Geotechnical Engineering Structure Report
Final Design Submission
Retaining Walls
State Project 155-H025
Hartford, Connecticut

Vol. I

Prepared for:
URS Corporation
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Rocky Hill, CT 06067

Prepared by:

Geo**Design**, Inc.
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STATE OF CONN
No. 14435
LICENSED
PROFESSIONAL

September 20, 2010
w/ Addendum October 15, 2010

GEODESIGN
INCORPORATED

GEOTECHNICAL / CONSTRUCTION / ENVIRONMENTAL
ENGINEERS and SCIENTISTS

October 15, 2010
File 0380-004.0

URS Corporation
500 Enterprise Drive
Suite 3B
Rocky Hill, CT 06067

Attention: Robert Aloise, P.E.

Re: Retaining Wall Report Addendum
New Britain - Hartford Busway
State Project No. 155-H025

Dear Rob:

Geo**Design,** Inc. is providing this addendum to the Retaining Walls Report with the following recommendations for Retaining Walls 118 and 119:

<u>Wall 118 - Sta. 358+75 to Sta. 361+00 (similar to recommendations for Retaining Wall 108)</u>
Max Fill Height = 6 feet (per Peter Grandy, URS)
Nominal Bearing Resistance = 6 KSF
Bearing Resistance Factor = 0.45
Sliding Resistance Factor = 0.8
Estimated Settlement with Regular Fill: 2-inches
Estimated Settlement with Light Weight Fill: 1-inch

<u>Wall 119 - Sta. 393+75 to 395+75 (similar to recommendations for Retaining Wall 108)</u>
Previously Designated "RW-TBD"
Max Fill Height = 4 feet (per Table 7 of Retaining Wall Report)
Nominal Bearing Resistance = 6 KSF
Bearing Resistance Factor = 0.45
Sliding Resistance Factor = 0.8
Estimated Settlement with Regular Fill: 1.5-inches
Estimated Settlement with Light Weight Fill: 0.8-inch

Sincerely,

Marie G. Bartels, P.E.
Project Engineer

Ulrich La Fosse, P.E.
Project Manager / Reviewer

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Appendix:

Vol. I

1.0 GENERAL INFORMATION

1.1 General

This report summarizes the final design subsurface exploration program, inferred subsurface conditions, and geotechnical analyses; it also provides geotechnical engineering recommendations for foundation design and backfill and other special geotechnical requirements for the retaining walls for the proposed Busway in Hartford and West Hartford, Connecticut.

The Flatbush portion of Busway is State Project No. 63-643. The remaining Busway project is State Project No. 155-H025. The entire Busway project entails the design and construction of a 9.4-mile roadway connecting downtown New Britain and downtown Hartford. The Busway will be part of a dedicated Bus Rapid Transit System. The Busway will be adjacent to and west of the existing Amtrak railroad tracks.

The 63-643 and 155-H025 sections of the Busway begins at the intersection of Oakwood Avenue and the Amtrak railroad (Station 332+00), and ends at the intersection of Sigourney Street and Amtrak Railroad (Station 450+00). The alignment is shown in Figure 1 (Appendix 1, Vol. I.) The locations of the proposed retaining walls are shown on Figures within each wall-specific tab (appended).

URS Corporation is the Prime Designer for these sections of the Busway. Geo**Design,** Inc. (Geo**Design)** is the Geotechnical Subconsultant to URS.

Data and analyses, common to all retaining walls are included in this section. Additional wall-specific data and recommendations are provided for each wall or group of walls (typically in pairs) in separate sections (tabs) which follow this section.

1.2 Datum

All elevations referenced in this report are in feet and are based on NGVD 1929. The coordinates are based on Connecticut Coordinate System, NAD 1983.

1.3 Existing Conditions and Proposed Construction

The Busway alignment generally runs southwest to northeast. For this report the railroad and Busway are considered to run along a general south-north alignment, and the proposed Busway is to be constructed west of the tracks. The Busway base line Stationing increases from south to north.

Refer to the retaining wall-specific tabs (appended) for existing conditions and proposed construction in the vicinity of each proposed retaining wall location.

1.4 Design Criteria

Foundation design recommendations are based on AASHTO Load and Resistance Factor Bridge Design Specifications 2004, 3rd Edition with 2006 interims and Connecticut Department of Transportation Geotechnical Engineering Manual, 2005 Edition. Seismic design recommendations are based on AASHTO Load and Resistance Factor Bridge Design Specifications 4th Edition, 2007 (AASHTO LRFD) with 2008 Interims. Recommendations are also based on State of Connecticut Department of Transportation (ConnDOT) Standard Specifications for Roads, Bridges, and Incidental Construction, Form 816 (2004). American Society for Testing and Materials (ASTM) publications were followed as the reference standards for all field and laboratory tests applicable.

2.0 GEOLOGY

Published geologic data for this locale indicate that an Alluvial deposit overlies a Glaciolacustrine deposit, the prevalent surficial material in this area, below fill. A Glaciofluvial deposit and Glacial Till underlie the Glaciolacustrine deposit. These unconsolidated materials overlie bedrock of the Portland Arkose formation. These layers were formed in a bottom to top sequence. Thus, the shallower a layer the younger its geological age.

2.1 Alluvial Deposit

Alluvial deposits consist of sediments deposited by present day streams. This deposit is a non-continuous layer with a varying thickness. It consists of fine to medium grained Sand/Silt, with some Clay and little Gravel.

2.2 Glaciolacustrine Deposit

When the late Pleistocene ice sheet in New England retreated about fifteen thousand (15,000) years ago, the Glaciolacustrine deposit was formed in Glacial Lake Hitchcock. The Glaciolacustrine deposit in this area is distinctively featured by alternating layers of clay and silt. Each pair of clay and silt layers is called a “varve”, which corresponds to glacial lake deposit of a year: when the glacier melted, melt water streams brought soil particles into Glacial Lake Hitchcock. During the summer, a larger volume of water formed a more turbulent flow. This flow was capable of carrying silt particles (sometimes even larger particles) and settling them on the lake bottom. During the winter, when the volume of melt water decreased and frozen lake surface calmed the water, clay particles were deposited out of suspension. As a result of many years’ deposit, the “varved” structure dominated the Glaciolacustrine deposit in this region. The deposit could contain several hundred or even several thousand varves. The thickness of the varves is variable.

Although this deposit contains significant amount of Silt, the literature typically refers to the Glaciolacustrine deposit in this area as Varved Clay. Conforming to tradition, the term “Varved Clay” will be used in this report.

2.3 Glaciofluvial Deposit

Streams of melt water carried and deposited particles and formed this layer the before Varved Clay layer was deposited. The Glaciofluvial deposit consists of a heterogeneous mixture of sand, gravel, silt and clay in order of decreasing quantity.

2.4 Glacial Till

Glacial Till consists of a heterogeneous mixture of different sized particles. The composition of Till demonstrates a wide range of variation in particle size as well as in percentage of each size. Two extremes of these variations are stony till and clayey till. The former contains more than fifty percent of gravels, pebbles, cobbles and boulders. The latter consists of more than fifty percent of clay size particles.

2.5 Bedrock

The Portland Arkose formation, a sedimentary bedrock unit, is the dominating formation in this locale. Its texture ranges from coarse conglomerate to shale.

3.0 EXISTING GEOTECHNICAL INFORMATION

During the preliminary design phase (in 2003), Baker Engineering N.Y. (Baker) and their subcontractors, performed borings and laboratory tests.

3.1 General

Pilot Borings SB-56 through SB-87, SB-105 and RB-35 through RB-41 were drilled along the proposed Busway alignment (from Station 332+00 to Station 450+00).

3.2 Laboratory Test Data

Baker conducted the following laboratory tests on samples retrieved from the pilot borings: Moisture Contents, Atterberg Limits, Sieve Analyses, Hydrometer Analyses, Incremental Load Consolidation Tests (without unload-reload procedure), Triaxial Tests, and Corrosivity Tests. These tests are summarized in Table 3 (Appendix 2, Vol I). Details and interpretation of each test are provided in Vol. III and in Section 5 of this report, respectively.

4.0 SUBSURFACE EXPLORATIONS

Geo**Design** conducted additional subsurface explorations during final design. Details of these explorations are described in this section.

4.1 Test Borings

Geo**Design** coordinated the services of New England Boring Contractors of CT, Inc. (NEBC) to perform Standard Penetration Test (SPT, ASTM D1586) borings at the site. Geo**Design** also coordinated the services of ConeTec to perform six Cone Penetration Tests (CPT, ASTM D3441) in the vicinity of the proposed retaining walls.

Boring and CPT locations were staked out by Connecticut Department of Transportation (ConnDOT). Offsets (if any) from staked out locations were measured in the field. ConnDOT survey crews recorded the locations and elevations of the borings by surveying the as drilled boring locations. Selected structure, retaining wall, and roadway borings, as well as CPT locations are shown on the Figures included in each wall-specific tab (appended). Corresponding logs are included in Vol. II.

4.2 Field Vane Shear Testing

Eight field vane shear tests (ASTM D2573) were performed along the proposed Busway alignment. These data are included in Table 6 (Appendix 2, Vol. I).

4.3 Observation Wells

Eight observation wells were installed along Busway alignment. Observation well readings are summarized in Table 1 (Appendix 2, Vol. I).

5.0 LABORATORY TESTING RESULTS AND INTERPRETATION

Geo**Design** performed laboratory tests to obtain several important engineering properties of the Varved Clay, including compressibility and strength. These data were also used to verify field classifications, determine material drainage properties, and assess frost susceptibility. Of particular importance was the need to conclusively determine the stress history of the Varved Clay stratum because preliminary design findings provided conflicting results in this critical matter. A SHANSEP approach was also adopted to allow correlation of soil parameters at this intersection with other locations along Busway alignment. More limited testing was also performed on other materials. Test results are included in Vol. III.

5.1 Constant Rate of Strain (CRS) Consolidation Tests

Constant rate of strain (CRS) consolidation testing (ASTM D4186) was developed in the early 1970's and since has gradually become more popular because it generates continuous data instead of isolated data points as increment load (IL) testing. In the past CRS tests were significantly more costly than IL testing because the continuous load steps and constant strain control both require close monitoring. As computers and automation technology have improved and become more common, CRS test has become more widely accepted as the state of the art methodology for consolidation test. Seventeen CRS tests on soil samples taken along the Busway alignment were selected for this report. Results are included in Table 2 (Appendix 2, Vol. I) and in Appendix 1, Vol. III.

5.2 Incremental Load (IL) Consolidation Tests

Increment load (IL) consolidation testing (ASTM D2435) has been used much longer than CRS testing. As a result, the testing equipment for IL testing is more widely available and the IL tests are easier to perform than CRS tests. IL tests were performed by three testing labs: TestCon, GTX and UMass. Results from these three laboratories are comparable.

The ability to predict maximum past pressures using IL tests depends in part on the selection of the load increments. Loading increments of 4,000 and 8,000 psf were used and at this site the predicted maximum past pressures fall between these values. As a result, the predicted maximum past pressures end up converging around 4,000 psf. Thus, the IL tests yield lower bound (conservative) predicted maximum past pressures as compared to CRS tests. Test results are included in Table 2 (Appendix 2, Vol. I) and in Appendix 1, Vol. III.

5.3 Direct Simple Shear (DSS) Tests

Direct Simple Shear (DSS) tests (ASTM D6528) can simulate shear forces acting horizontally on soil. Because of the horizontal-layered structure of Varved Clay, the shear strength along the varves is lower than the shear strength across the varves. Thus, DSS testing on Varved Clay samples often yield the most conservative shear strength values compared to Triaxial Compression (TC) or Triaxial Extension (TE) tests.

Geo**Design** retained UMass to perform the DSS tests on four soil samples taken with Shelby tubes in Boring SB-01-1 and SB-01-4. DSS test results are included in Appendix 2, Vol. III. The tests were performed using the Stress History And Normalized Soil Engineering Properties (SHANSEP) Method recommended by Ladd & DeGroot in their 2003 article *"Recommended Practice for Soft Ground Site Characterization: Arthur Casagrande Lecture"*.

A SHANSEP correlation was developed from these test results. The expression, $S_{u,DSS}/\sigma'_{vc} = 0.17(OCR)^{0.75}$, provides a way to estimate shear strength in the Varved Clay layer by correlating S_u as determined by DSS testing with its over consolidation ratio (OCR) as determined by consolidation testing with the effective stress (σ'_{vc}). Plots depicting the shear strength and normalized shear strength ratio developed from the referenced laboratory tests are included as Chart 1 (Appendix 2, Vol. I).

A tabulation of the resulting calculated minimum shear strength, (S_u), for each tube sample that was tested for consolidation is included in Table 2 (Appendix 2, Vol. I).

Another DSS test taken in a sample AARW-10-40 from nearby Amtrak Access Road Project is also included for reference. But this test is not included in the analysis mentioned above.

5.4 Unconfined Rock Compression Tests

Unconfined Compression Tests (ASTM D2938) provide an indication of intact rock core strength. Corrected strengths ranges from 4,082 psi to 6,374 psi. The test results are included in Appendix 2, Vol III.

5.5 Atterberg Limits

Atterberg Limits (ASTM 4318) provide the Liquid Limit (LL), the Plastic Limit (PL) and the Plasticity Index (PI) of cohesive soil samples. These tests can characterize cohesive soils and provide a reference to compare soil properties at different depths and locations.

Forty-eight Atterberg Limit tests were performed. The LL ranges from 28% to 62%, the PL ranges from 19% to 50%, and the PI ranges from 3% to 31%. Data are included in Table 4 (Appendix 2, Vol. I) and in Appendix 3, Vol. III.

Baker performed 65 Atterberg Limits in 2003 on samples included in this report. The Liquid Limits (LL) range from 15% to 64%, the Plastic Limits (PL) range from 14% to 29%, and the Plasticity Indices (PI) range from 0% to 41%. These data are provided in Table 3 (Appendix 2, Vol. I) and in Appendix 3, Vol. III. These results correlate well with the above listed results. Additional Atterberg Limits were performed in samples taken from pilot borings farther away from the proposed structure. These results are also included in Appendix 3, Vol. III for reference.

5.6 Moisture Contents

Moisture contents (ASTM 2216), like Atterberg Limits, provide an easy way to characterize and compare cohesive soils. These tests were performed in larger numbers vertically and horizontally throughout the Varved Clay layer to rapidly and economically determine vertical and horizontal trends of soil property variations. Geo**Design** performed one hundred-sixteen moisture content tests in 2008. The moisture contents range from 25% to 70%. Baker performed 59 moisture content tests in 2003, on samples included in this report. The moisture contents range from 20.9% to 66.1%. Test results are included in Appendix 3 Vol. III.

5.7 Minus No. 200 Sieve Tests

Minus No. 200 Sieve Test (ASTM D1140) provides a means of determining the percentage of soil particles finer than 75 um (clay and silt particles).

Forty-three SPT jar samples taken from the Fill layer were washed with No. 200 sieve and weighted according to ASTM D1140 Method A. The results indicate a silt/clay content range of 4.2% to 99.4%. The data are included in Appendix 4, Vol. III.

5.8 Sieve Analyses

Sieve Analyses (ASTM C136) provide the gradation of soil particles larger than the 75um (or No. 200 sieve). The results are useful for evaluating reusability of existing soils and calibrating visual field description of soil samples.

Sieve analyses were performed in both the preliminary design and final design phase. In 2003, Baker performed 94 sieve analyses tests on samples along the proposed Busway alignment. Results are included in Appendix 4, Vol. III.

In 2008, Geo**Design** performed sieve analyses on 19 samples in borings along the proposed Busway alignment. Results are included in Appendix 4, Vol. III. These results indicate that shallow soils are granular and include from 5.1% to 32.2% silt.

5.9 Hydrometer Analyses

In 2008, hydrometer analyses (ASTM D422) were performed on 16 jar samples and nine tube samples of the Varved Clay layer. The results showed a fairly consistent pattern in gradation. Data are included in Table 5 (Appendix 2, Vol. I) and in Appendix 4, Vol. III.

In 2003, Baker performed hydrometer tests on 60 samples from this area. These data are included in Table 3 (Appendix 2, Vol. I) and in Appendix 4, Vol. III.

5.10 Corrosivity Tests

In 2008, pH and Sulfides tests were performed to estimate the corrosion potential of granular fill soils. Ten pH tests and ten Sulfide tests were performed in samples taken from Borings: SB-012, SB-01-3, SB-01-4, SB-02-1, SB-02-2, SB-02-3, SB-03-2, and SB-03-3. No Sulfide was detected in these samples. The pH values range from 3.7 to 7.5, with and average value of 5.86. Comparing to neutral pH value of 7, the average pH value indicates slightly acidic soils. Test results are included in Appendix 5, Vol. III.

In 2003, Baker Engineering performed corrosivity tests on four soil samples. One sample was tested for pH values, resistivity, sulfate content, sulfide content and chloride content. The other one was tested for pH values and resistivity only. The pH values vary from 7.0 to 7.7. The average pH is 7.35. Resistivity values ranged from 0.013 to 0.014 megohm-cm. A summary of the tests is included in Appendix 5, Vol. III.

6.0 SUBSURFACE CONDITIONS

6.1 Subsurface Profiles

One or more subsurface profiles are included in each wall-specific tab (appended). These profiles depict the generalized subsurface conditions at each wall or group of walls based on the pre-existing and recent subsurface exploration data. The legend for the subsurface profiles is included as Figure No. 2, Appendix 1.

Despite differences in subsurface conditions at each wall or group of walls, geologic conditions along the alignment are quite similar from wall to wall. The soil and rock profiles can be generalized as follows:

- **Top Soil/Asphalt** - 0 to 0.5 foot thick;
- **Fill -** 0 to 34 feet thick;
- **Silt/Fine Sand (Alluvium Deposit) -** 0 to 11 feet thick;
- **Varved Clay (Glaciolacustrine Deposit)** - 60 to 120 feet thick;
- **Silt/Fine Sand (Glaciofluvial Deposit) -** 0 to 45 feet thick;
- **Glacial Till** - 10 to 25 feet thick;
- **Bedrock (Siltstone/Shale)** -110 to 160 feet deep.

The **Fill** consists of loose to very dense, poorly graded (fine to medium) Sand, trace to some Silt, and (where present) some fine to coarse Gravel, little Asphalt fragments, trace Ash, trace Cinders, trace Brick/Concrete fragments, and trace Organic Fibers.

Most of the soil samples indicate that the Fill layer is medium dense, poorly graded and widespread.

The **Silt/Fine Sand** layer is erratic and generally medium dense. The layer typically consisted of loose to dense Silt and/or fine Sand. SPT N-values indicate that most of the samples have a medium density.

The **Varved Clay** layer was encountered in borings advanced sufficiently deep to penetrate the fill and Silt/Fine Sand layer. The stiffness of the Varved Clay stratum generally decreases toward the middle of the layer. Except for the desiccated zones, the SPT “N” values typically range from Weight of Rod (WOR), to Weight of Hammer (WOH), to 2, indicating a very soft consistency.

The **Silt/Fine Sand (Glaciofluvial Deposit)** underlies the Varved Clay. This stratum consists of medium dense to very dense fine Sand and Silt and with a thickness of up to 45 feet.

Glacial Till varies from 10 to 25 feet. SPT “N” values indicate the density of this layer ranges from dense to very dense.

Bedrock (Shale) depth ranges from 110 to 160 feet below the ground surface. Rock cores were taken in selected borings to confirm and characterize bedrock. Rock Quality Designation (RQD) values range from 0 to 53, indicating very poor to fair quality generally improving with depth.

6.2 Groundwater

Stabilized readings made in the observation wells indicate groundwater levels at approximately 2 to 14 feet below ground surface. Groundwater conditions will vary depending on factors such as temperature, season, precipitation, construction activity and other conditions, which may be different from those at the time of these readings.

7.0 ANALYSES AND DESIGN RECOMMENDATIONS

7.1 Settlements of Embankments and Wall Backfill

7.1.1 Magnitude of Settlements

Estimated consolidation settlements of the proposed retaining walls resulting from compression of the Varved Clay stratum are summarized in Table 7, (Appendix 2, Vol I). A recompression ratio of 0.04 was used to estimate consolidation settlements because none of the proposed stresses imparted by the wall and embankments will exceed the estimated maximum past pressure of the Varved Clay stratum. Predicted consolidation settlements (using conventional 125pcf fill) range from approximately 1 to 9 inches. These are significant.

7.1.2 Rate of Settlement

Because the proposed embankments will be relatively narrow (about 10 to 80 feet) as compared to the thickness of the Varved Clay (80 to 120 feet), and because of the anisotropic properties of Varved Clay, horizontal drainage will greatly affect the rate of consolidation.

In estimating the rate of consolidation, we used *“Field Consolidation of Varved Clay”*, a report by Professor Richard P. Long, Professor Kent A. Healy and Mr. Peter J. Carey from University of Connecticut. Figure 13 of this report depicts the field-measured apparent coefficient of consolidation for different loading geometries quantified as the ratio of the Varved Clay Thickness and to the Embankment Width (dimension ratio). This figure is reproduced as Chart 2, Appendix 2, Vol I. For a dimension ratio of one, the field-measured apparent coefficient of consolidation is 4 ft^2/day. We conservatively chose this value because most embankment widths are narrower than the thickness of clay. As a result, the apparent coefficient of consolidation is no less than 4 ft^2/day. Consolidation rates are summarized in Table 7 (Appendix 2, Vol I) for all proposed walls.

7.2 Retaining Wall Foundations and Backfill

The following design options were considered for the retaining wall foundations and backfill.

7.2.1 Foundation and Backfill Options

As discussed in Section 7.1 and as shown on Table 7, we predict significant consolidation (delayed) settlements in the vicinity of and/or behind the proposed retaining walls.

A significant factor which controls the selection of shallow vs. deep foundations for the retaining walls is the consolidation characteristics of the compressible Varved Clay layer. This affects predicted differential settlement between pile-supported abutments and retaining walls as well as total settlement of the walls, and differential settlement along the walls. Another factor is the adverse impact of wall backfill and wall loading in imparting new stresses on the varved clays in the vicinity of existing foundations.

In order of increasing effort and economic impacts, we considered the following options for support of retaining walls and retaining wall backfill:

- Normal shallow foundations (Spread Footings) and normal (125 pcf) backfill.
- Normal shallow foundations (Spread Footings) and lightweight granular (60 pcf) backfill.
- Deep Foundations (piles) and normal (125 pcf) backfill.
- Deep Foundations (piles) and lightweight granular (60 pcf) backfill.
- Deep Foundations (piles) and lightweight granular (60 pcf) backfill, with waiting periods.
- Deep Foundations (piles) and lightweight granular (60 pcf) backfill, with wick drains and waiting periods.
- Spread Footings and lightweight GeoFoam (1.8 pcf) backfill, with over-excavation to a depth of three feet to offset loading from the upper three feet of granular fill (required as a separation layer) and roadway base / pavement.

We also considered relieving platforms (pile supported structural slabs) in lieu of GeoFoam to eliminate new stresses that can cause unacceptable down-drag of existing piles. Preliminary cost comparisons between the relieving platforms and GeoFoam solution revealed that the latter option is more economical and quicker to construct. Thus, relieving platforms were not pursed further.

7.2.2 Criteria for Selection of Wall Foundations and Backfill

At locations where proposed fill thickness will not exceed about two to three feet, very small settlements will result. Retaining walls at these locations should be supported on shallow spread footings and be backfilled with regular fill (125 pcf)

In other areas, where the fill thickness is greater than three feet, large predicted wall settlements will preclude the use of spread footings and require special requirements to limit settlements to acceptable levels.

To determine the need for special requirements, we assumed a maximum allowable wall consolidation settlement of approximately one inch, and a maximum roadway pavement settlement (behind the walls) of approximately two inches.

7.2.3 Foundation Type and Backfill Recommendations

Recommended wall foundation types and special requirements are provided in on Table 7 (Appendix 2, Vol. I). Table 7 includes four sections as follows:

- Wall information
- Retaining wall settlements and time rates
- Roadway settlements and time rates
- Recommendations

Wall information (first five columns) includes retaining wall number, baseline, stationing and average fill height.

Retaining wall settlements and time rates are included in columns 6 through 9. We assume walls supported on spread footings. Columns 8 and 9 provide the percentage consolidation and the time required to limit wall settlement to about one inch. We prepared these settlement and rate of settlement calculations to determine which walls require deep foundations and/or special requirements.

Predicted roadway pavement settlements and time rates assuming regular and lightweight fill are included in columns 10 through 13. Columns 12 and 13 provide the percentage consolidation and the time required for pavement settlement to about two inches.
Recommendations are provided in columns 14 through 18. These include recommended wall foundation types (column 14), and special requirements for wall backfill, waiting periods or wick drains (columns 15 and 16).

Note that we estimate that pile-supported retaining walls will settle about 1/8 to 1/4 inches due to elastic compression of the piles.

In some cases, the recommended minimum waiting period (column 16) exceeds one month. Based on discussions during a working design meeting (with URS, ConnDOT and GeoDesign) there was consensus that the project schedule will likely not be able accommodate lengthy waiting periods. At that meeting, the risk that the actual rate of consolidation may be slower than predicted was also discussed. As a result, it was decided to employ measures to accelerate the consolidation and shorten the waiting periods. This should be accomplished by installing wick drains at the locations listed in Columns 17 and 18 of Table 7 under "alternate fill recommendations". In addition to reducing the waiting period, the use of wick drains will also reduce the risks associated with a slower actual consolidation rates as compared to theoretical predictions.

7.3 Retaining Wall Types

We recommend using cast-in-place reinforced concrete for walls supported on piles and for walls supporting city streets, e.g. Flatbush Ave. For walls supported on spread footings and constructed along the Busway alignment, we recommend use of proprietary walls or cast-in-place walls.

7.4 Pile Foundations

Refer to the retaining wall-specific tabs (appended).

7.5 Parameters for Spread Footing-supported Walls

Refer to the retaining wall-specific tabs (appended).

7.6 Fill and Backfill Design Parameters

Refer to the retaining wall-specific tabs (appended).

7.7 Seismic Design

Refer to the retaining wall-specific tabs (appended).

7.8 Drainage

Refer to the retaining wall-specific tabs (appended).

7.9 Wall Stability

Refer to the retaining wall-specific tabs (appended).

8.0 CONSTRUCTION RECOMMENDATIONS

Refer to the retaining wall-specific tabs (appended).

9.0 SPECIAL PROVISIONS

Refer to the retaining wall-specific tabs (appended).

10.0 LIMITATIONS

This report is subject to the limitations attached in Appendix 3, Vol. I.

INSERT TAB 101 and 104 HERE

Retaining Walls 101 and 104

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 101 and 104

Attached Figures:

RETAINING WALLS 101 and 104

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

These two walls will retain the approach embankments west of the Flatbush Ave. Bridge Over Busway. Flatbush Avenue currently intersects the two Amtrak railroad tracks at an at-grade crossing. The existing grade gradually slopes downward from Elevation (Elev.) 74 (west) to Elev. 70 (east).

Overhead utilities exist along the south side of Flatbush Avenue and along the west side of Amtrak's right-of-way, north of Flatbush Avenue. A buried fiber optic cable is also present along the west side of the tracks, within the Amtrak right-of-way. These utilities and related appurtenances will need to be relocated and/or replaced to accommodate the construction of these walls. Amtrak railroad crossing lights, gates and signalization exist along both sides of the existing at-grade crossing on Flatbush Avenue. These utilities will also be removed since they will no longer be required.

Existing utilities, including water, gas and sewer, extend under and/or near to Flatbush Avenue. These will be affected by embankment fill up to 26 feet in height. These utilities are to remain until new utilities are constructed along the new alignment.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

7.4.1 Pile Lengths

Estimated pile lengths at each proposed pile-supported retaining wall are included in Table 8 (Appendix 2, Vol I). We recommend an additional 20 feet be added for test pile lengths. We then recommend the contractor use the information from the test piles to determine the order length.

7.4.2 Down-drag Loads

Down-drag load is an important factor in pile design. Wall backfill and embankment loading will result in compression of the Varved Clays. Significant downdrag loads will be imparted to the piles.

7.4.3 Down-drag Load Reduction

Bitumen is very effective as a coating to reduce the friction between the soil and piles. A one-millimeter thick bitumen coating is sufficient to significantly reduce the down-drag force. The bitumen coating need not be applied to the section of the pile below the bottom of the Varved Clay stratum. We estimate that the down-drag load on a bitumen-coated pile will be 10 percent of the downdrag force on an uncoated pile (90% reduction). We estimate that bitumen coating of the piles will add approximately 10 percent to the pile's material cost. Thus, we recommend bitumen coating for the piles.

Precautions must be taken to prevent damage the bitumen coating with extreme temperatures and while driving through granular soils. To avoid damage to the coating, we recommend that a hole be pre-drilled through the surficial Fill and Silt/Sand layers to the top of the Varved Clay.

7.4.4 Corrosion Protection of Steel Piles

As noted in Section 5.10, pH values of 3.7 to 7.7 indicate some corrosive potential. However, due to the recommendation to use of bitumen coating, which will provide some protection from corrosion, we do not recommend using a corrosion allowance for the steel piles.

7.4.5 Pile Type and Size Selection

To limit down-drag forces, we recommend steel piles because compared to piles of other materials, steel piles typically provide higher strength, smaller perimeter, and smaller friction coefficients.

