

# PRELIMINARY DESIGN REPORT

18-0774

November 29, 2018

## Explorations and Geotechnical Engineering Services

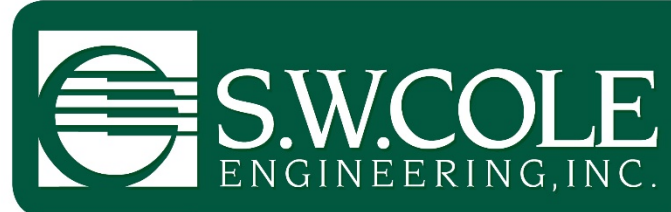
Routes 43/150 over Wesserunsett Stream  
Wesserunsett Bridge #2925 Wingwall  
Athens, Maine  
WIN 022825.00

### PREPARED FOR:

Maine Department of Transportation  
Attention: Laura Krusinski, P.E.  
State House Station 16  
Augusta, ME 04333-0016

### PREPARED BY:

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- *Geotechnical Engineering*
- *Construction Materials Testing and Special Inspections*
- *GeoEnvironmental Services*
- *Test Boring Explorations*

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Dear Laura:

In accordance with our Proposal dated June 13, 2018 and project specific Assignment Letter #20 dated July 9, 2018, we have made the requested subsurface explorations for the subject project. The purpose of our services was to obtain subsurface information in order to provide preliminary geotechnical design parameters, evaluations and recommendations for foundations and earthwork associated with wingwall rehabilitation and replacement alternatives. The services provided by S. W. Cole Engineering, Inc. (S.W.COLE) were conducted in accordance with our Multi-PIN Agreement with the Maine Department of Transportation (MaineDOT), No. 20150722000000000089, dated July 21, 2015. The contents of this report are subject to the limitations in Appendix A.

## **1.0 INTRODUCTION**

### **1.1 Site Conditions**

The site is located on Routes 43/150 at the crossing of the Wesserunsett Stream in Athens, Maine. The site location is shown on the "Site Location Map" attached in Appendix B. Based on the provided Historic Bridge Plans, we understand the existing bridge was originally constructed in 1924 (inner  $\pm 23.5$  feet) and widened  $\pm 23$  feet with

new wingwalls in 1982. We understand the existing substructure consists of concrete abutments and concrete wingwalls on the northwest, northeast and southwest corners. The southeast wingwall consists of an angled Gabion basket wall. The 1982 Historic Plans indicate the existing Gabion wall is about 40 feet long and tapers from 24.7 feet high adjacent to the abutment to 9.7 feet high at the east end. The base of the Gabion wall adjacent to the abutment is reportedly at Elevation 322 feet (project datum) and steps up along its length to El. 330.5 feet on the east end. The Gabion wall supports a  $\pm 2H:1V$  approach embankment slope that tapers in height from west to east along the length of the wall.

We understand, in 1996, 'several' grout bags and piles were used to repair undermining and spilling stones in the western corner of the Gabion wall. We understand the 'ledge' surface slopes down into the stream at the toe of wall. Additionally, an Underwater Dive Inspection Field Report, dated May 5, 2005 and revised June 17, 2014 indicates a 7-foot deep, isolated scour hole on the west corner adjacent to the east abutment. The Historic Plans and Inspection Reports are attached in Appendix C.

## **1.2 Proposed Construction**

Based on information provided by MaineDOT, we understand the existing Gabion wingwall will be either rehabilitated or replaced. We understand the following rehabilitation or replacement options are under consideration:

- Install driven sheet piles in front of existing wall with tie-back deadmen anchors and fill void between new and existing walls with Crushed Stone;
- Buttress western end of Gabion wall by placing heavy riprap slope protection to support failing gabion and improve hydraulics of stream;
- Buttress western end of Gabion wall with heavy riprap and reface Gabion wall with drainage geotextile connected to weep drains and shotcrete;
- Construct rock-socketed soldier pile retaining wall in-front of existing wall;
- Place cofferdam and construct cast-in-place (CIP) concrete wall with tie-back anchors behind Gabion wall; and
- Construct prefabricated wall system (metal bin-wall or precast segmental wall).

We understand alternating one-way traffic will be maintained at all times during construction.

## **2.0 EXPLORATIONS AND TESTING**

### **2.1 Explorations**

Four test borings (BB-AWS-101, BB-AWS-101A, BB-AWS-102 and BB-AHMB-103) were made at the site on July 30 and August 1, 2018 by S. W. Cole Explorations, LLC. The as-drilled exploration locations were selected and established in the field by S.W.COLE using taped measurements from existing site features. The ground surface elevations of the test borings were provided by MaineDOT. The approximate exploration locations are shown on the “Boring Location Plan” attached in Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix D.

### **2.2 Testing**

The test borings were drilled using a combination of solid-stem auger, cased wash boring and rock core drilling techniques. The soils were sampled at 2 to 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) techniques using a calibrated automatic hammer. Upon encountering refusal on bedrock, borings BB-AWS-101 through BB-AHMB-103 and BB-AWS-101A were advanced about 4 to 10 feet into bedrock using a NQ2 rock coring. The uncorrected SPT blow counts, uncorrected and corrected SPT N-values and rock core intervals are shown on the logs.

Soils samples recovered from the test borings were visually classified in our laboratory and transported to the MaineDOT Laboratory in Bangor, Maine for testing to assist soil classification and identification. Laboratory testing was performed on disturbed SPT samples obtained during the explorations. Laboratory testing was performed by the MaineDOT Materials Testing and Exploration Central Laboratory in Bangor, Maine in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) testing procedures. Laboratory testing included four natural water content tests and four grain size analyses (without hydrometer). Laboratory test results are shown on the boring logs in Appendix D and are provided in Appendix E.

## **3.0 SUBSURFACE CONDITIONS**

### **3.1 Surficial and Bedrock Geology**

Based on available geologic mapping (Maine Geologic Survey, Surficial Geology of the Skowhegan Quadrangle, Open-File 86-38) the soils in the project vicinity are mapped

as Presumpscot Formation (silt and clay) and glacial till. Bedrock in the site vicinity is mapped as Sangerville Formation, Patch Mountain Member (Bedrock Geology Map of Maine, 1985). The Sangerville Formation, Patch Mountain Member is generally composed of thinly interbedded, gray micritic metalimestone, limy metasandstone, and metasiltstone.

The observed subsurface conditions are generally consistent with the mapped surficial and bedrock geology; however, the explorations also encountered a surface deposit of fill soils from previous site development.

### **3.2 Subsurface Conditions**

The test borings encountered a soils profile generally consisting of a surface layer of pavement overlying fill overlying glacial till overlying bedrock. The principal strata encountered in the explorations are summarized below; refer to the attached logs for more detailed subsurface information at the exploration locations.

Fill: Below an approximate 7.5 to 8 inch layer of pavement, the borings encountered fill extending to depths of about 11.1 to 22.0 feet below ground surface (bgs), corresponding to Elevation (El.) 334.8 to 323.8 feet. The fill generally consisted of:

- Brown, SAND, some to little gravel, little silt;
- Brown, Gravelly SAND, little silt; and
- Brown, SAND, some silt, little gravel, with brick fragments.

Cobbles and an occasional boulder were noted within the fill. The fill was generally loose to very dense with SPT  $N_{60}$  values ranging from 6 blows per foot (bpf) to 100 blows for 4 inches (sampler refusal).

Glacial Till: Below the fill, the borings generally encountered 0.9 to 2 feet of glacial till extending to depths of about 12.1 to 24 feet bgs, corresponding to Elevation (El.) 333.8 to 321.8 feet. The glacial till generally consisted of:

- Brown, Silty SAND, little gravel, trace clay; and
- Brown, SAND, some gravel, little silt.

The glacial till was generally medium dense to very dense with SPT  $N_{60}$  values ranging from 13 bpf to 100 blows for 5 inches (sampler refusal).

**Bedrock:** Bedrock was encountered and sampled in each boring. The top of bedrock generally sloped down from east to west from about 12.1 to 24 feet bgs. The bedrock consisted of grey, moderately hard, fresh, meta-siltstone. Joints were generally vertical and along bedding planes to low angle along stress fractures, tight and very close to close.

RQD values for the bedrock generally ranged from 0 to 96 percent correlating to a Rock Mass Quality (RMQ) of very poor to excellent. The following table summarizes the approximate depths to bedrock, corresponding top of bedrock elevations and Rock Quality Designation (RQD) where encountered.

Boring Number (Substructure)	Approximate Depth to Bedrock (feet)	Approximate Bedrock Elevation (feet)	RQD (RMQ)
BB-AWS-101	24.0	321.8	0 to 42% (Very Poor to Poor)
BB-AWS-101A	23.1	322.7	0 to 75% (Very Poor to Fair)
BB-AWS-102	12.1	333.8	0 to 96% (Very Poor to Excellent)
BB-AWS-103	19.1	326.3	0 to 74% (Very Poor to Fair)

### **3.3 Groundwater Conditions**

The soils encountered at the test borings were damp to moist from the ground surface. The measured water levels following drilling ranged from 10 to 20.6 feet below ground surface. It should be noted that water was introduced during drilling; therefore, water levels observed may not represent stabilized ground water conditions. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate seasonally, particularly in response to periods of snowmelt and precipitation, as well as changes in site use and the water level of Wesserunsett Stream.

### **4.0 PRELIMINARY GEOTECHNICAL EVALUATION**

S.W.COLE conducted preliminary geotechnical engineering evaluations in accordance with 2017 AASHTO LRFD Bridge Design Specifications, 8<sup>th</sup> Edition (AASHTO LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through August 2014 (MaineDOT BDG) and offers the following:

#### **4.1 General Considerations**

The site is underlain by fill and glacial till overlying bedrock at about El. 321.8 to 333.8 feet. The Historic Gabion wall profile indicates the bottom of wall steps from El. 322 feet on the west end to El. 330.5 feet on the east end. The bedrock profile appears to slope upwards from west to east and appears to generally correspond with the toe of the existing wall.

We understand the original structure was constructed in 1924 and widened in 1982. We understand the existing structure is generally in fair to satisfactory condition with a Federal Sufficiency Rating of 71.6. Based on the remaining design-life of the existing structure, we understand that rehabilitation and countermeasures are preferred over replacement.

#### **4.2 Wingwall Alternatives**

We understand proposed rehabilitation and replacement structure alternatives include:

- Install driven sheet piles in front of existing wall with tie-back anchors and fill void between new and existing walls with Crushed Stone;
- Buttress western end of Gabion wall by placing heavy riprap slope protection to support failing gabion and improve hydraulics of stream;
- Buttress western end of Gabion wall with heavy riprap and reface Gabion wall with drainage geotextile connected to weep drains and shotcrete;
- Construct rock-socketed soldier pile retaining wall in-front of existing wall;
- Place cofferdam and construct a CIP concrete wall with tie-back anchors behind Gabion wall; and
- Construct prefabricated wall system (metal bin-wall or precast segmental wall).

The following sections summarize preliminary design considerations to assess the rehabilitation and replacement alternatives. We understand the preferred rehabilitation concept consist of constructing a riprap buttress on the western end of Gabion wall and re-facing with shotcrete underlain by a drainage geotextile that is connected to weep drains.

#### **4.2.1 Sheet Pile Wall**

The sheet pile wall option would consist of installing driven sheet piles in front of the existing Gabion wall and installing tie-back anchors to resist lateral loads. The void between the two walls would be backfilled with Crushed Stone (MaineDOT Standard Specification 703.22 “Underdrain Backfill Material Type C”).

Based on the presence of shallow bedrock and limited overburden soils at the site, we anticipate the sheet piles will be driven to refusal on bedrock. Additionally, due to the limited overburden soils at the toe of wall, the sheet piles will not have enough embedment to develop fixity. Therefore, we anticipate at least two rows of tie-back anchors to resist lateral loads. We anticipate tie-back anchors will be located near the bottom third and top third of the wall. Due to the anticipated excavation required to install two rows of deadman anchors, we anticipate construction of the tie-back anchors will require horizontal drilling through the existing Gabion wall and into the roadway embankment. We anticipate the tie-back anchors will consist of grouted, solid or stranded anchors.

This option would require a single-lane closure during construction, the mobilization of specialty horizontal equipment and construction of equipment access to drill and install the tie-back anchors.

#### **4.2.2 Buttress Fill and Improved Hydraulics**

This option would consist of constructing a riprap buttress fill for slope protection on the western end of the Gabion wall. Placement of the riprap will encroach the flow of Wesserunsett Stream; therefore, this option will include in-water excavations to improve hydraulics of the stream.

The riprap buttress fill would be constructed by placing a nonwoven erosion control geotextile against the Gabions followed by Plain and Hand Laid Riprap and capped with Heavy Riprap. Riprap fill shall conform to MaineDOT Standard Specification 703.28 “Heavy Riprap” and Standard Specification 703.26 “Plain and Hand Laid Riprap” and shall be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

The toe of the new riprap slopes should be keyed into the existing soils at least 2 feet. In accordance with MaineDOT BDG Section 2.3.11.3, the top of the riprap shall be located at or above, the  $Q_{50}$  elevation.

This option would require the least impact to the existing structure and roadway as well as future bridge replacement options.

#### **4.2.3 Buttress Fill and Reface Wall**

This option would consist of placing a riprap buttress fill for scour protection on the western end of the Gabion wall where undermining has been observed. Similar to the previous buttress option, riprap buttress fill would be constructed by placing a nonwoven erosion control geotextile against the Gabions followed by Plain and Hand Laid Riprap and capped with Heavy Riprap. Riprap fill shall conform to MaineDOT Standard Specification 703.28 "Heavy Riprap" and Standard Specification 703.26 "Plain and Hand Laid Riprap" and shall be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

The toe of the new riprap slopes should be keyed into the existing soils at least 2 feet. In accordance with MaineDOT BDG Section 2.3.11.3, the top of the riprap shall be located at or above, the  $Q_{50}$  elevation.

Additional scour countermeasures would include re-facing the Gabion wall, placement of new curbing at the roadway and constructing a new riprap lined drainage swale. The wall will be re-faced with shotcrete underlain by a Class 2 drainage geotextile per MaineDOT Standard Specification 722.02. The drainage geotextile will drain through the shotcrete fascia using weep drains. We anticipate the new fascia will require mechanical connection to the existing Gabion wall.

This option would require minimal impacts to the existing structure and roadway as well as future bridge replacement options.

#### **4.2.4 Soldier Pile Wall**

The soldier pile retaining wall option would consist of rock-socketed H-piles installed in-front of the existing Gabion wall and placing either wood or concrete lagging between the piles to retain the existing embankment fills. In addition, tie-back anchors may be

needed to resist lateral loads. The need for tie-back anchors would be dependent on the rock-socket length, selected H-pile section and lateral loads. We anticipate tie-back anchors may consist of deadman anchors.

To prevent undermining, a Class 1 nonwoven erosion control geotextile (MaineDOT Standard Specification 722.03) would be placed behind the lagging and wrapped up the failing portion of the Gabion wall. The void between the two walls would be backfilled with Crushed Stone (MaineDOT Standard Specification 703.22 "Underdrain Backfill Material Type C").

This option would require the use of specialty drilling equipment and construction of an access roadway for drilling equipment either behind or in front of the existing Gabion wall. If the construction access is constructed behind the existing wall, the Gabion wall will need to be evaluated to support the anticipate construction loads

#### **4.2.5 Cast-In-Place Concrete Wall**

The CIP concrete wall option would consist of constructing a CIP concrete wall with tie-back anchors behind the existing Gabion wall. Based on conversations with sheet pile contractors, due to the limited overburden soils, cofferdam construction may need to consist of a double row of sheet piles. The void between the two sheet pile walls would be filled with concrete in order to create a seal.

Based on the scour issues at the site, the retaining wall will need to be cast directly on the bedrock surface. Based on MaineDOT BDG Section 5.2.2, anchorage of the footing to a concrete seal, if used, is required. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. Anchorage of concrete seals to bedrock may also be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Additionally, construction will require over-excavations to remove the existing Gabion wall, temporary shoring to reduce excavations into the existing embankment, possible cuts to bench new embankment fills, placement and compaction of new embankment fills, and provide surface slope erosion control. Depending on the footing size, we

anticipate tie-back anchors that may be needed to resist lateral loads and improve stability would consist of deadman anchors.

#### **4.2.6 Prefabricated Wall System**

The prefabricated wall option would consist of constructing a prefabricated wall system such as a metal bin-wall, T-wall or mechanically-stabilized earth (MSE) wall behind the existing Gabion wall. Similar to the CIP concrete wall option, cofferdam construction consisting of a double row of sheet piles filled with concrete would be needed to construct in the dry.

Based on the scour issues and variable bedrock surface, we anticipate a concrete leveling mat would be needed. Based on MaineDOT BDG Section 5.2.2, anchorage of the footing to a concrete seal, if used, is required. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. Anchorage of concrete seals to bedrock may also be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

Additionally, construction will require over-excavations to remove the existing Gabion wall, temporary shoring to reduce excavations into the existing embankment, possible cuts to bench new embankment fills, placement and compaction of new embankment fills, and provide surface slope erosion control. We anticipate geogrid reinforcing would be needed to resist lateral loads and improve internal stability.

#### **4.3 Construction Considerations**

Earth support systems, such as sheet piles, will be required if laying back construction slopes in order to maintain a single lane of alternating traffic is not feasible. Regardless of excavation methods, excavations and earth support systems must be properly shored or sloped in accordance with OSHA regulations to prevent sloughing and caving of slopes during construction. The design and planning of excavations, excavation support systems, and dewatering is the responsibility of the Contractor.

The Contractor shall control surface water infiltration using temporary ditches, sumps, granular drainage blankets, stone ditch protection or hand-laid riprap with geotextile underlayment to divert groundwater and surface water as necessary.

Construction activities will likely include construction of cofferdams and earth support systems to support the approach fills and control stream flow during construction of concrete seals and spread footings for the CIP wall and prefabricated wall options. Construction activities will also include common earth and rock excavation and structural earth and rock excavation for the foundation construction and possible stream hydraulic improvements

Backfills shall be placed in horizontal lifts and compacted such that the desired density is achieved throughout the lift thickness with 2 to 5 passes of the compaction equipment. Loose lift thicknesses for grading, fill and backfill activities should be limited to 12 inches. Small, hand operated or walk-behind compaction equipment shall be used within 3 feet of the existing abutment wall and wing walls to avoid over-compaction of material adjacent to the existing walls. We recommend fill and backfill be compacted to at least 95 percent of its maximum dry density as determined by AASHTO T-180.

#### **4.4 Bedrock Removal and Bedrock Subgrade Preparation**

The nature, slope and degree of fracturing in the bedrock bearing surfaces will not be evident until the foundation excavations are made. The bedrock surface shall be cleared of all loose fractured bedrock, loose decomposed bedrock and soil to expose sound, intact bedrock. The final bearing surface shall be solid. If the bedrock surface is observed to slope steeper than 4H:1V at the subgrade elevation in any direction, the bedrock shall be benched to create level steps or excavated to be completely level. Excavation of highly sloped and loose fractured bedrock material shall be made using all conventional excavation methods (digging bucket, ripper tooth, hoe-ramming) possible in attempt to create level steps or be completely level. Based on the proximity to existing foundations and structures, we recommend bedrock excavation by blasting be avoided. Anchors or dowels may also be designed and employed to improve sliding resistance where the prepared bedrock surface is steeper than 4H:1V in any direction. The bottom of footing or concrete seal elevation may vary based on the presence of fractured bedrock and the variability of the bedrock surface.

#### **4.5 Earth Pressure and Surcharge**

##### **4.5.1 Earth Pressure**

The selected wingwall option shall be designed for active earth pressure over the wall height unless restrained from movement. Walls restrained from movement should be

designed for at-rest active earth pressure over the wall height. For design of gravity and semi-gravity walls backfilled with granular soil and drained (e.g. no hydrostatic pressures), we recommend the following earth pressure coefficients:

- Active Earth Pressure Coefficient,  $k_a = 0.28$  (for horizontal backslope)  
 $k_a = 0.45$  (for 2H:1V backslope)
- At-rest Earth Pressure Coefficient,  $k_o = 0.47$

The resultant earth pressure is orientated at an angle  $\delta$  of 21.3 degrees from a perpendicular line to the wall back-face, where  $\delta$  is the angle of friction between the abutment backfill soil and the wall back-face.

Based on MaineDOT BDG Section 3.6.1, the designer may assume Soil Type 4 for the backfill material with the following soil properties:

- Internal Friction Angle,  $\phi = 32$  degrees
- Total Unit Weight,  $\gamma = 125$  pcf

#### **4.5.2 Surcharge Pressure**

Lateral earth pressure due to construction surcharge or live load surcharge is required per MaineDOT BDG Section 3.6.8 for the wingwall. The live load surcharge on the wingwall may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil ( $h_{eq}$ ) of 2.0 feet, per LRFD Table 3.11.6.4-2. .

Wingwall modifications and design shall include a drainage system to ensure that drainage of water behind the structure is maintained. Drainage behind the structures shall be in accordance with MaineDOT BDG Section 5.4.1.4 Drainage.

#### **4.6 Frost**

It is anticipated that the wingwall footings will be founded directly on bedrock or mud slab on bedrock. For foundations on bedrock, heave due to frost is not a design concern; therefore, requirements for minimum depth of embedment are not necessary.

Although not anticipated, foundations on soil will need to be founded below frost depth. Based on the Maine Design Freezing Index Map<sup>1</sup>, the design freezing index for the Athens, Maine area is approximately 1,250 freezing degree-days. Based on Section 5.2.1 of the MaineDOT BDG and subsurface soils encountered, the maximum seasonal frost penetration is estimated to be on the order of about 7.6 feet. Considering this, we recommend foundations on soil should have at least 7.6 feet of soil cover to provide frost protection.

#### **4.7 Seismic Design**

Seismic site class was evaluated in accordance with LRFD Section 3.10.3.1 Method B (average N-value method). An N-value of 100 bpf was assumed for the profile below the refusal surface. Based on the subsurface information, the average N-value fell between 50 and 100 bpf corresponding to a Site Class C as defined in LRFD Table 3.10.3.1-1.

The USGS online Seismic Design Maps Tool was used to obtain the seismic design parameters for the site. Based on the assigned site class (Site Class C) and site coordinates, the software provides the recommended LRFD Response Spectrum for a 7% probability of exceedance in 75 years. The results for the project site are summarized below:

<b>Recommended Seismic Design Parameters<sup>2</sup></b>	
Site Class	C
PGA	0.074 g
S <sub>s</sub>	0.160 g
S <sub>1</sub>	0.048 g
F <sub>pga</sub>	1.2
F <sub>a</sub>	1.2
F <sub>v</sub>	1.7
A <sub>s</sub>	0.089 g
S <sub>DS</sub>	0.192 g
S <sub>D1</sub>	0.081 g
Seismic Zone (LRFD Table 3.10.6-1)	Zone 1

NOTE: Site Coordinates: N44.92316, W69.67457

<sup>1</sup> Maine Department of Transportation, Bridge Design Guide (BDG), August 2003, with Revisions through 2018, Figure 5-1.

<sup>2</sup> U.S. Geological Survey, Seismic Design Map, accessed September 12, 2018  
<https://earthquake.usgs.gov/designmaps/us/application.php>

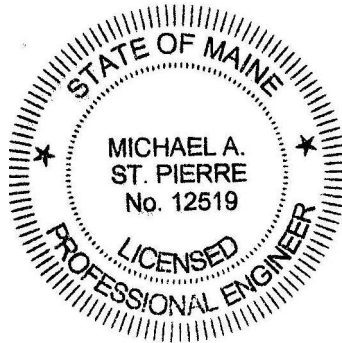
## 5.0 CLOSURE

We trust this information meets your present needs. Please contact us if you have any questions or need further assistance.

Sincerely,

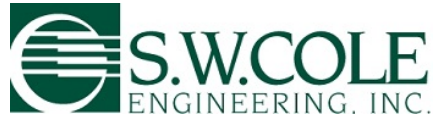
**S. W. Cole Engineering, Inc.**

Michael A. St. Pierre, P.E.  
Senior Geotechnical Engineer

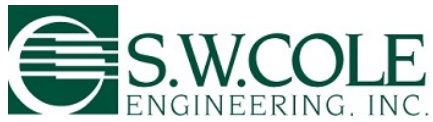


Timothy J. Boyce, P.E.  
Senior Geotechnical Engineer

MAS/tjb-rec



**APPENDIX A**  
**Limitations**



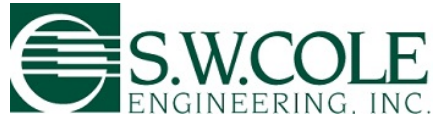
This report has been prepared for the exclusive use of the Maine Department of Transportation for specific application to the Wesserunsett Bridge #2925 Wingwall Rehabilitation or Replacement Project (MaineDOT WIN 022825.00) on Routes 43/150 over Wesserunsett Stream in Athens, Maine. S. W. Cole Engineering, Inc. (S.W.COLE) has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

The soil profiles described in the report are intended to convey general trends in subsurface conditions. The boundaries between strata are approximate and are based upon interpretation of exploration data and samples.

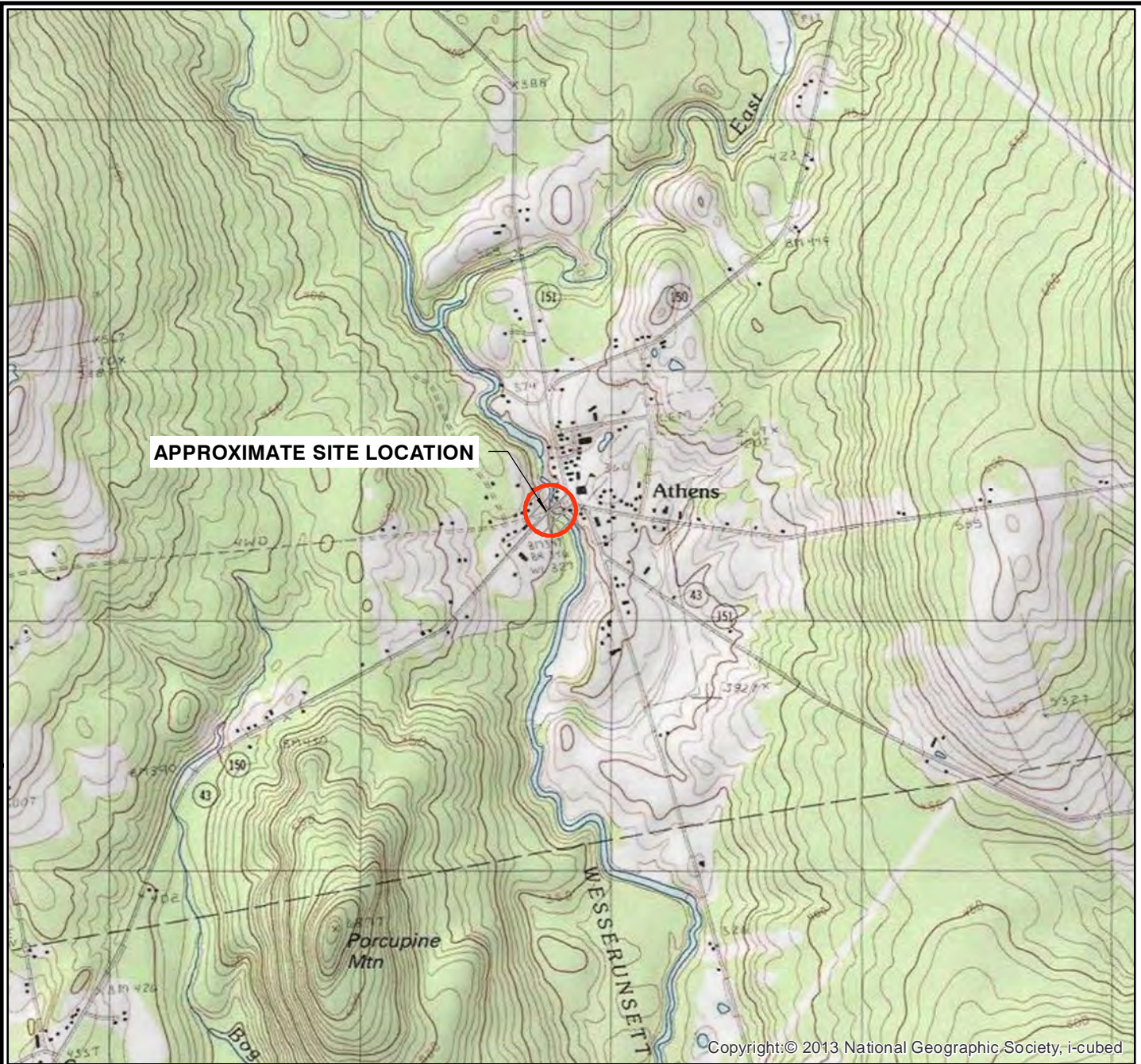
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and to review the recommendations of this report.

Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S.W.COLE should review such changes as they relate to analyses associated with this report. Recommendations contained in this report shall not be considered valid unless the changes are reviewed by S.W.COLE



**APPENDIX B**  
**Figures**



2,000 0 2,000 4,000



Scale in Feet



MAINE DEPARTMENT OF TRANSPORTATION

**SITE LOCATION MAP**

WESSERUNSETT BRIDGE #2925 WINGWALL  
 ROUTES 43/150 OVER WESSERUNSETT STREAM  
 ATHENS, MAINE  
 WIN 022825.00

**NOTE:**  
 SITE LOCATION MAP PREPARED FROM  
 ESRI ArcGIS ONLINE AND DATA PARTNERS  
 INCLUDING USGS AND © 2007 NATIONAL  
 GEOGRAPHIC SOCIETY.

Job No.	18-0774	Scale	1:24000
Date:	09/13/2018	Sheet	1





**APPENDIX C**  
**Historic Plans and Reports**

ATHENS 241 11

Soils Report 81-10  
Athens - Somerset County  
Wesserunsett Stream Bridge  
Project 241(11)  
April 1981

81 10

RE. Bob Zimmerman

Maine Department of Transportation

Materials & Research Division

Soils Section

Athens off 654-2635  
Detroit off 368-5633

---

SUBSURFACE INVESTIGATION FOR THE PROPOSED

WIDENING OF WESSERUNSETT STREAM

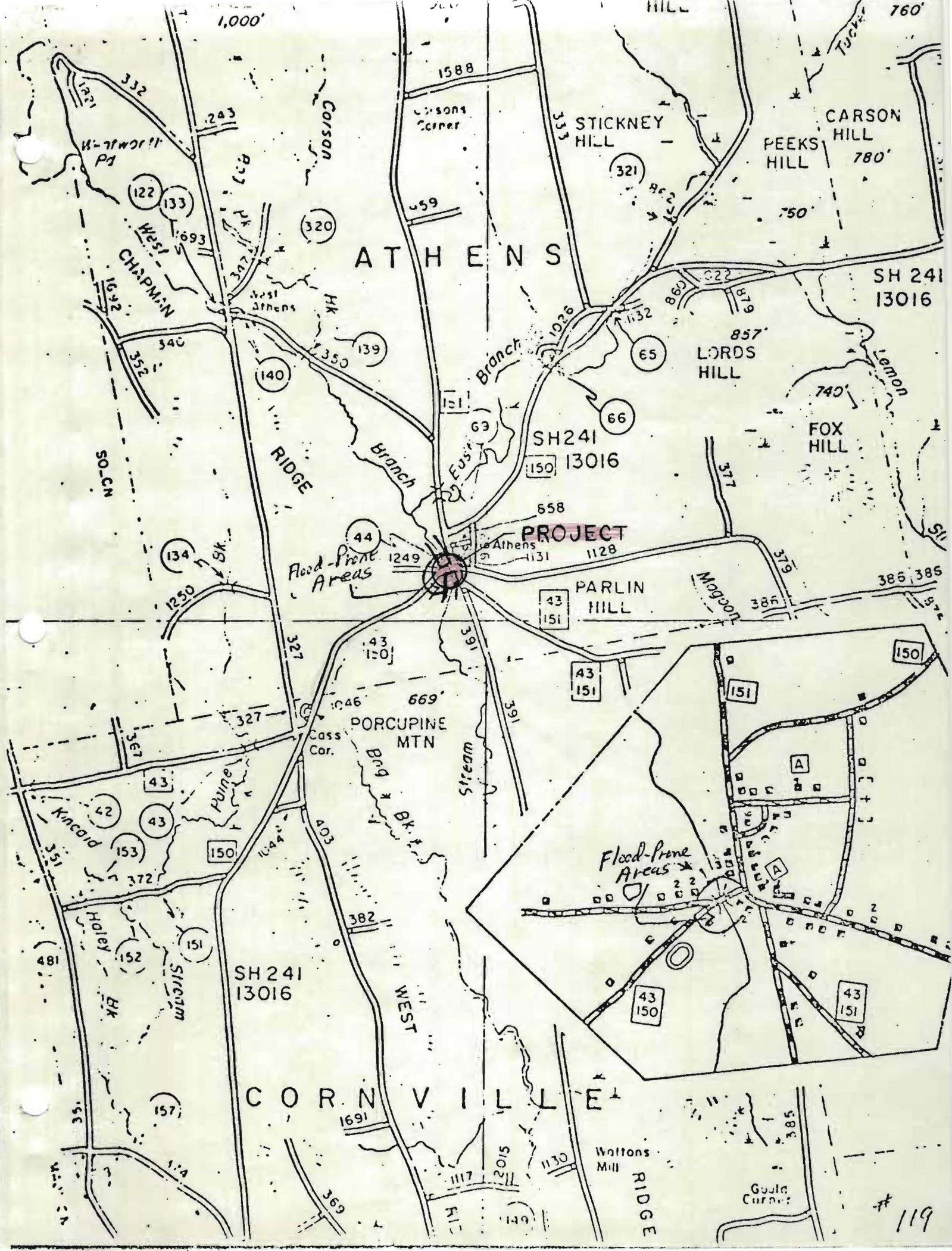
BRIDGE IN THE TOWN OF ATHENS

Somerset County

Project 241(11)

Soils Report 81-10

April 1981



# 119

TABLE OF CONTENTS

<u>TEXT</u>	<u>PAGE</u>
Introduction.....	1
General Conditions.....	1
Detailed Conditions	
West Abutment.....	1
East Abutment.....	2
Summary.....	2

<u>ILLUSTRATIONS</u>	<u>SHEET</u>
Foundation Survey Sheet.....	1

## INTRODUCTION

An investigation to determine soils conditions has been completed for Wesserunsett Stream Bridge in the Town of Athens. It is proposed to widen the existing structure and add a sidewalk. New wingwalls will also be required. The investigation consisted of an on-site inspection and a single hand sounding. The location of the sounding is shown on a plan on the sheet following the text of this report. Also shown on this sheet are two transverse sections. The original of Sheet 1 is being provided to the Bridge Design Section for inclusion in the construction plans.

## GENERAL CONDITIONS

The proposed project is located in the Town of Athens, Somerset County. The existing bridge is a single-span, narrow concrete structure and is in good condition. The existing abutments are believed to be supported directly on ledge. It is proposed to widen the structure approximately ten feet upstream and downstream for a total additional width of approximately twenty feet. A sidewalk is proposed for the upstream extension. New wingwalls will also be required. Exposed ledge is visible upstream and downstream from the existing bridge. It is believed that ledge is at or near the ground surface within the vicinity of the bridge site.

## DETAILED CONDITIONS

### West Abutment:

The centerline of bearing for this abutment intersects the survey centerline (existing bridge centerline) at Station 229+96+. Exposed ledge is visible at the right end of the abutment in the vicinity of the proposed downstream

extension. A ledge outcrop is also present beside the existing upstream granite wingwall between Stations 229+76<sub>+</sub> and 229+90<sub>+</sub>. No ledge is visible at the left end of the abutment but the riverbed is rocky and it is believed that the ledge surface is near the ground surface. A transverse section near the abutment at Station 229+90 is shown on Sheet 1. It is recommended to support the proposed west abutment extensions directly on ledge. The new wingwalls could be supported on the soil overlying the ledge surface, but to get adequate soil cover for frost and scour protection it seems highly probable that these wingwalls will also be built on ledge.

#### East Abutment

The centerline of bearing for this abutment intersects the survey centerline at Station 230+39<sub>+</sub>. Exposed ledge is visible at the left end of the abutment in the vicinity of the upstream extension. A hand sounding was made near the right end of the east abutment at Station 230+51, nineteen feet right, at a ground surface elevation of 327.70. Refusal was encountered at Elevation 322.70 and is believed to be on ledge. A transverse section drawn for the east abutment at Station 230+40 is shown on Sheet 1. It is recommended to support the east abutment extensions directly on ledge. Although it would be acceptable to support the wingwall on the soil overburden above ledge, it seems likely that the need for adequate soil cover to protect against frost problems and scour will dictate founding the wingwalls on ledge.

#### SUMMARY

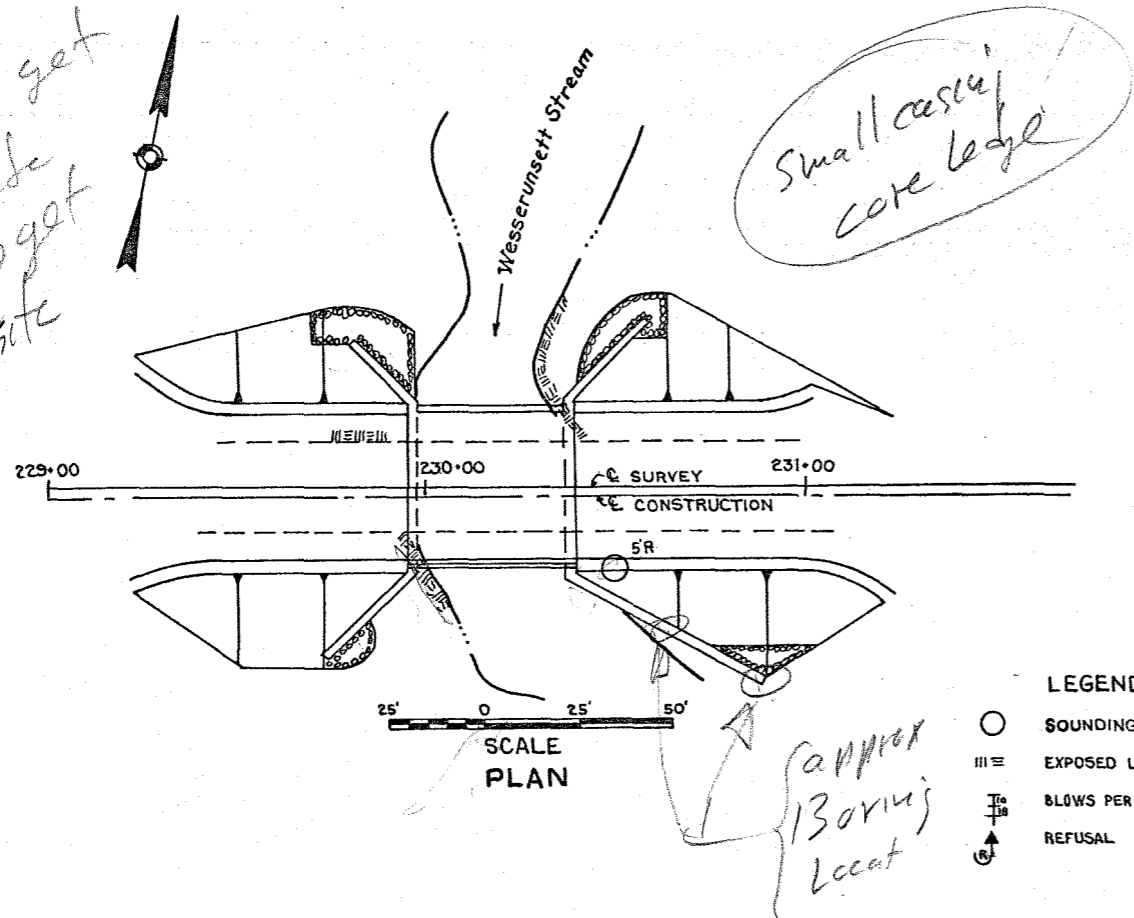
A site investigation, consisting of an on-site inspection and a single hand sounding, has been completed for the proposed project. Shown on Sheet 1 are a plan and profile of the bridge project as well as two transverse sections. Ledge

is exposed at the downstream end of the west abutment and at the upstream end of the east abutment extension locations and it is believed that ledge is near the ground surface at the remaining two abutment extension locations. It is recommended to support the abutment extensions directly on ledge. Wingwalls could be supported on the soil overburden above the ledge if there is sufficient depth of soil to provide adequate scour protection and frost protection.

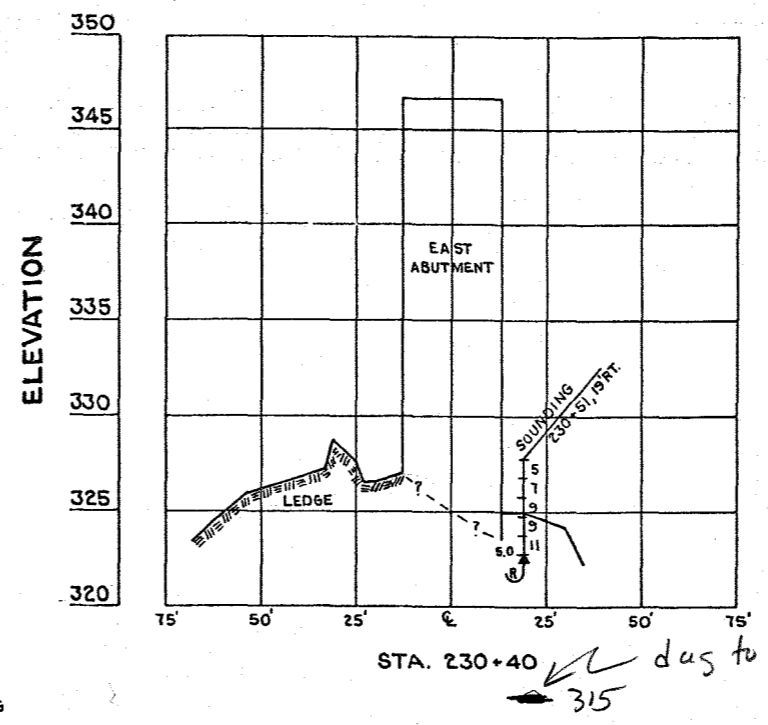
Prepared by David B. Miller  
David B. Miller  
Assistant Engineer - Soils

Approved by Guy L. Baker  
Guy L. Baker  
Assistant Soils Engineer

RE will get contractor to help get on site

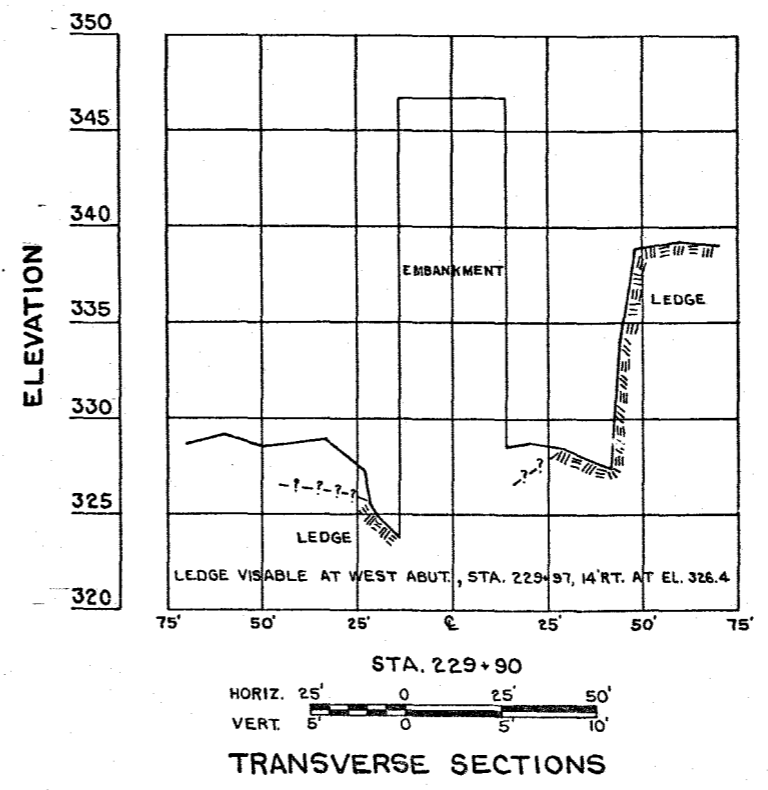
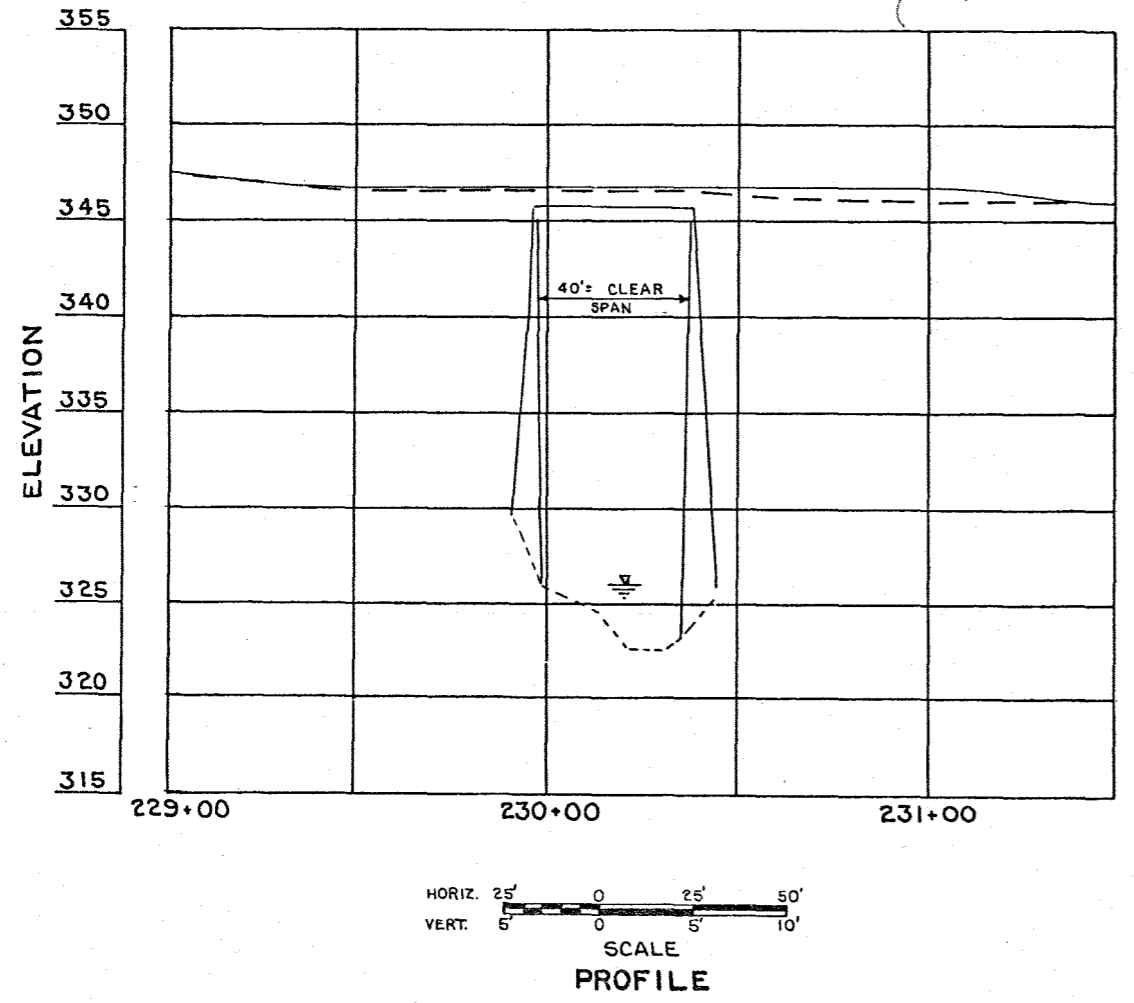


- LEGEND
- SOUNDING
  - ≡ EXPOSED LEDGE
  - ⊥ BLOWS PER FOOT-ROD SOUNDING
  - ↑ REFUSAL



el 315 No ledge due to

Cont. by Simpson



PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED		
CHECKED		
REVISIONS		
FIELD CHANGES		

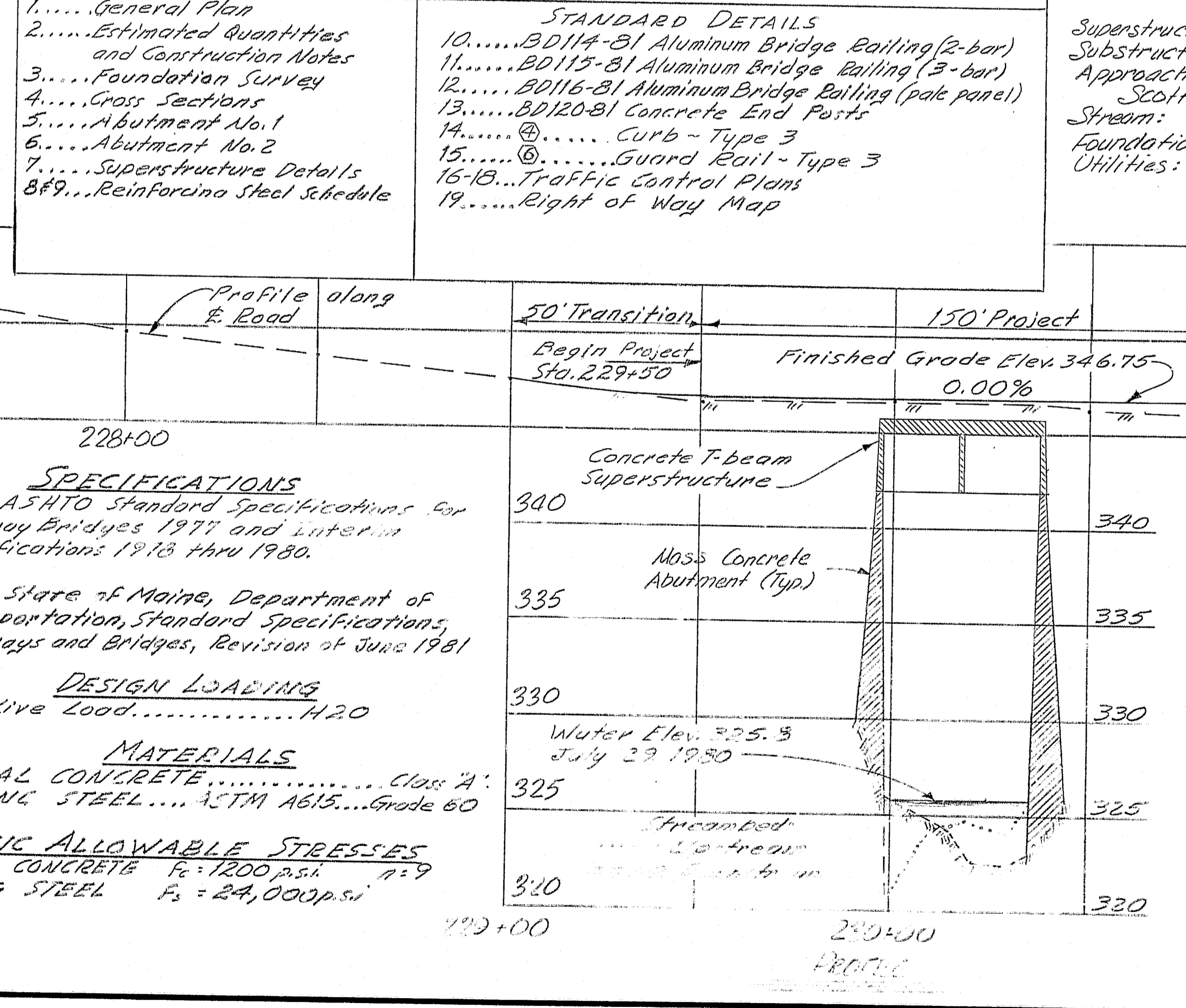
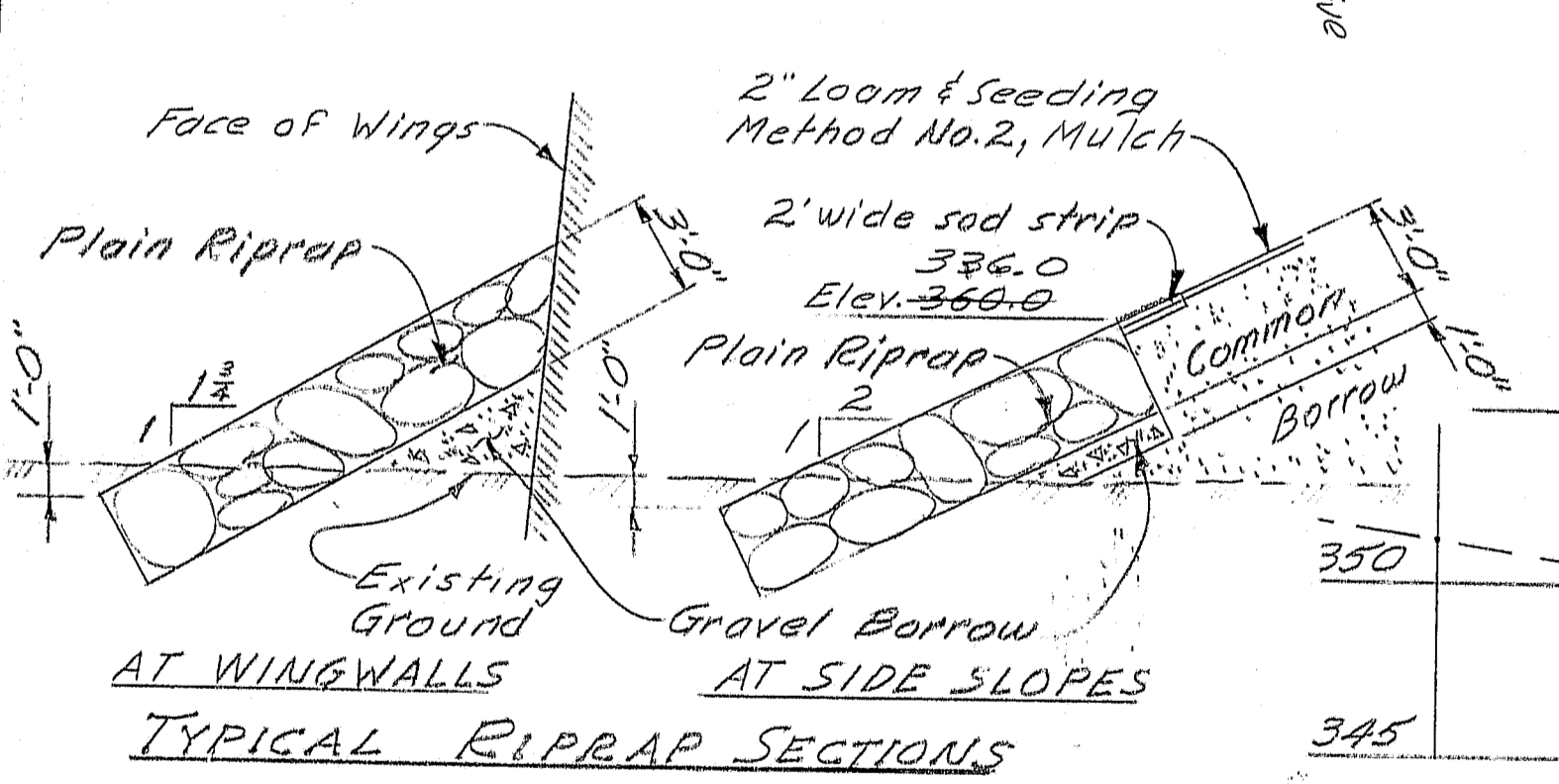
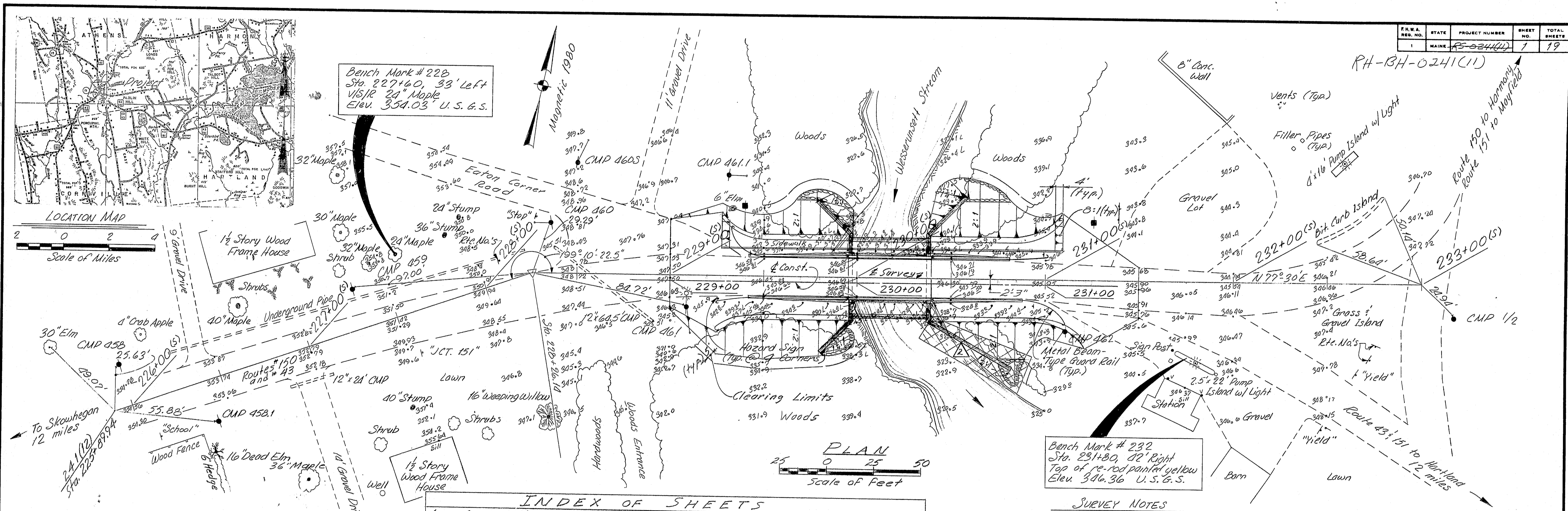
PLANS

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

**WESSERUNSETT  
STREAM BRIDGE**  
IN THE TOWN OF  
**ATHENS**  
SOMERSET COUNTY  
FOUNDATION SURVEY

SHEET OF AUGUSTA, MAINE

F.R.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	FE-0241(1)	7	19



**Br. #2925**

APPROVED: \_\_\_\_\_ DATE: 12/1/81

STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION  
 George N. Campbell, COMMISSIONER  
 Richard A. Coleman, CHIEF ENGINEER

References:  
 Bridge No. 2925  
 Field Notebook No. 8848

**NOTE:**  
 All work contemplated under this contract shall be governed by and in conformity with the Standard Specifications (Revision of June 1981) and supplements thereto except as modified on the plans and in the Special Provisions.  
 Plans of the existing bridge and a hydrologic report of the bridge site are available for the Contractor's reference at the Bridge Design Office in Augusta. The plans are reproductions of original drawings as prepared for the construction of the bridge and it is very unlikely that the plans will show any construction field changes or any alterations which have occurred since the original drawings were prepared. The Contractor shall be responsible for the interpretation of the drawings and for the accuracy of the information or conclusions of the Report which will be represented on the drawings at the time of construction.

