lanes, adjacent to Base Line Rd.
210 Sta-1830	Grand Ave. Crossing	WB	360	Top edge of fill slope, fwy shoulder
210 Sta-1870	Base Line Rd. On/Off-Ramps	WB	240	Ditch, fwy shoulder, top of fill slope 3 m from edge
210 Sta-1890	Base Line Rd. Off-Ramp/ Glendora Ave.	WB	440	Ditch, fwy shoulder, top of fill slope 3 m from edge

Note: 1. EB - Eastbound, WB - Westbound

2.3 GEOLOGY AND GROUNDWATER

The surface geology of the project area was mapped by California Division of Mines and Geology (CDMG) as Quaternary alluvial sediments. The project lies within the northeastern area of the San Gabriel Valley, approximately 1½ to 3 kilometer (km) south of the southern base of the San Gabriel Mountains. The area is characterized by coarse granular alluvium associated with stream and alluvial fan deposits at the base of the mountains. According to the 2001 groundwater contour map provided by the Main San Gabriel Valley Watermaster Web site, groundwater within the project area is deeper than 60 m below ground surface (bgs).

3.0 FIELD INVESTIGATION

3.1 PERMITS AND PLANNING

Preliminary design information regarding noise barrier foundation designs (DYA 2001) were reviewed and used to prepare a work plan. Caltrans documents pertaining to ADL sampling guidelines were also reviewed (Caltrans 2001). A list of the documents reviewed is presented in the bibliography (Section 7.0).

An encroachment permit was obtained from Caltrans for permission to perform sampling within the Caltrans ROW. A soil sampling work plan dated January 10, 2001 was prepared and reviewed and approved by Caltrans District 6 Environmental Department. (District 6 was selected to review the ADL aspects of this project.) A site-specific health and safety plan prepared by DYA for use by field workers was reviewed and signed by an independent Certified Industrial Hygienist.

Thirty-seven boring locations were planned at approximately 100-m intervals over the approximate 3760-m length of the noise barriers proposed by Tetra Tech, Inc.

3.2 SOIL SAMPLING

As indicated on the Site Plans, 35 boring locations were sampled for lead analyses. Two of the proposed locations were located within an asphalt pavement median between SR 210 and an access ramp, and were not sampled. The soil sampling, conducted on January 14 and 15, 2002, consisted of excavating one composite soil sample at each boring location with manual tools from the ground surface to a depth of approximately 500 mm. Surface debris and vegetation were cleared before collecting each sample. Tools were cleaned with distilled water before each boring excavation. For each boring, excavated soil was placed in a 19-liter bucket lined with a clean plastic bag. This soil was thoroughly mixed before collecting a composite bag sample of the soil for ADL analysis. The composite soil sample was collected and stored in a sealed, one-liter plastic bag. The remaining soil was backfilled into the borehole.

Soils encountered in the borings were noted to be granular consisting of sand and gravel, gravelly sand, or silty sand with gravel. Groundwater was not encountered during sampling excavations.

3.3 LABORATORY ANALYSES

At the end of each day of soil sampling, the soil samples were submitted to Del Mar Analytical Laboratories, Irvine, California, under chain-of-custody protocol. The 35 soil samples were initially analyzed for total lead concentration using EPA Method 6010. Laboratory test results are listed in Table 2. As specified in Caltrans guidelines (2001), soil samples with total lead concentrations greater than 50 milligrams per kilogram (mg/kg) were analyzed for extractable water soluble threshold limit concentration (STLC) of lead by the California wet extraction test (WET). Soil samples with STLC WET concentrations greater than 5 milligrams per liter (mg/L) were analyzed for extractable lead by the California WET using deionized water (DI-WET) for the extraction solution. Thirteen of the 35 soil samples tested had total lead concentrations greater than 50 mg/kg, and 10 samples tested had STLC WET lead concentrations greater than 5 mg/L; these samples were analyzed as noted above. Results of the WET and DI-WET tests are included in Table 2. As specified in Caltrans guidelines (2001), four samples were also analyzed for soil pH and are also presented in Table 2. Laboratory reports with analytical procedures and quality assurance data are presented in Appendix A.

Table 2 - LABORATORY LEAD TEST RESULTS

SAMPLE NO.	TOTAL LEAD (mg/kg)	STLC WET (mg/L)	DI-WET (mg/L)	pH
3-852	24			
3-861	120	9.3	0.30	7.87
3-874	31			
3-885	8.4			
3-895	180	11	0.29	
3-903	33			
3-913	57	5.2	0.19	
3-931	490	19	0.48	
3-941	20			
3-1688-S	13			
3-1699-S	7.2			
3-1711-S	19			7.7
3-1722-S	38			
3-1732-S	15			
3-1745-S	15			
3-1755-S	13			
3-1736-~~S~~ N	33			
3-1745-N	97	2.6		
3-1755-N	19			
3-1772-N	59	2.4		7.14
3-1797-N	21			
3-1784-N	18			
3-1805-N	260	7.4	0.16	
3-1814-N	120	5.9	0.18	
3-1822-N	530	38	0.77	
3-1828-N	190	12	0.39	
3-1836-N	88	5.1	0.17	8.02
3-1856-N	120	5.9	0.38	
3-1865-N	20			
3-1876-N	46			
3-1880-N	5.2			
3-1890-N	31			
3-1900-N	27			
3-1910-N	20			
3-1920-N	96	3.4		

Notes:

- Total lead = EPA Method 6010.
- STLC = soluble threshold limit concentration.
- DI-WET = California STLC method with deionized water.
- mg/kg = milligrams per kilogram.
- mg/L = milligrams per liter.

