December 4, 2017

STATE OF RHODE ISLAND AND PROVIDENCE PLANTATION DEPARTMENT OF ADMINISTRATION
DIVISION OF PURCHASES **BID NO. 7565501**

RHODE ISLAND DEPARTMENT OF TRANSPORTATION RHODE ISLAND CONTRACT NO. 2017-DB-027
FEDERAL-AID PROJECT NO. FAP No. BRO-2950(003)

CRANSTON I-295 BRIDGES CONTRACT 1

CITY/TOWN OF Cranston

COUNTY OF Providence

NOTICE TO PROSPECTIVE BIDDERS

ADDENDUM NO. 2: Prospective bidders and all concerned are hereby notified of the following changes in the Plans, Specifications, Proposal and Distribution of Quantities for this contract. These changes shall be incorporated in the Plans, Specifications, Proposal and Distribution of Quantities, and shall become an integral part of the Contract Documents

A. Contract Documents

1. Required Forms: Schedule of Participation by DBE and DBE Letter of Intent to Perform

   Use the revised/updated Forms attached.

2. Part A Instructions for Respondents: Section 2.3 Project Milestone Schedule, Page 5 of 32

   Revise the schedule and liquidated damages as shown in the attached document.

3. Part A Instructions for Respondents: Section 4.7a Respondent's Qualifications, Organizational Chart, Page 18 of 32

   Clarification regarding the D-B Team's ability to meet the SUBSTANTIAL Completion date as shown in the attached document.

4. Part B Project Technical Requirements: Section 2.4.3.1b Description of Structural Elements, Page 15 of 40

   Revise the requirements regarding the use of Stay-In-Place Forms as shown in the attached document.

5. Part B Project Technical Requirements: Section 2.6.2 Wetland and Water Quality Permits, Page 23 of 40

   Revise the requirements regarding the attenuation of peak stormwater flow as shown in the attached document and per revised BTC Plan Sheets 51-53 & 95 (attached.)

6. Part D Appendicies: Geotechnical Reports

   Revised (PE Stamp provided) Geotechnical Reports attached.

ADDENDUM NO. 2 David W. Fish

Manager, Division of Project Management

DBE Letter of Intent to Perform

PROJECT:

NAME OF PROPOSER: _____

FROM: _____
(Disadvantaged Business Enterprise)

TO: _____
(Name of Contractor)

1. My company is currently certified as a Disadvantaged Business Enterprise (DBE) by the state of Rhode Island. There have been no changes affecting the ownership, control or independence of my company since my last certification review.

2. If any such change occurs prior to my company's completion of this proposed work, I will give written notification to your firm and RIDOT.

3. My firm will provide to you, upon request, for the purpose of obtaining subcontractor approval: (a) a resume stating the qualifications and experience of the superintendent or foreperson who will supervise on-site work; (b) a list of equipment owned or leased by my firm for use on the project; and (c) a list of all projects (public or private) which my firm is currently performing, is committed to perform, or intends to make a commitment to perform. I shall include for each project the names and telephone number of a contact person for the contracting organization, the dollar value of the work, a description of the work, and my firm's work schedule for the project.

4. If you are awarded the contract, my company intends to enter into an agreement with your firm to perform the items of work or other activity described on the following sheet for the prices indicated.

5. My firm has the ability to manage, supervise and perform the activity described on the following page.

_____ _____
DBE Signature Date

Schedule of Participation by Disadvantaged Business Enterprises (DBE)

PROJECT:

BID NO.:

NAME OF PROPOSER:_____

Name, Address, and Phone Number of DBE	Name of Activity	DBE Contractor Activity Percent	Total Percent Eligible for Credit

DBE Percentage Total: _____%

SIGNATURE:_____ DATE:_____ TEL. NO.:_____

NAME AND TITLE (PRINT):_____

The RFP response requested will entail **ONE (1)** simultaneous submission including **TECHNICAL** proposals along with a **separately sealed** envelope containing the **PRICE PROPOSAL** for RIDOT's evaluation and final selection recommendation.

Submittals received must be in accordance with guidelines as outlined in this RFP and the State's General Conditions of Purchase which can be accessed online through the *Rhode Island Vendor Information Program, or "RIVIP*" as it is known at:

http://www.purchasing.ri.gov

All Respondents are advised to review all sections of this RFP thoroughly and to follow the instructions carefully. Failure to make a complete submission as described elsewhere herein may result in rejection of Respondent's submission.

2.3 Project Milestone Schedule

RIDOT currently anticipates conducting this procurement in accordance with the following list of milestones. This schedule is subject to revision and RIDOT reserves the right to modify this schedule as it finds necessary, at its sole discretion.

Advertise RFP	September 22, 2017
Deadline for Questions & Comments	December 8, 2017 @ Midnight
Proposal Submission Date	December 15, 2017 (11:30AM)
Evaluate Technical Proposals (Score and Rank)	January 10, 2018
Open Price Proposals	January 17, 2018 (Estimated)
Notice of Tentative Award	February 7, 2018 (Estimated)
Notice to Proceed	February 21, 2018 (Estimated)
Substantial Completion	October 15, 2020
~~Project Completion~~	~~December 15, 2020~~

Respondents are on notice that any earlier substantial completion date(s) identified by Respondent in response to this RFP will be deemed by RIDOT as the contractual substantial completion date(s) for this Project. Liquidated damages shall be based on the Schedule of Liquidated Damages in Standard Specification Section 108.08. The rate for liquidated damages shall be $2,350 per Calendar Day.

The D/B Team may propose a schedule that includes work through the winter shutdown period between December 15th and April 15th to the extent practical. The D/B Team must demonstrate that they can

on education and experience of individual.

The Environmental Manager will also be responsible for coordination with the RIDOT Cultural Resources Unit (CRU) to ensure the Project's compliance with all State and Federal Cultural Resource laws, regulations and procedures throughout the entire design/build process. This individual must be familiar with all stipulations that result from the cultural resource review process and ensure that all design changes arising after RI Historical Preservation & Heritage Commission approval of final design plans are coordinated through the CRU. In addition, this individual will be responsible for coordinating the review of all required samples, shop drawings, etc. with the CRU.

Organizational Chart: Furnish an organizational chart showing the "chain of command" and identifying major functions to be performed and their reporting relationships in managing, designing and constructing the Project. Additionally, furnish a narrative describing the functional relationships among participants listed on the organizational chart **(LIMITED TO ONE (1) PAGE).**

The Respondent shall provide an affirmative statement that the resources shown or indicated in the Proposal will be available if awarded the Contract. The Respondent shall discuss the current backlog of the Primary D-B Team Members and their capacity to perform the Project to achieve the Substantial Completion Date listed in Section 2.3.

The Respondent shall also explain the differing work locations, where key personnel will be located while working on this project, address how the coordination between the various D-B Team Members and Subcontractors will be managed and explain how the engineering process will be integrated with the construction process.

RELEVANT WORK EXPERIENCE:

The Respondent shall provide a description of EACH D-B TEAM Member's experience and qualifications, particularly with respect to experience in similar design-build bridge replacement/rehabilitation projects with a particular focus on the ability to deliver such projects on time and on budget. ***The LEAD CONTRACTOR and LEAD DESIGNER shall each identify and describe UP TO 3 PROJECTS with a construction value of a minimum of $5 MIL or more completed within the past TEN (10) YEARS*** which demonstrate adequate experience in areas such as:

- Construction using prefabricated bridge elements.
- Experience in successfully executing heavy lift construction.
- Traffic Management on Interstate/divided highways of comparable scale to this project.
- Relevant and verifiable evidence of good performance or lessons learned from previous projects and how these will benefit this Project.

For each project, provide the project name, owner's name, address, principal contact with current phone number and email address, dates of design/construction, construction value and description of the work

both the field and laboratory testing programs utilized. The plan shall also include a traffic control plan, a safety/hazard analysis plan, and a list of all permits required to perform the geotechnical investigation.

If additional geotechnical investigations are performed, a Final Geotechnical Report shall be developed by the D/B Team in accordance with Section 10.2.5 of the RIDOT LRFD Bridge Manual.

2.4.3 Description of Structural Elements

This Section covers the specific design and construction elements for the new bridge structures. The goal of the design and construction of all structural systems and components is to provide functionality, durability, constructability, ease of maintenance, safety, and aesthetics consistent with the context of the Project Site.

1. **Bridge Elements**

   a. **Structural Steel (073201/073221 only)**
      - As designated on the BTC Plans and in this RFP.

   b. **Decks (073201/073221 only)**
      - The bridge deck shall be comprised of high performance (HP) concrete with a minimum bridge deck thickness of 8 inches.
      - The bridge decks shall be protected with a Cold Spray-Applied Liquid Membrane Waterproofing system.
      - The deck and membrane shall be overlaid by a 3" minimum thickness Modified Class 9.5 HMA wearing surface in conformance with current RIDOT pavement standards.
      - For deck construction, stay-in-place (SIP) forms WILL~~will not~~ be allowed.

   c. **Prefabricated Bridge System (073001/073021 only)**
      - The prefabricated bridge system shall be a buried structure type and may consist of precast concrete arch/frame elements or corrugated steel structural plate systems capable of supporting the required design loads and providing a 75 year service life.
      - The system shall not require expansion or relief joints in the overlying pavement.
      - Beam-slab bridge types shall not be used.
      - The prefabricated systems shall be waterproofed in accordance with the manufacturer's recommendations to ensure the required service life.
      - If the system selected has discrete joints along its length, the joints shall be sealed with a waterproofing joint system in accordance with the manufacturer's requirements.

   d. **Deck Joints (073201/073221 only)**
      - Abutments: At the deck ends at the abutments, the new slab shall extend over the new backwalls per the "Fixed Joints at Abutments" details for backwall Type I shown in the RIDOT Bridge Design Standard Details. The standard details shall be modified as

2.6.2 Wetland and Water Quality Permits

The Department's coordination with regulatory agencies to date has determined that several wetland and water quality permits will be required including a Freshwater Wetlands Permit from the Rhode Island Department of Environmental Management (RIDEM) and Authorization under the Rhode Island Pollutant Discharge Elimination System (RIPDES) General Permit for Stormwater Discharge Associated with Construction Activity from the RIDEM; inclusive of preparing and complying with the requirements of a Stormwater Pollution Prevention Plan(SWPPP) (template attached). The D/B Team shall utilize the March 2015 Rhode Island Stormwater Design and Installation Standards Manual in the design of this project. The D/B Team is responsible for reviewing and understanding the performance standards and commitments made in all permits and approvals for the Project, as well as the standards and prohibitions of the respective regulations of these programs. The D/B Team shall be responsible for the preparation of all permit applications and supporting documentation, based on the D/B Team's final design. RIDOT as owner, will be the Permittee. Upon RIDOT review and approval of the necessary permit applications, RIDOT will submit them to the regulatory agencies. Should the D/B Team propose design changes acceptable to RIDOT, permitting requirements may also change. The D/B Team also remains responsible for obtaining any and all necessary amended permits required by the regulatory agencies.

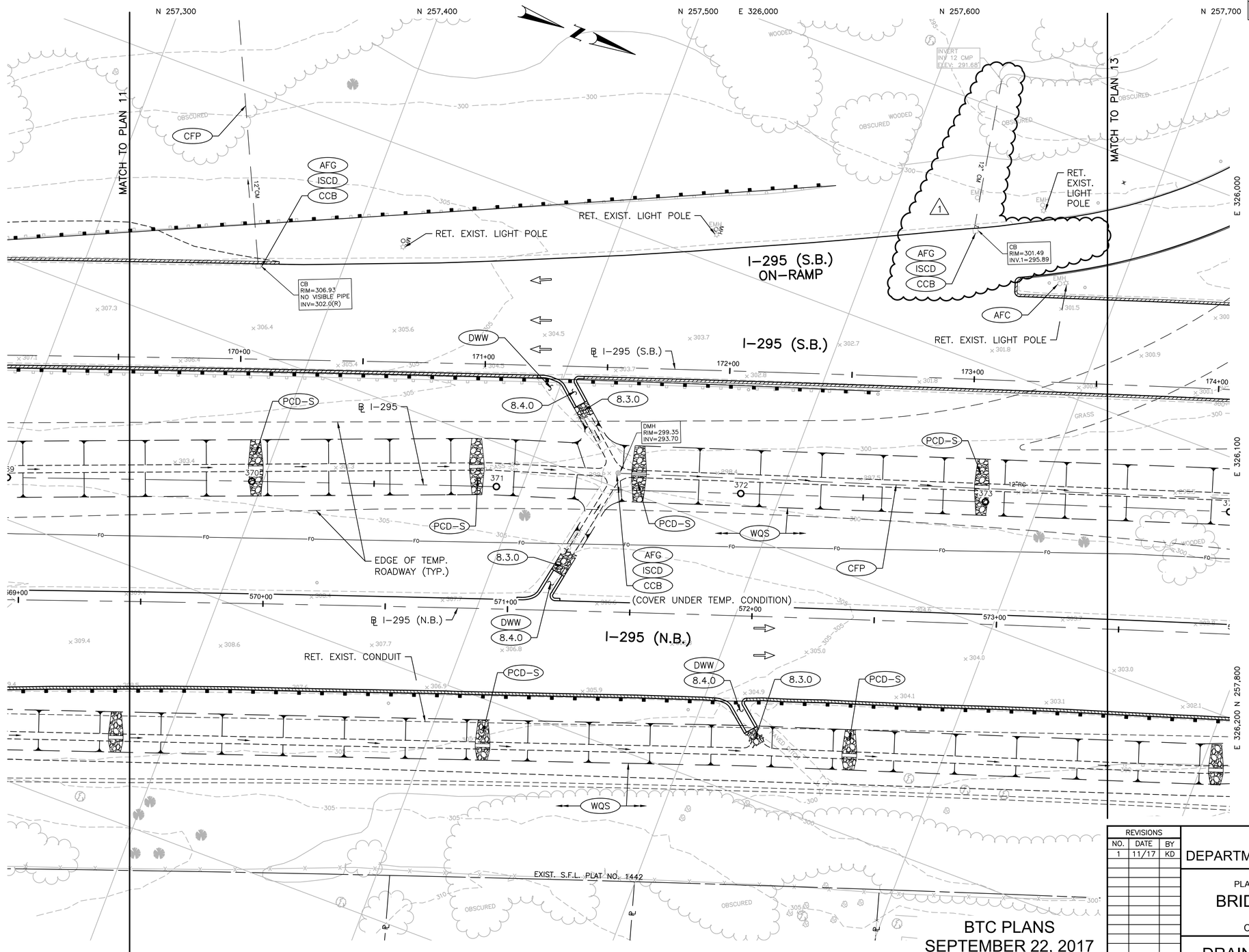
One of the goals of this project is to improve stormwater treatment at the site. The BTC Plans illustrate a concept to provide water quality infiltration swales along the I-295 approaches at select locations as well as infiltration basins within the ramp infields. The Respondent shall design and detail all proposed Stormwater BMP's. The stormwater BMP's shall satisfy the standards required for a redevelopment project and shall, at a minimum, be capable of treating at least 50% of the first flush water quality volume for the impervious area within the project's paving limits. In addition, a reduction in post-construction peak stormwater flow at the northwest quadrant of the Plainfield Pike interchange is required to help alleviate downstream flooding issues. At a minimum, the post-construction peak flow shall be reduced compared to the pre-construction (existing) flow by 10% at the 30" pipe outlet in this quadrant for the 2, 10, 50 and 100 year events. Providing more stormwater treatment than the minimum required will be considered as part of the Technical Proposal Evaluation.

The existing drainage structures and pipes within the limits of paving shall be cleaned and flushed of all sediment. All sediment removed shall be disposed of in accordance with State and Federal regulations.

The D/B Team shall utilize the March 2015 Rhode Island Stormwater Design and Installation Standards Manual and the Rhode Island Soil Erosion and Sediment Control Handbook, 1989 (revised 2014) in the design of this project. The D/B Team shall prepare and submit as soon as practicable prior to the 75/90% Design, a Preliminary Environmental Design Submission that conforms to the following:

- March 2015 Rhode Island Stormwater Design and Installation Standards Manual, Appendix A Checklist (as complete as possible at this design stage).
- Preliminary design strategy for environmental permitting (i.e. anticipated permit submissions based on the D/B Team's design concept).

FED. ROAD DIV. NO.	STATE	FEDERAL AID PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
	RI	BRO-2950(003)		51	102



BTC PLANS
SEPTEMBER 22, 2017

WSP USA Inc.
2348 POST ROAD
SUITE 27
WARWICK, RI 02886
TEL: +1 401.738.6600

REVISIONS		
NO.	DATE	BY
1	11/17	KD

RHODE ISLAND
DEPARTMENT OF TRANSPORTATION

PLANS FOR BRIDGE REPLACEMENT

BRIDGE NOS. 730 & 732

CRANSTON, RHODE ISLAND

DRAINAGE & UTILITY PLAN
NO. 12

CHECKED BY _____ DATE _____ SCALE 1"=20'

0114T_V1_051_DRAINUTL012_ADD02

FED. ROAD DIV. NO.	STATE	FEDERAL AID PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
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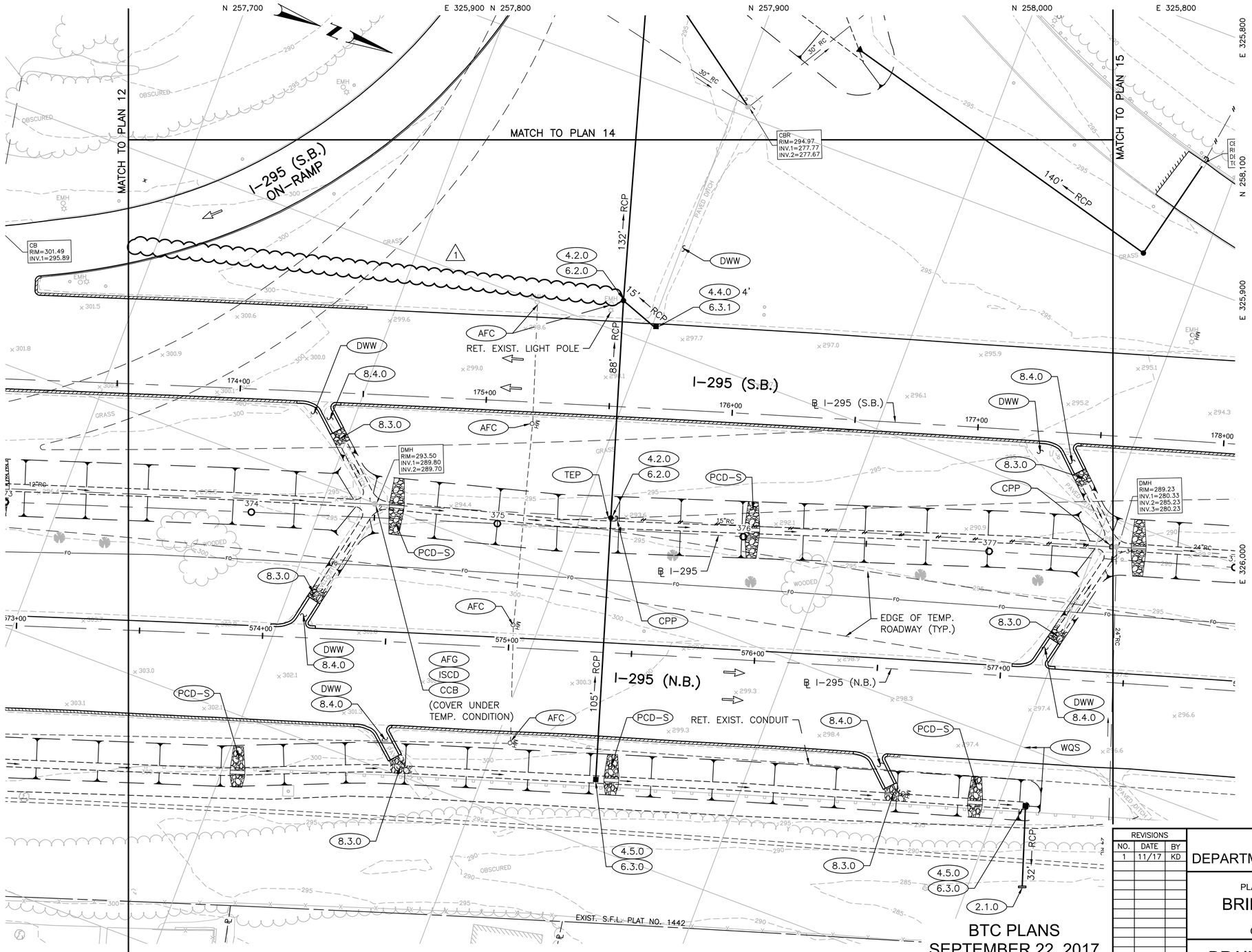

BTC PLANS
SEPTEMBER 22, 2017

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REVISIONS		
NO.	DATE	BY
1	11/17	KD

RHODE ISLAND
DEPARTMENT OF TRANSPORTATION

PLANS FOR BRIDGE REPLACEMENT

BRIDGE NOS. 730 & 732

CRANSTON, RHODE ISLAND

DRAINAGE & UTILITY PLAN NO. 13

CHECKED BY _____ DATE _____ SCALE 1"=20'

0114T_V1_052_DRAINUTL013_ADD02

FED. ROAD DIV. NO.	STATE	FEDERAL AID PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
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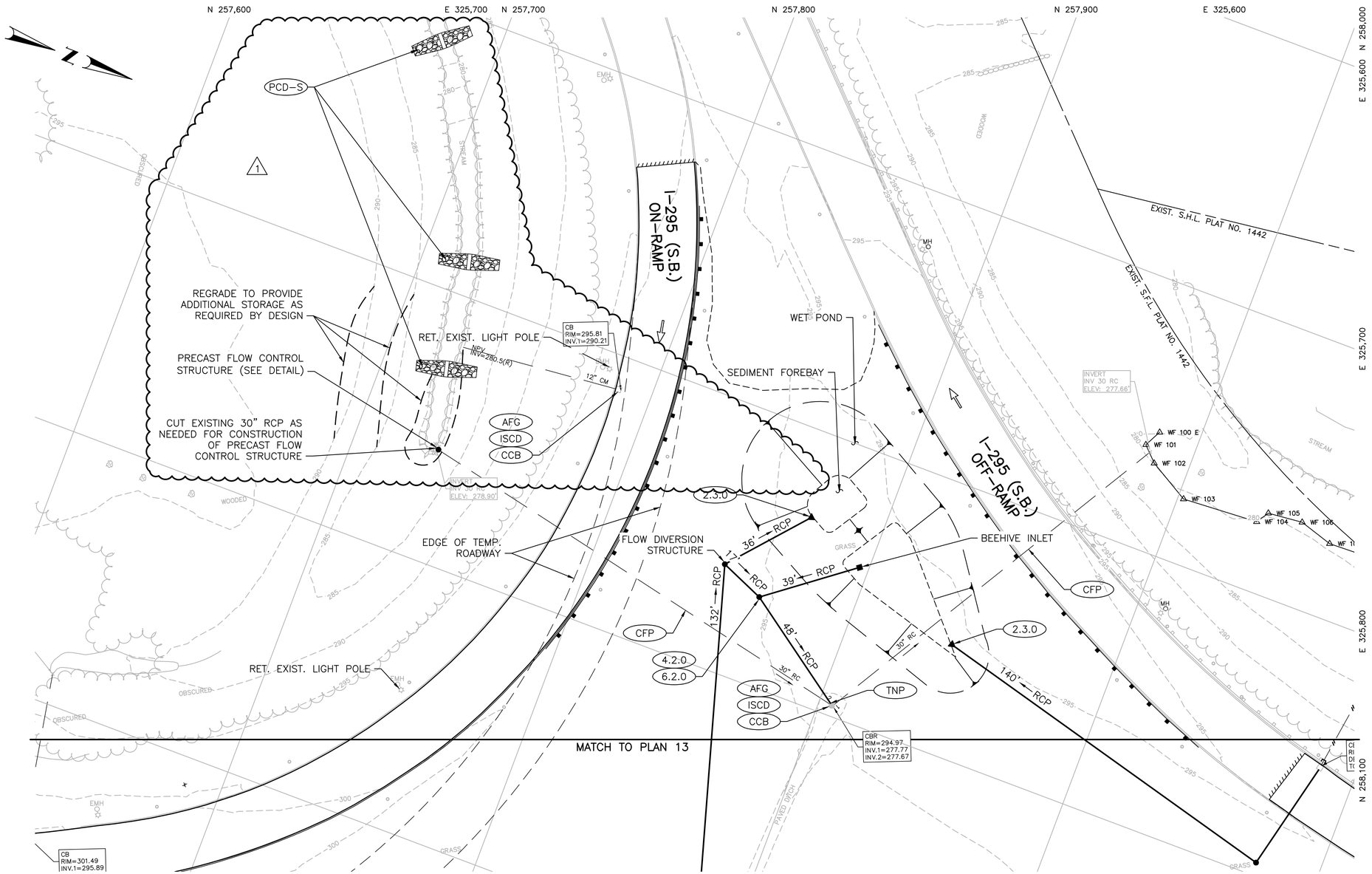

REVISIONS		
NO.	DATE	BY
1	11/17	KD

RHODE ISLAND
DEPARTMENT OF TRANSPORTATION

PLANS FOR BRIDGE REPLACEMENT

BRIDGE NOS. 730 & 732

CRANSTON, RHODE ISLAND

DRAINAGE & UTILITY PLAN
NO. 14

CHECKED BY _____ DATE _____ SCALE 1"=20'

BTC PLANS
SEPTEMBER 22, 2017

WSP USA Inc.
2348 POST ROAD
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WARWICK, RI 02886
TEL: +1 401.738.6600

0114T_V1_053_DRAINUTL014_ADD02

FED. ROAD DIV. NO.	STATE	FEDERAL AID PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
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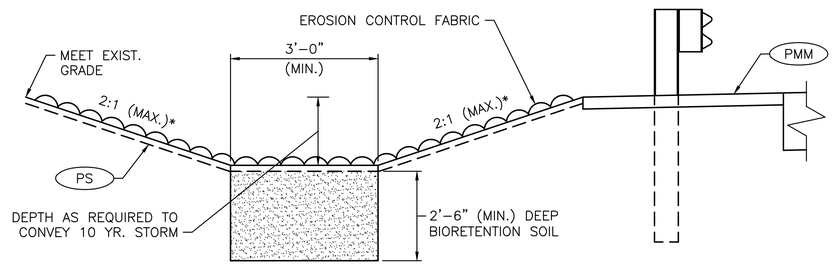

* – SIDE SLOPES STEEPER THAN 4:1 REQUIRE GUARD RAIL PROTECTION.

WATER QUALITY SWALE DETAIL

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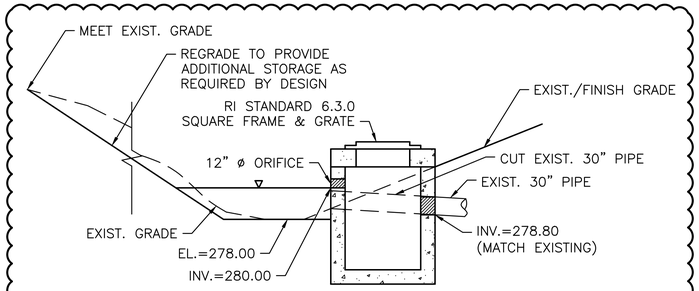

PRECAST FLOW CONTROL STRUCTURE DETAIL

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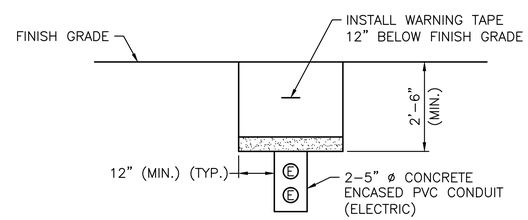

CONDUIT TRENCH DETAIL

SCALE: N.T.S.

NOTE:

CONDUIT MATERIAL AND INSTALLATION SHALL BE IN ACCORDANCE WITH THE UTILITY OWNER'S REQUIREMENTS.

BTC PLANS
SEPTEMBER 22, 2017

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REVISIONS		
NO.	DATE	BY
1	11/17	KD

RHODE ISLAND
DEPARTMENT OF TRANSPORTATION

PLANS FOR BRIDGE REPLACEMENT

BRIDGE NOS. 730 & 732

CRANSTON, RHODE ISLAND

MISCELLANEOUS DETAILS

CHECKED BY _____ DATE _____ SCALE AS NOTED

0114T_V1_095_DETAIL_ADD02

Geotechnical Report

for

Rhode Island Department of Transportation

Proposed Superstructure Replacement Project

for

Bridge No. 732

I-295 Southbound & Northbound

Scituate Avenue (C) to Scituate Avenue (J)

Cranston - Johnston, Rhode Island

Date: 8/31/2017

Prepared for:
WSP USA Inc.
2348 Post Road, Suite 27
Warwick, Rhode Island 02886
Phone: 401-738-6600

Prepared by:
Lamson Engineering Corporation
437 Cherry Street, Room # 109
Newton, Massachusetts 02465
Phone: 617-558-0101

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1.0 EXECUTIVE SUMMARY

Based on the 1964 and 1966 boring exploration results and our investigation, the existing foundations can support the design loads exerting from the Proposed Bridge Superstructure.

The nominal and factored bearing resistance values for the existing foundations are included in Section 6.0, Summary, of this report.

The elevations shown on the attached existing bridge plans (Appendix 1) were based on 1929 NGVD datum. The current design is based on 1988 NAVD datum. To obtain elevation values based on the 1988 NAVD datum, subtract 0.81' from those based on the 1929 NGVD datum. Elevations shown in this report, except those in the existing Plans, are in reference to the 1988 NAVD Datum.