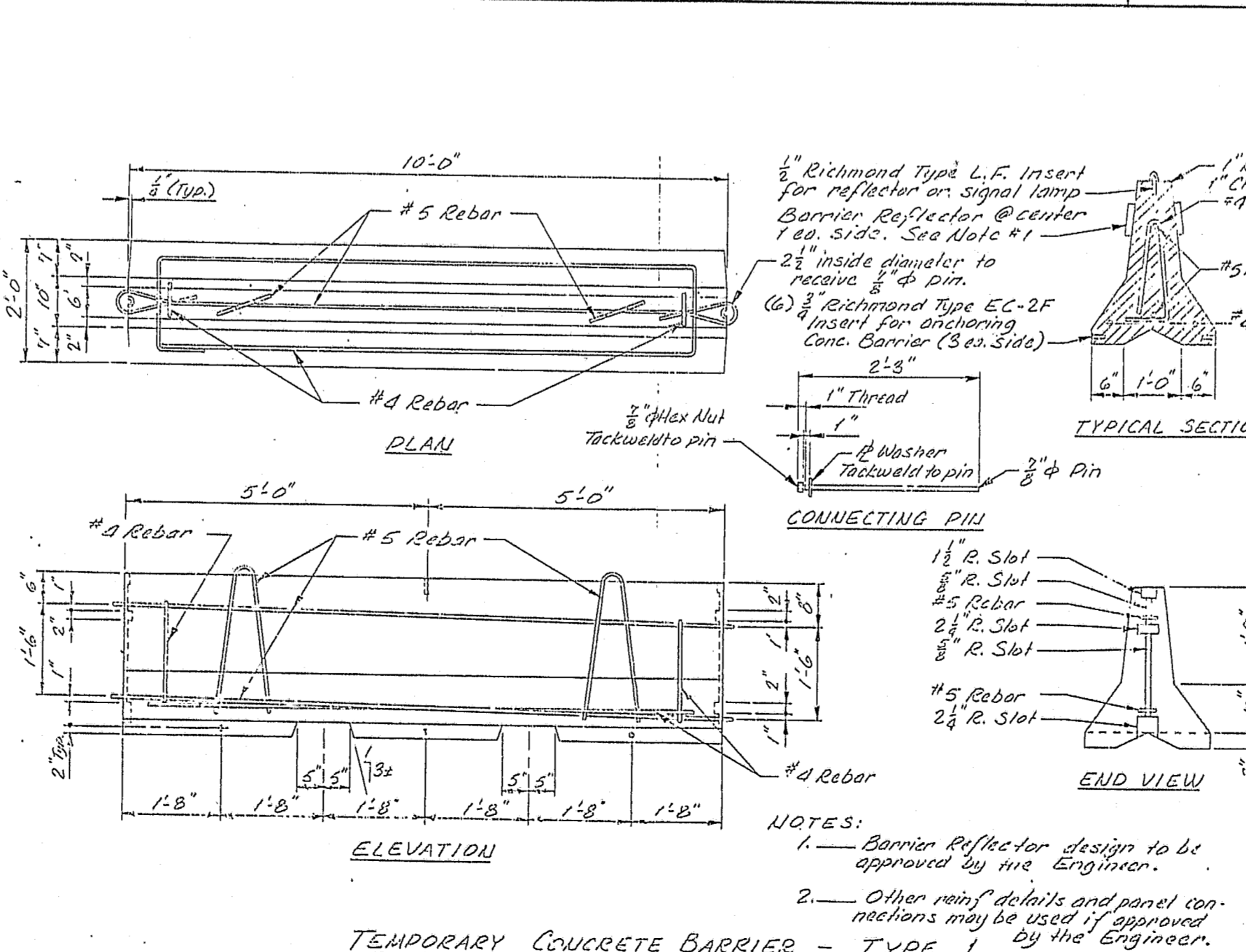
**STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION  
 WESSERUNSETT  
 STREAM BRIDGE  
 IN THE TOWN OF  
 ATHENS  
 SOMERSET COUNTY  
 GENERAL PLAN**

SHEET 7 OF 19 AUGUSTA, MAINE Jan. 1982

Survey Plotted: DMD 8-2  
 Survey Checked: \_\_\_\_\_  
 PROJECT ENGINEER: \_\_\_\_\_  
 DESIGN - DETAILED: \_\_\_\_\_  
 CHECKED: \_\_\_\_\_  
 REVISIONS: \_\_\_\_\_  
 FIELD CHANGES: \_\_\_\_\_

ESTIMATED QUANTITIES			
ITEM No.	DESCRIPTION	QUANTITY	UNIT
202.17	Removal of Existing Str. Concrete	1	Lump Sum
203.20	Common Excavation	315	Cu. Yds.
203.24	Common Borrow	800	Cu. Yds.
203.25	Granular Borrow	115	Cu. Yds.
203.26	Gravel Borrow	525	Cu. Yds.
206.081	Structural Earth Excavation-Abutments, Retaining Walls, Box Culverts and Structural Plate Units	200	Cu. Yds.
206.091	Structural Rock Excavation-Abutments, Retaining Walls, Box Culverts and Structural Plate Units	10	Cu. Yds.
304.10	Aggregate Subbase Course-Gravel	800	Cu. Yds.
307.20	Sand Leveling	13	Cu. Yds.
403.07	Hot Bituminous Pavement, Grading B	130	Ton
403.08	Hot Bituminous Pavement, Grading C	80	Ton
403.101	Hot Bituminous Pavement, Grading D (sidewalk)	15	Ton
502.21	Structural concrete, Abutments & Retaining Walls	450	Cu. Yds.
502.27	Structural Concrete Superstructure T-beam Type	1	Lump Sum
503.12	Reinforcing Steel Fabricated & Delivered	16,500	Lbs.
503.13	Reinforcing Steel Placing	16,500	Lbs.
507.092	Aluminum Bridge Railing, 2 Bar	33	Lin. Ft.
507.094	Aluminum Bridge Railing, 3 Bar, with pales	33	Lin. Ft.
511.0701	Cofferdam - Abut. #1	1	Lump Sum
511.0702	Cofferdam - Abut. #2	1	Lump Sum
512.08	French Drains	170	Lin. Ft.
514.06	Curing Box for Concrete Cylinders	1	Each
515.20	Protective Coating for Concrete Surfaces	115	Sq. Yds.
526.30	Temporary Concrete Barrier, Type 1	100	Lin. Ft.
526.40	Resetting Temporary Concrete Barrier, Type 1	100	Lin. Ft.
606.265	Terminal End-Single Rail-Galvanized Steel	4	Each
606.35	Guard Rail Delineator Post	4	Each
606.55	Guard Rail Type 3-Single Rail	213	Lin. Ft.
606.60	Guard Rail Type 3-over 15 Ft. Radius	88	Lin. Ft.
609.13	Vertical Bridge Curb-Type 1	81	Lin. Ft.
609.31	Curb Type 3	130	Lin. Ft.
610.08	Plain Riprap	120	Cu. Yds.
615.07	Loom	26	Cu. Yds.
616.08	Sodding	12	Sq. Yds.
618.14	Seeding Method No. 2	4	Unit
618.15	Temporary Seeding	3	Lbs.
619.12	Mulch	4	Unit
627.63	4 inch Solid Yellow Pavement Marking Line	500	Lin. Ft.
627.67	Removing Pavement Markings	188	Sq. Ft.
629.05	Hand Labor, Straight Time	10	Man Hr.
630.0606	Traffic Officers	25	Man Hr.
631.10	Air Compressor (including operator)	10	Hour
631.11	Air Tool (including operator)	10	Hour
631.12	All Purpose Excavator (including operator)	10	Hour
631.132	Small Bulldozer (including operator)	10	Hour

ESTIMATED QUANTITIES			
ITEM No.	DESCRIPTION	QUANTITY	UNIT
631.171	Truck-small (including operator)	10	Hour
631.22	Front End Loader (including operator)	10	Hour
637.07	Sprinkling	10	M. Gal.
637.08	Calcium Chloride	1	Ton
639.20	Field Office Type C	1	Each
643.72	Temporary Traffic Signal	1	Lump Sum
652.31	Type I Barricades	10	Each
652.34	Cones	10	Each
652.35	Construction Signs	300	Sq. Ft.
652.36	Maintenance of Traffic Control Devices	120	Col. Days
652.37	Warning Lights	2	Group
652.38	Flaggers	100	Man Hr.
659.10	Mobilization	1	Lump Sum
502.27	Structural Concrete Superstructure T-beam Type	78	Cu. Yds.



- ### CONSTRUCTION NOTES
- All utility facilities shall be adjusted by the respective utilities unless noted.
  - For easements, construction limits and right-of-way lines refer to Right of Way Map.
  - The Contractor shall maintain a minimum 14 foot roadway at all times.
  - Place a 2 foot wide strip of sod on the side slopes along the top of the riprap.
  - The clearing limits as shown on the plans are approximate. The exact limits shall be established in the field by the Engineer. Payment for clearing shall be incidental to contract items.
  - Place loam, 2 inches deep, on slopes between Station 229+0 and Station 231+50.
  - Do not excavate for Aggregate Subbase Course where existing material is suitable as determined by the Engineer. Shaping and compacting of the existing subbase and layers of new subbase 6 inches or less thick, in areas where the Engineer directs the Contractor not to excavate to the subgrade line shown on the plans, will be paid for with appropriate equipment rental items.
  - One guard rail delineator post and one terminal end shall be installed at each guard rail end.
  - All embankment material, except as otherwise shown, placed below Elevation 328.0, shall be granular borrow meeting the requirements of Subsection 702.19, Material for Underwater Backfill.
  - Payment for removal of the existing concrete curbs and portions of the T-beams, where shown on the plans, shall be paid for under Item 202.17.
  - The Contractor shall remove the 4 granite mill wheels, located in the existing wingwalls, with care and shall store them within the right of way. They are to become the property of the Town of Athens. Payment for removing and storing of the granite mill wheels shall be incidental to contract items.
  - Sodded gutters shall be constructed after paving and shoulder work is completed, where it is apparent that runoff will cause continual erosion.
  - Payment for the removal of the existing beam type guard rail over superstructure curbs to be considered incidental to item 202.17.
  - Chamfer all exposed edges of concrete a consistent dimension between 1/8 inch and 3/8 inch inclusive, unless otherwise indicated.
  - Reinforcing steel shall have 2 inches cover unless otherwise indicated.
  - Break bond at vertical contraction joints by a method approved by the Engineer.
  - Waterstops are not required in horizontal construction joints.
  - Protective coating for concrete surfaces shall be applied to the following areas:
    - Top of concrete sidewalk and curb.
    - Fascias and outside face of exterior T-beams.
    - Ends of sidewalk and curb to 1 foot below finished grade.
    - Top and back face of parapets to 1 foot below finished grade.
    - All exposed surfaces of concrete and posts.
  - Place 4 inch diameter drains in breastwall and wings at 20 foot maximum spacing. Exact location to be determined by the Engineer in the field.
  - Form a 1 inch V-groove on the fascias at the horizontal joint between the curb and the slab.
  - Mortar for bedding and for joints in the granite curb, and for the grout used in setting dowels (5508) shall contain an approved non-shrink additive.
  - Payment for drilling and grouting of dowels shall be considered incidental to contract items.
  - The superstructure slab concrete shall be placed monolithically with the beam section and shall be kept plastic until the entire slab concrete, on at least one side of the bridge, has been placed. Approved set retarding admixtures shall be used when authorized by the Engineer, in accordance with the Standard Specifications.
  - All deteriorated concrete on the exposed faces of the existing T-beams shall be removed and shall be patched with a non-shrink grout as directed by the Engineer. Payment for all materials and labor to be incidental to contract items.
  - The ends of the superstructure 4 inch PVC sections shall be plugged by a method approved by the Engineer.

As Built 1983

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

WESSERUNSETT  
STREAM BRIDGE

ATHENS

ESTIMATED QUANTITIES  
and CONSTRUCTION NOTES  
SHEET 2 OF 19 AUGUSTA, MAINE JAN 1982

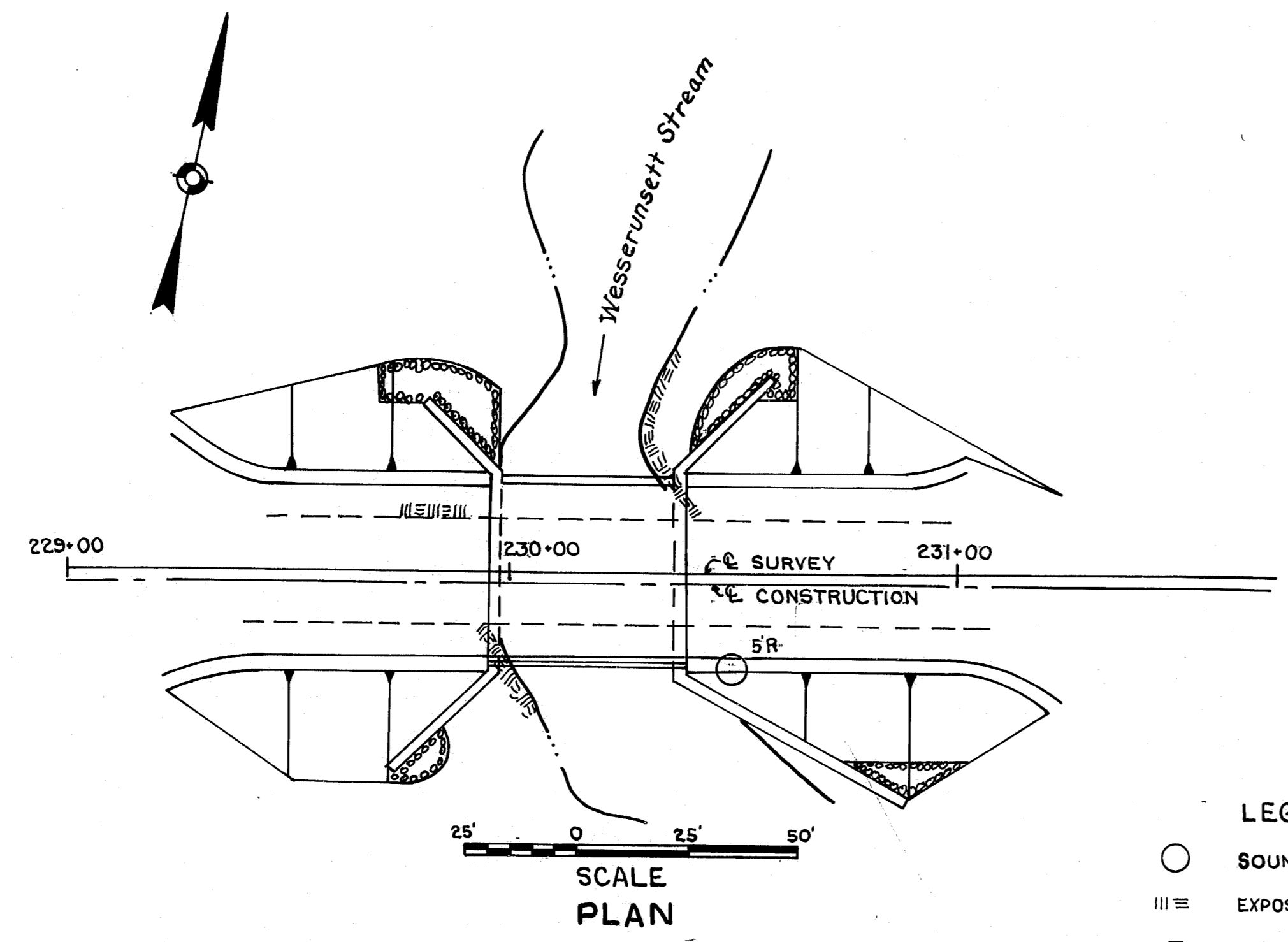
R89-459

PROJECT DESIGN ENGINEER  
DESIGN-DETAILED  
CHECKED  
REVISED  
FIELD CHANGES

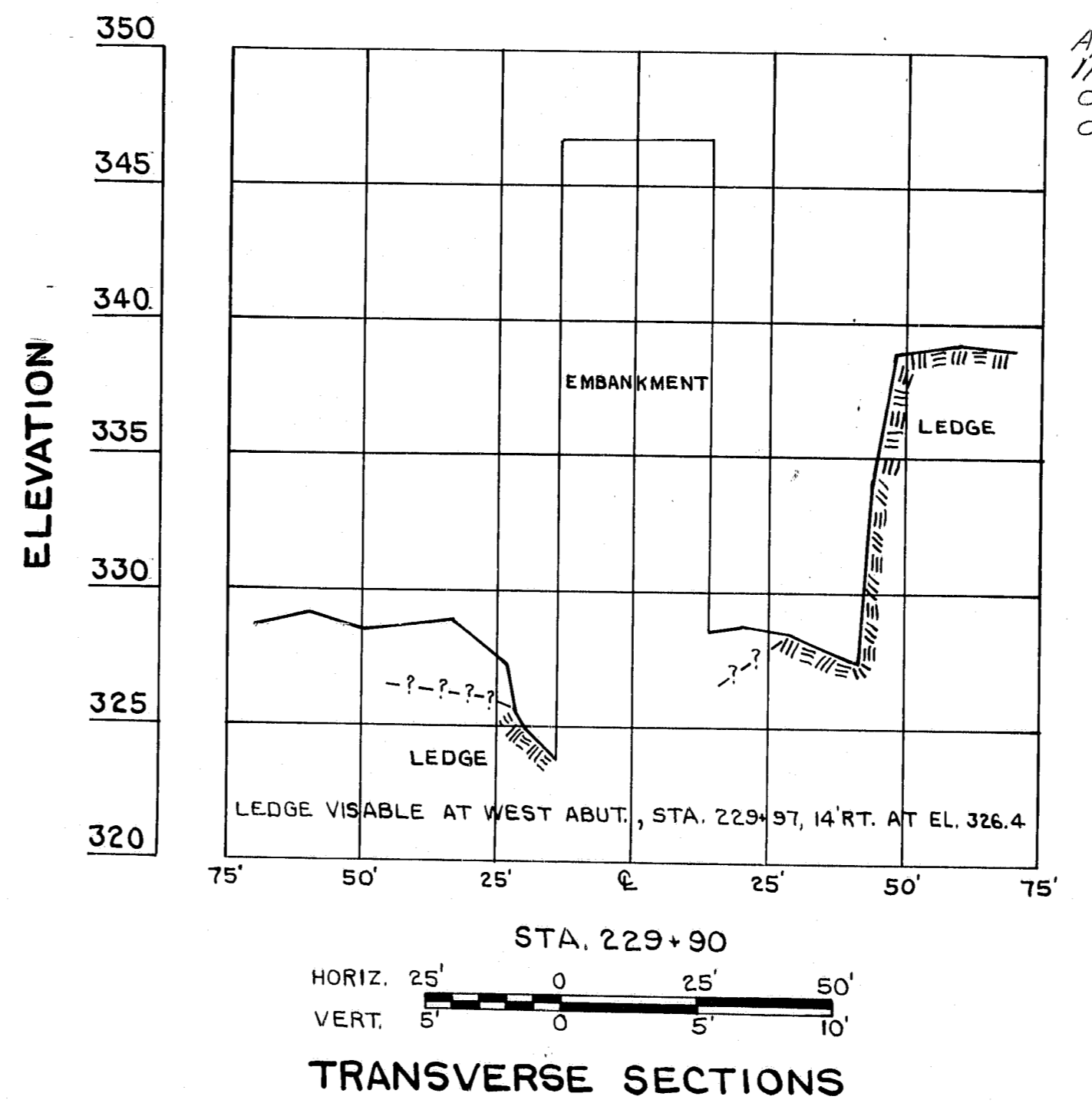
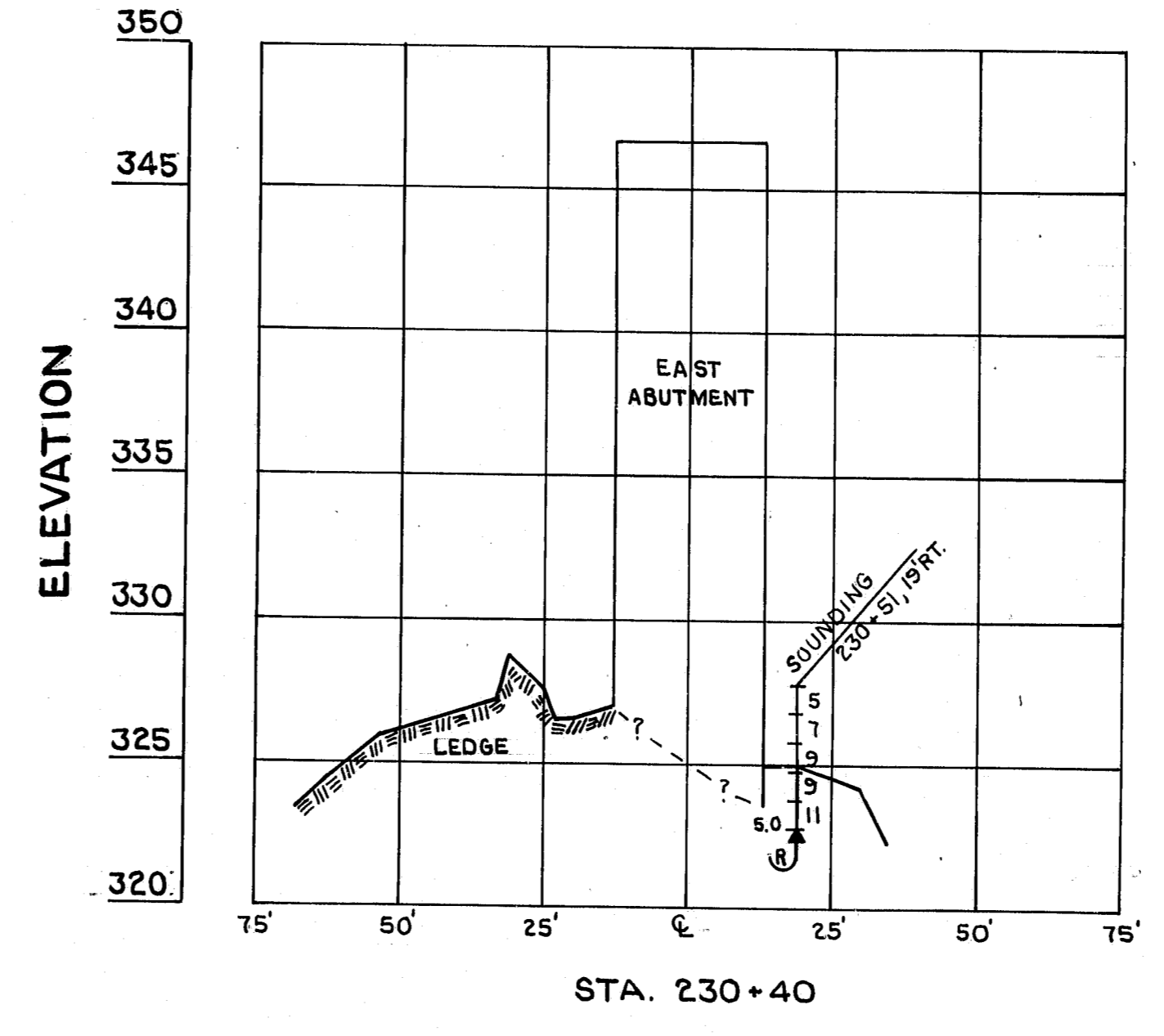
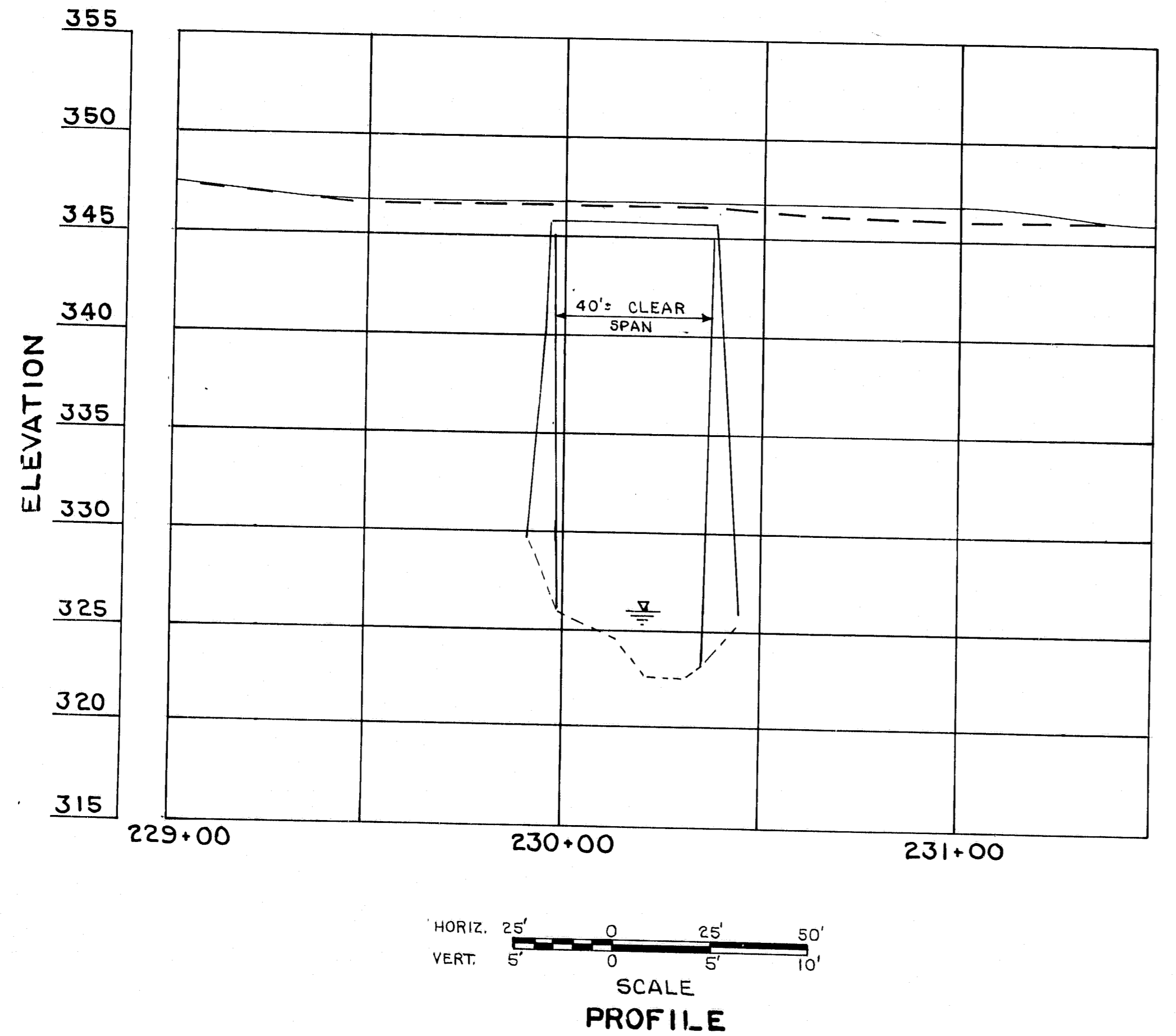
DATE  
BY  
7-81

