FOR CONTRACT NO.: 12-0G9604

INFORMATION HANDOUT

WATER QUALITY

1. CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
General Discharge Permit Order No. R8-2009-0045
(Amending Order No. R8-2007-0041, NPDES No. CAG918002)
SANTA ANA REGION

MATERIALS INFORMATION

1. Dimensions and Details for the External Cabinet, for Attaching the External Cabinet to the Model 332A Cabinet, and for Wiring the State-Furnished Equipment

2. Portions of Site Investigation Report (Petroleum Contaminated material)
Task Order No. 12-0G9601-29

3. Portions of Aerially Deposited Lead Site Investigation Report Task Order No. 12-0G9601-30

4. Foundation Report for Overhead Sign Structures

5. GeotechnicalDesign Recommendations for Retaining Wall No. 415

7. Second Revised Foundation Report for Dyer Road Undercrossing Widening, Bridge No. 55-0409 dated July 23, 2010.

8. Revised Foundation report for the Tieback Wall at Warner Avenue Overcrossing, Bridge No. 55-0394 dated May 6, 2010.

9. Foundation Review for Dyer Road Undercrossing Widening, Bridge # 55-0409 and Tieback Wall at Warner Avenue Overcrossing, Br# 55-0394 dated June 10, 2010.

ROUTE: 12-Ora-55-R7.8/ R9.4

California Regional Water Quality Control Board
Santa Ana Region

July 20, 2009

STAFF REPORT

ITEM: *9

SUBJECT: Amendment of Order No. R8-2007-0041, NPDES No. CAG918002, general discharge permit for discharges to surface waters of groundwater resulting from groundwater dewatering operations and/or groundwater cleanup activities at sites within the San Diego Creek/Newport Bay Watershed polluted by petroleum hydrocarbons, solvents, metals and/or salts - Order No. R8-2009-0045

DISCUSSION:

On November 30, 2007, the Regional Water Board adopted Order No. R8-2007-0041, NPDES No. CAG918002, prescribing general waste discharge requirements for discharges to surface waters of groundwater resulting from groundwater dewatering operations and/or groundwater cleanup activities at sites within the San Diego Creek/Newport Bay watershed polluted by petroleum hydrocarbons, solvents, metals and/or salts.

Order No. R8-2007-0041 consolidated the requirements of two general permits for discharges within the San Diego Creek/Newport Bay watershed: Order No. R8-2007-0008, NPDES No. CAG918001 (General Groundwater Cleanup Permit for Discharges to Surface Waters of Extracted and Treated Groundwater Resulting from the Cleanup of Groundwater Polluted by Petroleum Hydrocarbons, Solvents, Metals and/or Salts), and Order No. R8-2004-0021, NPDES No. CAG998001 (General Waste Discharge Requirements for Short-term Groundwater-Related Discharges and De Minimus Wastewater Discharges to Surface Waters within the San Diego Creek/Newport Bay Watershed). Specifically, Order No. 2007-0041 includes requirements to regulate groundwater-related discharges that may contain selenium, nutrients, volatile organic compounds, solvents or metals. The intent of this Order was to expedite the processing of applications and permitting for projects for which authorization under both Order No. 2007-0008 and Order No. R8-2004-0021 would otherwise have been necessary.

Order No. R8-2004-0021, NPDES No. CAG998002, regulates short-term groundwater-related discharges that are expected to last one year or less, and discharges that pose an insignificant threat to water quality *(de minimus* discharges) within the San Diego Creek/ Newport Bay watershed. This Order was amended by Order No. R8-2006-0065 to allow the discharge of wastewater effluent associated with pilot testing of selenium and nitrogen treatment technologies and BMPs and to prohibit the discharge of brine, resins, sludge or other secondary concentrates from treatment systems to surface waters. In summary, Order No. R8-2004-0021, as amended by Order No. R8-2006-0065, regulates the following types of discharges in the watershed:

a. Short-term (one year or less duration) discharges from activities involving groundwater extraction and discharge:

   (1) Wastes associated with well installation, development, test pumping and purging;
   (2) Aquifer testing wastes;
   (3) Dewatering wastes from subterranean seepage; and
   (4) Groundwater dewatering wastes at construction sites.

b. Discharges that pose an insignificant threat to water quality:

   (1) Construction dewatering wastes not involving groundwater (except storm water dewatering at construction sites)[1];
   (2) Discharges resulting from hydrostatic testing of vessels, pipelines, tanks, etc.;
   (3) Discharges resulting from the maintenance of potable water supply pipelines, tanks, reservoirs, etc.;
   (4) Discharges resulting from the disinfection of potable water supply pipelines, tanks, reservoirs, etc.;
   (5) Discharges from potable water supply systems resulting from system failures, pressure releases, etc.;
   (6) Discharges from fire hydrant testing or flushing;
   (7) Non-contact cooling water;
   (8) Air conditioning condensate;
   (9) Swimming pool drainage;
   (10) Discharges resulting from diverted stream flows;
   (11) Discharges from residential sump pumps; and
   (12) Other similar types of wastes, which pose a *de minimus* threat to water quality, yet technically must be regulated under waste discharge requirements.

c. Wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs.

In the process of consolidation of the requirements of Order No. R8-2004-0021, as amended by Order No. R8-2006-0065, into Order No. R8-2007-0041, certain types of discharges were inadvertently omitted. Specifically, Order No. R8-2007-0041 failed to include Items b. and c. of the above listing (i.e., de minimus types of discharges and wastewater associated with testing of selenium and nitrogen treatment technologies and BMPs). Order No. R8-2004-0021 is due to expire on December 20, 2009 and is not planned to be renewed since regulatory coverage can and will be provided under Order No. R8-2007-0041. However, it is necessary to amend Order No. R8-2007-0041 to include the discharges identified in items b. and c. above, as well as the discharge prohibition added by Order No. R8-2006-0065.

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1 *Storm water discharges are covered under separate permit.*

Furthermore, based on input from some of the Dischargers, it is necessary to clarify in Order No. R8-2007-0041, that for certain metals (including lead, cadmium, copper, chromium (II), nickel, silver, and zinc), the toxicity of which is hardness-dependent, the fifth percentile hardness value to be used in calculating the applicable criteria/effluent limits cannot exceed 400 mg/L, unless a site specific water effect ratio (WER) is developed and approved by the Regional Water Board. The California Toxic Rule, which specifies numeric criteria for these metals using equations in which hardness is a variable, explains that if the hardness is over 400 mg/L, two options are available to calculate the freshwater metals criteria: (1) Calculate the criterion using a default WER of 1.0 and using a hardness of 400 mg/L in the hardness equation; or (2) calculate the criterion using a WER and the actual ambient hardness of the surface water in the equation.

The following are the recommended changes to Order No. R8-2007-0041. Deleted text is struck out and added text is bold and highlighted.

1. Order No. R8-2007-0041, page 4, modify last paragraph of Section I. Discharge Information as follows:

   This general permit will regulate **de minimus discharges and wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs, and** discharges of treated wastewater from groundwater dewatering and/or groundwater remediation activities at sites polluted by petroleum hydrocarbons, solvents, metals and/or salts within the San Diego Creek/Newport Bay watershed.

2. Order No. R8-2007-0041, page 6, modify paragraph 5., as follows:

   5. The Discharger shall submit for approval by the Executive Officer of the Regional Water Board a fixed hardness value based on the 5th percentile of effluent hardness measurements or the average ambient receiving water hardness measurements for those sites polluted with metals (lead, cadmium, copper, chromium (III), nickel, silver, and zinc). **For purposes of calculating the applicable fresh water aquatic life criteria and effluent limitations for metals, the required fifth percentile hardness value has an upper limit of 400 mg/L as calcium carbonate, unless a site specific water effect ratio (WER) is developed and approved by the Regional Water Board. The California Toxic Rule explains that if the hardness is over 400 mg/L, two options are available to calculate the freshwater metals criteria (which are used as the basis for setting effluent limitations): (1) Calculate the criterion using a default WER of 1.0 and using a hardness of 400 mg/L in the hardness equation; or (2) calculate the criterion using a WER and the actual ambient hardness of the surface water in the equation.**

3. Order No. R8-2007-0041, page 8, modify paragraph II.B.3., as follows:

    3. For freshwater discharges, within forty five (45) days of the effective date of this Order, Dischargers from those sites polluted with leaded gasoline or metals shall submit for approval by the Regional Water Board Executive Officer the proposed hardness value based on 5th percentile of effluent hardness measurements or the average ambient freshwater receiving water hardness measurements. Once approved by the Executive Officer, this hardness value shall be the basis for determining the lead/metals effluent limits for the discharge from Attachment **"B~~J~~"** of this Order.

4. Order No. R8-2007-0041, page 10, modify last paragraph of Finding B., as follows:

    In summary, this general permit will regulate discharges from activities involving groundwater dewatering, **discharges that pose an insignificant threat to water quality, wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs** and groundwater remediation in areas where contamination from petroleum hydrocarbons, solvents, metals and/or salts may be present. These activities include the following:

    1. Wastes associated with well installation, development, test pumping and purging;
    2. Aquifer testing wastes;
    3. Dewatering wastes from subterranean seepage;
    4. Groundwater dewatering wastes at construction sites; ~~and~~
    5. Groundwater remediation.
    6. **Discharges resulting from hydrostatic testing of vessels, pipelines, tanks, etc.;**
    7. **Discharges resulting from the maintenance of potable water supply pipelines, tanks, reservoirs, etc.;**
    8. **Discharges resulting from the disinfection of potable water supply pipelines, tanks, reservoirs, etc.;**
    9. **Discharges from potable water supply systems resulting from initial system startup, routine startup, sampling of influent flow, system failures, pressure releases, etc.;**
    10. **Discharges from fire hydrant testing or flushing;**
    11. **Air conditioning condensate;**
    12. **Swimming pool discharge;**
    13. **Discharges resulting from diverted stream flows;**
    14. **Decanted filter backwash wastewater and/or sludge dewatering filtrate water from water treatment facilities;**
    15. **Discharges of wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs into surface water; and**
    16. **Other similar types of wastes as determined by the Regional Water Board Executive Officer, which pose a de minimus threat to water quality yet must be regulated under waste discharge requirements.**

5. Order No. R8-2007-0041, page 17, add new paragraph G. in Section IV., as follows:

   G. The discharge of brine, resins, sludge or other secondary concentrates from treatment systems to surface waters is prohibited.

RECOMMENDATION:

Adopt Order No. R8-2009-0045 as presented.

Comments were solicited from the following agencies:

U.S. Environmental Protection Agency, Permits Issuance Section (WTR-5) – Doug Eberhardt
U.S. Army District, Los Angeles, Corps of Engineers - Regulatory Branch
U.S. Fish and Wildlife Service, Carlsbad
State Water Resources Control Board, Office of the Chief Counsel – David Rice
State Department of Water Resources, Glendale
State Department of Fish and Game, San Diego – Dolores Duarte
California Department of Public Health, Santa Ana - Oliver Pacifico
Orange County Water District - Nira Yamachika/Greg Woodside
Orange County Public Works - Chris Crompton
Orange County Public Works, Flood Control – Andy Ngo
Orange County Health Care Agency – Larry Honeybourne
South Coast Air Quality Management District - – Dr. Barry R. Wallerstein
Orange County Coastkeeper - Garry Brown
Lawyers for Clean Water C/c San Francisco Baykeeper
Dr. Jack Skinner
Defend the Bay - Robert J. Caustin
Irvine Ranch Water District - Steve Malloy
California Department of Transportation, District 12 - Grace Pina-Garrett
City of Tustin - Dana R. Kasdan
Irvine Community Development Company – Tina Bachelder
City of Lake Forest - Robert L. Woodings
City of Laguna Hills – Kenneth Rosenfield
Golden State Water Company – Brandy O'Gorman, bogorman@gswater.com
City of Newport Beach - John Kappeler
City of Santa Ana Public Works Agency - James Ross
City of Irvine - Steve Ollo
City of Costa Mesa – Fariba Fazeli
Foothill Engineering & Dewatering - Wendell Bradford

California Regional Water Quality Control Board
Santa Ana Region

Order No. R8-2009-0045

Amending Order No. R8-2007-0041, NPDES No. CAG918002
General Discharge Permit For Discharges To Surface Waters Of Groundwater Resulting From Groundwater Dewatering Operations And/Or Groundwater Cleanup Activities At Sites Within The San Diego Creek/Newport Bay Watershed Polluted By Petroleum Hydrocarbons, Solvents, Metals And/Or Salts

The California Regional Water Quality Control Board, Santa Ana Region (hereinafter Regional Water Board), finds that:

1. On November 30, 2007, the Regional Water Board adopted Order No. R8-2007-0041, NPDES No. CAG918002, prescribing general waste discharge requirements for discharges to surface waters of groundwater resulting from groundwater dewatering operations and/or groundwater cleanup activities at sites within the San Diego Creek/Newport Bay watershed polluted by petroleum hydrocarbons, solvents, metals and/or salts.

2. Order No. R8-2007-0041 consolidated the requirements of two general permits for discharges within the San Diego Creek/Newport Bay watershed; Order No. R8-2007-0008, NPDES No. CAG918001, and Order No. R8-2004-0021, NPDES No. CAG998001. Specifically, Order No. R8-2007-0041 includes requirements to regulate groundwater-related discharges that may contain selenium, nutrients, volatile organic compounds, solvents or metals.

3. Order No. R8-2004-0021, NPDES No. CAG998002, regulates the short-term groundwater-related discharges that are expected to last one year or less, and discharges that pose an insignificant threat to water quality (de minimus discharges) within the San Diego Creek/Newport Bay watershed. This Order was amended by Order No. R8-2006-0065 to authorize discharges of wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs and to prohibit the discharge of brine, resins, sludge or other secondary concentrates from treatment systems to surface waters.

4. In the process of consolidation of permit requirements in Order No. R8-2007-0041, certain discharges regulated under Order No. R8-2004-0021, as amended, were omitted. Specifically, Order No. R8-2007-0041 failed to include de minimus discharges and wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs. Further, Order No. R8-2007-0041 failed to include the prohibition regarding the discharge of brine, resins, sludge or other secondary concentrates from treatment systems to surface waters. Order No. R8-2004-0021 is due to expire on December 20, 2009 and is not planned to

be renewed since regulatory coverage can and should be provided under Order No. R8-2007-0041. However, it is necessary to amend Order No. R8-2007-0041 to include the previously omitted de minimus discharges, discharges resulting from the testing of nitrogen and selenium treatment technologies and BMPs, and to include the prohibition specified in Order No. R8-2006-0065.

5. In accordance with California Water Code Section 13389, amending the general waste discharge requirements for the types of discharges regulated under Order No. R8-2007-0041 is exempt from those provisions of the California Environmental Quality Act contained in Chapter 3 (Commencing with Section 21100), Division 13 of the Public Resources Code.

6. The Regional Water Board has notified the dischargers and other interested agencies and persons of its intent to amend Order No. R8-2007-0041 and has provided them with an opportunity to submit their written views and recommendations.

7. The Regional Water Board, in a public meeting, heard and considered all comments pertaining to the amendment of general waste discharge requirements for de minimus discharges.

IT IS HEREBY ORDERED that Order No. R8-2007-0041 be amended as follows:

1. Order No. R8-2007-0041, page 4, modify last paragraph of Section I. Discharge Information as follows:

   This general permit will regulate de minimus discharges and wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs, and discharges of treated wastewater from groundwater dewatering and/or groundwater remediation activities at sites polluted by petroleum hydrocarbons, solvents, metals and/or salts within the San Diego Creek/Newport Bay watershed.

2. Order No. R8-2007-0041, page 6, modify paragraph 5., as follows:

   5. The Discharger shall submit for approval by the Executive Officer of the Regional Water Board a fixed hardness value based on the 5th percentile of effluent hardness measurements or the average ambient receiving water hardness measurements for those sites polluted with metals (lead, cadmium, copper, chromium (III), nickel, silver, and zinc). For purposes of calculating the applicable fresh water aquatic life criteria and effluent limitations for metals, the required fifth percentile hardness value has an upper limit of 400 mg/L as calcium carbonate, unless a site specific water effect ratio (WER) is

developed and approved by the Regional Water Board. The California Toxic Rule explains that if the hardness is over 400 mg/L, two options are available to calculate the freshwater metals criteria (which are used as the basis for setting effluent limitations): (1) Calculate the criterion using a default WER of 1.0 and using a hardness of 400 mg/L in the hardness equation; or (2) calculate the criterion using a WER and the actual ambient hardness of the surface water in the equation.

3. Order No. R8-2007-0041, page 8, modify paragraph II.B.3., as follows:

   3. For freshwater discharges, within forty five (45) days of the effective date of this Order, Dischargers from those sites polluted with leaded gasoline or metals shall submit for approval by the Regional Water Board Executive Officer the proposed hardness value based on 5th percentile of effluent hardness measurements or the average ambient freshwater receiving water hardness measurements. Once approved by the Executive Officer, this hardness value shall be the basis for determining the lead/metals effluent limits for the discharge from Attachment "B" of this Order.

4. Order No. R8-2007-0041, page 10, modify last paragraph of Finding B., as follows:

   In summary, this general permit will regulate discharges from activities involving groundwater dewatering, discharges that pose an insignificant threat to water quality, wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs and groundwater remediation in areas where contamination from petroleum hydrocarbons, solvents, metals and/or salts may be present. These activities include the following:

   1. Wastes associated with well installation, development, test pumping and purging;
   2. Aquifer testing wastes;
   3. Dewatering wastes from subterranean seepage;
   4. Groundwater dewatering wastes at construction sites;
   5. Groundwater remediation.
   6. Discharges resulting from hydrostatic testing of vessels, pipelines, tanks, etc.;
   7. Discharges resulting from the maintenance of potable water supply pipelines, tanks, reservoirs, etc.;
   8. Discharges resulting from the disinfection of potable water supply pipelines, tanks, reservoirs, etc.;
   9. Discharges from potable water supply systems resulting from initial system startup, routine startup, sampling of influent flow, system failures, pressure releases, etc.;
   10. Discharges from fire hydrant testing or flushing;
   11. Air conditioning condensate;
   12. Swimming pool discharge;

13. Discharges resulting from diverted stream flows;
14. Decanted filter backwash wastewater and/or sludge dewatering filtrate water from water treatment facilities;
15. Discharges of wastewater effluent associated with testing of selenium and nitrogen treatment technologies and BMPs into surface water; and
16. Other similar types of wastes as determined by the Regional Water Board Executive Officer, which pose a de minimus threat to water quality yet must be regulated under waste discharge requirements.

5. Order No. R8-2007-0041, page 17, add new paragraph G. in Section IV., as follows:

   G. The discharge of brine, resins, sludge or other secondary concentrates from treatment systems to surface waters is prohibited.

6. All other conditions and requirements of Order No. R8-2007-0041 shall remain unchanged

I, Gerard J. Thibeault, Executive Officer, do hereby certify that the foregoing is a full, true, and correct copy of an order adopted by the California Regional Water Quality Control Board, Santa Ana Region, on July 20, 2009.

Gerard J. Thibeault
Executive Officer

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Caltrans

FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY		REVISED BY	
	CHECKED BY		DATE REVISED	

Dist	COUNTY	LOCATION CODE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-20-07
REGISTERED ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-10 ELECT STATE OF CALIFORNIA

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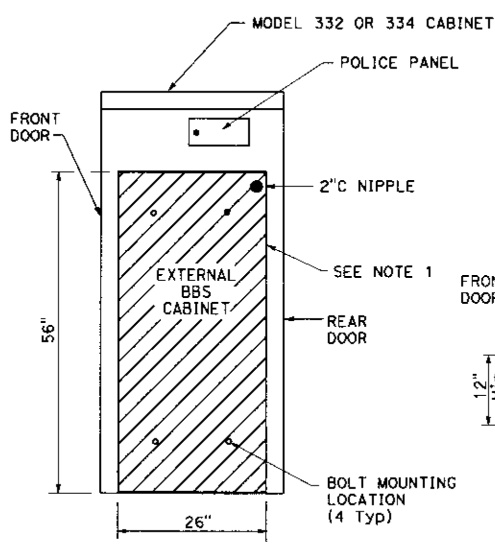
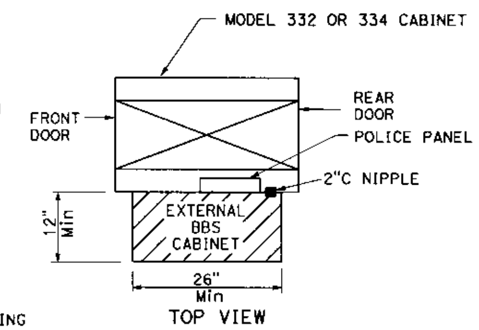
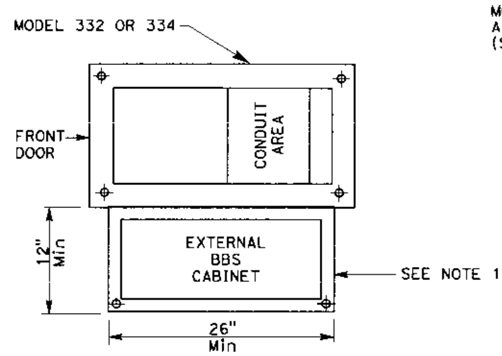


SIDE VIEW



TOP VIEW

EXTERNAL BBS CABINET MOUNTED TO THE MODEL 332 OR 334 CABINET

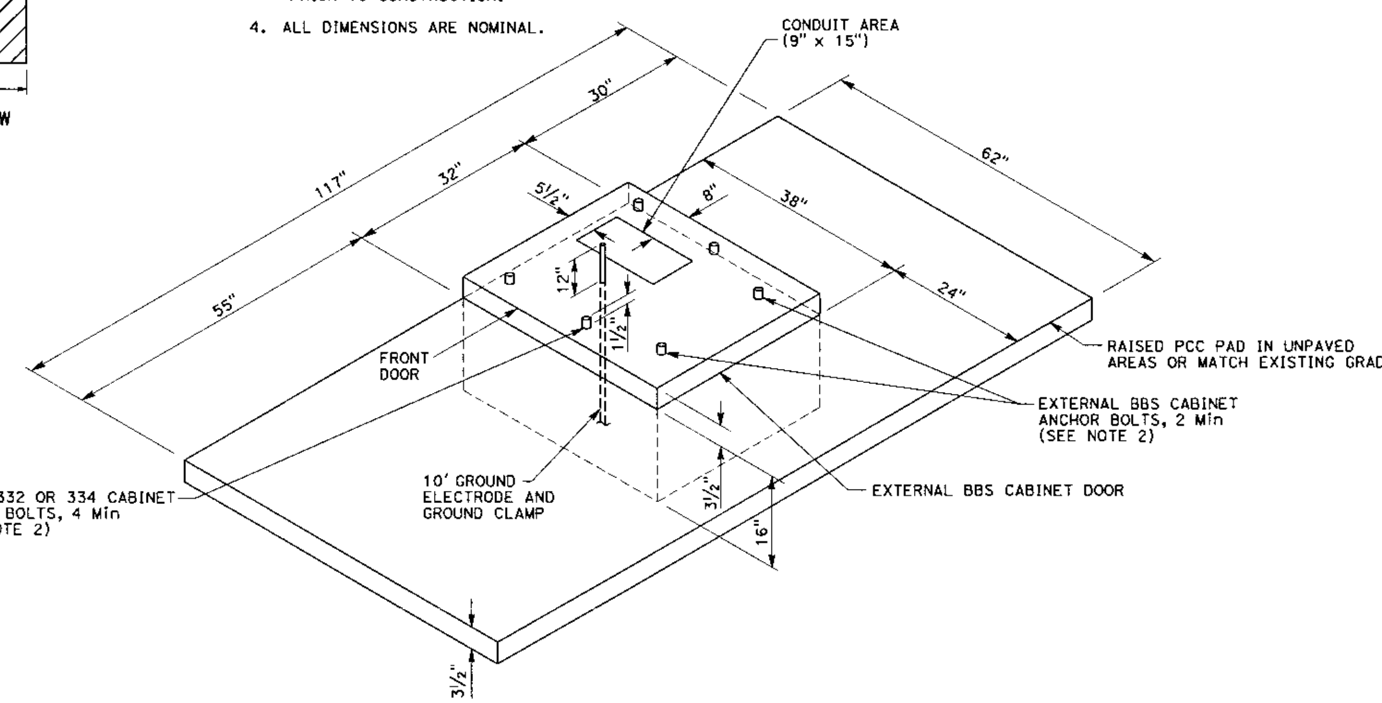


BASE PLAN FOR BBS MOUNTED TO THE MODEL 332 OR 334 CABINET

(FOR DIMENSIONS AND DETAILS NOT SHOWN, SEE SHEET A6-1 TO A6-4, CABINET HOUSING DETAILS OF THE TRANSPORTATION ELECTRICAL EQUIPMENT SPECIFICATION (TEES))

NOTE: (THIS SHEET ONLY)

1. THE EXTERNAL BBS CABINET SHALL BE MOUNTED TO THE MODEL 332 OR 334 CABINET WITH FOUR 18-8 STAINLESS STEEL HEX HEAD, FULLY-THREADED, ⅜"-16 X 1" BOLTS; TWO WASHERS PER BOLT, DESIGNED FOR ⅜" BOLTS AND ARE 18-8 STAINLESS STEEL, 1" OUTSIDE DIAMETER, ROUND, AND FLAT; AND ONE K-LOCK NUT PER BOLT THAT IS 18-8 STAINLESS STEEL AND A HEX-NUT. THE ENGINEER WILL HAVE TO APPROVE THE BOLT MOUNTING LOCATION PRIOR TO INSTALLATION.
2. THE ANCHOR BOLTS SHALL BE ¾" Dia X 15" WITH A 2"-90° BEND. THE CABINET MANUFACTURER'S SPECIFICATION SHALL DETERMINE THE LOCATION OF THE ANCHOR BOLTS IN THE FOUNDATION. THE ENGINEER WILL HAVE TO APPROVE THE ANCHOR BOLTS AND ITS LOCATION IN THE FOUNDATION PRIOR TO CONSTRUCTION.
3. THE CONTRACTOR SHALL VERIFY THE DIMENSIONS OF THE BBS CABINET PRIOR TO CONSTRUCTING THE FOUNDATION OF THE MODIFIED PORTION OF THE Std MODEL 332 AND 334 CABINET FOUNDATION. THE ENGINEER WILL HAVE TO APPROVE ANY NECESSARY DEVIATIONS PRIOR TO CONSTRUCTION.
4. ALL DIMENSIONS ARE NOMINAL.



MODIFIED MODEL 332 AND 334 CABINET FOUNDATION DETAIL FOR BATTERY BACKUP SYSTEM (BBS)

(FOR DIMENSIONS AND DETAILS NOT SHOWN AND ADDITIONAL NOTES, SEE SHEET ES-3C OF THE STANDARD PLANS FOR MODEL 332 AND 334 CABINETS)

ELECTRICAL SYSTEMS
(BBS FOUNDATION DETAILS)

NO SCALE

THIS PLAN IS ACCURATE FOR ELECTRICAL WORK ONLY.

LAST REVISION 2-2-09 | DATE PLOTTED => 13-MAR-2009 | TIME PLOTTED => 09:11

BORDER LAST REVISED 4/11/2008 | RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3 | USERNAME => trcoroi | DGN FILE => BBS Foundation.dgn | CU 00000 | EA 000000

Theresa Gabriel 12-20-07
REGISTERED ELECTRICAL ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-10 ELECT. STATE OF CALIFORNIA

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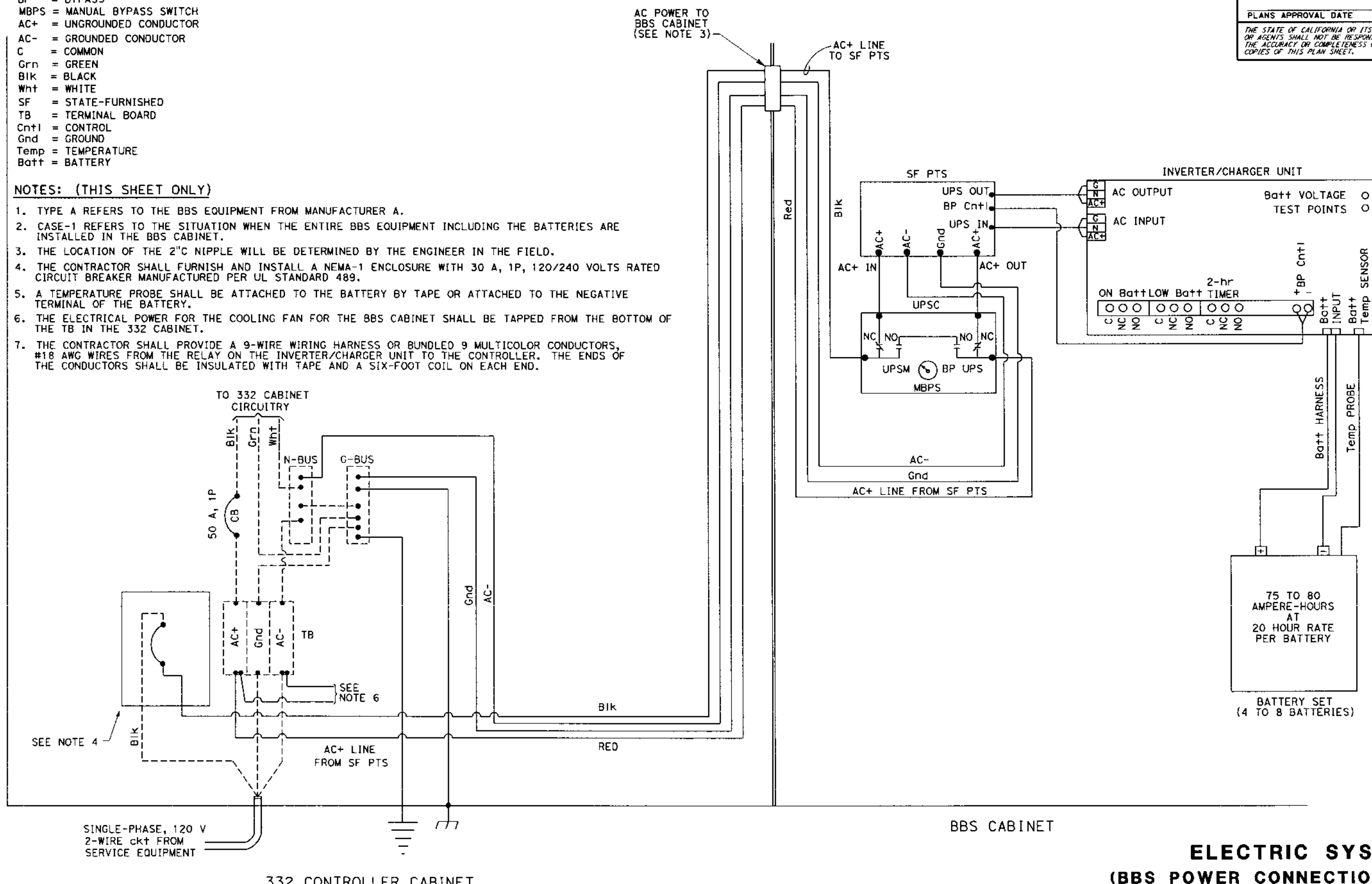
LEGEND: (THIS SHEET ONLY)

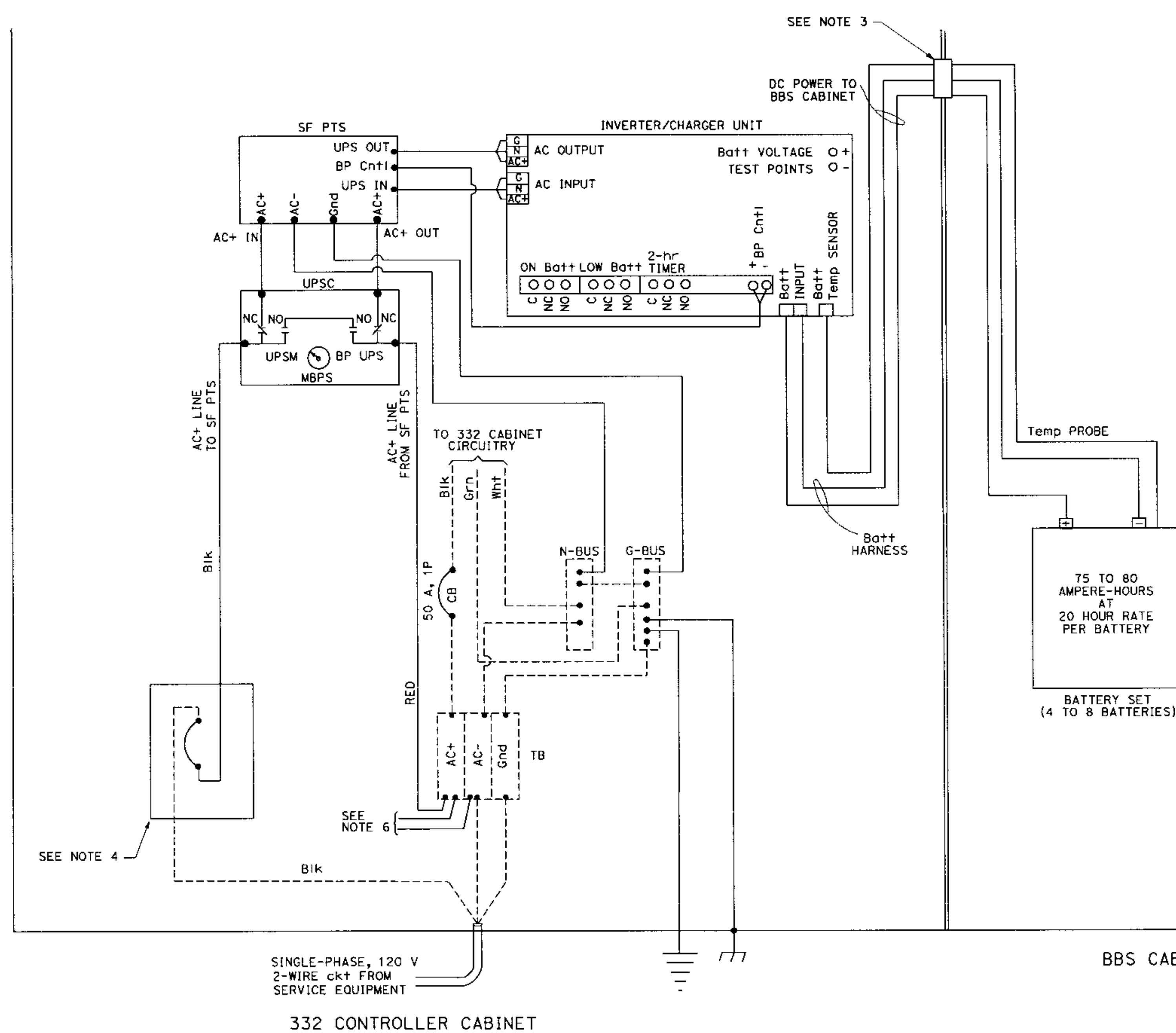
PTS = POWER TRANSFER SWITCH
UPS = UNINTERRUPTIBLE POWER SUPPLY
UPSC = UNINTERRUPTIBLE POWER SUPPLY CONTROLLER
UPSM = UPS MODE
BP = BYPASS
MBPS = MANUAL BYPASS SWITCH
AC+ = UNGROUNDED CONDUCTOR
AC- = GROUNDED CONDUCTOR
C = COMMON
Grn = GREEN
Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND
Temp = TEMPERATURE
Batt = BATTERY

NOTES: (THIS SHEET ONLY)

1. TYPE A REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER A.
2. CASE-1 REFERS TO THE SITUATION WHEN THE ENTIRE BBS EQUIPMENT INCLUDING THE BATTERIES ARE INSTALLED IN THE BBS CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OF THE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.



