FOR CONTRACT NO. 07-215934

# INFORMATION HANDOUT

# MATERIALS INFORMATION

FOUNDATION REPORT FOR SOUNDWALLS 160, 204 AND 205

**ROUTE: 07-LA-5-PM 2.5/4.0**

**ADDED PER ADDENDUM NO. 1 DATED JULY 13, 2012**

State of California | Business, Transportation and Housing Agency

# Memorandum

*Flex your power!*
*Be energy efficient!*

To: Ms. TRACI MENARD, CHIEF
Structures Design Branch 15
Office of Bridge Design-South 1
Attention: Mr. John Lane

Date: July 11, 2012
File: 07-LA-005-PM 2.5/4.0
0700001833 (07-215931)

Soundwalls 160, 204, and 205

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

Subject: Foundation Report for Soundwalls 160, 204, and 205

The Office of Geotechnical Design-South 1 was requested to provide foundation recommendations for portions of soundwalls 160, 204, and 205. The portions of soundwalls were specially designed to span over new bent footings of Shoemaker Avenue Overcrossing (No. 53-3039) and Silverbow Pedestrian Overcrossing (No. 53-3043) and underground reinforced concrete (RC) pipe and box culvert. It should be noted that Caltrans Standard Type 736SV soundwall on barrier was used for the walls except these specific portions.

## PROJECT DESCRIPTION

As part of Interstate 5 Corridor Improvement Project (Segment 3), the soundwalls will be constructed in Los Angeles County in order to reduce the impact of traffic noise on adjacent residential areas. In accordance with Geotechnical Design Report-Revision 1 dated April 7, 2010, the soundwalls will be founded on sixteen (16) inch cast-in-drilled-hole (CIDH) piles except two segments of SW 160, one segment of 204, and three segments of SW 205 where the walls are required to cross the proposed bent footings or underground RC box culverts. In order to span over the footings or underground RC box culverts, the portions of SW 160, 204, and 205 will be founded on 30 inch CIDH piles. Detailed information of the special designs is presented below:

**Table 1 – Special Foundation Designs for Soundwalls 160, 204, and 205**

| Wall No | Station (I-5 CL) | | Span Length (feet) | Pile Type | Remarks |
|---|---|---|---|---|---|
| | From | To | | | |
| SW 160 | 161+30.4 | 161+55.49 | 25.1 | 30 inch CIDH | Bent 4 (Shoemaker Ave OC) |
| SW 160 | 161+55.49 | 161+80.58 | 25.1 | 30 inch CIDH | Bent 4 (Shoemaker Ave OC) |
| SW 204 | 210+49.0 | 210+68.9 | 20.0 | 30 inch CIDH | RC Pipe and Box |
| SW 205 | 209+42.2 | 209+61.6 | 19.4 | 30 inch CIDH | Bent 7 (Silverbow Ave POC) |
| SW 205 | 209+61.6 | 209+79.5 | 17.9 | 30 inch CIDH | RC Box |
| SW 205 | 209+79.5 | 210+04.6 | 25.1 | 30 inch CIDH | RC Box |

## SCOPE OF WORK

The following tasks were performed in preparing foundation recommendations:

- Review of the pertinent reports and plans
- Field reconnaissance to observe the existing conditions at the site
- Field exploration and laboratory testing
- Interpretation of subsurface soil and groundwater conditions at the project site
- Engineering analyses and preparation of foundation recommendations for design and construction of the proposed structures

## PERTINENT DOCUMENTS AND FIELD INVESTIGATIONS

For foundation design of the proposed Silverbow Pedestrian Overcrossing (No. 53-3043), one cone penetration test (CPT) was conducted by URS Corporation in 2008 and three exploratory borings were performed by Caltrans personnel in 2010 in order to fully investigate subsurface conditions at each support location of the bridge. The results of CPT and exploratory borings were used to prepare foundation recommendations for the special designs of soundwalls 204 and 205 because of proximity to the location of those two walls. For special design of soundwall 160, one exploratory boring close to the wall was selected among borings and cone penetration tests performed for Showmaker Avenue Overcrossing (No. 53-3039).