BRUNING 64132 8/70

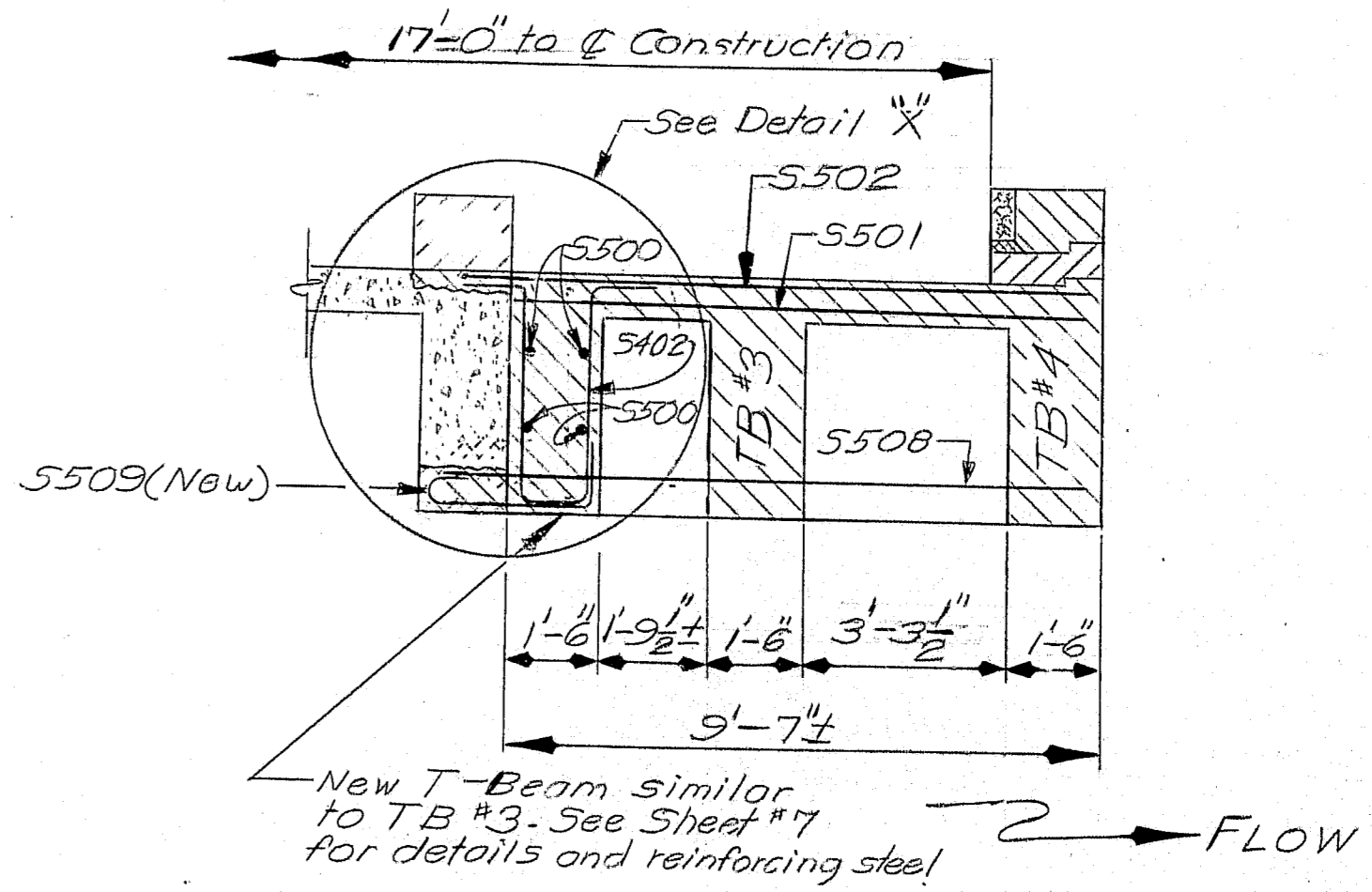
F.R.M.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	25-0241(1)	3	19



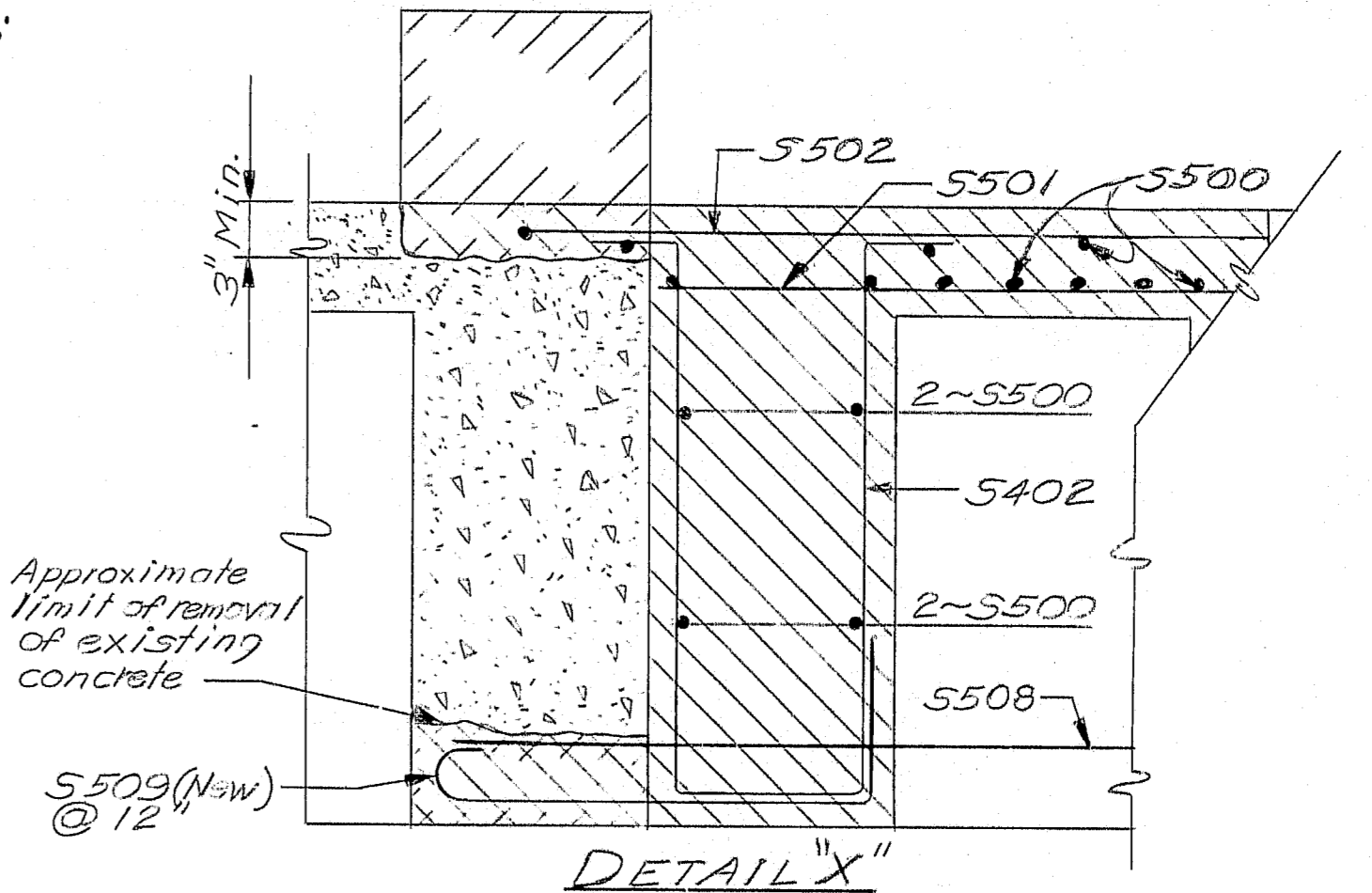
- LEGEND**
- SOUNDING
  - ≡ EXPOSED LEDGE
  - ⊥ BLOWS PER FOOT-ROD SOUNDING
  - ⊥ REFUSAL



TRANSVERSE SECTIONS



DOWNSTREAM TRANSVERSE SECTION



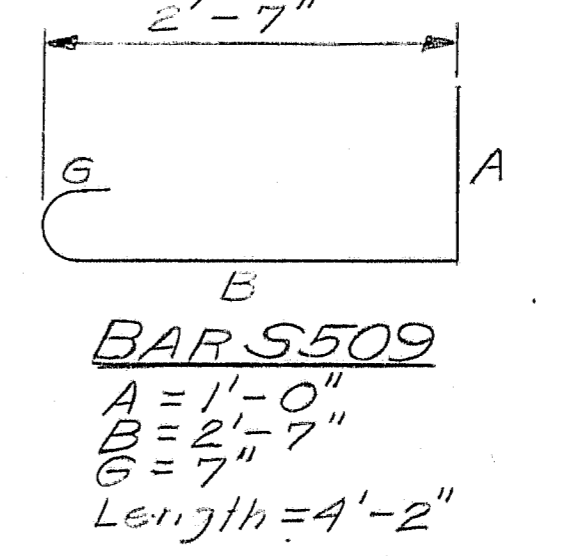
DETAIL "X"

**ADDITIONAL REINFORCING STEEL REQUIRED**

MARK	NUMBER
S500	17
S501	5
S502	16
S501	16
S502	26
S501	3
S509	40 (maximum, if full length of existing T-Beam concrete removed at bottom)

NOTE: See Sheet 19 for Reinforcing Steel Schedule. Bar S509 is on this sheet. Additional S500 bars are required due to previous omission from the reinforcing steel schedule of bars required in the T-Beams.

Detail of T-Beam revision on downstream side of bridge with additional reinforcing steel required. 10-1-82  
As Built 1983 28



BAR S509  
A = 1'-0"  
B = 2'-7"  
C = 7"  
Length = 4'-2"

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

**WESSERUNSETT  
STREAM BRIDGE**  
IN THE TOWN OF  
**ATHENS**  
SOMERSET COUNTY  
FOUNDATION SURVEY

**R89-455**

PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED		
REVISIONS		
FIELD CHANGES		
<b>PLANS</b>		

BRIDGE 44132-45710

✓ R89-456  
DESIGN: PJM

ORIGINAL SURVEY	DATE
BY	1-23-11
APPROVED	
DATE	

FINAL SURVEY	DATE
BY	
APPROVED	
DATE	

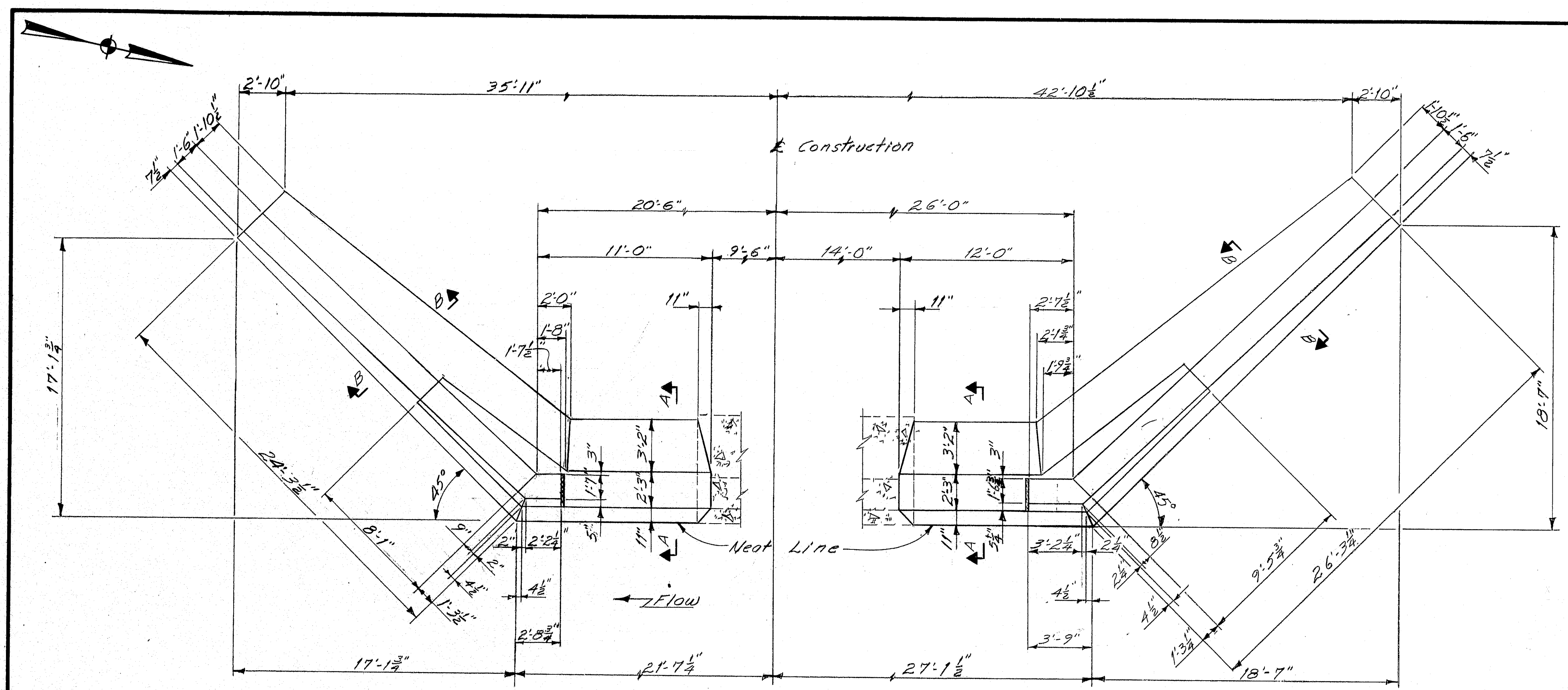


FED. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
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**R89-456**

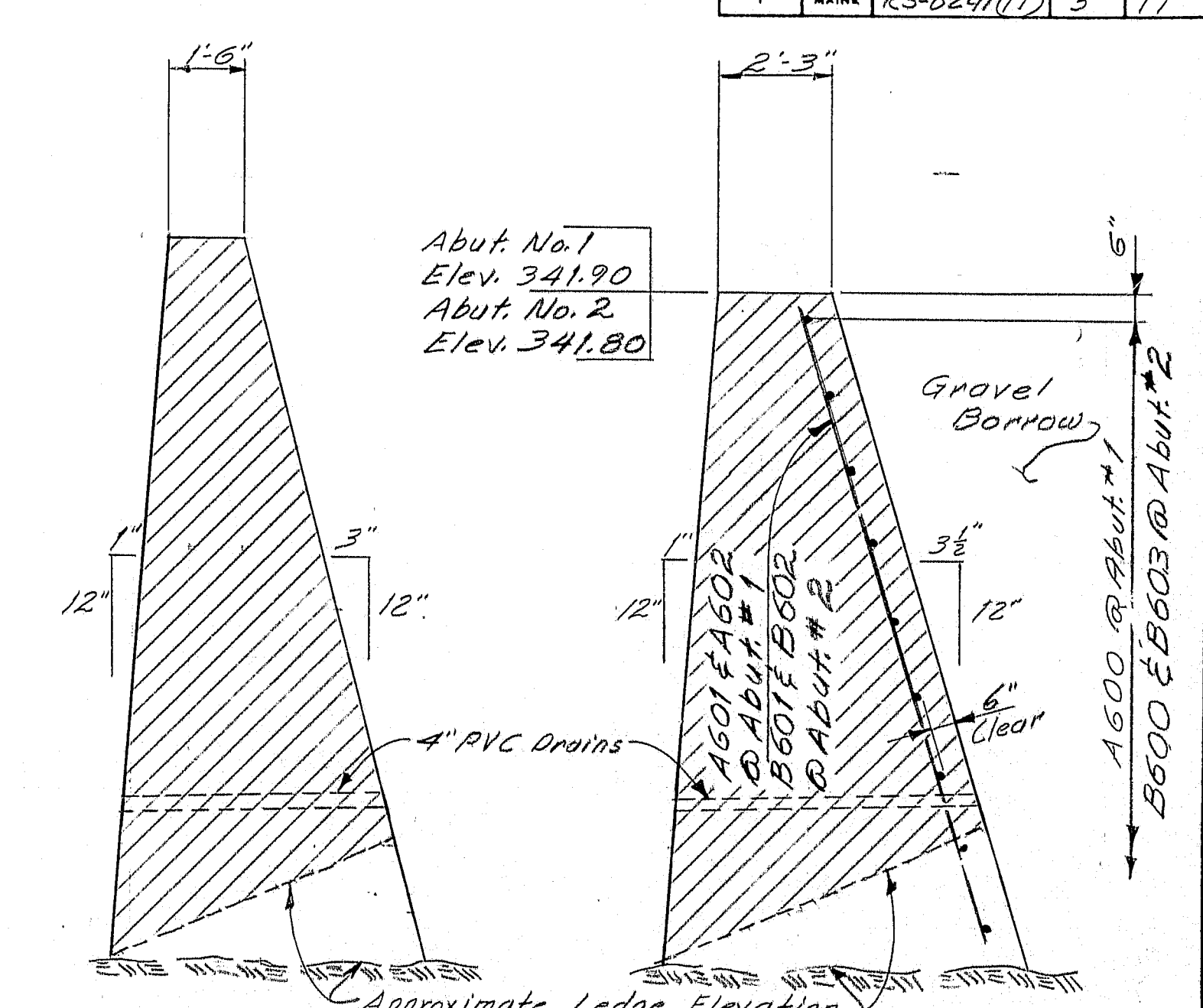
ATHENS  
WESSERUNSETT STREAM  
X-SECTIONS 4 Sheet of 19

F.R.W.A. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
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**PLAN**

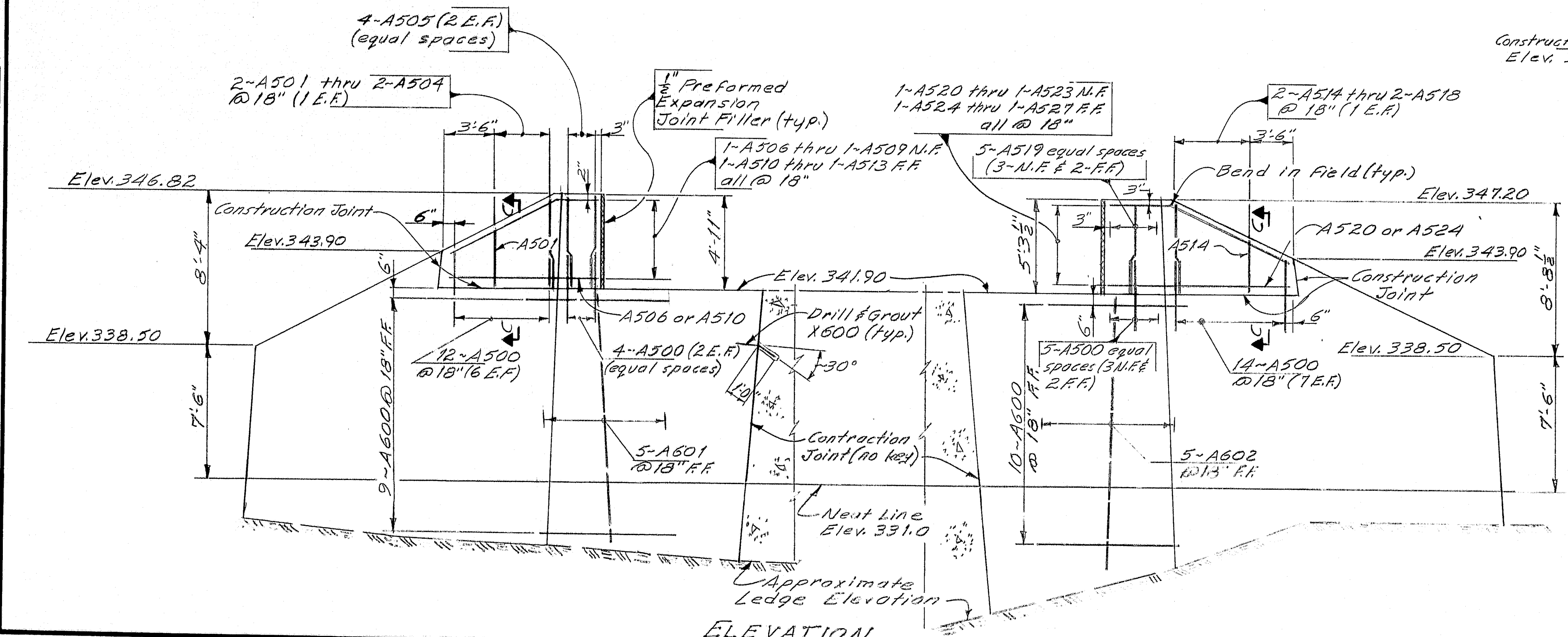
The face of the new abutment to meet the face of the existing abutment.



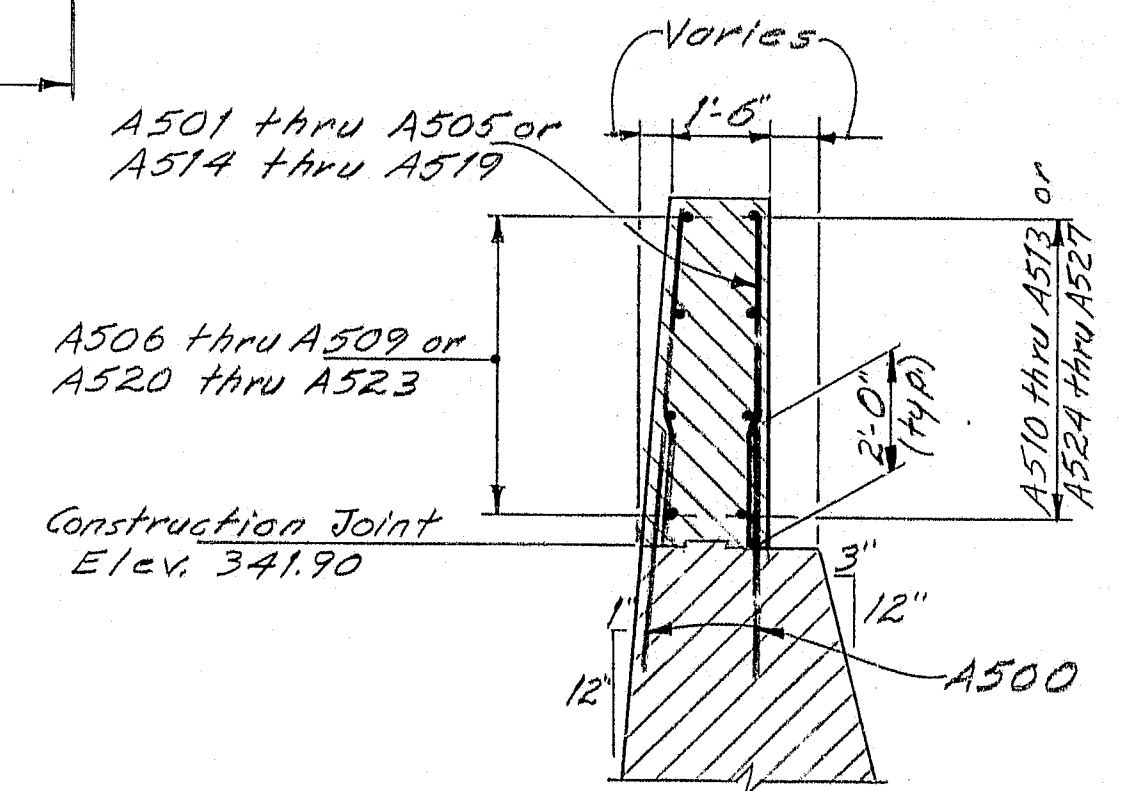
**SECTION B-B**

Sloping ledge in the vicinity of the structure shall be stepped where directed by the Engineer. Payment for the removal of ledge shall be made under Item 206.091

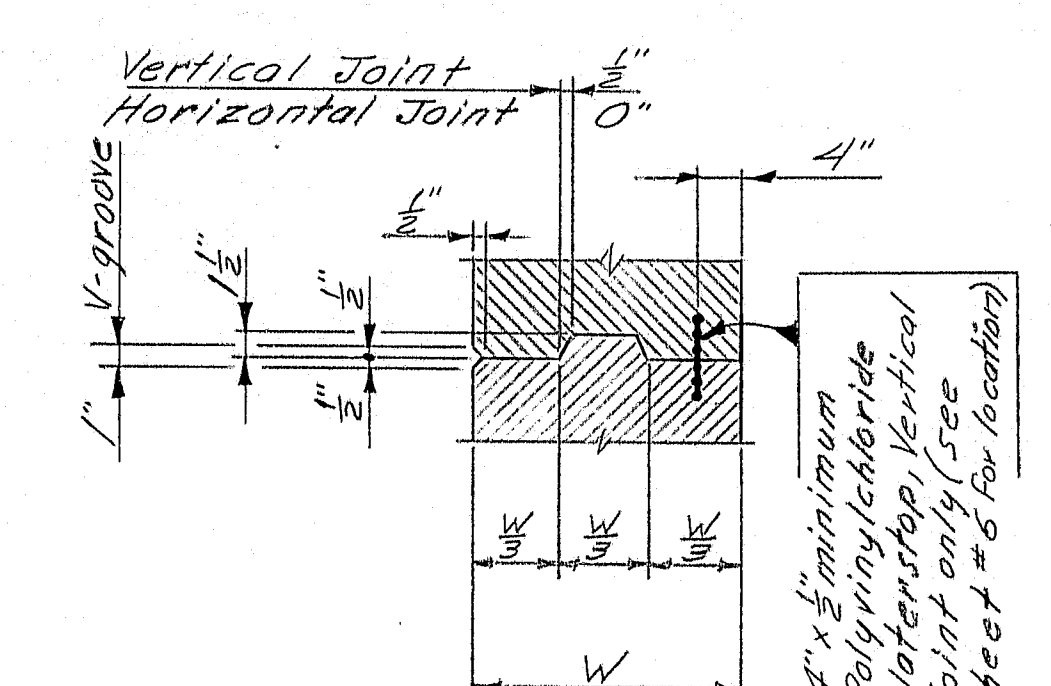
**SECTION A-A**



**ELEVATION**



**SECTION C-C**



**TYPICAL CONSTRUCTION AND CONTRACTION JOINTS**

- REFERENCES**
- For Construction Notes see Sheet #2.
  - For French Drain see Sheet #6.
  - For Gravel Borrow Limits see Typical Section sheet #6
- LEGEND**
- N.F. = Near Face
  - F.F. = Far Face
  - E.F. = Each Face
- SYMBOLS**
- Plan or Elev.
  - ▨ Section
  - ▤ Existing Concrete (to remain)
  - ▥ Approximate Ledge
  - ▧ New Concrete

