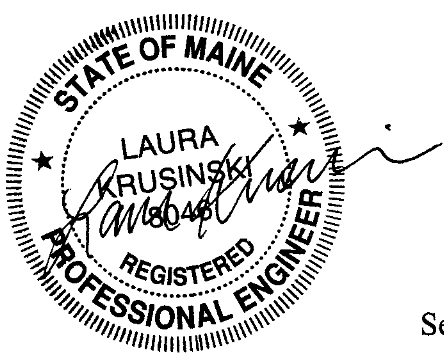
MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
STATE HOUSE STATION 16
AUGUSTA, MAINE

Feasibility Study and Preliminary Geotechnical Design Report
For the rehabilitation/replacement of:

# VERANDA STREET BRIDGE
# OVER THE ST. LAWRENCE AND ATLANTIC RAILROAD
# PORTLAND, MAINE

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# GEOTECHNICAL REPORT SUMMARY

A Feasibility Study and Preliminary Geotechnical Design Report for the proposed replacement or rehabilitation of the Veranda Street Bridge over the St. Lawrence and Atlantic Railroad, in Portland, Maine has been completed. Potential bridge modifications include a 0.91 to 1.677 m (3.0 to 5.5 foot) raise in the roadway profile to provide a 6.9 m (22.5 ft) railroad under clearance in the future. This report has been prepared to present:

- subsurface data collected from the preliminary geotechnical subsurface investigation;
- geophysical data collected to verify existing abutment geometry;
- subsurface data collected to characterize of the backfill soils of the existing abutments, and the soils the abutments bear upon;
- stability analyses for the existing abutments with the proposed roadway profile;
- foundation alternatives:
  - substructure reinforcement/rehabilitation
  - substructure replacement, and,
- foundation recommendations.

## RECOMMENDATIONS

Due to the condition of the abutments, the results of stability analyses, and the potential cost of abutment reinforcement, replacement of the bridge substructures is recommended.

Based on the visual observations of the substructures, verified abutment dimensions, and substructure stability analyses, it has been determined that re-use of the existing bridge substructures under the proposed roadway profile raise *is feasible only with significant reinforcement.* However, due to the condition of the abutments and the results of stability analyses, the potential cost of abutment reinforcement can be significant. Prior to any scope decision, a life cycle cost analysis is recommended, as the cost of a properly engineered substructure reinforcement project can likely exceed that of substructure replacement. For the purpose of such an analysis, two abutment reinforcement alternatives have been provided in this report, and these reinforcement alternatives must be engineered to provide the substructures with AASHTO required factors of safety of 1.5 and 2.0, against sliding and overturning, respectively. The life cycle cost analysis task is beyond the scope of this report.

## CONDITION OF EXISTING SUBSTRUCTURES

<u>Overall Substructure Condition</u>. The Veranda Street Bridge substructures consist of stone masonry ashlar abutments dating to a bridge constructed prior to 1930, and unreinforced concrete abutment and wingwall modifications constructed circa 1930.

All are founded on spread footings. The concrete abutments and wingwalls are in good condition, but the concrete is unreinforced. The masonry stone abutments are visually in good condition, however, the geophysical data show that behind the stone abutment face the blocks are unbonded and moisture has infiltrated the individual stones.

MaineDOT Bridge Inspection Records (2001) assign a condition rating of 6 to the substructures. A rating of 6, on a scale of 0 to 9 indicates "satisfactory condition".

Pointing. The pointing has failed at scattered locations in the courses of the stone masonry. There is some deterioration of the concrete caps.

Drainage. The pre-1930 stone substructures were constructed without a drainage system. The concrete abutment modifications were constructed with two weepholes per abutment.

Abutment Foundation Soils and Backfill - The data indicate that the stone masonry abutments, concrete abutment extensions and concrete wingwalls bear on dense to very dense, fine to coarse sand, some to trace of gravel, with trace of silt. The soil backfill behind the abutments is generally described as loose to dense, damp to moist, silty fine to coarse sand, trace of brick wood and gravel. The absence of a high groundwater table and source of water implies that prolonged cold weather will not result in ice lenses. With the exception of one backfill sample which is classified as having a high degree of frost susceptibility, all backfill samples have a negligible to low degree of frost susceptibly.

Abutment Footing Embedment. A test pit was hand dug in front of the stone masonry section of the north (Falmouth) abutment but did not encounter the bottom of the stone footing. Future investigations should confirm the footing embedment and toe dimensions assumed in this report.

## INFERRED ABUTMENT GEOMETRY

A ground penetrating radar (GPR) survey and sonic/ultrasonic investigation of the Veranda Street Bridge abutments was performed by NDT Corporation of Worcester, Massachusetts, under the direction of the MaineDOT. The objective of the geophysical investigation was to determine the approximate thickness and nature of the concrete and stone masonry abutments.

## ABUTMENT STABILITY ANALYSIS

The overall stability of the Portland Veranda Street Bridge south and north abutments was assessed considering the geometry and backfill soils confirmed by the GPR study and soil sampling. The overall stability of each abutment was analyzed considering the confirmed geometry, the earth loads due to the potential raise in the roadway profile by 910 mm, and the stabilizing moment of superstructure dead load.

The overall stability of the north stone masonry abutment is calculated to be approximately 1.3 against overturning and 1.4 against sliding, compared with the AASHTO required factors of safety of 2.0 and 1.5. The overall stability of the south stone masonry abutment is calculated to be approximately 1.2 against overturning and 1.6 against sliding, which are also lower than the factors of safety required by AASHTO. These factors of safety represent the higher range, as the design assumptions are unconservative.

Based on the stability analyses, the factors of safety against overturning, sliding and bearing capacity for the existing substructures are unacceptable if the roadway profile is raised 910 mm. The overfill would result in increased earth pressure loads and footing bearing pressures. These factors of safety will be even lower for an analysis assuming a 1.677 m raise in grade. The stability analyses demonstrate that both abutments require significant reinforcement for re-use or replacement. Replacement is preferred.

## FOUNDATION ALTERNATIVES

Based on the stability analyses, reinforcement or replacement of the existing abutments is required:

**Alternative #1 - Abutment and Wingwall Reinforcement.** The recommended abutment reinforcing system may consist of either:

- Excavation of the abutment backfill, and thickening the abutment section by casting a thicker backwall with reinforced concrete. This essentially increases the mass of the abutment by building-up the gravity section of the abutment. The end result would be acceptable factors of safety against sliding and overturning. Furthermore, construction of a backfill drainage system, and construction of weep holes is required.

- Drill soil or rock anchors through the abutment face, in two to three rows, and grout into the soil behind the abutments. This system essentially increases the mass of the abutment by engaging the soil mass beyond the potential failure wedge. Should the abutment and soil begin to move, the anchors would become tensioned to stabilize the wall. This option would be very difficult to construct, as there is limited room between the railroad tracks and the abutments and limited headroom.

  These reinforcement systems must be engineered to raise the factors of safety of the substructures to 1.5 for sliding and 2.0 for overturning. Design of the reinforcing system is beyond the scope of this report.

**Alternative #2 - Foundation Replacement.** Based on the findings of this report, replacement of the bridge substructures is preferred. Replacement substructures may be supported by shallow foundations. Spread footings should be embedded a sufficient depth to bear on the glacial till unit, which generally consists of medium dense to dense sand. The selection of bottom of footing elevations should be based on an assessment of suitable bearing soil and the depth of frost penetration, and is beyond the scope of this report.

# 1.0 INTRODUCTION

A preliminary subsurface investigation, geophysical investigation and geotechnical recommendations have been completed for the improvement of the Veranda Street Bridge spanning the St. Lawrence and Atlantic Railroad in Portland, Maine. The purpose of this investigation was to explore subsurface conditions at the site and develop geotechnical recommendations for the proposed bridge substructure replacement or bridge substructure rehabilitation.

This Feasibility Study and Preliminary Geotechnical Design Report for the proposed replacement or rehabilitation of the Veranda Street Bridge report presents:

- subsurface data collected from the preliminary geotechnical subsurface investigation;
- geophysical data collected to verify existing abutment geometry;
- subsurface data collected to characterize of the backfill soils of the existing abutments, and the soils the abutments bear upon;
- stability analyses for the existing abutments considering the proposed roadway profile;
- foundation alternatives:
    - substructure reinforcement/rehabilitation
    - substructure replacement and,
- foundation recommendations.

The Project Description for this project, described in the 2004-2005 Biennial Transportation Improvement Program, is *"Bridge Improvement"*. This report has been prepared for preliminary engineering for improvement of the bridge.

Potential bridge modifications include a 910 mm (3.0 ft) raise in grade to provide a 6.1 m (20-foot) under clearance. Jacking the bridge superstructure an additional 760 mm (2.5 ft) to provide a 6.9 m (22.5 ft) under clearance is possible in the future. Hence, the substructures may potentially need to be designed or reinforced to resist the additional earth pressure due to a 1.677 m (5.5 ft) raise in grade.

The Veranda Street Bridge is a simply supported single span, riveted thru girder truss bridge. The superstructure consists of a concrete deck on concrete encased stringers on thru girder floor beams. The condition of the superstructure is very poor. The floor beams are badly deteriorated, and there is severe section loss on webs of main girders. The bottom of the slab is covered with efflorescence and delaminations and scaling has begun.

The substructures of the Veranda Street Bridge are gravity abutments made of unreinforced concrete and older stone ashlar masonry. The original bridge was built before 1930 and consisted of stone masonry abutments with a timber and steel superstructure. In 1930, the timber and steel superstructure was removed and replaced with the current steel superstructure. The stone masonry abutments were symmetrically widened with unreinforced concrete abutments and new wingwalls. The 1930 construction plans for the bridge modifications were obtained from the Railroad (14 sheets, Canadian National Railways, Verandah St. O.H. Bridge, Deering, Maine, Office of the Bridge Engineer, Toronto, dated March 1930) for this evaluation and report. Pictures of the existing abutments are included as Appendix A – Photos of this report.

A MaineDOT Bridge Inspection Report, dated April 24, 2001, states that the bridge has a sufficiency rating of 29.7, on a scale of 0 to 100. The Federal Highway Administration considers a bridge eligible for replacement if its sufficiency rating is less than 50.

## 2.0 GEOLOGIC SETTING

The Veranda Street Bridge carries Veranda Street over the St. Lawrence and Atlantic Railway, in Portland, in Cumberland County, as shown on Sheet 1 – Location Map presented at the end of this report.

According to the Surficial Geology Map, Portland West Quadrangle, Maine, Maine Geological Survey, 1997 (Open-File No. 97-51), the surficial soils in the vicinity of the site consist of the Presumpscot Formation. The Presumpscot Formation is a glaciomarine deposit, which accumulated on the ocean floor during the late-glacial marine submergence of lowland areas in southern Maine. These soils are comprised of silt, clay and minor amounts of sand. The most common component is the clayey silt known as the Presumpscot Formation. Sand is dominant in some areas. The unit also contains areas of till.

According to the Bedrock Geologic Map of Maine, Maine Geological Survey, 1985, the bedrock in the vicinity of the site consists of calcareous pelite of the Macworth Formation, bounded by calcareous sandstone, interbedded sandstone and impure limestone of the Vassalboro Formation.

## 3.0 SUBSURFACE INVESTIGATION

Subsurface explorations were performed to provide information related to the subsurface conditions, abutment backfill and foundation soils.

Subsurface conditions in the vicinity of the existing abutments were explored by drilling five (5) cased wash borings (BB-PRR-101, BB-PRR-102A, BB-PRR-102B, BB-PRR-102C, BB-PRR-102D) and digging one test pit (TP-PRR-101).

The locations of the explorations are shown on Sheet 2 - Boring Location Plan and Sheet 3 - Interpretive Subsurface Profile found at the end of this report. The test borings were drilled by the MaineDOT Materials, Testing and Exploration Division, from July 20, 2004 through July 21, 2004. Borehole logging was completed by the MaineDOT Geotechnical Team Engineer. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the boring logs provided in Appendix B – Boring Logs and on Sheets 4 and 5 – Boring Logs, found at the end of this report.

The borings were drilled using cased wash boring and solid stem auger techniques. Soil samples were obtained at 1.5-meter (5-ft) intervals using Standard Penetration Test (SPT) methods. Bedrock was cored in borings using NQ core barrel and the Rock Quality Designation (RQD) of the core was calculated. The MaineDOT Geotechnical Engineer selected the boring locations and drilling methods, designated type and depth of sampling techniques, identified field and lab testing requirements and maintained the field logs of the subsurface conditions encountered. The borings were located in the field by use of a tape after completion of the drilling program.

## 4.0 LABORATORY TESTING

Laboratory testing consisted of six (6) standard grain size analyses and six (6) natural water content analyses. The results of these laboratory tests are provided in Appendix C – Laboratory Data at the end of this report. Moisture content information and other soil test results are also included on the Boring Logs in Appendix B and on Sheets 4 and 5 - Boring Logs, found at the end of this report.

## 5.0 SUBSURFACE CONDITIONS

An interpretation of the subsurface conditions at the Veranda Street Bridge is graphically illustrated on Sheet 3 - Interpretive Subsurface Profile, found at the end of this report. In general, the soil statigraphy encountered at the site consists of two principle soil units, fill and glacial till, overlying bedrock.

### 5.1 NORTH ABUTMENT

The following paragraphs discuss the soils encountered in order of increasing depth behind the north (Falmouth) abutment:

**Fill –** A layer of fill was encountered in boring BB-PRR-101. The fill materials are highly variable and most likely represent backfill soils for the pre-1930 stone masonry abutments. Four (4) different fill units were encountered behind the abutment:

- Brown, damp, fine to coarse SAND, trace silt, little gravel.
- Brown, damp to moist, fine to coarse silty SAND, trace brick, wood and gravel.
- Olive brown, moist, fine silty SAND, trace medium to coarse sand and gravel, with iron staining.
- Red brown, wet, fine to coarse SAND, some gravel, trace to some silt with iron staining.

SPT N-values in the upper fill layer ranged from 6 to 114 blows per foot, indicating that the soil is loose to very dense in consistency. The high blow counts are attributed to obstructions. The thickness of the fill unit is 7.3 m (24 feet).

One sample of the fill unit from BB-PRR-101 was tested and was classified as Unified SM (AASHTO A-4) and described as silty SAND. The measured water content was 21.3%. A second sample was tested and classified as Unified SW-SM (AASHTO A-1-b) and described as fine to coarse SAND, some gravel, little silt. The measured water content was 8.4%.

Fill soils were also encountered in test pit TP-PRR-101, which was hand dug in front of the north stone masonry abutment. The test pit extended to a depth of 1.2 m (4 ft), and did not encounter the bottom of the stone footing. The soils that embed the footing at the location of the TP-PRR-101 consist of 1.2 m (4.0 feet) of brown and black, dry, fine to coarse sand, some gravel, little silt, trace of brick, cobbles, coal and slag.

**Glacial Till –** Native glacial till deposits were encountered underlying the fill. The till unit generally consists of:

- Brown and red-brown, wet, fine to coarse SAND, trace silt, trace to no gravel, and,
- Grey, wet, fine to coarse silty SAND.

SPT N-values in the glacial till layer ranged from 43 to 80 blows per foot, indicating that the glacial till unit is dense to very dense in consistency.

One soil sample of the till unit from BB-PRR-101 was tested and is classified as Unified SP-SM (AASHTO A-3), and described as fine to coarse sand, trace silt, trace of gravel. The measured water content was 15.5 %.

### 5.2 SOUTH ABUTMENT

The following paragraphs discuss the soils encountered in order of increasing depth behind the south (Portland) abutment.

**Fill –** Fill materials from the 1930 bridge widening were encountered in boring BB-PRR-102A. This boring terminated with a concrete core of the heel of the abutment footing. Boring BB-PRR-102D sampled the backfill between the face of the older, buried stone wingwall and the newer 1930 wingwall. One consistent fill unit was encountered behind the south abutment:

- Brown, damp to wet, fine to coarse SAND, some to trace of silt, little to trace gravel.

SPT N-values in the fill layer ranged from 4 to 47 blows per foot, indicating that the soil is loose to dense in consistency. The thickness of the fill unit is approximately 7.5 m (25 ft)

Borings BB-PRR-102B and BB-PRR-102C were both abandoned after hitting the remains of the abandoned southeast wingwall from the pre-1930 stone masonry substructure, which was left in place and buried in backfill with the bridge widening in 1930.

**Glacial Till –** Glacial till deposits were encountered underlying the fill. The deposit generally consisted of:

- Brown, moist, fine to coarse SAND, little gravel, trace silt.
- Red/brown to grey, wet, fine to coarse SAND, trace silt, trace gravel.

SPT N-values in the fill layer ranged from 47 to 59 blows per foot, indicating that the till unit is dense to very dense in consistency.

Three soil samples of the till unit from BB-PRR-102D were tested and classified as Unified SW-SM (AASHTO A-1-b), Unified SP-SM (AASHTO A-3) and Unified SM (A-2-4). Measured water contents for samples of the till unit range from 10.7 to 18.2%.

### 5.3 BEDROCK AND GROUNDWATER CONDITIONS

**Bedrock -** The site is underlain by the Macworth Formation. The bedrock surface was encountered and cored at a depth of 12.5 m (40.9 ft) bgs in boring BB-PRR-101, and at a depth of 12.3 m (40.2 ft) bgs in boring BB-PRR-102D. The bedrock observed in the core samples recovered from the explorations is described as grey and white, fine grained, hard, fresh, slightly metamorphosed SILTSTONE. The RQD of the bedrock ranged from 76 to 82%, indicating a bedrock quality of fair to good.

**Groundwater** - The depth of groundwater measured in the boring BB-PRR-101 was 8.84 m (29 ft) bgs (el. 2.86 m (9.38 ft)). Groundwater levels may fluctuate due to conditions other than those present at the time measurements were made.

## 6.0 GEOPHYSICAL INVESTIGATION

A ground penetrating radar (GPR) survey and sonic/ultrasonic investigation of the Veranda Street Bridge substructures was conducted on August 31, 2004. The investigation and data analysis was performed by NDT Corporation of Worcester, Massachusetts. The objective of the geophysical investigation was to determine the approximate thickness of the concrete and stone masonry abutments. The results of the geophysical investigation are discussed in Section 8.0 of this report. The report prepared by NDT Corporation is included as Appendix D of this report.

## 7.0 CONDITION OF SUBSTRUCTURES

The Veranda Street Bridge abutments and retaining walls consist of stone masonry abutments with unreinforced concrete abutment modifications and unreinforced concrete wingwalls. The stone masonry abutments date to a bridge constructed prior to 1930. The unreinforced concrete abutment and wingwall modifications were constructed in 1930 when the earlier superstructure was replaced with a wider superstructure. All substructures are supported on spread footings founded on the glacial till layer. See Appendix A – Photos for photographs of the abutments.

Overall Substructure Condition. The newer concrete abutments and wingwalls are in good condition, but the concrete is unreinforced, with the exception of steel rails cast into the footings. The rails are shown on the 1930 construction drawings obtained from the railroad. The rails are 2.5 m (8.25 ft) to 2.6 m (8.5 ft) long with a spacing of 900 mm (3 ft). The rails are placed in the toes and centers of the footings of the abutments, with 75 mm (3 in) of concrete cover.

The concrete-capped masonry stone abutments from the pre-1930 bridge which carried an electric trolley and are visually in good condition, however, the GPR geophysical data show that behind the stone abutment face the blocks are unbonded and moisture has infiltrated the individual stones. The unbonded and moisture infiltrated nature of the stone abutments compromises the integrity of the construction.

MaineDOT Bridge Inspection Records (2001) assign a condition rating of 6 out of 9 to the substructures. This indicates that the substructures are in “satisfactory condition”.

Pointing. The pointing has failed at scattered locations in the courses of the stone masonry. There is some deterioration of the concrete caps.

Drainage. The pre-1930 stone substructures were constructed without a drainage system. The potential exists for hydrostatic pressures to develop behind the abutments from surface runoff might collects in the backfill. The unreinforced concrete substructures were constructed with 2 weep holes per abutment. The weep holes are 100 mm (4 in) diameter cast iron pipes and are located in the extreme corners where the abutment face meets the wingwall. The wingwalls are backfilled with 300 to 460 mm (12 to 18 in) of crushed stone with a 150 mm (6 in) “tile drain”.

Frost. The water table is low enough that abutment backfill soils will not become saturated from capillarity, and the roadway profile allows surface water to be removed rapidly without saturating the underling backfill soils. The absence of a high groundwater table and source of water implies that prolonged cold weather will not result in ice lenses. With the exception of one backfill sample which is classified as having a high degree of frost susceptibility (Frost Class III[1]), all backfill samples have a negligible to low degree of frost susceptibly (Frost Class 0 to I).

Stone Masonry Abutment Foundation Soils and Backfill. The data indicate that both stone masonry abutments bear on dense to very dense, fine to coarse sand, some to trace of gravel, with trace of silt. The soil backfill behind the stone abutments is generally described as loose to dense, damp to moist, silty fine to coarse sand, trace of brick wood and gravel, overlying a layer of olive brown, wet, medium dense to dense, fine silty sand with iron staining.

Unreinforced Concrete Abutment Foundation Soils and Backfill. The data indicate that the concrete abutment extensions and concrete wingwalls bear on brown, wet to moist, dense to very dense, fine to coarse sand with some to no gravel, with trace of silt. The backfill is generally described as loose, damp, sand, trace of silt overlying very loose to medium dense, silty sand, trace of gravel.

Abutment Footing Embedment. Test pit TP-PRR-101 was hand dug in front of the stone masonry section of the north abutment. The test pit extended to a depth of 1.2 m (4 ft), and did not encounter the bottom of the stone footing. The soils that embed the footing at the location of the test pit consisted of brown and black, dry, fine to coarse sand, some gravel, little silt, trace of brick, cobbles, coal and slag.

[1] MaineDOT and U. S. Army Corps of Engineers Classification System.

# 8.0 INFERRED ABUTMENT GEOMETRY

A ground penetrating radar (GPR) survey and sonic/ultrasonic investigation of the Veranda Street Bridge substructures was conducted on August 31, 2004. The investigation and data analysis was performed by NDT Corporation of Worcester, Massachusetts. The objective of the geophysical investigation was to determine the approximate thickness of the concrete and stone masonry abutments. The report prepared by NDT Corporation is included as Appendix D of this report.

## 8.1 METHODS OF INVESTIGATION

The GPR method uses a pulsed electromagnetic signal that is transmitted to and reflected by subsurface features, back to the point of transmission. Metal reinforcing or changes in the electrical properties of subsurface materials (e.g. moisture) produce strong reflections of radar signals. GPR data was used to determine the thickness of the abutments and wingwalls by detecting the interface of the abutment backface and granular backfill.

Sonic/ultrasonic reflection measurements were used to calibrate the GPR data and verify the thickness of the concrete and masonry abutments determined with GPR. Sonic/ultrasonic measurements determine the characteristics of the abutment with stress waves created by an impact energy source. Direct measurements are made of the compression and shear wave velocities and the reflected compression wave phases. The time it takes for the compressional wave to propagate to the end of a subsurface feature and be reflected back to the source is related directly to the length of the path to the feature,[2] assuming predetermined compression wave velocity values for the concrete and rock. The computed length of the path is related to abutment thickness.

A complete discussion of the methods of investigation is included in Appendix D.

## 8.2 GEOPHYSICAL TEST RESULTS

GPR data was collected along 6 vertical lines and 2 horizontal lines. The GPR survey lines are illustrated in Figures 2 and 4 of Appendix D. Survey lines were selected by the MaineDOT Team Geotechnical Engineer.

The geophysical results and inferred abutment geometries for the south abutment are presented as cross sections in Figures 2 and 3 of Appendix D. The geophysical results and inferred abutment geometries for the north abutment are presented as cross sections in Appendix D - Figures 4 and 5. These findings are summarized in Tables 1 and 2.

[2] The time of reflection (t) is two times the length of the structure divided by the wave's velocity ($t=2 \times L/v$)

| Abutment | Abutment Section | Abutment Thickness at top m (ft) | Abutment Thickness at Ground Surface m (ft) |
|---|---|---|---|
| South | Western Concrete | 1.62 (5.3) | 2.0 (6.6) |
| South | Stone Masonry | 1.62 (5.3) | 1.77 to 2.0 (5.8 to 6.6) |
| South | Eastern Concrete | 1.0 (3.3) | 1.5 (5.0) |

**Table 1.** South Abutment Geometries based on GPR Survey.

| Abutment | Abutment Section | Abutment Thickness at Top m (ft) | Abutment Thickness at Ground Surface m (ft) |
|---|---|---|---|
| North | Western concrete | 0.762 (2.5) | 1.5 (5.0) |
| North | Stone Masonry | 0.762 (2.5) | 2.0 (6.6) |
| North | Eastern concrete | 0.762 (2.5) | 2.0 (6.6) |

**Table 2.** North Abutment Geometries based on GPR Survey.

The geophysical report is attached as Appendix D of this report. It should be noted that actual abutment geometries may vary from the inferred geometries. The GPR and sonic/ultrasonic data also indicated, or failed to provide, the following:

1. GPR data collected at the south stone masonry abutments showed strong reflectors 1 m (3.3 ft) to 1.27 m (4.17 ft) behind the abutment face in middle of the abutments, indicating the bonding of the masonry blocks is fractured and filled with moisture. The moisture blocked GRP signals, thus Appendix D - Figure 3, reports that the abutment is only 1 to 1.27 m (40 to 50 inches) thick in this area.

2. In general, the frequency of irregular reflectors in the stone abutment was high, indicating that there are extensive areas of “un-bonded” blocks.

3. In the unreinforced concrete sections of the abutments and wingwalls, systematic vertical GRP reflectors were detected at 460 to 600 mm (1.5 to 2 ft) deep, at 1.2 m (4 ft) spacing. MaineDOT Bridge Maintenance was consulted, and the reflectors are consistent with the practice of casting vertical steel “bars” in the concrete as reinforcement. These bars are not shown on the 1930 construction plans.[3]

4. The GPR and sonic/ultrasonic data did not fully corroborate the 1930 construction plans[4] for the concrete abutment sections. The data indicate that the actual abutment cross section is thinner in some areas is than that shown on the 1930 plans.

5. No GPR data was collected to confirm depth of footing.

6. The data shows that the wingwalls of the older south masonry abutments are buried behind the 1930 concrete abutments and wingwalls.

7. GPR data was collected for the wingwalls. The findings are reported in Appendix D.

## 9.0 STONE MASONRY ABUTMENT STABILITY ANALYSIS

A 910 mm (3 ft) to 1677 mm (5.5 ft) raise in the roadway profile is being considered as part of the current Veranda Street Bridge project.

The overall stability of the south and north stone masonry abutments was assessed considering the geometry and backfill soils confirmed by the ground penetrating radar study and the borings. Inferred abutment geometries used in the abutment stability analyses are shown in Table 3 and shown in Appendix E – Calculations for Stone Masonry Abutment Stability Analysis.

[3] 14 sheets, Canadian National Railways, Verandah St. O.H. Bridge, Deering, Maine, Office of the Bridge Engineer, Toronto, dated March 1930.
[4] 14 sheets, Canadian National Railways, Verandah St. O.H. Bridge, Deering, Maine, Office of the Bridge Engineer, Toronto, dated March 1930.

| Stone Masonry Abutments | | | |
|---|---|---|---|
| Abutment | Abutment Thickness at top m (ft) | Abutment Thickness at Ground Surface m (ft) | Basis of Model |
| North | 0.84 (2.75) | 2.0 (6.6) | Average of GPR Scan Lines #266 and #267, Figure 4 of Appendix D. |
| South | 1.60 (5.25) | 2.13 (7.0) | GPR Scan Line #250, Figure 2 of Appendix D. |

**Table 3**. Inferred Abutment Cross Sectional Thickness Assumed in Stability Analyses.

Earth pressures were computed using Coulomb theory. Abutment stability was analyzed considering the earth pressures due to the potential raise in the roadway profile by 910 mm (3.0 ft). A conventional method of overturning and sliding analysis was used for the calculations. The calculated factors of safety were compared with the AASHTO specified factors of safety of 2.0 and 1.5 against overturning and sliding, respectively. The calculations are provided in Appendix E – Calculations for Stone Masonry Abutment Stability Analysis.

The following assumptions were made for the analysis:

- The restoring load and moment of the proposed superstructure and live load was included as a stabilizing force and resisting moment in the analysis.
- Horizontal components of the superstructure dead load (DL) and live load (LL) were excluding from destabilizing forces and moments.
- No traffic surcharge was included, assuming use of an approach slab.
- A depth of footing embedment of 1.2 m (4.0 ft) and a toe of 122 mm (0.4 ft) was assumed based test pit record TP-PRR-101.

With the raise in roadway profile, the overall stability of the north stone masonry abutment is calculated to be approximately 1.3 against overturning and 1.4 against sliding, compared with the AASHTO required factors of safety of 2.0 and 1.5, respectively. The overall stability of the south stone masonry abutment is calculated to be approximately 1.2 against overturning and 1.6 against sliding. These factors of safety represent the higher range, as the design assumptions are unconservative.

| Abutment | FS Against Overturn-ing | FS Against Sliding | Maximum Toe Bearing Pressure kPa (ksf) | Assumed Load Conditions |
|---|---|---|---|---|
| North | 1.1 | 1.2 | 957 (20) | 1. 910 mm ( 3 ft) raise in roadway profile<br>2. approach slab included<br>3. superstructure DL and LL stabilizing forces included. |
| South | 1.0 | 1.4 | 1101 (23) | 1. 910 mm (3 ft) raise in roadway profile<br>2. approach slab included<br>3. superstructure DL and LL stabilizing forces included. |

**Table 4.** Factors of Safety of Stone Masonry Abutments.

The ultimate bearing capacity of the foundation soils is estimated to be 24 ksf. The calculations are provided in Appendix E – Calculations for Stone Masonry Abutment Stability Analysis. With the proposed raise in the roadway profile, the maximum bearing pressures at the footing toe approach the ultimate soil bearing capacity.

Based on the stability analyses in this Section, the factors of safety against overturning, sliding and bearing capacity for the existing stone substructures are unacceptable if the roadway profile is raised 910 mm (3 ft). The overfill results in increased earth pressure loads on the abutment and increased footing bearing pressures. The stability analyses indicate that in order to reuse the existing substructures significant reinforcement and rehabilitation is required to resist the additional earth pressure loads.

## 13.0 UNREINFORCED CONCRETE ABUTMENT STABILITY ANALYSIS

A 910 mm (3 ft) to 1677 mm (5.5 ft) raise in the roadway profile is being considered as part of the current Veranda Street Bridge project.

The overall stability of the south and north concrete abutments was assessed considering the geometry determined from the GPR study. The inferred abutment geometries used in the abutment stability analyses are shown in Table 4 and graphically illustrated in Appendix F – Circa 1930 Concrete Abutment Stability Analyses.

| Unreinforced Concrete Abutment (circa 1930) | | | |
|---|---|---|---|
| | Abutment Thickness at top m (ft) | Abutment Thickness at Ground Surface m (ft) | Basis of Abutment Cross Section |
| North | 1.3 (4.25 ) | 2.2 (7.2) | GPR Scan Line #270, Figure 4 of Appendix D. |
| South | 1.6 (5.25) | 2.0 (6.6) | GPR Scan Line #247, Figure 2 of Appendix D. |

**Table 5**. Inferred Abutment Cross Sectional Thickness assumed in Stability Calculations.

Earth pressures were computed using Coulomb theory. Abutment stability was analyzed considering the earth pressures due to the potential raise in the roadway profile by 910 mm. A conventional method of overturning and sliding analysis was used for the calculations. The calculated factors of safety were compared with the AASHTO specified factors of safety of 2.0 and 1.5 against overturning and sliding, respectively. The calculations are provided in Appendix F – Calculations -Circa 1930 Concrete Abutment Stability Analyses.
.
The following assumptions were made for the analysis:

- The restoring load and moment of the proposed superstructure and live load was included as a stabilizing force and resisting moment in the analysis.
- Horizontal components of the superstructure dead load (DL) and live load (LL) were excluded from destabilizing forces and moments.
- No traffic surcharge was applied assuming addition of an approach slab.
- A depth of footing embedment of 1.8 m (6.0 ft) and a toe length of 910 mm (3 ft) was assumed based the 1930 bridge plans.
- The GPR data indicates that some cross sections of the concrete abutments may be approximately 760 mm (2.5 ft) at the top and 1.52 m (5.0 ft) at the base. A thicker, more representative average cross section was used the analysis.

With the raise is the roadway profile, the overall stability of the north concrete abutment sections (assuming both eastern and western widened sections to be identical for the purposes of the analysis) is calculated to be approximately 1.8 against overturning and 1.2 against sliding compared with the AASHTO required factors of safety of 2.0 and 1.5. The overall stability of the south abutment concrete sections is calculated to be approximately 2.0 against overturning and 1.3 against sliding.
These factors of safety represent the higher range, as the design assumptions are unconservative. These stability analyses indicate that the south and north unreinforced concrete abutments require some reinforcement to improve the factors of safety against sliding and overturning in order to be reused.

| Abutment | FS Against Overturn-ing | FS Against Sliding | Maximum Toe Bearing Pressure kPa (ksf) | Assumed Load Conditions |
|---|---|---|---|---|
| North | 1.7 | 1.2 | 445 (9.3) | 1. 900 mm raise in roadway profile<br>2. approach slab included<br>3. superstructure DL and LL stabilizing forces included. |
| South | 2.0 | 1.3 | 303 (8) | 1. 900 mm raise in roadway profile.<br>2. approach slab included<br>3. superstructure DL and LL stabilizing forces included. |

**Table 6.** Factors of Safety for circa 1930 Unreinforced Concrete Abutments.

The ultimate bearing capacity of the foundation soil is estimated to be 24 ksf. With the proposed raise in the roadway profile of 910 mm, the resulting bearing pressure at the footing toe results in an approximate factor of safety of 3.0 against bearing capacity failure.

Based on the stability analyses in this Section, the factors of safety against sliding, overturning and bearing capacity for the existing substructures are marginally acceptable if the roadway profile is raised 910 mm (3 ft). The overfill results in increased earth pressure loads on the abutment and increased footing bearing pressures. The stability analyses indicate that in order to reuse the existing substructures minor reinforcement and rehabilitation will be required to resist the additional earth pressure loads.

## 11.0 WINGWALL STABILITY ANALYSIS

The overall stability of the existing unreinforced concrete wingwalls is not assessed in this report. However, the proposed raise in the roadway profile will result in increased earth pressure loads on the walls and increased footing bearing pressures. In order to reuse the existing wingwalls, it is likely that some reinforcement and rehabilitation will be required to resist the additional earth pressure loads. If the wingwalls are to be reused, to retain the additional fill on the side slopes U-shaped wingwalls will be required above the existing walls, or the existing wingwalls extended vertically with caps. It is recommended that the stability of the wingwalls be assessed assuming the proposed raise in roadway profile if the decision is made to reuse the abutments.

## 12.0 FOUNDATION ALTERNATIVES

Based on the abutment stability analyses, reinforcement for reuse or replacement is required. Both options are discussed below.

### 12.1 SUBSTRUCTURE REINFORCEMENT

The recommended abutment and wingwall reinforcing system may consist of either:

1. Excavation of the abutment backfill, <u>thickening the cross section</u> of the abutment mass with cast-in-place concrete, construction of a backfill drainage system and weep holes. This system increases the restoring moment and restoring forces of the abutment by building up the gravity section of the abutment, and reduces the potential for hydrostatic pressure with inclusion of a drainage system.

2. <u>Drilling and installing soil anchors</u> through the abutment face. The design is beyond the scope of this report, however, it is estimated that a minimum of 2 rows of 180 kN (40 kip) soil anchors, at 1.2 m (4 ft) on-center spacing, at 15 degree angles, and grouted into the soil behind the abutments, will be required. This system essentially increases the mass of the abutment by engaging the soil mass beyond the potential failure wedge. Should the abutment and soil begin to move, the anchors would become tensioned to stabilize the wall. Limited headroom due to the superstructure, and railroad traffic will complicate installation of anchors.

3. <u>Spread footings improvements.</u> Substructure rehabilitation should verify that the abutment and wingwall footings are embedded for frost. It is also important to verify that footing dimensions and embedment depths for the purpose of better estimating the applied footing pressures. In light of only limited test pit data, recommendations about footing improvement are beyond the scope of this report.

### 12.2 SUBSTRUCTURE REPLACEMENT

Based on the findings and conclusions of this report, replacement bridge substructures are preferred. The new structure may be supported by shallow foundations:

***Spread Footings.*** Spread footings should be embedded a sufficient depth to bear on the glacial till unit, which generally consists of medium dense sand. The selection of bottom of footing elevations should be based on an assessment of suitable bearing soil and depth of frost penetration, and is beyond the scope of this report. In general, a minimum embedment of 1.2 m (4 ft) will be required for frost protection.

## 13.0 GEOTECHNCIAL DESIGN RECOMMENDATIONS

Due to the condition of the abutments, the results of stability analyses, and the potential cost of abutment reinforcement, replacement of the bridge substructures is recommended.

Based on verified abutment dimensions, and substructure stability analyses, it has been determined that re-use of the existing bridge substructures under the proposed roadway profile raise *is feasible only with significant reinforcement.* However, due to the condition of the abutments and the results of stability analyses, the potential cost of abutment reinforcement can be significant. Prior to any scope decision, a life cycle cost analysis is recommended, as the cost of a properly engineered substructure reinforcement project can likely exceed that of substructure replacement. For the purpose of such an analysis, two abutment reinforcement alternatives have been provided in this report, and these reinforcement alternatives must be engineered to provide the substructures with the AASHTO required factors of safety of 1.5 and 2.0, against sliding and overturning, respectively. The life cycle cost analysis task is beyond the scope of this report.

## 14.0 FUTURE GEOTECHNICAL WORK

In the situation that substructure reuse and rehabilitation is chosen, more geotechnical work is necessary. Some assumptions were made in the stability analyses in this report, which need confirmation. A depth of footing embedment of 1.8 m (6.0 ft) and a toe length of 910 mm (3 ft) was assumed for the analysis of the concrete abutments, based the 1930 bridge plans. This assumption must be confirmed with test pits. Confirmed footing dimensions and embedment depths should be used to refine estimates of factors of safety against overturning, and factor of safety against bearing capacity failure under the proposed conditions. It is also important to verify that the both the stone masonry and concrete abutment and wingwall footings are embedded for frost protection.

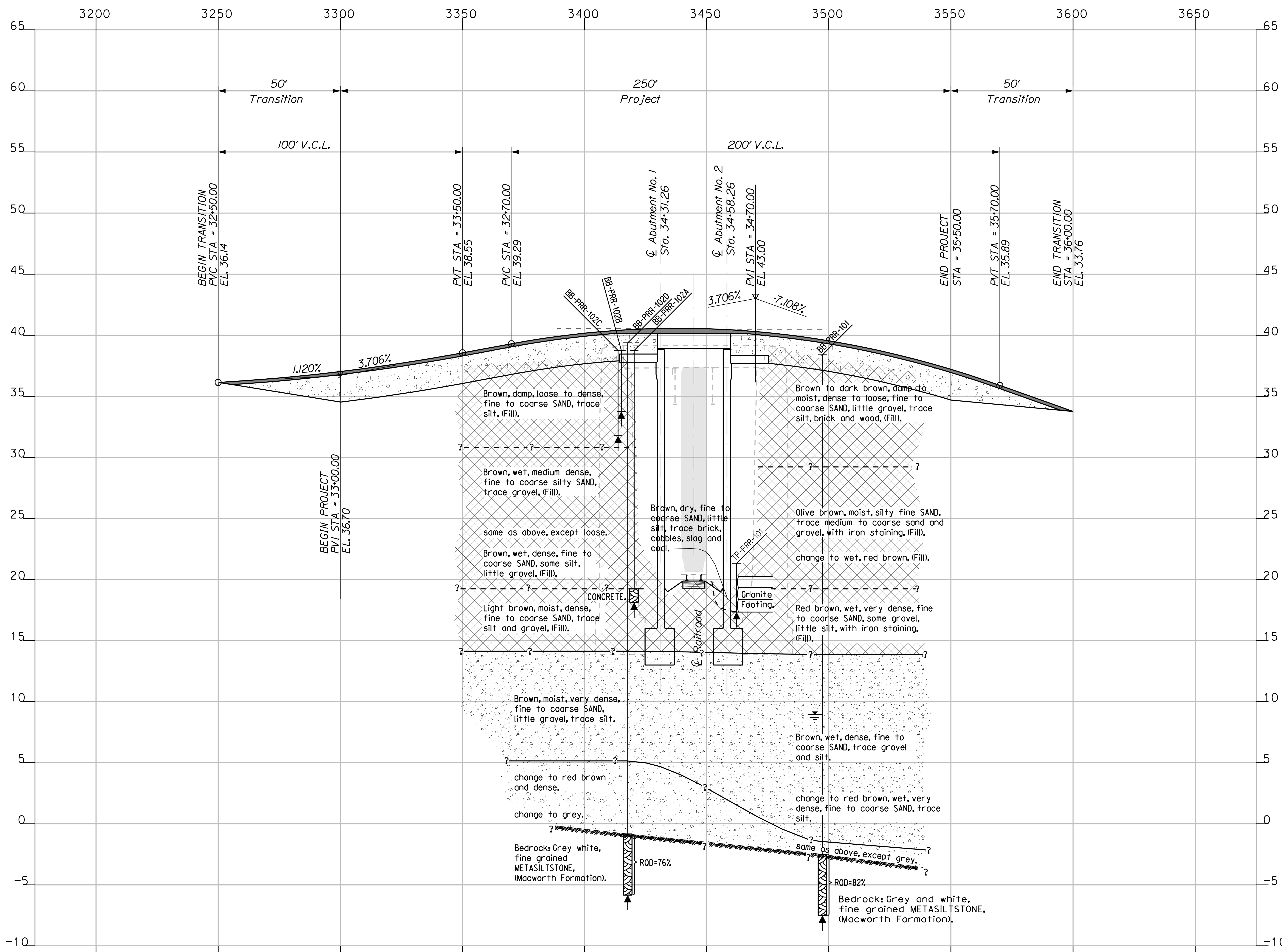
In the situation that the final project scope is substructure replacement, geotechnical design criteria for bearing capacity, settlement, frost protection and seismic loads shall be developed.

## 15.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program and the Bureau of Planning, for the specific application to the proposed rehabilitation or replacement of the Veranda Street Bridge in the city of Portland, Maine, in accordance with generally accepted soil and foundation engineering practices. No other intended use is implied. In the event that any changes in the nature, design or location of the proposed project are planned this report should be reviewed by the geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also be necessary to re-evaluate the recommendations made in this report.

It is recommended that the geotechnical engineer be provided the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly implemented in the design.