During exploratory borings, Standard Penetration Tests (SPTs) and relatively undisturbed sampling were performed. The SPTs were performed in accordance with ASTM Test Method D1586 using a standard 1.4 inch sampler with a 140 pound hammer dropped 30 inches. Relatively undisturbed soil samples were also obtained using a 2.0 inch California modified sampler. The information from the field exploration is summarized in Table 2. In addition to the above field investigation and testing, the following documents were reviewed for preparation of the recommendations:

- Liquefaction Potential Analysis Report for I-5 HOV Widening (Segment 3), URS Corporation, June 2011.
- Log of Test Boring for Silver Bow Avenue Pedestrian Overcrossing (No. 53-1003), November 1951.
- Log of Test Boring for Silver Bow Avenue Pedestrian Overcrossing (No. 53-1003), November 1995.
- Log of Test Boring for Shoemaker Avenue Overcrossing (No. 53-1015), January 1953.
- Log of Test Boring for Shoemaker Avenue Overcrossing (No. 53-1005), May 1996.
- Surface Street and State Right-of-Way Evaluation Report prepared for District 7 OEECS, Interstate 5/Segment 3 Improvement Project, AMEC Geomatrix, Inc., August 2010.

**Table 2 – Summary of Subsurface Exploration**

| Borehole ID | Date Drilled | Total Depth (ft) | Surface Elevation (ft) | Station (I-5 CL) | Offset (ft) | Remarks |
|---|---|---|---|---|---|---|
| R-08-019 | 09/25/2008 | 101.5 | 97.40 | 161+83.12 | Rt 177.5 | URS |
| CPT-08-095 | 05/28/2008 | 60.7 | 94.76 | 209+87.98 | Lt 75.70 | URS |
| R-10-303 | 10/05/2010 | 101.5 | 94.61 | 209+70.01 | Lt 116.39 | Caltrans |
| R-10-304 | 10/06/2010 | 111.5 | 94.73 | 210+04.13 | Lt 41.16 | Caltrans |
| R-10-305 | 10/18/2010 | 101.5 | 94.92 | 210+53.91 | Rt 138.06 | Caltrans |

It should be noted that the three digit sequence number in boring identification starts with the segment number per approved exception to 2007 Caltrans Soil and Rock Logging Manual (e.g. R-10-**301**, not R-10-**001** for the I-5/Segment 3).

## LABORATORY TESTING

Selected samples taken during the field investigation were tested at Caltrans Headquarters Soil Testing Laboratory and District 7 Material Testing Laboratory in order to obtain or derive relevant physical and engineering soil properties. The following laboratory tests were conducted to supplement the observations recorded during the field investigation:

- In-situ Moisture Content and Unit Weight
- Sieve analysis
- Atterberg Limits
- Unconsolidated-undrained test (UU test)
- Consolidation
- Minimum Resistivity, pH, Sulfate and chloride content

The laboratory tests were conducted in general accordance with California Test Methods or American Society for Testing and Material (ASTM) Standards.

## GEOLOGY AND SUBSURFACE CONDITIONS

### Regional Geology

The subject site is located within the Peninsular Range Geomorphic Province. The Peninsular Ranges are characterized by northerly and northwesterly trending mountain ranges and associated valleys. The site is located within the Coastal Plain of Los Angeles County, which is comprised of shallow Pleistocene marine sediments overlain by Holocene alluvial deposits (Department of Water Resources, 1961). The Coastal Plain is bounded by the Santa Monica Mountains, Elysian Hills, Repetto Hills, Merced Hills and Puente Hills on the north and bounded by the Palos Verdes Hills on the south. Northwest-southeast trending strike-slip faults are present within and bordering the Coastal Plain (Newport Inglewood Fault and Whittier Fault). Reverse and thrust faults including the Santa Monica-Hollywood-Raymond Fault and Puente Hills Blind Thrust Fault are present and associated with shortening or compression of the Coastal Plain. The active fault(s) nearby the site are discussed in Seismic Recommendations, Faulting and Seismicity section of this report.

**Site Geology**

The entire project site is relatively flat and directly underlain by recent Holocene age alluvium. This alluvium was deposited primarily by floods emanating from the Los Angeles River and the San Gabriel River and from the mountains and hills to the north of the Coastal Plain adjacent to the project location. The alluvium consists of predominantly medium dense sand with varying amount of silt and/or clay, and soft to medium stiff sandy silt, silt, silty clay and sandy clay. Depth to bedrock or bedrock like material should be estimated at greater than 400 feet for this project. The proposed foundations will be founded on alluvium. The closest fault to the site is the Puente Hills Blind Thrust Fault oriented as a low angle north dipping thrust fault approximately 1.9 miles northwest of the site (Caltrans, 2009).