ELECTRIC SYSTEM
(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-1)



LEGEND: (THIS SHEET ONLY)

PTS = POWER TRANSFER SWITCH
UPS = UNINTERRUPTIBLE POWER SUPPLY
UPSC = UNINTERRUPTIBLE POWER SUPPLY CONTROLLER
UPSM = UPS MODE
BP = BYPASS
MBPS = MANUAL BYPASS SWITCH
AC+ = UNGROUNDED CONDUCTOR
AC- = GROUNDED CONDUCTOR
C = COMMON
Grn = GREEN
Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
Batt = BATTERY
Temp = TEMPERATURE
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND

NOTES: (THIS SHEET ONLY)

1. TYPE B REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER B.
2. CASE-2 REFERS TO THE SITUATION WHEN ONLY THE BATTERIES ARE INSTALLED IN THE BBS CABINET. THE REMAINING EQUIPMENT IS PLACED IN THE 332 CONTROLLER CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OFTHE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.

ELECTRICAL SYSTEMS
(BBS POWER CONNECTION DIAGRAM, TYPE A, CASE-2)

NO SCALE

Dist	COUNTY	LOCATION CODE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-20-07
REGISTERED CIVIL ENGINEER DATE

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp. 6-30-10 ELECT STATE OF CALIFORNIA

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION
Caltrans

FUNCTIONAL SUPERVISOR | CALCULATED-DESIGNED BY | CHECKED BY | REVISED BY | DATE REVISED

BORDER LAST REVISED 4/11/2008

RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3

USERNAME => rcorol
DGN FILE => BBS 250FSP.dgn

CU 00000

EA 000000

LAST REVISION 2-2-09
DATE PLOTTED => 13-MAR-2009
TIME PLOTTED => 09:07

Dist	COUNTY	LOCATION CODE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-20-07
REGISTERED ~~CIVIL ENGINEER~~ DATE

PLANS APPROVAL DATE

THE STATE OF CALIFORNIA OR ITS OFFICERS OR AGENTS SHALL NOT BE RESPONSIBLE FOR THE ACCURACY OR COMPLETENESS OF SCANNED COPIES OF THIS PLAN SHEET.

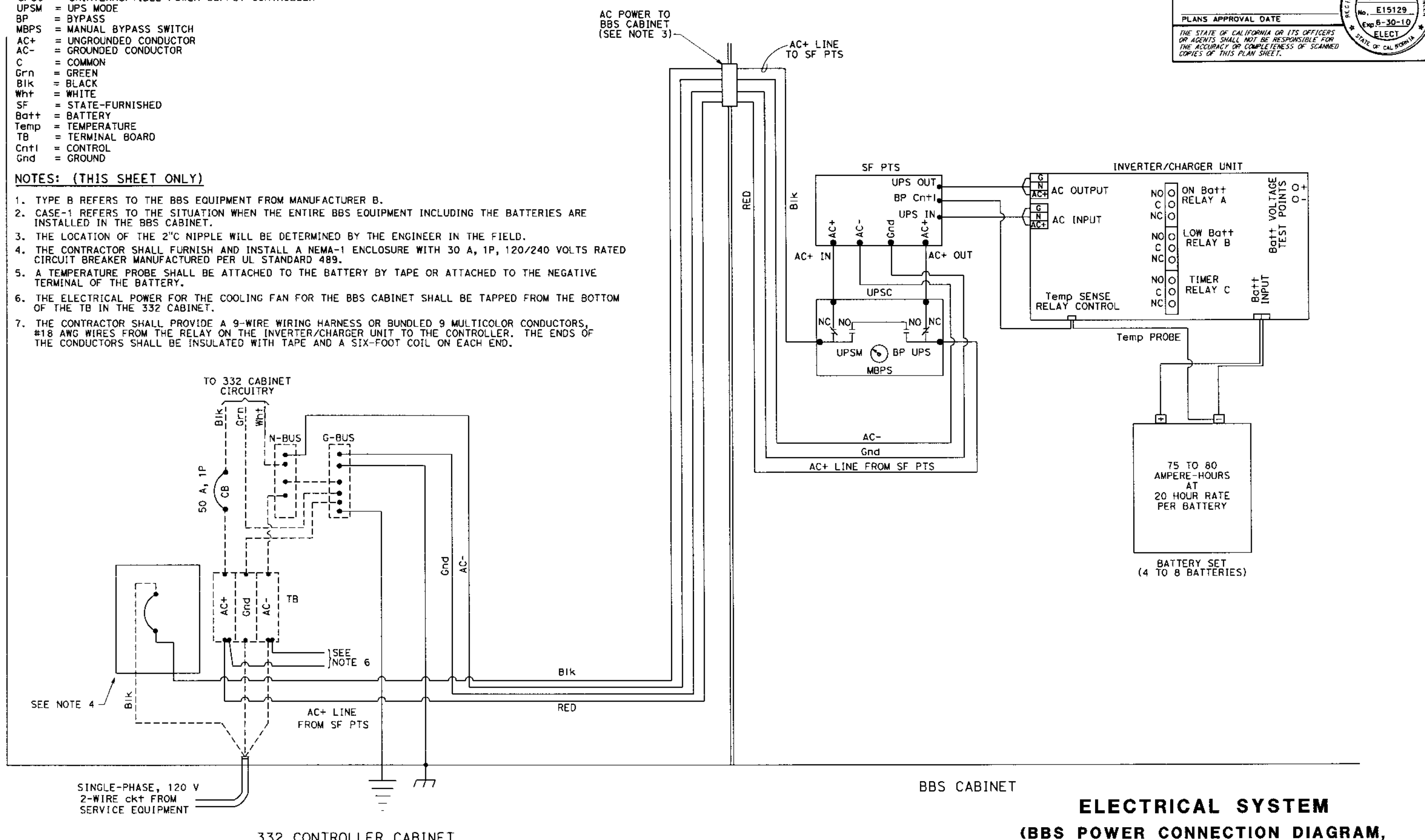
REGISTERED PROFESSIONAL ENGINEER
Theresa A. Gabriel
No. E15129
Exp. 6-30-10
ELECT
STATE OF CALIFORNIA

LEGEND: (THIS SHEET ONLY)

PTS = POWER TRANSFER SWITCH
UPS = UNINTERRUPTIBLE POWER SUPPLY
UPSC = UNINTERRUPTIBLE POWER SUPPLY CONTROLLER
UPSM = UPS MODE
BP = BYPASS
MBPS = MANUAL BYPASS SWITCH
AC+ = UNGROUNDED CONDUCTOR
AC- = GROUNDED CONDUCTOR
C = COMMON
Grn = GREEN
Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
Batt = BATTERY
Temp = TEMPERATURE
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND

NOTES: (THIS SHEET ONLY)

1. TYPE B REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER B.
2. CASE-1 REFERS TO THE SITUATION WHEN THE ENTIRE BBS EQUIPMENT INCLUDING THE BATTERIES ARE INSTALLED IN THE BBS CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OF THE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.



ELECTRICAL SYSTEM
(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-1)

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION	FUNCTIONAL SUPERVISOR	CALCULATED-DESIGNED BY		REVISED BY		
Caltrans		CHECKED BY		DATE REVISED		

Dist	COUNTY	LOCATION CODE	POST MILES TOTAL PROJECT	SHEET No.	TOTAL SHEETS

Theresa Gabriel 12-20-07
REGISTERED ~~CIVIL ENGINEER~~ ENGINEER DATE

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER Theresa A. Gabriel No. E15129 Exp 6-30-10 ELECT STATE OF CALIFORNIA

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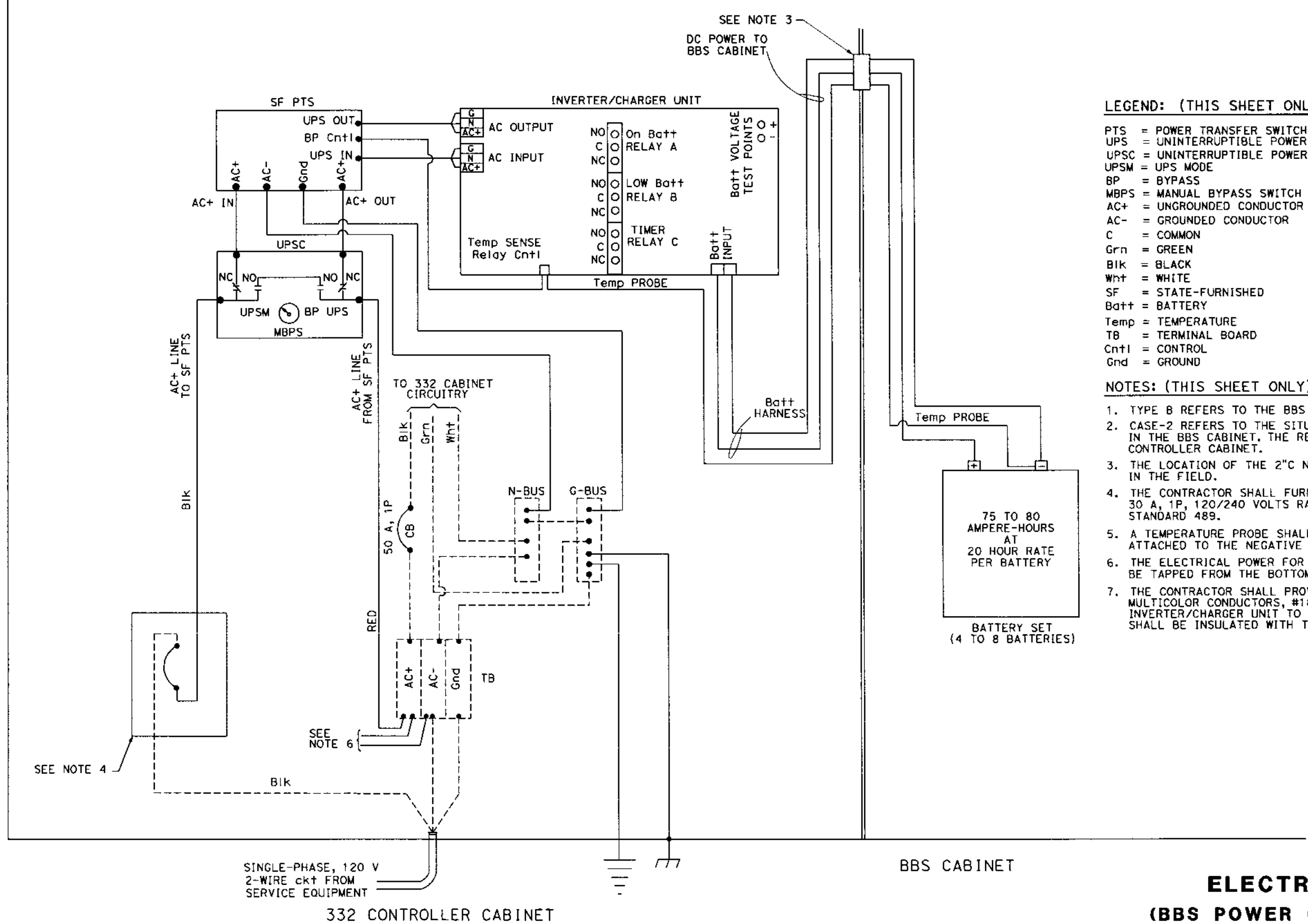

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Blk = BLACK
Wht = WHITE
SF = STATE-FURNISHED
Batt = BATTERY
Temp = TEMPERATURE
TB = TERMINAL BOARD
Cntl = CONTROL
Gnd = GROUND

NOTES: (THIS SHEET ONLY)

1. TYPE B REFERS TO THE BBS EQUIPMENT FROM MANUFACTURER B.
2. CASE-2 REFERS TO THE SITUATION WHEN ONLY THE BATTERIES ARE INSTALLED IN THE BBS CABINET. THE REMAINING EQUIPMENT IS PLACED IN THE 332 CONTROLLER CABINET.
3. THE LOCATION OF THE 2"C NIPPLE WILL BE DETERMINED BY THE ENGINEER IN THE FIELD.
4. THE CONTRACTOR SHALL FURNISH AND INSTALL A NEMA-1 ENCLOSURE WITH 30 A, 1P, 120/240 VOLTS RATED CIRCUIT BREAKER MANUFACTURED PER UL STANDARD 489.
5. A TEMPERATURE PROBE SHALL BE ATTACHED TO THE BATTERY BY TAPE OR ATTACHED TO THE NEGATIVE TERMINAL OF THE BATTERY.
6. THE ELECTRICAL POWER FOR THE COOLING FAN FOR THE BBS CABINET SHALL BE TAPPED FROM THE BOTTOM OF THE TB IN THE 332 CABINET.
7. THE CONTRACTOR SHALL PROVIDE A 9-WIRE WIRING HARNESS OR BUNDLED 9 MULTICOLOR CONDUCTORS, #18 AWG WIRES FROM THE RELAY ON THE INVERTER/CHARGER UNIT TO THE CONTROLLER. THE ENDS OFTHE CONDUCTORS SHALL BE INSULATED WITH TAPE AND A SIX-FOOT COIL ON EACH END.

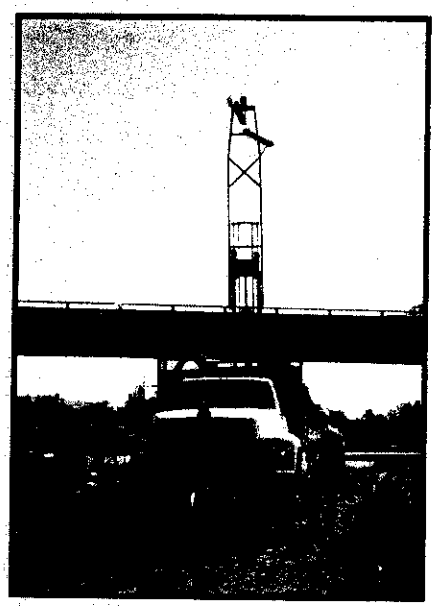
ELECTRICAL SYSTEM

(BBS POWER CONNECTION DIAGRAM, TYPE B, CASE-2)

LAST REVISION 3-11-09 | DATE PLOTTED => 13-MAR-2009 | TIME PLOTTED => 09:10

BORDER LAST REVISED 4/11/2008 | RELATIVE BORDER SCALE IS IN INCHES 0 1 2 3 | USERNAME => trcarol | DGN FILE => BBS DUIFSP.dgn | CU 00000 | EA 000000

**SITE INVESTIGATION
LANE ADDITION OF SOUTHBOUND
STATE ROUTE 55
SANTA ANA, CALIFORNIA
TASK ORDER NO. 12-0G9601-29
EA NO. 0G9601, CONTRACT NO. 12A1139**



Geotechnical

and

Environmental

Sciences

Consultants

Ninyo & Moore

EXECUTIVE SUMMARY

In accordance with the State of California, Department of Transportation Contract No. 12A1139, Task Order No. 29, Ninyo & Moore has performed a site investigation (SI) along East Dyer Road and State Route 55 (SR-55) in the city of Santa Ana, California (site; Figure 1).

An ExxonMobil Service Station (Station) at 1351 East Dyer Road (approximately 120 feet northwest and upgradient of the site) has a known fuel release to soil and groundwater. The dissolved plume has extended downgradient (southeast) at least as far as the site.

The objective of the SI is to evaluate the potential concerns associated with the Station in soil and groundwater at the site in areas of proposed earthmoving. The results of this SI will be used to evaluate worker safety and soil and groundwater handling procedures. Sampling locations were screened for contaminants of concern in soil and groundwater including: total petroleum hydrocarbons as gasoline, total petroleum hydrocarbons as diesel, Title 22 Metals, and volatile organic compounds (VOCs).

Based on the results of this SI the following conclusions have been made:

- Concentrations of Title 22 Metals detected in the surface samples at the site do not cause the soil to require special handling.
- Soil beneath the site is impacted with fuel products to the maximum depth explored (20 feet below ground surface [bgs]). Because groundwater was encountered at approximately 13 feet bgs and fuel products are less dense than water (i.e. they float and dissolve near the surface of the water table), it is conservative to assume that the soil below the water table to the maximum expected construction depth is also impacted with fuel products.
- Groundwater beneath the site is impacted with fuel products at levels that exceed the maximum contaminant levels. Because groundwater was encountered at approximately 13 feet bgs, it is likely that construction dewatering will occur and contaminated groundwater handling will be needed at this site.
- The soil and groundwater impacts most likely resulted from a release from the Station upgradient from the site.

RECOMMENDATIONS

The following recommendations are based on the results and conclusions of the SI:

- A site-specific Health and Safety Plan for the construction work should be prepared and reviewed by a Certified Industrial Hygienist.
- Occupational Safety and Health Administration hazardous waste operations and emergency response trained field personnel should be used for subsurface activities associated with this project.
- Due to the presence of soil impacted with fuel products at the site, earthwork associated with this project should be conducted in accordance with South Coast Air Quality Management District (SCAQMD) Rule 1166.
- Monitoring for the presence of VOCs should be conducted with a photo-ionization detector (PID) as required in the SCAQMD Rule 1166 permit.
- Excavated soil should be stockpiled on heavy tarpaulins or plastic sheeting and kept moist during working hours to control potential vapor emissions. Stockpiles should be covered with plastic sheeting at the end of the day. The edges of the plastic should have an overlap of at least 24 inches. The plastic should be secured at the base of the stockpile and along the seams of overlapping plastic sheeting with sandbags or equivalent. Completed stockpiles should remain covered until load-out or reuse.
- Soils with PID readings under 5 parts per million (ppm) above ambient levels and exhibit no odors or soil staining should be considered potentially clean and placed in a "potentially clean" stockpile for confirmation soil sampling and potential reuse.
- Soils with PID readings greater than 5 ppm above ambient levels should be considered VOC-impacted and should be stockpiled for off-site treatment or disposal. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."
- Soils with PID readings greater than 50 ppm above ambient levels should be considered VOC-impacted and stockpiled and covered or direct-loaded into trucks for off-site treatment or disposal and the SCAQMD should be notified within 24 hours of discovery. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."
- Soils with PID readings greater than 1,000 ppm above ambient levels should be considered VOC-impacted and sprayed with water or suppression foam, placed directly into covered contains for off-site treatment or disposal, and the SCAQMD should be notified within 1 hour of discovery. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."

- Soil placed in the "potentially clean" stockpile should be sampled at a rate of:

Volume (cubic yards)	Sampling Frequency
0 - 500	1 sample per 100 cubic yards
501 - 1,000	1 sample per 250 cubic yards
1,001 - 5,000	1 sample per 250 cubic yards for first 1000 cubic yards 1 sample per 500 cubic yards thereafter
5,001 - 20,000	12 samples for first 5,000 cubic yards 1 sample per 1,000 cubic yards thereafter
>20,000	1 sample per 2,000 cubic yards for first 20,000 cubic yards 1 sample per 2,500 cubic yards thereafter

The samples should be analyzed for total petroleum hydrocarbons (TPHs), VOCs, and oxygenates using United States Environmental Protection Agency (EPA) Methods 8015M and 8260B, respectively. If results are non-detect, the soil can be considered clean and be reused on site. Otherwise, the soil should be transported off site for treatment or disposal as a "petroleum-contaminated non-hazardous waste."

- Soil that is considered a "petroleum-contaminated non-hazardous waste" will need to be profiled by a receiving facility licensed to receive this type of waste (Crosby & Overton in Long Beach, California is such a facility). Typical transportation and disposal costs are $70 to $100 per ton.

- If groundwater dewatering is needed for the construction activities, the water will need to be containerized and disposed or treated at a facility licensed to receive the waste. Alternatively, the water could be treated and discharged at the site in accordance with a National Pollutant Discharge Elimination System permit, if adequate treatment can be designed.

1. INTRODUCTION

In accordance with the State of California, Department of Transportation (Department) Contract No. 12A1139, Task Order No. 29 (TO-29), Ninyo & Moore has performed a site investigation (SI) along East Dyer Road and State Route 55 (SR-55) in the city of Santa Ana, California (site; Figure 1). This report is based on conditions at the site at the time of the sampling activities and provides documentation of our findings and recommendations.

1.1. Project Location

The Department is currently preparing the plans, specifications, and estimates (PS&E) to add an auxiliary lane in the southbound direction of SR-55 between Edinger Avenue on-ramp and East Dyer Road off-ramp.

1.2. Proposed Project

Dyer Road under-crossing (UC) (Bridge No 55-409) will be widened and a new standard retaining wall will be constructed to accommodate the proposed widening. In support of the project, Ninyo & Moore has conducted a SI at the site (Figure 2).

2. BACKGROUND

An ExxonMobil Service Station (Station) at 1351 East Dyer Road is approximately 120 feet northwest of the Dyer Road UC Bridge No 55-409. According to information on the State Water Resources Control Board (SWRCB) GeoTracker website (GeoTracker) an unauthorized gasoline release to groundwater was discovered at the Station in 1981. A Groundwater Monitoring and Remedial Progress Report prepared by ETIC Engineering (ETIC) dated September 25, 2009, was reviewed. According to the report, quarterly monitoring was initiated under the direction of the Regional Water Quality Control Board (RWQCB) in 1989.

Monitoring wells were installed at the Station as well as to the southeast of the Station in down-gradient locations (in the vicinity of this SI). Historical maximum concentrations of total petroleum hydrocarbons as gasoline (TPHg) 5,900,000 micrograms per liter (μg/l) and benzene of 30,000 μg/l were reported to have been detected in groundwater samples collected at the Station in 1992. During the most recent groundwater monitoring event (September 25, 2009), TPHg was detected at 430 μg/l and benzene at 1.7 μg/l in groundwater samples collected at the Station.

Monitoring well EW3A is in the vicinity of the site and was installed in 1998, on the north side of East Dyer Road (Figure 2). Depth to groundwater was reported to range from approximately 10 to 14 feet below ground surface (bgs). Historical maximum concentrations of TPHg (12,900 μg/l) and benzene (805 μg/l) were detected in 1998 and 1999, respectively. Concentrations of TPHg and benzene have been reported to be non-detect in this well since 2006.

A groundwater pump and treatment system was installed at the Station in 2000 to remediate the contaminated groundwater. The system was operated through 2006. Underground storage tanks (USTs) were replaced at the Station in 2002. The Station is currently undergoing post-remedial monitoring activities.

3. GEOLOGY/HYDROGEOLOGY

Based on our review of documents published by the State of California Division of Mines and Geology (CDMG), the site area is in the Orange County Plain within the Transverse Ranges Geomorphic Province of California. The site vicinity is underlain by a thick sequence of alluvial deposits derived from the Santa Ana River. These deposits are predominately gravels, sands, and silts. Below the alluvium are consolidated sedimentary rocks and older crystalline basement rocks. The crystalline rocks are of two types: the eastern igneous and metamorphic, and the western Catalina schist. Younger (late Cretaceous, lower Miocene, Pliocene and Pleistocene) sedimentary rocks of marine origin overlie the crystalline basement rocks. These sedimentary rocks are composed of conglomerates, sandstones, and shale accumulating to 17,000 feet thick (CDMG, 1966).

Sediments encountered during the SI consist of sandy silt and silty sand from the surface to 13 feet bgs and clay from 13 to 20 feet bgs (maximum depth explored). Groundwater was encountered at approximately 13 feet bgs and had a petroleum odor. Refer to Appendix B for copies of boring logs.

Based on review of records associated with the Station, groundwater was reported to flow from the northwest to the southeast.

4. OBJECTIVE

The objective of the SI is to evaluate the potential concerns associated with the Station in soil and groundwater at the site in areas of proposed earthmoving. The results of this SI will be used to evaluate worker safety and soil and groundwater handling procedures. Sampling locations were screened for contaminants of concern (COC) in soil and groundwater including: TPHg, total petroleum hydrocarbons as diesel (TPHd), Title 22 Metals, and volatile organic compounds (VOCs).

5. SCOPE OF WORK

The following scope of work was performed in accordance with the work plan.

5.1. Site-Specific Health and Safety Plan (HSP)

Ninyo & Moore prepared and provided a site-specific HSP under separate cover, based on the scope of work and potential hazards observed during a site reconnaissance. The HSP was prepared in accordance with applicable local, state, and federal regulations. The HSP covered the field activities conducted by Ninyo & Moore personnel and was approved by a California Certified Industrial Hygienist (CIH).

5.2. Site Investigation

5.2.1. Site Reconnaissance

Ninyo & Moore and the Department conducted a site walk on November 4, 2009. Three boring locations were selected by the Department. Ninyo & Moore marked the locations with white spray paint at the approximate locations shown on Figure 2.

5.2.2. Underground Service Alert (USA)

Ninyo & Moore obtained the inquiry identification number (A93010618) from USA at least 48 hours prior to start of work at the site. This number was obtained for the proposed SI borings.

5.2.3. Soil Sampling

Three direct-push borings (B1, B2, and B3) were advanced on November 11, 2009 at the approximate locations shown on Figure 2. Soil samples were collected at the surface, 5, 10, 15, and 20 feet bgs. A total of 15 soil samples were collected. The borings were advanced to a depth of 20 feet bgs. Groundwater was encountered at approximately 13 feet bgs in each boring.

Soil samples were collected using clean acetate liners and direct push methods. Excess soil not collected as a sample was placed in a Department of Transportation (DOT) approved container and stored at the site pending removal. Refer to Appendix A for sampling procedures.

Sample containers were labeled with the boring number and sample depth. Sampling information, time, date of sample collection, sample matrix type, turn-around-time, container type, requested analysis, and other information was recorded on the chain-of-custody form. Soil samples were stored in an ice chest for transport within 24 hours of collection to a laboratory certified by the State of Department of Health Services Environmental Laboratory Accreditation Program (ELAP).

5.2.4. Groundwater Sampling

After the borings reached total depth (20 feet bgs), they were converted to temporary groundwater sampling points. Grab groundwater samples were collected from borings B2 and B3 boring as described in Appendix A. A grab groundwater sample was not collected from boring B1 due to poor groundwater recovery at this location.

Sample containers were labeled with the boring number. Sampling information, time, date of sample collection, sample matrix type, turn-around-time, container type, requested analysis, and other information was recorded on the chain-of-custody. Samples were stored in an ice chest for transport within 24 hours of collection to a state-certified ELAP laboratory.

5.2.5. Decontamination

Clean and decontaminated sampling equipment was used for each borehole location. Sampling equipment was new or decontaminated between boreholes to prevent introduction of foreign materials and cross-contamination. Specific decontamination procedures are described in Appendix A.

5.2.6. Investigative Derived Wastes (IDW)

Decontamination water and soil generated from the SI was placed in a DOT-approved drum and stored at the site. The drum was subsequently transported to Crosby & Overton in Long Beach, California, under a waste manifest. A copy of the manifest is presented in Appendix C.

Discarded equipment/items, such as gloves and pails, were not considered hazardous and can be disposed at a permitted disposal facility. Discarded equipment that is to be disposed, which can still be re-used, was rendered inoperable prior to its disposal in the refuse facility at the direction of the Department.

5.3. Global Positioning Satellite (GPS) Data Collection

Borings were located using a GPS. Approximate latitude and longitude of the North American Datum (NAD 83) were recorded. GPS location data is presented in Table 1.

5.4. Laboratory Analysis

Soil samples were analyzed for TPHg and TPHd by modified United States Environmental Protection Agency (EPA) Method 8015B, and VOCs by EPA Method 8260B/5035. The surface samples were also analyzed for Title 22 Metals by EPA Method 6010B.

Groundwater samples were analyzed for TPHg and THPd, by modified EPA Method 8015B, and VOCs by EPA Method 8260B.

The laboratory limit on the analysis is reported as Method Detection Limit (MDL) and Practical Quantitation Limit (PQL). Soil and groundwater samples were analyzed by Advanced Technology Laboratories (ATL), a state-certified ELAP laboratory in Signal Hill, California. Copies of laboratory reports are presented in Appendix D.

5.5. Quality Control And Quality Assurance (QA/QC)

5.5.1. Field QA/QC

Field procedures, including decontamination of field sampling equipment, described in Appendix A, were used to ensure quality of samples during field sampling. Duplicate samples were not collected.

5.5.2. Laboratory QA/QC

ATL analyzed samples in accordance with the requirements of their in-house QA/QC program (a copy of which will be provided to the Department upon request) and the requirements of contract 12A1139.

6. RESULTS

6.1. Physical Results

Sediments encountered consist of sandy silt and silty sand from 0 to 13 feet bgs and clay from 13 to 20 feet bgs (maximum depth explored). Groundwater was encountered at approximately 13 feet bgs and had a petroleum odor. Refer to Appendix B for copies of boring logs.

6.2. Chemical and Metals Results for Soil Samples

Results of the chemical analyses of soil samples are summarized in Tables 2, 3, and 4, and selected results on Figure 3. Laboratory reports are presented in Appendix D. Chemical results for the soil samples are summarized as follows:

- Concentrations of TPHg were detected soil samples B1-15, B1-20, B2-15, B2-20, B3-15, and B3-20 ranging from 1.2 to 490 milligrams per kilogram (mg/kg). Concentrations of TPHd were detected in soil samples B2-05, B2-20, and B3-0.5 ranging from 14 to 140 mg/kg. The concentrations of TPHg detected in samples B2-15 and B3-15 and TPHd detected in sample B3-0.5 exceed the soil screening levels (SSLs) for the protection of groundwater published by the Los Angeles Regional Water Quality Control Board (RWQCB, 1996). The SSLs are not criteria for classifying soil as a hazardous waste.