**SHEETS**




PROFILE

HORIZ 25 0 25 50

VERT 5 0 5 10

SCALE

Note: This generalized interpretive soil profile 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 transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

Filename: ...\00\GEOTECH\MSTA\007_ISP1.dgn Division: GEOTECH Username: laura.krusinski Date:1/26/2009

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1015(800)X

PIN 010158.00

BRIDGE NO. 5052

BRIDGE PLANS

| | BY | DATE |
|---|---|---|
| PROJ. MANAGER | J. WENTWORTH | |
| DESIGN-DETAILED | R. NADUS | TEAM SOUTH |
| CHECKED-REVIEWED | | |
| DESIGN2-DETAILED2 | K. MAGUIRE | T. WHITE | JAN 2009 |
| DESIGN3-DETAILED3 | | |
| REVISIONS 1 | | |
| REVISIONS 2 | | |
| REVISIONS 3 | | |
| REVISIONS 4 | | |
| FIELD CHANGES | | |

SIGNATURE

P.E. NUMBER

DATE

VERANDA STREET OVERPASS
ST. LAWRENCE & ATLANTIC R.R.
PORTLAND CUMBERLAND COUNTY

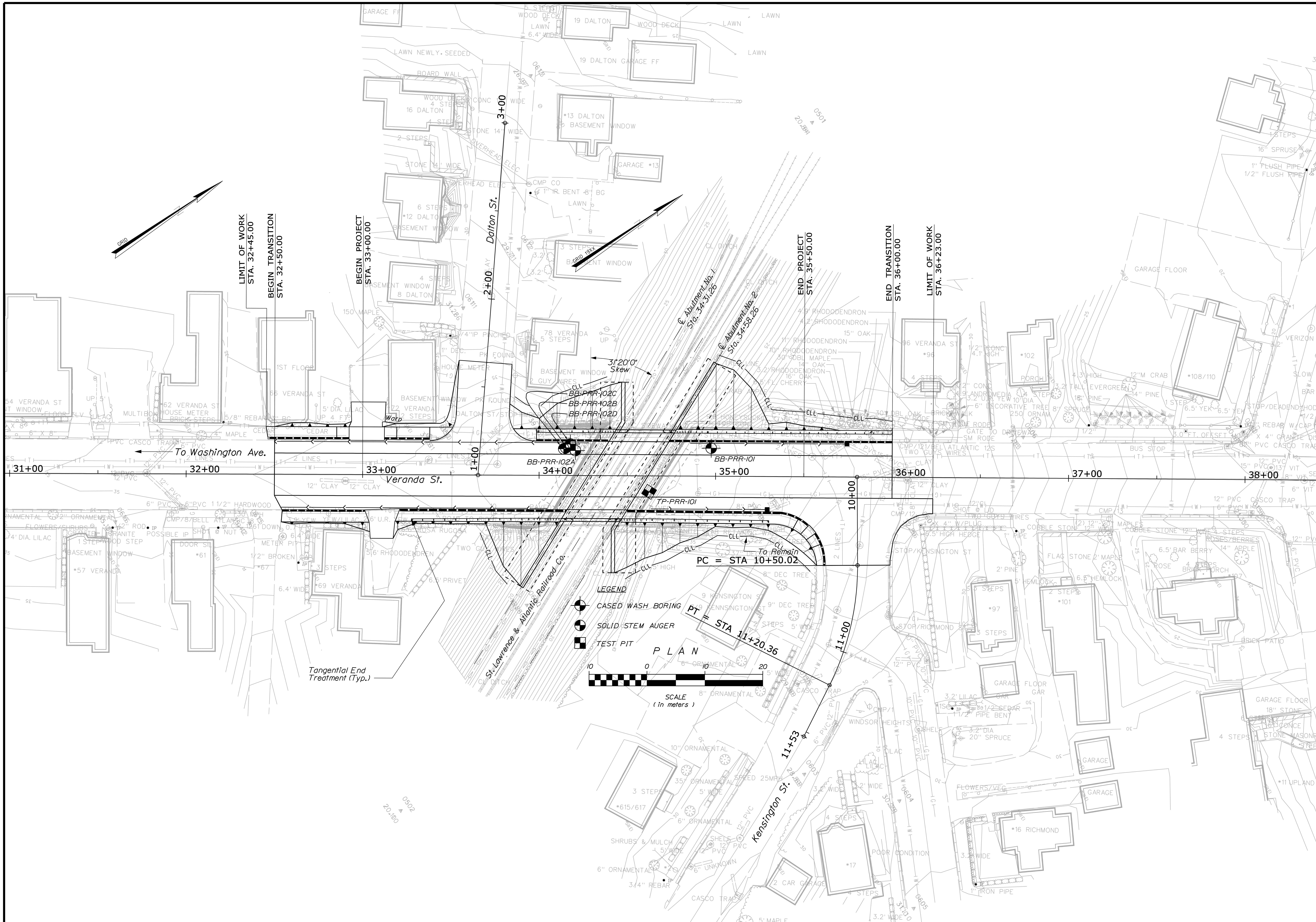
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER

7

OF 36

Filename: ...\00\GEOTECH\MSTA\006_BLP1.dgn Division: GEOTECH Username: terry.white Date:1/14/2009




STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1015(800)X

PIN 010158.00

BRIDGE PLANS

BRIDGE NO. 5052

| | BY | DATE | SIGNATURE |
|---|---|---|---|
| PROJ. MANAGER | J. WENTWORTH | | |
| DESIGN-DETAILED | R. NAOUS | TEAM SOUTH | |
| CHECKED-REVIEWED | | | |
| DESIGN2-DETAILED2 | K. MAGUIRE | T. WHITE | JAN 2009 |
| DESIGN3-DETAILED3 | | | |
| REVISIONS 1 | | | |
| REVISIONS 2 | | | |
| REVISIONS 3 | | | |
| REVISIONS 4 | | | |
| FIELD CHANGES | | | |

P.E. NUMBER

DATE

VERANDA STREET OVERPASS
ST. LAWRENCE & ATLANTIC R.R.
PORTLAND CUMBERLAND COUNTY

# BORING LOCATION PLAN

SHEET NUMBER

2

OF 5

Filename: ...\MSTA\008_BORING LOGS1.dgn    Division: GEOTECH    Username: laura.krusinski    Date:1/26/2009

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | BB-PRR-102C |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 38.71 | Auger ID/OD: | 125 mm |
| Operator: | C. Mann | Datum: | NGVD | Sampler: | N/A |
| Logged By: | K. Maguire | Rig Type: | CME 45C | Hammer Wt./Fall: | N/A |
| Date Start/Finish: | 7/21/04-7/21/04 | Drilling Method: | Solid Stem Auger | Core Barrel: | N/A |
| Boring Location: | 34+13.9, 15.1 Lt. | Casing ID/OD: | N/A | Water Level*: | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | | | No sampling conducted in boring. | |
| 7.000 | | | | | | | 31.71 | | Bottom of Exploration at 7.00 feet below ground surface. Obstruction. Could not auger through, abandon hole. | |

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-PRR-102C

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | BB-PRR-102B |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 38.71 | Auger ID/OD: | 125 mm |
| Operator: | C. Mann | Datum: | NGVD | Sampler: | N/A |
| Logged By: | K. Maguire | Rig Type: | CME 45C | Hammer Wt./Fall: | N/A |
| Date Start/Finish: | 7/21/04-7/21/04 | Drilling Method: | Solid Stem Auger | Core Barrel: | N/A |
| Boring Location: | 34+15, 15.6 Lt. | Casing ID/OD: | N/A | Water Level*: | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | | | No sampling conducted in boring. | |
| 5.000 | | | | | | | 33.71 | | Bottom of Exploration at 5.00 feet below ground surface. Casing very crooked, abandon hole. | |

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-PRR-102B

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | BB-PRR-102D |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 39.37 | Auger ID/OD: | 125 mm SSA |
| Operator: | C. Mann | Datum: | NGVD | Sampler: | Standard Split Spoon |
| Logged By: | K. Maguire | Rig Type: | CME 45C | Hammer Wt./Fall: | 63.5 kg/760 mm |
| Date Start/Finish: | 7/20/04-7/20/04 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ |
| Boring Location: | 34+17.6, 17.7 Lt. | Casing ID/OD: | HW | Water Level*: | None Observed |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows |
|---|---|---|---|---|---|
| | | | | | SSA |
| | | | | | 57 |
| | | | | | 32 |
| | | | | | 28 |
| | | | | | 34 |
| | | | | | 24 |
| | | | | | 21 |
| | | | | | 17 |
| | | | | | 26 |
| | | | | | 90 |
| 1D | 24/10 | 19.00 - 21.00 | 51/26/21/16 | 47 | 44 |
| | | | | | 45 |
| | | | | | 29 |
| | | | | | 36 |
| | | | | | 53 |
| 2D | 24/9 | 24.00 - 26.00 | 18/24/15/15 | 39 | 27 |
| | | | | | 45 |
| | | | | | 73 |
| | | | | | 102 |
| | | | | | 103 |
| 3D | 24/12 | 29.00 - 31.00 | 25/28/31/30 | 59 | 25 |
| | | | | | 42 |
| | | | | | 92 |
| | | | | | 110 |
| | | | | | 71 |
| 4D | 24/15 | 34.00 - 36.00 | 19/20/27/37 | 47 | 59 |
| | | | | | 72 |
| | | | | | 110 |
| | | | | | 150 |
| | | | | | 170 |
| 5D | 14.3/8 | 39.00 - 40.19 | 21/15/50(50) | --- | 78 |
| R1 | 60/58 | 40.20 - 45.20 | RQD = 76% | | a43 NQ |

| Depth (ft.) | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|
| 0 | | Refer to BB-PRR-102A for sampling information on upper 19.0'. | |
| 19.000 | 20.37 | Brown, wet, dense, fine to coarse SAND, some silt, little gravel. (Fill). | |
| 22.000 | 17.37 | El 17.39' (depth bgs 21.98') ESTIMATED B.O.F. of ashlar stone abutment | |
| | | Light brown, moist, dense, fine to coarse SAND, trace silt and gravel. (Fill). | |
| 25.000 | 14.37 | El. 14.76' (depth of 24.6') ESTIMATED B.O.F. of concrete abutment | |
| | | Brown, moist, very dense, fine to coarse SAND, little gravel, trace silt. | G#176158 A-1-b, SW-SM WC=10.7% |
| 34.000 | 5.37 | Red/brown, moist, dense, fine to coarse SAND, trace silt, trace gravel. | G#176159 A-3, SP-SM WC=18.2% |
| | | Grey, wet, fine to coarse SAND, little silt, trace gravel. | G#176160 A-2-4, SM WC=16.2% |
| | | a43 blows for 2". | |
| 40.200 | -0.83 | Bedrock: Grey white, fine grained METASILTSTONE. (Macworth Formation). R1: Core Times (min:sec) 40.2-41.2' (7:42) 41.2-42.2' (7:17) 42.2-43.2' (7:00) 43.2-44.2' (7:43) 44.2-45.2' (8:31) 96% Recovery | |
| 45.200 | -5.83 | Bottom of Exploration at 45.20 feet below ground surface. | |

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-PRR-102D

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | BB-PRR-102A |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 38.71 | Auger ID/OD: | 125 mm SSA |
| Operator: | C. Mann | Datum: | NGVD | Sampler: | Standard Split Spoon |
| Logged By: | K. Maguire | Rig Type: | CME 45C | Hammer Wt./Fall: | 63.5 kg/760 mm |
| Date Start/Finish: | 7/20/04-7/20/04 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ |
| Boring Location: | 34+20.5, 14.1 Lt. | Casing ID/OD: | NW | Water Level*: | None Observed |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows |
|---|---|---|---|---|---|
| 1D | 24/15 | 0.50 - 2.50 | 18/21/14/11 | 35 | SSA |
| 2D | 24/14 | 5.00 - 7.00 | 2/2/6/6 | 8 | 10 |
| | | | | | 14 |
| | | | | | 17 |
| | | | | | 36 |
| | | | | | 24 |
| 3D | 24/4 | 10.00 - 12.00 | 10/8/6/6 | 14 | 18 |
| | | | | | 18 |
| | | | | | 13 |
| | | | | | 11 |
| 4D | 24/6 | 14.00 - 16.00 | 5/2/2/1 | 4 | 6 |
| | | | | | 7 |
| | | | | | 10 |
| | | | | | 11 |
| | | | | | a24 |
| R1 | 13/13 | 19.50 - 20.58 | RQD = N/A% | | NQ |

| Depth (ft.) | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|
| 0.500 | 38.21 | 6" PAVEMENT. | |
| | | Brown, damp, loose to dense, fine to coarse SAND, trace silt, pavement in nose of spoon. (Fill). | |
| | | Similar to above, loose. | |
| 8.000 | 30.71 | Brown, wet, medium dense, fine to coarse silty SAND, trace gravel. (Fill). | |
| | | Similar to above, but very loose. | |
| | | a24 blows for 6". | |
| 19.500 | 19.21 | R1: Concrete. R1: Core Times (min:sec) 19.5-20.5' (8:43) 20.5-20.58' (1:20) Losing water below casing, with recirculation can't pump enough water. | |
| 20.600 | 18.11 | Bottom of Exploration at 20.60 feet below ground surface. Abandon hole, pulled forward to BB-PRR-102B location. | |

Remarks:

This boring cored through the heel of the concrete abutment built in 1930.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-PRR-102A

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | TP-PRR-101 |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 21.33 | Auger ID/OD: | N/A |
| Operator: | G. Lidstone | Datum: | NGVD | Sampler: | N/A |
| Logged By: | K. Maguire | Rig Type: | N/A | Hammer Wt./Fall: | N/A |
| Date Start/Finish: | 7/21/04-7/21/04 | Drilling Method: | Hand Dug Test Pit | Core Barrel: | N/A |
| Boring Location: | 34+62.4, 9.1 Rt. | Casing ID/OD: | N/A | Water Level*: | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|
| 0 | | Embankment grade in front of wall. | |
| | | Brown, dry, fine to coarse SAND, little silt, trace brick, some gravel, trace cobbles, trace slag and coal. | |
| | | Brown as above, mixed with black fine to coarse SAND, little silt, some gravel, trace cobbles, slag and coal. | |
| 4.000 | 17.33 | Bottom of Exploration at 4.00 feet below ground surface. Block at bottom of pit. Bottom of Test Pit at 4.0' bgs. Not bottom of block wall. | |
| | | Elev. 16.57' is ESTIMATED to be the BOF of the older ashlar stone abutment (Falmouth bound). Test pit terminated 9" above the estimated BOF. | |

Remarks:

Test Pit hand dug by G. Lidstone on North Abutment 20.0' from abutment corner, (right).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: TP-PRR-101

## Maine Department of Transportation — Soil/Rock Exploration Log — US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| Project: | Veranda Street Overpass Bridge | Location: | Portland, Maine | Boring No.: | BB-PRR-101 |
| | | | | PIN: | 10158.00 |
| Driller: | MaineDOT | Elevation (ft.) | 38.39 | Auger ID/OD: | 125 mm SSA |
| Operator: | C. Mann | Datum: | NGVD | Sampler: | Standard Split Spoon |
| Logged By: | K. Maguire | Rig Type: | CME 45C | Hammer Wt./Fall: | 63.5 kg/760 mm |
| Date Start/Finish: | 7/19/04-7/20/04 | Drilling Method: | Cased Wash Boring | Core Barrel: | NQ |
| Boring Location: | 34+97.4, 15.8 Lt. | Casing ID/OD: | HW | Water Level*: | '-8.84 m bgs on 7/20/04 |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger
Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods  WOC = weight of casing
Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows |
|---|---|---|---|---|---|
| | | | | | SSA |
| 1D | 24/18 | 2.00 - 4.00 | 10/13/31/26 | 44 | |
| 2D | 24/15 | 5.00 - 7.00 | 3/4/2/5 | 6 | 27 |
| | | | | | 23 |
| | | | | | 30 |
| | | | | | 40 |
| 3D | 24/17 | 9.00 - 11.00 | 57/47/10/10 | 57 | 29 |
| | | | | | 43 |
| | | | | | 95 |
| | | | | | 152 |
| | | | | | 191 |
| 4D | 24/16 | 14.00 - 16.00 | 15/30/35/26 | 65 | 81 |
| | | | | | 177 |
| | | | | | 205 |
| | | | | | 141 |
| | | | | | 122 |
| 5D | 24/15 | 19.00 - 21.00 | 20/65/49/40 | 114 | 157 |
| | | | | | 203 |
| | | | | | 184 |
| | | | | | 200 |
| | | | | | 226 |
| 6D | 24/16 | 24.00 - 26.00 | 28/22/21/30 | 43 | 138 |
| | | | | | 265 |
| | | | | | 438 |
| | | | | | 516 |
| | | | | | 87 aWA |
| 7D | 13/4 | 29.00 - 30.08 | 38/40/60(25) | --- | 355 |
| | | | | | 621 |
| | | | | | 67 aWA |
| | | | | | 55 |
| | | | | | 57 |
| 8D | 24/14 | 34.00 - 36.00 | 19/26/25/26 | 51 | 50 |
| | | | | | 68 |
| | | | | | 70 |
| | | | | | 73 |
| | | | | | 93 |
| 9D-A 9D-B | 23/13 | 39.00 - 40.92 | 16/28/52/50(100) | 80 | 61 |
| 9D-C R1 | 60/60 | 40.90 - 45.90 | RQD = 82% | | 140 NQ |

| Depth (ft.) | Elevation (ft.) | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class |
|---|---|---|---|
| 0.500 | 37.89 | 6" PAVEMENT. | |
| | | Brown, damp, dense, fine to coarse SAND, trace silt, little gravel, (Fill). | |
| | | Obstruction at 4.0' bgs. | |
| | | Dark brown, damp to moist, loose, fine to coarse silty SAND, trace brick, wood and gravel, (Fill). | |
| 9.000 | 29.39 | Olive brown, moist, medium dense, fine silty SAND, trace medium to coarse sand, trace of gravel, with iron staining and sand layers. (Fill). | G#176155 A-4, SM WC=21.3% |
| | | Red brown, wet, dense, fine SAND, some silt, with iron staining. (fill). | |
| 19.000 | 19.39 | Red brown, wet, very dense, fine to coarse SAND, some gravel, little silt with iron staining. (Fill). | G#176156 A-1-b, SW-SM WC=8.4% |
| | | El. 16.57' estimated B.O.F. of ashlar stone abutment. | |
| 22.000 | 16.39 | El. 14.76' estimated B.O.F. of 1930 concrete abutment. | |
| 24.000 | 14.39 | Brown, wet, dense, fine to coarse SAND, trace gravel and silt. | |
| | | aWashed Ahead | |
| | | Similar to above. | |
| | | Brown, wet, very dense, fine to coarse SAND, trace silt, trace gravel. | G#176157 A-3, SP-SM WC=15.5% |
| | | Similar to above. | |
| 39.500 | -1.11 | Red brown, wet, very dense, fine to coarse SAND, trace silt. | |
| | | Grey, wet, very dense, fine to coarse silty SAND. | |
| 40.900 | -2.51 | Bedrock: Grey and white, fine grained METASILTSTONE, (Macworth Formation). R1: Core Times (min:sec) 40.9-41.9' (9:45) 41.9-42.9' (12:00) 42.9-43.9' (10:15) 43.9-44.9' (10:23) 44.9-45.9' (11:07) 100% Recovery | |
| 45.900 | -7.51 | Bottom of Exploration at 45.90 feet below ground surface. | |

Remarks:

Boring BB-PRR-101 sampled backfill of older, pre-1930 ashlar stone abutments.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1 — Boring No.: BB-PRR-101

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1015(800)X
PIN 010158.00
BRIDGE NO. 5052    BRIDGE PLANS

| | | BY | DATE |
|---|---|---|---|
| PROJ. MANAGER | J. WENTWORTH | | |
| DESIGN-DETAILED | R. NAOUS | TEAM SOUTH | |
| CHECKED-REVIEWED | ------ | ------ | ------ |
| DESIGN2-DETAILED2 | K. MAGUIRE | T. WHITE | JAN 2009 |
| DESIGN3-DETAILED3 | ------ | ------ | ------ |
| REVISIONS 1 | ------ | ------ | ------ |
| REVISIONS 2 | ------ | ------ | ------ |
| REVISIONS 3 | ------ | ------ | ------ |
| REVISIONS 4 | ------ | ------ | ------ |
| FIELD CHANGES | ------ | ------ | ------ |

SIGNATURE
P.E. NUMBER
DATE

VERANDA STREET OVERPASS
ST. LAWRENCE & ATLANTIC R.R.
PORTLAND    CUMBERLAND COUNTY

# BORING LOGS

SHEET NUMBER
8
OF 36

# APPENDIX A

Photos

# APPENDIX B

Boring Logs

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Veranda Street Overpass Bridge | **Boring No.:** | BB-PRR-101 | | |
| **Location:** | Portland, Maine | **PIN:** | 10158.00 | | |
| **Driller:** | MaineDOT | **Elevation (ft.)** | 38.39 | **Auger ID/OD:** | 125 mm SSA |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | Standard Split Spoon |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 63.5 kg/760 mm |
| **Date Start/Finish:** | 7/19/04-7/20/04 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ |
| **Boring Location:** | 34+97.4, 15.8 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | ~8.84 m bgs on 7/20/04 |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | 37.89 | | 6" PAVEMENT. 0.500 | |
| | | | | | | | | | | |
| | 1D | 24/18 | 2.00 - 4.00 | 10/13/31/26 | 44 | | | | Brown, damp, dense, fine to coarse SAND, trace silt, little gravel, (Fill). | |
| | | | | | | | | | | |
| | | | | | | | | | Obstruction at 4.0' bgs. | |
| 5 | 2D | 24/15 | 5.00 - 7.00 | 3/4/2/5 | 6 | 27 | | | Dark brown, damp to moist, loose, fine to coarse silty SAND, trace brick, wood and gravel, (Fill). | |
| | | | | | | 23 | | | | |
| | | | | | | 30 | | | | |
| | | | | | | 40 | 29.39 | | 9.000 | |
| 10 | 3D | 24/17 | 9.00 - 11.00 | 57/47/10/10 | 57 | 29 | | | Olive brown, moist, medium dense, fine silty SAND, trace medium to coarse sand, trace of gravel, with iron staining and sand layers. (Fill). | G#176155 A-4, SM WC=21.3% |
| | | | | | | 43 | | | | |
| | | | | | | 95 | | | | |
| | | | | | | 152 | | | | |
| | | | | | | 191 | | | | |
| 15 | 4D | 24/16 | 14.00 - 16.00 | 15/30/35/26 | 65 | 81 | | | Red brown, wet, dense, fine SAND, some silt, with iron staining. (fill). | |
| | | | | | | 177 | | | | |
| | | | | | | 205 | | | | |
| | | | | | | 141 | | | | |
| | | | | | | 122 | 19.39 | | 19.000 | |
| 20 | 5D | 24/15 | 19.00 - 21.00 | 20/65/49/40 | 114 | 157 | | | Red brown, wet, very dense, fine to coarse SAND, some gravel, little silt with iron staining. (Fill). | G#176156 A-1-b, SW-SM WC=8.4% |
| | | | | | | 203 | | | | |
| | | | | | | 184 | 16.39 | | El. 16.57' estimated B.O.F. of ashlar stone abutment. 22.000 | |
| | | | | | | 200 | | | | |
| | | | | | | 226 | 14.39 | | El. 14.76' estimated B.O.F. of 1930 concrete abutment. 24.000 | |
| 25 | 6D | 24/16 | 24.00 - 26.00 | 28/22/21/30 | 43 | 138 | | | Brown, wet, dense, fine to coarse SAND, trace gravel and silt. | |

**Remarks:**

Boring BB-PRR-101 sampled backfill of older, pre-1930 ashlar stone abutments.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.



**Boring No.:** BB-PRR-101

| **Maine Department of Transportation**<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | **Project:** Veranda Street Overpass Bridge<br>**Location:** Portland, Maine | **Boring No.:** BB-PRR-101<br>**PIN:** 10158.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 38.39 | **Auger ID/OD:** | 125 mm SSA |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | Standard Split Spoon |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 63.5 kg/760 mm |
| **Date Start/Finish:** | 7/19/04-7/20/04 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ |
| **Boring Location:** | 34+97.4, 15.8 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | ~8.84 m bgs on 7/20/04 |

| Definitions: | Definitions: | Definitions: |
|---|---|---|
| D = Split Spoon Sample | $S_u$ = Insitu Field Vane Shear Strength (psf) | WC = water content, percent |
| MD = Unsuccessful Split Spoon Sample attempt | $T_v$ = Pocket Torvane Shear Strength (psf) | LL = Liquid Limit |
| U = Thin Wall Tube Sample | $q_p$ = Unconfined Compressive Strength (ksf) | PL = Plastic Limit |
| R = Rock Core Sample | $S_{u(lab)}$ = Lab Vane Shear Strength (psf) | PI = Plasticity Index |
| V = Insitu Vane Shear Test | WOH = weight of 140lb. hammer | G = Grain Size Analysis |
| SSA = Solid Stem Auger | WOR = weight of rods WOC = weight of casing | C = Consolidation Test |

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | 265 | | | | |
| | | | | | | 438 | | | | |
| | | | | | | 516 | | | | |
| | | | | | | 87 | | | aWashed Ahead | |
| | | | | | | aWA | | | | |
| | 7D | 13/4 | 29.00 - 30.08 | 38/40/60(25) | --- | 355 | | | Similar to above. | |
| 30 | | | | | | 621 | | | | |
| | | | | | | 67 | | | | |
| | | | | | | aWA | | | | |
| | | | | | | 55 | | | | |
| | | | | | | 57 | | | | |
| | 8D | 24/14 | 34.00 - 36.00 | 19/26/25/26 | 51 | 50 | | | Brown, wet, very dense, fine to coarse SAND, trace silt, trace gravel. | G#176157<br>A-3, SP-SM<br>WC=15.5% |
| 35 | | | | | | 68 | | | | |
| | | | | | | 70 | | | | |
| | | | | | | 73 | | | | |
| | | | | | | 93 | | | Similar to above. | |
| | 9D-A | 23/13 | 39.00 - 40.92 | 16/28/52/50(100) | 80 | 61 | -1.11 | | 39.500 | |
| 40 | 9D-B | | | | | | | | Red brown, wet, very dense, fine to coarse SAND, trace silt. | |
| | 9D-C<br>R1 | 60/60 | 40.90 - 45.90 | RQD = 82% | | 140<br>NQ | -2.51 | | Grey, wet, very dense, fine to coarse silty SAND.<br>40.900 | |
| | | | | | | | | | Bedrock: Grey and white, fine grained METASILTSTONE, (Macworth Formation).<br>R1: Core Times (min:sec)<br>40.9-41.9 (9:45)<br>41.9-42.9 (12:00)<br>42.9-43.9 (10:15)<br>43.9-44.9 (10:23)<br>44.9-45.9 (11:07) 100% Recovery | |
| 45 | | | | | | | -7.51 | | 45.900 | |
| | | | | | | | | | **Bottom of Exploration at 45.90 feet below ground surface.** | |
| 50 | | | | | | | | | | |

**Remarks:**

Boring BB-PRR-101 sampled backfill of older, pre-1930 ashlar stone abutments.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Veranda Street Overpass Bridge | **Boring No.:** | BB-PRR-102A | | |
| **Location:** | Portland, Maine | **PIN:** | 10158.00 | | |
| **Driller:** | MaineDOT | **Elevation (ft.)** | 38.71 | **Auger ID/OD:** | 125 mm SSA |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | Standard Split Spoon |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 63.5 kg/760 mm |
| **Date Start/Finish:** | 7/20/04-7/20/04 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ |
| **Boring Location:** | 34+20.5, 14.1 Lt. | **Casing ID/OD:** | NW | **Water Level*:** | None Observed |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1D | 24/15 | 0.50 - 2.50 | 18/21/14/11 | 35 | SSA | 38.21 | | 6" PAVEMENT. ——0.500 Brown, damp, loose to dense, fine to coarse SAND, trace silt, pavement in nose of spoon, (Fill). | |
| 5 | 2D | 24/14 | 5.00 - 7.00 | 2/2/6/6 | 8 | 10 | | | Similar to above, loose. | |
| | | | | | | 14 | | | | |
| | | | | | | 17 | 30.71 | | ——8.000 | |
| | | | | | | 36 | | | | |
| | | | | | | 24 | | | | |
| 10 | 3D | 24/4 | 10.00 - 12.00 | 10/8/6/6 | 14 | 18 | | | Brown, wet, medium dense, fine to coarse silty SAND, trace gravel. (Fill). | |
| | | | | | | 18 | | | | |
| | | | | | | 13 | | | | |
| | | | | | | 11 | | | | |
| 15 | 4D | 24/6 | 14.00 - 16.00 | 5/2/2/1 | 4 | 6 | | | Similar to above, but very loose. | |
| | | | | | | 7 | | | | |
| | | | | | | 10 | | | | |
| | | | | | | 11 | | | | |
| | | | | | | b24 | | | b24 blows for 6". | |
| 20 | R1 | 13/13 | 19.50 - 20.58 | RQD = N/A% | | NQ | 19.21 | | ——19.500 R1: Concrete. R1: Core Times (min:sec) 19.5-20.5 (8:43) 20.5-20.6 (1:20) Losing water below casing, with recirculation can't pump enough water. | |
| | | | | | | | 18.11 | | ——20.600 **Bottom of Exploration at 20.60 feet below ground surface.** Abandon hole, pulled forward to BB-PRR 102B location.- | |
| 25 | | | | | | | | | | |

**Remarks:**

This boring cored through the heel of the concrete abutment built in 1930.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

| | | | | | |
|---|---|---|---|---|---|
| **Project:** | Veranda Street Overpass Bridge | | | **Boring No.:** | BB-PRR-102B |
| **Location:** | Portland, Maine | | | **PIN:** | 10158.00 |
| **Driller:** | MaineDOT | **Elevation (ft.)** | 38.71 | **Auger ID/OD:** | 125 mm |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | N/A |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | N/A |
| **Date Start/Finish:** | 7/21/04-7/21/04 | **Drilling Method:** | Solid Stem Auger | **Core Barrel:** | N/A |
| **Boring Location:** | 34+15, 15.6 Lt. | **Casing ID/OD:** | N/A | **Water Level*:** | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | | | No sampling conducted in boring. | |
| 5 | | | | | | | 33.71 | | 5.000<br>**Bottom of Exploration at 5.00 feet below ground surface.**<br>Casing very crooked, abandon hole. | |
| 10 | | | | | | | | | | |
| 15 | | | | | | | | | | |
| 20 | | | | | | | | | | |
| 25 | | | | | | | | | | |

**Remarks:**

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.



**Boring No.:** BB-PRR-102B

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Veranda Street Overpass Bridge
**Location:** Portland, Maine

**Boring No.:** BB-PRR-102C
**PIN:** 10158.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 38.71 | **Auger ID/OD:** | 125 mm |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | N/A |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | N/A |
| **Date Start/Finish:** | 7/21/04-7/21/04 | **Drilling Method:** | Solid Stem Auger | **Core Barrel:** | N/A |
| **Boring Location:** | 34+13.9, 15.1 Lt. | **Casing ID/OD:** | N/A | **Water Level*:** | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | | | No sampling conducted in boring. | |
| 5 | | | | | | | | | | |
| 7.00 | | | | | | | 31.71 | | 7.000 **Bottom of Exploration at 7.00 feet below ground surface.** Obstruction. Could not auger through, abandon hole. | |
| 10 | | | | | | | | | | |
| 15 | | | | | | | | | | |
| 20 | | | | | | | | | | |
| 25 | | | | | | | | | | |

**Remarks:**

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Veranda Street Overpass Bridge
**Location:** Portland, Maine

**Boring No.:** BB-PRR-102D
**PIN:** 10158.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 39.37 | **Auger ID/OD:** | 125 mm SSA |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | Standard Split Spoon |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 63.5 kg/760 mm |
| **Date Start/Finish:** | 7/20/04-7/20/04 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ |
| **Boring Location:** | 34+17.6, 17.7 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | None Observed |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | SSA | | | Refer to BB-PRR-102A for sampling information on upper 19.0'. | |
| 10 | | | | | | 57 | | | | |
| | | | | | | 32 | | | | |
| | | | | | | 28 | | | | |
| | | | | | | 34 | | | | |
| | | | | | | 24 | | | | |
| 15 | | | | | | 21 | | | | |
| | | | | | | 17 | | | | |
| | | | | | | 26 | | | | |
| | | | | | | 90 | 20.37 | | 19.000 | |
| | 1D | 24/10 | 19.00 - 21.00 | 51/26/21/16 | 47 | 44 | | | Brown, wet, dense, fine to coarse SAND, some silt, little gravel. (Fill). | |
| 20 | | | | | | 45 | | | | |
| | | | | | | 29 | 17.37 | | 22.000 | |
| | | | | | | 36 | | | El 17.39' (depth bgs 21.98') ESTIMATED B.O.F. of ashlar stone abutment | |
| | | | | | | 53 | | | | |
| 25 | 2D | 24/9 | 24.00 - 26.00 | 18/24/15/15 | 39 | 27 | | | Light brown, moist, dense, fine to coarse SAND, trace silt and gravel. (Fill). | |

**Remarks:**

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

| Maine Department of Transportation<br>Soil/Rock Exploration Log<br>US CUSTOMARY UNITS | Project: Veranda Street Overpass Bridge<br>Location: Portland, Maine | Boring No.: BB-PRR-102D<br>PIN: 10158.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 39.37 | **Auger ID/OD:** | 125 mm SSA |
| **Operator:** | C. Mann | **Datum:** | NGVD | **Sampler:** | Standard Split Spoon |
| **Logged By:** | K. Maguire | **Rig Type:** | CME 45C | **Hammer Wt./Fall:** | 63.5 kg/760 mm |
| **Date Start/Finish:** | 7/20/04-7/20/04 | **Drilling Method:** | Cased Wash Boring | **Core Barrel:** | NQ |
| **Boring Location:** | 34+17.6, 17.7 Lt. | **Casing ID/OD:** | HW | **Water Level*:** | None Observed |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | 45 | 14.37 | | El. 14.76' (depth of 24.6') ESTIMATED B.O.F. of concrete abutment 25.000 | |
| | | | | | | 73 | | | | |
| | | | | | | 102 | | | | |
| | | | | | | 103 | | | | |
| | 3D | 24/12 | 29.00 - 31.00 | 25/28/31/30 | 59 | 25 | | | Brown, moist, very dense, fine to coarse SAND, little gravel, trace silt. | G#176158<br>A-1-b, SW-SM<br>WC=10.7% |
| 30 | | | | | | 42 | | | | |
| | | | | | | 92 | | | | |
| | | | | | | 110 | | | | |
| | | | | | | 95 | | | | |
| | 4D | 24/15 | 34.00 - 36.00 | 19/20/27/37 | 47 | 59 | 5.37 | | 34.000<br>Red/brown, moist, dense, fine to coarse SAND, trace silt, trace gravel. | G#176159<br>A-3, SP-SM<br>WC=18.2% |
| 35 | | | | | | 72 | | | | |
| | | | | | | 110 | | | | |
| | | | | | | 150 | | | | |
| | | | | | | 170 | | | | |
| | 5D | 14.3/8 | 39.00 - 40.19 | 21/15/50(50) | --- | 78 | | | Grey, wet, fine to coarse SAND, little silt, trace gravel. | G#176160<br>A-2-4, SM<br>WC=16.2% |
| 40 | R1 | 60/58 | 40.20 - 45.20 | RQD = 76% | | a43<br>NQ | -0.83 | | a43 blows for 2".<br>40.200<br>Bedrock: Grey white, fine grained METASILTSTONE, (Macworth Formation).<br>R1: Core Times (min:sec)<br>40.2-41.2 (7:42)<br>41.2-42.2 (7:17)<br>42.2-43.2 (7:00)<br>43.2-44.2 (7:43)<br>44.2-45.2 (8:31) 96% Recovery | |
| 45 | | | | | | | -5.83 | | 45.200<br>**Bottom of Exploration at 45.20 feet below ground surface.** | |
| 50 | | | | | | | | | | |

**Remarks:**

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

# Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

**Project:** Veranda Street Overpass Bridge
**Location:** Portland, Maine

**Boring No.:** TP-PRR-101
**PIN:** 10158.00

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | MaineDOT | **Elevation (ft.)** | 21.33 | **Auger ID/OD:** | N/A |
| **Operator:** | G. Lidstone | **Datum:** | NGVD | **Sampler:** | N/A |
| **Logged By:** | K. Maguire | **Rig Type:** | N/A | **Hammer Wt./Fall:** | N/A |
| **Date Start/Finish:** | 7/21/04-7/21/04 | **Drilling Method:** | Hand Dug Test Pit | **Core Barrel:** | N/A |
| **Boring Location:** | 34+62.4, 9.1 Rt. | **Casing ID/OD:** | N/A | **Water Level*:** | N/A |

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
R = Rock Core Sample
V = Insitu Vane Shear Test
SSA = Solid Stem Auger

Definitions:
$S_u$ = Insitu Field Vane Shear Strength (psf)
$T_v$ = Pocket Torvane Shear Strength (psf)
$q_p$ = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WOH = weight of 140lb. hammer
WOR = weight of rods WOC = weight of casing

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

| Depth (ft.) | Sample No. | Pen./Rec. (in.) | Sample Depth (ft.) | Blows (/6 in.) Shear Strength (psf) or RQD (%) | N-value | Casing Blows | Elevation (ft.) | Graphic Log | Visual Description and Remarks | Laboratory Testing Results/ AASHTO and Unified Class. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | | | Embankment grade in front of wall.<br><br>Brown, dry, fine to coarse SAND, little silt, trace brick, some gravel, trace cobbles, trace slag and coal.<br>Brown as above, mixed with black fine to coarse SAND, little silt, some gravel, trace cobbles, slag and coal. | |
| | | | | | | | 17.33 | | 4.00 | |
| | | | | | | | | | **Bottom of Exploration at 4.00 feet below ground surface.**<br>Block at bottom of pit. Bottom of Test Pit at 4.0' bgs. Not bottom of block wall.<br><br>Elev. 16.57 is ESTIMATED to be the BOF of the older ashlar stone abutment (Falmouth bound). Test pit terminated 9" above the estimated BOF. | |
| 5 | | | | | | | | | | |
| 10 | | | | | | | | | | |
| 15 | | | | | | | | | | |
| 20 | | | | | | | | | | |
| 25 | | | | | | | | | | |

**Remarks:**

Test Pit hand dug by G. Lidstone on North Abutment 20.0' from abutment corner, (right).

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.