As Built 1983 R3

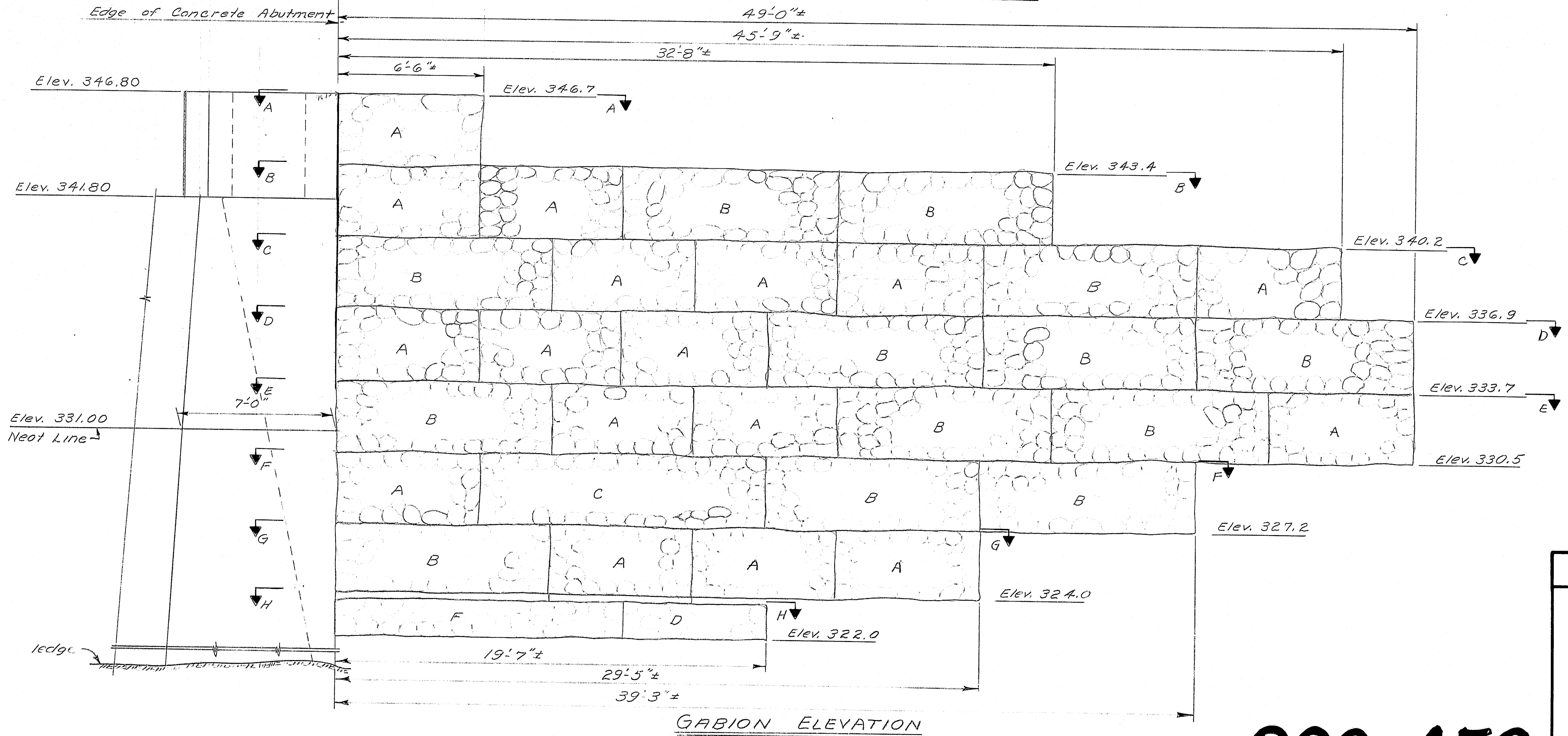
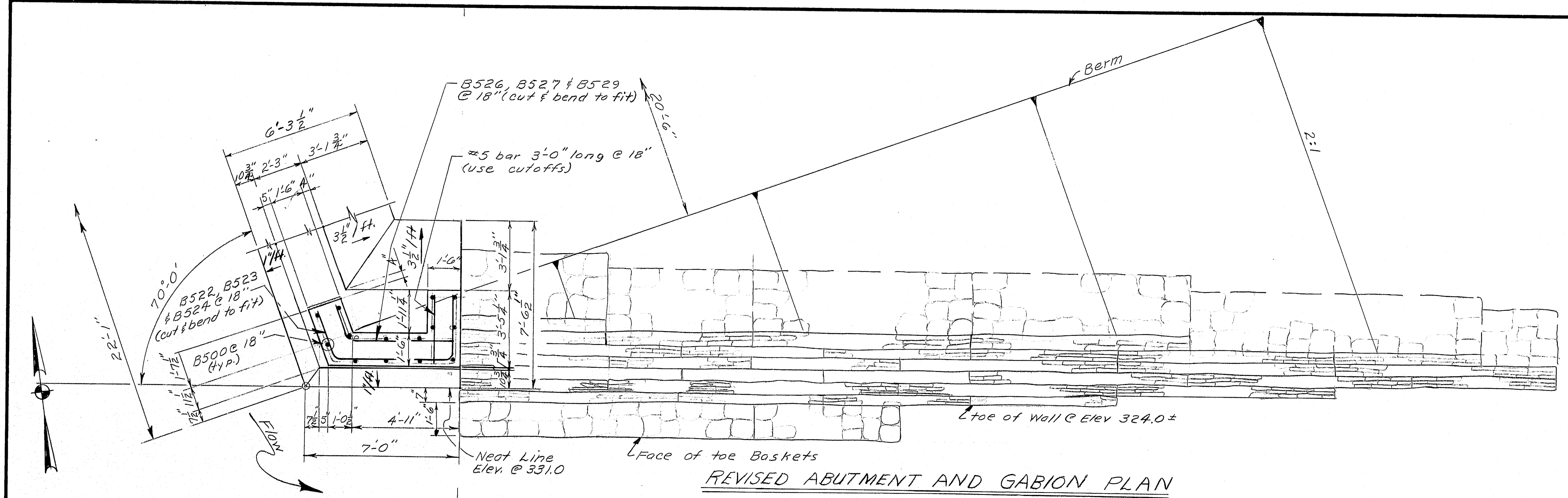
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
WESSERUNSETT  
STREAM BRIDGE  
ATHENS  
ABUTMENT NO. 1  
SHEET 5 OF 19 AUGUSTA, MAINE Jan. 1982

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	7-97
CHECKED	7-97
FIELD CHANGES	

**R89-457**



F.R.A. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	RS-027(11)	6a	



PROJECT DESIGN ENGINEER	BY	DATE
PLANS	W. M. Mousley	10/27/82
DESIGN DETAILING		
CHECKED	R. H. H.	
REVISIONS		
FIELD CHANGES		

BRUNING 44-152-28710

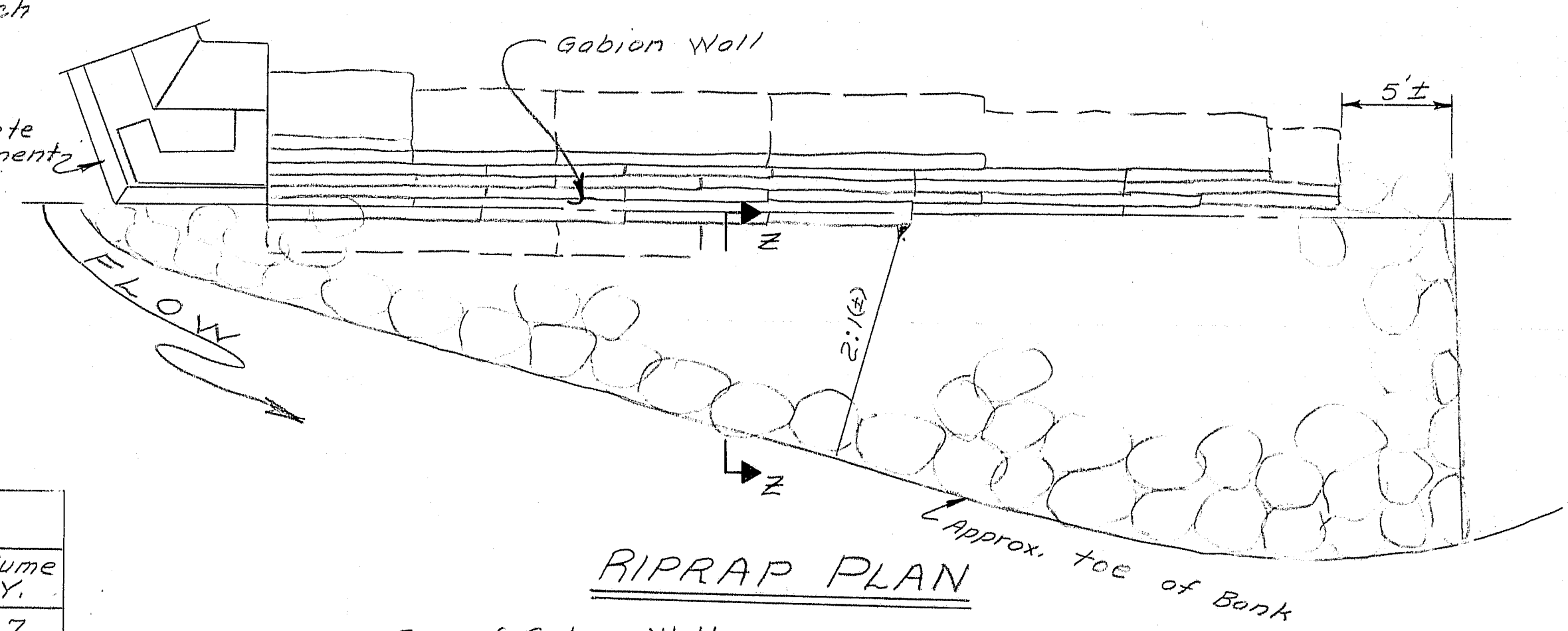
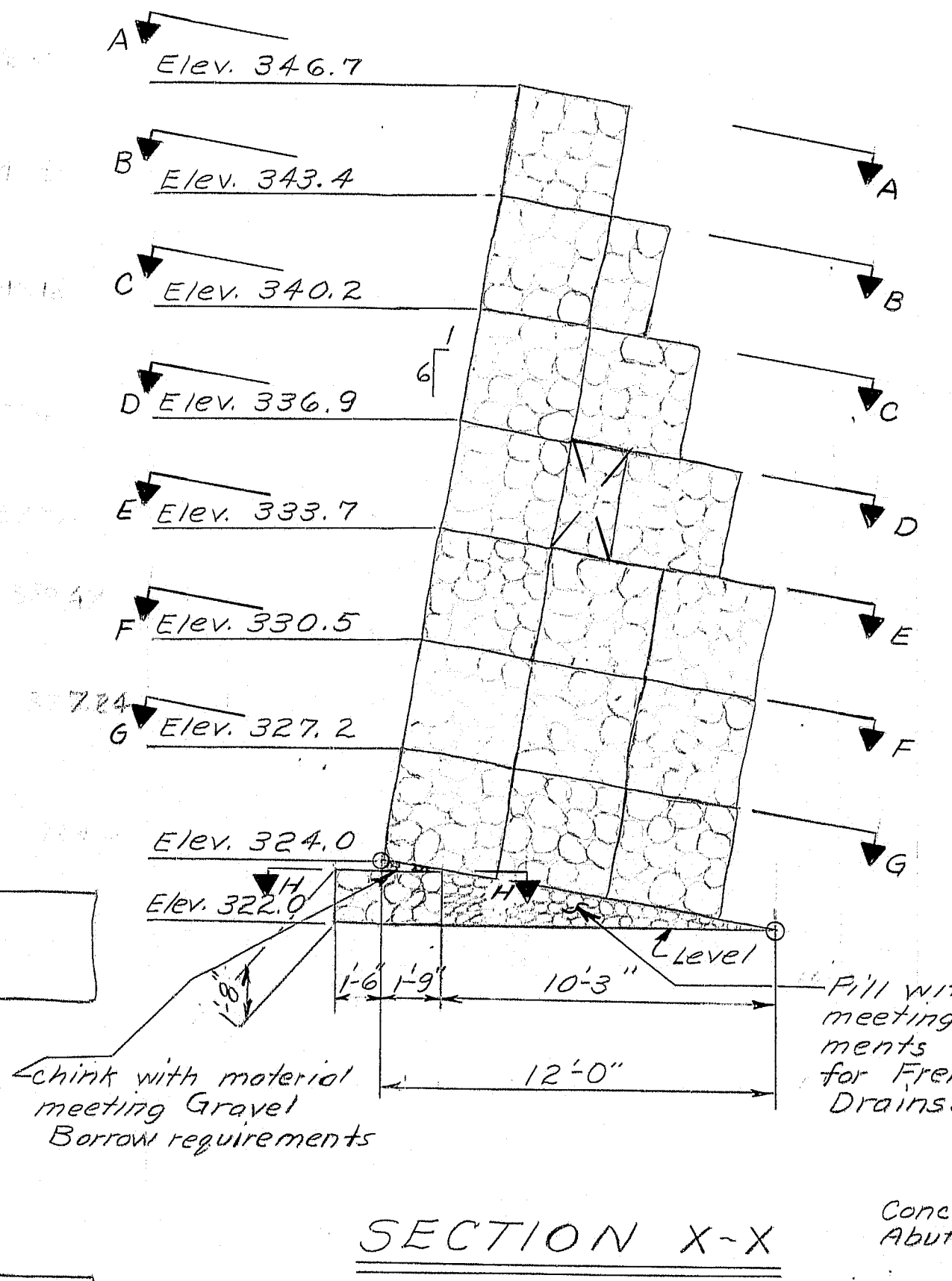
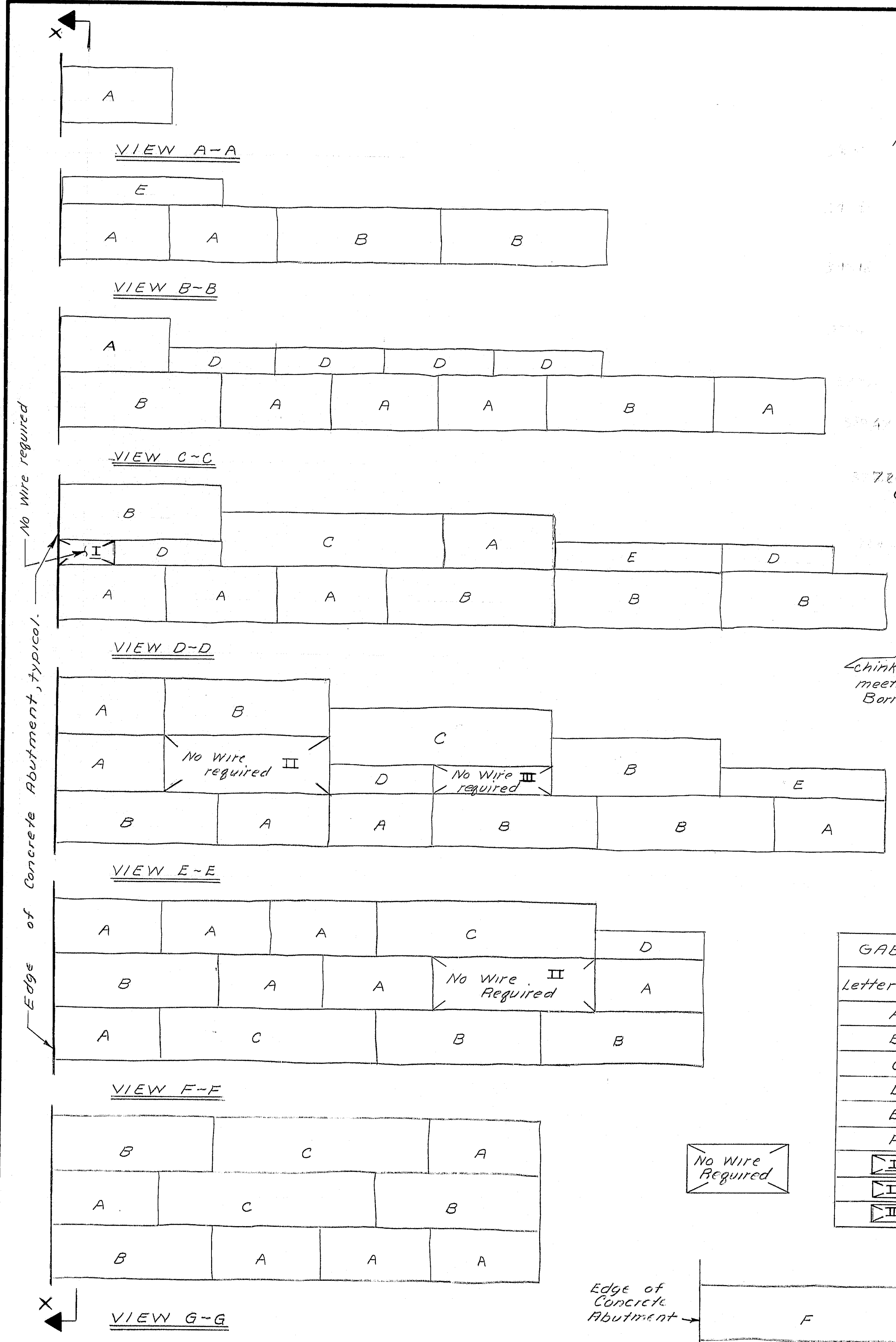
AS BUILT 1983  
 STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION  
 WESSERUNSETT  
 STREAM BRIDGE  
 ATHENS  
 ABUTMENT #2 REVISIONS  
 & GABIONS  
 SHEET 6a OF AUGUSTA, MAINE Oct 17, 1982

R89-459

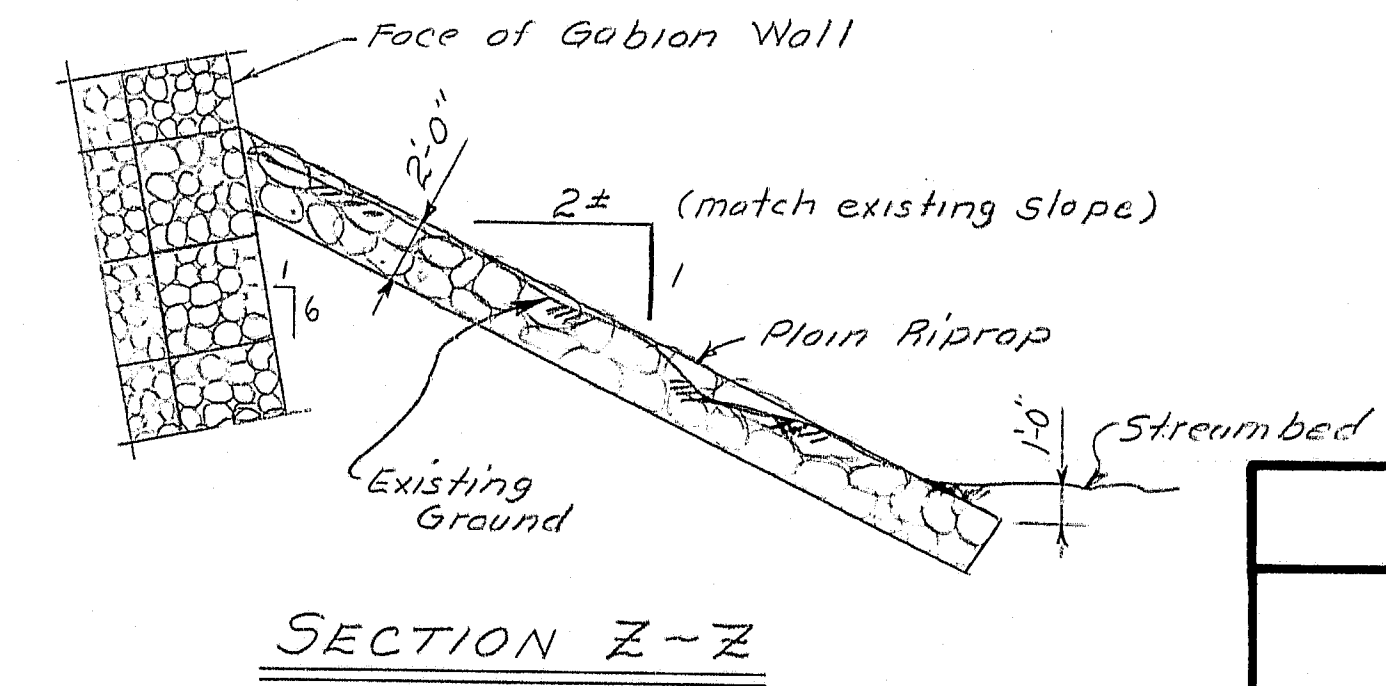
F.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	RS-0241(11)	66	

### GABION NOTES

- 1) The Gabions shall be constructed in accordance with the Department of Transportation Standard Specifications for Highways and Bridges Section 601.
- 2) All excavation required to construct the Gabion wall to neat lines 1'-6" out from the face of baskets will be paid for under Item 206.08, Structural Earth Excavation Abuts., Ret. Walls, etc.
- 3) Payment for all materials equipment and labor required to construct the Gabion foundation, as shown on the plans, will be considered incidental to the Gabions.
- 4) Place a 2 foot wide sod strip along the top of the Gabions.



GABION BASKET TABLE				QUANTITIES		
Letter Code	length	width	height	Volume C.Y.	No. of Baskets	Volume C.Y.
A	6'-6"	3'-3"	3'-3"	2.54	29	73.7
B	9'-9"	3'-3"	3'-3"	3.81	19	72.5
C	13'-1"	3'-3"	3'-3"	5.12	6	30.7
D	6'-6"	1'-8"	3'-3"	1.30	9	11.7
E	9'-9"	1'-8"	3'-3"	1.95	3	5.8
F	13'-1"	3'-3"	1'-8"	2.61	1	2.6
I	3'-3"	1'-8"	3'-3"	0.65	1	0.6
II	9'-9"	3'-3"	3'-3"	3.81	2	7.6
III	6'-6"	1'-8"	3'-3"	1.30	1	1.3
<b>TOTAL VOLUME</b>						<b>206.5</b>



As Built  
1983 R.M.J.

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

WESSERUNSETT  
STREAM BRIDGE

ATHENS

GABION DETAILS

SHEET 66 OF AUGUSTA, MAINE Oct. 7, 1982

PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED	PLM	10/2/82
CHECKED	ELB	10/2/82
REVISIONS		
FIELD CHANGES		

R89-460



**REINFORCING STEEL SCHEDULE**

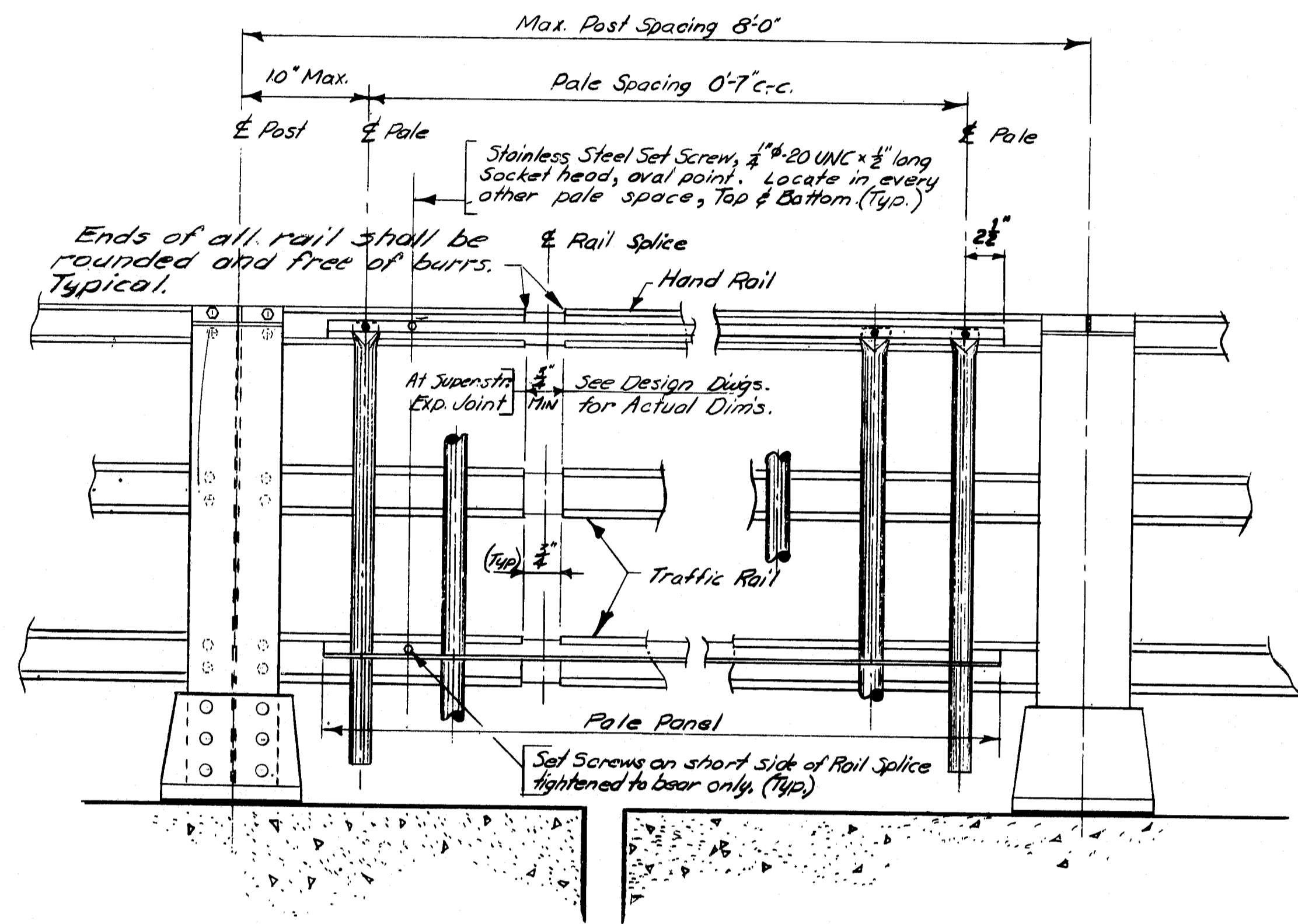
STRAIGHT BARS												BENT BARS														
MARK	NO.	LENGTH	LOCATION	MARK	NO.	LENGTH	LOCATION	MARK	NO.	LENGTH	LOCATION	MARK	NO.	LENGTH	TYPE	A	B	C	D	E	F	G	H	O	R	LOCATION
<b>ABUTMENT NO. 1</b>												<b>ABUTMENT NO. 2</b>														
A500	35	4'-0"	Dowels	B500	39	4'-0"	Dowels	A506	1	10'-11"	V	-	-	-	-	-	-	8'-9"	2'-2"	-	-	-	1'-6 1/2"	-	-	Downstream Wing
A501	2	3'-1"	Downstream Wing	B501	2	3'-0"	Upstream Wing	A507	1	10'-9"	V	-	-	-	-	-	-	8'-8"	2'-1"	-	-	-	1'-5 3/4"	-	-	
A502	1	3'-7"		B502	1	3'-5"		A508	1	6'-11"	V	-	-	-	-	-	-	4'-10"	2'-1"	-	-	-	1'-5 3/4"	-	-	
A503	1	4'-1"		B503	1	4'-1"		A509	1	11'-0"	V	-	-	-	-	-	-	9'-0"	2'-0"	-	-	-	1'-5"	-	-	
A504	2	4'-7"		B504	1	4'-7"		A510	1	9'-5"	V	-	-	-	-	-	-	7'-11"	1'-6"	-	-	-	1'-0 3/4"	-	-	
A505	4	4'-9"	Downstream Wing	B505	2	5'-1"		A511	1	9'-5"	V	-	-	-	-	-	-	7'-11"	1'-6"	-	-	-	1'-0 3/4"	-	-	
A514	2	3'-0"	Upstream Wing	B506	5	5'-2"	Upstream Wing	A512	1	5'-7"	V	-	-	-	-	-	-	4'-1"	1'-6"	-	-	-	1'-0 3/4"	-	-	
A515	1	3'-7"		B515	2	2'-8"	Downstream Wing	A513	1	9'-10"	V	-	-	-	-	-	-	8'-4"	1'-6"	-	-	-	1'-0 3/4"	-	-	Downstream Wing
A516	1	4'-1"		B516	1	3'-1"		A520	1	13'-3"	V	-	-	-	-	-	-	10'-1"	3'-2"	-	-	-	2'-3"	-	-	Upstream Wing
A517	1	4'-7"		B517	1	3'-6"		A521	1	12'-11"	V	-	-	-	-	-	-	9'-10"	3'-1"	-	-	-	2'-2"	-	-	
A518	2	5'-0"		B518	1	3'-10"		A522	1	8'-6"	V	-	-	-	-	-	-	5'-5"	3'-1"	-	-	-	2'-2"	-	-	
A519	5	5'-1"	Upstream Wing	B519	1	4'-2"		A523	1	13'-5"	V	-	-	-	-	-	-	10'-6"	2'-11"	-	-	-	2'-0 3/4"	-	-	
				B520	2	4'-7"		A524	1	11'-9"	V	-	-	-	-	-	-	9'-3"	2'-6"	-	-	-	1'-9 1/4"	-	-	
				B521	4	4'-10"	Downstream Wing	A525	1	11'-7"	V	-	-	-	-	-	-	9'-1"	2'-6"	-	-	-	1'-9 1/4"	-	-	
A601	5	13'-0"	Walls					A526	1	7'-2"	V	-	-	-	-	-	-	4'-9"	2'-5"	-	-	-	1'-8 1/2"	-	-	
A602	5	15'-0"	Walls					A527	1	11'-11"	V	-	-	-	-	-	-	9'-6"	2'-5"	-	-	-	1'-8 1/2"	-	-	Upstream Wing
				B601	5	12'-0"	Walls	A600	19	8'-0"	V	-	-	-	-	-	-	4'-0"	4'-0"	-	-	-	2'-10"	-	-	Walls
				B602	5	15'-0"	Walls																			
							</																			