- Concentrations of VOCs were detected in six soil samples. Benzene was detected in soil samples B1-15, B1-20, B2-15, B2-20, B3-15, and B3-20 at concentrations ranging from 27 to 6,600 micrograms per kilograms (µg/kg). Toluene was detected in soil sample B3-20 at 15 µg/kg. Ethylbenzene was detected in soil samples B1-15, B1-20, B2-15, B2-10, B3-15 and B3-20 at concentrations ranging from 81 to 10,000 µg/kg. Xylene was detected in soil samples B1-20, B2-15, B3-15, and B3-20 at concentrations ranging from 10 to 7,720 µg/kg. Other fuel related VOCs were detected in six other soil samples. The concentrations of benzene in B2-15 and B2-20 are in excess of the EPA Region 9 regional screening levels (RSLs) of 5,600 µg/kg (EPA, 2009). Other VOCs detected were either below their respective RSLs, or did not have an established RSL.

- Detected Title 22 Metals concentrations were below respective State of California Total Threshold Limit Concentrations (TTLCs) and below 10 times the State of California Soluble Threshold Limit Concentrations (STLC).

6.3. Chemical Results for Groundwater Samples

Results of the chemical analyses of groundwater samples are summarized in Tables 5 and 6 and selected results on Figure 3. A copy of the laboratory report is included in Appendix D. Results for the groundwater samples are summarized as follows:

- Dissolved TPHg was detected at 110 milligrams per liter (mg/l) in sample B2 and 5.6 mg/l in sample B3. Dissolved TPHd was detected at 6.4 mg/l in sample B2 and 1.7 mg/l in sample B3. There are no established maximum contaminant levels (MCL) for TPHg or TPHd.

- Dissolved VOCs were detected in both samples, B2 and B3. Benzene was detected at 31,000 μg/l in B2 and 870 μg/l in B3. The MCL for benzene is 1 μg/l. Toluene was detected at 1,300 μg/l in B2 and 74 μg/l in B3. The MCL for toluene is 150 μg/l. Ethylbenzene was detected at 2,900 μg/l in B2 and 54 μg/l in B3. The MCL for ethylbenzene is 300 μg/l. Xylene was detected at 5,300 μg/l in B2 and 181 μg/l in B3. The MCL for xylene is 1,750 μg/l. Other VOCs were detected, however they were either below their corresponding MCL or did not have an assigned MCL.

7. CONCLUSIONS

Based on the results of this SI the following conclusions have been made:

- Concentrations of Title 22 Metals detected in the surface samples at the site do not cause the soil to require special handling.

- Soil beneath the site is impacted with fuel products to the maximum depth explored (20 feet bgs). Because groundwater was encountered at approximately 13 feet bgs and fuel products are less dense than water (i.e. they float and dissolve near the surface of the water table), it is conservative to assume that the soil below the water table to the maximum expected construction depth is also impacted with fuel products.

- Groundwater beneath the site is impacted with fuel products at levels that exceed the MCLs. Because groundwater was encountered at approximately 13 feet bgs it is likely that construction dewatering will occur and contaminated groundwater handling will be needed at this site.

- The soil and groundwater impacts most likely resulted from a release from the Station upgradient from the site.

8. RECOMMENDATIONS

The following recommendations are based on the findings of this assessment.

- A site-specific HSP for the construction work should be prepared. This HSP should be reviewed by a CIH.

- Occupational Safety and Health Administration (OSHA) hazardous waste operations and emergency response (HAZWOPER) trained field personnel should be used for subsurface activities associated with this project.

- Due to the presence of soil impacted with fuel products at the site, earthwork associated with this project should be conducted in accordance with South Coast Air Quality Management District (SCAQMD) Rule 1166.

- Monitoring for the presence of VOCs should be conducted with a PID as required in the SCAQMD Rule 1166 permit.

- Excavated soil should be stockpiled on heavy tarpaulins or plastic sheeting and kept moist during working hours to control potential vapor emissions. Stockpiles should be covered with plastic sheeting at the end of the day. The edges of the plastic should have an overlap of at least 24 inches. The plastic should be secured at the base of the stockpile and along the seams of overlapping plastic sheeting with sandbags or equivalent. Completed stockpiles should remain covered until load-out or reuse.

- Soils with PID readings under 5 ppm above ambient levels and exhibit no odors or soil staining should be considered potentially clean and placed in a "potentially clean" stockpile for confirmation soil sampling and potential reuse.

- Soils with PID readings greater than 5 ppm above ambient levels should be considered VOC-impacted and should be stockpiled for off-site treatment or disposal. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."

- Soils with PID readings greater than 50 ppm above ambient levels should be considered VOC-impacted and stockpiled and covered or direct-loaded into trucks for off-site treatment or disposal and the SCAQMD should be notified within 24 hours of discovery. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."

- Soils with PID readings greater than 1,000 ppm above ambient levels should be considered VOC-impacted and sprayed with water or suppression foam, placed directly into covered contains for off-site treatment or disposal, and the SCAQMD should be notified within 1 hour of discovery. Based on existing sampling results, the soil would be considered "petroleum-contaminated non-hazardous waste."

- Soil placed in the "potentially clean" stockpile should be sampled at a rate of:

Volume (cubic yards)	Sampling Frequency
0 - 500	1 sample per 100 cubic yards
501 - 1,000	1 sample per 250 cubic yards
1,001 - 5,000	1 sample per 250 cubic yards for first 1000 cubic yards 1 sample per 500 cubic yards thereafter
5,001 - 20,000	12 samples for first 5,000 cubic yards 1 sample per 1,000 cubic yards thereafter
>20,000	1 sample per 2,000 cubic yards for first 20,000 cubic yards 1 sample per 2,500 cubic yards thereafter

The samples should be analyzed for TPHs, VOCs, and oxygenates using EPA Methods 8015M and 8260B, respectively. If results are non-detect, the soil can be considered clean and be reused on site. Otherwise, the soil should be transported off site for treatment or disposal as a "petroleum-contaminated non-hazardous waste."

- Soil that is considered a "petroleum-contaminated non-hazardous waste" will need to be profiled by a receiving facility licensed to receive this type of waste (Crosby & Overton in Long Beach, California is such a facility). Typical transportation and disposal costs are $70 to $100 per ton.

- If groundwater dewatering is needed for the construction activities, the water will need to be containerized and disposed or treated at a facility licensed to receive the waste. Alternatively, the water could be treated and discharged at the site in accordance with a National Pollutant Discharge Elimination System permit, if adequate treatment can be designed.

9. LIMITATIONS

The services outlined in this report have been conducted in a manner generally consistent with current regulatory guidelines. No warranty, expressed or implied, is made regarding the professional opinions presented in this report. Ninyo & Moore's opinions are based on an analysis of observed conditions and on information obtained from third parties. It is likely that variations in soil conditions may exist which were beyond the scope of work.

The samples collected and chemically analyzed and the observations made are believed to be representative of the general area evaluated; however, conditions can vary significantly between sampling locations. The interpretations and opinions contained in this report are based on the results of laboratory tests and analyses intended to detect the presence and measure the concentration of certain chemical or physical constituents in samples collected from the site. The analyses have been conducted by an independent laboratory, which is accredited by the United States EPA and/or certified by the State of California to conduct such analyses. Ninyo & Moore has no involvement in, or control over, such analyses and has no means of confirming the accuracy of laboratory results. Ninyo & Moore, therefore, disclaims any responsibility for inaccuracy in such laboratory results.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires any additional information, or has questions regarding content, interpretations presented, or completeness of this document. Opinions and judgments expressed herein, which are based on our understanding and interpretation of current regulatory standards, should not be construed as legal opinions.

10. REFERENCES

California Division of Mines and Geology (CDMG), 1966, Geologic Map of California, Santa Ana Sheet, dated 1966.

Environmental Protection Agency (EPA), Region 9, 2009, Regional Screening Levels Industrial Soil, dated April, 2009.

ETIC Engineering (ETIC), 2009, Groundwater Monitoring, and Remedial Progress Report, Exxon/Mobil Station, 1351 East Dyer Road, Santa Ana, California, dated September 25, 2009.

Los Angeles Regional Water Quality Control Board (RWQCB), 1996, Interim Site Assessment & Cleanup Guidebook, dated May 1996.

State Water Resources Control Board (SWRCB), 2009, GeoTracker webpage.

APPENDIX A

APPENDIX B

REFERENCE: GOOGLE AERIAL PHOTO, 2009.

N

APPROXIMATE SCALE IN FEET

0 40 80

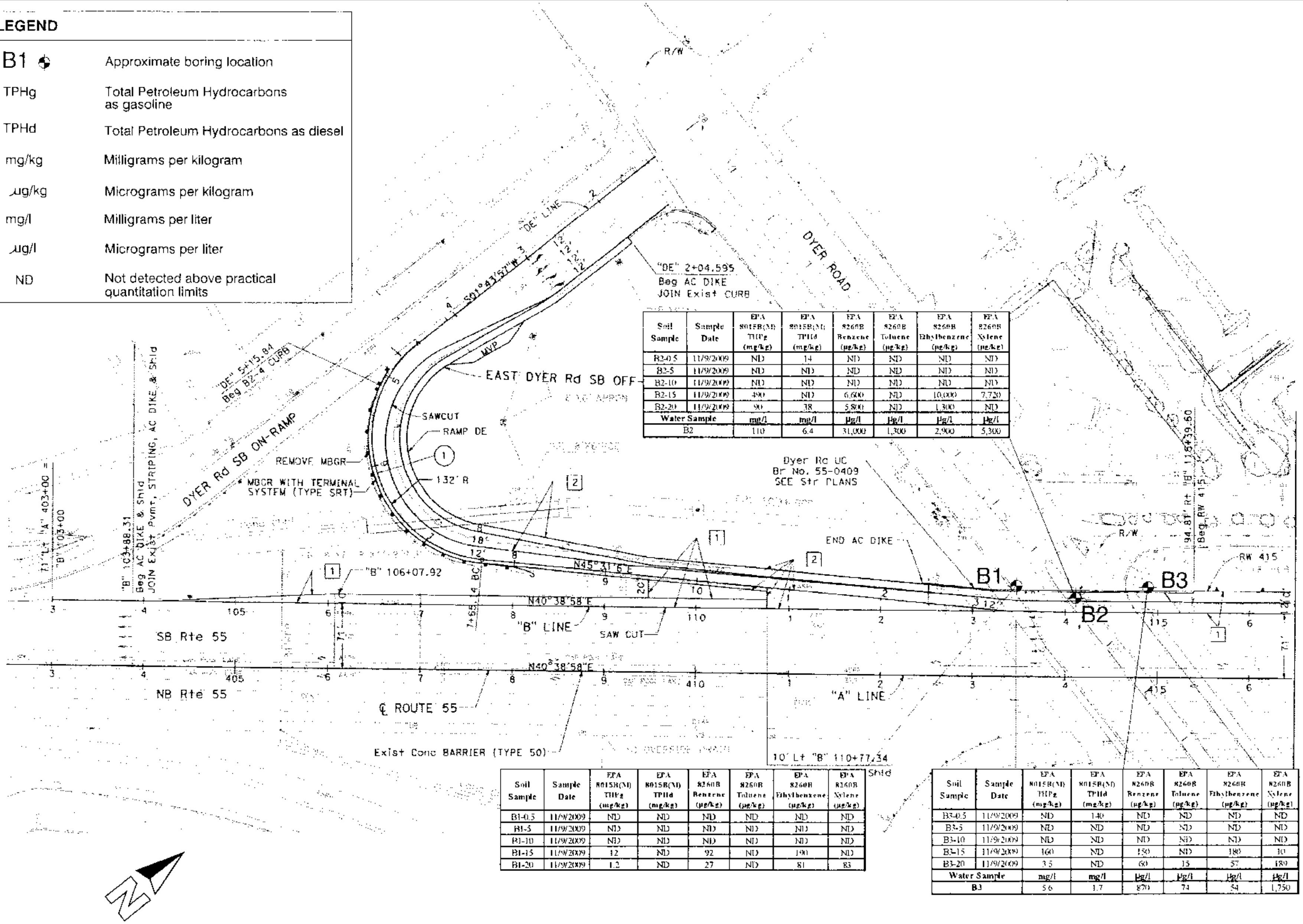
NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

LEGEND	
B3	APPROXIMATE LOCATION OF BORING
EW3A	EXISTING MONITORING WELL

Ninyo & Moore		BORING LOCATIONS	FIGURE
PROJECT NO.	DATE	EAST DYER ROAD AND SR-55 SANTA ANA, CALIFORNIA	2
207384029	1/10		

LEGEND

B1 ⊕	Approximate boring location
TPHg	Total Petroleum Hydrocarbons as gasoline
TPHd	Total Petroleum Hydrocarbons as diesel
mg/kg	Milligrams per kilogram
µg/kg	Micrograms per kilogram
mg/l	Milligrams per liter
µg/l	Micrograms per liter
ND	Not detected above practical quantitation limits



Soil Sample	Sample Date	EPA 8015B(M) TPHg (mg/kg)	EPA 8015B(M) TPHd (mg/kg)	EPA 8260B Benzene (µg/kg)	EPA 8260B Toluene (µg/kg)	EPA 8260B Ethylbenzene (µg/kg)	EPA 8260B Xylene (µg/kg)
B2-0.5	11/9/2009	ND	14	ND	ND	ND	ND
B2-5	11/9/2009	ND	ND	ND	ND	ND	ND
B2-10	11/9/2009	ND	ND	ND	ND	ND	ND
B2-15	11/9/2009	490	ND	6,600	ND	10,000	7,720
B2-20	11/9/2009	90	38	5,800	ND	1,300	ND
Water Sample		mg/l	mg/l	µg/l	µg/l	µg/l	µg/l
B2		110	6.4	31,000	1,300	2,900	5,300

Soil Sample	Sample Date	EPA 8015B(M) TPHg (mg/kg)	EPA 8015B(M) TPHd (mg/kg)	EPA 8260B Benzene (µg/kg)	EPA 8260B Toluene (µg/kg)	EPA 8260B Ethylbenzene (µg/kg)	EPA 8260B Xylene (µg/kg)
B1-0.5	11/9/2009	ND	ND	ND	ND	ND	ND
B1-5	11/9/2009	ND	ND	ND	ND	ND	ND
B1-10	11/9/2009	ND	ND	ND	ND	ND	ND
B1-15	11/9/2009	12	ND	92	ND	190	ND
B1-20	11/9/2009	1.2	ND	27	ND	81	83

Soil Sample	Sample Date	EPA 8015B(M) TPHg (mg/kg)	EPA 8015B(M) TPHd (mg/kg)	EPA 8260B Benzene (µg/kg)	EPA 8260B Toluene (µg/kg)	EPA 8260B Ethylbenzene (µg/kg)	EPA 8260B Xylene (µg/kg)
B3-0.5	11/9/2009	ND	140	ND	ND	ND	ND
B3-5	11/9/2009	ND	ND	ND	ND	ND	ND
B3-10	11/9/2009	ND	ND	ND	ND	ND	ND
B3-15	11/9/2009	160	ND	150	ND	180	10
B3-20	11/9/2009	3.5	ND	60	15	57	189
Water Sample		mg/l	mg/l	µg/l	µg/l	µg/l	µg/l
B3		5.6	1.7	870	74	54	1,750

APPROXIMATE SCALE IN FEET

0 150 300

NOTE: ALL DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

Ninyo & Moore

PROJECT NO.	DATE
207384029	1/10

BORING DATA MAP

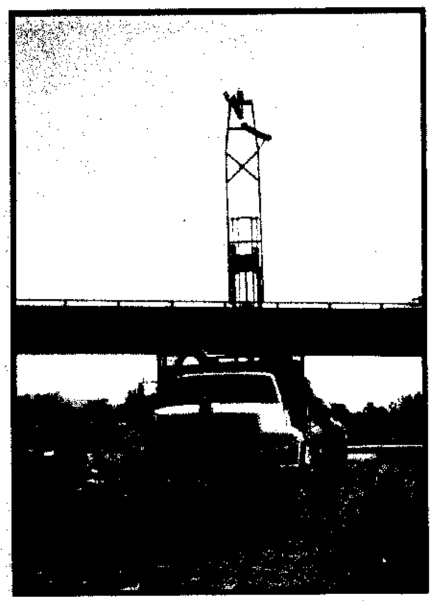
EAST DYER ROAD AND SR-55
SANTA ANA, CALIFORNIA

FIGURE

3

207384_A4.DWG

:RIALLY DEPOSITED LEAD SITE INVESTIGATION
R-55 BETWEEN THE EDINGER AVENUE ON-RAMP
AND THE EAST DYER ROAD OFF-RAMP
SANTA ANA, CALIFORNIA
TASK ORDER NO. 12-0G9601-30
EA NO. 0G9601, CONTRACT NO. 12A1139



Geotechnical

and

Environmental

Sciences

Consultants

Ninyo & Moore

EXECUTIVE SUMMARY

The State of California Department of Transportation (Department) authorized Ninyo & Moore to conduct an Aerially Deposited Lead (ADL) Site Investigation (SI) on the southbound (SB) State Route 55 (SR-55) between the Edinger Avenue on-ramp and the East Dyer Road off-ramp in the city of Santa Ana, California (site). Work was conducted in general accordance with the Department Contract No. 12A1139, Task Order No. 12-0G9601-30 (TO 30), dated December 10, 2009. It is our understanding that the Department is planning to construct an auxiliary lane in the southbound direction of SR-55 at the site.

This investigation was performed to evaluate the presence of lead in soil resulting from the combustion of leaded fuel from freeway traffic. Data collected during this investigation were used to develop recommendations for the potential reuse or disposal of soil excavated from the site and to inform the Department of potential health and safety issues concerning the presence of lead in soil for workers at the site during construction activities.

Ninyo & Moore collected 142 soil samples from forty-two borings at the site. Twenty-seven of the 142 samples contained a total lead concentration greater than or equal to 50 milligrams per kilogram (mg/kg) and less than 1,000 mg/kg and were subsequently analyzed for soluble lead using citric acid as the extractant. Ten of the results were above 5.0 milligrams per liter (mg/l) and the ten samples were subsequently analyzed for soluble lead using deionized water as the extractant and using the toxicity characteristic leaching procedure (TCLP). The results of the soluble lead analyses using deionized water as the extractant were below 1.5 mg/l and the TCLP results were below 5.0 mg/l. Fifteen samples were analyzed for pH. The pH levels ranged from 6.8 to 9.0, which would not be classified as Resource Conservation and Recovery Act (RCRA) hazardous waste and is greater than the California Environmental Protection Agency (Cal-EPA), Department of Toxic Substances Control (DTSC) lower limit of 5.0.

Our recommendations for soil reuse on site are based on the guidelines set forth by the DTSC, Lead Variance issued to the Department on June 30, 2009 (DTSC Variance). Laboratory analytical results for lead were compared to the guidelines of the DTSC Variance for potential reuse of the soil as fill within the Department right-of-way (ROW).

Our recommendations for off-site disposal were based on the comparison of lead concentrations in soil samples to the DTSC Variance thresholds, the California Health and Safety Code thresholds, and Title 40 Code of Federal Regulations (CFR) 261.24 thresholds.

Based on the analytical results and the statistical data evaluation, the on-site reuse and the off-site disposal recommendations are summarized below.

Recommendations for Soil for Reuse by the Department

Soil at the site can be reused on site with the following restrictions:

- Scenario A, soil in the surface layer (surface to 0.5 feet below ground surface [bgs]) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 1.5- to 4-foot layer (0.5 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.
- Scenario B, soil in the surface to 1.5-foot layer (surface to 1.5 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 3- to 4-foot layer (1.5 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.
- Scenario C, soil in the surface to 3-foot layer (surface to 3 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 4-foot layer (3 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.
- Scenario D, soil in the surface to 4-foot layer is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.

Recommendations for Soil to be Disposed Off Site

If the Department elects to dispose the soil off site, the following restrictions apply:

- Scenario A, soil in the surface layer (surface to 0.5 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 1.5- to 4-foot layer (0.5 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.
- Scenario B, soil in the surface to 1.5-foot layer (surface to 1.5 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 3- to 4-foot layer

(1.5 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.

- Scenario C, soil in the surface to 3-foot layer (surface to 3 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 4-foot layer (3 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.

- Scenario D, soil in the surface to 4-foot layer has no restrictions with respect to total and soluble lead concentrations.

The Department should notify the contractors performing the construction activities that hazardous concentrations of lead are present in on-site soil. Appropriate health and safety measures should be taken to minimize the potential exposure to lead.

1. INTRODUCTION

The State of California Department of Transportation (Department) authorized Ninyo & Moore to conduct an Aerially Deposited Lead (ADL) Site Investigation (SI) on the southbound (SB) State Route 55 (SR-55) between the Edinger Avenue on-ramp and the East Dyer Road off-ramp in the city of Santa Ana, California (site; Figure 1). Work was conducted in general accordance with the Department Contract No. 12A1139, Task Order No. 12-0G9601-30 (TO 30), dated December 10, 2009.

1.1. Project Description and Objective

It is our understanding that the Department is planning to construct an auxiliary lane on the SB SR-55 between the Edinger Avenue on-ramp and the East Dyer Road off-ramp. This report has been prepared by Ninyo & Moore to document the results of a study to evaluate the potential presence of ADL along the unpaved shoulder and slope in the area of the site. Forty-two borings were hand augered at the site (Figures 2 and 3).

1.2. Scope of Work

Ninyo & Moore performed the tasks described in the following sections.

1.2.1. Prefield Activities

Prefield activities included:

- Preparing a site specific health and safety plan (HSP).
- Marking boring locations at the site.
- Notifying Underground Service Alert (USA) that Ninyo & Moore would be advancing soil borings in the area (USA ticket numbers A251171 and A251181).
- Preparing a project schedule, and coordinating work with subcontractors.

1.2.2. Soil Sampling

Soil sampling was conducted from February 2 through 4, 2010. Forty-two sampling locations (B1 to B17 and B19 to B43) were used, as shown on Figures 2 and 3. Boring

B18 was not completed because it was located in asphalt and there was no alternative location in soil within a reasonable distance. The borings were advanced and sampled using a hand auger. Four soil samples were attempted for collection from depths of surface to ½ foot, 1½ to 2, 2½ to 3, and 3½ to 4 feet below ground surface (bgs) at each boring location.

1.2.3. Laboratory Analysis

Ninyo & Moore submitted the soil samples under chain of custody to Advanced Technology Laboratories (ATL) of Signal Hill, California, a laboratory certified by the State of California Department of Health Services Environmental Laboratory Accreditation Program (ELAP).

1.2.4. Global Positioning System (GPS) Surveying

Approximate latitude and longitude (North American Datum [NAD] 83) of sampling locations were recorded with a handheld global positioning system (GPS) unit (GeoXT, Trimble). The latitude and longitude data for each boring are presented on Table 1.

1.2.5. Report Preparation

This report was prepared in general accordance with Department Contract No. 12A1139 and TO 30 dated December 10, 2009.

1.3. Previous Site Investigations

Ninyo & Moore has not performed previous investigations at this site. In addition, the Department has not notified Ninyo & Moore of previous investigations performed at the site.

2. BACKGROUND

The Department obtained a variance (V09 HQSCD006) from the California Environmental Protection Agency (Cal-EPA), Department of Toxic Substances Control (DTSC), on June 30, 2009 (DTSC Variance). The DTSC Variance allows for conditional reuse of lead-impacted soil within the Department right-of-way (ROW). Background information regarding the source of ADL and the reuse or disposal of lead-impacted soil is discussed in the following sections.

2.1. Aerially Deposited Lead in Soil

Analyses for lead in soil along highways throughout the state of California have revealed that lead is commonly present along the shoulders of the highways as a result of automobile exhaust containing lead from the combustion of leaded gasoline. Elevated concentrations of lead are commonly found in the upper 2 feet of soil. Lead concentrations in soil are dependent on many variables; but in general, are a function of the age of the highway and the volume of traffic using the highway (DTSC, 2009).

2.2. Hazardous Waste Classification Criteria

Soil that exceeds the following limitations may be classified as hazardous waste with respect to lead concentrations:

- The soil contains more than 1,000 milligrams per kilogram (mg/kg) total lead, exceeding the Total Threshold Limit Concentration (TTLC) for California hazardous waste (Title 22 California Code of Regulations [CCR], Section 66261.24);
- The soil contains more than 5.0 milligrams per liter (mg/l) citric acid-extractable lead, exceeding the Soluble Threshold Limit Concentration (STLC) for California hazardous waste (Title 22 CCR, Section 66261.24);
- The soil contains more than 5.0 mg/l leachable lead using the Toxicity Characteristic Leaching Procedure (TCLP), exceeding the maximum concentration for the toxicity characteristic of the Resource, Conservation, and Recovery Act (RCRA; Title 40 Code of Federal Regulations [CFR] 261.24); or
- The soil pH is less than or equal to 2.0 or greater than or equal to 12.5, which exceeds the limits for the corrosivity characteristic of RCRA hazardous waste (40CFR 261.22) and California hazardous waste (Title 22 CCR, Section 66261.22).

2.3. DTSC Variance

In accordance with the DTSC Variance, soil that is subject to the guidelines presented below may be reused within the Department ROW. A chart presenting the different ADL soil type classifications is included in Appendix A.

2.3.1. Reuse – Condition 1

Soil containing less than 1.5 mg/l extractable lead by the Waste Extraction Test (WET) using de-ionized water as the extractant (WET-DI) and less than or equal to 1,411 mg/kg total lead (United States Environmental Protection Agency [EPA] Method 6010B) may be used as fill in the Department ROW provided the soil is placed a minimum of 5 feet above the maximum level of the water table and covered with at least 1 foot of non-hazardous soil.

2.3.2. Reuse – Condition 2

Soil containing greater than or equal to 1.5 mg/l but less than 150 mg/l extractable lead by WET-DI method, or more than 1,411 mg/kg total lead but less than 3,397 mg/kg total lead, may be used as fill in the Department ROW provided the soil is placed a minimum of 5 feet above the maximum level of the water table and protected from infiltration by a paved structure that will be maintained by the Department.

2.3.3. Reuse – Condition 3

Lead-contaminated soil with a pH less than 5.5 but greater than 5.0 shall only be used as fill material under the paved portion of the roadway. Lead-contaminated soil with a pH at or less than 5.0 shall be managed as a hazardous waste.

2.4. Criteria for Disposal of Soil Not Intended for Reuse On Site

If the Department elects to reuse soil within the Department ROW that has been excavated during construction activities, the soil may be classified either as hazardous waste or non-hazardous waste. The distinction is based on the total and soluble lead concentrations compared to the TTLC and STLC criteria. As mentioned in Section 2.2, the TTLC for total lead is 1,000 mg/kg and the STLC for citric acid extractable lead is 5.0 mg/l. Waste containing lead concentrations in excess of or equal to those listed must be disposed at a Class I hazardous waste disposal facility pursuant to State of California regulations.

3. INVESTIGATION METHODS

The investigation activities are described in the following subsections and were conducted in general accordance with TO 30 that was approved by the Department prior to beginning the field activities.

3.1. Health and Safety Plan (HSP)

A site-specific HSP dated January 28, 2010, was prepared by Ninyo & Moore and submitted to the Department for approval prior to commencing field work.

3.2. Utility Clearance

The boring locations were described to USA during the notification at least 48 hours prior to conducting the soil sampling. USA marked the member utilities known to be in the vicinity of the boring locations.

3.3. Hand-Auger Sampling

The field work was conducted on February 2 though 4, 2010. The boring locations were approved by the Department Task Order Manager and are shown on the attached Figures 2 and 3. Four samples were attempted for collection from each of the 42 boreholes at depths of 0 to ½ foot, 1½ to 2, 2½ to 3, and 3½ to 4 feet bgs unless refusal was encountered. The depths reached for each boring are presented on Table 1.

Samples were placed into new, 4-ounce, glass jars; capped with Teflon-coated plastic lids; labeled; placed in a resealable plastic bag; and stored in a cooler. The sampling equipment was decontaminated between each boring. Soil samples were transferred under chain-of-custody (COC) protocol to ATL within 24 hours of collection. In accordance with TO 30, soil sample homogenization was performed in the laboratory.

Traffic control was provided by American Barricade. Hand augering was conducted by Ninyo & Moore personnel.

3.4. Investigative-Derived Wastes

Soil cuttings generated by hand-auger drilling were returned to their corresponding bore-holes after collection of soil samples. Decontamination water was transported to Ninyo & Moore's Irvine office and placed in a drum pending chemical characterization. Based on the result of the decontamination water sample (non-detect), the decontamination water was subsequently disposed in the sanitary sewer.

3.5. Laboratory Analyses

Once the samples were received by ATL, the samples were homogenized and analyzed for the following:

- One hundred forty-two soil samples were analyzed for total lead using EPA Method 6010B;
- Twenty-seven of the soil samples contained a total lead concentration greater than or equal to 50 mg/kg and less than 1,000 mg/kg and were subsequently analyzed for soluble lead by WET using citric acid for comparison to the STLC;
- Ten of the soil samples contained a soluble lead concentration greater than or equal to 5.0 mg/l and were therefore analyzed for soluble lead by WET using de-ionized water for comparison to the STLC and soluble lead by TCLP.
- Approximately 10 percent of the soil samples (15 samples) were analyzed for pH using EPA Method 9045; and
- One sample of the decontamination water was analyzed for total lead using EPA Method 6010B.

4. ANALYTICAL RESULTS

The results of this investigation are described in the following subsections. The analytical results of lead and pH are summarized in Table 1, and the sampling locations with their corresponding data are shown on Figures 4 through 10. Laboratory reports and COC records are included in Appendix B.

4.1. Total Lead

One hundred forty-two samples were analyzed for total lead. The maximum total lead concentration was 360 mg/kg. The minimum total lead concentration was less than the laboratory practical quantitation limit of 5.0 mg/kg (Table 1).

The decontamination water sample did not contain a reportable concentration of lead.

4.2. Soluble Lead – Citric Acid

Twenty-seven of the 142 samples contained total lead at a concentration greater than or equal to 50 mg/kg and less than 1,000 mg/kg and were subsequently analyzed for soluble lead using a citric acid extraction. The maximum reported concentration was 15 mg/l. The minimum reported concentration was 1.9 mg/l.

4.3. Soluble Lead – Deionized Water

Ten of the 27 samples analyzed using the WET contained soluble lead at a concentration greater than or equal to 5.0 mg/l and were subsequently analyzed for soluble lead using deionized water extraction. The maximum reported concentration was 0.27 mg/l. The minimum reported concentration was less than the laboratory practical quantitation limit of 0.25 mg/l.

4.4. Soluble Lead – TCLP

Ten of the 27 samples analyzed using the WET contained soluble lead at a concentration greater than or equal to 5.0 mg/l or contained a total lead concentration greater than 1,000 mg/kg and were subsequently analyzed for soluble lead by the TCLP Method. The maximum reported concentration was 1.4 mg/l. The minimum reported concentration was 0.38 mg/l.

4.5. pH

Approximately 10 percent of the samples collected (15 samples) were analyzed for pH. The maximum pH level was 9.0 and the minimum pH level was 6.8. The soil pH value is not characteristic of RCRA hazardous waste and is above the lower limit of 5.0 specified in the DTSC Variance.

5. STATISTICAL EVALUATION

The following subsections describe the statistical methods used to evaluate the lead data set for the site.

5.1. Statistical Evaluation Methods

The analytical results were evaluated statistically to recommend the appropriate method of on-site reuse or off-site disposal of excavated soil. Prior to performing statistical calculations, concentrations below the laboratory reporting limit were assigned values equal to half the reporting limit. Statistical methods were applied to the data set to evaluate:

- The total lead data population distribution;
- The one-sided upper confidence limits (UCLs) of the means of the total lead concentrations; and
- If there is an acceptable correlation between total and soluble lead concentrations that would allow prediction of soluble lead concentrations based on calculated UCLs.

5.2. Population Distribution

A test for population distribution is necessary in order to apply the appropriate evaluation methods when estimating the UCLs on the total lead means. When evaluating the distribution of total lead concentrations, total lead data are treated as one data set. Distribution was evaluated in accordance with EPA SW-846, Chapter Nine (1986) by comparing the mean to the variance of the total lead data sets. If the mean is greater than the variance, the data set is normally distributed and no transformation is performed. If the mean is less than the variance, the data set is transformed using an arcsine conversion. If the mean is approximately equal to the variance, the data set is transformed using a square-root conversion. A histogram of the data is presented in Appendix D.

5.3. Upper Confidence Limits

The UCLs are used to address the uncertainty associated with estimating the true mean concentration of a population. As more data become available for a given site, the uncertainty of the estimate of a true statistical mean decreases and the UCLs move closer to the true mean of the population.

For this project, a 90 percent UCL is calculated for soil to be reused on site, while a 95 percent UCL is calculated for soil to be disposed off site. As described in Section 2.3.2, the maximum 90 percent UCL allowed for soil reuse on site is 3,397 mg/kg. A total lead concentration above 1,000 mg/kg is classified as hazardous for soil not reused on site, corresponding to a 95 percent UCL greater than or equal to 1,000 mg/kg.