**Boring No.:** TP-PRR-101

# APPENDIX C

Laboratory Data

State of Maine - Department of Transportation

## Laboratory Testing Summary Sheet

# Town(s): Portland    Project Number: 10158.00

| Boring & Sample Identification Number | Station (Feet) | Offset (Feet) | Depth (Feet) | Reference Number | G.S.D.C. Sheet | W.C. | L.L. | P.I. | Classification | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Unified | AASHTO | Frost |
| BB-PRR-102D, 3D | 34+17.6 | 17.7 Lt. | 29.0-31.0 | 176158 | 1 | 10.7 | | | SW-SM | A-1-b | 0 |
| BB-PRR-102D, 4D | 34+17.6 | 17.7 Lt. | 34.0-36.0 | 176159 | 1 | 18.2 | | | SP-SM | A-3 | 0 |
| BB-PRR-102D, 5D | 34+17.6 | 17.7 Lt. | 39.0-40.19 | 176160 | 1 | 16.2 | | | SM | A-2-4 | II |
| BB-PRR-101, 3D | 34+97.4 | 15.8 Lt. | 9.0-11.0 | 176155 | 2 | 21.3 | | | SM | A-4 | III |
| BB-PRR-101, 5D | 34+97.4 | 15.8 Lt. | 19.0-21.0 | 176156 | 2 | 8.4 | | | SW-SM | A-1-b | 0 |
| BB-PRR-101, 8D | 34+97.4 | 15.8 Lt. | 34.0-36.0 | 176157 | 2 | 15.5 | | | SP-SM | A-3 | 0 |

**Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MDOT and Corps of Engineers Classification Systems.**

GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

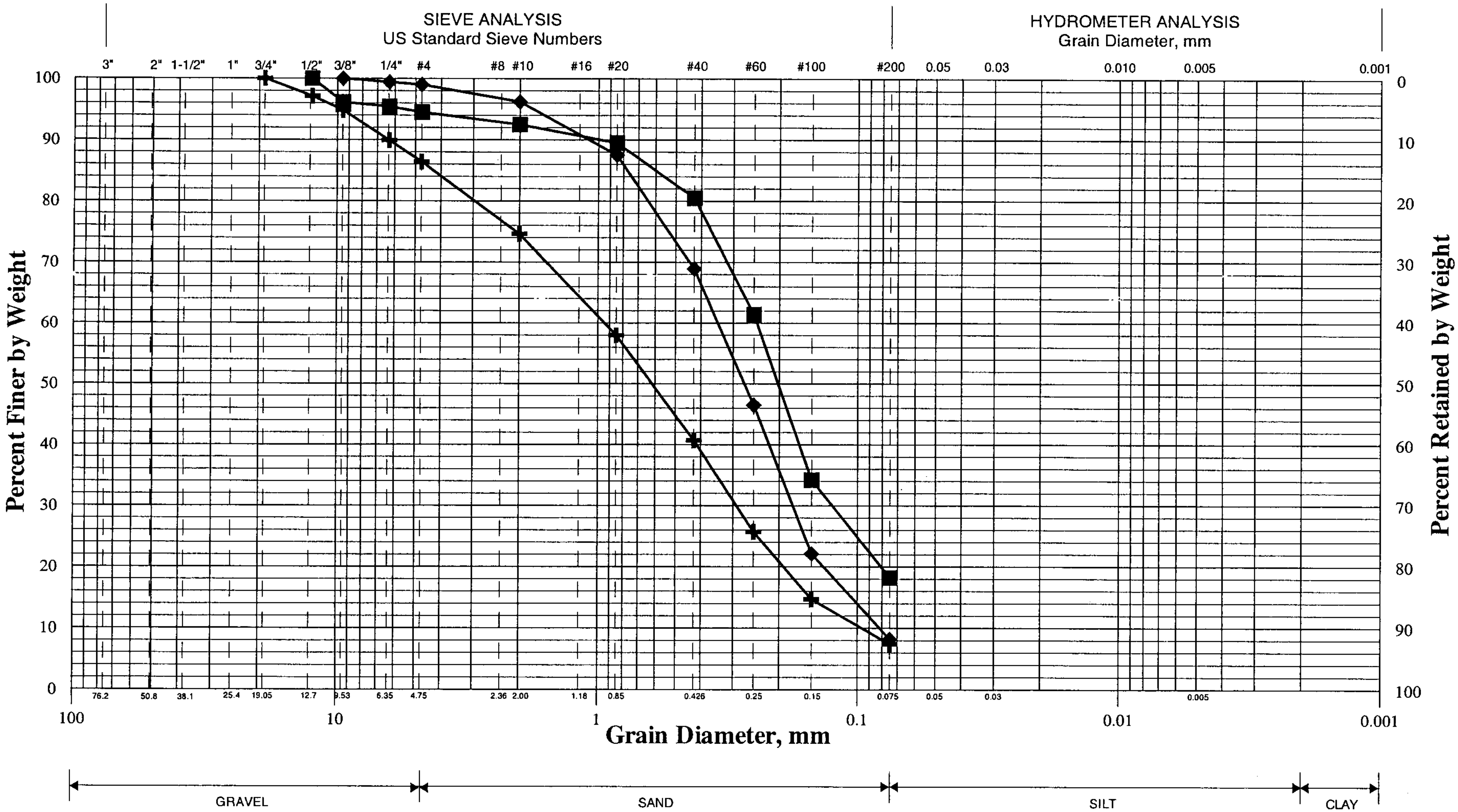
WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

SHEET NO.1

## *State of Maine Department of Transportation*
## GRAIN SIZE DISTRIBUTION CURVE



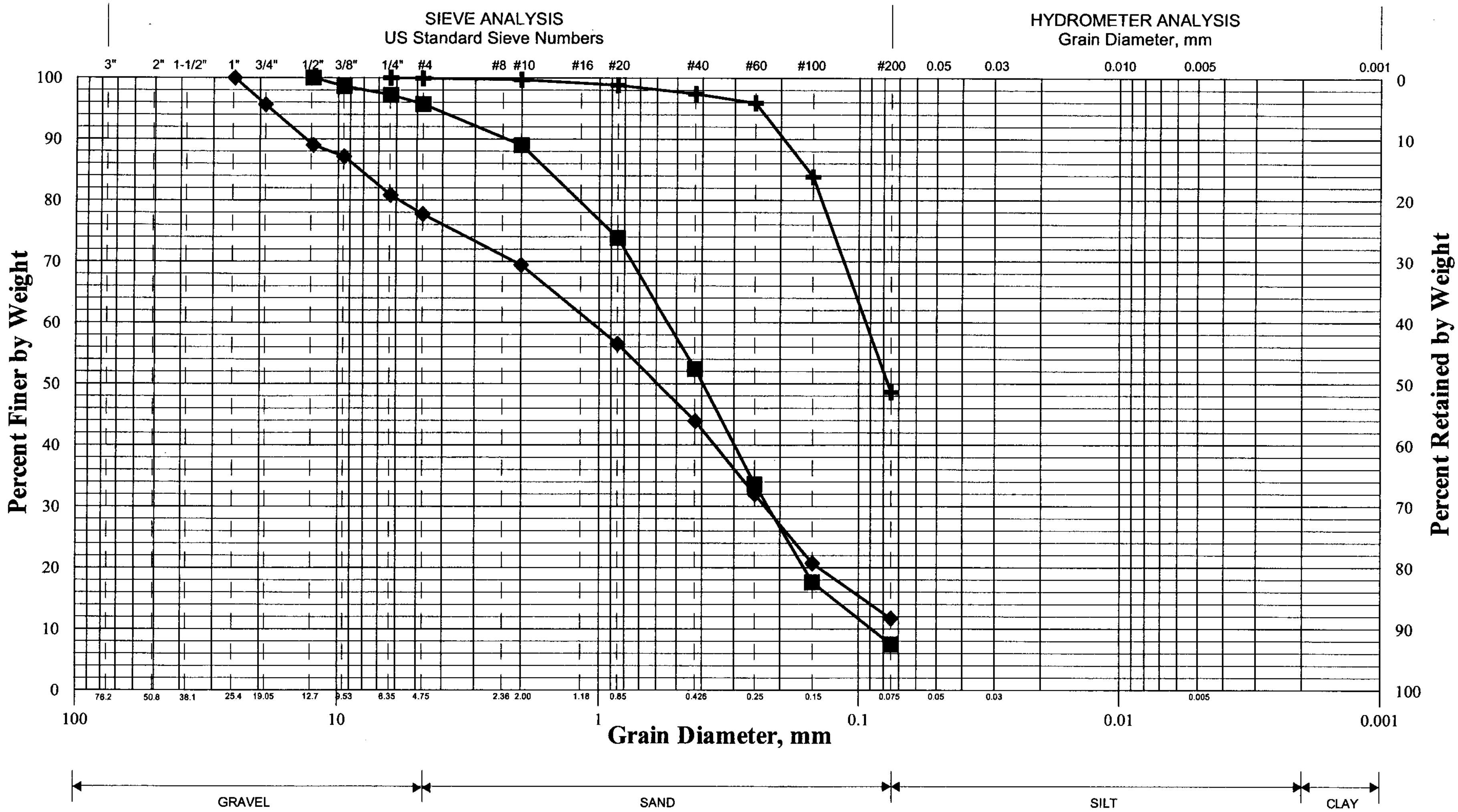

UNIFIED CLASSIFICATION

| | Boring No. | Sample No. | Depth (m) | Description | w% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|
| + | BB-PRR-102D | 3D | 8.84-9.45 | SAND, little gravel, trace silt. | 10.7 | | | |
| ◆ | BB-PRR-102D | 4D | 10.36-10.97 | SAND, trace silt, trace gravel. | 18.2 | | | |
| ■ | BB-PRR-102D | 5D | 11.89-12.25 | SAND, little silt, trace gravel. | 16.2 | | | |
| ● | | | | | | | | |
| ▲ | | | | | | | | |
| × | | | | | | | | |

| |
|---|
| PIN: 10158.00 |
| Town: Portland |
| Reported by: T. White |
| Date: 8/31/04 |

SHEET NO.2

# *State of Maine Department of Transportation*
## GRAIN SIZE DISTRIBUTION CURVE




UNIFIED CLASSIFICATION

| | Boring No. | Sample No. | Depth (m) | Description | w% | LL | PL | PI |
|---|---|---|---|---|---|---|---|---|
| + | BB-PRR-101 | 3D | 2.74-3.35 | Silty fine SAND, trace medium to coarse sand, trace gravel. | 21.3 | | | |
| ◆ | BB-PRR-101 | 5D | 5.79-6.4 | SAND, some gravel, little silt. | 8.4 | | | |
| ■ | BB-PRR-101 | 8D | 10.36-10.97 | SAND, trace silt, trace gravel. | 15.5 | | | |
| ● | | | | | | | | |
| ▲ | | | | | | | | |
| ✕ | | | | | | | | |

| |
|---|
| PIN: 10158.00 |
| Town: Portland |
| Reported by: T. White |
| Date: 8/31/04 |

# APPENDIX D

Nondestructive Testing Report, NDT Corporation

LAURA KRUSINSKI
FILE COPY

# NONDESTRUCTIVE TESTING

# VERANDA STREET BRIDGE

## Portland, Maine

Prepared For

## Maine Department of Transportation

September, 2004

NDT CORPORATION

NDT CORPORATION

September 24, 2004

Mrs. Laura Krusinski, PE
Maine Department of Transportation Bridge Program
16 State House Station
Augusta, Maine 04333-0016

Dear Mrs. Krusinski:

NDT Corporation conducted Ground Penetrating Radar (GPR) and Sonic/Ultrasonic (Sonic) survey of the Veranda St. Bridge abutments in Portland, Maine to determine the approximate dimensions of the concrete and masonry abutments. Field work was conducted on August 31, 2004

If you have any questions or need additional information, contact Paul Fisk at 508-754-0147.

Sincerely,
NDT Corporation

Paul Fisk

Paul S. Fisk

## Table of Contents

## 1.0 Summary

A Ground Penetrating radar (GPR) and Sonic/Ultrasonic nondestructive testing investigation was conducted on August 31, 2004. The objective of the investigation was to determine the approximate thickness of the concrete and masonry abutments of the Veranda St. Bridge over the St. Lawrence and Atlantic Railroad in Portland, Maine. GPR data was used to determine the thickness of the abutment and wing walls while Sonic/Ultrasonic frequency/reflection measurements were used to calibrate the GPR and also determine thickness of the abutments and wing walls.

## 2.0 Methods of Investigation

### 2.1 Location and Survey Control

The general location of the Veranda Street Bridge is shown on Figure 1. Ground penetrating radar (GPR) lines (Figures 2 and 4) were collected vertically from the top of the abutment down to the ground surface. The western most corners of the abutment were used as station 0+00. GPR lines were taken every 10 feet. Sonic/ultrasonic measurements were taken from top to bottom at approximately 2 foot increments along selected GPR lines.

### 2.2 Ground Penetrating Radar (GPR)

GPR data are acquired using a digital system coupled with a 400 MHz antenna. The GPR method uses a pulsed electromagnetic signal that is transmitted to and reflected by a target back to the point of transmission. The wave transmission and reflection is dependent on the electrical (dielectric constant and conductivity) properties of the material(s) being investigated. Since these electrical properties are highly dependent on moisture content, saturated or moist conditions provide both strong reflections and high attenuation. Metal reinforcing, pipes and beams also produce strong reflections of radar signals. GPR results are qualitative and depth interpretation is based on calibration(s). Appendix 1 is a discussion of the GPR survey method.

### 2.3 Sonic/Ultrasonic Measurements

Sonic/ultrasonic testing is the most definitive NDT testing technique for the assessment of concrete. Sonic/ultrasonic NDT measurements determine the characteristics of concrete by creating a stress wave generated by a relatively low energy projectile impact energy source. Stress wave measurements in the sonic/ultrasonic frequency band are used to make direct measurements of the compressional and shear wave transmission velocity and to measure reflected phases of the compressional wave. The transmission velocity values determine the elastic deformational characteristics of the concrete, including Young's, bulk, and shear moduli, as well as Poisson's ratio, and calculated strength values. Appendix 2 is a discussion of the sonic/ultrasonic survey method.

Sonic/ultrasonic reflection data measures the time required for a compressional wave transmitted at the front of a structure to propagate to the end of the structure and be reflected back to the front. The time of the reflection is two times the length of the structure divided by its velocity. The compressional wave velocity values are predetermined by measurements across the exposed top of the structure or by a back calculation from cylinder tests or design strength values.

The sonic/ultrasonic data are acquired with a system designed and built by NDT Corporation specifically for testing concrete structures. This system uses a projectile impact energy source and a mechanical sensor array. The signal is input to a set of amplifiers for signal conditioning, analog to digital conversion and finally to a portable PC for display and archiving of the data. The sensors are typically spaced 2, 6, 18, 30 inches (spacing is dependent on concrete thickness) from the energy impact point.

## 3.0 Discussion of Results

### SOUTH ABUTMENT AND WING WALLS

GPR data was collected along 6 vertical (top to ground surface) lines and 2 horizontal lines; one at approximately 2 feet above the ground surface and one at approximately 7 feet above the ground surface. GPR data was also collected on the west facing wing wall; 3 vertical and 2 horizontal. The locations of these lines are shown on Figure 2. Sonic/ultrasonic data was collected along selected GPR lines; four on the abutment and 1 on the west facing wing wall. Due to the complexity and variability of the structures GPR and Sonic results are presented in cross-sections along GPR lines and are shown in Figures 2 and 3.

| | |
|---|---|
| Western Concrete Abutment: | Approximately 64 inches at the top and approximately 80 inches near the ground surface. |
| Masonry Abutment* | Approximately 64 inches at the top and 70 to 80 inches at the ground surface. |
| Eastern Concrete Abutment | Approximately 40 inches at the top and approximately 60 inches near the ground surface. |

*Vertical GPR File 251, horizontal GPR Files 255 and 254 and sonic/ultrasonic data collected at this location indicated reflector depths of approximately 40 to 50 inches, it is believed the bonding of the masonry blocks, in the middle to eastern 20 feet of the masonry abutment, is fractured and filled with moisture for the 14 foot height tested. This high moisture/debonded area has blocked both the GPR and sonic signals and data from propagating past this point, thus it is reported the abutment is only 40 to 50 inches thick through this section.

## NORTH ABUTMENT AND WING WALLS

GPR data was collected along 6 vertical (top to ground surface) lines and 2 horizontal lines; one at approximately 2 feet above the ground surface and one at approximately 7 feet above the ground surface. GPR data was also collected on the west and east facing wing walls; 1 vertical and 1 horizontal. The locations of these lines are shown on Figure 4. Sonic/ultrasonic data was collected along selected GPR lines; four on the abutment and 1 on each of the wing walls. GPR and Sonic results are presented in cross-sections for each line and are shown in Figures 4.

| | |
|---|---|
| Western Concrete Abutment: | Approximately 30 inches at the top and approximately 60 inches near the ground surface. |
| Masonry Abutment/<br>Eastern Concrete Abutment | Approximately 30 inches at top, 66 inches at approximately 8 feet above the ground, and approximately 80 inches near the ground surface |

## INTERMEDIATE REFLECTORS

Intermediate GPR reflectors were detected in the data from both abutments and wing walls. In the wing-walls and concrete sections of the abutments, systematic reflectors, at approximately 18 to 24 inches deep, are consistent in number and spacing to vertical "rails" noted on the plans. In the masonry abutments intermediate reflectors were noted at irregular locations and depths, these reflectors are believed to be fractured (un-bonded) blocks where moisture has infiltrated and caused a reflector. It should be noted that the frequency of data representative of "un-bonded" blocks is high.

## SONIC/ULTRASONIC AND GPR CORELATION

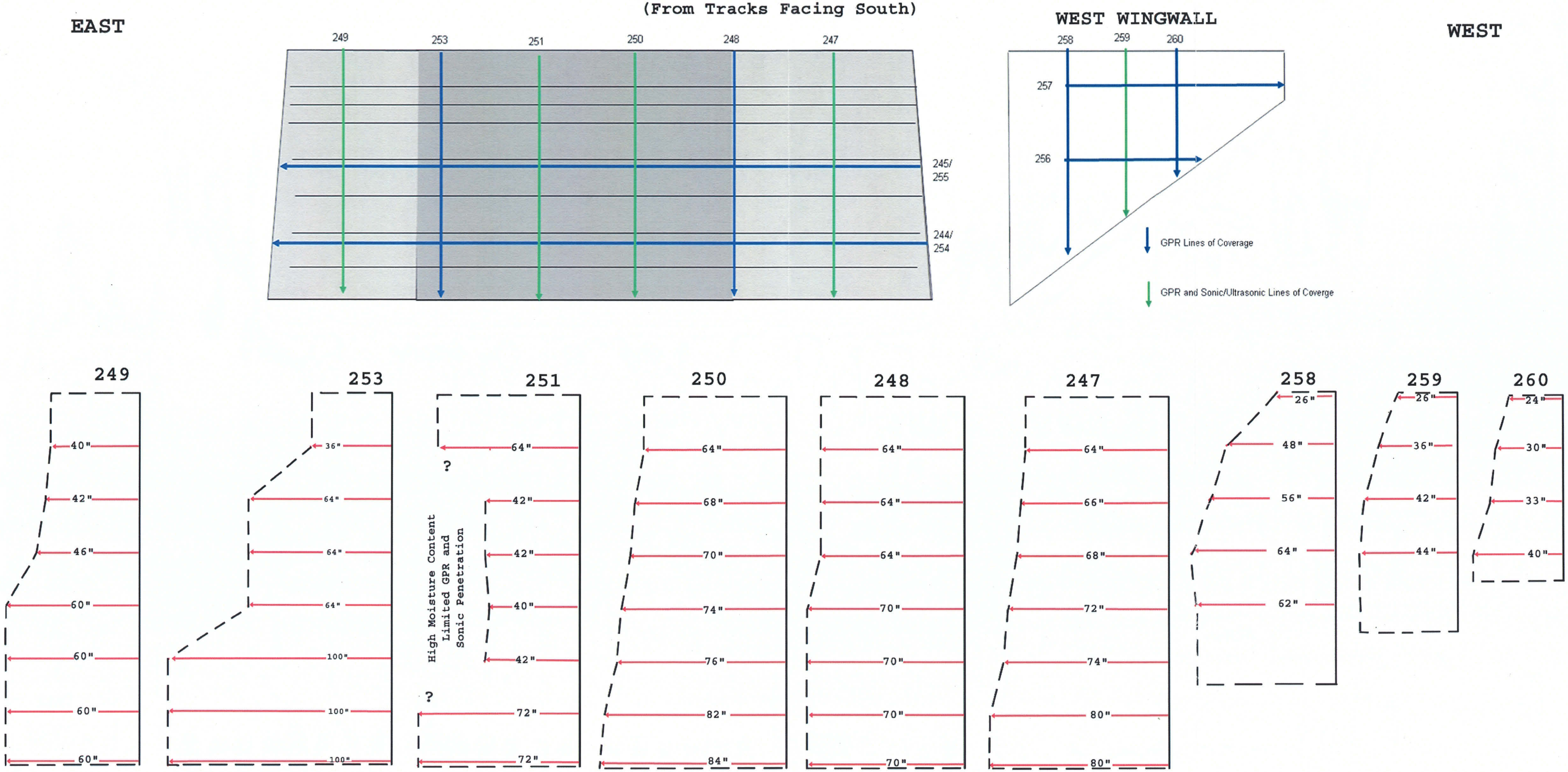
Sonic/ultrasonic frequency and reflection measurements were correlated with the GPR results and were used to determine the GPR signal velocity. Sonic/ultrasonic measurements directly measure the compressional velocity of the concrete (average of 14,000 ft/sec) and granite blocks (average of 13,000 ft/sec across joints, and average of 15,000 ft/sec for solid granite). Using the measured compressional velocity reflection times and measure frequency values can used to calculate thicknesses. The time to back of wall GPR reflectors and intermediate reflectors (back of blocks, "rails" in wing walls, etc) were compared to the sonic/ultrasonic thicknesses and it was determined that the typical GPR signal velocity of 2 inches/nanosecond was within 6 inches of most reflectors. Figure 5 is an annotated GPR record (File 250), which is typical for the masonry abutments, showing GPR back of wall reflector and intermediate reflectors. In this case it appears that cracking/deterioration of the mortar between masonry blocks has blocked the sonic/ultrasonic signal from propagating to the back of the wall and has also allowed moisture to collect at the block interfaces giving an intermediate GPR reflector and a shallow sonic/ultrasonic frequency thickness. Figure 6 is a GPR record (File 257), which is typical for the concrete wing walls and abutment sections, depicting the back of wall reflector and the intermediate reflectors these reflectors are believed to be "rails".

# FIGURES


| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of Transportation<br>by<br>NDT Corporation | Area of Investigation | |
|---|---|---|
| | Septemebr, 2004 | Figure 1 |

# SOUTH ABUTMENT

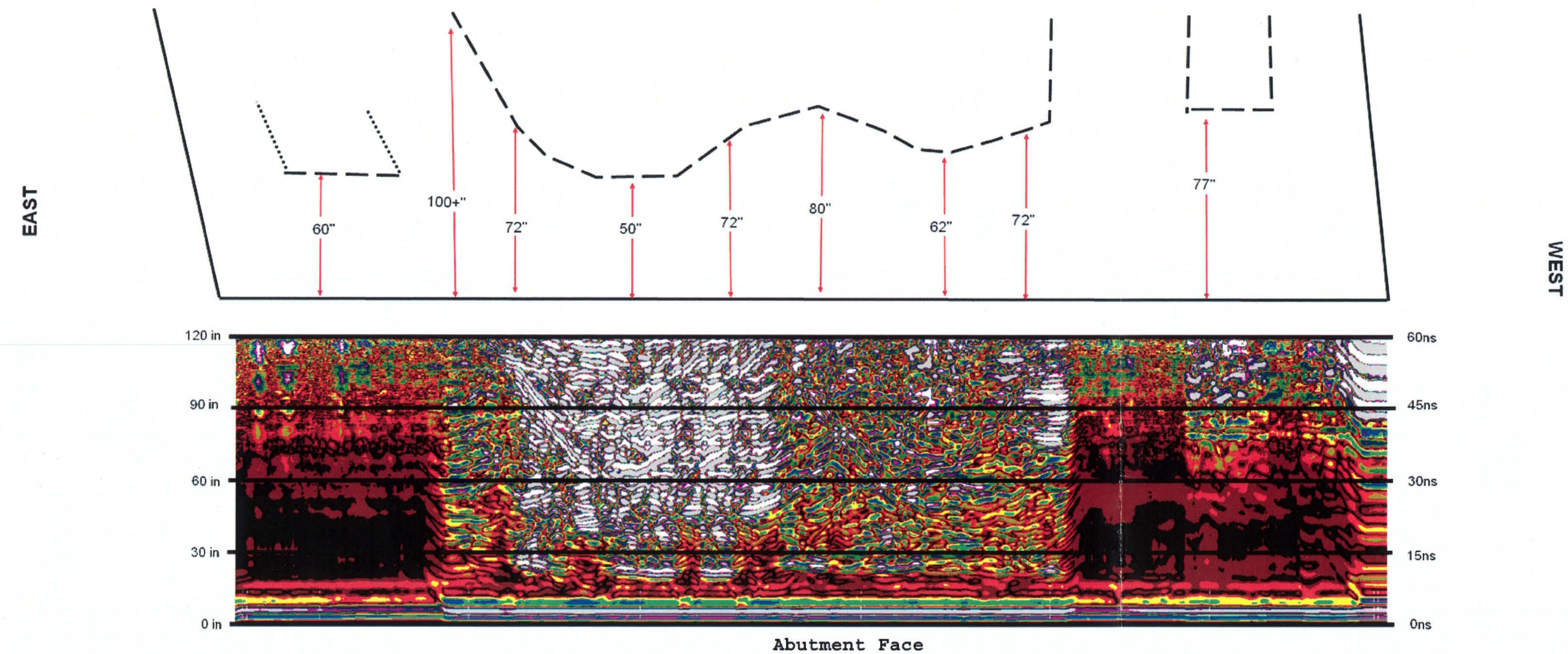
(From Tracks Facing South)




| | | |
|---|---|---|
| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of Transportation<br>by<br>NDT Corporation | Lines of Coverage and Cross Sections | |
| | September, 2004 | Figure 2 |

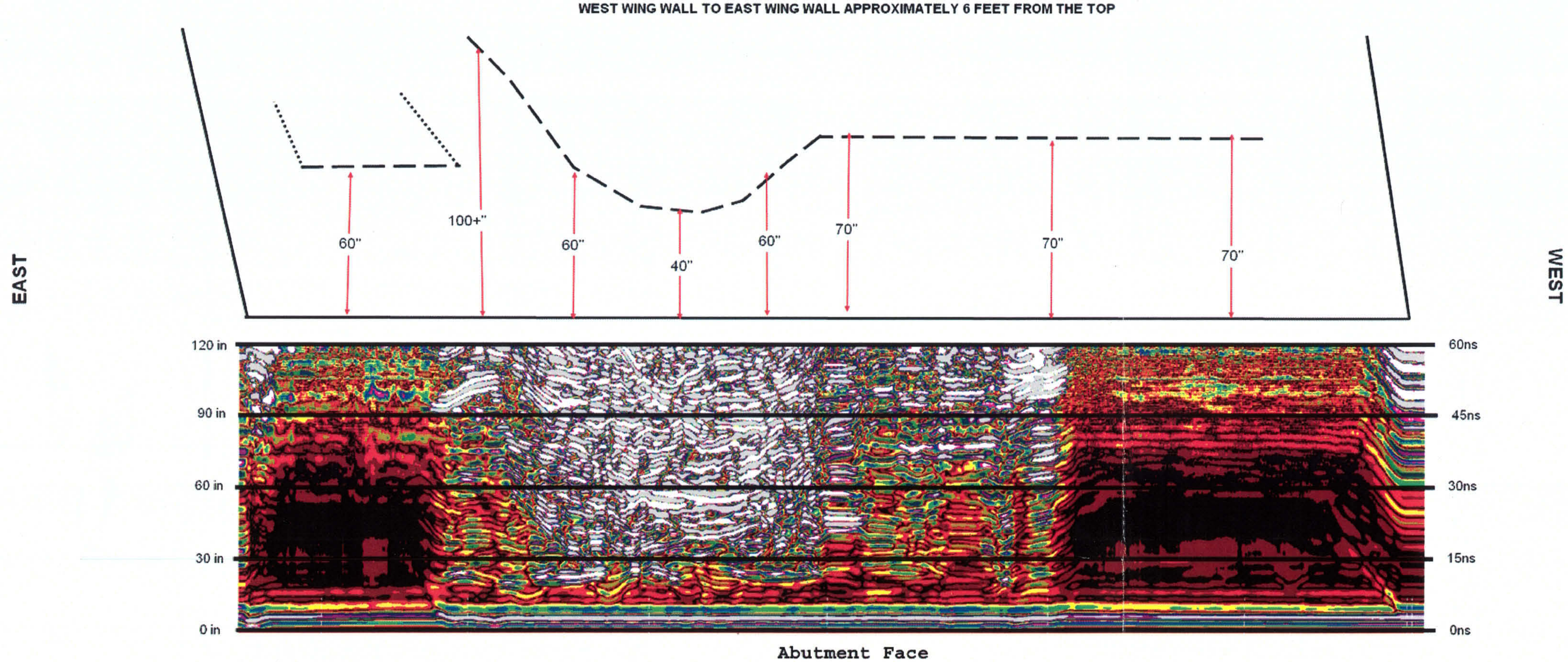
## SOUTH ABUTMENT CROSS SECTION ALONG FILE 254

WEST WING WALL TO EAST WING WALL APPROXIMATELY 10 FEET FROM THE TOP

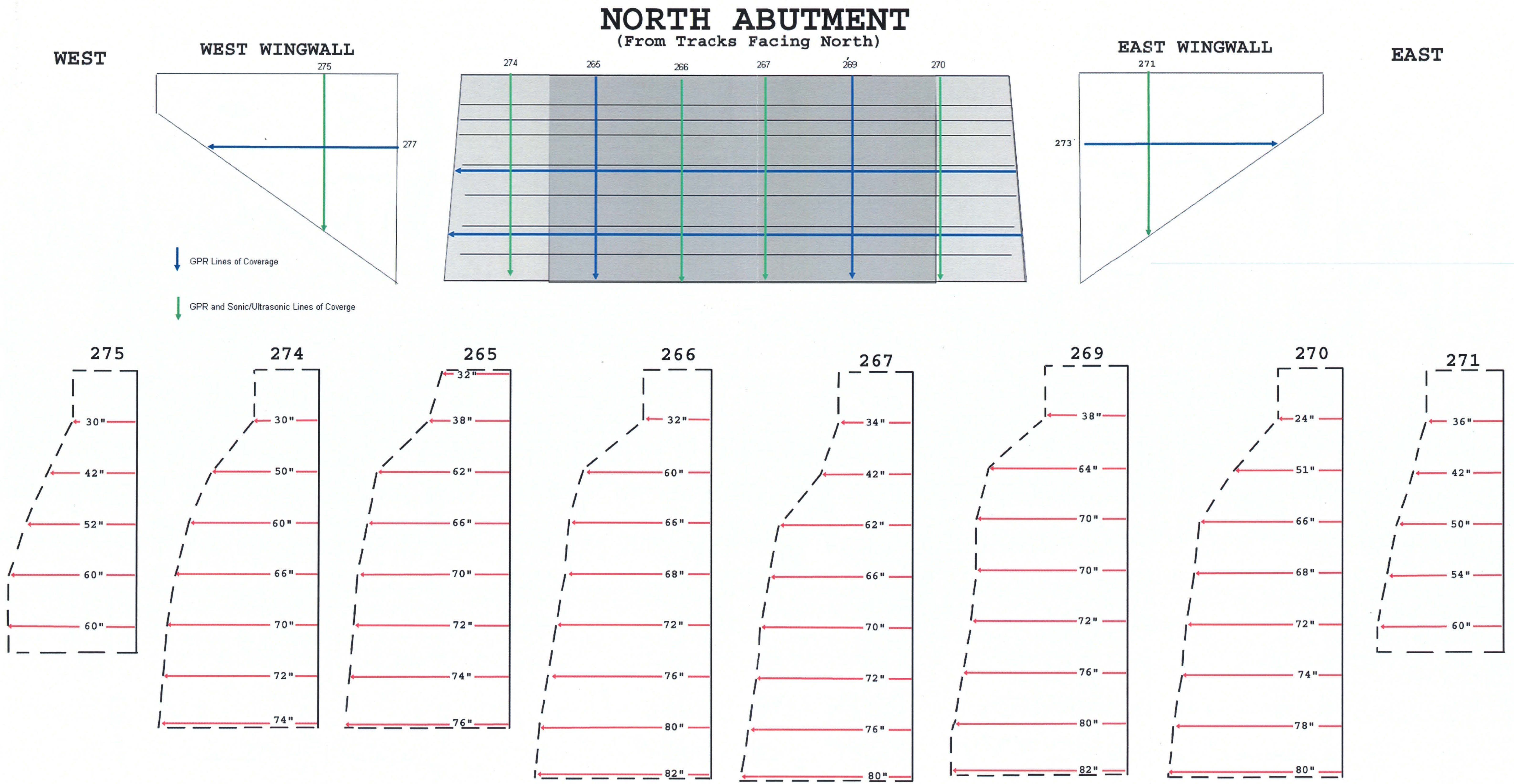



## SOUTH ABUTMENT CROSS SECTION ALONG FILE 255

WEST WING WALL TO EAST WING WALL APPROXIMATELY 6 FEET FROM THE TOP

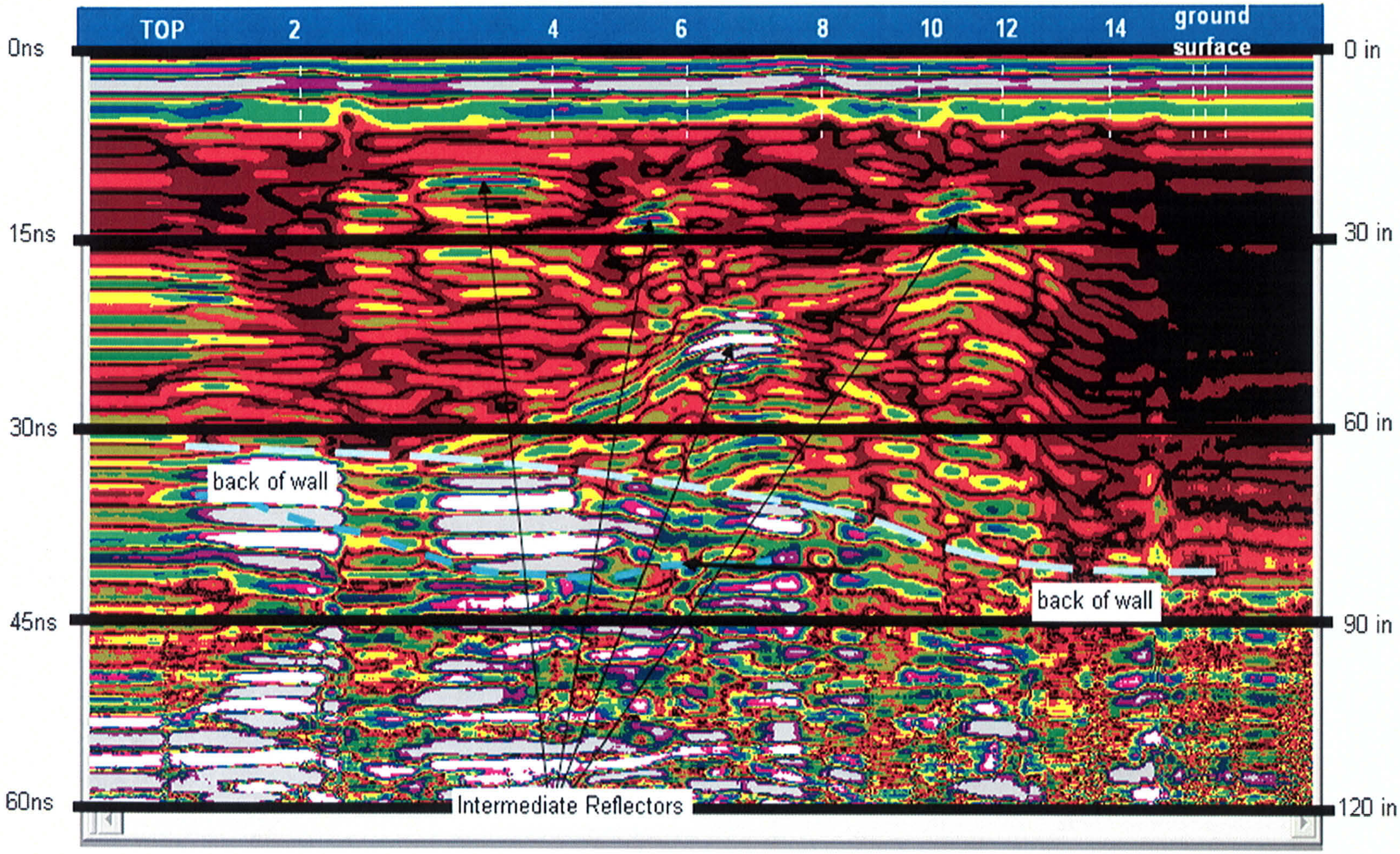



| | | |
|---|---|---|
| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of<br>Transportation<br>by<br>NDT Corporation | South Abutment<br>Cross Sections | |
| | Septemebr,<br>2004 | Figure 3 |




| | | |
|---|---|---|
| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of Transportation<br>by<br>NDT Corporation | Lines of Coverage and Cross Sections | |
| | September, 2004 | Figure 4 |

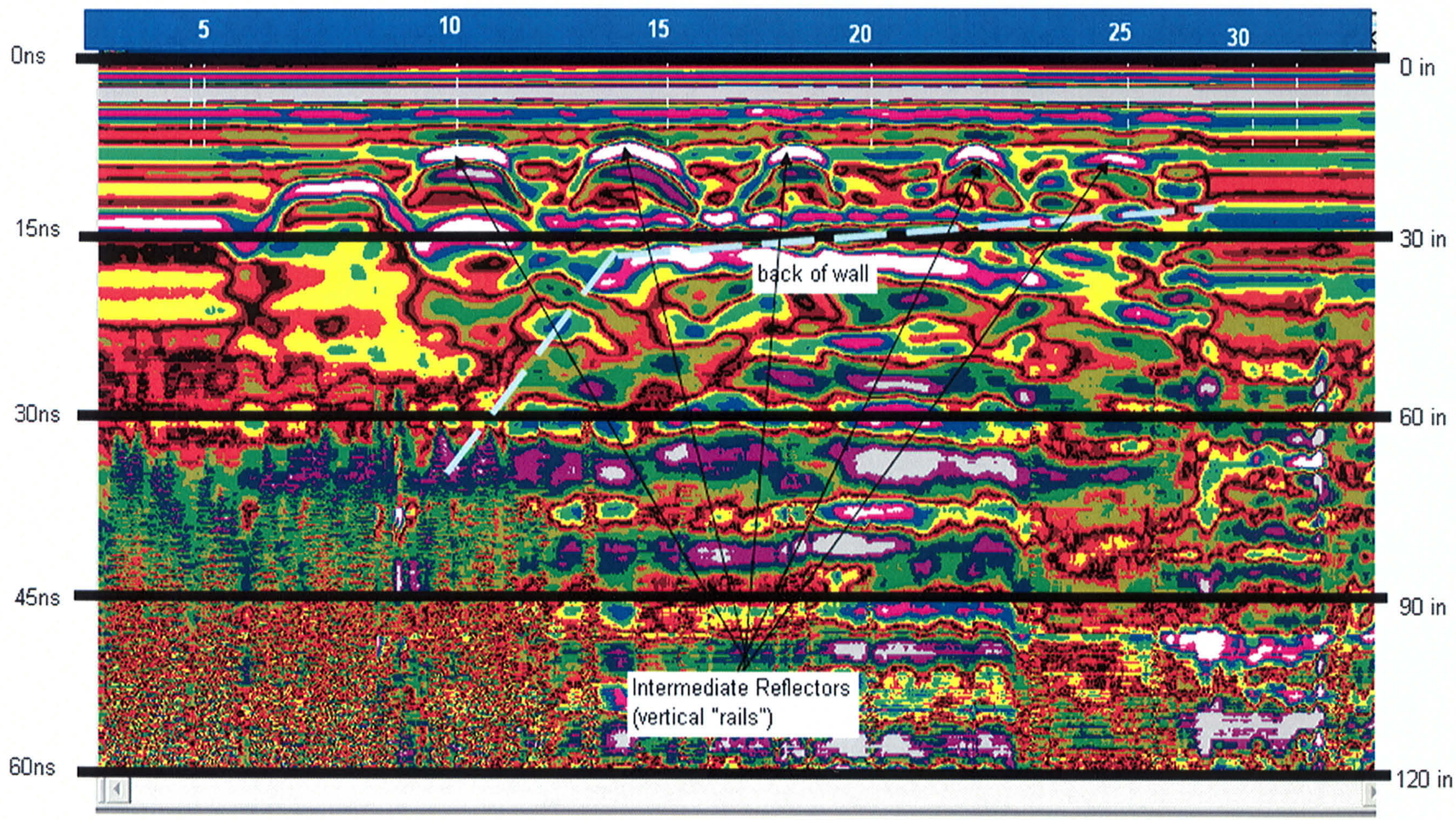
GPR DATA ALONG MASONRY SOUTH ABUMTMENT
from top of abutment to ground surface marks every 2 feet




| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of<br>Transportation<br>by<br>NDT Corporation | GPR Record along<br>Masonary Wall of<br>South Abutment | |
|---|---|---|
| | Septemebr, 2004 | Figure 5 |

## GPR DATA ALONG CONCRETE SOUTH ABUTMENT WEST WING WALL

horizontal line beginning 5 feet from abutment face back
apporimately 2 feet from top of wing wall (marks every 5 feet)




| | | |
|---|---|---|
| Nondestructive Testing<br>Veranda Street Bridge<br>Portland, ME<br>prepared for<br>Maine Department of<br>Transportation<br>by<br>NDT Corporation | GPR Record along<br>Concrete Wall of South<br>Abutment West Wing Wall | |
| | Septemebr,<br>2004 | Figure 6 |

# APPENDIX 1

# GROUND PENETRATING RADAR

# APPENDIX: GROUND PENETRATING RADAR

## OVERVIEW

Ground Penetrating Radar (GPR) is an electrical geophysical method which transmits high frequency electromagnetic waves into the ground and detects the energy reflected back to the surface. GPR utilizes various antennas (transmitter and receiver) with discrete frequencies ranging from 16 MHz to 2000 MHz. Electromagnetic signals are transmitted from the antenna (ground surface) and reflected back to the antenna (ground surface) from interfaces with differing electrical properties. Electrical properties such as dielectric permittivity (dielectric constant) and conductivity are the controlling parameters of the GPR. The greater the contrast in the dielectric constant between two materials, the more energy is reflected to the surface. Thus the greater the contrast in the dielectric constant between the host material and the "target" the more defined results.

GPR reflections typically occur at subsurface discontinuities, lithologic changes, and internal soil structures, such as:

- Buried metal objects (utilities, tanks, reinforcing)
- Open and Water filled voids
- Water table
- Top of bedrock
- Soil and rock stratification
- Seepage and leachate zones
- Bedrock Fractures
- Archaeological structures

The depth of penetration of GPR is site specific, limited by the attenuation of the electromagnetic energy. Signal attenuation is controlled by four different mechanisms listed below.

- Scattering: energy losses due to scattering occur when signals are dispersed in random direction, away from the receiving antenna, by large irregular shaped objects, such as boulders, tree stumps and closely spaced rebar.
- High conductivity layers: the greater the conductivity values of materials at a site, the more signal attenuation or less penetration. (mineral content, high moisture content, water table, metal plates, etc.
- Water/Moisture Content: water molecules polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar system when it is converted to kinetic and thermal energy as a result of rotation of water molecules.
- Clays, (Ion content): ions along clay surfaces polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar when migration and collisions of these charged particles causes electromagnetic energy to be converted to kinetic and thermal energy, which is lost to the radar system.

Signal penetration is also dependent of the frequency of the antenna. High frequency antennas have short wavelengths which are attenuated more rapidly with depth, but have better resolution. Low frequency antennas have long wavelengths which are attenuated slowly with depth but these antennas have lower resolution to details.

## APPLICATIONS:

Ground Penetrating Radar (GPR) is a shallow penetrating geophysical profiling system used where rapid and accurate surveys are desired. GPR can be used for both area and source detection studies. GPR has been used to locate underground pipes, buried drums, foundations, void in rock and concrete, lithologic contacts, determine stratigraphy, depth to water table, depth to bedrock, buried archeological artifacts, excavations, filled pits and lagoons, and numerous other site specific applications. GPR is also an excellent tool for concrete structures such as bridges, walls, beams, ceilings, etc where the GPR can locate rebar and conduits, quantify rebar spacing, cover variability over reinforcing, and even concrete thickness. GPR can be used to locate voiding behind walls, delaminations, and moisture conditions.

Laterally GPR can cover large areas relatively quickly. Using a grid pattern of survey lines it is very effective in collecting data over close to 100% of the survey area. GPR can not only map the lateral extents of targeted features but also can be used to calculate the depth to the features of interest. Typically to perform depth calculations an onsite calibration, to determine the electrical properties of the materials at the site, is need. Depth calibrations typically consist of collecting GPR data over a metal target with a known depth. Known utilities, and buried metal plates are great targets for calibrations. Calibration lines near boreholes that are geologically logged are also good calibrations for depth to bedrock, water table and lithology/stratigraphy surveys.

GPR surveys coupled with other geophysical surveys and/or ground truth methods are good ways to verify, correlate and extrapolate GPR results. GPR is a fast and effective method to cover large survey areas in a short amount of time. For example seismic refraction, boreholes, and/or test pits are good methods to verify depth to bedrock, water table and stratigraphy surveys. Magnetometer and electromagnetic induction methods are good methods to verify locations of metal tanks and metal utilities. Electromagnetic induction and electrical resistivity are good methods to verify the lateral extents of conductive plumes. GPR surveys are a fast and cost effective method to collect data over large or obstructed sites, and isolate anomalies and areas where borings or other methods can be focused for the best interest of a project.

## EQUIPTMENT:

- Control unit (pulse transmitter, digital recorder, data storage, monitor)
- Antenna (s)
- Coaxial Cable
- Printer

GPR Control Units are computers which set up the parameters, such as sampling rate, range, gain control, filtering, etc. the control units also visually displays the data, digitally archives the data, and allows for play back of the data.

The coaxial cable connects the control unit to the Antenna. The Antenna(s) are sealed and shielded fiberglass housing for the transmitter and receiver. (In some cases the transmitter and receiver are placed in separate housings, usually the very low frequency antennas). Radar systems are designed to use antennas of various electrical characteristics. Selection of the antenna is dictated by the requirements of the survey. If high resolution, near-surface data is desired, a small, high frequency antenna is used; if the survey requires deeper penetration, a larger, lower frequency antenna is used. Commercially available antennas have the following frequencies: 16, 20, 32, 40, 80, 100, 120, 300, 400, 500, 900, 1000, 1500 and 2500 MHz. The drawback of using the lower frequency antennas is that resolution of data is sacrificed for penetration. Typically the 80 to 300 MHz antennas are used for geologic, environmental and archaeological surveys; 300 to 900 are used for utility, tank, foundation, etc surveys while the high frequency antenna 900 to 2500 is used for concrete assessment.