2.0 INTRODUCTION

2.1 Scope of Report

The purpose of this report is to document the 1964 and 1966 subsurface exploration results and evaluate the existing substructure foundations based on the proposed superstructure system, provide recommendations, develop engineering design values, and identify possible geotechnical related construction issues. The design values will be prepared using the AASHTO Standard Specifications for Highway Bridges based on the Load Resistance Factor Design (LRFD) method with HL-93 truck loading.

2.2 Existing Bridge & Foundation

Bridge No. 732 is located on I-295 Southbound & Northbound over Scituate Avenue (C) to Scituate Avenue (J) in the City of Cranston/Johnston as illustrated on Figure 1, 'Project Location Plan'.

The existing southbound bridge is a single-span, steel rolled beam composited with 7¼" reinforced concrete deck structure supported by reinforced concrete abutments. The skewed span length of the structure is 118'-2½" ±. The structure carries three (3) lanes of traffic with a curb to curb width of 43'±. The existing abutments are supported by reinforced concrete spread footings. The existing wingwalls were also constructed of reinforced concrete supported by spread footings.

The existing northbound bridge, including the foundations, was constructed in a similar manner to the southbound bridge, as described above.

The existing northbound and southbound superstructures will be replaced. There are no known foundation settlements or performance issues with the existing foundations. Therefore, the existing substructure should be considered for reuse to support the proposed superstructure.

2.3 Proposed Bridge Superstructure Replacement

The design concept of the proposed superstructure replacement can be found on the proposed bridge plans prepared by WSP, who is the Prime Design Engineer for the entire project. Lamson

Engineering Corporation (LEC) is a sub-consultant to WSP responsible for the geotechnical design of the project.

The proposed southbound bridge is a single-span, steel plate girder with 8" precast concrete deck panel structure supported by existing reinforced concrete abutments. The skewed span length of the structure is 118'-2½" ±. The structure carries three (3) lanes of traffic with a curb to curb width of 43'±. The bridge seat will be reconstructed on top of the existing abutments.

The proposed northbound bridge superstructure and bridge seat will be constructed similar to the southbound bridge, as described above.

2.4 Temporary Bridge

A temporary Acrow Bridge will be constructed during the stage construction of the proposed superstructure replacement. The temporary bridge carries three (3) lanes of traffic with a curb to curb width of 42'. The temporary bridge will be removed after the proposed bridge superstructure construction is completed.

2.5 References

The following references were used in the preparation of this report:

- AASHTO LRFD Bridge Design Specifications, 2014, 7th Edition with 2015 & 2016 Interim Revisions.
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Edition with 2012, 2014, and 2015 Interim Revisions.

3.0 SUBSURFACE CONDITIONS

3.1 General

Based on the previous 1964 & 1966 boring program, refusal materials were encountered at depths varying from 4'-0" to 11'-0". Surficial geologic deposits above refusal consist of sandy loam on medium dense to very dense, fine to coarse sand, gravel materials on top of Granite / Quartz Bedrock.

A new boring program was not conducted for this superstructure replacement project.

3.2 1964 & 1966 Subsurface Exploration Program

A subsurface exploration program consisting of 11 test borings was performed during June 1964 and August 1966, as indicated on Figure 2, '1964 & 1966 Boring Location Plan'.

Southbound Bridge

Borings B-119 and B-120 were conducted for the North Abutment. Borings B-121 and B-122 were conducted for the South Abutment. Boring B-17 was conducted between the Abutments.

Northbound Bridge

Borings B-123 and B-211 were conducted for the North Abutment. Borings B-124 and B-125 were conducted for the South Abutment. Boring B-18 was conducted for the North Abutment.

The soil samples were generally taken using a 2" outside diameter split spoon sampler driven 18" into soil by a 140-pound hammer falling 30". Per Standard Penetration Test requirements, blows per 6" were recorded. The materials at the site, as indicated by the obtained samples, are included on the boring logs and are also included on the soil profile in Figures 3 to 6.

As indicated on the logs, the predominant materials at the site below the existing abutment foundation are medium dense to very dense, fine to coarse sand and gravel materials. No peat or clay materials were found at the site during the previous exploration operation. For detailed information on each boring, refer to the 1967 Existing Bridge Plan, included in Appendix 1 of this report.

3.3 Subsurface Profiles

The subsurface conditions, as reflected in the present boring exploration results, are provided below. Soil profiles of the site, based on the boring results, are included in Figures 3 to 6 of the report.

Boring B-119 (For Southbound North Abutment)
Ground Surface is at Elevation 289.7'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, sandy Topsoil
S2: 5'- 6'6"	Dry, medium dense, fine to coarse Sand
10'	Top of bedrock (Granite)

No ground water was recorded.

Boring B-120 (For Southbound North Abutment)
Ground Surface is at Elevation 291.7'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, sandy Loam
S2: 5'- 6'6"	Dry, dense, fine to coarse Sand
7'6"	Top of bedrock (Granite)

No ground water was recorded.

Boring B-121 (For Southbound South Abutment)
Ground Surface is at Elevation 286.0'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, sandy Loam
S2: 5'- 6'6"	Dry, dense, fine to coarse Sand
8'	Refusal

No ground water was recorded.

Boring B-122 (For Southbound South Abutment)
Ground Surface is at Elevation 287.0'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, sandy Loam
1'6"-4'	Dry, fine to coarse Sand
4'	Refusal

No ground water was recorded.

Boring B-123 (For Northbound North Abutment)
Ground Surface is at Elevation 293.2'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, dense, sandy Loam
S2: 5'- 6'	Dry, very dense, fine to coarse Sand
6'	Top of Bedrock (Granite)

No ground water was recorded.

Boring B-124 (For Northbound South Abutment)
Ground Surface is at Elevation 292.7'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, dense, sandy Loam
S2: 5'- 6'	Dry, very dense, fine to coarse Sand
6'	Refusal

No ground water was recorded.

Boring B-125 (For Northbound South Abutment)
Ground Surface is at Elevation 296.2'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, sandy Loam
1'6"-4'	Dry, fine to coarse Sand
4'	Top of Bedrock (Granite)

No ground water was recorded.

Boring B-211 (For Northbound North Abutment)
Ground Surface is at Elevation 294.2'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, dense, sandy Loam
S2: 5'- 6'6"	Dry, dense, fine to coarse Sand
7'	Refusal

No ground water was recorded.

Boring B-17 (For Southbound)
Ground Surface is at Elevation 231.2'

Sample Depth (ft)	Soil Description
S1: 0'- 1'6"	Dry, very dense, medium to coarse Sand and medium Gravel
S2: 5'- 6'6"	Dry, very dense, medium to coarse Sand and medium Gravel
8'	Refusal

No ground water was recorded.

Boring B-18 (For Northbound)
Ground Surface is at Elevation 296.0'

Sample Depth (ft)	Soil Description
S1: 0"- 1'6"	Dry, medium dense, fine to medium Sand, little trace gravel (Fill).
2' - 5'	Dry, fine to medium Sand, some medium to fine gravel.
5'	Top of Bedrock (Quartz)

No ground water was recorded.

3.4 Bedrock Information

Bedrock at the site was encountered ranging from Elev. 279.9 to Elev. 292.2 as shown in the table below, with its average top at approximately Elev. 286.9.

Table 3.4: Bedrock Information

Boring No.	Ground Elevation (Feet)	Depth from Previous Existing Ground	Top of Bedrock (Elev.)	% of Recovery
B-119	289.7	10'-0"	Elev. 279.7	77%
B-120	291.7	7'-6"	Elev. 284.2	80%
B-123	293.2	6'-0"	Elev. 287.2	80%
B-125	296.2	4'-0"	Elev. 292.2	82%
B-18	296.0	5'-0"	Elev. 291.0	62%

3.5 Groundwater Information

Based on the logs shown on the existing plans, no groundwater information was recorded.

3.6 Seismic Design Category Evaluation

All seismic analysis and design of bridges shall be performed in accordance with the 'AASHTO Guide Specifications for LRFD Seismic Bridge Design'. Based on 2011 Guide Specifications, Table 3.4.2.1-1 (same as AASHTO Table 3.10.3.1-1) for site soil type and profile, the site class is C (F_{pga} = 1.2, F_a = 1.2, F_v = 1.7). With these results and Appendix 3, 'USGS Seismic Hazard Map for the 2,500-year return period', the following can be obtained:

Acceleration Coefficient (A_s) = 0.092g

Design Spectral Acceleration Coefficient at 0.2-sec Period (S_{DS}) = 0.160g
Design Spectral Acceleration Coefficient at 1.0-sec Period (S_{D1}) = 0.087g

Since S_{D1} is less than 0.15g, the Seismic Design Category (SDC) is A. This is based on Guide Specifications Table 3.5-1 (same as AASHTO Table 4.7.4.3.1-1).

3.7 Liquefaction Potential

A liquefaction assessment is not required for structures in Seismic Design Category (SDC) A. The existing spread footing is supported by medium dense to very dense sand and gravel materials. Therefore, the existing foundation is not susceptible to the seismic soil liquefaction effect.

4.0 FOUNDATION RECOMMENDATIONS

4.1 Existing Bridge Foundation

Based on the information obtained from the 1964 & 1966 boring exploration program, the existing abutment footings can be reused to support the design loads for the proposed replacement superstructures.

The existing bottoms of footings for Northbound are at approximately Elev. 290.2 and Elev. 292.7 for North Abutment and South Abutment, respectively. The existing bottoms of footings for Southbound are at approximately Elev. 289.2 and Elev. 291.7 for North Abutment and South Abutment, respectively. The abutment footing bearing materials are predominately medium dense to very dense sand and gravel materials. These are competent materials capable of supporting the proposed bridge superstructure.

The nominal and factored bearing resistance values for the existing foundations are included in Section 6 and Appendix 2 of the report.

4.2 Temporary Bridge Foundation

Based on the information obtained from the 1964 & 1966 boring exploration program, presumptive bearing resistances for the temporary bridge foundation are provided in Section 6 and Appendix 2 of the report.

4.3 Proposed Substructure Design Parameters

The following parameters are recommended for design at the back of the abutment wall:

Moist Unit Weight of Soil = 120 pcf
Soil Friction Angle = 32°
Active Lateral Earth Pressure Coefficient = 0.275
At-Rest Lateral Soil Pressure Coefficient = 0.470
(Based on AASHTO Table 10.5.5.2.2-1)

4.4 Soil Parameters

Based on the information from the boring logs, the following soil parameters are recommended:

Table 4.4: Recommended Soil Parameters

Structure		**Soil below Footing**	**Soil Unit Weight, γ (below footing)**	**Soil Friction Angle, ϕ_{f2} (below footing)**	**Boring #**
Southbound Structure	South Abutment	Medium Dense Fine to Coarse Sand	120 pcf	34°	B-121
	North Abutment	Medium Dense Fine to Coarse Sand	120 pcf	34°	B-119
Northbound Structure	South Abutment	Medium Dense Fine to Coarse Sand	120 pcf	34°	B-125
	North Abutment	Dense Fine to Coarse Sand	125 pcf	36°	B-211

5.0 CONSTRUCTION CONSIDERATIONS

Since this is a superstructure replacement project, there are no major bridge foundation, earth excavation, or dewatering activities anticipated. Construction should not be performed until the affected overhead or underground utilities are relocated.

6.0 SUMMARY

Table 6.1: Summary of Bearing Resistance

Structure		Footing Width (ft)	Factored Bearing Pressure (ksf)	Nominal Bearing Resistance (ksf)	Factored Bearing Resistance (ksf)
Southbound Bridge	South Abutment	9.5	5.2	15.9	7.1
	North Abutment	10.5	5.2	15.9	7.1
Northbound Bridge	South Abutment	10.5	5.2	15.9	7.1
	North Abutment	11.0	5.2	15.9	7.1

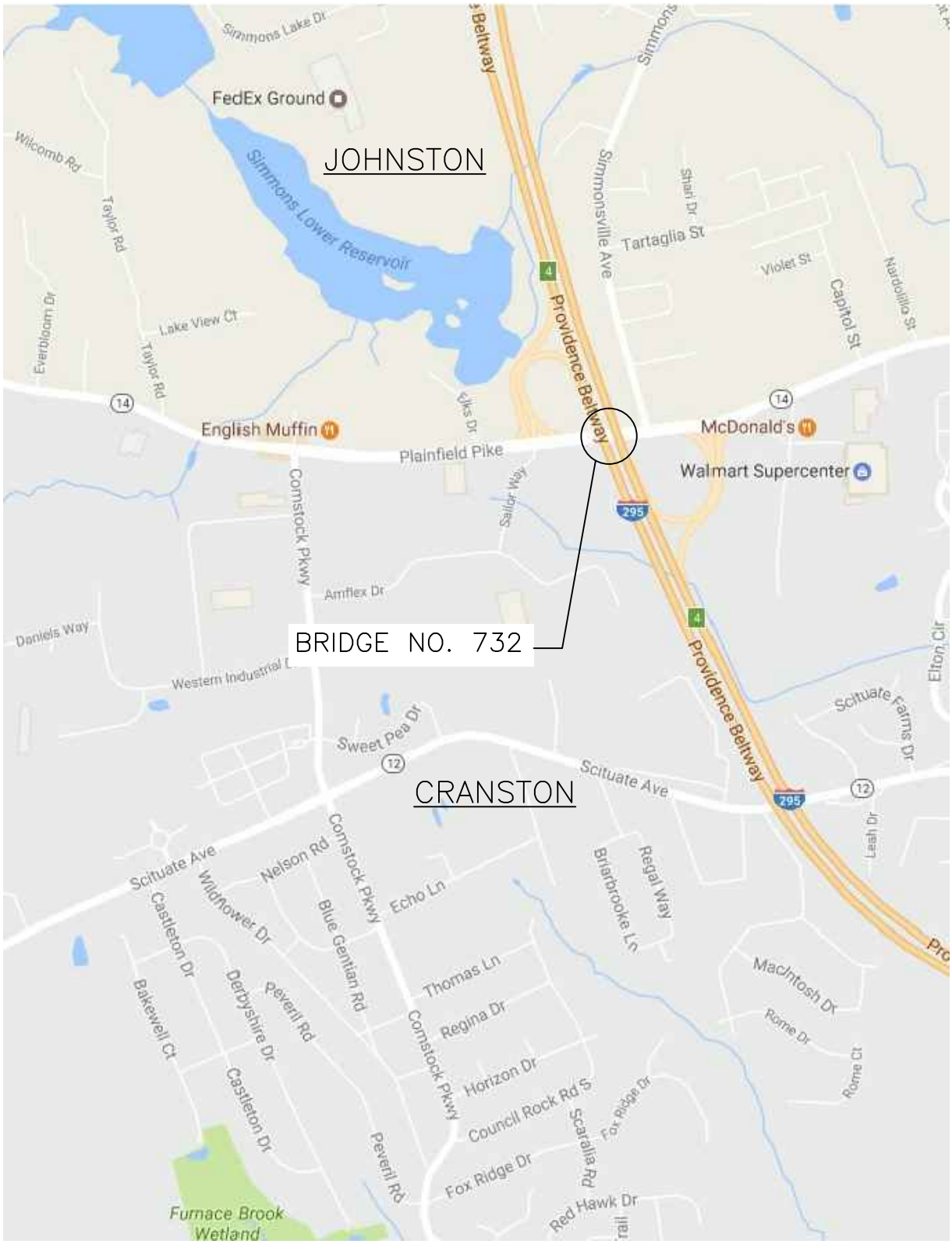
Notes:

1. Water table is assumed to be at the bottom of footing.
2. Resistance Factor = 0.45
3. The factored bearing pressures are shown for the controlling structure element (Southbound – South Abutment).
4. The bearing resistance values are shown for the controlling structure element (Southbound – South Abutment).

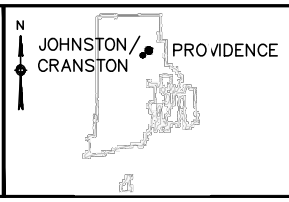
Table 6.2: Summary of Presumptive Bearing Resistance – Temporary Bridge

Structure		Soil Below Bottom of Footing	Max. Allowable Bearing Pressure - From Exist. 1967 Plan (ksf)	Presumptive Bearing Resistance - AASHTO 10.6.2.6 (ksf)	Recommended Allowable Soil Bearing Capacity (ksf)
Temporary Acrow Bridge	South Abutment	Medium Dense Fine to Coarse Sand	6.0	6.0	6.0
	North Abutment	Medium Dense Fine to Coarse Sand	6.0	6.0	6.0

FIGURES



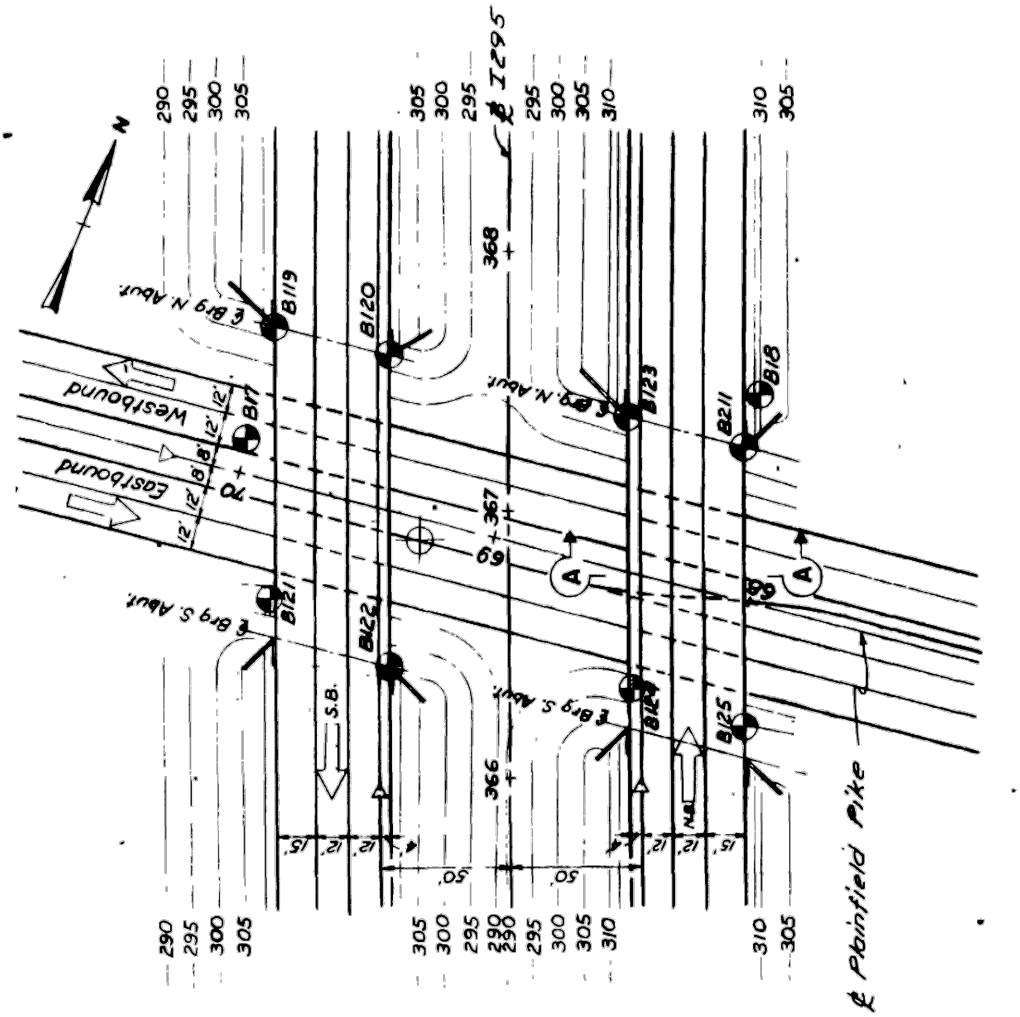
BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND



PROJECT LOCATION PLAN
NOT TO SCALE

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 1



B-XX/B-XXX 1964 & 1966 BORING LOCATION

SOURCE: FROM RIDOT EXISTING PLAN FOR BRIDGE NO. 732.

BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON-JOHNSTON, RHODE ISLAND

1964 & 1966 BORING LOCATION PLAN

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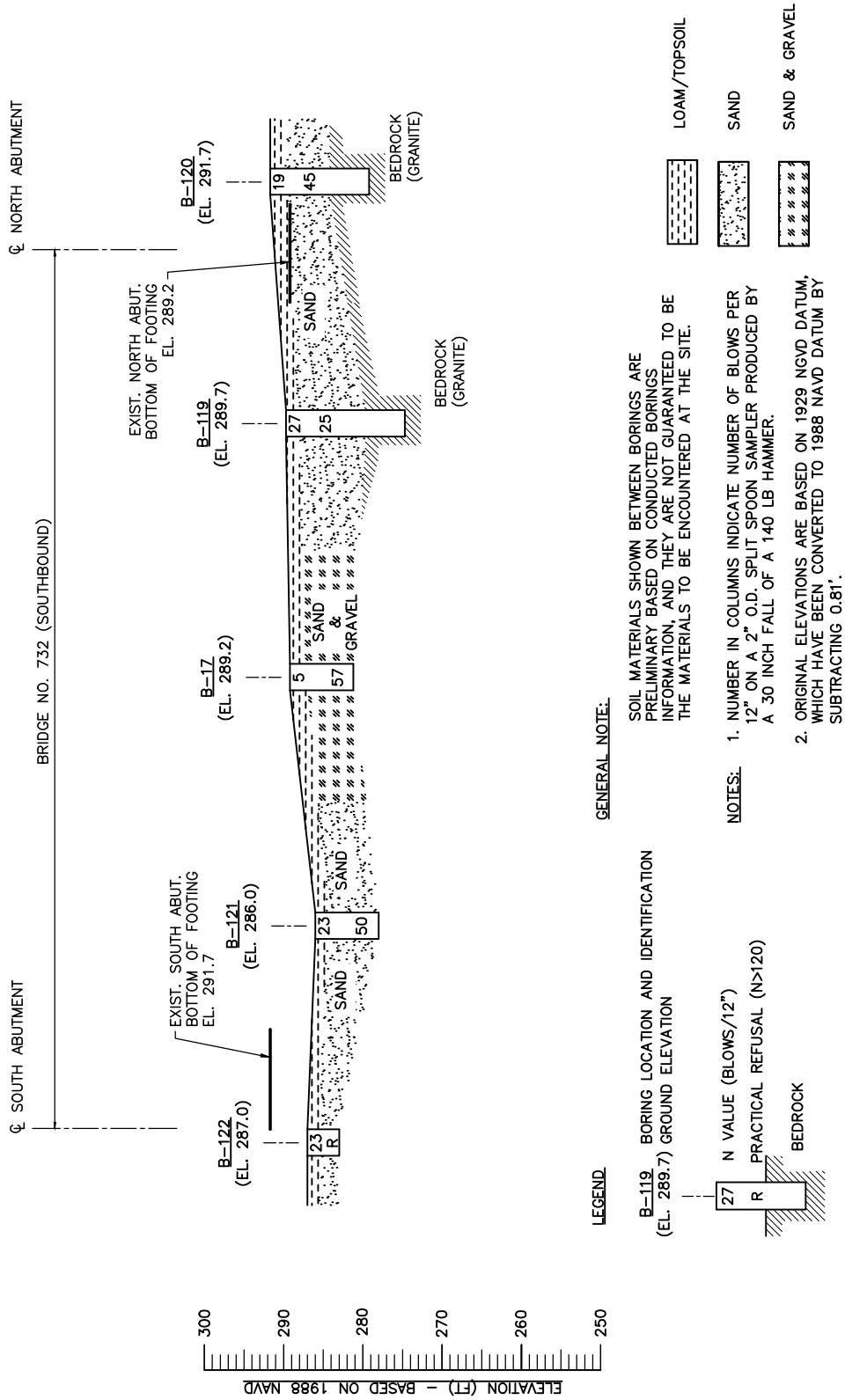
Fig. 2



BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND

SOIL PROFILE – SOUTHBOUND BRIDGE
(PICTORIAL VIEW)

SCALE: 3/64" = 1'–0"

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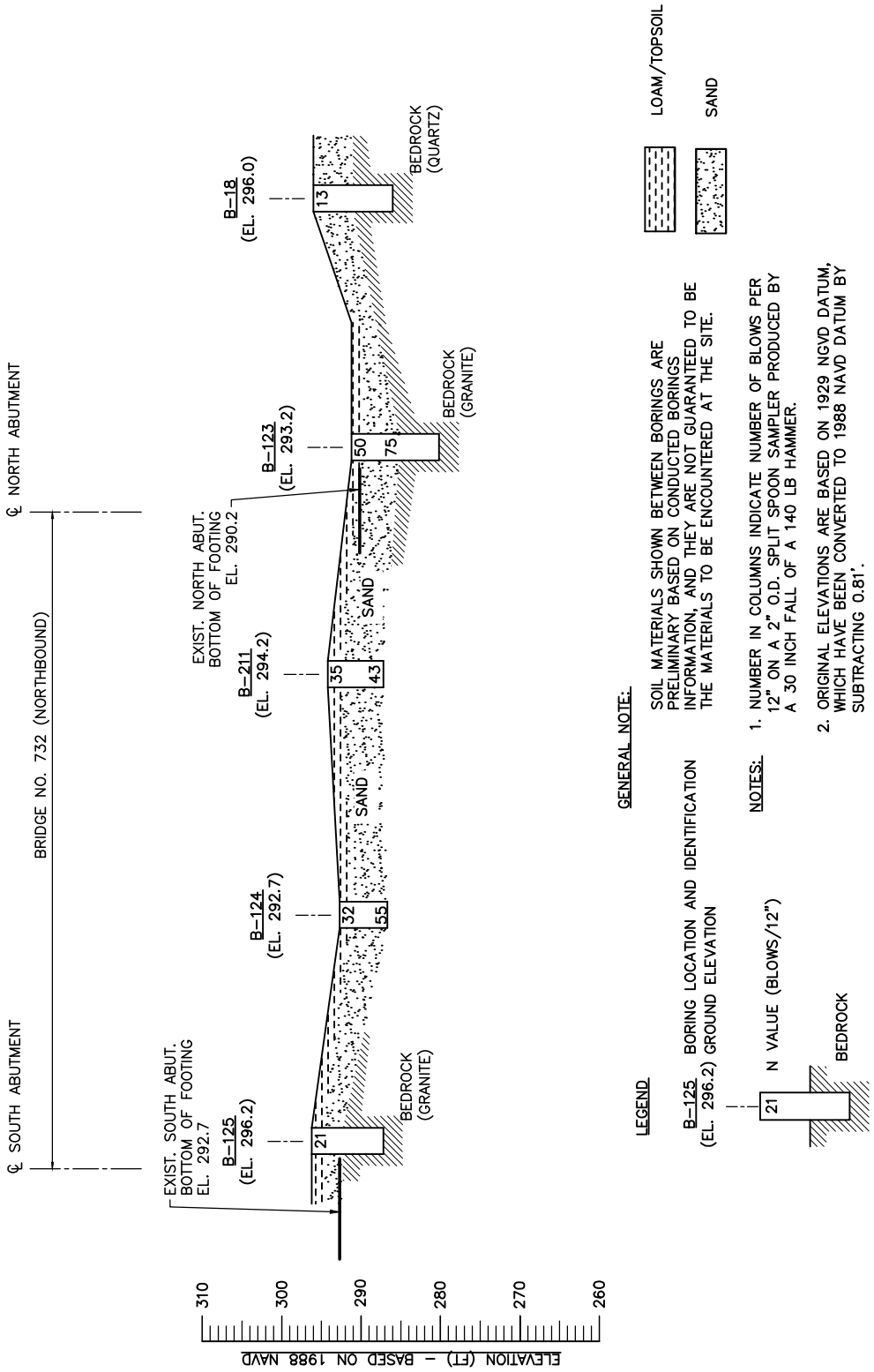
Fig. 3



BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND

SOIL PROFILE – NORTHBOUND BRIDGE
(PICTORIAL VIEW)

SCALE: 3/64" = 1'–0"