One-sided 90 and 95 percent UCLs of the true mean are defined as values that, when calculated repeated for randomly drawn subsets of data, equal or exceed the true mean 90 and 95 percent of the time, respectively. The following equation (EPA, 1986) was used to calculate the UCLs:

$$\mathrm{UCL} = x + t_p \frac{S}{\mathrm{sqrt}(n)}$$

Where:

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

The samples in this study were collected using a systematic random sampling approach. SW-846 Chapter Nine indicates that statistical transformation should be used if the data set is not normally distributed and that statistical evaluations should be performed on the transformed scale. The data for this project are not normally distributed and therefore must be transformed using the arcsine function.

Transformation using the arcsine function is accomplished by calculating the arcsine of the concentration normalized to the maximum concentration in the population. That is:

$$y_i = \text{arcsine} \frac{x_i}{x_{max}}$$

Where:

y_i = transformed value sample mean
x_i = reported concentration
x_{max} = maximum concentration reported for the data set

The final result is transformed back to a concentration by multiplying the sine of the transformed number by the maximum concentration:

$$z_i = x_{max} \sin y_i$$

In order to evaluate four of the possible soil excavation depth scenarios, several different UCLs for total lead concentrations were calculated:

- **Scenario A** – surface soil (0 to ½ foot) and underlying subsurface soil (½ foot to 4 feet bgs)
- **Scenario B** – the upper 1½ feet (0 to 1½ feet) and the underlying subsurface soil (1½ to 4 feet)
- **Scenario C** – the upper 3 feet (0 to 3 feet) and the underlying subsurface soil (3 to 4 feet)
- **Scenario D** – the entire 4-foot soil column

Results of this exercise are presented in Appendix C and are shown graphically on the block diagrams presented in Appendix F.

5.4. Regression Analysis

A linear regression analysis is used to create a soluble lead prediction model for use with the 90 and 95 percent UCLs. A line fit to the data using the equation:

$y = mx + b$

Where:

y = soluble lead by WET-citric acid, mg/l
x = total lead concentration, mg/kg
b = y-intercept
m = slope

$$\text{slope} = \frac{r \times s_t}{s_s}$$

Where:

r = correlation coefficient
s_t = standard deviation of the total lead concentrations
s_s = standard deviation of the soluble lead concentrations

The linear equation from the regression is used to predict soluble lead concentrations for the statistical total lead UCLs. The integrity of the equation is directly related to 'r,' the correlation coefficient, which should be greater than or equal to 0.8.

A regression analysis was performed for this data set and the correlation coefficient was 0.8. The regression analysis is included as Appendix E.

6. CONCLUSIONS

The analyses of the data indicate that the 4-foot layers tend to have the highest concentrations of total lead, followed by the surface, 1½-, and then the 3-foot layers. Assuming the soil has not been disturbed since construction of the routes in the site vicinities, concentrations of total lead would be expected to decrease with depth.

7. RECOMMENDATIONS

Based on the findings of this study, recommendations are summarized on block diagrams in Appendix F and discussed below.

7.1. Recommendations for Soil for Reuse by the Department

Soil at the site can be reused on site with the following restrictions:

- Scenario A, soil in the surface layer (surface to 0.5 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 1.5- to 4-foot layer (0.5 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.

- Scenario B, soil in the surface to 1.5-foot layer (surface to 1.5 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 3- to 4-foot layer (1.5 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.

- Scenario C, soil in the surface to 3-foot layer (surface to 3 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations. Soil in the 4-foot layer (3 to 4 feet bgs) is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.

- Scenario D, soil in the surface to 4-foot layer is suitable for on-site reuse by the Department with no restrictions based on total and soluble lead concentrations.

7.2. Recommendations for Soil to be Disposed Off Site

If the Department elects to dispose the soil off site, the following restrictions apply:

- Scenario A, soil in the surface layer (surface to 0.5 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 1.5- to 4-foot layer (0.5 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.

- Scenario B, soil in the surface to 1.5-foot layer (surface to 1.5 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 3- to 4-foot layer (1.5 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.

- Scenario C, soil in the surface to 3-foot layer (surface to 3 feet bgs) has no restrictions with respect to total and soluble lead concentrations. Soil in the 4-foot layer (3 to 4 feet bgs) has no restrictions with respect to total and soluble lead concentrations.

- Scenario D, soil in the surface to 4-foot layer has no restrictions with respect to total and soluble lead concentrations.

The Department should notify the contractors performing the construction activities that hazardous concentrations of lead are present in on-site soil. Appropriate health and safety measures should be taken to minimize the potential exposure to lead.

8. HEALTH EFFECTS OF LEAD

Concentrations of lead in soil at the site represent a potential threat to the health of site workers performing earthwork activities.

Lead in its element form is a heavy, ductile, soft, gray metal. The permissible exposure limit for lead is 0.05 milligrams per cubic meter in air based on an eight-hour time-weighted average. The immediately dangerous to life and health exposure limit is 100 mg/m^3 as established by the National Institute of Occupational Safety and Health. Exposure may produce several symptoms including weakness, eye irritation, facial pallor, pale eyes, lassitude, insomnia, anemia, tremors, malnutrition, constipation, paralysis of the wrists and ankles, abdominal pain, colic, nephropathy, encephalopathy, gingival lead line, hypertension, anorexia, and weight loss. Target organs are the central nervous system, kidneys, eyes, blood, gingival tissue, and the gastrointestinal tract.

Because of the potential hazard from exposure to lead-contaminated soil, a lead HSP should be prepared by a Certified Industrial Hygienist (CIH). In addition, all site workers (earthwork) should have completed a training program meeting the requirements of 29 CFR/910.120 and 8 CCR 1532.1. The plan developed by the CIH should include a hazard analysis, dust control measures, air monitoring, signage, work practices, emergency response plans, personal protective equipment, decontamination, and documentation.

9. LIMITATIONS

The services outlined in this report have been conducted in a manner generally consistent with current regulatory guidelines. No warranty, expressed or implied, is made regarding the professional opinions presented in this report. Ninyo & Moore's opinions are based on an analysis of observed conditions and on information obtained from third parties. It is likely that variations in soil conditions may exist.

The samples collected and chemically analyzed and the observations made are believed to be representative of the general area evaluated; however, conditions can vary significantly between sampling locations. The interpretations and opinions contained in this report are based on the results of laboratory tests and analyses intended to detect the presence and measure the concentration of selected chemical or physical constituents in samples collected from the site. The analyses have been conducted by an independent laboratory certified by the State of California to conduct such analyses. Ninyo & Moore has no involvement in, or control over, such analyses and has no means of confirming the accuracy of laboratory results. Ninyo & Moore, therefore, disclaims any responsibility for inaccuracy in such laboratory results.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader wants any additional information, or has questions regarding content, interpretations presented, or completeness of this document. Opinions and judgments expressed herein, which are based on our understanding and interpretation of current regulatory standards, should not be construed as legal opinions.

For individuals with sensory disabilities, this document is available in alternate formats upon request. For any questions regarding this document, please call or write Wayne Chiou, Environmental Engineering, 3347 Michelson Drive, Suite 100, Irvine, California 92612-1692. Phone Number (949) 724-2221.

10. REFERENCES

Department of Toxic Substance Control (DTSC), 2009, Variance (V69HQSCD006), dated June 30.

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. ADEL MALEK– DISTRICT 12
Senior Transportation Engineer
Design Branch I/Traffic Design

Date: January 14, 2010

File: 12-ORA-55-PM 7.8/9.4
EA 12-0G9601
Overhead Sign Nos. 1,3,5,7

Attention: Bang Hua

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

Subject: Foundation Report for 55 Southbound Widening Project, Overhead Sign Nos. 1, 3, 5, and 7

1.0 INTRODUCTION

The Office of Geotechnical Design South 1 (OGDS-1), Branch B has conducted a foundation investigation pursuant to the request by your office on July 30, 2009 for a foundation investigation and recommendations for the proposed overhead signs to be supported on Cast in Drilled Hole (CIDH) pile foundations. Mr. Bang Hua of Design Branch I/Traffic Design provided the pile head loading conditions for the subject overhead signs via e-mail dated August 24, 2009.

2.0 PROJECT DESCRIPTION

The proposed overhead signs are located on the southbound 55 freeway between Edinger Avenue and Dyer Road. Three of the signposts (#1, 3, and 5) are located on the western shoulder of the roadway, while signpost #7 is located in the center median of the roadway. See Appendix I: Site Vicinity Map for a map of the project location.

3.0 FIELD INVESTIGATION AND TESTING PROGRAM

Our geotechnical investigation consisted of drilling three exploratory borings and one Cone Penetrometer Test (CPT) at the proposed sign locations. The borings were advanced utilizing the mud rotary method with a Caltrans-operated drill rigs from the Office of Drilling Services, and logged by personnel from our office. Table 1 summarizes details of the boring information.

Table No. 1 – Summary of Boring Locations

Boring	Date	Station [1]	Offset [2]	Surface Elevation[2] ft	Drilled Depth ft	Bottom Elevation ft
R-09-004	9/15/09	415+96.17	74.73 Lt	72.83	61.5	11.33
R-09-301	11/4/09	465+58.01	83.30 Lt	75.68	41.5	34.18
R-09-501	11/4/09	434+16.17	87.83 Lt	61.11	51.5	9.61
CPT-101	11/17/09	493+56.70	103.18 Lt	103.93	39.0	64.93

Note: 1. Stationing and Offsets according to 55 Center Line.
2. Elevations are Above Mean Sea Level (MSL) (1988 NAVD Datum).

Stations, offsets, and elevations of the borings were surveyed by a District 12 Surveys Crew and provided on 12-16-09.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler and a California sampler alternating at typically 5-foot intervals. The SPT samples were driven using a 140-pound hammer falling freely for 30 inches for a total penetration of 18 inches. The Modified California Sampler is a 2" diameter sampler that retrieves undisturbed push samples. At the completion of the borings, the holes were backfilled with bentonite chips.

Boring locations will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTBs are presently being prepared by the Office of Geotechnical Support and will be submitted to Design Branch I.

4.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT and undisturbed samples from the borings. Laboratory testing included unconfined compression. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 2 below). The laboratory results are shown in Appendix II: Laboratory Data.

Table No. 2 – Laboratory Test Methods

Test	Standard
Unconfined Compression of Soils	CTM 221

5.0 GEOLOGY

5.1 Regional Geology

The project is located within the Peninsular Ranges geomorphic province at the center of the Los Angeles Basin. A thick Cenozoic sedimentary section underlies the Los Angeles Basin that can be several miles thick. The Peninsular Ranges Province is characterized by northwest-southeast trending mountain ranges and valleys that are parallel to the San Andreas Fault.

5.2 Site Geology

Signposts 1 and 7 are located in clayey abutment fill, while signposts 3 and 5 are located in native alluvium. The abutment fill is approximately 20 feet deep. The underlying alluvium consists of predominantly clays and sandy clays with a layer of loose sand approximately 10-15 feet below the roadway. The alluvium is soft at the surface, but increases in density with depth.

5.3 Ground Water

At Signpost 1, ground water is located at an approximate elevation of 42 ft. above sea level as noted in ground water monitoring records for the nearby gas station. Water was measured at an elevation of 54.19 ft. at a boring drilled between Signposts 3 and 5. This places ground water at approximately 10 feet below native grade along the project limits.

5.4 Seismicity

The site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on the Caltrans ARS Online site, the controlling faults are the San Joaquin Hills Blind Thrust, the Compton–Los Alamitos Blind Thrust, and the Newport-Inglewood/Rose Canyon Fault Zone. The average shear wave velocity of the upper 30 meters (Vs30) is approximately 270 m/sec based on correlations with SPT data collected during our geotechnical investigation. The Peak Ground Acceleration (PGA) calculated for this site is 0.5g for signposts 1 through 5 and 0.4g for signpost 7. A summary of the contributing fault parameters as given by ARS Online is listed for each signpost in Tables 4 through 7. ARS curve data for each signpost are given in Appendix III: ARS Curve Data.

Table No. 4 – Signpost 1 Fault and Design Ground Motion Parameters

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	6.6	Reverse	23	SW	3.01	2.25	2.25
Newport – Inglewood	427	7.5	Strike Slip	90	N/A	7.40	7.40	7.38
USGS 5% in 50 year probabilistic model	NA	NA	NA	NA	NA	NA	NA	NA

Table No. 5 – Signpost 3 Fault and Design Ground Motion Parameters

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	6.6	Reverse	23	SW	3.46	2.75	2.75
Compton-Los Alamitos	291	6.8	Reverse	20	NE	9.61	0.64	14.36
Newport – Inglewood	427	7.5	Strike Slip	90	N/A	7.96	7.96	7.95
USGS 5% in 50 year probabilistic model	NA	NA	NA	NA	NA	NA	NA	NA

Table No. 6 – Signpost 5 Fault and Design Ground Motion Parameters

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	6.6	Reverse	23	SW	4.24	3.54	3.54
Compton-Los Alamitos	291	6.8	Reverse	20	NE	9.41	0.02	13.76
Newport – Inglewood	427	7.5	Strike Slip	90	N/A	8.90	8.90	8.88
USGS 5% in 50 year probabilistic model	NA	NA	NA	NA	NA	NA	NA	NA

Table No. 7 – Signpost 7 Fault and Design Ground Motion Parameters

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	6.6	Reverse	23	SW	5.12	4.44	4.44
Compton-Los Alamitos	291	6.8	Reverse	20	NE	9.74	0.99	14.73
Newport – Inglewood	427	7.5	Strike Slip	90	N/A	9.88	9.88	9.86
USGS 5% in 50 year probabilistic model	NA	NA	NA	NA	NA	NA	NA	NA

6.0 GEOTECHNICAL ENGINEERING ANALYSIS

6.1 Liquefaction Potential

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 and the soil borings produced for this project, there is one possibly liquefiable layer present at the signpost 3 and 5 locations. The liquefiable layer is at approximately 15 ft below roadway grade and is approximately 5 ft thick. A liquefaction analysis yields a result of approximately one inch of settlement for a seismic event with an M_{max} of 7.5. Therefore, the effects of downdrag were considered in the analyses for these two sign foundations.

6.2 Corrosion

A composite bulk soil sample from a nearby exploratory boring conducted for the widening project was tested at the District 7 Transportation Laboratory in Los Angeles for corrosion potential. The Results of the tests are not yet available at this time, however, due to the clayey characteristic of the subsurface soils encountered, it is likely that on-site soils are corrosive. The corrosion results will be provided at a later date.

7.0 FOUNDATION RECOMMENDATIONS

7.1 Axial and Lateral Pile Capacity Analysis

In performing our analysis, we assumed that the Pedestal is an integral part of the pile, and loads provided by the designer as shown in Table 8, are applied at the top of the pedestal. With the exception of Sign 7, the top of the pedestal is assumed to be the adjacent ground finish elevation. Therefore the pile lengths as shown in Table 10 include the pedestal height.

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, V5.0m by ENSOFT Inc. The depth of sign foundation was computed based on the boundary conditions shown in Table 8. Pile data is shown in Table 9. Recommended pile depths are given in Table 10. Maximum bending moments and maximum shear forces computed are presented in Table 11.

Table 8 – Unfactored Loads

Sign Post No.	Design Axial Load (Kips)	Shear Force at Pile Head (Kips)	Bending Moment at Pile Head (Kip-ft)
1	23.5	15.6	462
3	20.6	14.1	402
5	15.6	10.1	266
7	3.4	5.8	125

Table 9- Pile Data

Sign Post No.	Pile Type	Ground Surface Elevation* (ft)	Design Loading (Kips)	Nominal Resistance		Design Tip Elevation (ft)	Specified Tip Elevation (ft)
				Compression (Kips)	Tension (Kips)		
1	5.0' CIDH	72.81	23.5	47	N/A	59 (1) 38 (2) N/A (3)	38
3	5.0' CIDH	61.00	20.6	41.2	N/A	52 (1) 26 (2) 41 (3)	26
5	5.0' CIDH	76.04	15.9	31.8	N/A	66 (1) 54 (2) 44 (3)	44
7	3.0' CIDH	113.98**	3.4	6.8	N/A	110 (1) 92 (2) N/A (3)	92

- Elevations as shown on sheets SD-4 and SD-5.
- **: The boundary conditions are applied at the top of the pedestal (Elev. 116.98).

(1) Compression Load based on skin friction capacity only.
(2) Lateral Loads
(3) Liquefaction

Table 10- Recommended Pile Depths

Sign Post No.	Pile Diameter/ Pile Type	Pile Depth (Length from pile head to pile tip) (feet)
1	5.0' / CIDH	35
3	5.0' / CIDH	35
5	5.0' / CIDH	32
7	3.0' / CIDH	25

Table 11-Maximum Bending Moments (BM) and Maximum Shear Forces

Sign Post No.	Max. BM (Kip-in)	Depth of Max BM Below the Pile Head (feet)	Max. Shear (Kips)	Depth of Max Shear Below the Pile Head (feet)	Maximum Lateral Pile Head Deflection (inches)
1	6136	5.9	41.8	17.8	0.07
3	1965	12.2	14.1	0	0.04
5	3345	2.6	20.0	8.8	0.02
7	1700	4.6	18.9	12.2	0.07

8.0 CONSTRUCTION CONSIDERATIONS

The following recommendations are made for CIDH piles installation and construction and are recommended to be incorporated in the Special Provisions of the project.

- Ground water is expected to be encountered during the drilling for signposts 3 and 5. The wet method must be utilized for CIDH installation.

- Caving soils are expected to be encountered during the drilling for signposts 3 and 5.

- 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 piling shall be completed within the same day that excavation of the drilled hole has been completed.

8.0 CONCLUSIONS

The proposed CIDH foundations for the overhead signs are feasible from a Geotechnical point of view. All earthwork must be implemented in accordance to Caltrans Standard Specifications (2006) edition. The construction of the CIDH piling shall follow section 49-4 (Cast-In–Place Concrete Piles) of the 2006 Caltrans Standard Specifications.

If you have any questions or comments, please call Kristopher Barker at 213-620-2334 or Nadeem Srour at or 213-620-2377.

Prepared by: Date: 1-20-10

Reviewed by: Date:

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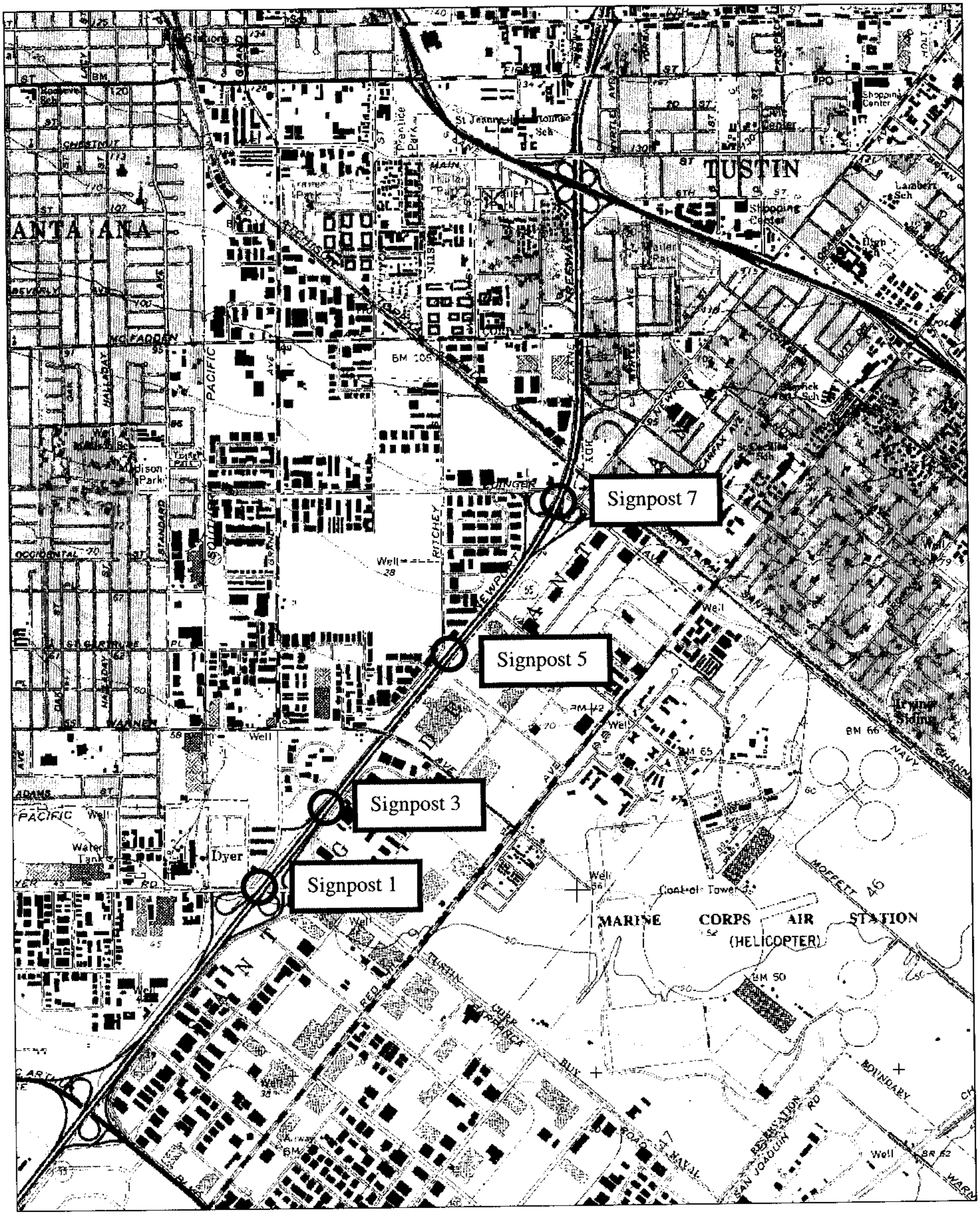
District Project Engineer – Bang H. Nguyen
GS Corporate – Mark Willian
Structure Construction R.E. Pending File
DES Office Engineer, Office of PS&E
District Materials Engineer

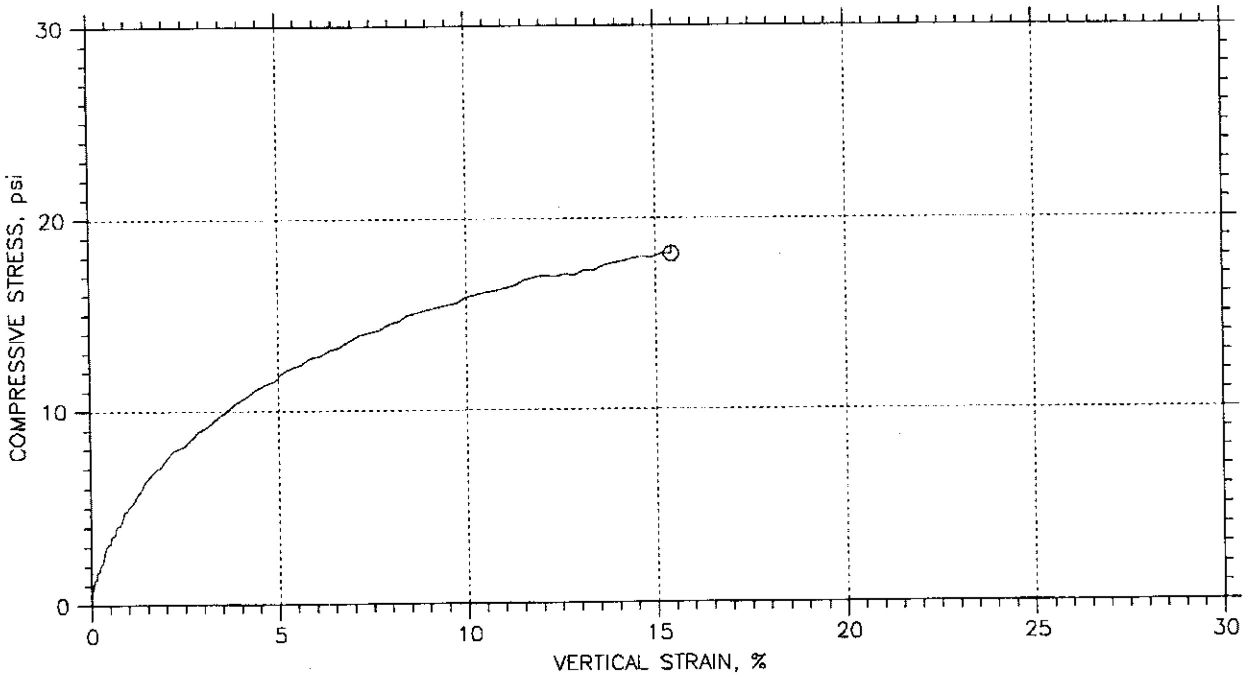
APPENDIX I: SITE VICINITY MAP



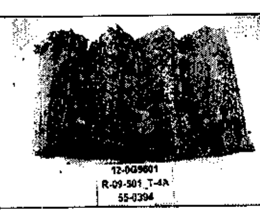
Signpost 7
Signpost 5
Signpost 3
Signpost 1
TUSTIN
SANTA ANA
MARINE CORPS AIR STATION
(HELICOPTER)
Control Tower
Dyer
Madison Park
Irvine Siding
NEWPORT
FREEWAY
PACIFIC
MCFADDEN
OCCIDENTAL
RITCHEY
WARNER
ADAMS
MOFFETT
BOUNDARY
SAN JOAQUIN RD
NAVAL

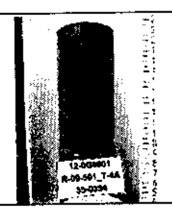
APPENDIX II: LABORATORY DATA

UNCONFINED COMPRESSION TEST REPORT



Symbol		⊙
Test No.		Q09-134
Initial	Diameter, in	1.91
	Height, in	3.87
	Water Content, %	21.89
	Dry Density, pcf	107.7
	Saturation, %	---
	Void Ratio	---
Unconfined Compressive Strength, psi		18.16
Undrained Shear Strength, psi		---
Time to Failure, min		---
Strain Rate, %/min		1
Implied Specific Gravity		---
Liquid Limit		---
Plastic Limit		---
Plasticity Index		---
Failure Sketch		





Caltrans	Project: Warner Ave OC Tieback Wal
	Location: 12-ORA-55-R8.5
	Project No.: 12-0G9601
	Boring No.: R-09-501
	Sample Type: BRASS
	Description: Moist, Very Stiff, Brown, Clay with Silt
	Remarks: ASTM D 2166. WP 12/14

Thu, 10-DEC-2009 15:13:22

APPENDIX III: ARS CURVE DATA

ARS Data for Signpost 1

Period	Acceleration	Period	Acceleration	Period	Acceleration	Period	Acceleration
0.01	0.51	0.09	0.689	0.36	0.99	1.5	0.504
0.02	0.517	0.095	0.703	0.38	0.991	1.6	0.464
0.022	0.521	0.1	0.717	0.4	0.991	1.7	0.428
0.025	0.527	0.11	0.744	0.42	0.986	1.8	0.404
0.029	0.533	0.12	0.768	0.44	0.981	1.9	0.385
0.03	0.535	0.13	0.79	0.45	0.978	2	0.368
0.032	0.54	0.133	0.796	0.46	0.975	2.2	0.334
0.035	0.547	0.14	0.809	0.48	0.97	2.4	0.304
0.036	0.55	0.15	0.827	0.5	0.964	2.5	0.291
0.04	0.558	0.16	0.846	0.55	0.944	2.6	0.279
0.042	0.563	0.17	0.864	0.6	0.927	2.8	0.258
0.044	0.568	0.18	0.881	0.65	0.911	3	0.24
0.045	0.571	0.19	0.897	0.667	0.906	3.2	0.223
0.046	0.573	0.2	0.911	0.7	0.896	3.4	0.208
0.048	0.578	0.22	0.931	0.75	0.883	3.5	0.202
0.05	0.583	0.24	0.949	0.8	0.859	3.6	0.195
0.055	0.595	0.25	0.956	0.85	0.836	3.8	0.183
0.06	0.607	0.26	0.961	0.9	0.814	4	0.173
0.065	0.619	0.28	0.971	0.95	0.794	4.2	0.166
0.067	0.625	0.29	0.975	1	0.775	4.4	0.16
0.07	0.632	0.3	0.978	1.1	0.706	4.6	0.154
0.075	0.645	0.32	0.984	1.2	0.646	4.8	0.149
0.08	0.66	0.34	0.988	1.3	0.593	5	0.144
0.085	0.674	0.35	0.989	1.4	0.546		

ARS Data for Signpost 3

Period	Acceleration	Period	Acceleration	Period	Acceleration	Period	Acceleration
0.01	0.495	0.09	0.689	0.36	0.961	1.5	0.477
0.02	0.501	0.095	0.707	0.38	0.961	1.6	0.439
0.022	0.505	0.1	0.725	0.4	0.96	1.7	0.417
0.025	0.511	0.11	0.756	0.42	0.954	1.8	0.399
0.029	0.518	0.12	0.784	0.44	0.947	1.9	0.382
0.03	0.52	0.13	0.81	0.45	0.944	2	0.367
0.032	0.525	0.133	0.817	0.46	0.941	2.2	0.332
0.035	0.532	0.14	0.832	0.48	0.935	2.4	0.303
0.036	0.534	0.15	0.852	0.5	0.928	2.5	0.291
0.04	0.546	0.16	0.868	0.55	0.907	2.6	0.279
0.042	0.552	0.17	0.881	0.6	0.889	2.8	0.258
0.044	0.558	0.18	0.893	0.65	0.872	3	0.24
0.045	0.561	0.19	0.903	0.667	0.866	3.2	0.223
0.046	0.563	0.2	0.913	0.7	0.856	3.4	0.208
0.048	0.569	0.22	0.92	0.75	0.842	3.5	0.201
0.05	0.574	0.24	0.929	0.8	0.818	3.6	0.195
0.055	0.586	0.25	0.936	0.85	0.796	3.8	0.183
0.06	0.597	0.26	0.94	0.9	0.775	4	0.173
0.065	0.608	0.28	0.949	0.95	0.755	4.2	0.166
0.067	0.612	0.29	0.951	1	0.737	4.4	0.16
0.07	0.619	0.3	0.954	1.1	0.671	4.6	0.154
0.075	0.634	0.32	0.958	1.2	0.613	4.8	0.149
0.08	0.653	0.34	0.96	1.3	0.562	5	0.144
0.085	0.671	0.35	0.961	1.4	0.517		

ARS Data for Signpost 5

Period	Acceleration	Period	Acceleration	Period	Acceleration	Period	Acceleration
0.01	0.477	0.09	0.696	0.36	0.921	1.5	0.459
0.02	0.485	0.095	0.715	0.38	0.915	1.6	0.437
0.022	0.49	0.1	0.732	0.4	0.912	1.7	0.416
0.025	0.496	0.11	0.763	0.42	0.904	1.8	0.398
0.029	0.508	0.12	0.792	0.44	0.897	1.9	0.381
0.03	0.512	0.13	0.818	0.45	0.893	2	0.366
0.032	0.519	0.133	0.825	0.46	0.889	2.2	0.332
0.035	0.53	0.14	0.84	0.48	0.881	2.4	0.303
0.036	0.533	0.15	0.86	0.5	0.873	2.5	0.29
0.04	0.546	0.16	0.876	0.55	0.851	2.6	0.279
0.042	0.551	0.17	0.89	0.6	0.831	2.8	0.258
0.044	0.557	0.18	0.902	0.65	0.813	3	0.24
0.045	0.56	0.19	0.913	0.667	0.807	3.2	0.223
0.046	0.563	0.2	0.922	0.7	0.796	3.4	0.208
0.048	0.568	0.22	0.93	0.75	0.781	3.5	0.201
0.05	0.573	0.24	0.936	0.8	0.758	3.6	0.195
0.055	0.585	0.25	0.938	0.85	0.737	3.8	0.183
0.06	0.597	0.26	0.938	0.9	0.717	4	0.173
0.065	0.607	0.28	0.94	0.95	0.698	4.2	0.166
0.067	0.614	0.29	0.939	1	0.68	4.4	0.16
0.07	0.624	0.3	0.938	1.1	0.618	4.6	0.154
0.075	0.641	0.32	0.934	1.2	0.564	4.8	0.149
0.08	0.66	0.34	0.928	1.3	0.517	5	0.144
0.085	0.678	0.35	0.925	1.4	0.485		

ARS Data for Signpost 7

Period	Acceleration	Period	Acceleration	Period	Acceleration	Period	Acceleration
0.01	0.465	0.09	0.684	0.36	0.899	1.5	0.459
0.02	0.473	0.095	0.702	0.38	0.891	1.6	0.436
0.022	0.478	0.1	0.719	0.4	0.883	1.7	0.416
0.025	0.491	0.11	0.75	0.42	0.872	1.8	0.398
0.029	0.508	0.12	0.779	0.44	0.86	1.9	0.381
0.03	0.512	0.13	0.804	0.45	0.855	2	0.366
0.032	0.519	0.133	0.811	0.46	0.85	2.2	0.332
0.035	0.529	0.14	0.827	0.48	0.839	2.4	0.303
0.036	0.533	0.15	0.847	0.5	0.828	2.5	0.29
0.04	0.545	0.16	0.862	0.55	0.802	2.6	0.279
0.042	0.551	0.17	0.875	0.6	0.78	2.8	0.258
0.044	0.557	0.18	0.887	0.65	0.759	3	0.24
0.045	0.56	0.19	0.897	0.667	0.753	3.2	0.223
0.046	0.562	0.2	0.906	0.7	0.741	3.4	0.208
0.048	0.568	0.22	0.913	0.75	0.723	3.5	0.201
0.05	0.573	0.24	0.918	0.8	0.702	3.6	0.195
0.055	0.585	0.25	0.919	0.85	0.682	3.8	0.183
0.06	0.596	0.26	0.92	0.9	0.662	4	0.173
0.065	0.607	0.28	0.92	0.95	0.644	4.2	0.166
0.067	0.611	0.29	0.919	1	0.632	4.4	0.16
0.07	0.617	0.3	0.918	1.1	0.586	4.6	0.154
0.075	0.629	0.32	0.913	1.2	0.547	4.8	0.149
0.08	0.648	0.34	0.907	1.3	0.514	5	0.144
0.085	0.666	0.35	0.903	1.4	0.485		

Memorandum

Flex your power!
Be energy efficient!