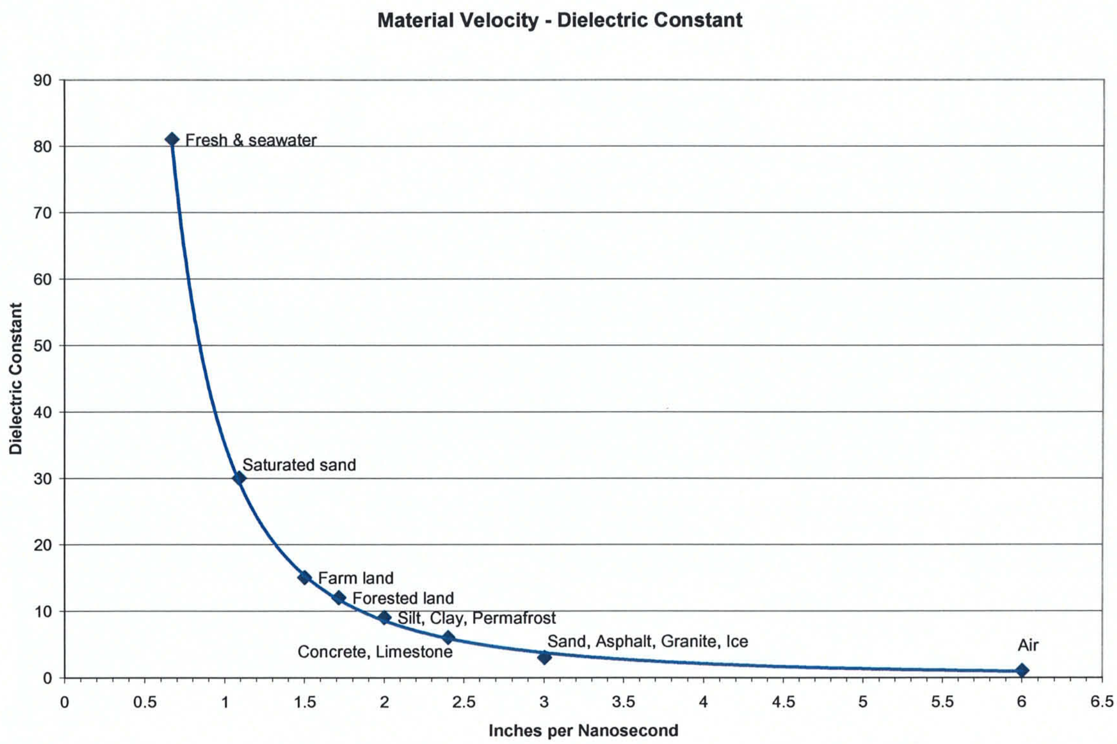
## ACQUISTION AND INTERPETATION:

Site logistics and specifics are the most time consuming part of a GPR survey. The first step in any GPR survey is to get as much information about the site as possible, (Pre-survey site visits are very valuable but not always possible. Site specifics such as definition of the intent of the survey (utility, tank, water table survey etc), depth of survey, calibration areas/targets, accessibility and obstructions, should be gathered before the survey. Background information such as host materials, boring logs, electrical properties of host and targets. These considerations will aid in antenna selection, survey grid size, and the onsite calibration.

### DIELECTRIC CONSTANTS OF SOME COMMON MATERIALS

| | | | |
|---|---|---|---|
| Air | 1 | BEDROCK | |
| Snow | 1-2 | Granite | 4-7 |
| PVC | 3 | Sandstone | 6 |
| Asphalt | 3-5 | Shale | 5-15 |
| Freshwater Ice | 4 | Limestone | 4-8 |
| Concrete | 4-11 | Basalt | 8-9 |
| Soil and Sediments | 4-30 | | |
| Fresh and Saltwater | 81 | | |

At the time of the survey the survey grid should be marked out (a survey wheel attached to the system may be a valuable tool). The onsite calibration should be conducted such that a velocity for the materials can be set, depth of penetration can be determined and the correct acquisition parameter can be adjusted. (Large site and even small sites can be very electrically variable so be aware that these setting may have to be adjusted and other onsite calibrations may be needed.) A good rule of thumb when beginning the calibration is to assume a soil/concrete velocity of 2 inches per nanosecond. Set the time window for 2X the depth of interest when ever possible such that if the depth of interest is around 10 feet set the time window for approximately 100 to 120 nanoseconds. This should put the calibration "target" near the middle of the record, once the "target" is noted then the settings should be adjusted for the best resolution.

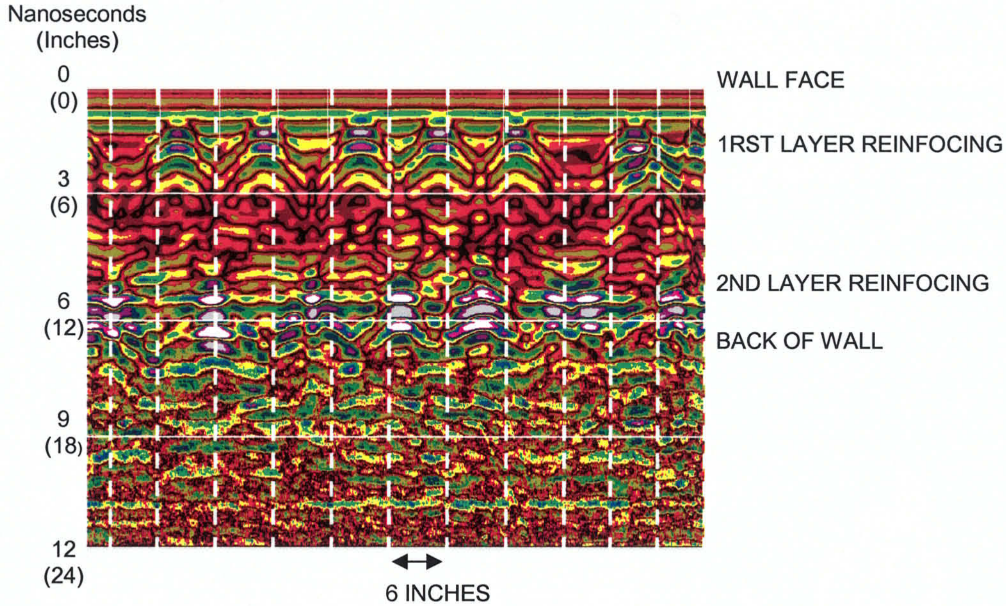



Once the settings are adjusted and a certain confidence level with the calibration is obtained then the survey can begin. Slowly walk the antenna along the grid lines. Speed at which the antenna is moved is determined by the type of survey and what the "target" is. Radar signals are propagated from the antenna in a 15 to 45 degree cone, thus the slower the speed of the antenna the greater the horizontal resolution. Slow walking speeds are recommended for most surveys, but if the target is a long continuous layer such as water table mapping the antenna may be towed from the back of a car of truck at speeds up to 10 miles/hours.

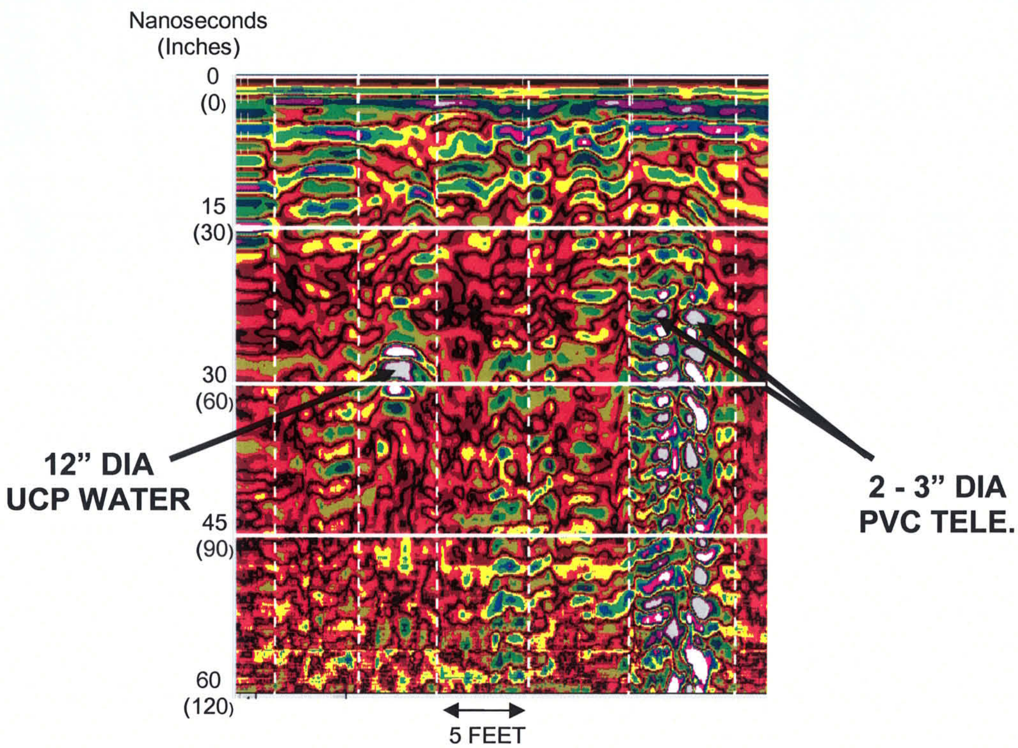
Each survey line should be printed and digitally saved to assure record security and for interpretation purposes. Station markers and any field notes can be written right on the printed copy and the digitally saved data can be used to reprint or to use with post processing software back at the office. Interpretation of GPR data is subjective, even among experienced interpreters. The strength of a reflected signal and/or the continuity of the reflector across the record may be indicative of a stratigraphic contact. FIGURES>>>>> Point targets, such as buried drums, pipes, boulders, tree stumps, create a distinctive parabolic feature on GPR records. Positive identification of point targets is subjective, as the GPR signature of a pipe is similar to that of a large boulder.

Computer processing in the form of filtering, deconvolution, migration, color tables, gain enhancement etc. is available though it is somewhat costly and in most cases not necessary, except for presentation purposes.

## GPR RECORD
## 12” THICK WALL WITH REINFORCING




## UNDER GROUND UTILITY LOCATION/MAPPING



# APPENDIX 2

# SONIC/ULTRASONIC MEASUREMENTS

# APPENDIX

## SONIC/ULTRASONIC NONDESTRUCTIVE TESTING OF CONCRETE

The sonic/ultrasonic measurements made to determine the characteristics of concrete (or rock) are generated by a relatively low energy source generally as a single discrete impulse caused by a pulsed transducer, projectile, mechanical hammer, laser, etc. as a single event wide band source. In the case of a pulsed transducer the energy may be input in the form of a series of tone bursts. Practical problems largely determine the source(s) to be used. Where concrete such as in an older tunnel liner is being investigated, then the condition of the concrete surface largely determines the selection. A rough concrete surface that has deposits of organic materials or mineral deposits generally requires a more powerful energy source whereas a relatively new or wet concrete may be inspected by the use of a pulsed transducer or other higher frequency source. In general high frequency sources that may work well in the laboratory may be unusable for the in-situ conditions. High frequency sources have the advantage of high resolution but the disadvantage of low penetration. While metals can be tested in the megahertz range, such signals in concrete will not have measurable signals for less than an inch in thickness. The energy source should be sufficient to maximize the resolution, have sufficient penetration to examine the concrete being tested and enough energy to excite the fundamental frequencies being sought.

The transmitted energy is in the form of three principal wave types, compressional (contraction expansion-spring like particle motion), shear (traction-sliding motion), and surface waves (combination of motions). Each boundary that has density and or velocity contrast will reflect and or refract these waves; for the present purposes, the compressional and shear waves will be discussed. The velocity values are determined by the Young's shear, and bulk moduli values as well as the density and Poisson's Ratio. In turn the velocity can be used to determine the moduli values and Poisson's Ratio given that the density is known. The moduli values measured are the dynamic moduli values at low strain. In general the difference between the dynamic values and the static values is almost entirely controlled by the crack densities of the concrete. Using the modulus values, a reasonable estimate of the unconfined compressive strength can be determined. The strength is largely dependent on the crack density of the concrete and for static tests the orientation of the cracks. Cracks perpendicular to the axis of the core and perpendicular to the directed stress will produce a strength (static) that is not greatly different from un-cracked concrete. The applied stress closes the cracks in compression. Cracks that are near 45° to the direction of stress will result in lower strength. The orientation of the cracks can be determined by measuring the velocity values in different directions.

NDT Engineering, Inc. makes several determinations from one energy impact. The velocity is measured directly from the energy point of impact to a linear array(s) of sensors on the surface their array length is usually in excess of the thickness of the concrete being tested. In addition to the velocity measurements, reflections are measured individually or more likely by examining the resonant frequency (multiple reflections) of the layered sequence in the frequency domain. Each reflecting surface (change of density and/or velocity) produces a multi-path reflection in the layer it bounds. For example the generated wave will travel to a delamination surface and then reflect back to the surface of the concrete in multi-reflections.

These become apparent in the frequency domain where processing can enhance their presence (along with their higher modes). These reverberations (echoes) are particularly diagnostic of delaminations and thickness of the concrete. They will readily distinguish the presence of local delaminations, cracked or decomposed inclusions by the particular frequency band generated at the mechanical discontinuity, This is a drum head effect where the inclusion of differing properties from the host material resonates in a relatively narrow band usually quite distinguishable. This is the basis of the 'chain drag" using the human ear as the sensor to recognize frequency differences. The ear however is limited in its perception and will distinguish within the hearing range which is relatively small population of potential problems.

DIRECT AND REFRACTED ENERGY

One of the advantages of the sonic/ultrasonic method is its ability to "look through" overlying materials coatings particularly decomposed "softer layers" when the array(s) is configured properly. This is done using refracted waves associated with the different layer velocities or by careful examination of the resonant frequencies associated with such layering.

The diagram below shows the wave path for refracted energy generated for a softer (1) over harder (2) layer. The wave is bent (similar to the appearance of a stick in water) toward and travel along the boundary between the lower velocity layer and the harder concrete and radiates back to the surface. The higher velocity of the good concrete assures that the refracted wave (2) will overtake the direct wave (1) at some distance designated as $D_{1|2}$. To the left of this point the surface velocity (lower) will be measured and beyond it the velocity of the deeper layer is measured.

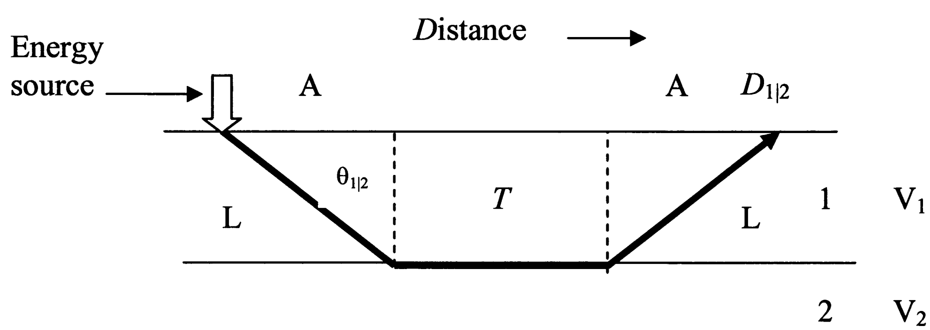



Figure A1

The time for the direct path is $D/V_1$; the refracted path time is $2L/V_1+(D_{1|2}-2A)/V_2$.
The array of sensors is placed in the distance direction and the time elapsed (travel time) from the time of energy impact to the sensor distance is measured. The velocity is determined from this time-distance measuremen(s). The angle θ is the angle between the perpendicular to the layer and the incident wave that is critically refracted. The sine of this angle is the velocity of the first layer divided by that of the second layer (Snell's Law). The

distance shown $D_{1|2}$ is the point on the surface where the refracted time arrival equals that of the direct wave (the refracted wave travels at a higher velocity than the direct wave

$$\frac{D}{T} = \frac{D - 2\tan\Theta}{V_2} + \frac{2T}{V_1 \cos\Theta}$$

With some substitutions and algebraic minipulations the thickness is expressed as:

$$T = \frac{D_{1|2}}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

*D* is the distance and *T* is the thickness. Since the times as well as the distances are measured, then $V_1$ and $V_2$ are determined. If a plot of distance versus time is made then the resulting graph will look like Figure A2.

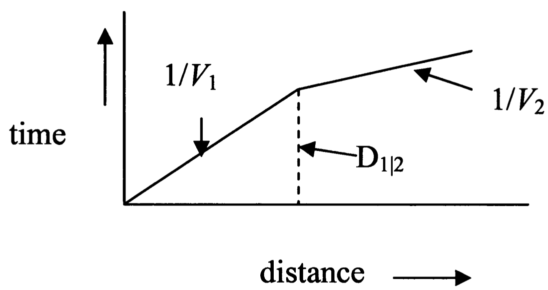



Figure A2

If there is no overlay then the velocity is simple $D/T$.

The resonant freuencies are determined by the thickness and the velocity of the material. Since the velocity is measured as above then the thickness can be determined directly.

The resonance of a simple beam is given by:

$$f = \frac{nV}{2L} \qquad (\mathit{fixed} - \mathit{fixed}, \quad \mathit{free} - \mathit{free}) \quad \mathit{where}\ n = 1,2,3 - - -$$

$$f = \frac{nV}{4L} \quad (\mathit{open} - \mathit{fixed}), \quad \mathit{where} \quad n = 1,3,5,7 - - -$$

Since the frequency and velocity are measured, the thickness is determined. This thickness can be the thickness of the concrete slab(floor, deck),column, being measured or it can be the thickness of a delamination. The computation of the dimensions of an included body a zone of cracked, deteriorated concrete or a delamination can be determined from the measured frequency and velocity.

While the refracted wave is dependent only on a contrast in velocity, a reflection can take place where there is a change in velocity or density or both. The impedance (RF reflection coefficient) which causes a wave to be reflected is given by:

$$\mathrm{RF} = \frac{\rho_2 V_2 - \rho_1 V_1}{\rho_2 V_2 + \rho V_{11}}$$

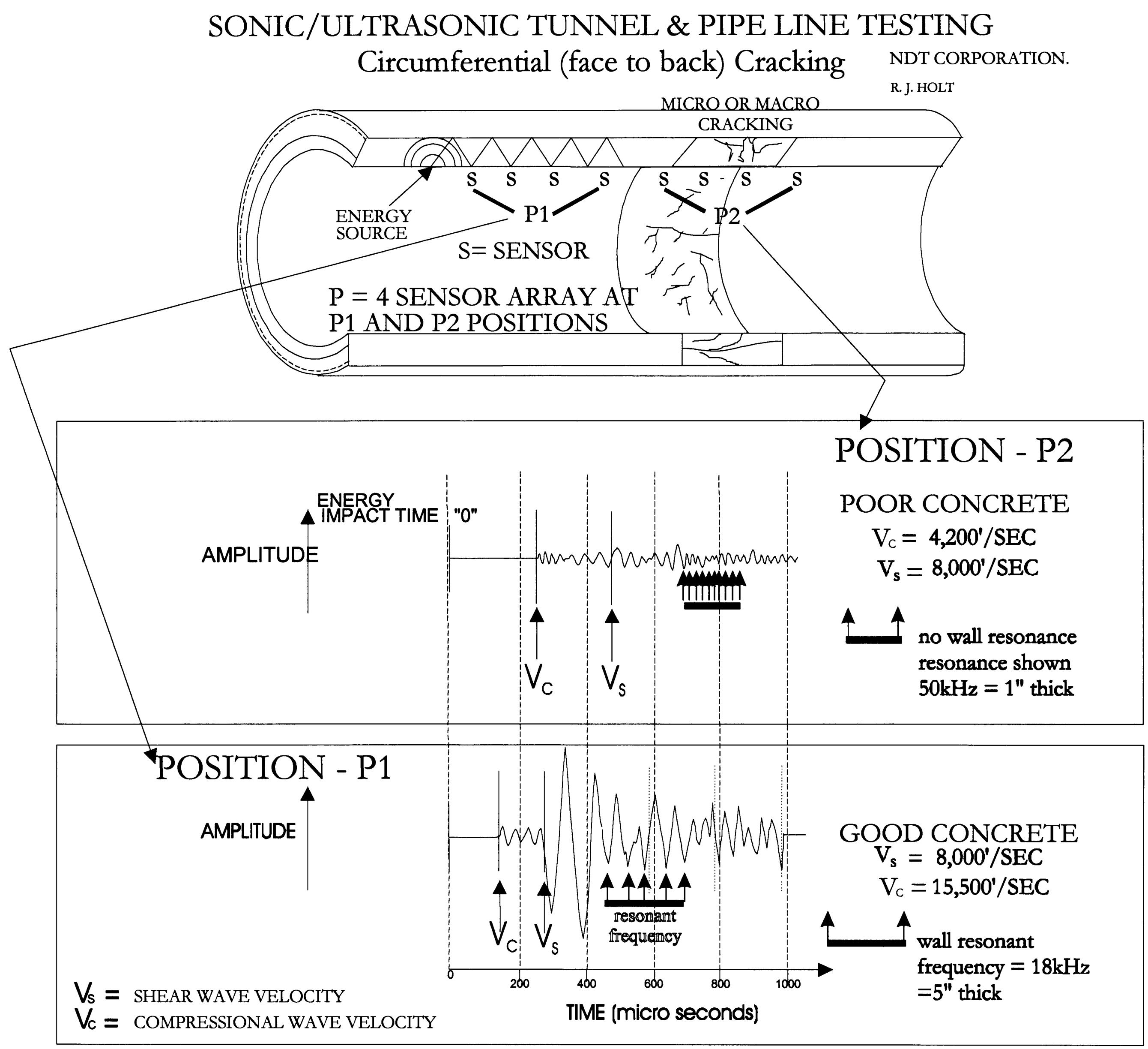
Where $\rho$ is the density and $V$ is the velocity of the material. The impedence determines the strength of the reflection. The contrast between an air filled void at the back of or within the concrete is significant, the velocity in air is 1,000ft/sec. and that in good concrete is 13,000ft/sec. The density differences are of course very large between the concrete and air. The same difference exists for a water filled void where the velocity in water is 5,000ft/sec. and concrete is nearly a factor of 2.5 denser. Voiding behind a liner or under a slab is usually well distinguished by a distinct resonant frequency.

MODULI VALUES AND STRENGTH

The moduli values as stated above are determined from the velocity values using an assumed or measured density. The density is usually the best known or best estimated value for the concrete, its variance generally does not affect the calculations significantly.

Figures are included with this appendix that show the relationships for Young's modulus vesrsus the compressional velocity Figure A4; shear modulus versus the shear velocity Figure A5; Poisson's Ratio versus the compressional and shear wave velocities Figure A6; and finally a relationship between the velocity values (compressional and shear) and the unconfined compressiive strength of concrete, Figure A7.

The figure below Figure A3 is illustrative of a tunnel liner or pipe investigation where there has been circumferential damage perhaps at a construction joint, an outside zone of weakness (rock shear or fault, soil washout etc.) that has affected the liner. The damage need not be visible; there can be a 20% reduction in the strength of the concrete from micro-cracking that is not visible to the naked eye. The process of deteroration of most concrete starts at the micro level and with continued stress the micro cracks coalesce into macro cracks and finally to spalling. The ability to measure at the micro level well in advance of future needed repairs provides a management tool for establishing proirities for repair, projected budgets, and asset valuation




SONIC/ULTRASONIC TIME AMPLITUDE RECORDS
FOR FOURTH SENSOR - POOR AND GOOD CONCRETE

FIGURE A3

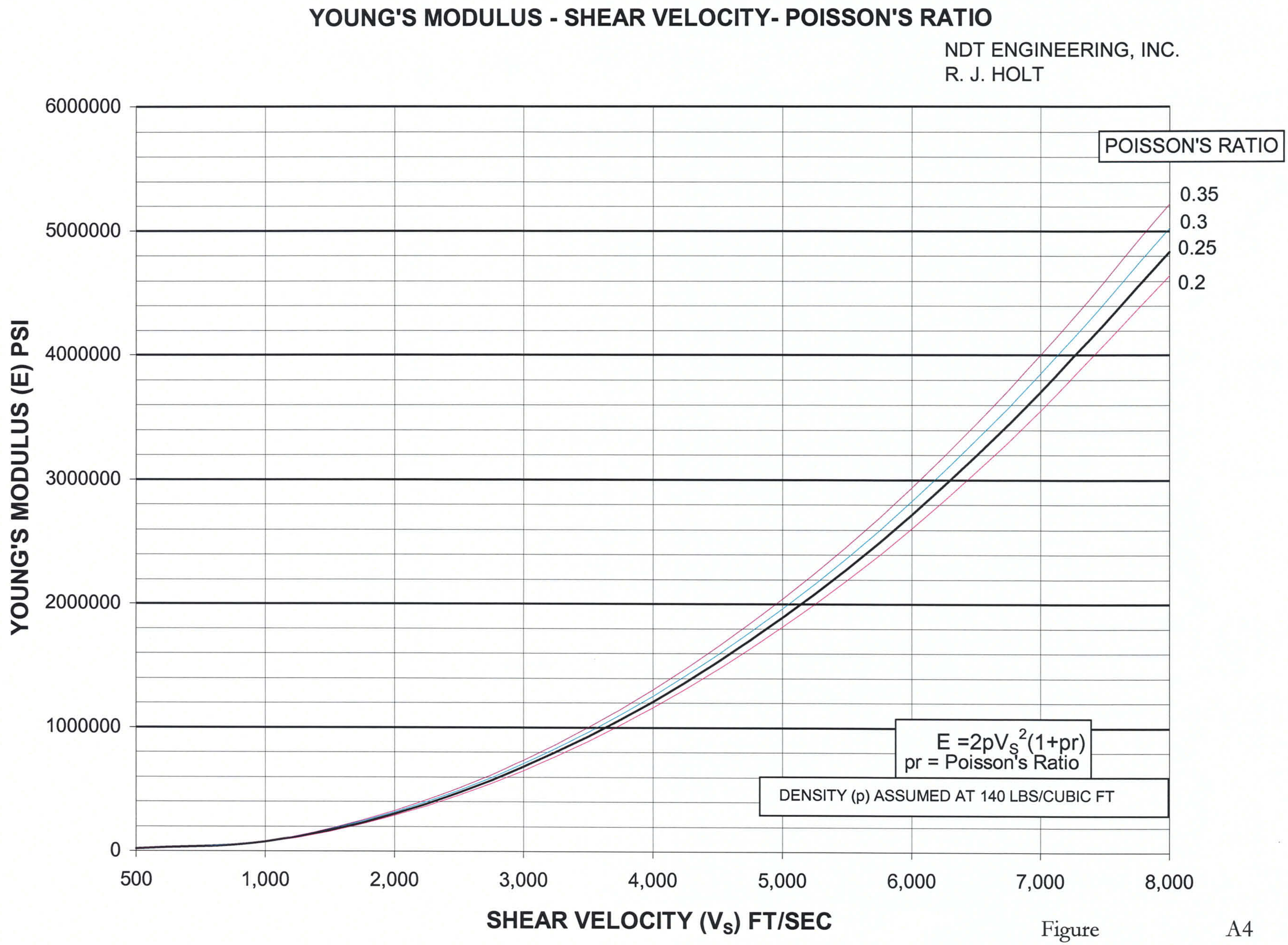

YOUNG'S MODULUS - SHEAR VELOCITY- POISSON'S RATIO
NDT ENGINEERING, INC.
R. J. HOLT
YOUNG'S MODULUS (E) PSI
6000000
5000000
4000000
3000000
2000000
1000000
0
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY ($V_S$) FT/SEC
POISSON'S RATIO
0.35
0.3
0.25
0.2
$E = 2pV_S^2(1+pr)$
pr = Poisson's Ratio
DENSITY (p) ASSUMED AT 140 LBS/CUBIC FT

Figure A4

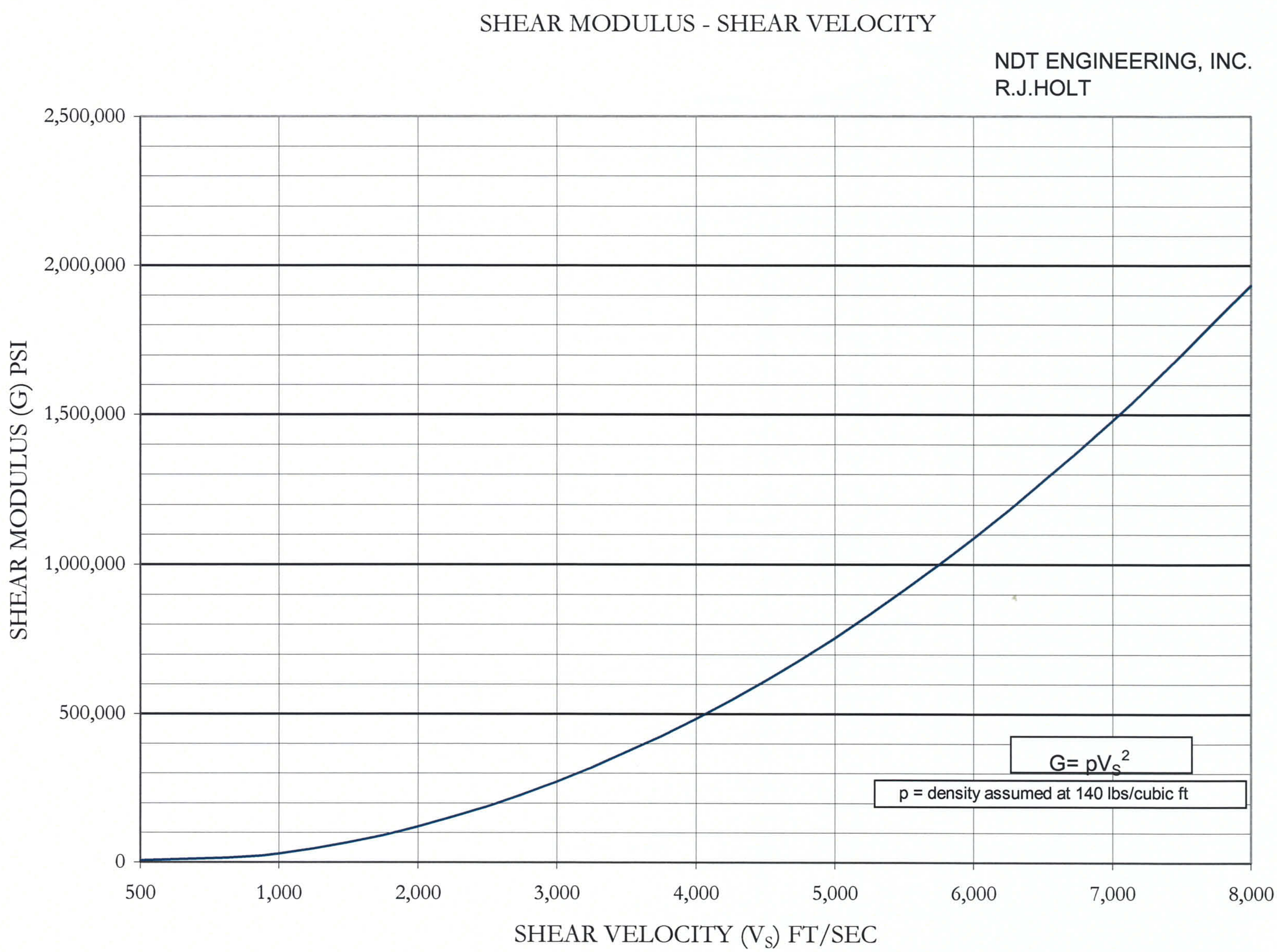
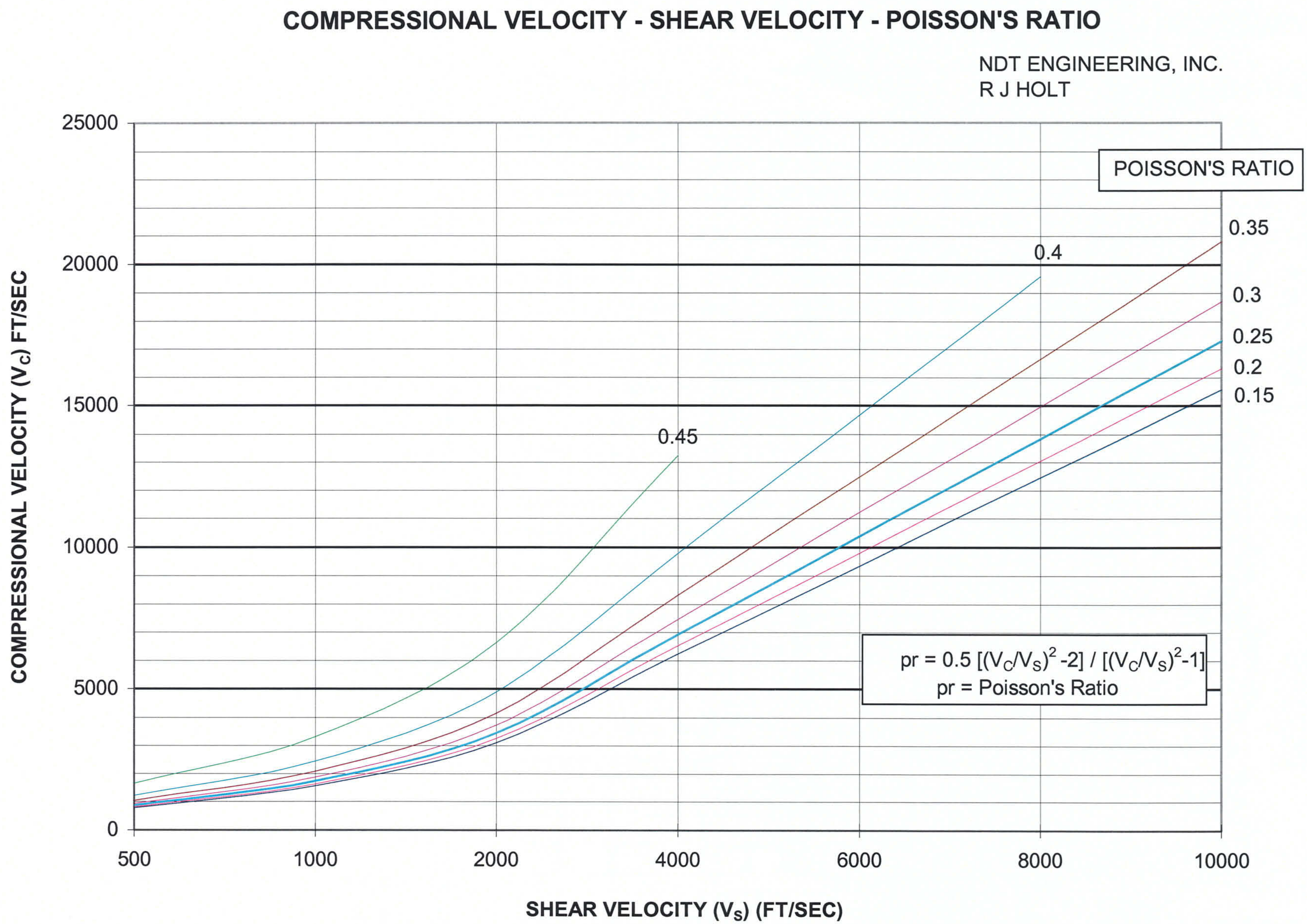

SHEAR MODULUS - SHEAR VELOCITY
NDT ENGINEERING, INC.
R.J.HOLT
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
SHEAR MODULUS (G) PSI
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY ($V_S$) FT/SEC
$G = pV_S^2$
p = density assumed at 140 lbs/cubic ft

Figure A5

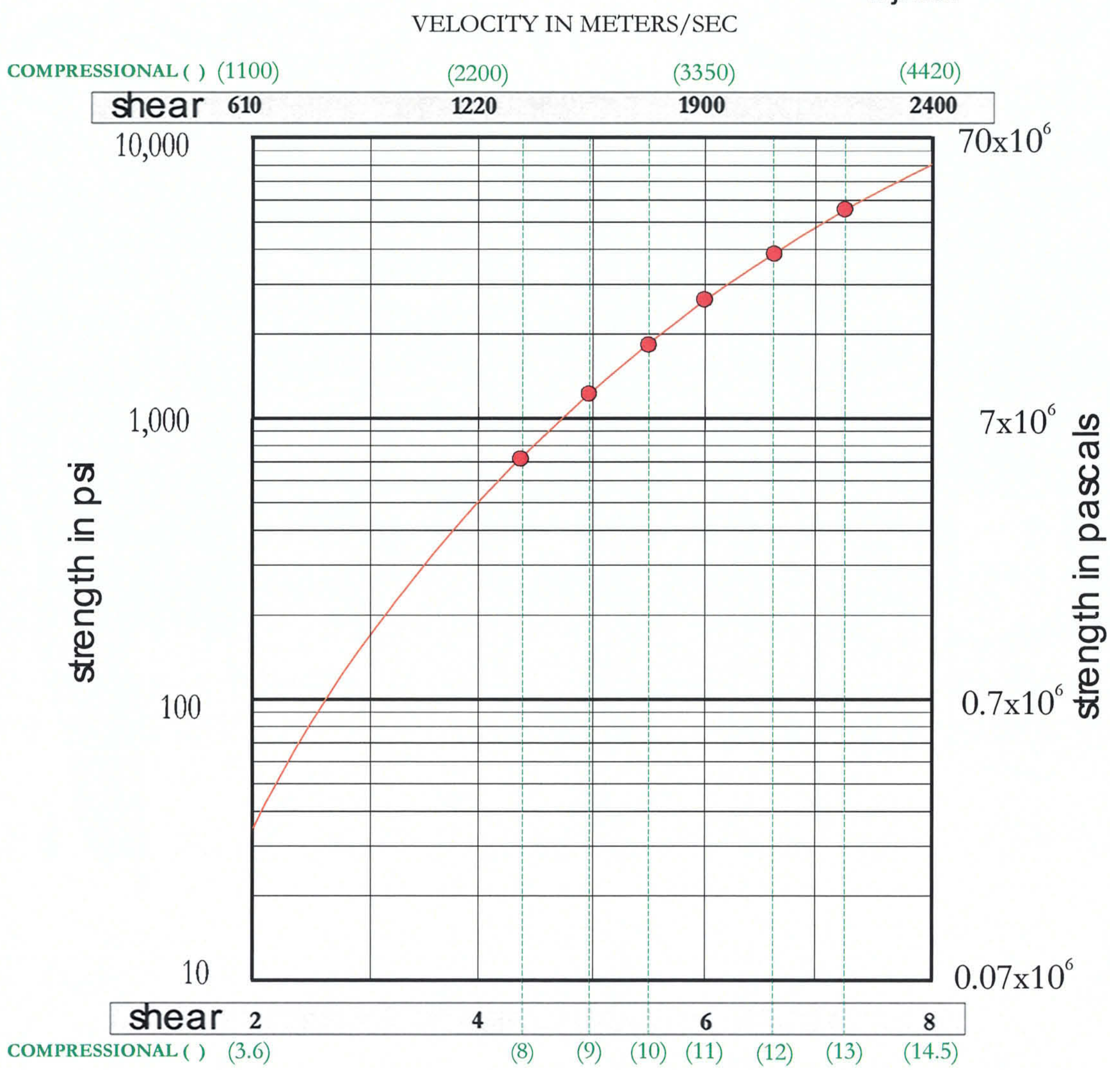


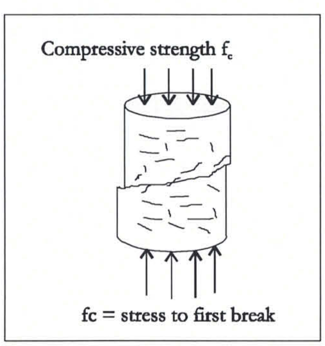
COMPRESSIONAL VELOCITY - SHEAR VELOCITY - POISSON'S RATIO
NDT ENGINEERING, INC.
R J HOLT
POISSON'S RATIO
0.45
0.4
0.35
0.3
0.25
0.2
0.15
pr = 0.5 [(V_C/V_S)^2 -2] / [(V_C/V_S)^2-1]
pr = Poisson's Ratio
COMPRESSIONAL VELOCITY (V_C) FT/SEC
25000
20000
15000
10000
5000
0
500
1000
2000
4000
6000
8000
10000
SHEAR VELOCITY (V_S) (FT/SEC)


Figure A6

# strength of concrete versus velocity

r. j. holt




VELOCITY IN FEET/SEC (X 1,000)

CURVE FOR RATIO: $V_{SHEAR} / V_{COMPRESSIONAL} = 0.55$
EQUALS POISSON'S RATIO OF 0.28

FIGURE A7

NDT ENGINEERING, INC.

# APPENDIX E

Calculations – Stone Masonry Abutment Stability Analyses

**Analysis:** **ALLOWABLE BEARING CAPACITY**
**Structure:** **Existing Abutments founded on Spread footings**
**Project Name:** Portland Veranda Street Bridge
**by:** L. Krusinski
**date:** November 2004
**Sheets:** 8 sheets
**check by:**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tonf := g \cdot ton$ $kip := 1000 \cdot lbf$

$ksf := \frac{kip}{ft^2}$ $ton := 2000 \cdot lbf$ $tsf := \frac{tonf}{ft^2}$ $psi := \frac{lbf}{in^2}$ $ksi := \frac{kip}{in^2}$

**Assumptions**

Minimum footing width is 7 ft

Base of stone footings are at approximately elev 17.0 ft (5.2 m). This results in 4.0 embedment for frost, assuming finished grade elevation of 21 feet. To be verified with a test pit during final design. Assume a footing embedment of 2 ft in this analysis

Supported on brown, wet dense, fine to coarse sand, trace silt and gravel, N=43, N=51 (BB-PRR-101)

Supported on Light brown, moist dense, fine to coarse SAND, trace silt and gravel (Fill - N=39) underlain by Brown moist very dense, fine to coarse SAND, little gravel, trace silt (N=59, N=47). Based on BB-PRR-102D.