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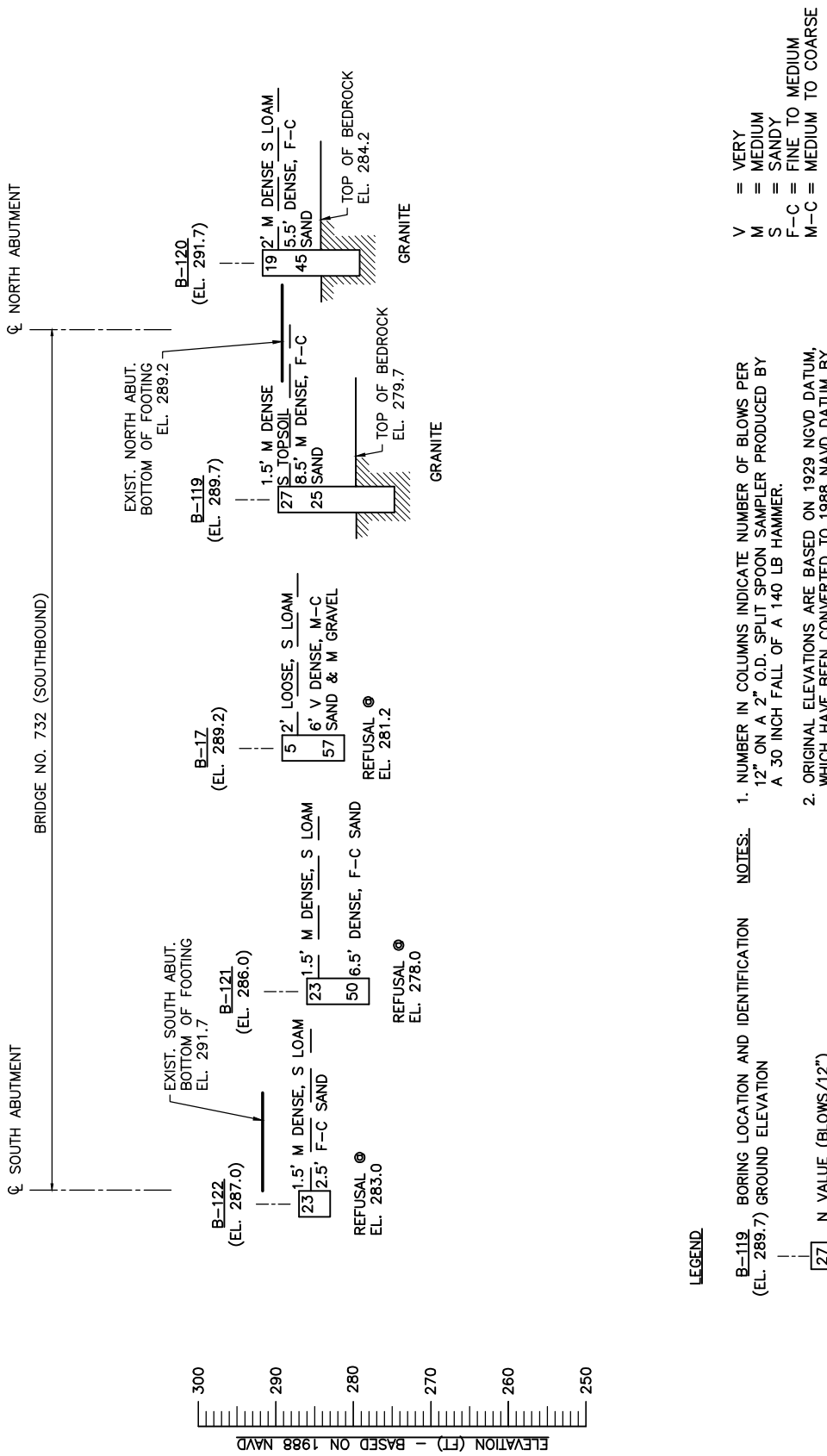
Fig. 4



BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND

SOIL PROFILE – SOUTHBOUND BRIDGE
(DESCRIPTIVE VIEW)

SCALE: 3/64" = 1'–0"

PREPARED BY
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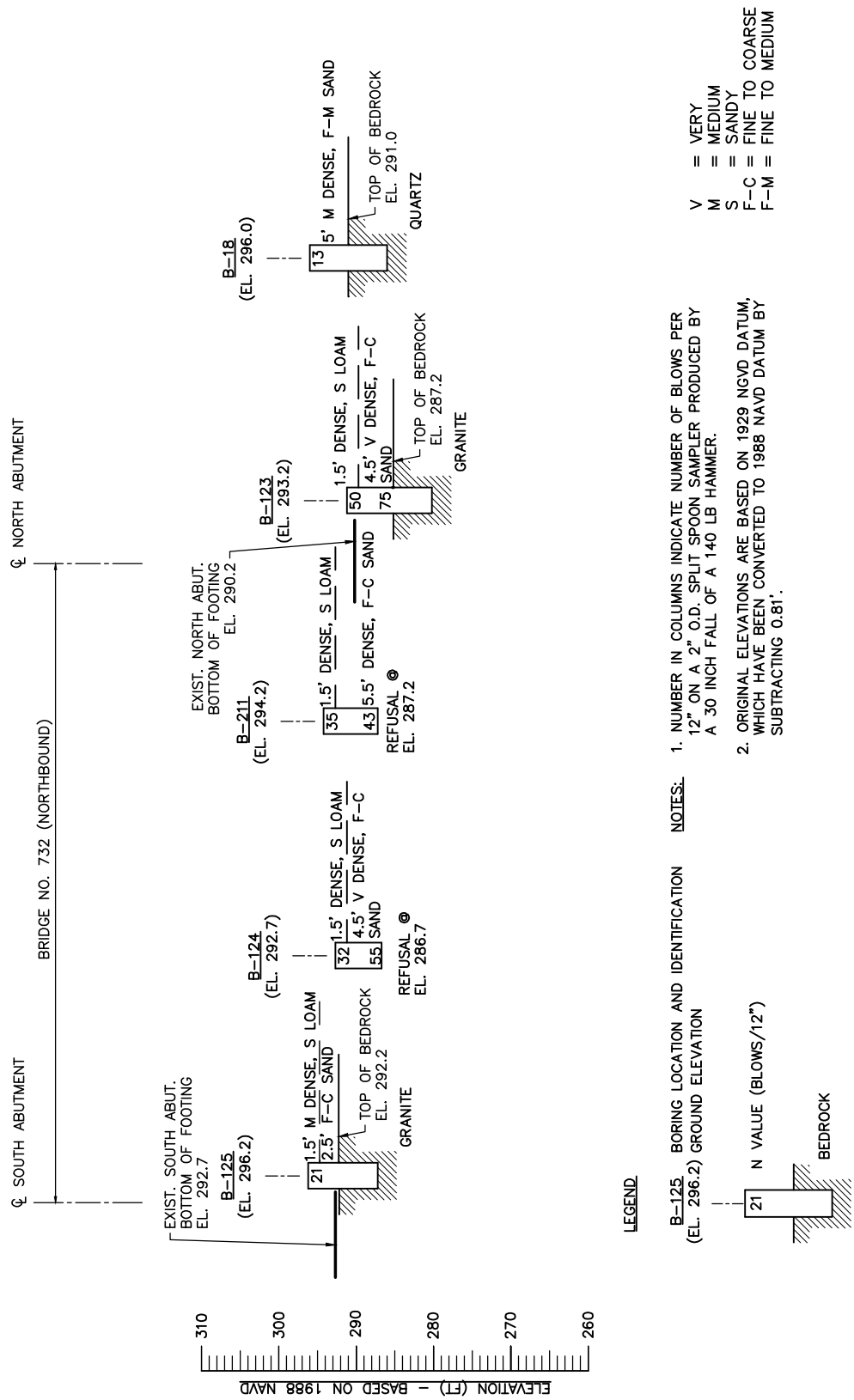
Fig. 5



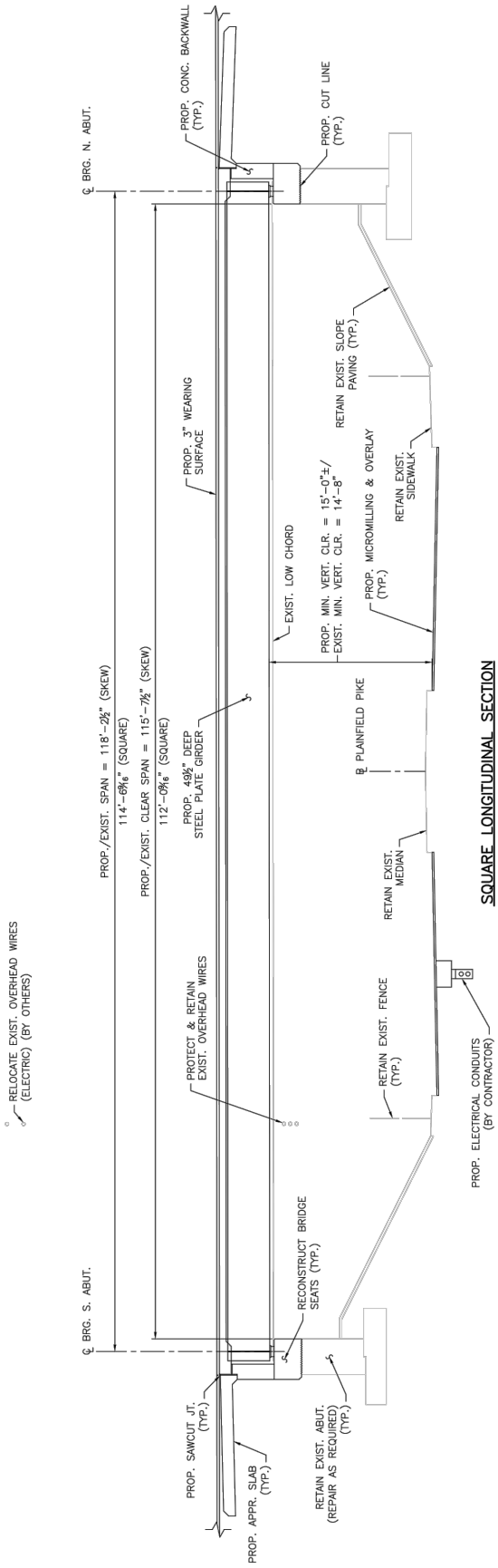
BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON-JOHNSTON, RHODE ISLAND

SOIL PROFILE – NORTHBOUND BRIDGE
(DESCRIPTIVE VIEW)

SCALE: 3/64" = 1'-0"

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 6



SQUARE LONGITUDINAL SECTION

SOURCE: PLANS FROM WSP.

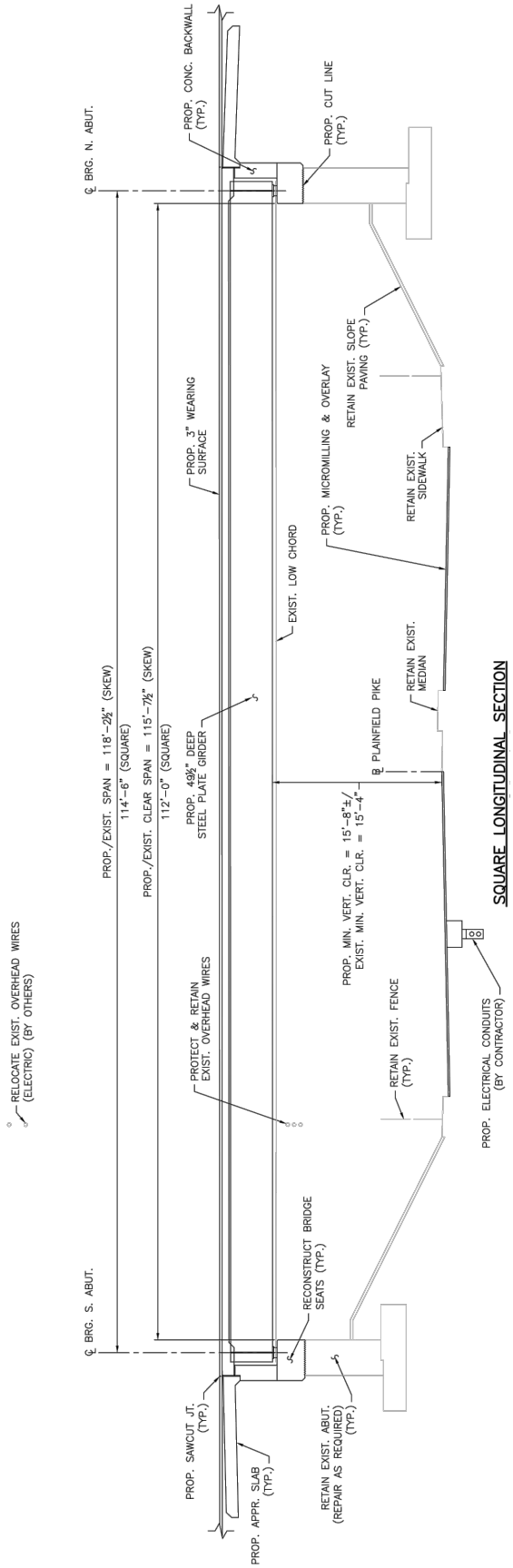
BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND

SQUARE LONGITUDINAL SECTION
(SOUTHBOUND BRIDGE)

NOT TO SCALE

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 7



SOURCE: PLANS FROM WSP.

BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON–JOHNSTON, RHODE ISLAND

SQUARE LONGITUDINAL SECTION
(NORTHBOUND BRIDGE)

NOT TO SCALE

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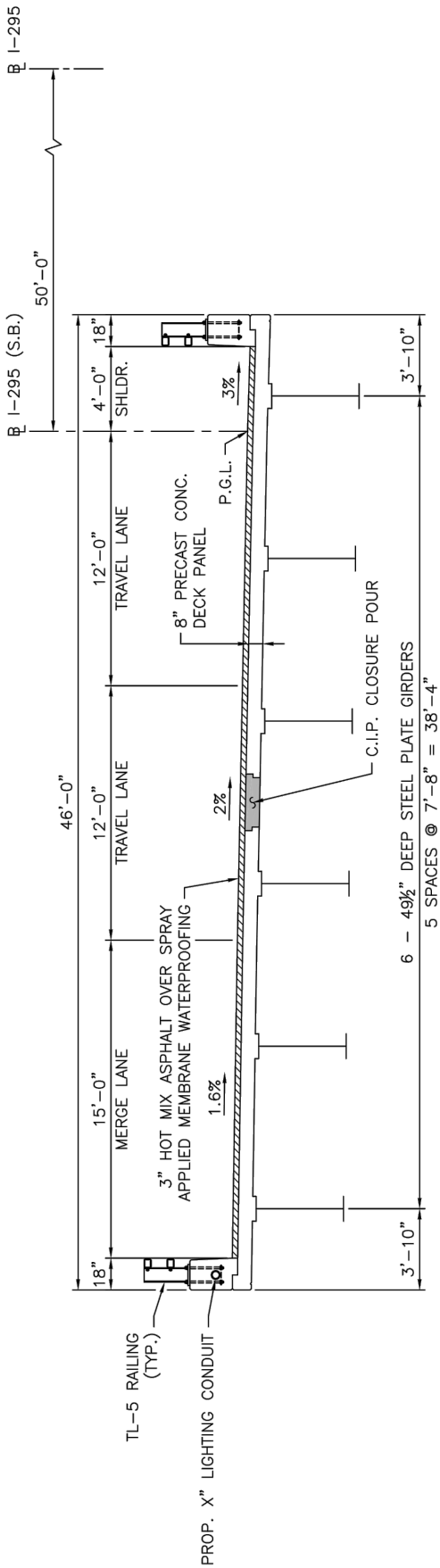
NEWTON, MASS.

Fig. 8



SOURCE: PLANS FROM WSP.

BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON-JOHNSTON, RHODE ISLAND

BRIDGE TRANSVERSE SECTION
(SOUTHBOUND BRIDGE)

NOT TO SCALE

PREPARED BY
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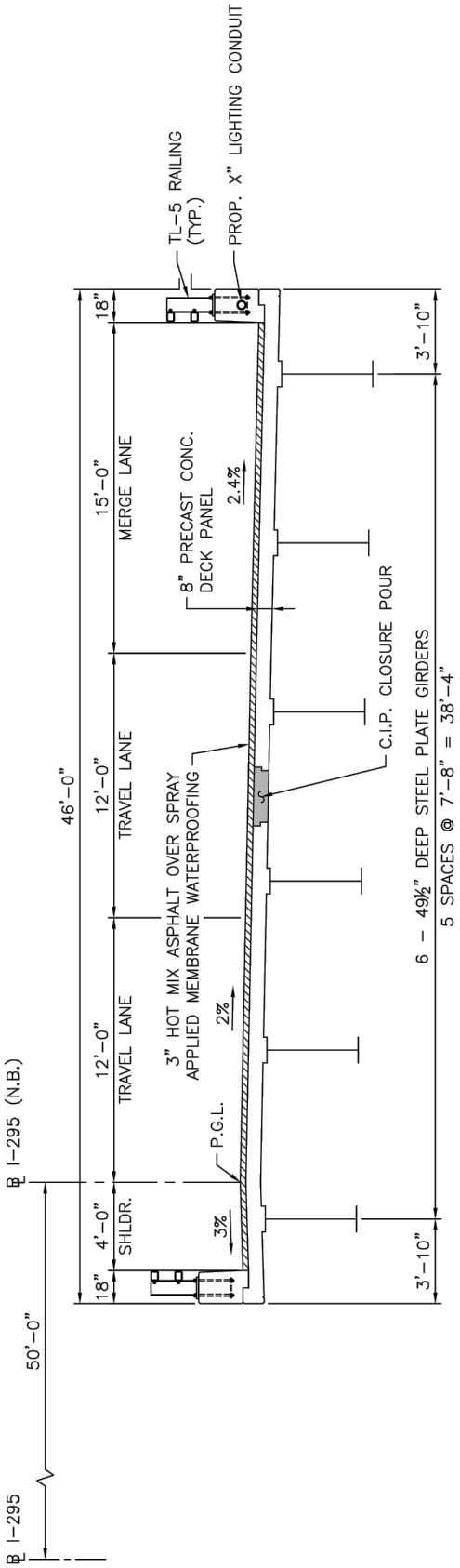
NEWTON, MASS.

Fig. 9



SOURCE: PLANS FROM WSP.

BRIDGE NO. 732 / INTERSTATE RTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)
CRANSTON-JOHNSTON, RHODE ISLAND

BRIDGE TRANSVERSE SECTION
(NORTHBOUND BRIDGE)

NOT TO SCALE

PREPARED BY
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NEWTON, MASS.

Fig. 10

APPENDIX 1

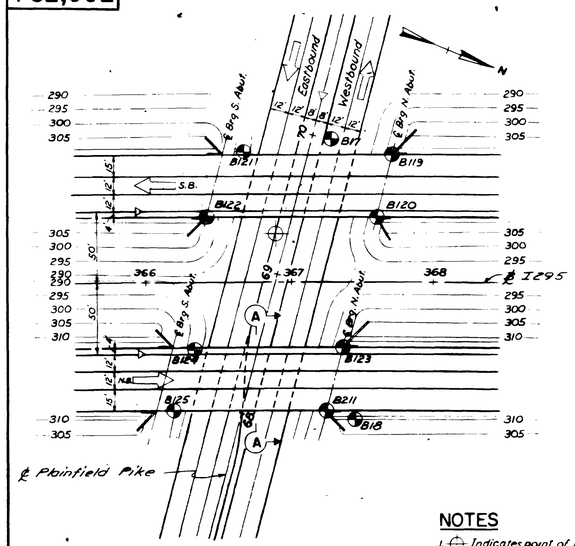
1967 Existing Bridge Plans

Note:

- The elevations shown on the attached Existing 1967 Plans are based on the 1929 NGVD Datum.

732,002

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	R. I.	I-295-5(241)33		B12	50

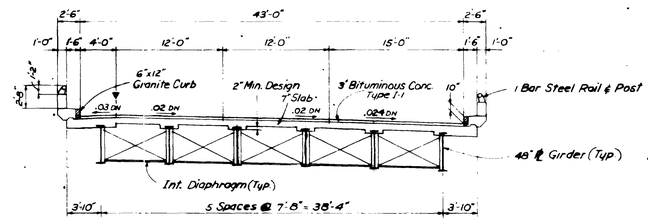


LOCATION PLAN

SCALE 1" = 50'

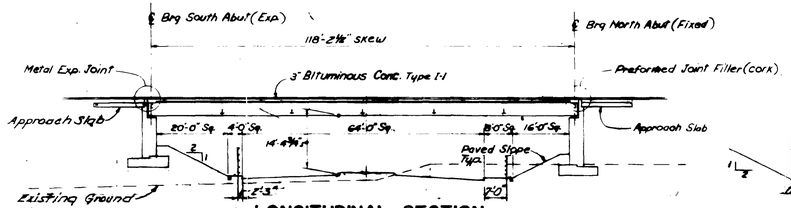
NOTES

1. ⊕ Indicates point of minimum vertical clearance
2. ⇨ Traffic Flow
3. Bore Hole



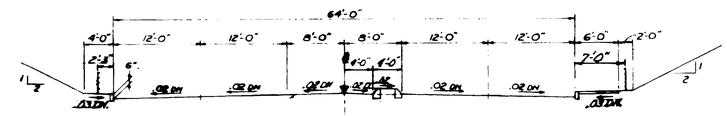
SECTION A-A

SCALE 1/8" = 1'-0"

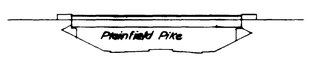


LONGITUDINAL SECTION

SCALE 1" = 20'

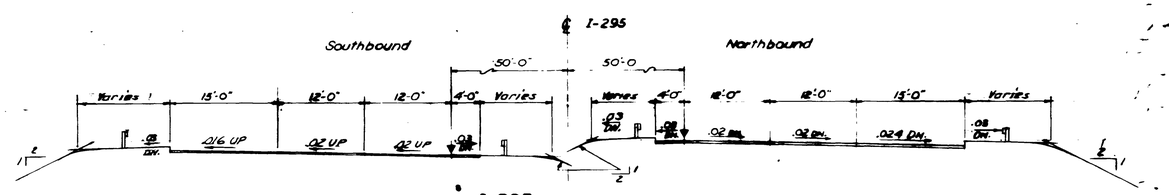


PLAINFIELD PIKE



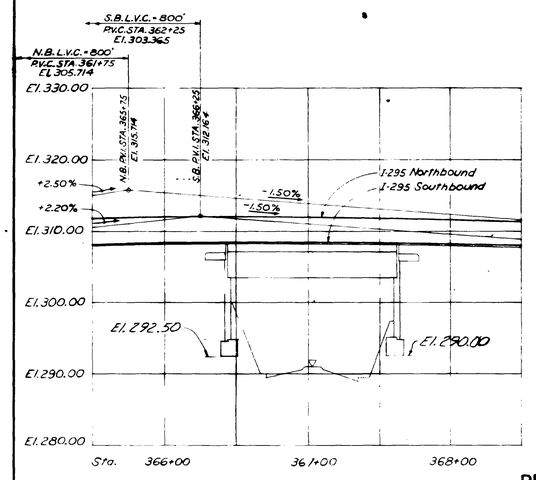
ELEVATION

SCALE 1" = 50'

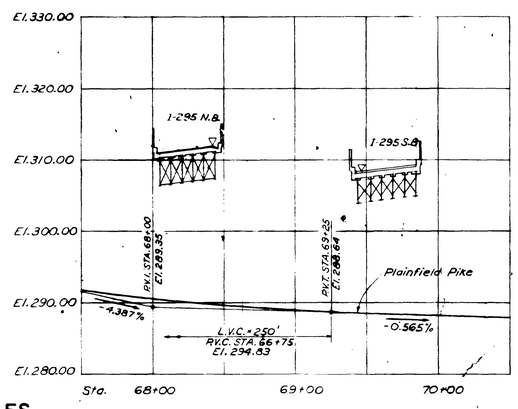


I-295 APPROACH SECTIONS

SCALE 1" = 10'-0"



I-295



PLAINFIELD PIKE

PROFILES

SCALES 1" = 50' HORIZONTAL
1" = 10' VERTICAL

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RHODE ISLAND
DEPARTMENT OF PUBLIC WORKS
DIVISION OF ROADS AND BRIDGES

INTERSTATE ROUTE 295

SCITUATE AVE (C) TO SCITUATE AVE (J)

CRANSTON - JOHNSTON RHODE ISLAND

PLAINFIELD PIKE BRIDGE
GENERAL PLAN & SECTIONS

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE, R. I. BOSTON, MASS.

DATE
SCALE AS SHOWN
DESIGNED BY
SHEET B-12 OF 50

APPROVED _____ SUPERVISING CIVIL ENGINEER (BRIDGE)
APPROVED _____ CHIEF ENGINEER
FINAL DATE

SET NO.
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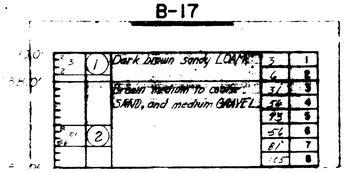
732,002

732 022

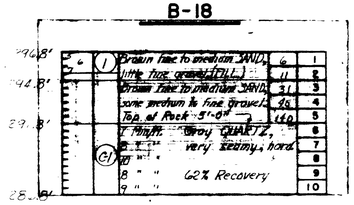
FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	R.I.	[illegible]		[illegible]	[illegible]

B-17



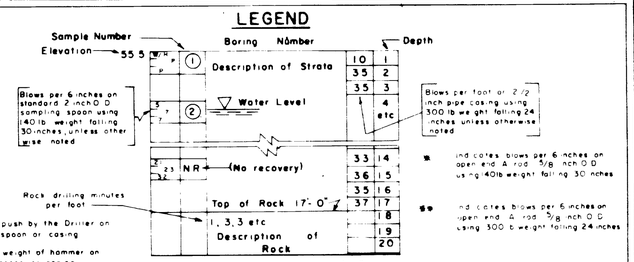
No water recorded

B-18



Bottom of Boring [illegible]'-0"
Date Completed [illegible] 64
No water recorded

LEGEND



Bottom of Boring 20'-1"
Date Completed 12-28-60

NOTES

1. Ground water levels indicated on the drawings are those existing at the time subsurface investigations were made and do not necessarily represent permanent ground water levels. It shall be the Contractors responsibility to determine annual and seasonal variations in ground water level which may effect his work. Variations in existing ground water levels from those indicated on the drawings shall under no conditions constitute grounds for revision in contract price or completion date.
2. Borings, probings, soundings, etc. were taken for the purpose of design and indicate subsurface conditions only at the locations shown. The Contractor shall be aware that subsurface conditions encounted during construction may vary from those shown on the Plans. See specifications.

NOTES

1. Borings B-17 and B-18 were done by American Drilling & Boring Co. Inc. on the dates indicated.
2. Borings B-119 through 125, and 211 were done by University Engineering Services Inc. on the dates indicated.
3. Datum is mean sea level.

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RHODE ISLAND
DEPARTMENT OF PUBLIC WORKS
DIVISION OF ROADS AND BRIDGES

INTERSTATE ROUTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)

CRANSTON-JOHNSTON RHODE ISLAND

PLAINFIELD PIKE BRIDGE
BORING LOGS B-17 & B-18

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE, R.I. BOSTON MASS

DATE
SCALE
DESIGNED BY
SHEET B-32 OF [illegible]

APPROVED [illegible] ASSISTANT CIVIL ENGINEER (BRIDGE)
APPROVED [illegible] CIVIL ENGINEER

SET NO
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732,022

732,023

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	R. I.	I-295-3(24)63		B33	50

Bottom of Footing, Southbound Lane North Abutment El. 290.33

B-119

Elev.	Depth	Description	Blows	No.
290.5	0.0	Brown sandy TOPSOIL	24	1
			20	2
			29	3
			34	4
			42	5
		Brown fine to coarse SAND	[illegible]	6
			[illegible]	7
			[illegible]	8
285.5		Top of Rock 10'0"		
		4 Min/ft. Gray GRANITE hard		
		4 "		
		5 "		
		5 " 77% Recovery		
		4		
275.5				

Bottom of boring 15'2"
Date completed 1-27-66
No water observed

B-120

Bottom of Footing, Southbound Lane North Abutment El. 290.00

Elev.	Depth	Description	Blows	No.
290.0		Brown sandy LOAM	20	1
			15	
			36	
		Brown fine to coarse SAND	42	
			55	
			70	
			128	
285.0		Top of Rock 7'6"		
		5 Min/ft.		
		7 " Gray GRANITE hard		
		6 "		
		7 " 80% Recovery		
		7		
280.0				

Bottom of boring 12'6"
Date completed 1-28-66
No water observed

B-121

Elev.	Depth	Description	Blows	No.
286.8		Brown sandy LOAM	17	1
285.3		Brown fine to coarse SAND		
278.8		300/ No pen.		

Refusal at 8'
Date completed 1-26-66
No water observed

Bottom of Footing, Southbound Lane South Abutment El. 292.50

B-122

Elev.	Depth	Description	Blows	No.
287.8		Brown sandy LOAM	17	1
286.3			35	2
		Brown fine to coarse SAND	58	3
281.8			100	4
		300/ No pen.		

Refusal at 4'
Date completed 1-29-66
No water observed

Bottom of Footing, North bound Lane North Abutment El. 291.00

B-123

Elev.	Depth	Description	Blows	No.
294.0		Brown Sandy LOAM	14	1
292.0			21	2
		Brown fine to coarse SAND	40	3
			28	4
			21	5
288.0		Top of Rock 6'0"	28	6
		5 Min/ft.		7
		6 " Gray GRANITE hard		8
		6 "		9
		7 "		10
282.0		6 " 80% Recovery		11

Bottom of boring 11'0"
Date completed 1-25-66
No water observed

B-124

Bottom of Footing, Northbound Lane South Abutment El. 293.50

Elev.	Depth	Description	Blows	No.
293.5		Brown sandy LOAM	17	1
292.0			26	
		Brown fine to coarse SAND	31	
			48	
			40	
287.5			285	
		300/No pen.		

Refusal at 6'0"
Date completed 1-27-66
No water observed

B-125

Elev.	Depth	Description	Blows	No.
297.0		Brown sandy [illegible]		
293.5		Brown fine to coarse [illegible]		
293.0		Top of Rock [illegible]		
		6 Min/ft.		
		6 " Gray GRANITE		
		6 "		
		7 "		
288.0		6 " 82% Recovery		

Bottom of boring 9'0"
Date completed 1-26-66
No water observed

B-211

Bottom of Footing, Northbound Lane North Abutment El. [illegible]

Elev.	Depth	Description	Blows	No.
295.0		Brown Sandy LOAM	15	1
292.5			27	2
		Brown fine to coarse SAND	35	3
			47	4
			72	5
			90	6
288.0			115	7
		300/ No pen.		

Refusal at [illegible]
Date completed 1-28-66
No water observed

NOTE:

For notes and legend see sheet No. B-32

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DIVISION OF ROADS AND BRIDGES

INTERSTATE ROUTE 295
SCITUATE AVE (C) TO SCITUATE AVE (J)

CRANSTON - JOHNSTON RHODE ISLAND

PLAINFIELD PIKE BRIDGE
BORING LOGS B-119 THRU B-125 & B-211

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE, R. I. BOSTON, MASS.