ConnDOT has requested that pile tip stresses not exceed 24 ksi. We therefore recommend end-bearing, bitumen coated, Grade 50 steel HP-Piles, with a maximum tip stress of 24 ksi. We further recommend pile tip reinforcement with integrally cast cutting teeth (or similar) be used. The following table provides nominal compressive resistances, down-drag loads, and nominal lateral capacities for a selection of HP pile sections. The down-drag loads are based on bitumen coated piles and must be added to the abutment load when determining the required number of piles. Nominal lateral capacities are based on a predicted lateral deflection of 0.6 inches.

Pile Selection	Nominal Compressive Resistance (kips/pile)	Design Down-drag Load (kips/bitumen coated pile)	Nominal Lateral Capacity (kips/pile)
HP 12x53	372	45	20
HP 12x74	523	45	20
HP 14x89	626	50	25
HP14x117	825	50	25

We recommend a resistance factor for compression, ($Ø_C$), of 0.6 (for good driving). Resistance factors for the service limit state shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. Resistance factors for the extreme limit state shall also be taken as 1.0, except for uplift resistance of piles, where resistance factor shall be taken as 0.8. Refer to Section 7.4.9 for use of resistance factors during pile testing.

Although larger piles are preferred to carry vertical load efficiently, four pile sizes are provided, for walls where the horizontal loading will control. In this case the smaller pile sizes may be more efficient overall.

The down-drag load must be added to the wall load when determining the required capacity and number of piles.

7.4.6 Pile Batter

Batter piles may be used to supplement the recommended lateral capacity of vertical piles if needed. We recommend a maximum pile batter of 1H:4V. In addition, if batter piles are used, the lateral capacity of piles (excluding the batter component) must be reduced to a maximum ultimate lateral capacity of 3 kips per pile.

7.4.7 Pile Spacing

In no case should the piles be spaced closer than three pile diameters. Pile group reduction factors, as applicable must be applied in accordance with AASHTO LRFD (2006 interims) Section 10.7.2.4, Table 10.7.2.4-1, and Figure 10.7.2.4-1.

7.4.8 Pile Splicing

Due to pile length, shipping, and handling constraints, piles will require at least one splice. Splices shall be made using pre-approved pre-fabricated splice connectors welded to provide the design pile vertical and lateral capacity. Splices shall not be the allowed within 15 feet of the pile cut-off and splices between adjacent piles shall be staggered at least 5 feet vertically and should conform to Form 816 7.02.03.

7.4.9 Pile Load Testing

We recommend the use of PDA testing, which can be completed quickly. We recommend one test pile be tested at each of these retaining walls. The pile load testing resistance factor for PDA Testing ($Ø_{dyn}$) is 0.65. The test pile selection should be based on a successfully tested indicator pile driving records, considered in relation to the test boring data, as determined by the Geotechnical Engineer.

Preliminary installation criteria for the piles should be based on wave equation analysis employing the characteristics of the pile type, soil conditions, and pile driving hammer and cushions proposed by the Contractor. This installation criteria analysis may be performed by Geo**Design,** or by the Contractor’s engineer and submitted for review.

Production pile installation criteria should be based on the passing of a successfully tested indicator pile.

7.4.10 Pile Supported Wall Settlement

Settlement of pile-supported retaining wall is expected to range from 1/8 to 1/4 inches and will occur largely during wall construction.

7.5 Parameters for Spread Footings-Supported Walls

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick granular fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls.

7.6 Fill and Backfill Design Parameters

7.6.1 Regular Fill

For design of the walls backfilled with regular fill (e.g. not lightweight), we recommend the following static design parameters:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, phi = 34°

7.6.2 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.6.3 Wick Drain Design Parameters

Recommended design parameters for wick drains, where used to accelerate the rate of consolidation of the Varved Clay stratum under proposed embankment loads, are as follows:

- Drain length from existing ground surface (after pavement/topsoil/subsoil removal) to approximate bottom of Varved Clay layer, See following table.
- Triangular Drain Spacing = 8 feet.
- Minimum thickness of Pervious Structure Fill or Drainage Sand layer above top of wick drains = 12 inches

Retaining Wall No.	Roadway Baseline	From Station	To Station	Width of Wick Drain Area (ft)	Estimated Top El. Of Wick Drain (ft)	Estimated Bottom El. Of Wick Drain (ft)	Estimated Wick Drain Length (ft)
RW-101	Flatbush Ave.	1400	1500	100	75	-30	105
RW-101	Flatbush Ave.	1500	1550	100	75	-30	105
RW-104	Flatbush Ave.	1400	1550	100	75	-20	95

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer’s approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of “unsuitable” materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a distance of over 15 feet from railroad tracks, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Vibrations from pile driving may impact the tracks and nearby utilities. See Section 8.5 below for recommendations regarding the utilities. We recommend that other structures be surveyed prior to construction and closely monitored. The threshold/action criteria should be defined and coordinated in advance with Amtrak. The railroad tracks should be monitored for vibration in accordance with Amtrak requirements.

8.5 Monitoring of Utilities

Existing utilities are present at Flatbush Ave. where a new bridge is to be constructed. Refer to specific Structure Layout for Design Reports for details.

8.6 Monitoring of Amtrak Railway Tracks

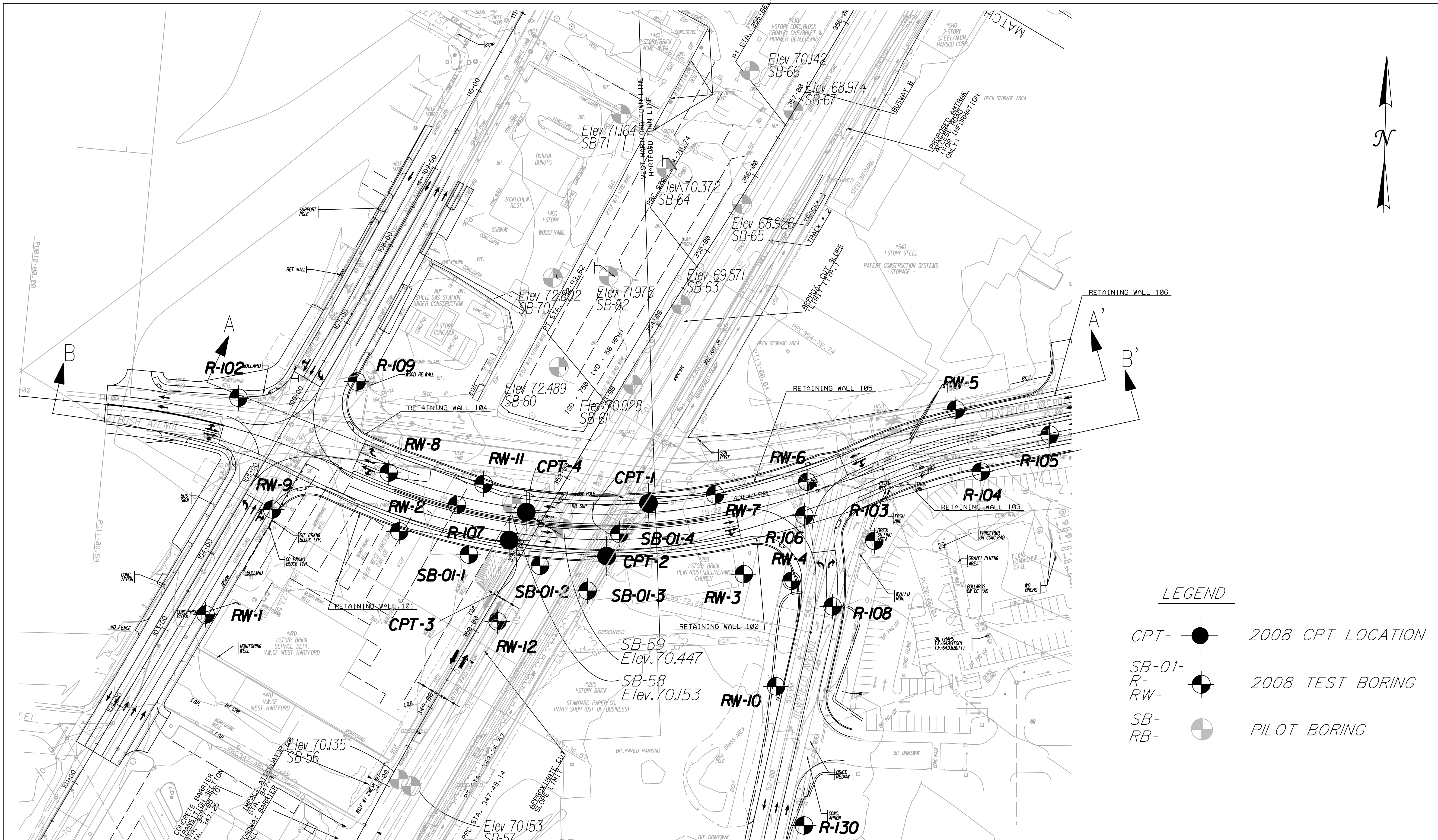
Two active railway tracks extend perpendicular to these proposed retaining walls. They are over 15 feet east of proposed retaining walls. We recommend that monitoring points be established on both tracks at 50-foot intervals along these retaining wall embankments since they need Special Requirements B or C (Column 15 or 17, Table 7, Appendix 2, Vol I).

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address bitumen coating of piles, pile splicing, pile testing, and lightweight fill.



DESIGNED BY	MGB
DRAWN BY	SMC
CHECKED BY	MGB
APPROVED BY	ULF

NO.	DATE		DRWN.	CHKD	APPVD

REVISIONS

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

GEODESIGN INCORPORATED
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS
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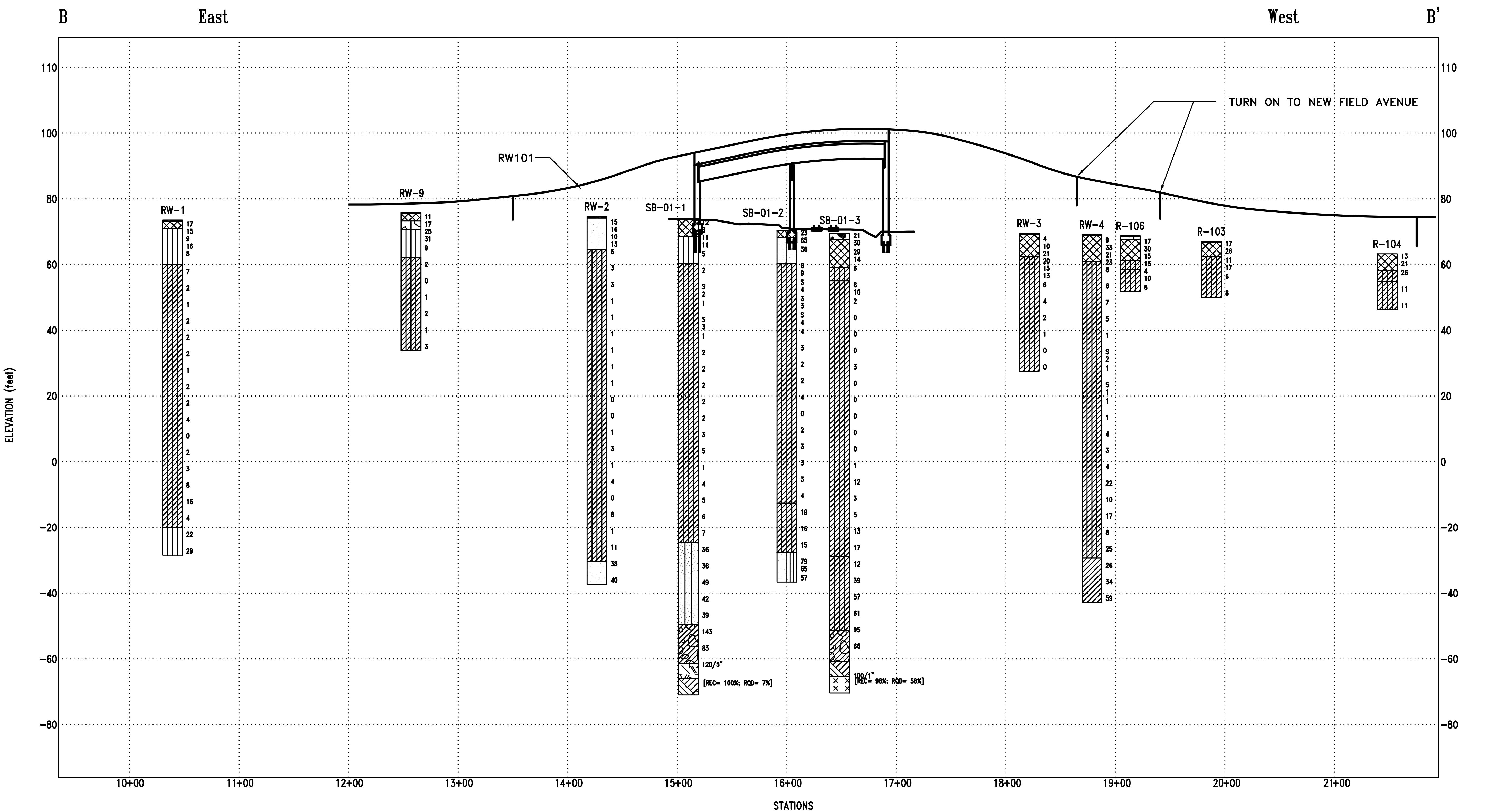
DWG. TITLE
RETAINING WALLS 101/104
AS-DRILLED EXPLORATION
LOCATION PLAN

PROJECT
NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

FILE NO. 380-04

SCALE 1" = 100' DATE 5/26/09

FIGURE NO. 1



GEODESIGN INCORPORATED

Geotechnical Engineers and Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 90
Horizontal Scale (feet)
Vertical Exaggeration: 3x

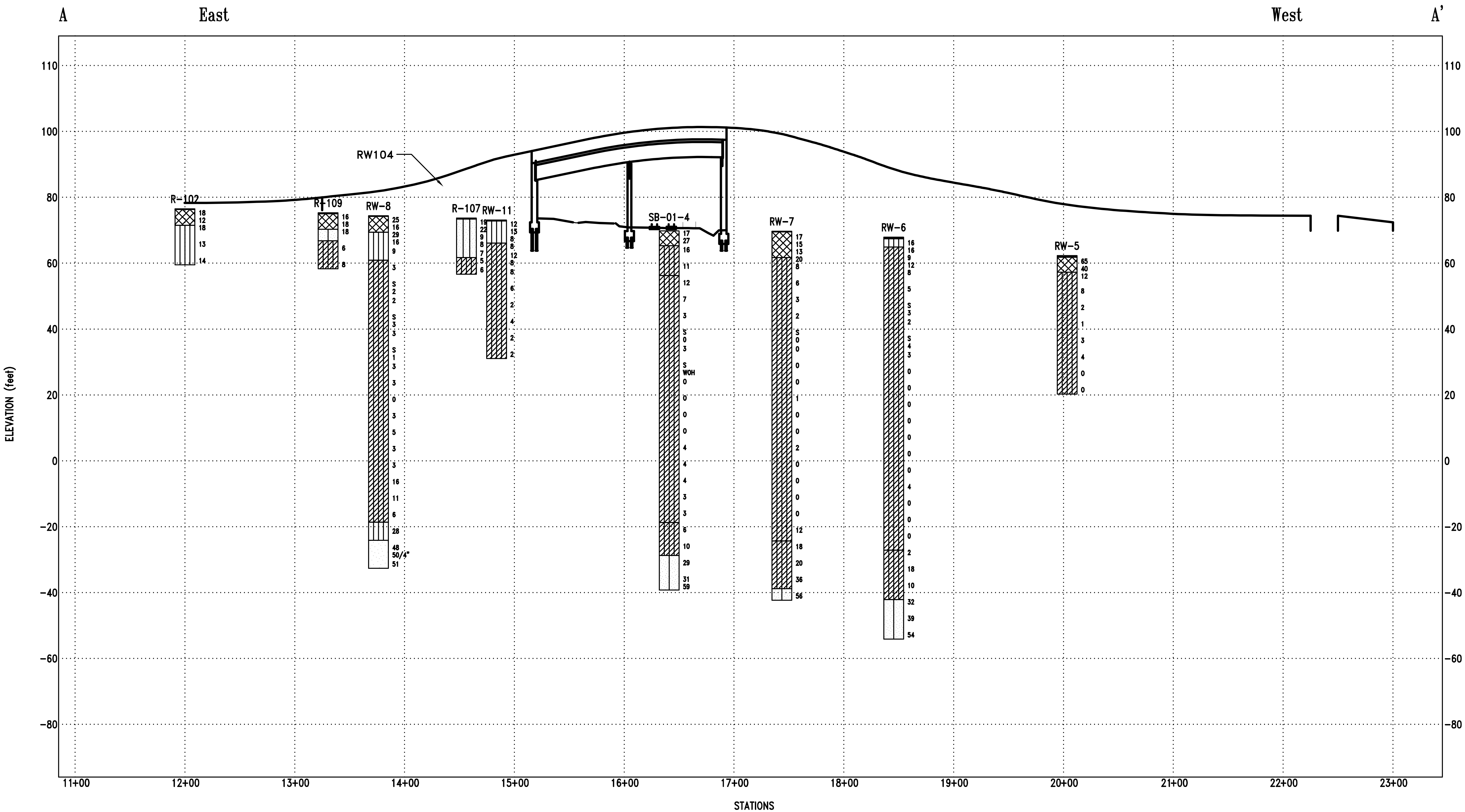
Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.

SUBSURFACE PROFILE B-B'

Retaining Wall RW101
New Britain - Hartford Busway /
State Project No. 155-H025

Date: 5/26/09	Drawn By: MBF	Reviewed By: MGB	File No.: 0380-004.0	Figure No.: 2

M:\CL\0380 BUSWAY\04 HTFD SOUTH\gINT\Flatbush Ave (1-19-09)\profile b.dwg



GEODESIGN
INCORPORATED
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984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 90

Horizontal Scale (feet)

Vertical Exaggeration: 3x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.

SUBSURFACE PROFILE A-A'

Retaining Wall RW104
New Britain - Hartford Busway /
State Project No. 155-H025

Date: 5/26/09	Drawn By: MBF	Reviewed By: MGB	File No.: 0380-004.0	Figure No.: 3

M:\CL\0380 BUSWAY\04 HTFD SOUTH\gINT\Flatbush Ave (1-19-09)\profile a.dwg

INSERT TAB
102, 103, 105, and ~~106~~
HERE

Retaining Walls 102, 103, and 105

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 102, 103, and 105

Attached Figures:

RETAINING WALLS 102, 103, and 105

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

These three walls will retain the approach embankments east of the proposed Flatbush Ave. Bridge Over Busway, and align mainly with Flatbush Avenue. Flatbush Avenue currently intersects the two Amtrak railroad tracks at an at-grade crossing. The existing grade gradually slopes downward from Elevation (Elev.) 74 (west) to Elev. 70 (east).

Overhead utilities exist along the south side of Flatbush Avenue and along the west side of Amtrak's right-of-way, north of Flatbush Avenue. These utilities and related appurtenances will need to be relocated and/or replaced to accommodate the construction of these walls. Amtrak railroad crossing lights, gates and signalization exist along both sides of the existing at-grade crossing on Flatbush Avenue. These utilities will also be removed since they will no longer be required.

Existing utilities, including water, gas and sewer, extend under and/or near to Flatbush Ave and Newfield Ave. These will be affected by embankment fill up to 26 feet in height. These utilities are to remain until new utilities are constructed along the new alignment.

A one-story steel building is located to the northeast of the intersection of Newfield and Flatbush Avenues. The building footprint is about 30,000 square feet. One corner of the building is as close as 15 feet to the proposed embankment.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

7.4.1 Pile Lengths

Estimated pile lengths at each proposed pile-supported retaining wall are included in Table 8 (Appendix 2, Vol I). We recommend an additional 20 feet be added for test pile lengths. We then recommend the contractor use the information from the test piles to determine the order length.

7.4.2 Down-drag Loads

Down-drag load is an important factor in pile design. Wall backfill and embankment loading will result in compression of the Varved Clays. Significant downdrag loads will be imparted to the piles.

7.4.3 Down-drag Load Reduction

Bitumen is very effective as a coating to reduce the friction between the soil and piles. A one-millimeter thick bitumen coating is sufficient to significantly reduce the down-drag force. The bitumen coating need not be applied to the section of the pile below the bottom of the Varved Clay stratum. We estimate that the down-drag load on a bitumen-coated pile will be 10 percent of the downdrag force on an uncoated pile (90% reduction). We estimate that bitumen coating of the piles will add approximately 10 percent to the pile's material cost. Thus, we recommend bitumen coating for the piles.

Precautions must be taken to prevent damage the bitumen coating with extreme temperatures and while driving through granular soils. To avoid damage to the coating, we recommend that a hole be pre-drilled through the surficial Fill and Silt/Sand layers to the top of the Varved Clay.

7.4.4 Corrosion Protection of Steel Piles

As noted in Section 5.10, pH values of 3.7 to 7.7 indicate some corrosive potential. However, due to the recommendation to use of bitumen coating, which will provide some protection from corrosion, we do not recommend using a corrosion allowance for the steel piles.

7.4.5 Pile Type and Size Selection

To limit down-drag forces, we recommend steel piles because, compared to piles of other materials, steel piles typically provide higher strength, smaller perimeter, and smaller friction coefficients.

ConnDOT has requested that pile tip stresses not exceed 24 ksi. We therefore recommend end-bearing, bitumen coated, Grade 50 steel HP-Piles, with a maximum tip stress of 24 ksi. We further recommend pile tip reinforcement with integrally cast cutting teeth (or similar) be used. The following table provides nominal compressive resistances, down-drag loads, and nominal lateral capacities for a selection of HP pile sections. The down-drag loads are based on bitumen coated piles and must be added to the abutment load when determining the required number of piles. Nominal lateral capacities are based on a predicted lateral deflection of 0.6 inches.

Pile Selection	Nominal Compressive Resistance (kips/pile)	Design Down-drag Load (kips/bitumen coated pile)	Nominal Lateral Capacity (kips/pile)
HP 12x53	372	45	20
HP 12x74	523	45	20
HP 14x89	626	50	25
HP14x117	825	50	25

We recommend a resistance factor for compression, ($Ø_c$), of 0.6 (for good driving). Resistance factors for the service limit state shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. Resistance factors for the extreme limit state shall also be taken as 1.0, except for uplift resistance of piles, where resistance factor shall be taken as 0.8. Refer to Section 7.4.9 for use of resistance factors during pile testing.

Although larger piles are preferred to carry vertical load efficiently, four pile sizes are provided, for walls where the horizontal loading will control. In this case the smaller pile sizes may be more efficient overall.

The down-drag load must be added to the wall load when determining the required capacity and number of piles.

7.4.6 Pile Batter

Batter piles may be used to supplement the recommended lateral capacity of vertical piles if needed. We recommend a maximum pile batter of 1H:4V. In addition, if batter piles are used, the lateral capacity of piles (excluding the batter component) must be reduced to a maximum ultimate lateral capacity of 3 kips per pile.

7.4.7 Pile Spacing

In no case should the piles be spaced closer than three pile diameters. Pile group reduction factors, as applicable must be applied in accordance with AASHTO LRFD (2006 interims) Section 10.7.2.4, Table 10.7.2.4-1, and Figure 10.7.2.4-1.

7.4.8 Pile Splicing

Due to pile length, shipping, and handling constraints, piles will require at least one splice. Splices shall be made using pre-approved pre-fabricated splice connectors welded to provide the design pile vertical and lateral capacity. Splices shall not be the allowed within 15 feet of the pile cut-off and splices between adjacent piles shall be staggered at least 5 feet vertically and should conform to Form 816 7.02.03.

7.4.9 Pile Load Testing

We recommend the use of PDA testing, which can be completed quickly. We recommend one test pile be tested at each of these retaining walls. The pile load testing resistance factor for PDA Testing ($Ø_{dyn}$) is 0.65. The test pile selection should be based on a successfully tested indicator pile driving records, considered in relation to the test boring data, as determined by the Geotechnical Engineer.

Preliminary installation criteria for the piles should be based on wave equation analysis employing the characteristics of the pile type, soil conditions, and pile driving hammer and cushions proposed by the Contractor. This installation criteria analysis may be performed by Geo**Design,** or by the Contractor's engineer and submitted for review.

7.4.10 Pile Supported Wall Settlement

Settlement of pile-supported retaining wall is expected to range from 1/8 to 1/4 inches and will occur largely during wall construction.

7.5 Parameters for Spread Footings-Supported Wall (Walls 103 & 105)

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls.

7.6 Fill and Backfill Design Parameters

7.6.1 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.6.2 Wick Drain Design Parameters

Recommended design parameters for wick drains which are used to accelerate the rate of consolidation of the Varved Clay stratum under proposed embankment loads, are as follows:

- Drain length from existing ground surface (after pavement/topsoil/subsoil removal) to approximate bottom of Varved Clay layer, see following table.
- Triangular Drain Spacing = 8 feet.
- Minimum thickness of Pervious Structure Fill or Drainage Sand layer above top of wick drains = 12 inches

Retaining Wall No.	Roadway Baseline	From Station	To Station	Width of Wick Drain Area (ft)	Estimated Top El. Of Wick Drain (ft)	Estimated Bottom El. Of Wick Drain (ft)	Estimated Wick Drain Length (ft)
RW-102	Flatbush Ave.	1690	1800	100	75	-30	105
RW-102	Flatbush Ave.	1800	1950	100	75	-30	105
RW-102	Newfield Ave.	5380	5528	100	75	-30	105
RW-103	Newfield Ave.	5380	5500	100	75	-30	105
RW-103	Flatbush Ave.	1950	2100	100	75	-30	105
RW-105	Flatbush Ave.	1750	1850	100	70	-40	110
RW-105	Flatbush Ave.	1850	1950	100	70	-40	110
RW-105	Flatbush Ave.	1950	2050	100	70	-40	110
RW-105	Flatbush Ave.	2050	2150	100	70	-40	110

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundations and the distance to the nearest railroad track, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Vibrations from pile driving may impact the tracks and nearby utilities. See Section 8.5 below for recommendations regarding the utilities. We recommend that other structures be surveyed prior to construction and closely monitored. The threshold/action criteria should be defined and coordinated in advance with Amtrak. The railroad tracks should be monitored for vibration in accordance with Amtrak requirements.

8.5 Monitoring of Utilities

Existing utilities are present at the Flatbush Ave. crossing where, in addition to these retaining walls a new bridge will be constructed. Refer to the Flatbush Avenue Over Busway Structure Layout for Design Report for details.

8.6 Monitoring of Amtrak Railway Tracks

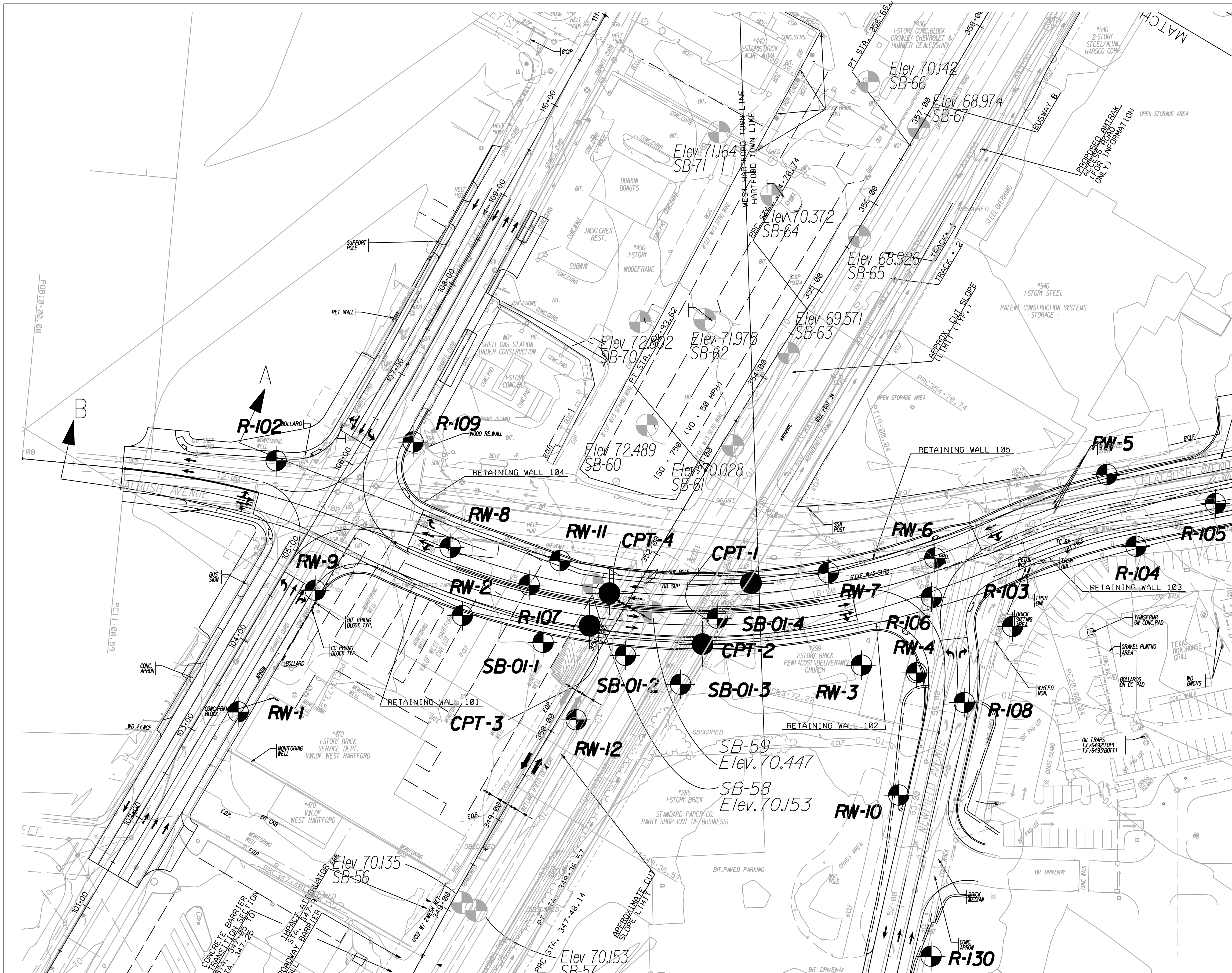
We recommend that monitoring points be established on both tracks at 50-foot intervals along these retaining wall embankments since they need Special Requirements B or C (Column 15 or 17, Table 7, Appendix 2, Vol I).