Method used: Terzaghi, use strip footing equations since L>5B

Examine 1 conditions: (1) effective stress (unconservative)

**Foundation soil values**

$\phi$=30-34 degrees at ultimate strength for an effective stress analysis for a drained, effective stress analysis. Phi is 38 to 40 degrees at peak strength. (Lamb and Whitman, Table 11.3). Based on Bowles table 3-4 $\phi$=40 for dense granular soils. Use 35 degrees

**Available References:**

$\phi$ : Lambe & Whitman Table 11.3 based on Hough, Basic Soils Engr, 1967
$\phi$, SPT correlation, Lambe & Whitman, Fig 11.14, (from Peck, Hanson, Thornburn).
$\phi$ and $\gamma$ correlations to soil description and N values, Bowles 1977 Table 3-4
$\phi$: Bowles (4 th Ed) Table 2-6
Mass Highway unnamed Table for $\gamma$sat
$\gamma$ sat : Holtz, Kovacs, Table 2-1 1981

**Footing Width and Depth**

$$B := \begin{pmatrix} 10 \\ 8 \\ 7 \end{pmatrix} \cdot ft \qquad D_f := 2 \cdot ft$$

## Soil Statigraphy

Depth to water table $D_w := 5 \cdot ft$ $\gamma_w := 62.4 \cdot pcf$

Fill

$\gamma1_{sat} := 19.6 \cdot \frac{kN}{m^3}$ $\gamma1_d := 18.9 \cdot \frac{kN}{m^3}$ $\gamma1_{sat} = 124.771\ pcf$ $H_1 := 7 \cdot ft$ $N1 := 20$

$\gamma1_t := \gamma1_{sat}$ $\phi := 35 \cdot deg$ $c_1 := 0 \cdot psf$ $\gamma1_d = 120.315\ pcf$

## Bearing Capacity

***Method 1 : Terzaghi Method - Drained, Effective Stress Analysis (unconservative)***

$$N_c := \cot(\phi) \cdot \left[ \frac{e^{2 \cdot \left(3 \cdot \frac{\pi}{4} - \frac{\phi}{2}\right) \cdot \tan(\phi)}}{2 \cdot \left(\cos\left(\frac{\pi}{4} + \frac{\phi}{2}\right)\right)^2} - 1 \right]$$

$$N_q := \frac{e^{2 \cdot \left(3 \cdot \frac{\pi}{4} - \frac{\phi}{2}\right) \cdot \tan(\phi)}}{2 \cdot \cos\left(45 \cdot deg + \frac{\phi}{2}\right)^2}$$

$$N_\gamma := \frac{1}{2} \cdot \left( \frac{K_p}{\cos(\phi)^2} - 1 \right) \cdot \tan(\phi)$$

**where**

$$K_p := \tan\left[(45) \cdot deg + \frac{\phi}{2}\right]^2$$

***the Kp is not fully explained in Terzaghi - back computed Kp from curve fit to data. Use Meyerhof Nγ if ϕ<40; or use Vesic or Spangler and Handy per Bowles page 187.***

**Vesic** $N_\gamma := 2 \cdot (N_q + 1) \cdot \tan(\phi)$ $N_\gamma = 59.433$

**Spangler and Handy** $N_\gamma := 1.1 \cdot (N_q - 1) \cdot \tan(1.3 \cdot \phi)$ $N_\gamma = 45.267$

**SOUTH ABUTMENT**

**STONE MASONRY**
**ABUTMENT SECTION**

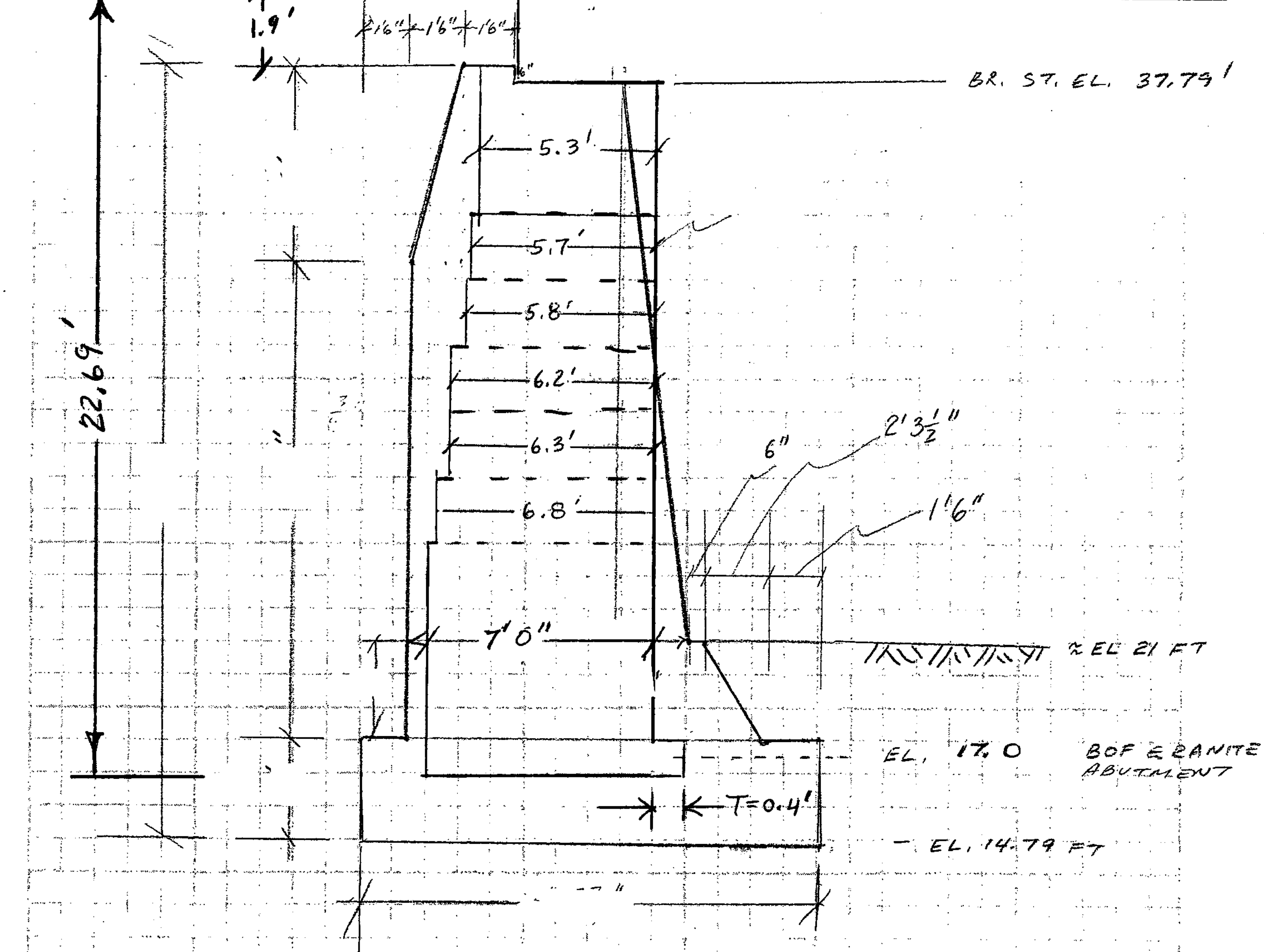
## STABILITY ANALYSES

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 110158.00 FILE NO. ____ OF ____ KM 12/04

FINAL CHK. BY ____ DATE ____ LOCATION PORTLAND SH. NO. ____ OF ____

ITEM NO. ____ SUBJECT SOUTH ABUTMENT - STONE MASONRY

STONE MASONRY GEOMETRY BASED ON GPR / SONIC SCAN LINE # 250 PER FIGURE 2, NDT CORPORATION, 9-2004




ASHLAR STONE ABUT ≈ 32' FT LONG

## Terzaghi equation for continuous foundations (Bowles, Table 4-1, 4th Ed.)

$q := (D_f)\cdot(\gamma 1_d)$ $\quad q = 0.12\,\text{tsf}$

$q_u := 1.0\cdot c_1\cdot N_c + q\cdot N_q + 1.0\cdot(\gamma 1_d)\cdot B\cdot N_\gamma$

***Solution***

$N_c = 57.754$ $\quad N_q = 41.44$ $\quad N_\gamma = 45.267$

$q_u = \begin{pmatrix} 32.217 \\ 26.771 \\ 24.048 \end{pmatrix}\text{tsf}$ $\quad q_{allow} := \frac{q_u}{3}$ $\quad q_{allow} = \begin{pmatrix} 10.739 \\ 8.924 \\ 8.016 \end{pmatrix}\text{tsf}$

## Terzaghi modified procedure with Vesic modification for Nq

$N_{q\_vesic} := e^{(3.8\cdot\phi)\cdot\tan(\phi)}\cdot\tan\left(45\cdot\deg + \frac{\phi}{2}\right)^2$ $\quad N_{q\_vesic} = 18.747$

**continuous foundations**

$q_u := (1.0\cdot c_1\cdot N_c) + q\cdot N_{q\_vesic} + 1.0\cdot\gamma 1_d\cdot B\cdot N_\gamma$ $\quad q_u = \begin{pmatrix} 29.487 \\ 24.041 \\ 21.318 \end{pmatrix}\text{tsf}$

$q_{allow} := \frac{q_u}{3}$ $\quad q_{allow} = \begin{pmatrix} 9.83 \\ 8.01 \\ 7.11 \end{pmatrix}\text{tsf}$

## Presumptive Bearing Capacity

4 tsf based on Table 1201, Massachusetts Building Code 1990
4 tsf based on NavFac DM 7.3

**Use 4 tsf for allowable bearing capacity**

T·Y·LIN INTERNATIONAL

To: Kate Maguire, P.E.
Maine Department of Transportation
State House Station 16
Augusta, Maine 04333

From: Tim Merritt, P.E.
T.Y. Lin International
5 Fundy Road
Falmouth, Maine 04105

Date: July 9, 2004

Subject: Portland – Veranda St. over St. Lawrence & Atlantic Railway
PIN 10158.00 (Bridge # 5052)
**Preliminary Abutment Reactions**

Copy: Mike Wight, File

## MEMORANDUM

Dear Kate:

T. Y. Lin International (TYLI) has estimated preliminary abutment reactions for the subject project for your use in evaluating the re-use of the abutments. We have assumed a 3ft profile raise and included the extra dead load of the raised abutment seat and backwall, as well as half of a new approach slab. The voided slab alternative is the heaviest at this point. The unfactored Group I loading per abutment is:

DL=3,125kN (700k)
LL (HL-93)=960kN (215k)

As we previously mentioned, the 1930 existing bridge plans do not contain any soils information and there are notes that indicate that the footing elevation may have been changed during construction. There also is a note that indicates that piling should be used at the engineer's discretion. The existing plans also indicate that an approach slab was detailed, but we see no field evidence that would indicate this.

Please note that there is the possibility of locating new abutments inside the existing ones if the railroad indicates that is acceptable for their horizontal clearance needs and future track use. It is our understanding that a combination of soil borings and probes will be taken to identify soil conditions and abutment limits.

Please do not hesitate to call or email with any questions, concerns, or suggestions.

Thank you for your assistance,

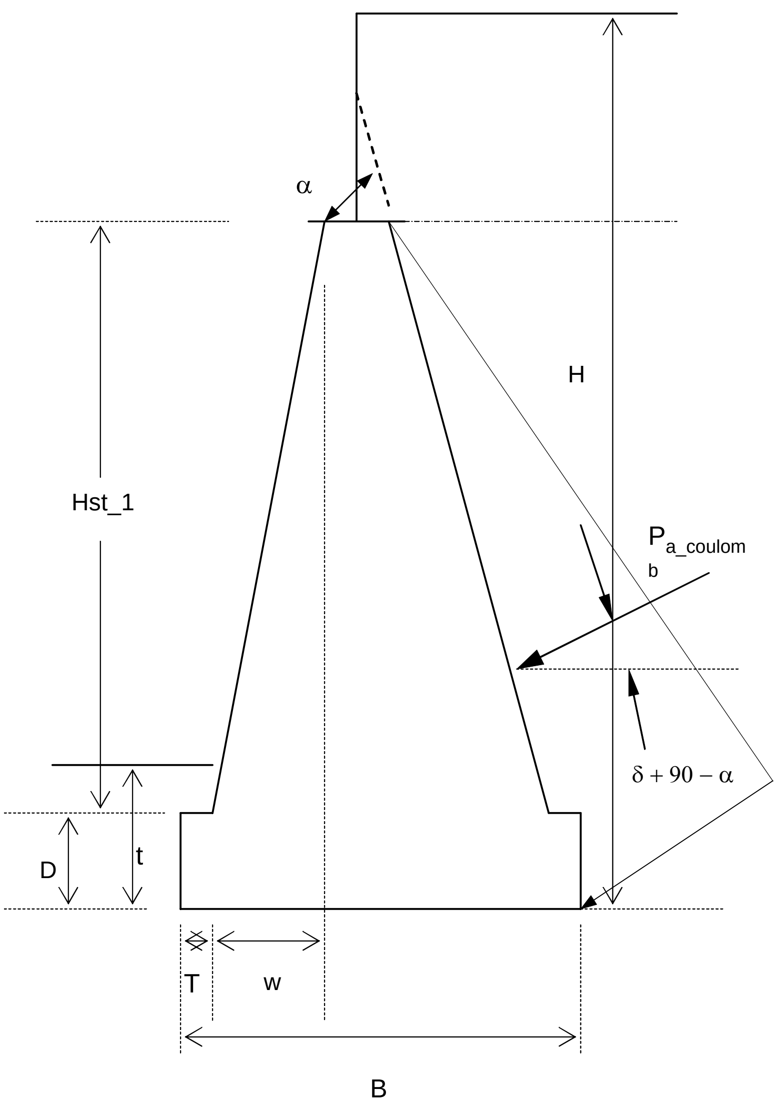
T. Y. Lin International
Tim Merritt
Tim Merritt, P.E.
Associate

LetterKM070104.doc / twm

RECEIVED
JUL 12 2004

**South Abutment Analysis - using field verified abutment dimensions and field-verified backfill.**
**Uses Coulomb theory. Traffic Surcharge added (Coulomb). Existing conditions - no raise in grade**
**Assuming no batter.**
**Footing toe of 0.4 ft based on test pit.**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 22.69 \cdot ft$

Width of footing $B := 7. \cdot ft$

Length of toe $T := 0.4 \cdot ft$

Footing thickness $D := 3 \cdot ft$

Depth of toe $t := 0 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 20.79 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

**Assumed backfill and abutment properties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

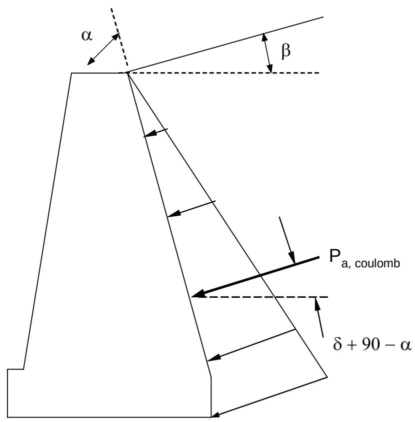
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$$K_{a\_rank} = 0.307$$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$$K_{a\_coulomb} = 0.313$$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 10.057\ \text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{avert} = 4.409 \times 10^3\, lbf$ $\quad E_{avert} = 4.409\, kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{ahoriz} = 9039.227\, lbf$ $\quad E_{ahoriz} = 9.039\, kip$ per lin ft of wall

### No approach slab; Force due traffic - model w/ surcharge of 2' of soil ( using Coulomb earth pressure theory)

$s := 2 \cdot ft \cdot \gamma_1$ $\quad s = 250\, psf$

$E_s := K_{a\_coulomb} \cdot s \cdot H \cdot b$ $\quad E_s = 1.773\, kip$

### Vertical Surcharge Earth Pressure, Resultant acting at H/2:

$E_{surch\_vert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_vert} = 0.777\, kip$ per lnr foot of wall

### Horizontal Surcharge Earth Pressure, Resultant acting at H/2:

$E_{surch\_horiz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{ahoriz} = 9.039\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 7.0 \cdot 7.0 \cdot ft^2$ $\quad F_1 := A_1 \cdot \gamma_c \cdot b$ $\quad x_1 := \frac{7.0 \cdot ft}{2}$ $\quad M_{r1} := F_1 \cdot x_1$ $\quad M_{r1} = 29.155\, kip \cdot ft$

$A_2 := 6.8 \cdot 2 \cdot ft^2$ $\quad F_2 := A_2 \cdot \gamma_c \cdot b$ $\quad x_2 := \frac{6.8 \cdot ft}{2} + T$ $\quad M_{r2} := F_2 \cdot x_2$ $\quad M_{r2} = 8.786\, ft \cdot kip$

$A_3 := 6.3 \cdot ft \cdot 2 \cdot ft$ $\quad F_3 := A_3 \cdot \gamma_c \cdot b$ $\quad x_3 := \frac{6.3}{2} \cdot ft + T$ $\quad M_{r3} := F_3 \cdot x_3$ $\quad M_{r3} = 7.604\, ft \cdot kip$

$A_4 := (6.2 \cdot 2) \cdot ft^2$ $\quad F_4 := A_4 \cdot \gamma_c \cdot b$ $\quad x_4 := \frac{6.2}{2} \cdot ft + T$ $\quad M_{r4} := F_4 \cdot x_4$ $\quad M_{r4} = 7.378\, ft \cdot kip$

$A_5 := 5.8 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.8}{2} \cdot ft + T$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 6.508\ ft \cdot kip$

$A_6 := 5.7 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.7}{2} \cdot ft + T$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 6.299\ ft \cdot kip$

$A_7 := 5.3 \cdot 4 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.3}{2} \cdot ft + T$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 10.992\ ft \cdot kip$

$A_8 := 0 \cdot 0 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{3.92}{2} \cdot ft + T$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 0\ ft \cdot kip$

$A_9 := 0 \cdot 0 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{2.75}{2} \cdot ft + T$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 0\ ft \cdot kip$

$A_{10} := 0 \cdot ft \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 0\ ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := 0 ft \cdot 0 \cdot ft$ $\quad F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $\quad x_{11} := 7 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 0\ ft \cdot kip$

$A_{12} := 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\ ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + cl) \cdot b$ $\quad M_{rDL} = 10.5\ ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (7 \cdot ft)$ $\quad M_{rSCH} = 5.44\ ft \cdot kip$

3. $M_{r\_Pa} := E_{avert} \cdot (6.0) \cdot ft$ $\quad M_{r\_Pa} = 26.452\ ft \cdot kip$ $\quad$ acts downward on backface at point x = H/3

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 18.078\,ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad M_{d\_Pa} = 6.837 \times 10^{4}\,ft \cdot lbf$ $\qquad M_{d\_Pa} = 68.367\,kip \cdot ft$ $\qquad M_{d\_Pa} = 68.367\,kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad M_{d3} = 28.262\,ft \cdot kip$

**DO NOT INCLUDE driving moment due to horizontal component of LL and DL in the load group, OVERRIDE with the following values:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.926 \times 10^{4}\,lbf$

$\Sigma V = 39.259\,kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO INCLUDE horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 10.633\,kip$

$\Sigma H = 10.633\,kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.191 \times 10^{5}\,ft \cdot lbf$

$\Sigma M_r = 119.114\,ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 86.445\,ft \cdot kip$

$\Sigma M_d = 86.445\,kip \cdot ft$

### Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$FS_{ot} = 1.378$

AASHTO required factor of safety against overturning is 2.0

### Factor of safety against sliding

friction at base + adhesion

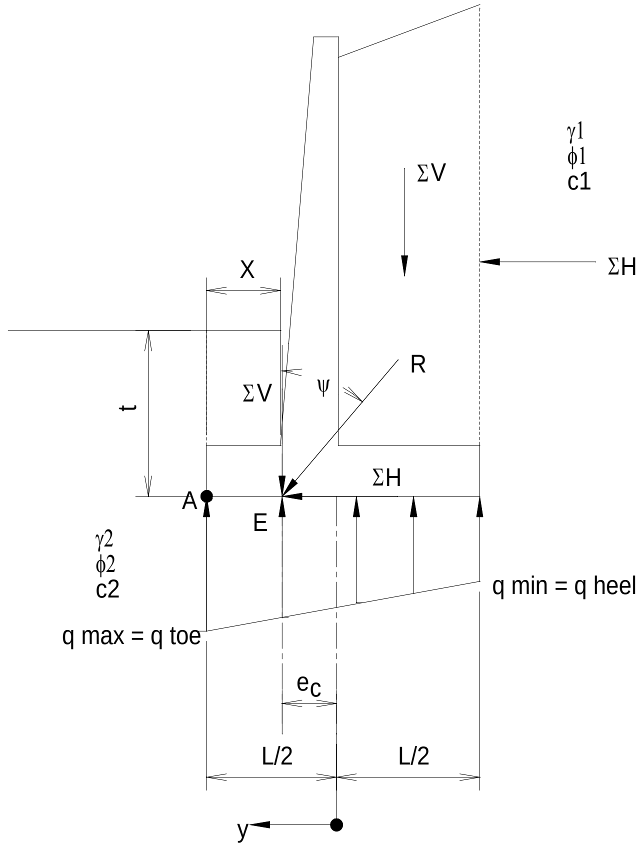
$\tan(\delta_2) = 0.445$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$FS_{sl} = 1.644$

AASHTO required factor of safety against sliding is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 3.267 \times 10^4\, lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 0.832\, ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 2.668\, ft$

$\dfrac{B}{6} = 1.167\, ft$ $\qquad$ NO GOOD !!!!!!!

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 18433\, psf$ $\qquad q_{max} = 18.433\, ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = -7216\, psf$ $\qquad q_{min} = -7.216\, ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

## Allowable Bearing Pressure:

$q_u := 24 \cdot ksf$ $\quad$ $q_{allow} := \dfrac{q_u}{3}$ $\quad$ $q_{allow} = 8\,ksf$

## Applied Bearing Pressure:

$q_a := \dfrac{\Sigma V}{B_e \cdot b}$ $\quad$ $q_a = 1.129 \times 10^3\,kPa$ $\quad$ $q_a = 24\,ksf$

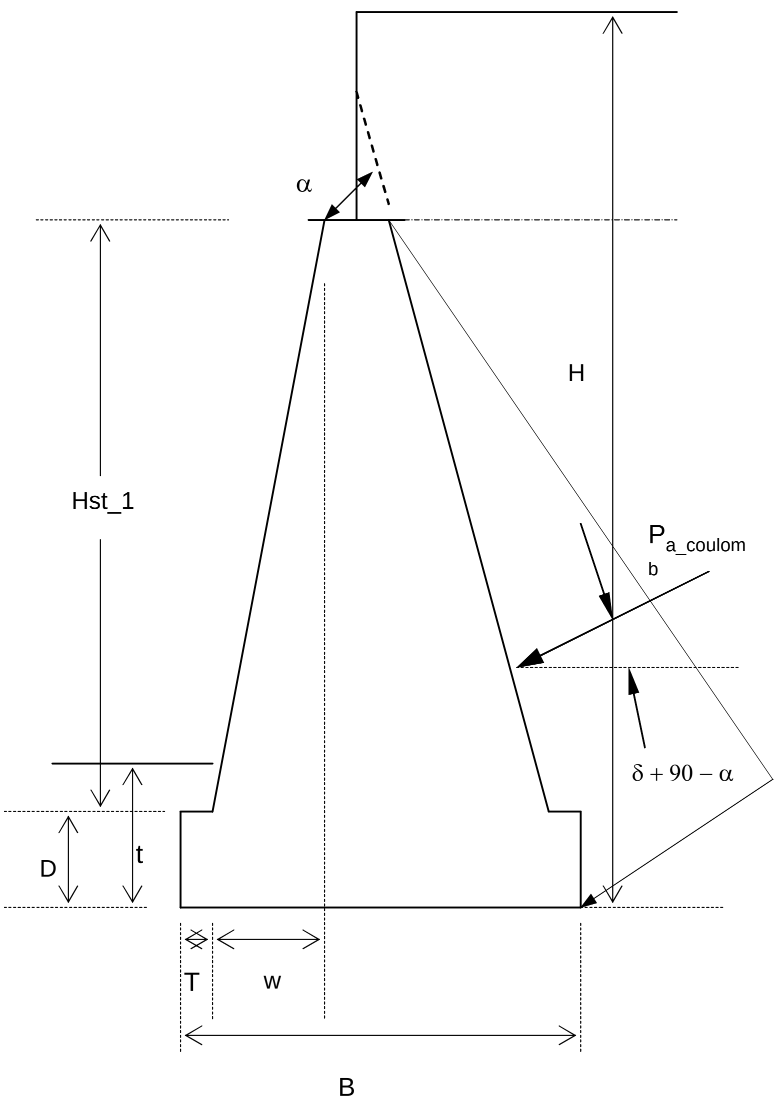
## Factor of Safety against BC failure:

$FS_{bc} := \dfrac{q_u}{q_a}$ $\quad$ $FS_{bc} = 1.017$

A factor of safety of 2 to 3 against bearing capacity failure is recommended.

**South Abutment Analysis - using field verified abutment dimensions and field-verified backfill.**
**Uses Coulomb theory. Traffic Surcharge added (Coulomb).**
**3 foot raise in profile**
**Assuming no batter.**
**Footing toe of 0.4 ft based on test pit.**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 25.69 \cdot ft$

Width of footing $B := 7. \cdot ft$

Length of toe $T := 0.4 \cdot ft$

Footing thickness $D := 3 \cdot ft$

Depth of toe $t := 0 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 20.79 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

**Assumed backfill and abutment proporties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi up to 24 degrees*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

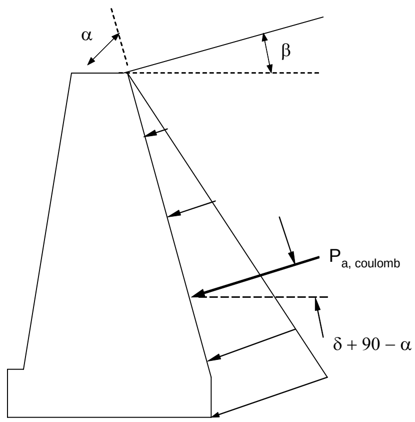
$P_{dl} := 700 \cdot \dfrac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \dfrac{kip}{ft}$

$P_{ll} := 215 \cdot \dfrac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \dfrac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.313$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 12.892\,\text{kip}$ per linear foot of abutment

**Vertical Earth Pressure:**

$E_{avert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{avert} = 5.652 \times 10^{3}\, lbf$ $\quad E_{avert} = 5.652\, kip$ per linear foot of wall

**Horizontal Earth Pressure:**

$E_{ahoriz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{ahoriz} = 11587.52\, lbf$ $\quad E_{ahoriz} = 11.588\, kip$ per lin ft of wall

**No approach slab; Force due traffic - model w/ surcharge of 2' of soil ( using Coulomb earth pressure theory)**

$s := 2 \cdot ft \cdot \gamma_1$ $\quad s = 250\, psf$

$E_s := K_{a\_coulomb} \cdot s \cdot H \cdot b$ $\quad E_s = 2.007\, kip$

**Vertical Surcharge Earth Pressure, Resultant acting at H/2:**

$E_{surch\_vert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_vert} = 0.88\, kip$ per lnr foot of wall

**Horizontal Surcharge Earth Pressure, Resultant acting at H/2:**

$E_{surch\_horiz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_horiz} = 1.804\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

**Resisting moments - abutment composed of granite stone masonry**

$A_1 := 7.0 \cdot 7.0 \cdot ft^2$ $\quad F_1 := A_1 \cdot \gamma_c \cdot b$ $\quad x_1 := \frac{7.0 \cdot ft}{2}$ $\quad M_{r1} := F_1 \cdot x_1$ $\quad M_{r1} = 29.155\, kip \cdot ft$

$A_2 := 6.8 \cdot 2 \cdot ft^2$ $\quad F_2 := A_2 \cdot \gamma_c \cdot b$ $\quad x_2 := \frac{6.8 \cdot ft}{2} + T$ $\quad M_{r2} := F_2 \cdot x_2$ $\quad M_{r2} = 8.786\, ft \cdot kip$

$A_3 := 6.3 \cdot ft \cdot 2 \cdot ft$ $\quad F_3 := A_3 \cdot \gamma_c \cdot b$ $\quad x_3 := \frac{6.3}{2} \cdot ft + T$ $\quad M_{r3} := F_3 \cdot x_3$ $\quad M_{r3} = 7.604\, ft \cdot kip$

$A_4 := (6.2 \cdot 2) \cdot ft^2$ $\quad F_4 := A_4 \cdot \gamma_c \cdot b$ $\quad x_4 := \frac{6.2}{2} \cdot ft + T$ $\quad M_{r4} := F_4 \cdot x_4$ $\quad M_{r4} = 7.378\, ft \cdot kip$

$A_5 := 5.8 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.8}{2} \cdot ft + T$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 6.508\, ft \cdot kip$

$A_6 := 5.7 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.7}{2} \cdot ft + T$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 6.299\, ft \cdot kip$

$A_7 := 5.3 \cdot 4 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.3}{2} \cdot ft$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 9.551\, ft \cdot kip$

$A_8 := 0 \cdot 0 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{3.92}{2} \cdot ft + T$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 0\, ft \cdot kip$

$A_9 := 0 \cdot 0 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{2.75}{2} \cdot ft + T$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 0\, ft \cdot kip$

$A_{10} := 0 \cdot ft \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 0\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := ft \cdot 0 \cdot ft$ $\quad F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $\quad x_{11} := 7 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 0\, ft \cdot kip$

$A_{12} := 0 \cdot 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + cl) \cdot b$ $\quad M_{rDL} = 10.5\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (7 \cdot ft)$ $\quad M_{rSCH} = 6.16\, ft \cdot kip$

3. $M_{r\_Pa} := E_{avert} \cdot (6.0) \cdot ft$ $\quad M_{r\_Pa} = 33.91\, ft \cdot kip$ $\quad$ acts downward with a moment arm at point on the backface H/3 high

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad$ $M_{d\_surch} = 23.175\,ft \cdot kip$

$M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H$ $\qquad$ $M_{d\_Pa} = 9.923 \times 10^{4}\,ft \cdot lbf$ $\qquad$ $M_{d\_Pa} = 99.228\,kip \cdot ft$ $\qquad$ $M_{d\_Pa} = 99.228\,kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad$ $M_{d3} = 28.262\,ft \cdot kip$

**DO NOT INCLUDE driving moment due to horizontal component of LL and DL in the load group, OVERRIDE with the following values:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad$ $H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 4.06 \times 10^{4}\,lbf$

$\Sigma V = 40.604\,kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO INCLUDE horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 13.392\,kip$

$\Sigma H = 13.392\,kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.258 \times 10^{5}\,ft \cdot lbf$

$\Sigma M_r = 125.849\,ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 122.403\,ft \cdot kip$

$\Sigma M_d = 122.403\,kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$$FS_{ot} = 1.028$$

AASHTO required FS is 2.0

## Factor of safety against sliding

friction at base + adhesion

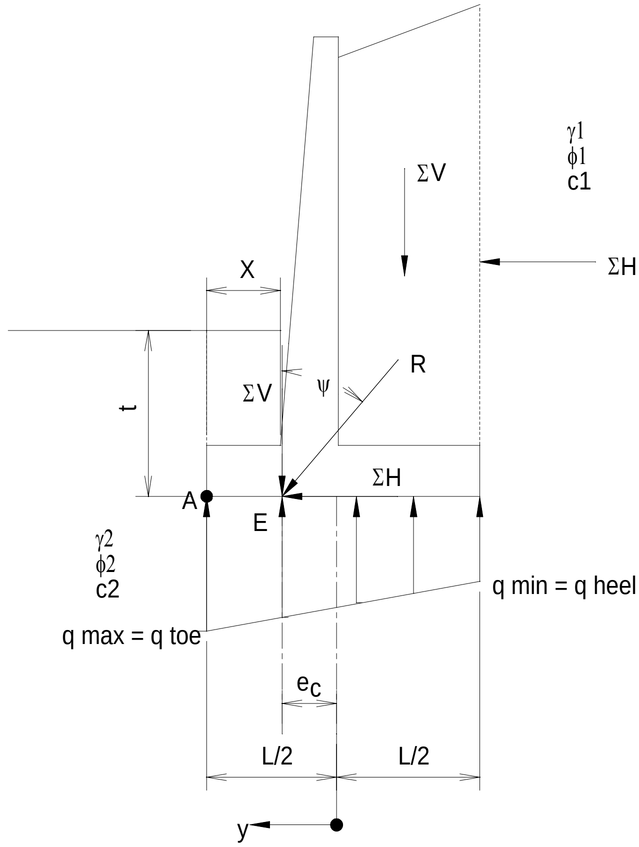
$$\tan(\delta_2) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$$FS_{sl} = 1.35$$

AASHTO required FS is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 3.446 \times 10^3\, lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 0.085\, ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 3.415\, ft$

$\dfrac{B}{6} = 1.167\, ft$ $\qquad$ NO GOOD

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 22780\, psf$ $\qquad q_{max} = 22.78\, ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = -11179\, psf$ $\qquad q_{min} = -11.179\, ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

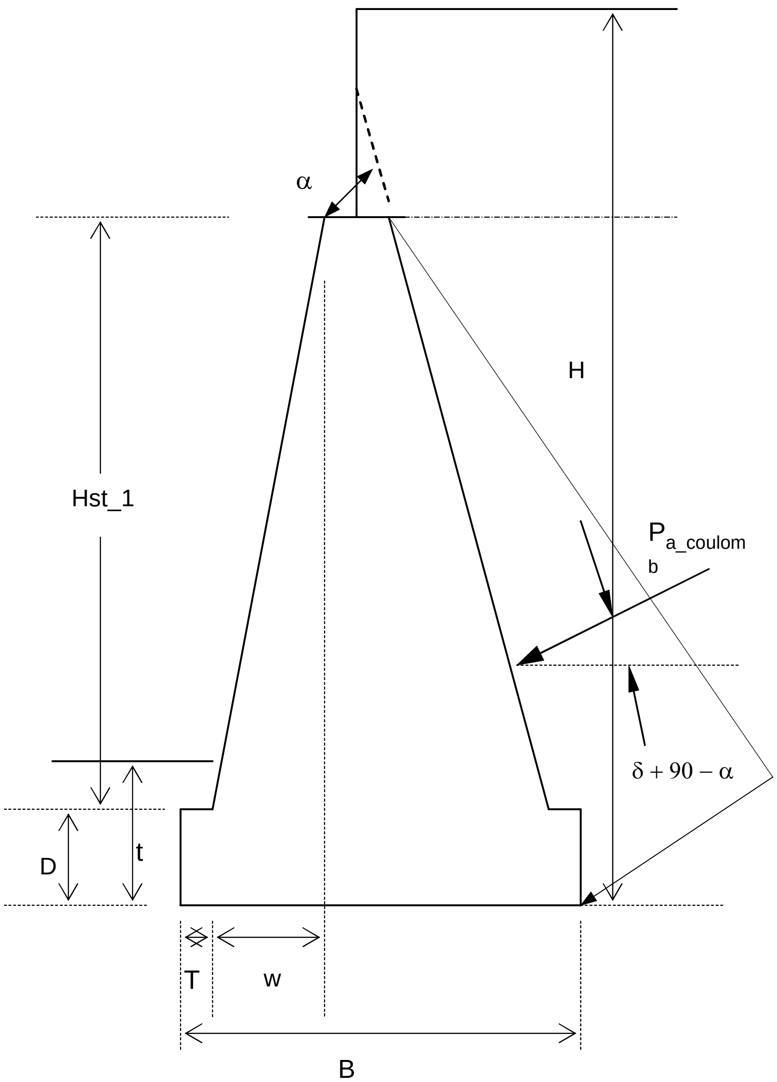
**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \dfrac{q_u}{3}$ $\qquad q_{allow} = 8\, ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\qquad FS_{bc} = 1.054$

AASHTO recommends a FS of 3

**South Abutment Analysis - using field verified abutment dimensions and field-verified backfill.**
**Uses Coulomb theory. Assume approach slab is added, so ignore Traffic Surcharge.**
**3 foot raise in profile**
**Assuming no batter.**
**Footing toe of 0.4 ft based on test pit.**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




| | |
|---|---|
| Height of wall, | $H := 25.69 \cdot ft$ |
| Width of footing | $B := 7. \cdot ft$ |
| Length of toe | $T := 0.4 \cdot ft$ |
| Footing thickness | $D := 3 \cdot ft$ |
| Depth of toe | $t := 0 \cdot ft$ |
| Depth of seat | $d_s := 1.9 \cdot ft$ |
| Height of stem 1 | $H_{st\_1} := 20.79 \cdot ft$ |
| Unit width | $b := 1 \cdot ft$ |
| centerline distance | $cl := 6 \cdot in$ |

**Assumed backfill and abutment proporties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi to maximum of 24 degrees*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

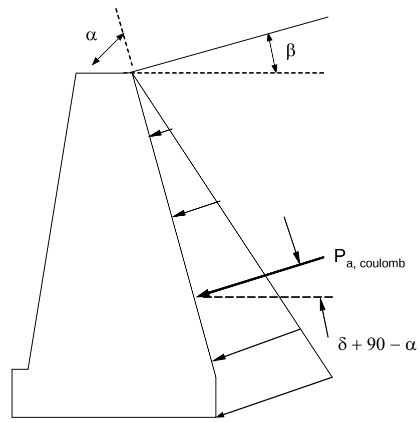
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

~~Coulomb Ka for granular backfill is very similar to the Rankine Value~~

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.313$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 12.892\ \text{kip}$

per linear foot of abutment

**Vertical Earth Pressure:**

$E_{avert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{avert} = 5.652 \times 10^{3}\,lbf$ $\quad E_{avert} = 5.652\,kip$ per linear foot of wall

**Horizontal Earth Pressure:**

$E_{ahoriz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{ahoriz} = 11587.52\,lbf$ $\quad E_{ahoriz} = 11.588\,kip$ per lin ft of wall

**Assume approach slab is added; neglect Force due traffic - ~~model w/ surcharge of 2' of soil (using Coulomb earth pressure theory)~~**

$s := 0 \cdot ft \cdot \gamma_1$ $\quad s = 0\,psf$

$E_s := K_{a\_coulomb} \cdot s \cdot H \cdot b$ $\quad E_s = 0\,kip$

**Vertical Surcharge Earth Pressure, Resultant acting at H/2:**

$E_{surch\_vert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_vert} = 0\,kip$ per lnr foot of wall

**Horizontal Surcharge Earth Pressure, Resultant acting at H/2:**

$E_{surch\_horiz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_horiz} = 0\,kip$ per lin ft of wall

## Factor of safety against overturning and sliding

**Resisting moments - abutment composed of granite stone masonry**

$A_1 := 7.0 \cdot 7.0 \cdot ft^2$ $\quad F_1 := A_1 \cdot \gamma_c \cdot b$ $\quad x_1 := \dfrac{7.0 \cdot ft}{2}$ $\quad M_{r1} := F_1 \cdot x_1$ $\quad M_{r1} = 29.155\,kip \cdot ft$

$A_2 := 6.8 \cdot 2 \cdot ft^2$ $\quad F_2 := A_2 \cdot \gamma_c \cdot b$ $\quad x_2 := \dfrac{6.8 \cdot ft}{2} + T$ $\quad M_{r2} := F_2 \cdot x_2$ $\quad M_{r2} = 8.786\,ft \cdot kip$

$A_3 := 6.3 \cdot ft \cdot 2 \cdot ft$ $\quad F_3 := A_3 \cdot \gamma_c \cdot b$ $\quad x_3 := \dfrac{6.3}{2} \cdot ft + T$ $\quad M_{r3} := F_3 \cdot x_3$ $\quad M_{r3} = 7.604\,ft \cdot kip$

$A_4 := (6.2 \cdot 2) \cdot ft^2$ $\quad F_4 := A_4 \cdot \gamma_c \cdot b$ $\quad x_4 := \dfrac{6.2}{2} \cdot ft + T$ $\quad M_{r4} := F_4 \cdot x_4$ $\quad M_{r4} = 7.378\,ft \cdot kip$

$A_5 := 5.8 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.8}{2} \cdot ft + T$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 6.508\ ft \cdot kip$

$A_6 := 5.7 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.7}{2} \cdot ft + T$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 6.299\ ft \cdot kip$

$A_7 := 5.3 \cdot 4 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.3}{2} \cdot ft$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 9.551\ ft \cdot kip$

$A_8 := 0 \cdot 0 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{3.92}{2} \cdot ft + T$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 0\ ft \cdot kip$

$A_9 := 0 \cdot 0 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{2.75}{2} \cdot ft + T$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 0\ ft \cdot kip$

$A_{10} := ft \cdot 0 \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 0\ ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := 0 \cdot ft \cdot ft$ $\quad F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $\quad x_{11} := 7 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 0\ ft \cdot kip$

$A_{12} := 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\ ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + cl) \cdot b$ $\quad M_{rDL} = 10.5\ ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (6 \cdot ft)$ $\quad M_{rSCH} = 0\ ft \cdot kip$ $\quad$ acts at H/2 $\quad \frac{H}{2} = 12.845\ ft$

3. $M_{r\_Pa} := E_{avert} \cdot (6) \cdot ft$ $\quad M_{r\_Pa} = 33.91\ ft \cdot kip$ $\quad$ acts at H/3 $\quad \frac{H}{3} = 8.563\ ft$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 0\ \text{ft} \cdot \text{kip}$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad M_{d\_Pa} = 9.923 \times 10^{4}\ \text{ft} \cdot \text{lbf}$ $\qquad M_{d\_Pa} = 99.228\ \text{kip} \cdot \text{ft}$ $\qquad M_{d\_Pa} = 99.228\ \text{kip} \cdot \text{ft}$

$M_{d3} := H_{ss} \cdot 21 \cdot \text{ft}$ $\qquad M_{d3} = 28.262\ \text{ft} \cdot \text{kip}$

**Do not include driving moment due to horizontal component of LL and DL in the load group, OVERRIDE with the following values:**

$M_{d3} := 0 \cdot \text{ft} \cdot \text{kip}$ $\qquad H_{ss} := 0 \cdot \text{kip}$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.972 \times 10^{4}\ \text{lbf}$

$\Sigma V = 39.724\ \text{kip}$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**Do not include horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 11.588\ \text{kip}$

$\Sigma H = 11.588\ \text{kip}$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.197 \times 10^{5}\ \text{ft} \cdot \text{lbf}$

$\Sigma M_r = 119.689\ \text{ft} \cdot \text{kip}$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 99.228\ \text{ft} \cdot \text{kip}$

$\Sigma M_d = 99.228\ \text{kip} \cdot \text{ft}$

**Factor of safety against overturning compared to AASHTO required FS of 2.0**

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$$FS_{ot} = 1.206$$

AASHTO requirements specify a factor of safety of 2.0 against overturning

**Factor of safety against sliding - compared to AASHTO required FS of 1.5**

friction at base + adhesion

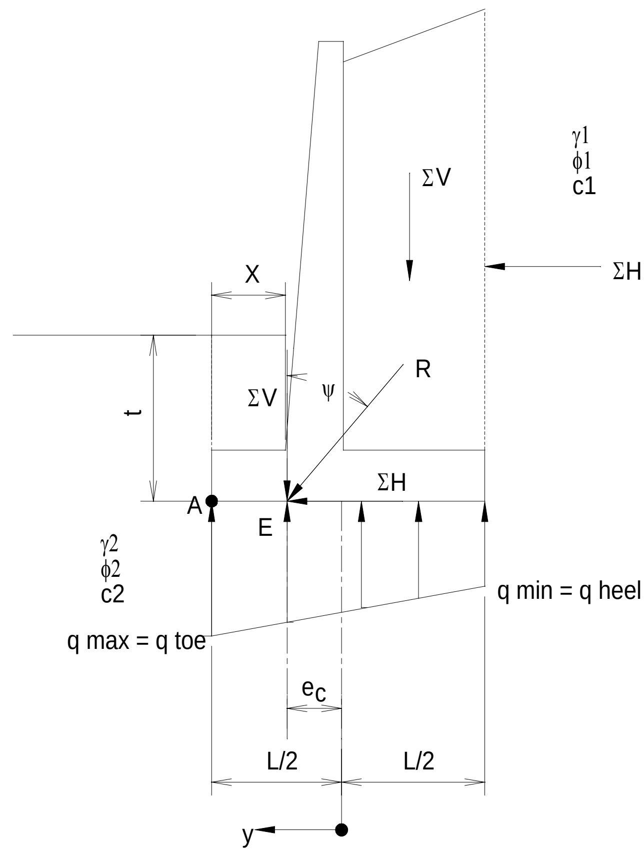
$$\tan(\delta_2) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$$FS_{sl} = 1.526$$

AASHTO requirements specify a factor of safety of 2.0 against overturning

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 2.046 \times 10^4\, lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 0.515\, ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 2.985\, ft$

$\dfrac{B}{6} = 1.167\, ft$ NO GOOD

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 20194\, psf$ $\qquad q_{max} = 20.194\, ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = -8844\, psf$ $\qquad q_{min} = -8.844\, ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \dfrac{q_u}{3}$ $\qquad q_{allow} = 8\, ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\qquad FS_{bc} = 1.188$