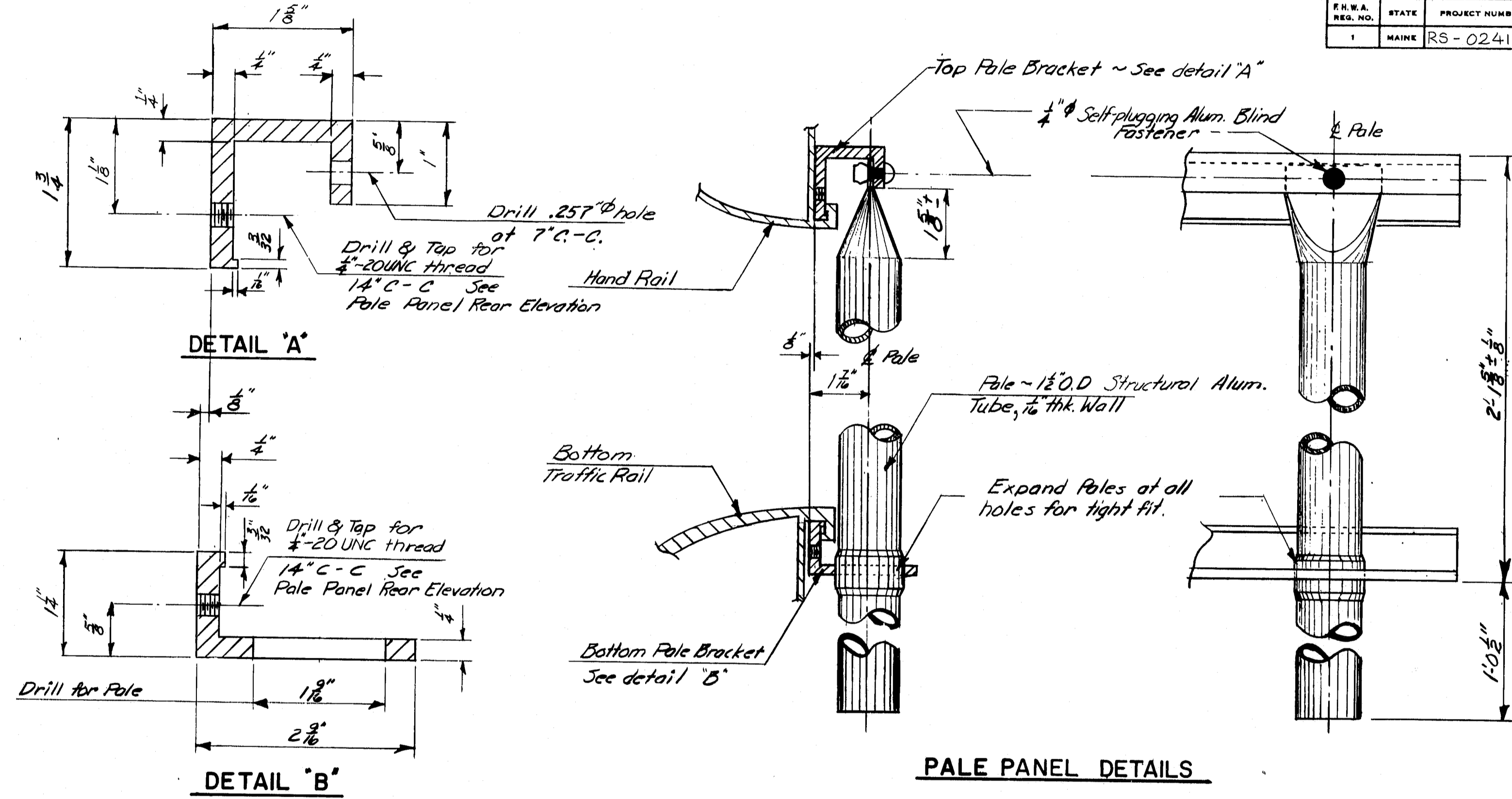




F.M.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	RS-0241(1)	12	19



PALE PANEL REAR ELEVATION



PALE PANEL

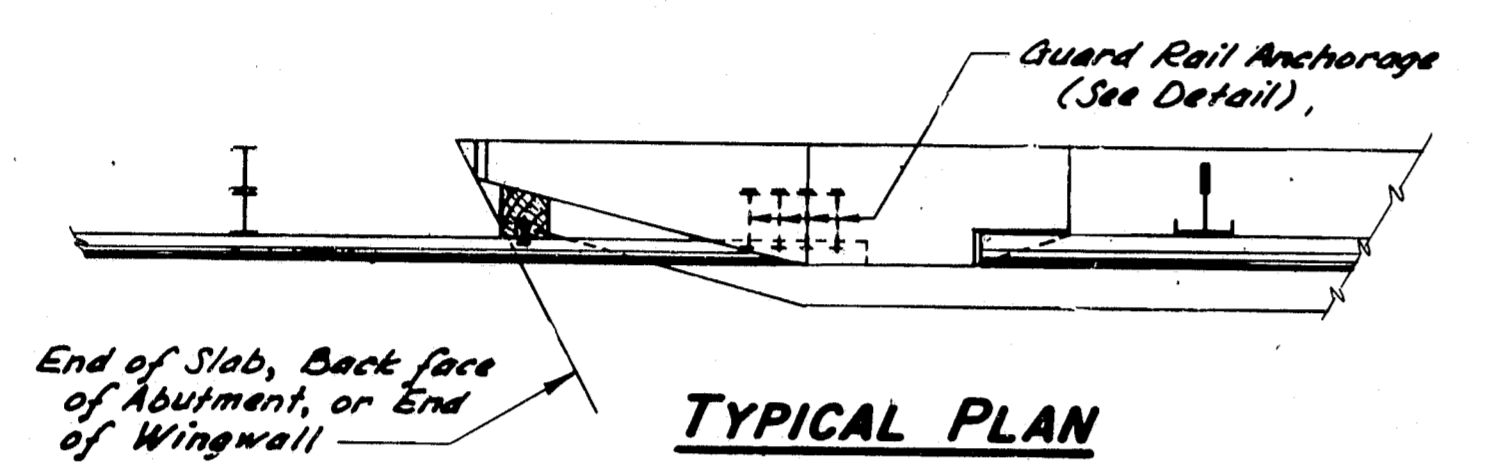
PLANS	DESIGN - DETAILED	DATE
	REVISIONS	BY
	FIELD CHANGES	DATE

As Built 1993 rmz	
REVISIONS	DATE
STATE OF MAINE DEPARTMENT OF TRANSPORTATION (SHEET BD 116 - 81 SHALL ACCOMPANY THIS SHEET) <b>STANDARD DETAILS</b> (BD 116 - 81) <b>ALUMINUM BRIDGE RAILING</b> PALE PANEL	
SHEET 12 OF 19 AUGUSTA, MAINE JUNE 1991	

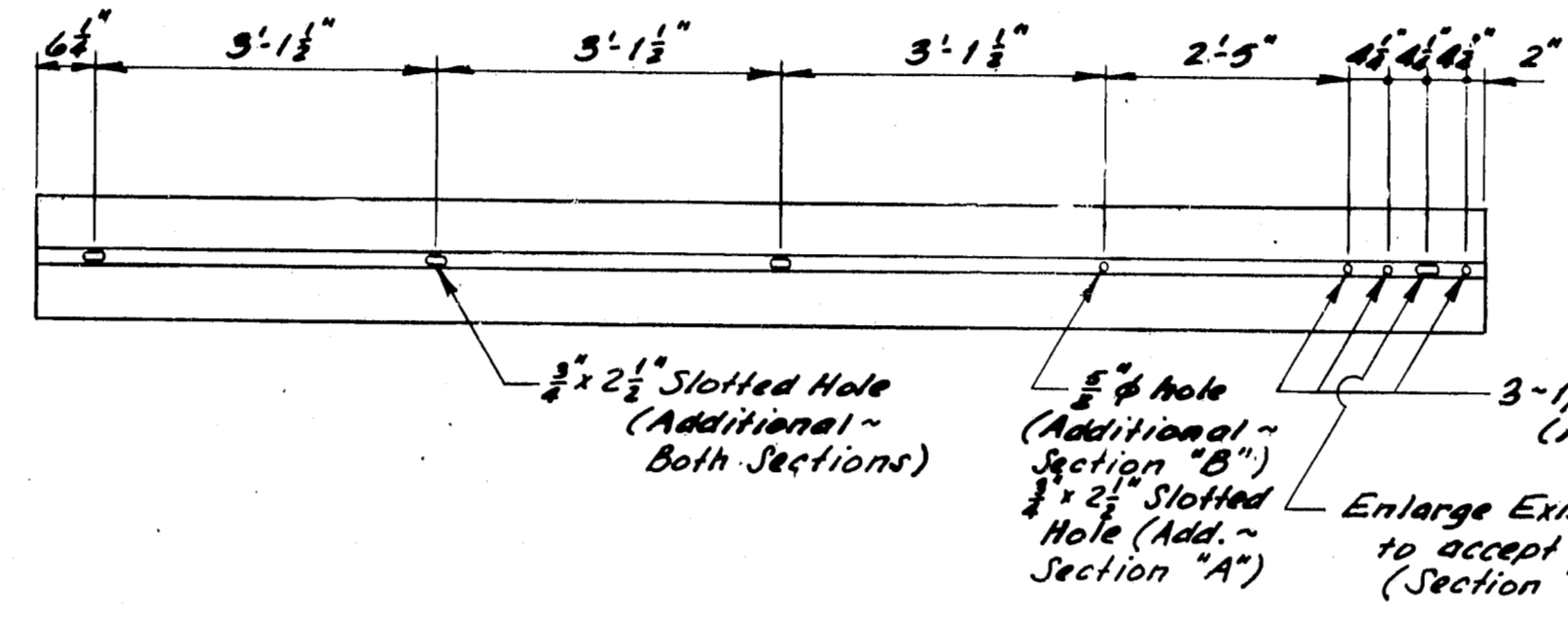
R89-466

ATKINS

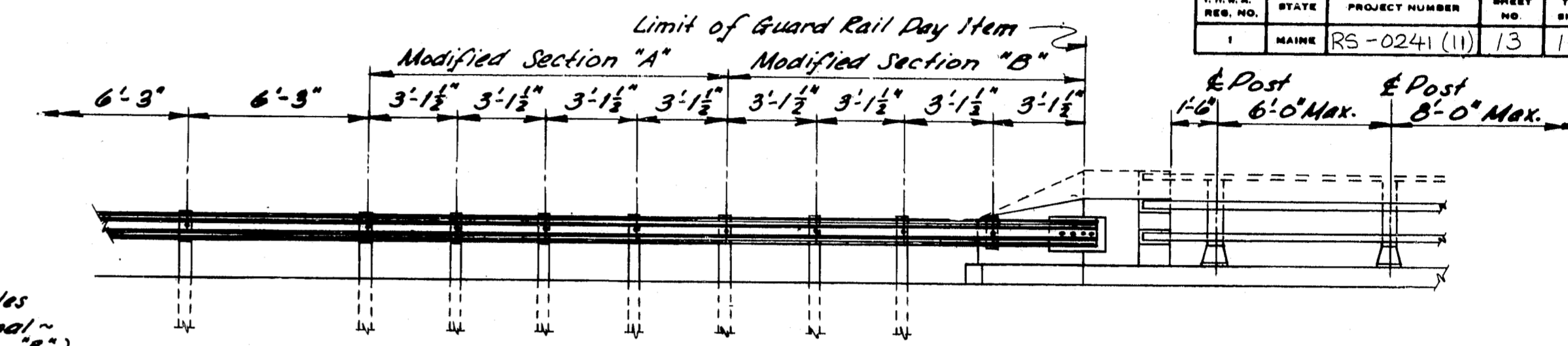
F.W.A. DIST. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	RS-0241 (11)	13	19



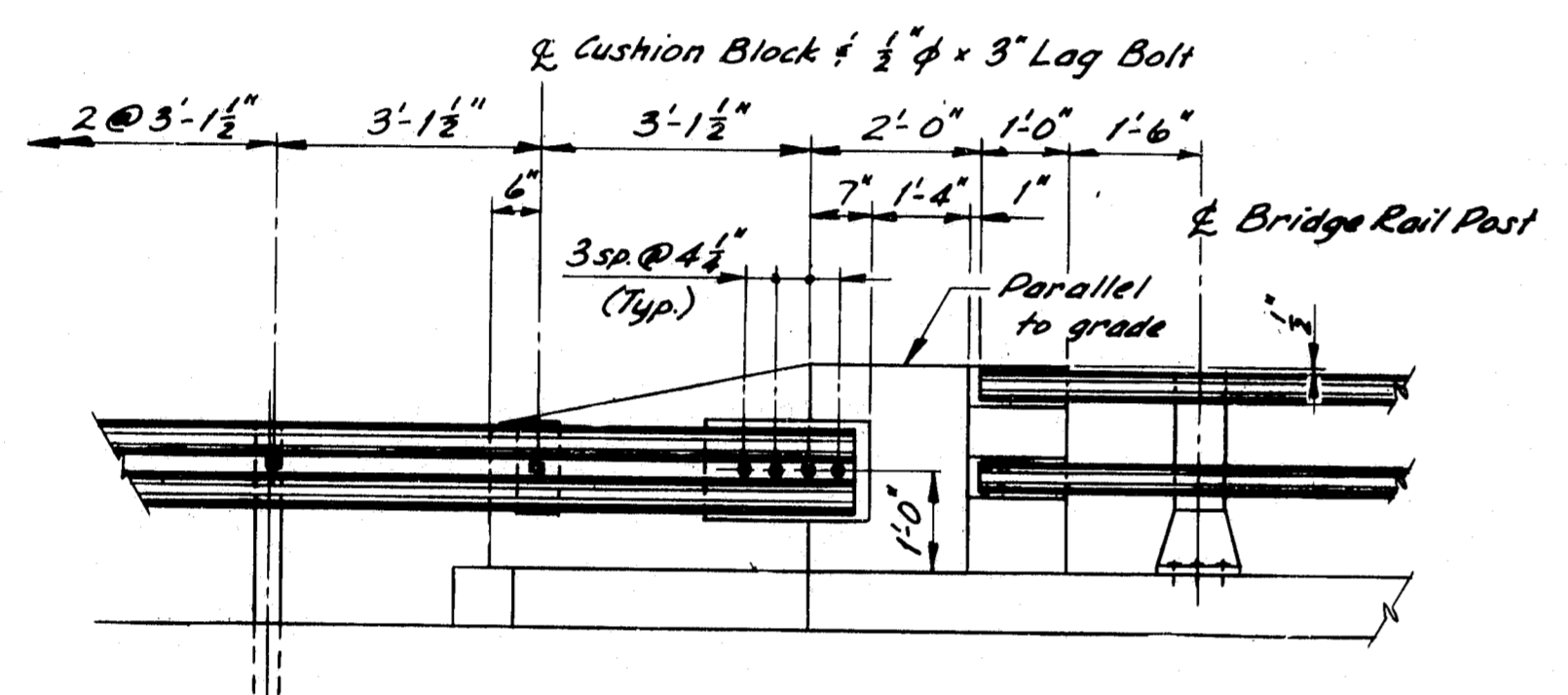
**TYPICAL PLAN**



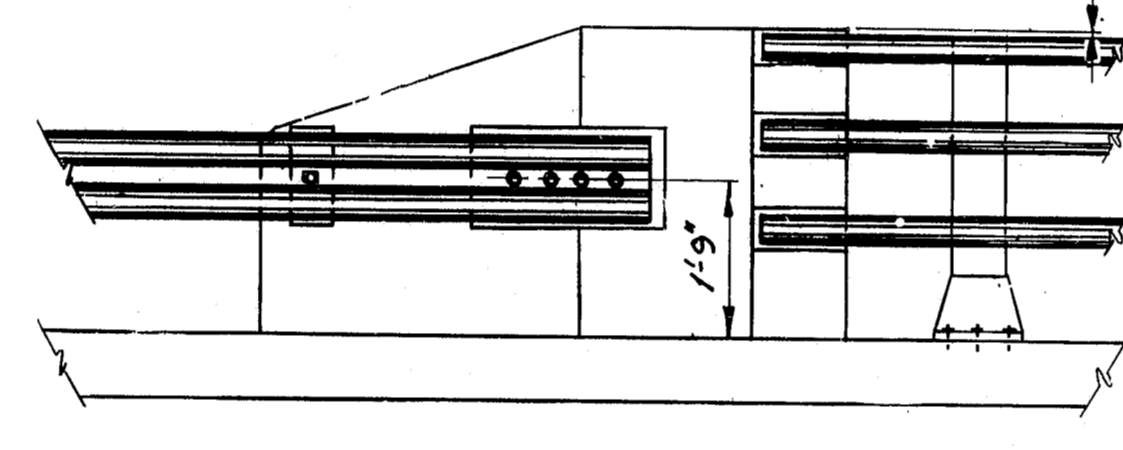
**MODIFIED GUARD RAIL SECTIONS**  
(See Note #6)



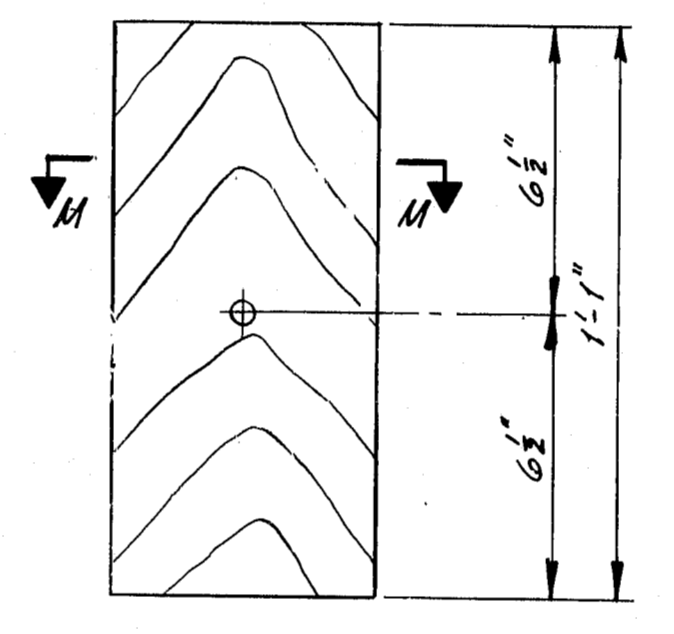
**RAILING - ELEVATION**



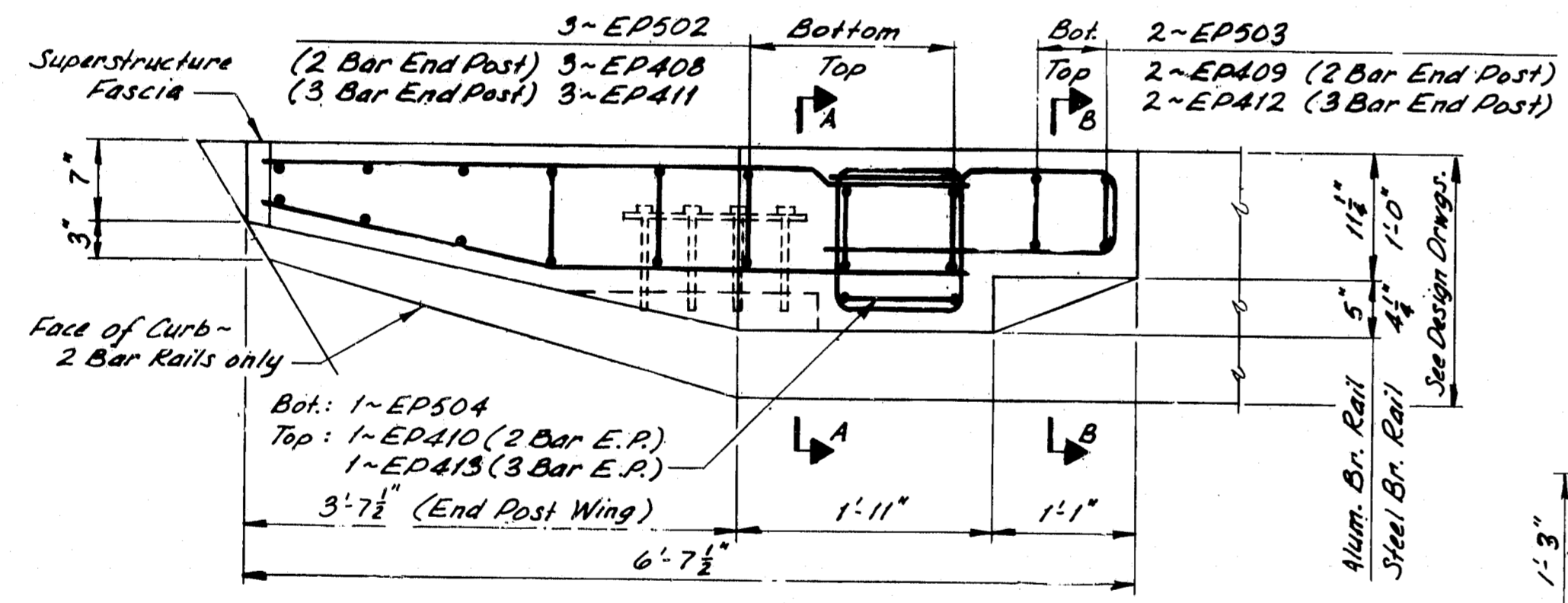
**ELEVATION**  
2-Bar Bridge Rail (Aluminum or Steel)



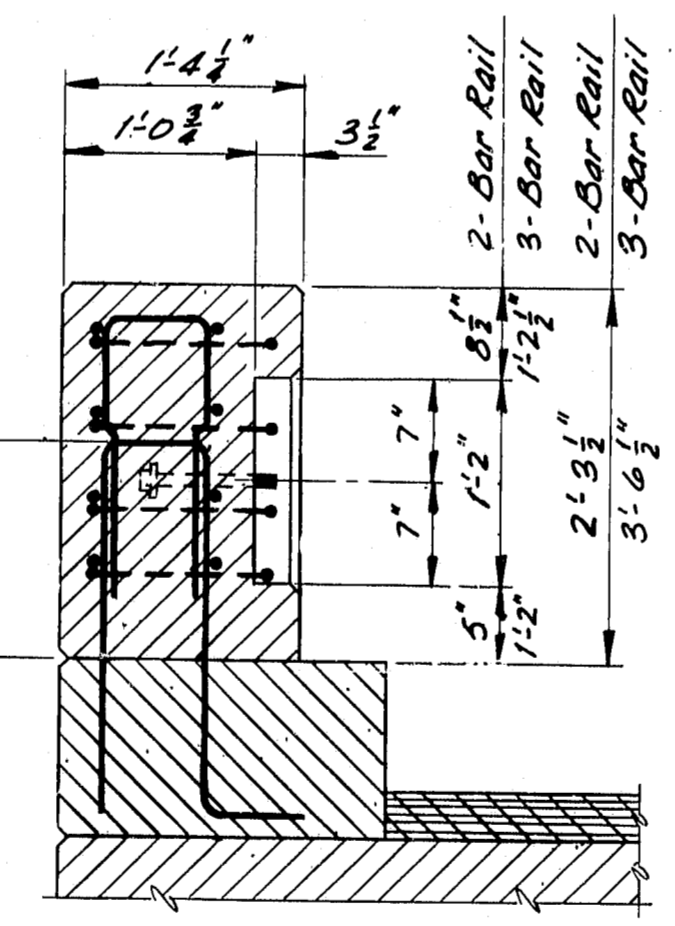
**ELEVATION**  
3-Bar Bridge Rail (Aluminum or Steel)