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address bitumen coating of piles, pile splicing, pile testing, and lightweight fill.



LEGEND

CPT- 2008 CPT LOCATION

SB-01- R- RW- 2008 TEST BORING

SB- RB- PILOT BORING

DESIGNED BY MGB						BORINGS LOCATED BY CT DOT SURVEY.
DRAWN BY SMC						
CHECKED BY MGB						
APPROVED BY ULF	NO.	DATE	DRWN.	CHKD	APPVD	
	REVISIONS					

PRIME DESIGNER:

URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

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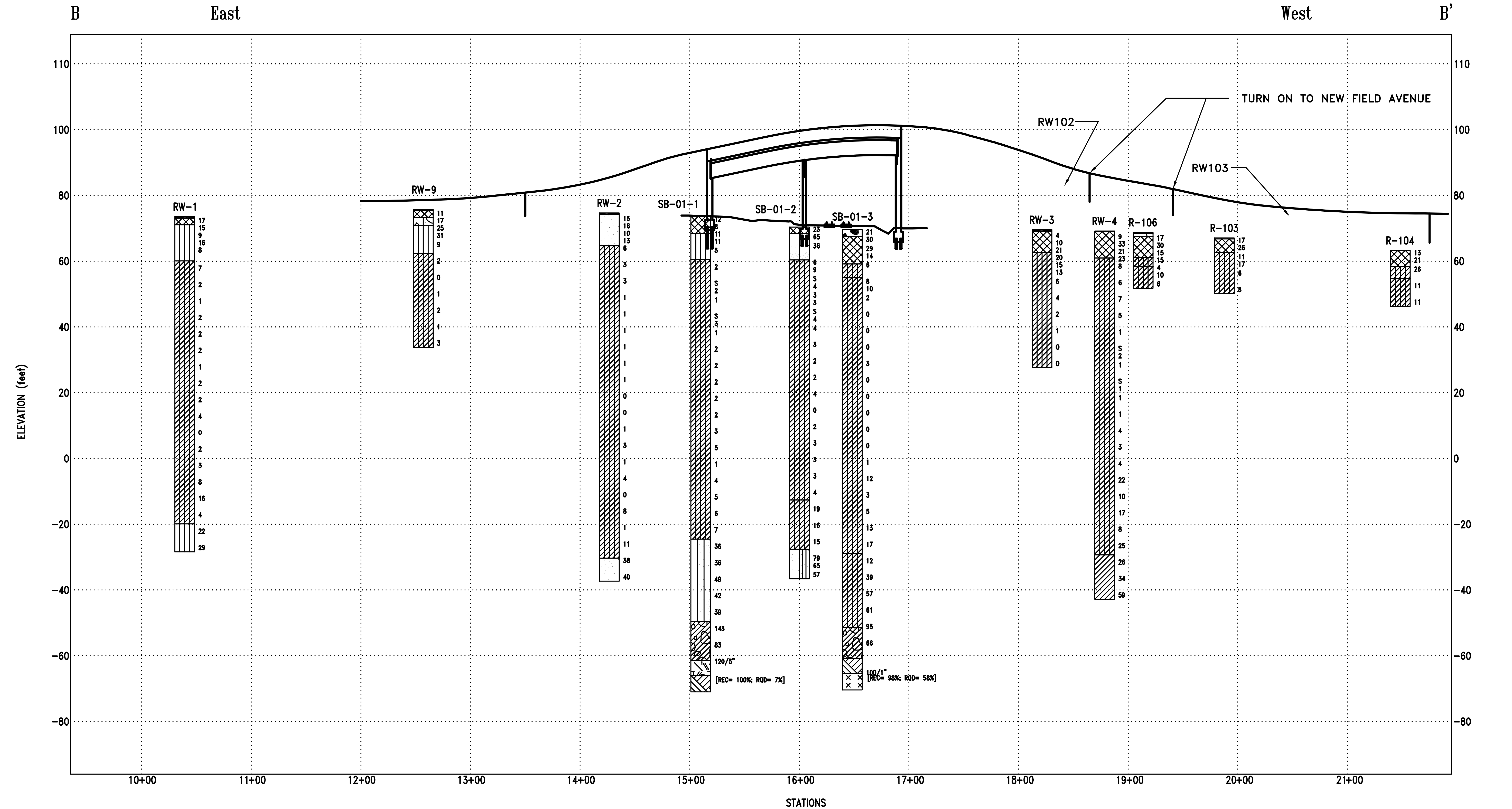
DWG. TITLE
RETAINING WALLS 102/103/105/106
AS-DRILLED EXPLORATION
LOCATION PLAN

PROJECT
NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

FILE NO. 380-04
SCALE 1" = 100'
DATE 5/26/09
FIGURE NO. 1



GEODESIGN INCORPORATED
Geotechnical Engineers and Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 90

Horizontal Scale (feet)

Vertical Exaggeration: 3x

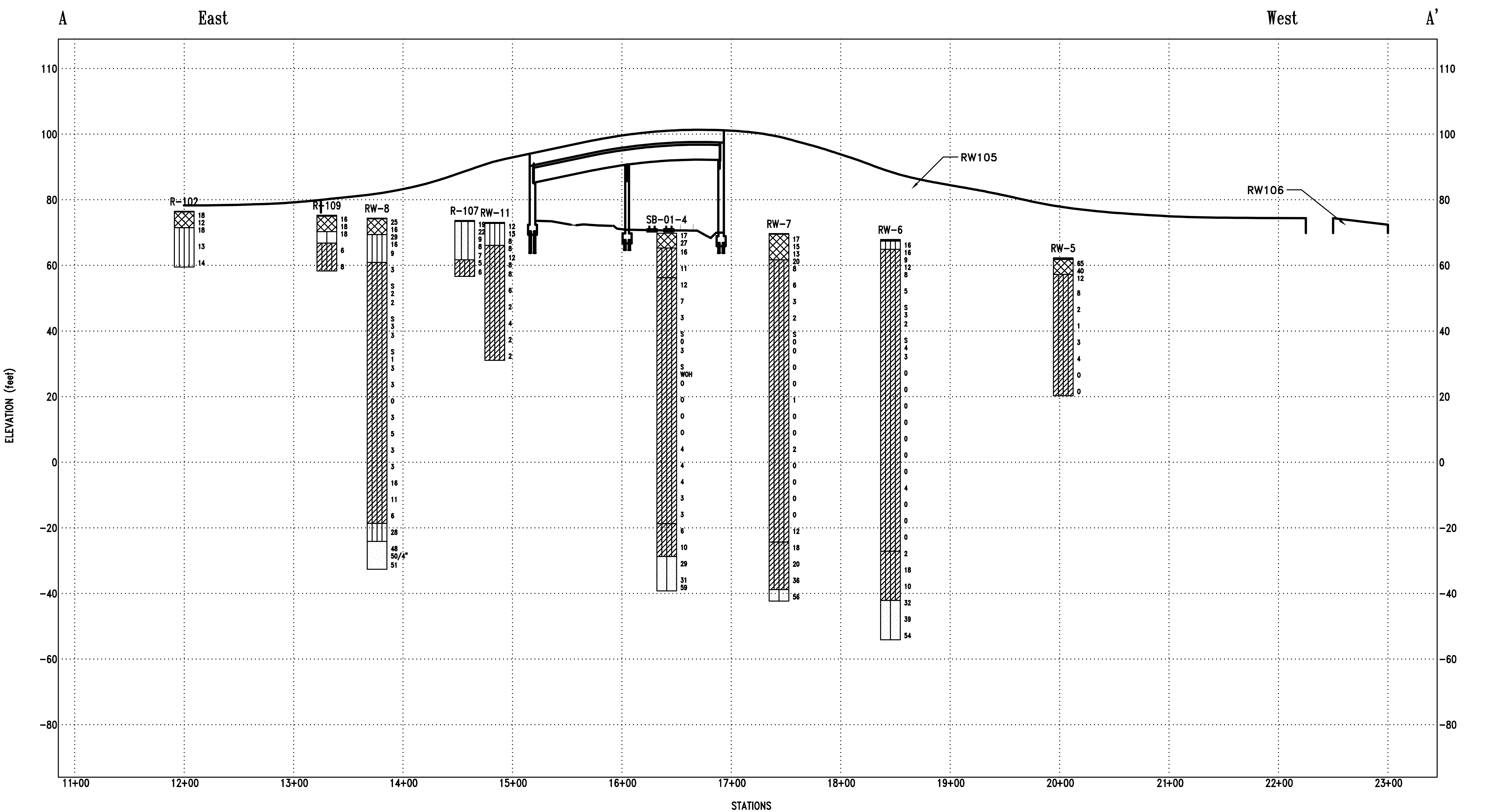
Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.

SUBSURFACE PROFILE B-B'

Retaining Walls RW102 and RW103
New Britain - Hartford Busway /
State Project No. 155-H025

Date: 5/26/09	Drawn By: MBF	Reviewed By: MGB	File No.: 0380-004.0	Figure No.: 2

M:\CL\0380 BUSWAY\04 HTFD SOUTH\gINT\Flatbush Ave (1-19-09)\profile b.dwg



GEODESIGN
INCORPORATED
Geotechnical Engineers and Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 90

Horizontal Scale (feet)

Vertical Exaggeration: 3x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.

SUBSURFACE PROFILE A-A'

Retaining Wall RW105 and RW106
New Britain - Hartford Busway /
State Project No. 155-H025

Date: 5/26/09	Drawn By: MBF	Reviewed By: MGB	File No.: 0380-004.0	Figure No.: 3

INSERT TAB
SW-1
HERE

Retaining Wall SW-1

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Wall SW-1

Attached Figures:

RETAINING WALL SW-1

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

West of the Amtrak RR tracks, the existing embankment slopes downward to the west at about 1V: 2H between approximate Stations 383+50 and 384+75. This proposed wall (crash wall) will be built adjacent to two existing I-84 piers and very close to two other I-84 existing piers. A chain link fence is located west of the proposed retaining wall between the existing piers.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls.

7.6 Fill and Backfill Design Parameters

7.6.1 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, bagged stone and weep holes should be utilized.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. This retaining wall will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed wall, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Not Applicable

8.6 Monitoring of Amtrak Railway Tracks

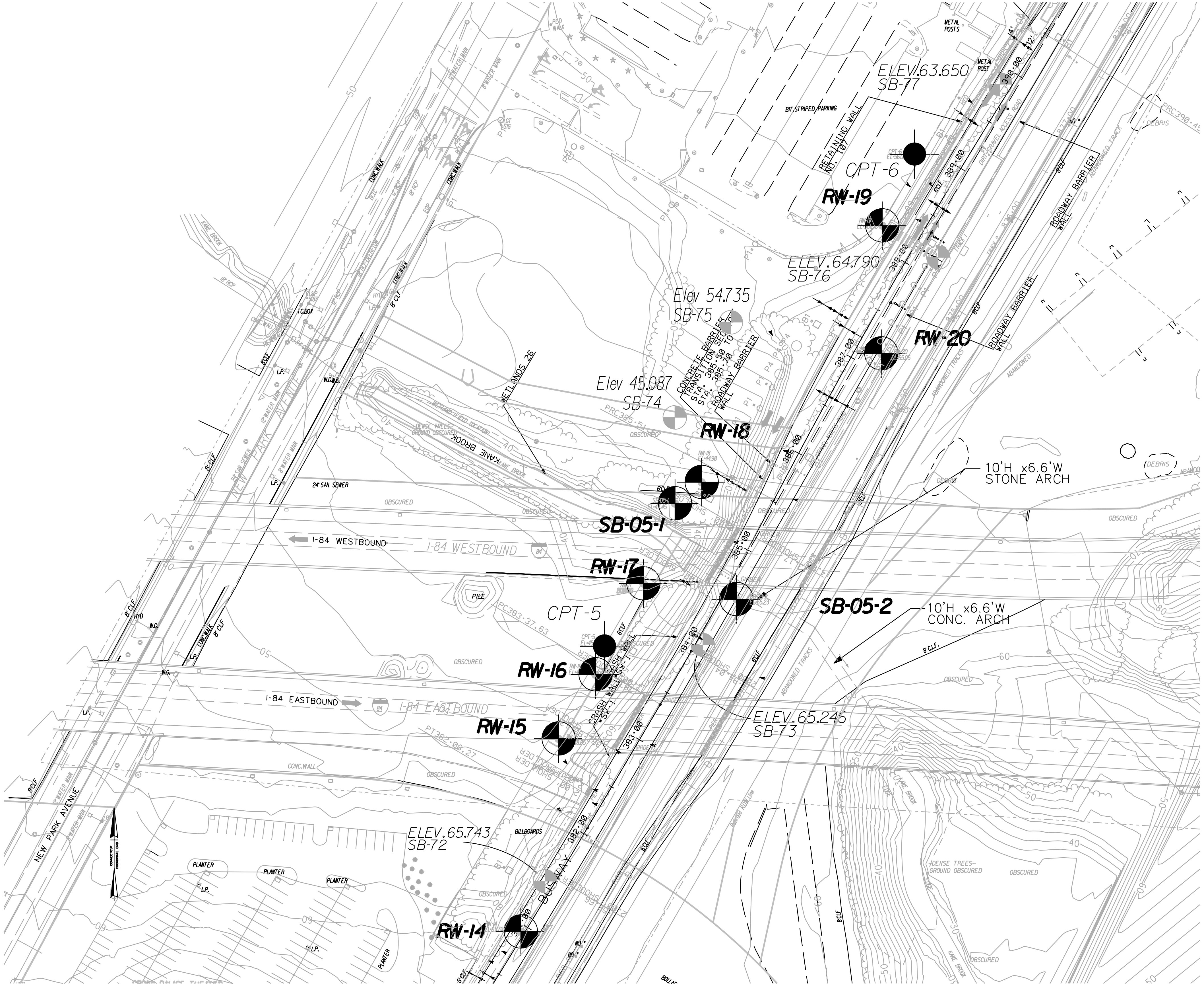
Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address lightweight fill.



LEGEND

CPT- 2008 CPT LOCATION

SB-01- R- RW- 2008 TEST BORING

SB- RB- PILOT BORING

DESIGNED BY	MGB	
DRAWN BY	SMC	
CHECKED BY	MGB	
APPROVED BY	ULF	

NO.	DATE		DRWN.	CHKD	APPVD

REVISIONS

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

GEODESIGN INCORPORATED
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS
984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762
TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842

DWG. TITLE
RETAINING WALL SW-1
AS-DRILLED EXPLORATION
LOCATION PLAN

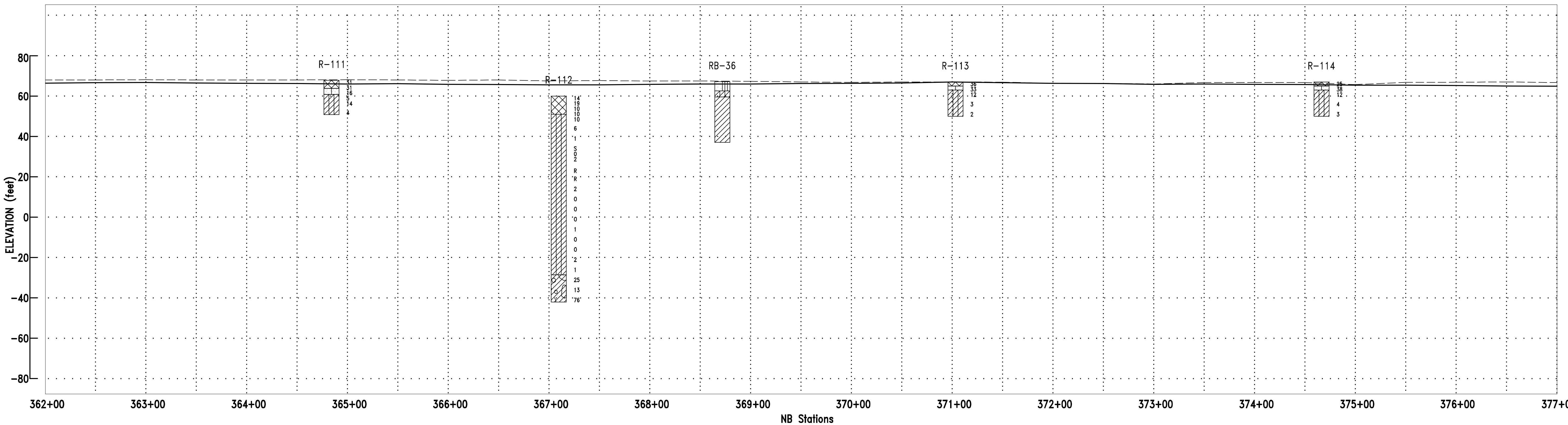
PROJECT
NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

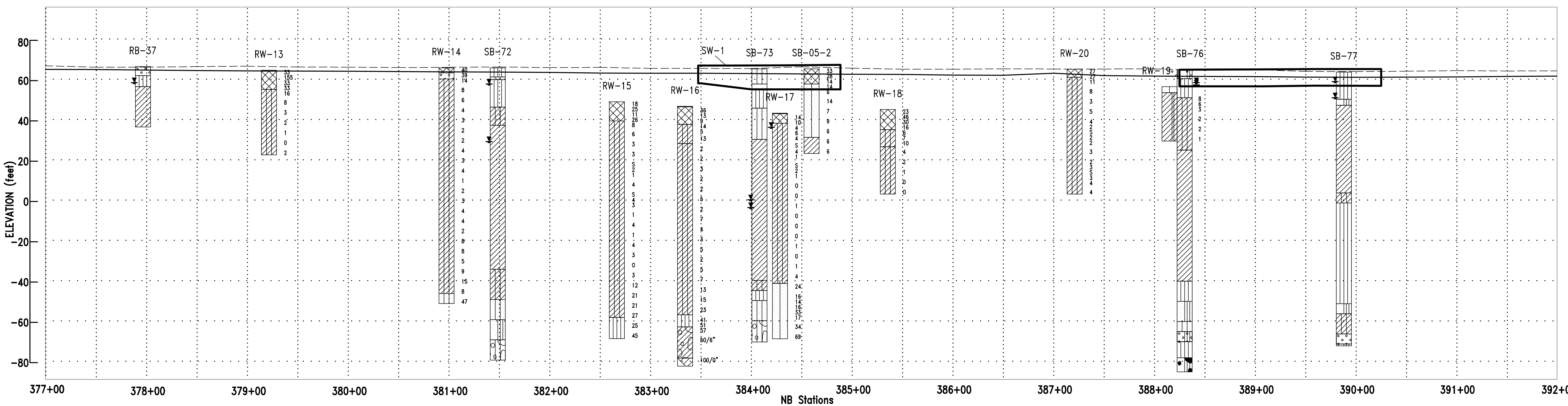
FILE NO. 380-04

SCALE 1" = 100'

DATE 5/26/09

FIGURE NO. 1





M:\C1\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 14)

GEODESIGN
INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL SW-1

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2

INSERT TAB
107
HERE

Retaining Wall 107

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Wall 107

Attached Figures:

RETAINING WALL 107

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

West of the Amtrak RR tracks, between approximate Stations 388+25 and 390+25, an approximate 5-foot height embankment slope of 1V: 4H is present. The first 150 feet (388+25 to 389+75) of this wall will require no fill. The last 50 feet (389+75 to 390+25) of this wall will require about 3 feet of fill. This wall will be located east of an existing parking lot. An existing chain link fence is present between this proposed wall and the parking lot.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls.

7.6 Fill and Backfill Design Parameters

7.6.1 Regular Fill

For design of the walls backfilled with regular fill (e.g. not lightweight), we recommend the following static design parameters:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, phi = 34°

7.6.2 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, bagged stone and weep holes should be utilized.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. This retaining wall will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed wall, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Not Applicable

8.6 Monitoring of Amtrak Railway Tracks

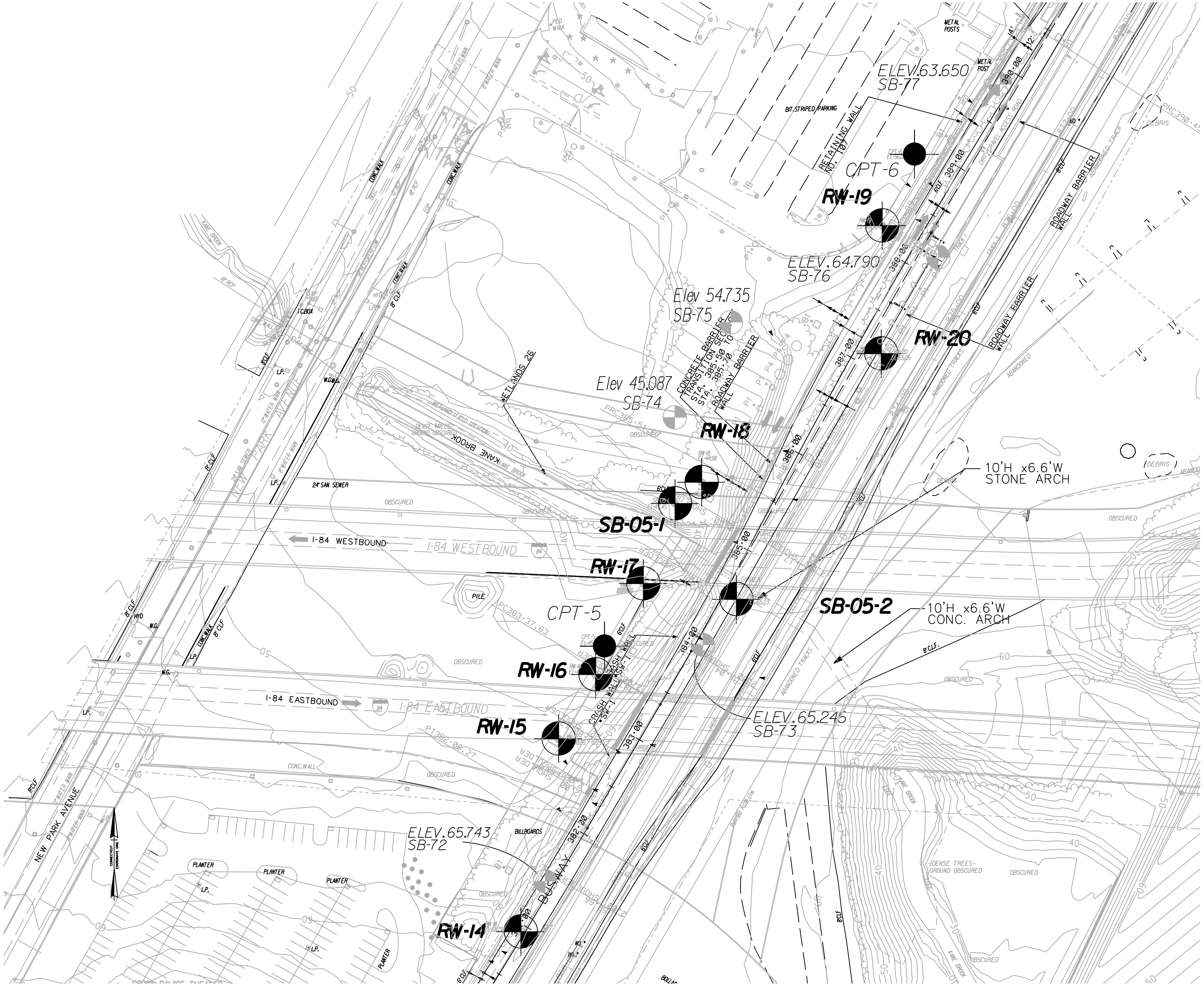
Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address lightweight fill.



LEGEND

CPT-	2008 CPT LOCATION
SB-01- R- RW-	2008 TEST BORING
SB- RB-	PILOT BORING

DESIGNED BY	MGB	
DRAWN BY	SMC	
CHECKED BY	MGB	
APPROVED BY	ULF	

NO.	DATE		DRWN.	CHKD	APPVD
		REVISIONS			

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

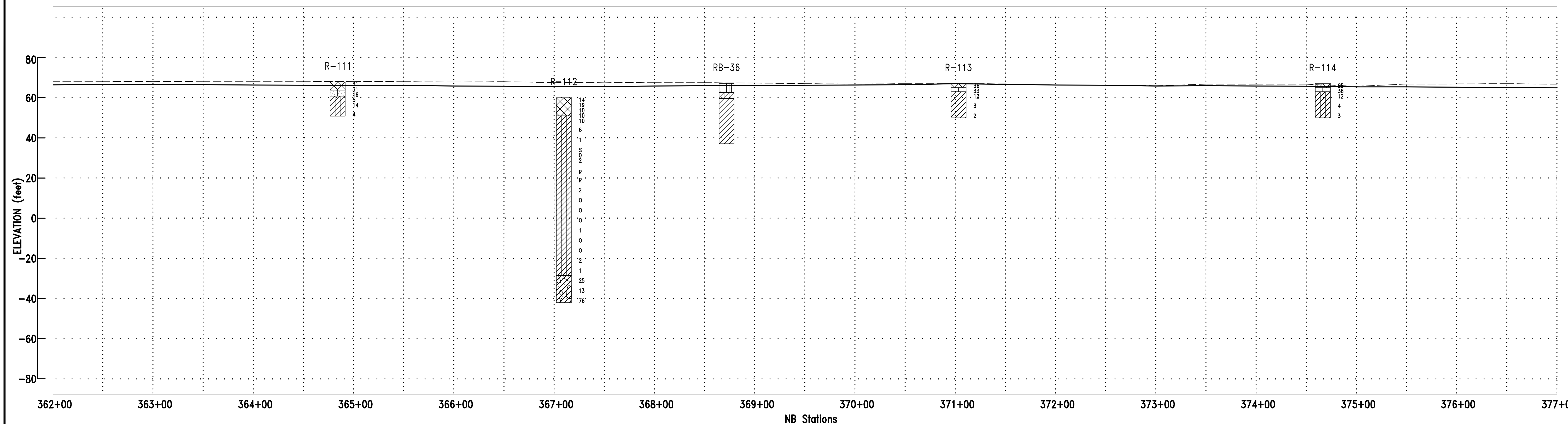
URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

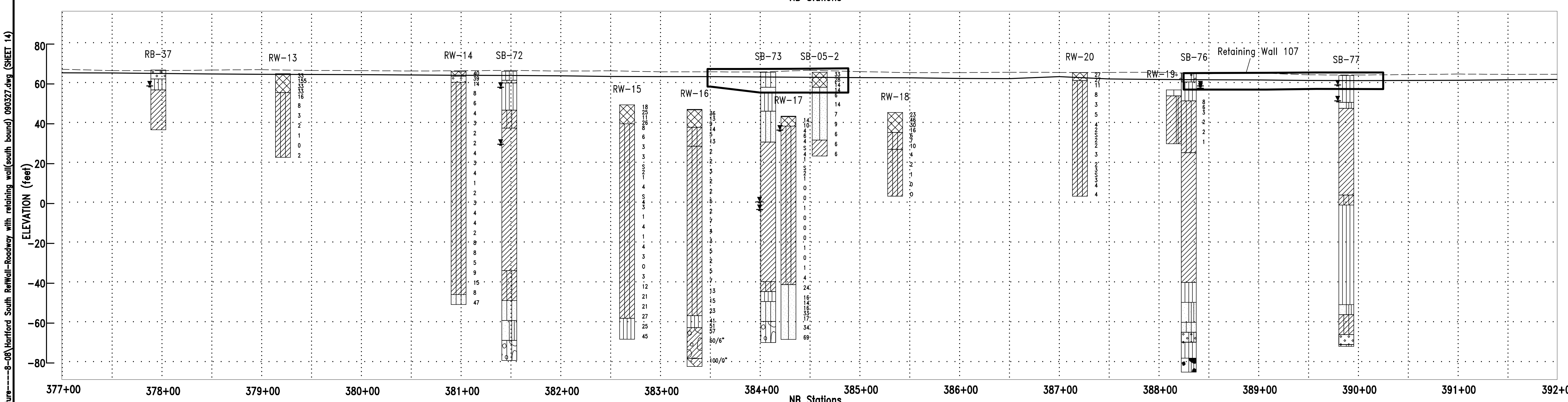
GEODESIGN INCORPORATED
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS
984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762
TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842

DWG. TITLE
RETAINING WALL 107
AS-DRILLED EXPLORATION
LOCATION PLAN

PROJECT
NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

FILE NO.	380-04
SCALE	1" = 100'
DATE	5/26/09
FIGURE NO.	1





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 14)

GEODESIGN INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100
Horizontal Scale (feet)
Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 107

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2

INSERT TAB 108 HERE

Retaining Wall 108

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Wall 108

Attached Figures:

RETAINING WALL 108

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

West of the Amtrak RR tracks, between Stations 411+20 and 412+60, a seven-foot high embankment with a slope of 1V: 1H to 1V: 4H is present. This proposed wall will be located between the existing Amtrak RR tracks (to the east) and a gas company property (to the west). An existing chain link fence is present west of the proposed wall.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick granular fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls .

7.6 Fill and Backfill Design Parameters

7.6.1 Regular Fill

For design of the walls backfilled with regular fill (e.g. not lightweight), we recommend the following static design parameters:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, phi = 34°

7.6.2 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. This retaining wall will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Not Applicable

8.6 Monitoring of Amtrak Railway Tracks

Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

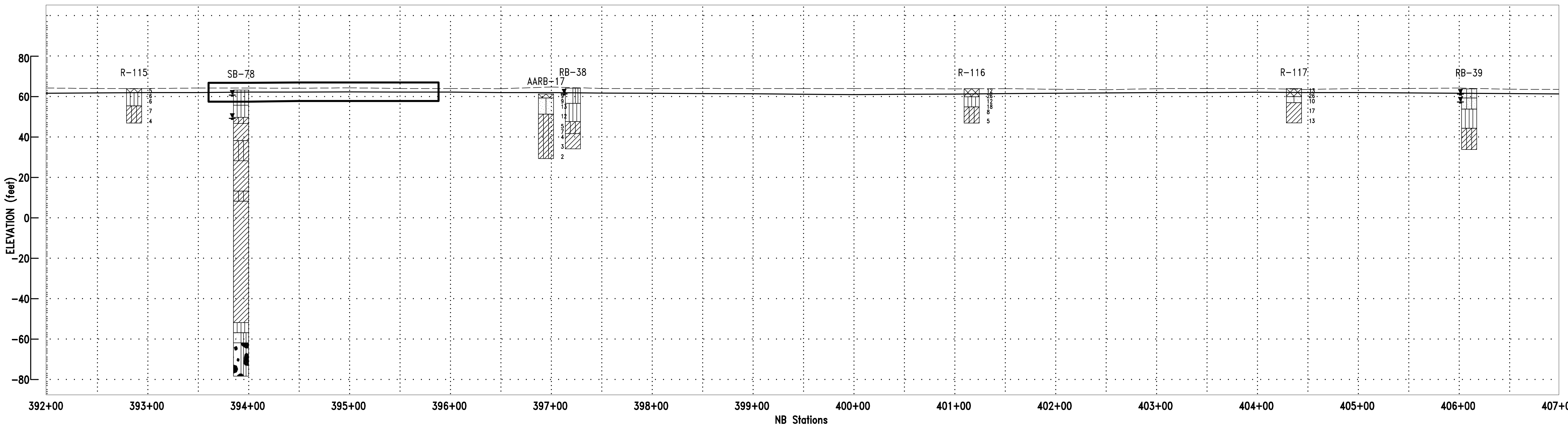
9.0 SPECIAL PROVISIONS

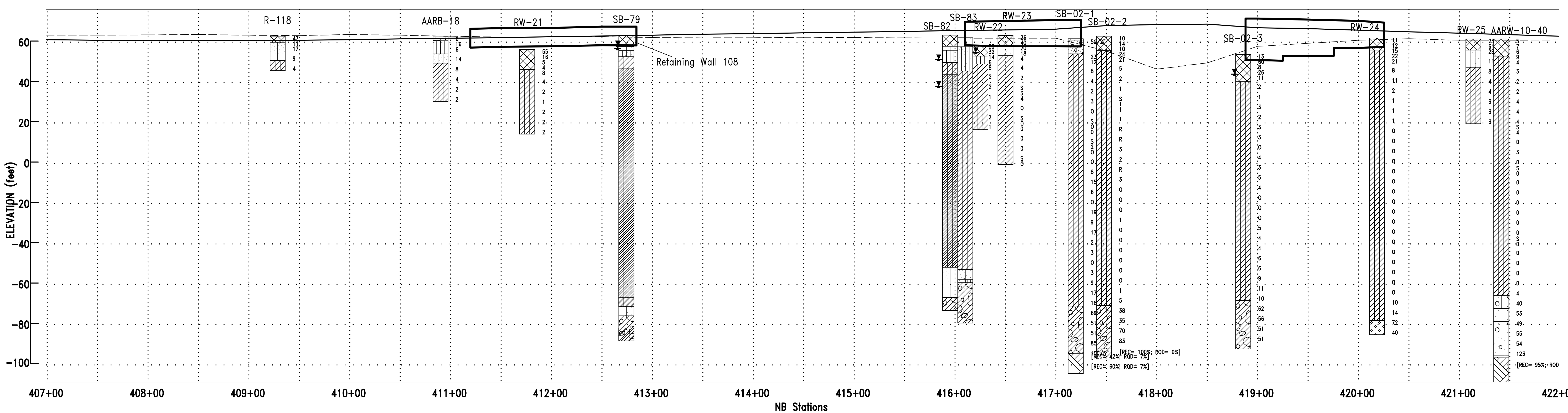
Special provisions will be required to address lightweight fill.

LEGEND

CPT-		2008 CPT LOCATION
SB-01- R- RW-		2008 TEST BORING
SB- RB-		PILOT BORING

DESIGNED BY MGB DRAWN BY SMC CHECKED BY MGB APPROVED BY ULF	NO. / DATE / DRWN. / CHKD / APPVD REVISIONS	BORINGS LOCATED BY CT DOT SURVEY.	PRIME DESIGNER: **URS Corporation** 500 Enterprise Dr. Suite 3B Rocky Hill, Ct.
GEODESIGN INCORPORATED GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS 984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762 TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842	DWG. TITLE RETAINING WALL 108 AS-DRILLED EXPLORATION LOCATION PLAN	PROJECT NEW BRITAIN – HARTFORD BUSWAY STATE PROJECT NO. 155-H025	FILE NO. 380-04 SCALE 1" = 100' DATE 5/26/09 FIGURE NO. 1





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 15)

GEODESIGN
INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 108

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2

INSERT TAB 109 and 110 HERE

Retaining Walls 109 and 110

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 109 and 110

Attached Figures:

RETAINING WALLS 109 and 110

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

These two walls are proposed to retain embankment fill for the Busway south of the proposed Park St. Bridge, between Stations 415+92 and 417+40. The existing Amtrak railroad currently crosses over Park Street on a single-span bridge. Existing grades are highest along the Amtrak Railroad (Elev. 64) and lowest at Park Street (Elev. 46). The present railroad embankment side slopes range from about 3H: 1V to 4H: 1V near the bridge. In this vicinity, a 36-inch diameter reinforced concrete pipe exists underneath the centerline of Park Street, and a 30-inch water main exists at the north edge of Park Street. Chain link fences are present next to proposed wall alignment. Proposed fill thickness will vary from about 6 to 8 feet.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations (RW 109)

7.4.1 Pile Lengths

Estimated pile lengths at each proposed pile-supported retaining wall are included in Table 8 (Appendix 2, Vol I). We recommend an additional 20 feet be added for test pile lengths. We then recommend the contractor use the information from the test piles to determine the order length.

7.4.2 Down-drag Loads

Down-drag load is an important factor in pile design. Wall backfill and embankment loading will result in compression of the Varved Clays. Significant downdrag loads will be imparted to the piles.

7.4.3 Down-drag Load Reduction

Bitumen is very effective as a coating to reduce the friction between the soil and piles. A one-millimeter thick bitumen coating is sufficient to significantly reduce the down-drag force. The bitumen coating need not be applied to the section of the pile below the bottom of the Varved Clay stratum. We estimate that the down-drag load on a bitumen-coated pile will be 10 percent of the downdrag force on an uncoated pile (90% reduction). We estimate that bitumen coating of

the piles will add approximately 10 percent to the pile's material cost. Thus, we recommend bitumen coating for the piles.

Precautions must be taken to prevent damage the bitumen coating with extreme temperatures and while driving through granular soils. To avoid damage to the coating, we recommend that a hole be pre-drilled through the surficial Fill and Silt/Sand layers to the top of the Varved Clay.

7.4.4 Corrosion Protection of Steel Piles

As noted in Section 5.10, pH values of 3.7 to 7.7 indicate some corrosive potential. However, due to the recommendation to use of bitumen coating, which will provide some protection from corrosion, we do not recommend using a corrosion allowance for the steel piles.

7.4.5 Pile Type and Size Selection

To limit down-drag forces, we recommend steel piles because, compared to piles of other materials, steel piles typically provide higher strength, smaller perimeter, and smaller friction coefficients.

ConnDOT has requested that pile tip stresses not exceed 24 ksi. We therefore recommend end-bearing, bitumen coated, Grade 50 steel HP-Piles, with a maximum tip stress of 24 ksi. We further recommend pile tip reinforcement with integrally cast cutting teeth (or similar) be used. The following table provides nominal compressive resistances, down-drag loads, and nominal lateral capacities for a selection of HP pile sections. The down-drag loads are based on bitumen coated piles and must be added to the abutment load when determining the required number of piles. Nominal lateral capacities are based on a predicted lateral deflection of 0.6 inches.

Pile Selection	Nominal Compressive Resistance (kips/pile)	Design Down-drag Load (kips/bitumen coated pile)	Nominal Lateral Capacity (kips/pile)
HP 12x53	372	55	20
HP 12x74	523	55	20
HP 14x89	626	65	25
HP14x117	825	65	25

We recommend a resistance factor for compression, ($Ø_c$), of 0.6 (for good driving). Resistance factors for the service limit state shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. Resistance factors for the extreme limit state shall also be taken as 1.0, except for uplift resistance of piles, where resistance factor shall be taken as 0.8. Refer to Section 7.4.9 for use of resistance factors during pile testing.

Although larger piles are preferred to carry vertical load efficiently, four pile sizes are provided, for walls where the horizontal loading will control. In this case the smaller pile sizes may be more efficient overall.

The down-drag load must be added to the wall load when determining the required capacity and number of piles.

7.4.6 Pile Batter

Batter piles may be used to supplement the recommended lateral capacity of vertical piles if needed. We recommend a maximum pile batter of 1H:4V. In addition, if batter piles are used, the lateral capacity of piles (excluding the batter component) must be reduced to a maximum ultimate lateral capacity of 3 kips per pile.

7.4.7 Pile Spacing

In no case should the piles be spaced closer than three pile diameters. Pile group reduction factors, as applicable must be applied in accordance with AASHTO LRFD (2006 interims) Section 10.7.2.4, Table 10.7.2.4-1, and Figure 10.7.2.4-1.

7.4.8 Pile Splicing

Due to pile length, shipping, and handling constraints, piles will require at least one splice. Splices shall be made using pre-approved pre-fabricated splice connectors welded to provide the design pile vertical and lateral capacity. Splices shall not be the allowed within 15 feet of the pile cut-off and splices between adjacent piles shall be staggered at least 5 feet vertically and should conform to Form 816 7.02.03.

7.4.9 Pile Load Testing

We recommend the use of PDA testing, which can be completed quickly. We recommend one test pile be tested at this retaining wall. The pile load testing resistance factor for PDA Testing ($Ø_{dyn}$) is 0.65. The test pile selection should be based on a successfully tested indicator pile driving records, considered in relation to the test boring data, as determined by the Geotechnical Engineer.

Preliminary installation criteria for the piles should be based on wave equation analysis employing the characteristics of the pile type, soil conditions, and pile driving hammer and cushions proposed by the Contractor. This installation criteria analysis may be performed by Geo**Design,** or by the Contractor’s engineer and submitted for review.

7.4.10 Pile Supported Wall Settlement

Settlement of pile-supported retaining wall is expected to range from 1/8 to 1/4 inches and will occur largely during wall construction.

7.5 Parameters for Spread Footings (Wall 110)

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls .

7.6 Fill and Backfill Design Parameters

7.6.1 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf

- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These

retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Vibrations from pile driving may impact the tracks and nearby utilities. See Section 8.5 below for recommendations regarding the utilities. We recommend that other structures be surveyed prior to construction and closely monitored. The threshold/action criteria should be defined and coordinated in advance with Amtrak. The railroad tracks should be monitored for vibration in accordance with Amtrak requirements.

8.5 Monitoring of Utilities

Existing utilities are present at Park Street where a new bridge is to be constructed. Refer to specific Structure Layout for Design Reports for details.

8.6 Monitoring of Amtrak Railway Tracks

Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address bitumen coating of piles, pile splicing, pile testing, and lightweight fill.

LEGEND

CPT-	2008 CPT LOCATION
SB-01- R- RW-	2008 TEST BORING
SB- RB-	PILOT BORING

DESIGNED BY	MGB
DRAWN BY	SMC
CHECKED BY	MGB
APPROVED BY	ULF

NO.	DATE		DRWN.	CHKD	APPVD

REVISIONS

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

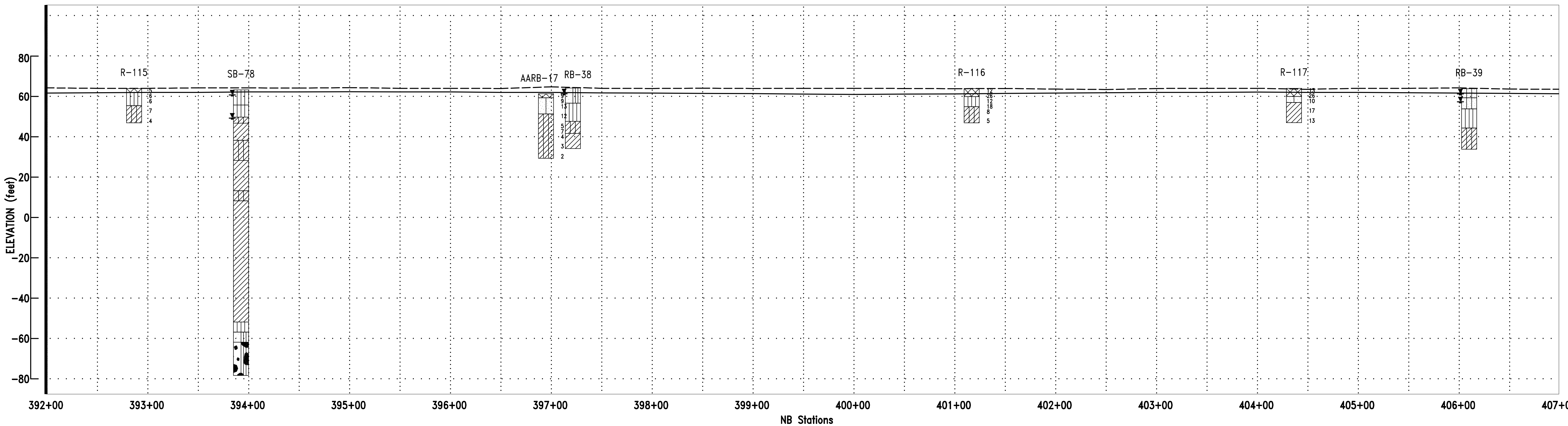
URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

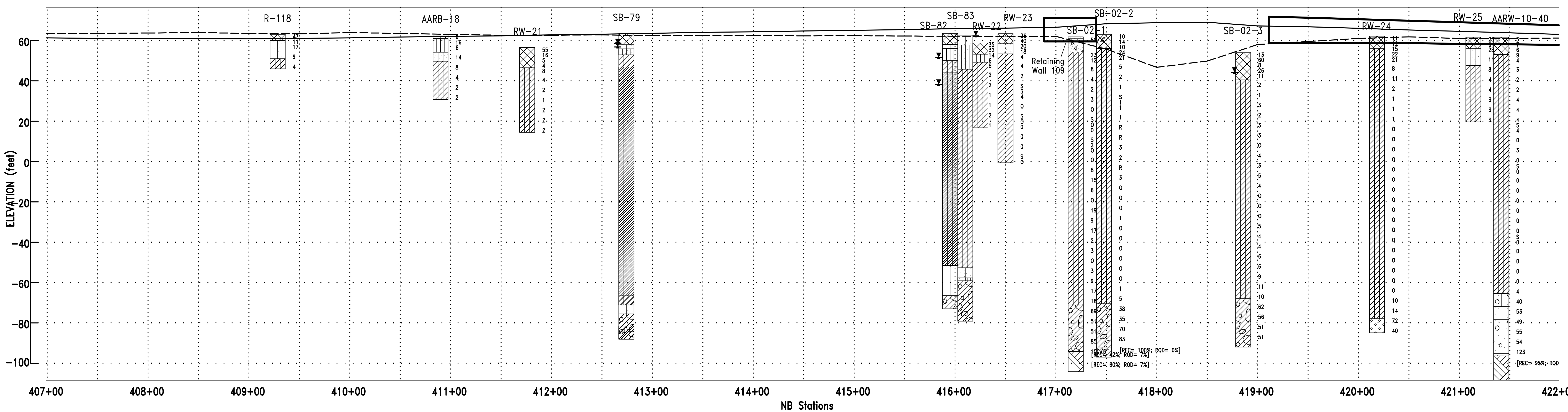
GEODESIGN INCORPORATED
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS
984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762
TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842

DWG. TITLE
RETAINING WALLS 109/110
AS-DRILLED EXPLORATION
LOCATION PLAN

PROJECT
NEW BRITAIN - HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

FILE NO.	380-04
SCALE	1" = 100'
DATE	5/26/09
FIGURE NO.	1





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(north bound) 090327.dwg (SHEET 15)

GEODESIGN INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

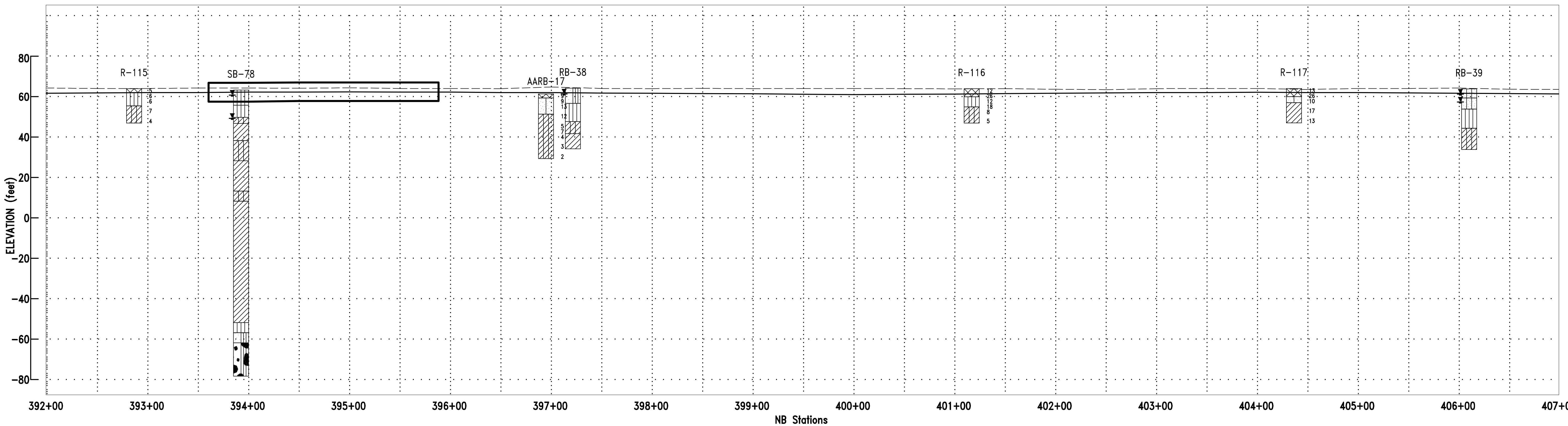
0 100
Horizontal Scale (feet)
Vertical Exaggeration: 2x

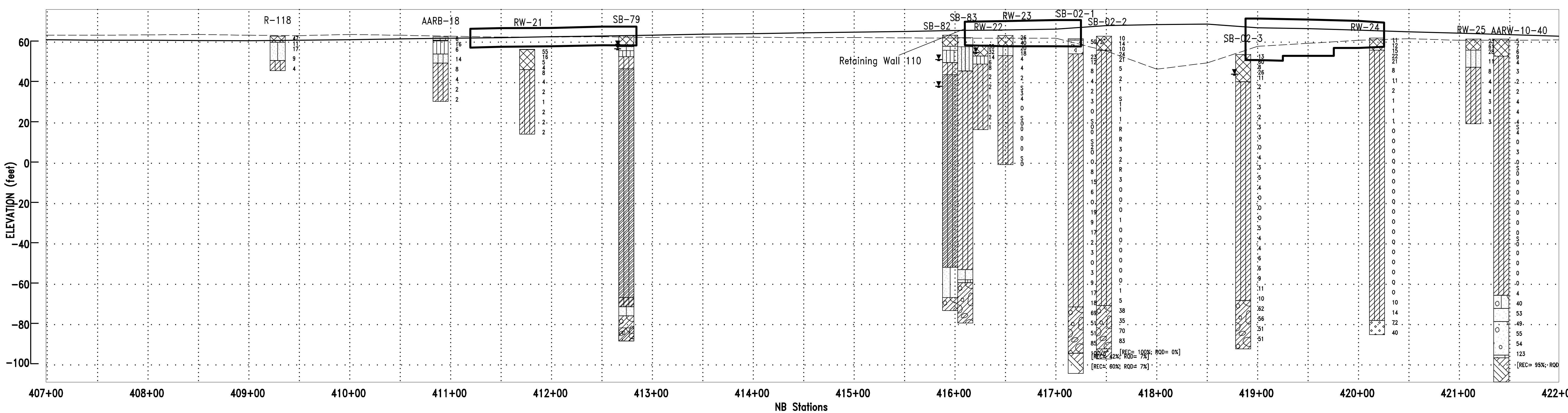
Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

NORTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 109

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 15)

GEODESIGN INCORPORATED
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984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 110

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	3

INSERT TAB
111 and 112
HERE

Retaining Walls 111 and 112

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 111 and 112

Attached Figures:

RETAINING WALLS 111 and 112

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

These two walls are proposed to retain embankment fill for the Busway north of the proposed Park St. Bridge, between Stations 415+92 and 418+90. The existing Amtrak railroad currently crosses over Park Street on a single-span bridge. Existing grades are highest along the Amtrak Railroad (Elev. 64) and lowest at Park Street (Elev. 46). The present railroad embankment side slopes range from about 3H: 1V to 4H: 1V near the bridge. In this vicinity, a 36-inch diameter reinforced concrete pipe exists underneath the centerline of Park Street, and a 30-inch water main exists at the north edge of Park Street. Chain link fences are present next to proposed wall alignment. Proposed fill thickness will vary from about 2 to 14 feet.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

7.4.1 Pile Lengths

Estimated pile lengths at each proposed pile-supported retaining wall are included in Table 8 (Appendix 2, Vol I). We recommend an additional 20 feet be added for test pile lengths. We then recommend the contractor use the information from the test piles to determine the order length.

7.4.2 Down-drag Loads

Down-drag load is an important factor in pile design. Wall backfill and embankment loading will result in compression of the Varved Clays. Significant downdrag loads will be imparted to the piles.

7.4.3 Down-drag Load Reduction

Bitumen is very effective as a coating to reduce the friction between the soil and piles. A one-millimeter thick bitumen coating is sufficient to significantly reduce the down-drag force. The bitumen coating need not be applied to the section of the pile below the bottom of the Varved Clay stratum. We estimate that the down-drag load on a bitumen-coated pile will be 10 percent of the downdrag force on an uncoated pile (90% reduction). We estimate that bitumen coating of the piles will add approximately 10 percent to the pile's material cost. Thus, we recommend bitumen coating for the piles.

Precautions must be taken to prevent damage the bitumen coating with extreme temperatures and while driving through granular soils. To avoid damage to the coating, we recommend that a hole be pre-drilled through the surficial Fill and Silt/Sand layers to the top of the Varved Clay.

7.4.4 Corrosion Protection of Steel Piles

As noted in Section 5.10, pH values of 3.7 to 7.7 indicate some corrosive potential. However, due to the recommendation to use of bitumen coating, which will provide some protection from corrosion, we do not recommend using a corrosion allowance for the steel piles.

7.4.5 Pile Type and Size Selection

To limit down-drag forces, we recommend steel piles because, compared to piles of other materials, steel piles typically provide higher strength, smaller perimeter, and smaller friction coefficients.

ConnDOT has requested that pile tip stresses not exceed 24 ksi. We therefore recommend end-bearing, bitumen coated, Grade 50 steel HP-Piles, with a maximum tip stress of 24 ksi. We further recommend pile tip reinforcement with integrally cast cutting teeth (or similar) be used. The following table provides nominal compressive resistances, down-drag loads, and nominal lateral capacities for a selection of HP pile sections. The down-drag loads are based on bitumen coated piles and must be added to the abutment load when determining the required number of piles. Nominal lateral capacities are based on a predicted lateral deflection of 0.6 inches.

Pile Selection	Nominal Compressive Resistance (kips/pile)	Design Down-drag Load (kips/bitumen coated pile)	Nominal Lateral Capacity (kips/pile)
HP 12x53	372	55	20
HP 12x74	523	55	20
HP 14x89	626	65	25
HP14x117	825	65	25

We recommend a resistance factor for compression, ($Ø_c$), of 0.6 (for good driving). Resistance factors for the service limit state shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. Resistance factors for the extreme limit state shall also be taken as 1.0, except for uplift resistance of piles, where resistance factor shall be taken as 0.8. Refer to Section 7.4.9 for use of resistance factors during pile testing.

Although larger piles are preferred to carry vertical load efficiently, four pile sizes are provided, for walls where the horizontal loading will control. In this case the smaller pile sizes may be more efficient overall.

The down-drag load must be added to the wall load when determining the required capacity and number of piles.

7.4.6 Pile Batter

Batter piles may be used to supplement the recommended lateral capacity of vertical piles if needed. We recommend a maximum pile batter of 1H:4V. In addition, if batter piles are used, the lateral capacity of piles (excluding the batter component) must be reduced to a maximum ultimate lateral capacity of 3 kips per pile.

7.4.7 Pile Spacing

In no case should the piles be spaced closer than three pile diameters. Pile group reduction factors, as applicable must be applied in accordance with AASHTO LRFD (2006 interims) Section 10.7.2.4, Table 10.7.2.4-1, and Figure 10.7.2.4-1.

7.4.8 Pile Splicing

Due to pile length, shipping, and handling constraints, piles will require at least one splice. Splices shall be made using pre-approved pre-fabricated splice connectors welded to provide the design pile vertical and lateral capacity. Splices shall not be the allowed within 15 feet of the pile cut-off and splices between adjacent piles shall be staggered at least 5 feet vertically and should conform to Form 816 7.02.03.

7.4.9 Pile Load Testing

We recommend the use of PDA testing, which can be completed quickly. We recommend one test pile be tested at each of these retaining walls. The pile load testing resistance factor for PDA Testing ($Ø_{dyn}$) is 0.65. The test pile selection should be based on a successfully tested indicator pile driving records, considered in relation to the test boring data, as determined by the Geotechnical Engineer.

Preliminary installation criteria for the piles should be based on wave equation analysis employing the characteristics of the pile type, soil conditions, and pile driving hammer and cushions proposed by the Contractor. This installation criteria analysis may be performed by Geo**Design,** or by the Contractor’s engineer and submitted for review.

7.4.10 Pile Supported Wall Settlement

Settlement of pile-supported retaining wall is expected to range from 1/8 to 1/4 inches and will occur largely during wall construction.

7.5 Parameters for Spread Footings

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls .

7.6 Fill and Backfill Design Parameters

7.6.1 Regular Fill

For design of the walls backfilled with regular fill (e.g. not lightweight), we recommend the following static design parameters:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf

- Soil Angle of Internal Friction, phi = 34°

7.6.2 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.6.3 Wick Drain Design Parameters

Recommended design parameters for wick drains, where used to accelerate the rate of consolidation of the Varved Clay stratum under proposed embankment loads, are as follows:

- Drain length = 95 to 140 ft from existing ground surface (after pavement/topsoil/subsoil removal) to approximate bottom of Varved Clay layer, (see following table).
- Triangular Drain Spacing = 8 feet.
- Minimum thickness of Pervious Structure Fill or Drainage Sand layer above top of wick drains = 12 inches

Retaining Wall No.	Roadway Baseline	From Station	To Station	Width of Wick Drain Area (ft)	Estimated Top El. Of Wick Drain (ft)	Estimated Bottom El. Of Wick Drain (ft)	Estimated Wick Drain Length (ft)
RW-111	Busway SB	41890	41950	50	60	-80	140

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer’s approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of “unsuitable” materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Vibrations from pile driving may impact the tracks and nearby utilities. See Section 8.5 below for recommendations regarding the utilities. We recommend that other structures be surveyed prior to construction and closely monitored. The threshold/action criteria should be defined and coordinated in advance with Amtrak. The railroad tracks should be monitored for vibration in accordance with Amtrak requirements.

8.5 Monitoring of Utilities

Existing utilities are present at Park Street where a new bridge is to be constructed. Refer to the Structure Layout for Design Reports for details.

8.6 Monitoring of Amtrak Railway Tracks

Two active railway tracks extend parallel to some proposed retaining walls. They are approximately 15 feet and 50 feet east of proposed retaining walls. We recommend that monitoring points be established on both tracks at 50-foot intervals along retaining wall embankments that need Special Requirements B or C (Column 15 or 17, Table 7, Appendix 2, Vol I).