AASHTO recommends a FS of 3

**NORTH ABUTMENT**

**STONE MASONRY**
**ABUTMENT SECTION**

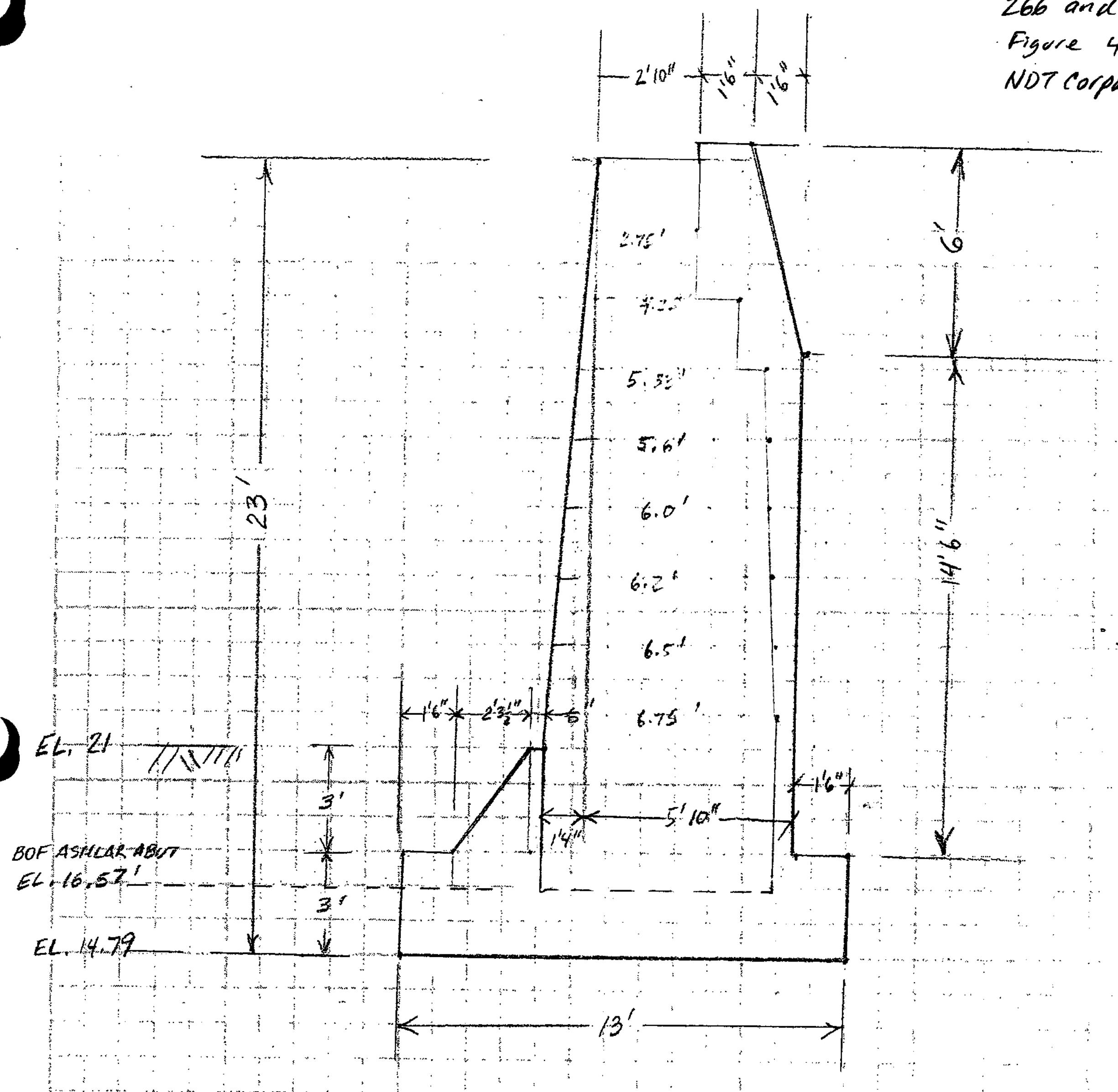
## STABILITY ANALYSES

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 10158.00 FILE NO. ______ OF ______ ICM 12/04

FINAL CHK. BY ______ DATE ______ LOCATION PORTLAND SH. NO. ______ OF ______

ITEM NO. ______ SUBJECT NORTH (FALMOUTH) ABUTMENT

GRANITE BLOCK ABUTMENT DIMENSIONS
Based on averaging GPR Scan Lines 266 and 267 per Figure 4 of NDT Corporation Report



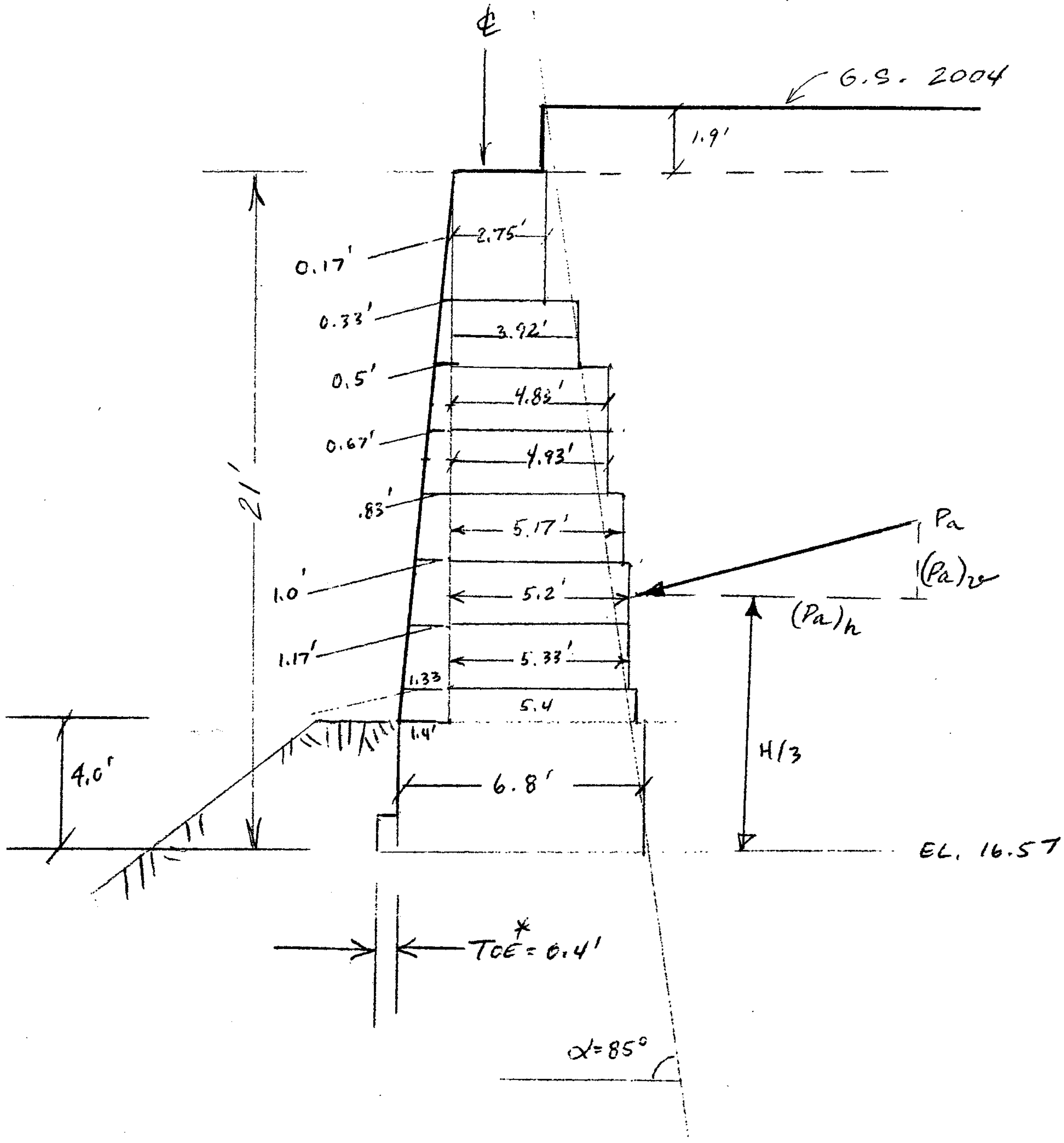

ASHLAR STONE ABUT ≈ 39'0" LONG

KW 12/04

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 10158.00 FILE NO. ____ OF ____
FINAL CHK. BY ____ DATE ____ LOCATION PORTLAND SH. NO. ____ OF ____
ITEM NO. ____ SUBJECT NORTH ABUTMENT.

STONE MASONRY ABUTMENT GEOMETRY BASED ON AVERAGE OF GPR SCAN LINES 266 AND 267.

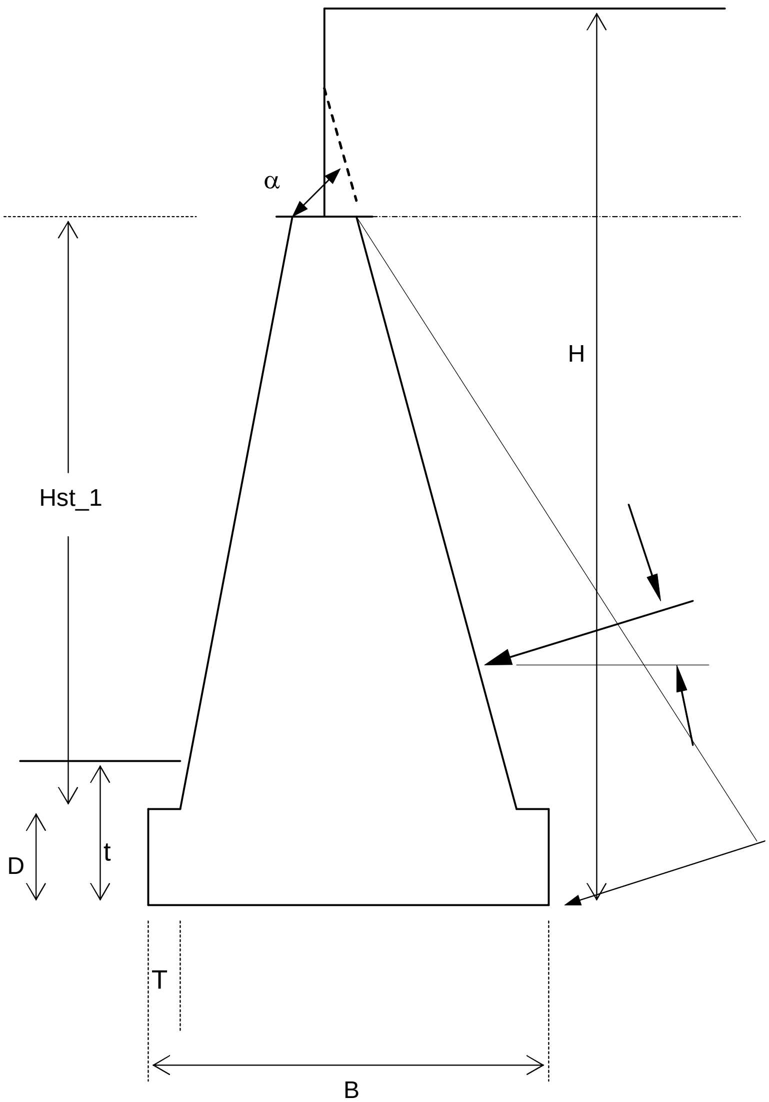
SIMPLIFIED GEOMETRY TO ACCOUNT FOR BACK BATTER OF FACE (1:12 FACE BATTER)




* TOE OF FOOTING AND DEPTH OF EMBEDMENT PER TEST PIT TP-PRR-101.

**North (Falmouth) Abutment Analysis**
**Using field verified abutment dimensions and field-verified backfill.**
**Uses Coulomb theory. Traffic Surcharge added (Coulomb).**
**Existing profile conditions**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 22.9 \cdot ft$

Width of footing $B := 7.2 \cdot ft$

Length of toe $T := 0.4 \cdot ft$

$w := 1.4 \cdot ft$

Footing thickness $D := 4.0 \cdot ft$

Depth of toe $t := 4 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 21 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

**Assumed backfill and abutment properties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot \mathrm{deg}$

Coulomb wall friction $\delta := 21 \cdot \mathrm{deg}$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot \mathrm{deg}$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot \mathrm{deg}$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot \mathrm{pcf}$ $\phi_2 := 32 \cdot \mathrm{deg}$ $c_2 := 0 \cdot \mathrm{psf}$

concrete - sand friction angle $\delta_2 := 24 \cdot \mathrm{deg}$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

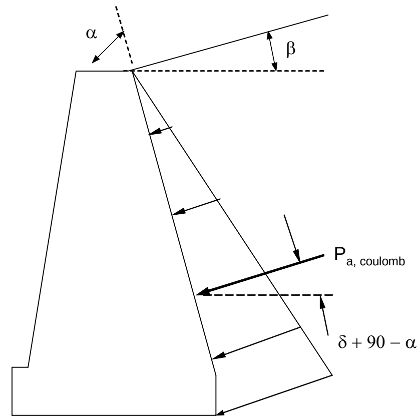
$P_{dl} := 700 \cdot \dfrac{\mathrm{kip}}{60 \cdot \mathrm{ft}}$ $P_{dl} = 11.667 \dfrac{\mathrm{kip}}{\mathrm{ft}}$

$P_{ll} := 215 \cdot \dfrac{\mathrm{kip}}{60 \cdot \mathrm{ft}}$ $P_{ll} = 3.583 \dfrac{\mathrm{kip}}{\mathrm{ft}}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4\,\mathrm{lbf}$ $V_{ss} = 15.25\,\mathrm{kip}$

$H_{ss} := \left[(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})\right] \cdot b$ $H_{ss} = 1.346 \times 10^3\,\mathrm{lbf}$ $H_{ss} = 1.346\,\mathrm{kip}$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.313$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 10.244\,\text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{avert} = 4.491 \times 10^{3}\, lbf$ $\quad E_{avert} = 4.491\, kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{ahoriz} = 9207.321\, lbf$ $\quad E_{ahoriz} = 9.207\, kip$ per lin ft of wall

### No approach slab; Force due traffic - model w/ surcharge of 2' of soil (using Coulomb earth pressure theory)

$s := 2\cdot ft\cdot \gamma_1$ $\quad s = 250\, psf$

$E_s := K_{a\_coulomb}\cdot s\cdot H\cdot b$ $\quad E_s = 1.789\, kip$

**Vertical Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:**

$E_{surch\_vert} := \sin(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_vert} = 0.784\, kip$ per lnr foot of wall

**Horizontal Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:**

$E_{surch\_horiz} := \cos(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{ahoriz} = 9.207\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 6.8\cdot 4\cdot ft^2$ $\quad F_1 := A_1\cdot \gamma_c\cdot b$ $\quad x_1 := \frac{6.8\cdot ft}{2} + T$ $\quad M_{r1} := F_1\cdot x_1$ $\quad M_{r1} = 17.571\, kip\cdot ft$

$A_2 := 5.4\cdot 1\cdot ft^2$ $\quad F_2 := A_2\cdot \gamma_c\cdot b$ $\quad x_2 := \frac{5.4\cdot ft}{2} + T + w$ $\quad M_{r2} := F_2\cdot x_2$ $\quad M_{r2} = 4.131\, ft\cdot kip$

$A_3 := 5.33\cdot ft\cdot 2\cdot ft$ $\quad F_3 := A_3\cdot \gamma_c\cdot b$ $\quad x_3 := \frac{5.33}{2}\cdot ft + T + w$ $\quad M_{r3} := F_3\cdot x_3$ $\quad M_{r3} = 8.091\, ft\cdot kip$

$A_4 := (5.2\cdot 2)\cdot ft^2$ $\quad F_4 := A_4\cdot \gamma_c\cdot b$ $\quad x_4 := \frac{5.2}{2}\cdot ft + T + w$ $\quad M_{r4} := F_4\cdot x_4$ $\quad M_{r4} = 7.779\, ft\cdot kip$

$A_5 := 5.17 \cdot 2 \cdot ft^2$ $F_5 := A_5 \cdot \gamma_c \cdot b$ $x_5 := \frac{5.17}{2} \cdot ft + T + w$ $M_{r5} := F_5 \cdot x_5$ $M_{r5} = 7.708\, ft \cdot kip$

$A_6 := 4.39 \cdot 2 \cdot ft^2$ $F_6 := A_6 \cdot \gamma_c \cdot b$ $x_6 := \frac{4.93}{2} \cdot ft + T + w$ $M_{r6} := F_6 \cdot x_6$ $M_{r6} = 6.366\, ft \cdot kip$

$A_7 := 4.83 \cdot 2 \cdot ft^2$ $F_7 := A_7 \cdot \gamma_c \cdot b$ $x_7 := \frac{4.83}{2} \cdot ft + T + w$ $M_{r7} := F_7 \cdot x_7$ $M_{r7} = 6.922\, ft \cdot kip$

$A_8 := 3.92 \cdot 2 \cdot ft^2$ $F_8 := A_8 \cdot \gamma_c \cdot b$ $x_8 := \frac{3.92}{2} \cdot ft + T + w$ $M_{r8} := F_8 \cdot x_8$ $M_{r8} = 5.011\, ft \cdot kip$

$A_9 := 2.75 \cdot 4 \cdot ft^2$ $F_9 := A_9 \cdot \gamma_c \cdot b$ $x_9 := \frac{2.75}{2} \cdot ft + T + w$ $M_{r9} := F_9 \cdot x_9$ $M_{r9} = 5.937\, ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $F_{10} := A_{10} \cdot \gamma_c \cdot b$ $x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + w$ $M_{r10} := F_{10} \cdot x_{10}$ $M_{r10} = 4.72\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := 0 \cdot ft \cdot 0 \cdot ft$ $F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $x_{11} := 7 \cdot ft$ $M_{r11} := F_{11} \cdot x_{11} \cdot 0$ $M_{r11} = 0\, ft \cdot kip$

$A_{12} := 0 \cdot ft \cdot ft$ $F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $x_{12} := 4.5 \cdot ft$ $M_{r12} := F_{12} \cdot x_{12} \cdot 0$ $M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (1.4 \cdot ft + cl) \cdot b$ $M_{rDL} = 22.167\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (5 \cdot ft)$ $M_{rSCH} = 3.922\, ft \cdot kip$

3. $M_{r\_Pa} := E_{avert} \cdot (5.5 \cdot ft)$ $M_{r\_Pa} = 24.699\, ft \cdot kip$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 18.415\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad M_{d\_Pa} = 7.028 \times 10^4\ ft \cdot lbf$ $\qquad M_{d\_Pa} = 70.283\ kip \cdot ft$ $\qquad M_{d\_Pa} = 70.283\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad M_{d3} = 28.262\ ft \cdot kip$

**DO NOT INCLUDE horizontal component of LL and DL in the load group as a driving moment nor a horizontal force, Override with these values:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.618 \times 10^4\ lbf$

$\Sigma V = 36.182\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO NOT INCLUDE horizontal component of LL and DL in the load group (Hss) or Sum of driving moments**

$\Sigma H = 10.816\ kip$

$\Sigma H = 10.816\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.25 \times 10^5\ ft \cdot lbf$

$\Sigma M_r = 125.025\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 88.697\ ft \cdot kip$

$\Sigma M_d = 88.697\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$FS_{ot} = 1.41$

AASHTO required minimum FS = 2.0

## Factor of safety against sliding

friction at base + adhesion

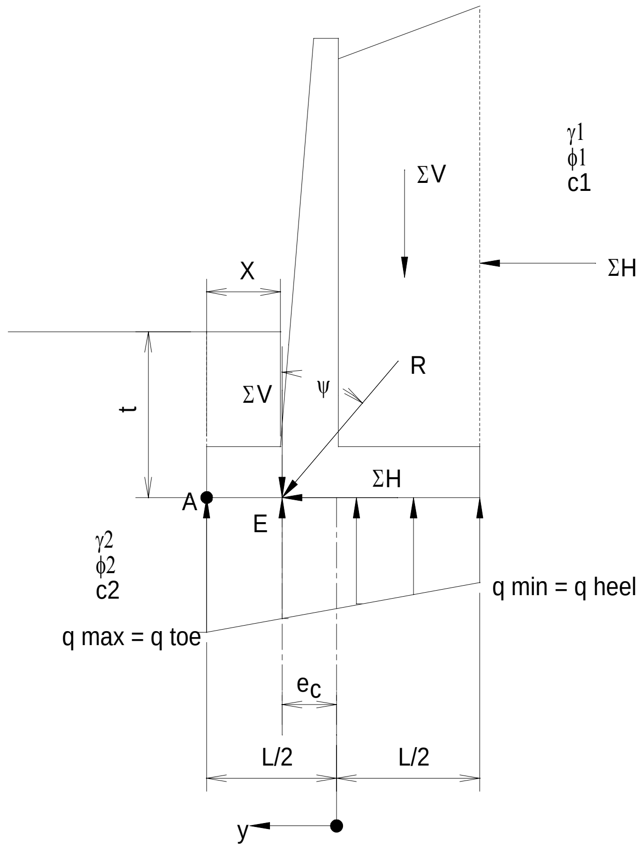
$\tan(\delta_2) = 0.445$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$FS_{sl} = 1.489$

AASHTO required minimum FS = 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\quad M_{net} = 3.633 \times 10^4 \, lbf \cdot ft$

### location of resultant

$AE := \frac{M_{net}}{\Sigma V}$ $\quad AE = 1.004\,ft$ $\quad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \frac{B}{2} - AE$ $\quad e_c = 2.596\,ft$

$\frac{B}{6} = 1.2\,ft$ $\quad$ NOT GOOD

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\frac{\Sigma V}{B} \cdot \left(1 + \frac{6 \cdot e_c}{B}\right)\right] \cdot \frac{1}{b}$ $\quad q_{max} = 15897\,psf$ $\quad q_{max} = 15.897\,ksf$ $\quad q_{toe} := q_{max}$

$q_{min} := \left[\frac{\Sigma V}{B} \cdot \left(1 - \frac{6 \cdot e_c}{B}\right)\right] \cdot \frac{1}{b}$ $\quad q_{min} = -5846\,psf$ $\quad q_{min} = -5.846\,ksf$ $\quad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

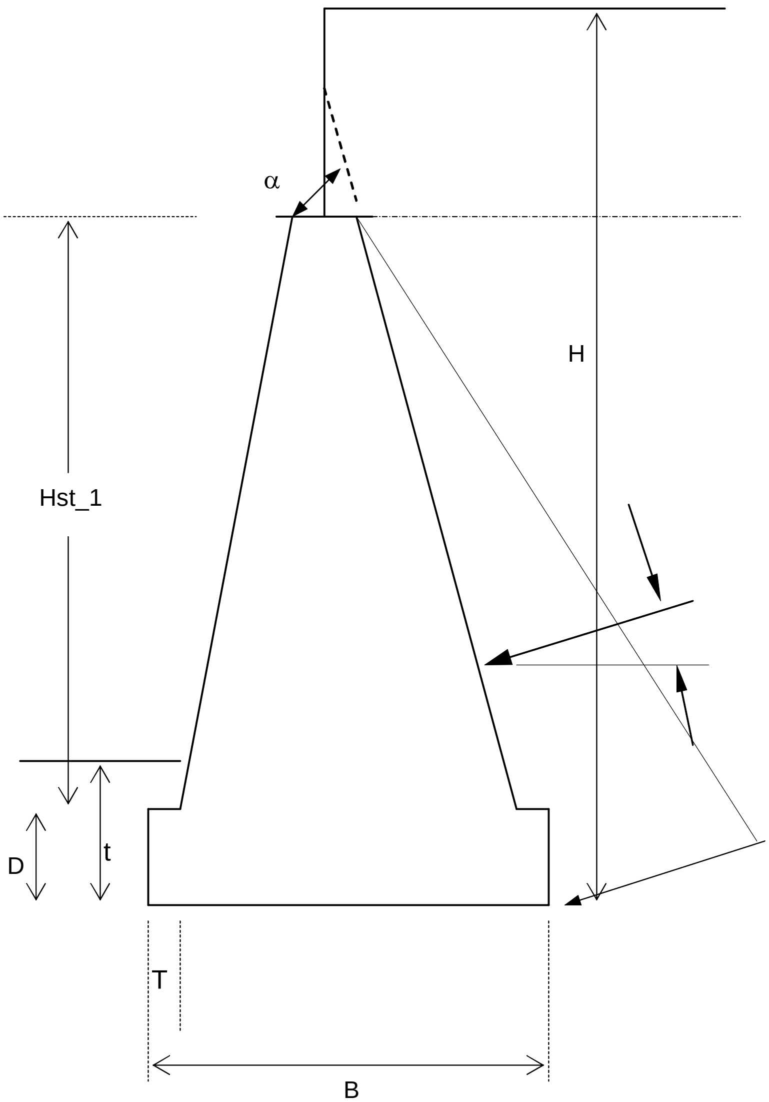
**Allowable Bearing Pressure:** $\quad q_u := 24 \cdot ksf$ $\quad q_{allow} := \frac{q_u}{3}$ $\quad q_{allow} = 8\,ksf$

**Factor of Safety against BC failure:** $\quad FS_{bc} := \frac{q_u}{q_{max}}$ $\quad FS_{bc} = 1.51$

AASHTO recommends a FS of 3

**North (Falmouth) Abutment Analysis**
**Using field verified abutment dimensions and field-verified backfill.**
**Uses Coulomb theory. Traffic Surcharge added (Coulomb).**
**Assuming proposed profile raise by 3 ft**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 25.9 \cdot ft$

Width of footing $B := 7.2 \cdot ft$

Length of toe $T := 0.4 \cdot ft$

$w := 1.4 \cdot ft$

Footing thickness $D := 4.0 \cdot ft$

Depth of toe $t := 4 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 21 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

**Assumed backfill and abutment properties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

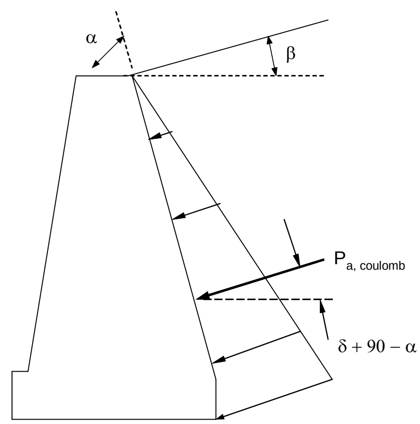
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin\left(\phi_1 + \alpha\right)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin\left(\phi_1 + \delta\right) \cdot \sin\left(\phi_1 - \beta\right)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.313$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 13.104\,\text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\qquad E_{avert} = 5.744 \times 10^{3}\, lbf$ $\qquad E_{avert} = 5.744\, kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\qquad E_{ahoriz} = 11777.736\, lbf$ $\qquad E_{ahoriz} = 11.778\, kip$ per lin ft of wall

### No approach slab; Force due traffic - model w/ surcharge of 2' of soil ( using Coulomb earth pressure theory)

$s := 2 \cdot ft \cdot \gamma_1$ $\qquad s = 250\, psf$

$E_s := K_{a\_coulomb} \cdot s \cdot H \cdot b$ $\qquad E_s = 2.024\, kip$

### Vertical Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:

$E_{surch\_vert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\qquad E_{surch\_vert} = 0.887\, kip$ per lnr foot of wall

### Horizontal Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:

$E_{surch\_horiz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\qquad E_{ahoriz} = 11.778\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 6.8 \cdot 4 \cdot ft^2$ $\qquad F_1 := A_1 \cdot \gamma_c \cdot b$ $\qquad x_1 := \frac{6.8 \cdot ft}{2} + T$ $\qquad M_{r1} := F_1 \cdot x_1$ $\qquad M_{r1} = 17.571\, kip \cdot ft$

$A_2 := 5.4 \cdot 1 \cdot ft^2$ $\qquad F_2 := A_2 \cdot \gamma_c \cdot b$ $\qquad x_2 := \frac{5.4 \cdot ft}{2} + T + w$ $\qquad M_{r2} := F_2 \cdot x_2$ $\qquad M_{r2} = 4.131\, ft \cdot kip$

$A_3 := 5.33 \cdot ft \cdot 2 \cdot ft$ $\qquad F_3 := A_3 \cdot \gamma_c \cdot b$ $\qquad x_3 := \frac{5.33}{2} \cdot ft + T + w$ $\qquad M_{r3} := F_3 \cdot x_3$ $\qquad M_{r3} = 8.091\, ft \cdot kip$

$A_4 := (5.2 \cdot 2) \cdot ft^2$ $\qquad F_4 := A_4 \cdot \gamma_c \cdot b$ $\qquad x_4 := \frac{5.2}{2} \cdot ft + T + w$ $\qquad M_{r4} := F_4 \cdot x_4$ $\qquad M_{r4} = 7.779\, ft \cdot kip$

$A_5 := 5.17 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.17}{2} \cdot ft + T + w$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 7.708\ ft \cdot kip$

$A_6 := 4.39 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{4.93}{2} \cdot ft + T + w$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 6.366\ ft \cdot kip$

$A_7 := 4.83 \cdot 2 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{4.83}{2} \cdot ft + T + w$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 6.922\ ft \cdot kip$

$A_8 := 3.92 \cdot 2 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{3.92}{2} \cdot ft + T + w$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 5.011\ ft \cdot kip$

$A_9 := 2.75 \cdot 4 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{2.75}{2} \cdot ft + T + w$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 5.937\ ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + w$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 4.72\ ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := 0 \cdot ft \cdot ft$ $\quad F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $\quad x_{11} := 7 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11} \cdot 0$ $\quad M_{r11} = 0\ ft \cdot kip$

$A_{12} := 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12} \cdot 0$ $\quad M_{r12} = 0\ ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (1.4 \cdot ft + cl) \cdot b$ $\quad M_{rDL} = 22.167\ ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (5 \cdot ft)$ $\quad M_{rSCH} = 4.436\ ft \cdot kip$

3. $M_{r\_Pa} := E_{avert} \cdot (5.5 \cdot ft)$ $\quad M_{r\_Pa} = 31.594\ ft \cdot kip$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\quad$ $M_{d\_surch} = 23.555\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\quad$ $M_{d\_Pa} = 1.017 \times 10^{5}\ ft \cdot lbf$ $\quad$ $M_{d\_Pa} = 101.681\ kip \cdot ft$ $\quad$ $M_{d\_Pa} = 101.681\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\quad$ $M_{d3} = 28.262\ ft \cdot kip$

**DO NOT INCLUDE horizontal component of LL and DL in the load group as a driving moment nor a horizontal force, Override with these values:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\quad$ $H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.754 \times 10^{4}\ lbf$

$\Sigma V = 37.539\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO NOT INCLUDE horizontal component of LL and DL in the load group (Hss) or Sum of driving moments**

$\Sigma H = 13.597\ kip$

$\Sigma H = 13.597\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.324 \times 10^{5}\ ft \cdot lbf$

$\Sigma M_r = 132.434\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 125.237\ ft \cdot kip$

$\Sigma M_d = 125.237\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$$FS_{ot} = 1.057$$

compared to AASHTO required factor of safety of 2.0

## Factor of safety against sliding

friction at base + adhesion

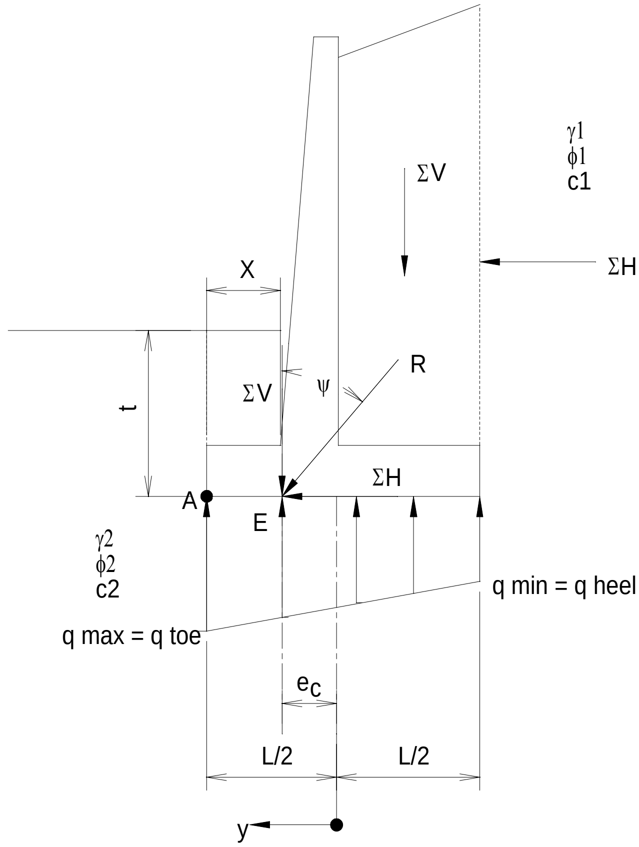
$$\tan(\delta_2) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$$FS_{sl} = 1.229$$

compared to AASHTO required factor of safety of 2.0

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 7.198 \times 10^3\ lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 0.192\ ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 3.408\ ft$

$\dfrac{B}{6} = 1.2\ ft$ $\qquad$ NOT GOOD

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 20022\ psf$ $\qquad q_{max} = 20.022\ ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = -9594\ psf$ $\qquad q_{min} = -9.594\ ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \dfrac{q_u}{3}$ $\qquad q_{allow} = 8\ ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\qquad FS_{bc} = 1.199$

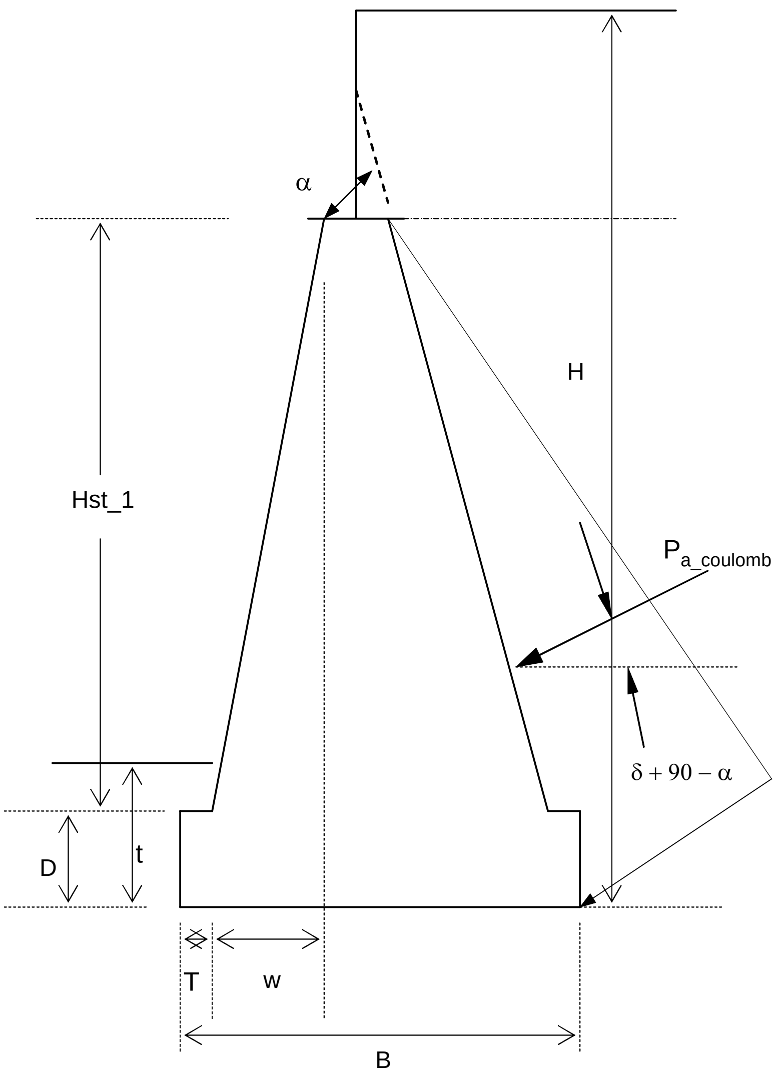
AASHTO recommends a FS of 3

**North (Falmouth) Abutment Analysis. Using field verified abutment dimensions and field-verified backfill. Uses Coulomb theory.**

**Assume approach slab is added, so removed Traffic Surcharge.**

**Assuming proposed profile raise by 3 ft**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 25.9 \cdot ft$

Width of footing $B := 7.2 \cdot ft$

Length of toe $T := 0.4 \cdot ft$

$w := 1.4 \cdot ft$

Footing thickness $D := 4.0 \cdot ft$

Depth of toe $t := 4 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 21 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

**Assumed backfill and abutment properties**

granite unit weight $\gamma_c := 170 \cdot pcf$ $\gamma_c = 26.705 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 85 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

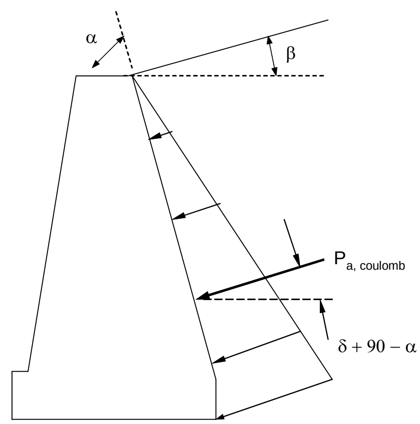
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.313$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 13.104\,\text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{avert} = 5.744 \times 10^{3}\,lbf$ $\quad E_{avert} = 5.744\,kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{ahoriz} = 11777.736\,lbf$ $\quad E_{ahoriz} = 11.778\,kip$ per lin ft of wall

### Assume approach slab; so ignore Force due traffic - model w/ surcharge of 2' of soil ( using Coulomb earth pressure theory)

$s := 0\cdot ft\cdot\gamma_1 \quad s = 0\,psf$

$E_s := K_{a\_coulomb}\cdot s\cdot H\cdot b \quad E_s = 0\,kip$

### Vertical Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:

$E_{surch\_vert} := \sin(\delta + 90\cdot deg - \alpha)\cdot E_s \quad E_{surch\_vert} = 0\,kip$ per lnr foot of wall

### Horizontal Surcharge Earth Pressure, Resultant acting at H/2 on backface of wall:

$E_{surch\_horiz} := \cos(\delta + 90\cdot deg - \alpha)\cdot E_s \quad E_{surch\_horiz} = 0\,kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 6.8\cdot 4\cdot ft^2 \quad F_1 := A_1\cdot\gamma_c\cdot b \quad x_1 := \frac{6.8\cdot ft}{2} + T \quad M_{r1} := F_1\cdot x_1 \quad M_{r1} = 17.571\,kip\cdot ft$

$A_2 := 5.4\cdot 1\cdot ft^2 \quad F_2 := A_2\cdot\gamma_c\cdot b \quad x_2 := \frac{5.4\cdot ft}{2} + T + w \quad M_{r2} := F_2\cdot x_2 \quad M_{r2} = 4.131\,ft\cdot kip$

$A_3 := 5.33\cdot ft\cdot 2\cdot ft \quad F_3 := A_3\cdot\gamma_c\cdot b \quad x_3 := \frac{5.33}{2}\cdot ft + T + w \quad M_{r3} := F_3\cdot x_3 \quad M_{r3} = 8.091\,ft\cdot kip$

$A_4 := (5.2\cdot 2)\cdot ft^2 \quad F_4 := A_4\cdot\gamma_c\cdot b \quad x_4 := \frac{5.2}{2}\cdot ft + T + w \quad M_{r4} := F_4\cdot x_4 \quad M_{r4} = 7.779\,ft\cdot kip$

$A_5 := 5.17 \cdot 2 \cdot ft^2$ $F_5 := A_5 \cdot \gamma_c \cdot b$ $x_5 := \frac{5.17}{2} \cdot ft + T + w$ $M_{r5} := F_5 \cdot x_5$ $M_{r5} = 7.708\, ft \cdot kip$

$A_6 := 4.39 \cdot 2 \cdot ft^2$ $F_6 := A_6 \cdot \gamma_c \cdot b$ $x_6 := \frac{4.93}{2} \cdot ft + T + w$ $M_{r6} := F_6 \cdot x_6$ $M_{r6} = 6.366\, ft \cdot kip$

$A_7 := 4.83 \cdot 2 \cdot ft^2$ $F_7 := A_7 \cdot \gamma_c \cdot b$ $x_7 := \frac{4.83}{2} \cdot ft + T + w$ $M_{r7} := F_7 \cdot x_7$ $M_{r7} = 6.922\, ft \cdot kip$

$A_8 := 3.92 \cdot 2 \cdot ft^2$ $F_8 := A_8 \cdot \gamma_c \cdot b$ $x_8 := \frac{3.92}{2} \cdot ft + T + w$ $M_{r8} := F_8 \cdot x_8$ $M_{r8} = 5.011\, ft \cdot kip$

$A_9 := 2.75 \cdot 4 \cdot ft^2$ $F_9 := A_9 \cdot \gamma_c \cdot b$ $x_9 := \frac{2.75}{2} \cdot ft + T + w$ $M_{r9} := F_9 \cdot x_9$ $M_{r9} = 5.937\, ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $F_{10} := A_{10} \cdot \gamma_c \cdot b$ $x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + w$ $M_{r10} := F_{10} \cdot x_{10}$ $M_{r10} = 4.72\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{11} := \frac{1}{2} \cdot 0 \cdot ft \cdot ft$ $F_{11} := A_{11} \cdot \gamma_1 \cdot b$ $x_{11} := 7 \cdot ft$ $M_{r11} := F_{11} \cdot x_{11}$ $M_{r11} = 0\, ft \cdot kip$

$A_{12} := 0 \cdot ft^2$ $F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $x_{12} := 4.5 \cdot ft$ $M_{r12} := F_{12} \cdot x_{12}$ $M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) ~~vertical component of the Traffic Surcharge acting on the backface,~~ and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (1.4 \cdot ft + cl) \cdot b$ $M_{rDL} = 22.167\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (5 \cdot ft)$ $M_{rSCH} = 0\, ft \cdot kip$

3. $M_{r\_Pa} := E_{avert} \cdot (5.5 \cdot ft)$ $M_{r\_Pa} = 31.594\, ft \cdot kip$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$    $M_{d\_surch} = 0\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$    $M_{d\_Pa} = 1.017 \times 10^{5}\ ft \cdot lbf$    $M_{d\_Pa} = 101.681\ kip \cdot ft$    $M_{d\_Pa} = 101.681\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$    $M_{d3} = 28.262\ ft \cdot kip$

**DO NOT INCLUDE horizontal component of LL and DL in the load group as a driving moment nor a horizontal force, Override with these values:**

$M_{d3} := 0 \cdot ft \cdot kip$    $H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.665 \times 10^{4}\ lbf$

$\Sigma V = 36.652\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO NOT INCLUDE horizontal component of LL and DL in the load group (Hss) or Sum of driving moments**

$\Sigma H = 11.778\ kip$

$\Sigma H = 11.778\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 1.28 \times 10^{5}\ ft \cdot lbf$

$\Sigma M_r = 127.998\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 101.681\ ft \cdot kip$

$\Sigma M_d = 101.681\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$\boxed{FS_{ot} = 1.259}$

AASHTO required Factor of Safety against overturning is 2.0

## Factor of safety against sliding

friction at base + adhesion

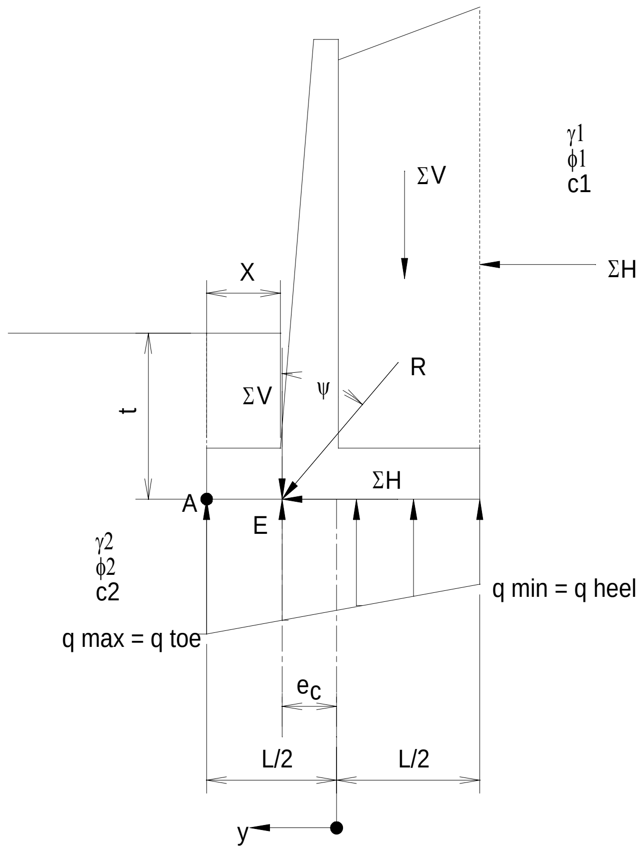
$$\tan\left(\delta_2\right) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan\left(\delta_2\right)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$\boxed{FS_{sl} = 1.386}$

AASHTO required Factor of Safety against sliding is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 2.632 \times 10^4\, lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 0.718\, ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 2.882\, ft$

$\dfrac{B}{6} = 1.2\, ft$ $\qquad$ NO GOOD !!!!!!!!