DATE
SCALE
DESIGNED BY
SHEET B-33 OF 50

APPROVED _____ SUPERVISING CIVIL ENGINEER (BRIDGE)
APPROVED _____ CHIEF ENGINEER
FINAL DATE

SET NO.
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APPENDIX 2

Calculations

INDEX OF CALCULATIONS

Title	Page
Summary	1
Temporary Bridge	2
Shallow Foundation Design	
• Abutment Section	3
• Lateral Earth Pressure	4
• Bearing Resistance	7
• Unfactored Vertical Load	9
• Overturning and Bearing Pressure	10
• Sliding and Eccentricity	12
• Settlement	14
• Corrected SPT Blow Count	18
• Global Stability	19

LAMSON ENGINEERING CORPORATION			Final Page No.: 1	
Project:	I-295 Bridge No. 732, Cranston-Johnston	**Job No.:**	**Preliminary Sheet No.:**	
Subject:	Summary	**Prepared by:** JJL		**Date:** 8/2017
Detail:	Bearing Resistance	**Checked by:** WD		**Date:** 8/2017

Summary

Structure		**Footing Width (ft)**	**Factored Bearing Pressure (ksf)**	**Nominal Bearing Resistance (ksf)**	**Factored Bearing Resistance (ksf)**
Southbound Bridge	South Abutment	9.5	5.2	15.9	7.1
	North Abutment	10.5	5.2	15.9	7.1
Northbound Bridge	South Abutment	10.5	5.2	15.9	7.1
	North Abutment	11.0	5.2	15.9	7.1

Notes:

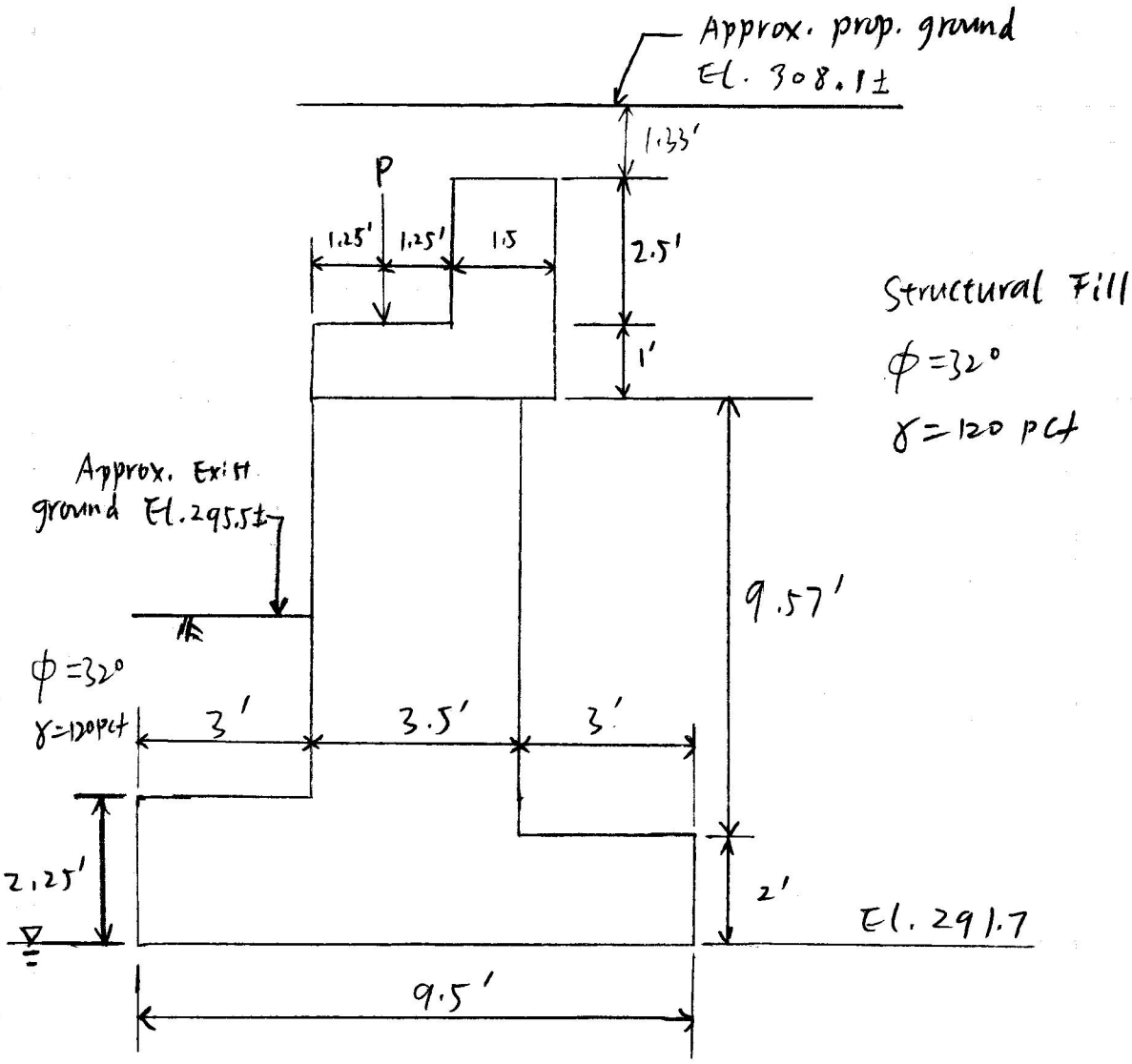
1. Water table is assumed to be at the bottom of footing.
2. Resistance Factor = 0.45
3. The factored bearing pressures are shown for the controlling structure elements (Southbound – South Abutment).
4. The bearing resistance values are shown for the controlling structure element (Southbound – South Abutment).

LAMSON ENGINEERING CORPORATION			Final Page No.: 2
Project:	I-295 Bridge No. 732, Cranston-Johnston	**Job No.:**	**Preliminary Sheet No.:**
Subject:	Temporary Bridge	**Prepared by:** JJL	**Date:** 8/2017
Detail:	Allowable Bearing Capacity	Checked by: WD	**Date:** 8/2017

Structure		**Soil Below Bottom of Footing**	**Max. Allowable Bearing Pressure - From Exist. 1967 Plan (ksf)**	**Presumptive Bearing Resistance - AASHTO 10.6.2.6 (ksf)**	**Recommended Allowable Soil Bearing Capacity (ksf)**
Temporary Acrow Bridge	South Abutment	Medium Dense Fine to Coarse Sand	6.0	6.0	6.0
	North Abutment	Medium Dense Fine to Coarse Sand	6.0	6.0	6.0

LAMSON ENGINEERING CORPORATION			Final Page No: 3	
Project: I-295 Bridge No. 732, Cranston-Johnston		Job No:	Preliminary Sheet No:	
Subject: Shallow Foundation Design		Prepared by: JJL		Date: 8/2017
Detail: Abutment Section		Checked by: WD		Date: 8/2017

Abutment Section — SB bridge S. Abut is the critical Abut.
Based on B-121



Medium Dense

Fine to coarse Sand

$\phi = 34°$

$\gamma = 120$ pcf

Determine Unfactored Horizontal Load

Effective angle of internal friction ϕ'_f	=	32	degree
Friction angle between fill and wall, δ	=	21	degree
Angle of fill to the horizontal, β	=	0.00	degree
Angle of back of wall to the horizontal, θ	=	90	degree
Unit weight of soil, γ_{soil}	=	0.120	kcf
Total unit weight of water, γ_w	=	0.0624	kcf
Height of Abutment	=	16.40	ft ±
Distance from back of wall to footing heel, B_{heel}	=	3.00	ft
Height of fill behind footing at heel, $h = H + B_{heel} \tan \beta$	=	16.40	ft ±
Height of water from bottom of footing ,Hw	=	0	ft

Lateral Earth Pressure (EH)

Active pressure coefficient, K_a

$$= \frac{\sin^2(\theta + \phi'_f)}{\sin^2\theta \sin(\theta - \delta)\left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2}$$

$= 0.275$

At-rest pressure coefficient, K_o $= 1 - \sin \phi'_f = 1 - \sin(32)$

$= 0.470$

Per *massDOT* LRFD BM 3.1.6

$K_e = K_a$

$= 0.275$

Lateral earth pressure, P_{e1} $= 0.5\, K_e \gamma_{soil} (h - H_w)^2$

$= 0.5 \times 0.275 \times 0.12$ kcf x (16.4 ft - 0 ft)^2

$= 4.44$ k/ft length of wall

Vertical component of P_{e1}, P_{ev1} $= P_{e1} \sin(90 - \theta + \delta)$

$= 4.44$ k /ft x sin (90 - 90 + 21)

$= 1.59$ k/ft

Horizontal component of P_{e1}, P_{eh1} = $P_{e1} \cos(90 - \theta + \delta)$

= 4.44 k/ft x cos (90 - 90 + 21)

= 4.15 k/ft

Lateral earth pressure, P_{e2} = $K_e \gamma_{soil} (h - H_w) H_w$

= 0.275 x 0.12 kcf x (16.4 ft - 0 ft) x 0 ft

= 0.00 k/ft length of wall

Vertical component of P_{e2}, P_{ev2} = $P_{e2} \sin(90 - \theta + \delta)$

= 0 k/ft x sin (90 - 90 + 21)

= 0.00 k/ft

Horizontal component of P_{e2}, P_{eh2} = $P_{e2} \cos(90 - \theta + \delta)$

= 0 k/ft x cos (90 - 90 + 21)

= 0.00 k/ft

Lateral earth pressure, P_{e3} = $0.5 K_e \gamma' (H_w)^2$

0.5 x 0.275 x (0.12 kcf - 0.0624 kcf) x 0 ft^2

= 0.00 k/ft length of wall

Vertical component of P_{e3}, P_{ev3} = $P_{e3} \sin(90 - \theta + \delta)$

= 0 k/ft x sin (90 - 90 + 21)

= 0.00 k/ft

Horizontal component of P_{e3}, P_{eh3} = $P_{e3} \cos(90 - \theta + \delta)$

= 0 k/ft x cos (90 - 90 + 21)

= 0.00 k/ft

Live Load Surcharge (LS) (AASHTO LRFD 3.11.6.4)

Per MassDOT LRFD Part I, Section 3.3.1.3

Equivalent height of soil for , h_{eq} = 0.000 ft

LAMSON ENGINEERING CORPORATION			Final Page No.: 6
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:
Subject:	Shallow Foundation Design	Prepared by: JJL	Date: 8/2017
Detail:	Lateral Earth Pressure	Checked by: WD	Date: 8/2017

Vertical live load Surcharge on heel, LS = 0.12 kcf x 0 ft x 3 ft

= 0.000 k / ft

Horizontal earth pressure due to live Δ_p = $K_e \gamma_{soil} h_{eq}$

load surcharge = 0.275 x 0.12 kcf x 0 ft

= 0.00 ksf

Live load lateral earth pressure, P_{LS} = Δ_p h

= 0 ksf x 16.4 ft

= 0.00 k/ft length of wall

Vertical component of P_{LS}, P_{LSV} = P_{LS} sin (90 - α + δ)

= 0 k/ft x sin (90 - 90 + 21)

= 0.00 k/ft length of wall

Horizontal component of P_{LS}, P_{LSH} = P_{LS} cos (90 - α + δ)

= 0 k/ft x cos (90 - 90 + 21)

= 0.00 k/ft length of wall

Unfactored horizontal load, EH + LSH = P_{eh1} + P_{eh2} + P_{eh3} + P_{LSH}

= 4.15 + 0 + 0 + 0

= 4.15 k/ft length of wall

Bearing Resistance

Bearing Resistance at the Service Limit State

Based on Boring B-121, Bottom of Footing Elev. 291.7 (NAVD 88)

Presumptive Bearing Resistance for Spread Footing (Table C10.6.2.6.1-1)

Bearing Resistance, q = 5 ksf

Bearing Resistance of Soil at the Strength Limit State Design

Footing length, L = 49.4 ft

Factored bearing resistance, q_R = $\varphi_b\ q_n$ (AASHTO 10.6.3.1.1-1)

Bearing resistance factor, φ_b = 0.45 (AASHTO Table 10.5.5.2.2-1)

Nominal bearing resistance, q_n = $c\ N_{cm} + \gamma\ D_f\ N_{qm}\ C_{wq} + 0.5\ \gamma\ B\ N_{\gamma m}\ C_{w\gamma}$

N_{cm} = $N_c\ s_c\ i_c$

N_{qm} = $N_q\ s_q\ d_q\ i_q$

$N_{\gamma m}$ = $N_\gamma\ s_\gamma\ i_\gamma$

Because of c = 0,

Nominal bearing resistance, q_n = $\gamma\ D_f\ N_{qm}\ C_{wq} + 0.5\ \gamma\ B\ N_{\gamma m}\ C_{w\gamma}$

i_q = $[\ 1 - H / V\ ]^n$

= See table next page

i_γ = $[\ 1 - H / V\ ]^{(n+1)}$

= See table next page

n = $[(2+L / B) / (1+ L / B)]\cos^2\theta + [(2+B / L) / (1+ B / L)]\sin^2\theta$

= See table next page

Unfactored horizontal load, H = 4.15 kips/ft

Unfactored vertical load, V = 33.33 kips/ft

Project direction of load in the plane of the footing, θ = 90 deg (measured from the side of L)

Soil properties above footing

Angle of internal friction, Φ_{f1} = 32 degree

Moist unit weight of soil, γ_1 = 0.12 kcf

LAMSON ENGINEERING CORPORATION			Final Page No.: 8
Project: I-295 Bridge No. 732, Cranston-Johnston		Job No.:	Preliminary Sheet No.:
Subject: Shallow Foundation Design		Prepared by: JJL	Date: 8/2017
Detail: Bearing Resistance		Checked by: WD	Date: 8/2017

Footing embedment depth, D_f = 3.8 ft (EL. 295.5 - EL. 291.7)

Surcharge bearing capacity factor, N_q = 23.2 (AASHTO Table 10.6.3.1.2a-1)

For $\Phi_{f1} > 0$ Shape factor, s_q = $1 + (B' / L) \tan \Phi_{f1}$ (AASHTO Table 10.6.3.1.2a-3)

= See table this page

Depth Correction Factor, d_q = 1

Correction factors to account for the location of the ground water table (AASHTO Table 10.6.3.1.2a-2)

C_{wq} = 1.00

Soil properties below footing

Angle of internal friction, Φ_{f2} = 34 degree

Moist unit weight of soil, γ_2 = 0.12 kcf ← γ_2

Unit Weight bearing capacity factor, N_γ = 41.1 (AASHTO Table 10.6.3.1.2a-1)

For $\Phi_{f2} > 0$ Shape factor, s_γ = $1 - 0.4 (B' / L)$ (AASHTO Table 10.6.3.1.2a-3)

See Table this page

Correction factors to account for the location of the ground water table

$C_{w\gamma}$ = 0.5 (AASHTO Table 10.6.3.1.2a-2)

B = 9.50 ft

e = 0.46 ft B' = B - 2e

B' (ft)	n	i_q	i_γ	s_q	s_γ	N_{qm}	$N_{\gamma m}$	$\gamma_1 D_f N_{qm} C_{wq}$ (ksf)	$0.5 \gamma_2 B' N_{\gamma m} C_{w\gamma}$ (ksf)	q_n (ksf)	q_R (ksf)
8.6	1.85	0.782	0.685	1.11	0.93	20.14	26.18	9.18	6.74	15.92	**7.1**

Factored bearing resistance

LAMSON ENGINEERING CORPORATION			Final Page No.: 9
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:
Subject:	Shallow Foundation Design	Prepared by: JJL	Date: 8/2017
Detail:	Unfactored Vertical Load	Checked by: WD	Date: 8/2017

Unfactored Vertical Load

Backwall	1.5 x 2.5 x 0.15	=	0.56 k/ft
Beam Seat	1 x 4 x 0.15	=	0.60 k/ft
Stem	9.57 x 3.5 x 0.15	=	5.02 k/ft
Concrete Footing	(9.5 x 2 + 0.25 x 3) x 0.15	=	2.96 k/ft
Soil on back of the wall	14.4 x 3 x 0.12	=	5.18 k/ft
From WSP			
Superstructure Dead Load (DC)	495.4 / 49.4	=	10.03 k/ft
Superstructure Dead Load (DW)	98.9 / 49.4	=	2.00 k/ft
Superstructure Live Load (LL)	265.6 / 49.4	=	5.38 k/ft

LAMSON ENGINEERING CORPORATION			Final Page No.: 10	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Strength I for Overturning and Bearing Pressure	Checked by: WD		Date: 8/2017

Summary of Factored Loads - Strength IA: 1.25DC + 1.50EH + 1.35EV + 1.75 LS

Vertical Loads and Vertical Moments

Item	Load Factor γ	Vertical load, V (k/ft)	Factored vertical load, V_u (k/ft)	Arm about ℄ Footing	Factored Moment	
Backwall	1.25	0.56	0.70	-1.50	-1.05	
Beam Seat	1.25	0.60	0.75	-0.25	-0.19	
Stem	1.25	5.02	6.28	0.00	0.00	
Concrete Footing	1.25	2.96	3.70	0.00	0.00	
Soil on back of the wall	1.35	5.18	7.00	-3.25	-22.74	
Superstructure DC	1.25	10.03	12.54	0.50	6.27	
Superstructure DW	1.50	2.00	3.00	0.50	1.50	
Superstructure LL	1.75	5.38	9.41	0.50	4.70	
P_{v1}	1.50	1.59	2.39	-4.75	-11.34	
P_{v2}	1.50	0.00	0.00	-4.75	0.00	
P_{v3}	1.50	0.00	0.00	-4.75	0.00	
P_{LSV}	1.75	0.00	0.00	-4.75	0.00	
LS	1.75	0.00	0.00	-3.25	0.00	
TOTAL		33.33	45.77		-22.85	ΣM_V

Horizontal Loads and Horizontal Moments

Item	Load Factor γ	Horizontal load H (k/ft)	Factored horizontal load, H_u (k/ft)	Arm about ℄ Footing	Factored Moment	
P_{h1}	1.50	4.15	6.22	5.47	33.99	
P_{h2}	1.50	0.00	0.00	0.00	0.00	
P_{h3}	1.50	0.00	0.00	0.00	0.00	
P_{LSH}	1.75	0.00	0.00	8.20	0.00	
TOTAL		4.15	6.22		33.99	ΣM_H

LAMSON ENGINEERING CORPORATION			Final Page No.: 11	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Strength I for Overturning and Bearing Pressure	Checked by: WD		Date: 8/2017

COEFFICIENT OF FRICTION FOR SLIDING

Resistance Factor, φ_t	=	0.8	Cast-in- Place Concrete on Sand
Effective angle of internal friction Φ_f below footing	=	34	degree
$\varphi_t \tan\Phi_f$	=	0.8 x tan 34	
	=	0.54	

BEARING PRESSURE

Footing width, B	=	9.500	ft
Distance from wall face to toe	=	3	ft
Height of soil over toe from top of footing	=	1.55	ft
Factored vertical soil over toe, V_{st}	=	1.35 x 3 ft x 1.6 ft x 0.120 kcf	
	=	0.753	k/ft
Factored Moment about ℄, M_{st}	=	0.753 k/ft x (9.5 ft - 3 ft) / 2	
	=	2.448	k-ft

Eccentricity of load, e

$$= \frac{\Sigma M_V + \Sigma M_H + M_{st}}{\Sigma V_U + V_{st}}$$

$$= \frac{-22.85 + 33.99 + 2.448}{45.769 + 0.753}$$

= 0.29 ft < B/3 OK

For Footing founded on Soil

$$q_{average} = \frac{(\Sigma V_u + V_{st})}{(B - 2e)}$$

$$= \frac{45.77 + 0.75}{(9.5 - 2 \times 0.29)} = 5.22 \text{ ksf}$$

Factored Bearing Resistance, q_R = 7.10 k/ft^2 > 5.22 OK

LAMSON ENGINEERING CORPORATION			Final Page No.: 12	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Strength I for Sliding and Eccentricity	Checked by: WD		Date: 8/2017

Summary of Factored Loads - Strength IB: 0.90DC + 1.50EH + 1.0EV + 1.75 LS

Vertical Loads and Vertical Moments

Item	Load Factor γ	Vertical load, V (k/ft)	Factored vertical load, V_u (k/ft)	Arm about ℄ Footing	Factored Moment	
Backwall	0.90	0.56	0.51	-1.50	-0.76	
Beam Seat	0.90	0.60	0.54	-0.25	-0.14	
Stem	0.90	5.02	4.52	0.00	0.00	
Concrete Footing	0.90	2.96	2.67	0.00	0.00	
Soil on back of the wall	1.00	5.18	5.18	-3.25	-16.85	
Superstructure DC	0.90	10.03	9.03	0.50	4.51	
Superstructure DW	0.65	2.00	1.30	0.50	0.65	
Superstructure LL	1.75	5.38	9.41	0.50	4.70	
P_{v1}	1.50	1.59	2.39	-4.75	-11.34	
P_{v2}	1.50	0.00	0.00	-4.75	0.00	
P_{v3}	1.50	0.00	0.00	-4.75	0.00	
P_{LSV}	1.75	0.00	0.00	-4.75	0.00	
LS	1.75	0.00	0.00	-3.25	0.00	
TOTAL		33.33	35.54		-19.21	ΣM_V

Horizontal Loads and Horizontal Moments

Item	Load Factor γ	Horizontal load H (k/ft)	Factored horizontal load, H_u (k/ft)	Arm about ℄ Footing	Factored Moment	
P_{h1}	1.50	4.15	6.22	5.47	33.99	
P_{h2}	1.50	0.00	0.00	0.00	0.00	
P_{h3}	1.50	0.00	0.00	0.00	0.00	
P_{LSH}	1.75	0.00	0.00	8.20	0.00	
TOTAL		4.15	6.22		33.99	ΣM_H

LAMSON ENGINEERING CORPORATION			Final Page No.: 13	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Strength I for Sliding and Eccentricity	Checked by: WD		Date: 8/2017

SLIDING

Resistance Factor, φ_t = 0.8 Cast-in- Place Concrete on Sand

Effective angle of internal friction Φ_f = 34 degree

below footing

Sliding Factor = $\dfrac{\varphi_t \tan\Phi_f (\Sigma V)}{(\Sigma H_u)}$

= $\dfrac{0.8 \times \tan 34 \times 35.54}{6.22}$

Sliding Factor = 2.9 > 1 OK

Eccentricity

Footing width, B = 9.500 ft

Distance from wall face to toe = 3 ft

Height of soil over toe from top of footing = 1.55 ft

Factored vertical soil over toe, V_{st} = 1.0 x 3 ft x 1.6 ft x 0.120 kcf

= 0.558 k/ft

Factored Moment about ℄ , M_{st} = 0.558 k/ft x (9.5 ft - 3 ft) / 2

= 1.814 k-ft

Eccentricity of load, e = $\dfrac{\Sigma M_V + \Sigma M_H + M_{st}}{\Sigma V_U + V_{st}}$

= $\dfrac{-19.212 + 33.99 + 1.814}{35.541 + 0.558}$

= 0.46 ft < B/3 OK

LAMSON ENGINEERING CORPORATION			Final Page No.: 14
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:
Subject:	Shallow Foundation Design	Prepared by: JJL	Date: 8/2017
Detail:	Service I for Settlement	Checked by: WD	Date: 8/2017

Summary of Factored Loads - Service I: 1.0DC + 1.0EH + 1.0EV

Vertical Loads and Vertical Moments

Item	Load Factor γ	Vertical load, V (k/ft)	Factored vertical load, V_u (k/ft)	Arm about ℄ Footing	Factored Moment	
Backwall	1.00	0.56	0.56	-1.50	-0.84	
Beam Seat	1.00	0.60	0.60	-0.25	-0.15	
Stem	1.00	5.02	5.02	0.00	0.00	
Concrete Footing	1.00	2.96	2.96	0.00	0.00	
Soil on back of the wall	1.00	5.18	5.18	-3.25	-16.85	
Superstructure DC	1.00	10.03	10.03	0.50	5.01	
Superstructure DW	1.00	2.00	2.00	0.50	1.00	
Superstructure LL	1.00	5.38	5.38	0.50	2.69	
P_{v1}	1.00	1.59	1.59	-4.75	-7.56	
P_{v2}	1.00	0.00	0.00	-4.75	0.00	
P_{v3}	1.00	0.00	0.00	-4.75	0.00	
P_{LSV}	1.00	0.00	0.00	-4.75	0.00	
LS	1.00	0.00	0.00	-3.25	0.00	
TOTAL		33.33	33.33		-16.70	ΣM_V

Horizontal Loads and Horizontal Moments

Item	Load Factor γ	Horizontal load H (k/ft)	Factored horizontal load, H_u (k/ft)	Arm about ℄ Footing	Factored Moment	
P_{h1}	1.00	4.15	4.15	5.47	22.66	
P_{h2}	1.00	0.00	0.00	0.00	0.00	
P_{h3}	1.00	0.00	0.00	0.00	0.00	
P_{LSH}	1.00	0.00	0.00	8.20	0.00	
TOTAL		4.15	4.15		22.66	ΣM_H

LAMSON ENGINEERING CORPORATION			Final Page No.: 15
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:
Subject:	Shallow Foundation Design	Prepared by: JJL	Date: 8/2017
Detail:	Service I for Settlement	Checked by: WD	Date: 8/2017

BEARING PRESSURE

Footing width, B	=	9.500	ft
Distance from wall face to toe	=	3	ft
Height of soil over toe from top of footing	=	1.55	ft
Factored vertical soil over toe, V_{st}	=	1.0 x 3 ft x 1.6 ft x 0.120 kcf	
	=	0.558	k/ft
Factored Moment about ℄ , M_{st}	=	0.558 k/ft x (9.5 ft - 3 ft) / 2	
	=	1.814	k-ft

Eccentricity of load, e

$$= \frac{\Sigma M_V + \Sigma M_H + M_{st}}{\Sigma V_U + V_{st}}$$

$$= \frac{-16.696 + 22.66 + 1.814}{33.331 + 0.558}$$

$$= 0.23 \text{ ft}$$

For Footing founded on Soil

$$q_{average} = \frac{(\Sigma V_u + V_{st})}{(B - 2e)}$$

$$= \frac{33.33 + 0.56}{(9.5 - 2 \times 0.23)} = 3.75 \text{ ksf}$$

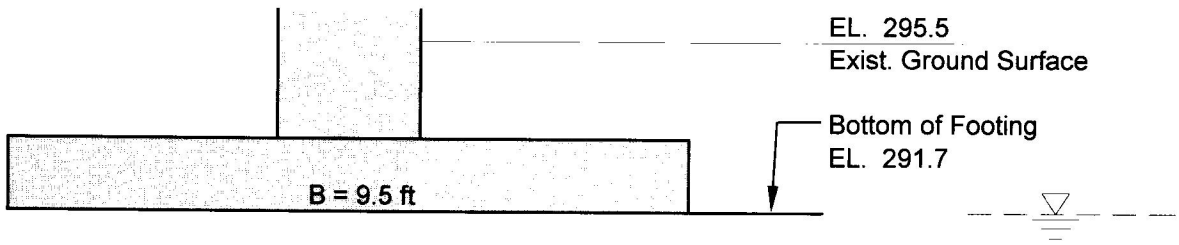
LAMSON ENGINEERING CORPORATION			Final Page No.: 16	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Service I for Settlement	Checked by: WD		Date: 8/2017

Abutment Settlement (AASHTO 10.6.2.4.2)

Based on B-121

Footing Width, B = 9.5 ft

Footing Length, L = 49.40 ft



Compacted Gravel γ_{soil} = 0.12 kcf

Average $N1$ = 64 blows per ft

EL. 284.9
Mid-Layer below footing

Refusal EL. 278.0

LAMSON ENGINEERING CORPORATION			Final Page No.: 17	
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:	
Subject:	Shallow Foundation Design	Prepared by: JJL		Date: 8/2017
Detail:	Service I for Settlement	Checked by: WD		Date: 8/2017

The total settlement, S_t = $S_e + S_c + S_s$

Total settlement, S_t = S_e for footing on cohesionless soils $S_c = S_s = 0$

Immediate settlement, S_e = $\sum_{i=1}^{n} \Delta H_i$

ΔH_i = elastic settlement of layer i (ft.)

= $H_c \frac{1}{C'} \log\left(\frac{\sigma_o' + \Delta\sigma_v}{\sigma_o'}\right)$

n = number of soil layers within zone of stress influence of the footing

= 1

H_c = initial heigh of layer i (ft.)

= 13.7 ft.

C' = bearing capacity index from Figure 10.6.2.4.2-1

= 185 (for F-C Sand and N1 = 64)

σ'_o = initial vertical effective stress at the midpoint of layer i

= 0.12x(EL.295.5 - EL.291.7)+(0.12-0.0624)x(EL.291.7 - EL.284.85)

= 0.851 ksf

Max. Service I
(No Exist. Soil Weight)

σ_v = 3.75 ksf

$\Delta\sigma_v$ = increase in vertical stress at the midspan of layer i

= $0.5\ \sigma_v$ From Figure 10.62.4.1-1,

$\Delta\sigma_v$ = 1.87 ksf

ΔH_i = (13.7 / 185) log [(0.851 + 1.874) / 0.851]

= 0.04 ft = 0.4 in.