To: MR. SON NGUYEN, CHIEF-D12
DESIGN BRANCH E

Date: Jan 29, 2010

File: 12-ORA-55-PMR7.8/9.4
EA: 12-0G9601
RW415, SB 55 Auxiliary Lane

Attn: Mr. Bang Nguyen

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS 5
GEOTECHNICAL DESIGN – SOUTH 1

Subject: **Geotechnical Design Recommendations for Retaining Wall #415**

In response to your request dated March 4, 2009, the Office of Geotechnical Design South-1 provides following geotechnical design recommendations for the Type 1 retaining walls #415 to be built for the proposed auxiliary lane and ramp improvement for southbound Route 55, between Edinger Ave on-ramp and East Dyer Rd off-ramp.

This office performed subsurface exploration work for the proposed wall near the subject site by June 2009. The following recommendations are based on the review of the preliminary geotechnical report, typical cross-sections/wall layouts, review of as-built logs of test borings (LOTBs) of 1963 for the initial construction of Dyer Rd UC, and the recent subsurface explorations.

PROPOSED IMPROVEMENT

The proposed retaining wall will be located at the southbound edge of Route 55, between Dyer Road Undercrossing and Grand Avenue Off-ramp. The wall will be 552 feet long, from “A” Line Stations 415+18.51 to 420+70.51. To accommodate the proposed 12 feet highway widening, the design wall height is to be 8 feet, with concrete barrier Type 736 on top. The bottom of the wall footing will be located at approximately 8 feet above the toe of the embankment near the existing bridge.

EXISTING SITE CONDITION

Existing embankment consists of 2:1 slope (horizontal to vertical), and is moderately landscaped with trees. No slope erosion was observed. The highway pavement on top of the embankment appears to be free of distress. Fourteen feet by eight feet underground reinforced concrete box culvert runs near parallel to the wall alignment, and is estimated to be approximately 22 feet outside of the proposed wall layout line.

SUBSURFACE CONDITION

The Site is situated in the Los Angeles basin, which is underlain by a thick sequence of sediments and sedimentary rocks. The existing natural ground surface of the subject site is generally flat. The highway embankment consists mainly of sandy lean clay or clay sand on top, with very stiff silt at the bottom portion of the embankment fills. From natural grade to approximately 40 feet below, subsurface materials are mostly stiff to medium stiff lean clay interbedded with layer (or layers) of silty fine sand with the thickness ranging from 1 to 4 feet. Below the depth of 40 feet, subsurface materials are mostly dense sand with very stiff silt binder as shown in the as-built LOTBs, 1963.

GROUNDWATER

Ground water was found from Elevations 38.9 ft (borehole #A-09-01) to 33.7 ft (borehole #A-09-02) above mean sea level (MSL) based on soil borings completed in June 2009. According to borehole #B-2 that was completed in 1963 for the initial bridge construction near the proposed wall, the groundwater table appeared to be at Elev. 40.0 ft (NGVD29), which is equivalent to 42.0 ft above MSL (NAVD88) after vertical datum adjustment. The design groundwater table for the improvement will be based on higher record of the 1963.

SEISMICITY

The nearest seismic source to the project site is San Joaquin Hills Fault. This reverse/thrust type fault is located about 1.8 miles from the proposed wall, and is capable of generating maximum credible earthquake (MCE) of 7.0. The design peak bedrock acceleration (PBA) is estimated to be 0.7g based on the Sadigh et al (1997) attenuation relationships. The corresponding peak ground acceleration (PGA) at the site is estimated to be 0.62g.

LIQUEFACTION

According to subsurface explorations conducted for the proposed wall and existing bridge, the subsurface materials below the groundwater are predominantly cohesive, and appeared to be underlain by medium dense to dense silty sand from 40 ft below the natural ground. The liquefaction potential is marginal, due to the existence of medium dense sand layer, which is relatively thin in its thickness and deep in its depth from original grade.

SUBSURFACE EXPLORATIONS

Subsurface exploration has been performed at the left offset within the longitudinal limits of the proposed retaining wall. A total of three Hollow-Stem-Auger (HSA) borings were drilled early June 2009. Two of them (A-09-01 and A-09-03) were located on the highway embankment, one (A-09-02) located in the flat area between the toe of the fill slope and right-of-way fence. The boring locations are presented in Figure 1 of the Attachment.

Standard penetration tests (SPT) were conducted at the selected depths of the borings. The relatively undisturbed samples were retrieved by pushing in split spoon sampler (with brass rings) into the ground. The soil samples recovered within the brass rings were sealed with plastic caps/tapes and transported to Caltrans laboratory for testing.

Laboratory testing program consisted of moisture-density determinations (California Test Method (CTM 226)), mechanical analysis (CTM 203), direct-shear (CTM 222), Atterberg-limit (CTM204), and unconsolidated-undrained tri-axial tests.

CORROSION EVALUATION

Bulk soil samples were also obtained at selected borehole locations during the site exploration and tested for corrosion potential following the guidelines of the Corrosion Technology Branch. Based on corrosion tests, the soils at the site are non-corrosive to reinforced concrete.

Table 1. Corrosion Test Results

Sample Loaction (Borehole No.)	Depth of Sample (ft)	pH	Soluble Sulfates	Soluble Chlorides	Minimum Resistivity
A-09-03	7 - 10	7.89	N/A	N/A	1047 ohm-cm
A-09-02	20 - 40	7.31	N/A	N/A	1464 ohm-cm
Caltrans Criteria for Non-corrosive Soil and Rock		> 5.5	< 2000 PPM	< 500 PPM	> 1000 Ohm-cm

Note: The tests for sulfate and chloride are usually not conducted unless the resistivity of the sample soil is 1000 Ohm-cm or less.

GEOTECHNICAL ANALYSIS

Settlement

The maximum total settlement of the proposed wall is estimated to be in the order of 2.5 inches. Most of the settlement is immediate settlement, which will take place during construction. Long-term settlement due to primary consolidation is estimated to be less than 1 inch. The differential settlement of the wall is expected to be below the threshold value (1/500, relative settlement/wall length) suggested by FHWA for reinforced concrete cantilever wall.

Bearing Capacity

The retaining wall footing will be located within the embankment fills. The bearing capacity evaluation is based on Meyerhof (1957) method, considering modified bearing capacity factors for footing adjacent to sloping ground. The horizontal minimum clearance between the footing and the slope surface is assumed to be 4 feet (BDS 4.4.5.1).

Based on the analysis, the minimum factor of safety will be close to 5.0 for the proposed Type 1 retaining wall.

Global Stability

Slope stability analyses were conducted for wall/embankment system. The most critical wall section was selected for such analysis using Morgenstern-Price method (*SLOPE/W 2004*).

Both static and seismic conditions were considered. The horizontal pseudostatic coefficient for seismic inertia force is assumed to be 0.2g.

The Calculated factors of safety (FOS) for global stability of the retaining walls are higher than the required 1.5 for static condition, and 1.1 for seismic condition, respectively.

The results of stability analysis are presented in Figures 2 and 3 of the Attachment.

GEOTECHNICAL RECOMMENDATIONS

- Based on the results of above analyses, shallow footing can be used for the proposed Type 1 retaining wall. The wall details can be found on Plate B3-1 of *Standard Plan (May 2006).* A minimum horizontal clearance of 4 feet should be maintained between edge of retaining wall footing and slope surface.

- The foundation soils are considered to be non-corrosive to the structural elements of the proposed wall.

- Even though liquefaction potential of the underlying soil may still exist at the wall location, seismic-induced settlement and the settlement due to liquefaction of the interbedded sandy layers is relatively low. In addition, the repair of distressed retaining walls and embankment after a major seismic event is feasible, and is more cost-effective than mitigation to liquefaction potential for the roadway portion of the project.

- The existing underground reinforced concrete box culvert is located beyond the zone of influence from the proposed wall footing. The load impact to the box culvert due to the existence of the future wall is negligible.

CONSTRUCTION CONSIDERATIONS

- To reduce the compaction-induced distress on the retaining wall, the backfill compaction should be performed using hand-operated compactors or other lightweight compaction equipment within no less than 5 feet from the wall. The selection of backfill materials and the backfill placement should be in conformance with Section 19-3.06 “Structure Backfill” of *Standard Specifications*.

Should you have any question regarding the above recommendations, please contact Haitao Liu at (916) 227-0992

Prepared by:

HAITAO LIU, P.E.
Transportation Engineer - Civil
Branch A

cc: R.E. Pending File
OGDS-1, Los Angeles
OGDS-1, Sacramento
GS File Room

ATTACHMENT

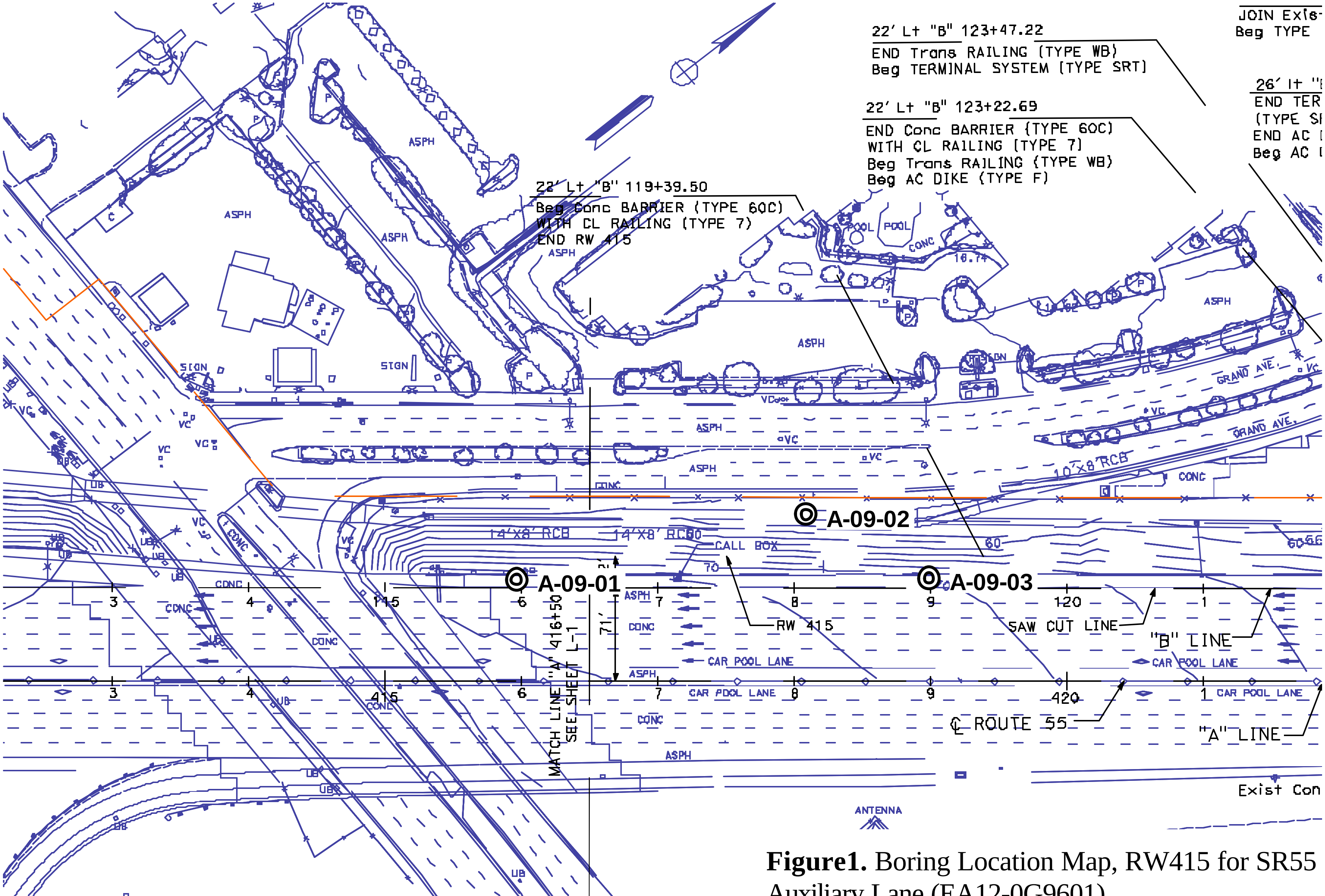


Figure1. Boring Location Map, RW415 for SR55 Auxiliary Lane (EA12-0G9601)

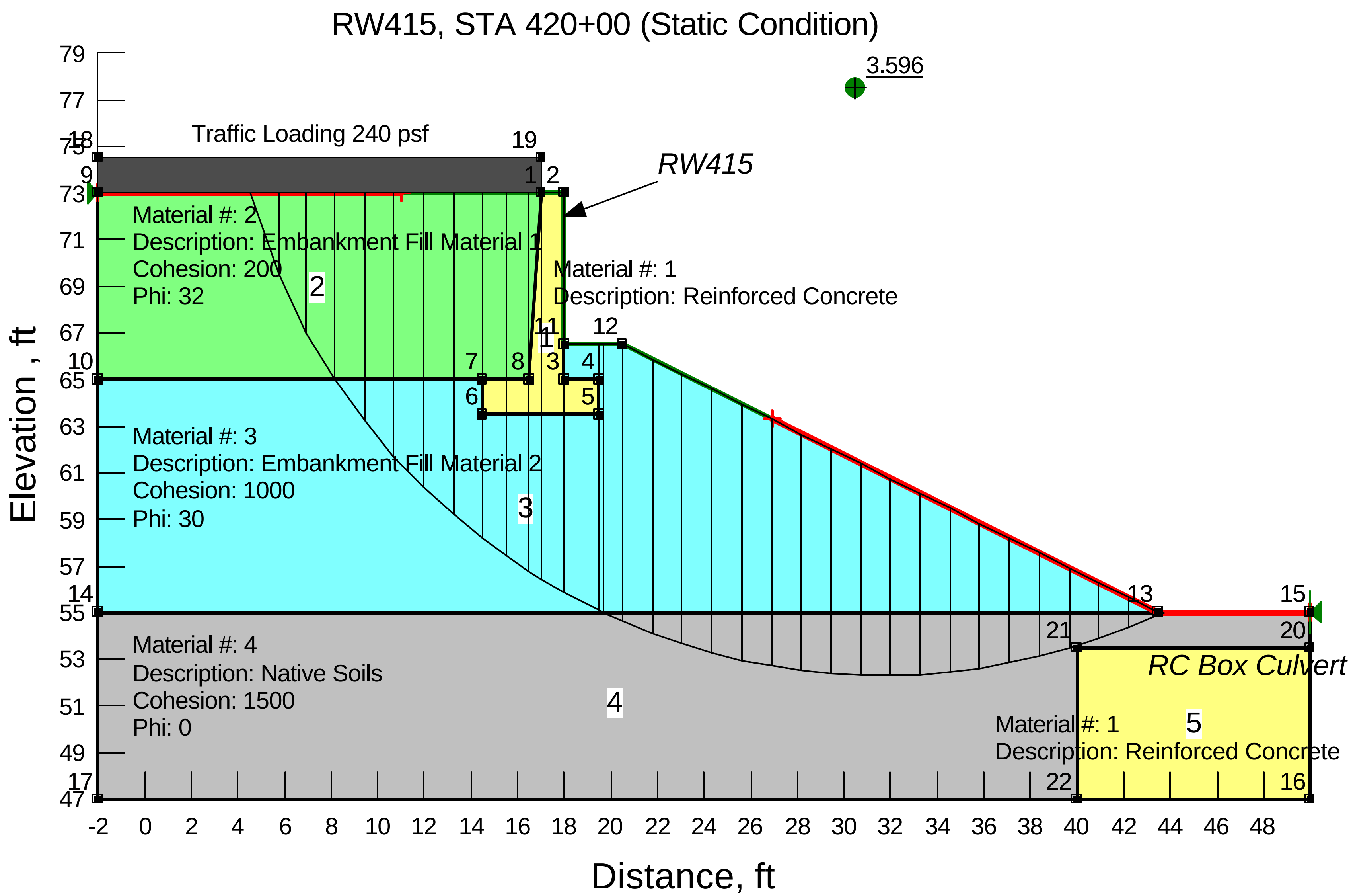


Figure 2. Global Stability for Static Case, RW415 for SB55 Auxiliary Lane (EA12-0G9601)

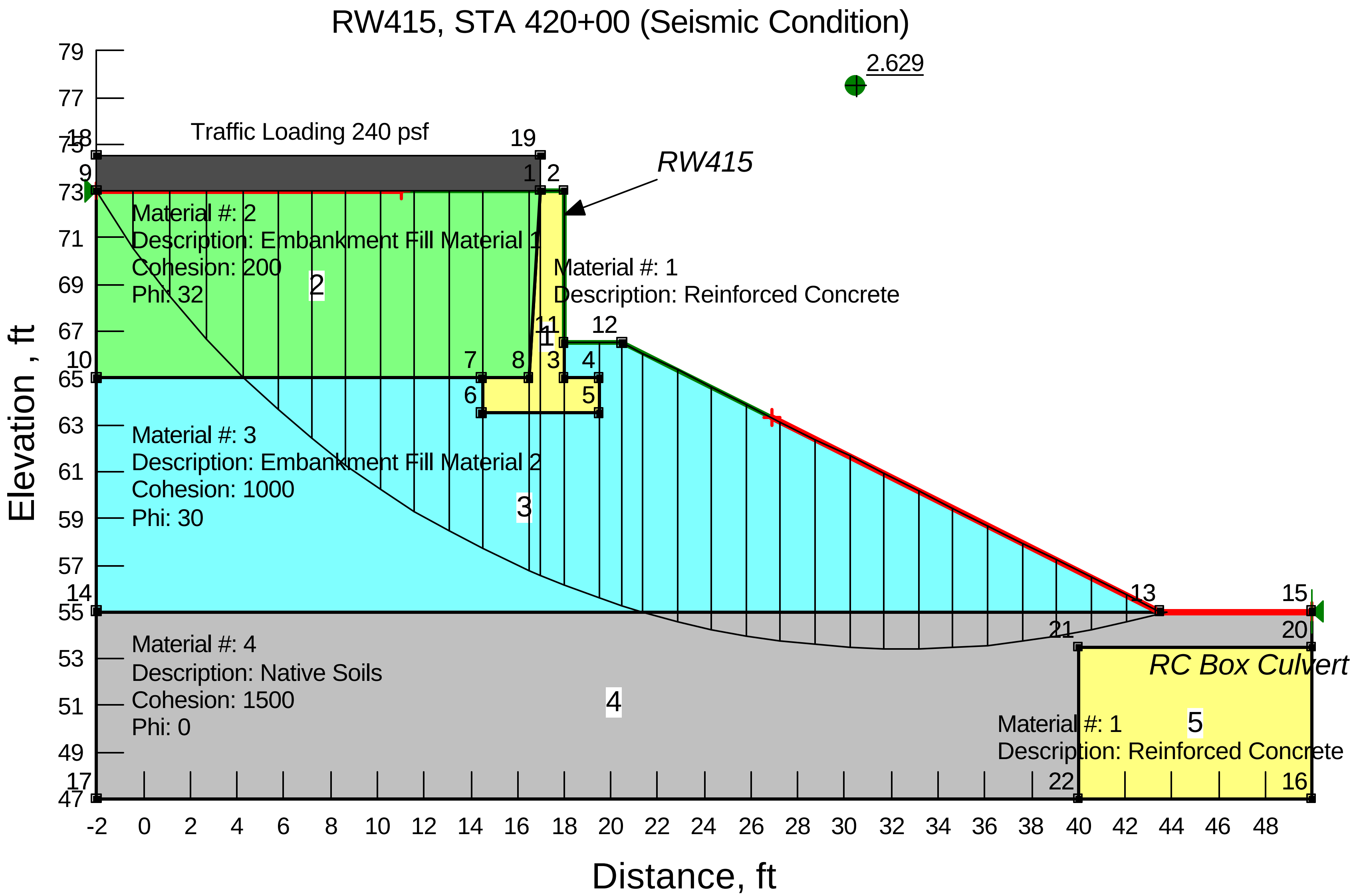


Figure 3. Global Stability for Psuedostatic Case, RW415 for SB55 Auxiliary Lane (EA12-0G9601)

M e m o r a n d u m

Flex your power!
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To: MR MOHAMMAD RAVANIPOUR
Branch Chief, Design Branch 19
Office of Bridge Design South 2

Date: July 23, 2010

File: 12-ORA-55-PM 7.87
EA: 12-0G9601
Dyer Rd. UC Widening
Br# 55-0409

Attn: **J.R. Torres**

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

Subject: Second Revised Foundation Report for Dyer Road Undercrossing widening, Bridge # 55-0409

This Second Revised Foundation Report is prepared to clarify corrosion conditions at the site. The foundation recommendations in this report are based on the latest plans provided by your office, dated April 01, 2010 as well as a Geotechnical Exploration program done for this project. This Second Revised Report supersedes our previous reports dated June 09, 2010 and February 26, 2010.

1.0 PROJECT DESCRIPTION

1.1 Location Existing Site Conditions

The existing bridge is located on State Route 55 in the City of Santa Ana, Orange County. Dyer Road UC is not just one bridge, but is made up of four bridge elements. The original structure is composed of 2 similar, but separate bridges, built 34' apart in 1965. In 1969, the abutments were expanded into the middle 34' space for additional lanes. The southbound side was widened in 1989 with a cast-in-place/prestressed box girder, and in 1999, a precast/prestressed I-Beam widening was performed on the northbound side. The existing Dyer Rd. UC is founded on piles. The following table shows pile data for the various bridge components. A Site Vicinity Map is located in Appendix I: Site Vicinity Map.

Table No. 1 – Existing Foundation Data

Bridge	Abutments 1 and 5	Pier Walls 2 and 4	Column Bent 3
RC Box (Original left and right)	Class II 45 ton	Class II 45 ton	Class II 45 ton
RC Box (center widening)	Class I 45 ton	Class I 45 ton	Class I 45 ton
PT Box (southbound widening)	Class I 45 ton	Class I 70 ton	Class I 70 ton
PC/PS (northbound widening)	Class 400 Alt X or Y (Class 90)	Class 625 Alt X or Y (Class 140)	Class 625 Alt X or Y (Class 140)

1.2 Proposed Structure

The proposed widening will be to the southbound side of the existing Dyer Road UC structure. DES proposes a 4-span precast/prestressed Box-girder bridge on short seat abutments similar to the 1999 northbound widening. All abutments and bents are to be supported on piles. According to the provided General Plans, the additional width of the proposed work is approximately 10 feet wide along the southbound edges of SR-55.

Table 2. Foundation Design Data Sheet

Support	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (kips)
Abut 1	Class 90	67 per pile
Bent 2	Class 140	114 per pile
Bent 3	Class 140	60 per pile
Bent 4	Class 140	114 per pile
Abut 5	Class 90	48 per pile

Notes:
1. *Estimate of maximum factored loads is not required for standard piles*
2. *Maximum factored loads will be estimated based on: Strength Limit State for bents and Service-I Limit State for abutments.*

2.0 FIELD INVESTIGATION AND TESTING PROGRAM

Six exploratory borings were drilled at the proposed widening location. One boring was drilled at each proposed support location, with the exception of the center bent. Two borings were drilled at the center bent, but the first was terminated at 26.5' due to encountering contaminated soil. Three borings were drilled by the Caltrans Office of Drilling Services and logged by personnel from our office. Three borings were drilled and logged by URS Corporation due to the contaminated soil conditions encountered at the site.

Borings R-09-004, R-09-006, and R-09-008 were drilled on 9/16/09, 11/3/09, and 9/15/09 respectively by Caltrans personnel. Borings R-10-001, R-10-002, and R-10-003 were drilled on 1/12/10, 1/13/10, and 1/14/10 respectively by URS. All borings were drilled using the mud rotary method. The Table below shows a summary of the boring data with elevations and locations.

Table No. 3 – Summary of Boring Locations

Boring	Station [1]	Offset [1]	Surface Elevation ft	Drilled Depth ft	Bottom Elevation ft
R-09-004	415+96.17	74.73 Lt	72.83	61.5	11.33
R-10-001	414+74.57	101.72 Lt	53.8	91.5	-37.7
R-10-002	414+5.17	107.76 Lt	53.2	91.5	-38.3
R-10-003	413+30.34	100.86 Lt	53.8	91.5	-37.7
R-09-008	412+34.38	95.52 Lt	71.29	66.5	4.79

Note: 1. Stationing and Offsets according to Route 55 Center Line.
2. Elevations are Above Mean Sea Level (MSL) (1988 NAVD Datum).

Stations, offsets, and elevations of the Caltrans borings were surveyed by a District 12 Surveys Crew and provided on 12/16/09. URS provided survey information for the borings logged by URS personnel. Elevation data for URS borings was estimated from plans and existing monitoring well borings. The URS borings will be surveyed with the other borings provided on the Log of Test Borings (LOTB) sheets which will be provided at a latter date.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler and a California sampler alternating at typically 5-foot intervals. The SPT samples were driven using a 140-pound hammer falling freely for 30 inches for a total penetration of 18 inches. The Modified California Sampler is a 2" sampler that retrieves undisturbed samples. At the completion of the borings, the holes were backfilled with bentonite chips.

3.0 LABORATORY TESTING PROGRAM

Laboratory testing was performed on selected SPT and undisturbed samples from the borings. Laboratory testing included unconfined compression and plasticity index. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 4 below). A complete summary of the geotechnical laboratory results is presented in Appendix II: Laboratory Data.

Table No. 4 – Laboratory Test Methods

Test	Standard
Unconfined Compression of Soils	CTM 221
Plasticity Index of Soils	CTM 204
Mechanical Analysis of Soils	CTM 203
Corrosion – Resistivity, pH	CTM 643
Corrosion – Chloride Content	CTM 422
Corrosion – Sulfate Content	CTM 417

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project is located within the Peninsular Ranges geomorphic province at the center of the Los Angeles Basin. A thick Cenozoic sedimentary section underlies the Los Angeles Basin that can be several miles thick. The Peninsular Ranges Province is characterized by northwest-southeast trending mountain ranges and valleys that are parallel to the San Andreas Fault.

4.2 Site Geology

The abutment fill consists of approximately 20 feet of clay and clayey sand at the northern abutment. The southern abutment is composed of sand with silt. The underlying alluvium consists of predominantly clays and sandy clays with thinner layers of silty sand and sand. The alluvium is soft at the surface, but increases in density with depth.

4.3 Ground Water

Ground water records for the gas station on the adjacent corner show continuous ground water monitoring data since 1998. Two monitoring wells are located directly adjacent to or under the existing structure. These wells show a maximum ground water elevation of about 44 feet, or 9 feet below native ground elevation.

5.0 CORROSION EVALUATION

A composite bulk sample from boring R-10-001 was tested for corrosion potential. The bulk sample is a composite of several individual specimens obtained from varying depths between 5 and 91.5 feet below the surface. The individual samples consisted of both sandy (low corrosion potential) and clayey (higher corrosion potential) soil units. Laboratory test results based on the “composite procedure” as presented herein, should serve as an indicator regarding the corrosivity of the soil. However, the results are an average of all soil units within the composite sample.

Table No. 5 – Corrosion Test Results

Boring	Depth (ft)	Minimum Resistivity (Ohm-cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)
R-10-001	5-91.5	1250	7.4	75	930

Note: Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 or less.

The Laboratory test results of the composite sample are within the range for non-corrosive. However, based on our local experience with clayey soils in Orange County coupled with the explanation presented above, <u>corrosion-resistant design practices and materials are recommended</u>, because individual units of soil are believed to be corrosive.

6.0 SEISMIC RECOMMENDATIONS

The bridge site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey; therefore, the risk of surface rupture is low. Based on the Caltrans ARS Online site, the controlling faults are the San Joaquin Hills Blind Thrust, the Newport-Inglewood/Rose Canyon Fault Zone, and the USGS 5% in 50 years probabilistic hazard. The average shear wave velocity of the upper 30 meters (Vs30) is approximately 270 m/sec based on correlations with SPT data collected during our geotechnical investigation. The Peak Ground Acceleration (PGA) calculated for this site is 0.5g. A summary of the contributing fault parameters as given by ARS Online is shown below. ARS curve data for each signpost are given in Appendix III: ARS Curve Data. The ARS curve data has been modified for near source effects per the Caltrans Seismic Design Criteria.

Table No. 6 – Fault and Design Ground Motion Parameters.

Fault	Fault ID	Type	Dip°	Dip Direction	M_{max}	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	Reverse	23	SW	6.6	2.98	2.22	2.22
Newport-Inglewood	427	Strike Slip	90	V	7.5	7.36	7.36	7.34
USGS 5%	NA	NA	NA	NA	NA	NA	NA	NA

Due to the high fines content of the native soils, liquefaction potential is considered to be low.

7.0 AS-BUILT FOUNDATION DATA

A foundation investigation was completed at the above named bridge site during September 1963. The geotechnical information was obtained from two (2) rotary wash borings extending to approximately 60 feet below ground surface (BGS) and three Cone Penetration tests extending to a maximum depth of 65 feet BGS. Since the original investigation, three (3) additional reports were issued based on the original investigation.

"Soft to stiff Silt and Clay interbeded with slightly compact to very fine Sand was encountered to elevation +15. Borings then revealed 28 feet of compact to dense very fine to coarse sand and very stiff Silt with occasional hard Calcium Carbonate concretion zones."

The original Foundation Report (FR) recommended Concrete driven piles designed for 45 Ton piles to be driven to elevations of +10 for all Foundation supports. It was also recommended that all Abutment piles be predrilled to elevation of +50, to penetrate the embankment fill. The report also recommended the preloading of the site to induce the expected 6-8 inches of settlement.

Subsequent reports stated similar parameters and recommended a tip elevation for the 70-Ton piles to be driven to an elevation of +5.

Pile-Driving data indicated a noticeable increase in end-bearing capacity between elevations +5 and +15.

8.0 FOUNDATION RECOMMENDATIONS

8.1 Pile Types and Bearing capacity

Class 90 and 140 driven Concrete piles are proposed for the Abutments and Bents consecutively. Based on the field investigation, laboratory test results, and geologic evaluation several soil units were identified to exist within the subsurface area for this project. A list of the soil parameters used in the calculation of the capacity of the proposed foundation types is summarized in Appendix. IV (Soil Parameters). The loading demand for the proposed foundations were obtained from Tables 7 and 8 (Below) provided by Structure design.

Ensoft software was used to calculate the axial (A-Pile) and lateral (L-Pile) pile tips. The provided loads were used to calculate Compression, Tension, Settlement and lateral Tip elevations, which are provided in Tables 9 to 11. In calculating the lateral pile tip elevations, shear loads were provided to our office by Structure Design for each support. The connection between the pile head and the pile cap was considered to be a pin connection; therefore no moment was applied to the pile head.

8.2 Design loads provided by Structure Design

Table 7. General Foundation Information from SD to GS

Foundation Design Data Sheet								
Support No.	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in)*	Number of Piles per Support
					B	L		
Abut 1	WSD	Class 90	66.0	61.5	7.25	10.66	1”	6
Bent 2	LRFD	Class 140	52.5	50.0	7.5	19.0	1”	8
Bent 3	LRFD	Class 140	53.0	48.0	9.0	12.0	1”	12
Bent 4	LRFD	Class 140	52.5	50.0	7.5	23.66	1”	8
Abut 5	WSD	Class 90	67.0	58.5	7.25	16.0	1”	8

Based on CALTRANS' current practice, the total permissible settlement is one inch for multi-span structures with continuous spans or multi-column bents. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Table 8. Design Loads from SD to GS

Foundation Design Loads											
	Service-I Limit State (kips)			**Strength Limit State (Controlling Group, kips)**				**Extreme Event Limit State (Controlling Group, kips)**			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
Support No.	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1	270	67	142	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	685	114	450	1021	169	0	0	648	145	0	-123
Bent 3	458	60	239	748	99	0	0	725	257	0	-95
Bent 4	685	114	450	1021	169	0	0	627	245	0	-89
Abut 5	317	48	189	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Notes:
1) Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load
2) Design tip elevations for Bents are controlled by: (a) Compression, (b) Tension, (c) Settlement (d) Lateral Load

8.3 Recommended Design Tip Elevations

Table-9 **Abutment Foundations Design Recommendations**

Support Location	Pile Type	Cut-off Elevation (ft)	LRFD Service-I Limit State Load (kips) per Support		LRFD Service-I Limit State Total Load (kips) per Pile (Compression)	Nominal Resistance (kips)	Design Tip Elevations (ft)	Specified Tip Elevation (ft)
			Total	Permanent				
Abut. 1	Class 90	61.5	270	142	67	140	(a) = 8.5 (c) = 26 (d) = 50.4	8.5
Abut. 5	Class 90	58.5	317	189	48	100	(a) =10.5 (c) = 27 (d) = 53	10.5

1) Notes: Design tip elevations are controlled by: (a) Compression, (c) Settlement, and (d) Lateral Load, respectively.

Table-10 **Bent Foundations Design Recommendations**

Support Location	Pile Type	Cut-off Elevation (ft)	Service-I Limit State Load per Support (kips)	Total Permissible Support Settlement (inches)	Required Factored Nominal Resistance (kips)				Design Tip Elevations (ft)	Specified Tip Elevation (ft)
					Strength Limit		Extreme Event			
					Comp. (φ=0.7)	Tension (φ=0.7)	Comp. (φ=1)	Tension (φ=1)		
Bent 2	Class 140	50.0	685	1	169/0.7 =250	0	145/1 =150	-123	(a-I) = 3.5 (b-I) = N/A (a-II)=14.5 (b-II)=19.5 (c) = 25.5 (d) =39.4	3.5
Bent 3	Class 140	48.0	458	1	99/0.7 =150	0	257/1 =260	-95	(a-I) =11 (b-I) = N/A (a-II) = 2.5 (b-II)= 14 (c) = 23 (d) =36.5	2.5
Bent 4	Class 140	50.0	685	1	169/0.7 =250	0	245/1 =250	-89	(a-I) = 5.5 (b-I) = N/A (a-II) = 5.5 (b-II) =25.5 (c) = 27 (d)=40.6	5.5

Notes: Design tip elevations are controlled by: (a-I) Compression (Strength Limit), (b-I) Tension (Strength Limit), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement, and (d) Lateral Load, respectively.

Table-11 **Pile Data Table**

Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)
		Compression	Tension		
Abut. 1	Class 90	140	N/a	(a) = 8.5 (c) = 26 (d) = 50.4	8.5
Bent 2	Class 140	250	-123	(a) = 3.5 (b) = 19.5 (c) = 25.5 (d) =39.4	3.5
Bent 3	Class 140	260	-95	(a) = 2.5 (b) = 14 (c) = 23 (d) = 36.5	2.5
Bent 4	Class 140	250	-89	(a) =5.5 (b) = 25.5 (c) = 27 (d) = 40.6	5.5
Abut. 5	Class 90	100	N/a	(a) = 10.5 (c) =27 (d) = 53	10.5

Notes:

1) Design tip elevations for Abutments are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load
2) Design tip elevations for Bents are controlled by: (a) Compression, (b) Tension, (c) Settlement (d) Lateral Load

8.3 Special Considerations

- Based on the provided general plan, no proposed embankment fill is proposed for the widening. However should any additional fill greater than five (5) in height be proposed, it is recommended that the embankments should be placed prior to the bridge widening to allow for the settlement to occur. Previous settlement calculations indicate that 6-8 inches of settlement were calculated as a result of the approach embankments. The estimated settlement period is less than 90 days.

- Abutment piles driven through fill, should be placed in predrilled holes to an elevation of +50, in accordance to Section 49-1.06 of the Standard Specifications.

- In order to reduce the potential impact on a near by utility line, it is proposed that Bent-3 piles be place in pre-drilled holes. The pre-drilled holes may be drilled with an auger with a diameter not exceeding the maximum dimension of the proposed pile (15 inches). Drilling may be advanced to an elevation of 31 feet or higher (not to exceed 17 feet below the cut off elevation). Pile driving should continue to the design tip elevation, or to achieve the Nominal Resistance Compression, as summarized in Table 11. Should a gap occur between the pile and the pre-drilled annuls during pile driving, this gap should be backfilled with fine silica sand (#20 to #30 sieve such as Ottawa Sand) or cement grout. The backfill is preferably done during pile driving to help fill all the voids.

- According to previous driving records in 1963, the average driving tip elevations varied between Elev. 9.35 and 10.95. However and should the contractor encounter difficulty reaching the design tip elevations, the piles could be cut, if the minimum demand for tension and lateral tips are met. The maximum allowable length of pile that could be cut is 10 feet; in this case Geotechnical Engineer should be contacted to evaluate the encountered case. Limited pre-drilling could be allowed help reaching the design tip elevations.

- Pile bearing will be assessed by the Gates formula (Nominal Resistance Ru) as specified in the Standard Specifications in Section 49-1.08. Piles achieving 150 % of Ru bearing under the hammer within 4 feet of the specified pile tip elevations, may be accepted at the Resident Engineer's discretion. This procedure should prevent damage to the piles. Driving tips may be necessary to insure pile integrity during hard driving conditions. Pile Heads must be protected from direct impact of the hammer by a cushion-driving block.

- If the Ru bearing is not achieved at the specified tip elevation, the contractor should allow the piles to set for a minimum period of 24 hours, then retap for bearing verification.

9.0 NOTES TO DESIGNER

It is recommended that our office be notified when pile driving begins to witness the initial work progress. Any problems with pile driving or achieving capacity or design tip elevations should be reported to our office for evaluation.

Project Plans and Specifications should be submitted to our office for review.

10.0 CONSTRUCTION CONSIDERATIONS

- All structural work associated with pile installation shall be implemented in accordance to the recommendations outlined in Section 49 in the Caltrans Standard Specifications.

- All earthwork shall be implemented in accordance to the recommendations outlined in Section 19 of the Caltrans Standard Specifications.

- Quality control must be practiced during pile installation to insure compliance with Caltrans Construction procedures.

- Contractor must become familiarized with the site conditions. Care must be exercised during pile driving to avoid damage to the close-by existing piles.

- Should any excavation occur below El. 40 MSL, contaminated soil conditions should be anticipated.

- Noise and Vibration from the pile driving operation should be studied prior to construction, due to the close proximity of adjacent structures lying outside the State Right of Way.

If you have any questions, please contact Kristopher Barker at (213) 620-2334 or Sam Sukiasian at (213) 620-2135.

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District Project Manager
GS Corporate
Structure Construction R.E. Pending File
DES Office Engineer, Office of PS&E
District Materials Engineer
(District Hydraulics/Structure Hydraulics)

Appendix I: Site Vicinity Map

3000 ft Scale: 1 : 100,000 Detail: 11-0 Datum: WGS84

Appendix II: Laboratory Data

Santa Ana Geotechnical Laboratory Testing Summary

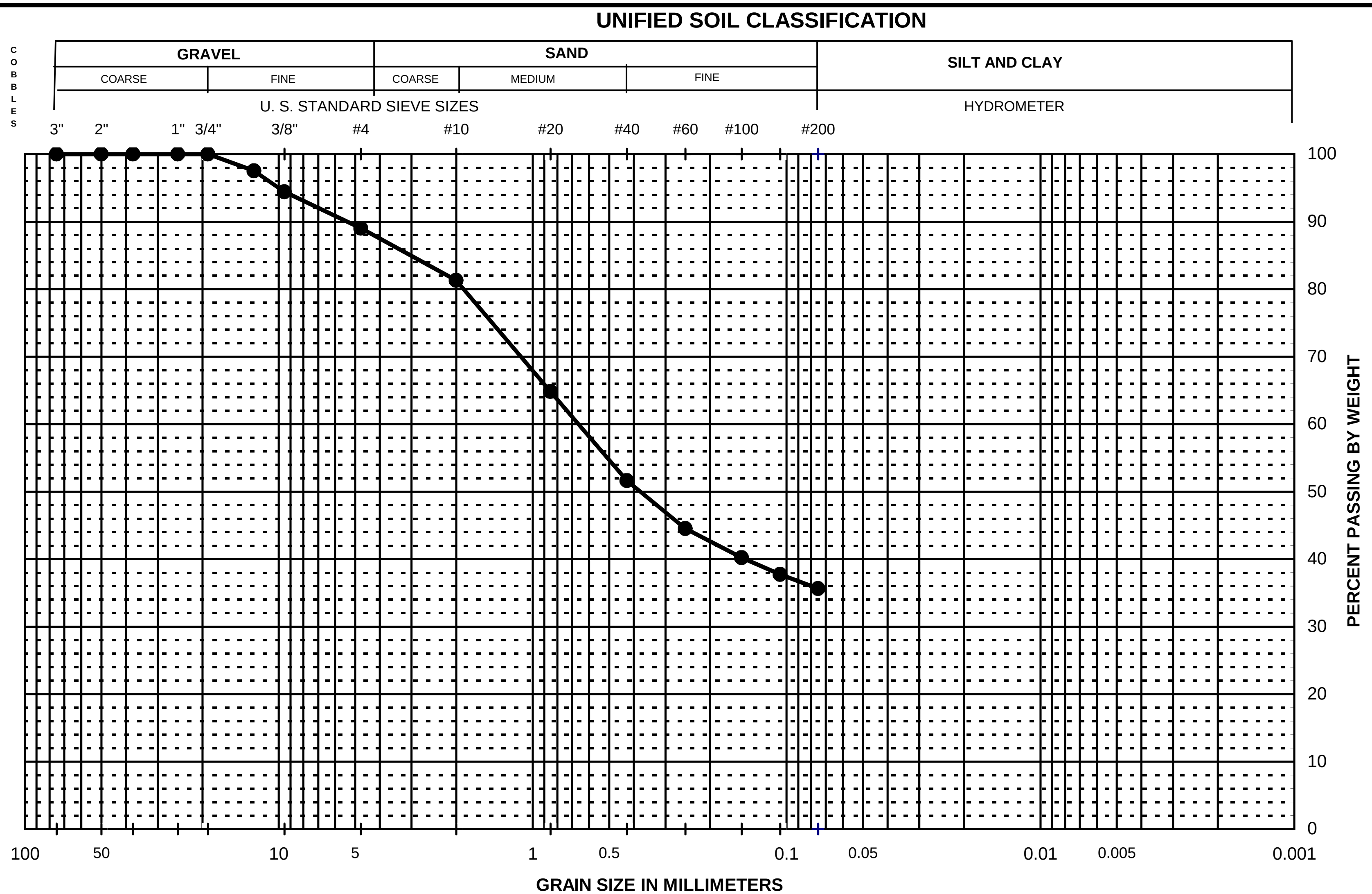
Project Name: SR-55 Dyer Road
Project Number: 30989831, EA 12-0G9601
Project Engineer: FM

LOCATION				INITIAL CONDITION			INDEX										
							Limits			Gradation			UC	CORROSIVITY			
Exploration Number	Sample/ Specimen Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit (%)	Plasticity Index (%)	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)	Unconfined Compression peak (ksf)	pH	Minimum Resistivity (ohm - cm)	Sulfate Content (ppm)	Chloride Content (ppm)
R-10-001	1	5.0															
R-10-001	2	10.0															
R-10-001	3	15.0	CL	24.4	127.3	102.3	39	21	0.30			78.6	2.45				
R-10-001	4	20.0															
R-10-001	5	30.0	SC									37.5					
R-10-001	7	40.0															
R-10-001	8	45.0															
R-10-001	9	50.0	SC	22.9	128.8	104.8						43.7	0.43	7.4	1250	930	75
R-10-001	10	55.0															
R-10-001	11	60.0															
R-10-001	12	65.0	CL	30.9			37	16	0.62								
R-10-001	13	70.0															
R-10-001	14	75.0															
R-10-001	15	80.0															
R-10-001	16	90.0															
R-10-002	1	7.0	SC	15.3			24	8	-0.09								
R-10-002	2	10.0	SC									13.8					
R-10-002	3	15.0	CL	26.5			37	20	0.48								
R-10-002	6	30.0	SC	15.8	138.4	119.5						44.7	0.77				
R-10-002	8	40.0	SC							11.0	53.4	35.6					
R-10-003	4	25.0	SC	17.4	135.7	115.6						35.6	1.93				
R-10-003	7	40.0	SC									19.2					

Santa Ana Geotechnical Laboratory Testing Summary

Project Name: SR-55 Dyer Road
Project Number: 30989831, EA 12-0G9601
Project Engineer: FM

LOCATION				INITIAL CONDITION			INDEX										
							Limits			Gradation			UC	CORROSIVITY			
Exploration Number	Sample/ Specimen Number	Depth (ft)	USCS Symbol	Water Content (%)	Total Unit Weight (pcf)	Dry Unit Weight (pcf)	Liquid Limit (%)	Plasticity Index (%)	Liquidity Index	Gravel (%)	Sand (%)	Fines (%)	Unconfined Compression peak (ksf)	pH	Minimum Resistivity (ohm - cm)	Sulfate Content (ppm)	Chloride Content (ppm)
R-10-003	13	70.0	CL	21.1	130.4	107.7						69.4	3.09				



Sieve No.	Dia. mm	% Finer
3"	75.0	100.0
2"	50.0	100.0
1.5"	37.5	100.0
1"	25.0	100.0
3/4"	19.00	100.0
1/2"	12.50	97.5
3/8"	9.50	94.4
#4	4.75	89.0
#10	2.00	81.3
#20	0.850	64.8
#40	0.425	51.6
#60	0.250	44.5
#100	0.150	40.2
#140	0.106	37.7
#200	0.075	35.6
Hydrometer Analysis		

% Cobbles	
% Gravel	11.0
% Sand	53.4
% Fines	35.6
D_{60}	
D_{30}	
D_{10}	
C_u	
C_c	

Exploration	Sample No.	Depth (ft)	SYMBOL	Wn (%)	LL	PI	% 5 mm	Description and Classification
R-10-002	8	40.0	I					**Yellowish brown clayey Sand (SC)**

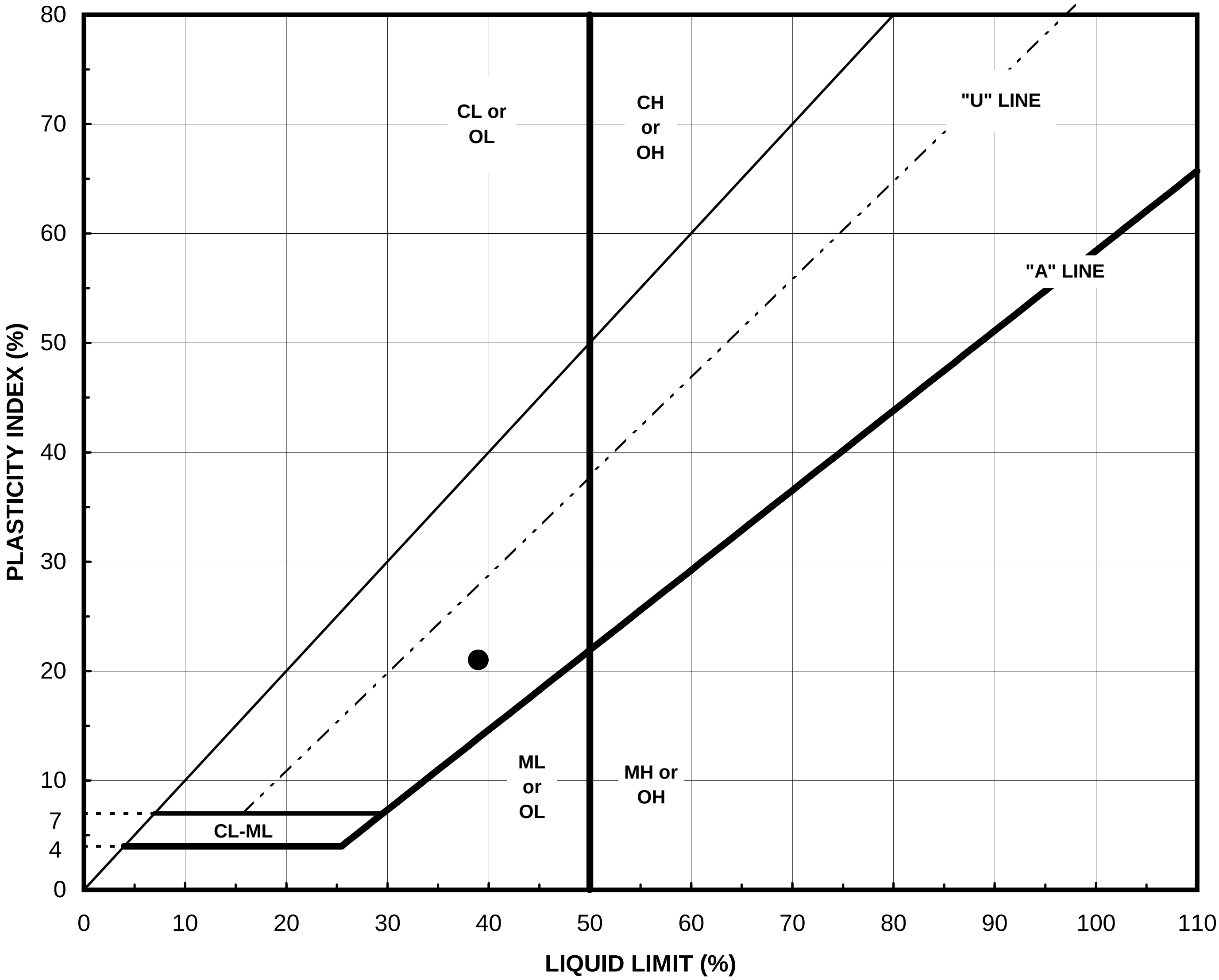
PROJECT NAME: **SR-55 Dyer Road**

PROJECT NUMBER: **30989831**

PARTICLE-SIZE DISTRIBUTION CURVES

Figure:

URS



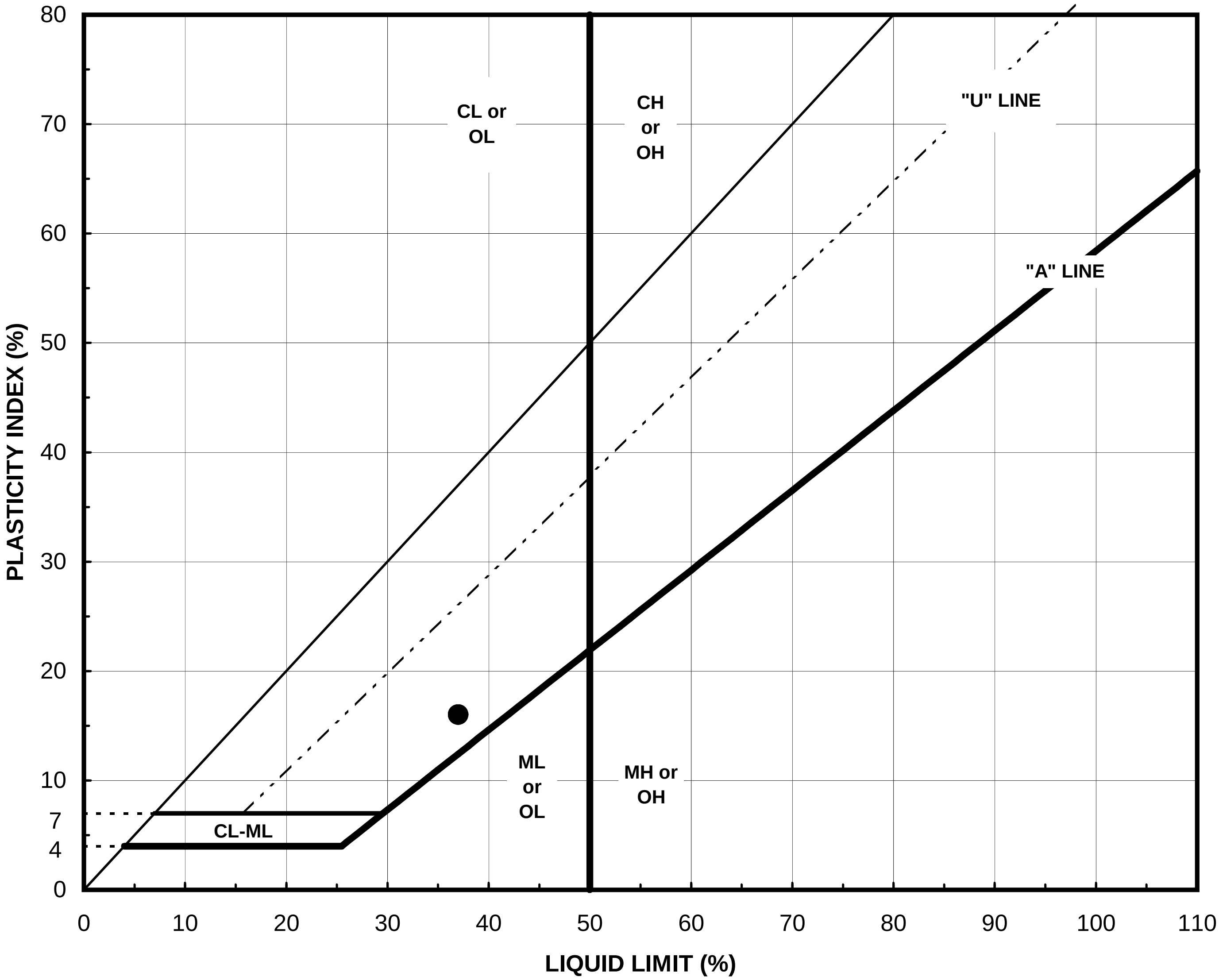
BORING / SAMPLE	DEPTH (ft.)	TEST SYMBOL	WATER CONTENT (%)	LL (%)	PI (%)	DESCRIPTION / CLASSIFICATION
R-10-001	15.0	●	24.4	39	21	Olive gray Clay (CL)
		■				
		◆				
		o				
		□				
		◇				

Project Name: **SR-55 Dyer Road**

Project Number: **30989831**

PLASTICITY CHART

Figure



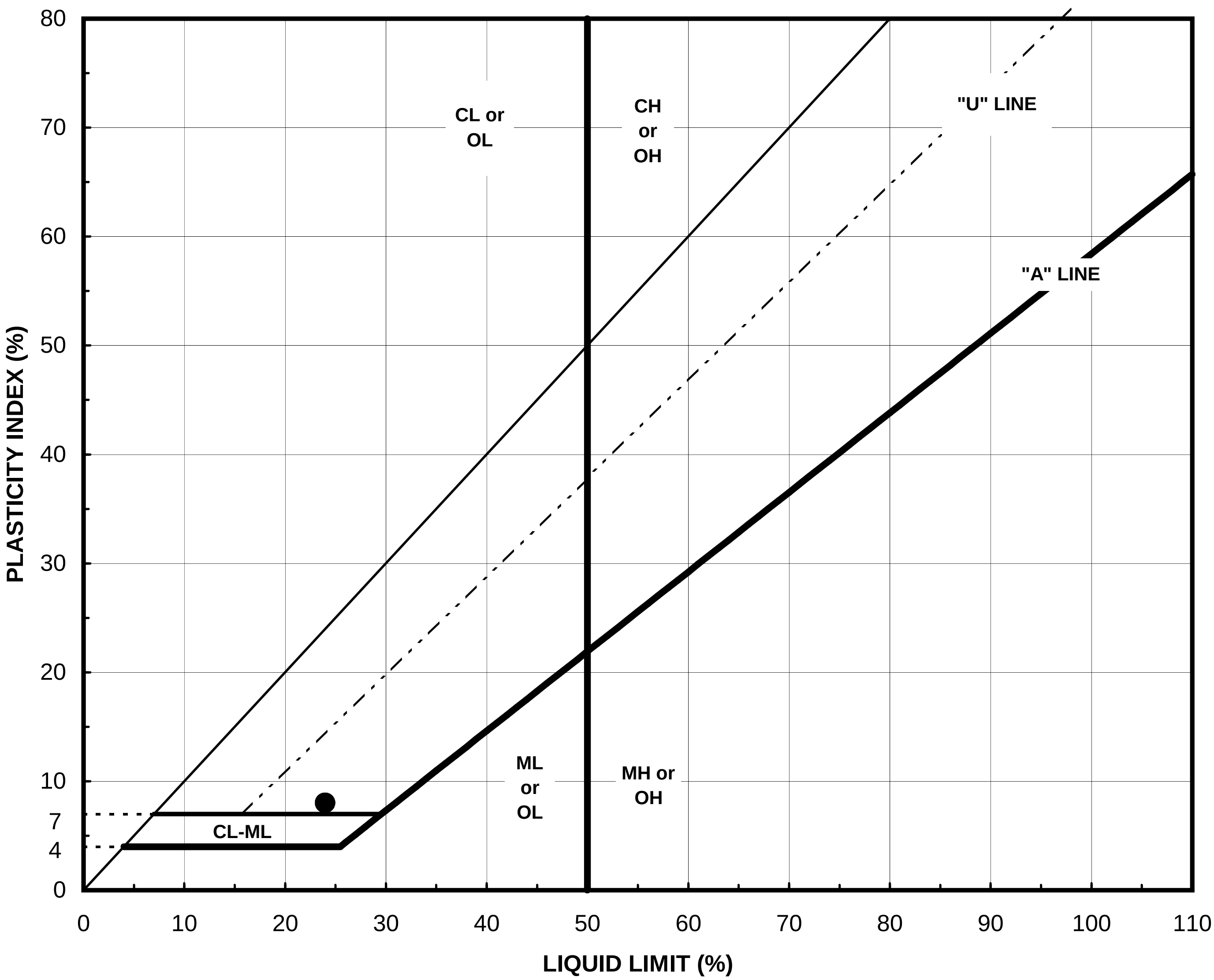
BORING / SAMPLE	DEPTH (ft.)	TEST SYMBOL	WATER CONTENT (%)	LL (%)	PI (%)	DESCRIPTION / CLASSIFICATION
R-10-001	65.0	●	30.9	37	16	Olive brown Clay (CL)
		■				
		◆				
		o				
		□				
		◇				

Project Name: **SR-55 Dyer Road**

Project Number: **30989831**

PLASTICITY CHART

Figure



BORING / SAMPLE	DEPTH (ft.)	TEST SYMBOL	WATER CONTENT (%)	LL (%)	PI (%)	DESCRIPTION / CLASSIFICATION
R-10-002	7.0	●	15.3	24	8	Yellowish brown clayey Sand (SC)
		■				
		◆				
		o				
		□				
		◇				

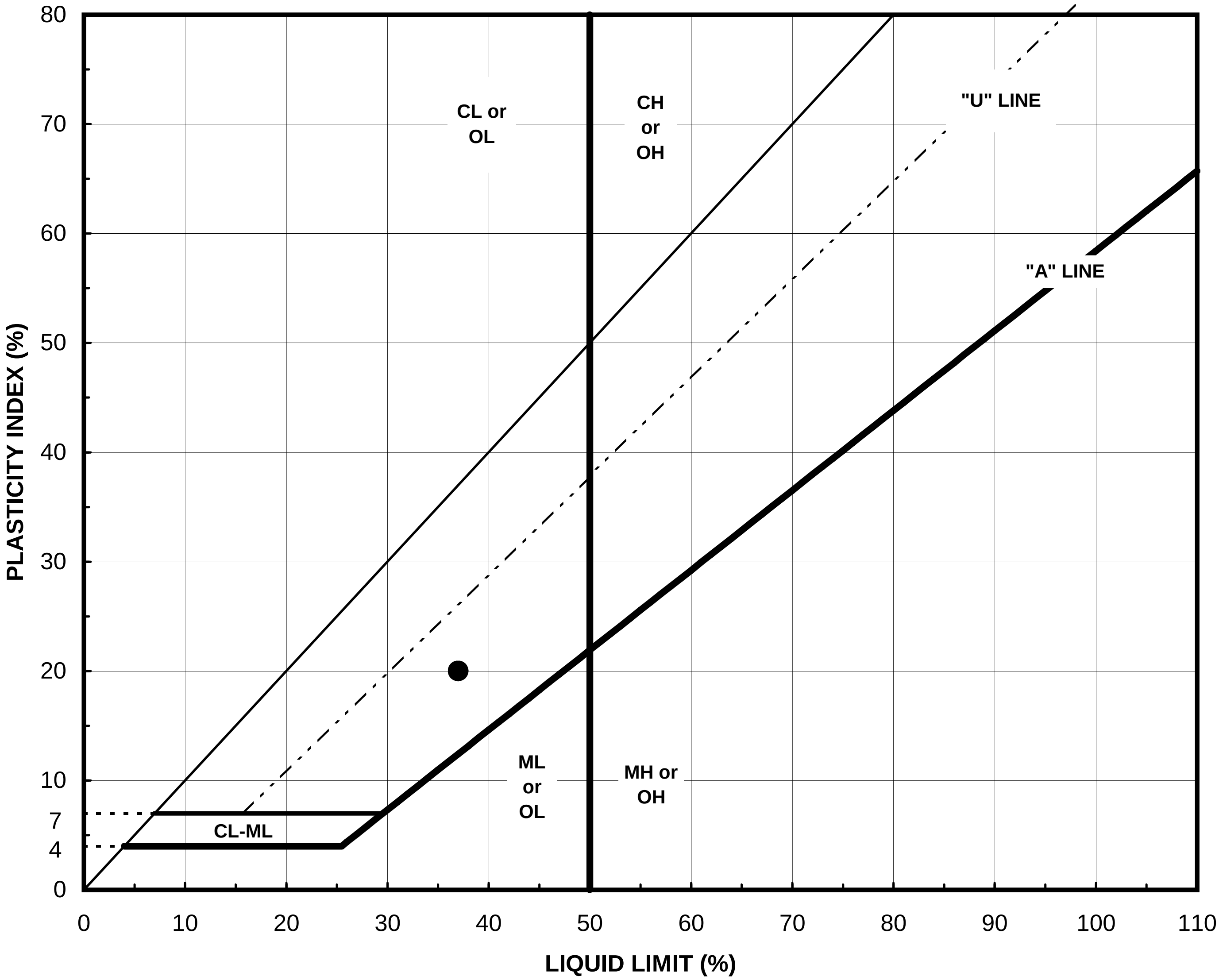
Project Name: **SR-55 Dyer Road**

Project Number: **30989831**

PLASTICITY CHART

Figure

URS



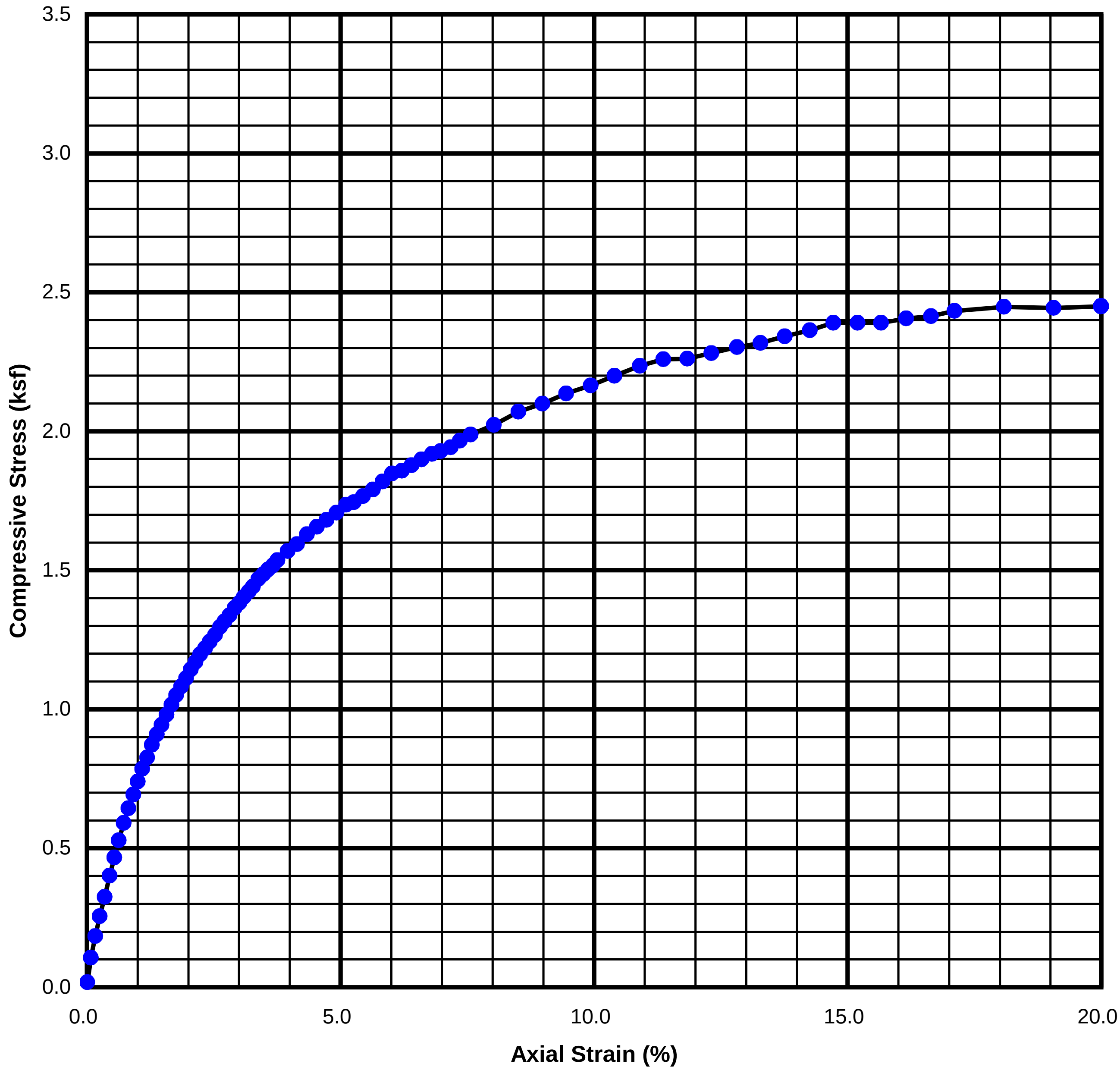
BORING / SAMPLE	DEPTH (ft.)	TEST SYMBOL	WATER CONTENT (%)	LL (%)	PI (%)	DESCRIPTION / CLASSIFICATION
R-10-002	15.0	●	26.5	37	20	Grayish brown Clay (CL)
		■				
		◆				
		○				
		□				
		◇				

Project Name: **SR-55 Dyer Road**

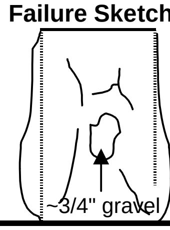
Project Number: **30989831**

PLASTICITY CHART

Figure



Failure Sketch



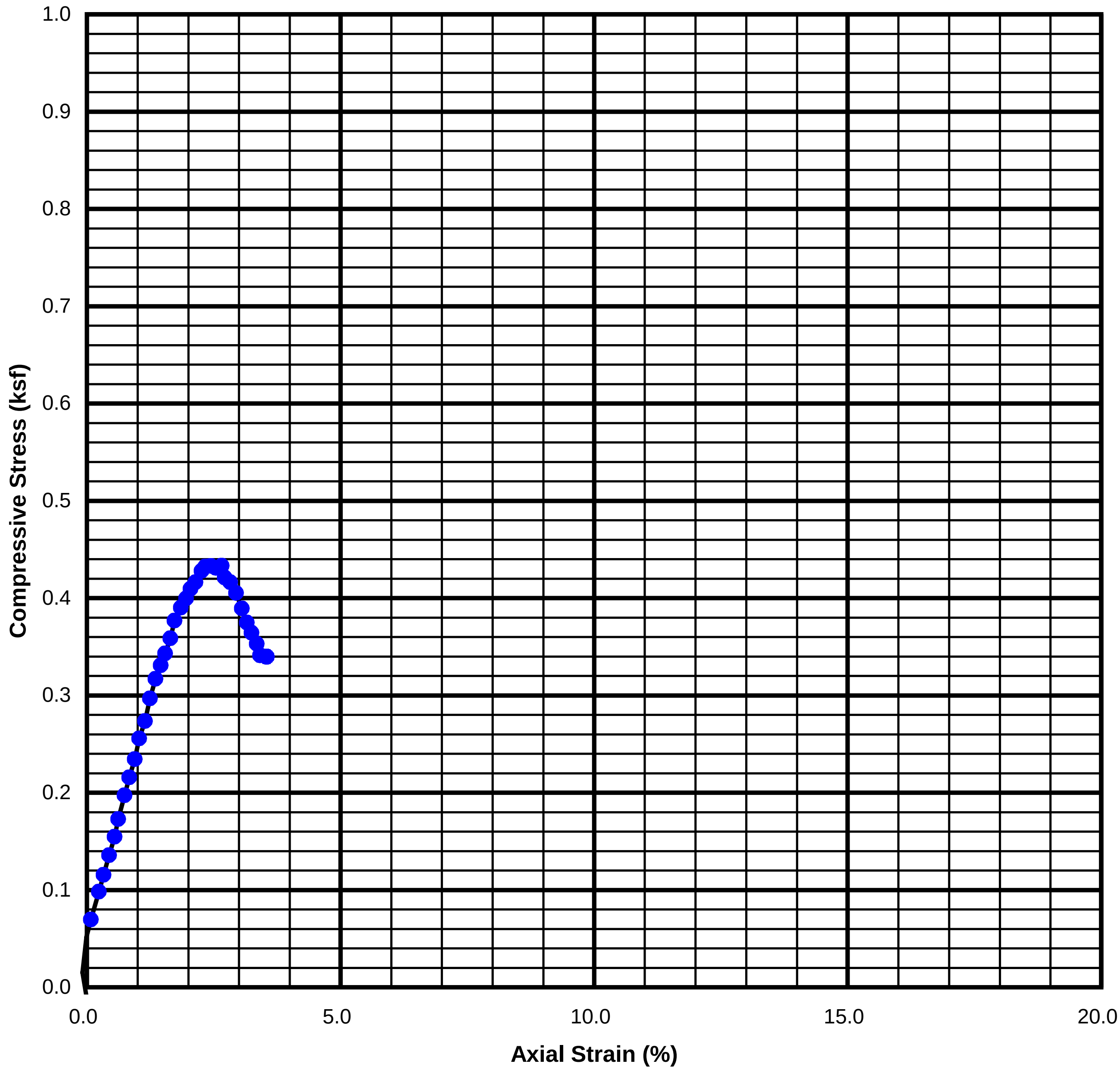
Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Density (ksf)	Degree of Saturation (%)	Peak Stress (ksf)
24.4	39	21	5.853	2.407	127.3	99.5	2.45

Project Name:	**SR-55 Dyer Road**				
Project Number:	**30989831**				
Exploration No:	**R-10-001**	**Sample No.:**	**3**	**Depth (ft):**	**15**
Description and/or Classification:	**Olive gray Clay (CL)**				

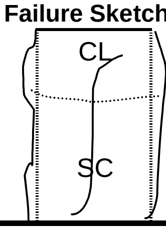
UNCONFINED COMPRESSION TEST

ASTM D 2166

Figure :

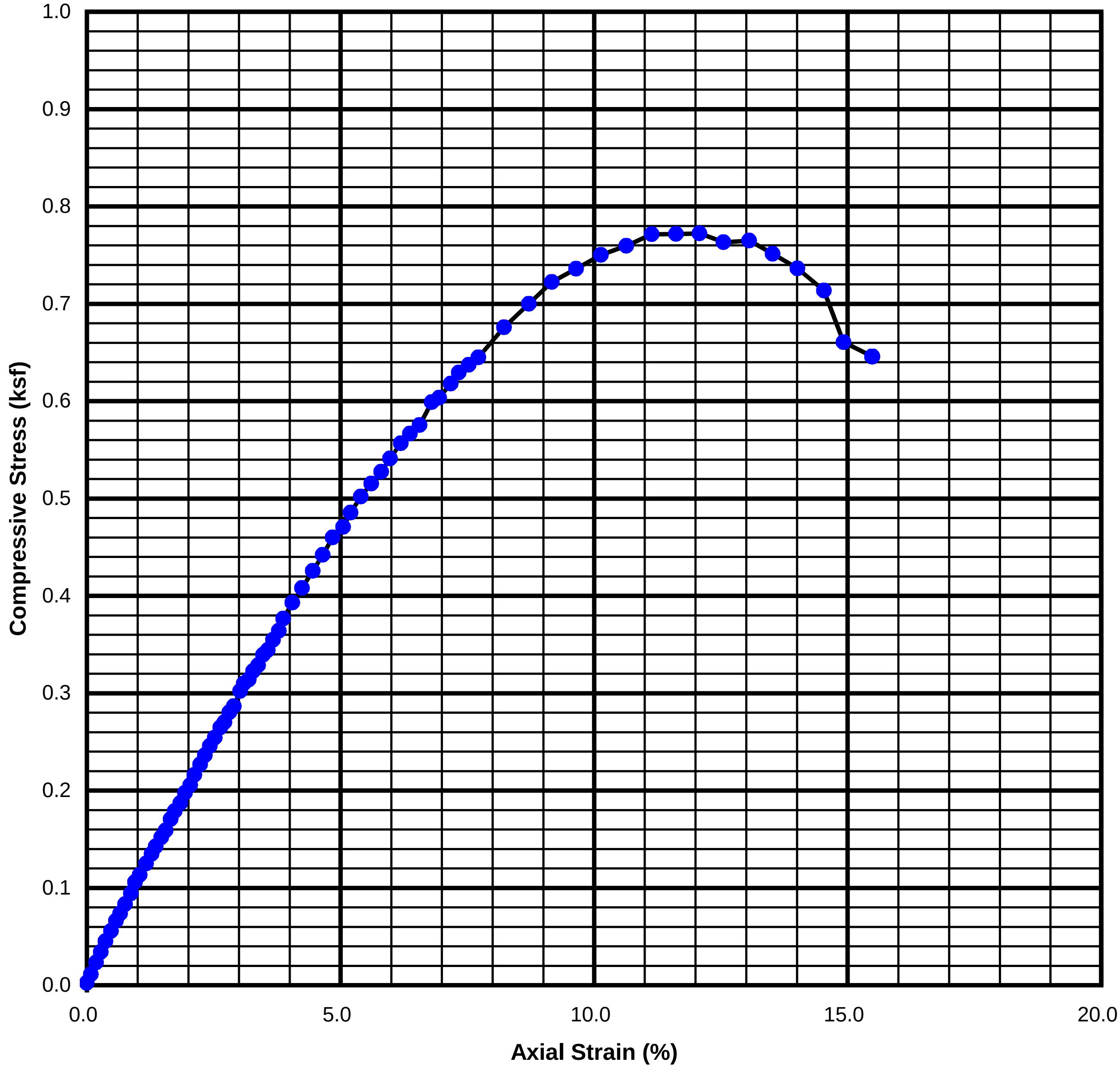


Failure Sketch

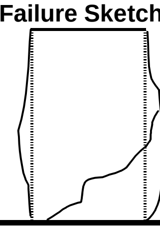


Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Density (ksf)	Degree of Saturation (%)	Peak Stress (ksf)
22.9			5.557	2.407	128.8	99.2	0.43

Project Name:	**SR-55 Dyer Road**	**UNCONFINED COMPRESSION TEST**
Project Number:	**30989831**	
Exploration No:	**R-10-001 Sample No.: 9 Depth (ft): 50**	**ASTM D 2166**
Description and/or Classification:	**Yellowish brown clayey Sand (SC)**	**Figure :**

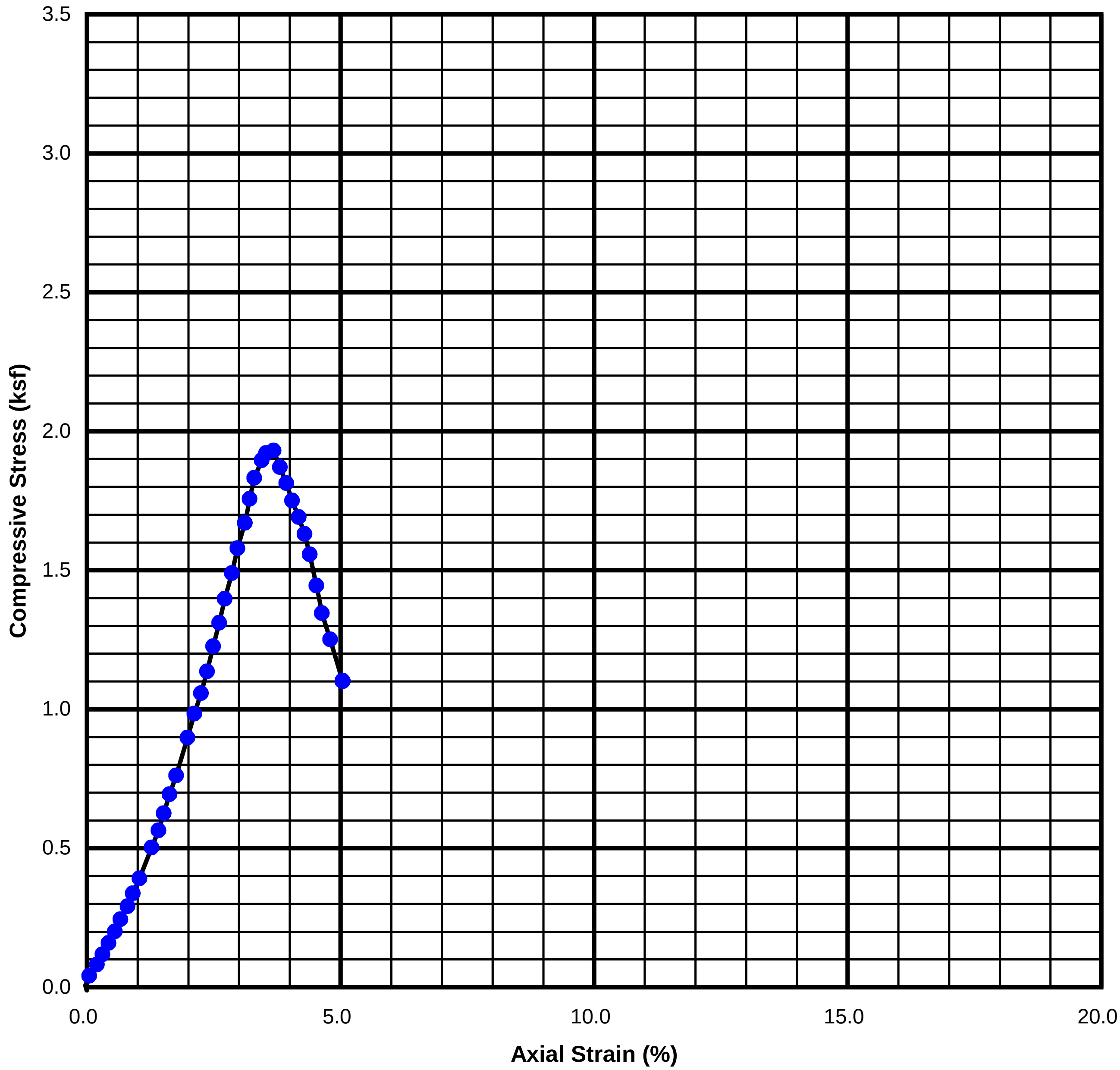


Failure Sketch

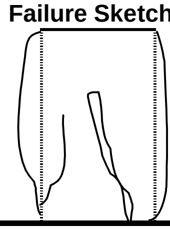


Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Density (ksf)	Degree of Saturation (%)	Peak Stress (ksf)
15.8			5.796	2.446	138.4	100.1	0.77

Project Name: SR-55 Dyer Road	**UNCONFINED COMPRESSION TEST**
Project Number: 30989831	
Exploration No: R-10-002 **Sample No.:** 6 **Depth (ft):** 30	**ASTM D 2166**
Description and/or Classification: Yellowish brown clayey Sand (SC)	**Figure :**



Failure Sketch



Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Density (ksf)	Degree of Saturation (%)	Peak Stress (ksf)
17.4			4.595	2.413	135.7	99.1	1.93

Project Name: SR-55 Dyer Road

Project Number: 30989831

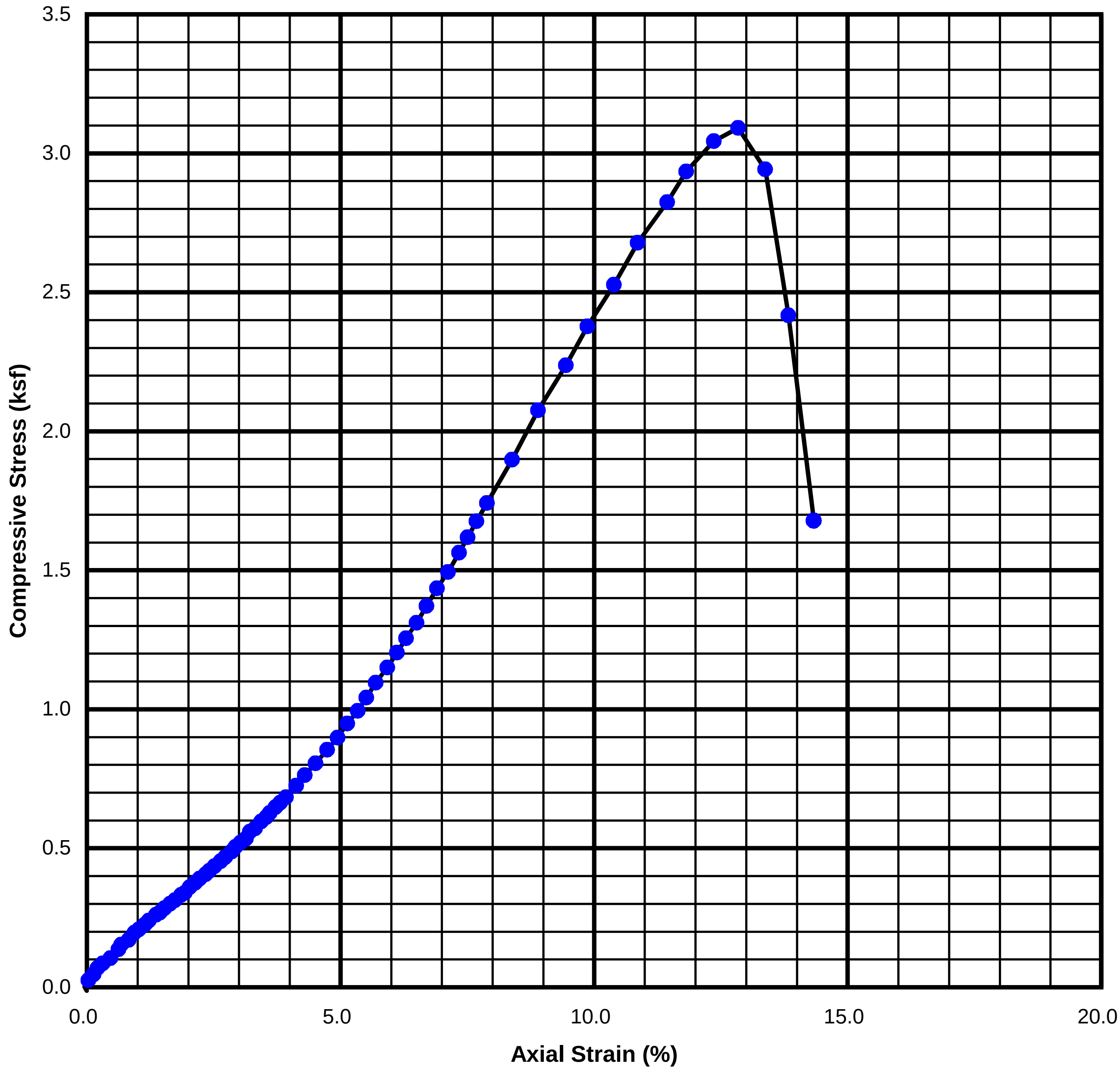
Exploration No: R-10-003 **Sample No.:** 4 **Depth (ft):** 25

Description and/or Classification: Yellowish brown clayey Sand (SC)

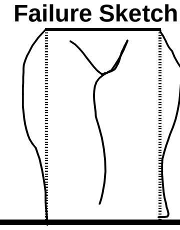
UNCONFINED COMPRESSION TEST

ASTM D 2166

Figure :



Failure Sketch



Water Content (%)	LL (%)	PI (%)	Length (in)	Diameter (in)	Wet Density (ksf)	Degree of Saturation (%)	Peak Stress (ksf)
21.1			5.623	2.416	130.4	98.0	3.09

Project Name:	**St-55 Dyer Road**
Project Number:	**30989831**
Exploration No:	**R-10-003** **Sample No.:** **13** **Depth (ft):** **70**
Description and/or Classification:	**Olive gray sandy Clay (CL)**

UNCONFINED COMPRESSION TEST

ASTM D 2166

Figure :

Appendix III: ARS Curve Data

Period	Acceleration	Period	Acceleration	Period	Acceleration	Period	Acceleration
0.01	0.511	0.09	0.69	0.36	0.992	1.5	0.506
0.02	0.518	0.095	0.704	0.38	0.993	1.6	0.465
0.022	0.522	0.1	0.718	0.4	0.993	1.7	0.43
0.025	0.528	0.11	0.745	0.42	0.989	1.8	0.399
0.029	0.534	0.12	0.769	0.44	0.983	1.9	0.382
0.03	0.536	0.13	0.791	0.45	0.981	2	0.367
0.032	0.541	0.133	0.797	0.46	0.978	2.2	0.332
0.035	0.549	0.14	0.81	0.48	0.972	2.4	0.304
0.036	0.551	0.15	0.828	0.5	0.967	2.5	0.291
0.04	0.56	0.16	0.847	0.55	0.947	2.6	0.279
0.042	0.564	0.17	0.865	0.6	0.93	2.8	0.258
0.044	0.569	0.18	0.882	0.65	0.914	3	0.24
0.045	0.572	0.19	0.898	0.667	0.909	3.2	0.223
0.046	0.574	0.2	0.912	0.7	0.899	3.4	0.208
0.048	0.579	0.22	0.933	0.75	0.886	3.5	0.202
0.05	0.584	0.24	0.95	0.8	0.862	3.6	0.195
0.055	0.596	0.25	0.958	0.85	0.839	3.8	0.183
0.06	0.608	0.26	0.963	0.9	0.817	4	0.173
0.065	0.62	0.28	0.973	0.95	0.797	4.2	0.166
0.067	0.626	0.29	0.976	1	0.778	4.4	0.16
0.07	0.633	0.3	0.98	1.1	0.709	4.6	0.154
0.075	0.646	0.32	0.986	1.2	0.649	4.8	0.149
0.08	0.661	0.34	0.99	1.3	0.595	5	0.144
0.085	0.675	0.35	0.991	1.4	0.548		

Appendix IV:
Soil Parameters
(For lateral pile analysis)

For Abutment 1

Cut off elevation is @ 61.5 feet.

According to our calculations the length of pile to develop a Nominal Resistance of 140 Kips is 53 feet below the cut-off elevation. The tip elevation is at 8.5

Between Elevation of 61.5 & 54 use the following:

Assume a Sand : γ = 125 PCF , K=90 lb/in³, θ =35°.

Between Elevation of 54.0 & 49 use the following:

Assume a Clay: γ = 122 PCF , C=7.63 PSI , ε50=0.005

Between Elevation of 49 & 44 use the following:

Assume a Sand : γ = 125 PCF , K=90 lb/in³, θ =33°.

Between Elevation of 44 & 29 use the following:

Assume a Clay: γ = 67.4 PCF , C= 5.55 PSI , ε50=0.01, K=95

Between Elevation of 29 & 24 use the following:

Assume a Clay: γ = 67.4 PCF , C= 13.9 PSI , ε50=0.004, K=800

Between Elevation of 24& 19 use the following:

Assume a Sand : γ = 63.4 PCF , K=125 lb/in³, θ =34°.

Between Elevation of 19 & -1 use the following:

Assume a Sand : γ = 67.4 PCF , K=125 lb/in³, θ =36°.

For Abutment 5

Cut off elevation is @ 62.5 feet.

According to our calculations the length of pile to develop a Nominal Resistance of 140 Kips is 52 feet below the cut-off elevation. The tip elevation is at 10.5

<u>Between Elevation of 62.5& 49 use the following:</u>

Assume a Clay: γ = 130 PCF , C=7.0 PSI , ε_{50}=0.004

<u>Between Elevation of 49 & 44 use the following:</u>

Assume a Sand : γ = 128 PCF , K=90 lb/in^3, θ =33°.

<u>Between Elevation of 44 & 39 use the following:</u>

Assume a Clay: γ = 126 PCF , C= 4.86 PSI , ε_{50}=0.01, K=92

<u>Between Elevation of 39 & 34 use the following:</u>

Assume a Clay: γ = 63.93 PCF , C= 4.17 PSI , ε_{50}=0.01, K=50

<u>Between Elevation of 34 & 24 use the following:</u>

Assume a Clay: γ = 67.4 PCF , C= 6.93 PSI , ε_{50}=0.005, K=250

<u>Between Elevation of 24& 0 use the following:</u>

Assume a Sand : γ = 67.4 PCF , K=125 lb/in^3, θ =36°.

For Bent-2

Cut off elevation is @ 50.5 feet.

According to our calculations the length of pile to develop the required loads is 47 feet below the cut-off elevation. The tip elevation is at 8.5

Between Elevation of 50.5 & 47.5 use the following:

Assume a Sand : γ = 125 PCF , K=90 lb/in³, θ =33°.

Between Elevation of 47.5 & 43.5 use the following:

Assume a Clay: γ = 120 PCF , C=5.21 PSI , ε_{50}=0.01

Between Elevation of 43.5 & 33 use the following:

Assume a Sand : γ = 72.6 PCF , K=125 lb/in³, θ =32°.

Between Elevation of 33 & 13 use the following:

Assume a Clay: γ = 47.52 PCF , C= 11.8 PSI , ε_{50}=0.005, K=500

Between Elevation of 13 & 08 use the following:

Assume a Sand : γ = 82.94 PCF , K=125 lb/in³, θ =36°.

Between Elevation of 08 & -2 use the following:

Assume a Clay: γ = 69.12 PCF , C=13.19 PSI , ε_{50}=0.005, K=500

Between Elevation of -2 & -7 use the following:

Assume a Sand : γ = 67.6 PCF , K=125 lb/in³, θ =36°.

Between Elevation of -7 & -39.5 use the following:

Assume a Clay: γ = 67.4 PCF , C= 12.32 PSI , ε_{50}=0.005, K=500

For Bent-3

Cut off elevation is @ 48 feet.

According to our calculations the length of pile to develop the required loads is 45.5 feet below the cut-off elevation. The tip elevation is at 2.5

Between Elevation of 48 & 43 use the following:

Assume a Sand : γ = 125 PCF , K=90 lb/in^3, θ =33°.

Between Elevation of 43 & 38 use the following:

Assume a Sand : γ = 126 PCF , K=90 lb/in^3, θ =33°.

Between Elevation of 38 & 35.5 use the following:

Assume a Sand : γ = 126 PCF , K=90 lb/in^3, θ =34°.

Between Elevation of 35.5 & 15.5 use the following:

Assume a Clay: γ =72.57 PCF , C= 8.33 PSI , ε_{50}=0.01, K=500

Between Elevation of 15.5& 10 use the following:

Assume a Sand : γ = 72.57 PCF , K=60 lb/in^3, θ =33°.

Between Elevation of 10 & -9.5 use the following:

Assume a Sand : γ = 65.6 PCF , K=125 lb/in^3, θ =37°.

Between Elevation of –9.5& -42 use the following:

Assume a Clay: γ = 67.39 PCF , C= 10.41 PSI , ε_{50}=0.004, K=800

For Bent-4

Cut off elevation is @ 50.5 feet.

According to our calculations the length of pile to develop the required loads is 45 feet below the cut-off elevation. The tip elevation is at 5.5

Between Elevation of 50.5 & 43 use the following:

Assume a Sand : γ = 130 PCF , K=90 lb/in³, θ =33˚.

Between Elevation of 43 & 38 use the following:

Assume a Clay: γ =130 PCF , C= 6.95 PSI , ε_{50}=0.005

Between Elevation of 38 & 28 use the following:

Assume a Clay: γ =43.54 PCF , C= 10.5 PSI , ε_{50}=0.005, K=450

Between Elevation of 28& 13 use the following:

Assume a Sand : γ = 72.57 PCF , K=125 lb/in³, θ =33˚.

Between Elevation of 13 & 4.5 use the following:

Assume a Sand : γ = 72.57 PCF , K=125 lb/in³, θ =34˚.

Between Elevation of 4.5 & -2 use the following:

Assume a Clay: γ = 72.57 PCF , C= 14.23 PSI , ε_{50}=0.005, K=750

Between Elevation of -2 & -7 use the following:

Assume a Sand : γ = 70.5 PCF , K=125 lb/in³, θ =34˚.

Between Elevation of –7& -41 use the following:

Assume a Clay: γ = 67.39 PCF , C= 13.89 PSI , ε_{50}=0.004, K=1000

State of California | Business, Transportation and Housing Agency

Memorandum

Flex your power!
Be energy efficient!

To: MR. MOHAMMAD RAVANIPOUR
Branch Chief, Design Branch 19
Office of Bridge Design South 2

Date: May 06, 2010

File: 12-ORA-55-PM 7.87
EA: 12-0G9601
Warner Ave OC Tieback Wall
Br# 55-0394

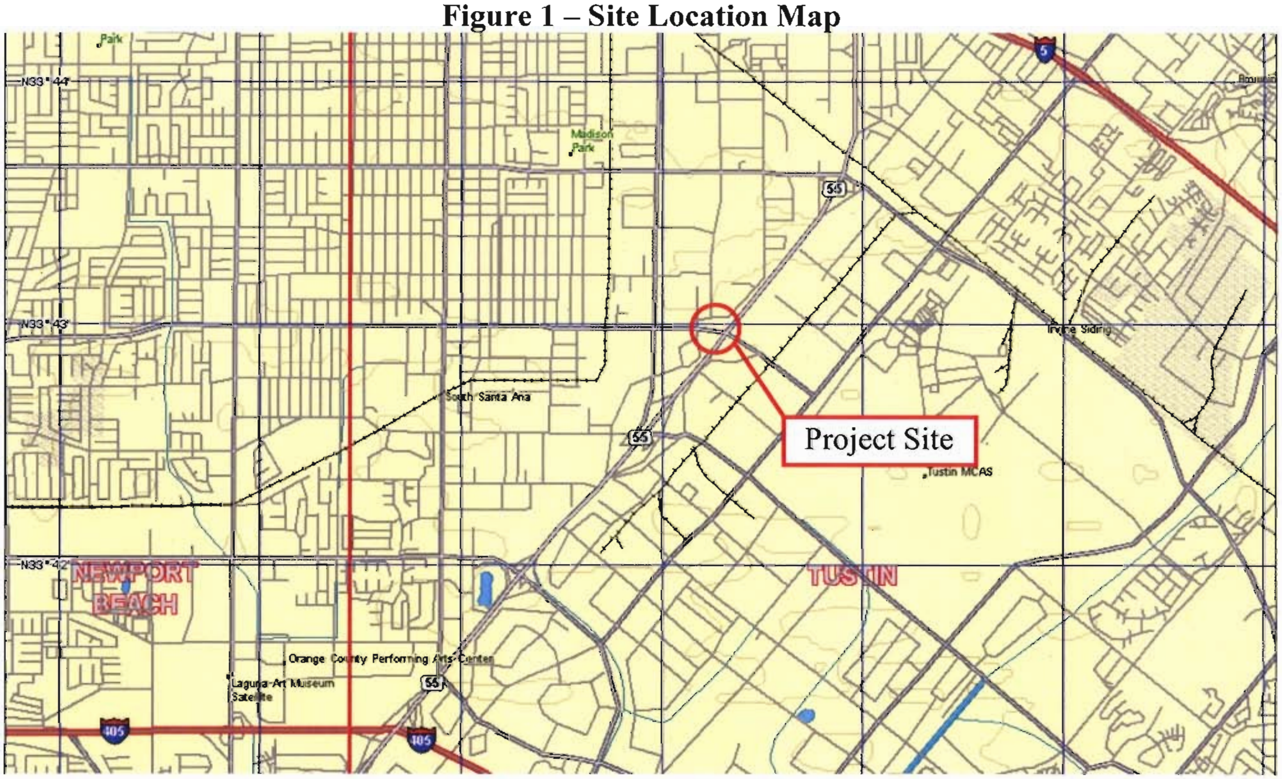
Attn: **J.R. Torres**

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

Subject: Revised Foundation Report for the Tieback Wall at Warner Avenue Overcrossing, Br# 55-0394

Per your request dated July 7, 2009, a final Foundation Report (FR) has been prepared for the proposed tieback wall underneath the western abutment of Warner Avenue Overcrossing on State Route 55. The foundation recommendations in this report are based on the latest plans provided by your office, dated 10/21/09, as well as a geotechnical exploration program done for this project.