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where: A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 17316\, psf$ $\qquad q_{max} = 17.316\, ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = -7135\, psf$ $\qquad q_{min} = -7.135\, ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \dfrac{q_u}{3}$ $\qquad q_{allow} = 8\, ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\qquad FS_{bc} = 1.386$

AASHTO recommends a FS of 3

# APPENDIX F

Calculations – Unreinforced Concrete Abutment Stability Analyses

**SOUTH ABUTMENT**

**CONCRETE ABUTMENT SECTION**

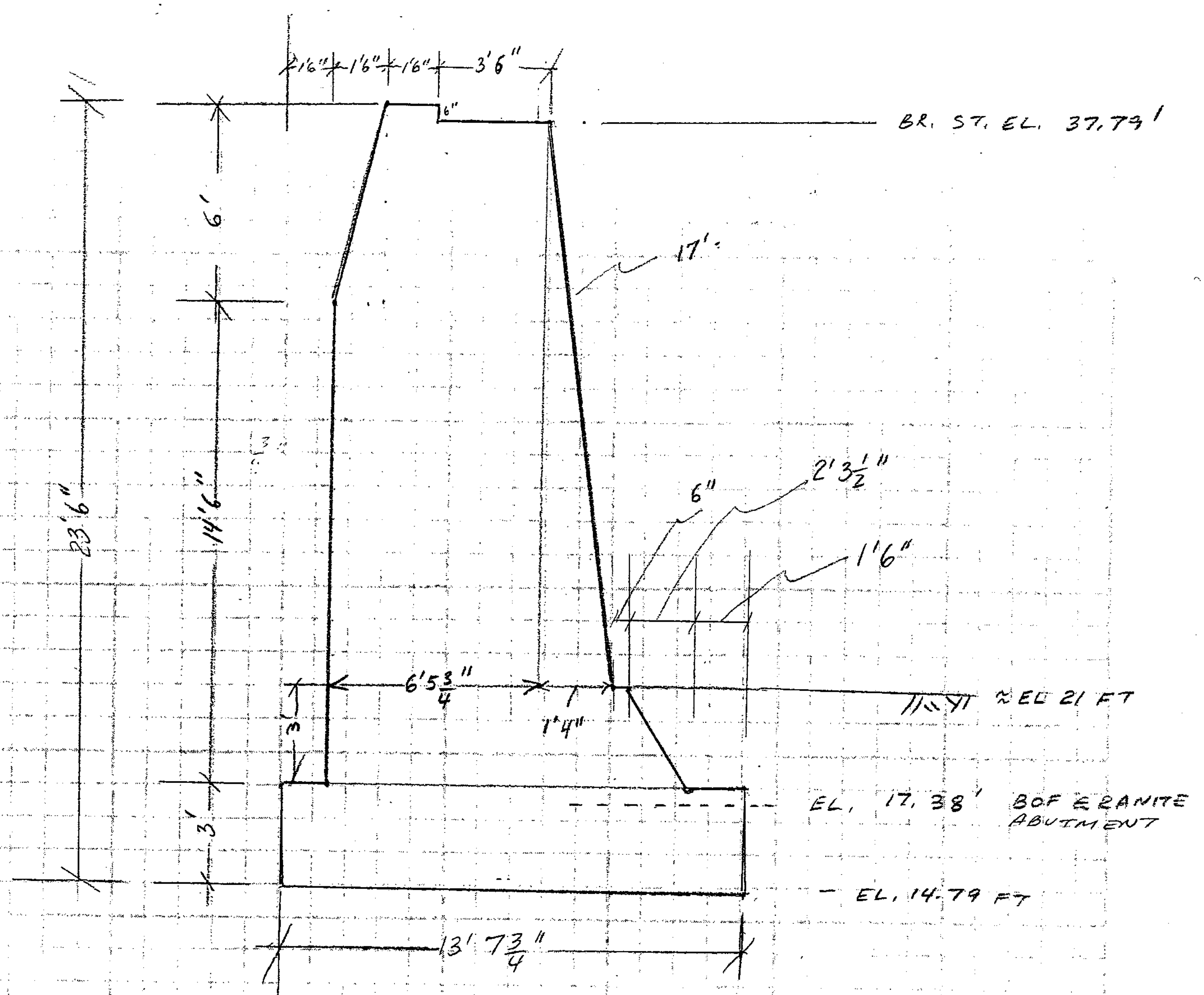
**<u>STABILITY ANALYSES</u>**

KM 12/04

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 110158.00 FILE NO. ______ OF ______

FINAL CHK. BY ______ DATE ______ LOCATION PORTLAND SH. NO. ______ OF ______

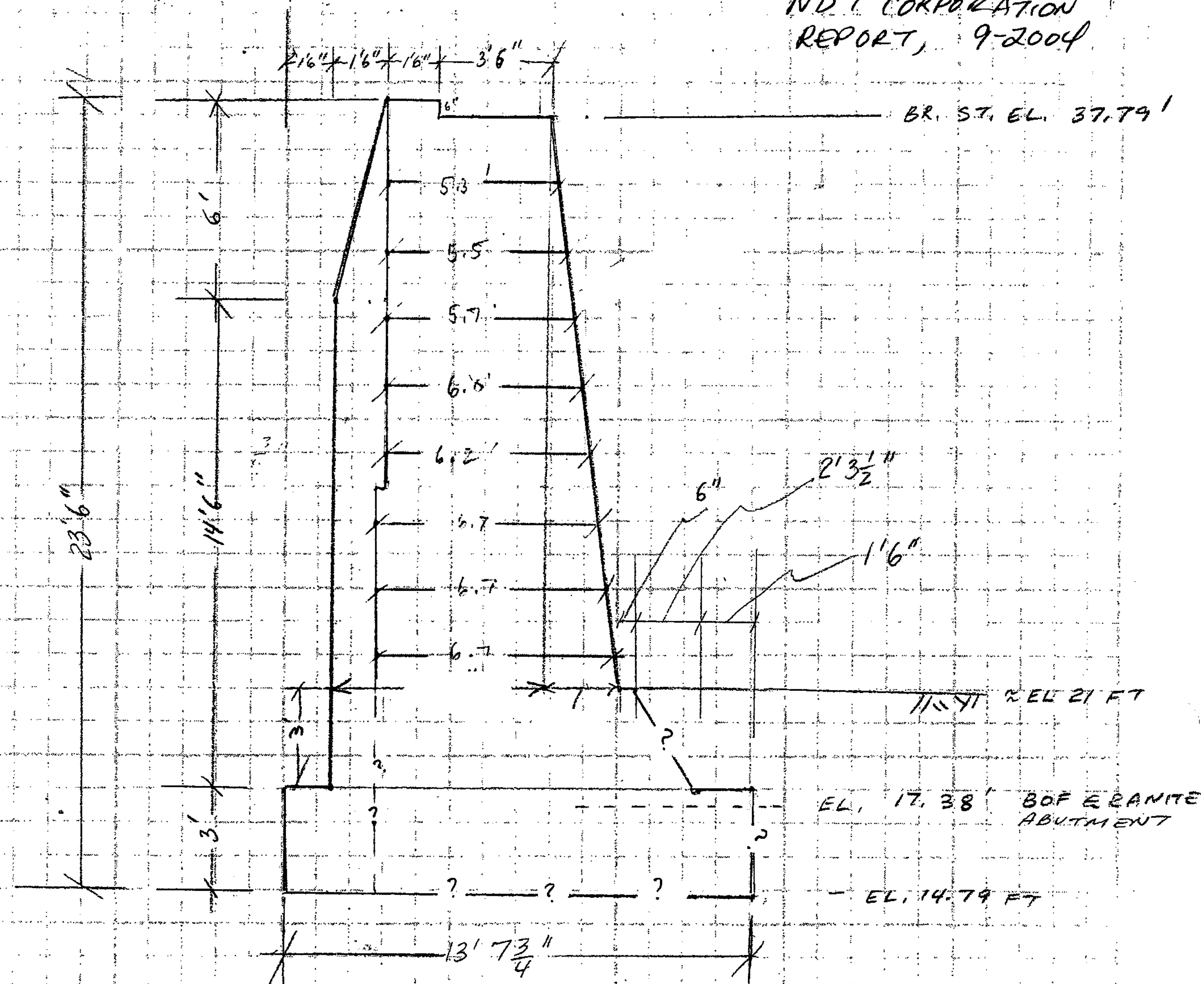
ITEM NO. ______ SUBJECT SOUTH ABUTMENT CONCRETE SECTION PER 1930 CONSTRUCTION PLANS




ASHLAR STONE ABUT ≈ 32' FT LONG

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 110158.00 FILE NO. ____ OF KM 12/04
FINAL CHK. BY ____ DATE ____ LOCATION PORTLAND SH. NO. ____ OF ____
ITEM NO. ____ SUBJECT SOUTH ABUTMENT

CONCRETE ABUTMENT DIMENSIONS based on GPR scan # 247, Figure 2, NDT CORPORATION REPORT, 9-2004

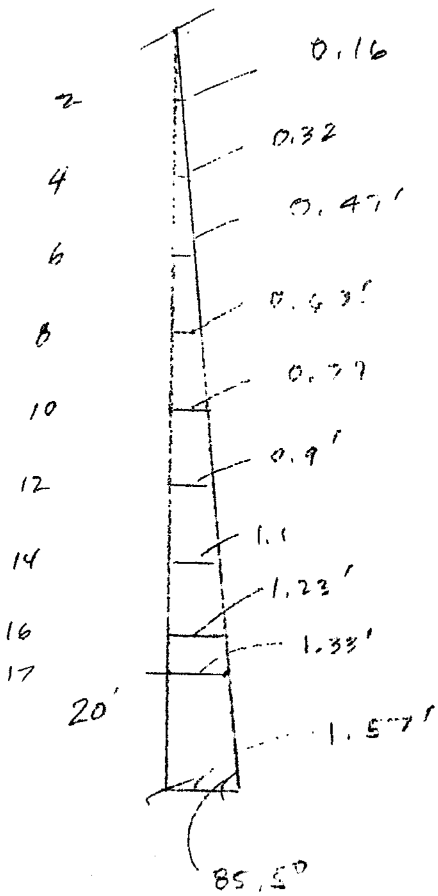



ASHLAR STONE ABUT ≈ 32' FT LONG

KM 12/04

PRELIM. BY ________ DATE ________ PROJ. NO. ________ FILE NO. ________ OF ________
FINAL CHK. BY ________ DATE ________ LOCATION ________ SH. NO. ________ OF ________
ITEM NO. ________ SUBJECT SOUTH ABUTMENT - CONCRETE X-SECTION

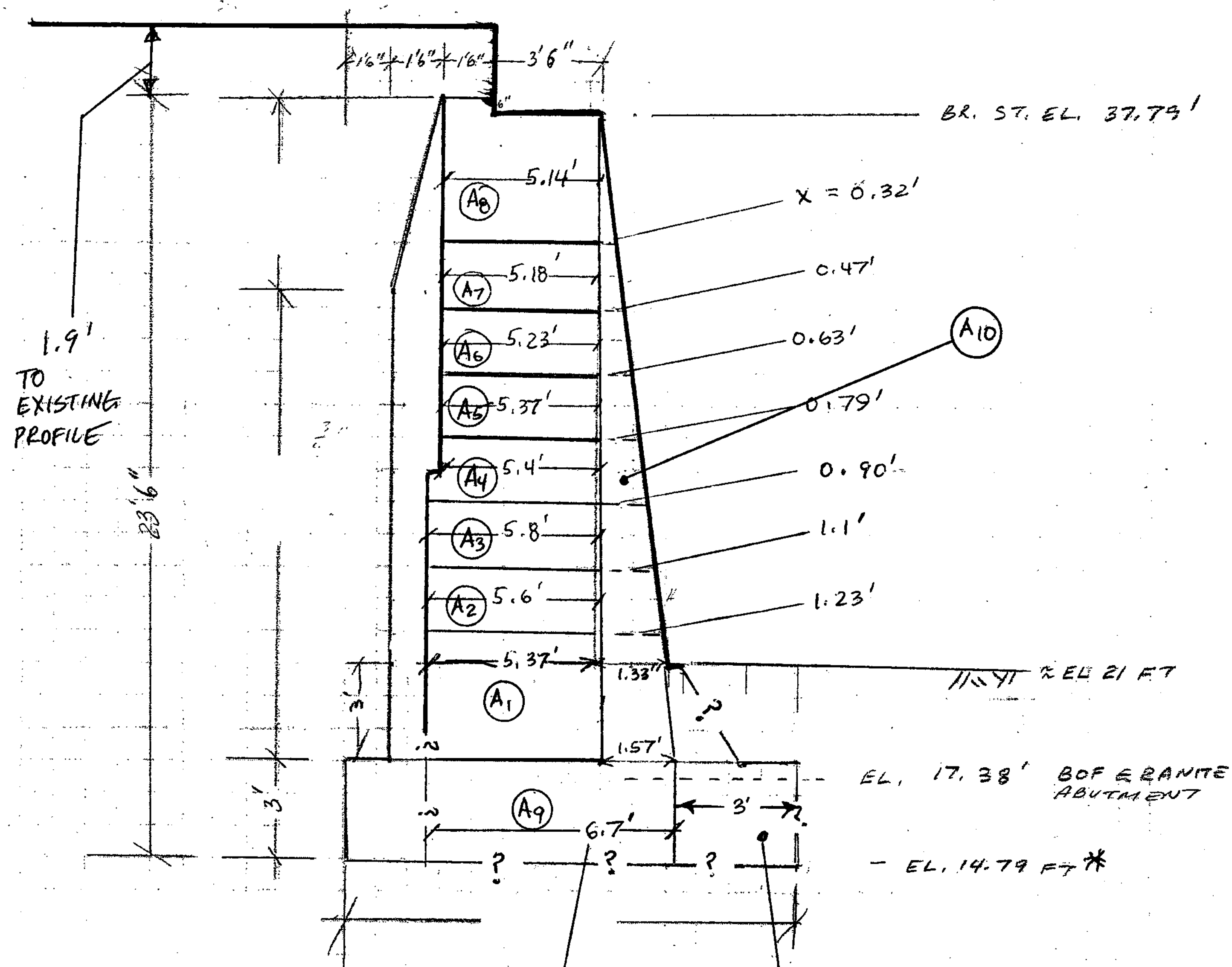
DEVELOP GEOMETRY AND AREA OF BATTERED SECTION FOR STABILITY ANALYSIS :




$$\tan 85.5 = \frac{\text{Wall Height}}{X}$$

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| PRELIM. BY | LK | DATE | 9-23-04 | PROJ. NO. | 110158.00 | FILE NO. | OF KA 11/2/04 |
| FINAL CHK. BY | | DATE | | LOCATION | PORTLAND | SH. NO. | OF |
| ITEM NO. | | SUBJECT | SOUTH ABUTMENT - CONCRETE SECTION | | | | |

CONCRETE ABUTMENT GEOMETRY BASED ON GPR SCAN #247, FIGURE 2, NDT CORP. REPORT, 9-2004



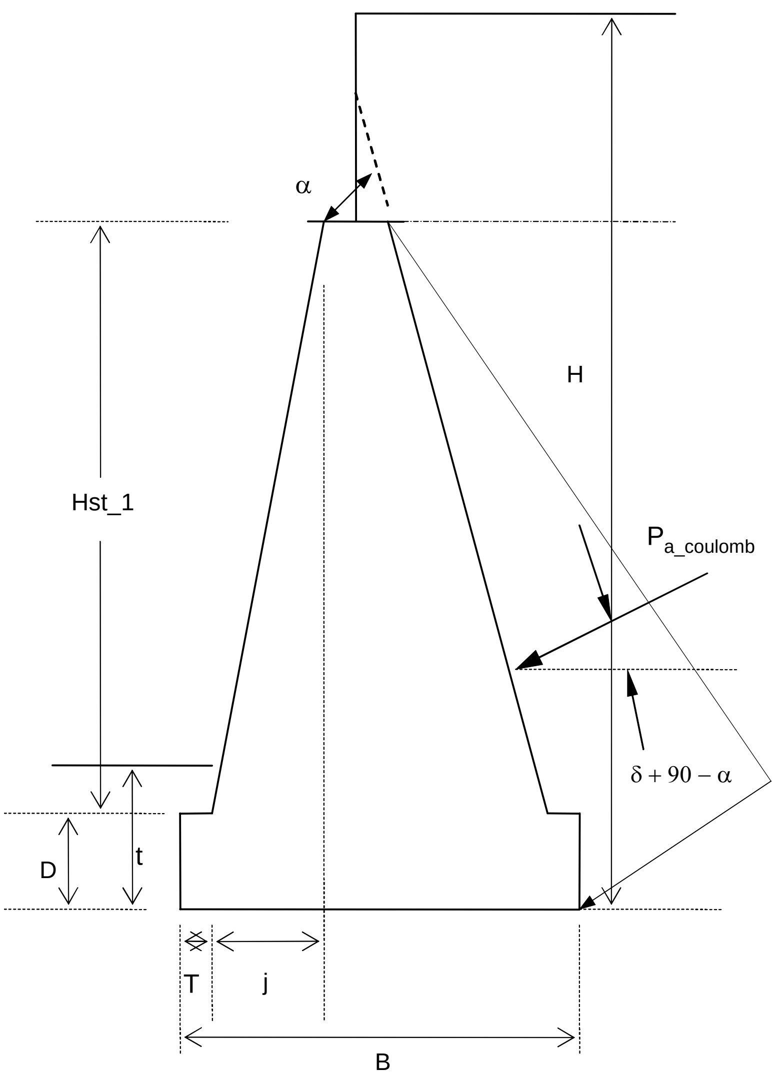

USE 6.7' (MAX. THICKNESS DETECTED BY GPR AS FOOTING WIDTH.

* EVERYTHING BELOW G.S. IS ASSUMED.

3' TOE ASSUMED. NEEDS TO BE CONFIRMED BY TEST PIT.

**South (Portland) Concrete Abutment (1930) Analysis**
**- using field verified abutment dimensions and 1930's plans for footing dimensions. field-verified backfill.**
**- Uses Coulomb theory. Traffic Surcharge added (Coulomb).**
**- Existing conditions - no raise in grade**
**- Assuming full footing toe as shown on the 1930 plans - but shorter footing (B) than shown on plans. I use max. value confirmed from GPR + <u>assumed 3' long toe- this needs to be confirmed with a test pit.</u>**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 25.5 \cdot ft$

Width of footing $B := 9.7 \cdot ft$

Length of toe $T := 3 \cdot ft$

Footing thickness $D := 3.0 \cdot ft$

Depth of toe $t := 3 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 23.5 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

$j := 1.57 \cdot ft$

## Assumed backfill and abutment proporties

concrete unit weight $\gamma_c := 150 \cdot pcf$ $\gamma_c = 23.563 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 90 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

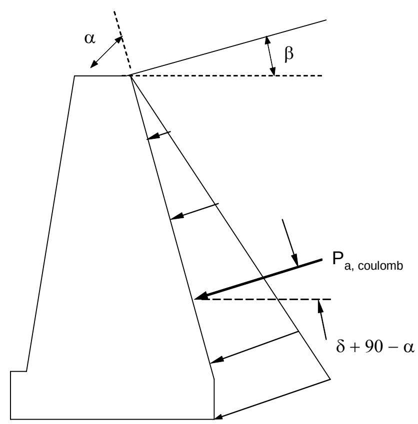
$$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft} \qquad P_{dl} = 11.667 \frac{kip}{ft}$$

$$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft} \qquad P_{ll} = 3.583 \frac{kip}{ft}$$

$$V_{ss} := (P_{dl} + P_{ll}) \cdot b \qquad V_{ss} = 1.525 \times 10^4 \, lbf \qquad V_{ss} = 15.25 \, kip$$

$$H_{ss} := \left[(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})\right] \cdot b \qquad H_{ss} = 1.346 \times 10^3 \, lbf \quad H_{ss} = 1.346 \, kip$$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.275$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 11.182\ \text{kip}$ per linear foot of abutment

**Vertical Earth Pressure:**

$E_{avert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{avert} = 4.007 \times 10^{3}\, lbf$ $\quad E_{avert} = 4.007\, kip$ per linear foot of wall

**Horizontal Earth Pressure:**

$E_{ahoriz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot P_{a1}$ $\quad E_{ahoriz} = 10438.979\, lbf$ $\quad E_{ahoriz} = 10.439\, kip$ per lin ft of wall

**No approach slab; Force due traffic - model w/ surcharge of 2' of soil ( using Coulomb earth pressure theory)**

$s := 2 \cdot ft \cdot \gamma_1$ $\quad s = 250\, psf$

$E_s := K_{a\_coulomb} \cdot s \cdot H \cdot b$ $\quad E_s = 1.754\, kip$

**Vertical Surcharge Earth Pressure:**

$E_{surch\_vert} := \sin(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_vert} = 0.629\, kip$ per lnr foot of wall

**Horizontal Surcharge Earth Pressure:**

$E_{surch\_horiz} := \cos(\delta + 90 \cdot deg - \alpha) \cdot E_s$ $\quad E_{surch\_horiz} = 1.637\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

**Resisting moments - abutment composed of granite stone masonry**

$A_1 := 5.37 \cdot 4 \cdot ft^2$ $\quad F_1 := A_1 \cdot \gamma_c \cdot b$ $\quad x_1 := \frac{5.37 \cdot ft}{2} + T + j$ $\quad M_{r1} := F_1 \cdot x_1$ $\quad M_{r1} = 23.376\, kip \cdot ft$

$A_2 := 5.6 \cdot 2 \cdot ft^2$ $\quad F_2 := A_2 \cdot \gamma_c \cdot b$ $\quad x_2 := \frac{5.6 \cdot ft}{2} + T + j$ $\quad M_{r2} := F_2 \cdot x_2$ $\quad M_{r2} = 12.382\, ft \cdot kip$

$A_3 := 5.8 \cdot ft \cdot 2 \cdot ft$ $\quad F_3 := A_3 \cdot \gamma_c \cdot b$ $\quad x_3 := \frac{5.8}{2} \cdot ft + T + j$ $\quad M_{r3} := F_3 \cdot x_3$ $\quad M_{r3} = 12.998\, ft \cdot kip$

$A_4 := (5.4 \cdot 2) \cdot ft^2$ $\quad F_4 := A_4 \cdot \gamma_c \cdot b$ $\quad x_4 := \frac{5.4}{2} \cdot ft + T + j$ $\quad M_{r4} := F_4 \cdot x_4$ $\quad M_{r4} = 11.777\, ft \cdot kip$

$A_5 := 5.37 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.37}{2} \cdot ft + T + j$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 11.688\, ft \cdot kip$

$A_6 := 5.23 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.23}{2} \cdot ft + T + j$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 11.273\, ft \cdot kip$

$A_7 := 5.18 \cdot 2 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.18}{2} \cdot ft + T + j$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 11.127\, ft \cdot kip$

$A_8 := 5.14 \cdot 4 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{5.14}{2} \cdot ft + T + j$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 22.02\, ft \cdot kip$

$A_9 := 3 \cdot 9.7 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{9.7}{2} \cdot ft$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 21.17\, ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + T$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 7.021\, ft \cdot kip$

$A_{11} := 3 \cdot 0 \cdot ft^2$ $\quad F_{11} := A_{11} \cdot \gamma_c \cdot b$ $\quad x_{11} := 6.5 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 0\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{12} := \frac{1}{2} \cdot 9.5 \cdot 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + j + cl) \cdot b$ $\quad M_{rDL} = 59.15\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (10 \cdot ft)$ $\quad M_{rSCH} = 6.286\, ft \cdot kip$ $\quad$ acts a point on backface H/2 above BOF

3. $M_{r\_Pa} := E_{avert} \cdot (10 \cdot ft)$ $\quad M_{r\_Pa} = 40.071\, ft \cdot kip$ $\quad$ acts at a point on backface H/3 above BOF

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 20.878\,ft \cdot kip$

$M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H$ $\qquad M_{d\_Pa} = 8.873 \times 10^{4}\,ft \cdot lbf$ $\qquad M_{d\_Pa} = 88.731\,kip \cdot ft$ $\qquad M_{d\_Pa} = 88.731\,kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad M_{d3} = 28.262\,ft \cdot kip$

**DO NOT INCLUDE horizontal component of LL and DL in the load group. Override lateral load and moment due to this component:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.853 \times 10^{4}\,lbf$

$\Sigma V = 38.532\,kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO NOT INCLUDE horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 12.076\,kip$

$\Sigma H = 12.076\,kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 2.503 \times 10^{5}\,ft \cdot lbf$

$\Sigma M_r = 250.338\,ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 109.609\,ft \cdot kip$

$\Sigma M_d = 109.609\,kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$FS_{ot} = 2.284$

AASHTO required factor of safety is 2.0

## Factor of safety against sliding

friction at base + adhesion

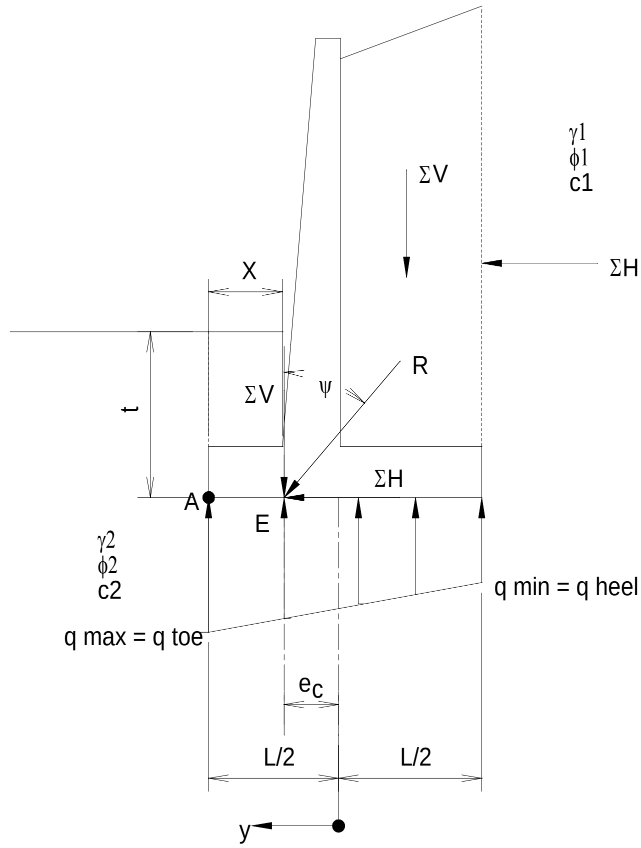
$\tan(\delta_2) = 0.445$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$FS_{sl} = 1.421$

AASHTO required factor of safety is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 1.407 \times 10^5\ lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\qquad AE = 3.652\ ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\qquad e_c = 1.198\ ft$

$\dfrac{B}{6} = 1.617\ ft$ $\qquad$ OK

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{max} = 6916\ psf$ $\qquad q_{max} = 6.916\ ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\qquad q_{min} = 1029\ psf$ $\qquad q_{min} = 1.029\ ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

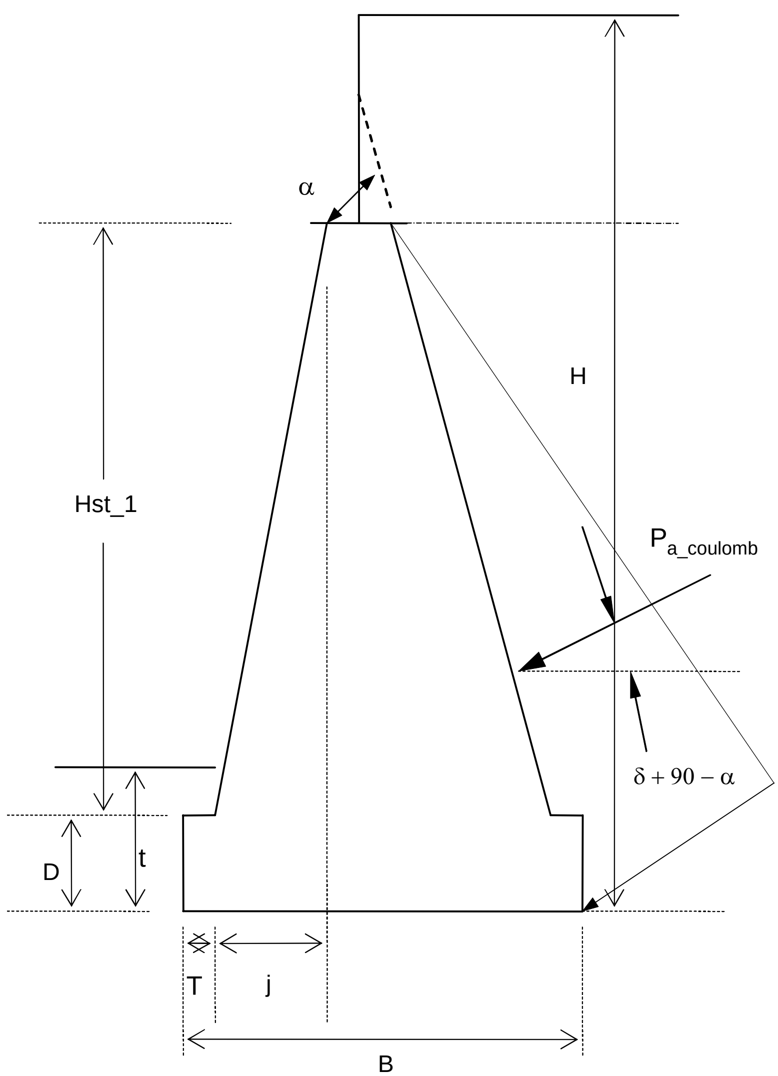
**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \dfrac{q_u}{3}$ $\qquad q_{allow} = 8\ ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\qquad FS_{bc} = 3.47$

AASHTO recommends a FS of 3

**South (Portland) Concrete Abutment (1930) Analysis**
**- using field verified abutment dimensions and 1930's plans for footing dimensions.  field-verified backfill.**
**- Uses Coulomb theory.  Assume approach slab is added.  So, no traffic surcharge.**
**- Proposed conditions - 3 foot raise in profile**
**- Assuming full footing toe as shown on the 1930 plans - but shorter footing (B) than shown on plans. I use max. value confirmed from GPR + <u>assumed 3' long toe- this needs to be confirmed with a test pit.</u>**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




| | |
|---|---|
| Height of wall, | $H := 28.5 \cdot ft$ |
| Width of footing | $B := 9.7 \cdot ft$ |
| Length of toe | $T := 3 \cdot ft$ |
| Footing thickness | $D := 3.0 \cdot ft$ |
| Depth of toe | $t := 3 \cdot ft$ |
| Depth of seat | $d_s := 1.9 \cdot ft$ |
| Height of stem 1 | $H_{st\_1} := 23.5 \cdot ft$ |
| Unit width | $b := 1 \cdot ft$ |
| centerline distance | $cl := 6 \cdot in$ |

$j := 1.57 \cdot ft$

**Assumed backfill and abutment proporties**

concrete unit weight $\gamma_c := 150 \cdot pcf$ $\gamma_c = 23.563 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 90 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

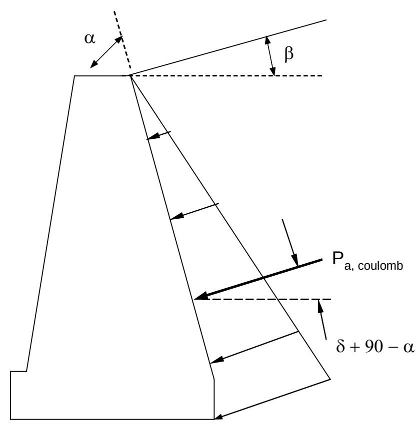
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin\left(\phi_1 + \alpha\right)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin\left(\phi_1 + \delta\right) \cdot \sin\left(\phi_1 - \beta\right)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.275$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 13.967\ \text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{avert} = 5.005 \times 10^3\, lbf$ $\quad E_{avert} = 5.005\, kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{ahoriz} = 13039.693\, lbf$ $\quad E_{ahoriz} = 13.04\, kip$ per lin ft of wall

### Add an approach slab; so can neglect force due traffic - model w/ surcharge of 2' of soil (using Coulomb earth pressure theory)

$s := 0\cdot ft\cdot\gamma_1$ $\quad s = 0\, psf$

$E_s := K_{a\_coulomb}\cdot s\cdot H\cdot b$ $\quad E_s = 0\, kip$

### Vertical Surcharge Earth Pressure:

$E_{surch\_vert} := \sin(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_vert} = 0\, kip$ per lnr foot of wall

### Horizontal Surcharge Earth Pressure:

$E_{surch\_horiz} := \cos(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_horiz} = 0\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 5.37\cdot 4\cdot ft^2$ $\quad F_1 := A_1\cdot\gamma_c\cdot b$ $\quad x_1 := \frac{5.37\cdot ft}{2} + T + j$ $\quad M_{r1} := F_1\cdot x_1$ $\quad M_{r1} = 23.376\, kip\cdot ft$

$A_2 := 5.6\cdot 2\cdot ft^2$ $\quad F_2 := A_2\cdot\gamma_c\cdot b$ $\quad x_2 := \frac{5.6\cdot ft}{2} + T + j$ $\quad M_{r2} := F_2\cdot x_2$ $\quad M_{r2} = 12.382\, ft\cdot kip$

$A_3 := 5.8\cdot ft\cdot 2\cdot ft$ $\quad F_3 := A_3\cdot\gamma_c\cdot b$ $\quad x_3 := \frac{5.8}{2}\cdot ft + T + j$ $\quad M_{r3} := F_3\cdot x_3$ $\quad M_{r3} = 12.998\, ft\cdot kip$

$A_4 := (5.4\cdot 2)\cdot ft^2$ $\quad F_4 := A_4\cdot\gamma_c\cdot b$ $\quad x_4 := \frac{5.4}{2}\cdot ft + T + j$ $\quad M_{r4} := F_4\cdot x_4$ $\quad M_{r4} = 11.777\, ft\cdot kip$

$A_5 := 5.37 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.37}{2} \cdot ft + T + j$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 11.688\, ft \cdot kip$

$A_6 := 5.23 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.23}{2} \cdot ft + T + j$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 11.273\, ft \cdot kip$

$A_7 := 5.18 \cdot 2 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.18}{2} \cdot ft + T + j$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 11.127\, ft \cdot kip$

$A_8 := 5.14 \cdot 4 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{5.14}{2} \cdot ft + T + j$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 22.02\, ft \cdot kip$

$A_9 := 6.7 \cdot 3 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{9.7}{2} \cdot ft$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 14.623\, ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + T$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 7.021\, ft \cdot kip$

$A_{11} := 3 \cdot 0 \cdot ft^2$ $\quad F_{11} := A_{11} \cdot \gamma_c \cdot b$ $\quad x_{11} := 6.5 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 0\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{12} := \frac{1}{2} \cdot 9.5 \cdot 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + j + cl) \cdot b$ $\quad M_{rDL} = 59.15\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (9 \cdot ft)$ $\quad M_{rSCH} = 0\, ft \cdot kip$ acts downward at point H/2 above BOF

3. $M_{r\_Pa} := E_{avert} \cdot (9 \cdot ft)$ $\quad M_{r\_Pa} = 45.049\, ft \cdot kip$ acts downward at point H/3 above BOF

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad$ $M_{d\_surch} = 0\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad$ $M_{d\_Pa} = 1.239 \times 10^{5}\ ft \cdot lbf$ $\qquad$ $M_{d\_Pa} = 123.877\ kip \cdot ft$ $\qquad$ $M_{d\_Pa} = 123.877\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad$ $M_{d3} = 28.262\ ft \cdot kip$

**DO NOT INCLUDE horizontal component of LL and DL in the load group. Override lateral load and moment due to this component:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad$ $H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 3.755 \times 10^{4}\ lbf$

$\Sigma V = 37.552\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**DO NOT INCLUDE horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 13.04\ kip$

$\Sigma H = 13.04\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 2.425 \times 10^{5}\ ft \cdot lbf$

$\Sigma M_r = 242.483\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 123.877\ ft \cdot kip$

$\Sigma M_d = 123.877\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$$FS_{ot} = 1.957$$

AASHTO required factor of safety is 2.0

## Factor of safety against sliding

friction at base + adhesion

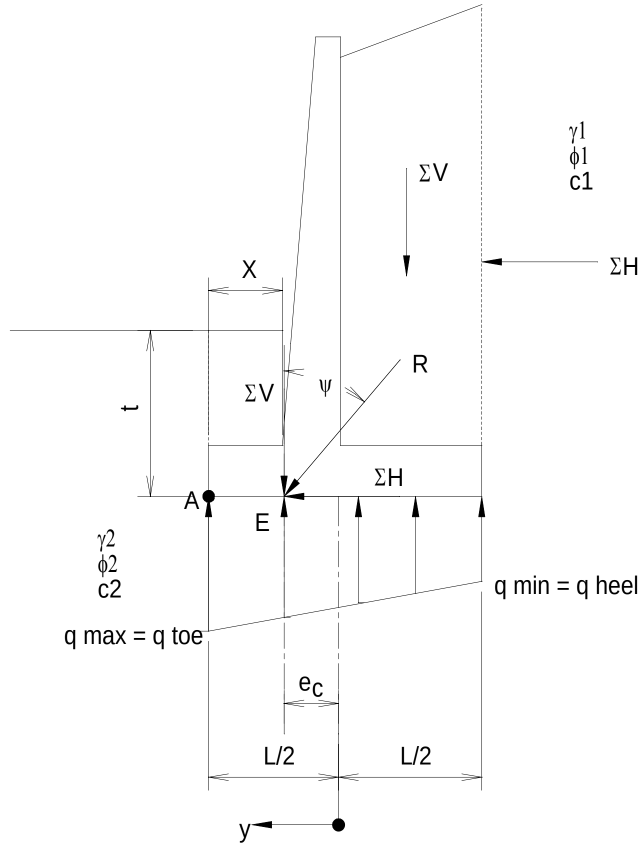
$$\tan(\delta_2) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$$FS_{sl} = 1.282$$

AASHTO required factor of safety is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d \qquad M_{net} = 1.186 \times 10^5\ \text{lbf}\cdot\text{ft}$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V} \qquad AE = 3.158\ \text{ft} \qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE \qquad e_c = 1.692\ \text{ft}$

$\dfrac{B}{6} = 1.617\ \text{ft}$ OK

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net}\cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

$$q = \frac{\Sigma V}{A} - \frac{M_{net}\cdot y}{I}$$

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B}\cdot\left(1 + \dfrac{6\cdot e_c}{B}\right)\right]\cdot\dfrac{1}{b} \qquad q_{max} = 7922\ \text{psf} \qquad q_{max} = 7.922\ \text{ksf} \qquad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B}\cdot\left(1 - \dfrac{6\cdot e_c}{B}\right)\right]\cdot\dfrac{1}{b} \qquad q_{min} = -179\ \text{psf} \qquad q_{min} = -0.179\ \text{ksf} \qquad q_{heel} := q_{min}$

$B_e := B - 2\cdot e_c$

**Allowable Bearing Pressure:** $q_u := 24\cdot\text{ksf} \qquad q_{allow} := \dfrac{q_u}{3} \qquad q_{allow} = 8\ \text{ksf}$

**Factor of Safety against BC failure:** $FS_{bc} := \dfrac{q_u}{q_{max}} \qquad FS_{bc} = 3.03$

AASHTO recommends a FS of 3

**NORTH ABUTMENT**

**CONCRETE ABUTMENT SECTION**

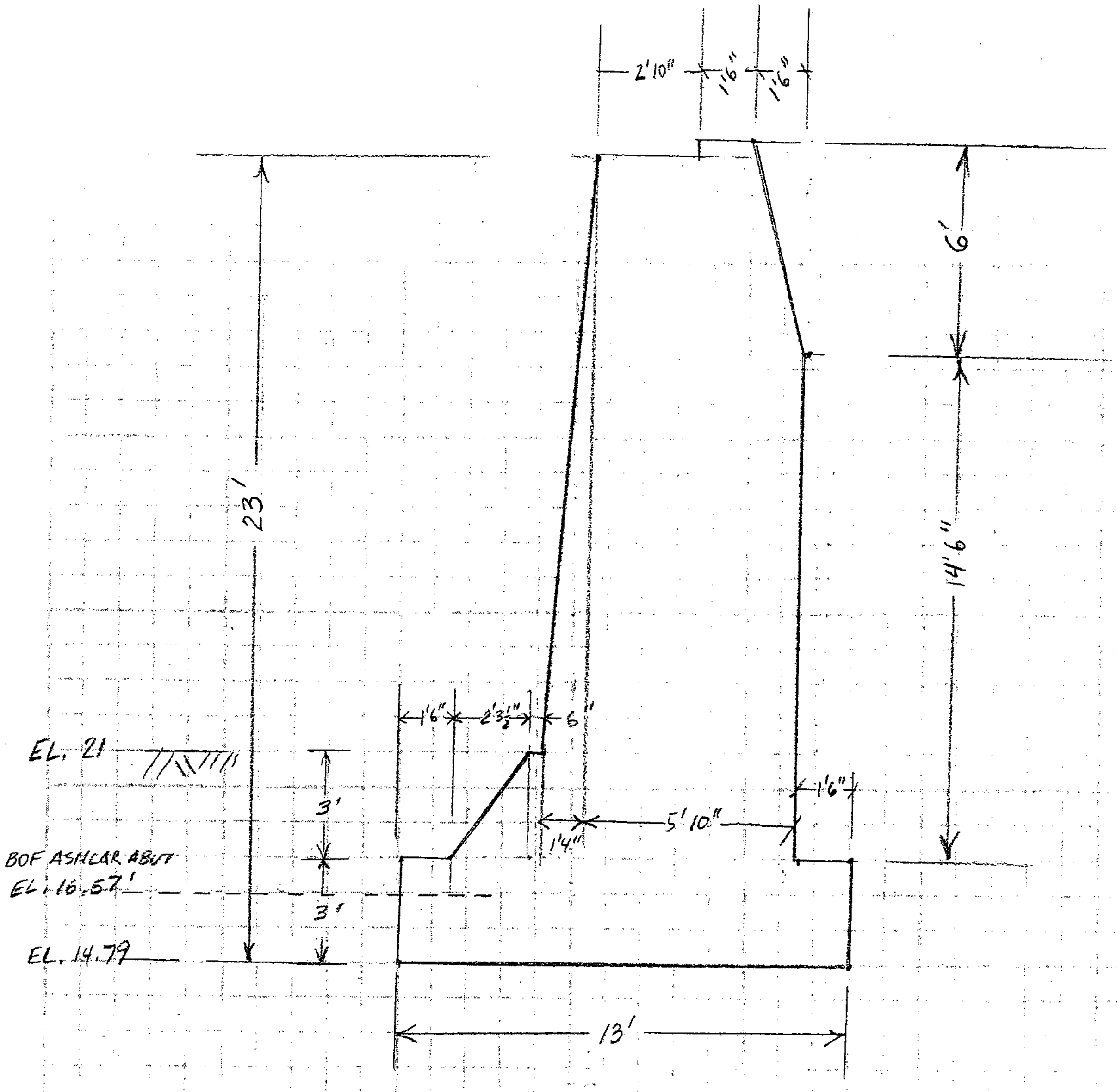
**STABILITY ANALYSES**

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 10158.00 FILE NO. ____ OF ____ KM 12/04