LAMSON ENGINEERING CORPORATION			Final Page No.: 18
Project:	I-295 Bridge No. 732, Cranston-Johnston	Job No.:	Preliminary Sheet No.:
Subject:	Shallow Foundation Design	Prepared by: JJL	Date: 08/2017
Detail:	Corrected *SPT* blow count	Checked by: WD	Date: 08/2017

B-121

Water table = 0 ft γ_w 0.0624 kcf
(Assume at bottom of footing) γ_m 0.125 kcf

Bottom of Footing
Elev. 291.7

Depth ft	σ'_v ksf	C_N	N blows/ft	$N1 = C_N N$ blows/ft
1.00	0.06	2.00	23	46
5.00	0.31	1.62	50	81

Top of Exist. Ground
Elev. 286.0±

Average, $N1$ 64 (below footing)

For example

Depth = 5.00 ft

C_N = $0.77 \log_{10} (40 / \sigma'_v)$ < 2 (AASHTO 10.4.6.2.4-1)

σ'_v = vertical effective stress (ksf)

= 0 ft x 0.125 kcf + (5 - 0) ft x (0.125 - 0.062) kcf

= 0.313 ksf

C_N = $0.77 \log_{10} (40 / 0.313)$

= 1.622 < 2

N = uncorrected *SPT* blow count (blows/ft.)

= 50 blows /ft

$N1$ = $C_N N$

= 1.622 x 50 blows/ft ≤ 100 blows/ft

= 81 blows/ft < 100 blows/ft

use $N1$ = 81 blows/ft

Overall Stability - S. Abutment

Report created by ReSSA(3.0): Copyright (c) 2001-2010, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title: Overall Stability - S. Abutment
Project Number: Bridge No. 732 -
Client: WSP / RIDOT
Designer: JJL
Station Number:

Description:
Overall Stability

Company's information:

Name: Lamson Engineering Coporation
Street: 437 Cherry Street
Newton, MA 02465
Telephone #: 617-558-0101
Fax #:
E-Mail:

Original file path and name: C:\Program Files\ADAMA\ReSSA(3.0)\I-295 Br. No. 732.MSE
Original date and time of creating this file: Feb 19 15:56:56 2017

PROGRAM MODE: Analysis of a General Slope using NO reinforcement material.

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INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

========== Soil Layer #: ==========	Unit weight, γ [lb/ft^3]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft^2]
....1.....Gravel Borrow.....	120.0	32.0	0.0
....2.....Backfill.....	120.0	32.0	0.0
....3..... Medium Dense Sand.....	120.0	34.0	0.0

REINFORCEMENT

Analysis of slope WITHOUT reinforcement.

WATER

Unit weight of water = 62.45 [lb/ft^3]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

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TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Yw (phreatic)
1	-300.00	0.00	0.00	0.00	0.00
2	0.00	0.00	0.00	0.00	0.00
3	3.00	2.25	0.00	0.00	0.00
4	9.50	16.40	0.00	0.00	0.00

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RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
. 1	35.00	16.40	-21.45	0.14	-5.47	50.79	53.11	1.57 .	On extreme X-entry
2	37.00	16.40	-24.57	0.16	-7.12	58.87	61.24	1.65	
3	39.00	16.40	-21.60	0.15	-5.62	61.69	63.58	1.73	
4	41.00	16.40	-24.29	0.07	-6.82	68.92	71.03	1.81	
5	43.00	16.40	-27.30	0.07	-8.67	79.38	81.47	1.89	
6	45.00	16.40	-27.06	0.01	-8.72	86.00	87.92	1.98	
7	47.00	16.40	-27.19	0.04	-8.00	89.41	91.40	2.06	
8	49.00	16.40	-30.37	0.13	-4.29	74.65	78.96	2.14	
9	51.00	16.40	-30.15	0.06	-2.40	71.91	77.02	2.22	
10	53.00	16.40	-30.26	0.10	-1.53	74.16	79.44	2.30	
11	55.00	16.40	-30.60	0.25	-0.17	73.84	79.64	2.38	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.) The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	-30.22	0.08	35.00	16.40	-10.05	57.94	61.28	1.61	
2	-27.46	0.15	35.00	16.40	-8.71	56.28	59.17	1.59	
3	-24.26	0.08	35.00	16.40	-6.87	52.72	55.43	1.58	
. 4	-21.45	0.14	35.00	16.40	-5.47	50.79	53.11	1.57 .	OK
5	-18.03	0.01	35.00	16.40	-4.03	48.71	50.67	1.57	
6	-15.17	0.04	35.00	16.40	-2.55	46.47	48.11	1.58	
7	-12.27	0.07	35.00	16.40	-1.01	44.07	45.42	1.60	
8	-9.06	0.01	35.00	16.40	0.58	41.52	42.61	1.64	
9	-6.22	0.04	35.00	16.40	1.78	40.01	40.76	1.68	
10	-3.20	0.04	35.00	16.40	3.54	37.08	37.65	1.75	
11	-0.28	0.04	35.00	16.40	4.92	35.04	35.39	1.83	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

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APPENDIX 3

Seismic Design Information

(4 Sheets)

LAMSON ENGINEERING CORPORATION			Final Page No.: 1
Project:	I-295 Bridge No. 732, Cranston-Johnston	**Job No.:**	**Preliminary Sheet No.:**
Subject:	Seismic Analysis	**Prepared by:** JJL	**Date:** 6/2017
Detail:	Site Class	Checked by: WD	**Date:** 6/2017

B-119

Depth ft	*Soil* Layer	*N* blows/ft	*Ave. N* blows/ft	d_i ft	d_i / N_i ft^2/blow
1.00	1	27	26	5	0.19
5.00		25			
100.00	2	120	120	95	0.79
			Sum	100	0.98

Ground Surface Elev. 289.7

$\bar{N}$ = 102 > 50

Therefore, the site class is C per Table 3.4.2.1-1 of AASHTO Guide Specification.

From Seismic Design Maps:

PGA = 0.077 g

S_s = 0.133 g

S_1 = 0.051 g

Per Tables 3.4.2.3-1 & 3.4.2.3-2 of AASHTO Guide Specification

F_{pga} = 1.2

F_a = 1.2

F_v = 1.7

Therefore,

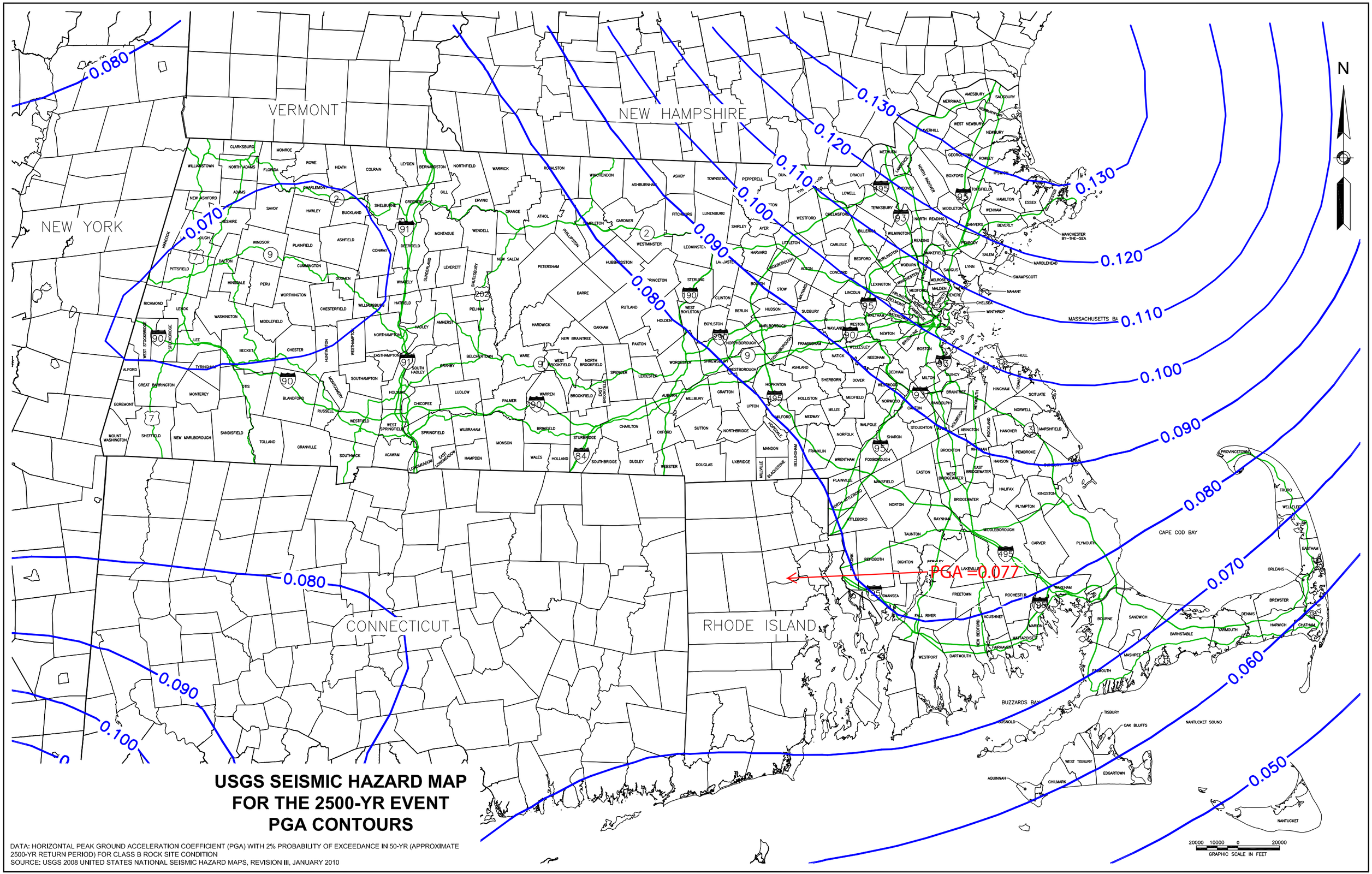
A_s = 0.092 g

S_{DS} = 0.160 g

S_{D1} = 0.087 g < 0.15g

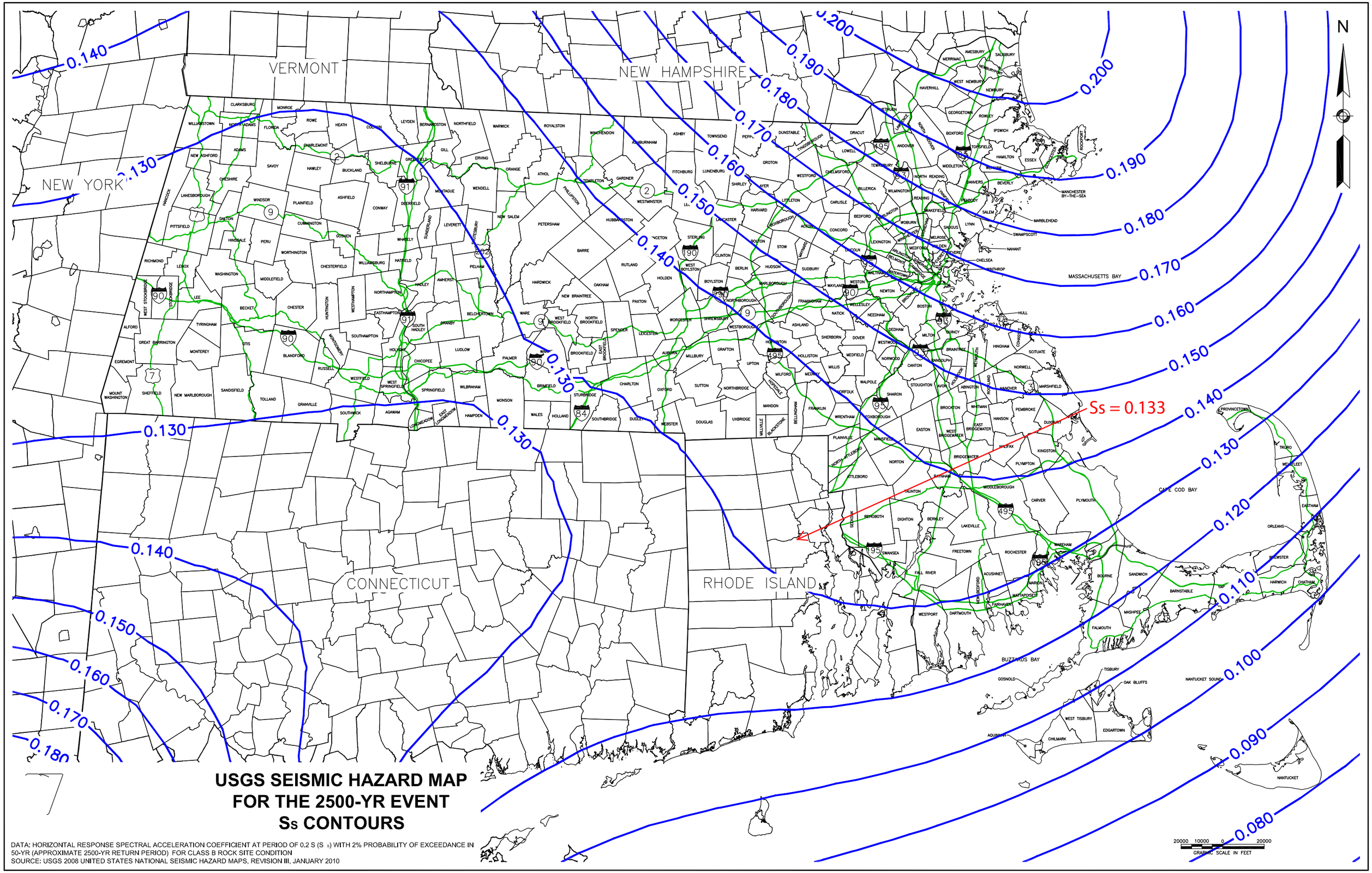
Seismic Design Category (SDC) = A

Summary: Based on AASHTO Guide Specifications Table 3.5-1, a liquefaction assessment is not required for structures in Seismic Design Category (SDC) A area.



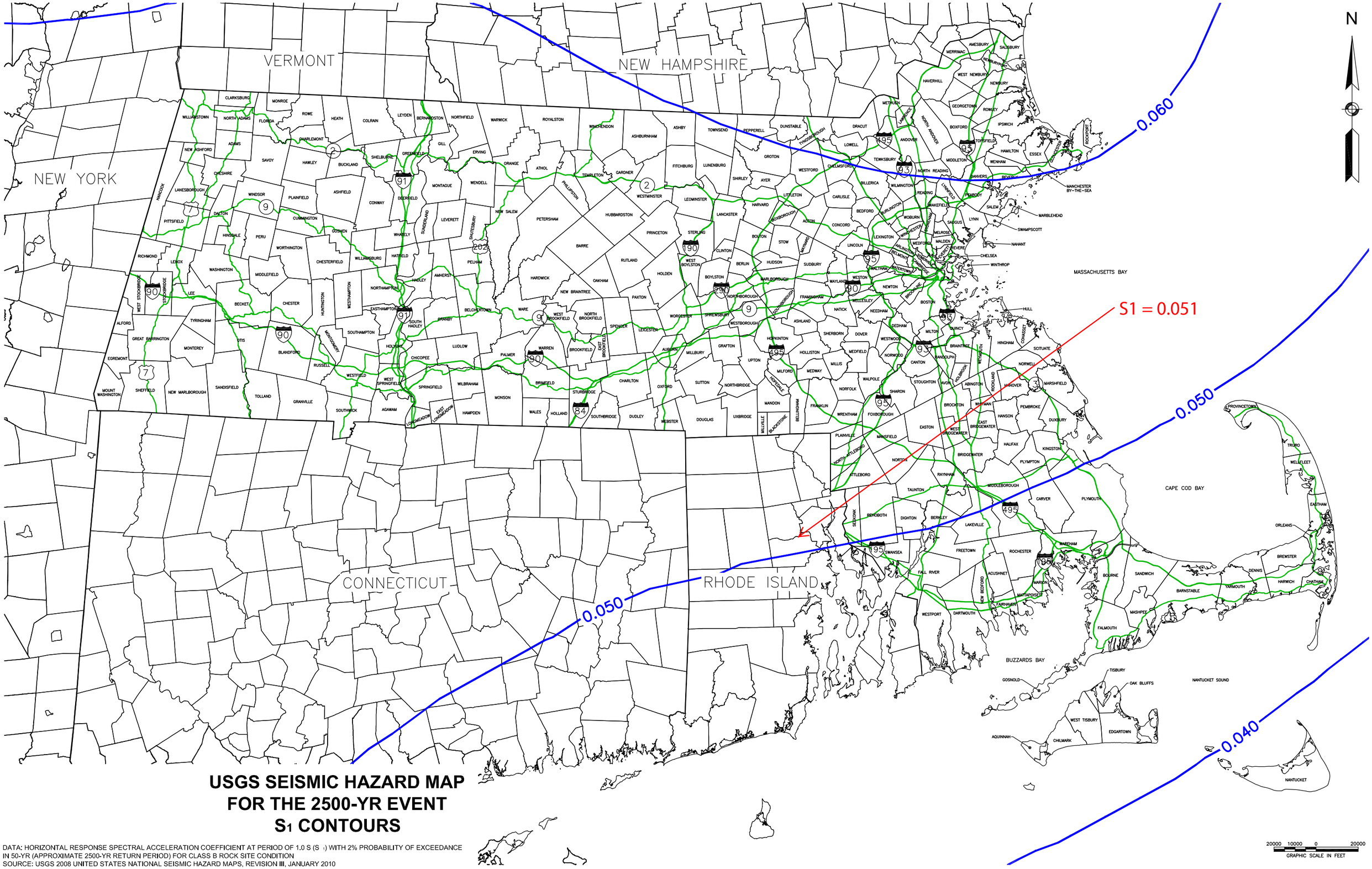
USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT PGA CONTOURS

DATA: HORIZONTAL PEAK GROUND ACCELERATION COEFFICIENT (PGA) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010



USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT S_S CONTOURS

DATA: HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT PERIOD OF 0.2 S (S_a) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010



USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT S_1 CONTOURS

DATA: HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT PERIOD OF 1.0 S (S_1) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010

APPENDIX 4

Loads from WSP

(1 Sheet)

Summary of Substructure Loads:

Substructure Dead Loads:

Abutment Loads	Service I Factors	Service I Loads (kip)	Strength I Factors	Strength I Loads (kip)
DW	**1.00**	98.934	**1.50**	148.400
DC	**1.00**	495.368	**1.25**	619.209
DC, Approach Slab	**1.00**	73.008	**1.25**	91.259

** Approach slab load determined separately since not applied at bearing*

Substructure Live Loads:

Abutment Loads	Service I Factors	Service I Loads (kip)	Strength I Factors	Strength I Loads (kip)
LL, Composite Loading 1	**1.00**	**265.562**	**1.75**	464.733

Geotechnical Report

for

Rhode Island Department of Transportation

Proposed Bridge Replacement Design-Build Project

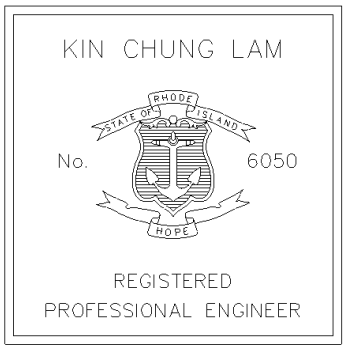
for

Bridge No. 730

I-295 Southbound & Northbound

Ralphs Pond to Scituate Avenue (C)

Cranston, Rhode Island



Date: 8/30/2017

Prepared for:
WSP USA Inc.
2348 Post Road, Suite 27
Warwick, Rhode Island 02886
Phone: 401-738-6600

Prepared by:
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Newton, Massachusetts 02465
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3.0 Subsurface Conditions

4.0 Foundation Recommendations

5.0 Construction Considerations

6.0 Summary

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2. 1964 & 1966 Boring Location Plan
3. Soil Profile – Southbound Bridge (Pictorial View)
4. Soil Profile – Northbound Bridge (Pictorial View)
5. Soil Profile – Southbound Bridge (Descriptive View)
6. Soil Profile – Northbound Bridge (Descriptive View)
7. Square Longitudinal Section

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1. 1967 Existing Bridge Plans
2. Calculations
3. Seismic Design Information
4. Loads from WSP

1.0 EXECUTIVE SUMMARY

Based on the 1964 & 1966 subsurface exploration results and our investigation, the proposed precast concrete arch can be supported by the proposed concrete spread footing.

The nominal and factored bearing resistance values for the proposed foundations are included in Section 6.0, Summary, of this report.

The elevations shown on the attached 1963 existing bridge plans (Appendix 1) were based on 1929 NGVD datum. The current design is based on 1988 NAVD datum. To obtain elevation values based on 1988 NAVD datum, subtract 0.81' from those based on 1929 NGVD datum. Elevations shown in this report, except those in the existing Plans, are in reference to the 1988 NAVD Datum.

Contractor should prepare a final geotechnical report based on his proposed superstructure system. Such report shall be prepared by a professional engineer licensed in the state of Rhode Island and shall be submitted to Engineer for review and approval.

2.0 INTRODUCTION

2.1 Scope of Report

The purpose of this report is to document the 1964 and 1966 subsurface exploration results and evaluate the proposed substructure foundations based on the proposed superstructure system, provide recommendations, develop engineering design values, and identify possible geotechnical related construction issues. The design values will be prepared using the AASHTO Standard Specifications for Highway Bridges based on the Load Resistance Factor Design (LRFD) method with HL-93 truck loading.

2.2 Existing Bridge & Foundation

Bridge No. 730 is located on I-295 Southbound & Northbound Ralphs Pond to Scituate Avenue (C) in the City of Cranston as illustrated on Figure 1, 'Project Location Plan'.

The existing southbound bridge is a three-span, steel rolled beam composited with 7¼" reinforced concrete deck structure supported by reinforced concrete abutments and piers. The overall length of the structure is 270'-0"± with three spans from west to east at 85'-0"±, 100'-0"±, 85'-0"± long, respectively. The structure carries three (3) lanes of traffic with a curb to curb width of 38'±. The existing abutments and piers are supported by reinforced concrete spread footings. The existing wingwalls were also constructed of reinforced concrete supported by spread footings.

The existing northbound bridge, including the foundations, was constructed in a similar manner to the southbound bridge, as described above.

The existing northbound and southbound superstructures will be replaced. There are no known foundation settlements or performance issues with the existing foundations. Therefore, the existing substructure should be considered for reuse to support the proposed superstructure.

2.3 Proposed Bridge Replacement

The design concept of the proposed superstructure replacement can be found on the proposed bridge plans prepared by WSP, who is the Prime Design Engineer for the entire project. Lamson Engineering Corporation (LEC) is a sub-consultant to WSP responsible for the preliminary geotechnical design of the project.

The proposed precast concrete arch structure has a clear span length of 29'-0". The minimum under-clearance of the arch is 8'-9' at crown.

2.4 References

The following references were used in the preparation of this report:

- AASHTO LRFD Bridge Design Specifications, 2014, 7th Edition with 2015, 2016, & 2017 Interim Revisions.
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2011, 2nd Edition with 2012, 2014, and 2015 Interim Revisions.

3.0 SUBSURFACE CONDITIONS

3.1 General

Based on previous 1964 & 1966 boring program, refusal materials were encountered at depths varying from 5'-0" to 21'-0". Surficial geologic deposits above refusal consist of loose to very dense, fine to coarse sand, gravel, and cobble materials on top of Granite Bedrock.

A new boring program was not conducted for this bridge replacement project.

3.2 1964 & 1966 Subsurface Exploration Program

A subsurface exploration program consisting of 11 test borings was performed in June 1964 and August 1966, as indicated on Figure 2, '1964 & 1966 Boring Location Plan'

Southbound Bridge

Boring B-222 was conducted for the existing South Pier. Borings B-224 and B-225 were conducted for the existing North Pier. Borings H-35 and H-36 were conducted between the existing Piers.

Northbound Bridge

Borings B-230 and B-231 were conducted for the existing South Pier. Borings B-232 and B-233 were conducted for the existing North Pier. Borings B-234 and B-235 were conducted for the existing North Abutment.

The soil samples were generally taken using a 2" outside diameter split spoon sampler driven 18" into soil by a 140-pound hammer falling 30". Per Standard Penetration Test requirements, blows per 6" were recorded. The materials at the site, as indicated by the obtained samples, are included on the boring logs and are also included on the soil profile in Figures 3 to 6.

As indicated on the logs, the predominant materials at the site below the proposed spread footing foundation are comprised of medium dense to very dense, fine to coarse sand, gravel, and cobble materials on top of granite bedrock. No peat or clay materials were found at the site during the previous exploration operation. For detailed information on each boring, refer to the 1967 Existing Bridge Plans, included in Appendix 1 of this report.

3.3 Subsurface Profiles

The subsurface conditions, as reflected in the present boring exploration results, are provided below. Soil profiles of the site, based on the boring results, are included in Figures 3 to 6 of the report.

Boring B-222

Ground Surface is at Elevation 236.2'

Sample Depth (ft)	Soil Description
0'- 6"	Sandy Topsoil
S1: 6"- 1'6"	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel, little boulder.
4'9"	Top of bedrock (Granite)

No ground water was recorded.

Boring B-224

Ground Surface is at Elevation 232.7'

Sample Depth (ft)	Soil Description
0'- 6"	Sandy Topsoil
S1: 6"- 1'6"	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S2: 4'6"- 6'	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
8'	Refusal

No ground water was recorded.

Boring B-225

Ground Surface is at Elevation 230.2'

Sample Depth (ft)	Soil Description
0'- 6"	Sandy Topsoil
S1: 6"- 1'6"	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S2: 4'6"- 6'	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
8'	Top of Bedrock (Granite)

No ground water was recorded.

Boring B-230

Ground Surface is at Elevation 232.2'

Sample Depth (ft)	Soil Description
0'- 6"	Sandy Topsoil

S1: 6”- 1’6”	Dry, very loose, fine to coarse Sand, some fine to medium gravel and cobbles.
S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to medium gravel and cobbles.
S3: 10’- 11’6”	Dry, medium dense, fine to coarse Sand, some fine gravel, little silt and cobbles.
12’	Refusal

No ground water was recorded.

Boring B-231

Ground Surface is at Elevation 232.2’

Sample Depth (ft)	Soil Description
0’- 6”	Sandy Topsoil
S1: 6”- 1’6”	Dry, loose, fine to coarse Sand, some fine to medium gravel, little silt and cobbles.
S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, little silt and cobbles.
S3: 10’- 11’6”	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, little silt and cobbles.
S4: 15’- 16’	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, little silt and cobbles.
16’	Top of Bedrock (Granite)

No ground water was recorded.

Boring B-232

Ground Surface is at Elevation 230.2’

Sample Depth (ft)	Soil Description
0’- 6”	Sandy Topsoil
S1: 6”- 1’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S3: 9’- 10’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S4: 14’- 15’6”	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, little silt and cobbles.
17’	Refusal

No ground water was recorded.

Boring B-233

Ground Surface is at Elevation 230.2’

Sample Depth (ft)	Soil Description
0’- 6”	Sandy Topsoil
S1: 6”- 1’6”	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, trace silt and cobbles.

S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, trace silt and cobbles.
S3: 10’- 11’6”	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, trace silt and cobbles.
S4: 15’- 16’6”	Dry, medium dense, fine Sand, some fine gravel, little silt.
18’	Refusal

No ground water was recorded.

Boring B-234

Ground Surface is at Elevation 230.2’

Sample Depth (ft)	Soil Description
0’- 6”	Sandy Topsoil
S1: 6”- 1’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S3: 9’- 10’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S4: 14’- 15’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S5: 19’- 20’	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
20’	Refusal

No ground water was recorded.

Boring B-235

Ground Surface is at Elevation 231.2’

Sample Depth (ft)	Soil Description
0’- 6”	Sandy Topsoil
S1: 6”- 1’6”	Dry, loose, fine to coarse Sand, some fine to coarse gravel and cobbles.
S2: 4’6”- 6’	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S3: 10’- 11’6”	Dry, medium dense, fine to coarse Sand, some fine to coarse gravel and cobbles.
S4: 15’- 16’6”	Dry, medium dense, fine to coarse Sand, some fine to medium gravel, little silt.
18’6”	Refusal

No ground water was recorded.

Boring H-35

Ground Surface is at Elevation 234.2’

Sample Depth (ft)	Soil Description
S1: 0”- 1’6”	Dry, loose, fine Sand, little silt and fine gravel (Fill).
S2: 5’- 6’6”	Dry, medium dense, fine Sand, little silt and fine gravel (Fill).
S3: 10’- 10’7”	Wet, very dense, fine Sand and Cobbles.

10’7” Top of Bedrock (Granite)
Ground water was recorded at depth of 7’ (El. 227.2).

Boring H-36
Ground Surface is at Elevation 234.2’

Sample Depth (ft)	Soil Description
S1: 0”- 1’6”	Dry, loose, fine Sand, little fine gravel (Fill).
S2: 4’- 5’	Dry, very dense, fine Sand and decomposed Rock.
5’	Refusal

No ground water was recorded.

3.4 Bedrock Information

Bedrock at the site was encountered ranging from Elev. 216.2 to Elev. 226.5 as shown in the table below, with its average top at approximately Elev. 222.1.

Table 3.4: Bedrock Information

Boring No.	Ground Elevation (Feet)	Depth from Previous Existing Ground	Top of Bedrock (Elev.)	% of Recovery
B-222	236.2	9’-9”	Elev. 226.5	70%
B-225	230.2	8’-0”	Elev. 222.2	83%
B-231	232.2	16’-0”	Elev. 216.2	65%
H-35	234.2	10’-7”	Elev. 223.6	85%

3.5 Groundwater Information

Based on the logs shown on the existing plans, no groundwater information was recorded except for boring H-35. The information from the washed boring operation, however, is not accurate. It is presented below for preliminary reference only.

Table 3.5: Groundwater Information Measured after Washed Boring Operation

Boring No.	**Ground Elev. (Feet)**	**Groundwater Depth**	**Estimated Groundwater Elev. (Feet)**
H-35	Elev. 234.2	7’	Elev. 227.2

The approximate historic ground water level, measured years ago, most likely does not represent the ground water level today.