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

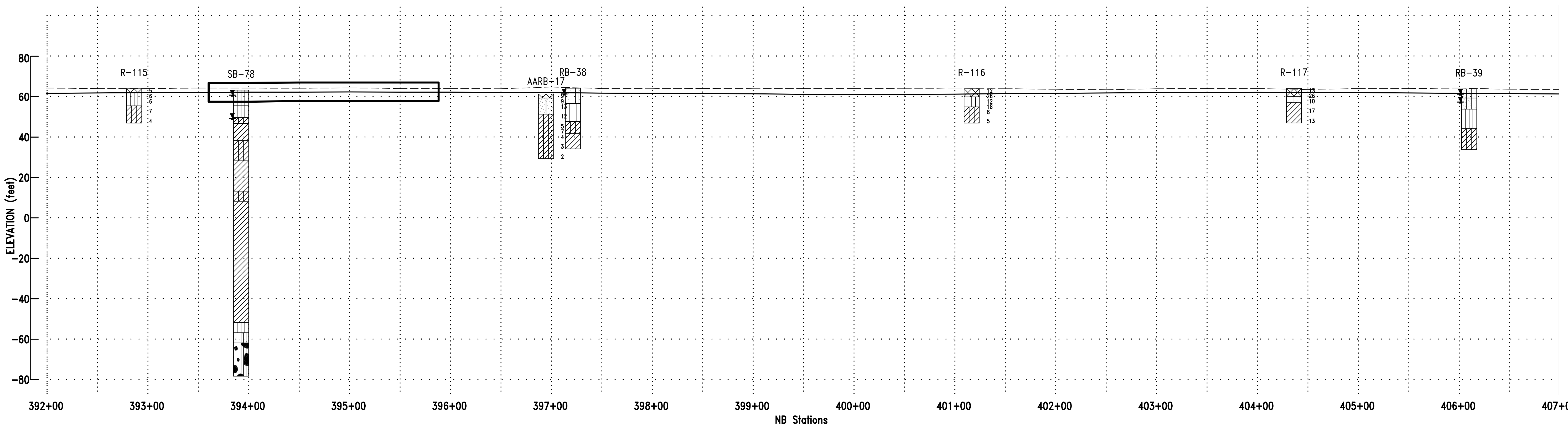
Special provisions will be required to address bitumen coating of piles, pile splicing, pile testing, and lightweight fill.

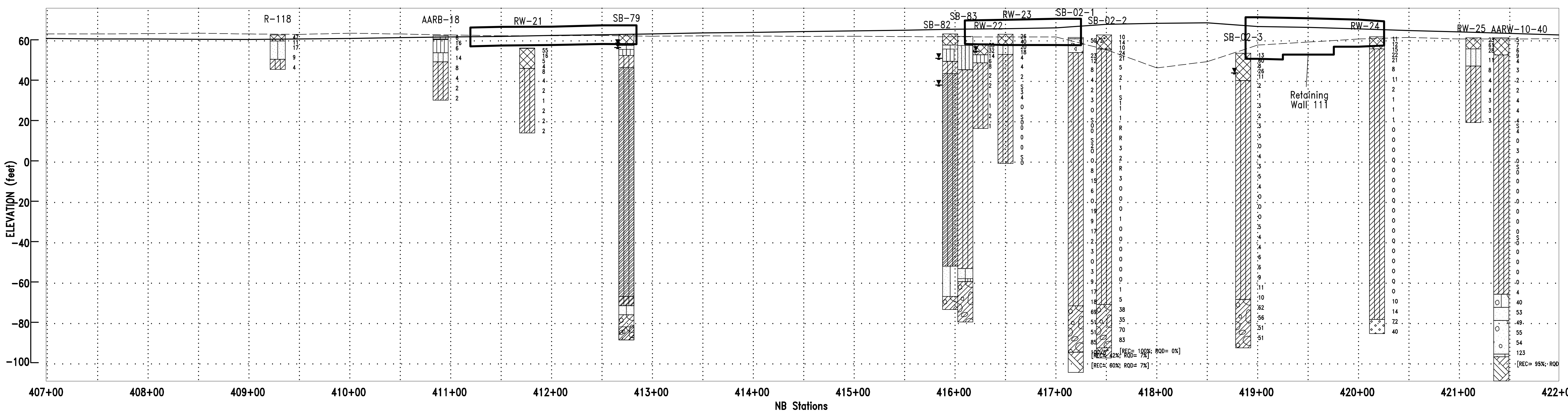
LEGEND

CPT-	2008 CPT LOCATION
SB-01- R- RW-	2008 TEST BORING
SB- RB-	PILOT BORING

DESIGNED BY MGB	NO. DATE DRWN. CHKD APPVD	BORINGS LOCATED BY CT DOT SURVEY.	PRIME DESIGNER: **URS Corporation** 500 Enterprise Dr. Suite 3B Rocky Hill, Ct.
DRAWN BY SMC	REVISIONS		GEODESIGN INCORPORATED — GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS, 984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762, TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842
CHECKED BY MGB			DWG. TITLE: RETAINING WALLS 111/112 AS-DRILLED EXPLORATION LOCATION PLAN
APPROVED BY ULF			PROJECT: NEW BRITAIN – HARTFORD BUSWAY STATE PROJECT NO. 155-H025

FILE NO. 380-04
SCALE 1" = 100'
DATE 5/26/09
FIGURE NO. 1





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 15)

GEODESIGN
INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

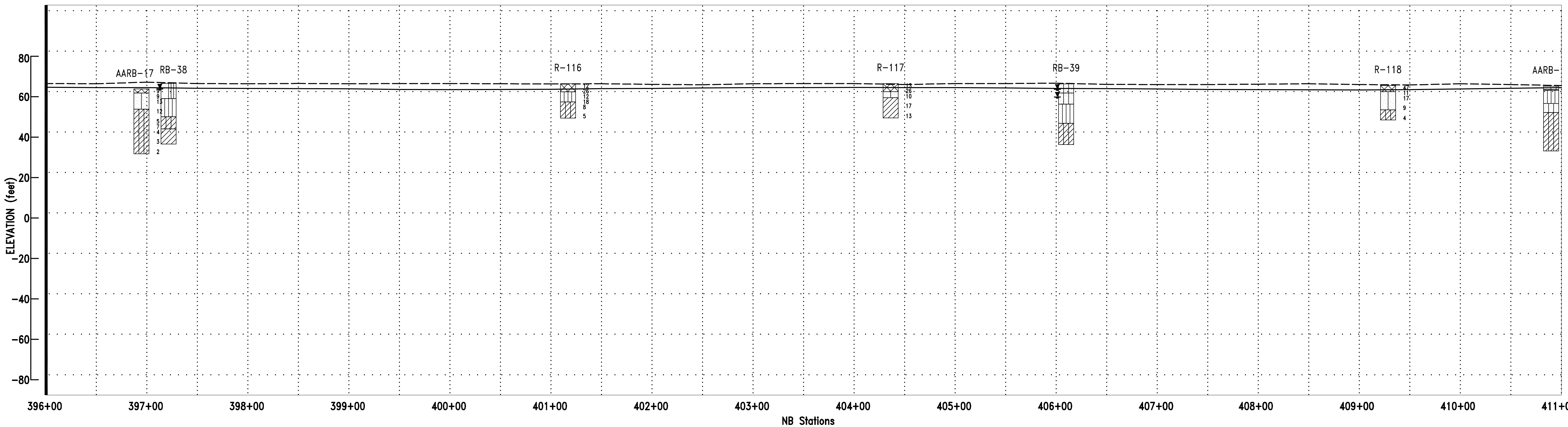
Vertical Exaggeration: 2x

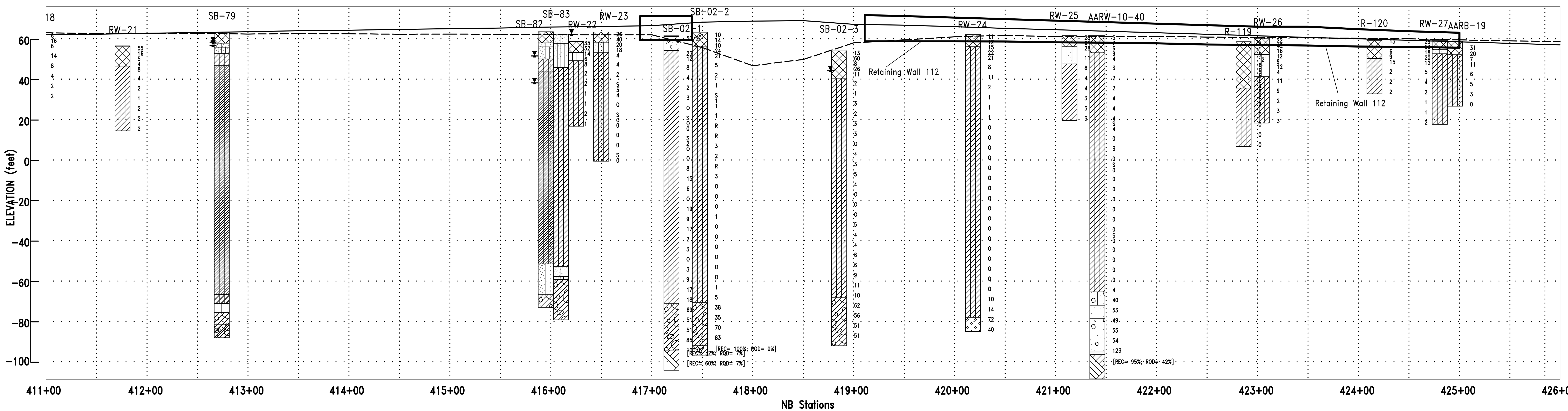
Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 111

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(north bound) 090327.dwg (SHEET 15)

GEODESIGN INCORPORATED
Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100
Horizontal Scale (feet)
Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

NORTHBOUND SUBSURFACE PROFILE FOR RETAINING WALL 112

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	3

INSERT TAB
113 and 114
HERE

Retaining Walls 113 and 114

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 113 and 114

Attached Figures:

RETAINING WALLS 113 and 114

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

These two walls will retain the proposed embankment west of proposed southbound Busway Over Capitol Ave., between Stations 436+00 and 437+40. Existing grades at Capitol Avenue in the area of the proposed South Bound bridge abutments range from Elev. 36 to 38.

The embankment in the area of this wall is located in close proximity to existing I-84 piers. To avoid imparting stresses and consolidation on the clay below these piers, the roadway embankment (including the wall backfill) will be constructed with geofoam.

A 21-inch diameter reinforced concrete (or clay) pipe is present underneath the centerline of Capitol Avenue. A 16-inch diameter water main is present along the south side of Capitol Avenue.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings-Supported Walls

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45
- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)

- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

We recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls .

7.6 Fill and Backfill Design Parameters

7.6.1 GeoFoam

GeoFoam is recommended behind these walls, and thus under the proposed Busway roadway and to the eastern and western extents of proposed fill limits. Figure 3 (appended) depicts the recommended transition of GeoFoam into the existing embankment. We recommend the GeoFoam block arrangement follow the arrangement depicted on Figure 4 (appended).

We recommend a depth of over excavation that will result in zero net stress (and thus zero settlement), plus 3-inches to account for the potential to moisture absorption. Therefore, assuming a three foot cover at the surface over the GeoFoam (granular fill and pavement), we recommend 3-feet 3-inches of over excavation prior to GeoFoam block placement. We recommend the granular fill cover consist of compacted normal 125 pcf backfill, followed by the chosen pavement section.

Groundwater is expected up to about three feet above bottom of geofoam elevations. Groundwater should be controlled using sump pumps. We recommend that water be lowered to bottom of geofoam until at least 24 inches of cover has been placed over the geofoam. We recommend a well be installed to provide a means to verify groundwater levels during construction.

Assuming an embankment width over 10 ft wide, a 2H:1V embankment, and an average clay shear strength of 700 psf, the factor of safety for external stability is greater than 3.5. Therefore, the pavement section type is not limited by the use of the GeoFoam and may consist of a flexible or rigid system.

We recommend a separation membrane between the GeoFoam and normal granular fill. This membrane will prevent migration of granular fill between the blocks, and will provide a barrier to protect the GeoFoam in the event of a petroleum spill. The separation membrane should consist of a 28mm minimum thickness, gasoline-resistant geomembrane. The membrane should be pitched at 1-percent toward the outside of the embankment.

For design of the walls backfilled with GeoFoam, we recommend the following static design parameters:

- Unit Weight of GeoFoam = 1.8 pcf
- Minimum Compressive Resistance @ 1% Deformation = 10.9 psi
- Minimum Compressive Resistance @ 5% Deformation = 24.7 psi
- Minimum Compressive Resistance @ 10% Deformation = 29.0 psi
- Minimum Elastic Modulus = 1,090 psi
- Minimum Flexural Strength = 50.0 psi
- Maximum Water Absorption by Total Immersion = 2.0% by volume
- ESCS Fill Angle of Internal Friction = 90°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0
- Coefficient of Passive Earth Pressure, we do not recommend using GeoFoam on the passive side of retaining wall
- Coefficient of Active Earth Pressure, $K_a = 0$

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

Where Geofoam is used we recommend a geo-composite type drainage used between the geofoam and back of retaining wall.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Existing utilities are present at Capitol Avenue where a new bridge is to be constructed. Refer to the Structure Layout for Design Report for details.

8.6 Monitoring of Amtrak Railway Tracks

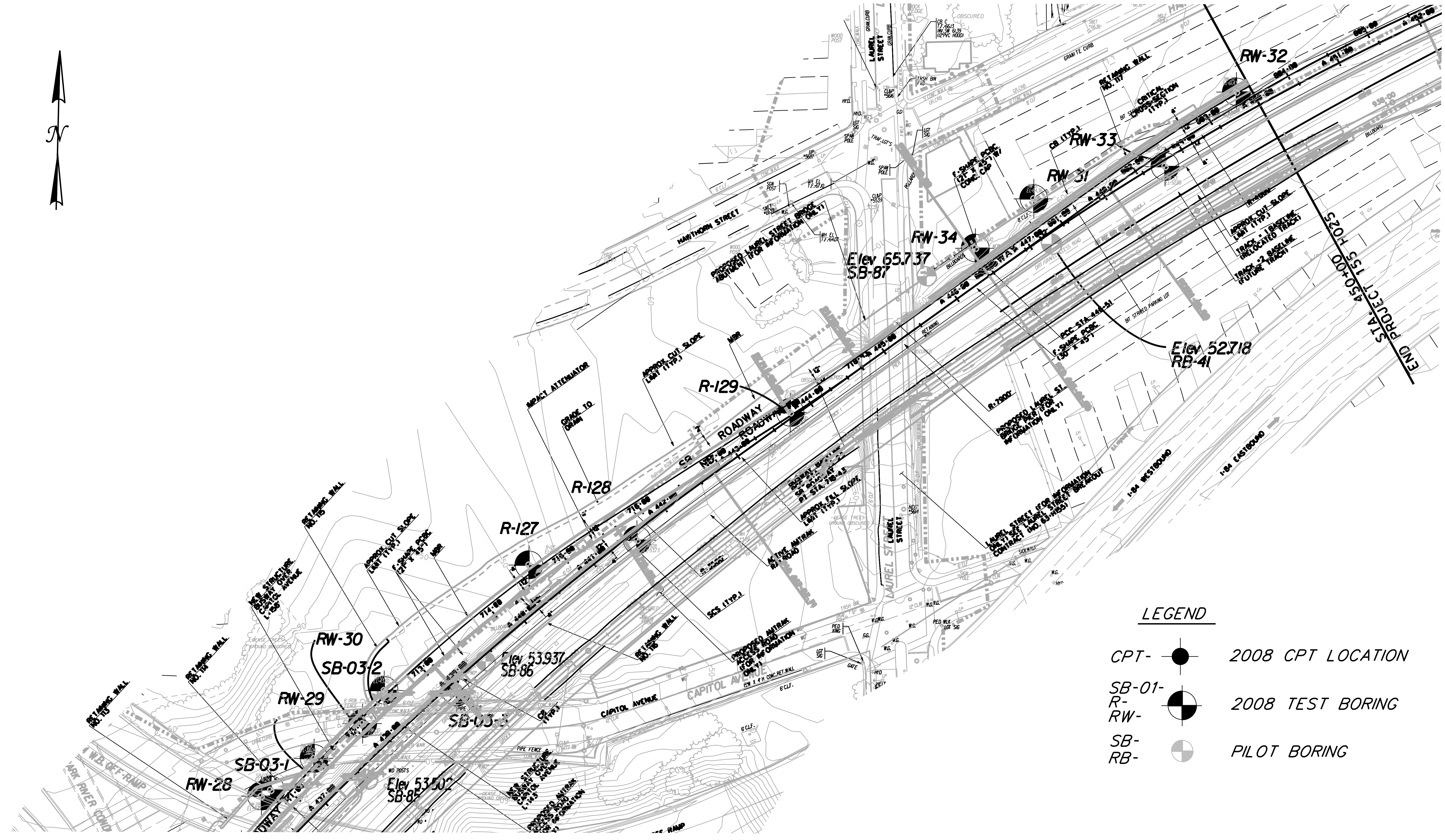
Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address GeoFoam.



DESIGNED BY	MGB
DRAWN BY	SMC
CHECKED BY	MGB
APPROVED BY	ULF

NO.	DATE		DRWN.	CHKD	APPVD

REVISIONS

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

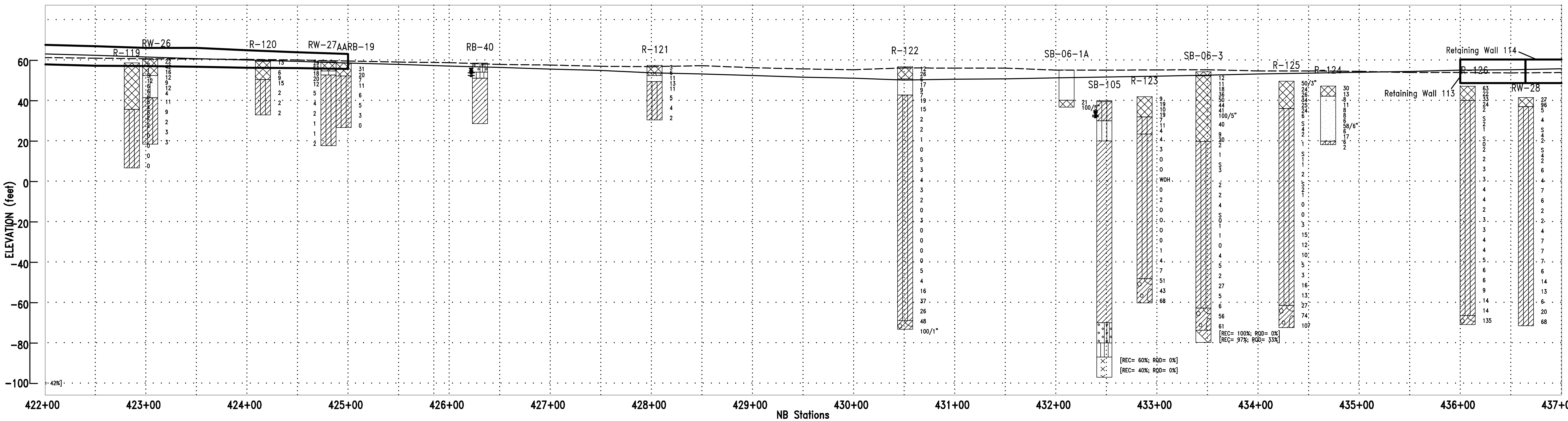
URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

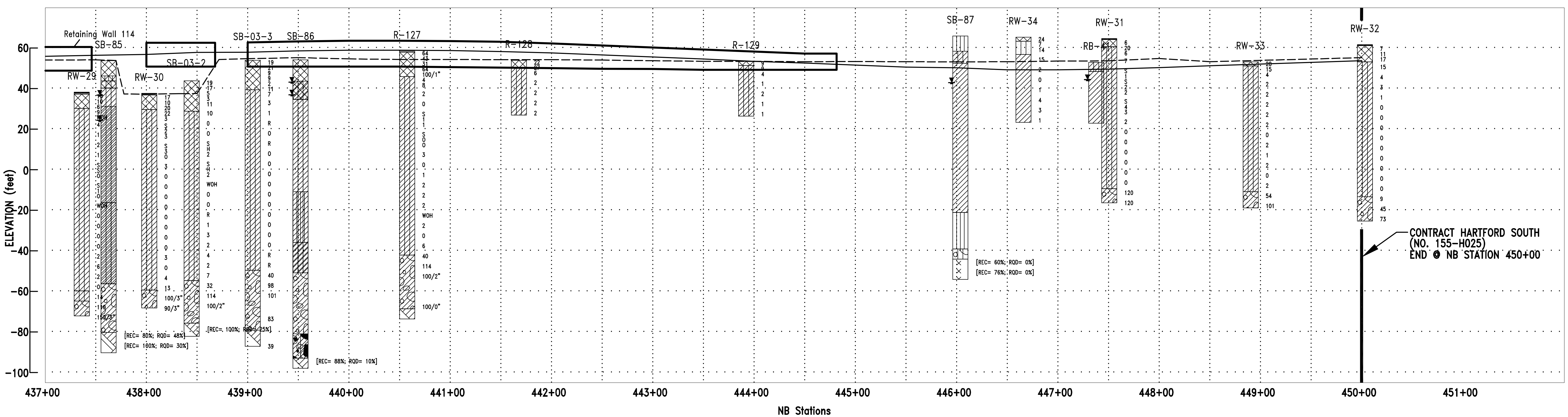
GEODESIGN INCORPORATED
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS
984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762
TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842

DWG. TITLE
RETAINING WALLS 113/114
AS-DRILLED EXPLORATION
LOCATION PLAN

PROJECT
NEW BRITAIN - HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

FILE NO. 380-04
SCALE 1" = 100'
DATE 5/26/09
FIGURE NO. 1





M:\CL\0380\04 CADD---NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(north bound) 090327.dwg (SHEET 16 NORTHBOUND)

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Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

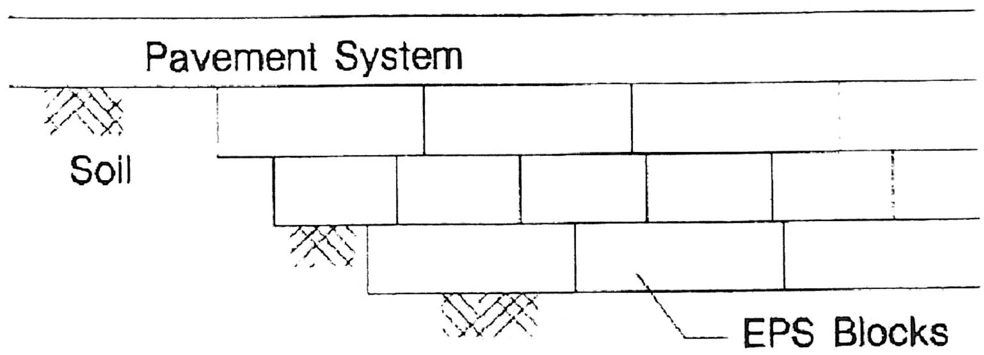
Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

NORTHBOUND SUBSURFACE PROFILE FOR RETAINING WALLS 113 AND 114

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2



(Source: Figure 8.2 NCHRP GeoFoam Applications in the Design and Construction of Highway Embankments)

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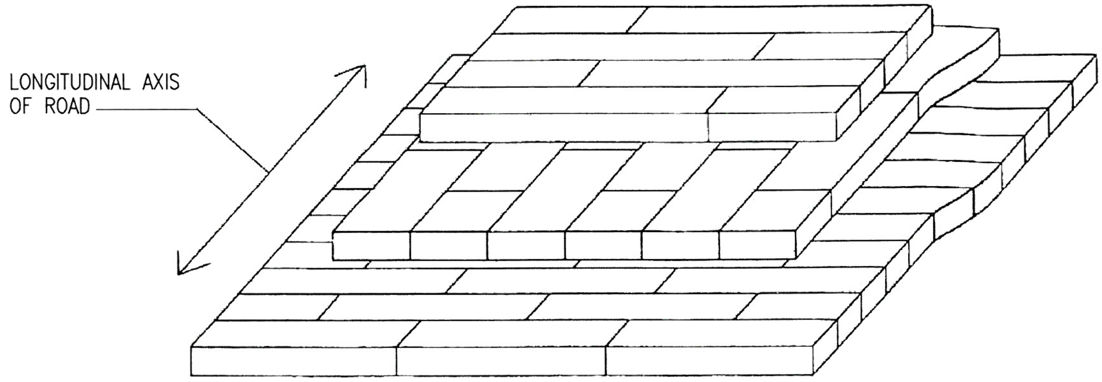
GEOFOAM DETAIL 1

Hartford Busway South
State Project No. 155-H025 & 63-643

DRAWN BY:	CHECKED BY:	DATE:	FIGURE NO.
SMC	MGB	12/23/2009	3



(Source: Figure 8.1 NCHRP GeoFoam Applications
in the Design and Construction of Highway Embankments)

GEODESIGN INCORPORATED	GEOFOAM DETAIL 2		
GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS 984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762 TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842	Hartford Busway South State Project No. 155-H025 & 63-643		
	DRAWN BY: SMC	CHECKED BY: MGB	DATE: 12/23/2009
	FIGURE NO. 4		

M:\CL\0380 - BUSWAY\04 HTFD SOUTH\FD Reports\Roadway\GEOFOAM DETAILS 1 & 2.dwg

INSERT TAB
115 and 116
HERE

Retaining Walls 115 and 116

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Walls 115 and 116

Attached Figures:

RETAINING WALLS 115 and 116

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

Wall 115 will retain the proposed embankment west of proposed southbound Busway Over Capitol Ave., between Stations 438+00 and 438+70. Wall 116 is West of the Amtrak RR tracks, between Stations 439+00 and 442+75. The existing ground surface ranges from Elev. 46 to 58 with vertical to 1H: 1V slopes varying from 2 to 8 feet high. To achieve proposed finished grades ranging from Elev. 55 to 60, embankment fill varying from 2 to 15 feet is proposed. The proposed fill is higher but will slope to the west on a 2H: 1V slope on the southbound side, and is lower but will be vertical on the northbound side. This wall will retain about 4 to 6.5 feet of vertical cut on the east side of the Busway alignment.

Existing grades at Capitol Avenue in the area of the proposed South Bound bridge abutments range from Elev. 36 to 38. A 21-inch diameter reinforced concrete (or clay) pipe is present underneath the centerline of Capitol Avenue. A 16-inch diameter water main is present along the south side of Capitol Avenue.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings

7.5.1 Design of Cast-in-Place Retaining Walls

We recommend the following static design parameters for cast-in-place walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ= 34° (regular fill) or 38° (lightweight fill)
- Load Factors should be selected from AASHTO LRFD Table 3.4.1-2.
- Factored Bearing Resistance (Service Limit State) = 2.7 ksf
- Factored Bearing Resistance (Strength Limit State) = 6.3 kips per square foot (ksf)
- Bearing Resistance Factor (φb) = 0.45

- Coefficient of Friction for Sliding = 0.55 (AASHTO LRFD Table 3.11.5.3-1)
- Coefficient of Friction for Soil against Wall (tan δ) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Sliding Resistance Factor (φ_τ) = 0.8 (AASHTO LRFD Table 10.5.5.2.2-1)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

The resistance factors provided above are for the Strength Limit State. In accordance with LRFD Section 10.5.5.1, resistance factors for the Service Limit State shall be taken as 1.0, except for global stability where the resistance factor shall be taken as 0.75. In accordance with LRFD Section 10.5.5.3.3, resistance factors for Extreme Limit State shall be taken as 1.0, except for uplift resistance of piles, where the resistance factor shall be taken as 0.80.

At wall 115, we recommend a 12-inch thick Granular Fill pad over granular soils or undisturbed, stiff to very stiff varved clay for all footing-supported retaining walls. At wall 116 we recommend a 24-inch thick granular fill pad over undisturbed, soft varved clay.

7.6 Fill and Backfill Design Parameters

7.6.1 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.6.2 GeoFoam

GeoFoam is recommended behind Retaining Wall 116, and thus under the proposed Busway roadway and to the eastern and western extents of proposed fill limits. Figure 3 (appended) depicts the recommended transition of GeoFoam into the existing embankment. We recommend the GeoFoam block arrangement follow the arrangement depicted on Figure 4 (appended).

We recommend a depth of over excavation that will result in zero net stress (and thus zero settlement), plus 3-inches to account for the potential to moisture absorption. Therefore, assuming a three foot cover at the surface over the GeoFoam (granular fill and pavement), we recommend 3-feet 3-inches of over excavation prior to GeoFoam block placement. We recommend the granular fill cover consist of compacted normal 125 pcf backfill, followed by the chosen pavement section.

Assuming an embankment width over 10 ft wide, a 2H:1V embankment, and an average clay shear strength of 700 psf, the factor of safety for external stability is greater than 3.5. Therefore, the pavement section type is not limited by the use of the GeoFoam and may consist of a flexible or rigid system.

We recommend a separation membrane between the GeoFoam and normal granular fill. This membrane will prevent migration of granular fill between the blocks, and will provide a barrier to protect the GeoFoam in the event of a petroleum spill. The separation membrane should consist of a 28mm minimum thickness, gasoline-resistant geomembrane. The membrane should be pitched at 1-percent toward the outside of the embankment.