**Subsurface Conditions**

Based on the recent field exploration and as-built Log of Test Borings, the site is underlain by alluvial deposits consisting of medium dense to very dense sand with varying amount of silt and/or clay, silt, clay and mixture of clay and silt. The subsurface soil at proposed CIDH pile locations were generalized using information revealed by the exploratory borings or CPT. The soil profile and design strength parameters of subsurface material for the CIDH piles are presented in Appendix 1.

**Groundwater**

Groundwater was encountered during the 1950's and 1990's field investigations for Shoemaker Avenue Overcrossing and Silverbow Pedestrian Overcrossing. In the 1950's Log of Test Borings, the highest groundwater table is recorded at an elevation of 67.2 feet at Shoemaker Avenue Overcrossing, but location of groundwater table ranges from an elevation of 43.9 to 52.9 feet in 1990's Log of Test Borings. For conventional and seismic geotechnical analysis, the design groundwater table is assumed to be at an elevation of 60.0 feet by averaging the highest groundwater measurements of 1950's and 1990's Log of Test Borings. District 7 Office of Environmental Engineering and Corridor Studies informed us that installation of the proposed CIDH piles for this structure will not cause migration of contamination in groundwater and subsurface soil.

## CORROSION EVALUATION

Selected samples were tested at District 7 Materials Testing Laboratory in order to obtain corrosivity parameters including pH, resistivity, sulfate and chloride content. The results are summarized in Table 3.

**Table 3 – Summary of Corrosion Test Results**

| Borehole ID | Depth (ft) | pH | Minimum Resistivity (ohm-cm) | Sulfate Content (ppm) | Chloride Content (ppm) |
|---|---|---|---|---|---|
| R-08-019 | 0.0 – 5.0 (combined) | 8.0 | 3500 | 12 | 75 |
| R-10-304 | 0.0 – 50.0 (combined) | 8.5 | 5300 | N/A | N/A |
| B-1 (1995) | 0.0-71.5 (combined) | 7.7 | 1500 | N/A | N/A |

Note: N/A = Not available

Caltrans currently considers a site to be corrosive to foundation elements if one or more of the following conditions exist for the representative soil and/or water samples taken at the site: Chloride concentration is greater than or equal to 500 ppm, sulfate concentration is greater than or equal to 2000 ppm, or the pH is 5.5 is less. District 7 Materials Laboratory informed us that subsurface material is identified as non-corrosive without sulfate and chloride content testing if minimum resistivity is larger than 1,000 ohm-cm. Based on the results of corrosion tests, the site is considered non-corrosive to foundation elements.

## SEISMIC RECOMMENDATIONS

### Faulting and Seismicity

The Puente Hills Blind Thrust fault (PHBT) is the controlling seismic source for these structures. The PHBT is a reverse fault dipping 25 degrees to the north. The design Acceleration Response Spectrum (ARS) curve was developed for the seismic design of this structure per the Appendix B of the Caltrans Seismic Design Criteria, Version 1.5 (August 2009) and the Caltrans Geotechnical Services-Design Manual, Version 1.0 (August 2009). In addition to the criteria, various tools including "Caltrans ARS Online" and "United States Geologic Survey-Interactive Deaggregation" were utilized to produce the curve. Based on the recent field investigation, the average shear wave velocity ($V_{s30}$) for the upper 100 feet (30 meters) of subsurface soils at the site was estimated to be about 787 ft/sec (240 m/sec). The information utilized to determine the curve is shown in Table 4:

**Table 4 – Fault Information**

| Fault Name | Type | $M_{max}$ | Dip direction (Dip angle) | $R_X$ | $R_{JB}$ | $R_{RUP}$ |
|---|---|---|---|---|---|---|
| Puente Hills Blind Thrust | R | 7.3 | North (25 degrees) | 3.1 km (1.9 miles) | 3.1 km (1.9 miles) | 4.3 km (2.7 miles) |