3.6 Seismic Design Category Evaluation

All seismic analysis and design of bridges shall be performed in accordance with the 'AASHTO Guide Specifications for LRFD Seismic Bridge Design'. Based on 2011 Guide Specifications, Table 3.4.2.1-1 (same as AASHTO Table 3.10.3.1-1) for site soil type and profile, the site class is C (F_{pga} = 1.2, F_a = 1.2, F_v = 1.7). With these results and Appendix 3, 'USGS Seismic Hazard Map for the 2,500-year return period', the following can be obtained:

Acceleration Coefficient (A_s) = 0.092g
Design Spectral Acceleration Coefficient at 0.2-sec Period (S_{DS}) = 0.160g
Design Spectral Acceleration Coefficient at 1.0-sec Period (S_{D1}) = 0.087g

Since S_{D1} is less than 0.15g, the Seismic Design Category (SDC) is A. This is based on Guide Specifications Table 3.5-1 (same as AASHTO Table 4.7.4.3.1-1).

3.7 Liquefaction Potential

A liquefaction assessment is not required for structures in Seismic Design Category (SDC) A. The proposed spread footing is supported by medium dense to very dense sand materials. Therefore, the proposed foundation is not susceptible to the seismic soil liquefaction effect.

4.0 FOUNDATION RECOMMENDATIONS

4.1 Proposed Bridge Foundation

Based on the information obtained from the 1964 & 1966 subsurface exploration program, the proposed spread footing foundation can be used to support the design loads for the proposed replacement superstructures.

The proposed bottom of footing is recommended to be at approximately Elev. 227.0. The footing bearing materials are predominately medium dense to very dense sand materials. These are competent materials capable of supporting the proposed precast concrete arch structure.

The nominal and factored bearing resistance values for the proposed foundations are included in Section 6 and Appendix 2 of the report.

The nominal and factored bearing resistance values are based on the previous 1964 & 1966 boring information before the existing bridges is constructed. After many years, the soil below the proposed spread footing could be more compacted than the soil based on the 1964 & 1966 boring information. Therefore, the nominal and factored bearing resistance values in this report are for preliminary design purposes only. It is the contractor's responsibility to verify the actual soil condition below the proposed spread footing.

4.2 Proposed Substructure Design Parameters

The following parameters are recommended for design at the back of the abutment wall:

Moist Unit Weight of Soil = 120 pcf
Soil Friction Angle = 32°
Active Lateral Earth Pressure Coefficient = 0.275
At-Rest Lateral Soil Pressure Coefficient = 0.470
(Based on AASHTO Table 10.5.5.2.2-1)

4.3 Soil Parameters

Based on the information from the boring logs, the following soil parameters are recommended:

Table 4.3: Recommended Soil Parameters

Precast Concrete Arch Structure	**Soil below Footing**	**Soil Unit Weight, γ (below footing)**	**Soil Friction Angle, ϕ_{f1} (above footing)**	**Soil Friction Angle, ϕ_{f2} (below footing)**	**Boring #**
East End	Medium Dense, Fine to Coarse San	120 pcf	32°	32°	B-231
Center	Medium Dense, Fine Sand	120 pcf	32°	32°	H-35
West End	Medium Dense, Fine to Coarse Sand	120 pcf	32°	32°	B-224

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Water Control

The water level recorded in the boring logs was not accurate due to the washed boring operation.

The following factors should be taken into consideration for the recorded water level:

- Groundwater levels obtained from the boring exploration operation are generally unreliable and are provided for reference only.
- Groundwater levels fluctuate over time due to seasonal changes.
- Contractor should not use the previous recorded water level for design until s/he has made an investigation to her/his satisfaction.

5.2 Earth Support / Cofferdam System

A temporary earth support is needed for the proposed foundation construction. The system shall be designed in accordance with the RIDOT Standard Specifications. The design of the system should consider that the top of the bedrock is high at the site.

5.3 Utilities

There are no utilities carried by the existing structures nor are there underground utilities at the bridge site, except an existing aqueduct buried below the middle of the proposed grade.

5.4 Obstruction

Abandoned underground utilities, previous excess construction materials, underground foundations, boulders, etc. are potential obstruction materials that may be encountered at the site during the proposed foundation excavation.

6.0 SUMMARY

Table 6.1: Summary of Bearing Resistance

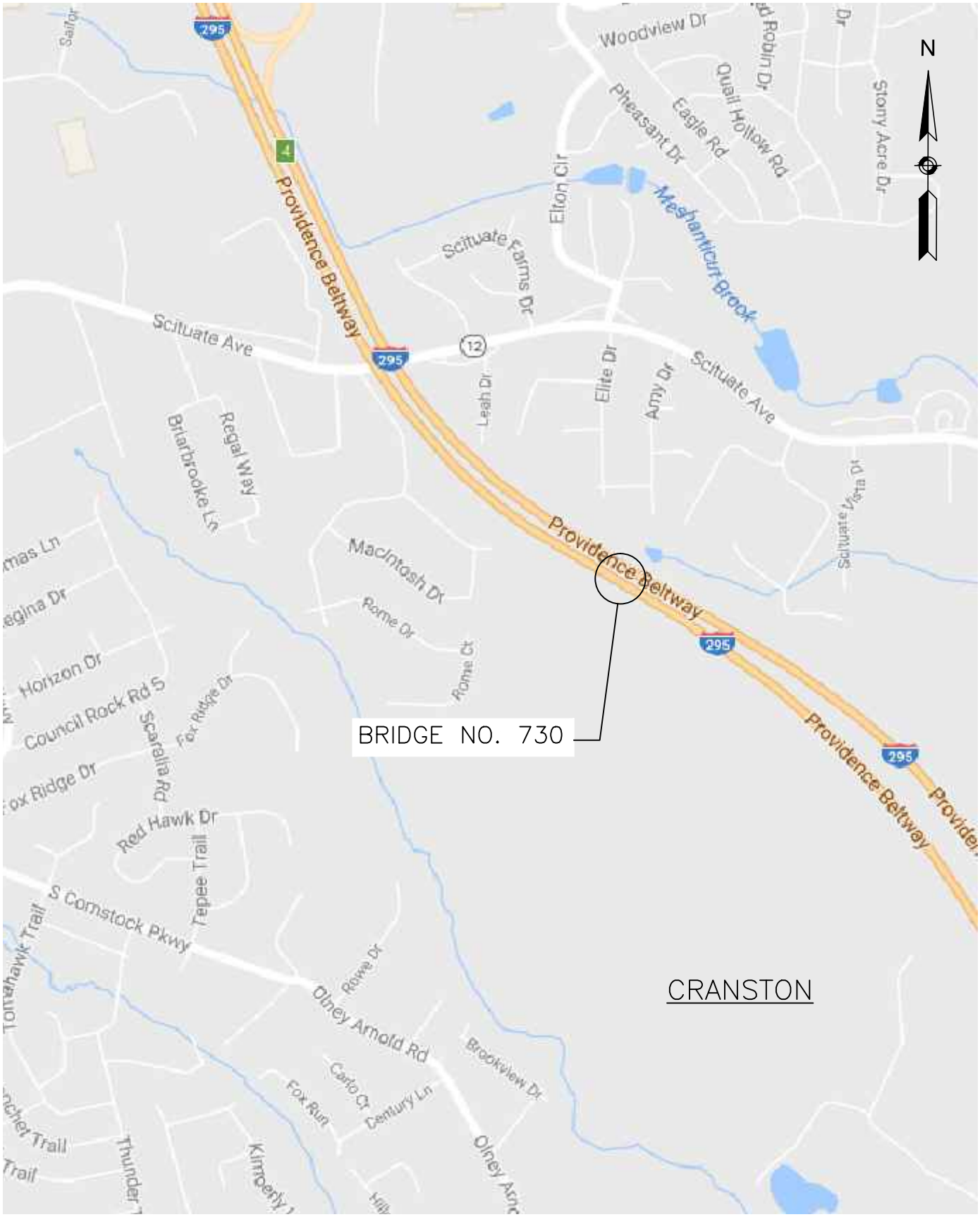
Effective Footing Width, B' (ft)	Nominal Bearing Resistance, q_n (ksf)	Factored Bearing Resistance, q_R (ksf)	Remarks
6.00	9.0	4.0	Based on Boring B-231 ϕ_{f1} = 32° ϕ_{f2} = 32°
7.00	9.4	4.2	
8.00	9.9	4.4	
9.00	10.3	4.6	
10.00	10.8	4.8	
12.00	11.2	5.0	
13.00	11.7	5.2	

Notes:

1. The water table is assumed to be at the bottom of footing.
2. Resistance Factor = 0.45
3. Contractor shall prepare resistance values based on the proposed loads in the final design.

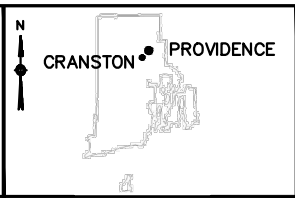
ϕ_{f1} = Angle of internal friction above bottom of footing
ϕ_{f2} = Angle of internal friction below bottom of footing

FIGURES



RI DOT

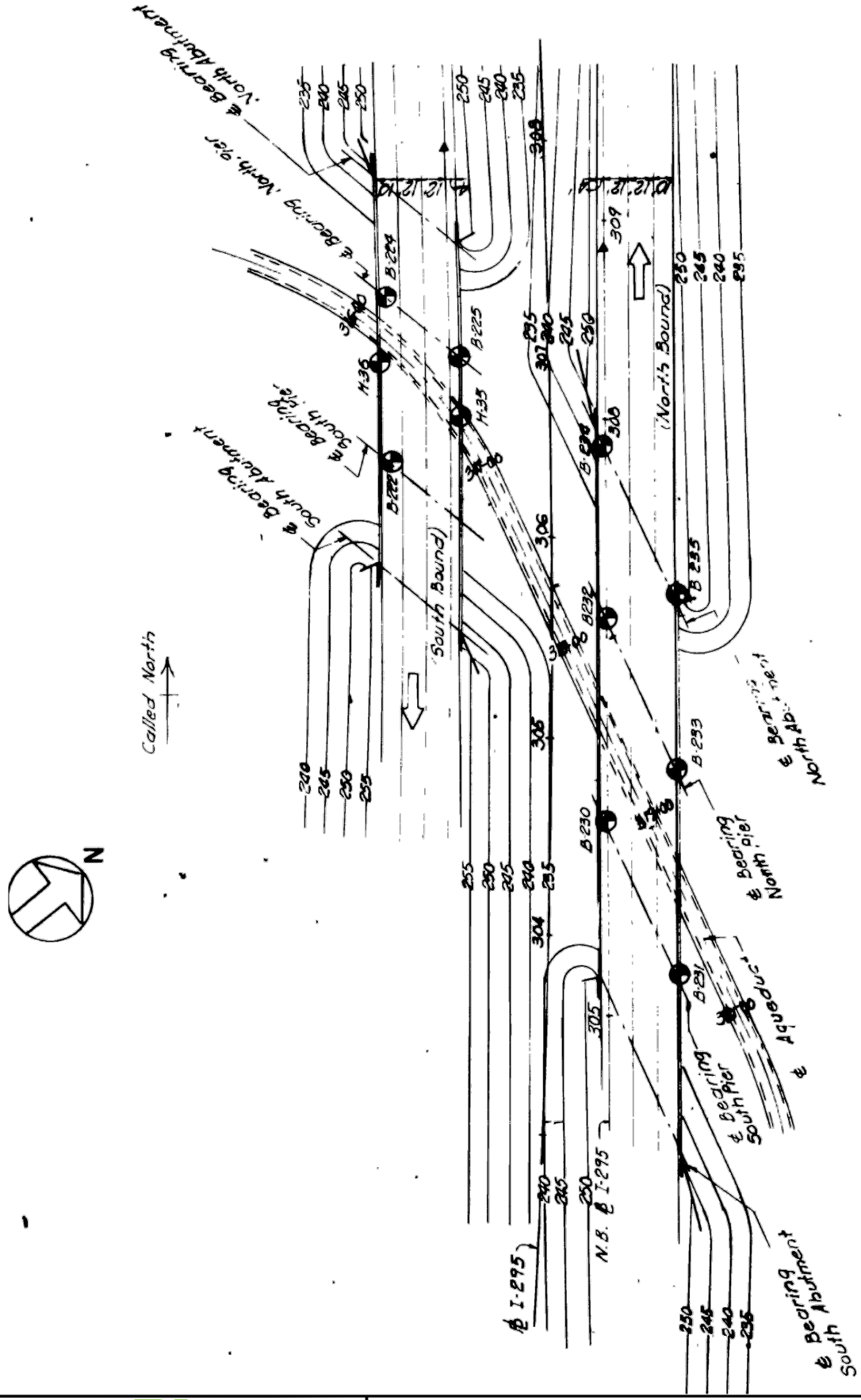
BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND



PROJECT LOCATION PLAN
NOT TO SCALE

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 1



B-XXX 1966 BORING LOCATION

H-XX 1964 BORING LOCATION

SOURCE: FROM RIDOT 1967 EXISTING PLAN FOR BRIDGE NO. 730.

RI DOT

BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

1964 & 1966 BORING LOCATION PLAN

NOT TO SCALE

PREPARED BY
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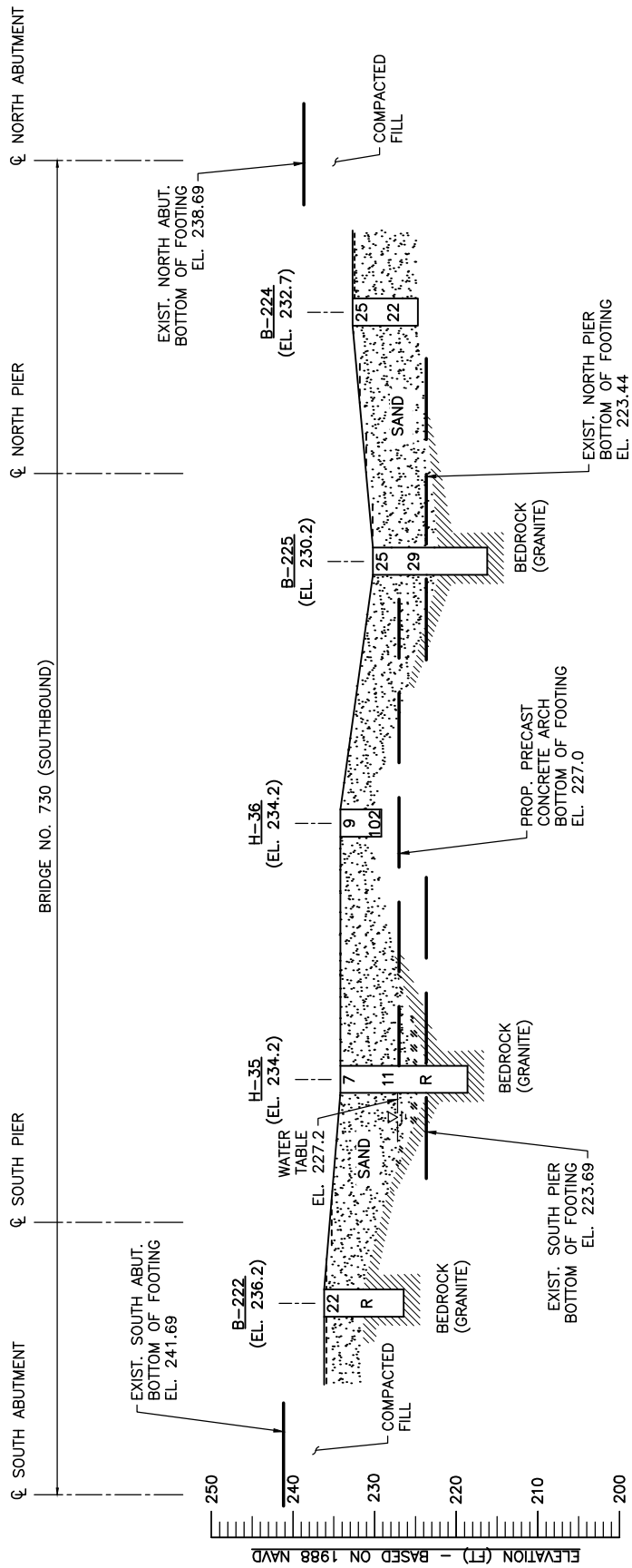
NEWTON, MASS.

Fig. 2



LEGEND

B-222 (EL. 236.2) BORING LOCATION AND IDENTIFICATION
GROUND ELEVATION

22 N VALUE (BLOWS/12")

R PRACTICAL REFUSAL (N>120)

BEDROCK

TOPSOIL

SAND

SAND & COBBLES

GENERAL NOTE:

SOIL MATERIALS SHOWN BETWEEN BORINGS ARE PRELIMINARY BASED ON CONDUCTED BORINGS INFORMATION, AND THEY ARE NOT GUARANTEED TO BE THE MATERIALS TO BE ENCOUNTERED AT THE SITE.

NOTES:

1. NUMBER IN COLUMNS INDICATE NUMBER OF BLOWS PER 12" ON A 2" O.D. SPLIT SPOON SAMPLER PRODUCED BY A 30 INCH FALL OF A 140 LB HAMMER.
2. ORIGINAL ELEVATIONS ARE BASED ON 1929 NGVD DATUM, WHICH HAVE BEEN CONVERTED TO 1988 NAVD DATUM BY SUBTRACTING 0.81'.

RI DOT

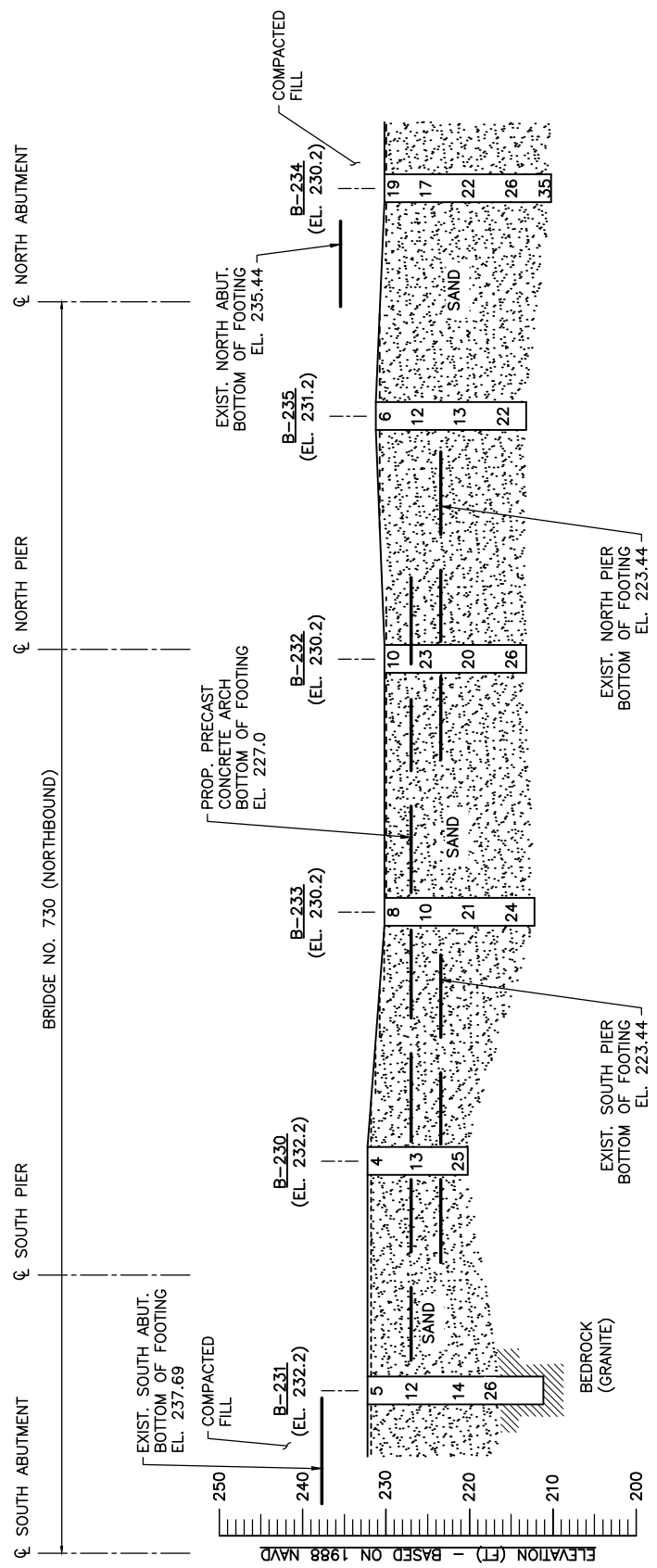
BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

SOIL PROFILE – SOUTHBOUND BRIDGE
(PICTORIAL VIEW)

SCALE: 3/64" = 1'-0"

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 3



GENERAL NOTE:

SOIL MATERIALS SHOWN BETWEEN BORINGS ARE PRELIMINARY BASED ON CONDUCTED BORINGS INFORMATION, AND THEY ARE NOT GUARANTEED TO BE THE MATERIALS TO BE ENCOUNTERED AT THE SITE.

NOTES:

1. NUMBER IN COLUMNS INDICATE NUMBER OF BLOWS PER 12" ON A 2" O.D. SPLIT SPOON SAMPLER PRODUCED BY A 30 INCH FALL OF A 140 LB HAMMER.
2. ORIGINAL ELEVATIONS ARE BASED ON 1929 NGVD DATUM, WHICH HAVE BEEN CONVERTED TO 1988 NAVD DATUM BY SUBTRACTING 0.81'.

LEGEND

B-231 (EL. 232.2) BORING LOCATION AND IDENTIFICATION GROUND ELEVATION

5 N VALUE (BLOWS/12")

BEDROCK

TOPSOIL

SAND

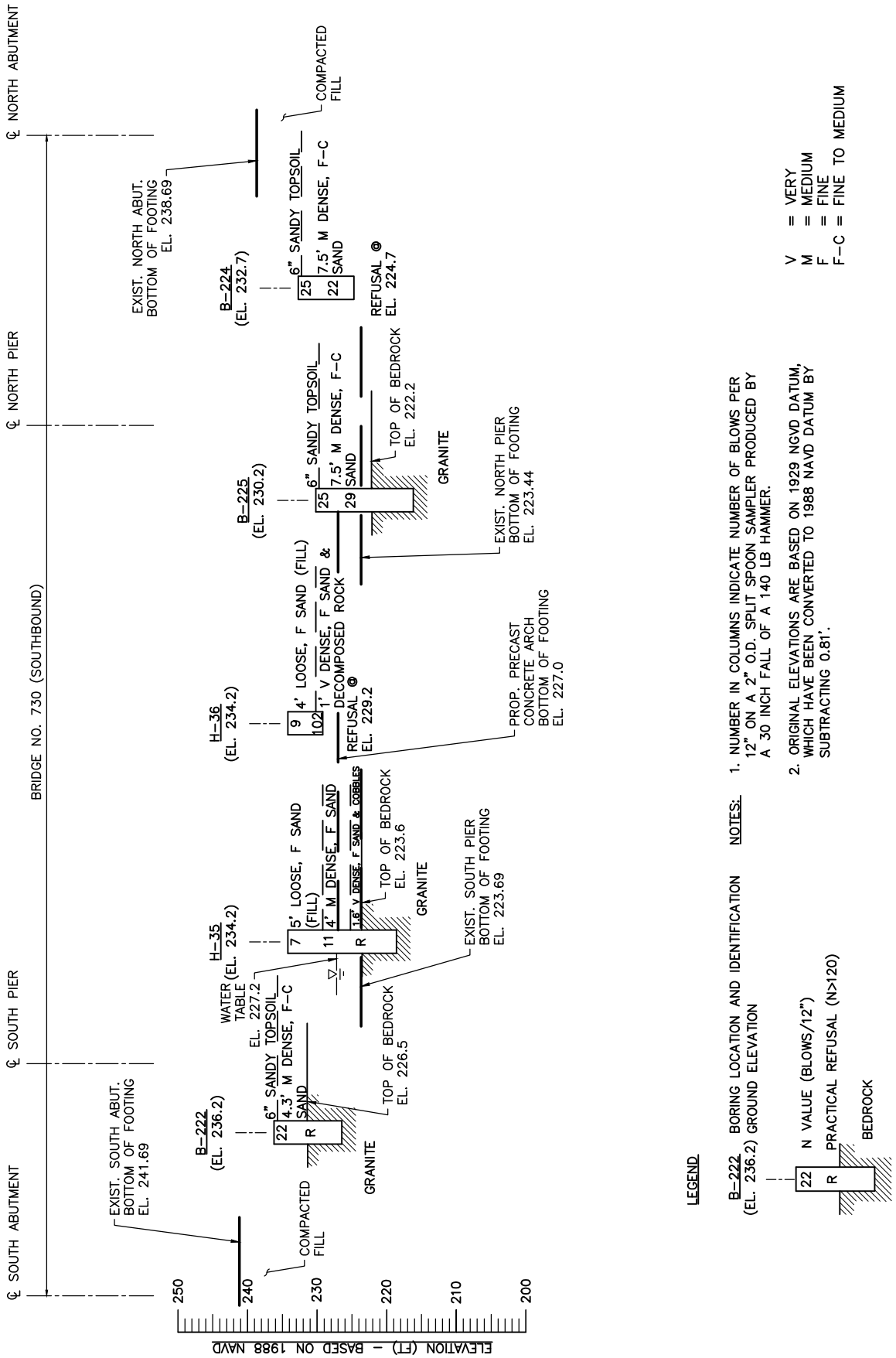
BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

SOIL PROFILE – NORTHBOUND BRIDGE
(PICTORIAL VIEW)

SCALE: 3/64" = 1'-0"

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 4



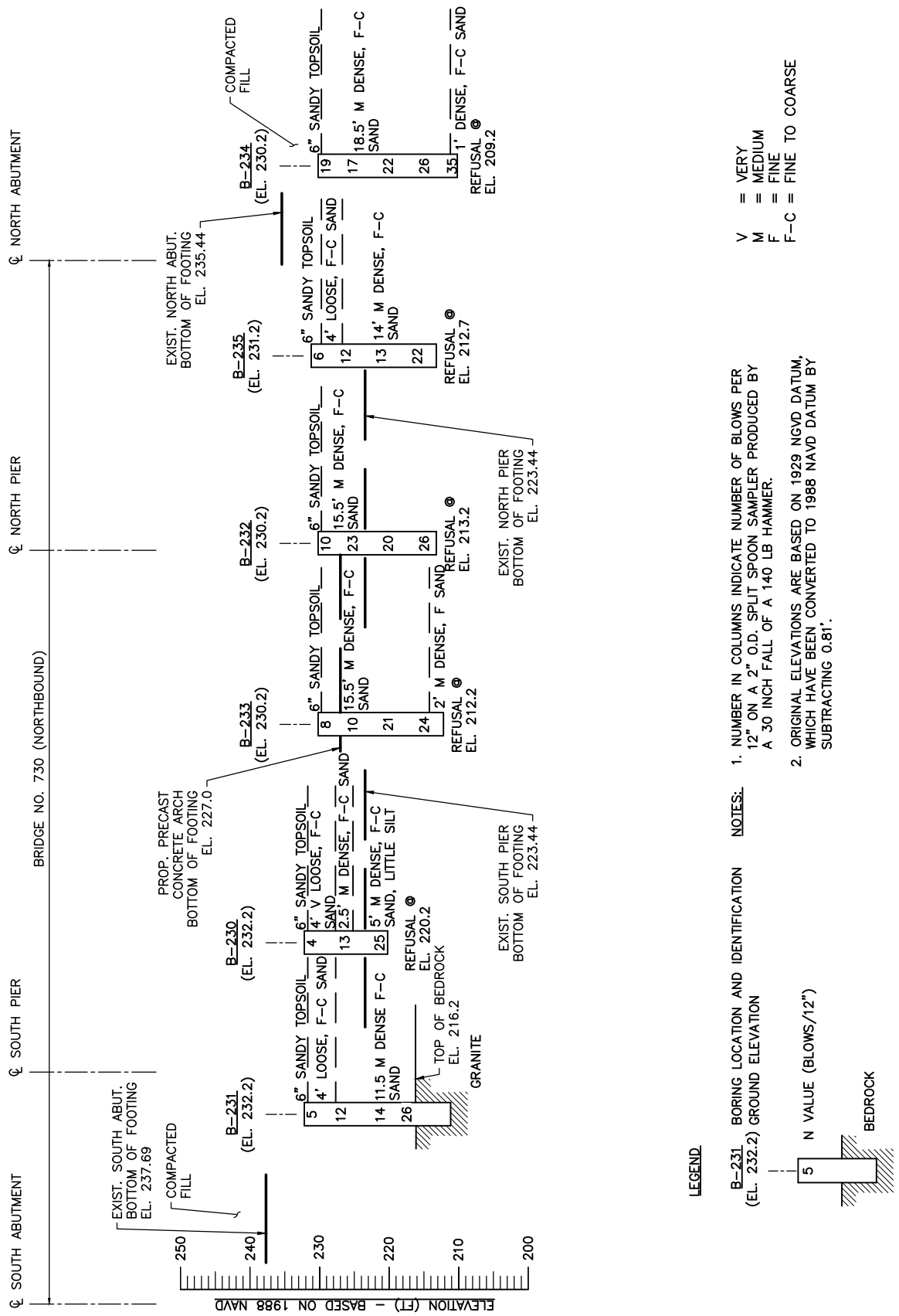
BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

SOIL PROFILE – SOUTHBOUND BRIDGE
(DESCRIPTIVE VIEW)

SCALE: 3/64" = 1'-0"

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 5



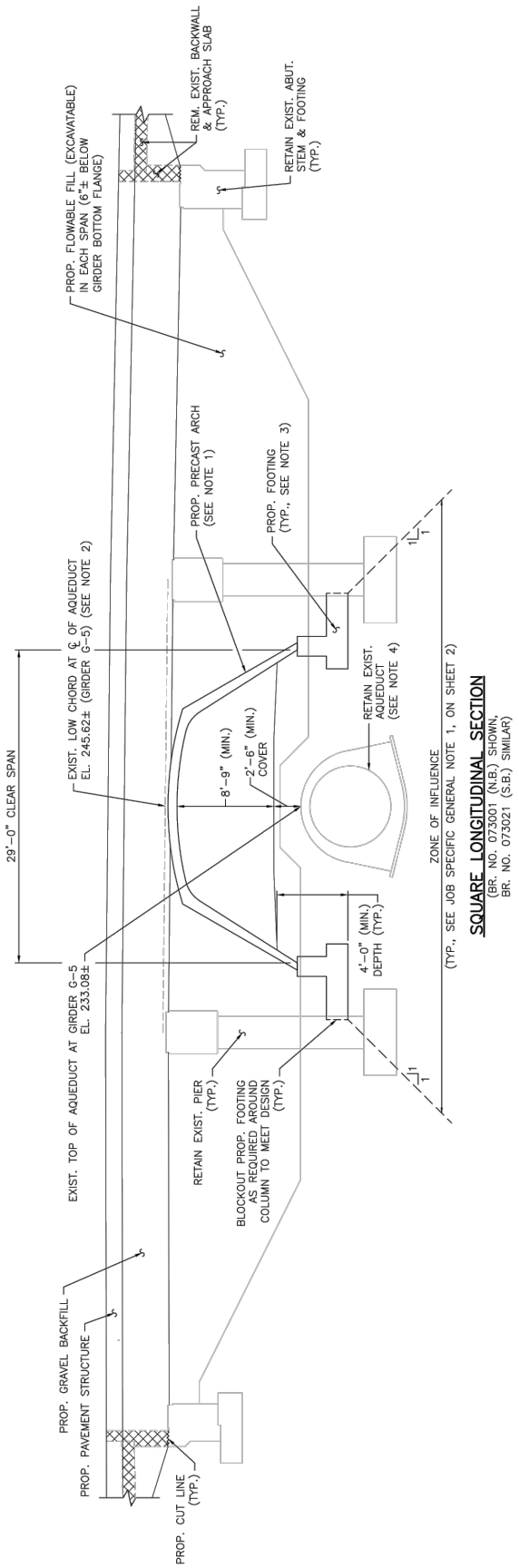
BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

SOIL PROFILE – NORTHBOUND BRIDGE
(DESCRIPTIVE VIEW)

SCALE: 3/64" = 1'-0"

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 6



SQUARE LONGITUDINAL SECTION
(BR. NO. 073001 (N.B.) SHOWN,
BR. NO. 073021 (S.B.) SIMILAR)

SOURCE: PLANS FROM WSP.

BRIDGE NO. 730 / INTERSTATE RTE 295
RALPHS POND TO SCITUATE AVE (C)
CRANSTON, RHODE ISLAND

SQUARE LONGITUDINAL SECTION
NOT TO SCALE

PREPARED BY
LAMSON ENGINEERING CORP.
NEWTON, MASS.

Fig. 7

APPENDIX 1

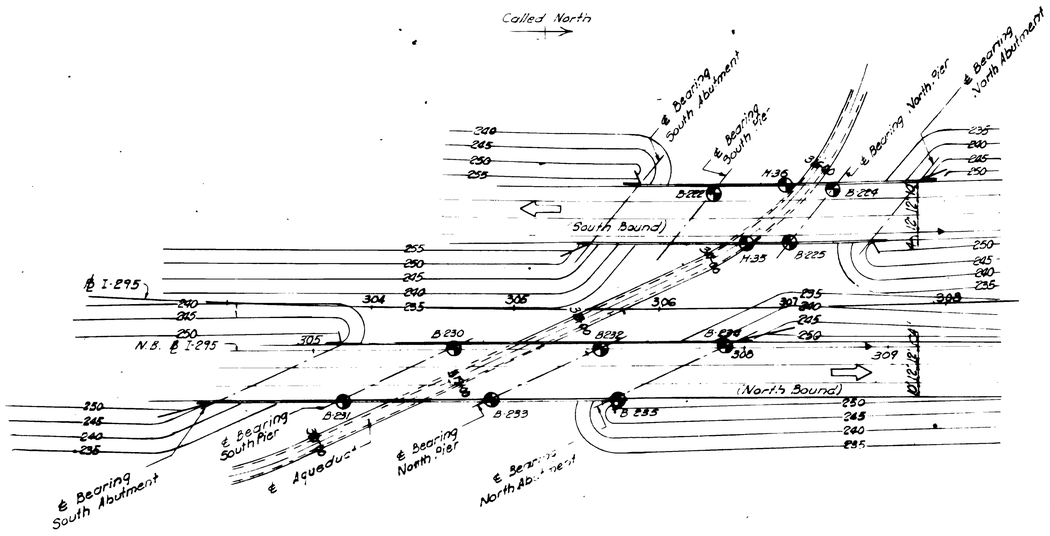
1967 Existing Bridge Plans

Notes:

- The elevations shown on the attached Existing 1967 Plans are based on the 1929 NGVD Datum.

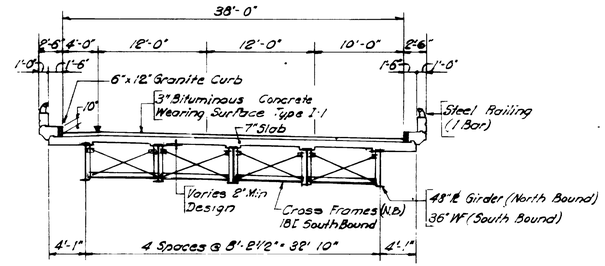
730,002

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	R. I.	I 295-5(23)32		B-47	95



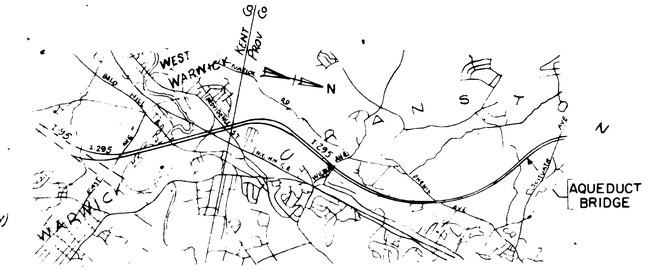
PLAN

SCALE: 1" = 50'



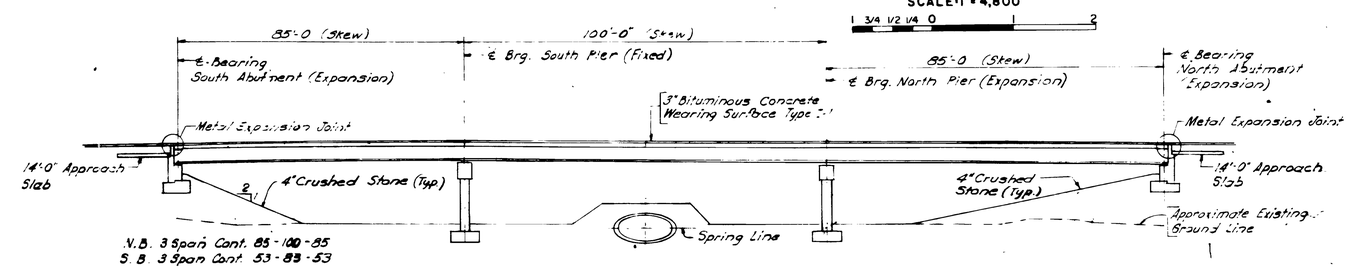
TRANSVERSE SECTION

SCALE: 1/8" = 1'-0"



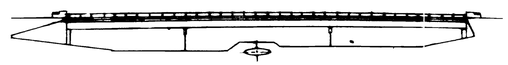
KEY MAP

SCALE: 1" = 4,800



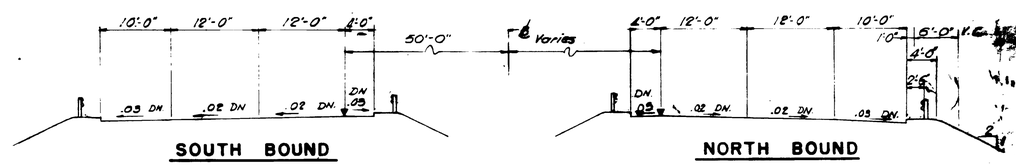
LONGITUDINAL SECTION (N.B.)

SCALE: 1" = 20'



ELEVATION

SCALE: 1" = 50'



SOUTH BOUND

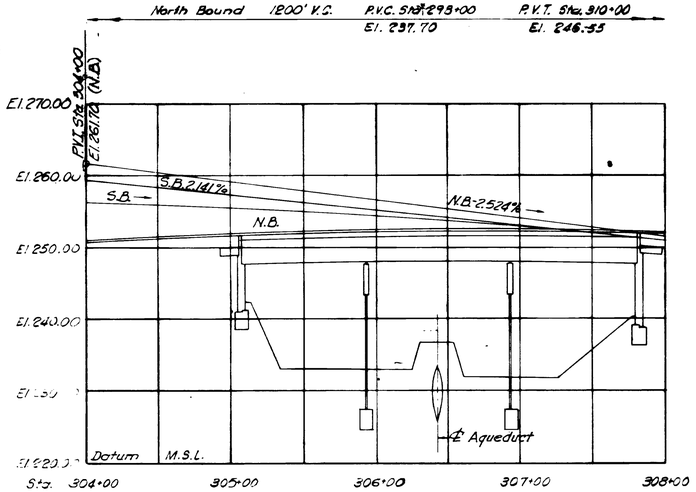
NORTH BOUND

I 295 APPROACH SECTIONS

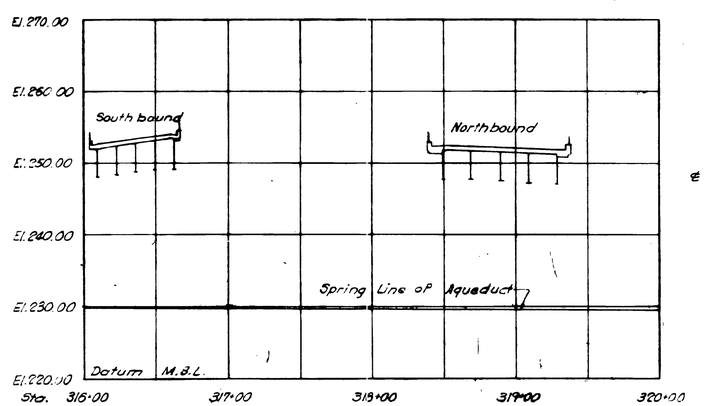
SCALE: 1" = 10'

South Bound 1100' V.C. P.V.C. Sta. 298+25 El. 242.34 P.V.I. Sta. 301+75 El. 254.34 P.V.T. Sta. 307+25 El. 252.56

North Bound 1200' V.C. P.V.C. Sta. 293+00 El. 237.70 P.V.T. Sta. 310+00 El. 246.55



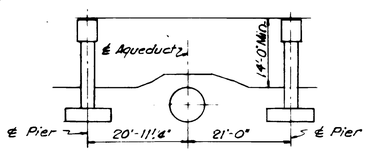
I 295



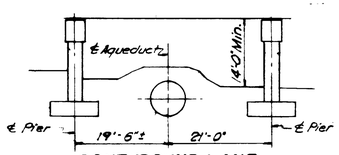
AQUEDUCT

PROFILES

SCALE – HORIZONTAL 1" = 50
VERTICAL 1" = 10'



NORTHBOUND LANE



SOUTHBOUND LANE

CLEARANCE DIAGRAMS

NOT TO SCALE

REVISIONS		
NO.	DATE	BY

RHODE ISLAND
DEPARTMENT OF PUBLIC WORKS
DIVISION OF ROADS AND BRIDGES

INTERSTATE ROUTE 295

RALPHS POND TO SCITUATE AVENUE (C)

CRANSTON RHODE ISLAND

AQUEDUCT BRIDGE
GENERAL PLAN & SECTIONS

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE, R. I. BOSTON, MASS.
DATE MARCH 1967 DESIGNED BY
SCALE AS SHOWN SHEET B-47 OF 95

APPROVED _____ SUPERVISING CIVIL ENGINEER (BRIDGE)
APPROVED _____ CHIEF ENGINEER
FINAL DATE

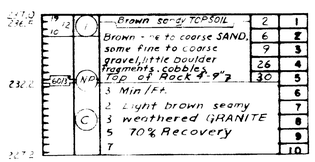
SET NO.
PRINTED
ISSUED

730,002

730,031

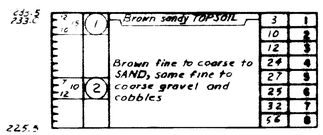
FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
1	R. I.			72	5

B-222



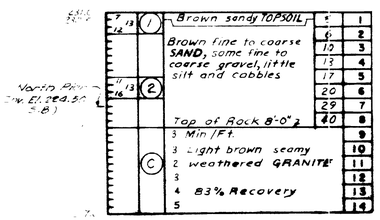
Bottom of Boring 9'-9"
Date Completed 8-22-66
No Water Recorded

B-224



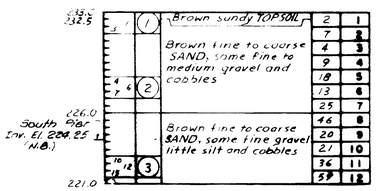
Refusal at 8'-0"
Date Completed None Recorded
No Water Recorded

B-225



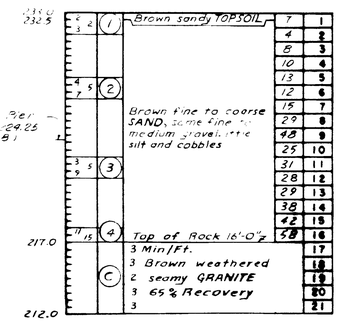
Bottom of Boring 14'-0"
Date Completed 8-25-66
No Water Recorded

B-230



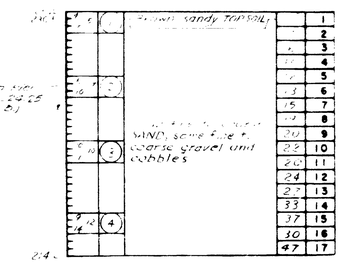
Refusal at 12'-0"
Date Completed 8-27-66
No Water Recorded

B-231



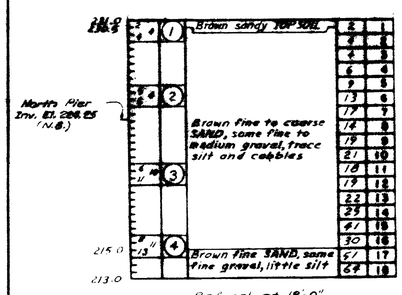
Bottom of Boring 21'-0"
Date Completed 8-29-66
No Water Recorded

B-232



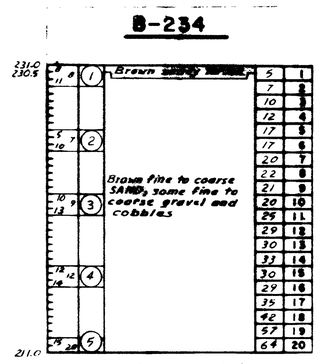
Refusal at 17'-0"
Date Completed None Recorded
No Water Recorded

B-233



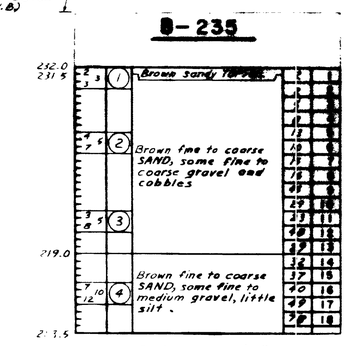
Refusal at 18'-0"
Date Completed 8-30-66
No Water Recorded

B-234



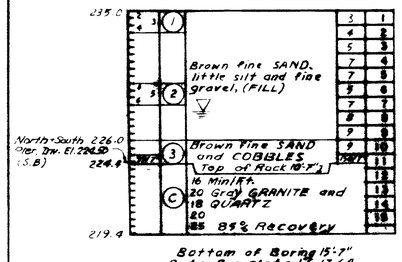
Refusal at 20'-0"
Date Completed 8-19-66
No Water Recorded

B-235



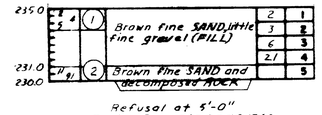
Refusal at 18'-6"
Date Completed 8-15-66
No Water Recorded

H-35



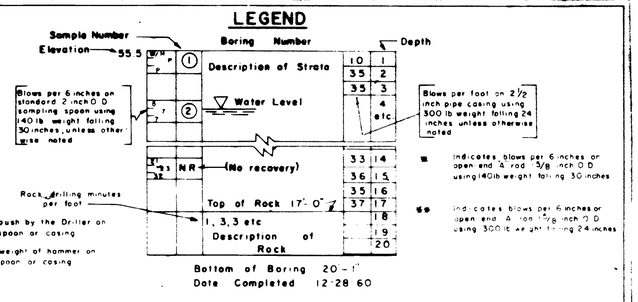
Bottom of Boring 15'-7"
Date Completed 6-17-64

H-36



Refusal at 5'-0"
Date Completed 6-17-64
No Water Recorded

LEGEND



Bottom of Boring 20'-1"
Date Completed 12-28-60

NOTES

1. Ground water levels indicated on the drawings are those existing at the time subsurface investigations were made and do not necessarily represent permanent ground water levels. It shall be the Contractors responsibility to determine annual and seasonal variations in ground water level which may effect his work. Variations in existing ground water levels from those indicated on the drawings shall under no conditions constitute grounds for revision in contract price or completion date.
2. Borings, probings, soundings, etc. were taken for the purpose of design and indicate subsurface conditions only at the locations shown. The Contractor shall be aware that subsurface conditions encounted during construction may vary from those shown on the Plans. See specifications.

NOTES:

1. Borings B-223, 226, 227, 228, and 229 were deleted from the program.
2. Borings B-222 through B-235 were done by the R. I. Dept. of Public Works on the dates indicated.
3. Borings H-35 and H-36 were done by American Drilling Co. Inc. on the dates indicated.

REVISIONS		
NO.	DATE	BY

RHODE ISLAND
DEPARTMENT OF PUBLIC WORKS
DIVISION OF ROADS AND BRIDGES

INTERSTATE ROUTE 295

RALPHS POND TO SCITUATE AVENUE (C)

CRANSTON RHODE ISLAND

AQUEDUCT BRIDGE
BORINGS-B-222, B-224, B-225 B-230 THRU B-235, H-35 & H-36

CHARLES A. MAGUIRE & ASSOCIATES
ENGINEERS
PROVIDENCE, R. I. BOSTON, MASS.

DATE MARCH 1967 DESIGNED BY
SCALE
EXCEPT AS NOTED SHEET 8-76 OF 95

APPROVED _____ SUPERVISING CIVIL ENGINEER (BRIDGE)
APPROVED _____ CHIEF ENGINEER
FINAL DATE

SET NO.
PRINTED
ISSUED TO

730,031

APPENDIX 2

Calculations

INDEX OF CALCULATIONS

Title	Page
Summary	1
Bearing Resistance	2

LAMSON ENGINEERING CORPORATION		Final Page No.: 1
Project: Bridge No. 730, Cranston	Job No.:	Preliminary Sheet No.:
Subject:	Prepared by: JJL	Date: 8/2017
Detail: Summary	Checked by: WD	Date: 8/2017

Summary of Bearing Resistance

Recommended Bottom of Footing = Elev. 227.0

Effective Footing Width, B' (ft)	Nominal Bearing Resistance, q_n (ksf)	Factored Bearing Resistance, q_R (ksf)	Remarks
6.00	9.0	4.0	Based on Boring B-231 ϕ_{f1} = 32° ϕ_{f2} = 32°
7.00	9.4	4.2	
8.00	9.9	4.4	
9.00	10.3	4.6	
10.00	10.8	4.8	
12.00	11.2	5.0	
13.00	11.7	5.2	

Notes:

1. Water is assumed to be at the bottom of footing elevation.

ϕ_{f1} = Angle of internal friction above bottom of footing
ϕ_{f2} = Angle of internal friction below bottom of footing

LAMSON ENGINEERING CORPORATION			Final Page No:	2
Project:	Bridge No. 730, Cranston	Job No:	Preliminary Sheet No:	
Subject:	Precast Concrete Arch	Prepared by: JJL	Date:	8/2017
Detail:	Bearing Resistance	Checked by: WD	Date:	8/2017

Bearing Resistance - East Abutment

Bearing Resistance at the Service Limit State

Presumptive Bearing Resistance for Spread Footing (Table C10.6.2.6.1-1)

Bearing Resistance, q = 5 ksf

Bearing Resistance of Soil at the Strength Limit State Design

Footing length, L = 60 ft (Assume exp. to exp. Joints)

Factored bearing resistance, q_R = $\varphi_b \, q_n$ (AASHTO 10.6.3.1.1-1)

Bearing resistance factor, φ_b = 0.45 (AASHTO Table 10.5.5.2.2-1)

Nominal bearing resistance, q_n = $c\, N_{cm} + \gamma\, D_f\, N_{qm}\, C_{wq} + 0.5\, \gamma\, B\, N_{\gamma m}\, C_{w\gamma}$

N_{cm} = $N_c\, s_c\, i_c$

N_{qm} = $N_q\, s_q\, d_q\, i_q$

$N_{\gamma m}$ = $N_\gamma\, s_\gamma\, i_\gamma$

Because of c = 0,

Nominal bearing resistance, q_n = $\gamma\, D_f\, N_{qm}\, C_{wq} + 0.5\, \gamma\, B\, N_{\gamma m}\, C_{w\gamma}$

i_q = $[\,1 - H / V\,]^n$

= See table next page

i_γ = $[\,1 - H / V\,]^{(n+1)}$

= See table next page

n = $[(2+L / B) / (1+ L / B)]\cos^2\theta + [(2+B / L) / (1+ B / L)]\sin^2\theta$

= See table next page

Unfactored horizontal load, H = 4.87 kips/ft (From WSP)

Unfactored vertical load, V = 19.19 kips/ft (From WSP)

Project direction of load in , θ = 90 deg (measured from the side of L)
the plane of the footing

Soil properties above footing

Angle of internal friction, Φ_{f1} = 32 degree

Moist unit weight of soil, γ_1 = 0.12 kcf

LAMSON ENGINEERING CORPORATION				Final Page No:	3
Project:	Bridge No. 730, Cranston	Job No:		Preliminary Sheet No:	
Subject:	Precast Concrete Arch	Prepared by:	JJL	Date:	8/2017
Detail:	Bearing Resistance	Checked by:	WD	Date:	8/2017

Footing embedment depth, D_f = 4 ft

Surcharge bearing capacity factor, N_q = 23.2 (AASHTO Table 10.6.3.1.2a-1)

For $\Phi_{f1} > 0$ Shape factor, s_q = $1 + (B' / L) \tan \Phi_{f1}$ (AASHTO Table 10.6.3.1.2a-3)

= See table this page

Depth Correction Factor, d_q = 1

Correction factors to account for the location of the ground water table (AASHTO Table 10.6.3.1.2a-2)

C_{wq} = 1.00

Soil properties below footing

Angle of internal friction, Φ_{f2} = 32 degree

Moist unit weight of soil, γ_2 = 0.12 kcf ← γ_2

Unit Weight bearing capacity factor, N_γ = 30.2 (AASHTO Table 10.6.3.1.2a-1)

For $\Phi_{f2} > 0$ Shape factor, s_γ = $1 - 0.4 (B' / L)$ (AASHTO Table 10.6.3.1.2a-3)

See Table this page

Correction factors to account for the location of the ground water table

$C_{w\gamma}$ = 0.5 (AASHTO Table 10.6.3.1.2a-2)

B' (ft)	n	i_q	i_γ	s_q	s_γ	N_{qm}	$N_{\gamma m}$	$\gamma_1 D_f N_{qm} C_{wq}$ (ksf)	$0.5 \gamma_2 B' N_{\gamma m} C_{w\gamma}$ (ksf)	q_n (ksf)	q_R (ksf)
6.0	1.91	0.572	0.427	1.06	0.96	14.07	12.38	6.75	2.23	8.98	**4.0**
7.0	1.90	0.574	0.428	1.07	0.95	14.25	12.28	6.84	2.58	9.42	**4.2**
8.0	1.88	0.576	0.43	1.08	0.95	14.43	12.34	6.93	2.96	9.89	**4.4**
9.0	1.87	0.579	0.432	1.09	0.94	14.64	12.26	7.03	3.31	10.34	**4.6**
10.0	1.86	0.581	0.433	1.10	0.93	14.83	12.16	7.12	3.65	10.77	**4.8**
11.0	1.85	0.583	0.435	1.11	0.93	15.01	12.22	7.20	4.03	11.23	**5.0**
12.0	1.83	0.585	0.436	1.12	0.92	15.20	12.11	7.30	4.36	11.66	**5.2**

Factored bearing resistance

APPENDIX 3

Seismic Design Information

(4 Sheets)

LAMSON ENGINEERING CORPORATION			Final Page No.: 1
Project:	I-295 Bridge No. 730, Cranston	**Job No.:**	**Preliminary Sheet No.:**
Subject:	Seismic Analysis	**Prepared by:** JJL	**Date:** 8/2017
Detail:	Site Class	Checked by: WD	**Date:** 8/2017

B-231

Depth ft	*Soil* Layer	*N* blows/ft	*Ave. N* blows/ft	d_i ft	d_i / N_i ft^2/blow
1.00	1	5	5	1	0.20
5.00	2	12	17	15	0.87
10.00		14			
16.00		26			
100.00		R	120	84	0.70
			Sum	100	1.77

Boring Ground Surface Elev. 232.2

$\bar{N}$ = 57 > 50

Therefore, the site class is C per Table 3.4.2.1-1 of AASHTO Guide Specification.