For design of the walls backfilled with GeoFoam, we recommend the following static design parameters:

- Unit Weight of GeoFoam = 1.8 pcf
- Minimum Compressive Resistance @ 1% Deformation = 10.9 psi
- Minimum Compressive Resistance @ 5% Deformation = 24.7 psi
- Minimum Compressive Resistance @ 10% Deformation = 29.0 psi
- Minimum Elastic Modulus = 1,090 psi
- Minimum Flexural Strength = 50.0 psi
- Maximum Water Absorption by Total Immersion = 2.0% by volume
- ESCS Fill Angle of Internal Friction = 90°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0
- Coefficient of Passive Earth Pressure, we do not recommend using GeoFoam on the passive side of retaining wall
- Coefficient of Active Earth Pressure, $K_a = 0$

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed at Retaining Wall 115 and connected to roadway drainage ; and bagged stone and weep holes should be utilized at Retaining Wall 116.

Where Geofoam is used we recommend a geo-composite type drainage used between the geofoam and back of retaining wall.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. These retaining walls will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

8.2 Reuse of Excavated Materials

Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Existing utilities are present at Capitol Avenue where a new bridge is to be constructed. Refer to specific Structure Layout for Design Reports for details.

8.6 Monitoring of Amtrak Railway Tracks

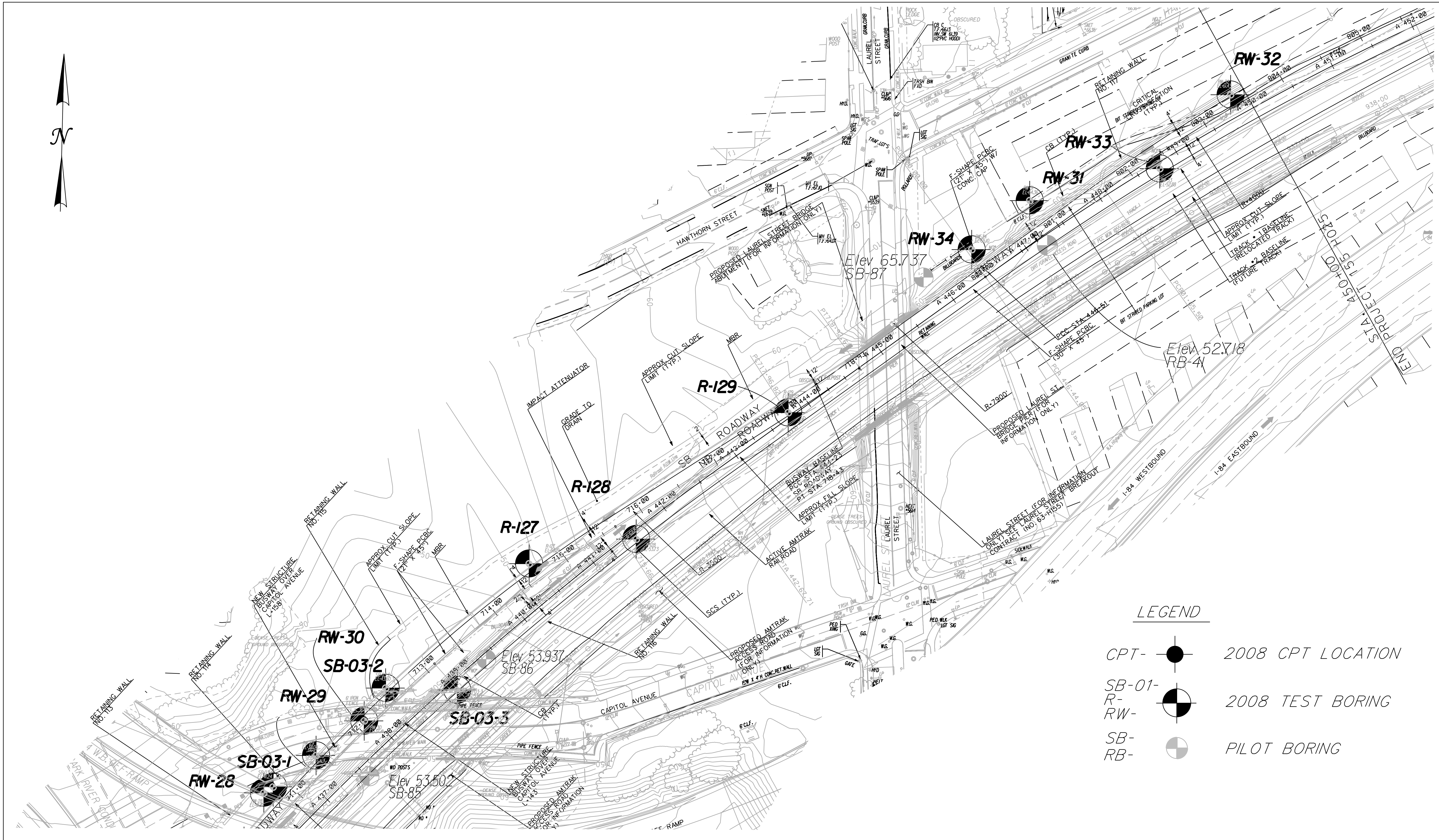
Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address GeoFoam and lightweight fill.



DESIGNED BY MGB	NO.	DATE	DRWN.	CHKD	APPVD	BORINGS LOCATED BY CT DOT SURVEY.	PRIME DESIGNER: **URS Corporation** 500 Enterprise Dr. Suite 3B Rocky Hill, Ct.	GEODESIGN INCORPORATED GEOTECHNICAL ENGINEERS • ENVIRONMENTAL CONSULTANTS 984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762 TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842
DRAWN BY SMC	REVISIONS							
CHECKED BY MGB								
APPROVED BY ULF								

DWG. TITLE: RETAINING WALLS 115/116 AS-DRILLED EXPLORATION LOCATION PLAN

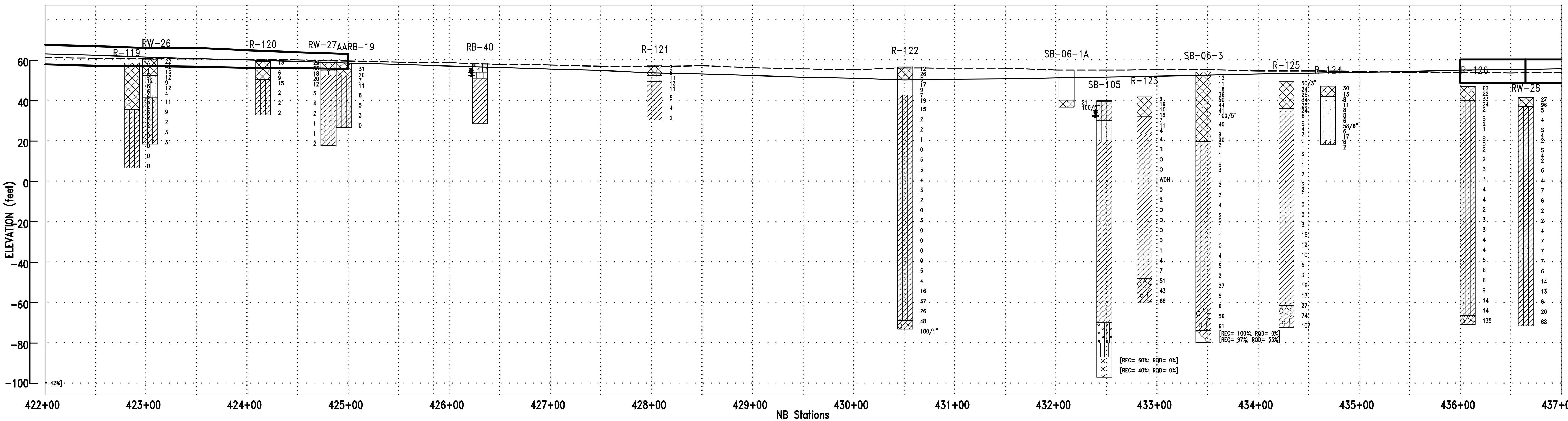
PROJECT: NEW BRITAIN – HARTFORD BUSWAY STATE PROJECT NO. 155-H025

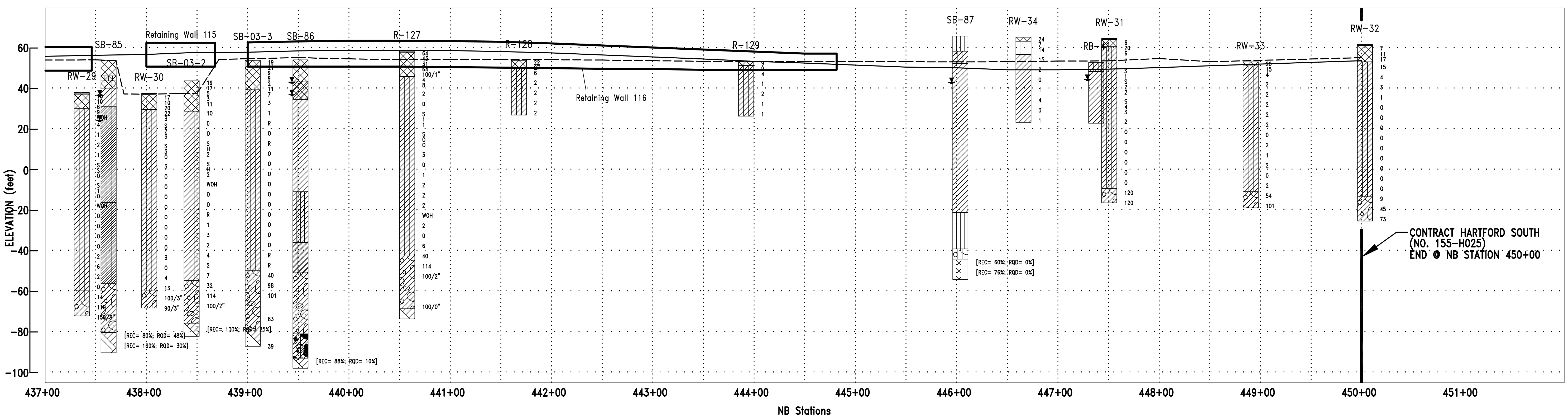
FILE NO. 380-04

SCALE 1" = 100'

DATE 5/26/09

FIGURE NO. 1





M:\CL\0380\04\CADD----NewDirectory Structure----8-08\Hartford South RetWall-Roadway with retaining wall(north bound) 090327.dwg (SHEET 16 NORTHBOUND)

GEODESIGN INCORPORATED

Geotechnical & Construction Engineers - Environmental Consultants
984 Southford Road
Middlebury, Connecticut 06762
Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

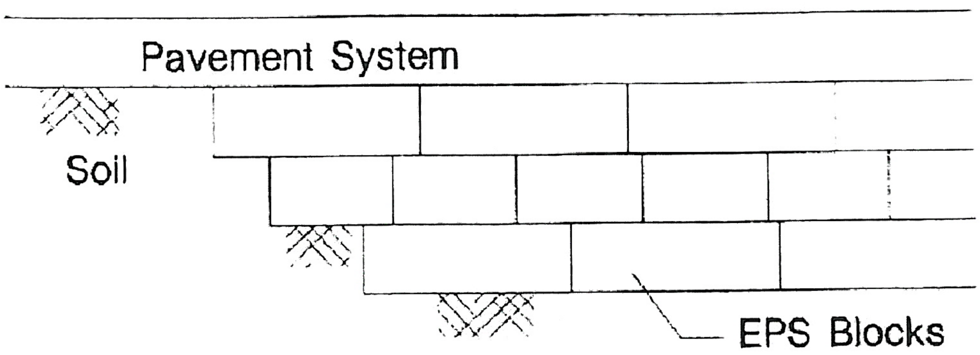
Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

NORTHBOUND SUBSURFACE PROFILE FOR RETAINING WALLS 115 AND 116

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2



(Source: Figure 8.2 NCHRP GeoFoam Applications in the Design and Construction of Highway Embankments)

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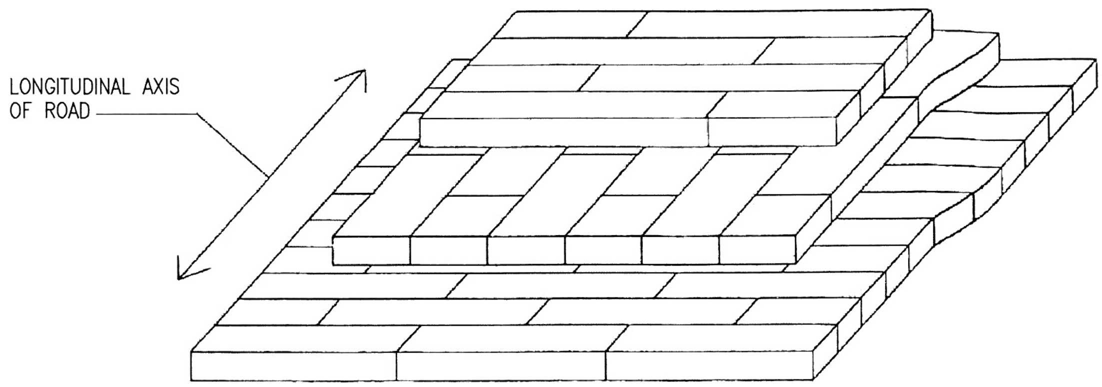
GEOFOAM DETAIL 1

Hartford Busway South
State Project No. 155-H025 & 63-643

DRAWN BY:	CHECKED BY:	DATE:	FIGURE NO.
SMC	MGB	12/23/2009	3

M:\62\0360 BUSWAY\04 HTFD SOUTH\FD Reports\Roadway\GEOFOAM DETAILS 1 & 2.dwg



(Source: Figure 8.1 NCHRP GeoFoam Applications in the Design and Construction of Highway Embankments)

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GEOFOAM DETAIL 2

Hartford Busway South
State Project No. 155-H025 & 63-643

DRAWN BY:	CHECKED BY:	DATE:	FIGURE NO.
SMC	MGB	12/23/2009	4

INSERT TAB
117
HERE

Retaining Wall 117

Wall-Specific Information & Recommendations

Wall-Specific Table of Contents
for Retaining Wall 117

Attached Figures:

RETAINING WALL 117

Wall-Specific Information & Recommendations

This section (tab) should be used in conjunction with the preceding section, which is common to all retaining walls.

1.0 GENERAL INFORMATION

1.3 Existing Conditions and Proposed Construction

West of the Amtrak RR tracks, between Stations 445+75 and 450+00, the ground surface slopes down from about Elev. 65 to Elev. 53 on a 1H: 1V slope. The slope will need to be cut to construct the proposed Busway. The east side of Retaining Wall 117 will retain about 7 to 10 feet of cut, while the west side adjacent to Laural Street Bridge will retain up to 21 feet.

7.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

7.4 Pile Foundations

Not Applicable

7.5 Parameters for Spread Footings

7.5.1 Design of Proprietary Retaining Walls

Mechanically Stabilized Earth (MSE) or prefabricated modular walls (such as Double-Wal or T-Wall) are feasible alternate wall types.

We recommend the following static design parameters for proprietary retaining walls:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, φ = 34° (regular fill) or 38° (lightweight fill)
- Allowable Bearing Capacity = 2.0 kips per square foot (ksf)
- Coefficient of Friction for Sliding = 0.55
- Coefficient of Friction for Soil against Wall ($\tan \delta$) = 0.40 (regular fill) or 0.45 (lightweight fill)
- Coefficient of Passive Earth Pressure, K_P = 3.5 (regular fill) or 4.0 (lightweight fill)
- Coefficient of Active Earth Pressure, Ka = 0.28 (regular fill) or 0.25 (lightweight fill)
- Earth pressure calculations should assume a surface surcharge of 24 inches soil depth or 250 psf.

We recommend a 24-inch thick granular fill pad over undisturbed, soft varved clay.

7.6 Fill and Backfill Design Parameters

7.6.1 Regular Fill

For design of the walls backfilled with regular fill (e.g. not lightweight), we recommend the following static design parameters:

- Unit weight of soil above the water table of 125 pcf
- Unit weight of soil below the water table of 62.6 pcf
- Soil Angle of Internal Friction, phi = 34°

7.6.2 Expanded Shale, Clay and Slate (ESCS) Fill

For design of the walls backfilled with 60 pcf lightweight fill, we recommend the following static design parameters:

- Assumed Expanded Shale, Clay and Slate (ESCS) Fill Backfill Material
- Unit Weight of ESCS Lightweight Fill = 60 pcf
- ESCS Fill Angle of Internal Friction = 38°
- Coefficient of Friction for Sliding of Footing over Sand or Crushed Stone = 0.55
- Coefficient of Friction for Soil against Wall, tan delta = 0.45

7.7 Seismic Design

AASHTO LRFD Section 4.7.4.1 states that the bridges in Seismic Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry; accordingly, recommendations for dynamic lateral earth pressures are not included. However, the minimum requirements, as specified in Section 4.7.4.4 and 3.10.9, shall apply.

7.8 Drainage

Drainage details for retaining walls should be constructed in accordance with ConnDOT Bridge Design Manual specifications for walls and abutments. Specifically, six-inch underdrains should be installed and connected to roadway drainage.

7.9 Wall Stability

As shown in Table 2, Appendix 2, Vol. I, the shear strengths of Varved Clay vary from 500 psf to 900 psf along the Busway alignment and averages at about 700 psf. The tallest retaining walls (Walls 102 and 105) are located at Flatbush Ave. Assuming 700 psf and 26 feet high of 60 pcf lightweight fill, the calculated factor of safety against global stability exceeds 1.5. This retaining wall will be smaller and will impart lower stresses to the Varved Clay stratum. Therefore, by inspection, the resulting safety factor against global stability will exceed 1.5.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Subgrade Preparation

Subgrade preparation for shallow spread footings should be conducted in such a way as to minimize disturbance. The final six inches of excavation should be performed with a smooth-edged bucket or a clip attached to the bucket of the excavator or, alternatively, hand shoveling of the loose, disturbed material such that the subgrade remains essentially undisturbed.

Construction operations should be planned to mitigate disturbance to the final subgrade. Disturbed subgrades should be over-excavated to firm stable ground and replaced by Granular Fill, Compacted Granular Fill, or crushed stone wrapped in a non-woven filter fabric. Granular Fill should be used when fill depth is less than two feet, and Compacted Granular Fill should be used when fill depth is greater than two feet.

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Some excavated existing granular materials are suitable for reuse as embankment fill (ConnDOT Form 816 Section 2.02.03.5) after testing and geotechnical engineer's approval. Excavated Silts and Clays are not expected to be suitable for reuse on the project, except for placement of "unsuitable" materials in the outer slopes of an embankment as indicated on ConnDOT Standard Drawing No. 201. No excavated materials are expected to be suitable for re-use as Granular Fill, Compacted Granular Fill, or Pervious Structure Backfill.

8.3 Protection of Existing Railroad

Base on anticipated depth of excavations required to construct the wall foundation and a range of approximately 15 to 50 feet between Track No. 2 and the proposed walls, we do not anticipate the need for protection of live railroad tracks.

8.4 Vibrations and Construction-Induced Settlements

Not Applicable

8.5 Monitoring of Utilities

Not Applicable

8.6 Monitoring of Amtrak Railway Tracks

Not Applicable

8.7 Dewatering

Groundwater will be encountered during foundation installation. Therefore, Contractors should be prepared to control groundwater. Dewatering will be especially critical in areas where proposed foundation subgrades will be close to or/and the Varved Clay stratum.

9.0 SPECIAL PROVISIONS

Special provisions will be required to address lightweight fill.

DESIGNED BY	MGB
DRAWN BY	SMC
CHECKED BY	MGB
APPROVED BY	ULF

NO.	DATE		DRWN.	CHKD	APPVD
		REVISIONS			

BORINGS LOCATED BY CT DOT SURVEY.

PRIME DESIGNER:

URS Corporation
500 Enterprise Dr.
Suite 3B
Rocky Hill, Ct.

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984 SOUTHFORD ROAD • MIDDLEBURY CONNECTICUT 06762
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DWG. TITLE
RETAINING WALL 117
AS-DRILLED EXPLORATION
LOCATION PLAN

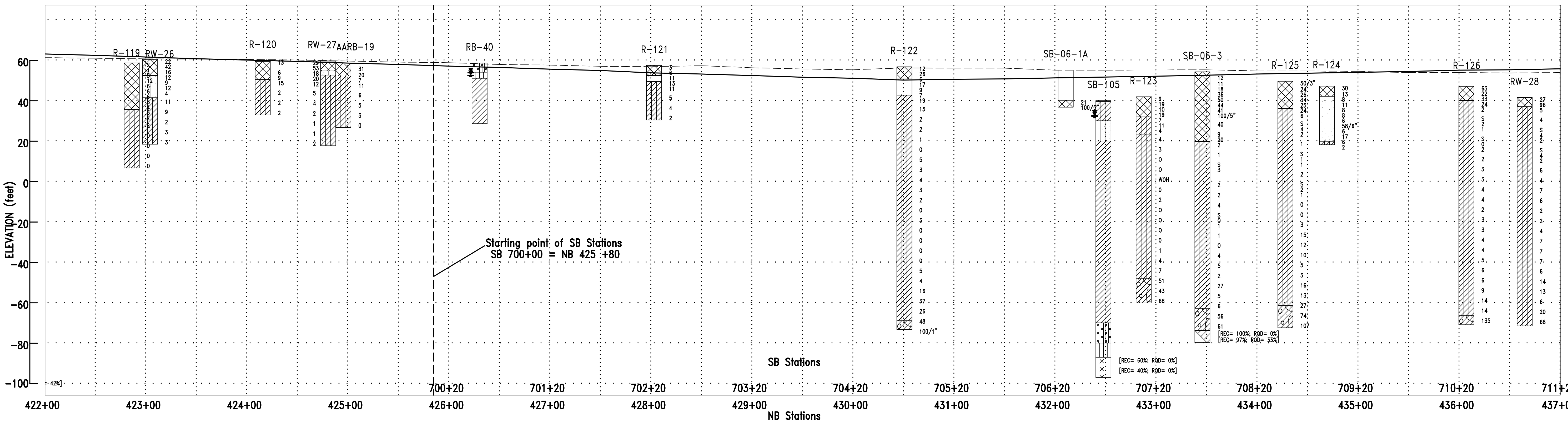
PROJECT
NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

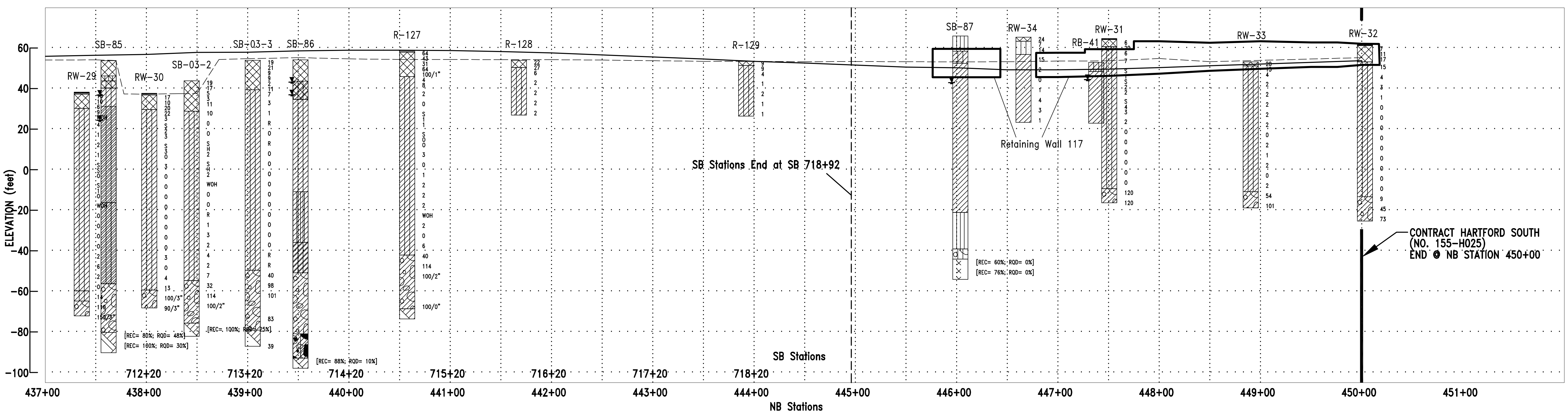
FILE NO. 380-04

SCALE 1" = 100'

DATE 5/26/09

FIGURE NO. 1





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Telephone: 203-758-8836 Fax: 203-758-8842

0 100

Horizontal Scale (feet)

Vertical Exaggeration: 2x

Notes:
1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown.
2. Refer to plan view for subsurface profile location. For strata details and symbol legend, see Subsurface Profile Legend and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval. Where coring was performed, numbers represent Recovery and RQD values.
4. Profile location is approximate centerline of proposed roadway.

SOUTHBOUND SUBSURFACE PROFILE FOR RETAINING WALLS 117

Busway Hartford South
GeoDesign Project No. 0380-004.0
CT DOT Project No. 155-H025

Date:	5/26/09	Drawn By:	DL/MBF	Reviewed By:	MGB	File No.:	0380-004.0	Figure No.:	2

M:\CL\0380\04\CADD---NewDirectory Structure---8-08\Hartford South RetWall-Roadway with retaining wall(south bound) 090327.dwg (SHEET 16 NORTHBOUND)

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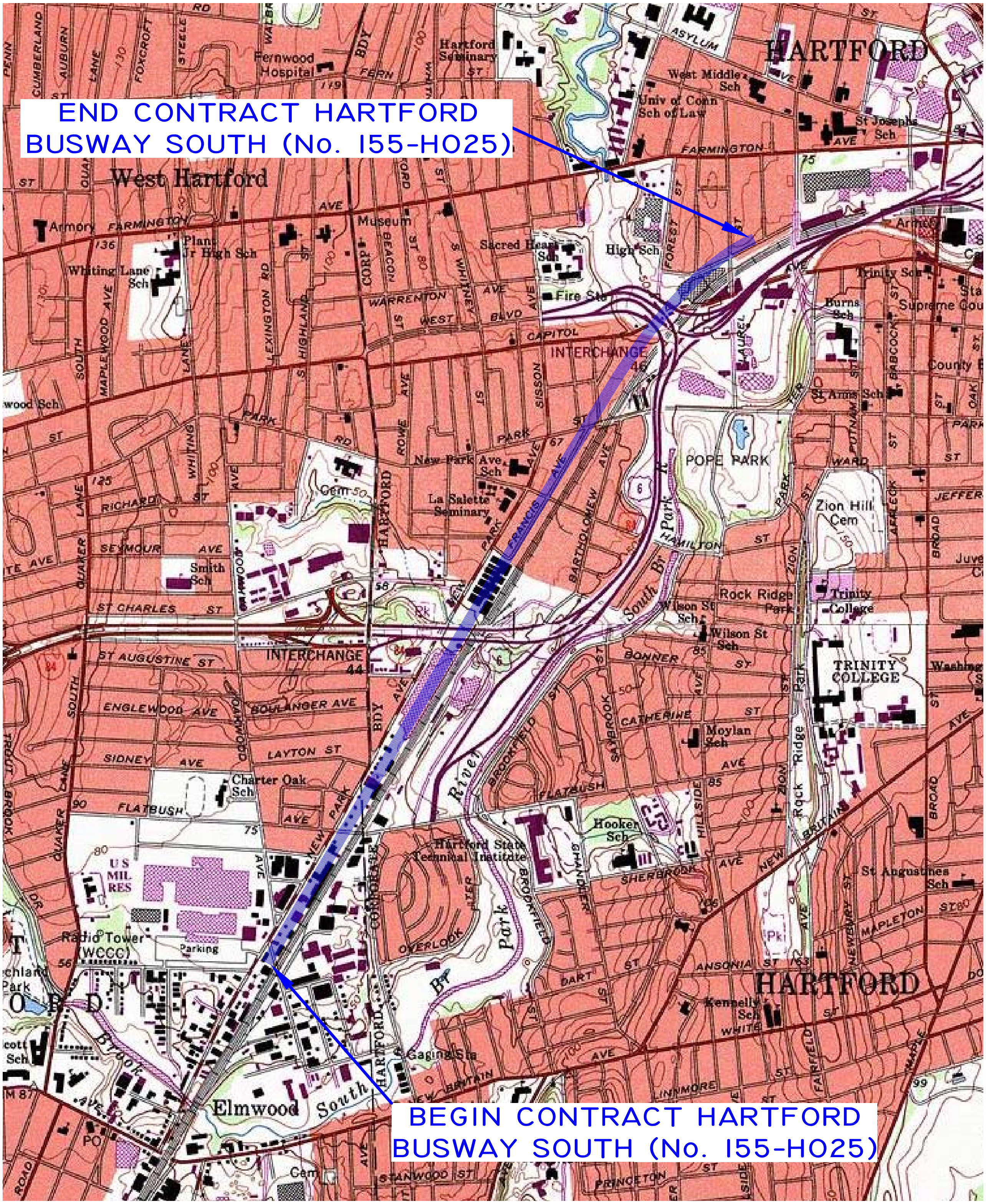
Appendix 1

Figures



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TELEPHONE: (203)758-8836 FACSIMILE: (203)758-8842

DRAWN BY: SMC | REVIEWED BY: CZ

NEW BRITAIN – HARTFORD BUSWAY
STATE PROJECT NO. 155-H025

REFERENCE:
U.S.G.S. 7.5 MINUTE QUADRANGLE: NEW BRITAIN, HARTFORD, CT. Figure was created using TOPO! 2003 software

SCALE IN FEET
0 1000 2000 4000

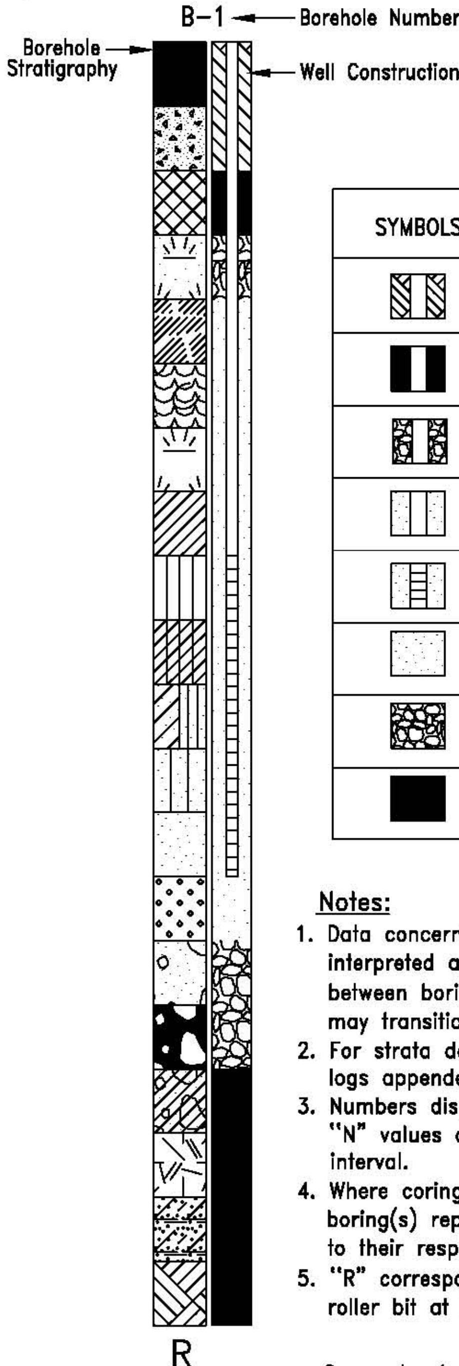
PROJECT NO. 380-04
DATE: 3-18-09
FIGURE NO. 1

\CL\0380 BUSWAY\04 HTFD SOUTH\------CADD------- (NEW DIRECTORY STRUCTURE – 8-08)\PROJECT AREA MAP.dwg, 8X11 LOCUS, 3/20/2009 3:26:51 PM, -SMC

STRATIGRAPHY SYMBOLS

SYMBOLS	TYPICAL DESCRIPTIONS OF PREDOMINENT MATERIAL TYPE
	ASPHALT
	CONCRETE
	FILL
	TOPSOIL
	SUBSOIL
	ORGANIC SILT OR CLAY WITH SHELLS
	PEAT
	CLAY
	SILT
	CLAY/SILT MIXTURE
	CLAY/SILT/SAND MIXTURE
	SILT/SAND MIXTURE
	SAND/SILT MIXTURE
	POORLY-GRADED SAND
	WELL-GRADED SAND
	SAND/GRAVEL MIXTURE
	SAND/GRAVEL/SILT MIXTURE
	BOULDERS AND/OR COBBLES
	GLACIAL TILL
	DECOMPOSED BEDROCK
	SANDSTONE
	BEDROCK

EXPLANATION OF BORING



WELL SYMBOLS

SYMBOLS	TYPICAL DESCRIPTIONS
	CEMENT SEAL: 1 PIPE
	BENTONITE SEAL: 1 PIPE
	SLOUGH BACKFILL: 1 PIPE
	FILTER PACK: 1 PIPE
	SLOTTED PIPE WITH FILTER PACK: 1 PIPE
	FILTER PACK AT BOTTOM OF HOLE
	SLOUGH AT BOTTOM OF HOLE
	BENTONITE AT BOTTOM OF HOLE

Notes:

1. Data concerning the various strata have been interpreted at boring locations only. The stratigraphy between borings may vary from that shown, and may transition more gradually within borings.
2. For strata details, see Report and boring logs appended to this report.
3. Numbers displayed beside boring(s) represent SPT "N" values corresponding to their respective sampling interval.
4. Where coring was performed, numbers displayed beside boring(s) represent Recovery and RQD values corresponding to their respective sampling interval.
5. "R" corresponds to refusal of sampler, casing and/or roller bit at bottom of boring.