Figure 1 – Site Location Map



1.0 PROJECT DESCRIPTION

1.1 Existing Site Conditions

The existing bridge is located on State Route 55 in the City of Santa Ana, Orange County. The existing structure is a two span cast-in-place box girder bridge with one 4-column reinforced concrete bent and open-end, reinforced concrete, seated abutments, all on driven concrete piles.

1.2 Proposed Structure

The proposed tieback wall will be built at the western (#1) abutment of Warner Avenue Overcrossing. The maximum height will be approximately 12 feet, with two 6-foot lifts. The total wall length is 200 feet, with 115 feet of tiebacks. In addition, a 35-foot long Type 1 wall on the left, and a 50-foot long Type 1 wall on the right will flank the tieback wall.

Table No. 1 – Tieback / Type-1 Wall - Required Foundation Data

Wall Section	Max. Design Height of Wall (ft)	Design Length (ft)	Bottom of Footing Elevation (ft)	Design Footing Width (ft)	Required Bearing (ksf)	
					q_{all}	q_n
Type 1	12	35	62.92	7' 3"	2.7	8.1
Tieback	12	115	64.25	N/A	N/A	N/A
Type 1	12	50	62.92	7' 3"	2.7	8.1

2.0 FIELD EXPLORATION PROGRAM

Three exploratory borings were drilled at the proposed wall location. Two borings were drilled on Route 55 on either side of Warner Ave. Overcrossing. One boring was drilled on Warner Ave. at the #1 abutment. Borings were drilled by Caltrans Office of Drilling Services and logged by personnel from our office.

Borings R-09-001 and R-09-002 were drilled on 9/1/09 and 9/2/09 respectively. Boring R-09-003 was drilled on 11/3/09. All borings were drilled using the mud rotary method. The Table below shows a summary of the Boring data with elevations and locations.

Table No. 2 – Summary of Boring Locations

Boring	Station [1]	Offset [2]	Surface Elevation[2] ft	Drilled Depth ft	Bottom Elevation ft
R-09-001	448+30.18	84.94 Lt	67.17	51.5	15.67
R-09-002	446+99.13	85.05 Lt	66.16	51.5	14.66
R-09-003	447+42.50	137.70	91.28	46.5	44.78

Note: 1. Stationing and Offsets according to 55 Center Line.
2. Elevations are Above Mean Sea Level (MSL) (1988 NAVD Datum).

Stations, offsets, and elevations of the borings were surveyed by a District 12 Surveys Crew and provided on 12-16-09.

Soil samples were logged and sampled using a Standard Penetration Test (SPT) sampler and a California sampler alternating at typically 5-foot intervals. The SPT samples were driven using a 140-pound hammer falling freely for 30 inches for a total penetration of 18 inches. The Modified California Sampler is a 2" diameter sampler that retrieves undisturbed push samples. At the completion of the borings, the holes were backfilled with bentonite chips.

Boring location will also be provided on the Log of Test Borings (LOTB), which is to be delivered at a later date. LOTBs are presently being prepared by the Office of Geotechnical Support and will be submitted to the Office of Structure Design.

3.0 LABORATORY TESTING

Laboratory testing was performed on selected SPT and undisturbed samples from the borings. Laboratory testing included unconfined compression and plasticity index. Geotechnical testing was performed in accordance with California Test Methods and/or ASTM procedures (see Table No. 3 below). The laboratory results are shown in Appendix I: Laboratory Data.

Table No. 3 – Laboratory Test Methods

Test	Standard
Unconfined Compression of Soils	CTM 221
Plasticity Index of Soils	CTM 204

4.0 SUBSURFACE CONDITIONS

According to As-Built Plans/survey data obtained for this project, an approximate elevation for the top of the borings ranges between 91.28 to 67.17 feet MSL. The deepest drilled depth of the borings was to an elevation of about 14.66 feet MSL.

The abutment fill to be retained is composed of stiff fat clays, lean clays, and sandy clays. The proposed tieback wall is underlain by medium dense, well-graded, sandy artificial fill to about el. 60 ft. The underlying native material is composed of fat clays, lean clays, and sandy clays with a 5' layer of soft sand at about el. 57 ft. The upper layers of clay are soft, but transition to hard by el. 37-32 ft.

Ground water was measured on 11/10/09 in boring R-09-001 at an elevation of 54.19 feet.

5.0 GEOLOGY

5.1 Regional Geology

The project is located within the Peninsular Ranges geomorphic province at the center of the Los Angeles Basin. A thick Cenozoic sedimentary section underlies the Los Angeles Basin that can be several miles thick. The Peninsular Ranges Province is characterized by northwest-southeast trending mountain ranges and valleys that are parallel to the San Andreas Fault.

5.2 Site Geology

The abutment fill consists of approximately 35 feet of sandy clay and lean clay. The underlying alluvium consists of predominantly clays and sandy clays with a layer of loose sand approximately 20 feet below the roadway. The alluvium is soft at the surface, but increases in density with depth.

6.0 SEISMICITY

The retaining wall site is not located within any Alquist-Priolo Earthquake Fault Zone as established by the California Geological Survey. Based on the Caltrans ARS Online site, the controlling faults are the San Joaquin Hills Blind Thrust, the Compton–Los Alamitos Blind Thrust, and the Newport-Inglewood/Rose Canyon Fault Zone. The average shear wave velocity of the upper 30 meters (Vs30) is approximately 270 m/sec based on correlations with SPT data collected during our geotechnical investigation. The Peak Ground Acceleration (PGA) calculated for this site is 0.5g. A summary of the contributing fault parameters as given by ARS Online is shown below.

Table No. 4 – Fault and Design Ground Motion Parameters.

Fault	Fault ID	M_{max}	Type	Dip°	Dip Direction	R_{rup} (km)	R_{JB} (km)	R_x (km)
San Joaquin Hills	7	6.6	Reverse	23	SW	3.83	3.12	3.12
Compton – Los Alamitos	291	6.8	Reverse	20	NE	9.66	0.77	14.50
Newport – Inglewood	427	7.5	Strike Slip	90	N/A	8.41	8.41	8.40

6.1 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 and the soil borings produced for this project, there is one possibly liquefiable layer present. The liquefiable layer is at approximately 20 ft below roadway grade and is approximately 5 ft thick. A liquefaction analysis yields a result of approximately one inch of settlement along the entire length of the proposed wall for a seismic event with an M_{max} of 7.5.

7.0 CORROSIVITY

As prescribed by the Caltrans Corrosion Technology Branch and the FHWA, all permanent anchor systems must have the standard corrosion protection applied regardless of test results. This includes protecting the full anchor length and anchor head as well. Corrosion resistant design is also recommended for the Type 1 walls as well. Corrosion test results are not yet available, and will be provided at a later date.

8.0 FOUNDATION RECOMMENDATIONS

8.1 Tie Back Wall Parameters

Based on Laboratory and field investigation data, soils behind the proposed tieback wall at the proposed tie back elevations are predominantly very stiff to hard Clays. For preliminary design, the resistance of the anchors may be estimated based on laboratory and field observations. Final design of the bonded length is the responsibility of the contractor and verified by load testing each ground anchor. An average ultimate bond stress (Between soil and concrete) of 1200 PSF may be used in preliminary design. For an 8-inch diameter anchor, we recommend a minimum bonded length of 16 feet be used.

Based on the provided Tieback Wall Plan, the minimum unbonded length of the 1st row is 20 feet, and 16 feet for the 2nd row. The unbonded length was based on an assumed failure plane acting at a 1:1 slope, and extending from the bottom of the tieback wall

8.2 Type-1 Walls (Shallow Foundations)

Based on the encountered subsurface conditions, the proposed walls may be supported on spread footings with preloading treatment, (see sections 8.2.1, 8.2.3 and 9.0), or on driven pile foundations. However, based on conversations with the Office of Structural Design, standard Type-1 retaining wall systems supported on spread footings are the preferred wall system to retain soils on either side of the tieback wall. Sections 8.2.1 to 8.2.3 provide detailed recommendations as well as design parameters for the proposed Type 1 walls.

8.2.1 Bearing Capacity

Allowable bearing capacity of the Retaining Wall footings was calculated using the total stress analysis method, using the undrained shear strength of the plastic soil. Table 5 summarizes the minimum spread footing dimensions and corresponding allowable bearing capacity for the Type 1

Retaining Walls. Minimum footing widths are based on Standard Type 1 Retaining Walls on Spread footings per the corresponding wall height, Plan Sheet B3-1 (May 2006).

Table No. 5 – Retaining Wall 116 - Recommended Spread Footing Data

Design Height of Wall (ft)	Bottom of Footing Elevation (ft)	Minimum Footing Width (ft)	Recommended Bearing Limits (kPa)	
			WSD Method (1) Allowable Bearing Capacity (q_{all})	LFD Method Nominal Soil Bearing Resistance (q_n)
H=6.0	62.92	4' 3"	1.5	4.5
H=8.0	62.92	5' 3"	2.0	6.0
H=10.0	62.92	6' 3"	2.3	6.9
H=12.0	62.92	7' 3"	2.7	8.1

Ground improvement will be required to densify the soils beneath the proposed Type-1 walls. Densification methods, such as surcharging will increase the strength and reduce the settlement of the subsurface soils. Surcharging is discussed in sections 9.0 and 10.0.

8.2.2 Lateral Active Earth Pressure

Passive earth pressures acting against the sides of the retaining wall footings may resist applied lateral loads. An allowable passive resistance value of Kp=2.66, may be used for foundations placed against compacted level clayey soil. A γ = 120 PCF may be used for the clayey on-site soils.

The sliding resistance along the bottom of retaining wall footings may be based on an allowable coefficient of friction of 0.3.

Assuming sandy soils (ø = 34°) backfill behind the wall, the active earth coefficient is (Ka=0.3) for level back fill, and (Ka = 0.65) for sloped backfill (35°) as shown on the plans. A γ = 125 PCF may be used for sandy soils.

8.2.3 Anticipated Settlement of Spread Footings

Total and differential settlements were calculated for the proposed retaining wall footings. Settlement was based on allowable bearing capacities at the retaining walls. The settlement parameters were estimated from generalized soil profiles for soils beneath the proposed retaining wall footings. Table No. 6 summarizes the estimated total and differential settlements for the proposed retaining structures.

Table 6 – Anticipated Settlement

Location		Footing Loads – q_{all}, ksf	Total Settlement, (in.)	Total Differential Settlement, (in.)
Type 1 Retaining Wall	H=6.0	1.5	1.9	1.0
	H=8.0	2.0	2.8	1.4
	H=10.0	2.3	3.9	1.9
	H=12.0	2.7	4.0	2.0

Given the above settlement magnitudes, coupled with low bearing capacities, ground improvement is required to increase bearing capacity and reduce settlement. Ground improvement includes surcharging the site, which involves stockpiling soil over the proposed footing footprint. The recommended height of soil to induce settlement is 12 feet. The recommended surcharge will simulate the net additional pressure applied to the subsurface soil, as a result of the proposed construction. The estimated time for 90% (t_{90}) of the above-estimated settlements to take place is between 70 and 90 days. The t_{90} is estimated based on a coefficient of consolidation c_v =0.33 ft^2/day. The estimated settlement time is also confirmed by actual as-built field measurements during the Warner Avenue bridge replacement in 1988.

9.0 EARTHWORK

- All earthwork should comply with general requirements outlined in Section 19 "Earthwork" of the 2009 Standard Specifications. Structural backfill to be placed behind the retaining walls should conform to Section 19-3.06 of the Standard Specifications (May 2006). Active coefficients provided in section 8.2 do not assume hydrostatic pressure build up behind the walls. It is therefore imperative that proper drainage be provided behind the walls as shown on Standard plan B3-8.

- Site preloading is recommended, and a monitoring period is required. The settlement magnitude and the required waiting period are dependant on the amount of fill placed. The settlement magnitudes were calculated based on basic consolidation theories. It should be noted that the estimated settlement values and durations are for planning purposes only. The actual settlement period will be determined in the field by the engineer based on the results of settlement monitoring. Surface monuments constructed in accordance with Caltrans Standard plan A74 or equivalent at the original ground surface and the top of the surcharge are required. Surcharge slopes should not be steeper than 1:1. Surcharge may be removed after the completion of the settlement-waiting period.

- Following the removal of the surcharge and excavation to the bottom of footing elevation, a representative of the RE must inspect the exposed sub-grade. Proof rolling is recommended to check for soft sub-grade areas. Should any soft areas be encountered, subject areas should be stabilized before the construction of the foundations.

10.0 CONSTRUCTION CONSIDERATIONS

- Ground water is expected within 10 feet of the existing surface. Over excavation should be minimized and implemented when necessary to achieve stabilization. It is recommended that no vibratory compaction equipment be used on this site, specifically at areas close to the bottom of footing elevations, as it may destabilize subsurface soils in creating a pumping condition.

- Temporary slopes during construction may be no steeper than 1:1 (Vertical: Horizontal). If any temporary slopes need to be steeper than 1:1 a temporary shoring system must be used and devised by the Contractor.

- The settlement monuments should be placed at 25-foot intervals, and should be monitored once a week for at least the first month after completion of fill or surcharge, then once every two weeks thereafter. Settlement monitoring should continue until such time that sufficient readings are obtained to indicate that Primary settlements are complete. It is imperative that the monuments placed on original ground be surveyed prior to the placement of any fill. Settlement within soils above ground water (unsaturated soils) is expected to take place, as soon as the fill (surcharge) is placed.

- The minimum bonded and unbonded lengths provided in this report should be provided in the contract plans and specifications. The contract plans and specifications should also state that the contractor must be responsible for determining the actual bond length in the field, provided that the actual bond length exceeds the specified minimum bond length.

- As discussed in section 7.0, all permanent anchors should be corrosion protected along the length of the anchor and the anchor head.

- Proof testing or performance testing must be done on all permanent tiebacks to verify tieback capacity. The specifications should also indicate that it is the contractor's responsibility to achieve the required design test loads. The test loads are generally 1.5 times the design force T.

- After successful testing, the tiebacks should be stressed to the specified design force and locked off against structure wall. The lock off force should equal 0.75 the design force T, per section 5.8.11.2 of the BDS.

- A sequence of backfilling, placing the tiebacks and stressing should be specified in detail to prevent overstressing any members during construction.

- After drilling tieback holes is complete, the holes should be probed to verify that no collapse has occurred, before the installation of the pre-stressing elements. Installation of pre-stressing elements and grouting should be done on the same day to avoid hole deterioration or complete collapse.

If you have any questions, please contact Kristopher Barker at (213) 620-2334 or Nadeem Srour at (213) 620-2377.

Prepared by:

M. Mustafa

FOR Kristopher Barker, C.E.G. 5/6/10
Engineering Geologist
Office of Geotechnical Design South 1
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REGISTERED PROFESSIONAL ENGINEER
NADEEM SROUR
No. 2801
Exp. 12/31/11
GEOTECHNICAL
STATE OF CALIFORNIA

5/6/2010
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Reviewed by:

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Senior Transportation Engineer
Office of Geotechnical Design South 1
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c.c. GS File Room
District Project Manager – Bob Bazargan
District Project Engineer – Bang H. Nguyen
GS Corporate – Mark Willian
Structure Construction R.E. Pending File
DES Office Engineer, Office of PS&E
District Materials Engineer

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1. Caltrans, 2007, Foundation Report for the LA-5/LA-60 Connector Merge Improvement Widening (Euclid Avenue Widening and Retaining Walls 104 and 106), Bridge No. 53-1389L, 8/28/2007.

2. Caltrans, 2007, Preliminary Seismic Design Recommendations for the LA-5/LA-60 Connector Merge Improvement and Euclid Street Offramp Undercrossing Widening, Bridge No. 53-1389L, 5/18/2007.

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7. Sadigh, K, Chang, C.Y., Egan, J.A., Makdisi, F. and Youngs, R.R., 1997, Attenuation Relationships for Shallow Crustal Earthquake Based on California Strong Motion Data, Seismological Research Letters, Vol. 68, No.1.

Appendix I: Laboratory Data

DISTRICT 7 SOIL & AGGREGATE TESTS

1	2	0	G	9	6	0	1	S	P	-	5	0857
CONTRACT NO.								SAMPLE NO.				LAB. NO.

Sample of: SOIL

Sampled from: WARNER AVENUE OC TIEBACK WALL DEPTH: 25' - 26.5'

Material Source: JOBSITE BR # 55-0394 @ BORING #R - 09 - 03

Owner / Mfr.: STATE 12 - OR - 55 - R 8.5

Date Sampled: 11/3/09

Materials Lab. Stamp

By PSR for

G. ANYANWU
District Materials Engineer

DATE RCV'D: 11/23/09
DATE OUT: 12-9
NUMBER OF CONTAINERS: 1 BAG
NORMAL ☐ PRIORITY ✓
DATE NEEDED:
By: FAX ✓ MAIL PHONE OTHER

R.E.: KRIS BARKER
Address:
Geotechnical Design - South 1
100 South Main Street
11th Floor, MS #18
Los Angeles, CA 90012
Phone No.: 213-620-2334
Fax. No.: 213-620-2316

DISTRICT MATERIALS LABORATORY
1616 S. Maple Street
Los Angeles, CA 90015
Phone: (213) 620-5692
Calnet: 8-640-5692
Fax: (213) 620-5540
Calnet: 8-640-5540

GRADING ANALYSIS

Total Wt | By: | Date:

Wt. Ret.	Size (mm)	Acc. Wt. Ret.	% Ret.	% Pass	Comb. % Pass	SPEC.
	87.5					
	75					
	62.5					
	50					
	37.5					
	25					
	19					
	12.5					
	9.5					
	4.75					

FINE GRADE / MECHANICAL ANALYSIS

Dry Wt. (g)	Size					
	2.36 mm					
	1.18 mm					
	600 µm					
	300 µm					
	150 µm					
	75 µm					
	Pan (-75 µm)					

MECH. / HYDRO.		R	Corr.	C.R.	Mat In Sus.	Comb % Pass
1hr.	5M					
24hr.	1M					

SAND EQUIVALENT

				Avg.	MIN. SPEC.
Sand R2					
Clay R1					
S.E. Value					

L.A.R.T.	Rev.	Wt.	Wt. Ret.	% Ret.	% Loss	% SPEC
A B	100	5000g				
C D	500	5000g				

No. of spheres = Wt. of spheres =

DURABILITY INDEX

			SPEC.
Dura-Coarse	Sed.Ht.=		
Dura-Fine	R2/R1 =		

R-VALUE BATCH

% Run	Size	Wt.
	25 mm	
	19 mm	
	12.5 mm	
	9.5 mm	
	4.75 mm	

% CRUSHED PARTICLES

%Ret.x(Wt.Cr./Tot.Wt.) = Prod. | SPEC.

Wtd. Avg. % CP
Ret. No.4 =
% CP = P/R

R-VALUE RESULT | SPEC.

SP. G. FINE (SSD)
(B) S.S. Dry
(A) Ov. Dry
ABS. %
Wt. S+C+H2O
Wt. S+C
W=Wt. H2O

$$Bulk = \frac{500}{500 - W} =$$

SP. GR. COARSE CT206/CT209, +4
(B) S.S. Dry
(A) Ov. Dry
ABS. %
(C) Wt. S. in H2O

$$App = \frac{A}{A - C} =$$

$$SSD = \frac{B}{B - C} =$$

$$OD = \frac{A}{B - C} =$$

MOISTURE CONTENT

Gr. Wet
Gr. Dry
H2O
Tare
Net Dry
% H2O

PLASTICITY INDEX

	SPEC.	
L.L.		35
P.L.		22
P.I.		13

pH / RESISTIVITY

	Field	Lab.	MIN. SPEC.
Soil pH			5.5~10.0
H2O			
Min. Resistivity			NL/ 1000

Based on 18 gauge CMP.
Estimated life:

CLEANNESS VALUE

NL	SED. HT.	RESULT

FILM STRIPPING	ORGANIC IMPURITIES	
NL	Satisfactory	
	Unsatisfactory	

TEST(S) REQUESTED

Test		✓
Fine Grade	202	
Coarse Grade	202	
Filler Material	202	
Mech. Analysis	203	
Plasticity Index	204	✓
% Crushed Particles	205	
SpG. Coarse	206	
SpG. Fine (SSD)	207	
SpG. of Soils	209	
L.A.R.T.	211	
Unit Wt.	212	
Organic Impurities	213	
Soundness	214	
Relative Compaction	216	
Sand Equivalent	217	
Moisture Content	226	
Cleaness Value	227	
Durability Fine	229	
Durability Coarse	229	
Reactivity	ASTM C-289	
R-Value	301	
Film Strip	302	
Mortar Strength	515	
pH (RC)	532	
Resistivity (RC)	532	
pH (CMP)	643	
Resistivity (CMP)	643	
Expansion Index	UBC-29-2	
Max. Dry Density/ Opt. Moist Content	ASTM-D1557	

SAMPLE TYPE

			✓
A.B.		PCC	
A.S.		Bk.Fill	
EMB.		MISC.	
O.G.		Sub-Grade	
A.C. Agg.		SOIL	✓

TL-101 S.I.C. NO.

Wet Unit Weight, pcf

Dry Unit Weight, pcf

Laboratory Remarks:

SPECIFIC GRAVITY OF SOILS

Wt Oven Dry Soil (Wo)
Wt Pycnometer + H_2O (Wa)
Wt Pycnometer + H2O + Soil (Wb)
Wo / (Wo + Wa - Wb)

Wo		Spec. Grav.	
Wa			
Wb			

CONTRACT NO. 1 2 - 0 G 9 6 0 1

LAB. NO. 0 8 5 7

DISTRICT 7 SOIL & AGGREGATE TESTS

CONTRACT NO.	SAMPLE NO.	LAB. NO.
1 2 - 0 G 9 6 0 1	S P - 7	0858

DATE RCV'D: 11/23/09 DATE OUT: 12-9

NUMBER OF CONTAINERS: 1 BAG By: FAX ✓ MAIL PHONE OTHER

NORMAL PRIORITY ✓ DATE NEEDED:

Sample of: SOIL

Sampled from: WARNER AVENUE OC TIEBACK WALL DEPTH : 35' - 36.5'

Material Source: JOBSITE BR # 55-0394 @ BORING #R - 09 - 03

Owner / Mfr.: STATE 12 - OR - 55 - R 8.5

Date Sampled: 11/3/09

Materials Lab. Stamp

By PSR for

G. ANYANWU

District Materials Engineer

R.E.: KRIS BARKER

Address:
Geotechnical Design - South 1
100 South Main Street
11th Floor, MS #18
Los Angeles, CA 90012

Phone No.: 213-620-2334

Fax. No.: 213-620-2316

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1616 S. Maple Street
Los Angeles, CA 90015

Phone: (213) 620-5692
Calnet: 8-640-5692
Fax: (213) 620-5540
Calnet: 8-640-5540

TEST(S) REQUESTED

TEST(S) REQUESTED		✓
Fine Grade	202	
Coarse Grade	202	
Filler Material	202	
Mech. Analysis	203	
Plasticity Index	204	✓
% Crushed Particles	205	
SpG. Coarse	206	
SpG. Fine (SSD)	207	
SpG. of Soils	209	
L.A.R.T.	211	
Unit Wt.	212	
Organic Impurities	213	
Soundness	214	
Relative Compaction	216	
Sand Equivalent	217	
Moisture Content	226	
Cleaness Value	227	
Durability Fine	229	
Durability Coarse	229	
Reactivity	ASTM C-289	
R-Value	301	
Film Strip	302	
Mortar Strength	515	
pH (RC)	532	
Resistivity (RC)	532	
pH (CMP)	643	
Resistivity (CMP)	643	
Expansion Index	UBC-29-2	
Max. Dry Density/ Opt. Moist Content	ASTM-D1557	

SAMPLE TYPE

SAMPLE TYPE			✓
A.B.		PCC	
A.S.		Bk.Fill	
EMB.		MISC.	
O.G.		Sub-Grade	
A.C. Agg.		SOIL	✓

TL-101 S.I.C. NO.

Wet Unit Weight, pcf

Dry Unit Weight, pcf

Laboratory Remarks:

GRADING ANALYSIS

Total Wt By: Date:

Wt. Ret.	Size (mm)	Acc. Wt. Ret.	% Ret.	% Pass	Comb. % Pass	SPEC.
	87.5					
	75					
	62.5					
	50					
	37.5					
	25					
	19					
	12.5					
	9.5					
	4.75					

FINE GRADE / MECHANICAL ANALYSIS

Dry Wt. (g)	Size					
	2.36 mm					
	1.18 mm					
	600 µm					
	300 µm					
	150 µm					
	75 µm					
	Pan (-75 µm)					

MECH. / HYDRO.		R	Corr.	C.R.	Mat In Sus.	Comb % Pass
1hr.	5M					
24hr.	1M					

SAND EQUIVALENT

				Avg.	MIN. SPEC.
Sand R2					
Clay R1					
S.E. Value					

L.A.R.T.	Rev.	Wt.	Wt. Ret.	% Ret.	% Loss	% SPEC
A B	100	5000g				
C D	500	5000g				

No. of spheres = Wt. of spheres =

DURABILITY INDEX

		SPEC.
Dura-Coarse	Sed.Ht.=	
Dura-Fine	R2/R1 =	

R-VALUE BATCH

% Run	Size	Wt.
	25 mm	
	19 mm	
	12.5 mm	
	9.5 mm	
	4.75 mm	

R-VALUE RESULT SPEC.

SP. G. FINE (SSD)

(B) S.S. Dry
(A) Ov. Dry
ABS. %
Wt. S+C+H2O
Wt. S+C
W=Wt. H2O

$Bulk = \frac{500}{500 - W} =$

SP. GR. COARSE CT206/CT209, +4

(B) S.S. Dry
(A) Ov. Dry
ABS. %
(C) Wt. S. in H2O

$App = \frac{A}{A-C} =$

$SSD = \frac{B}{B-C} =$

$OD = \frac{A}{B-C} =$

% CRUSHED PARTICLES

%Ret.x(Wt.Cr./Tot.Wt.) = Prod. SPEC.

Wtd. Avg. % CP Ret. No.4 =

% CP = P/R

MOISTURE CONTENT

Gr. Wet
Gr. Dry
H2O
Tare
Net Dry
% H2O

PLASTICITY INDEX

	SPEC.	
L.L.		23
P.L.		22
P.I.		1

pH / RESISTIVITY

	Field	Lab.	MIN. SPEC.
Soil pH			5.5~10.0
H2O			
Min. Resistivity			NL/ 1000

Based on 18 gauge CMP.
Estimated life:

CLEANNESS VALUE

NL	SED. HT.	RESULT

FILM STRIPPING

NL

ORGANIC IMPURITIES

Satisfactory
Unsatisfactory

SPECIFIC GRAVITY OF SOILS

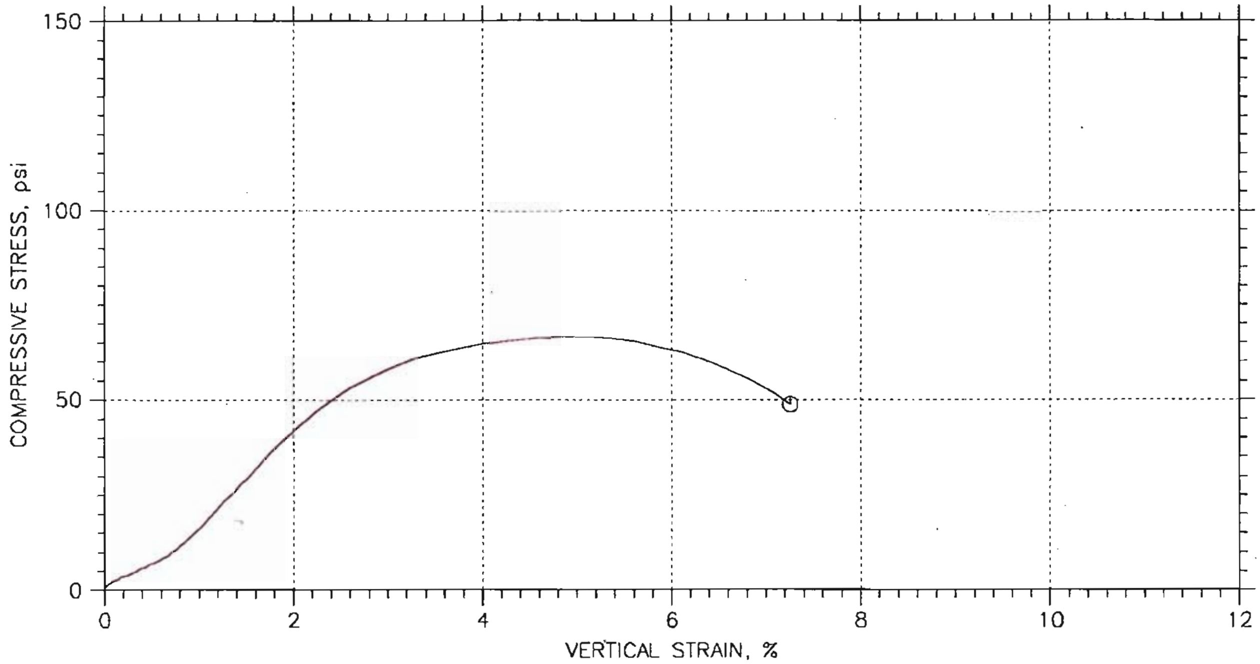
Wt Oven Dry Soil (Wo)
Wt Pycnometer + H_2O (Wa)
Wt Pycnometer + H2O + Soil (Wb)
Wo / (Wo + Wa - Wb)

Wo
Wa
Wb
Spec. Grav.

CONTRACT NO. 1 2 - 0 G 9 6 0 1

LAB. NO. 0 8 5 8

UNCONFINED COMPRESSION TEST REPORT



Symbol	⊕			
Test No.	Q09-133			
Initial — Diameter, in	1.92			
Initial — Height, in	4.04			
Initial — Water Content, %	13.18			
Initial — Dry Density, pcf	123.5			
Initial — Saturation, %	---			
Initial — Void Ratio	---			
Unconfined Compressive Strength, psi	66.65			
Undrained Shear Strength, psi	---			
Time to Failure, min	---			
Strain Rate, %/min	1			
Implied Specific Gravity	---			
Liquid Limit	---			
Plastic Limit	---			
Plasticity Index	---			
Failure Sketch				

Caltrans	Project: Warner Ave OC Tieback Wal
	Location: 12-ORA-55-R8.5
	Project No.: 12-0G9601
	Boring No.: R-09-003
	Sample Type: BRASS
	Description: Moist, Very Stiff, Dark Brown, Silty Clay with Sand.
	Remarks: ASTM D 2166.

Thu, 10-DEC-2009 15:12:37

FOUNDATION REVIEW

DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

To: **Structure Design**

1. Design
2. R.E. Pending File
3. Specifications & Estimates
4. File

Geotechnical Services

1. GD - North ; South ; West
2. GS File Room

Date: 6/10/10

Warner Ave. Tieback Wall
Structure Name

12 - Ora - 55 - R8.5
District County Route ~~km~~ Post mi

District Project Development
District Project Engineer

12-0G9601 55-0374
E.A. Number Structure Number

Foundation Report By: N. Srour Dated: 1/7/10

Reviewed By: J Torres (SD) R. Price (GS)

General Plan Dated: 6/3/10 Foundation Plan Dated:

☑ No changes. ☐ The following changes are necessary.

FOUNDATION CHECKLIST

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

Juan Torres J. 19
Structure Design Bridge Design Branch No.

Geotechnical Services

Rev. 06/02

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: 6/10/10

Dyer Rd. UC
Structure Name

12 - Ora - 55 - 7.8
District County Route ~~Post Km~~ M.

12 - 069601 55 - 0409
E.A. Number Structure Number

Foundation Report By: N. Srour Dated: 6/9/10

Reviewed By: J. Torres (OSD) R Price (GS)

General Plan Dated: 6/8/10 Foundation Plan Dated: 6/8/10

☑ 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

Juan Torres Jr. 19
Office of Structure Design Branch No.

[signature]
Geotechnical Services

Rev. 10/02