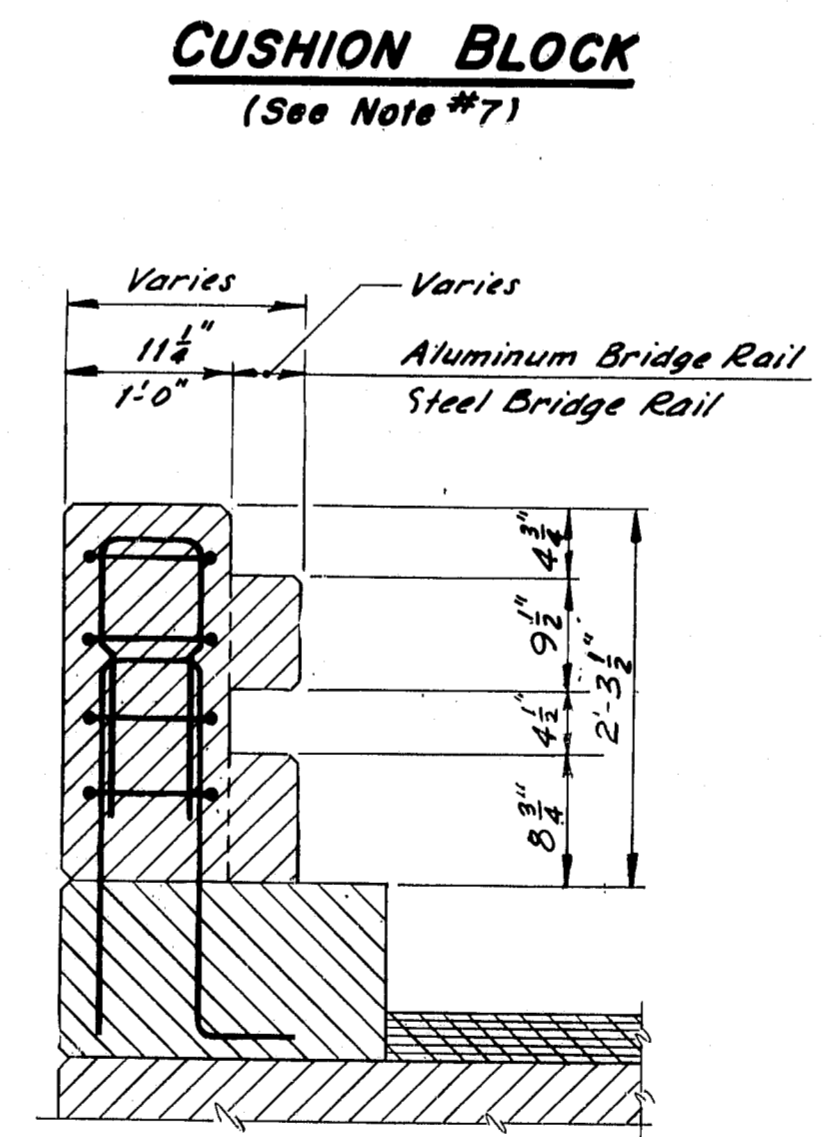
**SECTION M-M**



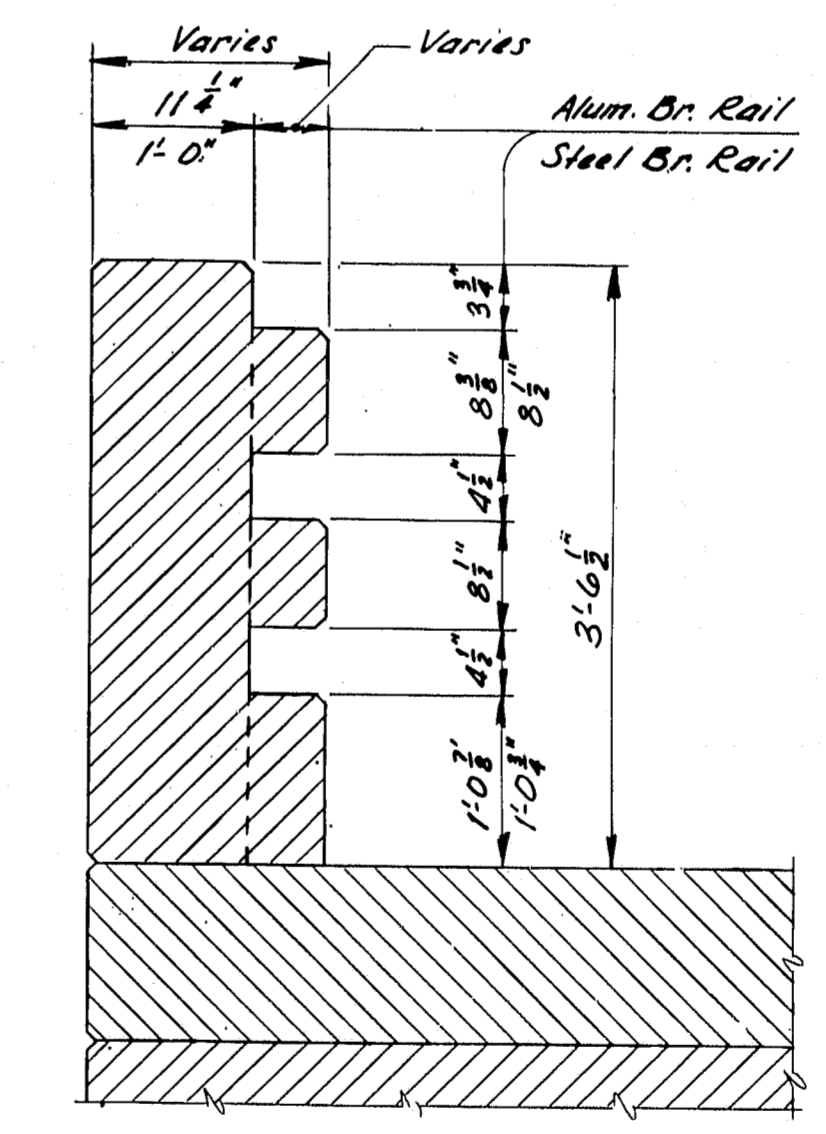
**PLAN**



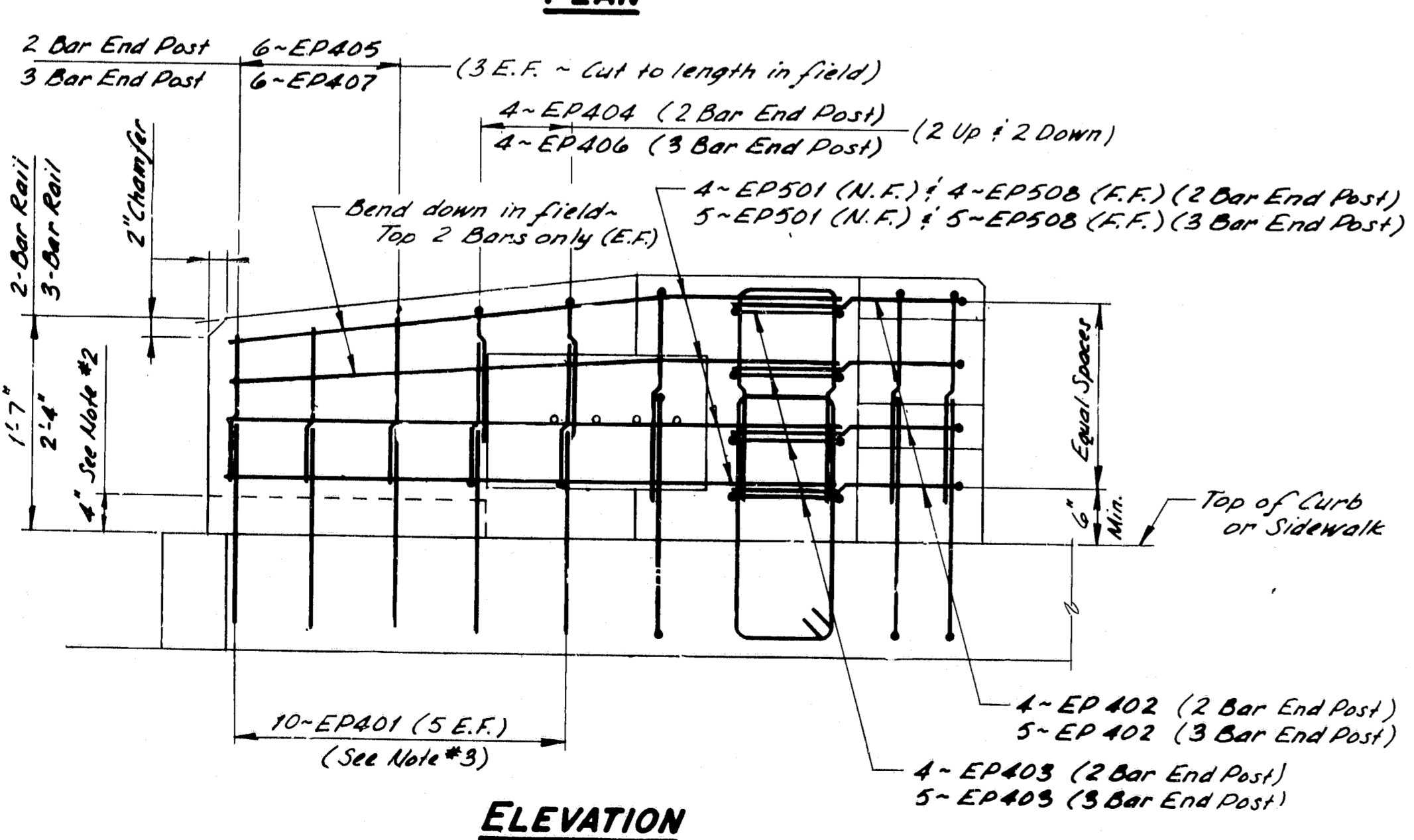
**SECTION A-A**



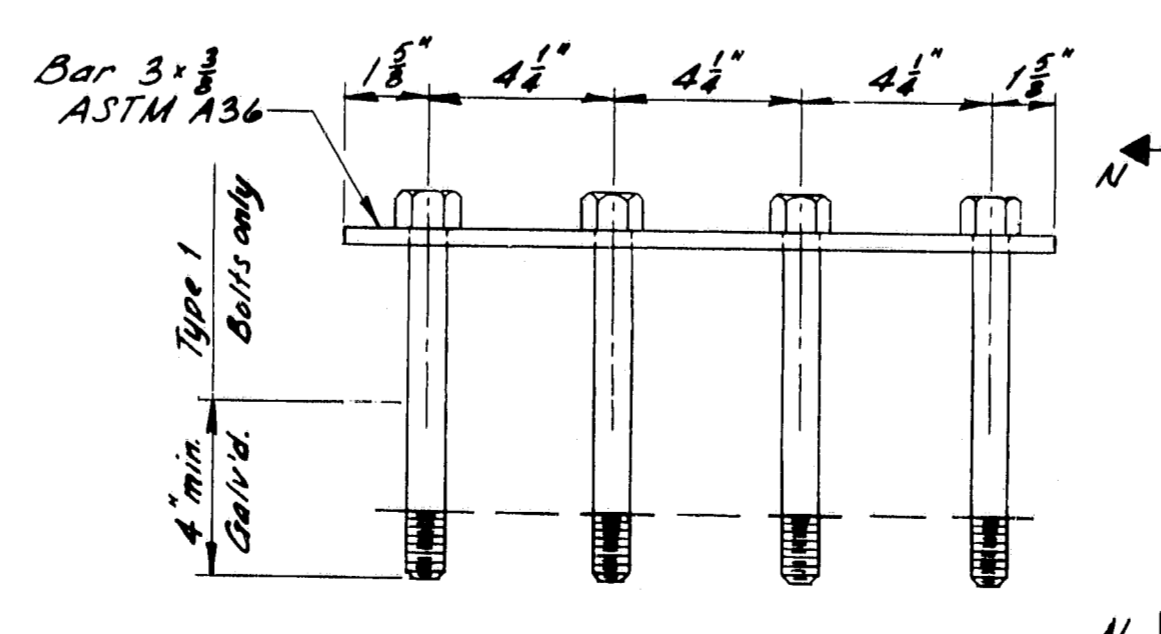
**SECTION B-B**  
2-Bar Bridge Rail (Aluminum or Steel)



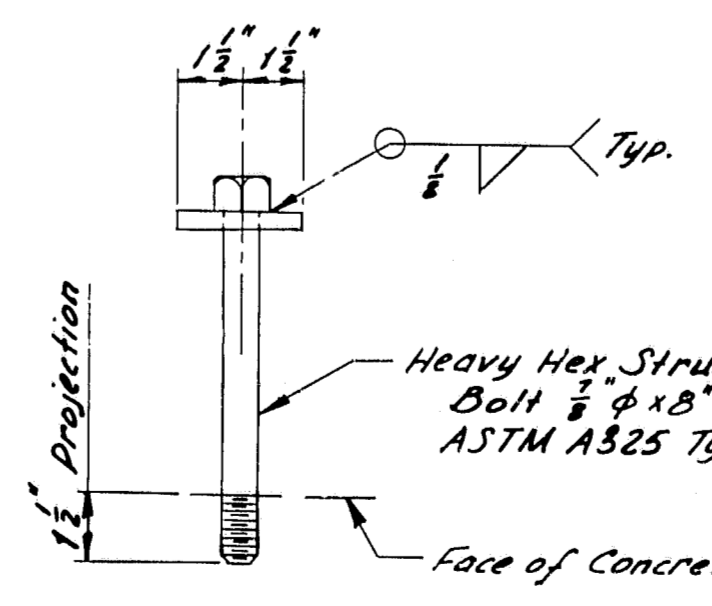
**SECTION B-B**  
3-Bar Bridge Rail (Aluminum or Steel)



**ELEVATION**



**GUARD RAIL ANCHORAGE**



**VIEW N-N**

- NOTES**
- For locations of End Posts on the structure, see Design Drawings.
  - At times, an End Post Wing may be cantilevered for all or part of its length. For details, see Design Drawings.
  - If an End Post Wing is cantilevered, bars EP401 to be omitted as needed.
  - When End Post Wing is cantilevered more than 2'-0", all #5 bars shall be replaced by #7 bars.
  - Nuts for 3/4" anchor bolts shall be incidental to Guard Rail Pay Items. Nuts shall conform to A.S.T.M. A363, Grade DH, galvanized in accordance with A.S.T.M. A153, or Grade C3, plain.
  - Additional holes in the Modified Guard Rail Sections may be made by drilling, punching, or any other method that produces a neat, clean hole of the required size. Burning of holes will not be allowed.
  - Cushion Block material shall be as specified for Wood Posts in Subsection 710.07 (a). Payment for Cushion Blocks and Lag Bolts shall be incidental to the Guard Rail Pay Items.
  - Reinforcing Steel shall have 2" min. concrete cover.
  - After installation of Guard Rail is complete, upset the thread on the anchor bolts in three places around each bolt, at the junction of the nut and the exposed thread, with a center punch or similar tool.
  - Guard Rail Anchorage shall be incidental to the applicable concrete pay item.
  - End Posts shall be constructed normal to grade unless otherwise shown on Design Drawings.

As Built 1903  
R117

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

**STANDARD DETAILS**  
(BC 120-81)

**CONCRETE END POSTS**

SHEET 13 OF 19 AUGUSTA, MAINE JUNE 1981  
Athens

**LEGEND**

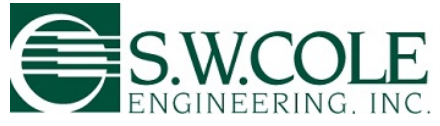
- N.F. = Near Face
- F.F. = Far Face
- E.F. = Each Face

**R89-467**

PROJECT DESIGN ENGINEER	DATE
CHECKED	BY
REVISIONS	NO.
FIELD CHANGES	







**APPENDIX D**  
**Boring Logs & Key to Soil and Rock Descriptions and Terms**

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)															
COARSE-GRAINED SOILS  (more than half of material is larger than No. 200 sieve size)	GRAVELS  (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS	GW Well-graded gravels, gravel-sand mixtures, little or no fines.	trace little some adjective (e.g. sandy, clayey)	0 - 10 11 - 20 21 - 35 36 - 50															
		(little or no fines)	GP Poorly-graded gravels, gravel sand mixtures, little or no fines.																	
	SANDS  (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM Silty gravels, gravel-sand-silt mixtures.			<b>TERMS DESCRIBING DENSITY/CONSISTENCY</b>														
		CLEAN SANDS	SW Well-graded sands, gravelly sands, little or no fines			<b>Coarse-grained soils</b> (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).														
		(little or no fines)	SP Poorly-graded sands, gravelly sand, little or no fines.			<table border="0"> <tr> <td style="text-align: center;"><u>Density of Cohesionless Soils</u></td> <td style="text-align: center;"><u>Standard Penetration Resistance N-Value (blows per foot)</u></td> </tr> <tr> <td>Very loose</td> <td>0 - 4</td> </tr> <tr> <td>Loose</td> <td>5 - 10</td> </tr> <tr> <td>Medium Dense</td> <td>11 - 30</td> </tr> <tr> <td>Dense</td> <td>31 - 50</td> </tr> <tr> <td>Very Dense</td> <td>&gt; 50</td> </tr> </table>			<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>	Very loose	0 - 4	Loose	5 - 10	Medium Dense	11 - 30	Dense	31 - 50	Very Dense	> 50
		<u>Density of Cohesionless Soils</u>	<u>Standard Penetration Resistance N-Value (blows per foot)</u>																	
Very loose	0 - 4																			
Loose	5 - 10																			
Medium Dense	11 - 30																			
Dense	31 - 50																			
Very Dense	> 50																			
SANDS WITH FINES (Appreciable amount of fines)	SM Silty sands, sand-silt mixtures	<b>Fine-grained soils</b> (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.																		
FINE-GRAINED SOILS  (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS  (liquid limit less than 50)	ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.	<u>Approximate Undrained Shear Strength (psf)</u>																	
		CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	<u>Consistency of Cohesive soils</u>																	
		OL Organic silts and organic silty clays of low plasticity.	<u>SPT N-Value (blows per foot)</u>																	
	SILTS AND CLAYS  (liquid limit greater than 50)	MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	<u>Field Guidelines</u>																	
		CH Inorganic clays of high plasticity, fat clays.	Very Soft WOH, WOR, WOP, <2 0 - 250 Fist easily penetrates																	
		OH Organic clays of medium to high plasticity, organic silts.	Soft 2 - 4 250 - 500 Thumb easily penetrates																	
HIGHLY ORGANIC SOILS	Pt Peat and other highly organic soils.	Medium Stiff 5 - 8 500 - 1000 Thumb penetrates with moderate effort																		
<b>Desired Soil Observations (in this order, if applicable):</b>				<b>Rock Quality Designation (RQD):</b>																
Color (Munsell color chart)				RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$																
Moisture (dry, damp, moist, wet)				*Minimum NQ rock core (1.88 in. OD of core)																
Density/Consistency (from above right hand side)				Correlation of RQD to Rock Mass Quality																
Texture (fine, medium, coarse, etc.)				<table border="0"> <tr> <td style="text-align: center;"><u>Rock Mass Quality</u></td> <td style="text-align: center;"><u>RQD (%)</u></td> </tr> <tr> <td>Very Poor</td> <td>≤25</td> </tr> <tr> <td>Poor</td> <td>26 - 50</td> </tr> <tr> <td>Fair</td> <td>51 - 75</td> </tr> <tr> <td>Good</td> <td>76 - 90</td> </tr> <tr> <td>Excellent</td> <td>91 - 100</td> </tr> </table>		<u>Rock Mass Quality</u>	<u>RQD (%)</u>	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100			
<u>Rock Mass Quality</u>	<u>RQD (%)</u>																			
Very Poor	≤25																			
Poor	26 - 50																			
Fair	51 - 75																			
Good	76 - 90																			
Excellent	91 - 100																			
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)				<b>Desired Rock Observations (in this order, if applicable):</b>																
Gradation (well-graded, poorly-graded, uniform, etc.)				Color (Munsell color chart)																
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)				Texture (aphanitic, fine-grained, etc.)																
Structure (layering, fractures, cracks, etc.)				Rock Type (granite, schist, sandstone, etc.)																
Bonding (well, moderately, loosely, etc.,)				Hardness (very hard, hard, mod. hard, etc.)																
Cementation (weak, moderate, or strong)				Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)																
Geologic Origin (till, marine clay, alluvium, etc.)				Geologic discontinuities/jointing:																
Groundwater level				-dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)																
<b>Maine Department of Transportation Geotechnical Section Key to Soil and Rock Descriptions and Terms Field Identification Information</b>				-spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)																
				-tightness (tight, open, or healed)																
<b>Sample Container Labeling Requirements:</b>				-infilling (grain size, color, etc.)																
				Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)																
<b>Sample Container Labeling Requirements:</b>				RQD and correlation to rock mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A																
				Recovery (inch/inch and percentage)																
<b>Sample Container Labeling Requirements:</b>				Rock Core Rate (X.X ft - Y.Y ft (min:sec))																
				WIN Blow Counts																
<b>Sample Container Labeling Requirements:</b>				Bridge Name / Town Sample Recovery																
				Boring Number Date																
<b>Sample Container Labeling Requirements:</b>				Sample Number Personnel Initials																
				Sample Depth																

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Wesserunsett Bridge #2925 carries Routes 43/150 over Wesserunsett Stream Location: Athens, Maine				Boring No.: BB-AWS-101 WIN: 22825.00							
Driller: S. W. Cole Explorations, LLC		Elevation (ft.): 345.8		Auger ID/OD: 5" Solid Stem		Operator: M. Leonard		Datum: NAVD88		Sampler: Standard Split-Spoon					
Logged By: N. Strout		Rig Type: Diedrich D-50		Hammer Wt./Fall: 140#/30"		Date Start/Finish: 7/31/2018		Drilling Method: Cased-Wash		Core Barrel: NQ2					
Boring Location: TBD		Casing ID/OD: 4" / 4.5"		Water Level*: 20.6 ft bgs (following coring)		Hammer Efficiency Factor: 0.918		Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				S <sub>u</sub> = Peak/Remolded Field Vane Undrained Shear Strength (psf) S <sub>u</sub> (lab) = Lab Vane Undrained Shear Strength (psf) q <sub>p</sub> = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N <sub>60</sub> = SPT N-uncorrected Corrected for Hammer Efficiency N <sub>60</sub> = (Hammer Efficiency Factor/60%)*N-uncorrected				T <sub>v</sub> = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.			
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows								
0	1D	24/18	0.80 - 2.80	14/11/22/17	33	50	SSA	345.2		7.5" of Pavement.	G#342676 A-1-b, SM WC=5.5%				
										Brown, damp, dense, SAND, some gravel, little silt, (Fill).					
	2D	24/12	2.80 - 4.80	12/12/10/9	22	34				Similar to above except medium dense.					
5	3D	24/19	5.00 - 7.00	7/7/5/4	12	18	15			Brown, damp, medium dense, Gravelly SAND, trace silt, with asphalt, (Fill).	G#342677 A-1-a, SW-SM WC=3.8%				
							32								
	4D	24/10	7.00 - 9.00	4/5/9/9	14	21	36			Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).					
							36								
							44								
10	5D	24/6	10.00 - 12.00	7/6/5/5	11	17	20			Similar to above except wet.					
							30								
							34								
							39								
							43								
15	6D	24/2	15.00 - 17.00	7/4/8/12	12	18	29			Similar to above.					
							48								
							104								
							78								
							79								
20	MD	24/0	20.00 - 22.00	5/2/2/1	4	6	35			No recovery, loose.					
							36								
	7D	11/3	22.00 - 22.92	5/100-5"	-	-	150	323.8	Brown, wet, loose, Silty SAND, little gravel, trace clay (Glacial Till). Wood fragments in wash water.						
							40								
25	R1	14/13	24.10 - 25.27	RQD = 0%			NQ2	321.8	Top of Bedrock at Elev 321.8 feet.						
<b>Remarks:</b> Automatic Hammer SN 367 Casing driven using Auto-Hammer Hole Abandoned at 28.2 ft, casing crooked; relocated 3 ft East. bgs = below ground surface															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 1 of 2					
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-AWS-101					

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS	<b>Project:</b> Wesserunsett Bridge #2925 carries Routes 43/150 over Wesserunsett Stream <b>Location:</b> Athens, Maine	<b>Boring No.:</b> BB-AWS-101 <b>WIN:</b> 22825.00
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<b>Driller:</b> S. W. Cole Explorations, LLC	<b>Elevation (ft.):</b> 345.8	<b>Auger ID/OD:</b> 5" Solid Stem
<b>Operator:</b> M. Leonard	<b>Datum:</b> NAVD88	<b>Sampler:</b> Standard Split-Spoon
<b>Logged By:</b> N. Strout	<b>Rig Type:</b> Diedrich D-50	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 7/31/2018	<b>Drilling Method:</b> Cased-Wash	<b>Core Barrel:</b> NQ2
<b>Boring Location:</b> TBD	<b>Casing ID/OD:</b> 4" / 4.5"	<b>Water Level*:</b> 20.6 ft bgs (following coring)

<b>Hammer Efficiency Factor:</b> 0.918	<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	
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Definitions: R = Rock Core Sample S<sub>u</sub> = Peak/Remolded Field Vane Undrained Shear Strength (psf) T<sub>v</sub> = Pocket Torvane Shear Strength (psf)  
 D = Split Spoon Sample SSA = Solid Stem Auger S<sub>u</sub>(lab) = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent  
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q<sub>0</sub> = Unconfined Compressive Strength (ksf) LL = Liquid Limit  
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value PL = Plastic Limit  
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140 lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PI = Plasticity Index  
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N<sub>60</sub> = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis  
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N<sub>60</sub> = (Hammer Efficiency Factor/60%)\*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows				
25	R2	23/23	25.30 - 27.22	RQD = 26%				317.6		Advanced by rollercone from 24.0 to 24.1 ft bgs. R1: Bedrock: Grey, aphanitic, METASILTSTONE some phyllite, moderately hard, fresh, joints are vertical (85 degrees) along bedding planes to low angle (5 degrees) along stress fracture planes, very close to close and tight, secondary mineralization infilling of calcite along bedding foliation stress planes. (Sangerville Formation, Patch Mountain Member). Rock Mass Quality = Very Poor. R1: Core Times (min:sec) 24.1-25.1 ft (4:13) 25.1-25.3 ft (1:10) 93% Recovery. R2: Bedrock: Similar to R1. Rock Mass Quality = Poor. R2: Core Times (min:sec) 25.3-26.1 ft (2:47) 26.1-27.1 ft (4:50) 27.1-27.2 ft (0:45) 100% Recovery. R2: Bedrock: Similar to R2. Rock Mass Quality = Poor. R2: Core Times (min:sec) 27.2-28.1 ft (3:01) 28.1-28.2 ft (1:46) 100% Recovery.	
	R3	12/12	27.20 - 28.20	RQD = 42%							
30											
35											
40											
45											
50											

**Remarks:**  
 Automatic Hammer SN 367  
 Casing driven using Auto-Hammer  
 Hole Abandoned at 28.2 ft, casing crooked; relocated 3 ft East.  
 bgs = below ground surface

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Wesserunsett Bridge #2925 carries Routes 43/150 over Wesserunsett Stream <b>Location:</b> Athens, Maine				<b>Boring No.:</b> BB-AWS-101A <b>WIN:</b> 22825.00							
<b>Driller:</b> S. W. Cole Explorations, LLC				<b>Elevation (ft.):</b> 345.8				<b>Auger ID/OD:</b> 5" Solid Stem							
<b>Operator:</b> M. Leonard				<b>Datum:</b> NAVD88				<b>Sampler:</b> Standard Split-Spoon							
<b>Logged By:</b> N. Strout				<b>Rig Type:</b> Diedrich D-50				<b>Hammer Wt./Fall:</b> 140#/30"							
<b>Date Start/Finish:</b> 8/1/2018				<b>Drilling Method:</b> Cased-Wash				<b>Core Barrel:</b> NQ2							
<b>Boring Location:</b> TBD				<b>Casing ID/OD:</b> 4" / 4.5"				<b>Water Level*:</b> 19.8 ft bgs (following coring)							
<b>Hammer Efficiency Factor:</b> 0.918				<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf) $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) $q_u$ = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value $N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency $N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected				$T_v$ = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows	Elevation (ft.)							
0								SSA		See BB-AWS-101 for description of strata from 0 to 23.1 ft bgs.					
5															
10															
15															
20															
25	R1	20/19	23.60 - 25.27	RQD = 0%				NQ2	322.7		Top of Bedrock at Elev 322.7 feet. Advanced by rollercone from 23.1 to 23.6 ft bgs. R1: Bedrock: Grey, aphanitic, METASILTSTONE some phyllite,				

**Remarks:**

Automatic Hammer SN 367  
 Casing driven using Auto-Hammer  
 bgs = below ground surface

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

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



<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Wesserunsett Bridge #2925 carries Routes 43/150 over Wesserunsett Stream <b>Location:</b> Athens, Maine				<b>Boring No.:</b> BB-AWS-102 <b>WIN:</b> 22825.00							
<b>Driller:</b> S. W. Cole Explorations, LLC				<b>Elevation (ft.):</b> 345.9				<b>Auger ID/OD:</b> 5" Solid Stem							
<b>Operator:</b> M. Leonard				<b>Datum:</b> NAVD88				<b>Sampler:</b> Standard Split-Spoon							
<b>Logged By:</b> N. Strout				<b>Rig Type:</b> Diedrich D-50				<b>Hammer Wt./Fall:</b> 140#/30"							
<b>Date Start/Finish:</b> 7/30/2018				<b>Drilling Method:</b> Cased-Wash				<b>Core Barrel:</b> NQ2							
<b>Boring Location:</b> TBD				<b>Casing ID/OD:</b> 4" / 4.5"				<b>Water Level*:</b> 10.0 ft bgs (following coring)							
<b>Hammer Efficiency Factor:</b> 0.918				<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf) $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) $q_p$ = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value $N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency $N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected				$T_v$ = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows								
0								345.2		8" of Pavement.					
	1D	24/19	1.00 - 3.00	14/15/11/11	26	40				Brown, damp, dense, Gravelly SAND, little silt, (Fill).					
	2D	24/16	3.00 - 5.00	14/11/15/16	26	40				Brown, damp, dense, SAND, some gravel, little silt, (Fill).					
5	3D	24/16	5.00 - 7.00	11/11/9/16	20	31				Similar to above except with brick fragments.					
	4D	24/6	7.00 - 9.00	14/13/26/56	39	60				Dark brown, moist, very dense, SAND, some silt, little gravel, with brick fragments, (Fill). Drill action suggests cobbles. Advanced by rollercone through boulder from 8.6 to 11 ft bgs.					
10															
	5D	13/1	11.10 - 12.18	31/17/100-1"	--			334.8		Brown, wet, very dense, Silty SAND, little gravel, trace clay, (Glacial Till).					
								333.8		Top of Bedrock at Elev 333.8 feet Advanced by rollercone from 12.1 to 13.0 ft bgs. R1: Bedrock: Grey, aphanitic, METASILTSTONE some phyllite, moderately hard, fresh, joints are vertical (85 degrees) along bedding planes to low angle (5 degrees) along stress fracture planes, very close to close and tight, secondary mineralization infilling of calcite along bedding foliation stress planes. (Sangerville Formation, Patch Mountain Member). Rock Mass Quality = Very Poor. R1: Core Times (min:sec) 13.0-14.0 ft (2:21) 100% Recovery. R2: Bedrock: Similar to R1. Rock Mass Quality = Excellent. R2: Core Times (min:sec) 14.0-15.0 ft (2:40) 15.0-16.0 ft (2:30) 16.0-17.0 ft (2:28) 17.0-18.0 ft (2:27) 100% Recovery. R3: Bedrock: Similar to R2. Rock Mass Quality = Fair. R3: Core Times (min:sec) 18.0-19.0 ft (2:31) 19.0-20.0 ft (2:35) 20.0-21.0 ft (2:29) 21.0-22.0 ft (2:30) 22.0-23.0 ft (2:44) 100% Recovery.					
	R1	12/12	13.00 - 14.00	RQD = 0%											
15	R2	48/48	14.00 - 18.00	RQD = 96%											
	R3	60/60	18.00 - 23.00	RQD = 73%											
20															
25								322.9							

**Remarks:**

Automatic Hammer SN 367  
Casing driven using Auto-Hammer  
bgs = below ground surface

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

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



<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS				<b>Project:</b> Wesserunsett Bridge #2925 carries Routes 43/150 over Wesserunsett Stream <b>Location:</b> Athens, Maine				<b>Boring No.:</b> BB-AWS-103 <b>WIN:</b> 22825.00							
<b>Driller:</b> S. W. Cole Explorations, LLC				<b>Elevation (ft.):</b> 345.4				<b>Auger ID/OD:</b> 5" Solid Stem							
<b>Operator:</b> M. Leonard				<b>Datum:</b> NAVD88				<b>Sampler:</b> Standard Split-Spoon							
<b>Logged By:</b> N. Strout				<b>Rig Type:</b> Diedrich D-50				<b>Hammer Wt./Fall:</b> 140#/30"							
<b>Date Start/Finish:</b> 7/31/2018 - 8/1/2018				<b>Drilling Method:</b> Cased-Wash				<b>Core Barrel:</b> NQ2							
<b>Boring Location:</b> TBD				<b>Casing ID/OD:</b> 4" / 4.5"				<b>Water Level*:</b> Soils wet below ±15 ft bgs							
<b>Hammer Efficiency Factor:</b> 0.918				<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				$S_u$ = Peak/Remolded Field Vane Undrained Shear Strength (psf) $S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf) $q_p$ = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value $N_{60}$ = SPT N-uncorrected Corrected for Hammer Efficiency $N_{60}$ = (Hammer Efficiency Factor/60%)*N-uncorrected				$T_v$ = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows								
0	MD	10/0	0.70 - 1.53	53/100-4"	--		SSA	344.8		7.5" of Pavement.					
										Sampler pushing cobble.					
	1D	15/13	2.00 - 3.25	9/13/100-3"	--					Brown, damp, very dense, SAND, little gravel, little silt, (Fill). Drill action suggests cobbles.					
5	2D	24/17	5.00 - 7.00	8/10/13/21	23	35				Brown, damp, dense, SAND, some gravel, little silt, (Fill).	G#342678 A-1-b, SM 5.8%				
10	3D	24/14	10.00 - 12.00	7/9/7/8	16	24	40			Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).					
15	4D	24/7	15.00 - 17.00	7/4/2/2	6	9	16			Brown, wet, loose, SAND, little gravel, little silt, (Fill).					
	5D	24/15	17.00 - 19.00	3/5/8/5	13	20	24			5D(A): similar to above except medium dense.					
								327.4							
										5D(B): Brown, wet, medium dense, SAND, some gravel, little silt, trace rootlets, (Glacial Till).	G#342679 A-1-b, SM 20.8%				
								326.3							
20	R1	14/5	20.20 - 21.37	RQD = 0%						Top of Bedrock at Elev 326.3 feet. Advanced by rollercone from 19.1 to 20.2 ft bgs. R1: Bedrock: Grey, aphanitic, METASILTSTONE some phyllite, moderately hard, fresh, joints are vertical (85 degrees) along bedding planes to low angle (5 degrees) along stress fracture planes, very close to close and tight, secondary mineralization infilling of calcite along bedding foliation stress planes. (Sangerville Formation, Patch Mountain Member). Rock Mass Quality = Very Poor. R1: Core Times (min:sec) 20.2-21.2 ft (1:53) 21.2-21.4 ft (1:37) 100% Recovery.					
	R2	10/10	21.40 - 22.23	RQD = 0%											
	R3	23/22	22.20 - 24.12	RQD = 74%											
25	R4	7/6	24.10 - 24.68	RQD = 0%											