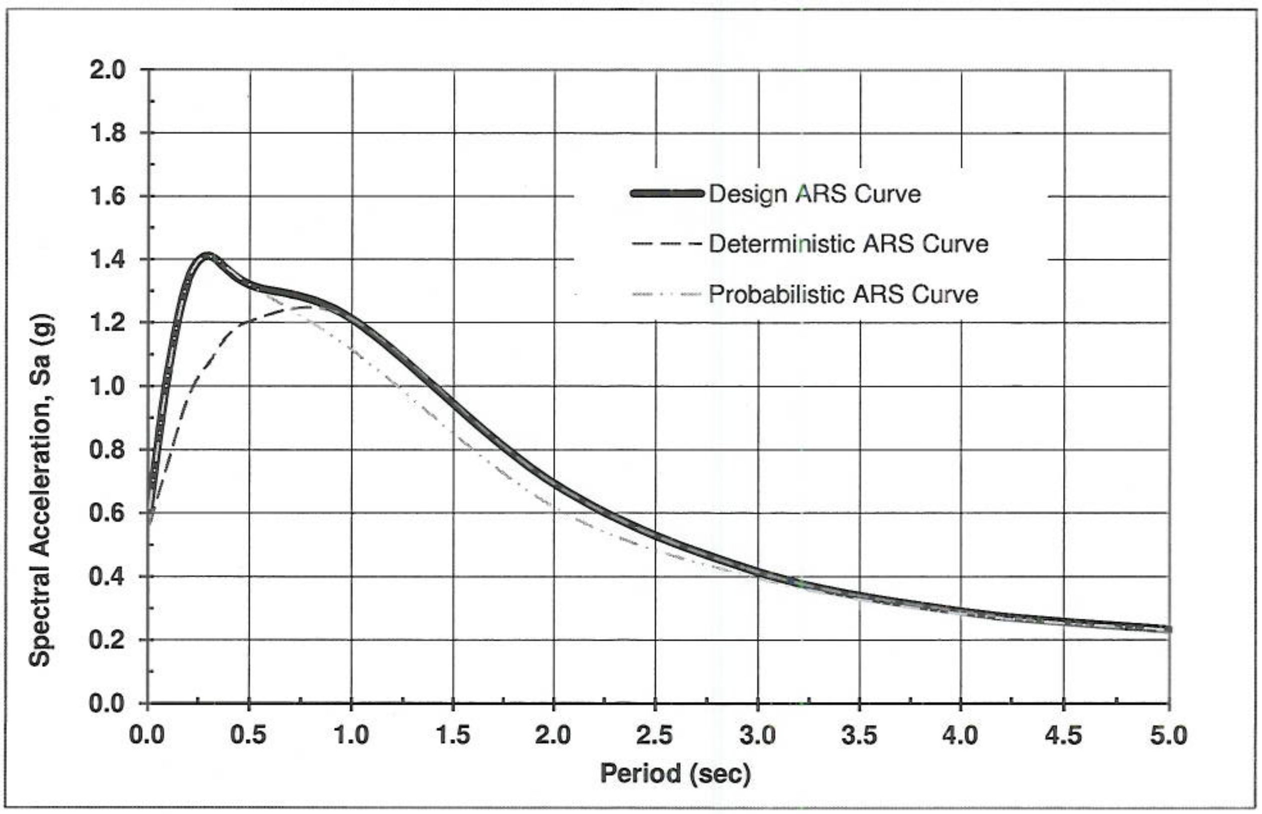
Notes: $R_X$ = Horizontal distance to the fault trace
$R_{JB}$ = Shortest horizontal distance to the surface projection of the rupture area
$R_{RUP}$ = Closest distance to the fault rupture plane

As shown in Figure 1, the design ARS curve is an envelope of deterministic and probabilistic acceleration response spectrum curves. The probabilistic ARS curve was developed with a ground motion return period of 975 year which is corresponding with 5% probability of exceedance in 50 years. The Next Generation Attenuation (NGA) was used for the deterministic ARS curve. The design Peak Ground Acceleration (PGA) has been evaluated as 0.62g from the design ARS curve.

**Table 5 – Design Spectral Acceleration for Special Design of SW 160, 204, and 205**

| Period (sec) | Spectral Acceleration (g) | | |
|---|---|---|---|
| | Deterministic | Probabilistic | Design |
| 0.010 | 0.564 | 0.617 | 0.617 |
| 0.100 | 0.759 | 1.040 | 1.040 |
| 0.200 | 0.965 | 1.337 | 1.337 |
| 0.300 | 1.072 | 1.410 | 1.410 |
| 0.500 | 1.204 | 1.319 | 1.319 |
| 1.000 | 1.211 | 1.114 | 1.211 |
| 2.000 | 0.693 | 0.619 | 0.693 |
| 3.000 | 0.415 | 0.395 | 0.415 |
| 4.000 | 0.290 | 0.282 | 0.290 |
| 5.000 | 0.223 | 0.232 | 0.232 |

**Figure 1 – Design ARS Curve for Special Design of SW 160, 204, and 205**



**Surface Fault Rupture Hazard Evaluation**

The site is not located within any CGS designated Earthquake Fault Zone (EFZ). Therefore, the site is not considered prone to surface fault rupture hazard and the possibility of surface fault rupture hazard at the bridge and soundwalls is considered very low.

**Liquefaction Potential**

Liquefaction is a phenomenon in which saturated, loose to medium dense sand and silt behave like a fluid when subjected to high intensity ground shaking. Liquefaction occurs when three general conditions exist: (1) shallow ground water (2) low-density, fine, sandy and/or silty soils and (3) high-intensity ground motion. The primary effects of liquefaction include sand boils, settlement and settlement-related downdrag to piles, lateral spreading and flow slides in areas with sloping ground. Cone penetration tests (CPT) performed in 2008 through 2011 were analyzed by URS Corporation to evaluate liquefaction potential for Segment 3 of I-5 Corridor Improvement Project. Based on the analysis results, the site is susceptible to liquefaction and a maximum vertical settlement of 3.5 inches is anticipated during the controlling seismic event.

## FOUNDATION RECOMMENDATIONS

As described previously in the beginning of the report, the special designs of soundwalls will be supported by 30 inch CIDH piles in order to cross over the bridge bent footing or underground RC box culverts.

In accordance with Caltrans LRFD Implementation Memo (December 2008), Load and Resistance Factor Design (LRFD) in AASHTO LRFD Bridge Design Specification (4$^{th}$ Edition) and California Amendment was utilized for geotechnical foundation design of the walls. The Office of Bridge Design-South 1 provided foundation information for the special designs. The detailed information of the foundations is shown in Table 6:

**Table 6 – General Foundation Information**

| Location | Design Method | Pile Type | FG Elev (ft) | Cut-off Elev (ft) | Pile Cap Size (ft) | | Permissible Settlement under Service Load (in) | Number of Piles |
|---|---|---|---|---|---|---|---|---|
| | | | | | B | L | | |
| SW 160 | LRFD | 30-inch CIDH | 80.4 | 78.5 | 3.0 | 56.6 | 1.0 | 3 |
| SW 204 | LRFD | 30-inch CIDH | 95.5 | 93.2 | 3.0 | 26.3 | 1.0 | 2 |
| SW 205 | LRFD | 30-inch CIDH | 95.0 | 93.0 | 3.0 | 69.5 | 1.0 | 4 |

The foundation loads for the three special designs are presented in Table 7:

**Table 7 – Foundation Design Loads provided by Structure Design**

| Location | Service-1 Limit State (kips) | | | Lateral Load (kips) |
|---|---|---|---|---|
| | Total Load | | Permanent Load | |
| | Per Support | Max Per Pile | | |
| SW 160 | 247 | 117 | 247 | N/A |
| SW 204 | 158 | 79 | 158 | N/A |
| SW 205 | 344 | 111 | 344 | N/A |

Note: N/A = Not available

| Location | Strength Limit State (kips) | | | | Extreme Event Limit State (kips) | | | |
|---|---|---|---|---|---|---|---|---|
| | Compression | | Tension | | Compression | | Tension | |
| | Per Support | Per Pile | Per Support | Per Pile | Per Support | Per Pile | Per Support | Per Pile |
| SW 160 | 308 | 146 | 0 | 0 | NA | NA | NA | NA |
| SW 204 | 196 | 98 | 0 | 0 | NA | NA | NA | NA |
| SW 205 | 431 | 139 | 0 | 0 | NA | NA | NA | NA |

Note: NA = Not applicable

Axial geotechnical pile resistance was calculated using only skin friction resistance. Pile end bearing was not considered for the resistance under the consideration of relatively large movement required for mobilizing the end bearing. The pile resistance was estimated using Static Analysis Method in AASHTO LRFD Bridge Design Specification ($4^{th}$ Edition) by employing the computer program Shaft 5.0 developed by Ensoft, Inc. The nominal resistance for each pile was calculated by dividing the load in strength limit state by a resistance factor of 0.7.

The liquefaction potential was not considered for foundation design of this structure because the anticipated maximum liquefaction settlement at this site is smaller than four (4) inches of maximum allowable settlement for this structure in the controlling seismic event.

As for design pile tip elevation for lateral load, this office was informed that the design pile tip elevation will be calculated and incorporated into pile data table by the Office of Bridge Design-South 1 if it is necessary.

### Table 8 – Foundation Recommendation

| Location | Pile Type | Cut-off Elevation (ft) | Service-1 Limit State Load (kips) per support | Total Permissible Support Settlement (inches) | Required Factored Nominal Resistance (kips) | | | | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Strength | | Extreme Event | | | |
| | | | | | Comp (φ=0.7) | Tension (φ=0.7) | Comp (φ=1.0) | Tension (φ=1.0) | | |
| SW 160 | 30-inch CIDH | 78.5 | 247 | 1 | 146 | 0 | NA | NA | (a-1) 48.5<br>(c) 66.0 | 48.5 |
| SW 204 | 30-inch CIDH | 93.2 | 158 | 1 | 98 | 0 | NA | NA | (a-1) 66.5<br>(c) 77.0 | 66.5 |
| SW 205 | 30-inch CIDH | 93.0 | 344 | 1 | 139 | 0 | NA | NA | (a-1) 64.5<br>(c) 77.0 | 64.5 |

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

### Table 9 – Pile Data Table

| Location | Pile Type | Cut-off Elevation (ft) | Nominal Resistance (kips) | | Design Tip Elevations (ft) | Specified Tip Elevation (ft) |
|---|---|---|---|---|---|---|
| | | | Compression | Tension | | |
| SW 160 | 30-inch CIDH | 78.5 | 210 | 0 | (a) 48.5<br>(c) 66.0 | 48.5 |
| SW 204 | 30-inch CIDH | 93.2 | 140 | 0 | (a) 66.5<br>(c) 77.0 | 66.5 |
| SW 205 | 30-inch CIDH | 93.0 | 200 | 0 | (a) 64.5<br>(c) 77.0 | 64.5 |

Notes:
1. Design tip elevations are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load, respectively.
2. Design tip elevation for lateral load will be provided and incorporated into pile data table by Structure Design.

## CONSTRUCTION CONSIDERATIONS

1. Temporary casing may be necessary for installation of the 30 inch cast-in-drilled-hole (CIDH) piles since there is relatively high caving potential at this site.
2. Encountering groundwater is anticipated during installation of CIDH piles at SW 160, and wet construction method may be required to avoid deterioration of the piles from groundwater.

If you have any questions or comments, please feel free to contact Chungkeun Lee at 213-620-2148 or Chi-Tseng (Ted) Liu at 213-620-2136.

Prepared by: Date: 07/11/2012

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## Appendix 1 – Generalized soil profile and Design strength parameters

### Special Design in SW 160

| Approximate Elev (ft) | Soil Type | $N1_{60}$ (Blow Counts) | Friction Angle (deg) | Undrained Shear Strength ($lbf/ft^2$) |
|---|---|---|---|---|
| +92.0 to 78.0 | Clayey sand (SC) | 38 | 37 | - |
| 78.0 to 71.0 | Sand with clay (SP-SC) | 22 | 33 | - |
| 71.0 to 66.0 | Clayey sand (SC) | 9 | 30 | - |
| 66.0 to 59.0 | Sand (SP) | 47 | 35 | - |
| 59.0 to 52.0 | Lean clay (CL) | 21 | - | 2000 |
| 52.0 to 46.0 | Sand with silt (SP-SM) | 42 | 35 | - |
| 46.0 to 42.0 | Silt (ML) | 21 | - | 1000 |
| 42.0 to 38.0 | Sand with clay (SP-SC) | 52 | 35 | - |
| 38.0 to 31.0 | Lean clay (CL) | 16 | - | 1200 |
| 31.0 to 12.0 | Sand with clay (SP-SC) | 70 | 40 | - |

**RAILROAD PRE-EMPTION OPERATION INFORMATION HANDOUT**

**ADVANCE WARNING RAILROAD PREEMPTION (USING RR1) WITH LIMITED SERVICE AND 75 SECONDS 2 STAGE PREEMPTION TIME**

1. WHEN THE TRAIN ENTERS THE TRACK CIRUCIT, THE TRAFFIC SIGNAL OPERATES NORMALLY FOR (42) FORTY TWO SECONDS, THEN THE YELLOW CLEARANCE BEGINS FOR THE PHASE ON GREEN INCLUDING "OLA" AND "OLB", UNLESS PHASE 2 & 5 ARE GREEN, IN WHICH CASE THEY STAY GREEN. PEDESTRIAN PHASES TURNS SOLID "DON'T WALK". R3-1 SIGNS ARE ACTIVATED.

2. FORTY THREE (43) SECONDS GREEN TRACK CLEARANCE(CLEAR 1) FOR PHASE 2 & 5 WITH RED INTERVAL FOR ALL OTHER PHASES AND ALL PEDESTRIAN SIGNALS SHOW SOLID "DON'T WALK" ELEVEN SECONDS (11) BEFORE THE END OF THIS TRACK CLEARANCE, RR SIGNAL FLASHES FOLLOWED BY LOWERING OF GATE FOR ROSECRANS AVENUE.

3. YELLOW CLEARANCE FOR PHASE 2 & 5 WHILE PEDESTRIAN SIGNALS ARE IN SOLID "DON'T WALK" INTERVAL.

4. THIRTY TWO (32) SECONDS GREEN TRACK CLEARANCE (CLEAR 2) FOR PHASE 8 & 3 WITH RED INTERVAL FOR ALL OTHER PHASES, AND ALL PEDESTRIAN SIGNALS REMAIN SOLID "DON'T WALK" ELEVEN (11) SECONDS BEFORE THE END OF THIS TRACK CLEARANCE, RR SIGNAL FLASHES FOLLOWED BY LOWERING OF GATE FOR BLOOMFIELD AVENUE.

5. YELLOW CLEARANCE FOR PHASE 8 & 3 WHILE PEDESTRIAN SIGNALS ARE IN SOLID "DON'T WALK" INTERVAL.

6. WHILE THE TRAIN IS ON THE TRACK CIRCUIT, LIMITED SERVICE SHALL BE IN EFFECT FOR PHASE 8 & PHASE 7.

7. WHEN THE TRAIN LEAVES THE TRACK CIRCUIT, PHASE 8 OR PHASE 7 WILL TURN YELLOW AND ALL RED. R3-1 SIGNS ARE DE-ACTIVATED.

8. SIGNAL RETURNS TO NORMAL OPERATION WITH GREEN INDICATION FOR PHASE 1 AND PHASES 5 AFTER THE TRAIN LEAVES THE TRACK CIRCUIT.

FOR CONTRACT NO. 07-215934

# INFORMATION HANDOUT

# **MATERIALS INFORMATION**

FOUNDATION REPORT FOR SOUNDWALLS 160, 204 AND 205

ROUTE 07-LA-5-PM 2.5/4.0

ADDED PER ADDENDUM NO. 1 DATED JULY 13, 2012

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5. YELLOW CLEARANCE FOR PHASE 8 & 3 WHILE PEDESTRIAN SIGNALS ARE IN SOLID "DON'T WALK" INTERVAL.

6. WHILE THE TRAIN IS ON THE TRACK CIRCUIT, LIMITED SERVICE SHALL BE IN EFFECT FOR PHASE 8 & PHASE 7.

7. WHEN THE TRAIN LEAVES THE TRACK CIRCUIT, PHASE 8 OR PHASE 7 WILL TURN YELLOW AND ALL RED. R3-1 SIGNS ARE DE-ACTIVATED.

8. SIGNAL RETURNS TO NORMAL OPERATION WITH GREEN INDICATION FOR PHASE 1 AND PHASES 5 AFTER THE TRAIN LEAVES THE TRACK CIRCUIT.