From Seismic Design Maps:

PGA = 0.077 g

S_s = 0.133 g

S_1 = 0.051 g

Per Tables 3.4.2.3-1 & 3.4.2.3-2 of AASHTO Guide Specification

F_{pga} = 1.2

F_a = 1.2

F_v = 1.7

Therefore,

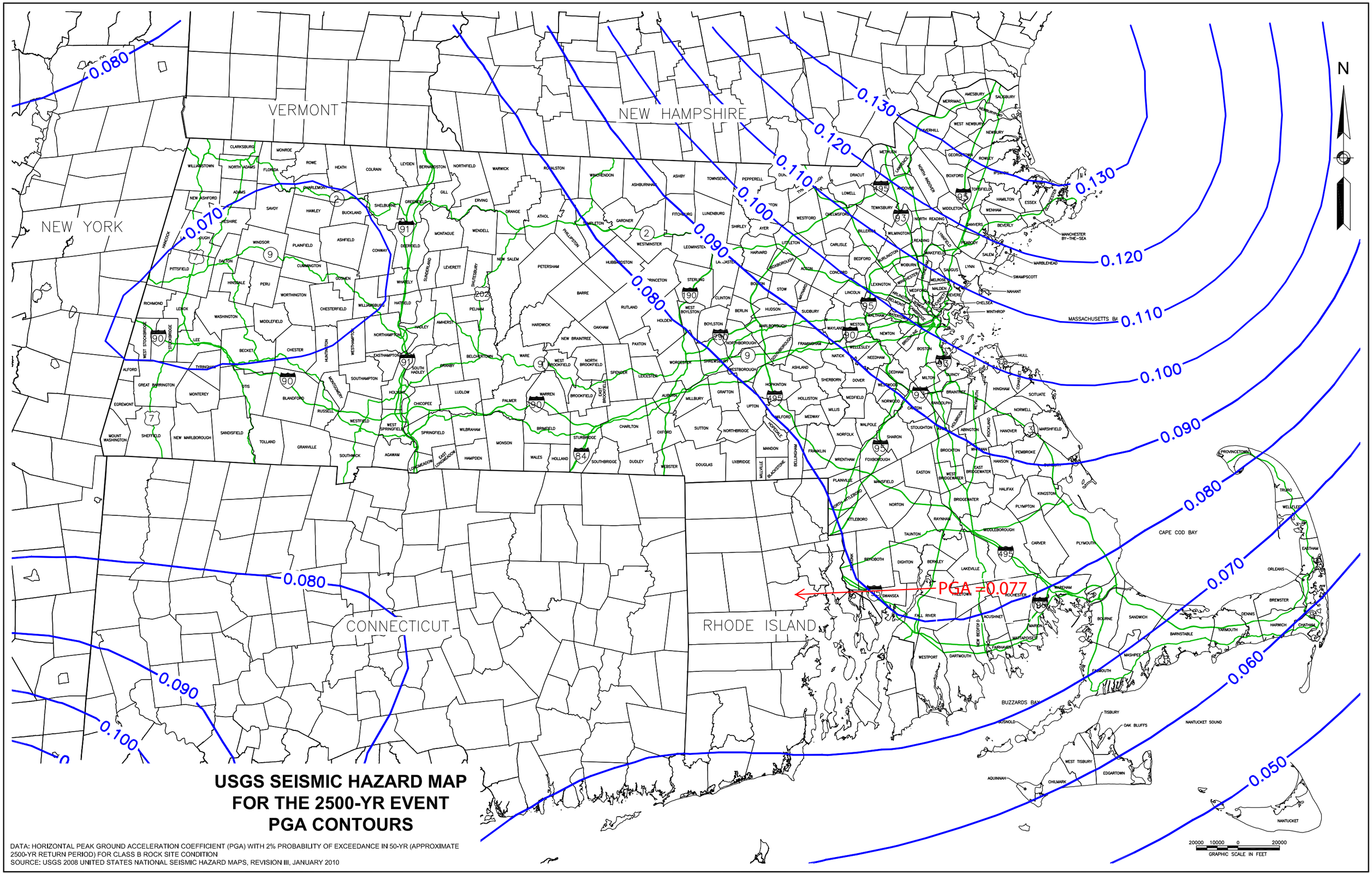
A_s = 0.092 g

S_{DS} = 0.160 g

S_{D1} = 0.087 g < 0.15g

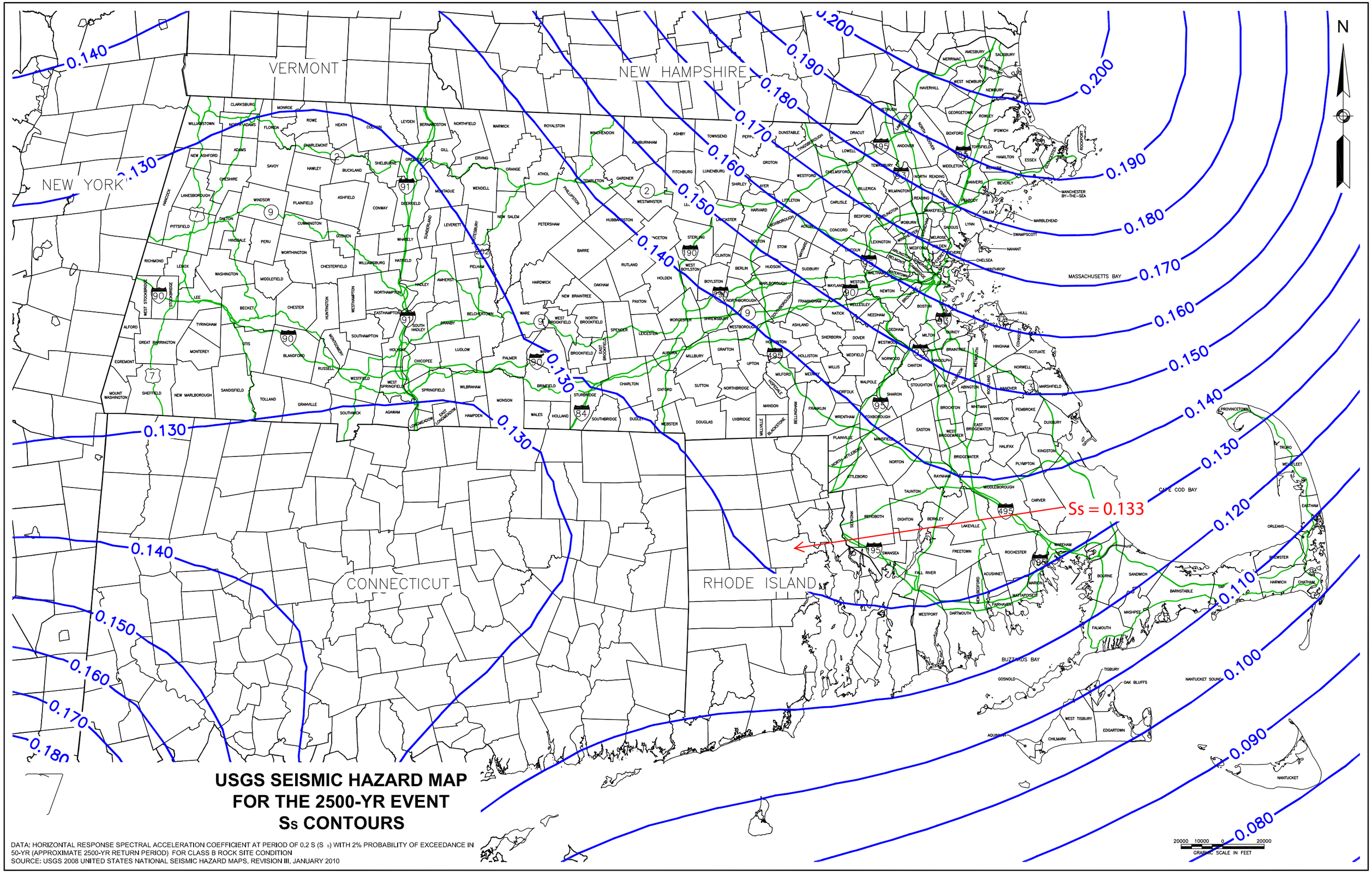
Seismic Design Category (SDC) = A

Summary: Based on AASHTO Guide Specifications Table 3.5-1, a liquefaction assessment is not required for structures in Seismic Design Category (SDC) A area.



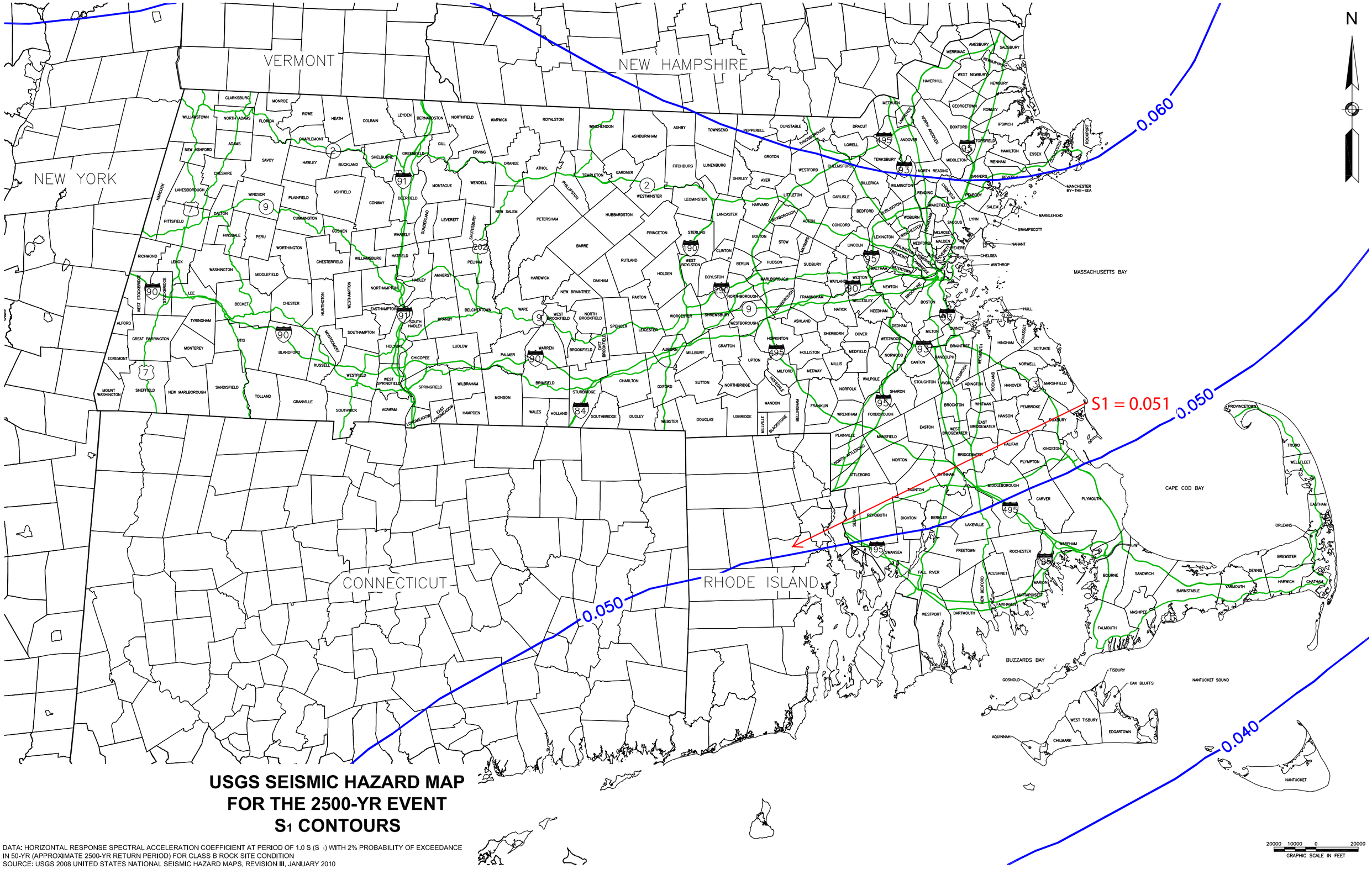
USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT PGA CONTOURS

DATA: HORIZONTAL PEAK GROUND ACCELERATION COEFFICIENT (PGA) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010



USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT S_S CONTOURS

DATA: HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT PERIOD OF 0.2 S (S_a) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010



USGS SEISMIC HAZARD MAP FOR THE 2500-YR EVENT S_1 CONTOURS

DATA: HORIZONTAL RESPONSE SPECTRAL ACCELERATION COEFFICIENT AT PERIOD OF 1.0 S (S_1) WITH 2% PROBABILITY OF EXCEEDANCE IN 50-YR (APPROXIMATE 2500-YR RETURN PERIOD) FOR CLASS B ROCK SITE CONDITION
SOURCE: USGS 2008 UNITED STATES NATIONAL SEISMIC HAZARD MAPS, REVISION III, JANUARY 2010

20000 10000 0 20000
GRAPHIC SCALE IN FEET

APPENDIX 4

Loads from WSP

(8 Sheets)

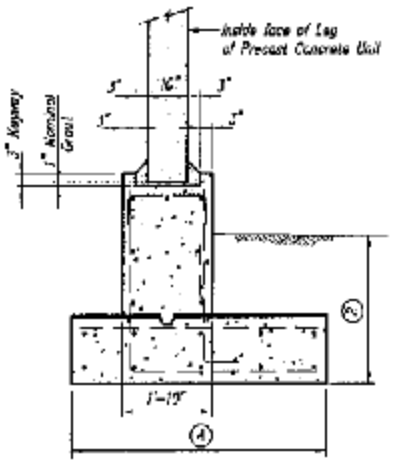
FOOTING STABILITY CHECK

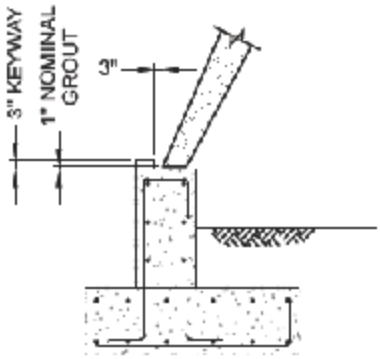
References

Design Properties:

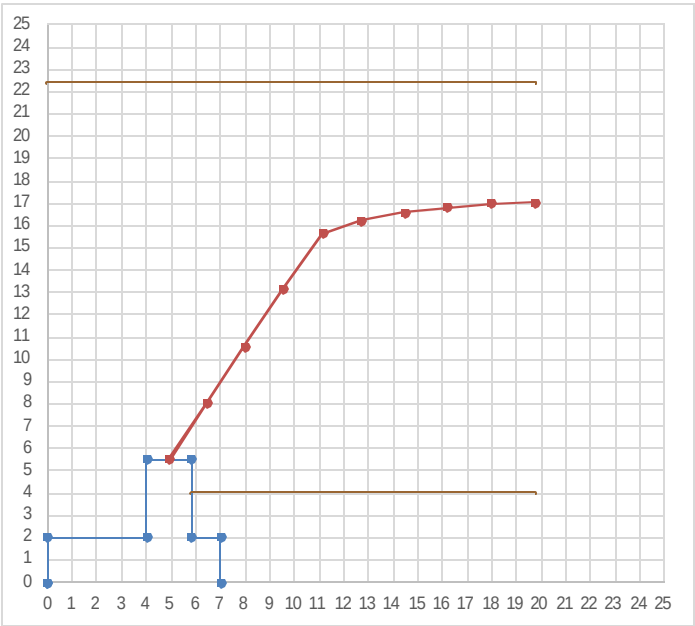
			Radians		References
Concrete Unit Weight, g_c =	**0.150**	kip/ft^3			MassDOT LRFD
Soil Unit Weight, g_s =	**0.120**	kip/ft^3			Br. Manual
Water Unit Weight, g_w =	**0.062**	kip/ft^3			Pt I - 3.1.5
Backfill Soil Internal Friction Angle, f =	**32.0**	° (typ. backfill)	0.5585	***- From abut design for 732***	
Soil Below Footing Internal Friction Angle, f_f =	**37.0**	° (for sliding check)		***- From abut design for 733***	
Slope Angle of Soil, b =	**0.0**	°	0.0000		
Angle of Backface of Wall, q =	**90.0**	°	1.5708		
Angle of Wall Friction, d =	**22.0**	° (gravel sand)	0.3840	***- From abut design for 73***	AASHTO LRFD
Abutment Height, H =	5.50	ft (for lateral earth pressure - (footing + pedestal height only))			Tbl. 3.11.5.3-1
Height of Water Table, h_w =	**0.00**	ft - drained backfill			
Surcharge Height, h_s =	**0.00**	ft			AASHTO LRFD
Factored Bearing Capacity, p =	**9.50**	kip/ft^2		***- From abut design for 73***	Tbl. 3.11.6.4-1

Footing Width =	**7.00**	ft
Footing Thickness =	**2.00**	ft
Pedestal Width =	**1.83**	ft
Pedestal Height =	**3.50**	ft
Toe Width =	**4.00**	ft
Heal Width (span side) =	**1.17**	ft
CL Brng to Toe =	4.92	ft





PEDESTAL WALL FOOTING



Summary of Results:

	Strength I	Strength IV	Service I
Bearing Resistance	**OK**	**OK**	**OK**
Bearing R_u/R_r	**1.09**	**1.22**	**1.84**
Overturning	**OK**	**OK**	**OK**
Sliding	**OK**	**OK**	**OK**

FOOTING STABILITY CHECK

References

Assumptions:

1. Analysis is done according to the methods outlined in AASHTO LRFD Manual.
2. Analysis performed checking per foot of wall length.
3. The ***design earth pressure coefficient*** following guidance in MassDOT Bridge Manual for buried structure less than 16' in height and not founded on piles or rock. — MassDOT LRFD Br. Manual Pt. I - 3.1.5
4. The loading on the footing consists of the superstructure horizontal and vertical reactions applied at the top of the footing pedestal, the soil weight located over the footing beyond the centerline of bearing of the arch and soil weight under the arch in front of the centerline of bearing.
4. Several loading conditions were considered to determine superstructure load, which resulted in different ratios of vertical and horizontal loads.

Active Earth Pressure Coefficient:

$K_a = [\sin^2(q + f)]/[(\sin^2 q * \sin(q - d)) * (1 + \sqrt{((\sin(f + d) * \sin(f - b))/(\sin(q - d) * \sin(q + b)))^2}]$ — AASHTO LRFD 3.11.5.3

where:

$\sin^2(q + f) =$	0.719	$\sin(f + d) =$	0.809
$\sin^2 q =$	1.000	$\sin(f - b) =$	0.530
$\sin(q - d) =$	0.927	$\sin(q + b) =$	1.000

$K_a =$ 0.275

At-rest Earth Pressure Coefficient:

$K_o = 1 - \sin f$ — AASHTO LRFD 3.11.5.2

$= 0.470$

Design Earth Pressure Coefficient: — MassDOT LRFD Br. Manual 3.1.5

$K_d = 0.5 * (K_o + K_a)$

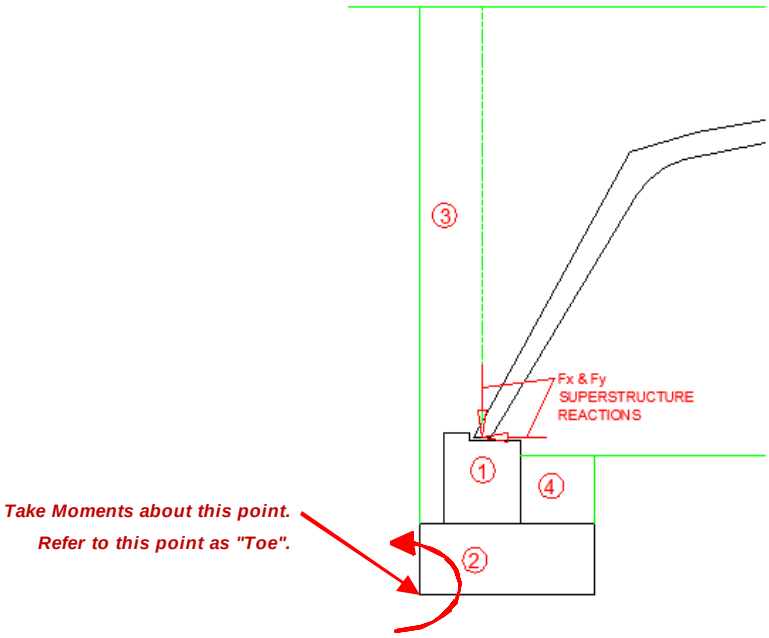
$= 0.372$ *(Less than 16', on spread footing)*

References

FOOTING STABILITY CHECK

Footing Loads:

Fill Height at Arch Bearing =	16.88	ft	- from DL calcs (fill height @ Node 1)
Fill Height Above Footing Toe =	20.38	ft	- above dimensions + Pedestal height
Fill Height @ Bottom of Footing =	22.38	ft	



Section	Base (ft)	Height (ft)	Material Density	Weight, V (kip)	Mom. Arm (ft)	Moment (kip-ft)
1-Pedestal	**1.83**	**3.50**	**0.150**	0.96	**4.92**	4.73
2-Footing	**7.00**	**2.00**	**0.150**	2.10	**3.50**	7.35
			V_{DC} =	3.06	M_{DC} =	12.08

Vertical Earth Pressure, EV1:

Section	Base (ft)	Height (ft)	Material Density	Weight, V (kip)	Mom. Arm (ft)	Moment (kip-ft)
3- "Toe" Soil	**4.00**	**20.38**	**0.120**	9.78	**2.00**	19.57
			V_{EV} =	9.78	M_{EV} =	19.57

Vertical Earth Pressure, EV2:

Section	Base (ft)	Height (ft)	Material Density	Weight, V (kip)	Mom. Arm (ft)	Moment (kip-ft)
4- "Heal" Soil	**1.17**	**2.00**	**0.120**	0.28	**6.42**	1.80
			V_{EV} =	0.28	M_{EV} =	1.80

Horizontal Earth Pressure, EH:

- Assuming footing rotates about the assumed location (see diagram above), the EH pressure on the back face of the pedestal and footing would be a passive earth pressure that resists overturning. Conservatively ignore this pressure. To avoid being overly conservative also ignore small EH overturning load on front face abutment (span side).

FOOTING STABILITY CHECK

References

Live Load Surcharge, LS:

- Assuming footing rotates about the assumed location (see diagram above), the LS pressure on the back face of the pedestal and footing would be a passive earth pressure that resists overturning. Conservatively ignore this pressure.

Thermal Uniform Load, TU: *- Assume no TU load*

Breaking Force, BR: *- Assume no BR load*

References

FOOTING STABILITY CHECK

Factored Rigid Frame Reactions:

		Max+ Hz	Max- Hz	Max Vt	
Unfactored HL-93 Tandem:	Horizontal Reaction =	1.527	0.000	1.074	kips
	Vertical Reaction =	1.778	0.867	2.302	kip-ft
Unfactored HL-93 Truck:	Horizontal Reaction =	1.385	0.000	1.000	kips
	Vertical Reaction =	2.034	1.110	2.292	kip-ft
Controlling Factored HL-93 Loads:	Horizontal Reaction =	2.672		1.880	kips
	Vertical Reaction =	3.112		4.029	kip-ft

(Use for Strength & Service)

Limit State Load Combination	Factored DEAD LOADS Vertical Force (kip)	Factored DEAD LOADS Horizontal Force (kip)	Factored LIVE LOADS Vertical Force (kip)	Factored LIVE LOADS Horizontal Force (kip)	
Strength I, Load Case 1	**23.24**	**8.96**	**3.11**	**2.67**	**<-- Use for Strength I**
Strength I & IV, Load Case 2	**15.66**	**0.72**	**0.00**	**0.00**	
Strength I, Load Case 3	**23.23**	**4.37**	**3.11**	**2.67**	
Strength I & IV, Load Case 4	**3.32**	**-5.12**	**0.00**	**0.00**	
Strength IV, Load Case 1	**23.91**	**9.31**	**0.00**	**0.00**	**<-- Use for Strength IV**
Strength IV, Load Case 3	**23.90**	**4.72**	**0.00**	**0.00**	
Service 1 Limit State	**17.41**	**3.34**	**1.78**	**1.53**	**<-- Use for Service I**

Vertical Superstructure Reaction Moment Arm = **4.917** ft *(Bearing to toe)*

Horizontal Superstructure Reaction Moment Arm = **5.500** ft *(Top of pedestal to bottom of footing)*

FOOTING STABILITY CHECK

References

NOTE: * Resisting" Forces = all Vertical Loads. Used to determine sliding capacity.
* Overturning" Forces = all Horizontal Loads. Used for Sliding Load.
* Net Moment / Resisting Forces = Eccentricity from "Toe".
* Overturning check satisfied if eccentricity of bearing pressure is withing middle 2/3rds of footing (entire bearing area in compression) AND bearing capacity check satisfied.
* For footings on soil, the vertical stress shall be calculated assuming a **uniformly** distributed pressure over an effective base area, which equals the total bearing area minus an area to account for the effects of the eccentric load.
* Loads and factors shall be combined to produce the maximum effect for bearing, sliding and eccentricity.
* For the bearing check the max load factors are applied to vertical loads and for the slidining/eccentricity check the min load factors are applied to the vertical loads (less vertical load = lower sliding capacity and greater eccentricity). See Figures C11.5.6-1 and C11.5.6-2

AASHTO LRFD C11.5.5

Bearing:

	Unfactored (Service)		Strength I			Strength IV		
	F (kip)	M (kip-ft)	Factor	F (kip)	M (kip-ft)	Factor	F (kip)	M (kip-ft)
RESISTING								
Vertical DL Superstructure	17.41	86	---	23.2	114	---	23.9	118
Vertical LL Superstructure	1.78	9	--	3.1	15	--	0.0	0
DC	3.06	12.08	**1.25**	3.8	15.1	**1.50**	4.6	18.1
EV1 (Toe Soil)	9.8	19.6	**1.35**	13.2	26.4	**1.35**	13.2	26.4
EV2 (Heel Soil)	0.3	1.8	**1.35**	0.4	2.4	**1.35**	0.4	2.4
LS-horizontal	0	0	**1.75**	0	0	**1.75**	0	0
LS-vertical	0	0	**1.75**	0	0	**1.75**	0	0
	32.31	**127.78**		**43.77**	**173.50**		**42.09**	**164.51**
OVERTURNING								
Horiz. DL Superstructure	3.34	18.39	--	9.0	49.27	--	9.3	51.20
Horiz. LL Superstructure	1.53	8.40	--	2.7	14.69	--	0.0	0.00
EH-h	0	0	**1.50**	0	0	**1.50**	0	0
	4.87	**26.78**		**11.63**	**63.96**		**9.31**	**51.20**
	Net Moment =	**100.99**			**109.54**			**113.31**

Sliding & Eccentricity:

	Unfactored (Service)		Strength I			Strength IV		
	F (kip)	M (kip-ft)	Factor	F (kip)	M (kip-ft)	Factor	F (kip)	M (kip-ft)
RESISTING								
Vertical DL Superstructure	17.41	86	---	23.24	114	---	23.91	118
Vertical LL Superstructure	1.78	9	--	3.11	15	--	0.00	0
DC	3.06	12.08	**0.90**	2.8	10.9	**0.90**	2.8	10.9
EV1 (Toe Soil)	9.78	19.57	**1.00**	9.8	19.6	**1.00**	9.8	19.6
EV2 (Heel Soil)	0	0	**1.00**	0	0	**1.00**	0	0
LS-horizontal	0	0	**1.75**	0	0	**1.75**	0	0
LS-vertical	0	0	**1.75**	0	0	**1.75**	0	0
	32.03	**125.98**		**38.89**	**160.00**		**36.45**	**147.99**
OVERTURNING								
Horiz. DL Superstructure	3.34	18.39	--	8.96	49.27	--	9.31	51.20
Horiz. LL Superstructure	1.53	8.40	--	2.67	14.69	--	0.00	0.00
EH-h	0	0	**1.50**	0	0	**1.50**	0	0
	4.87	**26.78**		**11.63**	**63.96**		**9.31**	**51.20**
	Net Moment =	**99.20**			**96.04**			**96.79**

- Consv. Ignore for sliding

FOOTING STABILITY CHECK

References

Abutment Stability Check

Resistance Factors:

Bearing Resistance, f_b =	**0.45**		AASHTO LRFD
Sliding Resistance, f_t =	**0.90**	- "Precast concrete placed on sand"	Tbl. 10.5.5.2.2-1
Overall Stability for Service I Limit State, f_{os} =	**0.65**	- limited geotechnical info.	& 11.6.2.3

Service I Limit State Check: AASHTO LRFD

Overall Stability $\geq$ 1 11.6.2.3

= (f_{os} * Resisting Moments)/(Overturning Moments)

= 3.06 **OK**

AASHTO LRFD

Bearing Resistance:	**Strength I**	**Strength IV**	**Service I**		
Net Moment (Resist. - Overturn), M =	109.54	113.31	101.0	k-ft	11.6.3.2
Resist. Vertical Forces for Bearing, V_b =	43.77	42.09	32.3	kips (= sum of all vertical loads)	
Resultant, R = M/V_b=	2.50	2.69	3.13	ft (from "toe")	AASHTO LRFD
Eccentricity, e = (W/2) - R =	1.00	0.81	0.37	ft (from center of footing)	Fig. 11.6.3.2.1
Bearing Stress, s_v = V_b/(W-2e) =	8.74	7.82	5.17	kip/ft^2	AASHTO LRFD
Capacity/Load =	1.09	1.22	1.84	*(resistance factor included in capacity)*	11.6.3.2
	OK	**OK**	**OK**		
Overturning:					
Net Moment (Resist. - Overturn), M =	96.04	96.79	99.20	k-ft	AASHTO LRFD
Resist. Vertical Forces for Bearing, V_e =	38.89	36.45	32.03	kips (= sum of all vertical loads)	11.6.3.3
Resultant, R = M/V_e=	2.47	2.66	3.10	ft (from "toe")	
Eccentricity, e = (W/2) - R =	1.03	0.84	0.40	ft (from center of footing)	
Acceptable Eccentricity (1/3 footing) =	2.33	2.33	2.33	ft	
Is Resultant within middle two-thirds?	**OK**	**OK**	**OK**		
Sliding:					
					AASHTO LRFD
Resisting Vertical Forces for Sliding, V_s =	38.89	36.45	32.03	kips	10.6.3.4 &
Internal Friction Angle, f_f =	**37.00**	**37.00**	**37.00**		RIDOT 10.5.4
tanf_f =	0.75	0.75	0.75		
R_t = 0.8 * V_s * (tanf_f) =	23.45	21.97	19.31	kips (for precast footings)	
f_t * R_t =	21.10	19.77	17.38	kips	
Capacity/Load =	1.81	2.12	3.57		
	OK	**OK**	**OK**		

FOOTING STABILITY CHECK

References

Footing Reinforcement Check:

- Assume that controlling location is flexure about the back face of the pedestal, therefore check bottom footing bar.

f'_c =	**4.0**	ksi
f_y =	**60.0**	ksi
β_1 =	**0.85**	
Bar Size =	**#6**	
Spacing, s =	**6.0**	in
A_s/Bar =	**0.44**	in^2
Diameter of Bar, d_b =	**0.750**	in
Cover =	**3.00**	in
$A_{s,\ provided}$ =	0.88	in^2/ft
d =	t_s - Cover - (d_b/2)	
=	20.63	in

Check Flexural Strength:

$f M_n = f * A_{s,\ provided} * f_s * (d - (a/2))$

f =	**0.9**	- tension controlled section	AASHTO LRFD
d =	20.63	in	5.5.4.2.1
a =	$(A_{s,\ provided} * f_y)/(0.85 * f'_c * b)$		
b =	**12.00**	in (1' unit width)	
=	1.29	in	
f_s =	f_y, if c/d $\leq$ 0.6		
c =	a / β_1		
=	1.52		
c/d =	0.074	**OK**	
=	60.00	ksi	
=	79.11	kip-ft	

$f_f M_n$ / M_u =	1.034
$f_f M_n > M_u$?	**OK**

Moment:

- Check Moment at Back Face of Pedestal

- According to AASHTO LRFD Section 10.6.5, the structural design of eccentrically loaded footings shall be based on triangular or trapezoidal bearing stress distributions, instead of "equivalent" uniform stress distribution, as assumed above for checking soil bearing capacity.

Strength I Limit State:

Net Moment, M =	109.54	kip-ft
Resisting Vertical Forces for Bearing, V_b =	43.77	
Resultant, R =	M/V_b	
=	2.50	ft
Eccentricity, e =	(W/2) - R	
=	1.00	ft

Resultant WITHIN middle third of base: (CASE 1)

$$\sigma_{vmax} = \frac{\sum V}{B}\left(1 + 6\frac{e}{B}\right)$$

$$\sigma_{vmin} = \frac{\sum V}{B}\left(1 - 6\frac{e}{B}\right)$$

Max Bearing Stress, s_{vmax} =	11.60	kip/ft^2	**OK, CASE 1**
Min Bearing Stress, s_{vmin} =	0.91	kip/ft^2	
Bearing Stress at Back Face of Stem, s_{vbk} =	5.49	kip/ft^2	

R (rect.) ARM R (triangle) ARM

Moment at Front Face of Stem, M = $(s_{vbk} * W_{toe}) * (W_{toe} / 2) + (s_{vmax} - s_{vbk}) * (W_{toe} / 2) * (2/3 * W_{toe})$

= 76.48 kip*ft (Service I)