4.0 DATA EVALUATION

4.1 CRITERIA

Waste soil is classified as hazardous waste if the total lead concentration is greater than 1000 mg/kg or greater than 5 mg/L extractable lead (STLC WET). Waste soil is considered a nonhazardous special waste if the total lead concentration is greater than 350 mg/kg but less than 1000 mg/kg and the extractable STLC WET concentration is less than 5 mg/L. The DTSC variance for Caltrans and Assembly Bill (AB) 414 allow reuse of soil with lead under 0.3 m of nonhazardous fill and where the groundwater depth is greater than 1.5 m. To reuse the lead containing soil, the total lead concentration must be less than 1496 mg/kg, and with either the WET extractable STLC lead concentration less than 5 mg/L or the DI-WET extractable lead concentration less than 0.5 mg/L. Soil pH values of less than 5 shall be used only as fill under pavement.

4.2 DATA ANALYSES

As shown in Table 2, none of the total lead tests exceeded 1000 mg/kg. Total lead concentrations ranged from 5.2 to 530 mg/kg, with 13 samples having lead concentrations greater than 50 mg/kg. On samples with a total lead concentration greater than 50 mg/kg, the 13 STLC WET test results ranged from 2.4 to 38 mg/L of lead, with 10 soil samples exceeding 5 mg/L of lead. On the ten soil samples that had STLC WET extractable lead greater than 5mg/L, the DI-WET test lead results ranged from 0.16 to 0.77 mg/L.

As suggested in the Caltrans ADL testing guidelines (2001), DYA analyzed the lead testing data summarized in Table 2 using statistical methods noted in U.S. EPA Guidance Document SW 846, Volume 1, Chapter 9. Based on equations for predetermined systemic sampling and the prescribed confidence interval of 80 percent, the mean, standard deviation, and upper confidence interval were calculated for total lead analyses results as shown in Table 3. Mean, standard deviation, and upper confidence interval values were calculated for STLC WET extractable lead by using the 13 STLC WET results. Values for extractable lead were assigned to the remaining soil samples not tested for extractable lead using a linear proportional relationship between total lead and STLC WET values from the 13 samples that were tested. Mean, standard deviation, and upper confidence intervals were calculated for the DI-WET results using the same method noted for the STLC WET results. Statistical calculations are presented in Appendix B.

Table 3 - STATISTICAL ANALYSES

	MEAN	STANDARD DEVIATION	UPPER CONFIDENCE INTERVAL
Total Lead (mg/kg)	82.4	122	110
STLC WET Test (mg/L) best fit	4.6	6.6	6.0
STLC WET Test (mg/L) upper limit	7.2	8.4	9.1
DI-WET Test (mg/L) best fit	0.18	0.12	0.21
DI-WET Test (mg/L) upper limit	0.25	0.13	0.28

5.0 CONCLUSIONS AND RECOMMENDATIONS

The statistical results indicate the waste soil generated by earthwork or excavation for standard spread footing for sound barrier construction is likely to contain STLC WET extractable lead concentrations that exceed the hazardous waste limit of 5 mg/L. Because the total lead statistical concentration was less than 1496 mg/kg and statistical DI-WET concentration was less than 0.5 mg/L, the soil excavated for the noise barrier foundations may be placed beneath 0.3 m of nonhazardous fill within this Caltrans corridor in accordance with the DTSC variance. If not used within this corridor as described above, the excavated soil must be disposed as hazardous waste.

The volume of soil to be disposed will depend on actual grading activities. As a minimum, the standard noise barrier foundations outlined in Bridge Standard Details sheets (Caltrans 2000) can be used to estimate the volume of soil requiring disposal. The actual type of noise barrier foundation will be determined during final design. For preliminary budgeting purposes, the noise barrier shallow footing excavation of 500 mm deep and 2 m wide can be used to estimate the volume of soil requiring disposal. If CIDH piles supported noise barriers are selected during final design, then that volume of soil excavated for the CIDH piles should also be assumed to be hazardous unless further ADL testing indicates otherwise.

6.0 LIMITATIONS

This report was prepared for this project in accordance with generally accepted geotechnical engineering practices common to the local area. No other warranty, expressed or implied, is made.

The analyses and recommendations contained in this report are based on the literature review, field investigation, and laboratory testing conducted in the area. The results of the field investigation indicate subsurface conditions only at the specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations.

The validity of our recommendations is based in part on assumptions about the stratigraphy. Observations during construction can help confirm such assumptions. If subsurface conditions different from those described are noted during construction, recommendations in this report must be reevaluated.

This report is intended for use only for the project described. In the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by DYA. We are not responsible for any claims, damages, or liability associated with the interpretation of subsurface data or reuse of the subsurface data or engineering analyses without our express written authorization.

7.0 BIBLIOGRAPHY

California Assembly Bill (AB) 414, October 14, 2001.

California Department of Toxic Substances Control, 2000, Variance 00-H-VAR-03, September 22, 2000.

California Department of Transportation, 2001, Aerially Deposited Lead Testing Guidance, March 16, 2001.

California Department of Transportation, 200, Bridge Standard Details, April 2000.

California Department of Transportation, 1998, Proposed Soil Lead Management Criteria as Part of Caltrans Highway Construction Maintenance, June 5, 1998.

California Division of Mines and Geology, 1986, Geologic Map of California, 1:250,000 Series, San Bernardino Sheet, 1998.

Diaz•Yourman & Associates, 2001, Geotechnical Services for NBSSR Package 3, State Route 210, Los Angeles County, California, December 20, 2001.

Main San Gabriel Valley Watermaster, 2002, Web site.

U.S. Environmental Protection Agency, 2001, Guidance Document SW-846, Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Volume 1, Chapter 9, Sampling Plan, 3rd Edition, August 2001.