FINAL CHK. BY ____ DATE ____ LOCATION PORTLAND SH. NO. ____ OF ____

ITEM NO. ____ SUBJECT NORTH (FALMOUTH) ABUTMENT

CONSTRUCTION PLANS dated 1930




ASHLAR STONE ABUT ≈ 39'0" LONG

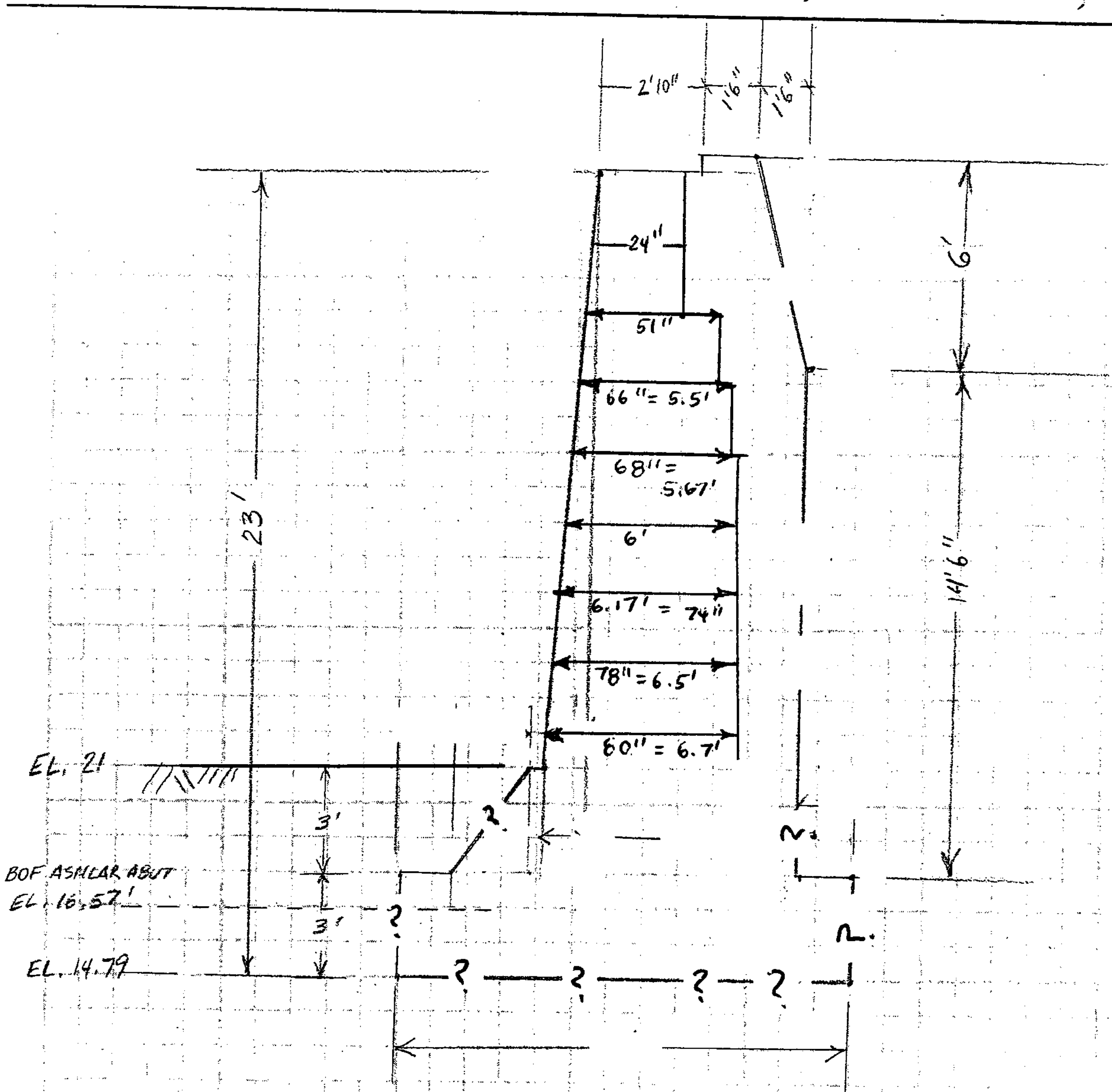
KM 12/04

PRELIM. BY LK DATE 9-23-04 PROJ. NO. 10158.00 FILE NO. ____ OF ____

FINAL CHK. BY ____ DATE ____ LOCATION PORTLAND SH. NO. ____ OF ____

ITEM NO. ____ SUBJECT NORTH (FALMOUTH) ABUTMENT

CONCRETE ABUTMENT DIMENSIONS BASED ON GPR / SONIC SCAN LINE #270, (FIGURE 4, NDT REPORT)



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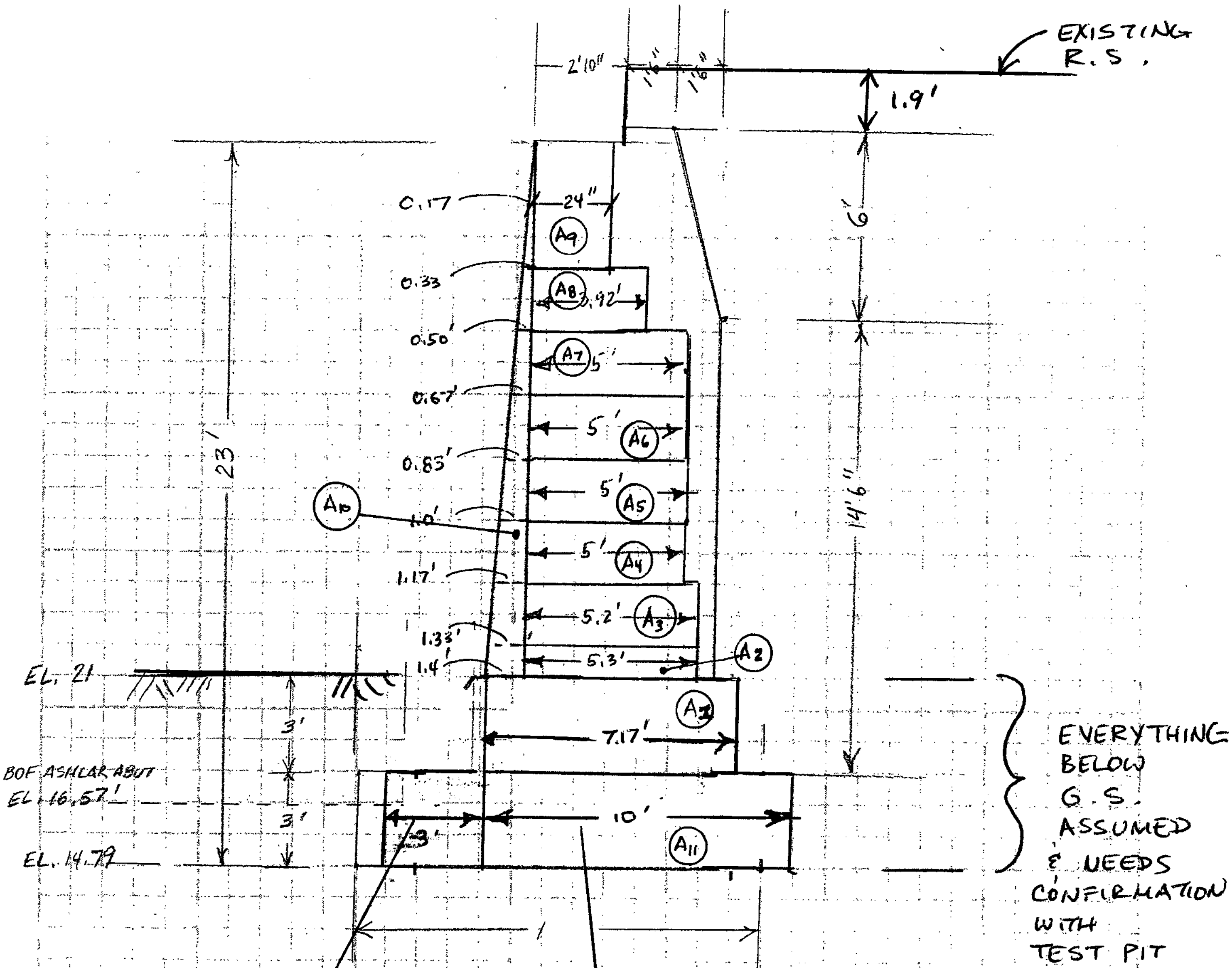
| PRELIM. BY LK | DATE 9-23-04 | PROJ. NO. 10158.00 | FILE NO. | OF | |
| FINAL CHK. BY | DATE | LOCATION PORTLAND | SH. NO. | OF | |
| ITEM NO. | SUBJECT NORTH (FALMOUTH) ABUTMENT | | | | |

CONCRETE ABUTMENT DIMENSIONS PER GPR # 270



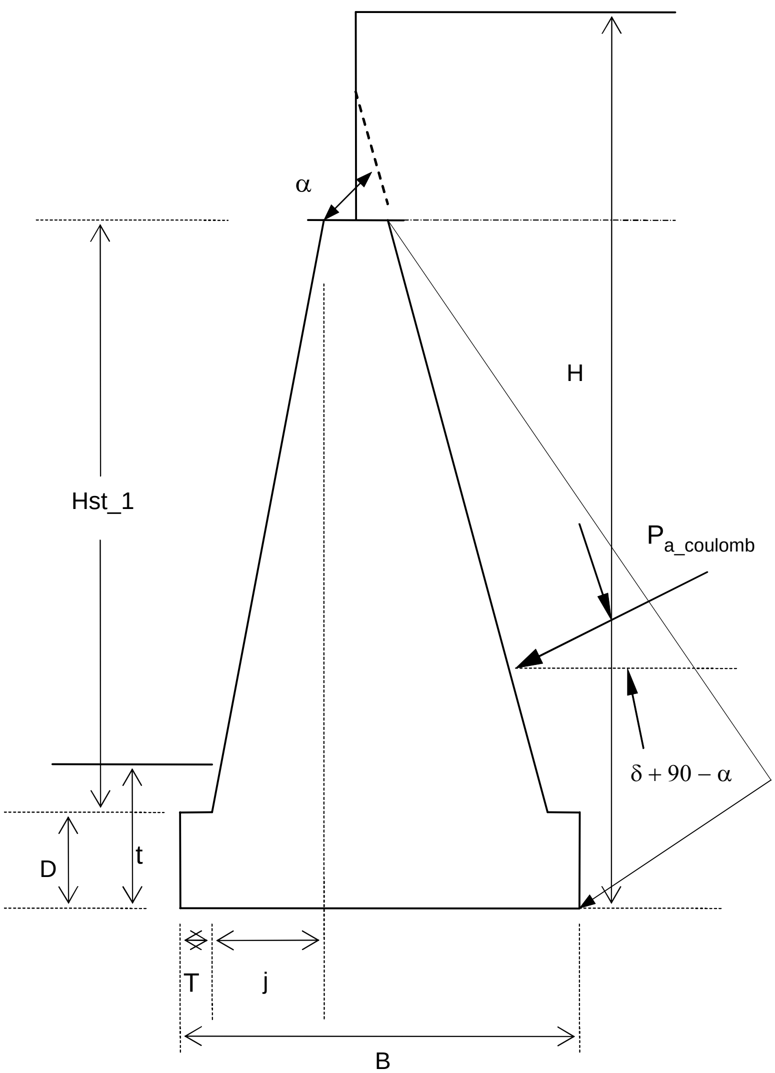

6.0 FT EMBEDMENT OF 13' FOOTING ASSUMED - FOOTING DIMENSION ASSUMED AND BASED ON 1930 CONSTRUCTION PLANS

3' TOE ASSUMED IN CALCULATION OF MOMENT ARMS FOR ALL RESTORING MASSES, WEIGHT OF TOE IGNORED.

**North (Falmouth) Concrete Abutment (1930) Analysis - using field verified abutment dimensions and 1930's plans for footing dimensions. field-verified backfill.**
**Uses Coulomb theory. Traffic Surcharge added (Coulomb). Existing conditions - no raise in grade**

**Assuming full footing toe and footing width as shown on the 1930 plans - this needs to be confirmed with a test pit**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 24.9 \cdot ft$

Width of footing $B := 13 \cdot ft$

Length of toe $T := 3 \cdot ft$

Footing thickness $D := 3.0 \cdot ft$

Depth of toe $t := 4 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 23 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

$j := 1.4 \cdot ft$

**Assumed backfill and abutment properties**

concrete unit weight $\gamma_c := 150 \cdot pcf$ $\gamma_c = 23.563 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 80 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

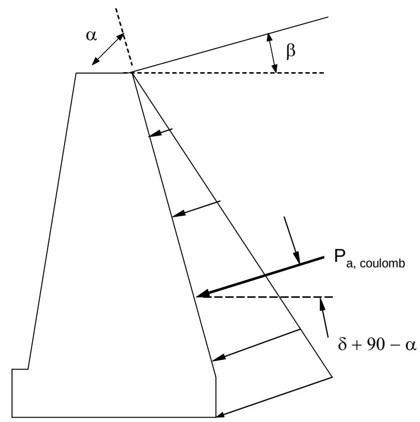
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - \left(\cos(\phi_1)\right)^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin\left(\phi_1 + \alpha\right)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin\left(\phi_1 + \delta\right) \cdot \sin\left(\phi_1 - \beta\right)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.354$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 13.735\,\text{kip}$ per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{avert} = 7.074 \times 10^3\, lbf$ $\quad E_{avert} = 7.074\, kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{ahoriz} = 11773.37\, lbf$ $\quad E_{ahoriz} = 11.773\, kip$ per lin ft of wall

### No approach slab; Force due traffic - model w/ surcharge of 2' of soil (using Coulomb earth pressure theory)

$s := 2\cdot ft\cdot \gamma_1$ $\quad s = 250\, psf$

$E_s := K_{a\_coulomb}\cdot s\cdot H\cdot b$ $\quad E_s = 2.206\, kip$

#### Vertical Surcharge Earth Pressure:

$E_{surch\_vert} := \sin(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_vert} = 1.136\, kip$ per lnr foot of wall

#### Horizontal Surcharge Earth Pressure:

$E_{surch\_horiz} := \cos(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_horiz} = 1.891\, kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 7.17\cdot 3\cdot ft^2$ $\quad F_1 := A_1\cdot\gamma_c\cdot b$ $\quad x_1 := \frac{7.17\cdot ft}{2} + T$ $\quad M_{r1} := F_1\cdot x_1$ $\quad M_{r1} = 21.247\, kip\cdot ft$

$A_2 := 5.3\cdot 1\cdot ft^2$ $\quad F_2 := A_2\cdot\gamma_c\cdot b$ $\quad x_2 := \frac{5.3\cdot ft}{2} + T + j$ $\quad M_{r2} := F_2\cdot x_2$ $\quad M_{r2} = 5.605\, ft\cdot kip$

$A_3 := 5.2\cdot ft\cdot 2\cdot ft$ $\quad F_3 := A_3\cdot\gamma_c\cdot b$ $\quad x_3 := \frac{5.2}{2}\cdot ft + T + j$ $\quad M_{r3} := F_3\cdot x_3$ $\quad M_{r3} = 10.92\, ft\cdot kip$

$A_4 := (5.0\cdot 2)\cdot ft^2$ $\quad F_4 := A_4\cdot\gamma_c\cdot b$ $\quad x_4 := \frac{5.0}{2}\cdot ft + T + j$ $\quad M_{r4} := F_4\cdot x_4$ $\quad M_{r4} = 10.35\, ft\cdot kip$

$A_5 := 5.0 \cdot 2 \cdot ft^2$ $\quad F_5 := A_5 \cdot \gamma_c \cdot b$ $\quad x_5 := \frac{5.0}{2} \cdot ft + T + j$ $\quad M_{r5} := F_5 \cdot x_5$ $\quad M_{r5} = 10.35\, ft \cdot kip$

$A_6 := 5.0 \cdot 2 \cdot ft^2$ $\quad F_6 := A_6 \cdot \gamma_c \cdot b$ $\quad x_6 := \frac{5.0}{2} \cdot ft + T + j$ $\quad M_{r6} := F_6 \cdot x_6$ $\quad M_{r6} = 10.35\, ft \cdot kip$

$A_7 := 5.0 \cdot 2 \cdot ft^2$ $\quad F_7 := A_7 \cdot \gamma_c \cdot b$ $\quad x_7 := \frac{5.}{2} \cdot ft + T + j$ $\quad M_{r7} := F_7 \cdot x_7$ $\quad M_{r7} = 10.35\, ft \cdot kip$

$A_8 := 3.92 \cdot 2 \cdot ft^2$ $\quad F_8 := A_8 \cdot \gamma_c \cdot b$ $\quad x_8 := \frac{3.92}{2} \cdot ft + T + j$ $\quad M_{r8} := F_8 \cdot x_8$ $\quad M_{r8} = 7.479\, ft \cdot kip$

$A_9 := 2 \cdot 4 \cdot ft^2$ $\quad F_9 := A_9 \cdot \gamma_c \cdot b$ $\quad x_9 := \frac{2}{2} \cdot ft + T + j$ $\quad M_{r9} := F_9 \cdot x_9$ $\quad M_{r9} = 6.48\, ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $\quad F_{10} := A_{10} \cdot \gamma_c \cdot b$ $\quad x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + T$ $\quad M_{r10} := F_{10} \cdot x_{10}$ $\quad M_{r10} = 7.021\, ft \cdot kip$

$A_{11} := 3 \cdot 13 \cdot ft^2$ $\quad F_{11} := A_{11} \cdot \gamma_c \cdot b$ $\quad x_{11} := 8 \cdot ft$ $\quad M_{r11} := F_{11} \cdot x_{11}$ $\quad M_{r11} = 46.8\, ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{12} := 0 \cdot ft^2$ $\quad F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $\quad x_{12} := 4.5 \cdot ft$ $\quad M_{r12} := F_{12} \cdot x_{12}$ $\quad M_{r12} = 0\, ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + j + cl) \cdot b$ $\quad M_{rDL} = 57.167\, ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (6 \cdot ft)$ $\quad M_{rSCH} = 6.818\, ft \cdot kip$ — acts downward on backface at H/2 above BOF $\quad \frac{H}{2} = 12.45\, ft$

3. $M_{r\_Pa} := E_{avert} \cdot (6.5 \cdot ft)$ $\quad M_{r\_Pa} = 45.982\, ft \cdot kip$ — acts downward on backface at H/3 above BOF $\quad \frac{H}{3} = 8.3\, ft$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 23.547\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad M_{d\_Pa} = 9.772 \times 10^4\ ft \cdot lbf$ $\qquad M_{d\_Pa} = 97.719\ kip \cdot ft$ $\qquad M_{d\_Pa} = 97.719\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad M_{d3} = 28.262\ ft \cdot kip$

**does not include horizontal component of LL and DL in the load group. Override lateral load and moment due to this component:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 4.147 \times 10^4\ lbf$

$\Sigma V = 41.47\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**does not include horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 13.665\ kip$

$\Sigma H = 13.665\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 2.569 \times 10^5\ ft \cdot lbf$

$\Sigma M_r = 256.919\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 121.266\ ft \cdot kip$

$\Sigma M_d = 121.266\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$$FS_{ot} = 2.119$$

AASHTO required factor of safety is 2.0

## Factor of safety against sliding

friction at base + adhesion

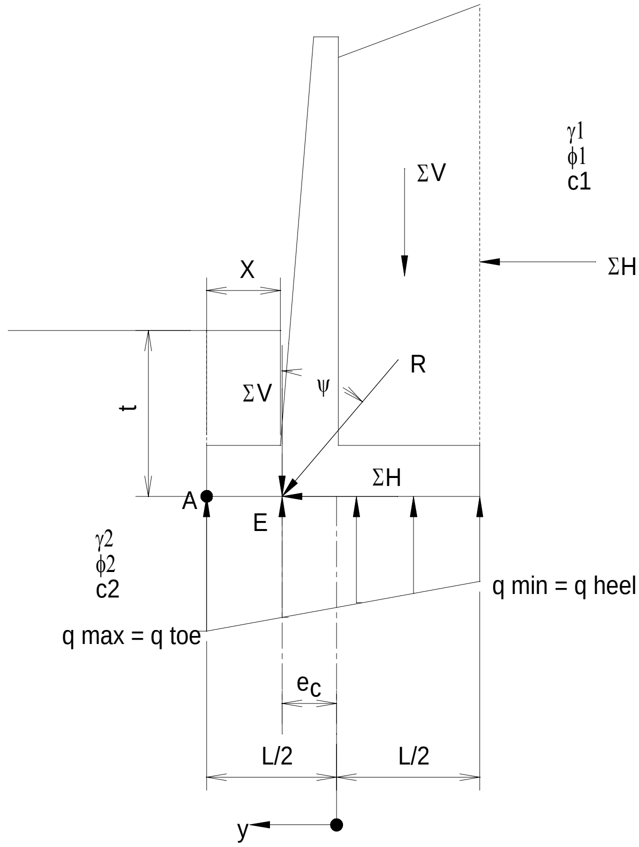
$$\tan(\delta_2) = 0.445$$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan(\delta_2)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$$FS_{sl} = 1.351$$

AASHTO required factor of safety is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\quad M_{net} = 1.357 \times 10^5\, lbf \cdot ft$

### location of resultant

$AE := \dfrac{M_{net}}{\Sigma V}$ $\quad AE = 3.271\, ft$ $\quad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \dfrac{B}{2} - AE$ $\quad e_c = 3.229\, ft$

$\dfrac{B}{6} = 2.167\, ft$ OK

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 + \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\quad q_{max} = 7944\, psf$ $\quad q_{max} = 7.944\, ksf$ $\quad q_{toe} := q_{max}$

$q_{min} := \left[\dfrac{\Sigma V}{B} \cdot \left(1 - \dfrac{6 \cdot e_c}{B}\right)\right] \cdot \dfrac{1}{b}$ $\quad q_{min} = -1564\, psf$ $\quad q_{min} = -1.564\, ksf$ $\quad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

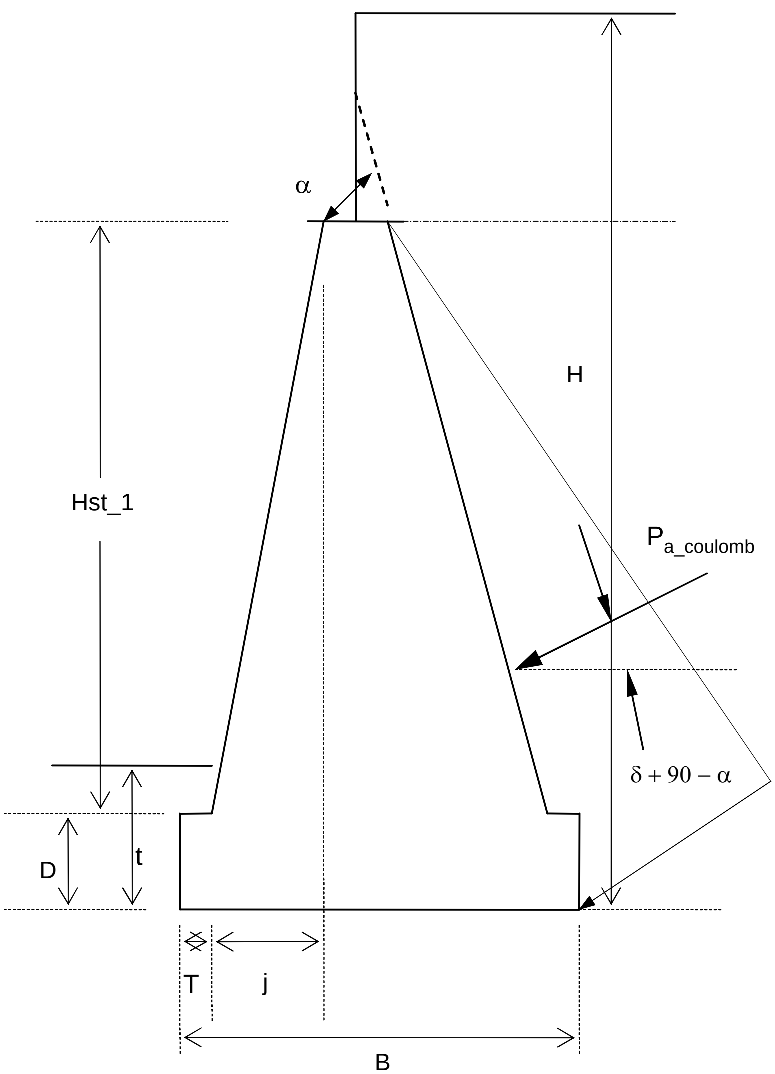
**Allowable Bearing Pressure:** $\quad q_u := 24 \cdot ksf$ $\quad q_{allow} := \dfrac{q_u}{3}$ $\quad q_{allow} = 8\, ksf$

**Factor of Safety against BC failure:** $\quad FS_{bc} := \dfrac{q_u}{q_{max}}$ $\quad FS_{bc} = 3.021$

AASHTO recommends a FS of 3

**North (Falmouth) Concrete Abutment (1930) Analysis - using field verified abutment dimensions and 1930's plans for footing dimensions and field-verified backfill. Uses Coulomb theory.**
**Traffic Surcharge neglected because assuming approach slab to be added.**
**Proposed conditions - 3 ft raise in grade.**
**Assuming full footing toe and footing width as shown on the 1930 plans - this needs to be confirmed with a test pit.**

$psf := \frac{lbf}{ft^2}$ $pcf := \frac{lbf}{ft^3}$ $Mg := 1000 \cdot kg$ $kN := 1000 \cdot newton$ $kPa := \frac{kN}{m^2}$ $tsf := \frac{ton}{ft^2}$ $kip := 1000 \cdot lbf$ $ksf := \frac{kip}{ft^2}$




Height of wall, $H := 28.9 \cdot ft$

Width of footing $B := 13 \cdot ft$

Length of toe $T := 3 \cdot ft$

Footing thickness $D := 3.0 \cdot ft$

Depth of toe $t := 4 \cdot ft$

Depth of seat $d_s := 1.9 \cdot ft$

Height of stem 1 $H_{st\_1} := 23 \cdot ft$

Unit width $b := 1 \cdot ft$

centerline distance $cl := 6 \cdot in$

$j := 1.4 \cdot ft$

**Assumed backfill and abutment properties**

concrete unit weight $\gamma_c := 150 \cdot pcf$ $\gamma_c = 23.563 \frac{kN}{m^3}$

backfill #1 $\gamma_1 := 125 \cdot pcf$ $\phi_1 := 32 \cdot deg$ $c_1 := 0 \cdot psf$ granular fill

Backfill #2 $\gamma_{1b} := 120 \cdot pcf$ $\phi_{1b} := 20 \cdot deg$ $c_{1b} := 700 \cdot psf$

Rankine wall friction $\delta := 0 \cdot deg$

Coulomb wall friction $\delta := 21 \cdot deg$ *2/3 phi*

Angle of backslope $\beta := 0 \cdot deg$

$\alpha$- Angle of abutment backwall (for Coulomb Analysis use true angle of gravity abutment backface) $\alpha := 80 \cdot deg$

$\alpha$ - Angle of abutment backface (for Rankine analyses use $\alpha$ = 90 as Rankine acts on a veritcal plane drawn from the back of the heel up to the GS)

$\alpha$ - For Coulomb Analysis on a Cantilever wall, use angle of line drawn from back of heel, to the back of the stem at the top of the wall.

Foundation material : sand $\gamma_2 := 125 \cdot pcf$ $\phi_2 := 32 \cdot deg$ $c_2 := 0 \cdot psf$

concrete - sand friction angle $\delta_2 := 24 \cdot deg$ $\tan(\delta_2) = 0.445$

**DL and LL forces per linear foot of wall:**

Tim Merritt, TYLin, calculate 700 kip per abutment of dead load.
Tim calculated 215 kip per abutment of LL. Bridge seat is roughly 18 meters or 60 ft.

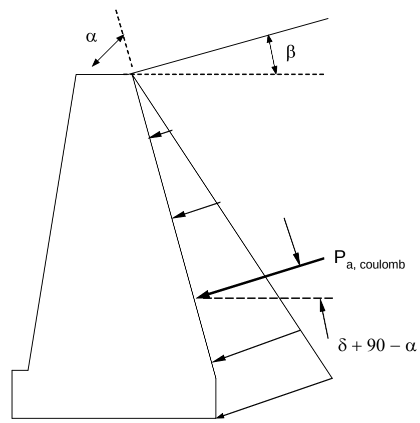
$P_{dl} := 700 \cdot \frac{kip}{60 \cdot ft}$ $P_{dl} = 11.667 \frac{kip}{ft}$

$P_{ll} := 215 \cdot \frac{kip}{60 \cdot ft}$ $P_{ll} = 3.583 \frac{kip}{ft}$

$V_{ss} := (P_{dl} + P_{ll}) \cdot b$ $V_{ss} = 1.525 \times 10^4 \, lbf$ $V_{ss} = 15.25 \, kip$

$H_{ss} := [(.1 \cdot P_{dl}) + (0.05 \cdot P_{ll})] \cdot b$ $H_{ss} = 1.346 \times 10^3 \, lbf$ $H_{ss} = 1.346 \, kip$

**Lateral Earth Pressure - use Coulomb - in failure, wedge of backfill soil slides upward along a plane matching the backwall of the gravity abutment**




$$K_{a\_rank} := \frac{\cos(\beta) - \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}}{\cos(\beta) + \left[(\cos(\beta))^2 - (\cos(\phi_1))^2\right]^{0.5}} \cdot \cos(\beta)$$

$K_{a\_rank} = 0.307$

**Coulomb Ka for granular backfill is very similar to the Rankine Value**

$$K_{a\_coulomb} := \frac{\sin(\phi_1 + \alpha)^2}{\left[\left(\sin(\alpha)^2 \cdot \sin(\alpha - \delta)\right) \cdot \left[1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{(\sin(\alpha - \delta) \cdot \sin(\alpha + \beta))}}\right]^2\right]}$$

$K_{a\_coulomb} = 0.354$

**Resultant Earth Pressure from backfill**

$$P_{a1} := \frac{1}{2} \cdot \gamma_1 \cdot H^2 \cdot K_{a\_coulomb} \cdot b$$

$P_{a1} = 18.503\,\text{kip}$

per linear foot of abutment

### Vertical Earth Pressure:

$E_{avert} := \sin(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{avert} = 9.53 \times 10^{3}\ lbf$ $\quad E_{avert} = 9.53\ kip$ per linear foot of wall

### Horizontal Earth Pressure:

$E_{ahoriz} := \cos(\delta + 90\cdot deg - \alpha)\cdot P_{a1}$ $\quad E_{ahoriz} = 15859.803\ lbf$ $\quad E_{ahoriz} = 15.86\ kip$ per lin ft of wall

**Assume approach slab; Neglect force due traffic - ~~model w/ surcharge of 2' of soil (using Coulomb earth pressure theory)~~**

$s := 0\cdot ft\cdot \gamma_1$ $\quad s = 0\ psf$

$E_s := K_{a\_coulomb}\cdot s\cdot H\cdot b$ $\quad E_s = 0\ kip$

### Vertical Surcharge Earth Pressure:

$E_{surch\_vert} := \sin(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_vert} = 0\ kip$ per lnr foot of wall

### Horizontal Surcharge Earth Pressure:

$E_{surch\_horiz} := \cos(\delta + 90\cdot deg - \alpha)\cdot E_s$ $\quad E_{surch\_horiz} = 0\ kip$ per lin ft of wall

## Factor of safety against overturning and sliding

### Resisting moments - abutment composed of granite stone masonry

$A_1 := 7.17\cdot 3\cdot ft^2$ $\quad F_1 := A_1\cdot \gamma_c\cdot b$ $\quad x_1 := \frac{7.17\cdot ft}{2} + T$ $\quad M_{r1} := F_1\cdot x_1$ $\quad M_{r1} = 21.247\ kip\cdot ft$

$A_2 := 5.3\cdot 1\cdot ft^2$ $\quad F_2 := A_2\cdot \gamma_c\cdot b$ $\quad x_2 := \frac{5.3\cdot ft}{2} + T + j$ $\quad M_{r2} := F_2\cdot x_2$ $\quad M_{r2} = 5.605\ ft\cdot kip$

$A_3 := 5.2\cdot ft\cdot 2\cdot ft$ $\quad F_3 := A_3\cdot \gamma_c\cdot b$ $\quad x_3 := \frac{5.2}{2}\cdot ft + T + j$ $\quad M_{r3} := F_3\cdot x_3$ $\quad M_{r3} = 10.92\ ft\cdot kip$

$A_4 := (5.0\cdot 2)\cdot ft^2$ $\quad F_4 := A_4\cdot \gamma_c\cdot b$ $\quad x_4 := \frac{5.0}{2}\cdot ft + T + j$ $\quad M_{r4} := F_4\cdot x_4$ $\quad M_{r4} = 10.35\ ft\cdot kip$

$A_5 := 5.0 \cdot 2 \cdot ft^2$ $F_5 := A_5 \cdot \gamma_c \cdot b$ $x_5 := \frac{5.0}{2} \cdot ft + T + j$ $M_{r5} := F_5 \cdot x_5$ $M_{r5} = 10.35\ ft \cdot kip$

$A_6 := 5.0 \cdot 2 \cdot ft^2$ $F_6 := A_6 \cdot \gamma_c \cdot b$ $x_6 := \frac{5.0}{2} \cdot ft + T + j$ $M_{r6} := F_6 \cdot x_6$ $M_{r6} = 10.35\ ft \cdot kip$

$A_7 := 5.0 \cdot 2 \cdot ft^2$ $F_7 := A_7 \cdot \gamma_c \cdot b$ $x_7 := \frac{5.}{2} \cdot ft + T + j$ $M_{r7} := F_7 \cdot x_7$ $M_{r7} = 10.35\ ft \cdot kip$

$A_8 := 3.92 \cdot 2 \cdot ft^2$ $F_8 := A_8 \cdot \gamma_c \cdot b$ $x_8 := \frac{3.92}{2} \cdot ft + T + j$ $M_{r8} := F_8 \cdot x_8$ $M_{r8} = 7.479\ ft \cdot kip$

$A_9 := 2 \cdot 4 \cdot ft^2$ $F_9 := A_9 \cdot \gamma_c \cdot b$ $x_9 := \frac{2}{2} \cdot ft + T + j$ $M_{r9} := F_9 \cdot x_9$ $M_{r9} = 6.48\ ft \cdot kip$

$A_{10} := 1.4 \cdot ft \cdot \frac{17}{2} \cdot ft$ $F_{10} := A_{10} \cdot \gamma_c \cdot b$ $x_{10} := \frac{2 \cdot 1.4}{3} \cdot ft + T$ $M_{r10} := F_{10} \cdot x_{10}$ $M_{r10} = 7.021\ ft \cdot kip$

$A_{11} := 3 \cdot 13 \cdot ft^2$ $F_{11} := A_{11} \cdot \gamma_c \cdot b$ $x_{11} := 8 \cdot ft$ $M_{r11} := F_{11} \cdot x_{11}$ $M_{r11} = 46.8\ ft \cdot kip$

**Resisting Moments - Soil over backwall and footing - neglect for Coulomb Analysis**

$A_{12} := 0 \cdot ft^2$ $F_{12} := A_{12} \cdot \gamma_1 \cdot b$ $x_{12} := 4.5 \cdot ft$ $M_{r12} := F_{12} \cdot x_{12}$ $M_{r12} = 0\ ft \cdot kip$

**Resisting moment due to (1) dead load on bridge seat, (2) vertical component of the Traffic Surcharge acting on the backface, and, (3) vertical component of Coulomb earth pressure acting on the backface.**

1. $M_{rDL} := P_{dl} \cdot (T + j + cl) \cdot b$ $M_{rDL} = 57.167\ ft \cdot kip$

2. $M_{rSCH} := E_{surch\_vert} \cdot (5.7 \cdot ft)$ $M_{rSCH} = 0\ ft \cdot kip$ acts downward on backface at H/2 above BOF $\frac{H}{2} = 14.45\ ft$

3. $M_{r\_Pa} := E_{avert} \cdot (6.17 \cdot ft)$ $M_{r\_Pa} = 58.797\ ft \cdot kip$ acts downward on backface at H/3 above BOF $\frac{H}{3} = 9.633\ ft$

## Driving moments

$M_{d\_surch} := E_{surch\_horiz} \cdot \frac{1}{2} \cdot H$ $\qquad M_{d\_surch} = 0\ ft \cdot kip$

$\boxed{M_{d\_Pa} := E_{ahoriz} \cdot \frac{1}{3} \cdot H}$ $\qquad M_{d\_Pa} = 1.528 \times 10^{5}\ ft \cdot lbf$ $\qquad M_{d\_Pa} = 152.783\ kip \cdot ft$ $\qquad M_{d\_Pa} = 152.783\ kip \cdot ft$

$M_{d3} := H_{ss} \cdot 21 \cdot ft$ $\qquad M_{d3} = 28.262\ ft \cdot kip$

**do not include horizontal component of LL and DL in the load group. Override lateral load and moment due to this component:**

$M_{d3} := 0 \cdot ft \cdot kip$ $\qquad H_{ss} := 0 \cdot kip$

## Summation of forces and moments

$\Sigma V := F_1 + F_2 + F_3 + F_4 + F_5 + F_6 + F_7 + F_8 + F_9 + F_{10} + F_{11} + F_{12} + E_{avert} + E_{surch\_vert} + P_{dl} \cdot b$

$\Sigma V = 4.279 \times 10^{4}\ lbf$

$\Sigma V = 42.789\ kip$

$\Sigma H := E_{ahoriz} + E_{surch\_horiz} + H_{ss}$

**do not include horizontal component of LL and DL in the load group (Hss)**

$\Sigma H = 15.86\ kip$

$\Sigma H = 15.86\ kip$

$\Sigma M_r := M_{r1} + M_{r2} + M_{r3} + M_{r4} + M_{r5} + M_{r6} + M_{r7} + M_{r8} + M_{r9} + M_{r10} + M_{r11} + M_{r12} + M_{rSCH} + M_{rDL} + M_{r\_Pa}$

$\Sigma M_r = 2.629 \times 10^{5}\ ft \cdot lbf$

$\Sigma M_r = 262.915\ ft \cdot kip$

$\Sigma M_d := M_{d\_Pa} + M_{d\_surch} + M_{d3}$

$\Sigma M_d = 152.783\ ft \cdot kip$

$\Sigma M_d = 152.783\ kip \cdot ft$

## Factor of safety against overturning

$$FS_{ot} := \frac{\Sigma M_r}{\Sigma M_d}$$

$FS_{ot} = 1.721$

AASHTO required factor of safety is 2.0

## Factor of safety against sliding

friction at base + adhesion

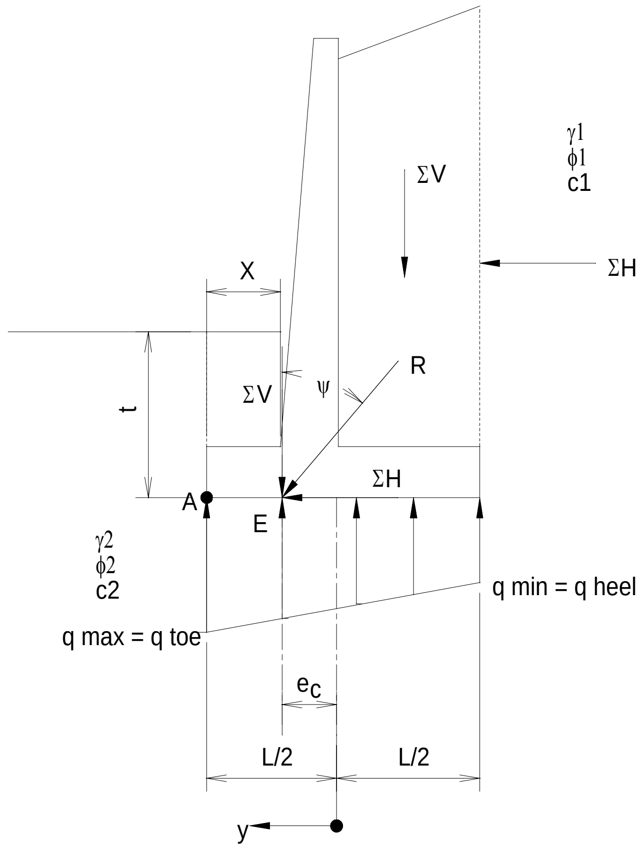
$\tan\left(\delta_2\right) = 0.445$

$$FS_{sl} := \frac{\left[(\Sigma V)\cdot\tan\left(\delta_2\right)\right] + \left[(B\cdot b)\cdot c_2\right]}{\Sigma H}$$

$FS_{sl} = 1.201$

AASHTO required factor of safety is 1.5

## Bearing Capacity Factor of Safety




### determine net moment

$M_{net} := \Sigma M_r - \Sigma M_d$ $\qquad M_{net} = 1.101 \times 10^5 \, lbf \cdot ft$

### location of resultant

$AE := \frac{M_{net}}{\Sigma V}$ $\qquad AE = 2.574\,ft$ $\qquad X := AE$

### determine eccentricity, if e > B/6, reproportion

$e_c := \frac{B}{2} - AE$ $\qquad e_c = 3.926\,ft$

$\frac{B}{6} = 2.167\,ft$ $\qquad$ Not good

### Determine pressure distribution under footing

$$q = \frac{\Sigma V}{A} + \frac{M_{net} \cdot y}{I}$$

$$q = \frac{\Sigma V}{A} - \frac{M_{net} \cdot y}{I}$$

where:A = area = b*B
I = moment of inertia = 1/12*B*2

### solving for $q_{max}$ and $q_{min}$

$q_{max} := \left[\frac{\Sigma V}{B} \cdot \left(1 + \frac{6 \cdot e_c}{B}\right)\right] \cdot \frac{1}{b}$ $\qquad q_{max} = 9256\,psf$ $\qquad q_{max} = 9.256\,ksf$ $\qquad q_{toe} := q_{max}$

$q_{min} := \left[\frac{\Sigma V}{B} \cdot \left(1 - \frac{6 \cdot e_c}{B}\right)\right] \cdot \frac{1}{b}$ $\qquad q_{min} = -2673\,psf$ $\qquad q_{min} = -2.673\,ksf$ $\qquad q_{heel} := q_{min}$

$B_e := B - 2 \cdot e_c$

**Allowable Bearing Pressure:** $\qquad q_u := 24 \cdot ksf$ $\qquad q_{allow} := \frac{q_u}{3}$ $\qquad q_{allow} = 8\,ksf$

**Factor of Safety against BC failure:** $\qquad FS_{bc} := \frac{q_u}{q_{max}}$ $\qquad FS_{bc} = 2.593$

AASHTO recommends a FS of 3