Groundwater Observations (where applicable)

▽ Water Level Reading at time of drilling.

▼ Water Level Reading after completing drilling.

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SUBSURFACE PROFILE LEGEND

Hartford Busway South
State Project No. 155-H025

DRAWN BY:	CHECKED BY:	DATE:	FIGURE NO.
MJV	CZ	3/04/2009	2

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Appendix 2

Tables and Charts

New Britain - Hartford Busway
Retaining Walls
CT DOT Project Number: 155-H025
West Hartford and Hartford, Connecticut
GeoDesign Project Number: 0380-004.0

Table 1 - Observation Well Readings

Well Location	Location	Ground Surface Elevation	June 10, 2008		June 12, 2008		June 16, 2008		June 18, 2008		June 20, 2008		June 27, 2008		October 24, 2008	
			Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)	Depth (ft.)	Elevation (ft.)
SB-01-3	Flat Bush Ave. Over Busway	69.6	**-**	-	**2.3**	67.3	**2.0**	67.6	**1.7**	67.9	**1.7**	67.9	**1.9**	67.8	**2.3**	67.3
RW-2	Flat Bush Ave. Over Busway	74.7	**-**	-	**-**	-	**-**	-	**-**	-	**2.1**	72.6	**3.9**	70.8	**4.2**	70.5
RW-7	Flat Bush Ave. Over Busway	69.7	**-**	-	**-**	-	**8.7**	61.0	**8.5**	61.2	**8.6**	61.1	**6.6**	63.1	**6.5**	63.2
SB-02-3	Busway Over Park St.	54.0	**8.7**	45.3	**9.0**	45.0	**8.7**	45.3	**8.5**	45.5	**8.7**	45.3	**8.4**	45.6	**8.8**	45.2
SB-03-1	Busway Over Capitol Ave.	41.2	**7.0**	34.2	**7.0**	34.2	**7.2**	34.0	**5.3**	35.9	**5.8**	35.5	**6.0**	35.3	**6.3**	34.9
SB-03-2	Busway Over Capitol Ave.	43.6	**8.2**	35.4	**9.7**	33.9	**8.8**	34.8	**8.8**	34.8	**9.0**	34.6	**9.1**	34.6	**8.8**	34.8
SB-05-1	Kane Brook Culvert	43.4	**9.4**	34.0	**9.3**	34.1	**8.8**	34.6	**9.0**	34.4	**9.0**	34.4	**9.1**	34.4	**9.0**	34.4
RW-16	Kane Brook Culvert	46.6	**12.5**	34.1	**11.0**	35.6	**11.0**	35.6	**11.0**	35.6	**4.0**	42.6	**11.0**	35.6	**11.2**	35.4
SB-06-4	Old Park River	47.8	**12.9**	34.9	**14.5**	33.3	**14.5**	33.3	**14.5**	33.3	**14.5**	33.3	**14.4**	33.4	**14.0**	33.8

New Britain - Hartford Busway
Retaining Walls
CT DOT Project Number: 155-H025
West Hartford and Hartford, Connecticut
GeoDesign Project Number: 0380-004.0

Table 2 - Consolidation Test Results and Calculated Shear Strength

Boring No.	Sample No.	Sample Depth (ft)	Tested By	Lab Test No.	Type of Test[1]	Strain at In-situ Stress	SQD[2]	Estimated In-situ Stress (psf)	RR	C_V[3] (ft2/day)	Wn(%)	σ'_p (psf)	OCR[4]	$S_{u,DSS}$[5] (psf)	E(ksf)	E_{ur}(ksf)	Estimated $C\alpha$[6]	Location
RW-4	UP2	46	GTX	C-1	IL	6.0%	D	2500	0.037	0.24	57.6	4000	1.60	605			0.0017	Flatbush Ave. Over Busway
RW-4	UP2	46	GTX	CRC-1	CRS	8.0%	E	2500	0.036	0.20	57.6	4800	1.92	693			0.0016	Flatbush Ave. Over Busway
RW-4	UP1	36	Test Con		IL	2.6%	C	2000	0.032	0.20	54.3	4000	2.00	572			0.00144	Flatbush Ave. Over Busway
RW-6	UP1	21	GTX	CRC-5	CRS	3.5%	C	1400	0.020	0.30	43.7	6500	4.64	753			0.0009	Flatbush Ave. Over Busway
RW-6	UP2	31	GTX	CRC-6A	CRS	3.2%	C	1800	0.030	0.50	61	5000	2.78	658			0.0014	Flatbush Ave. Over Busway
RW-7	UP1	31	GTX	CRC-2A	CRS	2.1%	C	1700	0.028	1.00	55.2	4000	2.35	549			0.0013	Flatbush Ave. Over Busway
RW-8	UP3	41	GTX	C-2	IL	10.0%	E	2500	0.032	0.42	59.5	4000	1.60	605			0.0014	Flatbush Ave. Over Busway
RW-8	UP2	31	Test Con		IL	2.3%	C	2000	0.030	0.23	50.9	3800	1.90	550			0.00135	Flatbush Ave. Over Busway
RW-8	UP1	21	Test Con		IL	2.5%	C	1650	0.035	0.20	55	2800	1.70	417			0.001575	Flatbush Ave. Over Busway
SB-01-1	UP1	21	Test Con		IL	1.8%	B	1300	0.030	0.28	46.2	4000	3.08	513			0.001368	Flatbush Ave. Over Busway
SB-01-1	UP2	31	UMass	CRS141	CRS	1.5%	B	1750	0.030	1.06	53	4835	2.76	638	120	450	0.0014	Flatbush Ave. Over Busway
SB-01-2	UP1	16	GTX	CRC-3	CRS	2.0%	B	1300	0.013	1.00	67.6	6300	4.85	722			0.0006	Flatbush Ave. Over Busway
SB-01-2	UP2	26	Test Con		IL	2.6%	C	1800	0.030	0.25	51.2	4000	2.22	557			0.00135	Flatbush Ave. Over Busway
SB-01-4	UP1	31	Test Con		IL	2.8%	C	2100	0.034	0.24	53.9	4000	1.90	579			0.00153	Flatbush Ave. Over Busway
SB-01-4	UP2	41	UMass	IL474	IL	2.0%	B	2100	0.031	0.80	54	4100	1.95	590			0.0014	Flatbush Ave. Over Busway
SB-01-4	UP2	41	UMass	CRS139	CRS	3.0%	C	2100	0.038	0.18	59	5159	2.46	700	110	240	0.0017	Flatbush Ave. Over Busway
SB-02-1	UP1	41	UMass	IL475	IL	3.0%	C	2500	0.040	0.20	69	3400	1.36	535			0.0018	Busway Over Park St.
SB-02-1	UP1	41	UMass	CRS143	CRS	2.3%	C	2500	0.048	0.12	69	5480	2.19	766			0.0022	Busway Over Park St.
SB-02-1	UP2	51	UMass	CRS142	CRS	3.2%	C	3000	0.042	0.10	70	6000	2.00	858			0.0019	Busway Over Park St.
RW-23	UP1	26	Test Con		IL	2.5%	C	1600	0.034	0.18	51.4	4400	2.75	581			0.00153	Busway Over Park St.
RW-23	UP2	41	Test Con		IL	3.5%	C	2200	0.033	0.24	57.7	4000	1.82	586			0.001485	Busway Over Park St.
SB-03-1	UP1	36	GTX	CRC-12	CRS	4.0%	D	1900	0.040	0.10	58	5500	2.89	717			0.0018	Busway Over Capitol Ave.
SB-03-1	UP2	46	Test Con		IL	3.0%	C	2400	0.035	0.09	48.5	5600	2.33	770			0.001575	Busway Over Capitol Ave.
SB-03-2	UP2	41	GTX	CRC-13	CRS	6.0%	D	2500	0.014	0.70	57	6800	2.72	900			0.0006	Busway Over Capitol Ave.
RW-28	UP1	16	GTX	CRC-8	CRS	3.2%	C	1000	0.026	0.20		4300	4.30	508			0.001188	Busway Over Capitol Ave.
RW-28	UP2	26	Test Con		IL	2.5%	C	1440	0.041	0.82	51.3	5800	4.03	696			0.001845	Busway Over Capitol Ave.
RW-29	UP3	48	Test Con		IL	3.0%	C	2300	0.039	0.08	50.8	6000	2.61	803			0.0017325	Busway Over Capitol Ave.
RW-30	UP1	16	Test Con		IL	2.0%	B	1680	0.035	0.18	58.8	3400	2.02	485			0.001575	Busway Over Capitol Ave.
SB-06-1A	UP1	46	Test Con		IL	6.0%	D	3000	0.025	0.08	51.1	2800	0.93	484			0.001125	Old Park River
SB-06-3	UP2	71	Test Con		IL	7.8%	E	3850	0.032	0.15	55	3000	0.78	543			0.00144	Old Park River
SB-06-3	UP1	46	UMass	CRS144	CRS	4.8%	D	2784	0.038	0.18	60	3000	1.08	501			0.0017	Old Park River
SB-06-3	UP1	46	UMass	CRS140	CRS	5.6%	D	2784	0.050	0.12	64	3000	1.08	501			0.0023	Old Park River
SB-06-4	UP1	26	GTX	CRC-11	CRS	5.5%	D	2200	0.011	0.10	46	3000	1.36	472			0.0005	Old Park River
SB-06-4	UP2	46	GTX	CRC-14	CRS	6.0%	D	3000	0.030	0.10	59	5200	1.73	770			0.0014	Old Park River
R-125	UP2	36	GTX	CRC-10	CRS	4.8%	D	3100	0.034	0.10	64	6000	1.94	865			0.00153	Old Park River
R-125	UP3	51	GTX	CRC-4	CRS	8.8%	E	3750	0.030	0.05	57.5	6000	1.60	907			0.00135	Old Park River
RW-17	UP1	16	GTX	CRC-7	CRS	2.7%	C	1200	0.025	0.10	43.9	7600	6.33	814			0.0011385	Retaining Wall
RW-31	UP1	16	GTX	CRC-9	CRS	1.5%	B	1250	0.028	0.30		4300	3.44	537			0.001242	Retaining Wall

Notes

1. CRS refers to constant strain consolidation test. IL refers to incremental load consolidation test.
2. SQD refers to sample quality evaluation. Sample quality evaluation method:

Terzaghi et al. (1996) Specimen Quality Designation (SQD): A (best) to E (worst)

ε_v at σ'_{vo}	<1	1-2	2-4	4-8	>8
SQD	A	B	C	D	E

3. Cv values were estimated using the proposed range of stresses (s'vo to s'final) from the maximum anticipated embankment load at mid-point of the varved clay layer.
4. OCR was determined using Casagrande Method.
5. $S_{u,DSS}$ is calculalted with SHANSEP correlation from four Direct Simple Shear tests
6. $C\alpha$ is estimated based on $C\alpha$=0.45CR

Table 3 - Summary of Test Results By Baker Engineering

	DEPTH (ft)	WATER CONTENT (%)	LL (%)	PL (%)	CLASS	AVE TOT UNIT WGT (pcf)	CONSOLIDATION TEST DEPTH (ft)	CLASS	e0	INITIAL WATER (%)	Cr	Cc	Cce	MAX PAST VERTICAL (psf)	OCR	TRIAXIAL TEST PISTON DEPTH* (ft)	PHI (deg)	c (psf)	TORSION VANE DEPTH (ft)	VALUE (tsf)
RB-34	1.5-3	27.4			SM															
	10-11.5	41.6	39	23	CL															
	25-26.5	60.6																		
RB-35	7.5-9	38.7																		
	13.5-15	36	21		CL															
	25.5-27	55.6																		
RB-36	4.5-6	31	34	24	CL															
	10-11.5	38.7																		
	25-26.5	58.9																		
RB-37	7.5-9	32	26	26	ML															
	13.5-15	40.1																		
	28.5-30	59.7																		
RB-38	1.5-3	5.2	NP	NP	SP-SM															
	13.5-15	35.3	29	22	CL-ML															
	22.5-24	49.6																		
RB-39	4.5-6	31.5	NP	NP	ML															
	16.5-18	35.8																		
RB-40	4.5-6	33																		
	13.5-15	43	49	24	CL															
	25-26.5	46.8																		
RB-41	4.5-6	59.4					*RB-41 NEAR HO25-117*													
	10-11.5	65.7	62	26	CH															
	16.5-18	66.1																		
	25-26.5	50.9																		
RB-42	7.5-9	10.6	NP	NP	SM															
	16.5-18	40.8																		
	22.5-28.5	10.4	25	14	SC															
SB-56	30.1	62.8	(Clay Portion)				31.7-31.8	CH	1.889	66.1	0.16	0.6	0.2	2500	0.72	30-32	20	210	30.1	0.2
	30.1-30.6	60.3																	30.6	0.2
	30.6	44.5	(Silt Portion)																31.1	0.21
	30.6-31.1	60.5																	37.7	0.2
	31.1	53.6	(Silt Portion)																	
	31.1-31.7	62																		
	31.7-31.8	66.1																		
	31-32	60.2	56	27																
	55-56.5	40.2	41	25	CL															
	75-76.5	27.3																		
	105-106.5	15.2	NP	NP	SM															
SB-57	28.5-30	57.1																		
	55-56.5	45.7	42	23	CL															
	90-91.5	22.5																		
SB-58	10-11.5	32.7					80.3-80.4	CH	1.835	67	0.19	0.66	0.22	6200	1.14				80.1	0.22
	35-36.5	59.5																	80.3	0.2
	50-51.5	56.6	53	25	CH		*SB-58 WITHIN FLATBUSH AVE ABUTMENT, FOR ALTERNATE LAYOUT*													
	80.1	65.1	(Clay Portion)			90.3														
	80.3	38.8	(Silt Portion)																	
	80.3-80.4	67																		
	80.4	44.5	(Entire Varve)																	
	80-80.5		53	25																
	90-91.5	34.2	24	24	ML															
	110-111.5	23.1	NP	NP	SM															
SB-59	7.5-9	27.8					*HAVE CORROSIVITY TEST FOR SB-59*													
	50-51.5	55.2					*SB-59 WITHIN FLATBUSH AVE PIER, FOR ALTERNATE LAYOUT*													
	105-11.5	23.7	NP	NP	ML															
SB-60	13.5-15	33.1	25	22	ML															
	35-36.5	55.4	60	27	CH															
	50-51.5	57.8																		
	70-71.5	45.8	48	23	CL															
	90-91.5	43.4																		
	110-111.5	22.7	NP	NP	ML															
	125-126.5	28	33	25	ML															
SB-61	40-41.5	58.1	64	23	CH															
	90-91.5	31.2	25	23	ML															
SB-62	10.-11	28.6					60.8-60.9	CH	2.02	71	0.23	0.83	0.26	5400	.56/1.03*				60.3	0.25
	35-36.5	59.2	60	26	CH															
	60.3	66.4	(Clay Portion)			108.53														
	60.4	40.7	(Silt Portion)																	
	60.5-60.7	49	(Entire Varve)																	
	60.8-60.9	71																		
	61.1	69.8	74	29																
	61.2	34.9	32	31																
	85-86.5	68.3	34	24	ML															
	115-116.5	11.9	16	14	SP-SM															
	120-121.5	21.6	NP	NP	SM															
SB-63	16.5-18	41.9	41	22	CL															
	50-51.5	57.5	56	25	CH															
	90-91.5	35.9	29	24	ML															
	135-136.5	11.7	NP	NP	SM															
SB-64	4.5-6	33																		
	10.5-12	24.8																		
	13.5-15	40.5																		
	16.5-18	47.4																		
	19.5-21	55																		
	22.5-24	58.3																		
	25-27	61.1																		
	28.5-30	55.2	63	27	CH															
	35-36.5	61.7																		
	40-41.5	57																		
	45-46.5	53.5																		
	50-51.5	48.3																		
	55-56.5	41.4																		
	60-61.5	40.1																		
	65-66.5	51.2																		

Table 3 - Summary of Test Results By Baker Engineering

							CONSOLIDATION TEST									TRIAXIAL TEST			TORSION VANE	
	DEPTH (ft)	WATER CONTENT (%)	LL (%)	PL (%)	CLASS	AVE TOT UNIT WGT (pcf)	DEPTH (ft)	CLASS	e0	INITIAL WATER (%)	Cr	Cc	Cce	MAX PAST VERTICAL (psf)	OCR	PISTON DEPTH* (ft)	PHI (deg)	c (psf)	DEPTH (ft)	VALUE (tsf)
	70-71.5	41.5	44	23	CL															
	75-76.5	41.8																		
	80-81.5	40.8																		
	85-86.5	42.7																		
	90-91.5	40.8																		
	95-96.5	24.1																		
	100-101.5	40.5																		
	110-111.5	23.5	NP	NP	SM															
	135-135.9	9.6			SM															
SB-65	13.5-15	28.9																		
	95-96.5	29.9	24	24	ML															
	125-126.5	14.6																		
SB-66	35-36.5	61.1	60	28	CH															
	70-71.5	47.8	43	24	CL															
	135-136.5	10.8	21	15	SC-SM															
SB-67	45-46.5	54.6	58	27	CH		71.3-71.4	CH	1.674	64.7	0.18	0.73	0.27	4800	1.06	70-72	30	250	70.4	0.25
	70.4	64.1	(Clay Portion)			110.5													70.9	0.27
	70.5-70.8	48.1																	71.3	0.24
	70.9	66.7	(Clay Portion)																	
	70.9-71.3	47																		
	71.3	76.8	(Clay Portion)																	
	71.3-71.4	64.7																		
	71.5-71.8	43.9																		
	71.8-71.9	43.1	(Entire Varve)																	
	71.9	39	(Silt Portion)																	
	70-72	67.8	70	32																
	95-96.5	46.2																		
	105-112	20.3	NP	NP	SM															
SB-68																				
SB-69	7.5-9	36.7	30	25	SM															
	50-51.5	44	56	26	CH															
	95-96.5	34.3	24	24	ML															
	145-146.5	12.1	18	15	GM															
SB-70	13.5-15	29.2					51.1-51.2	CH	1.681	59.6	.16*	0.62	0.23	4100	0.85	50-52	25	40	50.1	0.28
	28.5-30	54																	50.9	0.3
	50.1	71.5	(Clay Portion)			106.6													51.8	0.3
	50.2-50.5	61.3																		
	50.5-50.9	57.1																		
	50.9	70.4	(Clay Portion)																	
	51.1-51.2	59.6																		
	51.4-51.8	57.6																		
	51.8	71.6																		
	50-52	67.4	63	30																
	60-61.5	48.7	52	23	CH															
	105-106.5	24																		
SB-71	10.5-11	34.8																		
	13.5-15	39.2																		
	16.5-18	34.4																		
	19.5-21	43.5																		
	22.5-24	54.1																		
	25-26.5	53																		
	28.5-30	53.5																		
	35-36.5	51																		
	40-41.5	34.8																		
	45-46.5	54.9	56	26	CH															
	50.51.5	54																		
	55-56.5	46.3																		
	60.-61.5	39.5																		
	65-66.5	40																		
	70-71.5	48																		
	75-76.5	37.1																		
	80-81.5	50																		
	85-86.5	38.9	29	25	ML															
	90-91.5	41.4																		
	95.-96.5	23.7																		
	100-101.5	21.3																		
	115-116.5	12	17	16	SM															
SB-72	16.5-18	39.3																		
	25-26.5	49.6	47	24	CL															
	65-66.5	50.5	55	26	CH															
	105-106.5	24.5	20	20	ML															
	135-136.5	12.8			GP-GM															
SB-73	16.5-18	29.9					30.5-30.6	CL	0.853	31.2	0.035	0.18	0.11	4000	1.19	30-32	23	800	30.3	0.45
	28.5-30	33.1	NP	NP	ML														30.7	0.55
	30.3	30.5				127.6														
	30.3-30.6	29.5																		
	30.6-30.7	31.2																		
	30.7	24.3																		
	30.8-31.1	26.8																		
	31.2	26.9																	31.2	0.6
	31.3	24.3	29	21																
	31.4-31.7	23.1																		
	70-71.5	46.1	48	23	CL															
	115-116.5	21.4	NP	NP	SM															
SB-74	7.5-9	27.6	NP	NP	ML		29.5-29.6	CH	1.78*	65	0.17	0.7	0.23	4200	1.9				28.8	0.25
	25-26.5	63.5																		
	28.8	75.3	(Clay Portion)			110.47														
	28.9	34.8	(Silt Portion)																	
	28.9-29	56.2	(Entire Varve)																	
	29.1	68.1	83	31																
	29.5-29.6	65																		
	29.7-29.9	32.8	34	28																
	85-86.5	44.9	58	25	CH															

Table 3 - Summary of Test Results By Baker Engineering

	DEPTH (ft)	WATER CONTENT (%)	LL (%)	PL (%)	CLASS	AVE TOT UNIT WGT (pcf)	CONSOLIDATION TEST DEPTH (ft)	CLASS	e0	INITIAL WATER (%)	Cr	Cc	Cce	MAX PAST VERTICAL (psf)	OCR	TRIAXIAL TEST PISTON DEPTH* (ft)	PHI (deg)	c (psf)	TORSION VANE DEPTH (ft)	VALUE (tsf)
	120-121.5	12.8	22	17	GC-GM															
SB-75	16.5-18	43.8																		
	50-51.5	53.3	57	25	CH															
	110-111.5	24.8	NP	NP	ML															
SB-76	10.5-12	53																		
	28.5-30	51.6	59	26	CH															
	80-81.5	43.6	41	25	CL															
	130-131.5	19.2	NP	NP	SP-SM															
SB-77	10-11.5	25.8					41.6-41.7	CL	1.71	62.8	0.16	0.57	0.21	4200	1.84	40-42	13	800	40.2	0.32
	25-26.5	44.1	44	26	CL															
	40.2	76.1	(Clay Portion)			103.3													41.1	0.27
	40.2-40.6	61.1					*SB-77 NEAR WALL HO25-107*												41.5	0.25
	40.6-41.1	63.6																		
	41.1	39.5	(Silt Portion)																	
	41.1-41.5	57.5																		
	41.5	58.9	(Entire Varve)																	
	41.6-41.7	62.8																		
	40-42	72.1	74	43																
	85-86.5	42.3	35	25	ML															
	130-131.5	11.3	17	16	SM															
SB-78	13.5-15	32.1																		
	28.5-30	56.7	51	25	CH															
	80-81.5	43.9	38	24	CL															
	130-131.5	9.9	18	16	SM															
SB-79	13.5-15	41.2																		
	60-61.5	50.1	63	27	CH															
	145-146.5	11.5	20	17	SM															
SB-80	10-11.5	37.4	30	27	ML															
	22.5-24	48.8																		
	40-41.5	46.2	50	27	CH															
	70-71.5	51	55	28	CH															
	100-101.5	44.9	42	24	CL															
	130-131.5	19.1	NP	NP	SP-SM															
SB-81	7.5-9	31.9																		
	16.5-18	53.2																		
	28.5-30	45.9																		
	50-51.5	51	59	27	CH															
SB-82	16.5-18	39.1	36	24	CL															
	50-51.5	60.2	59	27	CH	100.4														
	70.3	46.5	(Disturbed)				*SB-82 NEAR SW-3*													
	70.9	56	(Disturbed)																	
	71.5	61.3	(Disturbed)																	
	95-96.5	46.9	42	25	CL															
	125-126.5	11	NP	NP	SW-SM															
SB-83	7.5-9	35.7	NP	NP	ML															
	28.5-30	44.8																		
	70-71.5	52	59	27	CH															
	130-131.5	10.6	NP	NP	SW-SM															
SB-84	4.5-6	4.5			SW-SM		31.6-31.7	CL/ML	1.27	47.1	0.09	0.31	0.16	3200	1.24	30-32	22	350	30.5	0.3
	25.5-27	46.8	44	25	CL														31.1	0.25
	30-30.5	45.3				106.2													31.5	0.27
	30.5	68	(Clay Portion)				*SB-84 WITHIN PARK STREET BRIDGE PIER*													
	30.6-31.1	50					*SB-84 NEAR HO25-112*													
	31.1	30.9	(Silt Portion)																	
	31.1-31.7	50.7																		
	31.5-31.6	47.2	(Entire Varve)																	
	31.6-31.7	47.1																		
	30-32	65.4	66	30																
	50-51.5	56.5	62	28	CH															
	90-91.5	46.9	58	26	CH															
	130-136.5	11.7	21	17	GP-GM															
SB-85	16.5-18	23.3					*SB-85 NEAR CAPITAL AVE BRIDGE ABUT*													
	25-26.5	46.6	59	29	CH															
	75-76.5	53.1	53	23	CH															
	110-11.5	10.8	17	16	ML															
SB-86	13.5-15	35.1	45	25	CL															
	40-41.5	42.9	56	28	CH															
	75-76.5	43.3	43	24	CL															
	110-111.5	11.6	15	14	SM															
SB-87	16.5-18	55.7	54	26	CH		*SB-87 NEAR HO25-116*													
	22.5-24	59.1																		
	55-56.5	57.4	55	26	CH															
SB-105	7.5-9	41.9					*SB-105 WITHIN OLD PARK RIVER AREA*													

* SB-62, e0 NOT GIVING IN LAB SHEET, BAKER CALCS e0=2.19. SHOULD BE 2.02 BY CGE CALC. SEE FB AVE REPORT.
* SB-62, OCR=.56 FROM LAB DATA, BAKER ASSUMES NORMAL CONSOL., ADJUST WATER TABLE, STRESS AND RECALCULATES OCR=1.03. SEE FB AVE REPORT.

* SB-70, BAKER CALCS Cc=.19. APPEARS TO BE MISCALCULATED. SEE FB AVE REPORT.

* SB-74, e0 NOT GIBING IN LAB SHEET, BAKER CALCS e0=2.04. SHOULD BE 1.78 BY CGE CALC. SEE NEW PARK AVE STATION/KANE BROOK CULVERT REPORT.

* THREE TRIAXIAL SAMPLES TESTED PER PISTON DEPTH SAMPLE

New Britain - Hartford Busway
Retaining Walls
CT DOT Project Number: 155-H025
West Hartford and Hartford, Connecticut
GeoDesign Project Number: 0380-004.0

Table 4 - Summary of Atterberg Limits

Boring No.	Tube No.	Depth From (ft)	Depth To (ft)	Moisture Content (%)	LL (%)	PL (%)	PI (%)	Tested By	Location
RW-4	UP-2	45	47	54	56	33	23	GeoTesting Express	Flatbush Ave. Over Busway
RW-4	UP-1	35	37	58.1	49	23	26	TestCon	Flatbush Ave. Over Busway
RW-6	S-7	22	24		42	25	17	TestCon	Flatbush Ave. Over Busway
RW-6	S-9	32	34		53	35	18	TestCon	Flatbush Ave. Over Busway
RW-7	UP-1	30	32	60	61	30	31	GeoTesting Express	Flatbush Ave. Over Busway
RW-7	S-9	32	34		51	41	10	TestCon	Flatbush Ave. Over Busway
RW-7	S-12	45	47		28	23	5	TestCon	Flatbush Ave. Over Busway
RW-7	S-14	55	57		29	21	8	TestCon	Flatbush Ave. Over Busway
RW-8	UP-2	30	32		49	24	25	TestCon	Flatbush Ave. Over Busway
RW-8	S-7	22	24		48	26	22	TestCon	Flatbush Ave. Over Busway
RW-8	S-11	42	44		62	48	14	TestCon	Flatbush Ave. Over Busway
RW-8	S-14	55	57		41	36	5	TestCon	Flatbush Ave. Over Busway
RW-9	S-5	10	12		29	26	3	TestCon	Flatbush Ave. Over Busway
RW-9	S-10	35	37		58	41	17	TestCon	Flatbush Ave. Over Busway
SB-01-1	UP-1	20	22	56.1	49	24	25	TestCon	Flatbush Ave. Over Busway
SB-01-1	S-7	22	24		32	34	-2	TestCon	Flatbush Ave. Over Busway
SB-01-1	S-11	40	42		61	50	11	TestCon	Flatbush Ave. Over Busway
SB-01-1	S-13	50	52		52	46	6	TestCon	Flatbush Ave. Over Busway
SB-01-2	UP-2	25	27	51.9	49	25	24	TestCon	Flatbush Ave. Over Busway
SB-01-3	S-10	30	32		53	28	25	TestCon	Flatbush Ave. Over Busway
SB-01-3	S-12	40	42		42	28	14	TestCon	Flatbush Ave. Over Busway
SB-01-4	S-6	20	22		50	40	10	TestCon	Flatbush Ave. Over Busway
SB-01-4	S-8	32	34		47	37	10	TestCon	Flatbush Ave. Over Busway
SB-01-4	S-10	42	44		57	44	13	TestCon	Flatbush Ave. Over Busway
SB-01-4	UP-1	30	32	51.7	50	24	26	TestCon	Flatbush Ave. Over Busway
RW-23	UP-2	40	42		44	23	21	TestCon	Busway Over Park St.
RW-23	UP-1	25	27		50	23	27	TestCon	Busway Over Park St.
SB-02-1	S-10	45	47		62	49	13	TestCon	Busway Over Park St.
SB-02-1	S-12	55	57		48	43	5	TestCon	Busway Over Park St.
SB-02-1	S-14	65	67		56	46	10	TestCon	Busway Over Park St.
SB-02-1	S-17	80	82		37	30	7	TestCon	Busway Over Park St.
SB-02-1	UP-1	40	42	67	58	35	23	UMass	Busway Over Park St.
SB-02-2	UP-1	30	32		51	30	21	UMass	Busway Over Park St.
SB-02-3	S-7	20	22		45	31	14	TestCon	Busway Over Park St.
SB-02-3	S-10	35	37		51	30	21	TestCon	Busway Over Park St.
SB-02-3	S-15	60	62		50	46	4	TestCon	Busway Over Park St.
RW-28	UP-2	25	27		48	24	24	TestCon	Busway Over Capitol Ave.
RW-29	UP-3	45	47		49	23	26	TestCon	Busway Over Capitol Ave.
R-123	S-10	30	32		53	36	17	TestCon	Busway Over Capitol Ave.
R-123	S-12	40	42		55	50	5	TestCon	Busway Over Capitol Ave.
R-125	S-8	20	22		49	27	22	TestCon	Busway Over Capitol Ave.
R-125	S-10	30	32		46	37	9	TestCon	Busway Over Capitol Ave.
SB-03-1	UP-2	45	47		45	22	23	TestCon	Busway Over Capitol Ave.
SB-03-1	S-16	70	72		45	36	9	TestCon	Busway Over Capitol Ave.
SB-03-2	UP-2	40	42	57	58	25	33	GeoTesting Express	Busway Over Capitol Ave.
SB-03-3	S-22	90	92		45	36	9	TestCon	Busway Over Capitol Ave.
SB-06-1A	UP-1	45	47		49	22	27	TestCon	Old Park River
SB-06-3	UP-1	45	47	64	58	30	28	UMass	Old Park River
SB-06-3	UP-2	70	72		47	23	24	TestCon	Old Park River
SB-06-4	S-9	20	22		41	33	8	TestCon	Old Park River
SB-06-4	S-14	47	49		46	38	8	TestCon	Old Park River
SB-06-4	S-20	75	77		41	32	9	TestCon	Old Park River
SB-06-4	UP-2	45	47	58	59	28	31	GeoTesting Express	Old Park River

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Table 5 - Summary of Hydrometer Tests

Boring No.	Tube No.	Depth From (ft)	Depth To (ft)	% Sand	% Silt	% Clay	Location
R-101	S-3	4	6	0	63.2	36.8	Flatbush Ave. Over Busway
R-112	S-5	10	12	2.7	26.9	70.4	Flatbush Ave. Over Busway
RW-4	UP-1	35	37	0.6	27.5	71.9	Flatbush Ave. Over Busway
RW-8	UP-2	30	32	0.4	19.9	79.7	Flatbush Ave. Over Busway
SB-01-1	UP-1	20	22	0.6	32.2	67.2	Flatbush Ave. Over Busway
SB-01-2	UP-2	25	27	0.6	29.5	69.9	Flatbush Ave. Over Busway
SB-01-4	UP-1	30	32	0.2	21.2	78.6	Flatbush Ave. Over Busway
SB-02-1	S-9	42	44	0.6	13.7	85.7	Busway Over Park St.
SB-02-1	S-11	52	54	0.2	15.7	84.1	Busway Over Park St.
SB-02-2	S-9	32	34	0.2	13.8	86	Busway Over Park St.
SB-02-2	S-13	50	52	0	13.4	86.6	Busway Over Park St.
SB-02-3	S-9	30	32	6	22.4	71.6	Busway Over Park St.
SB-02-3	S-12	45	47	0.8	15.1	84.1	Busway Over Park St.
RW-23	UP-2	40	42	0.3	28.4	71.3	Busway Over Park St.
RW-23	UP-1	25	27	0.6	21.5	77.9	Busway Over Park St.
RW-28	UP-2	25	27	0.4	21.1	78.5	Busway Over Capitol Ave.
RW-29	UP-3	45	47	0.5	22.6	76.9	Busway Over Capitol Ave.
SB-03-1	S-11	47	49	1	9.7	89.3	Busway Over Capitol Ave.
SB-03-1	S-9	37	39	18.6	14.5	66.9	Busway Over Capitol Ave.
SB-03-2	S-9	32	34	0.2	16.1	83.7	Busway Over Capitol Ave.
SB-03-2	S-11	42	44	0.4	11.6	88	Busway Over Capitol Ave.
SB-03-3	S-13	45	47	0.4	13.1	86.5	Busway Over Capitol Ave.
SB-03-3	S-21	85	87	0.6	19.9	79.5	Busway Over Capitol Ave.
R-121	S-3	5	7	75.2	8.1	16.7	Old Park River
R-115	S-2	2	4	13.6	55.5	30	Roadway

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Table 6 - Summary of Field Vane Shear Tests

Test Date	Boring No.	Drilled to Depth (ft)	Pushed Vane to Depth (ft)	Surface Elevation (ft)	Pushed to Elevation (ft)	Ultimate Shear Strength (psf)	Correction Factor u	Corrected Shear Strength (psf)	Remold Shear Strength (psf)	Sensitivity	Location
6/9/2008	SB-01-3	35	36	70	34	670	0.9	**603**	0	Very Sensitive	Flatbush Ave. Over Busway
5/30/2008	SB-02-3	30	31	54	23	282	0.9	**254**	22	13	Busway Over Park St.
5/23/2008	SB-03-2	25	26.5	44	17	476	0.9	**428**	0	Very Sensitive	Busway Over Capitol Ave.
6/2/2008	SB-06-1	40	41	55	14	454	0.9	**409**	22	21	Old Park Rvier
5/27/2008	SB-06-3	55	56	54	-2	83	0.9	**75**	61	1	Old Park Rvier
5/29/2008	R-126	20	21	47	26	239	0.9	**215**	22	11	Old Park Rvier
6/4/2008	R-112	36	37	60	23	281	0.9	**253**	9	32	Roadway
6/5/2008	RW-31	25	26	64	38	389	0.9	**350**	108	4	Retaining Wall

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Table 7 Estimated Magnitude and Rate of Consolidation Settlement and Design Recommendations for Retaining Walls

Retaining Wall No.	Roadway Baseline	From Station	To Station	Approximate Average Fill Height (ft.)	RETAINING WALLS (If founded on spread footingss): Estimated Max. Wall Settlement with 125 pcf Fill (in.)	Estimated Max. Wall Settlement with 60 pcf Light-weight Fill (in.)	To Achieve Less Than 1" Wall Settlement (With 60 pcf Light-weight Fill): Approximate % Consolidation	Approximate Time (Months)	ROADWAY: Estimated Max. Roadway Settlement with 125 pcf Fill (in.)	Estimated Max. Roadway Settlement with 60 pcf Light-weight Fill (in.)	To Achieve Less Than 2" Roadway Settlement (With 60pcf Light-weigh Fill): Approximate % Consolidation	Approximate Time (Months)	RECOMMENDATIONS: Recommended Foundation Type	Fill Recommendations: Special Requirement Key	Recommended Min. Waiting Period (Months)	Alternate Fill Recommendations Using Wick Drains: Special Requirement Key with Wick Drains	Recommended Min. Waiting Period with Wick Drains (Months)
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]	[18]
RW-101	Flatbush Ave.	1300	1400	4.5	1.8	0.8	-	-	2.5	1.2	-	-	Spread Footings	A	-	-	
RW-101	Flatbush Ave.	1400	1500	11	3.7	1.7	40%	2	5.0	3.3	40%	2	Piles	B	2	C	1
RW-101	Flatbush Ave.	1500	1550	16	5.2	2.5	60%	5	7.7	4.0	50%	3	Piles	B	3	C	1
RW-102	Flatbush Ave.	1690	1800	23	9.0	4.5	80%	11	13.2	6.5	70%	6	Piles	B	6	C	1
RW-102	Flatbush Ave.	1800	1950	18	6.4	3.0	70%	8	8.5	4.4	60%	5	Piles	B	5	C	1
RW-102	Newfield Ave.	5380	5528	13	4.3	2.8	60%	6	6.5	3.8	50%	3	Piles	B	3	C	1
RW-103	Newfield Ave.	5380	5500	13	4.3	2.7	60%	6	6.5	3.8	50%	3	Piles	B	3	C	1
RW-103	Flatbush Ave.	1950	2100	11	3.7	1.7	60%	6	5.0	3.3	40%	2	Piles	B	2	C	1
RW-103	Flatbush Ave.	2100	2200	5	1.8	1.0	-	-	3.5	2.1	10%	1	Spread Footings	B	1	-	-
RW-103	Flatbush Ave.	2200	2300	1.5	0.8	0.3	-	-	1.5	0.4	-	-	Spread Footings	-	-	-	-
RW-104	New Park Ave.	10600	10700	2	1.0	0.5	-	-	1.7	0.8	-	-	Spread Footings	-	-	-	-
RW-104	Flatbush Ave.	1300	1400	4.5	1.8	0.8	-	-	2.5	1.2	-	-	Spread Footings	A	-	-	-
RW-104	Flatbush Ave.	1400	1550	16	5.2	2.5	60%	6	7.7	4.0	50%	3	Piles	B	3	C	1
RW-105	Flatbush Ave.	1750	1850	23	9.0	4.5	80%	11	13.2	6.5	70%	6	Piles	B	6	C	1
RW-105	Flatbush Ave.	1850	1950	16	5.2	2.5	60%	6	7.7	4.0	50%	3	Piles	B	3	C	1
RW-105	Flatbush Ave.	1950	2050	12	4.0	2.3	70%	8	5.4	3.0	60%	5	Piles	B	5	C	1
RW-105	Flatbush Ave.	2050	2150	7	2.5	1.2	60%	8	3.3	1.7	-	-	Piles	A	-	-	-
RW-105	Flatbush Ave.	2150	2250	3	1.3	0.8	-	-	2.0	1.0	-	-	Spread Footings	A	-	-	-
RW-106	Flatbush Ave.	2250	2300	1.5	0.8	0.3	-	-	1.5	0.4	-	-	Spread Footings	-	-	-	-
SW-1	Busway NB **	38350	38475	4	1.5	0.8	-	-	1.9	1.2	-	-	Spread Footings	D	-	-	-
RW-107	Busway NB **	38825	38975	3	1.3	0.8	-	-	2.0	1.0	-	-	Spread Footings	D	-		
RW-107	Busway NB **	38975	39025	0	0.0	0.0	-	-	0.0	0.0	-	-	Spread Footings	-	-	-	-
RW-TBD	Busway NB **	39375	39575	4	1.5	0.8	-	-	1.9	1.1	-	-	Spread Footings	A	-	-	-
RW-108	Busway NB **	41120	41170	2	1.0	0.5	-	-	1.7	0.8	-	-	Spread Footings	-	-	-	-
RW-108	Busway NB **	41170	41260	6	2.1	1.1	10%	1	2.9	1.4	-	-	Spread Footings	A	-	-	-
RW-109	Busway NB	41690	41740	8	2.8	1.4	30%	1	3.7	1.9	-	-	Piles	A	-	-	-
RW-110	Busway NB **	41592	41705	6	2.1	1.1	10%	1	2.9	1.4	-	-	Spread Footings	A	-	-	-
RW-111	Busway NB **	41890	41950	14	6.0	3.0	70%	8	8.0	4.1	50%	6	Piles	B	6	C	1
RW-111	Busway NB **	41950	42000	12	5.7	2.6	40%	2	7.6	3.9	50%	6	Piles	B	6	C	1
RW-111	Busway NB **	42000	42100	7	2.5	1.1	10%	1	3.3	1.7	-	-	Piles	A	-	-	-
RW-111	Busway NB **	42100	42300	5	1.8	1.0	-	-	2.5	1.2	-	-	Spread Footings	A	-	-	-
RW-112	Busway NB	41910	42000	9	3.1	1.6	40%	8	5.8	2.9	10%	1	Piles	B	1	-	-
RW-112	Busway NB	42000	42100	6	2.1	1.1	10%	1	2.9	1.4	-	-	Spread Footings	A	-	-	-
RW-112	Busway NB	42100	42300	4	1.5	0.8	-	-	1.9	1.1	-	-	Spread Footings	A	-	-	-
RW-112	Busway NB	42300	42350	3	1.3	0.6	-	-	1.7	0.7	-	-	Spread Footings	A	-	-	-
RW-112	Busway NB	42350	42500	2	1.0	0.5	-	-	1.7	0.8	-	-	Spread Footings	-	-	-	-
Roadway Barrier Wall	Busway NB	43175	43325	14	4.6	3.0	70%	8	6.2	3.8	50%	6	Spread Footings	E	-	-	-
Roadway Barrier Wall	Busway NB	43325	43625	10	3.4	1.7	40%	5	4.6	3.2	40%	2	Spread Footings	E	-	-	-
RW-113	Busway NB **	43600	43725	11	3.7	1.8	40%	5	5.0	3.3	40%	2	Spread Footings	E	-	-	-
RW-114	Busway NB **	43665	43740	5	1.8	1.0	-	-	2.5	1.2	-	-	Spread Footings	E	-	-	-
RW-115	Busway NB **	43800	43870	5	1.8	1.0	-	-	2.5	1.2	-	-	Spread Footings	D	-	-	-
RW-116	Busway NB	43900	44150	6.5	2.3	1.1	10%	1	3.1	1.6	-	-	Spread Footings	E	-	-	-
RW-116	Busway NB	44150	44250	6	2.1	1.1	10%	1	2.9	1.4	-	-	Spread Footings	E	-	-	-
RW-116	Busway NB	44250	44475	4	1.5	0.8	-	-	1.9	1.1	-	-	Spread Footings	D	-	-	-
RW-117	Busway NB **	44575	45000	Cut	-	-	-	-	-	-	-	-	Spread Footings	-	-	-	-

Notes: 1 Approximate Clay Thickness
From RW 101 to RW 106 and from RW 113 to RW 117 — 90 ft.
Other Walls — 120 ft.
2 Triangular Wick Drain Spacing — 8 ft.
3 Estimated roadway (pavement) settlements and wall backfill recommendations are provided in this table because they control the design in some cases.
4 ** NB Busway Stationning is used for Walls located adjacent to SB Roadway.

Key	Special Requirement
A	Light-weight Fill
B	Light-weight Fill +Waiting Period
C	Light-weight Fill +Waiting Period +Wick Drains
D	Light-weight Fill + Over Excavation
E	GeoFoam + Over Excavation
-	None

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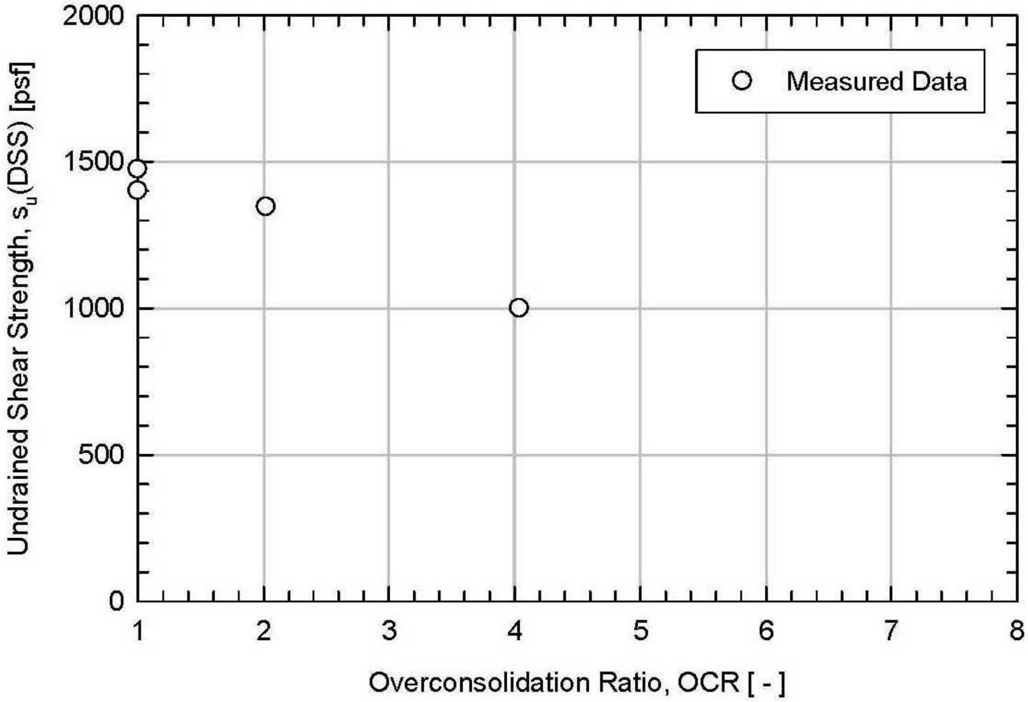
Table 8 Estimated Pile Length For Retaining Walls

Retaining Wall No.	Roadway Baseline	From Station	To Station	Estimated Minimum Pile Length (ft)	Estimated Maximum Pile Length (ft)	Estimated Average Pile Length (ft)
RW-101	Flatbush Ave.	1400	1500	137	152	145
RW-101	Flatbush Ave.	1500	1550	137	152	145
RW-102	Flatbush Ave.	1690	1800	137	152	145
RW-102	Flatbush Ave.	1800	1950	137	152	145
RW-102	Newfield Ave.	5380	5528	137	152	145
RW-103	Newfield Ave.	5380	5500	137	152	145
RW-103	Flatbush Ave.	1950	2100	137	152	145
RW-104	Flatbush Ave.	1400	1550	137	152	145
RW-105	Flatbush Ave.	1750	1850	137	152	145
RW-105	Flatbush Ave.	1850	1950	137	152	145
RW-105	Flatbush Ave.	1950	2050	137	152	145
RW-105	Flatbush Ave.	2050	2150	137	152	145
RW-109	Busway NB	41690	41740	157	158	158
RW-111	Busway SB	41890	41950	157	160	159
RW-111	Busway SB	41950	42000	157	160	159
RW-111	Busway SB	42000	42100	157	160	159
RW-112	Busway NB	41910	42000	157	160	159

Chart 1

Shear Strength vs. Overconsolidation Ratio



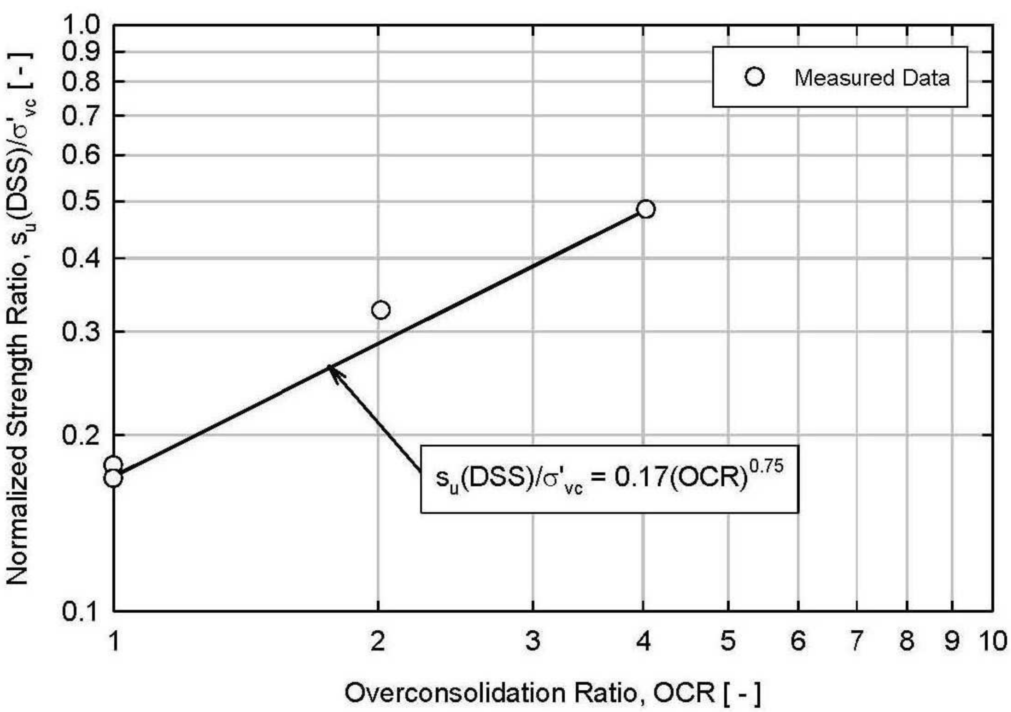
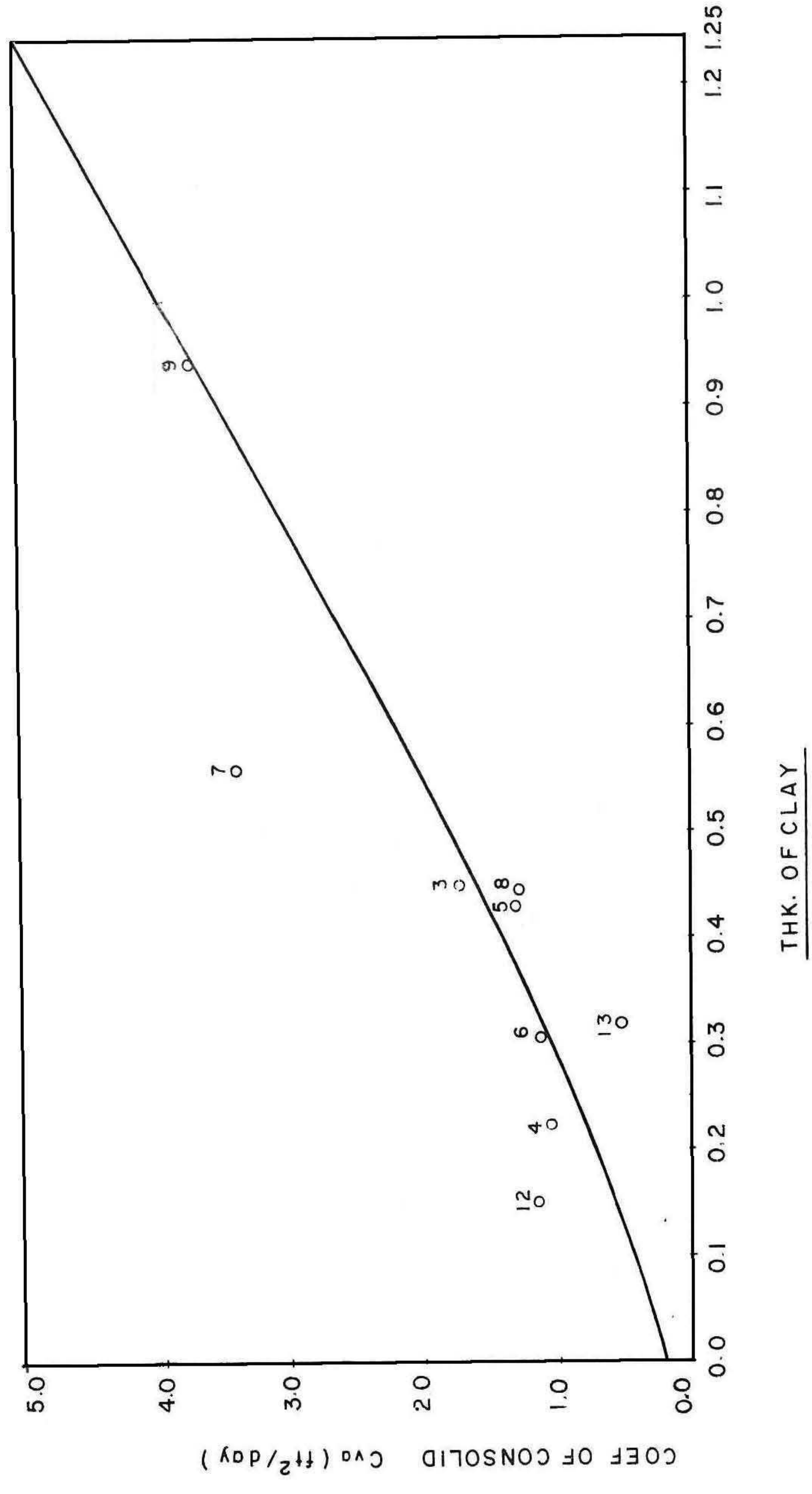


Chart 2



FIELD VALUES OF THE APPARENT COEFFICIENT OF CONSOLIDATION (EHE)

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Appendix 3
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Appendix 3

Limitations

GEOTECHNICAL LIMITATIONS

Explorations

1. The analyses and recommendations submitted in this report are based in part upon the data obtained from widely spaced subsurface explorations. The nature and extent of variations between these explorations may not become evident until further explorations are made and construction occurs. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

2. The generalized soil and bedrock profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil and bedrock transitions are probably more erratic. For specific information, refer to the boring logs.

3. The geologic and geomorphologic settings at this site are complex and the uncertain historic site usage has resulted in the varied distribution and stress history of compressible soils across the site. Limited spacing of borings and lab testing can at best, only allow for estimates to be developed for duration and magnitude of consolidation settlements.

4. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in river levels, rainfall, temperature, and other factors occurring since the time measurements were made.

Review

5. In the event that any changes in the nature, design or location, of the proposed retaining walls, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by Geo**Design**, Inc. It is recommended that this firm be provided the opportunity for a general review of design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

Use of Report

6. This report has been prepared for the exclusive use URS/Washington Group (URS/WGI), the Connecticut Department of Transportation (ConnDOT), and other members of the design team for specific application to the construction of retaining walls on New Britain - Hartford Busway in Hartford/West Hartford, Connecticut, in accordance with generally accepted soil and foundation engineering practices. No other warranty, express or implied, is made.

7. This final design soil and foundation engineering report has been prepared for this project by Geo**Design**. This report is for design purposes only and is not sufficient to prepare an accurate bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.