**Remarks:**

Automatic Hammer SN 367  
Casing driven using Auto-Hammer  
bgs = below ground surface

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

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

<b>Maine Department of Transportation</b> Soil/Rock Exploration Log US CUSTOMARY UNITS	<b>Project:</b> Wesserunett Bridge #2925 carries Routes 43/150 over Wesserunett Stream <b>Location:</b> Athens, Maine	<b>Boring No.:</b> BB-AWS-103 <b>WIN:</b> 22825.00
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<b>Driller:</b> S. W. Cole Explorations, LLC	<b>Elevation (ft.):</b> 345.4	<b>Auger ID/OD:</b> 5" Solid Stem
<b>Operator:</b> M. Leonard	<b>Datum:</b> NAVD88	<b>Sampler:</b> Standard Split-Spoon
<b>Logged By:</b> N. Strout	<b>Rig Type:</b> Diedrich D-50	<b>Hammer Wt./Fall:</b> 140#/30"
<b>Date Start/Finish:</b> 7/31/2018 - 8/1/2018	<b>Drilling Method:</b> Cased-Wash	<b>Core Barrel:</b> NQ2
<b>Boring Location:</b> TBD	<b>Casing ID/OD:</b> 4" / 4.5"	<b>Water Level*:</b> Soils wet below ±15 ft bgs

<b>Hammer Efficiency Factor:</b> 0.918	<b>Hammer Type:</b> Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>
--	--

Definitions:  
 D = Split Spoon Sample  
 MD = Unsuccessful Split Spoon Sample Attempt  
 U = Thin Wall Tube Sample  
 MU = Unsuccessful Thin Wall Tube Sample Attempt  
 V = Field Vane Shear Test, PP = Pocket Penetrometer  
 MV = Unsuccessful Field Vane Shear Test Attempt  
 R = Rock Core Sample  
 SSA = Solid Stem Auger  
 HSA = Hollow Stem Auger  
 RC = Roller Cone  
 WOH = Weight of 140 lb. Hammer  
 WOR/C = Weight of Rods or Casing  
 WO1P = Weight of One Person  
 S<sub>u</sub> = Peak/Remolded Field Vane Undrained Shear Strength (psf)  
 S<sub>u(lab)</sub> = Lab Vane Undrained Shear Strength (psf)  
 q<sub>u</sub> = Unconfined Compressive Strength (ksf)  
 N-uncorrected = Raw Field SPT N-value  
 Hammer Efficiency Factor = Rig Specific Annual Calibration Value  
 N<sub>60</sub> = SPT N-uncorrected Corrected for Hammer Efficiency  
 N<sub>60</sub> = (Hammer Efficiency Factor/60%)\*N-uncorrected  
 T<sub>v</sub> = Pocket Torvane Shear Strength (psf)  
 WC = Water Content, percent  
 LL = Liquid Limit  
 PL = Plastic Limit  
 PI = Plasticity Index  
 G = Grain Size Analysis  
 C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows/(6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N <sub>60</sub>	Casing Blows				
25	R5	30/29	24.70 - 27.20	RQD = 53%				315.2		R2: Bedrock: Similar to R1. Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 21.4-22.2 ft (4:51) 100% Recovery. R3: Bedrock: Similar to R2. Rock Mass Quality = Fair. R3:Core Times (min:sec) 22.2-23.2 ft (3:10) 23.2-24.1 ft (2:55) 96% Recovery. R4: Bedrock: Similar to R3. Rock Mass Quality = Very Poor. R4:Core Times (min:sec) 24.1-24.2 ft (0:17) 24.2-24.7 ft (4:37) 96% Recovery. R5: Bedrock: Similar to R4. Rock Mass Quality = Fair. R5:Core Times (min:sec) 24.7-25.2 ft (1:45) 25.2-26.2 ft (4:11) 26.2-27.2 ft (5:06) 97% Recovery. R6: Bedrock: Similar to R5. Rock Mass Quality = Very Poor. R6:Core Times (min:sec) 27.2-27.9 ft (4:18) 100% Recovery. R7: Bedrock: Similar to R6. Rock Mass Quality = Very Poor. R7:Core Times (min:sec) 27.9-28.2 ft (0:48) 28.2-29.2 ft (3:20) 29.2-30.2 ft (5:01) 96% Recovery.	
	R6	8/8	27.20 - 27.87	RQD = 0%							
	R7	28/27	27.90 - 30.23	RQD = 21%							
30											
35											
40											
45											
50											

**Remarks:**  
 Automatic Hammer SN 367  
 Casing driven using Auto-Hammer  
 bgs = below ground surface

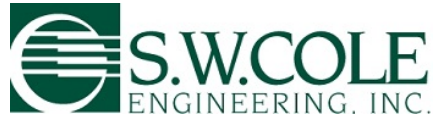
WIN # 22825 8-1-18  
 Athens-Wesserunsett Bridge # 2925  
 Route 43 over Wesserunsett Stream

BB-AWS-103	recov.	RQD
R1 20.2-21.4'	5"	0
R2 21.4-22.2'	10"	0
R3 22.2-24.1'	22"	17/23" = 74%
R4 24.1-24.7'	6"	0
R5 24.7-27.2'	29"	16/30" = 53%
R6 27.2-27.9'	8"	0
R7 27.9-30.2'	27"	6/28" = 21%

BB-AWS-101A	recov.	RQD
R1 23.6-25.3'	19"	0
R2 25.3-30.2'	58"	44/59" = 75%
R3 30.2-33.6'	41"	26/41" = 63%

BB-AWS-103	BB-AWS-103	BB-AWS-103	BB-AWS-103	BB-AWS-103
R1; 20.2-21.4'	R2; 20.4-22.2'	R3; 22.2-24.1'	R4; 24.1-24.7'	R5; 24.7-25.9'
BB-AWS-103	BB-AWS-103	BB-AWS-103, R7; 27.9-30.2'		
R5; 25.9-27.2'	R6; 27.2-27.9'	BB-AWS-101A; R2, 25.3-28.6'		
BB-AWS-101A	BB-AWS-101A; R3; 30.2-33.6'			
R1; 23.6-25.3'	BB-AWS-101A; R2, 28.6-30.2'			

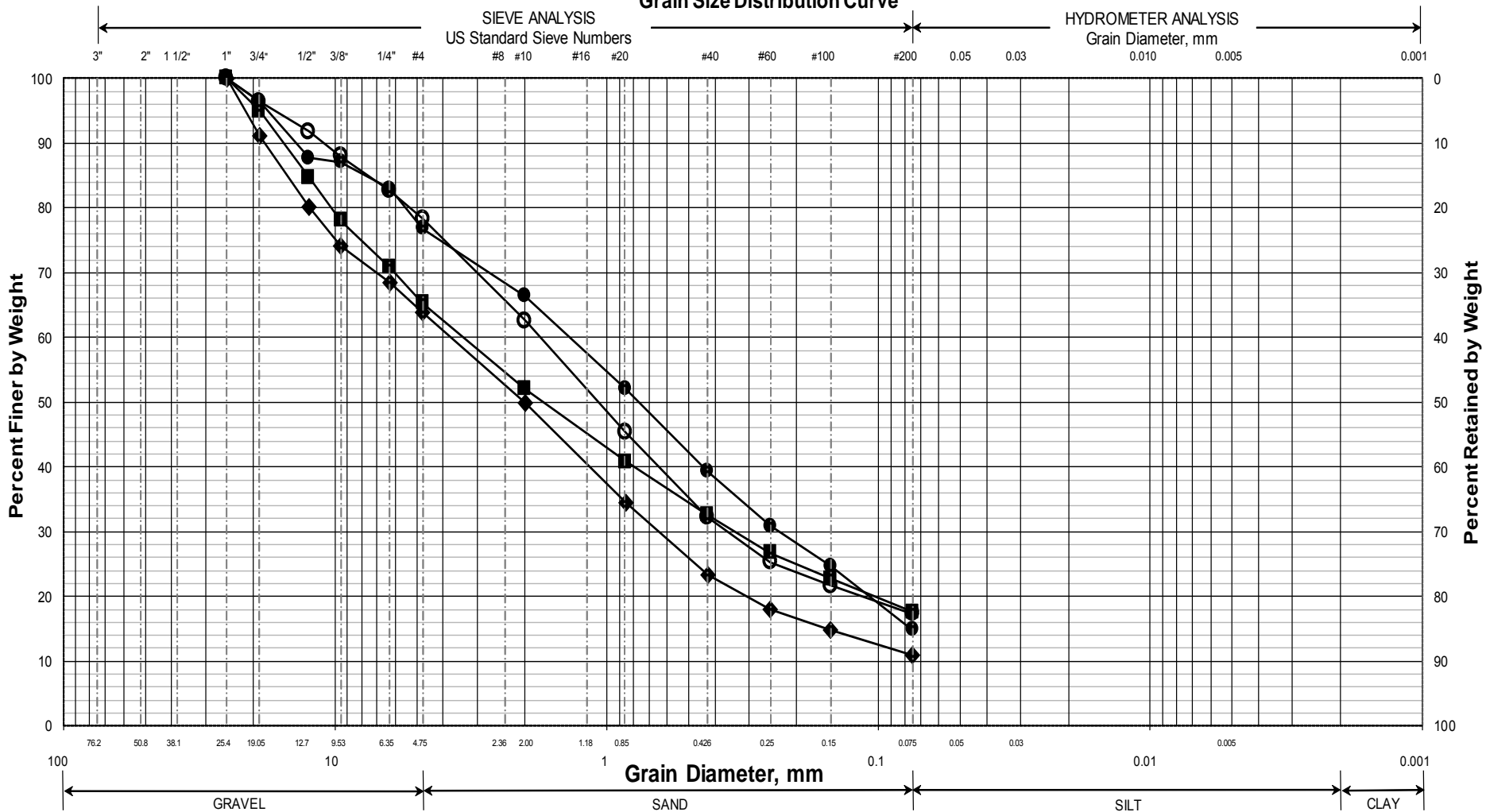




**APPENDIX E**  
**Laboratory Test Results**



## Maine Department of Transportation Grain Size Distribution Curve



### UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	WC, %	LL	PL	PI
○	BB-AWS-101/1D	TBD		0.8-2.8	SAND, some gravel, little silt.	5.5			
◆	BB-AWS-101/3D	TBD		5.0-7.0	Gravelly SAND, trace silt.	3.8			
■	BB-AWS-103/2D	TBD		5.0-7.0	SAND, some gravel, little silt.	5.8			
●	BB-AWS-103/5D(B)	TBD		18.0-19.0	SAND, some gravel, little silt.	20.8			
▲									
X									

WIN
022825.00
Town
Athens
Reported by/Date
WHITE, TERRY A      9/24/2018

WIN # 22825.00  
 Athens - Wasserunsicht Bridge (# 2925)  
 Rt. 43 over Wasserunsicht Stream

BB-AWS-102  
 cases RBD

R1 13.0 → 14.0' 12" 0  
 R2 14.0 → 18.0' 48" 46/48 = 96%  
 R3 18.0 → 23.0' 60" 44/60 = 73%

BB-AWS-101  
 D

R1 24.1 → 25.3' 13" 6/23 = 26%  
 R2 25.3 → 27.2' 23" 5/12 = 42%  
 R3 27.2 → 28.2' 12" 0

BB-AWS-102	BB-AWS-102, R2, 14 → 18'
R1, 13 → 14'	
BB-AWS-102, R3, 18 → 23'	
BB-AWS-101	BB-AWS-101
R1, 24.1 → 25.3'	R2, 25.3 → 27.2'
	R3, 27.2 → 28.2'





**APPENDIX F**  
**Calculations**

## Evaluation of Earth Pressure Coefficients for Design

### Assumed Backfill Values

MaineDOT BDG Section 3.6.1 - Soil Type 4

$$\gamma_1 := 125 \text{ } \mathit{pcf}$$

Unit Weight

$$\phi_1 := 32 \text{ } \mathit{deg}$$

Friction Angle

$$c_1 := 0 \text{ } \mathit{psf}$$

Cohesion

### Wall Parameters

$$\theta := 90 \text{ } \mathit{deg}$$

Angle of back face of wall  
(from horizontal)

$$\delta := \frac{2}{3} \cdot \phi_1 \quad \delta = 21.3 \text{ } \mathit{deg}$$

Interface Friction between Fill and Wall  
LRFD Table 3.11.5.3-1,  $\delta = 19$  to  $24$  deg

$$\beta := \begin{bmatrix} 0 \\ 26.4 \end{bmatrix} \mathit{deg}$$

Continous Backslope Angle(s)  
(from horizontal)

### Coulomb Active Earth Pressure Coefficient (LRFD Eq. 3.11.5.3-1 and 3.11.5.3-2)

$$\Gamma_a := \left( 1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}} \right)^2$$

$$k_a := \frac{\sin(\theta + \phi_1)^2}{\Gamma_a \cdot (\sin(\theta))^2 \cdot (\sin(\theta - \delta))}$$

$$k_a = \begin{bmatrix} 0.28 \\ 0.45 \end{bmatrix} \text{ for } \beta = \begin{bmatrix} 0 \\ 26.4 \end{bmatrix} \mathit{deg}$$

### At-Rest Earth Pressure Coefficient (LRFD Eq. 3.11.5.2-1)

$$k_o := 1 - \sin(\phi_1)$$

$$k_o = 0.47$$

**Estimated Frost Penetration Depth**

Based on MaineDOT Bridge Design Guide Section 5.2.1

Site Location: Athens, Maine

Soil Conditions: SAND, some gravel, little to trace silt  
(Coarse Grained)

**Step 1.** From Figure 5-1: Design Freezing Index (DFI) = ±1850 freezing degree-days

**Step 2.** From laboratory testing moisture contents (w) range from 3.8 to 5.8% within upper 10 feet of ground surface.

Assume w=10%

**Step 3.** From Table 5-1: Interpolate frost penetration for w = 10%

$$DFI := 1850$$

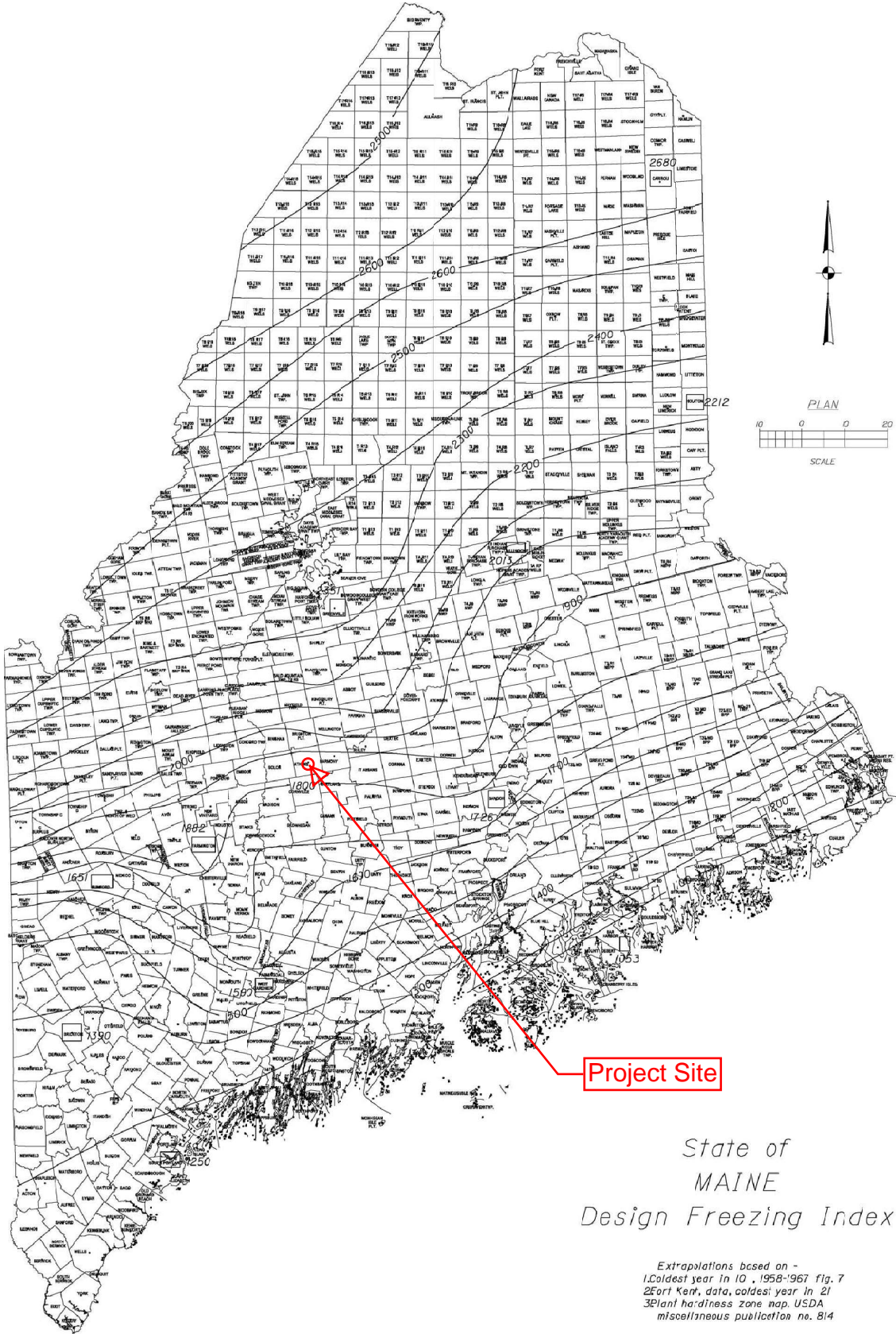
$$DFI_1 := 1800 \quad d_1 := 90.1 \text{ in}$$

$$DFI_2 := 1900 \quad d_2 := 92.6 \text{ in}$$

$$d_{frost} := d_1 + (d_2 - d_1) \cdot \left( \frac{DFI - DFI_1}{DFI_2 - DFI_1} \right) = 91.4 \text{ in}$$

$$d_{frost} = 7.6 \text{ ft}$$

Figure 5-1 Maine Design Freezing Index Map



Project Site

State of  
MAINE  
Design Freezing Index

Extrapolations based on -  
1) Coldest year in 10, 1958-1967 fig. 7  
2) Fort Kent, data, coldest year in 21  
3) Plant hardiness zone map, USDA  
miscellaneous publication no. 814

## 5.2 General

### 5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

**Table 5-1 Depth of Frost Penetration**

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

## CHAPTER 5 - SUBSTRUCTURES

- Notes:
1.  $w$  = water content
  2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

**Determine Seismic Site Classification per AASHTO LRFD Table C3.10.3.1-1 - Method B**

Data From Boring BB-AWS-101

Layer No.	Layer Description	Depth Range (ft)		N <sub>60</sub> values recorded within layer								Average N <sub>60</sub> value	Layer Thickness	d <sub>i</sub> /N <sub>i</sub>
		Top	End									N <sub>i</sub>	d <sub>i</sub>	
1	Overburden	0	24	50	34	18	21	17	18	9	23.9	24	1.01	
3	Bedrock	24	100	100							100.0	76	0.76	
<b>Σ =</b>											<b>100</b>	<b>1.77</b>		

**N<sub>bar</sub> = d<sub>i</sub>/d<sub>i</sub>/N<sub>i</sub> = 56.63**  
**Site Class = C**

Data From Borings BB-AWS-102

Layer No.	Layer Description	Depth Range (ft)		N <sub>60</sub> values recorded within layer								Average N <sub>60</sub> value	Layer Thickness	d <sub>i</sub> /N <sub>i</sub>
		Top	End									N <sub>i</sub>	d <sub>i</sub>	
1	Overburden	0	12.1	40	40	31	60				42.8	12.1	0.28	
3	Bedrock	12.1	100	100							100.0	87.9	0.88	
<b>Σ =</b>											<b>100</b>	<b>1.16</b>		

**N<sub>bar</sub> = d<sub>i</sub>/d<sub>i</sub>/N<sub>i</sub> = 86.06**  
**Site Class = C**

Data From Borings BB-AWS-103

Layer No.	Layer Description	Depth Range (ft)		N <sub>60</sub> values recorded within layer								Average N <sub>60</sub> value	Layer Thickness	d <sub>i</sub> /N <sub>i</sub>
		Top	End									N <sub>i</sub>	d <sub>i</sub>	
1	Overburden	0	19.1	100	100	24	9	20			50.6	19.1	0.38	
3	Bedrock	19.1	100	100							100.0	80.9	0.81	
<b>Σ =</b>											<b>100</b>	<b>1.19</b>		

- Notes:**
1. Refusal N60 values taken as N=100
  2. N60 value for bedrock taken as N=100

**N<sub>bar</sub> = d<sub>i</sub>/d<sub>i</sub>/N<sub>i</sub> = 84.28**  
**Site Class = C**

# USGS Design Maps Summary Report

## User-Specified Input

**Report Title** AWS\_2925

Wed September 12, 2018 19:58:52 UTC

**Building Code Reference Document** 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design  
(which utilizes USGS hazard data available in 2002)

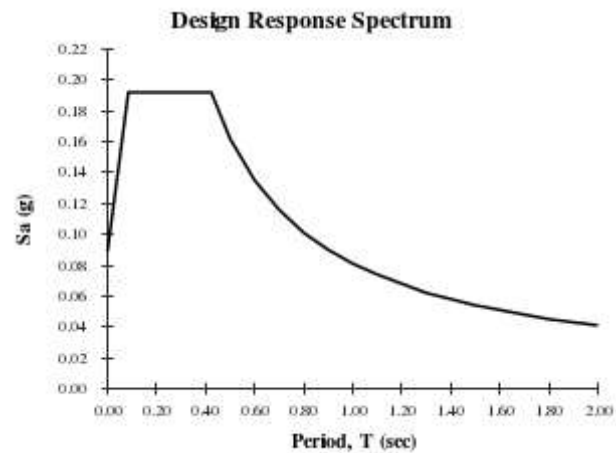
**Site Coordinates** 44.92316°N, 69.67457°W

**Site Soil Classification** Site Class C – “Very Dense Soil and Soft Rock”



## USGS-Provided Output

<b>PGA</b> = 0.074 g	<b>A<sub>s</sub></b> = 0.089 g
<b>S<sub>s</sub></b> = 0.160 g	<b>S<sub>DS</sub></b> = 0.192 g
<b>S<sub>1</sub></b> = 0.048 g	<b>S<sub>D1</sub></b> = 0.081 g



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From [Figure 3.4.1-2](#)<sup>[1]</sup> PGA = 0.074 g

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From [Figure 3.4.1-3](#)<sup>[2]</sup>  $S_s = 0.160$  g

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From [Figure 3.4.1-4](#)<sup>[3]</sup>  $S_1 = 0.048$  g

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## Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1-1 Site Class Definitions

<b>SITE CLASS</b>	<b>SOIL PROFILE NAME</b>	<b>Soil shear wave velocity, <math>\bar{v}_s</math>, (ft/s)</b>	<b>Standard penetration resistance, <math>\bar{N}</math></b>	<b>Soil undrained shear strength, <math>\bar{s}_u</math>, (psf)</b>
A	Hard rock	$\bar{v}_s > 5,000$	N/A	N/A
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	N/A	N/A
C	Very dense soil and soft rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$>2,000$ psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1,200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$<1,000$ psf
E	—	Any profile with more than 10 ft of soil having the characteristics: <ol style="list-style-type: none"> <li>1. Plasticity index <math>PI &gt; 20</math>,</li> <li>2. Moisture content <math>w \geq 40\%</math>, and</li> <li>3. Undrained shear strength <math>\bar{s}_u &lt; 500</math> psf</li> </ol>		
F	—	Any profile containing soils having one or more of the following characteristics: <ol style="list-style-type: none"> <li>1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.</li> <li>2. Peats and/or highly organic clays (<math>H &gt; 10</math> feet of peat and/or highly organic clay where <math>H</math> = thickness of soil)</li> <li>3. Very high plasticity clays (<math>H &gt; 25</math> feet with plasticity index <math>PI &gt; 75</math>)</li> <li>4. Very thick soft/medium stiff clays (<math>H &gt; 120</math> feet)</li> </ol>		

For SI: 1ft/s = 0.3048 m/s 1lb/ft<sup>2</sup> = 0.0479 kN/m<sup>2</sup>

### Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for  $F_{pga}$ )—Values of  $F_{pga}$  as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of PGA

**For Site Class = C and PGA = 0.074 g,  $F_{PGA} = 1.200$**

Table 3.4.2.3-1 (for  $F_a$ )—Values of  $F_a$  as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of  $S_s$

**For Site Class = C and  $S_s = 0.160$  g,  $F_a = 1.200$**

Table 3.4.2.3-2—Values of  $F_v$  as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of  $S_1$

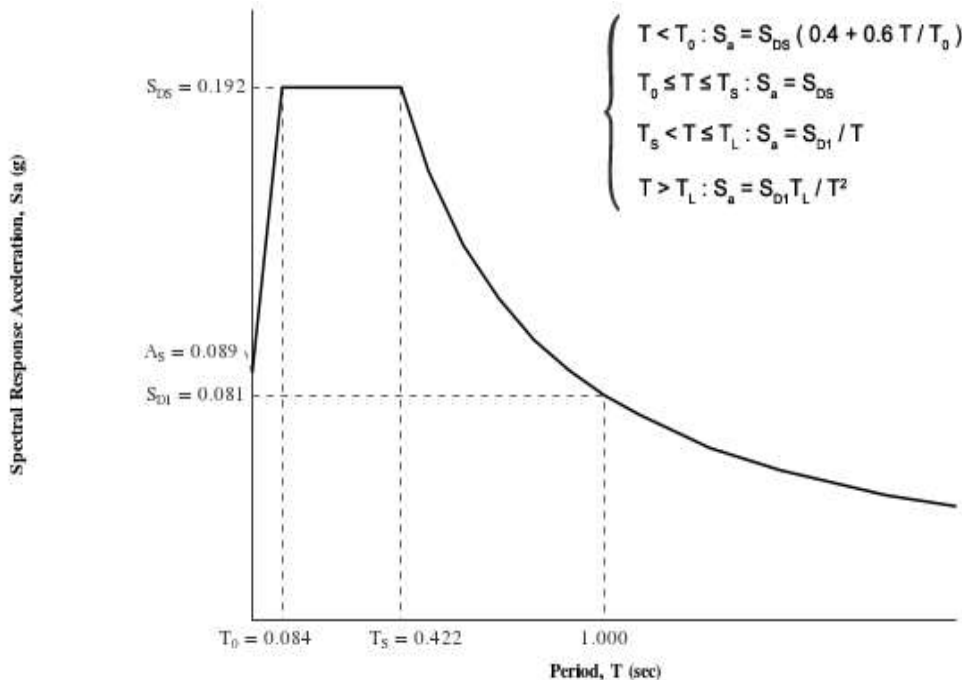
For Site Class = C and  $S_1 = 0.048$  g,  $F_v = 1.700$

**Equation (3.4.1-1):**  $A_S = F_{PGA} \text{ PGA} = 1.200 \times 0.074 = 0.089 \text{ g}$

**Equation (3.4.1-2):**  $S_{DS} = F_a S_S = 1.200 \times 0.160 = 0.192 \text{ g}$

**Equation (3.4.1-3):**  $S_{D1} = F_v S_1 = 1.700 \times 0.048 = 0.081 \text{ g}$

Figure 3.4.1-1: Design Response Spectrum



## Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

<b>VALUE OF <math>S_{D1}</math></b>	<b>SDC</b>
<b><math>S_{D1} &lt; 0.15g</math></b>	A
<b><math>0.15g \leq S_{D1} &lt; 0.30g</math></b>	B
<b><math>0.30g \leq S_{D1} &lt; 0.50g</math></b>	C
<b><math>0.50g \leq S_{D1}</math></b>	D

**For  $S_{D1} = 0.081 g$ , Seismic Design Category = A**

Seismic Design Category  $\equiv$  "the design category in accordance with Table 3.5-1" = A

---

## References

1. *Figure 3.4.1-2*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf>
2. *Figure 3.4.1-3*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf>
3. *Figure 3.4.1-4*: <https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf>