

# Bridge Load Rating

*Prepared for*

## Maine Department of Transportation

Bridge No. 1563

Brewer

Green Point Rd

OVER

I-395

Date of Inspection: 1/5/2015

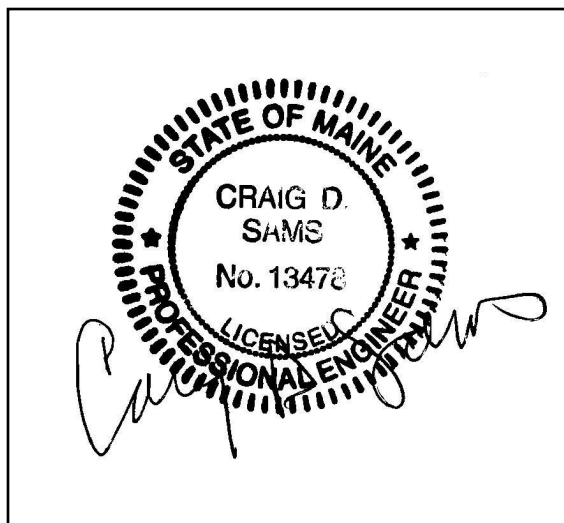
Date of Rating: 7/15/2016

Prepared By: Matthew M. Styckiewicz, P.E.

Checked By: Craig D. Sams, P.E.

Q.C. Review By: Charlie M. Roberts, P.E.

CEC – Childs Engineering Corporation





Bridge No: 1563  
 Town/City: Brewer  
 Route Carried: Green Point Rd  
 Crosses: I-395

Owner: MaineDOT  
 Maintainer: MaineDOT  
 Year Built: 1984  
 Year(s) Rebuilt/Rehab: N/A

## **SUMMARY OF BRIDGE RATING**

VEHICLE TYPE		RF	RT (TONS)	POSTING LOAD (TONS)
HL-93	INVENTORY	0.766		
	OPERATING	0.993		
CONFIGURATION 1		1.682	84.10	OK
CONFIGURATION 2		1.782	83.75	OK
CONFIGURATION 3		1.822	80.17	OK
CONFIGURATION 4		1.826	80.34	OK
CONFIGURATION 5		1.835	80.74	OK
CONFIGURATION 6		2.030	77.04	OK
CONFIGURATION 7		2.589	76.38	OK
CONFIGURATION 8		4.110	76.86	OK
2 Legal Loads in Same Direction		1.371	60.32	OK

### **Group 1 Posting Analysis (Configuration 1)**

Governing Posting: 84.1  
 Governing Load Model: Configuration 1

### **Group 2 Posting Analysis (Configuration 2 - 5)**

Governing Posting: 80.17  
 Governing Load Model: Configuration 3

### **Group 3 Posting Analysis (Configuration 6 - 8)**

Governing Posting: 76.38  
 Governing Load Model: Configuration 7

*Please Check all the boxes that apply:*

### **LRFR Evaluation Factors:**

Live Load Distribution Factor: Varies (see calculations)  
 Legal Load Factor: 1.4  
 Impact Factor: 1.33  
 Governing Condition Factor,  $\phi_c$ : 1.0  
 System Factor,  $\phi_s$ : 1.0  
 ADTT (one-way): 83

- ☐ Bridge load rating is governed by substructure rating
- ☐ Connection control the load rating
- ☒ Exterior girder control load rating
- ☒ As-built load rating
- ☐ As-inspected load rating
- ☐ One Lane Loaded
- ☒ Advanced Analysis Used
- ☐ Actual Measurements Taken
- ☐ Finite Fatigue life \_\_\_\_ years



## **BREAKDOWN OF BRIDGE RATING**

Town/City: Brewer  
 Bridge No.: 1563

Route Carried: Green Point Road  
 Crosses: I-395

## **LOAD RATING POINTS OF INTEREST**

<u><b>Bridge Component</b></u>	HL-93		MaineDOT Truck Configurations									
	Inv 72.0 kip	Oper 72.0 kip	1 100.0	2 94.0 kip	3 88.0 kip	4 88.0 kip	5 88.0 kip	6 75.9 kip	7 59.0 kip	8 37.4 kip	2 Legal Loads Same Direction	
Interior Girder Strength I Positive Bending	1.570	2.036										
Interior Girder Service II Positive Bending	1.558											
Interior Girder Strength I Negative Bending	0.917	1.189	2.015	2.134	2.182	2.187	2.198	2.431	3.100	4.923		
Interior Girder Service II Negative Bending	1.428											
Exterior Girder Strength I Positive Bending	1.587	2.057										
Exterior Girder Service II Positive Bending	1.574											
Exterior Girder Strength I Negative Bending	0.766	0.993	1.682	1.782	1.822	1.826	1.825	2.030	2.589	4.110		
Exterior Girder Service II Negative Bending	1.189											
Girder Strength I Shear - Interior Support	3.965	5.140										
Girder Strength I Shear - End Support	4.133	5.357										
Interior Girder Two Legal Load spaced @ 30' Negative Bending											1.642	
Exterior Girder Two Legal Load spaced @ 30' Negative Bending											1.371	
[COMPONENT] [LIMIT STATE] [ANALYZED CONDITION]												
CONTROLLING RATING FACTORS (STRENGTH I)	0.766	0.993	1.682	1.782	1.822	1.826	1.825	2.030	2.589	4.110	1.371	



## **DESCRIPTION OF BRIDGE**

Bridge Number:	1563
Owner:	MaineDOT
Maintained By:	MaineDOT
Location:	Brewer
Route Carried:	Green Point Rd
Featured Intersection:	I-395
Latest NBI Inspection Date:	1/5/2015
Field Verification Date:	7/5/2016
Date of Construction:	1984
Bridge Type:	Steel Girder with Composite Concrete Deck
Material Properties:	Fy = 50 ksi (A572) Fy (rebar) = 40 ksi, f'c = 3.0 ksi
Original Design Loading:	HS-25
Date(s) of Rebuild/Rehab:	N/A
Description of Rebuild/Rehab:	N/A
Posting:	N/A
Superstructure:	Steel Girder with Concrete Deck
Substructure:	Reinforced Concrete Abutments
Bearings:	Fixed Hinged Pedestal with Expansion Roller Pedestals
Bridge Spans:	Both Spans = 100'
Bridge Skew:	18°03'00"
Bridge Width:	36.7 ft
Roadway Surface:	33.0 ft inside curb to inside curb
Curbs:	Concrete with Granite Block
Sidewalk/Walkway/Median:	No walkway
Utilities:	No utilities
Bridge Railing:	Steel posts and railings in concrete curb
Approach Railings:	Steel Guardrail
Wearing Surface Condition:	
Bridge Railing Condition:	-- from inspection report dated 1/5/2015 --
Deck Condition:	Deck – 6 SATISFACTORY
Beam Condition:	Superstructure – 7 GOOD
Bearing Condition:	Substructure – 5 FAIR
Abutment Condition:	
Pier Condition:	



# **NOTES AND ASSUMPTIONS**

## **References Used:**

- *The Manual for Bridge Evaluation (MBE)*, Second Edition, 2011 (MBE) w/ 2013 Interims
- *AASHTO LRFD Bridge Design Specifications*, Sixth Edition, 2012
- MaineDOT Load Rating Guide, April 2015

## **General Notes:**

This load rating was performed in accordance with MaineDOT guidelines.

Bridge No. 1563 is a continuous 2 span bridge with 6 steel W-shape girders which act compositely with a steel reinforced concrete deck. The concrete deck thickness is 8.5 inches with various depth beam haunches and a 3 inch thick bituminous wearing surface. The bridge has concrete curbs on both sides of the roadway with no walkway. Both sides have a steel guardrail.

The bridge has an overall width of 36.7 ft with a roadway width of 33.0 ft carrying 2 lanes of traffic. The current ADT of the bridge was recorded as 2130 from the previous inspection. The ADTT for the bridge was calculated at 83 with a Truck ADT of 5%.

The bridge was evaluated at the Inventory and Operating levels for the Strength I limit state, as well as the Service II limit state. The superstructure has an inventory and operating rating factor of less than 1 for the AASHTO HL-93 load condition during negative moments at the interior support. Due to the inventory rating factor of less than 1 at the HL-93 condition, it was necessary to rate the structure for the MaineDOT legal loads. The superstructure was rated for the MaineDOT legal loads including 2 legal loads spaced 30 feet apart traveling in the same direction.

STAAD.Pro V8i was used to generate moment and shear forces in the girders due to the various load conditions. The bridge girders and deck were then evaluated at those forces using the distribution factors and capacity calculations provided by the AASHTO LRFD Bridge Design Specifications.

## **Condition Factors:**

The condition factor used for this bridge was 1.0 due to the “Good” rating that the superstructure received in the most recent (2015) inspection report.

## **Assumptions:**

- Steel specified as A572 steel, was  $F_y = 50$  ksi
- Concrete strength specified as Grade “A” ( $f'_c$ ) assumed to be 4.0 ksi
- Reinforcing steel yield strength ( $F_y$ ) assumed to be 60 ksi
- Superimposed dead loads such as wearing surface and guardrails distributed evenly between all girders.



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## APPENDIX A

### Structural Inventory and Appraisal Sheets



## Structure Inventory and Appraisal Sheet (English Units)

Bridge Key: 1563	Agency ID: 1563	SR: 86.6	SD/FO: ND
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### IDENTIFICATION

State 1: 23 Maine      Struc Num 8: 1563  
Facility Carried 7: GREEN POINT RD      Location 9: 0.4 MI S OF ROUTE 1A  
Rte.(On/Under)5A: Route On Structure      Rte. Signing Prefix 5B: 5 City Street  
Level of Service 5C: 0 None of the below      Rte. Number 5D: 00000  
Directional Suffix 5E: 0 N/A (NBI)      % Responsibility : 0  
SHD District 2: 04 Eastern      County Code 3: 019 Penobscot  
Place Code 4: 19050 Brewer      Mile Post 11: 0.520 mi  
Feature Intersected 6: I-395  
Latitude 16: 44d 46' 26"      Longitude 17: 068d 44' 46"  
Border Bridge Code 98: Not Applicable (P)  
Border Bridge Number 99: n/a

### INSPECTION

Frequency 91: 24 months      Inspection Date 90: 1/5/2015      Next Inspection: 01/05/2017  
FC Frequency 92A: NA      FC Inspection Date 93A: NA      Next FC Inspection: NA  
UW Frequency 92B: NA      UW Inspection Date 93B: NA      Next UW Inspection: NA  
SI Frequency 92C: NA      SI Date 93C: NA      Next SI: NA  
Element Frequency: 24 months      Element Inspection Date: 01/05/2015      Next Elem. Insp. Due:01/05/2017

### STRUCTURE TYPE AND MATERIALS

Number of Approach Spans 46: 0      Number of Spans Main Unit 45: 2  
Main Span Material/Design 43A/B:  
4 Steel Continuous      02 Stringer/Girder  
Deck Type 107: 1 Concrete-Cast-in-Place  
Wearing Surface 108A: 6 Bituminous  
Membrane 108B: 2 Preformed Fabric  
Deck Protection 108C: None

### CLASSIFICATION

Defense Highway 100: 0 Not a STRAHNET hwy      Parallel Structure 101: No || bridge exists  
Direction of Traffic 102: 2 2-way traffic      Temporary Structure 103: Not Applicable (P)  
Highway System 104: 0 Not on NHS      NBIS Length 112: Long Enough  
Toll Facility 20: 3 On free road      Functional Class 26: 19 Urban Local  
Defense Hwy 110: 0 Not a STRAHNET hwy      Historical Significance 37: 4 Hist sign not determin  
Owner 22: 01 State Highway Agency  
Custodian 21: 01 State Highway Agency

### AGE AND SERVICE

Year Built 27: 1984      Year Reconstructed 106: 0  
Type of Service on 42A: 1 Highway  
Type of Service under 42B: 1 Highway  
Lanes on 28A: 2      Lanes Under 28B: 4      Detour Length 19: 2.5 mi  
ADT 29: 2,130      Truck ADT 109: 5 %      Year of ADT 30: 2014

### CONDITION

Deck 55: 6 Satisfactory      Super 59: 7 Good      Sub 60: 5 Fair  
Culvert 62: N N/A (NBI)      Channel/Channel Protection 61: N N/A (NBI)

### GEOMETRIC DATA

Length Max Span 48: 100.0 ft      Structure Length 49: 207.0 ft  
Curb/Sdwk Width L 50A: 0.6 ft      Curb/Sidewalk Width R 50B: 0.6 ft  
Width Curb to Curb 51: 33.0 ft      Width Out to Out 52: 36.7 ft  
Approach Roadway Width 32: 32.0 ft      Median 33: 0 No median (w/ shoulders)  
Deck Area: 7,596.8 sq. ft  
Skew 34: 18.00 °      Structure Flared 35: 0 No flare  
Vertical Clearance 10: 99.99 ft      Horiz. Clearance 47: 33.00 ft  
Minimum Vertical Clearance Over Bridge 53: 327.8 ft  
Minimum Vertical Underclearance Reference 54A: H Hwy beneath struct  
Minimum Vertical Underclearance 54B: 17.6 ft  
Minimum Lateral Underclearance Reference R 55A: H Hwy beneath struct  
Minimum Lateral Underclearance R 55: 18.0 ft  
Minimum Lateral Underclearance L 56: 6.9 ft

### LOAD RATING AND POSTING

Inventory Rating Method 65: 1 LF Load Factor      Operating Rating Method 63: 1 LF Load Factor  
Inventory Rating 66: HS25.5      Operating Rating 64: HS42.8  
Design Load 31: MS 22.5 or greater      Posting 70: 5 At/Above Legal Loads  
Posting status 41: A Open, no restriction

### APPRAISAL

Bridge Rail 36A: 1 Meets Standards      Approach Rail 36C: 1 Meets Standards  
Transition 36B: 1 Meets Standards      Approach Rail Ends 36D: 1 Meets Standards  
Str. Evaluation 67: 5      Deck Geometry 68: 4 Tolerable  
Underclearance, Vertical and Horizontal 69: 6 Equal Minimum  
Waterway Adequacy 71: N Not applicable      Approach Alignment 72: 6 Equal Min Criteria  
Scour Critical 113: N Not Over Waterway

### PROPOSED IMPROVEMENTS

Bridge Cost 94: NA      Type of Work 75: Unknown (P)  
Roadway Cost 95: Unknown      Length of Improvement 76:  
Total Cost 96: Unknown      Future ADT 114: 2,982  
Year of Cost Estimate 97: Unknown      Year of Future ADT 115: 2034

### NAVIGATION DATA

Navigation Control 38: N NA-no waterway  
Vertical Clearance 39: 0.0 ft      Horizontal Clearance 40: 0.0 ft  
Pier Protection 111: Not Applicable (P)      Lift Bridge Vertical Clearance 116: 0.0 ft

## ELEMENT CONDITION STATE DATA

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	14/2	P Conc Deck/AC Ovly	(SF)	7,597	100 %	7,597	0 %	0	0 %	0	0 %	0	0 %	0
1	107/2	Paint Stl Opn Girder	(LF)	1,035	90 %	931	10 %	104	0 %	0	0 %	0	0 %	0
1	210/2	R/Conc Pier Wall	(LF)	37	90 %	33	10 %	4	0 %	0	0 %	0	0 %	0
1	215/2	R/Conc Abutment	(LF)	73	70 %	51	20 %	15	10 %	7	0 %	0	0 %	0
1	218/2	Undefined Wall Elem.	(LF)	120	0 %	0	50 %	60	50 %	60	0 %	0	0 %	0
1	302/2	Compressn Joint Seal	(LF)	74	0 %	0	50 %	37	50 %	37	0 %	0	0 %	0



## Structure Inventory and Appraisal Sheet (English Units)

Str Unit	Elm/Env	Description	Units	Total Qty	% in 1	Qty. St. 1	% in 2	Qty. St. 2	% in 3	Qty. St. 3	% in 4	Qty. St. 4	% in 5	Qty. St. 5
1	311/2	Moveable Bearing	(EA)	10	50 %	5	50 %	5	0 %	0	0 %	0	0 %	0
1	313/2	Fixed Bearing	(EA)	5	100 %	5	0 %	0	0 %	0	0 %	0	0 %	0
1	330/2	Metal Rail Uncoated	(LF)	414	80 %	331	20 %	83	0 %	0	0 %	0	0 %	0
1	383/2	Wear.Surf- AC+Membr.	(SF)	6,831	100 %	6,831	0 %	0	0 %	0	0 %	0	0 %	0
1	388/2	Paint	(SF)	15,675	90 %	14,108	5 %	784	5 %	784	0 %	0	0 %	0
Str Unit	Elm/Env	Description	Element Notes											
1	14/2	Concrete Deck - Protected w/ AC	Minor cracking of bituminous wearing surface. Curbs & fascias have isolated moderate areas of deterioration. Large spall at south abutment west end of joint needs repairs. Bottom of deck is in good condition.											
1	107/2	Painted Steel Open Girder/Beam	Moderate paint system failures and distress at high corrosion areas. Beams under south failed seal are rusting.											
1	210/2	Reinforced Conc Pier Wall	Minor map cracking & staining exterior pier walls. Large crack in center of pier.											
1	215/2	Reinforced Conc Abutment	Heavy cracking with efflo staining & light delaminations exterior abutment breastwalls, backwall, and bridge seats. Remaining abutments have minor cracking only.											
1	218/2	Undefined Wall Elem (Incl. Wing-,	Approximately 50% heavy cracking & staining wing walls with delaminations and isolated areas of spalling.											
1	302/2	Compression Joint Seal	Vertical & horizontal shifting of joint with seal, armor and concrete damage. South seal failed.											
1	311/2	Moveable Bearing (roller, sliding, e	Paint loss and active corrosion present											
1	313/2	Fixed Bearing	No problems noted.											
1	330/2	Metal Bridge Railing - Uncoated (A	Minor plow scraping only											
1	383/2	Wearing Surface - AC & Membrane	< none >											
1	388/2	Paint (Dummy Element)	<none>											

## BRIDGE NOTES

Continuous two span welded 5 steel beams with concrete deck, center pier, abutments and wingwalls. Bituminous wearing surface with granite curb and 2 rail aluminum bridge rail.

ABUTMENTS WERE CORED TO EVALUATE FOR ASR AND DETERMINE POSSIBLE SOLUTIONS - BWF

## PAST INSPECTION

Inspection Date: 01/05/2015

Type: 1 Regular NBI

Inspector: DT2HARR

Pontis User Key: DT2HARR - SCO1

Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

Structure is in overall satisfactory condition with moderate/ isolated heavy areas of deterioration. largest concern at this time is the south end compression seal that has failed. Recommend new seal, concrete repairs to joint fascia areas and treating of adjacent superstructure beam end areas. Abutments & wings suffer from ASR and a future rehab is desired. See individual elements & photos for details.



**Structure Inventory and Appraisal Sheet (English Units)**

## PAST INSPECTION

Inspection Date: 12/11/2012

Type: 1 Regular NBI

Inspector: DTCEDWA

Pontis User Key: DTCEDWA - CAR

Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

CHANNEL: Minor slumping of concrete slope away from abutments 1'-2" average. Center pier bituminous slope protection in slightly better condition.

## SUBSTRUCTURE:

Abutments: Bridge seats in generally good condition with large scale map cracking and scattered areas of delaminations and corrosion staining and efflorescence along face. Corners are heavily cracked with delaminations and efflorescence and isolated areas of spalling. Large vertical cracks at corners. Coring evident on southerly abutment in couple of locations. Northerly abutment has vertical crack at construction joint that gets wider at top.

Wingwalls: fair to poor condition with map cracking, cracking with efflorescence, staining and delaminations with spalls. Wingwalls and abutments appear to have slightly shifted relative to each other and curb and rail on roadway show deflections and offsets.

Pier good condition with medium vertical crack at or near center

## PAST INSPECTION

Inspection Date: 04/07/2011

Type: 1 Regular NBI

Inspector: DTJHARR

Pontis User Key: DTJHARR - STEV

Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

CHANNEL: Minor slumping of concrete slope away from abutments 1'-2" average. Center pier bituminous slope protection in slightly better condition.

## SUBSTRUCTURE:

Abutments: Bridge seats in generally good condition with large scale map cracking and scattered areas of delaminations and corrosion staining and efflorescence along face. Corners are heavily cracked with delaminations and efflorescence and isolated areas of spalling. Large vertical cracks at corners. Coring evident on southerly abutment in couple of locations. Northerly abutment has vertical crack at construction joint that gets wider at top. Measured @ horizontal architectural detail and found to be 1/4" at this time. Small diagonal cracking above slope armor below.

Wingwalls: fair to poor condition with map cracking, cracking with efflorescence, staining and delaminations with spalls. Wingwalls and abutments appear to have slightly shifted relative to each other and curb and rail on roadway show deflections and offsets.

Pier good condition with medium vertical crack at or near center



**Structure Inventory and Appraisal Sheet (English Units)**

## PAST INSPECTION

Inspection Date: 08/04/2010

Type: 1 Regular NBI

Inspector: DT2HARR

Pontis User Key: DT2HARR - SCOT

## Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

CHANNEL: Minor slumping of concrete slope away from abutments 1'-2" average. Center pier bituminous slope protection in slightly better condition.

## SUBSTRUCTURE:

Abutments: Bridge seats in generally good condition with large scale map cracking and scattered areas of delaminations and corrosion staining and efflorescence along face. Corners are heavily cracked with delaminations and efflorescence and isolated areas of spalling. Large vertical cracks at corners. Coring evident on southerly abutment in couple of locations. Northerly abutment has vertical crack at construction joint that gets wider at top. Measured @ horizontal architectural detail and found to be 1/4" at this time. Small diagonal cracking above slope armor below.

Wingwalls: fair to poor condition with map cracking, cracking with efflorescence, staining and delaminations with spalls.

Wingwalls and abutments appear to have slightly shifted relative to each other and curb and rail on roadway show deflections and offsets.

Pier good condition with medium vertical crack at or near center

## PAST INSPECTION

Inspection Date: 07/20/2009

Type: 1 Regular NBI

Inspector: DTRLANP

Pontis User Key: DTRLANP - ROBE

## Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

CHANNEL: Minor slumping of concrete slope away from abutments 1'-2" average. Center pier bituminous slope protection in slightly better condition.

## SUBSTRUCTURE:

Abutments: Bridge seats in generally good condition with large scale map cracking and scattered areas of delaminations and corrosion staining and efflorescence along face. Corners are heavily cracked with delaminations and efflorescence and isolated areas of spalling. Large vertical cracks at corners. Coring evident on southerly abutment in couple of locations. Northerly abutment has vertical crack at construction joint that gets wider at top. Measured @ horizontal architectural detail and found to be 1/4" at this time. Small diagonal cracking above slope armor below.

Wingwalls: fair to poor condition with map cracking, cracking with efflorescence, staining and delaminations with spalls.

Wingwalls and abutments appear to have slightly shifted relative to each other and curb and rail on roadway show deflections and offsets.

Pier good condition with medium vertical crack at or near center



**Structure Inventory and Appraisal Sheet (English Units)**

## PAST INSPECTION

Inspection Date: 12/27/2007

Type: 1 Regular NBI

Inspector: DTDBRYA

Pontis User Key: DTDBRYA - DARF

Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

Superstructure & deck are in overall good condition with minor deterioration only. Abutments and wings have heavy cracking & staining at high exposure areas. No immediate work needed. 2007 No major changes since last inspection, still a high % of cracking on abutments and wings, pier appears to be in good condition at this time.

## PAST INSPECTION

Inspection Date: 05/04/2005

Type: 1 Regular NBI

Inspector: DT2HARR

Pontis User Key: DT2HARR - SCO1

Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

Superstructure & deck are in overall good condition with minor deterioration only. Abutments and wings have heavy cracking & staining at high exposure areas. No immediate work needed.



**Structure Inventory and Appraisal Sheet (English Units)**

## PAST INSPECTION

Inspection Date: 12/31/2003

Type: 1 Regular NBI

Inspector: -1

Pontis User Key: PWV

## Scope:

NBI: ☒Other: ☐Element: ☒Underwater: ☐Fracture Critical: ☐

## INSPECTION NOTES

## INSPECTOR WORK CANDIDATES

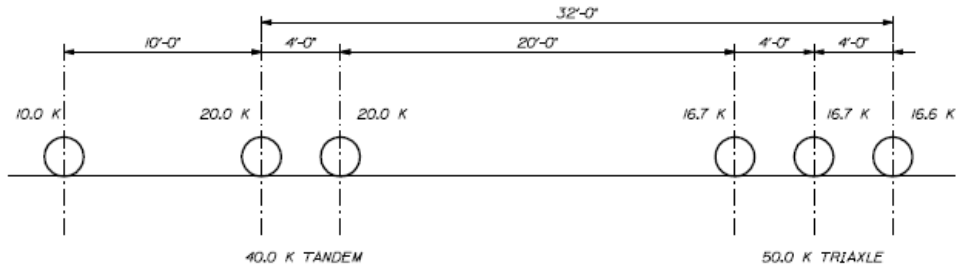
Work Candidate ID	Action	Object	Agency Status	Agency Priority	Assigned to a Project	Rec. Date
A-DOT001-11F70C8C-0000001B	Part Paint	Paint Stl Opn Girder	Approved	Medium	No	1/5/2015
A-DOT001-0A0B3E95-00000052	Rehab Elem	Undefined Wall Elem.	Approved	Medium	No	1/5/2015
A-DOT001-11F70C8C-00000017	Repl Elem	Compressn Joint Seal	Approved	High	No	1/5/2015



## APPENDIX B

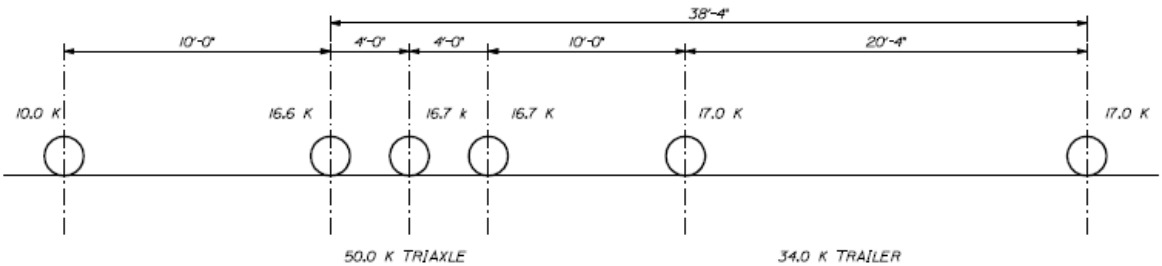
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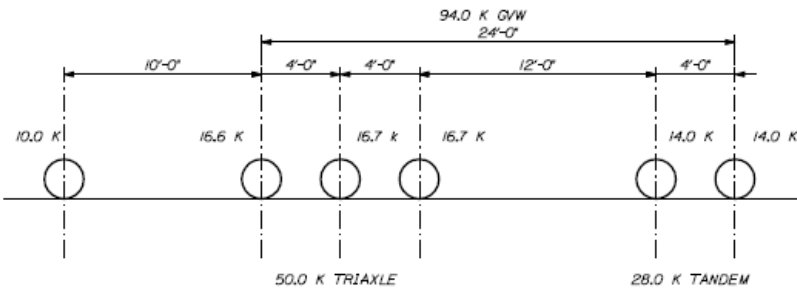
CONFIGURATON 1

SIX AXLE  
3 AXLE TRACTOR  
TRIAxLE SEMI-TRAILER  
100.0 K GVW



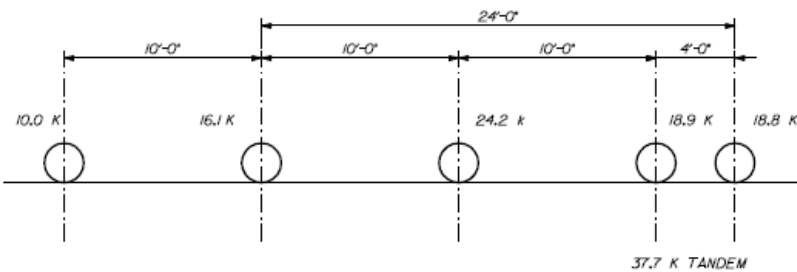
CONFIGURATON 2

SIX AXLE  
TRIAxLE TRUCK  
2 AXLE TRAILER  
94.0 K GVW



CONFIGURATON 3

SIX AXLE  
TRIAxLE TRUCK/TRACTOR  
TANDEM SEMI-TRAILER  
88.0 K GVW



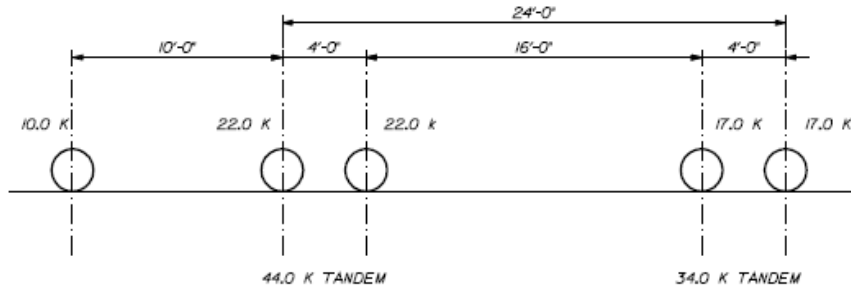
CONFIGURATON 4

FIVE AXLE  
TWO AXLE TRUCK/TRACTOR  
THREE AXLE TRAILER  
88.0 K GVW

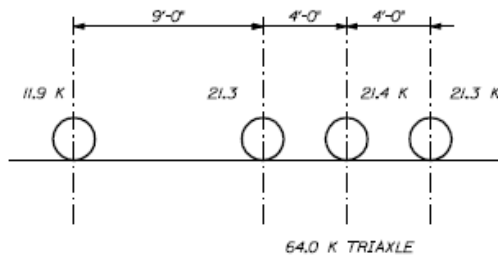
REVISION 03-07-11

**Figure 1. Maine DOT Legal Loads**

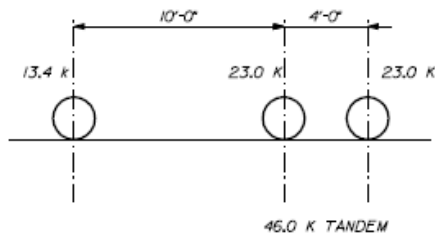




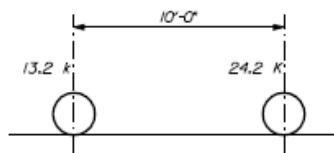
CONFIGURATON 5 FIVE AXLE  
TANDEM AXLE TRUCK  
OR TRACTOR SEMI-TRAILER  
88.0 K GVW



CONFIGURATON 6 FOUR AXLE TRUCK  
75.9 K GVW



CONFIGURATON 7 THREE AXLE TRUCK  
59.0 K GVW



CONFIGURATON 8 TWO AXLE TRUCK  
37.4 K GVW

**Figure 2. Maine DOT Legal Loads**



## APPENDIX C

### Load Ratings Calculations



Project: MaineDOT Bridge Load Rating  
Job No.: 2582-15.02

Rated By: MMS  
Checked By: CDS

Date: 7/15/16

## GREEN POINT ROAD OVER 395 No. 1563

### BRIDGE INFO

$Year := 1984$

Year the bridge was constructed

$Bridge_L := 207 \cdot ft$

Overall bridge length

$Bridge_W := 36.7 \cdot ft$

Overall bridge width

$Skew := 18^\circ - 03'$

Degree of skew

$Curvature := 0$

Degree of horizontal curvature

$Roadway_W := 33.0 \cdot ft$

Overall width of the roadway from curb to curb

$N_{spans} := 2$

Number of bridge spans

$Span_{L1} := 100 \cdot ft$

Length of 1st span

$Span_{L2} := 100 \cdot ft$

Length of 2nd span

$Span_{L3} := 0 \cdot ft$

Length of 3rd span

$Span_{total} := Span_{L1} + Span_{L2} + Span_{L3} = 200 \cdot ft$

$N_{beams} := 5$

Number of bridge girders

$S_{girders} := 7.66 \cdot ft$

Spacing between girders

$H := 2 \cdot in$

Haunch depth

$OVH := 3 \cdot ft$

Length deck overhangs exterior girders

$Deck_T := 8.5 \cdot in$

Deck Thickness

$WearingConc_T := 0 \cdot in$

Concrete wearing surface thickness (0 if none)

$WearingBit_T := 3 \cdot in$

Bituminous wearing surface thickness (0 if none)



$$S_{brace\_neg} := 11.5 \cdot ft$$

$$S_{brace\_pos} := 25 \cdot ft$$

$$N_{vehlanes} := 2$$

$$N_{deslanes} := 2$$

$$Curb_H := .75 \cdot ft$$

$$Curb_W := 1.833 \cdot ft$$

$$Sidewalk_W := 0 \cdot ft$$

$$ADT_{cur} := 2130$$

$$ADT_{fut} := 2982$$

$$ADT_{Dir\_Split} := 0.55$$

$$PerTrucks := 5\%$$

$$ADTT := \text{ceil}(ADT_{fut} \cdot ADT_{Dir\_Split} \cdot PerTrucks) = 83$$

$$\varphi_c := 1.0$$

$$\varphi_s := 1.0$$

Spacing of braces in neg bending region

Spacing of braces in pos bending region

Number of possible vehicle lanes according to AASHTO LRFD SPECS

Number of designated lanes on the bridge

Curb Height

Curb Width

Sidewalk Width

Condition factor - Satisfactory = 1.0 / Fair = 0.95 / Poor = 0.85

System Factor - 0.85 for 3 girder bridges with girder spacing = 6'  
0.95 for 4 girder bridges with girders spacing less than 4 '  
1.0 for all other girder bridges

## **LIMIT STATES**

### **Strength I Limit State - Inventory**

Limit states required for a steel girder bridge

$$\gamma_{STI\_inv} := \begin{bmatrix} 1.25 \\ 1.50 \\ 1.75 \end{bmatrix}$$

### **Strength I Limit State - Operating**

$$\gamma_{STI\_op} := \begin{bmatrix} 1.25 \\ 1.50 \\ 1.35 \end{bmatrix}$$

### **Service II Limit State**

$$\gamma_{SEH} := \begin{bmatrix} 1.00 \\ 1.00 \\ 1.30 \end{bmatrix}$$

### **Legal Load Load Factors**

$$\gamma_{LL} := \begin{bmatrix} 1.25 \\ 1.50 \\ 1.40 \end{bmatrix}$$

Dependant on ADTT



## BRIDGE MATERIAL PROPERTIES

$$\gamma_{steel} := 490 \text{ pcf}$$

$$\gamma_{conc} := 150 \text{ pcf}$$

$$\gamma_{bit} := 140 \text{ pcf}$$

Unit weight of steel

Unit weight of concrete

Unit weight of bit. wearing surface

$$f'_c := 4 \text{ ksi}$$

$$F_u := 50 \text{ ksi}$$

$$F_{uflange} := 50 \cdot \text{ksi}$$

$$F_{y\_rebar} := 60 \cdot \text{ksi}$$

Compressive strenght of concrete

Yield strength of steel

Yield strength of steel of flanges (use if flanges are differing grades)

Yield strength of rebars

$$E_{steel} := 29000 \text{ ksi}$$

$$E_{conc} := 1820 \cdot \sqrt{f'_c \cdot \text{ksi}} = (3.64 \cdot 10^3) \text{ ksi}$$

$$n := \frac{E_{steel}}{E_{conc}} = 7.967$$

## GIRDER SECTION PROPERTIES

### ***Positive Flexture --- Rolled Beam W36x182***

$$A_{girder} := 53.6 \text{ in}^2$$

Area of the steel section

$$I_x := 11300 \cdot \text{in}^4$$

Moment of inertia of the section

$$D_{girder} := 36.3 \cdot \text{in}$$

Depth of the girder

$$W_{tflange} := 12.1 \cdot \text{in}$$

Width of the top flange

$$t_{tflange} := 1.18 \cdot \text{in}$$

Thickness of the top flange

$$W_{bflange} := 12.1 \cdot \text{in}$$

Width of the bottom flange

$$t_{bflange} := 1.18 \cdot \text{in}$$

Thickness of the bottom flange

$$t_{web} := .725 \cdot \text{in}$$

Thickness of the web



$$S := \min \left( 0.25 \cdot \text{Span}_{L1}, \begin{cases} \text{if } t_{web} > \frac{t_{bflange}}{2} \\ 12 \cdot \text{Deck}_T + t_{web} \\ \text{else} \\ 12 \cdot \text{Deck}_T + \frac{t_{bflange}}{2} \end{cases}, S_{girders} \right) = 7.66 \text{ ft}$$

Effective deck slab width

$$S_{ext} := \min \left( S, \frac{S_{girders}}{2} + OVH \right) = 6.83 \text{ ft}$$

Effective deck slab width for exterior beams

$$W_{topplate} := 0 \cdot \text{in}$$

Width of additional plate on top flange of girder

$$t_{topplate} := 0 \cdot \text{in}$$

Thickness of additional plate on top flange of girder

$$A_{topplate} := W_{topplate} \cdot t_{topplate} = 0 \text{ in}^2$$

$$I_{topplate} := \frac{W_{topplate} \cdot t_{topplate}^3}{12} = 0 \text{ in}^4$$

$$W_{botplate} := 0 \cdot \text{in}$$

Width of additional plate on bottom flange of girder

$$t_{botplate} := 0 \cdot \text{in}$$

Thickness of additional plate on bottom flange of girder

$$A_{botplate} := W_{botplate} \cdot t_{botplate} = 0 \text{ in}^2$$

$$I_{botplate} := \frac{W_{botplate} \cdot t_{botplate}^3}{12} = 0 \text{ in}^4$$

$$d_0 := 300 \cdot \text{in}$$

Transverse stiffener spacing

$$d_{0\_end} := 300 \cdot \text{in}$$

Distance from end support to transverse stiffener

$$y := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \frac{t_{botplate}}{2}}{A_{topplate} + A_{girder} + A_{botplate}} = 18.15 \text{ in}$$

Distance to neutral axis of non-composite girder



$$I := I_x + A_{girder} \cdot \left( t_{botplate} + \frac{D_{girder}}{2} - y \right)^2 + I_{topplate} + A_{topplate} \cdot \left( t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y \right)^2 + I_{botplate} + A_{botplate} \cdot \left( y - \frac{t_{botplate}}{2} \right)^2 = (1.13 \cdot 10^4) \text{ in}^4$$

$$S_{x\_pos} := \frac{I}{y} = 622.59 \text{ in}^3$$

Section modulus of non-composite girder

Moment of inertia of non-composite girder

$$e_g := t_{botplate} + D_{girder} + H - t_{tflange} + \frac{Deck_T}{2} - y = 23.22 \text{ in}$$

$$K_{g\_pos} := n \cdot (I + (A_{girder} + A_{topplate} + A_{botplate}) \cdot e_g^2) = (3.203 \cdot 10^5) \text{ in}^4$$

### Composite Section Properties Interior Girder

#### Short Term / Transient (n)

$$A_{deck} := \frac{S \cdot Deck_T}{n} \quad I_{deck} := \frac{S \cdot Deck_T^3}{12 \cdot n}$$

$$y_{STC} := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \left( \frac{t_{botplate}}{2} \right) + A_{deck} \cdot \left( t_{botplate} + D_{girder} + H - t_{tflange} + \frac{Deck_T}{2} \right)}{A_{topplate} + A_{girder} + A_{botplate} + A_{deck}} = 33.164 \text{ in}$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y_{STC}$$

Distance to neutral axis of composite girder for short term loads

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y_{STC}$$

$$d_{botplate} := y_{STC} - \frac{t_{botplate}}{2}$$

$$d_{deck} := t_{botplate} + D_{girder} - t_{tflange} + H + \frac{Deck_T}{2} - y_{STC}$$

$$I_{STC} := I_x + A_{girder} \cdot (d_{girder})^2 + I_{topplate} + A_{topplate} \cdot (d_{topplate})^2 + I_{botplate} + A_{botplate} \cdot (d_{botplate})^2 + I_{deck} + A_{deck} \cdot (d_{deck})^2 = (3.058 \cdot 10^4) \text{ in}^4$$

$$S_{x_{STC}} := \frac{I_{STC}}{y_{STC}} = 921.987 \text{ in}^3$$

Section modulus of composite girder for short term loads

Moment of inertia of composite girder for short term loads

#### Long Term (3n)

$$A_{deck} := \frac{S \cdot Deck_T}{3 \cdot n} \quad I_{deck} := \frac{S \cdot Deck_T^3}{12 \cdot 3 \cdot n}$$



$$y_{LTC} := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \left( \frac{t_{botplate}}{2} \right) + A_{deck} \cdot \left( t_{botplate} + D_{girder} + H - t_{tflange} + \frac{Deck_T}{2} \right)}{A_{topplate} + A_{girder} + A_{botplate} + A_{deck}} = 26.947 \text{ in}$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y_{LTC}$$

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y_{LTC}$$

$$d_{botplate} := y_{LTC} - \frac{t_{botplate}}{2}$$

$$d_{deck} := t_{botplate} + D_{girder} - t_{tflange} + H + \frac{Deck_T}{2} - y_{LTC}$$

$$I_{LTC} := I_x + A_{girder} \cdot (d_{girder})^2 + I_{topplate} + A_{topplate} \cdot (d_{topplate})^2 + I_{botplate} + A_{botplate} \cdot (d_{botplate})^2 + I_{deck} + A_{deck} \cdot (d_{deck})^2 = (2.244 \cdot 10^4) \text{ in}^4$$

$$S_{xLTC} := \frac{I_{LTC}}{y_{LTC}} = 832.943 \text{ in}^3$$

Section modulus of composite girder for long term loads

Distance to neutral axis of composite girder for long term loads

Moment of inertia of composite girder for long term loads

### Plastic Moments Interior Girder ----- AASHTO Table D6.1-1

$$P_t := W_{bflange} \cdot t_{bflange} \cdot F_{yflange} = 713.9 \text{ kip}$$

$$P_w := (D_{girder} - t_{tflange} - t_{bflange}) \cdot t_{web} \cdot F_y = (1.23 \cdot 10^3) \text{ kip}$$

$$P_c := W_{tflange} \cdot t_{tflange} \cdot F_y = 713.9 \text{ kip}$$

$$P_{topplate} := A_{topplate} \cdot F_y = 0 \text{ kip}$$

$$P_{botplate} := A_{botplate} \cdot F_y = 0 \text{ kip}$$

$$P_s := 0.85 \cdot f'_c \cdot S \cdot Deck_T = (2.656 \cdot 10^3) \text{ kip}$$

$$D := D_{girder} - t_{tflange} - t_{bflange} = 33.94 \text{ in}$$

Forces in each component of the composite girder at the plastic moment

### Case I - PNA in Web

$$Y_{web} := \frac{(D)}{2} \cdot \left( \frac{P_t + P_{botplate} - P_c - P_{topplate} - P_s}{P_w} + 1 \right) = -19.671 \text{ in}$$

$$y_{web} := t_{botplate} + D_{girder} - t_{tflange} - Y_{web} = 54.791 \text{ in}$$

$$d_{slab} := \frac{Deck_T}{2} + H + Y_{web}$$



$$d_{comp} := \frac{t_{tflange}}{2} + Y_{web}$$

$$d_{topplate} := \frac{t_{topplate}}{2} + t_{tflange} + Y_{web}$$

$$d_{ten} := y_{web} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{web} - \frac{t_{botplate}}{2}$$

$$M_{p\_web} := \frac{P_w}{2 \cdot D} \cdot (Y_{web}^2 + (D - Y_{web})^2) + (P_s \cdot \langle d_{slab} \rangle + P_c \cdot \langle d_{comp} \rangle + P_{topplate} \cdot \langle d_{topplate} \rangle + P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle) = (4.044 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Plastic moment when  
PNA is in the web

### Case II - PNA in Flange

$$Y_{flange} := \left( \frac{t_{tflange} + t_{topplate}}{2} \right) \cdot \left( \frac{P_w + P_t + P_{botplate} - P_s}{P_c + P_{topplate}} + 1 \right) = 0.001 \text{ in}$$

$$y_{flange} := t_{botplate} + D_{girder} + t_{topplate} - Y_{flange} = 36.299 \text{ in}$$

$$d_{slab} := \frac{Deck_T}{2} + H - (t_{tflange} + t_{topplate} - Y_{flange})$$

$$d_{ten} := y_{flange} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{flange} - \frac{t_{botplate}}{2}$$

$$d_{web} := \frac{D_{girder}}{2} - Y_{flange}$$

$$M_{p\_flange} := \frac{P_c + P_{topplate}}{2 \cdot (t_{tflange} + t_{topplate})} \cdot (Y_{flange}^2 + (t_{tflange} + t_{topplate} - Y_{flange})^2) + (P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle + P_w \cdot \langle d_{web} \rangle + P_s \cdot \langle d_{slab} \rangle) = (5.143 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Plastic moment when  
PNA is in the flange

### Case III - PNA in Slab

$$Y_{slab} := Deck_T \cdot \left( \frac{P_c + P_{topplate} + P_w + P_t + P_{botplate}}{P_s} \right) = 8.505 \text{ in}$$

$$y_{slab} := t_{botplate} + D_{girder} + H - t_{tflange} + Deck_T - Y_{slab} = 37.115 \text{ in}$$

$$d_{comp} := y_{slab} - t_{botplate} - D_{girder} + \frac{t_{tflange}}{2}$$

$$d_{topplate} := y_{slab} - t_{botplate} - D_{girder} - \frac{t_{topplate}}{2}$$



$$d_{ten} := y_{slab} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{slab} - \frac{t_{botplate}}{2}$$

$$d_{web} := y_{slab} - t_{botplate} - \frac{D_{girder}}{2}$$

$$M_{p\_slab} := \left( \frac{Y_{slab}^2 \cdot P_s}{2 Deck_T} \right) + (P_{topplate} \cdot \langle d_{topplate} \rangle + P_c \cdot \langle d_{comp} \rangle + P_w \cdot \langle d_{web} \rangle + P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle) = (5.143 \cdot 10^3) \text{ kip} \cdot ft$$

Plastic moment when  
PNA is in the slab

$$M_{p\_Location} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_s \\ \quad \text{“PNA in Flange”} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_s \\ \quad \quad \text{“PNA in Flange”} \\ \quad \text{else} \\ \quad \quad \text{“PNA in Slab”} \end{cases} = \text{“PNA in Flange”}$$

$$M_{p\_pos} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_s \\ \quad M_{p\_web} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_s \\ \quad \quad M_{p\_flange} \\ \quad \text{else} \\ \quad \quad M_{p\_slab} \end{cases} = (5.143 \cdot 10^3) \text{ (kip} \cdot ft)$$

Plastic Moment of the  
composite section

### Nominal Flexural Resistance - Compression Flange Continuously Braced by Deck

#### Service Limit State

$$f_{f\_top\_pos} := 0.95 \cdot F_y = 47.5 \text{ ksi}$$

Allowable Stress in the top flange

$f_t$  term can be ignored

$$f_{f\_bot\_pos} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi}$$

Allowable Stress in the bottom flange



### Strength Limit State

$$D_{cp} := \begin{cases} \text{if } M_{p\_Location} = \text{"PNA in Web"} \\ \left\| \frac{D}{2} \cdot \left( \frac{F_y \cdot (A_{botplate} - A_{topplate}) - (0.85 \cdot f'_c \cdot S \cdot Deck_T)}{F_y \cdot D \cdot t_{web}} + 1 \right) \right\| \\ \text{else} \\ \left\| 0 \cdot \text{in} \right\| \end{cases} = 0 \text{ in}$$

$$D_t := t_{botplate} + D_{girder} - t_{tflange} + H + Deck_T = 45.62 \text{ in}$$

$$D_p := \begin{cases} \text{if } M_{p\_Location} = \text{"PNA in Web"} \\ \left\| D_t - y_{web} \right\| \\ \text{else if } M_{p\_Location} = \text{"PNA in Flange"} \\ \left\| D_t - y_{flange} \right\| \\ \text{else} \\ \left\| D_t - y_{slab} \right\| \end{cases} = 9.321 \text{ in}$$

$$Compactness := \begin{cases} \text{if } \frac{D}{t_{web}} > 150 \\ \left\| \text{"non compact"} \right\| \\ \text{else if } \frac{2 \cdot D_{cp}}{t_{web}} > 3.76 \cdot \sqrt{\frac{E_{steel}}{F_y}} \\ \left\| \text{"non compact"} \right\| \\ \text{else} \\ \left\| \text{"compact"} \right\| \end{cases} = \text{"compact"}$$

Compactness check

$$M_{n\_pos} := \begin{cases} \text{if } Compactness = \text{"compact"} \\ \left\| \begin{cases} \text{if } D_p \leq 0.1 \cdot D_t \\ \left\| M_{p\_pos} \right\| \\ \text{else} \\ \left\| M_{p\_pos} \cdot \left( 1.07 - 0.7 \frac{D_p}{D_t} \right) \right\| \end{cases} \right\| \\ \text{else} \\ \left\| \text{"noncompact - check code"} \right\| \end{cases} = (4.767 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Moment Capacity of the Composite Girder in Positive Bending for an Interior Girder



## Nominal Shear Resistance

### Strength Limit State

$$V_p := 0.58 \cdot F_y \cdot D \cdot t_{web} = 713.589 \text{ kip}$$

Shear Capacity of the web

$$k := 5 + \frac{5}{\left(\frac{d_0}{D}\right)^2} = 5.064$$

k value for typical web section with stiffeners

$$k_{end} := 5 + \frac{5}{\left(\frac{d_{0\_end}}{D}\right)^2} = 5.064$$

k value for end section of web above supports

$$C := \begin{cases} \text{if } \frac{D}{t_{web}} \leq 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \quad \parallel 1 \\ \text{else if } 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} < \frac{D}{t_{web}} \leq 1.40 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \quad \parallel \frac{1.12}{\left(\frac{D}{t_{web}}\right)} \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \text{else} \\ \quad \parallel \frac{1.57}{\left(\frac{D}{t_{web}}\right)^2} \left(\frac{E_{steel} \cdot k}{F_y}\right) \end{cases} = 1$$

C value for typical web section with stiffeners



$$C_{end} := \begin{cases} \text{if } \frac{D}{t_{web}} \leq 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ 1 \\ \text{else if } 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} < \frac{D}{t_{web}} \leq 1.40 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ \frac{1.12}{\left(\frac{D}{t_{web}}\right)} \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ \text{else} \\ \frac{1.57}{\left(\frac{D}{t_{web}}\right)^2} \left(\frac{E_{steel} \cdot k_{end}}{F_y}\right) \end{cases} = 1$$

C value for end section of web above supports

$$V_{n\_int} := \begin{cases} \text{if } \frac{2 D \cdot t_{web}}{(W_{tflange} \cdot t_{tflange} + W_{bflange} \cdot t_{bflange})} \leq 2.5 \\ V_p \cdot \left( C + \frac{0.87 (1 - C)}{\sqrt{1 + \left(\frac{d_0}{D}\right)^2}} \right) \\ \text{else} \\ V_p \cdot \left( C + \frac{0.87 (1 - C)}{\sqrt{1 + \left(\frac{d_0}{D}\right)^2} + \frac{d_0}{D}} \right) \end{cases} = 713.589 \text{ kip}$$

Shear capacity of girder with stiffeners

$$V_{n\_end} := C_{end} \cdot V_p = 713.589 \text{ kip}$$

Shear capacity of end section of girder with stiffeners



## Composite Section Properties Exterior Girder

### Short Term / Transient (n)

$$A_{deck} := \frac{S_{ext} \cdot Deck_T}{n} \quad I_{deck} := \frac{S_{ext} \cdot Deck_T^3}{12 \cdot n}$$

$$y_{STC} := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \left( \frac{t_{botplate}}{2} \right) + A_{deck} \cdot \left( t_{botplate} + D_{girder} + H - t_{tflange} + \frac{Deck_T}{2} \right)}{A_{topplate} + A_{girder} + A_{botplate} + A_{deck}} = 32.546 \text{ in}$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y_{STC}$$

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y_{STC}$$

$$d_{botplate} := y_{STC} - \frac{t_{botplate}}{2}$$

$$d_{deck} := t_{botplate} + D_{girder} - t_{tflange} + H + \frac{Deck_T}{2} - y_{STC}$$

$$I_{STC} := I_x + A_{girder} \cdot (d_{girder})^2 + I_{topplate} + A_{topplate} \cdot (d_{topplate})^2 + I_{botplate} + A_{botplate} \cdot (d_{botplate})^2 + I_{deck} + A_{deck} \cdot (d_{deck})^2 = (2.974 \cdot 10^4) \text{ in}^4$$

$$S_{xSTC\_ext} := \frac{I_{STC}}{y_{STC}} = 913.893 \text{ in}^3$$

Section modulus of composite girder for short term loads

Distance to neutral axis of composite girder for short term loads

Moment of inertia of composite girder for short term loads

### Long Term (3n)

$$A_{deck} := \frac{S_{ext} \cdot Deck_T}{3 \cdot n} \quad I_{deck} := \frac{S_{ext} \cdot Deck_T^3}{12 \cdot 3 \cdot n}$$

$$y_{LTC} := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \left( \frac{t_{botplate}}{2} \right) + A_{deck} \cdot \left( t_{botplate} + D_{girder} + H - t_{tflange} + \frac{Deck_T}{2} \right)}{A_{topplate} + A_{girder} + A_{botplate} + A_{deck}} = 26.329 \text{ in}$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y_{LTC}$$

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y_{LTC}$$

$$d_{botplate} := y_{LTC} - \frac{t_{botplate}}{2}$$

Distance to neutral axis of composite girder for long term loads

Deck<sub>T</sub>



$$d_{deck} := t_{botplate} + D_{girder} - t_{tflange} + H + \frac{Deck_T}{2} - y_{LTC}$$

$$I_{LTC} := I_x + A_{girder} \cdot (d_{girder})^2 + I_{topplate} + A_{topplate} \cdot (d_{topplate})^2 + I_{botplate} + A_{botplate} \cdot (d_{botplate})^2 + I_{deck} + A_{deck} \cdot (d_{deck})^2 = (2.166 \cdot 10^4) \text{ in}^4$$

$$S_{xLTC\_ext} := \frac{I_{LTC}}{y_{LTC}} = 822.48 \text{ in}^3$$

Section modulus of composite girder for  
slong term loads

Moment of inertia of composite  
girder for long term loads

### Plastic Moments Exterior Girder ----- AASHTO Table D6.1-1

$$P_t := W_{bflange} \cdot t_{bflange} \cdot F_{yflange} = 713.9 \text{ kip}$$

$$P_w := (D_{girder} - t_{bflange} - t_{tflange}) \cdot t_{web} \cdot F_y = (1.23 \cdot 10^3) \text{ kip}$$

$$P_c := W_{tflange} \cdot t_{tflange} \cdot F_y = 713.9 \text{ kip}$$

$$P_{topplate} := A_{topplate} \cdot F_y = 0 \text{ kip}$$

$$P_{botplate} := A_{botplate} \cdot F_y = 0 \text{ kip}$$

$$P_s := 0.85 \cdot f'_c \cdot S_{ext} \cdot Deck_T = (2.369 \cdot 10^3) \text{ kip}$$

$$D := D_{girder} - t_{bflange} - t_{tflange} = 33.94 \text{ in}$$

Forces in each component of the composite girder at the  
plastic moment

#### Case I - PNA in Web

$$Y_{web} := \frac{(D)}{2} \cdot \left( \frac{P_t + P_{botplate} - P_c - P_{topplate} - P_s}{P_w} + 1 \right) = -15.701 \text{ in}$$

$$y_{web} := t_{botplate} + D_{girder} - t_{tflange} - Y_{web} = 50.821 \text{ in}$$

$$d_{slab} := \frac{Deck_T}{2} + H + Y_{web}$$

$$d_{comp} := \frac{t_{tflange}}{2} + Y_{web}$$

$$d_{topplate} := \frac{t_{topplate}}{2} + t_{tflange} + Y_{web}$$

$$d_{ten} := y_{web} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{web} - \frac{t_{botplate}}{2}$$

Plastic moment when  
PNA is in the web

$$M_{p\_web} := \frac{P_w}{2 \cdot D} \cdot (Y_{web}^2 + (D - Y_{web})^2) + (P_s \cdot (d_{slab}) + P_c \cdot (d_{comp}) + P_{topplate} \cdot (d_{topplate}) + P_t \cdot (d_{ten}) + P_{botplate} \cdot (d_{botplate})) = (4.318 \cdot 10^3) \text{ kip} \cdot \text{ft}$$



### Case II - PNA in Flange

$$Y_{flange} := \left( \frac{t_{tflange} + t_{topplate}}{2} \right) \cdot \left( \frac{P_w + P_t + P_{botplate} - P_s}{P_c + P_{topplate}} + 1 \right) = 0.239 \text{ in}$$

$$y_{flange} := t_{botplate} + D_{girder} - Y_{flange} = 36.061 \text{ in}$$

$$d_{slab} := \frac{Deck_T}{2} + H - (t_{tflange} + t_{topplate} - Y_{flange})$$

$$d_{ten} := y_{flange} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{flange} - \frac{t_{botplate}}{2}$$

$$d_{web} := \frac{D_{girder}}{2} - Y_{flange}$$

$$M_{p\_flange} := \frac{P_c + P_{topplate}}{2 (t_{tflange} + t_{topplate})} \cdot (Y_{flange}^2 + (t_{tflange} + t_{topplate} - Y_{flange})^2) + (P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle + P_w \cdot \langle d_{web} \rangle + P_s \cdot \langle d_{slab} \rangle) = (5.018 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

### Case III - PNA in Slab

Plastic moment when  
PNA is in the flange

$$Y_{slab} := Deck_T \cdot \left( \frac{P_c + P_{topplate} + P_w + P_t + P_{botplate}}{P_s} \right) = 9.539 \text{ in}$$

$$y_{slab} := t_{botplate} + D_{girder} + H - t_{tflange} + Deck_T - Y_{slab} = 36.081 \text{ in}$$

$$d_{comp} := y_{slab} - t_{botplate} - D_{girder} + \frac{t_{tflange}}{2}$$

$$d_{topplate} := y_{slab} - t_{botplate} - D_{girder} - \frac{t_{topplate}}{2}$$

$$d_{ten} := y_{slab} - t_{botplate} - \frac{t_{bflange}}{2}$$

$$d_{botplate} := y_{slab} - \frac{t_{botplate}}{2}$$

$$d_{web} := y_{slab} - t_{botplate} - \frac{D_{girder}}{2}$$

$$M_{p\_slab} := \left( \frac{Y_{slab}^2 \cdot P_s}{2 Deck_T} \right) + (P_{topplate} \cdot \langle d_{topplate} \rangle + P_c \cdot \langle d_{comp} \rangle + P_w \cdot \langle d_{web} \rangle + P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle) = (5.028 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Plastic moment when  
PNA is in the slab



$$M_{p\_Location} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_s & \text{= "PNA in Flange"} \\ \quad \text{"PNA in Web"} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_s & \text{"PNA in Flange"} \\ \quad \text{else} \\ \quad \quad \text{"PNA in Slab"} \end{cases}$$

$$M_{p\_pos\_ext} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_s & = (5.018 \cdot 10^3) \text{ (kip} \cdot \text{ft)} \\ \quad M_{p\_web} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_s & \\ \quad \quad M_{p\_flange} \\ \quad \text{else} \\ \quad \quad M_{p\_slab} \end{cases}$$

Plastic Moment of the composite section in negative bending

### Nominal Flexural Resistance - Compression Flange Continuously Braced by Deck

#### Service Limit State

$$f_{f\_top\_pos\_ext} := 0.95 \cdot F_y = 47.5 \text{ ksi} \quad \text{Allowable Stress in the top flange}$$

$$f_{f\_bot\_pos\_ext} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi} \quad \text{Allowable Stress in the bottom flange}$$

$f_l$  term can be ignored

#### Strength Limit State

$$D_{cp} := \begin{cases} \text{if } M_{p\_Location} = \text{"PNA in Web"} \\ \quad \left| \frac{D}{2} \cdot \left( \frac{F_y \cdot (A_{botplate} - A_{topplate}) - (0.85 \cdot f'_c \cdot S_{ext} \cdot Deck_T)}{F_y \cdot D \cdot t_{web}} + 1 \right) \right| & = 0 \text{ in} \\ \text{else} \\ \quad 0 \cdot \text{in} \end{cases}$$



$$D_t := t_{botplate} + D_{girder} - t_{tflange} + H + Deck_T = 45.62 \text{ in}$$

$$D_p := \begin{cases} \text{if } M_{p\_Location} = \text{"PNA in Web"} \\ \quad D_t - y_{web} \\ \text{else if } M_{p\_Location} = \text{"PNA in Flange"} \\ \quad D_t - y_{flange} \\ \text{else} \\ \quad D_t - y_{slab} \end{cases} = 9.559 \text{ in}$$

$$Compactness := \begin{cases} \text{if } \frac{D}{t_{web}} > 150 \\ \quad \text{"non compact"} \\ \text{else if } \frac{2 \cdot D_{cp}}{t_{web}} > 3.76 \cdot \sqrt{\frac{E_{steel}}{F_y}} \\ \quad \text{"non compact"} \\ \text{else} \\ \quad \text{"compact"} \end{cases} = \text{"compact"}$$

Compactness check

$$M_{n\_pos\_ext} := \begin{cases} \text{if } D_p \leq 0.1 \cdot D_t \\ \quad M_{p\_pos\_ext} \\ \text{else} \\ \quad M_{p\_pos\_ext} \cdot \left( 1.07 - 0.7 \frac{D_p}{D_t} \right) \end{cases} = (4.633 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Moment Capacity of the  
Composite Girder in Positive  
Bending for an Interior Girder



### Negative Flexure--- Rolled Beam W36x210

$$A_{girder} := 61.8 \text{ in}^2$$

Area of the steel section

$$I_x := 13200 \cdot \text{in}^4$$

Moment of inertia of the section

$$D_{girder} := 36.7 \cdot \text{in}$$

Depth of the girder

$$W_{flange} := 12.2 \cdot \text{in}$$

Width of the flange

$$t_{flange} := 1.36 \cdot \text{in}$$

Thickness of the flange

$$t_{web} := .830 \cdot \text{in}$$

Thickness of the web

$$W_{topplate} := 1.125 \cdot \text{in}$$

Width of additional plate added to top flange

$$t_{topplate} := 10 \cdot \text{in}$$

Thickness of additional plate added to top flange

$$A_{topplate} := W_{topplate} \cdot t_{topplate} = 11.25 \text{ in}^2$$

$$I_{topplate} := \frac{W_{topplate} \cdot t_{topplate}^3}{12} = 93.75 \text{ in}^4$$

$$W_{botplate} := 1.125 \cdot \text{in}$$

Width of additional plate added to bottom flange

$$t_{botplate} := 10 \cdot \text{in}$$

Thickness of additional plate added to bottom flange

$$A_{botplate} := W_{botplate} \cdot t_{botplate} = 11.25 \text{ in}^2$$

$$I_{botplate} := \frac{W_{botplate} \cdot t_{botplate}^3}{12} = 93.75 \text{ in}^4$$

$$A_{rt} := 5.95 \cdot \text{in}^2$$

Area of upper long. deck steel

$$d_{rt} := 3 \cdot \text{in}$$

Depth of upper long. deck steel

$$A_{rb} := 3.72 \cdot \text{in}^2$$

Area of lower long. deck steel



$$d_{rb} := 6.5 \cdot \text{in}$$

Depth of lower long. deck steel

$$A_{rt\_ext} := 6.57 \cdot \text{in}^2$$

Area of upper long. deck steel in exterior girder

$$A_{rb\_ext} := 2.79 \cdot \text{in}^2$$

Area of lower long. deck steel in exterior girder

$$d_0 := 138 \cdot \text{in}$$

Transverse stiffener spacing

$$d_{0\_end} := 138 \cdot \text{in}$$

Distance from support to transverse stiffener (if stiffener is at support then set  $d_{0\_end}$  to  $d_0$ )

$$y := \frac{A_{topplate} \cdot \left( \frac{t_{topplate}}{2} + D_{girder} + t_{botplate} \right) + A_{girder} \cdot \left( \frac{D_{girder}}{2} + t_{botplate} \right) + A_{botplate} \cdot \frac{t_{botplate}}{2}}{A_{topplate} + A_{girder} + A_{botplate}} = 28.35 \text{ in}$$

Distance to neutral axis of non-composite section

$$I_{neg} := I_x + A_{girder} \cdot \left( t_{botplate} + \frac{D_{girder}}{2} - y \right)^2 + I_{topplate} + A_{topplate} \cdot \left( t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y \right)^2 + I_{botplate} + A_{botplate} \cdot \left( y - \frac{t_{botplate}}{2} \right)^2 = (2.566 \cdot 10^4) \text{ in}^4$$

$$S_{x\_neg} := \frac{I_{neg}}{t_{botplate} + D_{girder} + t_{topplate} - y} = 904.938 \text{ in}^3$$

Section modulus of non-composite section

Moment of inertia of non-composite section

$$e_g := t_{botplate} + D_{girder} + H - t_{flange} + \frac{Deck_T}{2} - y = 23.24 \text{ in}$$

$$K_{g\_neg} := n \cdot \left( I + (A_{girder} + A_{topplate} + A_{botplate}) \cdot e_g^2 \right) = (4.528 \cdot 10^5) \text{ in}^4$$

### Composite Section Properties Interior Girder

$$d_{topplate} := \frac{t_{topplate}}{2} + D_{girder} + t_{botplate}$$

$$d_{girder} := \frac{D_{girder}}{2} + t_{botplate}$$

$$d_{botplate} := \frac{t_{botplate}}{2}$$

$$dd_{rt} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt}$$

$$dd_{rb} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rb}$$



$$y := \frac{A_{topplate} \cdot (d_{topplate}) + A_{girder} \cdot (d_{girder}) + A_{botplate} \cdot (d_{botplate}) + A_{rt} \cdot (dd_{rt}) + A_{rb} \cdot (dd_{rb})}{A_{topplate} + A_{girder} + A_{botplate} + A_{rt} + A_{rb}} = 30.732 \text{ in}$$

Distance to neutral axis of composite section

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y$$

$$d_{botplate} := y - \frac{t_{botplate}}{2}$$

$$dd_{rt} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt} - y$$

$$dd_{rb} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rb} - y$$

Moment of inertia of composite section

$$I_{comp\_neg} := I_x + A_{girder} \cdot (d_{girder})^2 + I_{topplate} + A_{topplate} \cdot (d_{topplate})^2 + I_{botplate} + A_{botplate} \cdot (d_{botplate})^2 + A_{rt} \cdot (dd_{rt})^2 + A_{rb} \cdot (dd_{rb})^2 = (3.033 \cdot 10^4) \text{ in}^4$$

$$S_{x\_comp\_neg\_rt} := \frac{I_{comp\_neg}}{(t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt} - y)} = (1.372 \cdot 10^3) \text{ in}^3$$

Section Modulus to top reinforcing steel

$$S_{x\_comp\_neg\_tf} := \frac{I_{comp\_neg}}{(t_{botplate} + D_{girder} + t_{topplate} - y)} = (1.168 \cdot 10^3) \text{ in}^3$$

Section Modulus to top flange

$$S_{x\_comp\_neg\_bf} := \frac{I_{comp\_neg}}{y} = 986.917 \text{ in}^3$$

Section Modulus to bottom flange

### Plastic Moments Interior Girder----- AASHTO Table D6.1-2

$$P_t := W_{flange} \cdot t_{flange} \cdot F_{yflange} = 829.6 \text{ kip}$$

$$P_w := (D_{girder} - 2 \cdot t_{flange}) \cdot t_{web} \cdot F_y = (1.41 \cdot 10^3) \text{ kip}$$

$$P_c := P_t = 829.6 \text{ kip}$$

$$P_{topplate} := A_{topplate} \cdot F_y = 562.5 \text{ kip}$$

$$P_{botplate} := A_{botplate} \cdot F_y = 562.5 \text{ kip}$$

$$P_{rt} := A_{rt} \cdot F_{y\_rebar} = 357 \text{ kip}$$

$$P_{rb} := A_{rb} \cdot F_{y\_rebar} = 223.2 \text{ kip}$$

Forces in each component of the composite girder at the plastic moment in negative bending



$$D := D_{girder} - 2 t_{flange} = 33.98 \text{ in}$$

### Case I - PNA in Web

$$Y_{web} := \frac{(D)}{2} \cdot \left( \frac{P_t + P_{botplate} - P_c - P_{topplate} - P_{rt} - P_{rb}}{P_w} + 1 \right) = 10 \text{ in}$$

$$y_{web} := t_{botplate} + D_{girder} - t_{flange} - Y_{web} = 35.34 \text{ in}$$

Distance to the plastic neutral axis if it is located in the web

$$d_{comp} := \frac{t_{flange}}{2} + Y_{web}$$

$$d_{topplate} := \frac{t_{topplate}}{2} + t_{flange} + Y_{web}$$

$$d_{ten} := y_{web} - t_{botplate} - \frac{t_{flange}}{2}$$

$$d_{botplate} := y_{web} - \frac{t_{botplate}}{2}$$

$$dd_{rt} := Y_{web} + H + Deck_T - d_{rt}$$

$$dd_{rb} := Y_{web} + H + Deck_T - d_{rb}$$

Plastic moment if the PNA is located in the web

$$M_{p_{web}} := \frac{P_w}{2 \cdot D} \cdot \left( Y_{web}^2 + (D - Y_{web})^2 \right) + (P_c \cdot \langle d_{comp} \rangle + P_{topplate} \cdot \langle d_{topplate} \rangle + P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle + P_{rt} \cdot \langle dd_{rt} \rangle + P_{rb} \cdot \langle dd_{rb} \rangle) = (6.581 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

### Case II - PNA in Flange

$$Y_{flange} := \left( \frac{t_{flange} + t_{topplate}}{2} \right) \cdot \left( \frac{P_w + P_t + P_{botplate} - P_{rt} - P_{rb}}{P_c + P_{topplate}} + 1 \right) = 14.746 \text{ in}$$

$$y_{flange} := t_{botplate} + D_{girder} - Y_{flange} = 31.954 \text{ in}$$

Distance to the plastic neutral axis if it is located in the flange

$$d_Y := t_{flange} + t_{topplate} - Y_{flange}$$

$$d_{web} := \frac{D_{girder}}{2} - Y_{flange}$$

$$d_{ten} := y_{flange} - t_{botplate} - \frac{t_{flange}}{2}$$

$$d_{bot} := y_{flange} - \frac{t_{botplate}}{2}$$



$$dd_{rt} := H - t_{topplate} - t_{flange} + Y_{flange} + Deck_T - d_{rt}$$

$$dd_{rb} := H - t_{topplate} - t_{flange} + Y_{flange} + Deck_T - d_{rb}$$

Plastic moment if the PNA is located in the flange

$$M_{p\_flange} := \frac{P_c + P_{topplate}}{2 \langle t_{flange} + t_{topplate} \rangle} \cdot (Y_{flange}^2 + \langle d_Y \rangle^2) + \langle P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{bot} \rangle + P_w \cdot \langle d_{web} \rangle + P_{rt} \cdot \langle dd_{rt} \rangle + P_{rb} \cdot \langle dd_{rb} \rangle \rangle = (4.853 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

$$M_{p\_Location} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_{rt} + P_{rb} \\ \quad \text{"PNA in Web"} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_{rt} + P_{rb} \\ \quad \quad \text{"PNA in Flange"} \\ \quad \text{else} \\ \quad \quad \text{"Otherwise"} \end{cases} = \text{"PNA in Web"}$$

$$M_{p\_neg} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_{rt} + P_{rb} \\ \quad M_{p\_web} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_{rt} + P_{rb} \\ \quad \quad M_{p\_flange} \\ \quad \text{else} \\ \quad \quad M_{p\_slab} \end{cases} = (6.581 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Plastic moment of the composite section in negative bending

### Nominal Flexural Resistance - Compression Flange Discretely Braced by Braces

#### Service Limit State

$$f_{f\_top\_neg} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi}$$

$$f_{f\_bot\_neg} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi}$$

Allowable Stress in the top flange  
Allowable Stress in the bottom flange  
 $f_l$  term can be ignored



### Strength Limit State

$$\lambda_f := \frac{W_{flange}}{2 \cdot t_{flange}} = 4.485$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 9.152$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 13.487$$

$$\lambda_{rw} := 5.7 \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 137.274$$

$$D_c := y - t_{flange} - t_{botplate} = 19.372 \text{ in}$$

$$a_{wc} := \frac{2 D_c \cdot t_{web}}{W_{flange} \cdot t_{flange}} = 1.938$$

$$R_h := 1$$

$$D_n := \frac{D_{girder}}{2} - t_{flange} = 16.99 \text{ in}$$

$$A_{fn} := t_{flange} \cdot W_{flange} = 16.592 \text{ in}^2$$

$$\rho := \frac{F_y}{F_{yflange}} = 1 \quad \beta := \frac{2 \cdot D_n \cdot t_{web}}{A_{fn}} = 1.7$$

$$R_h := \frac{12 + \beta \cdot (3 \rho - \rho^3)}{12 + 2 \beta} = 1$$

Hybrid Factor, 1 if Fy of flange and web are the same, use value above if different

$$R_b := \left\| \begin{array}{l} \text{if } \frac{2 \cdot D_c}{t_{web}} \leq \lambda_{rw} \\ 1 \\ \text{else} \\ 1 - \left( \frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \left( \frac{2 D_c}{t_{web}} - \lambda_{rw} \right) \end{array} \right\| = 1$$

$$F_{nc\_LB} := \left\| \begin{array}{l} \text{if } \lambda_f \leq \lambda_{pf} \\ R_h \cdot R_b \cdot F_{yflange} \\ \text{else} \\ \left( 1 - \left( 1 - \frac{0.7 F_{yflange}}{R_h \cdot F_{yflange}} \right) \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right) \cdot R_b \cdot R_h \cdot F_{yflange} \end{array} \right\| = 50 \text{ ksi}$$

Allowable stress in the compression flange for local bucking

$$r_t := \frac{W_{flange}}{\sqrt{12 \left( 1 + \frac{1}{3} \cdot \frac{D_c \cdot t_{web}}{W_{flange} \cdot t_{flange}} \right)}} = 3.062 \text{ in}$$



$$L_b := S_{brace\_neg} = 11.5 \text{ ft}$$

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 6.145 \text{ ft}$$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 19.305 \text{ ft}$$

$$C_b := 1$$

$$F_{nc\_LTB} := \begin{cases} \text{if } L_b \leq L_p \\ R_b \cdot R_h \cdot F_{yflange} \\ \text{else if } L_p < L_b \leq L_r \\ C_b \cdot \left( 1 - \left( 1 - \frac{F_{yflange}}{R_h \cdot F_{yflange}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right) \cdot R_h \cdot R_b \cdot F_{yflange} \\ \text{else} \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_{steel}}{\left( \frac{L_b}{r_t} \right)^2} \end{cases} = 50 \text{ ksi}$$

$$F_{nc} := \min(F_{nc\_LB}, F_{nc\_LTB}, R_h \cdot R_b \cdot F_{yflange}) = 50 \text{ ksi}$$

$$F_{nt} := R_h \cdot F_{yflange} = 50 \text{ ksi}$$

### Nominal Shear Resistance

#### Strength Limit State

$$V_p := 0.58 \cdot F_y \cdot D \cdot t_{web} = 817.899 \text{ kip}$$

$$k := 5 + \frac{5}{\left( \frac{d_0}{D} \right)^2} = 5.303$$

$$k_{end} := 5 + \frac{5}{\left( \frac{d_{0\_end}}{D} \right)^2} = 5.303$$

Unbraced length of the girders

Beam bending coefficient

Allowable stress in the compression flange for lateral torsional bucking

Allowable stress in the compression flange for negative bending

Allowable stress in the tension flange for negative bending

Shear capacity of the web

k value for interior of stiffened web

k value for end section of stiffened web



$$C := \begin{cases} \text{if } \frac{D}{t_{web}} \leq 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \quad \parallel 1 \\ \text{else if } 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} < \frac{D}{t_{web}} \leq 1.40 \cdot \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \quad \parallel \frac{1.12}{\left(\frac{D}{t_{web}}\right)} \sqrt{\frac{E_{steel} \cdot k}{F_y}} \\ \text{else} \\ \quad \parallel \frac{1.57}{\left(\frac{D}{t_{web}}\right)^2} \left(\frac{E_{steel} \cdot k}{F_y}\right) \end{cases} = 1$$

C value for interior of stiffened web

$$C_{end} := \begin{cases} \text{if } \frac{D}{t_{web}} \leq 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ \quad \parallel 1 \\ \text{else if } 1.12 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} < \frac{D}{t_{web}} \leq 1.40 \cdot \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ \quad \parallel \frac{1.12}{\left(\frac{D}{t_{web}}\right)} \sqrt{\frac{E_{steel} \cdot k_{end}}{F_y}} \\ \text{else} \\ \quad \parallel \frac{1.57}{\left(\frac{D}{t_{web}}\right)^2} \left(\frac{E_{steel} \cdot k_{end}}{F_y}\right) \end{cases} = 1$$

C value for end section of stiffened web

$$V_{n\_int\_neg} := \begin{cases} \text{if } \frac{2 D \cdot t_{web}}{(2 W_{flange} \cdot t_{flange})} \leq 2.5 \\ \quad \parallel V_p \cdot \left( C + \frac{0.87 (1 - C)}{\sqrt{1 + \left(\frac{d_0}{r}\right)^2}} \right) \end{cases} = 817.899 \text{ kip}$$

Shear capacity of interior section of stiffened girder



$$\left\| \left\| \begin{array}{l} \text{else} \\ V_p \cdot \left( C + \frac{0.87 (1-C)}{\sqrt{1 + \left( \frac{d_0}{D} \right)^2} + \frac{d_0}{D}} \right) \end{array} \right\| \right\|$$

### Composite Section Properties Exterior Girder

$$d_{topplate} := \frac{t_{topplate}}{2} + D_{girder} + t_{botplate}$$

$$d_{girder} := \frac{D_{girder}}{2} + t_{botplate}$$

$$d_{botplate} := \frac{t_{botplate}}{2}$$

$$dd_{rt} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt}$$

$$dd_{rb} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rb}$$

$$y := \frac{A_{topplate} \cdot \langle d_{topplate} \rangle + A_{girder} \cdot \langle d_{girder} \rangle + A_{botplate} \cdot \langle d_{botplate} \rangle + A_{rt\_ext} \cdot \langle dd_{rt} \rangle + A_{rb\_ext} \cdot \langle dd_{rb} \rangle}{A_{topplate} + A_{girder} + A_{botplate} + A_{rt\_ext} + A_{rb\_ext}} = 30.693 \text{ in}$$

Distance to neutral axis of composite section

$$d_{topplate} := t_{botplate} + D_{girder} + \frac{t_{topplate}}{2} - y$$

$$d_{girder} := t_{botplate} + \frac{D_{girder}}{2} - y$$

$$d_{botplate} := y - \frac{t_{botplate}}{2}$$

$$dd_{rt} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt} - y$$

$$dd_{rb} := t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rb} - y$$

Moment of inertia of composite section

$$I_{comp\_neg\_ext} := I_x + A_{girder} \cdot \langle d_{girder} \rangle^2 + I_{topplate} + A_{topplate} \cdot \langle d_{topplate} \rangle^2 + I_{botplate} + A_{botplate} \cdot \langle d_{botplate} \rangle^2 + A_{rt\_ext} \cdot \langle dd_{rt} \rangle^2 + A_{rb\_ext} \cdot \langle dd_{rb} \rangle^2 = (3.031 \cdot 10^4) \text{ in}^4$$



$$S_{x\_comp\_neg\_ext\_rt} := \frac{I_{comp\_neg\_ext}}{(t_{botplate} + D_{girder} - t_{flange} + H + Deck_T - d_{rt} - y)} = (1.369 \cdot 10^3) \text{ in}^3$$

Section Modulus to top reinforcing steel

$$S_{x\_comp\_neg\_ext\_tf} := \frac{I_{comp\_neg\_ext}}{(t_{botplate} + D_{girder} + t_{topplate} - y)} = (1.165 \cdot 10^3) \text{ in}^3$$

Section Modulus to top flange

$$S_{x\_comp\_neg\_ext\_bf} := \frac{I_{comp\_neg\_ext}}{y} = 987.53 \text{ in}^3$$

Section Modulus to bottom flange

### Plastic Moments Exterior Girder----- AASHTO Table D6.1-2

$$P_t := W_{flange} \cdot t_{flange} \cdot F_{yflange} = 829.6 \text{ kip}$$

$$P_w := (D_{girder} - 2 \cdot t_{flange}) \cdot t_{web} \cdot F_y = (1.41 \cdot 10^3) \text{ kip}$$

$$P_c := P_t = 829.6 \text{ kip}$$

$$P_{topplate} := A_{topplate} \cdot F_y = 562.5 \text{ kip}$$

$$P_{botplate} := A_{botplate} \cdot F_y = 562.5 \text{ kip}$$

Forces in each component of the composite girder at the plastic moment in negative bending

$$P_{rt} := A_{rt\_ext} \cdot F_{y\_rebar} = 394.2 \text{ kip}$$

$$P_{rb} := A_{rb\_ext} \cdot F_{y\_rebar} = 167.4 \text{ kip}$$

$$D := D_{girder} - 2 \cdot t_{flange} = 33.98 \text{ in}$$

### Case I - PNA in Web

$$Y_{web} := \frac{(D)}{2} \cdot \left( \frac{P_t + P_{botplate} - P_c - P_{topplate} - P_{rt} - P_{rb}}{P_w} + 1 \right) = 10.224 \text{ in}$$

$$y_{web} := t_{botplate} + D_{girder} - t_{flange} - Y_{web} = 35.116 \text{ in}$$

Distance to the plastic neutral axis if it is located in the web

$$d_{comp} := \frac{t_{flange}}{2} + Y_{web}$$

$$d_{topplate} := \frac{t_{topplate}}{2} + t_{flange} + Y_{web}$$

$$d_{ten} := y_{web} - t_{botplate} - \frac{t_{flange}}{2}$$

$$d_{botplate} := y_{web} - \frac{t_{botplate}}{2}$$



$$dd_{rt} := Y_{web} + H + Deck_T - d_{rt}$$

$$dd_{rb} := Y_{web} + H + Deck_T - d_{rb}$$

Plastic moment if the PNA is located in the web

$$M_{p\_web} := \frac{P_w}{2 \cdot D} \cdot (Y_{web}^2 + (D - Y_{web})^2) + (P_c \cdot \langle d_{comp} \rangle + P_{topplate} \cdot \langle d_{topplate} \rangle + P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{botplate} \rangle + P_{rt} \cdot \langle dd_{rt} \rangle + P_{rb} \cdot \langle dd_{rb} \rangle) = (6.569 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

### Case II - PNA in Flange

$$Y_{flange} := \left( \frac{t_{flange} + t_{topplate}}{2} \right) \cdot \left( \frac{P_w + P_t + P_{botplate} - P_{rt} - P_{rb}}{P_c + P_{topplate}} + 1 \right) = 14.822 \text{ in}$$

$$y_{flange} := t_{botplate} + D_{girder} - Y_{flange} = 31.878 \text{ in}$$

Distance to the plastic neutral axis if it is located in the flange

$$d_Y := t_{flange} + t_{topplate} - Y_{flange}$$

$$d_{web} := \frac{D_{girder}}{2} - Y_{flange}$$

$$d_{ten} := y_{flange} - t_{botplate} - \frac{t_{flange}}{2}$$

$$d_{bot} := y_{flange} - \frac{t_{botplate}}{2}$$

$$dd_{rt} := H - t_{topplate} - t_{flange} + Y_{flange} + Deck_T - d_{rt}$$

$$dd_{rb} := H - t_{topplate} - t_{flange} + Y_{flange} + Deck_T - d_{rb}$$

Plastic moment if the PNA is located in the flange

$$M_{p\_flange} := \frac{P_c + P_{topplate}}{2 \cdot \langle t_{flange} + t_{topplate} \rangle} \cdot (Y_{flange}^2 + \langle d_Y \rangle^2) + (P_t \cdot \langle d_{ten} \rangle + P_{botplate} \cdot \langle d_{bot} \rangle + P_w \cdot \langle d_{web} \rangle + P_{rt} \cdot \langle dd_{rt} \rangle + P_{rb} \cdot \langle dd_{rb} \rangle) = (4.787 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

$$M_{p\_Location} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_{rt} + P_{rb} \\ \quad \text{"PNA in Web"} \\ \text{else} \\ \quad \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_{rt} + P_{rb} \\ \quad \quad \text{"PNA in Flange"} \\ \quad \text{else} \\ \quad \quad \text{"Otherwise"} \end{cases} = \text{"PNA in Web"}$$



$$M_{p\_neg\_ext} := \begin{cases} \text{if } P_t + P_w + P_{botplate} \geq P_c + P_{topplate} + P_{rt} + P_{rb} \\ \quad \parallel M_{p\_web} \\ \text{else} \\ \quad \parallel \text{if } P_t + P_{botplate} + P_w + P_c + P_{topplate} \geq P_{rt} + P_{rb} \\ \quad \quad \parallel M_{p\_flange} \\ \quad \quad \parallel \text{else} \\ \quad \quad \parallel \text{"check PNA location"} \end{cases} = (6.569 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Plastic moment of the composite section in negative bending

## Nominal Flexural Resistance - Compression Flange Discretely Braced by Braces

### Service Limit State

$$f_{f\_top\_neg\_ext} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi} \quad \text{Allowable Stress in the top flange}$$

$$f_{f\_bot\_neg\_ext} := 0.95 \cdot F_{yflange} = 47.5 \text{ ksi} \quad \text{Allowable Stress in the bottom flange}$$

$f_l$  term can be ignored

### Strength Limit State

$$D_c := y - t_{flange} - t_{botplate} = 19.333 \text{ in}$$

$$a_{wc} := \frac{2 D_c \cdot t_{web}}{W_{flange} \cdot t_{flange}} = 1.934$$

$$R_h := 1$$

$$R_b := \begin{cases} \text{if } \frac{2 \cdot D_c}{t_{web}} \leq \lambda_{rw} \\ \quad \parallel 1 \\ \text{else} \\ \quad \parallel 1 - \left( \frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \left( \frac{2 D_c}{t_{web}} - \lambda_{rw} \right) \end{cases} = 1$$



$$F_{nc\_LB} := \begin{cases} \text{if } \lambda_f \leq \lambda_{pf} \\ R_h \cdot R_b \cdot F_{yflange} \\ \text{else} \\ \left( 1 - \left( 1 - \frac{0.7 F_{yflange}}{R_h \cdot F_{yflange}} \right) \left( \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right) \cdot R_b \cdot R_h \cdot F_{yflange} \end{cases} = 50 \text{ ksi}$$

Allowable stress in the compression flange for local bucking

$$r_t := \frac{W_{flange}}{\sqrt{12 \left( 1 + \frac{1}{3} \cdot \frac{D_c \cdot t_{web}}{W_{flange} \cdot t_{flange}} \right)}} = 3.063 \text{ in}$$

$$L_b := S_{brace\_neg} = 11.5 \text{ ft}$$

Unbraced length of the girders

$$L_p := 1.0 \cdot r_t \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 6.146 \text{ ft}$$

$$L_r := \pi \cdot r_t \cdot \sqrt{\frac{E_{steel}}{F_{yflange}}} = 19.31 \text{ ft}$$

$$C_b := 1$$

Beam bending coefficient

$$F_{nc\_LTB} := \begin{cases} \text{if } L_b \leq L_p \\ R_b \cdot R_h \cdot F_{yflange} \\ \text{else if } L_p < L_b \leq L_r \\ C_b \cdot \left( 1 - \left( 1 - \frac{F_{yflange}}{R_h \cdot F_{yflange}} \right) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right) \cdot R_h \cdot R_b \cdot F_{yflange} \\ \text{else} \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_{steel}}{\left( \frac{L_b}{r_t} \right)^2} \end{cases} = 50 \text{ ksi}$$

Allowable stress in the compression flange for lateral torsional bucking

$$F_{nc\_ext} := \min(F_{nc\_LB}, F_{nc\_LTB}, R_h \cdot R_b \cdot F_{yflange}) = 50 \text{ ksi}$$

Allowable stress in the compression flange for negative bending

$$F_{nt\_ext} := R_h \cdot F_{yflange} = 50 \text{ ksi}$$

Allowable stress in the tension flange for negative bending



## Dead Loads

### Steel Weight

#### End Girders

$$wt_{girder1} := 182 \cdot \text{plf}$$

$$l_{girder1} := 148 \cdot \text{ft}$$

# per linear foot of the girder section  
overall length of the girder section

#### Middle Girders

$$wt_{girder2} := 210 \cdot \text{plf}$$

$$l_{girder2} := 52 \cdot \text{ft}$$

# per linear foot of the girder section  
overall length of the girder section

$$wt_{girder} := \frac{(wt_{girder1} \cdot l_{girder1} + wt_{girder2} \cdot l_{girder2})}{Span_{total}} = 189.28 \text{ plf}$$

#### Girder Cover Plates

$$l_{topplate} := 17.33 \cdot \text{ft}$$

$$l_{botplate} := 17.33 \cdot \text{ft}$$

length of the top plates on the girders  
length of the bottom plates on the girders

$$wt_{plates} := \frac{(l_{topplate} \cdot W_{topplate} \cdot t_{topplate} + l_{botplate} \cdot W_{botplate} \cdot t_{botplate}) \cdot \gamma_{steel}}{Span_{total}} = 6.634 \text{ plf}$$

#### Diaphragms

$$N_{xframes} := 9$$

$$N_{sections} := 2.5$$

$$wt_{xframe} := 10 \cdot \text{plf}$$

$$l_{xframe} := 8 \cdot \text{ft}$$

# of diaphragms per girder  
# of members per brace (2 for cross brace)  
LB per lf of each brace member  
Length of each brace member

$$wt_{diaphragm} := \frac{N_{xframes} \cdot N_{sections} \cdot wt_{xframe} \cdot l_{xframe}}{Span_{total}} = 9 \text{ plf}$$

$$wt_{diaphragm\_ext} := \frac{0.5 \cdot N_{xframes} \cdot N_{sections} \cdot wt_{xframe} \cdot l_{xframe}}{Span_{total}} = 4.5 \text{ plf}$$

#### Connection Plates

##### Girder Splice Plates

$$w_{sp} := 16 \cdot \text{in}$$

$$t_{sp} := .75 \cdot \text{in}$$

Width of the girder splice plates  
Thickness of the girder splice plates



$$l_{sp} := 33 \cdot \text{in}$$

$$N_{sp} := 12$$

$$wt_{sp} := w_{sp} \cdot t_{sp} \cdot l_{sp} \cdot \gamma_{steel} = (1.101 \cdot 10^3) \frac{m}{s^2} \cdot lb$$

### Diaphragm Connection Plates

$$w_{cp} := 8 \cdot \text{in}$$

$$t_{cp} := \frac{3}{8} \cdot \text{in}$$

$$l_{cp} := 33 \cdot \text{in}$$

$$N_{cp} := N_{frames} \cdot 2 = 18$$

$$wt_{cp} := w_{cp} \cdot t_{cp} \cdot l_{cp} \cdot \gamma_{steel} = 275.301 \frac{m}{s^2} \cdot lb$$

$$wt_p := \frac{wt_{sp} \cdot N_{sp} + wt_{cp} \cdot N_{cp}}{Span_{total}} = 9.264 \text{ plf}$$

### Miscellaneous

$$wt_{misc\_ext} := 5 \cdot \text{plf}$$

$$wt_{misc\_int} := 10 \cdot \text{plf}$$

### Concrete Weight

#### Deck

$$wt_{deck} := \frac{Deck_T \cdot Bridge_W \cdot \gamma_{conc}}{N_{beams}} = 779.875 \text{ plf}$$

#### Haunches

$$H_w := 12 \cdot \text{in}$$

$$wt_{haunch} := (H \cdot H_w - W_{flange} \cdot t_{flange}) \cdot \gamma_{conc} = 7.717 \text{ plf}$$

length of the girder splice plates  
# of girder splices plates per girder

Width of the diaphragm connection plates

Thickness of the diaphragm connection plates  
Length of the diaphragm connection plates

# of diaphragms per girder

Weight of additional steel per  
girder (bolts, welds, studs)

$$Deck_T = 8.5 \text{ in}$$

$$Bridge_W = 440.4 \text{ in}$$

$$\gamma_{conc} = 150 \text{ pcf}$$

Haunch width



## Superimposed Dead Loads

### Curbs

$$CurbR_w := 22 \cdot \text{in}$$

$$CurbL_w := 22 \cdot \text{in}$$

$$Curb_{ht} := 12 \cdot \text{in}$$

Width of right curb including sidewalk

Width of left curb including sidewalk

Height of curbs

$$wt_{curb} := \frac{(CurbR_w + CurbL_w) \cdot Curb_{ht} \cdot \gamma_{conc}}{N_{beams}} = 110 \text{ plf}$$

### Guardrail Loads

$$Rail := 20 \cdot \text{plf}$$

$$N_{rails} := 2$$

Weight of each guardrails

No. of guardrails

$$wt_{rail} := \frac{N_{rails} Rail}{N_{beams}} = 8 \text{ plf}$$

### Wearing Surface

$$wt_{ws} := \left\{ \begin{array}{l} \text{if } WearingBit_T \neq 0 \\ \frac{WearingBit_T \cdot Roadway_W \cdot \gamma_{bit}}{N_{beams}} \\ \text{else} \\ \frac{WearingConc_T \cdot Roadway_W \cdot \gamma_{conc}}{N_{beams}} \end{array} \right\} = 231 \text{ plf}$$

Weight of wearing surface

$$wt_{DC1} := wt_{girder} + wt_{plates} + wt_{diaphragm} + wt_p + wt_{deck} + wt_{haunch} + wt_{misc\_int} = (1.012 \cdot 10^3) \text{ plf} \quad \text{Weight of long term dead loads (interior girder)}$$

$$wt_{DC1\_ext} := wt_{girder} + wt_{plates} + wt_{diaphragm\_ext} + wt_p + wt_{deck} + wt_{haunch} + wt_{misc\_ext} = (1.002 \cdot 10^3) \text{ plf} \quad \text{Weight of long term dead loads (exterior girder)}$$

$$wt_{DC2} := wt_{curb} + wt_{rail} = 118 \text{ plf} \quad \text{Weight of superimposed dead loads}$$



## CAPACITY CHECKS

### Interior Girder - Positive Bending

$$L := \text{Span}_{L1} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 707 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 82.5 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 161 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{ws}$

$$M_{LL} := 1550 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{pos\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_pos} \cdot \text{ft}}{12.0 L \cdot \text{Deck}_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_pos} \cdot \text{ft}}{12.0 L \cdot \text{Deck}_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.559$$

$$DC1 := M_{DC1}$$

$$DC2 := M_{DC2}$$

$$DW := M_{DW}$$

$$LL\_IM := M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{pos\_int}$$

Load Effects due to bridge moments

### Strength I - Inventory

$$F_{ad} := F_{yflange} - \gamma_{STI\_inv_0} \cdot \frac{M_{DC1}}{S_{x\_pos}} - \gamma_{STI\_inv_0} \cdot \frac{M_{DC2}}{S_{xLTC}} - \gamma_{STI\_inv_1} \cdot \frac{M_{DW}}{S_{xLTC}} = 28.001 \text{ ksi}$$

Additional stress in the girder before yielding occurs

$$M_{ad} := \frac{F_{ad} \cdot S_{xSTC}}{\gamma_{STI\_inv_2}} = (1.229 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Additional moment on girder before yielding occurs

$$M_y := \gamma_{STI\_inv_0} \cdot M_{DC1} + \gamma_{STI\_inv_0} \cdot M_{DC2} + \gamma_{STI\_inv_1} \cdot M_{DW} + \gamma_{STI\_inv_2} \cdot M_{ad} = (3.38 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Moment on girder when yielding occurs

$$M_n := \min (M_{n\_pos}, 1.3 \cdot R_h \cdot M_y) = (4.394 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Moment capacity of composite girder in positive bending

$$\varphi := 1$$

LRFD Resistance Factor

$$C := \varphi_c \cdot \varphi_c \cdot \varphi \cdot M_n = (4.394 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Strength Limit State Capacity



$$RF_{posM\_int\_STI\_INV} := \frac{C - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 1.57$$

Rating Factor for positive bending of an interior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$F_{ad} := F_{yflange} - \gamma_{STI\_op_0} \cdot \frac{M_{DC1}}{S_{x\_pos}} - \gamma_{STI\_op_0} \cdot \frac{M_{DC2}}{S_{xLTC}} - \gamma_{STI\_op_1} \cdot \frac{M_{DW}}{S_{xLTC}} = 28.001 \text{ ksi}$$

Additional stress in the girder before yielding occurs

$$M_{ad} := \frac{F_{ad} \cdot S_{xSTC}}{\gamma_{STI\_op_2}} = (1.594 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Additional moment on girder before yielding occurs

$$M_y := \gamma_{STI\_op_0} \cdot M_{DC1} + \gamma_{STI\_op_0} \cdot M_{DC2} + \gamma_{STI\_op_1} \cdot M_{DW} + \gamma_{STI\_op_2} \cdot M_{ad} = (3.38 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Moment on girder when yielding occurs

$$M_n := \min(M_{n\_pos}, 1.3 \cdot R_h \cdot M_y) = (4.394 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Moment capacity of composite girder in positive bending

$$\phi := 1$$

LRFD Resistance Factor

$$C := \phi_c \cdot \phi_c \cdot \phi \cdot M_n = (4.394 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

Strength Limit State Capacity

$$RF_{posM\_int\_STI\_OP} := \frac{C - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 2.036$$

Rating Factor for positive bending of an interior girder in the STRENGTH I - OPERATING load case

### Service II

$$C := f_{f\_bot\_pos} = 47.5 \text{ ksi}$$

Service Limit State Capacity - Allowable stress in tension flange

$$DC1 := \frac{M_{DC1}}{S_{x\_pos}} = 13.627 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{xLTC}} = 1.189 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{xLTC}} = 2.319 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{pos\_int}}{S_{xSTC}} = 14.99 \text{ ksi}$$



$$RF_{posM\_int\_SEII} := \frac{C - \gamma_{SEII_0} \cdot DC1 - \gamma_{SEII_0} \cdot DC2 - \gamma_{SEII_1} \cdot DW}{\gamma_{SEII_2} \cdot LL\_IM} = 1.558$$

Rating Factor for positive bending of an interior girder in the SERVICE II load case.

### Exterior Girder - Positive Bending

$$L := Span_{L1} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 700 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

$$M_{DC2} := 82.5 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 161 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 1550 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

$$lever\_rule := .55$$

Distribution factor of Exterior Girder determined using the Lever Rule

$$d_e := 0 \cdot \text{ft}$$

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier. Taken as positive if web is inboard of curb or traffic barrier.

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 0.77$$

$$g_{pos\_ext} := \max(e_{dis} \cdot g_{pos\_int}, lever\_rule) = 0.55$$

Moment distribution factor for exterior steel girders

$$DC1 := M_{DC1\_ext}$$

$$DC2 := M_{DC2}$$

$$DW := M_{DW}$$

$$LL\_IM := M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{pos\_ext}$$

Load Effects due to bridge moments

Moment distribution factor for interior steel girders

### Strength I - Inventory

$$F_{ad} := F_{yflange} - \gamma_{STI\_inv_0} \cdot \frac{M_{DC1\_ext}}{S_{x\_pos}} - \gamma_{STI\_inv_0} \cdot \frac{M_{DC2}}{S_{xLTC\_ext}} - \gamma_{STI\_inv_1} \cdot \frac{M_{DW}}{S_{xLTC\_ext}} = 28.107 \text{ ksi}$$

Additional stress in the girder before yielding occurs

$$M_{ad} := \frac{F_{ad} \cdot S_{xSTC\_ext}}{\gamma_{STI\_inv_2}} = (1.223 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Additional moment on girder before yielding occurs

$$M_y := \gamma_{STI\_inv_0} \cdot M_{DC1\_ext} + \gamma_{STI\_inv_0} \cdot M_{DC2} + \gamma_{STI\_inv_1} \cdot M_{DW} + \gamma_{STI\_inv_2} \cdot M_{ad} = (3.36 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

Moment on girder when yielding occurs



$$M_n := \min(M_{n\_pos\_ext}, 1.3 \cdot R_h \cdot M_y) = (4.368 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

$$\varphi := 1$$

$$C := \varphi_c \cdot \varphi_c \cdot \varphi \cdot M_n = (4.368 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

LRFD Resistance Factor  
Strength Limit State Capacity

Moment capacity of composite girder in positive bending

$$RF_{posM\_ext\_STI\_INV} := \frac{C - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 1.587$$

Rating Factor for positive bending of an exterior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$F_{ad} := F_{yflange} - \gamma_{STI\_op_0} \cdot \frac{M_{DC1\_ext}}{S_{x\_pos}} - \gamma_{STI\_op_0} \cdot \frac{M_{DC2}}{S_{xLTC\_ext}} - \gamma_{STI\_op_1} \cdot \frac{M_{DW}}{S_{xLTC\_ext}} = 28.107 \text{ ksi}$$

$$M_{ad} := \frac{F_{ad} \cdot S_{xSTC\_ext}}{\gamma_{STI\_op_2}} = (1.586 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

$$M_y := \gamma_{STI\_op_0} \cdot M_{DC1\_ext} + \gamma_{STI\_op_0} \cdot M_{DC2} + \gamma_{STI\_op_1} \cdot M_{DW} + \gamma_{STI\_op_2} \cdot M_{ad} = (3.36 \cdot 10^3) \text{ kip} \cdot \text{ft}$$

$$M_n := \min(M_{n\_pos\_ext}, 1.3 \cdot R_h \cdot M_y) = (4.368 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

$$\varphi := 1$$

$$C := \varphi_c \cdot \varphi_c \cdot \varphi \cdot M_n = (4.368 \cdot 10^3) \text{ (kip} \cdot \text{ft)}$$

LRFD Resistance Factor  
Strength Limit State Capacity

$$RF_{posM\_ext\_STI\_OP} := \frac{C - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 2.057$$

Rating Factor for positive bending of an exterior girder in the STRENGTH I - OPERATING load case

### Service II

$$C := f_{f\_bot\_pos\_ext} = 47.5 \text{ ksi}$$

Service Limit State Capacity - Allowable stress in tension flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_pos}} = 13.492 \text{ ksi}$$

$$M_{DC2}$$



$$DC2 := \frac{M_{DC2}}{S_{xLTC\_ext}} = 1.204 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{xLTC\_ext}} = 2.349 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{pos\_ext}}{S_{xSTC\_ext}} = 14.888 \text{ ksi}$$

$$RF_{posM\_ext\_SEI} := \frac{C - \gamma_{SEI0} \cdot DC1 - \gamma_{SEI0} \cdot DC2 - \gamma_{SEI1} \cdot DW}{\gamma_{SEI2} \cdot LL\_IM} = 1.574$$

Rating Factor for positive bending of an exterior girder in the SERVICE II load case.

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

$$M_{LL} := 1541 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$M_{DW}$$



$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 14.347 \text{ ksi}$$

### Strength I - Inventory

$$RF_{negM\_CF\_int\_STI\_INV} := \frac{F_{nc} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 0.917$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$RF_{negM\_CF\_int\_STI\_OP} := \frac{F_{nc} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 1.189$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the STRENGTH I - OPERATING load case

### Service II

$$RF_{negM\_CF\_int\_SEII} := \frac{f_{f\_bot\_neg} - \gamma_{SEII_0} \cdot DC1 - \gamma_{SEII_0} \cdot DC2 - \gamma_{SEII_1} \cdot DW}{\gamma_{SEII_2} \cdot LL\_IM} = 1.428$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the SERVICE II load case

### Tension Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_tf}} = 13.11 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_tf}} = 1.531 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_tf}} = 2.99 \text{ ksi}$$

-- ( IM \



$$LL_{IM} := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_tf}} = 12.123 \text{ ksi}$$

### Strength I - Inventory

$$RF_{negM\_TF\_int\_STI\_INV} := \frac{F_{nt} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL_{IM}} = 1.283$$

Rating Factor for allowable stress in the tension flange during negative bending of an interior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$RF_{negM\_TF\_int\_STI\_OP} := \frac{F_{nt} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL_{IM}} = 1.663$$

Rating Factor for allowable stress in the tension flange during negative bending of an interior girder in the STRENGTH I - OPERATING load case

### Service II

$$RF_{negM\_TF\_int\_SEII} := \frac{f_{f\_top\_neg} - \gamma_{SEII_0} \cdot DC1 - \gamma_{SEII_0} \cdot DC2 - \gamma_{SEII_1} \cdot DW}{\gamma_{SEII_2} \cdot LL_{IM}} = 1.895$$

Rating Factor for allowable stress in the tension flange during negative bending of an interior girder in the SERVICE II load case

### Reinforcing Steel

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_rt}} = 11.162 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_rt}} = 1.303 \text{ ksi}$$

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_rt}} = 2.545 \text{ ksi}$$

$$LL_{IM} := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_rt}} = 10.321 \text{ ksi}$$

Load Effects due to bridge moments



### Strength I - Inventory

$$RF_{negM\_RS\_int\_STI\_INV} := \frac{F_{y\_rebar} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 2.248$$

Rating Factor stress in the rebar during negative bending of an interior girder in the STRENGTH I - INVENTORY load case (not a requirement of AASHTO or MaineDOT load ratings)

### Strength I - Operating

$$RF_{negM\_RS\_int\_STI\_OP} := \frac{F_{y\_rebar} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 2.914$$

Rating Factor for stress in the rebar during negative bending of an interior girder in the STRENGTH I - OPERATING load case (not a requirement of AASHTO or MaineDOT load ratings)

### ***Exterior Girder - Negative Bending***

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 1541 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Factor

$$lever\_rule := .55$$

Distribution factor of Exterior Girder determined using the Lever Rule

$$d_e := 4 \cdot \text{ft}$$

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier. Taken as postivite if web is inboard or curb or traffic barrier.

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

### Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$



$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 17.343 \text{ ksi}$$

### Strength I - Inventory

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 0.766$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$RF_{negM\_CF\_ext\_STI\_OP} := \frac{F_{nc\_ext} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 0.993$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the STRENGTH I - OPERATING load case

### Service II

$$RF_{negM\_CF\_ext\_SEII} := \frac{f_{f\_bot\_neg\_ext} - \gamma_{SEII_0} \cdot DC1 - \gamma_{SEII_0} \cdot DC2 - \gamma_{SEII_1} \cdot DW}{\gamma_{SEII_2} \cdot LL\_IM} = 1.189$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the SERVICE II load case

### Tension Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_ext\_tf}} = 13.138 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_tf}} = 1.534 \text{ ksi}$$

Load Effects due to bridge moments

$$M_{DW}$$



$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_tf}} = 2.996 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_tf}} = 14.695 \text{ ksi}$$

### Strength I - Inventory

$$RF_{negM\_TF\_ext\_STI\_INV} := \frac{F_{nt\_ext} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 1.056$$

Rating Factor for allowable stress in the tension flange during negative bending of an exterior girder in the STRENGTH I - INVENTORY load case

### Strength I - Operating

$$RF_{negM\_TF\_ext\_STI\_OP} := \frac{F_{nt\_ext} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 1.369$$

Rating Factor for allowable stress in the tension flange during negative bending of an exterior girder in the STRENGTH I - OPERATING load case

### Service II

$$RF_{negM\_TF\_ext\_SEII} := \frac{f_{f\_top\_neg\_ext} - \gamma_{SEII_0} \cdot DC1 - \gamma_{SEII_0} \cdot DC2 - \gamma_{SEII_1} \cdot DW}{\gamma_{SEII_2} \cdot LL\_IM} = 1.562$$

Rating Factor for allowable stress in the tension flange during negative bending of an exterior girder in the SERVICE II load case

### Reinforcing Steel

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_ext\_rt}} = 11.188 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_rt}} = 1.306 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_rt}} = 2.551 \text{ ksi}$$

-- ( IM )



$$LL_{IM} := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_rt}} = 12.514 \text{ ksi}$$

### Strength I - Inventory

$$RF_{negM\_RS\_ext\_STI\_INV} := \frac{F_{y\_rebar} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL_{IM}} = 1.852$$

Rating Factor for stress in the rebar during negative bending of an exterior girder in the STRENGTH I - INVENTORY load case (not a requirement of AASHTO or MaineDOT load ratings)

### Strength I - Operating

$$RF_{negM\_RS\_ext\_STI\_OP} := \frac{F_{y\_rebar} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL_{IM}} = 2.401$$

Rating Factor for stress in the rebar during negative bending of an exterior girder in the STRENGTH I - OPERATING load case (not a requirement of AASHTO or MaineDOT load ratings)

### Girder Shear - Interior Support

$$L := Span_{L1} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$V_{DC1} := 63.4 \cdot \text{kip}$$

$$V_{DC2} := 7.39 \cdot \text{kip}$$

$$V_{DW} := 14.5 \cdot \text{kip}$$

$$V_{LL} := 97 \cdot \text{kip}$$

Max Shear output from Bridge Load Analysis using  $wt_{DC1}$

Max Shear output from Bridge Load Analysis using  $wt_{DC2}$

Max Shear output from Bridge Load Analysis using  $wt_{DW}$

Max Shear output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Factor

$$g_{shear\_int} := \max \left( 0.36 + \left( \frac{S_{girders}}{25 \cdot \text{ft}} \right), 0.2 + \left( \frac{S_{girders}}{12 \cdot \text{ft}} \right) - \left( \frac{S_{girders}}{35 \cdot \text{ft}} \right)^2 \right) = 0.79$$

Shear distribution factor for interior steel girders

$$DC1 := V_{DC1}$$

$$DC2 := V_{DC2}$$

$$DW := V_{DW}$$

$$LL_{IM} := V_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{shear\_int}$$

Load Effects due to bridge shears



## Strength I - Inventory

$$RF_{shear\_int\_STI\_INV} := \frac{V_{n\_int\_neg} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 3.965$$

Rating Factor for allowable shear of an interior web section STRENGTH I - INVENTORY load case

## Strength I - Operating

$$RF_{shear\_int\_STI\_OP} := \frac{V_{n\_int\_neg} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 5.14$$

Rating Factor for allowable shear of an interior web section STRENGTH I - OPERATING load case

## Girder Shear - End Support

$$L := Span_{L1} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$V_{DC1\_ext} := 37.8 \cdot kip$$

Max Shear output from Bridge Load Analysis using  $wt_{DC1\_ext}$

$$V_{DC2} := 4.41 \cdot kip$$

Max Shear output from Bridge Load Analysis using  $wt_{DC2}$

$$V_{DW} := 8.64 \cdot kip$$

Max Shear output from Bridge Load Analysis using  $wt_{DW}$

$$V_{LL} := 81 \cdot kip$$

Max Shear output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Impact Factor

$$lever\_rule := .715$$

Distribution factor of Exterior Girder determined using the Lever Rule

$$d_e := 4 \cdot ft$$

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier. Taken as postivite if web is inboard or curb or traffic barrier.

$$e_{dis} := 0.6 + \frac{d_e}{10 \cdot ft} = 1$$

$$g_{shear\_ext} := \max(e_{dis} \cdot g_{shear\_int}, lever\_rule) = 0.79$$

Shear distribution factor for an exterior girder

$$DC1 := V_{DC1}$$

$$DC2 := V_{DC2}$$

$$DW := V_{DW}$$

$$LL\_IM := V_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{shear\_ext}$$

Load Effects due to bridge shears



### Strength I - Inventory

$$RF_{shear\_end\_STI\_INV} := \frac{V_{n\_end} - \gamma_{STI\_inv_0} \cdot DC1 - \gamma_{STI\_inv_0} \cdot DC2 - \gamma_{STI\_inv_1} \cdot DW}{\gamma_{STI\_inv_2} \cdot LL\_IM} = 4.133$$

Rating Factor for allowable shear of  
an end web section STRENGTH I -  
INVENTORY load case

### Strength I - Operating

$$RF_{shear\_end\_STI\_OP} := \frac{V_{n\_end} - \gamma_{STI\_op_0} \cdot DC1 - \gamma_{STI\_op_0} \cdot DC2 - \gamma_{STI\_op_1} \cdot DW}{\gamma_{STI\_op_2} \cdot LL\_IM} = 5.357$$

Rating Factor for allowable shear of  
an end web section STRENGTH I -  
OPERATING load case



## CAPACITY CHECKS - MDOT LEGAL LOADS

### Configuration 1

#### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

$$M_{LL} := 877 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor  
for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

#### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 8.165 \text{ ksi}$$

#### Legal Load Configuration 1

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.015$$

Rating Factor for allowable stress in  
the compression flange during  
negative bending of an interior girder  
in the Legal Load Configuration 1



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 877 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$lever\_rule := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 9.87 \text{ ksi}$$

## Legal Load Configuration 1

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL_0} \cdot DC1 - \gamma_{LL_0} \cdot DC2 - \gamma_{LL_1} \cdot DW}{\gamma_{LL_2} \cdot LL\_IM} = 1.682$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 1.



## Configuration 2

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 828 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 7.709 \text{ ksi}$$

### Legal Load Configuration 2

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.134$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 2



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 828 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$lever\_rule := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 9.319 \text{ ksi}$$

## Legal Load Configuration 2

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL_0} \cdot DC1 - \gamma_{LL_0} \cdot DC2 - \gamma_{LL_1} \cdot DW}{\gamma_{LL_2} \cdot LL\_IM} = 1.782$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 2.



### Configuration 3

#### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 810 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

#### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 7.541 \text{ ksi}$$

#### Legal Load Configuration 3

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.182$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 3



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 810 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$\text{lever\_rule} := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, \text{lever\_rule}) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 9.116 \text{ ksi}$$

## Legal Load Configuration 3

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL_0} \cdot DC1 - \gamma_{LL_0} \cdot DC2 - \gamma_{LL_1} \cdot DW}{\gamma_{LL_2} \cdot LL\_IM} = 1.822$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 3.



## Configuration 4

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 808 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 7.523 \text{ ksi}$$

### Legal Load Configuration 4

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.187$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 4



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 808 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$lever\_rule := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 9.093 \text{ ksi}$$

## Legal Load Configuration 4

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL_0} \cdot DC1 - \gamma_{LL_0} \cdot DC2 - \gamma_{LL_1} \cdot DW}{\gamma_{LL_2} \cdot LL\_IM} = 1.826$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 4.



## Configuration 5

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 804 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 7.485 \text{ ksi}$$

### Legal Load Configuration 5

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.198$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 5



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 804 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$lever\_rule := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as positive if web is inboard of curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 9.048 \text{ ksi}$$

## Legal Load Configuration 5

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 1.835$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 5



## Configuration 6

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 727 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 6.769 \text{ ksi}$$

### Legal Load Configuration 6

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.431$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 6



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 727 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$\text{lever\_rule} := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, \text{lever\_rule}) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 8.182 \text{ ksi}$$

## Legal Load Configuration 6

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 2.03$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 6



## Configuration 7

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 570 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 5.307 \text{ ksi}$$

### Legal Load Configuration 7

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 3.1$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 7



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 570 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$\text{lever\_rule} := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, \text{lever\_rule}) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 6.415 \text{ ksi}$$

## Legal Load Configuration 7

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL_0} \cdot DC1 - \gamma_{LL_0} \cdot DC2 - \gamma_{LL_1} \cdot DW}{\gamma_{LL_2} \cdot LL\_IM} = 2.589$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 7



## Configuration 8

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 359 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 3.342 \text{ ksi}$$

### Legal Load Configuration 8

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 4.923$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Legal Load Configuration 8



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 359 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

$$lever\_rule := .55$$

$$d_e := 4 \cdot \text{ft}$$

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Dynamic Load Factor

Distribution factor of Exterior Girder determined using the Lever Rule

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

Taken as postivite if web is inboard or curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 4.04 \text{ ksi}$$

## Legal Load Configuration 8

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 4.11$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Legal Load Configuration 8



## Double Legal Load Condition for Neg. Bending

### Interior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1} := 1276 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 1076 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Allowance

Moment distribution factor for interior steel girders

$$g_{neg\_int} := \max \left( 0.06 + \left( \frac{S_{girders}}{14 \cdot \text{ft}} \right)^{0.4} \left( \frac{S_{girders}}{L} \right)^{0.3} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1}, 0.075 + \left( \frac{S_{girders}}{9.5 \cdot \text{ft}} \right)^{0.6} \left( \frac{S_{girders}}{L} \right)^{0.2} \left( \frac{K_{g\_neg} \cdot \text{ft}}{12.0 L \cdot Deck_T^3 \cdot \text{in}} \right)^{0.1} \right) = 0.576$$

### Compression Flange

$$DC1 := \frac{M_{DC1}}{S_{x\_comp\_neg\_bf}} = 15.515 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_bf}} = 1.812 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_bf}} = 3.538 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left( 1 + \frac{IM}{100} \right) \cdot g_{neg\_int}}{S_{x\_comp\_neg\_bf}} = 10.018 \text{ ksi}$$

### Double Legal Load Condition

$$RF_{negM\_CF\_int\_LLC1} := \frac{F_{nc} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 1.642$$

Rating Factor for allowable stress in the compression flange during negative bending of an interior girder in the Double Legal Load Condition



## Exterior Girder - Negative Bending

$$L := \frac{Span_{L1} + Span_{L2}}{2} = 100 \text{ ft}$$

Length of span in which moments are being evaluated

$$M_{DC1\_ext} := 1263 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC1\_ext}$

$$M_{DC2} := 149 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DC2}$

$$M_{DW} := 291 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using  $wt_{DW}$

$$M_{LL} := 1076 \cdot \text{kip} \cdot \text{ft}$$

Max Moment output from Bridge Load Analysis using HL-93

$$IM := 33$$

Dynamic Load Factor

$$lever\_rule := .55$$

Distribution factor of Exterior Girder determined using the Lever Rule

$$d_e := 4 \cdot \text{ft}$$

Horizontal Distance from web of exterior girder to interior edge of curb or traffic barrier.

$$e_{dis} := 0.77 + \frac{d_e}{9.1 \cdot \text{ft}} = 1.21$$

Taken as positive if web is inboard of curb or traffic barrier.

$$g_{neg\_ext} := \max(e_{dis} \cdot g_{neg\_int}, lever\_rule) = 0.696$$

Moment distribution factor for exterior steel girders

## Compression Flange

$$DC1 := \frac{M_{DC1\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 15.347 \text{ ksi}$$

$$DC2 := \frac{M_{DC2}}{S_{x\_comp\_neg\_ext\_bf}} = 1.811 \text{ ksi}$$

Load Effects due to bridge moments

$$DW := \frac{M_{DW}}{S_{x\_comp\_neg\_ext\_bf}} = 3.536 \text{ ksi}$$

$$LL\_IM := \frac{M_{LL} \cdot \left(1 + \frac{IM}{100}\right) \cdot g_{neg\_ext}}{S_{x\_comp\_neg\_ext\_bf}} = 12.11 \text{ ksi}$$

## Double Legal Load Condition

$$RF_{negM\_CF\_ext\_STI\_INV} := \frac{F_{nc\_ext} - \gamma_{LL0} \cdot DC1 - \gamma_{LL0} \cdot DC2 - \gamma_{LL1} \cdot DW}{\gamma_{LL2} \cdot LL\_IM} = 1.371$$

Rating Factor for allowable stress in the compression flange during negative bending of an exterior girder in the Double Legal Load Condition



## APPENDIX D

### STAAD Input Data and Moment/Shear Summary Report





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## Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0.000	0.000	0.000
2	74.000	0.000	0.000
3	100.000	0.000	0.000
4	126.000	0.000	0.000
5	200.000	0.000	0.000

## Beams

Beam	Node A	Node B	Length (ft)	Property	$\beta$ (degrees)
1	1	2	74.000	1	0
2	2	3	26.000	2	0
3	3	4	26.000	2	0
4	4	5	74.000	1	0

## Section Properties

Prop	Section	Area (in <sup>2</sup> )	I <sub>yy</sub> (in <sup>4</sup> )	I <sub>zz</sub> (in <sup>4</sup> )	J (in <sup>4</sup> )	Material
1	Prismatic General	95.000	29.5E+3	29.5E+3	29.5E+3	STEEL
2	Prismatic General	95.000	30.3E+3	30.3E+3	30.3E+3	STEEL

## Materials

Mat	Name	E (kip/in <sup>2</sup> )	$\nu$	Density (kip/in <sup>3</sup> )	$\alpha$ (1/°F)
1	STEEL	29E+3	0.300	0.000	6.5E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	9.9E -6
3	ALUMINUM	10E+3	0.330	0.000	12.8E -6
4	CONCRETE	3.15E+3	0.170	0.000	5.5E -6

## Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip*ft/deg)	rY (kip*ft/deg)	rZ (kip*ft/deg)
1	Fixed	Fixed	Fixed	-	-	-
3	Fixed	Fixed	Fixed	-	-	-
5	Fixed	Fixed	Fixed	-	-	-





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## Basic Load Cases

Number	Name
1	DC1
2	DC2
3	DW
4	DC1_EXT
5	DESIGN LANE
406	90% DESIGN LANE
1237	LEGAL LOAD DESIGN LANE

## Moving Load Definition : Type 1

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	14.000
8.000	14.000

## Moving Load Definition : Type 2

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	22.000
8.000	14.000

## Moving Load Definition : Type 3

Width (ft)
-

Force (kip)	Distance (ft)
32.000	-
32.000	30.000
8.000	14.000





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### **Moving Load Definition : Type 4**

Width (ft)
-

Force (kip)	Distance (ft)
25.000	-
25.000	4.000

### **Moving Load Definition : Type 5**

Width (ft)
-

Force (kip)	Distance (ft)
7.200	-
28.800	14.000
28.800	14.000
7.200	100.000
28.800	14.000
28.800	14.000

### **Moving Load Definition : Type 6**

Width (ft)
-

Force (kip)	Distance (ft)
10.000	-
20.000	10.000
20.000	4.000
16.700	20.000
16.700	4.000
16.600	4.000

### **Moving Load Definition : Type 7**

Width (ft)
-





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Force (kip)	Distance (ft)
10.000	-
16.600	10.000
16.700	4.000
16.700	4.000
17.000	10.000
17.000	20.333

### Moving Load Definition : Type 8

Width (ft)
-

Force (kip)	Distance (ft)
10.000	-
16.600	10.000
16.700	4.000
16.700	4.000
14.000	12.000
14.000	4.000

### Moving Load Definition : Type 9

Width (ft)
-

Force (kip)	Distance (ft)
10.000	-
16.100	10.000
24.200	10.000
18.900	10.000
18.800	4.000

### Moving Load Definition : Type 10

Width (ft)
-





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Force (kip)	Distance (ft)
10.000	-
22.000	10.000
22.000	4.000
17.000	16.000
17.000	4.000

### **Moving Load Definition : Type 11**

Width (ft)
-

Force (kip)	Distance (ft)
11.900	-
21.300	9.000
21.400	4.000
21.300	4.000

### **Moving Load Definition : Type 12**

Width (ft)
-

Force (kip)	Distance (ft)
13.400	-
23.000	10.000
23.000	4.000

### **Moving Load Definition : Type 13**

Width (ft)
-

Force (kip)	Distance (ft)
13.200	-
24.200	10.000





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## Moving Load Definition : Type 14

**Width**

(ft)

-

Force (kip)	Distance (ft)
8.925	-
15.975	9.000
16.050	4.000
15.975	4.000
8.925	30.000
15.975	9.000
16.050	4.000
15.975	4.000

## Beam Force Detail Summary

Sign convention as diagrams:- positive above line, negative below line except Fx where positive is compression. Distance d is given from beam end A.

	L/C	Beam	d (ft)	Axial	Shear		Torsion	Bending	
				Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
Max Fx	1:DC1	1	0.000	<b>0.000</b>	37.843	0.000	0.000	0.000	-0.000
Min Fx	1:DC1	1	0.000	<b>0.000</b>	37.843	0.000	0.000	0.000	-0.000
Max Fy	57:LOAD GENI	3	0.000	0.000	<b>96.968</b>	0.000	0.000	0.000	907.777
Min Fy	453:LOAD GENI	1	74.000	-0.000	<b>-64.567</b>	-0.000	-0.000	-0.000	-801.529
Max Fz	1:DC1	1	0.000	0.000	37.843	<b>0.000</b>	0.000	0.000	-0.000
Min Fz	1:DC1	1	0.000	0.000	37.843	<b>0.000</b>	0.000	0.000	-0.000
Max Mx	1:DC1	1	0.000	0.000	37.843	0.000	<b>0.000</b>	0.000	-0.000
Min Mx	1:DC1	1	0.000	0.000	37.843	0.000	<b>0.000</b>	0.000	-0.000
Max My	1:DC1	1	0.000	0.000	37.843	0.000	0.000	<b>0.000</b>	-0.000
Min My	1:DC1	1	0.000	0.000	37.843	0.000	0.000	<b>0.000</b>	-0.000
Max Mz	411:LOAD GENI	2	26.000	-0.000	-60.819	-0.000	-0.000	-0.000	<b>1.54E+3</b>
Min Mz	78:LOAD GENI	4	29.600	0.000	11.392	0.000	0.000	0.000	<b>-1.53E+3</b>

## Beam Maximum Forces by Section Property

Section		Axial	Shear		Torsion	Bending	
		Max Fx (kip)	Max Fy (kip)	Max Fz (kip)	Max Mx (kip-ft)	Max My (kip-ft)	Max Mz (kip-ft)
Prismatic General	Max +ve	0.000	81.631	0.000	0.000	0.000	649.135
	Max -ve	0.000	-64.567	0.000	0.000	0.000	-1.55E+3
Prismatic General	Max +ve	0.000	96.968	0.000	0.000	0.000	1.54E+3
	Max -ve	0.000	-64.567	0.000	0.000	0.000	-801.529





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## Beam Maximum Moments

*Distances to maxima are given from beam end A.*

L/C	Beam	Node A	Length (ft)		d (ft)	Max My (kip-ft)	d (ft)	Max Mz (kip-ft)
1:DC1	1	1	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	37.000	-707.472
	2	2	26.000	Max -ve	0.000	0.000	26.000	1.28E+3
				Max +ve	0.000	0.000	0.000	-29.515
	3	3	26.000	Max -ve	0.000	0.000	0.000	1.28E+3
				Max +ve	0.000	0.000	26.000	-29.515
	4	4	74.000	Max -ve	0.000	0.000	74.000	0.000
				Max +ve	0.000	0.000	37.000	-707.472
2:DC2	1	1	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	37.000	-82.492
	2	2	26.000	Max -ve	0.000	0.000	26.000	148.749
				Max +ve	0.000	0.000	0.000	-3.442
	3	3	26.000	Max -ve	0.000	0.000	0.000	148.749
				Max +ve	0.000	0.000	26.000	-3.442
	4	4	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	37.000	-82.492
3:DW	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	37.000	-161.488
	2	2	26.000	Max -ve	0.000	0.000	26.000	291.196
				Max +ve	0.000	0.000	0.000	-6.737
	3	3	26.000	Max -ve	0.000	0.000	0.000	291.196
				Max +ve	0.000	0.000	26.000	-6.737
	4	4	74.000	Max -ve	0.000	0.000	74.000	-0.000
				Max +ve	0.000	0.000	37.000	-161.488
4:DC1_EXT	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	37.000	-700.481
	2	2	26.000	Max -ve	0.000	0.000	26.000	1.26E+3
				Max +ve	0.000	0.000	0.000	-29.224
	3	3	26.000	Max -ve	0.000	0.000	0.000	1.26E+3
				Max +ve	0.000	0.000	26.000	-29.224
	4	4	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	37.000	-700.481
5:DESIGN LAN	1	1	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	37.000	-336.259
	2	2	26.000	Max -ve	0.000	0.000	26.000	606.342
				Max +ve	0.000	0.000	0.000	-14.029
	3	3	26.000	Max -ve	0.000	0.000	0.000	606.342
				Max +ve	0.000	0.000	26.000	-14.029
	4	4	74.000	Max -ve	0.000	0.000	74.000	0.000
				Max +ve	0.000	0.000	37.000	-336.259
6:LOAD GENE	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	30.833	-740.559
	2	2	26.000	Max -ve	0.000	0.000	26.000	769.635
				Max +ve	0.000	0.000	0.000	-67.974





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**Beam Maximum Moments Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max My (kip-ft)	d (ft)	Max Mz (kip-ft)
	3	3	26.000	Max -ve	0.000	0.000	0.000	769.635
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	106.808
				Max +ve	0.000	0.000	37.000	-275.841
57:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	74.000	209.033
				Max +ve	0.000	0.000	30.833	-233.002
	2	2	26.000	Max -ve	0.000	0.000	26.000	907.777
				Max +ve	0.000	0.000		
	3	3	26.000	Max -ve	0.000	0.000	0.000	907.777
				Max +ve	0.000	0.000	26.000	-362.807
	4	4	74.000	Max -ve	0.000	0.000	74.000	-0.000
				Max +ve	0.000	0.000	24.667	-555.867
78:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	74.000	461.205
				Max +ve	0.000	0.000	24.667	-138.927
	2	2	26.000	Max -ve	0.000	0.000	26.000	1.25E+3
				Max +ve	0.000	0.000		
	3	3	26.000	Max -ve	0.000	0.000	0.000	1.25E+3
				Max +ve	0.000	0.000	26.000	-412.395
	4	4	74.000	Max -ve	0.000	0.000	74.000	-0.000
				Max +ve	0.000	0.000	30.833	-1.55E+3
411:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	74.000	105.948
				Max +ve	0.000	0.000	37.000	-1.17E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	1.54E+3
				Max +ve	0.000	0.000		
	3	3	26.000	Max -ve	0.000	0.000	0.000	1.54E+3
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	128.412
				Max +ve	0.000	0.000	37.000	-1.17E+3
453:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	43.167	-1.39E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	877.209
				Max +ve	0.000	0.000	0.000	-801.529
	3	3	26.000	Max -ve	0.000	0.000	0.000	877.209
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	649.135
				Max +ve	0.000	0.000	74.000	-0.000
554:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	49.333	-1.41E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	817.233
				Max +ve	0.000	0.000	0.000	-604.043
	3	3	26.000	Max -ve	0.000	0.000	0.000	817.233
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	604.752
				Max +ve	0.000	0.000	74.000	-0.000
656:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000





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Job No

Sheet No

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Job Title

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Date 7/15/16

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Client

File 1563.std

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**Beam Maximum Moments Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max My (kip-ft)	d (ft)	Max Mz (kip-ft)
				Max +ve	0.000	0.000	49.333	-1.37E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	810.021
				Max +ve	0.000	0.000	0.000	-685.089
	3	3	26.000	Max -ve	0.000	0.000	0.000	810.021
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	599.415
				Max +ve	0.000	0.000	74.000	-0.000
755:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	55.500	-1.3E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	807.783
				Max +ve	0.000	0.000	0.000	-707.233
	3	3	26.000	Max -ve	0.000	0.000	0.000	807.783
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	597.759
				Max +ve	0.000	0.000		
856:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	49.333	-1.36E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	802.972
				Max +ve	0.000	0.000	0.000	-695.401
	3	3	26.000	Max -ve	0.000	0.000	0.000	802.972
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	594.199
				Max +ve	0.000	0.000	74.000	-0.000
960:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	55.500	-1.29E+3
	2	2	26.000	Max -ve	0.000	0.000	26.000	726.894
				Max +ve	0.000	0.000	0.000	-586.182
	3	3	26.000	Max -ve	0.000	0.000	0.000	726.894
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	537.902
				Max +ve	0.000	0.000		
1061:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	55.500	-991.241
	2	2	26.000	Max -ve	0.000	0.000	26.000	569.699
				Max +ve	0.000	0.000	0.000	-463.255
	3	3	26.000	Max -ve	0.000	0.000	0.000	569.699
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	421.577
				Max +ve	0.000	0.000		
1162:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	55.500	-631.488
	2	2	26.000	Max -ve	0.000	0.000	26.000	359.373
				Max +ve	0.000	0.000	0.000	-283.184
	3	3	26.000	Max -ve	0.000	0.000	0.000	359.373
				Max +ve	0.000	0.000		





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**Beam Maximum Moments Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max My (kip-ft)	d (ft)	Max Mz (kip-ft)
	4	4	74.000	Max -ve	0.000	0.000	0.000	265.936
				Max +ve	0.000	0.000		
1244:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	61.667	-357.415
	2	2	26.000	Max -ve	0.000	0.000	26.000	1.08E+3
				Max +ve	0.000	0.000	0.000	-280.005
	3	3	26.000	Max -ve	0.000	0.000	0.000	1.08E+3
				Max +ve	0.000	0.000	26.000	-267.093
	4	4	74.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	6.167	-320.139

**Beam Maximum Shear Forces***Distances to maxima are given from beam end A.*

L/C	Beam	Node A	Length (ft)		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
1:DC1	1	1	74.000	Max -ve	0.000	0.000	0.000	37.843
				Max +ve	0.000	0.000	74.000	-37.045
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-63.357
	3	3	26.000	Max -ve	0.000	0.000	0.000	63.357
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	37.045
				Max +ve	0.000	0.000	74.000	-37.843
2:DC2	1	1	74.000	Max -ve	0.000	0.000	0.000	4.413
				Max +ve	0.000	0.000	74.000	-4.319
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-7.387
	3	3	26.000	Max -ve	0.000	0.000	0.000	7.387
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	4.319
				Max +ve	0.000	0.000	74.000	-4.413
3:DW	1	1	74.000	Max -ve	0.000	0.000	0.000	8.638
				Max +ve	0.000	0.000	74.000	-8.456
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-14.462
	3	3	26.000	Max -ve	0.000	0.000	0.000	14.462
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.456
				Max +ve	0.000	0.000	74.000	-8.638
4:DC1_EXT	1	1	74.000	Max -ve	0.000	0.000	0.000	37.469
				Max +ve	0.000	0.000	74.000	-36.679
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-62.731
	3	3	26.000	Max -ve	0.000	0.000	0.000	62.731





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**Beam Maximum Shear Forces Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	36.679
				Max +ve	0.000	0.000	74.000	-37.469
5:DESIGN LAN	1	1	74.000	Max -ve	0.000	0.000	0.000	17.987
				Max +ve	0.000	0.000	74.000	-17.607
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-30.113
	3	3	26.000	Max -ve	0.000	0.000	0.000	30.113
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	17.607
				Max +ve	0.000	0.000	74.000	-17.987
6:LOAD GENE	1	1	74.000	Max -ve	0.000	0.000	0.000	81.631
				Max +ve	0.000	0.000	74.000	-25.963
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-38.469
	3	3	26.000	Max -ve	0.000	0.000	0.000	31.746
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	19.240
				Max +ve	0.000	0.000	74.000	-16.354
57:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	14.972
				Max +ve	0.000	0.000	74.000	-20.622
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-33.128
	3	3	26.000	Max -ve	0.000	0.000	0.000	96.968
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	20.462
				Max +ve	0.000	0.000	74.000	-23.132
78:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	11.565
				Max +ve	0.000	0.000	74.000	-24.029
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-36.535
	3	3	26.000	Max -ve	0.000	0.000	0.000	70.135
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	57.629
				Max +ve	0.000	0.000	74.000	-49.965
411:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	47.281
				Max +ve	0.000	0.000	74.000	-49.561
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-60.819
	3	3	26.000	Max -ve	0.000	0.000	0.000	59.955
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	48.697
				Max +ve	0.000	0.000	74.000	-48.145
453:LOAD GENI	1	1	74.000	Max -ve	0.000	0.000	0.000	35.433
				Max +ve	0.000	0.000	74.000	-64.567





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File 1563.std

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**Beam Maximum Shear Forces Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-64.567
	3	3	26.000	Max -ve	0.000	0.000	0.000	8.772
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.772
				Max +ve	0.000	0.000		
554:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	33.887
				Max +ve	0.000	0.000	67.833	-43.113
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	8.667	-60.113
	3	3	26.000	Max -ve	0.000	0.000	0.000	8.172
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.172
				Max +ve	0.000	0.000		
656:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	30.496
				Max +ve	0.000	0.000	74.000	-57.504
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-57.504
	3	3	26.000	Max -ve	0.000	0.000	0.000	8.100
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.100
				Max +ve	0.000	0.000		
755:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	29.730
				Max +ve	0.000	0.000	74.000	-58.270
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-58.270
	3	3	26.000	Max -ve	0.000	0.000	0.000	8.078
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.078
				Max +ve	0.000	0.000		
856:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	30.370
				Max +ve	0.000	0.000	74.000	-57.630
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-57.630
	3	3	26.000	Max -ve	0.000	0.000	0.000	8.030
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	8.030
				Max +ve	0.000	0.000		
960:LOAD GEN	1	1	74.000	Max -ve	0.000	0.000	0.000	25.397
				Max +ve	0.000	0.000	74.000	-50.503
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-50.503
	3	3	26.000	Max -ve	0.000	0.000	0.000	7.269
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	7.269





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Job No

Sheet No

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File 1563.std

Date/Time 15-Jul-2016 12:17

**Beam Maximum Shear Forces Cont...**

L/C	Beam	Node A	Length (ft)		d (ft)	Max Fz (kip)	d (ft)	Max Fy (kip)
				Max +ve	0.000	0.000		
1061:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	19.671
				Max +ve	0.000	0.000	67.833	-39.729
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-39.729
	3	3	26.000	Max -ve	0.000	0.000	0.000	5.697
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	5.697
				Max +ve	0.000	0.000		
1162:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	12.686
				Max +ve	0.000	0.000	61.667	-24.714
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-24.714
	3	3	26.000	Max -ve	0.000	0.000	0.000	3.594
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	3.594
				Max +ve	0.000	0.000		
1244:LOAD GE	1	1	74.000	Max -ve	0.000	0.000	0.000	10.421
				Max +ve	0.000	0.000	74.000	-25.579
	2	2	26.000	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	26.000	-61.504
	3	3	26.000	Max -ve	0.000	0.000	0.000	62.118
				Max +ve	0.000	0.000		
	4	4	74.000	Max -ve	0.000	0.000	0.000	17.268
				Max +ve	0.000	0.000	74.000	-9.807



## APPENDIX E

### Bridge Inspection Photos





Looking northerly on Green Point Rd.



Looking southerly on Green Point Rd.



General view



General view

1563

Brewer

Green Point Rd./I-395

12/11/2012





Joint (Typ.)



Pier



Southerly abutment



Northerly abutment

1563

Brewer

Green Point Rd./I-395

12/11/2012





Southeasterly wing



Major spall at SEly joint end



Southerly abutment – note cracking & staining



Pier cap end – note cracking





Roadway looking north



Rail type



North seal



heavy spalling at east fascia southjoint area

1563

Brewer

Parkway South/ I-395

1-5-2015





Failed south seal

1563

Brewer

100

Failed seal

Parkway South/I-395

1-5-2015





south abutment



Rusty south beam ends due to failed seal



south span view



Pier view





north span view

general





north abutment view

general

1563

Brewer

103

Parkway South/I-395

1-5-2015



## APPENDIX F

### Original Plans and Drawings



182-146-171 B&H 1563

GREEN POINT ROAD OVER

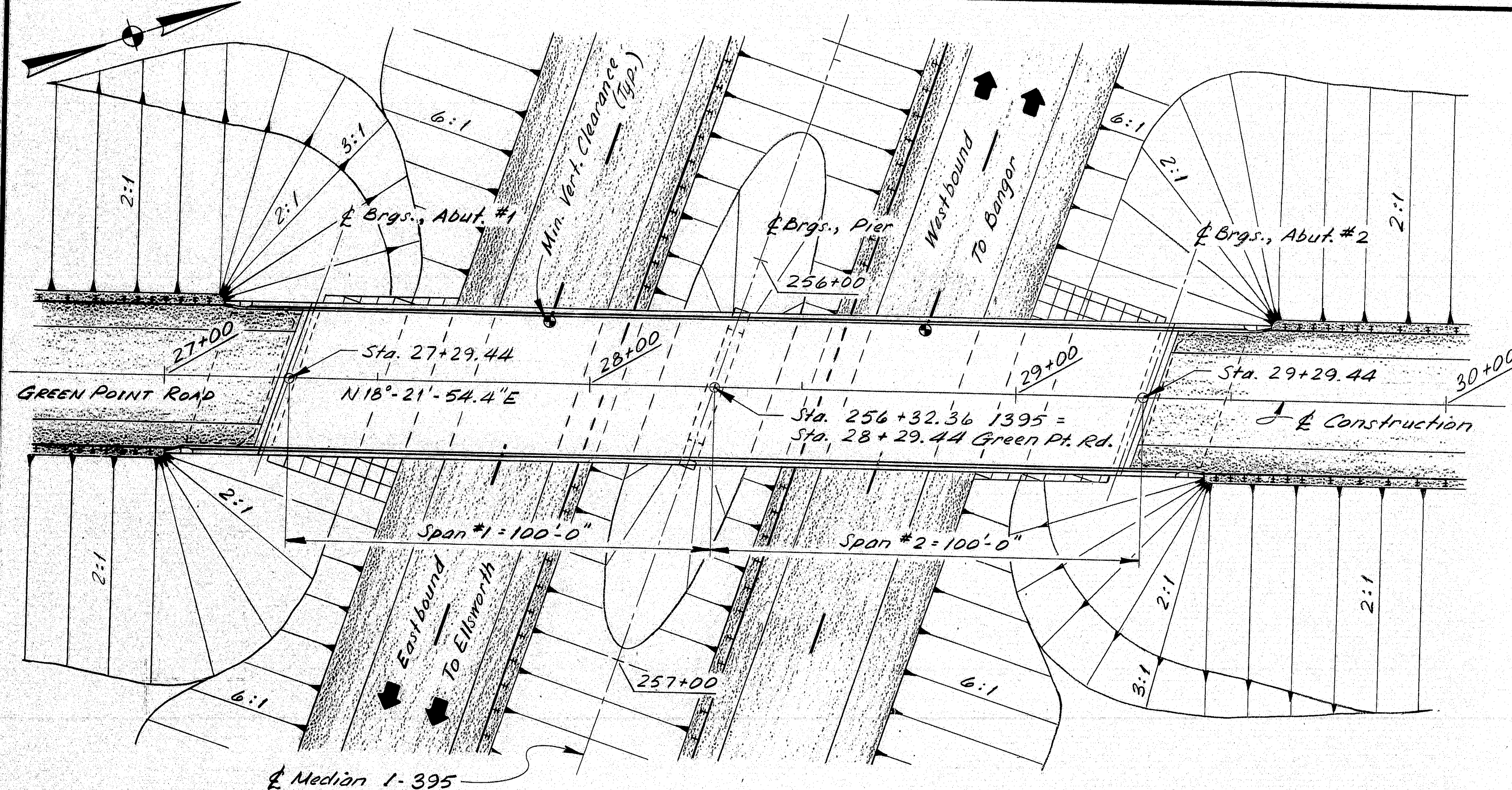
I-95 - BREWER

STEEL ACT

1

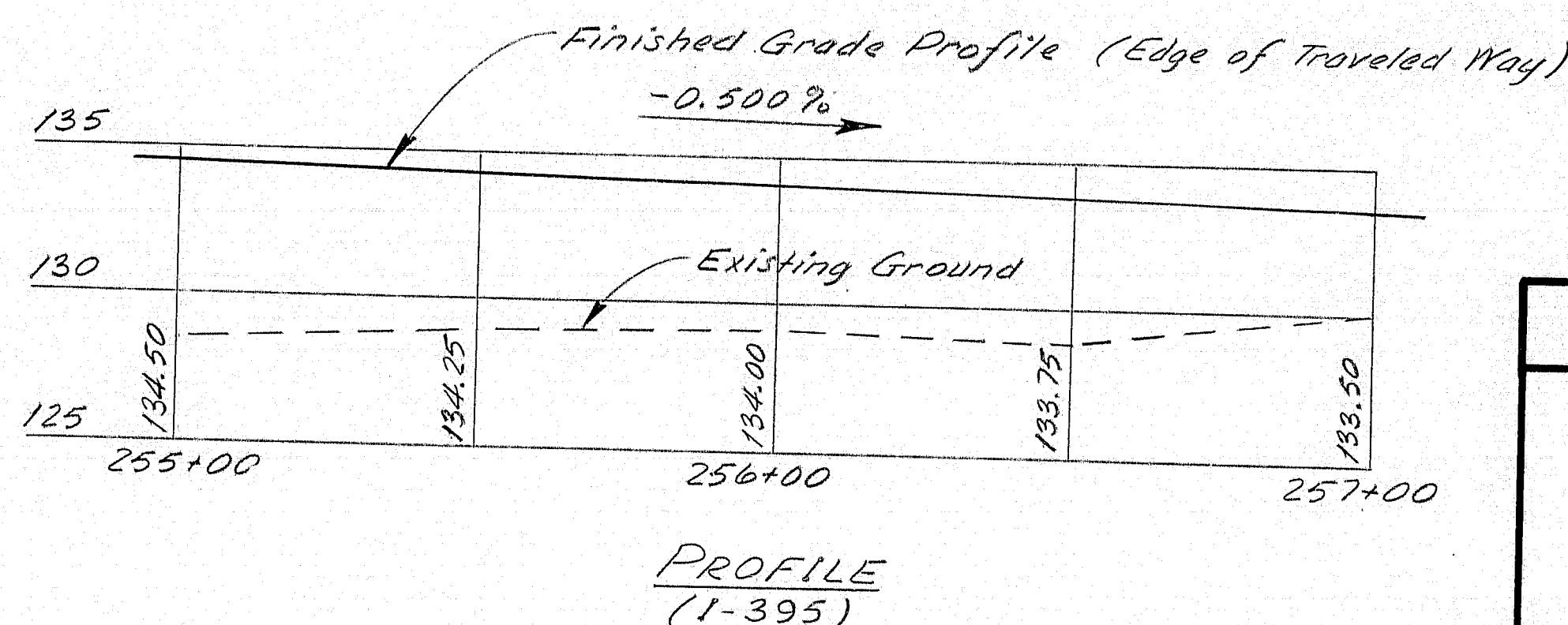
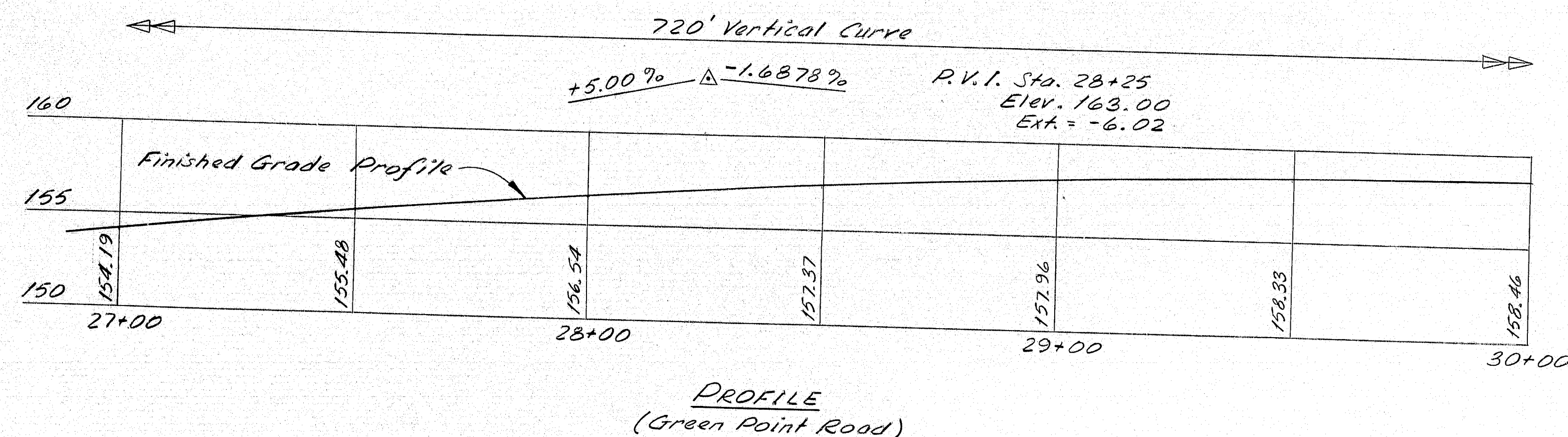
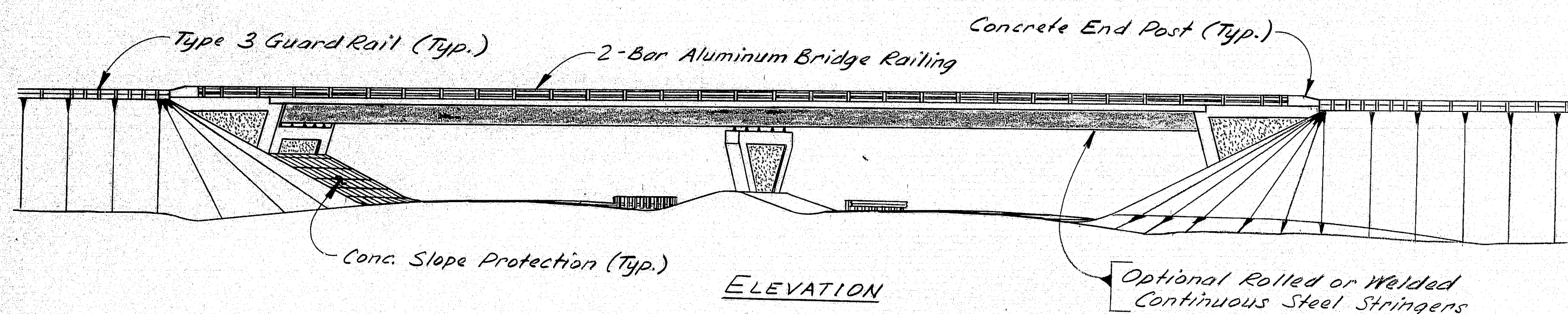


F.R.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87) 176	21	84



MINIMUM VERTICAL CLEARANCE:  
1-395 E.B. = 17'-0"  
1-395 W.B. = 18'-3"

GENERAL PLAN  
20 0 20 40  
Scale of Feet



### SPECIFICATIONS

DESIGN: AASHTO Standard Specifications for Highway Bridges 1977 and interim specifications thru 1982.

CONTRACT: State of Maine, Department of Transportation, Standard Specifications, Highways and Bridges, Revision of June 1981.

### DESIGN LOADING

LIVE LOAD: HS 25 Stress Cycles: 500,000

### INDEX OF SHEETS

GENERAL PLAN	1
ABUTMENT FOOTINGS	2
ABUTMENT No. 1	3
ABUT. No. 1 WINGS	4
ABUTMENT No. 2	5
ABUT. No. 2 WINGS	6
PIER	7
RECESSED PANEL DETAILS	8
STRUCTURAL STEEL (ROLLED BEAM OPTION)	9
STRUCTURAL STEEL (WELDED BEAM OPTION)	10-11
SUPERSTRUCTURE	12
CONCRETE SLOPE PROTECTION	13
REINFORCING STEEL SCHEDULE	14-15

### BRIDGE STANDARD DETAILS

BD 100-81...BEARING PEDESTALS
BD 101-81...BEARING PEDESTALS
BD 103-81...BEAM SPLICES (ROLLED BEAMS)
BD 113-81...DIAPHRAGMS AND CROSSFRAMES
BD 114-81...ALUMINUM BRIDGE RAILING:
2-BAR SEMI-ELLIPSE
BD 120-81...CONCRETE END POSTS
BD 125-82...EXPANSION DEVICE:
COMPRESSION SEAL
BD 126-81...BRIDGE DRAINS, SHEAR CON-
NECTORS, HAUNCH DETAILS,
CURB SECTION TYPE 1-B, AND
APPROACH SLAB
BD 127-81...PILE DETAILS, CONCRETE JOINTS

### MATERIALS

CONCRETE:..... Class "A"  
REINFORCING STEEL: ASTM A615,  
Grade 60

STRUCTURAL STEEL:  
All Material (except as noted).....  
ASTM A572  
High Strength Bolts.....ASTM A325,  
Type 1

### MINIMUM ULTIMATE YIELD STRENGTHS

ASTM A572.....F<sub>y</sub> = 50,000 psi  
ASTM A36.....F<sub>y</sub> = 36,000 psi

### BASIC ALLOWABLE STRESSES

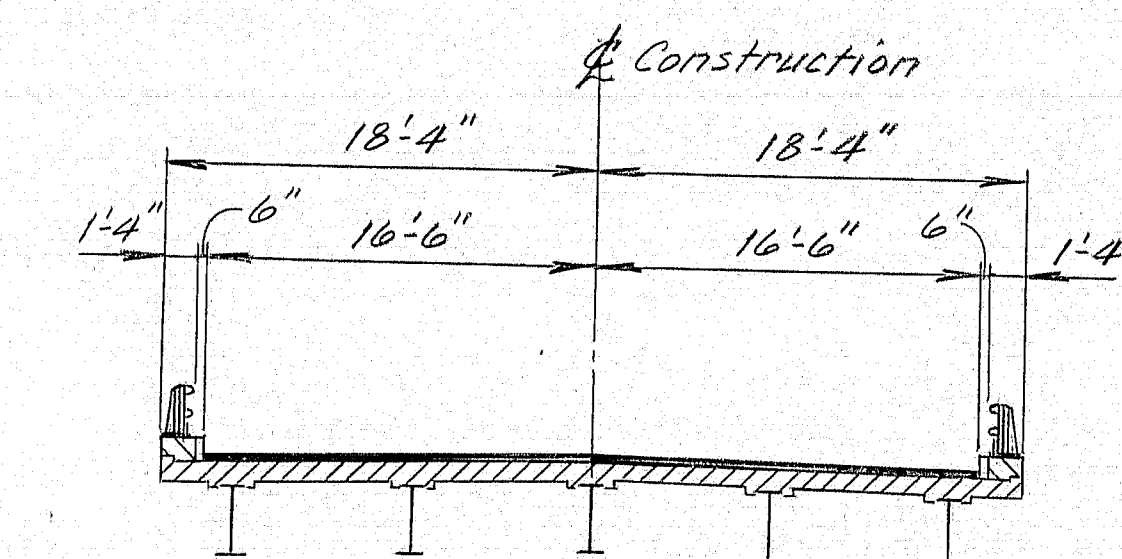
ASTM A325.....f<sub>v</sub> = 33,000 psi

### TRAFFIC DATA (G.P.R.)

AADT (1983)	1080
AADT (2003)	1290
DHV	129
T(%)	6
D(%)	55
18 KIP P2.5	16

### TRAFFIC DATA (1-395)

AADT (1983)	7000
AADT (2003)	13,000
DHV	1950
T(%)	7
D(%)	55
18 KIP P2.5	244



### TRANSVERSE SECTION

Revised "As Built" 1984  
R. Zimmerman

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
GENERAL PLAN

183-146

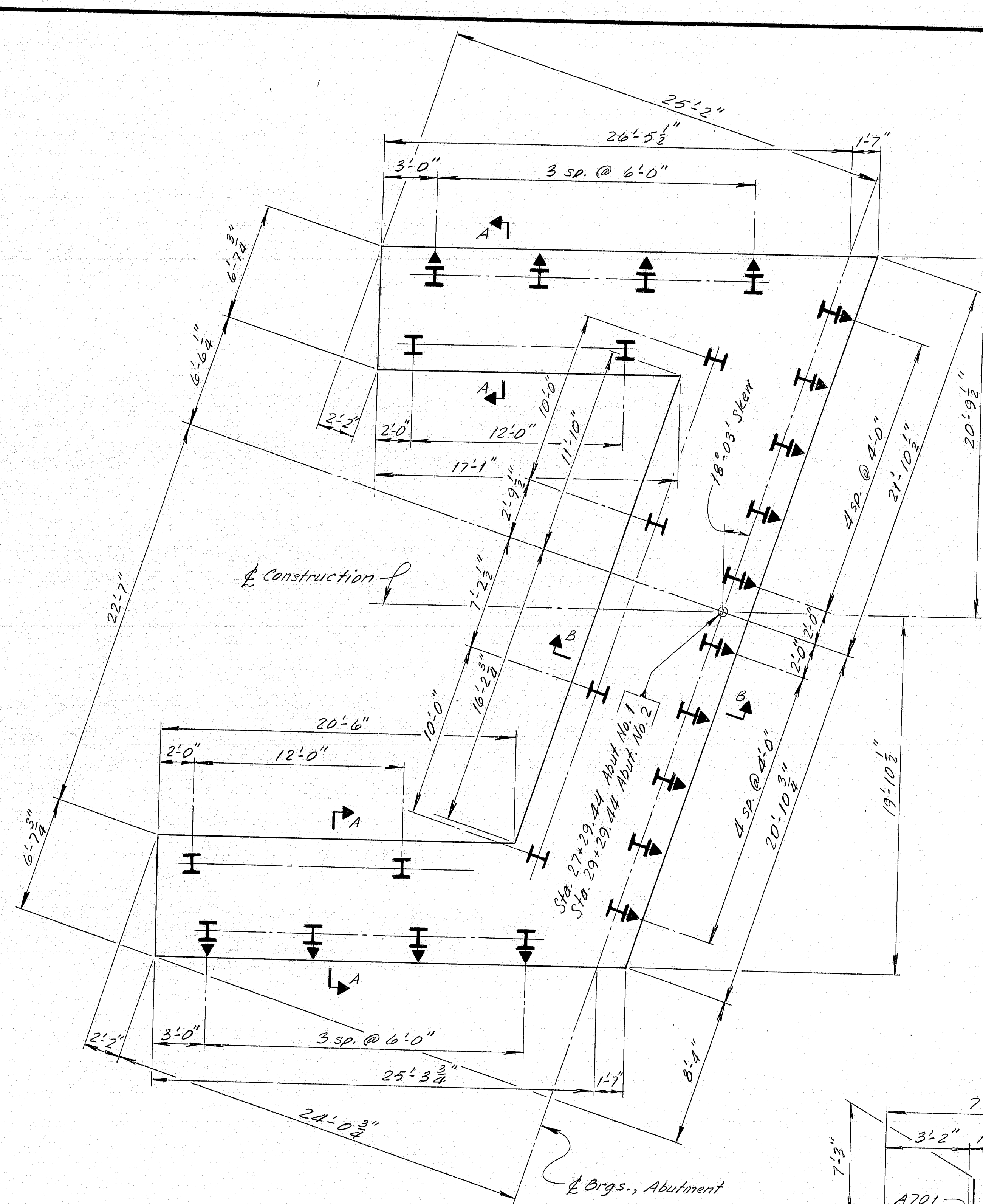
SHEET 1 OF 15 AUGUSTA, MAINE

Structural Steel Alternate



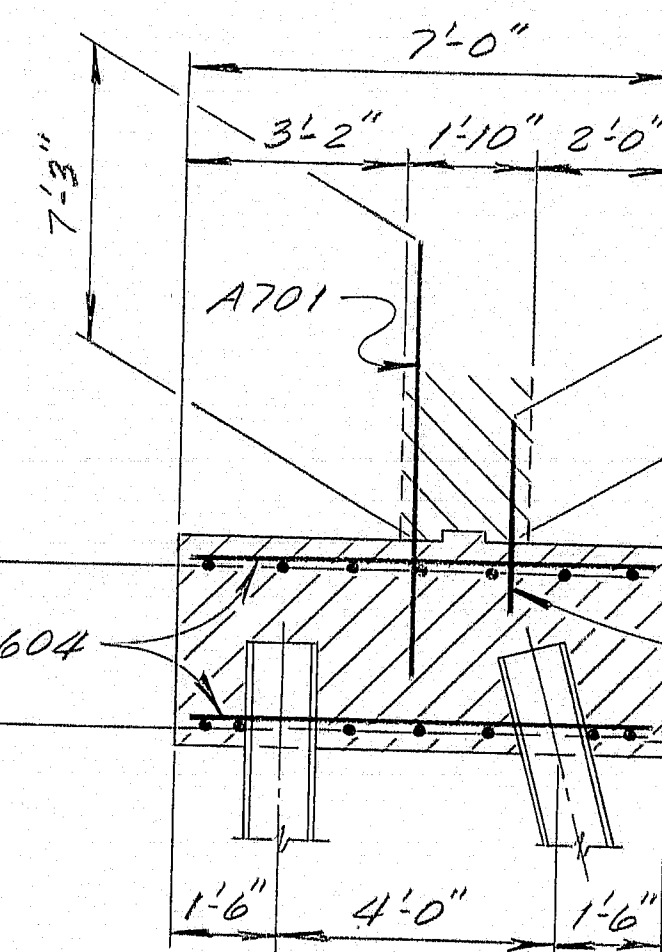
PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED	W. C. BROWN	12/23/83
CHECKED	W. C. BROWN	12/23/83
REVISIONS		
FIELD CHANGES		

BRUNING 44-132-45710

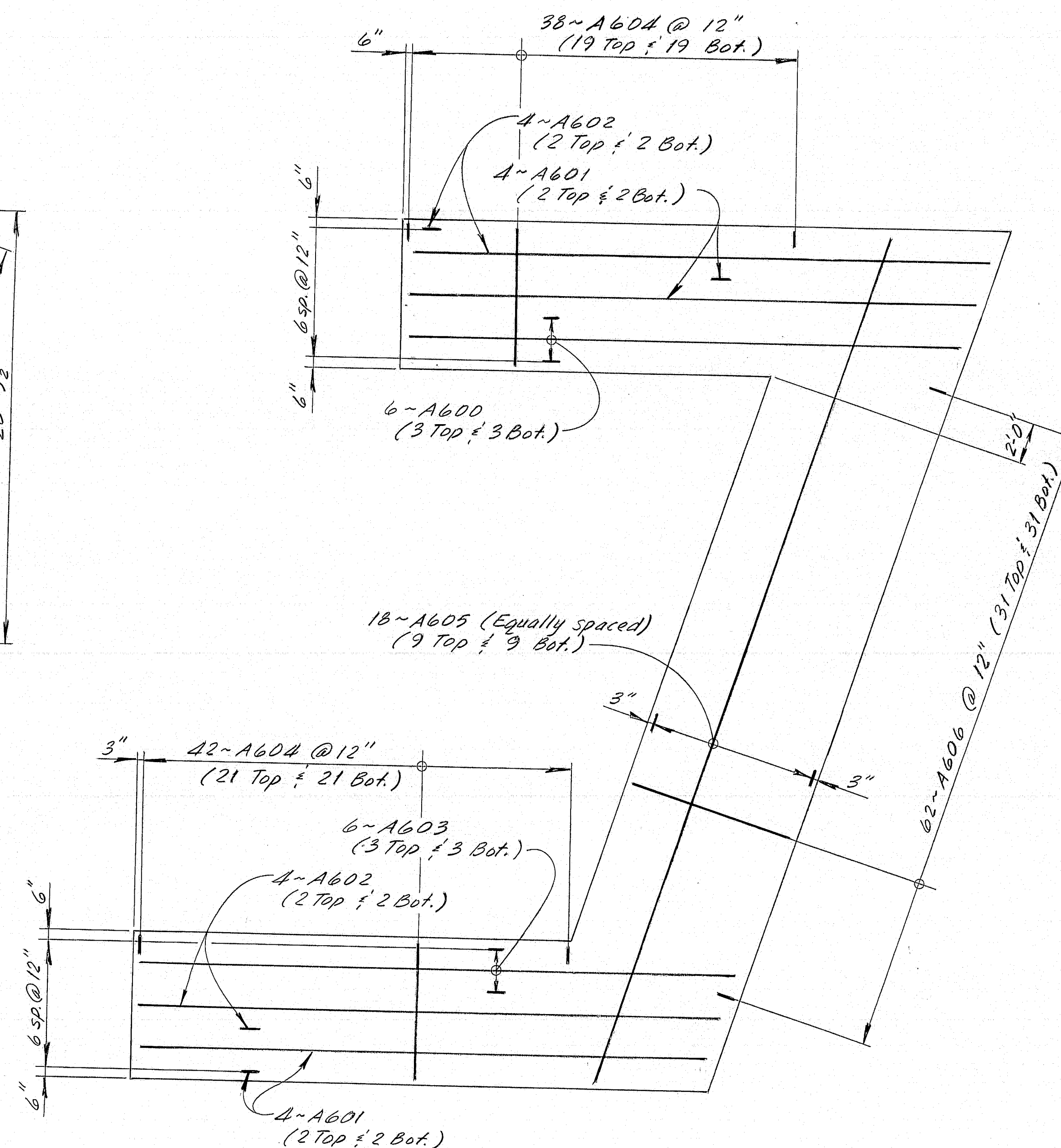


FOOTING PLAN

A600 thru A602 or  
A601 thru A603

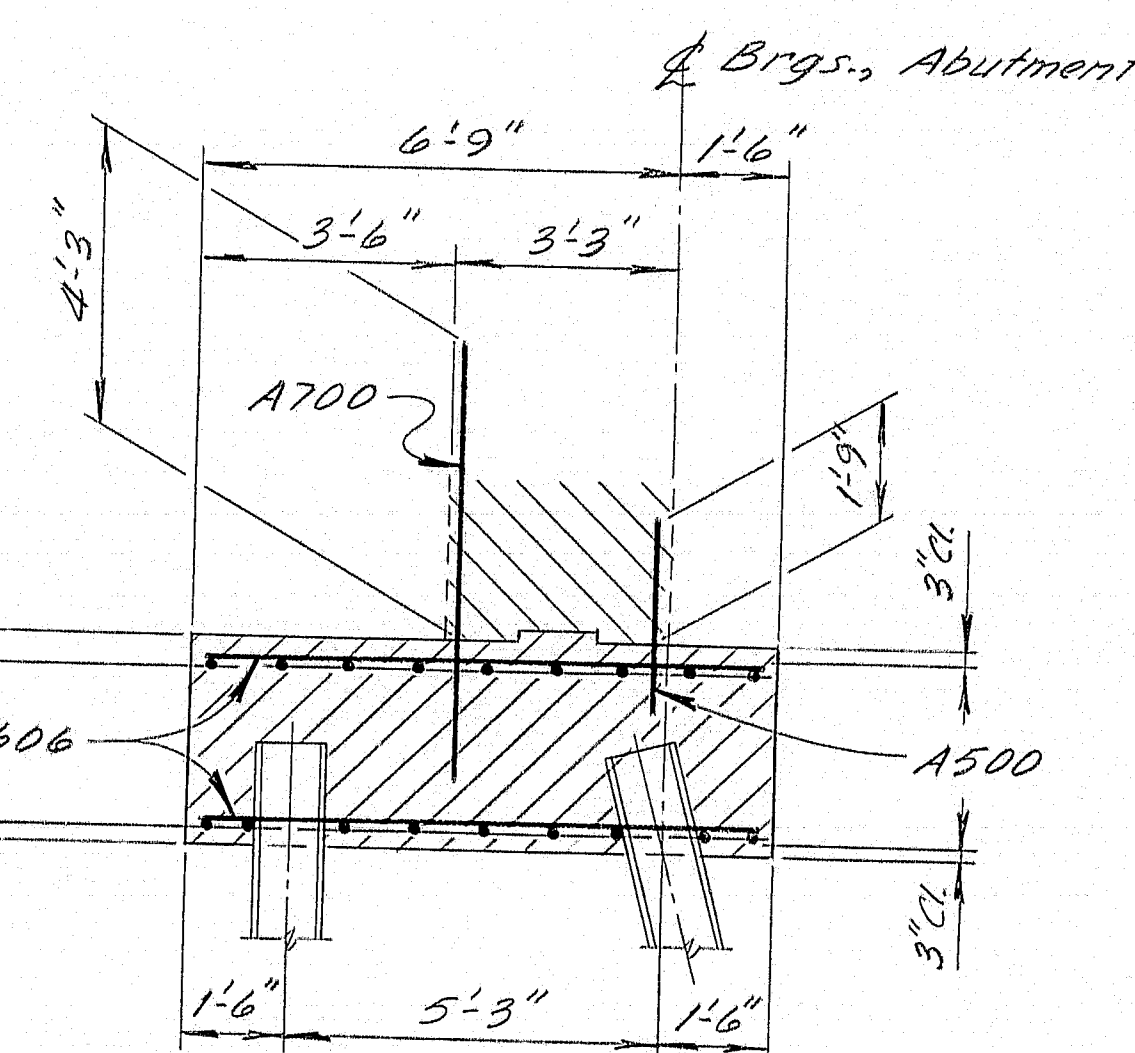


SECTION A-A



REINFORCING PLAN

Abut. No. 1 Elev. 136.50  
Abut. No. 2 Elev. 139.00  
A605  
Abut. No. 1 Elev. 133.50  
Abut. No. 2 Elev. 136.00



SECTION B-B

F.H.W.A.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87)176	22	84

### PILE NOTES

- Piles marked thus  $\nabla$  shall be battered  $3\frac{1}{2}$ " per foot in the direction of the arrow.
- Maximum calculated pile loads:  
Abutment No. 1 } 71 Tons  
Abutment No. 2 }
- Estimate of piles required:  
Abutment No. 1  
26 ~ HP12 x 53 @ 30 feet  
Abutment No. 2  
26 ~ HP12 x 53 @ 32 feet

### FOOTING NOTES

- Reinforcing steel shall have 2" minimum concrete cover.

As Built 1984 12mz

Note:  
Abutment No. 1 shown ~  
Rotate 180° for Abutment  
No. 2  
In all cases, substitute "B" bars  
for "A" bars in Abutment  
No. 2 Footing:  
A600 = B600  
A601 = B601, etc.

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

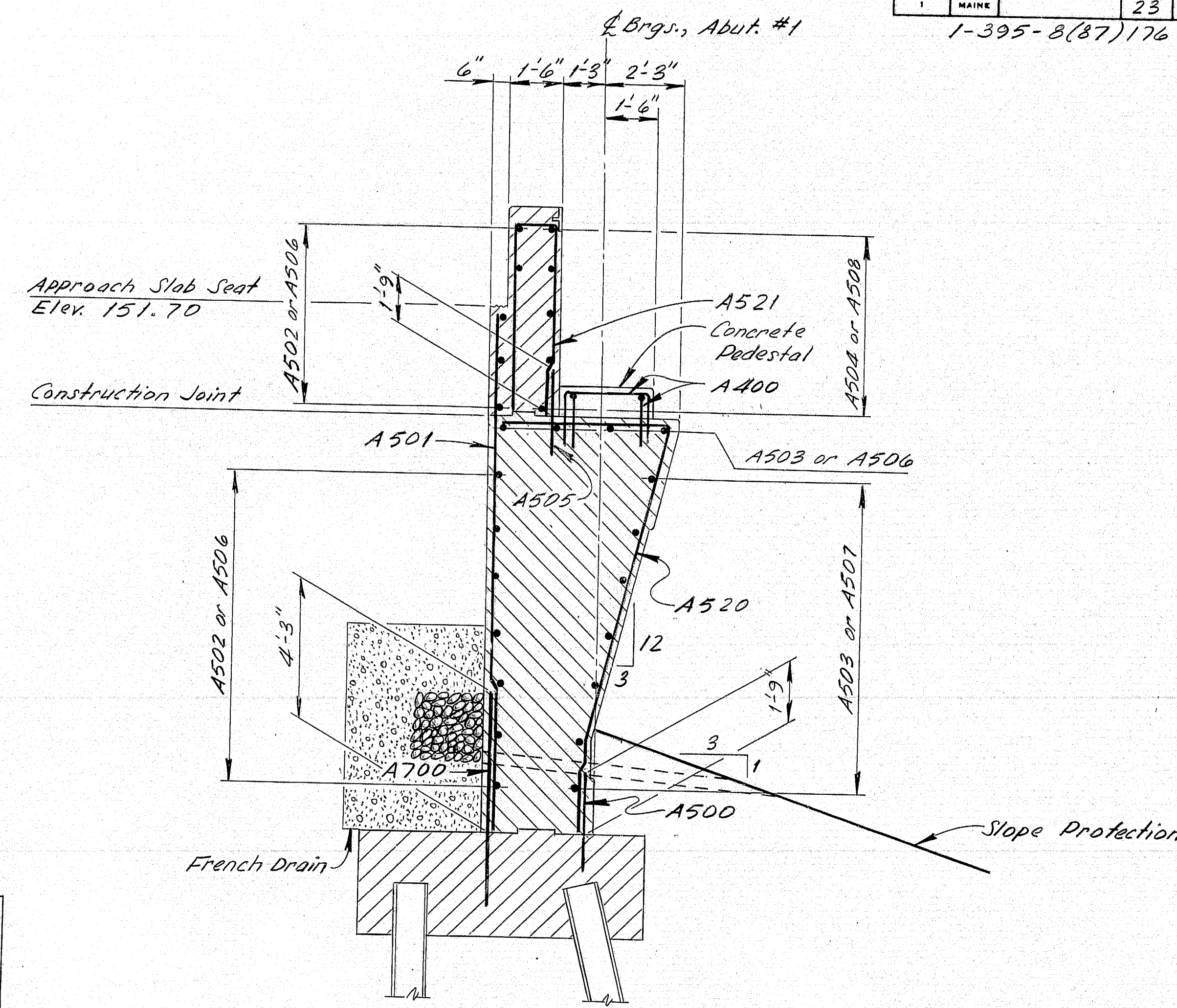
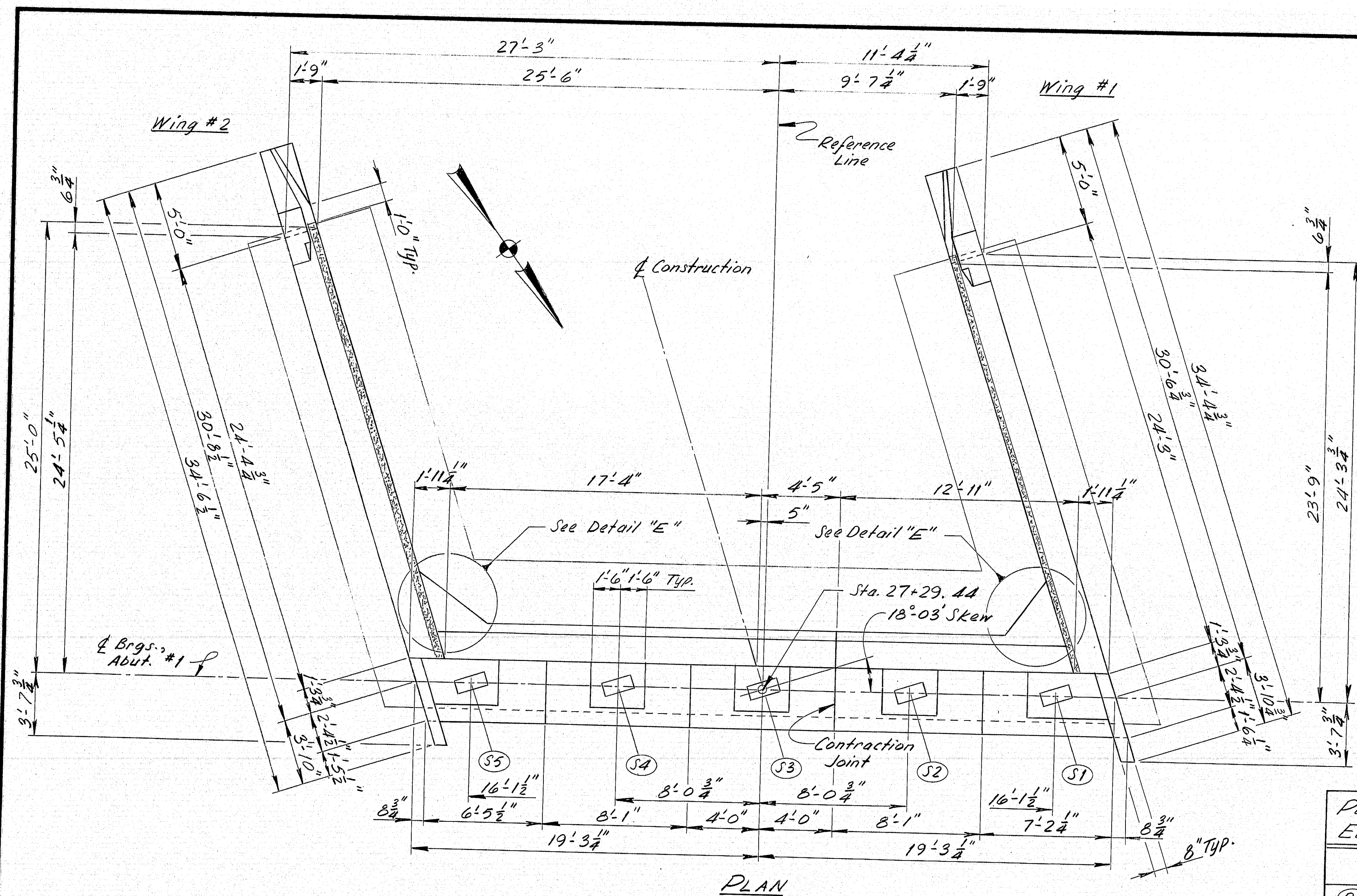
GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
ABUTMENT FOOTINGS

SHEET 2 OF 15 AUGUSTA, MAINE  
Structural Steel Alternate

183-147





PEDESTAL ELEVATIONS	
	ROLLED STEEL BEAM
(P1)	149.48
(P2)	149.58
(P3)	149.68
(P4)	149.45
(P5)	149.23

BRIDGE SEAT ELEVATIONS	
	Both Options
(B1)	148.35
(B2)	148.45
(B3)	148.55
(B4)	148.32
(B5)	148.10

- ABUTMENT NOTES**
- Reinforcing Steel shall have 2" minimum cover unless otherwise noted.
  - Protective Coating for Concrete Surfaces shall be applied to tops of concrete curbs, top of Abutment backwalls and one foot below top of back-wall on the back side, and all exposed surfaces of Concrete End Posts.
  - Place 4" drains in Breastwall and Wings at 20'0" maximum spacing. Exact location to be determined in the field by the Engineer.
  - The concrete pedestals as detailed are for use with the rolled beam option only. If the welded beam option is selected, the concrete pedestals shall be omitted.

#### SYMBOLS

- New Concrete (Plan or Elevation)
- New Concrete (Section)
- Granite Curb

Revised As Built 1984  
EMZ

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
ABUTMENT No. 1

183-148

SHEET 3 OF 15 AUGUSTA, MAINE

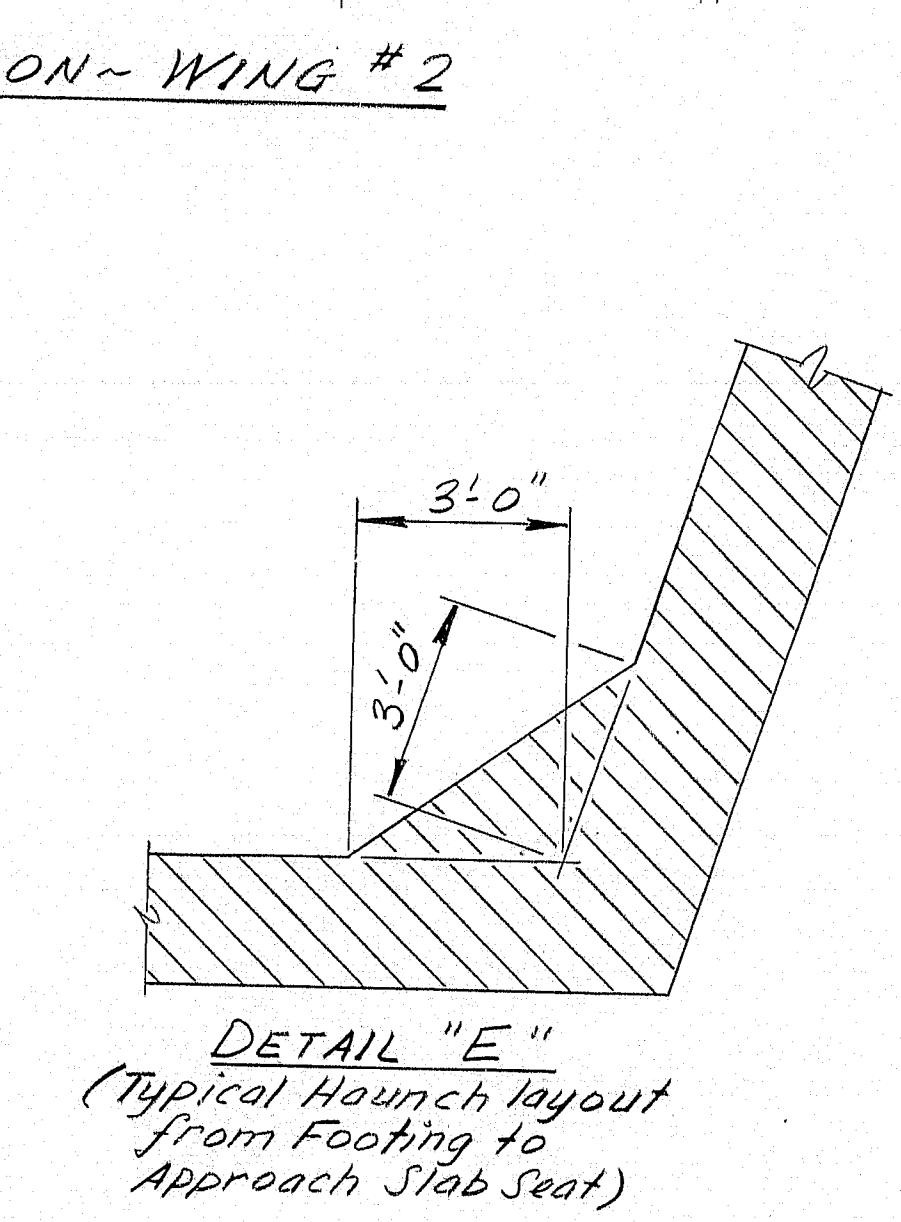
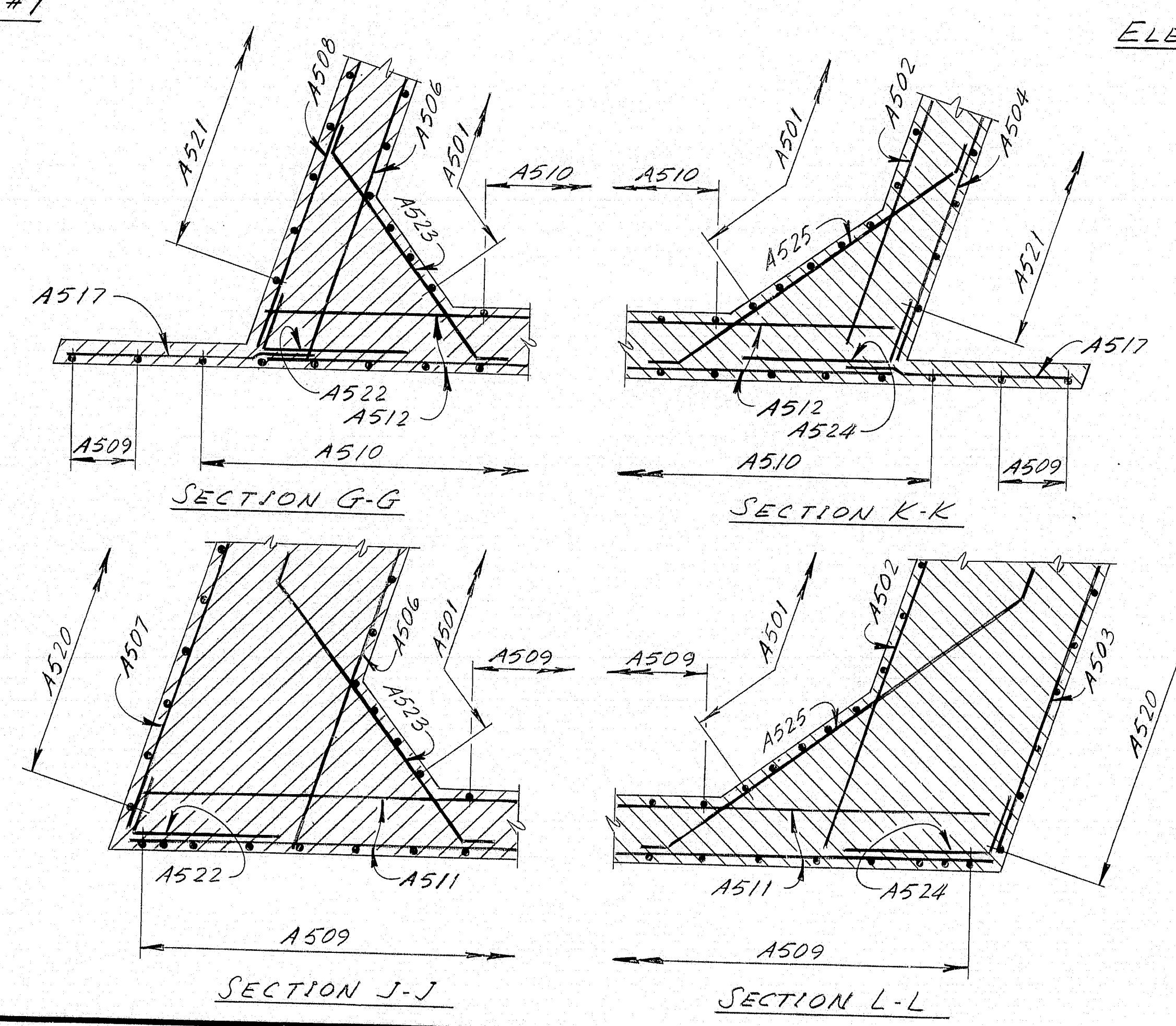
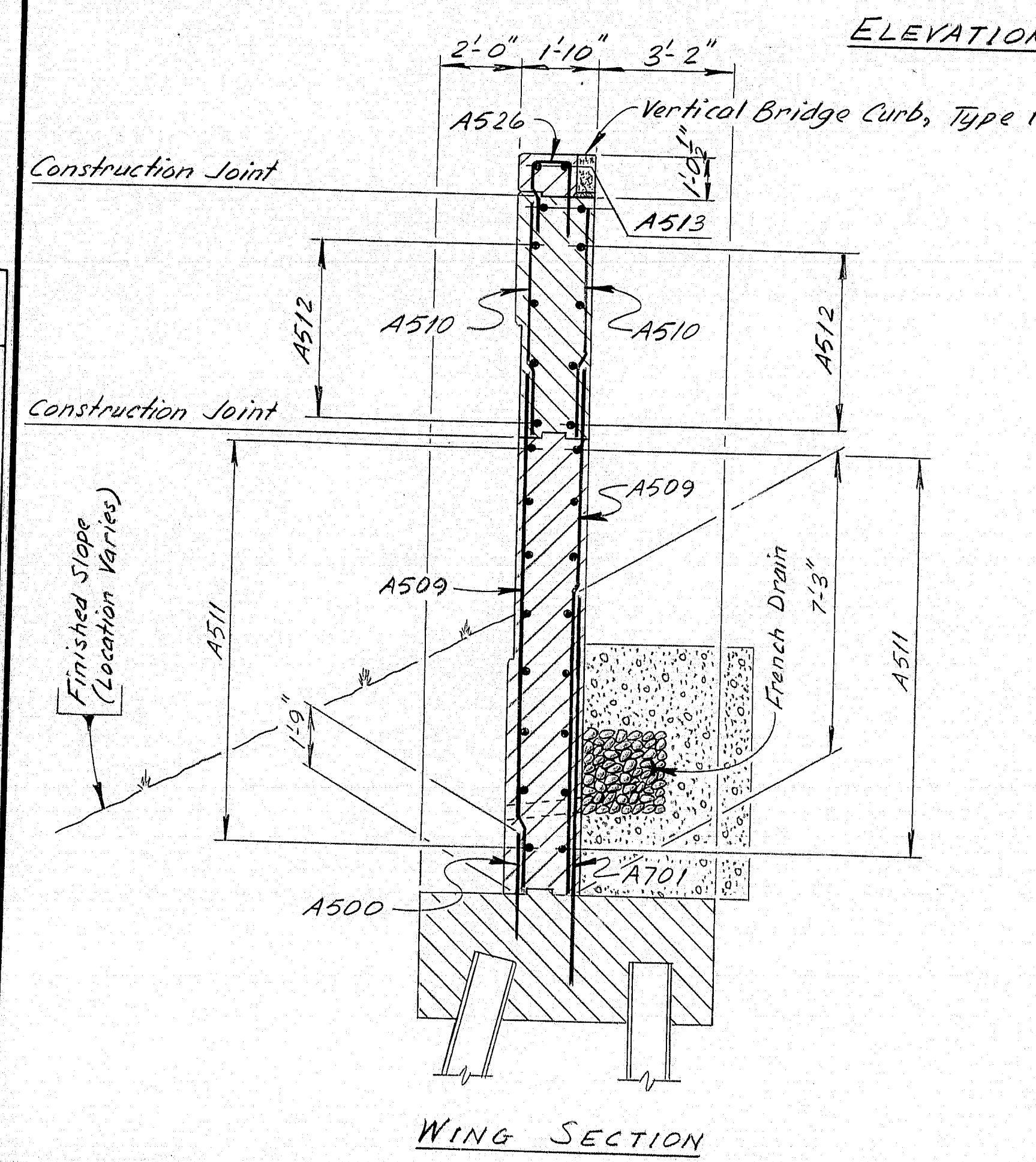
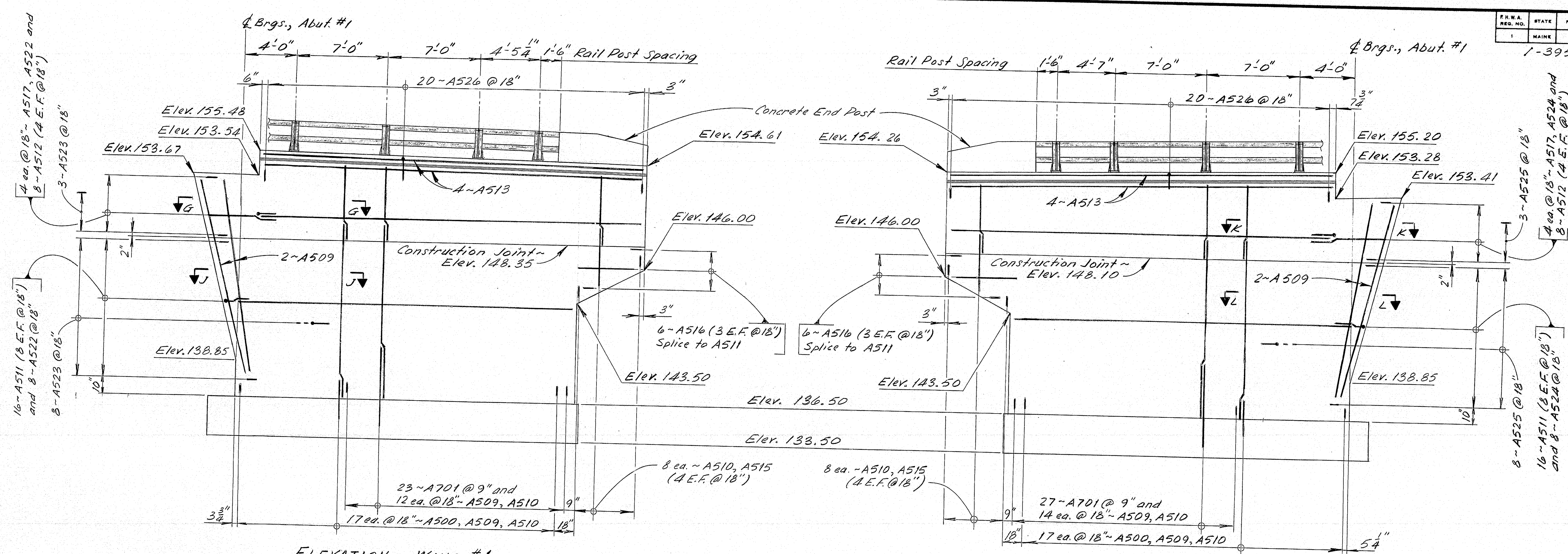
Structural Steel Alternate

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	10/27/82
CHECKED	11/1/82
REVISIONS	
FIELD SOURCES	

BRUING 44-132-45710



F.H.W.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87)176	24	84



PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	12/1/82
CHECKED	12/1/83
REVISIONS	
FIELD CHANGES	

**SYMBOLS**

New Concrete (Plan or Elevation)

New Concrete (Section)

"A" BUILT 1984 RMZ

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY

ABUTMENT No. 1 WINGS AND DETAILS

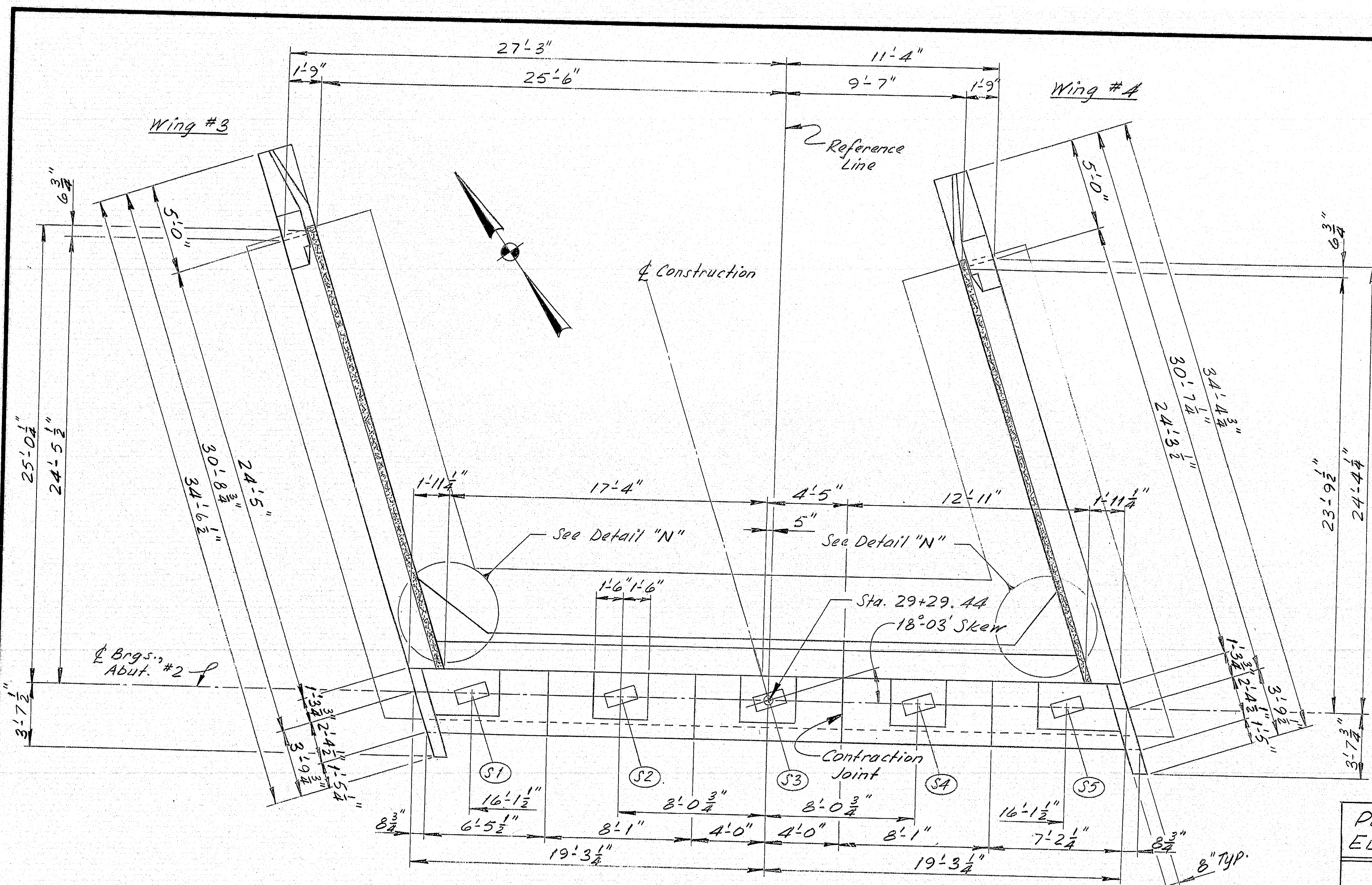
SHEET 4 OF 15 AUGUSTA, MAINE

Structural Steel Alternate

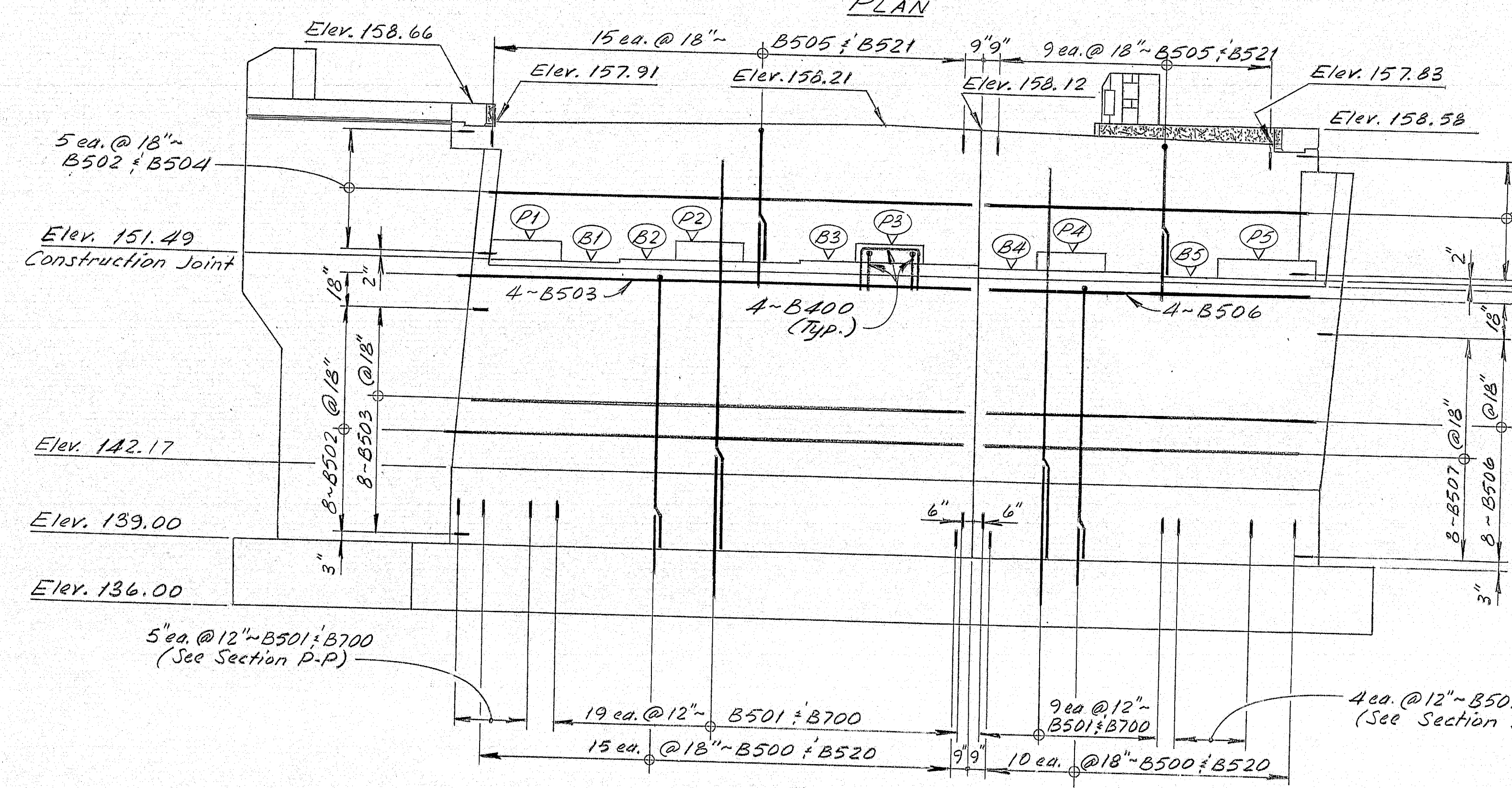
183-149



F.H.A. REQ. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87)176	25	84



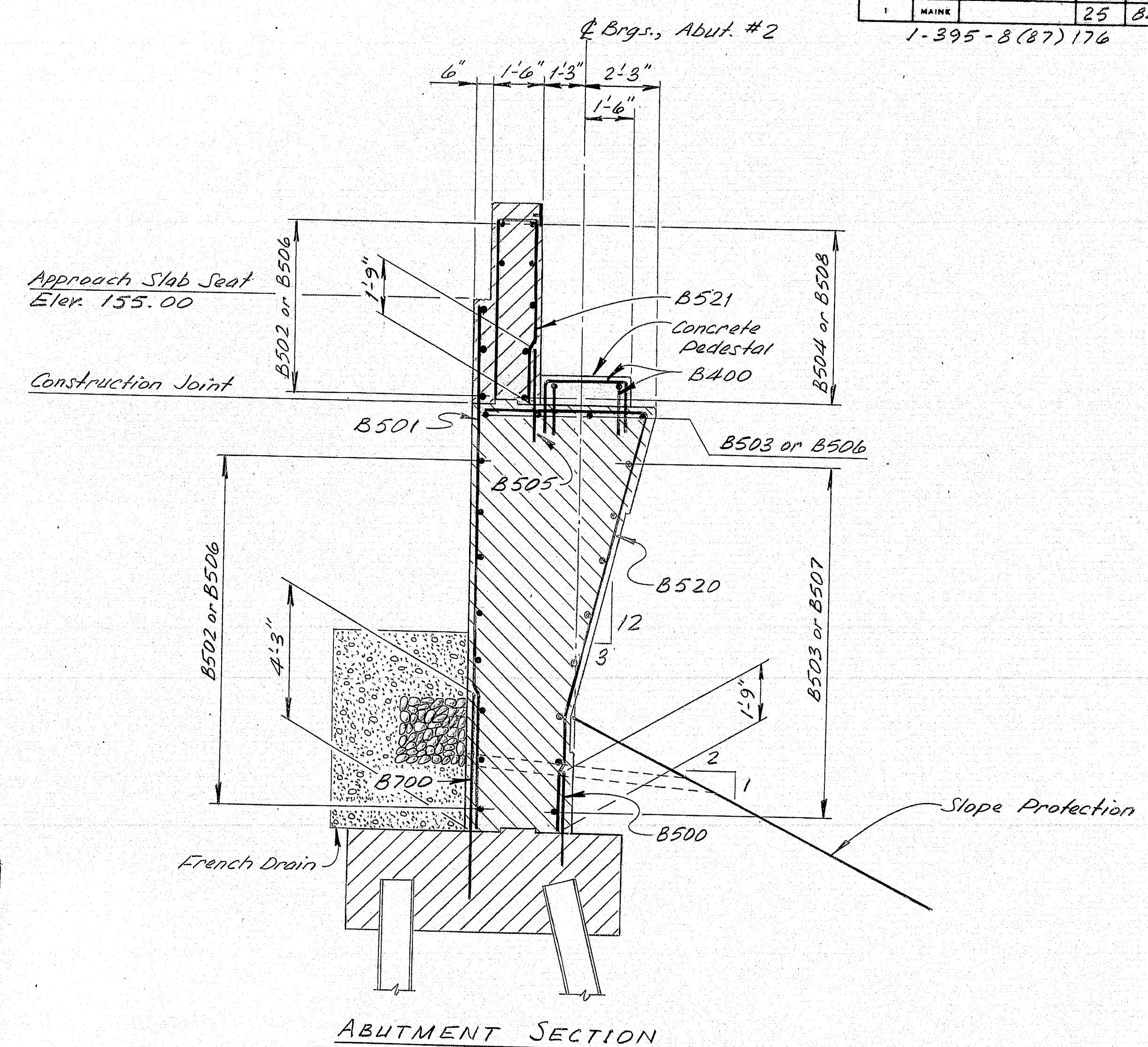
PLAN



ELEVATION

PEDESTAL ELEVATIONS	
	ROLLED STEEL BEAM
(P1)	152.62
(P2)	152.76
(P3)	152.91
(P4)	152.73
(P5)	152.55

BRIDGE SEAT ELEVATIONS	
	Both Options
(B1)	151.49
(B2)	151.63
(B3)	151.78
(B4)	151.60
(B5)	151.42



ABUTMENT SECTION

- SYMBOLS
- New Concrete (Plan or Elevation)
  - New Concrete (Section)
  - Granite Curb

As Built 1984 E.M.F.

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY

ABUTMENT No. 2

183-150

SHEET 5 OF 15 AUGUSTA, MAINE

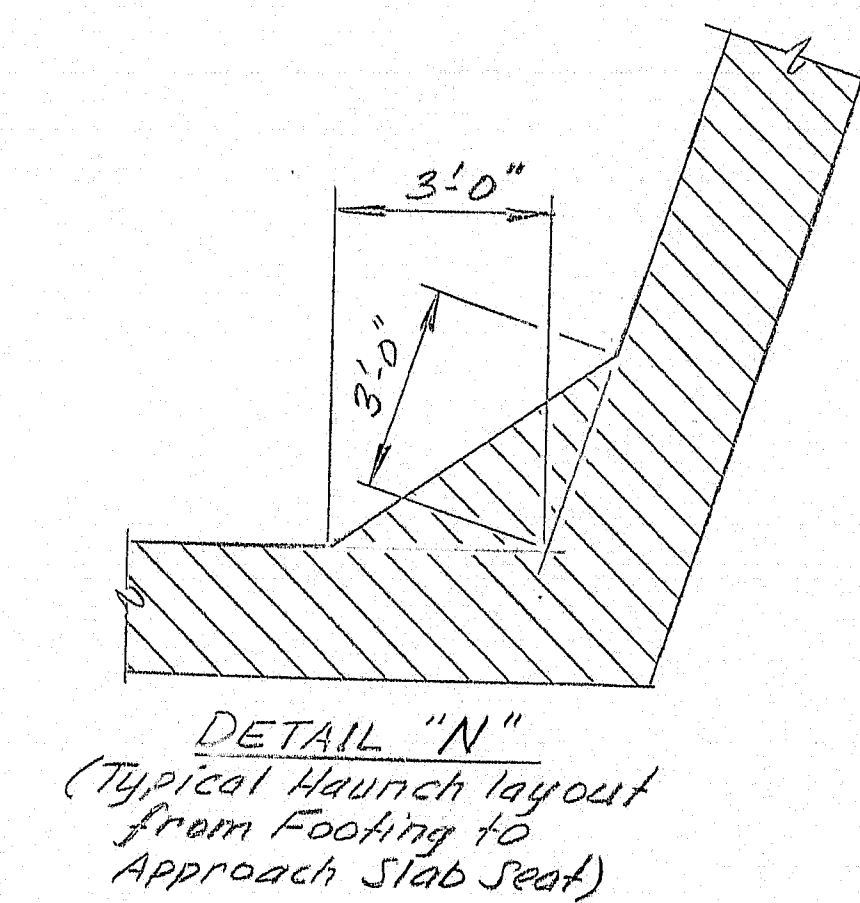
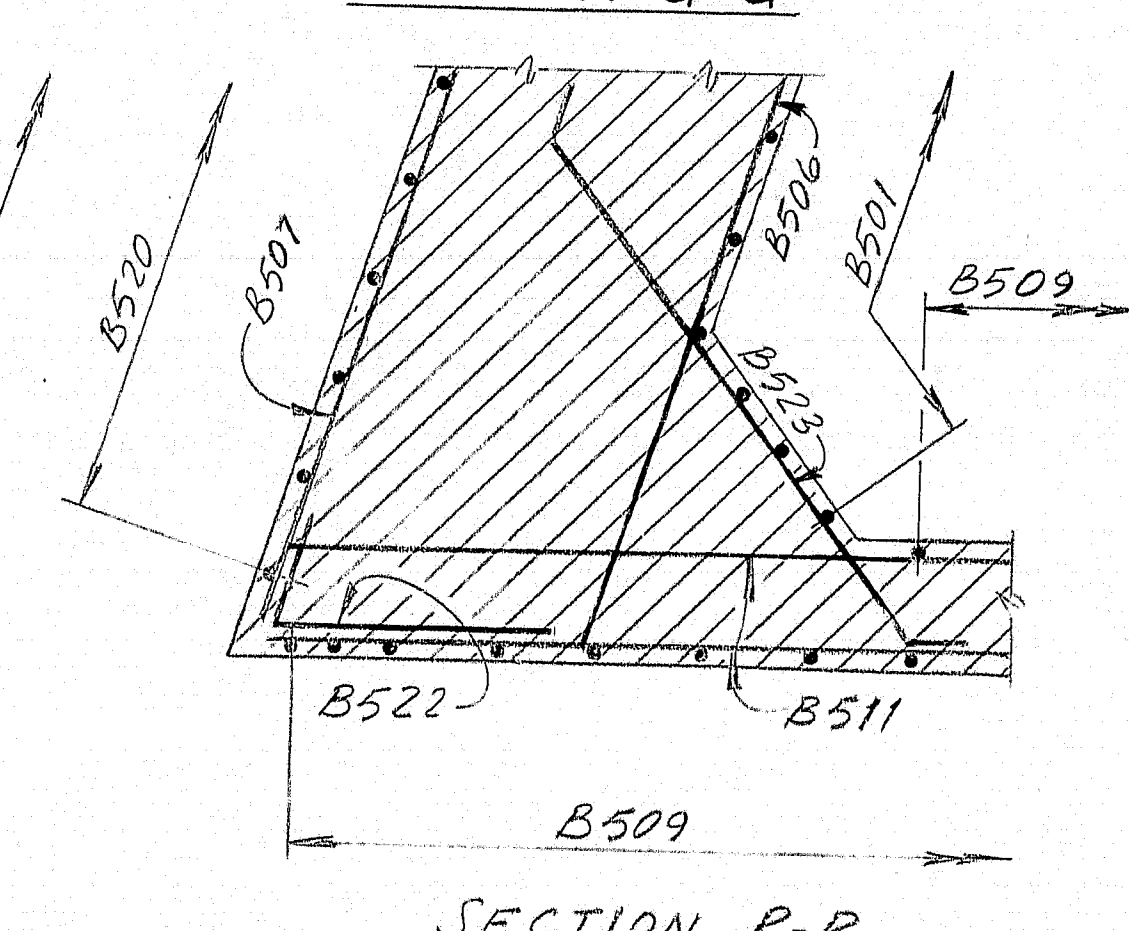
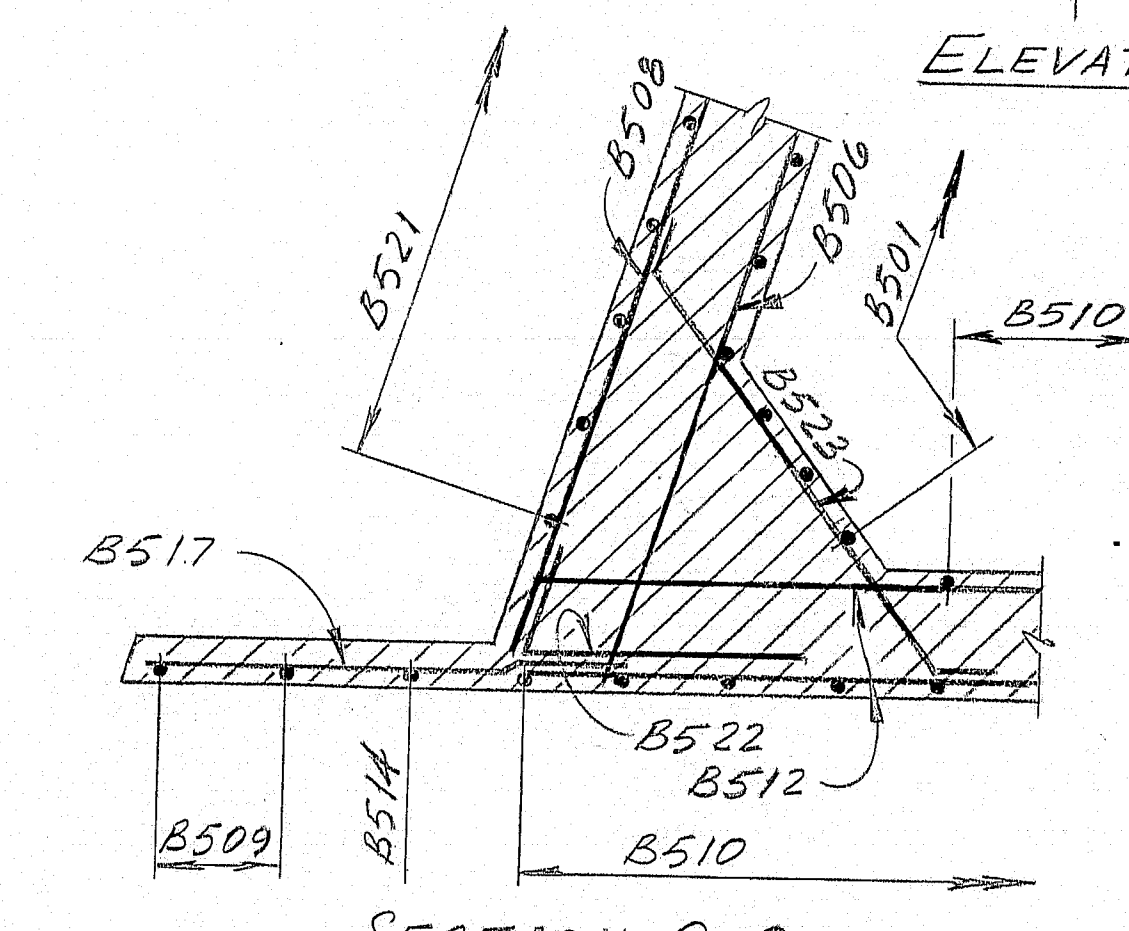
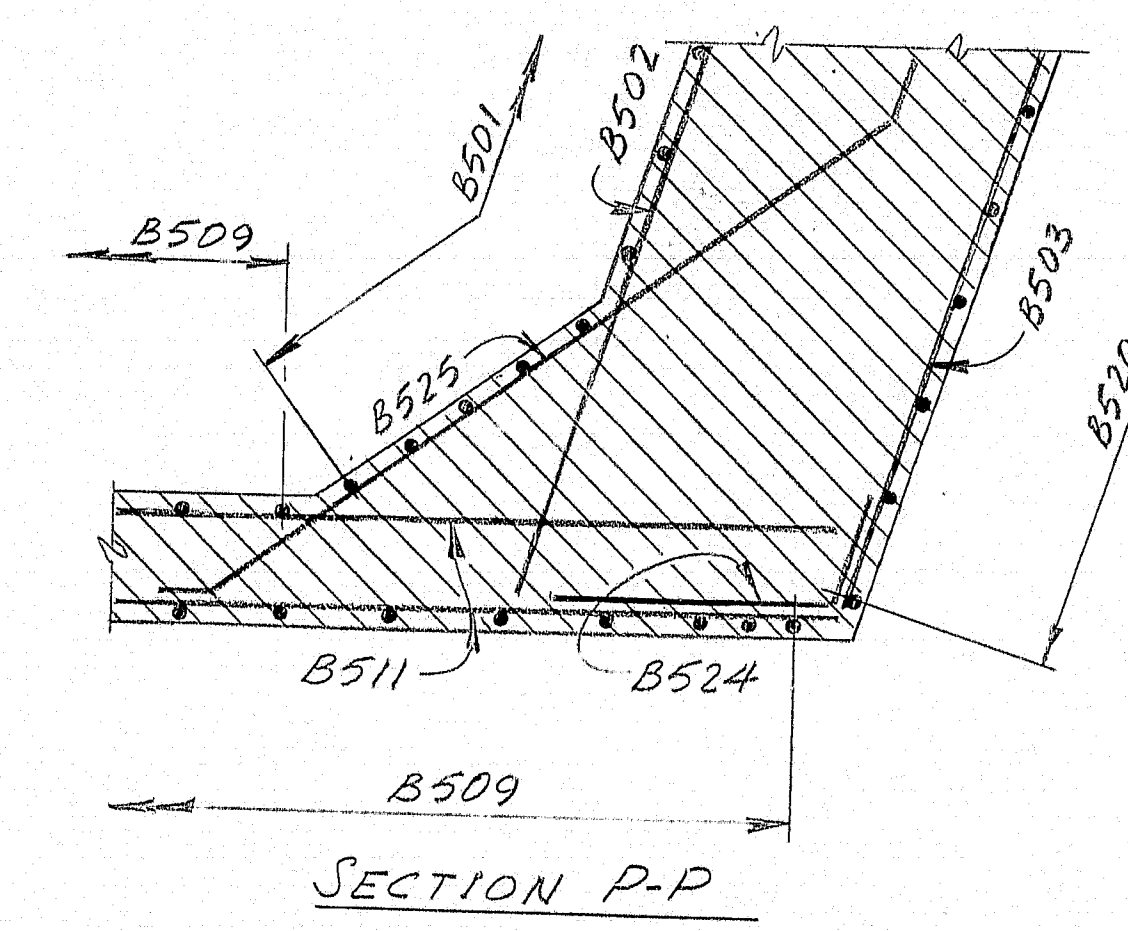
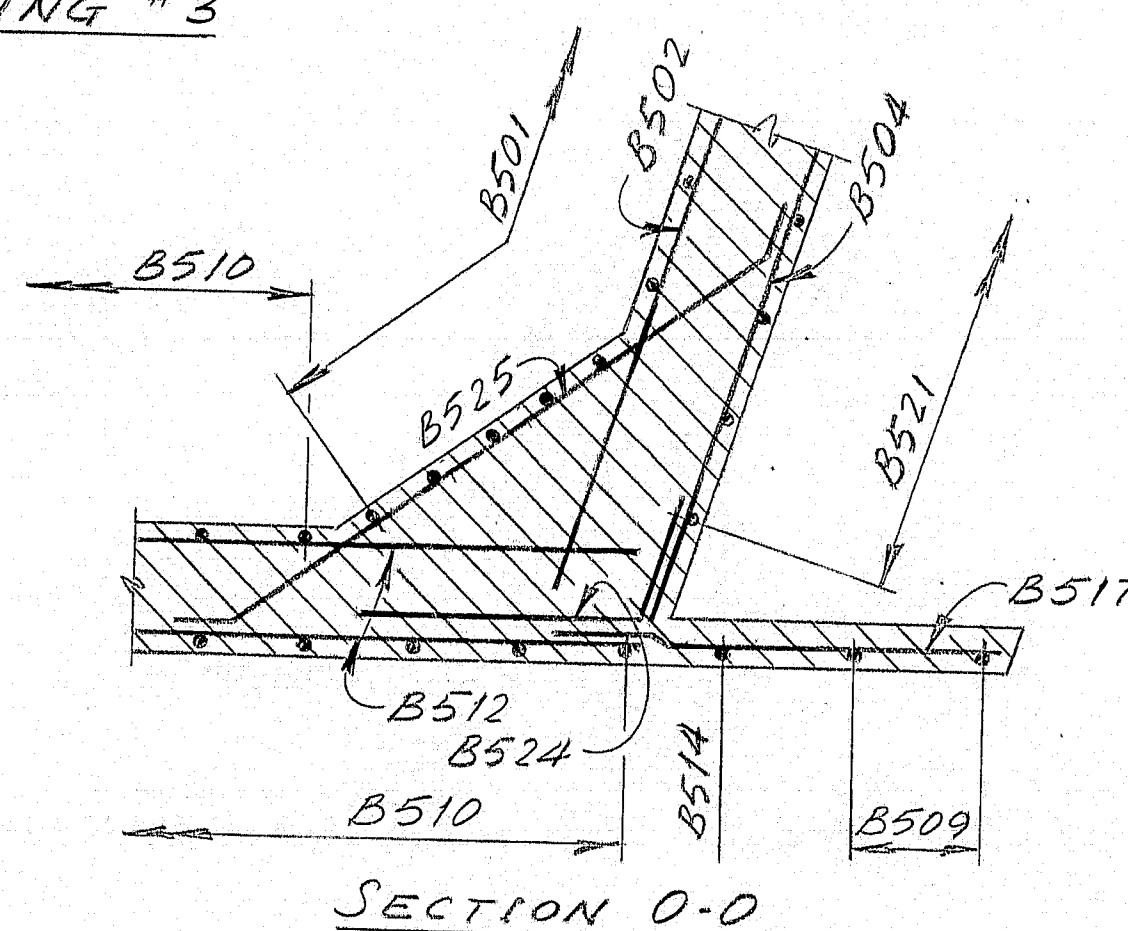
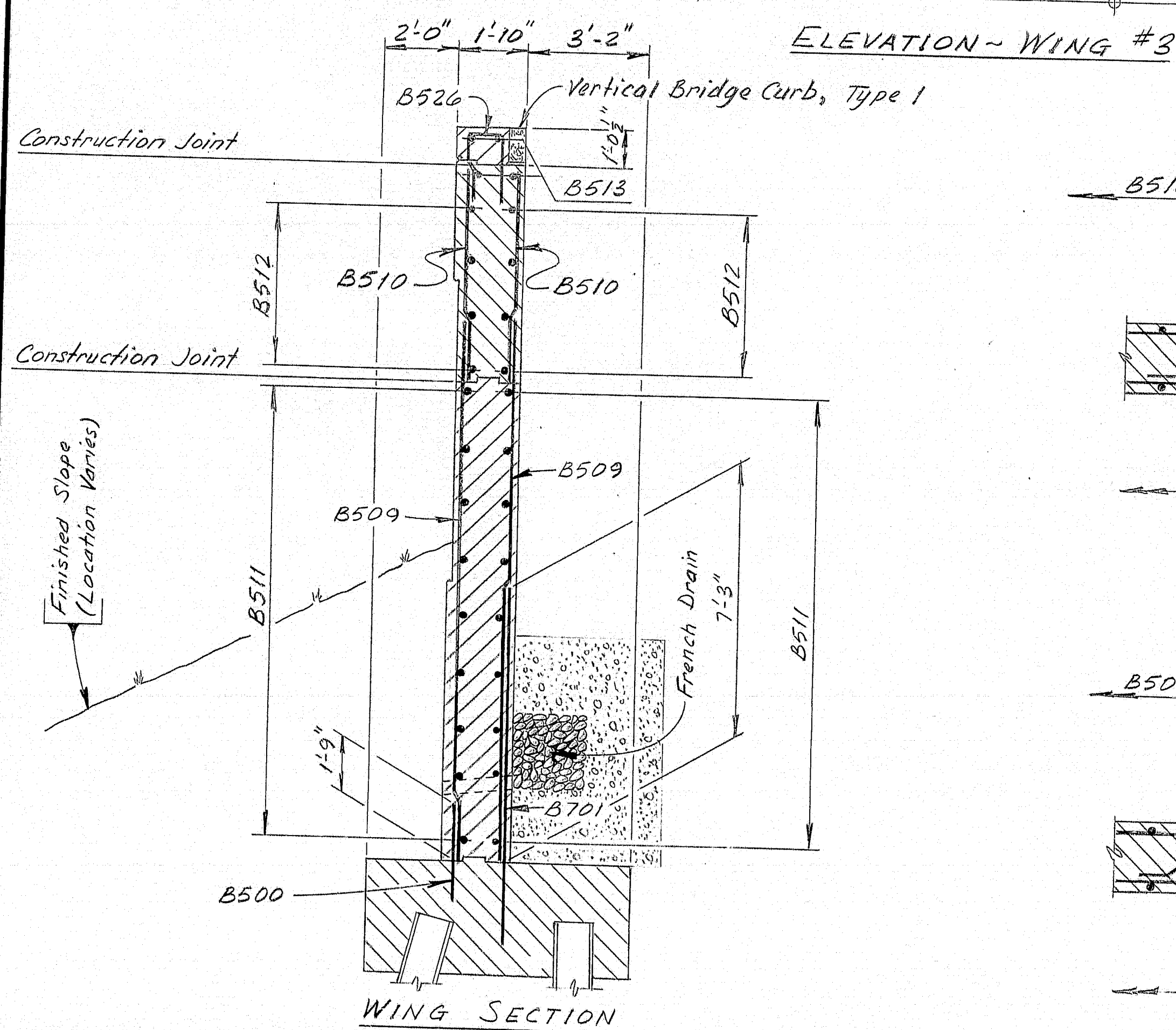
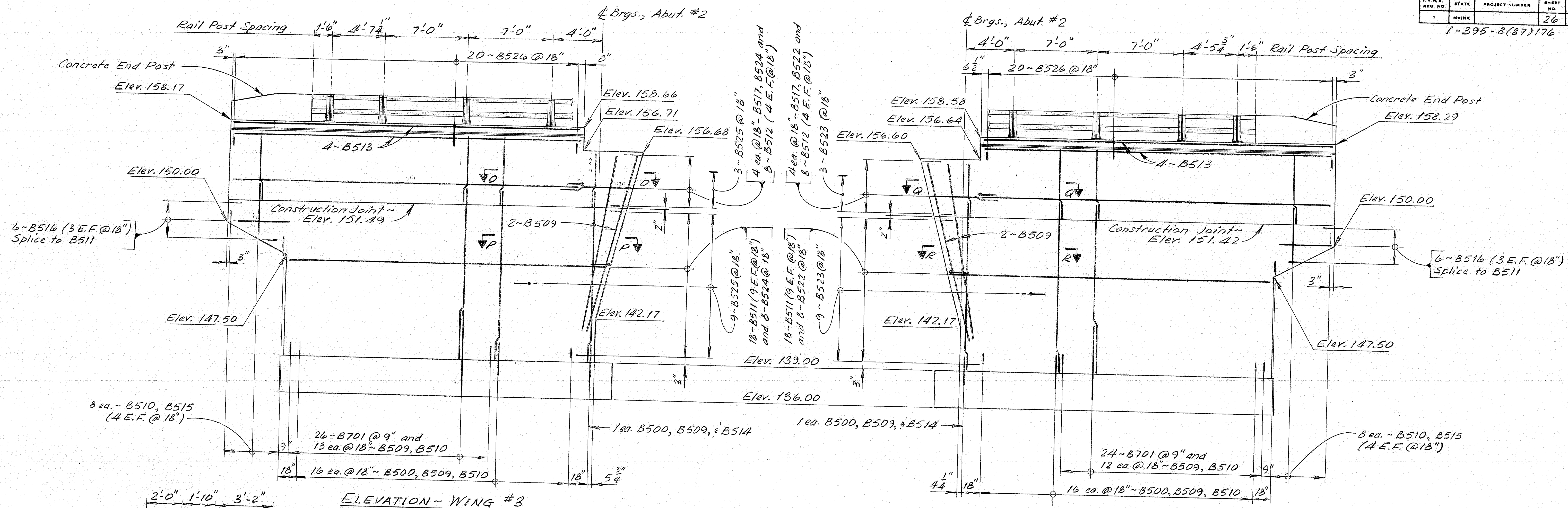
Structural Steel Alternate

PROJECT DESIGN ENGINEER	DATE
DESIGN: D. D. D.	12/1/82
CHECKED: D. D. D.	12/1/82
REVISIONS:	
FIELD CHANGES:	

BRUNING 44-132-45710



F.R.A. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87)176	26	84



SYMBOLS  
 [Hatched Box] New Concrete (Plan or Elevation)  
 [Solid Box] New Concrete (Section)

As Built 1984 R.M.J.  
 STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION  
 GREEN POINT ROAD  
 OVER  
 INTERSTATE 395  
 BREWER  
 PENOBSCOT COUNTY  
 ABUTMENT No. 2 WINGS AND DETAILS  
 SHEET 6 OF 15 AUGUSTA, MAINE  
 Structural Steel Alternate

183-151

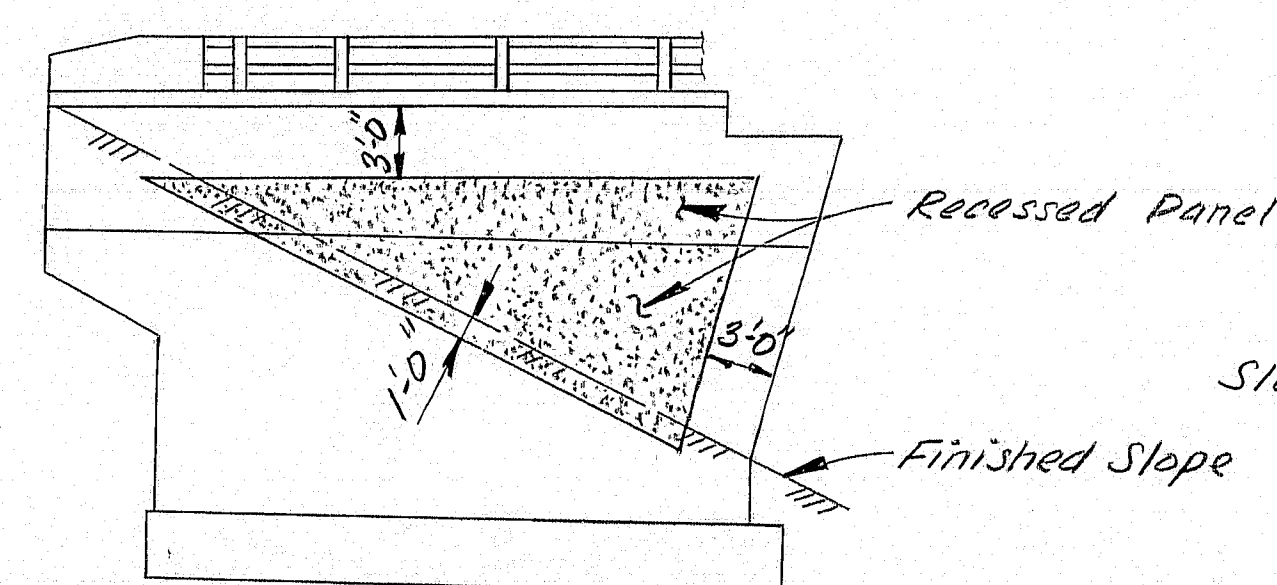
PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED	Lee, D. D. D. D. D.	12/1/83
CHECKED	Lee, D. D. D. D. D.	12/1/83
REVISIONS		
FIELD CHANGES		

BRIDGE 44-122-25710

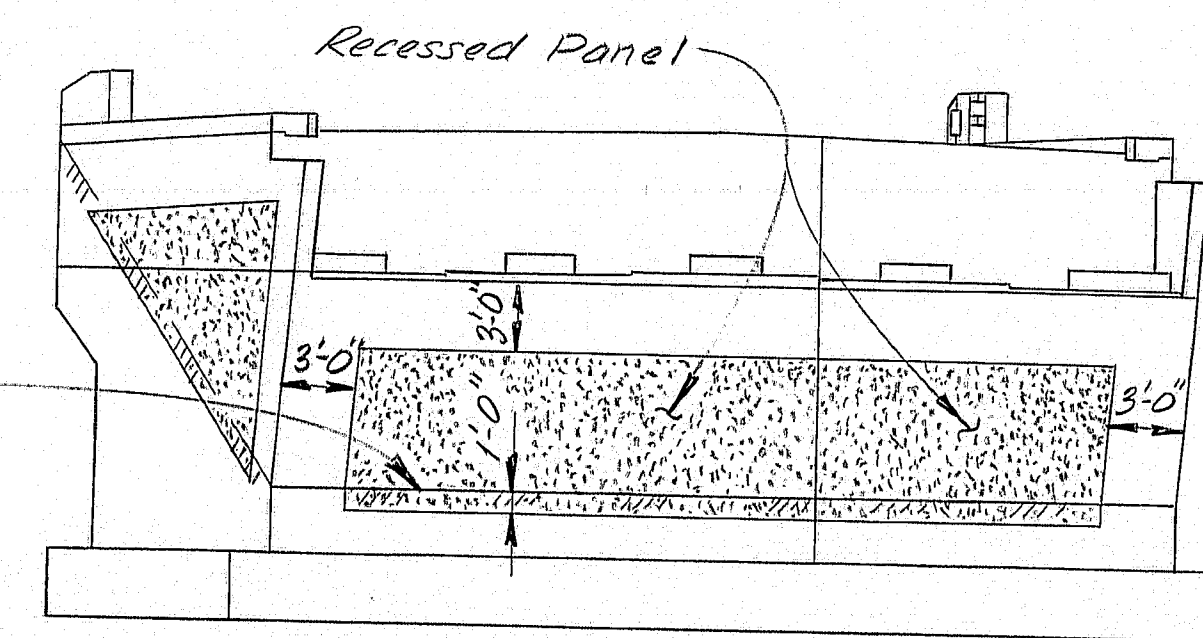




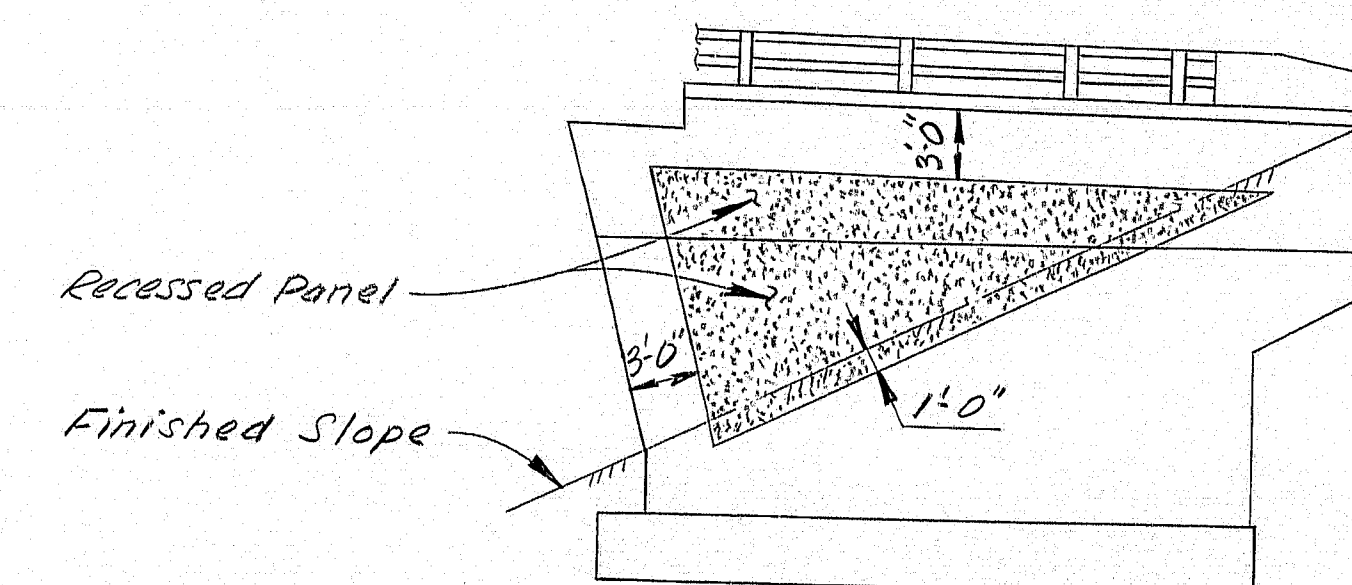




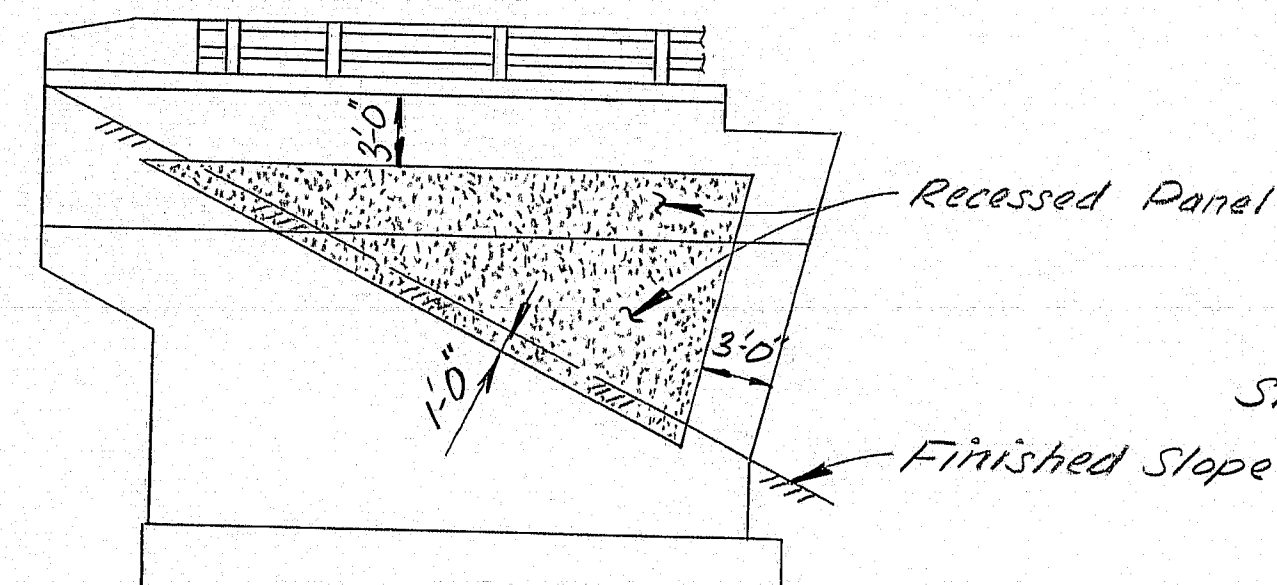
ELEVATION ~ RIGHT WING



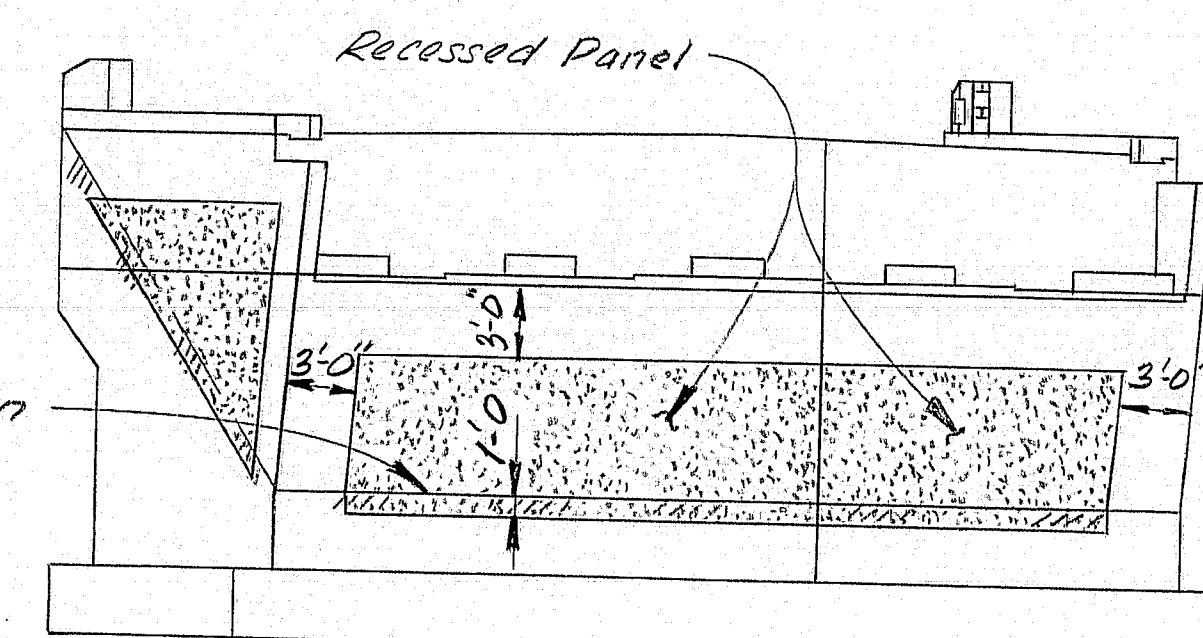
ELEVATION ~ ABUTMENT No. 1



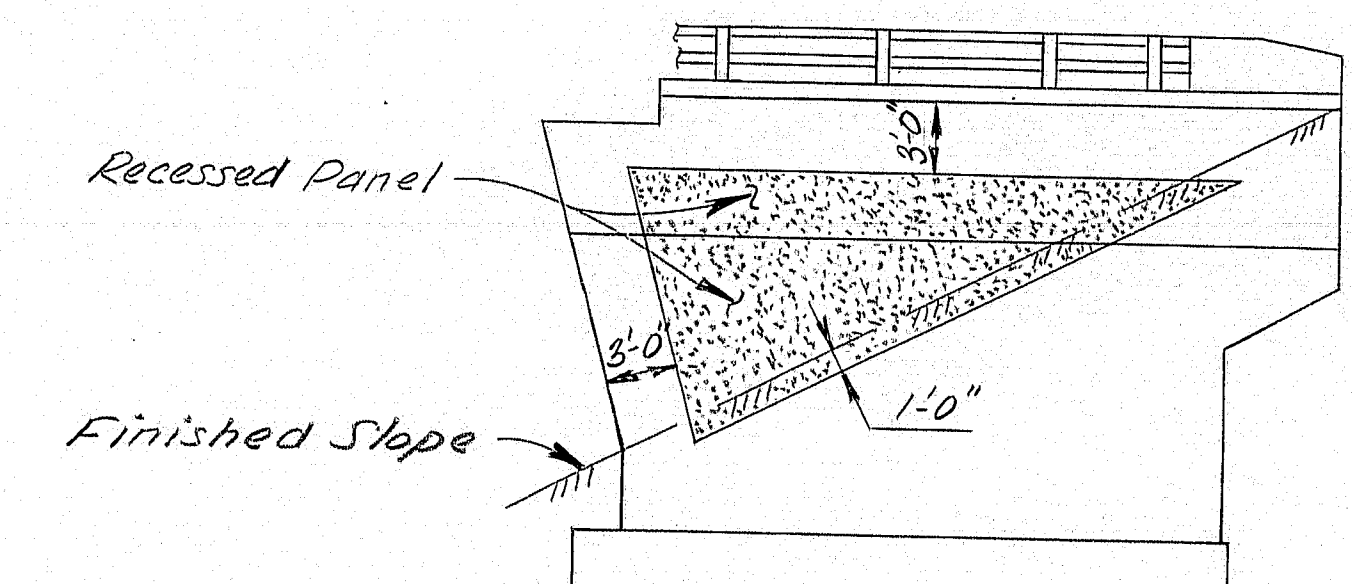
ELEVATION ~ LEFT WING



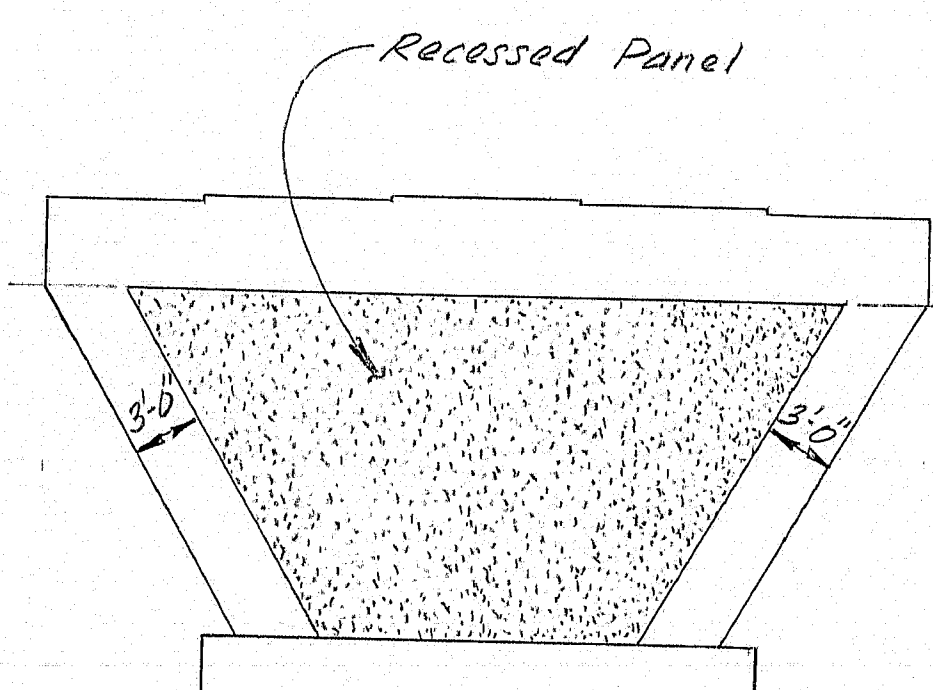
ELEVATION ~ LEFT WING



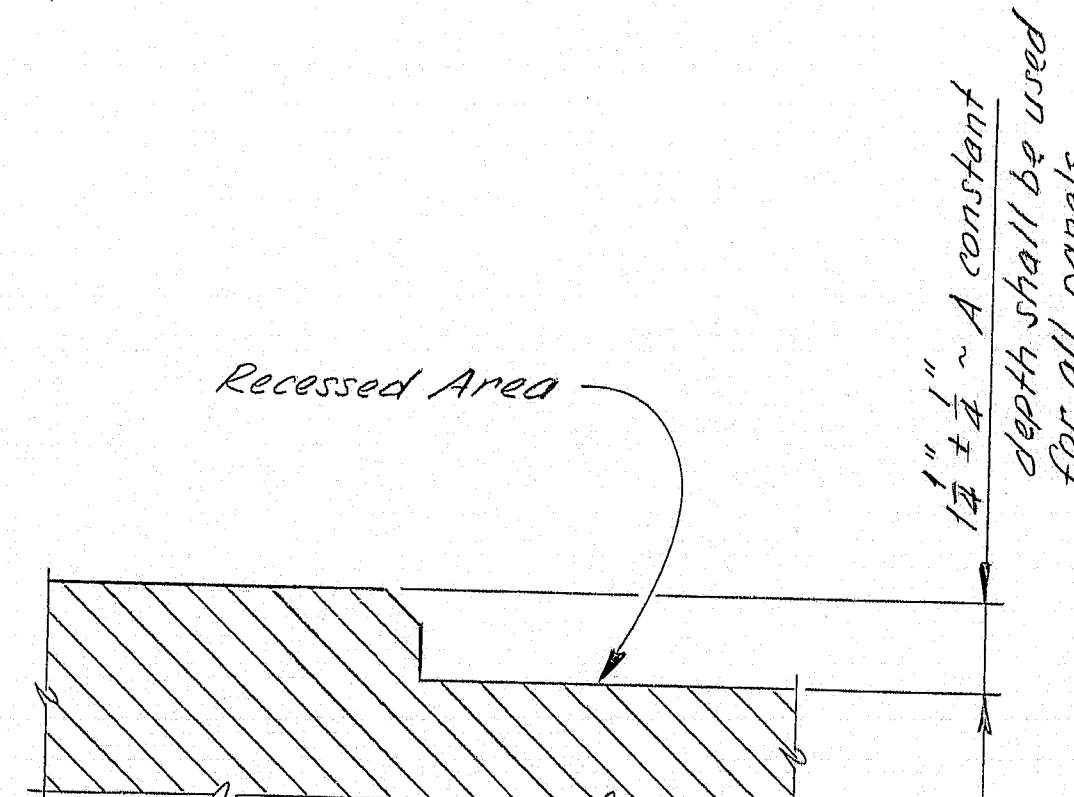
ELEVATION ~ ABUTMENT No. 2



ELEVATION ~ RIGHT WING



ELEVATION ~ PIER  
(Typ. for both sides)



RECESS DETAIL

RECESSED PANEL NOTES

1. Special care shall be exercised so that form joints at the exposed face of concrete shall be tight.
2. To insure a consistent surface texture, concrete aggregate shall be from the same source and portland cement from the same manufacturer throughout placement of the exposed substructure.
3. All fins and projections in the exposed face of concrete shall be removed and all holes patched to create a surface of uniform texture.
4. No deductions in the concrete pay volumes shall be made for the recessed panels.

As Built 1984 RME

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
RECESSED PANEL DETAILS

SHEET 8 OF 15 AUGUSTA, MAINE

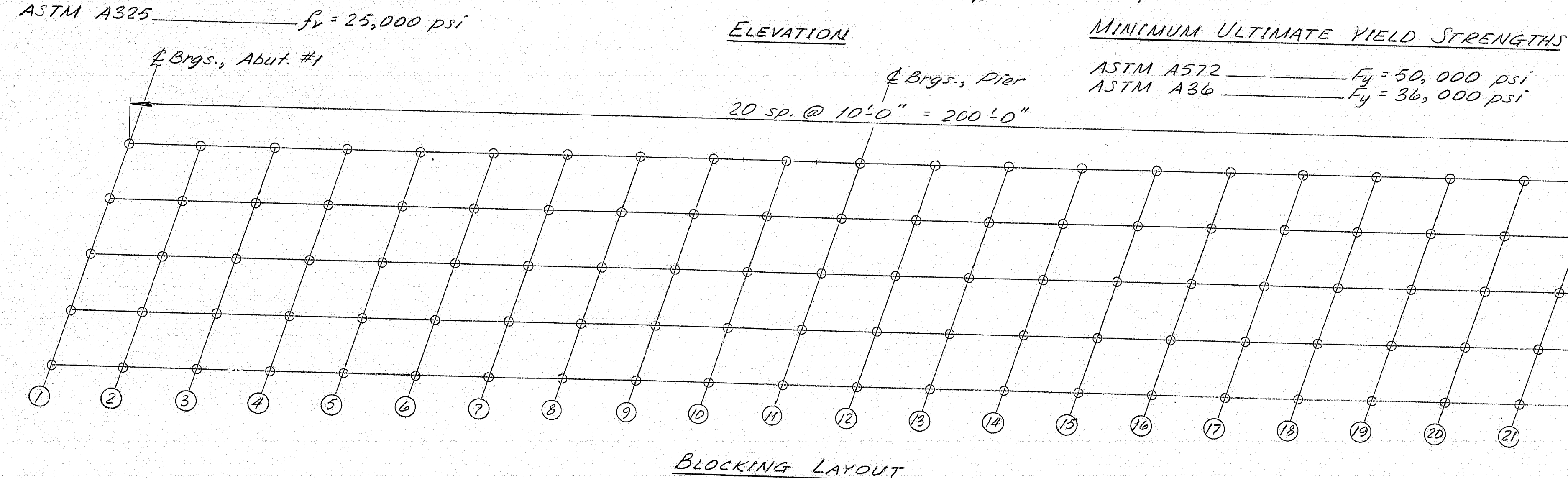
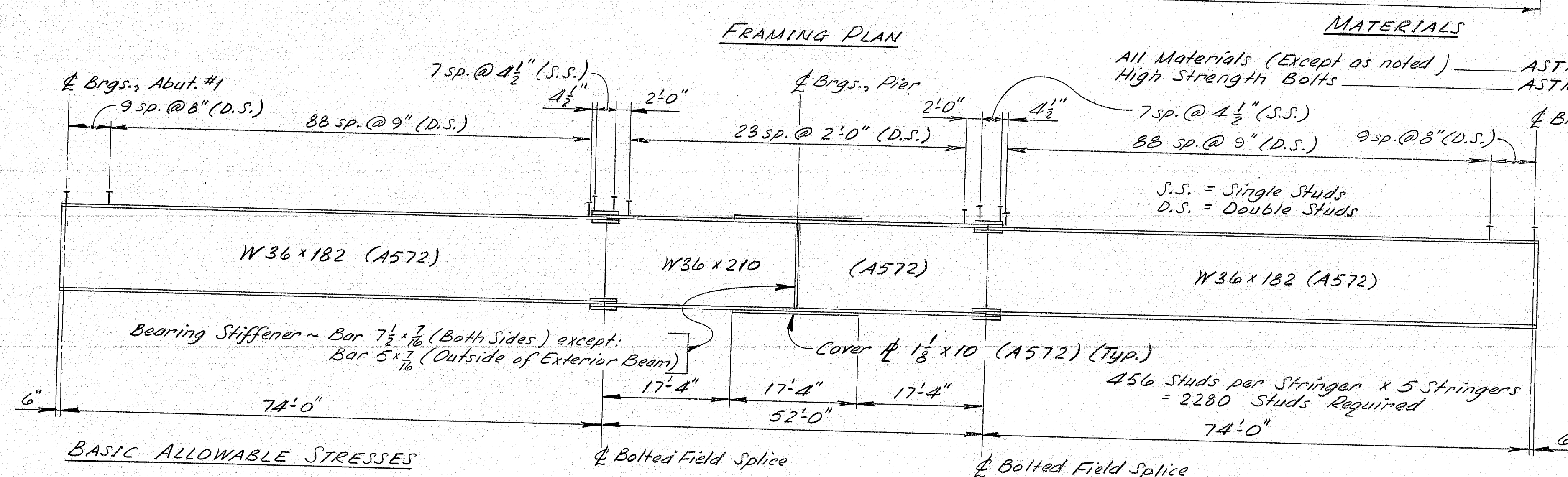
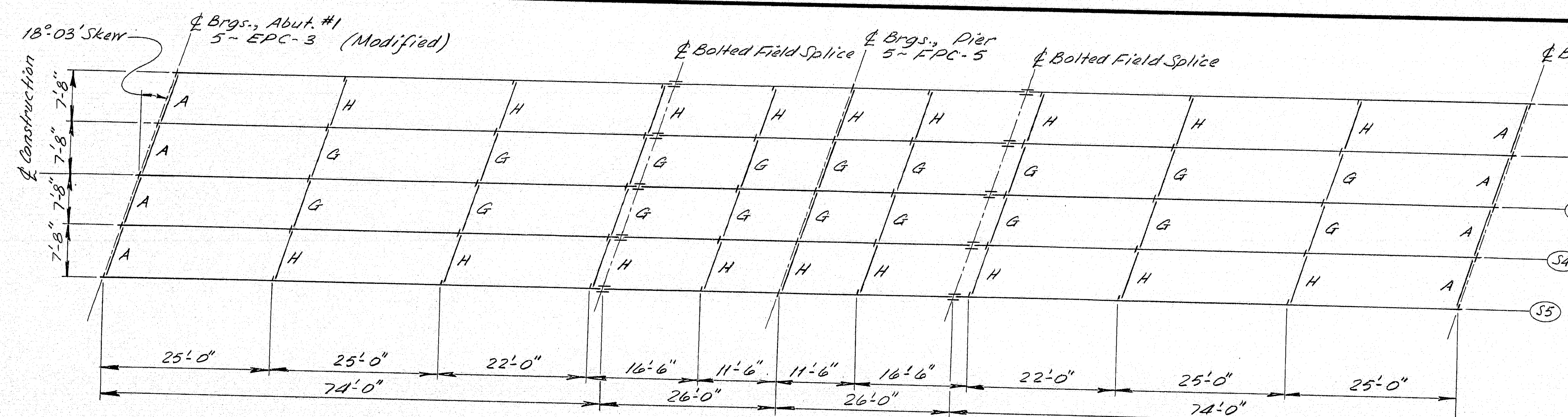
Structural Steel Alternate

183-153

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	02/82
CHECKED	02/82
REVISIONS	12/83
FIELD CHANGES	

BRUNING 44-132-45710

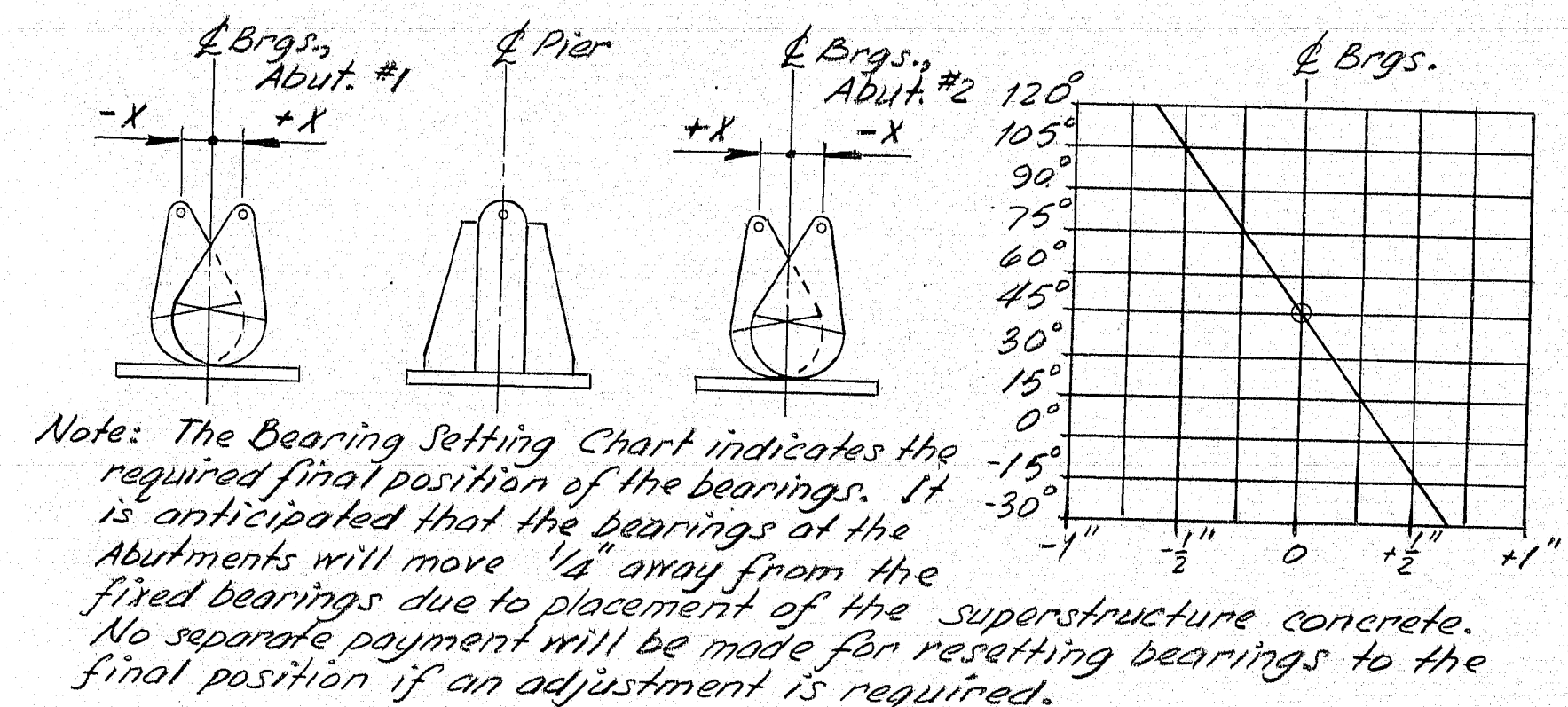
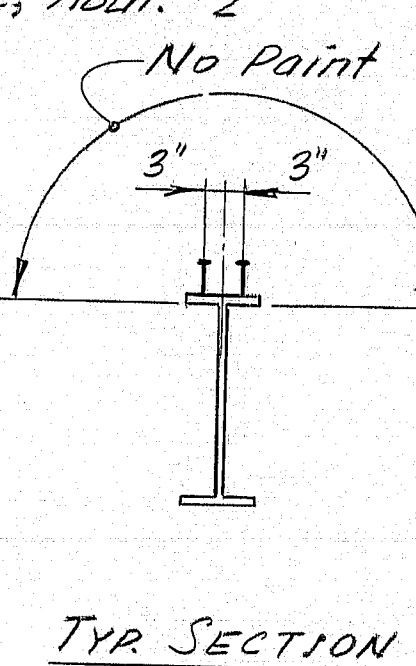
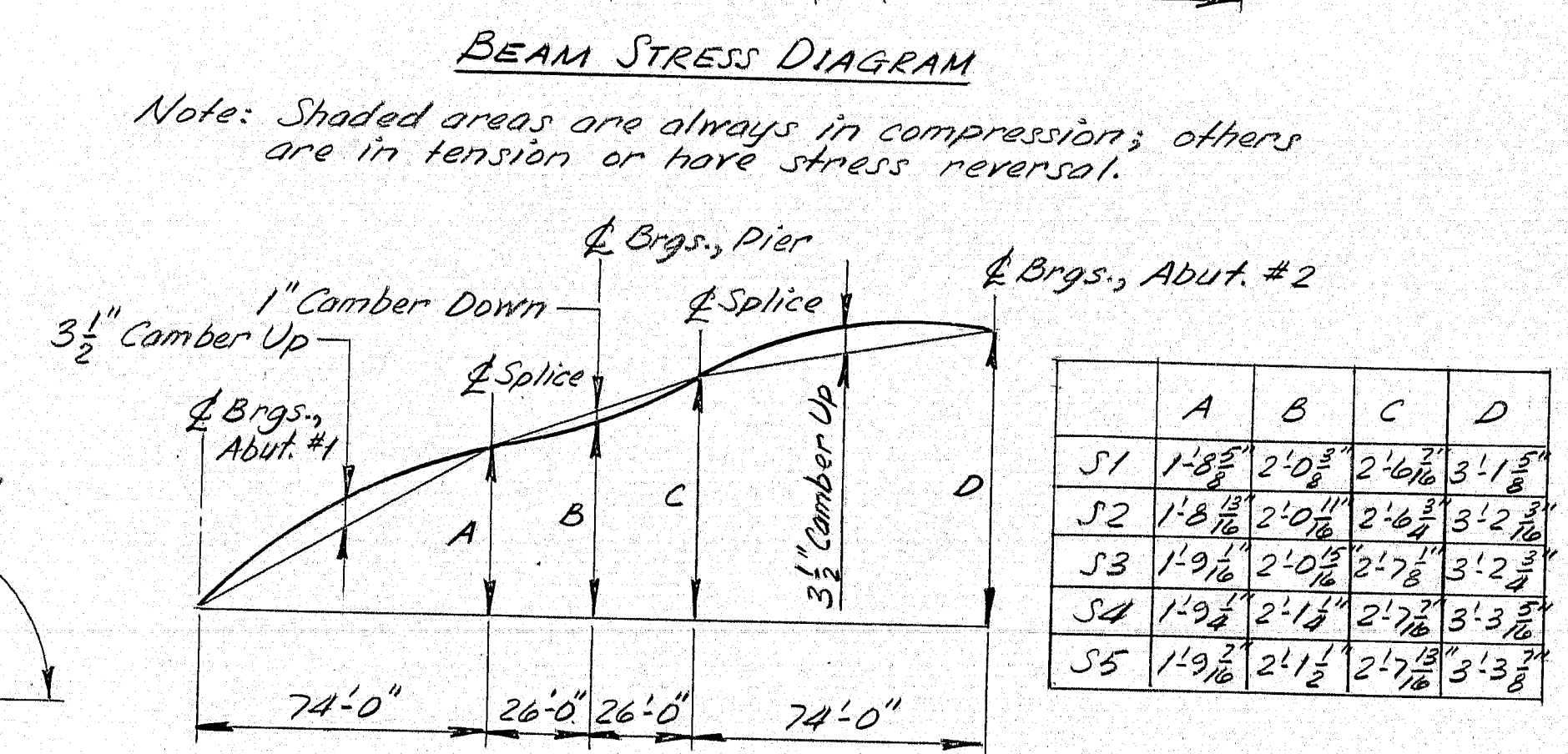
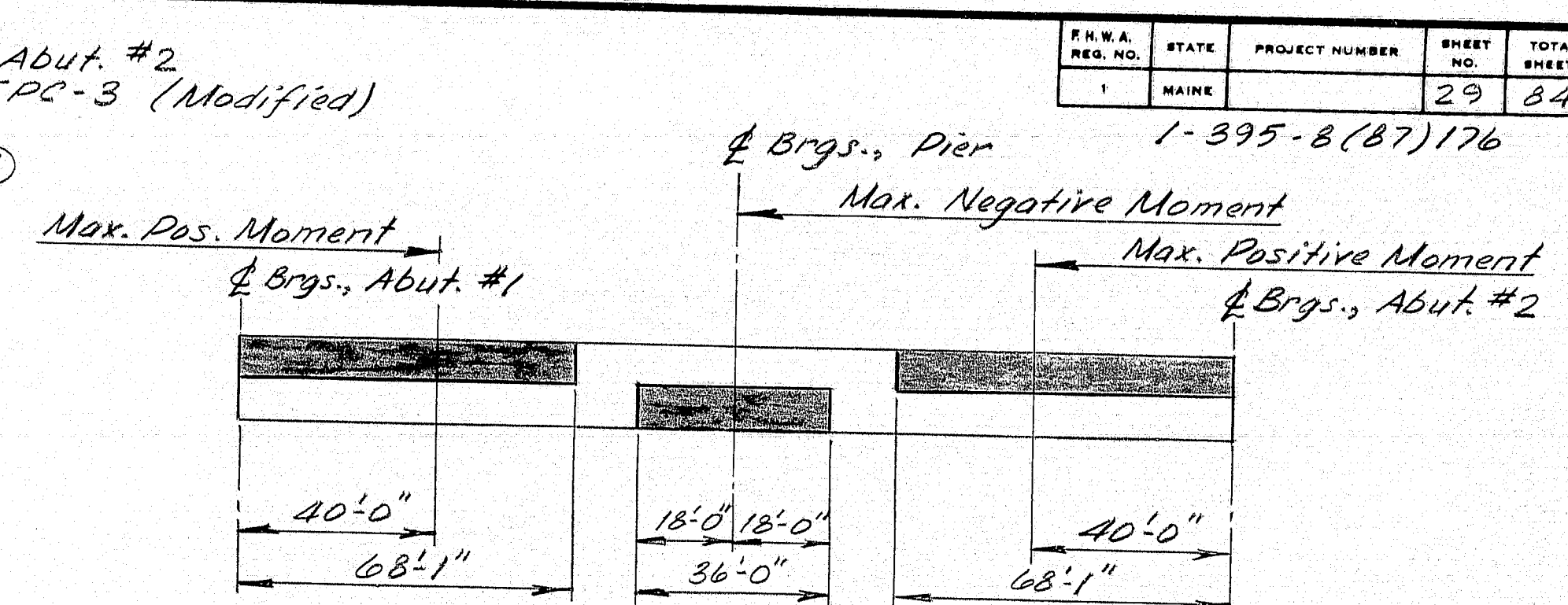




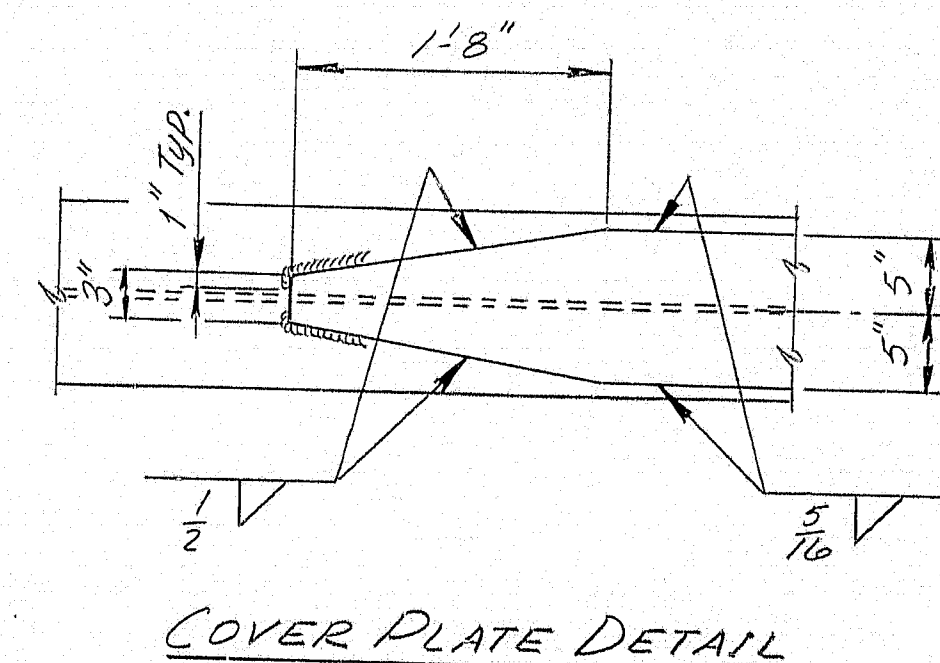
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Stringer 1	153.80	154.14	154.45	154.72	154.96	155.16	155.32	155.45	155.57	155.69	155.84	156.00	156.20	156.39	156.57	156.73	156.84	156.92	156.96	156.96	156.96
Stringer 2	153.90	154.24	154.55	154.83	155.07	155.27	155.43	155.56	155.68	155.81	155.96	156.13	156.32	156.52	156.70	156.86	156.98	157.06	157.10	157.10	157.20
Stringer 3	154.00	154.33	154.65	154.93	155.17	155.37	155.54	155.68	155.80	155.93	156.08	156.25	156.45	156.64	156.83	156.99	157.11	157.19	157.23	157.24	157.20
Stringer 4	153.77	154.11	154.43	154.71	154.96	155.16	155.33	155.47	155.59	155.72	155.87	156.05	156.25	156.45	156.64	156.80	156.92	157.01	157.05	157.06	157.00
Stringer 5	153.55	153.89	154.21	154.50	154.74	154.95	155.12	155.26	155.39	155.52	155.67	155.85	156.05	156.26	156.45	156.61	156.74	156.82	156.87	156.88	156.80

EPC-3 BEARING MODIFICATIONS			
C	D	H	J
9"	12 9"	3 1/2"	3 1/4"

Anchor Bolt Mod.,  
EPC-3 and EPC-5:  
1 1/2"  $\phi$  x 17" long with  
10" Swedged Embed.

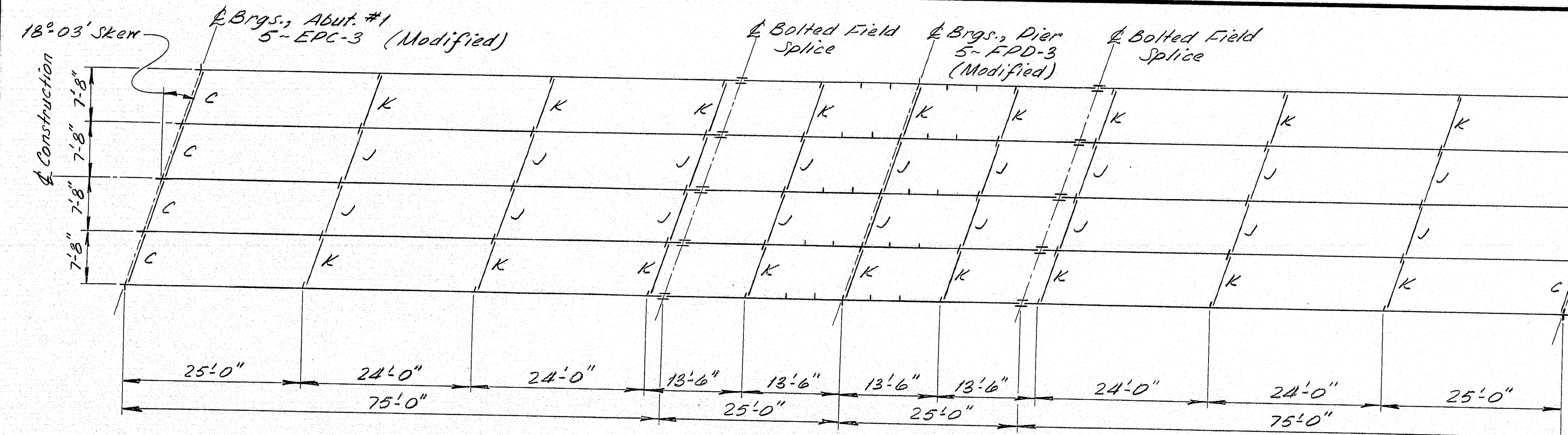


- ### STRUCTURAL STEEL NOTES
1. Camber ordinates, as shown, are computed to compensate for all dead load deflections and for the curvature of the finished grade profile.
  2. Bearing stiffeners shall be plumb after erection and dead loading of the structure.
  3. Cross-frame or diaphragm connection plates may be either plumb or normal to the top flange.
  4. Filler plates may be A36 steel and mill tests for filler plate material will not be required.
  5. Theoretical blocking shall be  $1\frac{1}{2}$ " nom. at  $\frac{1}{4}$  Brgs., Abutments and Piers.

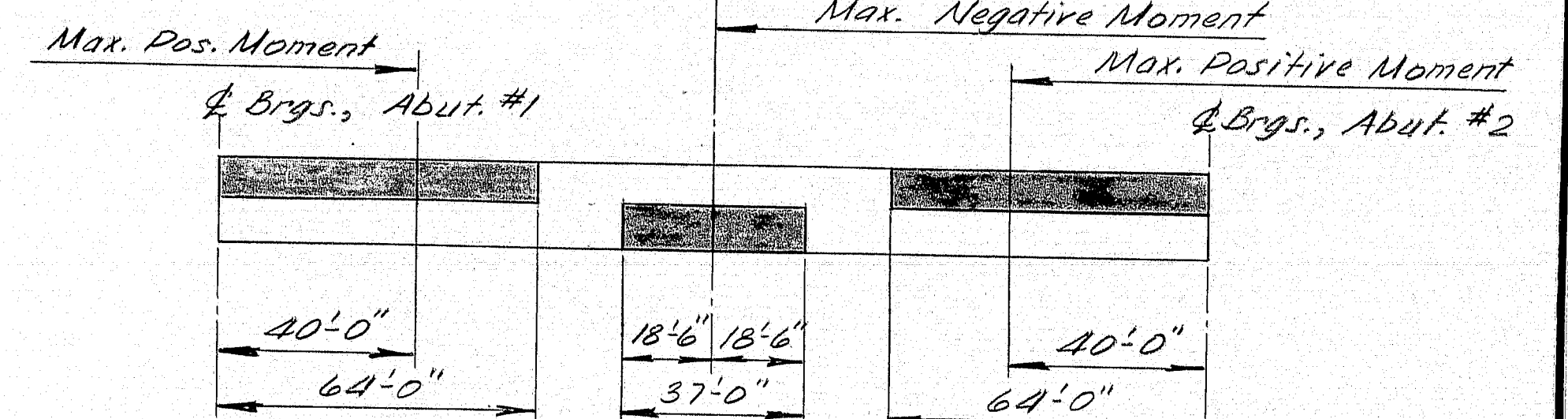




F.R.A. REQ. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(87)176	30	84

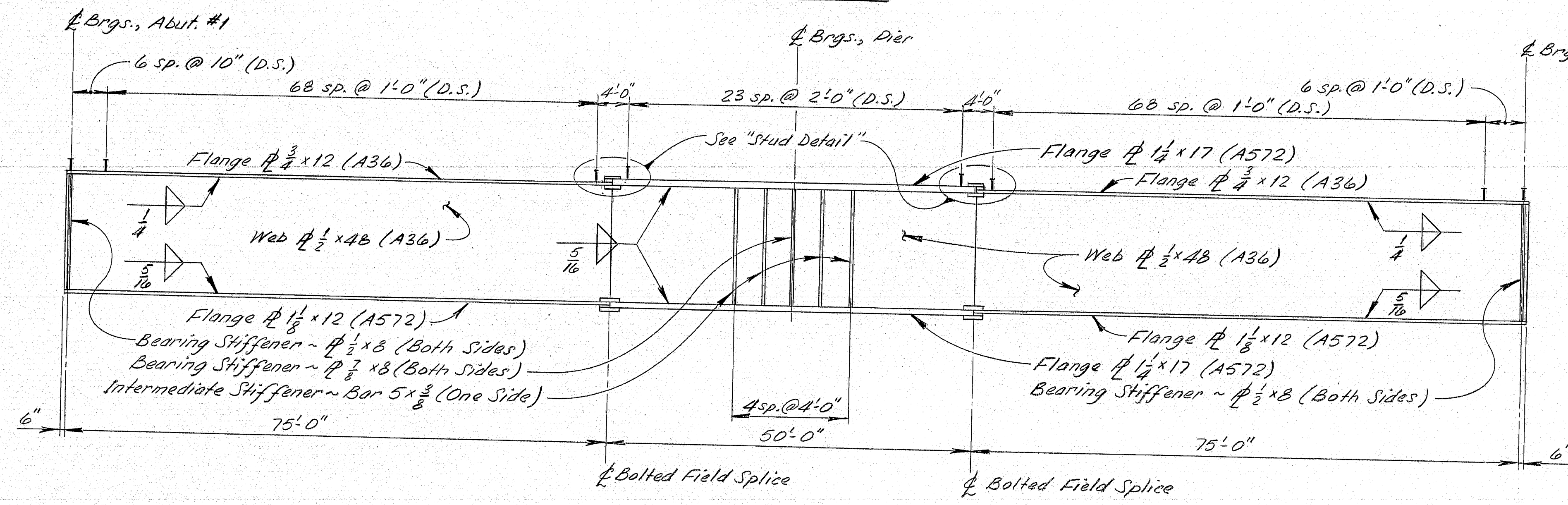


FRAMING PLAN



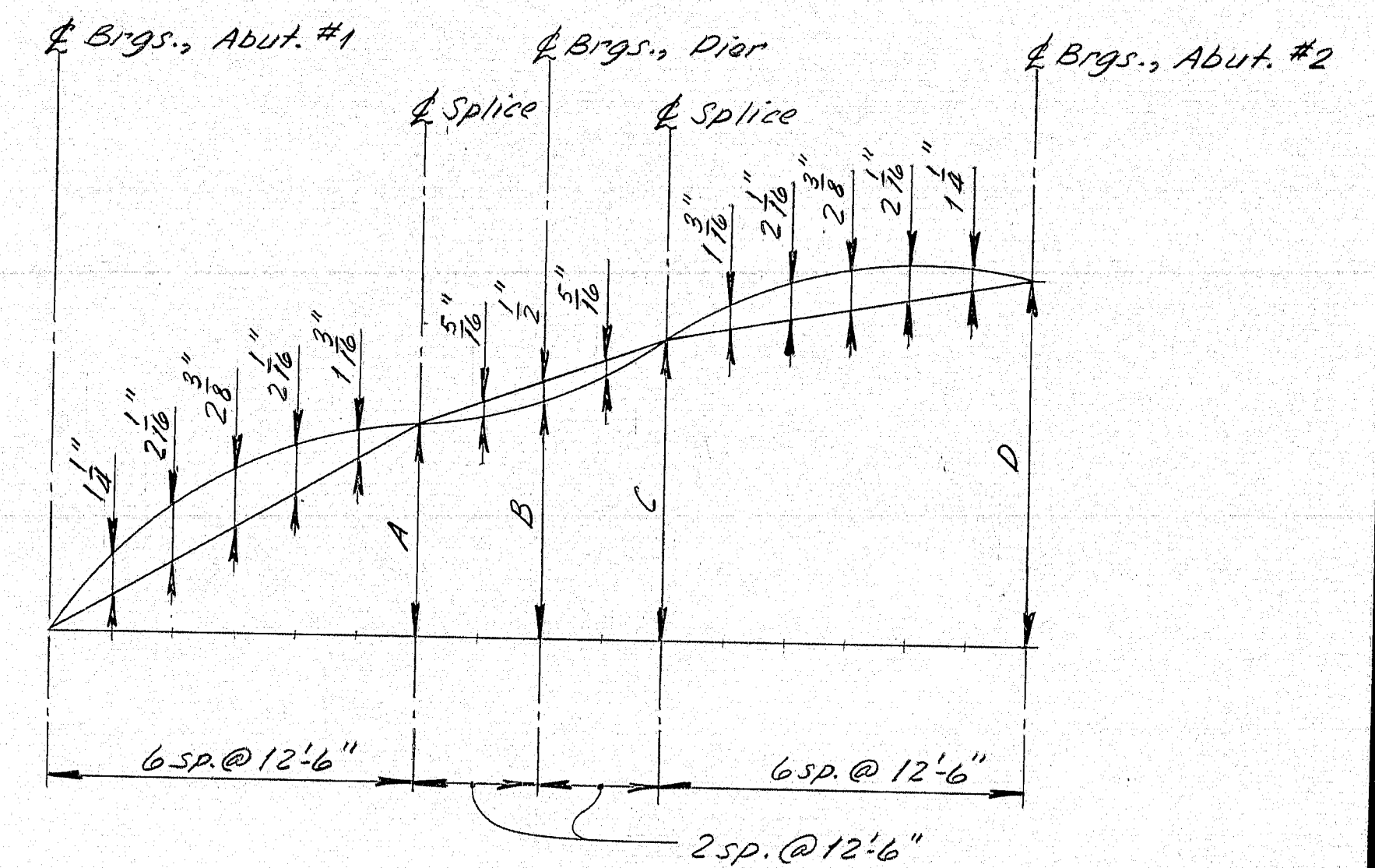
BEAM STRESS DIAGRAM

Note: Shaded areas are always in compression; others are in tension or have stress reversal.

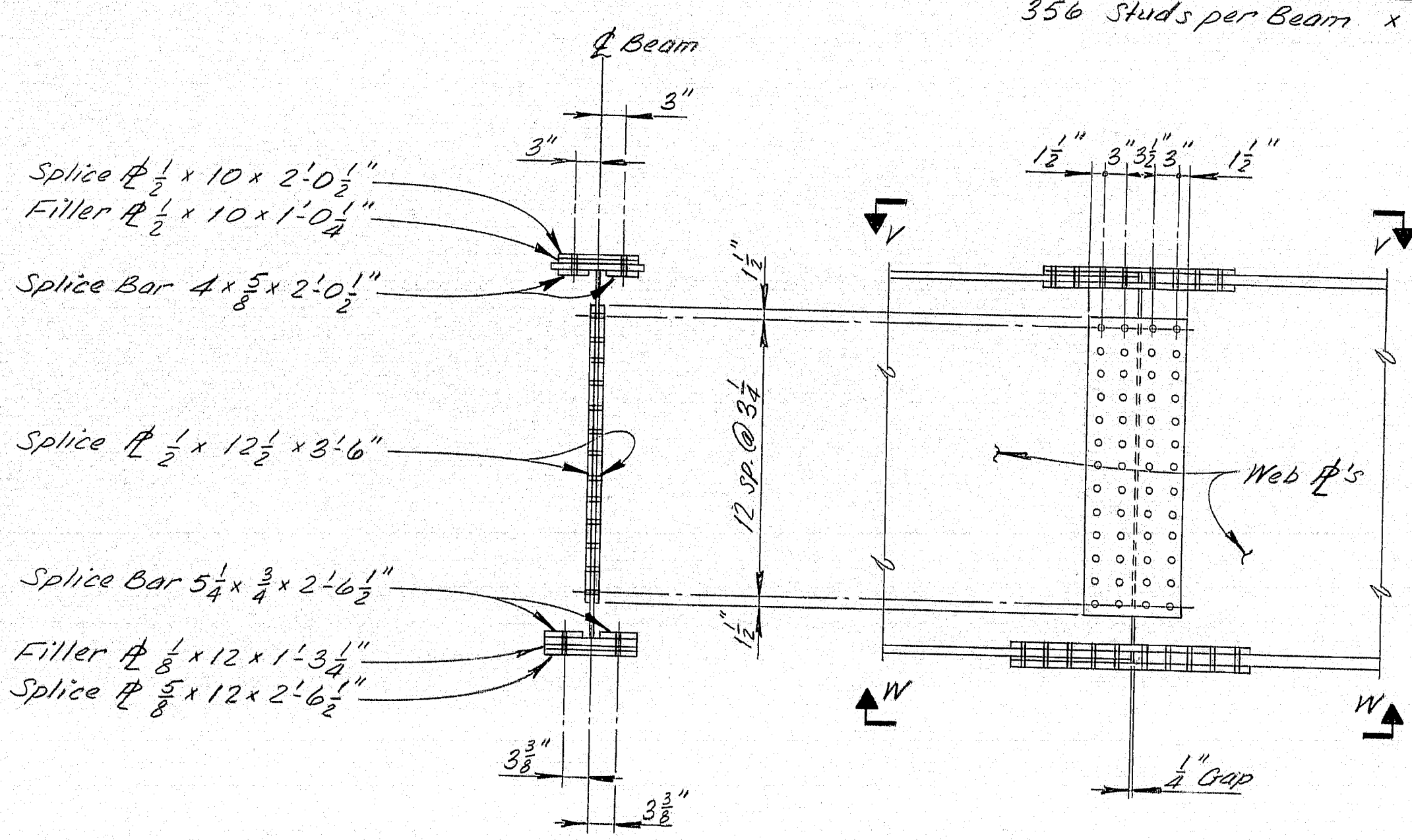


ELEVATION

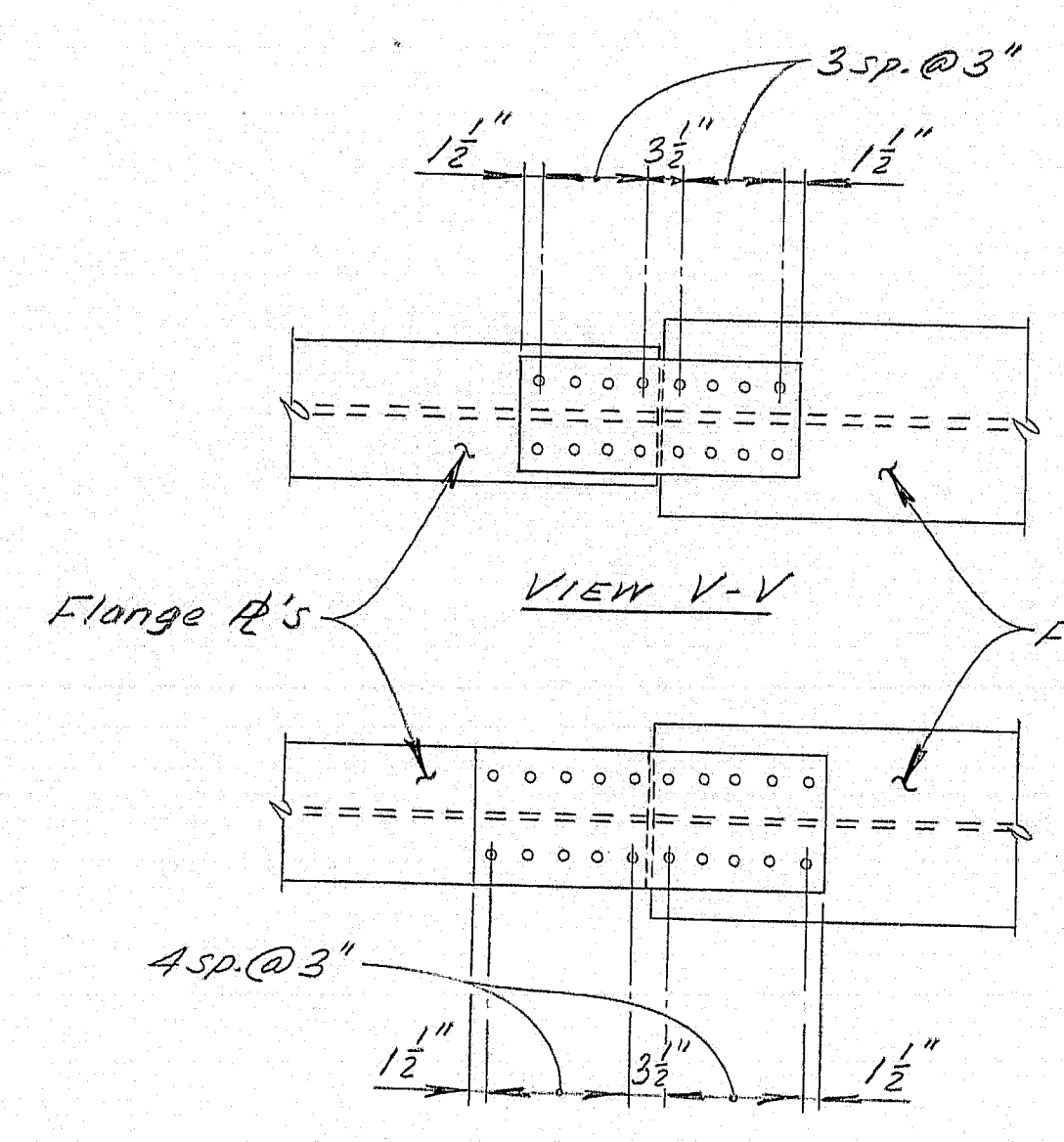
356 Studs per Beam x 5 Beams = 1780 Studs



CAMBER DIAGRAM



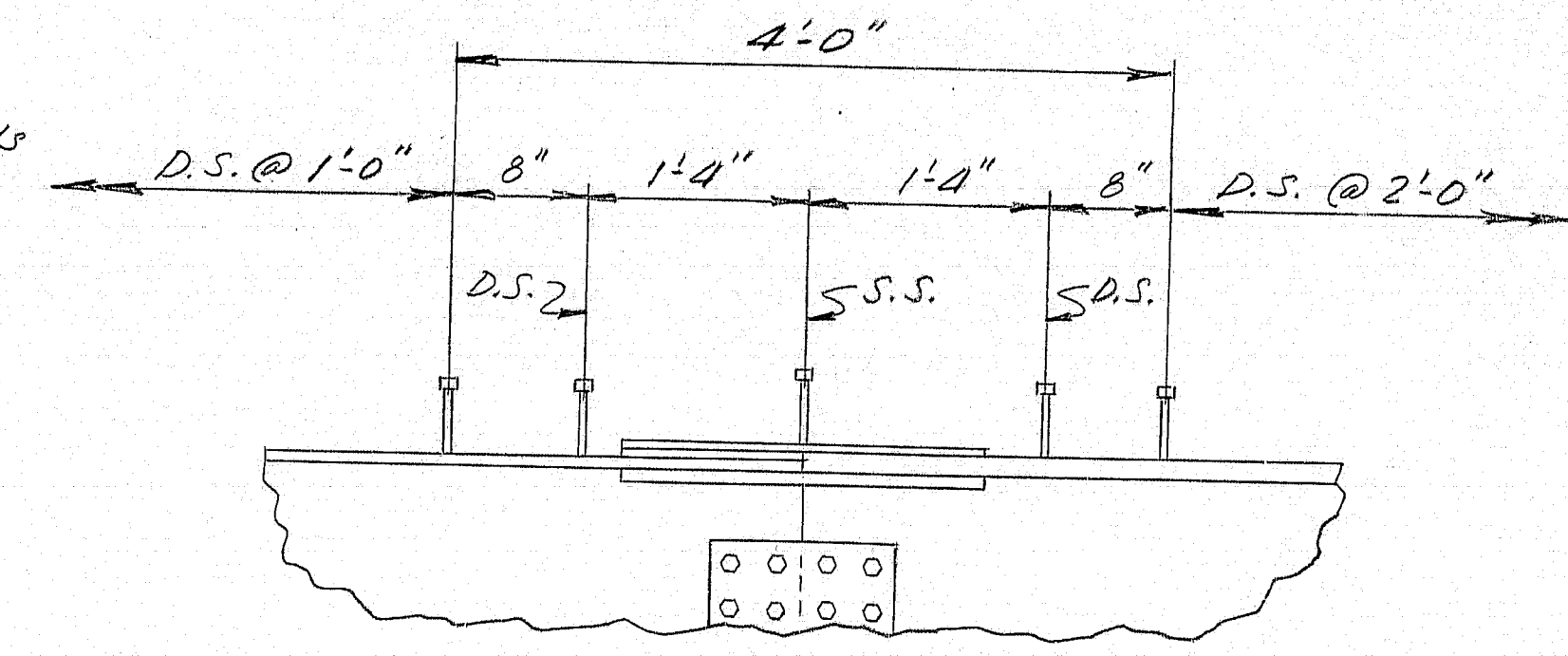
FIELD SPICE DETAILS



Note: All field splice connections shall be made with 3/4" φ ASTM A325, Type 1 High Strength Bolts. Holes shall be 15/16" φ.

BEARING PEDESTAL MODIFICATIONS				
	C	D	H	J
EPC-3	10"	1'-10"	3 1/2"	3 1/2"
FPD-3	1'-3"	2'-1"	10 1/2"	-
EPC-3 Anchor Bolt Mod: 1 1/4" φ x 17" long with 10" Sredged Embedment)				

CAMBER TABLE				
	A	B	C	D
S1	1'-8 3/8"	2'-0 3/8"	2'-5 3/8"	3'-1 3/8"
S2	1'-8 3/8"	2'-0 3/8"	2'-5 3/8"	3'-1 3/8"
S3	1'-8 3/8"	2'-0 3/8"	2'-6 3/8"	3'-2 3/8"
S4	1'-8 3/8"	2'-0 3/8"	2'-6 3/8"	3'-2 3/8"
S5	1'-9"	2'-1 1/8"	2'-7"	3'-3 1/8"



STUD DETAIL  
S.S. = Single Studs  
D.S. = Double Studs

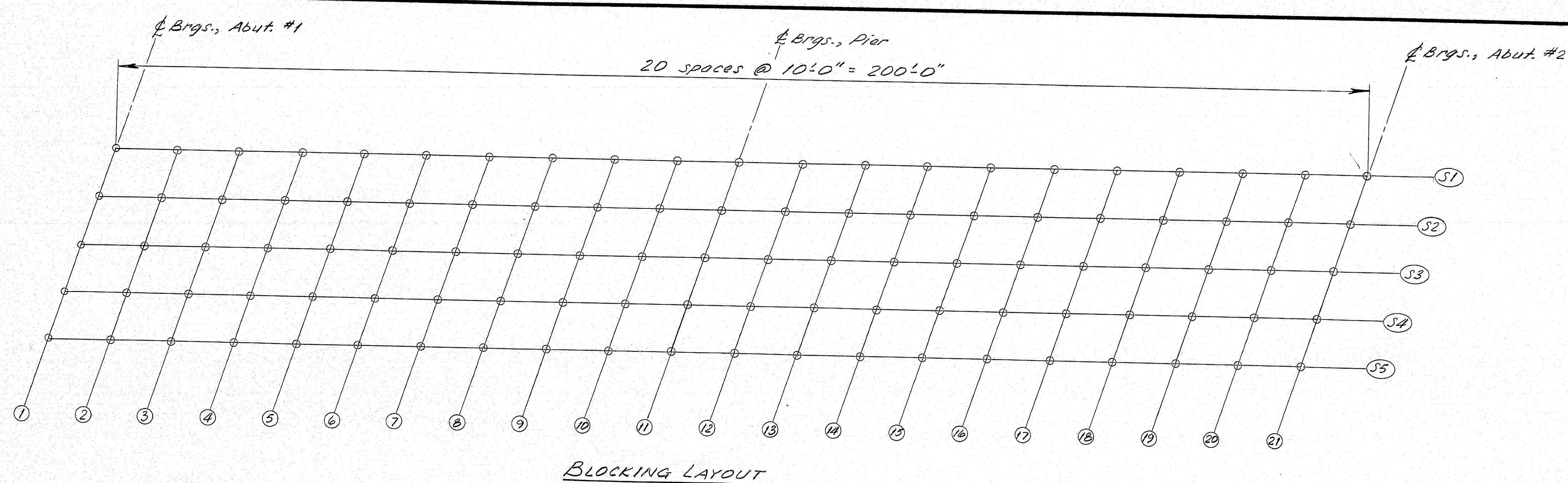
THIS OPTION NOT USED WITH  
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD  
OVER  
INTERSTATE 395  
BREWER  
PENOBSCOT COUNTY  
STRUCTURAL STEEL  
(Welded Beam Option)  
SHEET 10 OF 15 AUGUSTA, MAINE  
Structural Steel Alternate

183-155

PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	Nov 82
CHECKED	12/8/83
REVISIONS	
FIELD CHANGES	

BRUNING 44-132 45710





BLOCKING LAYOUT

BOTTOM OF SLAB ELEVATIONS																					
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
S1	153.80	154.10	154.38	154.64	154.87	155.07	155.24	155.40	155.54	155.69	155.83	156.00	156.17	156.34	156.50	156.64	156.75	156.84	156.89	156.92	156.94
S2	153.90	154.20	154.48	154.74	154.97	155.18	155.35	155.51	155.66	155.80	155.95	156.12	156.30	156.47	156.63	156.77	156.88	156.97	157.03	157.07	157.08
S3	154.00	154.30	154.59	154.85	155.08	155.29	155.47	155.63	155.77	155.92	156.07	156.24	156.42	156.59	156.76	156.90	157.02	157.11	157.17	157.21	157.23
S4	153.77	154.08	154.37	154.63	154.87	155.07	155.26	155.42	155.57	155.72	155.87	156.05	156.22	156.40	156.57	156.71	156.83	156.92	156.99	157.03	157.05
S5	153.55	153.86	154.15	154.41	154.65	154.86	155.05	155.21	155.36	155.51	155.67	155.85	156.03	156.21	156.37	156.52	156.64	156.74	156.80	156.85	156.87

STRUCTURAL STEEL NOTES

1. Camber ordinates, as shown, are computed to compensate for all dead load deflections and for the curvature of the finished grade profile.
2. No transverse butt weld splices will be allowed in the flange plates or web plates within ten feet from the points of maximum negative moment or maximum positive moment.
3. Sections of flange plates or web plates between transverse shop splices or between a transverse shop splice and a field splice shall not be less than 20 feet in length unless otherwise shown on the plans.
4. Butt weld splices in flanges shall be not less than one foot from transverse welds in the web plates.
5. Bearing stiffeners shall be plumb after erection and dead loading of the structure. Intermediate web stiffeners may be either plumb or normal to the top flange.
6. Cross-frame or diaphragm connection plates may be either plumb or normal to the top flange.
7. Filler plates may be ASTM A36 steel and mill tests for filler plate material will not be required.

MINIMUM ULTIMATE YIELD STRENGTHS

ASTM A572 \_\_\_\_\_  $F_y = 50,000$  psi  
 ASTM A36 \_\_\_\_\_  $F_y = 36,000$  psi

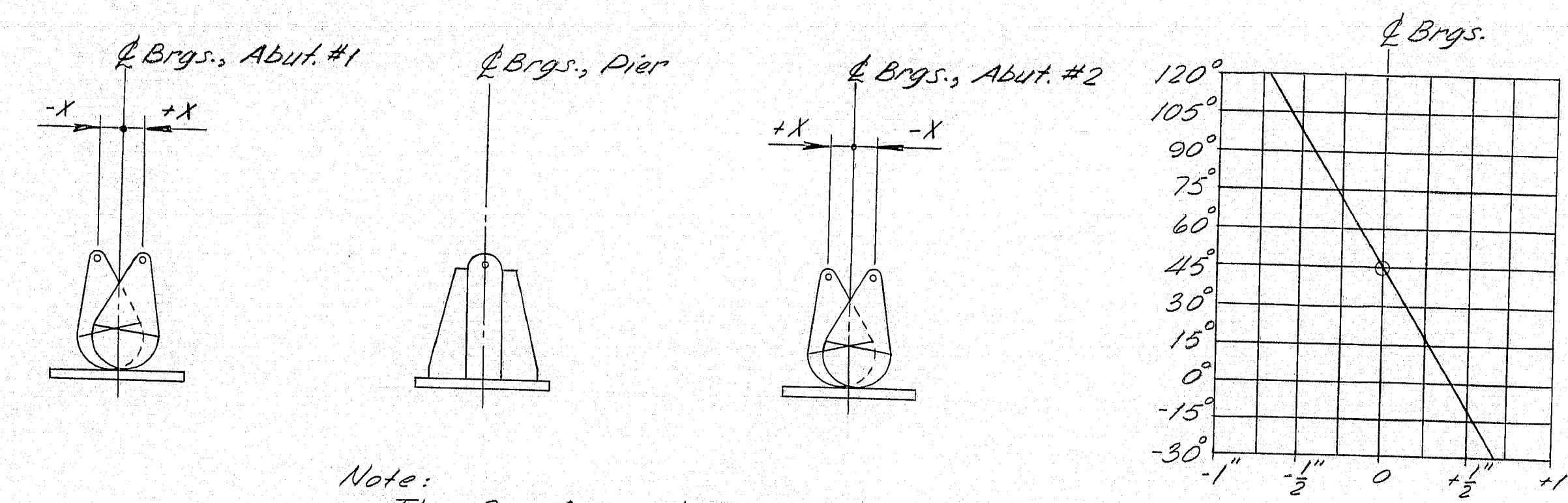
BASIC ALLOWABLE STRESSES

ASTM A325 \_\_\_\_\_  $F_r = 33,000$  psi

MATERIALS

All Material  
 (Except as otherwise noted) — ASTM A572  
 High Strength Bolts — ASTM A325,  
 Type 1

Note: Theoretical Blocking shall be  $2\frac{1}{2}$ " nominal at Brgs., Abutments and Pier.



Note:  
 The Bearing Setting Chart indicates the required final position of the bearings. It is anticipated that the bearings at the Abutments will move  $\frac{3}{16}$ " away from the fixed bearings due to placement of the Superstructure concrete. No separate payment will be made for resetting bearings to the final position if an adjustment is required.

BEARING SETTING DIAGRAM AND CHART

PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED	10/1/84	10/1/84
CHECKED	10/1/84	10/1/84
REVISIONS		
FIELD CHANGES		

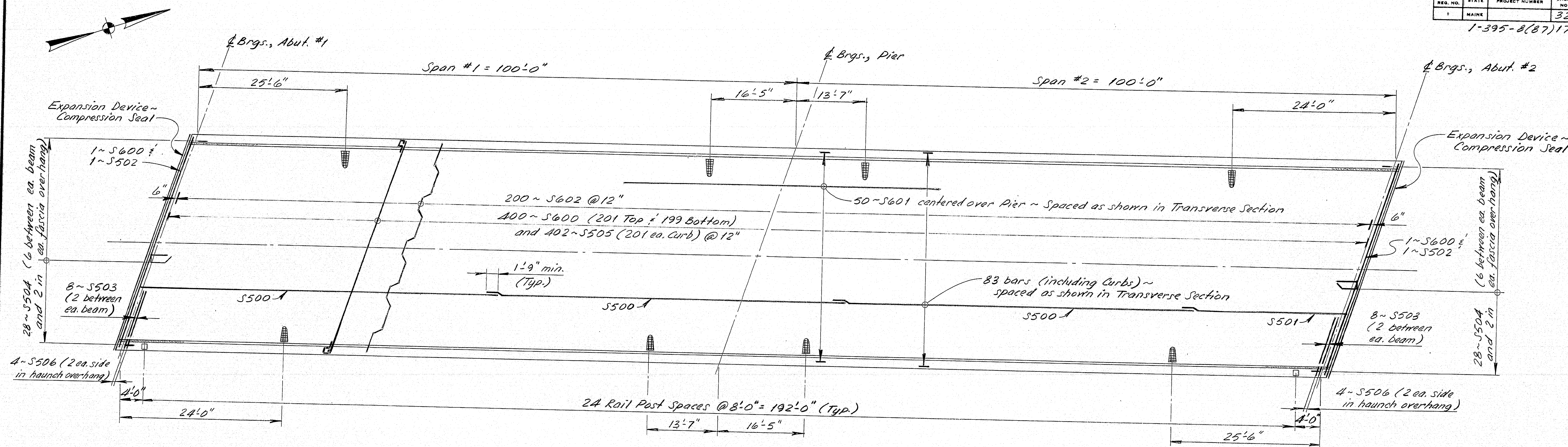
As Built 1984 RMZ

STATE OF MAINE DEPARTMENT OF TRANSPORTATION
GREEN POINT ROAD OVER INTERSTATE 395
BREWER PENOBSCOT COUNTY STRUCTURAL STEEL (Welded Beam Option)
SHEET 11 OF 15 AUGUSTA, MAINE

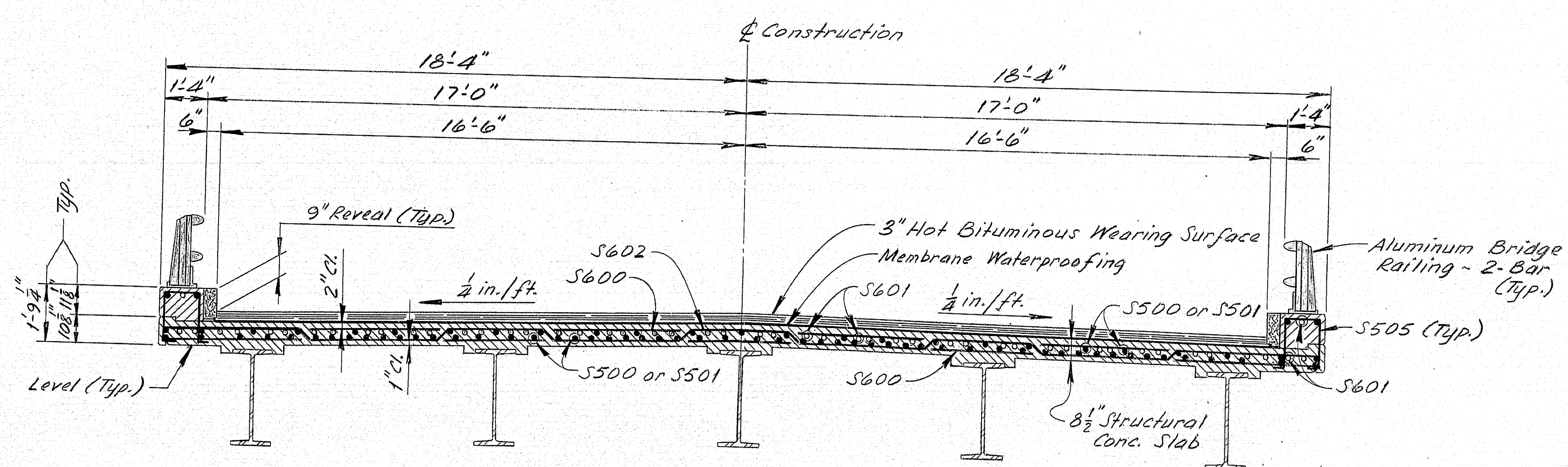
Structural Steel Alternate

183-156

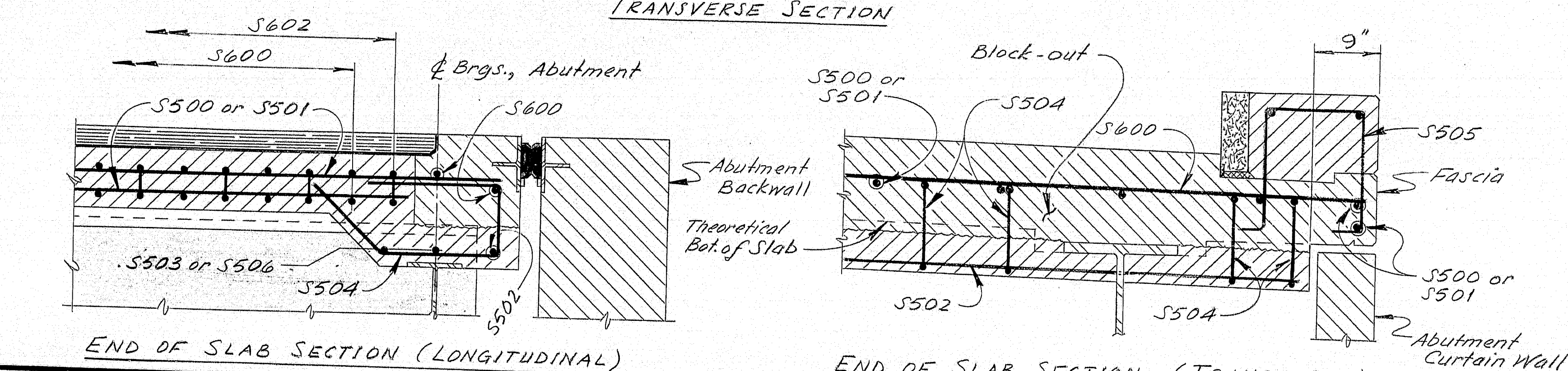




SUPERSTRUCTURE PLAN



TRANSVERSE SECTION



END OF SLAB SECTION (LONGITUDINAL)

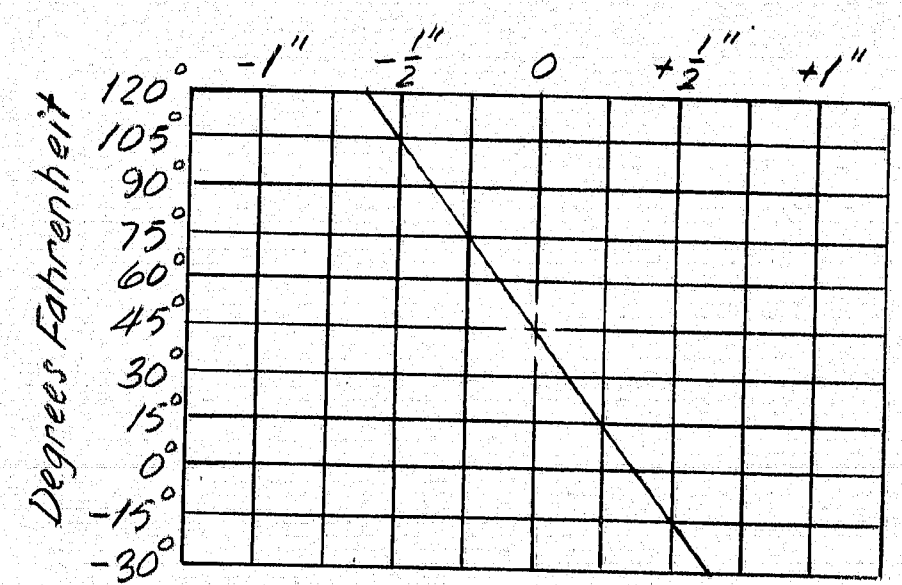
END OF SLAB SECTION (TRANSVERSE)

SUPERSTRUCTURE NOTES

- Form a 1" V-groove on the fascias at the horizontal joint between the curb and slab.
- Reinforcing steel shall have a minimum cover of 2" unless otherwise noted.
- Adjust reinforcing steel to fit around the drains in a manner approved by the Engineer. Do not cut transverse reinforcing bars.
- The Superstructure slab concrete shall be placed in one continuous operation and shall be kept plastic until the entire Superstructure slab has been placed.
- Mortar for bedding and for joints in the granite curb shall contain an approved non-shrink additive.
- Protective coating for concrete surfaces shall be applied to tops of concrete curbs and fascias down to the drip notch.

COMPRESSION SEAL NOTES

- The seals to be furnished shall have a minimum movement rating of 1.25".
- The seal shall be approved by the Engineer prior to fabrication of the joint armor.
- The joint opening will vary depending on the dimensions of the seal selected by the Contractor. The joint opening shall be set according to the opening shown on the approved shop detail drawings.
- It is anticipated that the slab and backwall concrete will be in place before the final adjustment to the joints is made and no allowance for movement due to dead load deflections is needed.
- The Compression Seal Adjustment Chart shows the adjustment necessary to set the joint opening shown on the shop detail drawings for temperatures other than 45°F. Adjustment is to be measured parallel to the centerline of construction.



COMPRESSION SEAL ADJUSTMENT CHART

SYMBOLS

- New Concrete (Plan or Elevation)
- New Concrete (Section)
- Hot Bituminous Prmt.
- Granite Curb

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
SUPERSTRUCTURE

183-157

SHEET 12 OF 15 AUGUSTA, MAINE  
Structural Steel Alternate

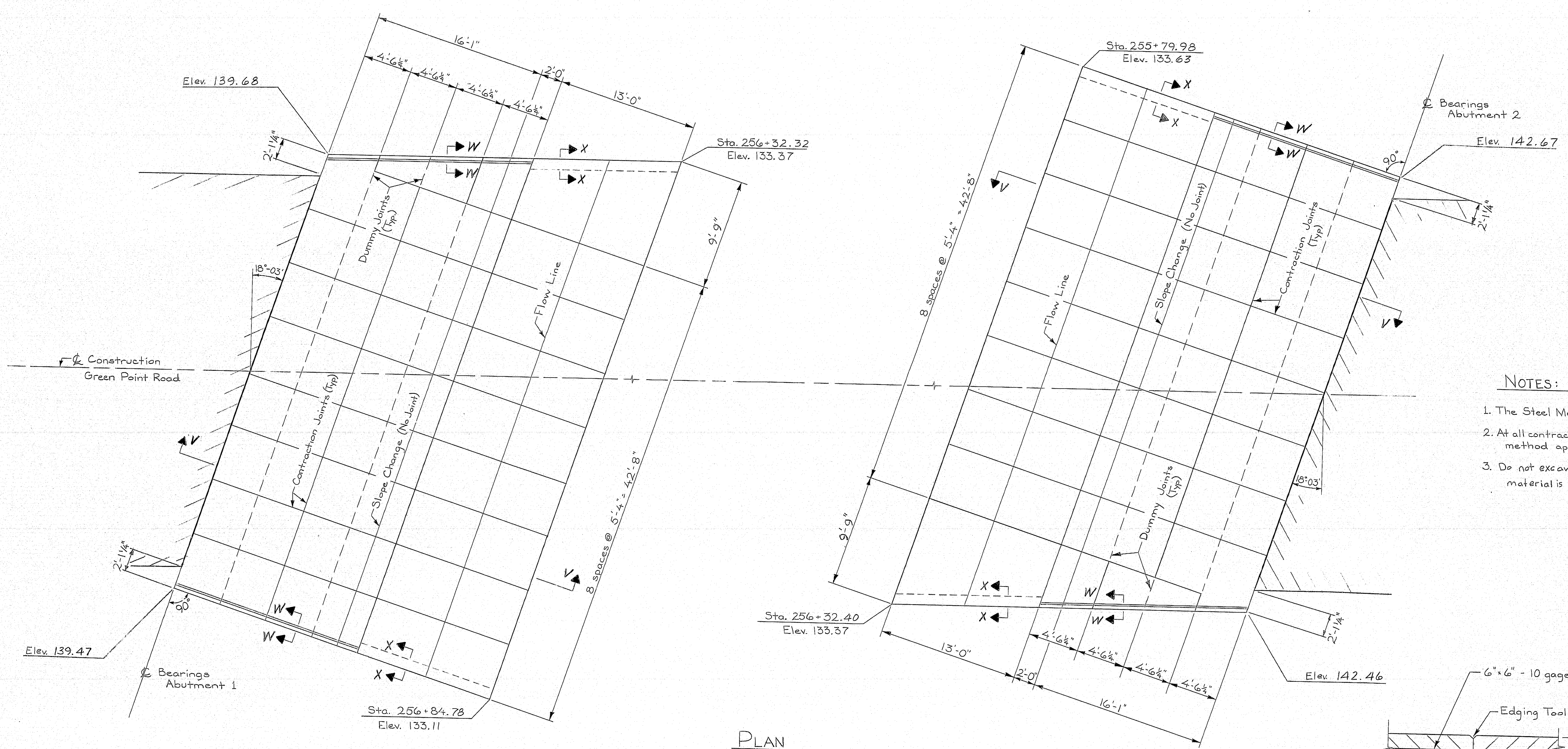
PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	12/8/83
CHECKED	
REVISIONS	
FIELD CHANGES	

BURNING 44-132 45710



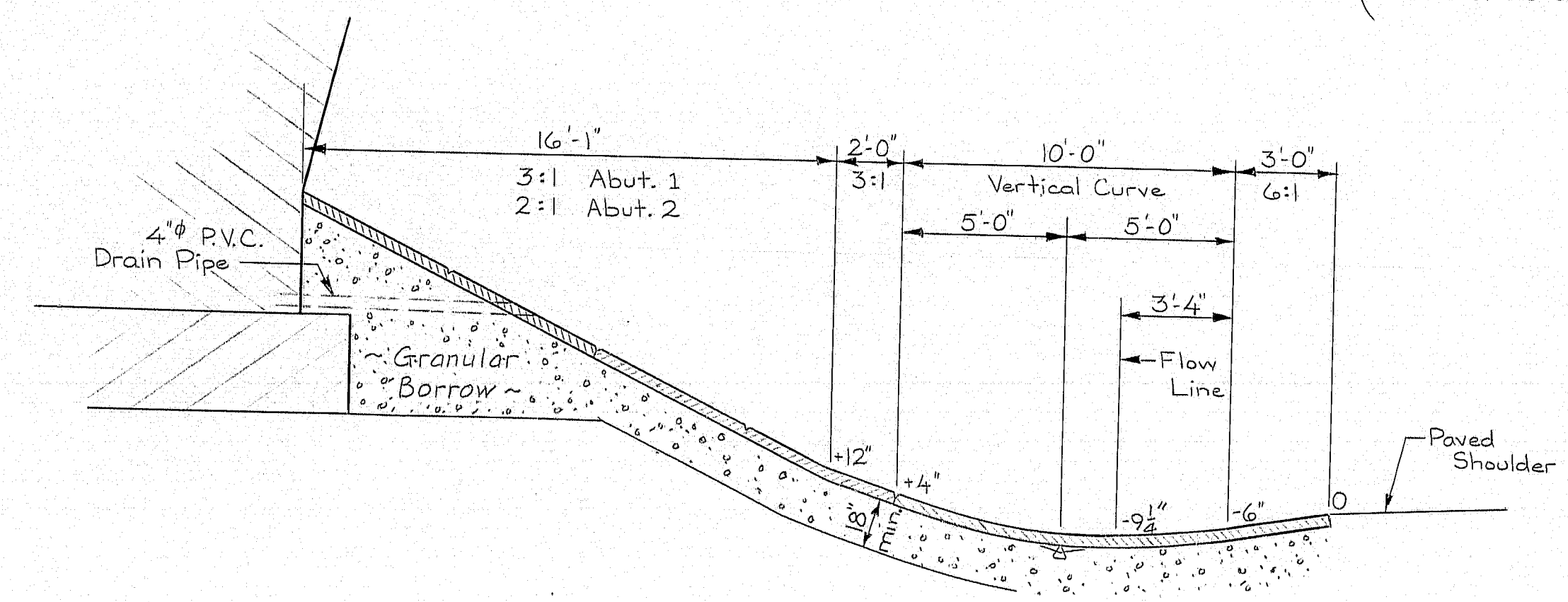
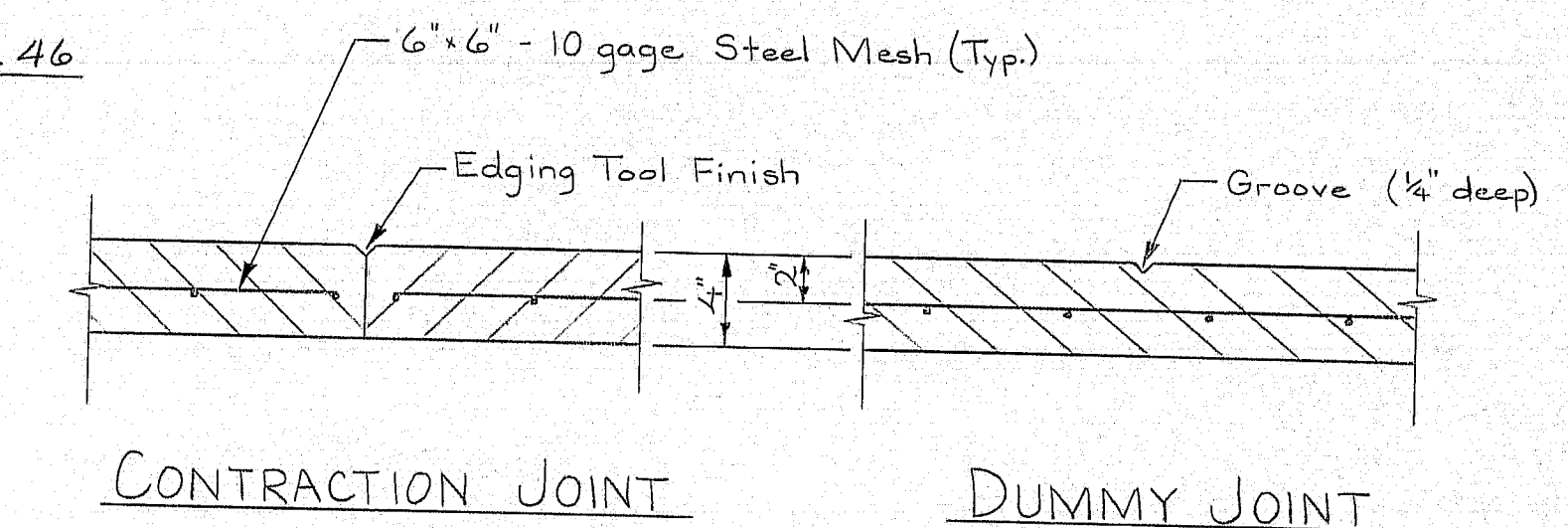
F.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE		33	84

1-395-8(87)176

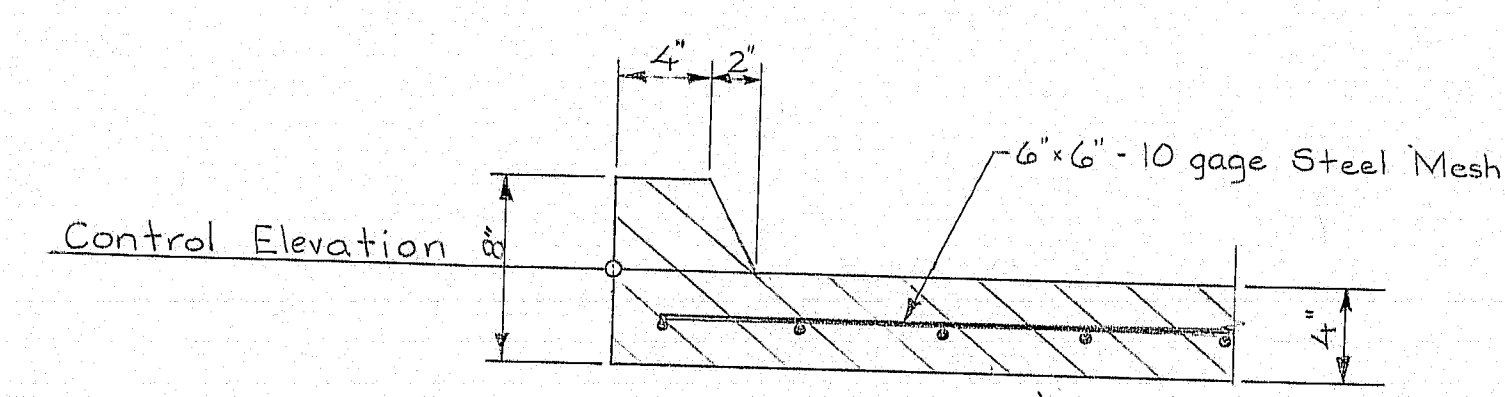


PLAN  
(Dimensions are Horizontal)

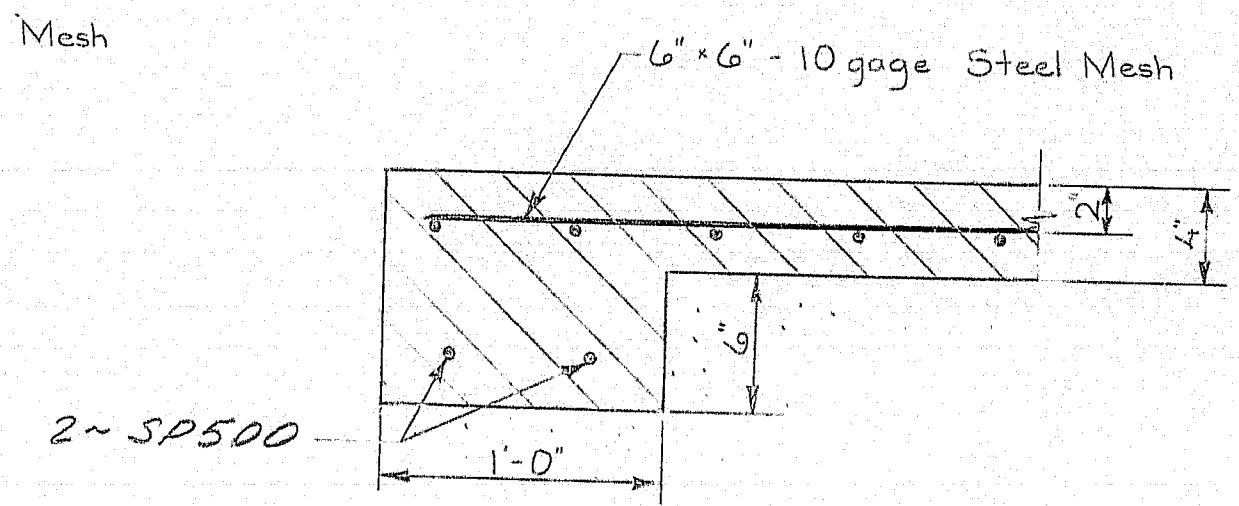
- NOTES:
1. The Steel Mesh shall not pass through any contraction joints.
  2. At all contraction joints, break band between sections by a method approved by the engineer.
  3. Do not excavate for granular borrow where the existing material is found suitable in the opinion of the engineer.



SECTION V-V



SECTION W-W



SECTION X-X

As Built 1984 EMB  
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD  
OVER  
INTERSTATE 395  
BREWER  
PENOBSCOT COUNTY  
CONCRETE SLOPE PROTECTION  
SHEET 13 OF 15 AUGUSTA, MAINE  
Structural Steel Alternate

183-158

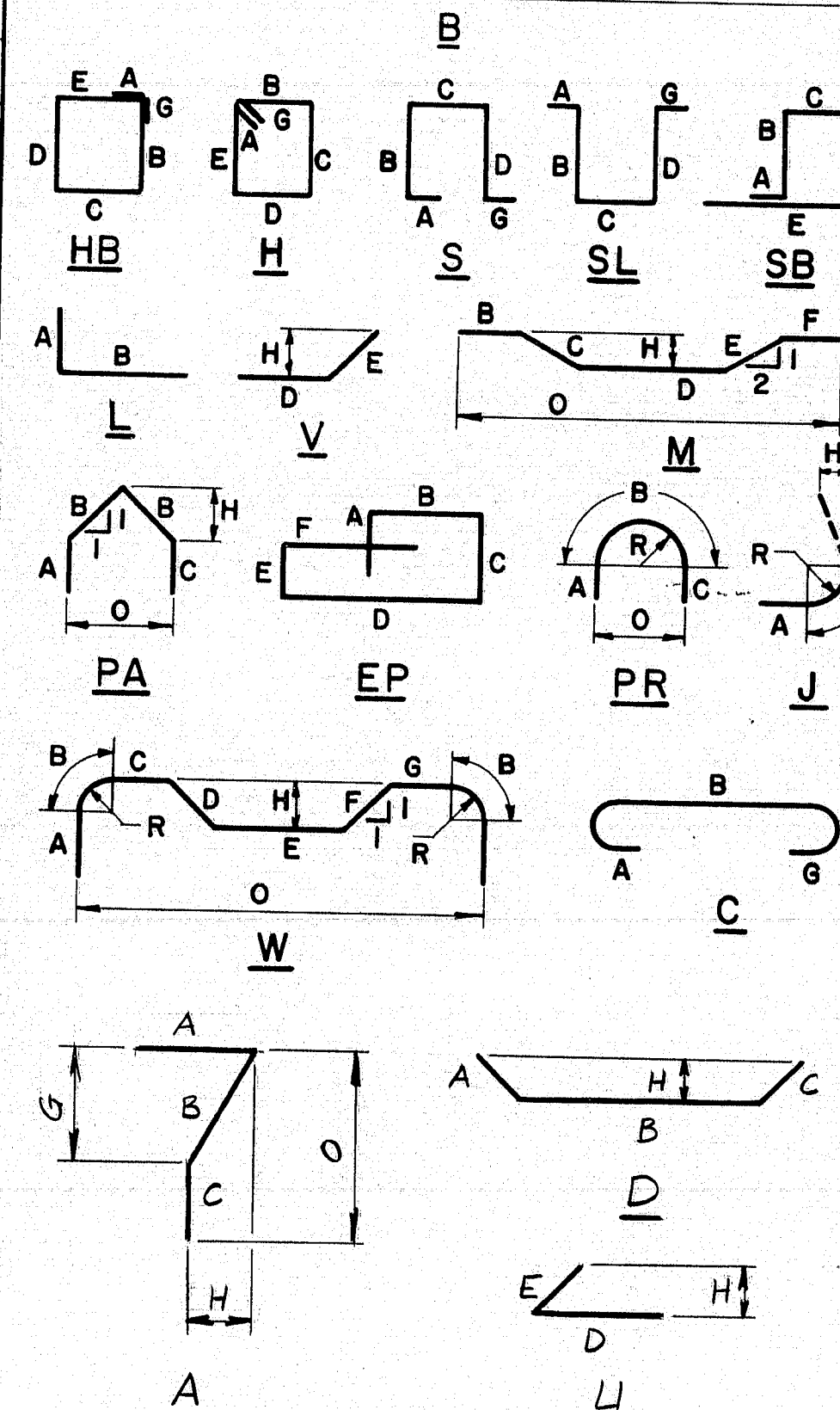
PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	5/82
CHECKED	12/83
REVISIONS	
FIELD CHANGES	

BRUNING 44-132-45710



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	
ABUTMENT No. 1				ABUTMENT No. 2				PIER				ABUTMENT No. 1															
A500	59	2'-9"	Dowels	B500	59	2'-9"	Dowels	P500	24	21'-0"	Horizontal	A400	20	6'-0"	S	0	1'-8"	2'-8"	1'-8"	—	—	0	—	—	—	—	Pedestals
A501	37	15'-0"	Vertical - Abutment	B501	37	15'-10"	Vertical - Abutment	P501	4	16'-0"	Vertical	A520	25	16'-8"	A	5'-1"	9'-3"	2'-4"	—	—	—	9'-0"	2'-3"	11'-4"	—	—	Vertical - Abutment
A502	12	23'-10"	Horiz. - Abutment	B502	13	23'-10"	Horiz. - Abutment	P502	4	14'-3"	↑	A521	24	12'-10"	S	0	5'-10"	1'-2"	5'-10"	—	—	0	—	—	—	—	Vertical - Abutment
A503	11	22'-2"	Horiz. - Abutment	B503	12	22'-2"	Horiz. - Abutment	P503	4	13'-4"	↑	A522	12	7'-11"	L	—	—	—	4'-0"	3'-11"	—	—	3'-9"	—	—	—	Corners
A504	5	23'-4"	Horiz. - Abutment	B504	5	23'-4"	Horiz. - Abutment	P504	4	10'-10"	↑	A523	11	11'-9"	D	1'-9"	8'-3"	1'-9"	—	—	—	—	1'-5"	—	—	—	Haunches
A505	24	3'-3"	Vertical - Backwall	B505	24	3'-2"	Vertical - Backwall	P505	4	9'-2"	↑	A524	12	6'-6"	V	—	—	—	4'-0"	2'-6"	—	—	2'-4 1/2"	—	—	—	Corners
A506	16	13'-10"	Horiz. - Abutment	B506	17	13'-10"	Horiz. - Abutment	P506	4	7'-5"	↑	A525	11	11'-9"	D	1'-9"	8'-3"	1'-9"	—	—	—	—	1'-0"	—	—	—	Haunches
A507	7	14'-10"	Horiz. - Abutment	B507	8	14'-10"	Horiz. - Abutment	P507	4	5'-8"	↓	A526	40	8'-7"	S	0	3'-9"	1'-1"	3'-9"	—	—	0	—	—	—	Top of Wing	
A508	5	14'-5"	Horiz. - Abutment	B508	5	14'-5"	Horiz. - Abutment	P508	4	4'-0"	Vertical																
A509	64	13'-7"	Vertical - Wing	B509	63	14'-3"	Vertical - Wing	P509	2	2'-6"	Horizontal																
A510	76	5'-0"	Vertical - Wing	B510	73	5'-6"	Vertical - Wing																				
A511	32	24'-8"	Horizontal - Wing	B511	36	24'-8"	Horizontal - Wing	P600	48	5'-6"	Footing																
A512	16	28'-6"	Horizontal - Wing	B512	16	28'-4"	Horizontal - Wing	P601	14	23'-0"	Footing																
A513	8	28'-4"	Horizontal - Wing	B513	8	28'-4"	Horizontal - Wing																				
				B514	2	5'-0"	Vertical - Wing	P700	42	4'-9"	Dowels	B400	20	6'-0"	S	0	1'-8"	2'-8"	1'-8"	—	—	0	—	—	—	—	Pedestals
A515	16	6'-5"	Vertical - Wing	B515	16	5'-9"	Vertical - Wing	P701	42	17'-4"	Vertical																
A516	12	6'-7"	Horizontal - Wing	B516	12	6'-7"	Horizontal - Wing					B520	25	17'-6"	A	5'-1"	9'-3"	3'-2"	—	—	—	9'-0"	2'-3"	12'-6"	—	—	Vertical - Abutment
A517	8	6'-10"	Curtain Wall	B517	8	6'-10"	Curtain Wall					B521	24	12'-10"	S	0	5'-10"	1'-2"	5'-10"	—	—	0	—	—	—	—	Vertical - Abutment
												B522	13	7'-4"	L	—	—	—	4'-8"	2'-8"	—	—	2'-4 1/2"	—	—	—	Corners
A600	6	25'-7"	Footing	B600	6	25'-7"	Footing					B523	12	11'-9"	D	1'-9"	8'-3"	1'-9"									

FHWA REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(87)	34	84



*All dimensions are out to out of reinf. bar*

*Bending details and hooks shall conform to the recommendations of ACI Standard 315-65.*

Reinforcing Bar: ASTM A615 Grade 60

### GENERAL NOTES

1. First digit(s) following the letter of the Mark indicates size of reinf. bar.  
Mark (A 502) bar size - #5  
Mark (P 1001) bar size - #10  
Mark (S 603) bar size - #6
2. Letter of Marks A, P & S locates bars of Abutments, Piers, and Superstructure parts respectively.

As Built 1984 emf

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
REINFORCING STEEL SCHEDULE

SHEET 14 OF 15      AUGUSTA, MAINE

**183-159**

PLANS	DESIGN - DETAIL	BY	DATE
	CHECKED	J.E. BROWN	Dec. '83
	REVISIONS	Bump	12/83
	FIELD CHANGES		



[illegible]

GENERAL NOTES

- As B01/H 1904 Rem.

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
INTERSTATE 395

BREWER  
PENOBSCOT COUNTY  
REINFORCING STEEL SCHEDULE

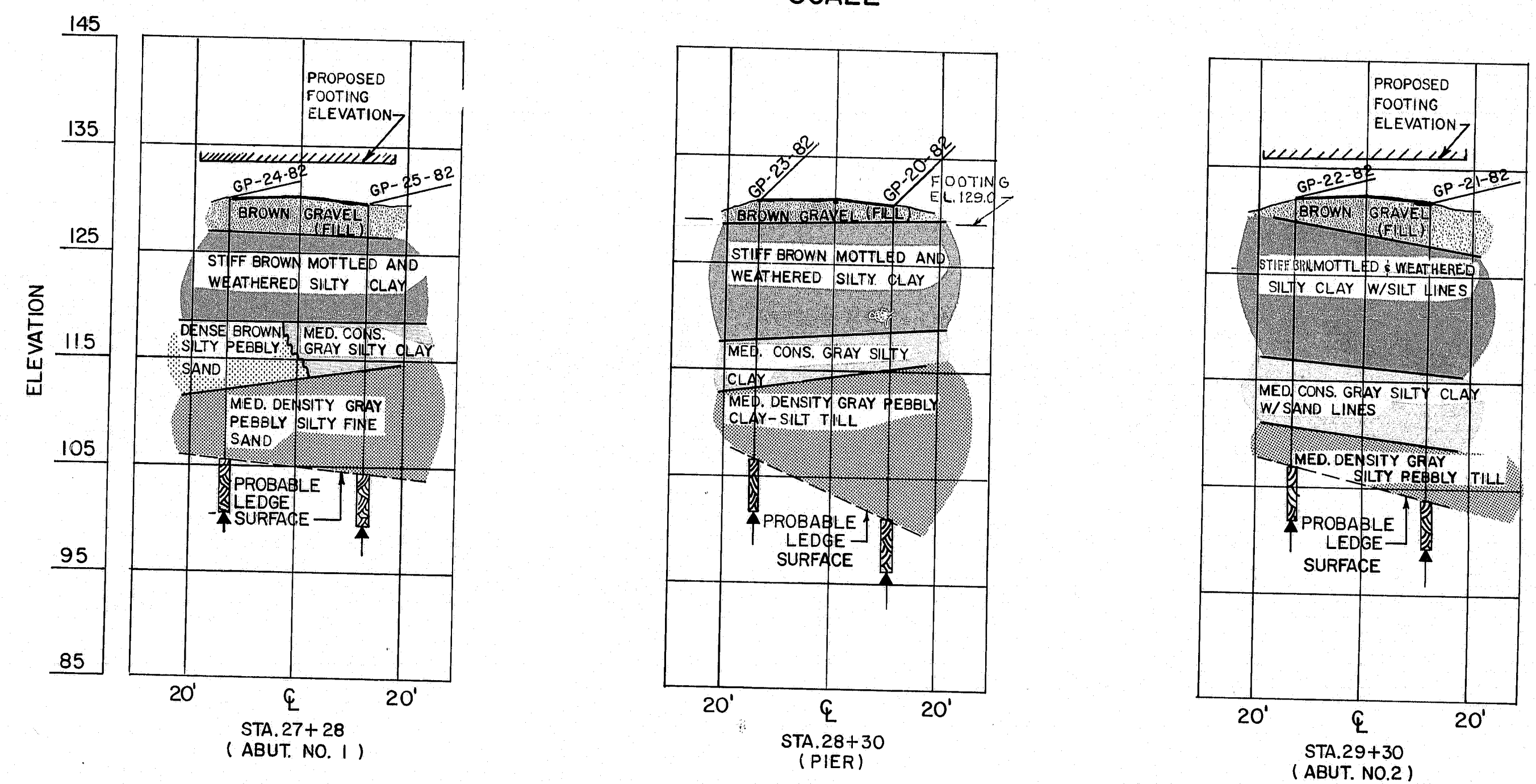
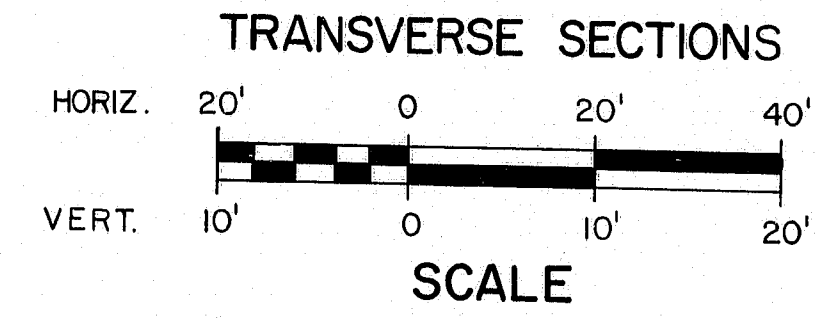
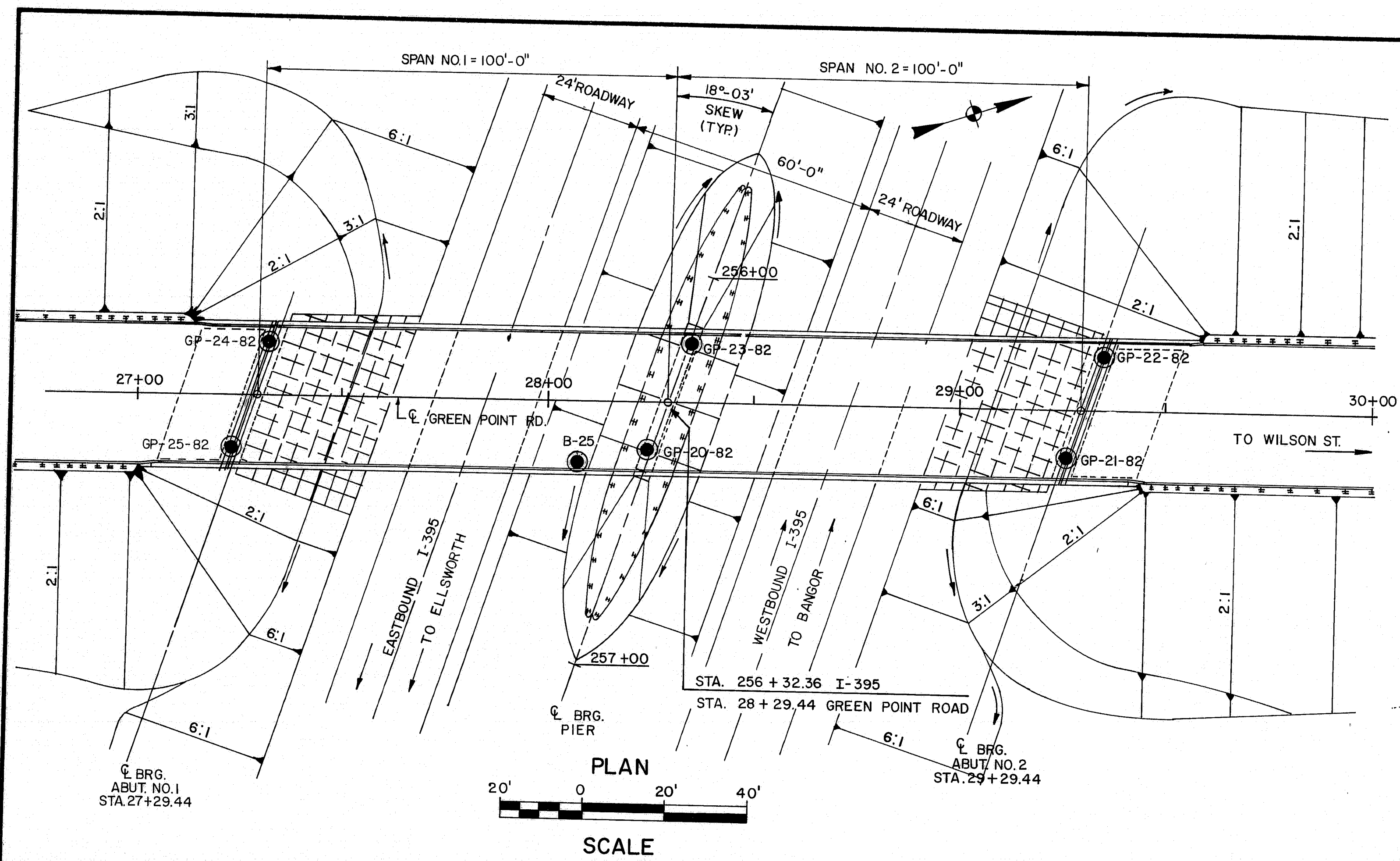
SHEET 15 OF 15 AUGUSTA, MAINE

**183-160**

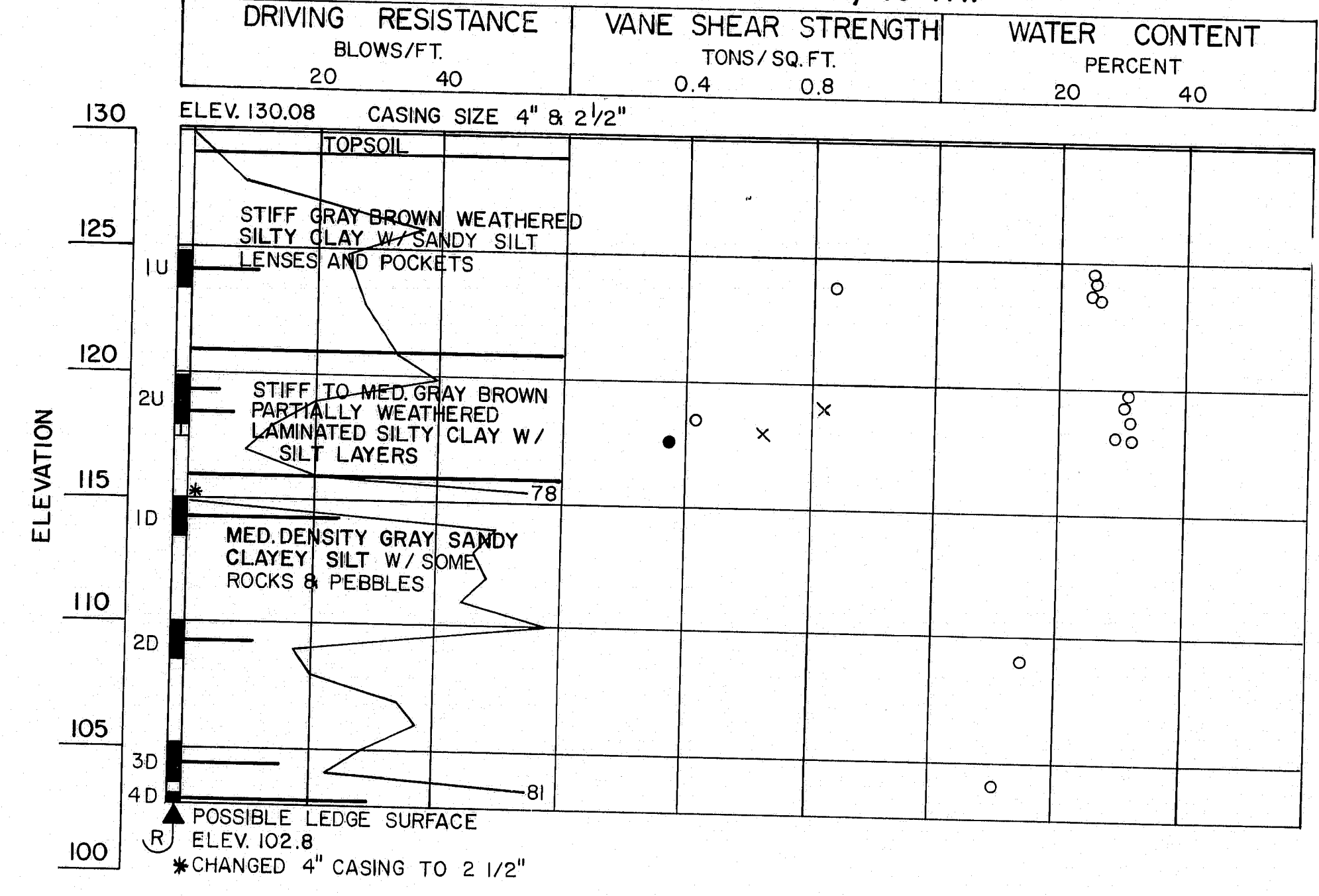
PLANS	DESIGN - DETAIL	BY	DATE
	CHECKED	J.E. Bazzard, D. Dammann	Dec. '83
	REVISIONS	Burt	12/18/83
	FIELD CHANGES		



F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1-395-8(79)	66	84



BORING GP-59-78(B-25) STA. 28+08, 15' RT.



BORING NOTES

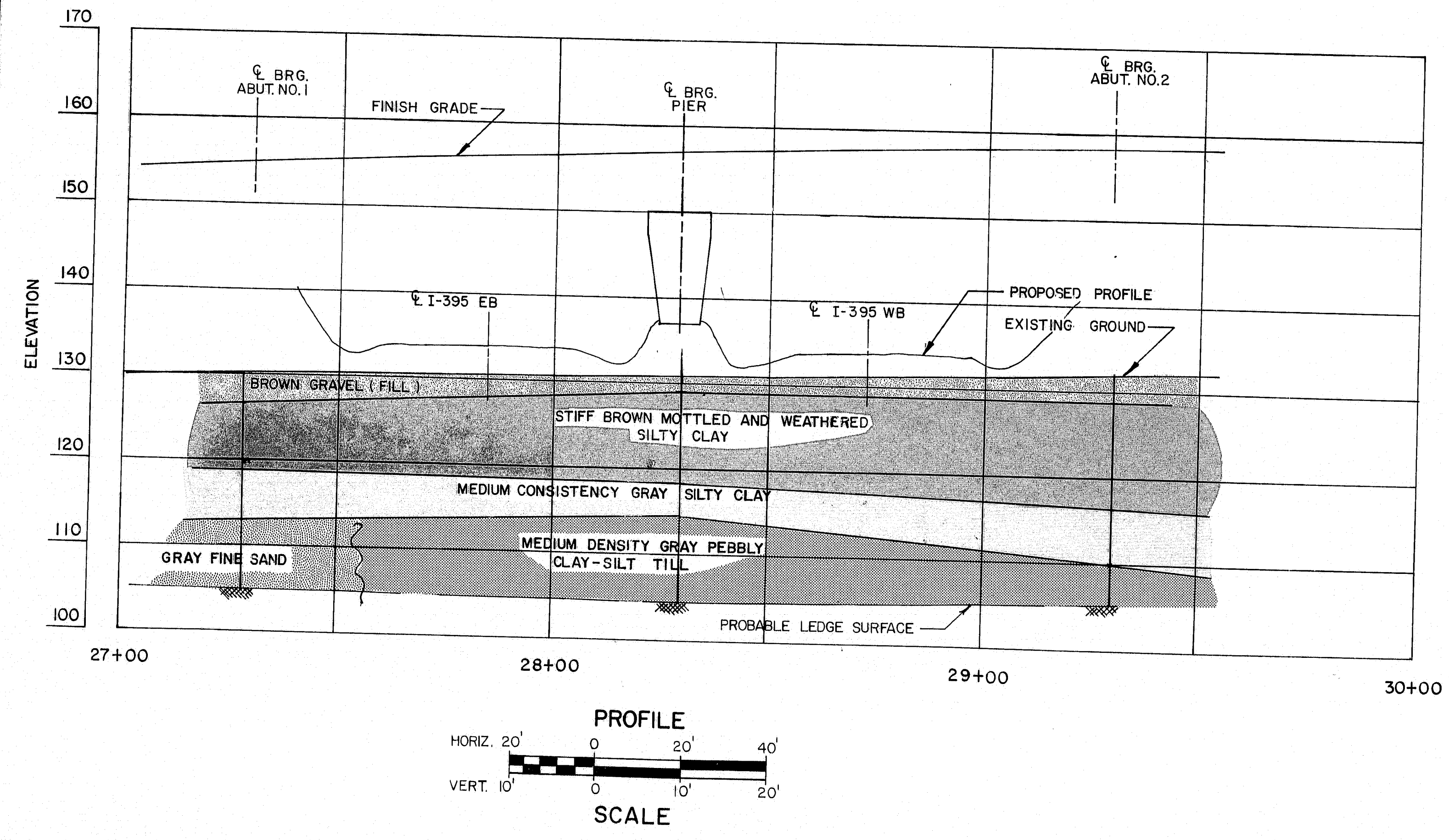
All samples and vane are made ahead of casing  
 Number of blows required to drive extra heavy casing one foot with 400 ft. lbs. of energy per blow  
 Location of sample or sample attempt  
 Number and type of dry sample  
 S & H Sampler #1290's  
 3/2" O.D. 16 ga. seamless tubing  
 Unsuccessful sample attempt and type of sampler  
 Number of blows required to drive spoon or tubing one foot with 350 ft. lb. of energy per blow  
 Field vane test  
 Refusal of drill rods or casing (may not be ledge)

SHEAR NOTES

Field vane shear strengths  
 Laboratory vane shear strengths  
 Shear strengths in excess of capacity of equipment  
 One half unconfined compressive strengths

WATER CONTENT NOTES

Natural water contents, given as percent of dry weight



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

BRUNING 44-132-45710

STATE OF MAINE  
 DEPARTMENT OF TRANSPORTATION

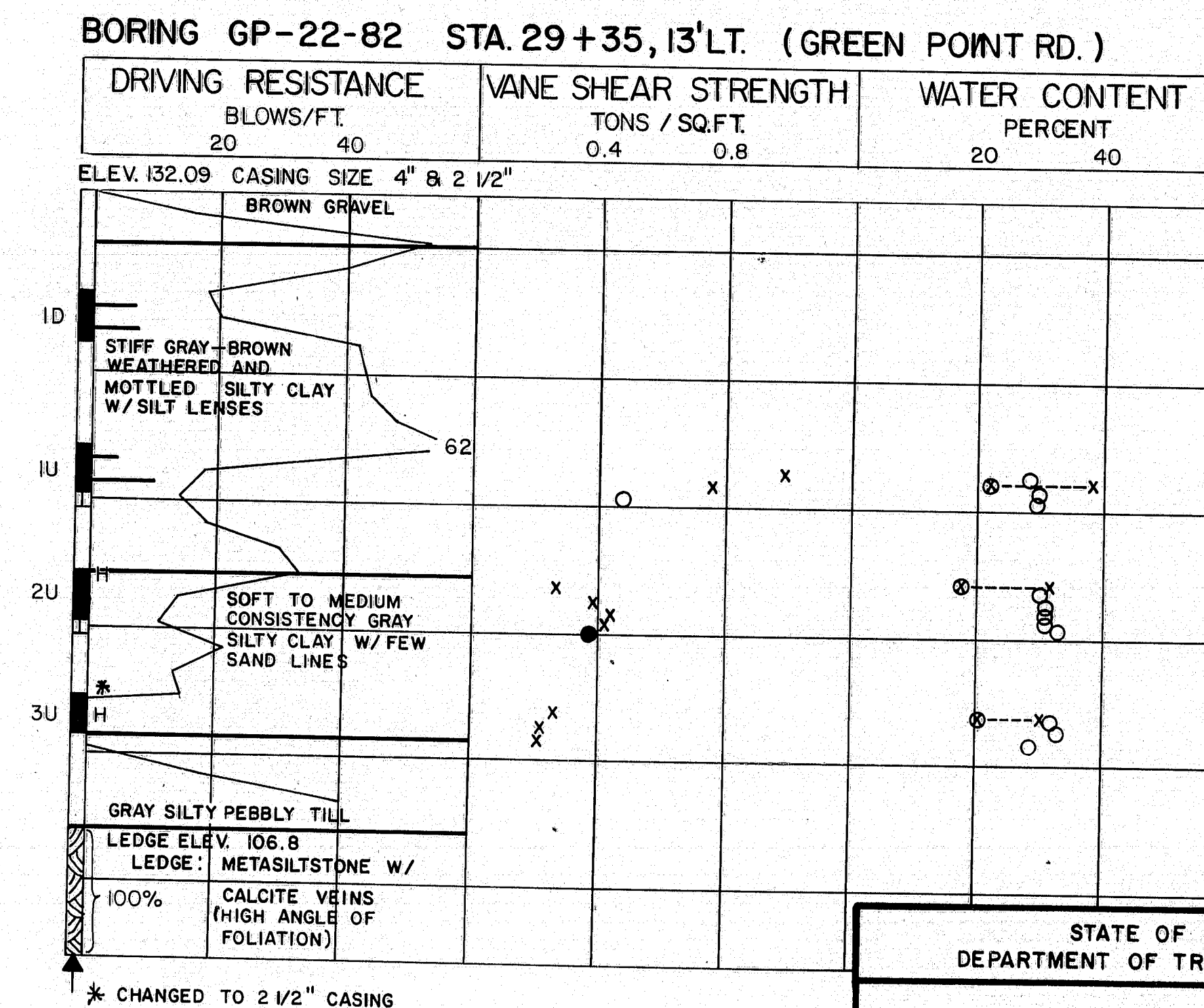
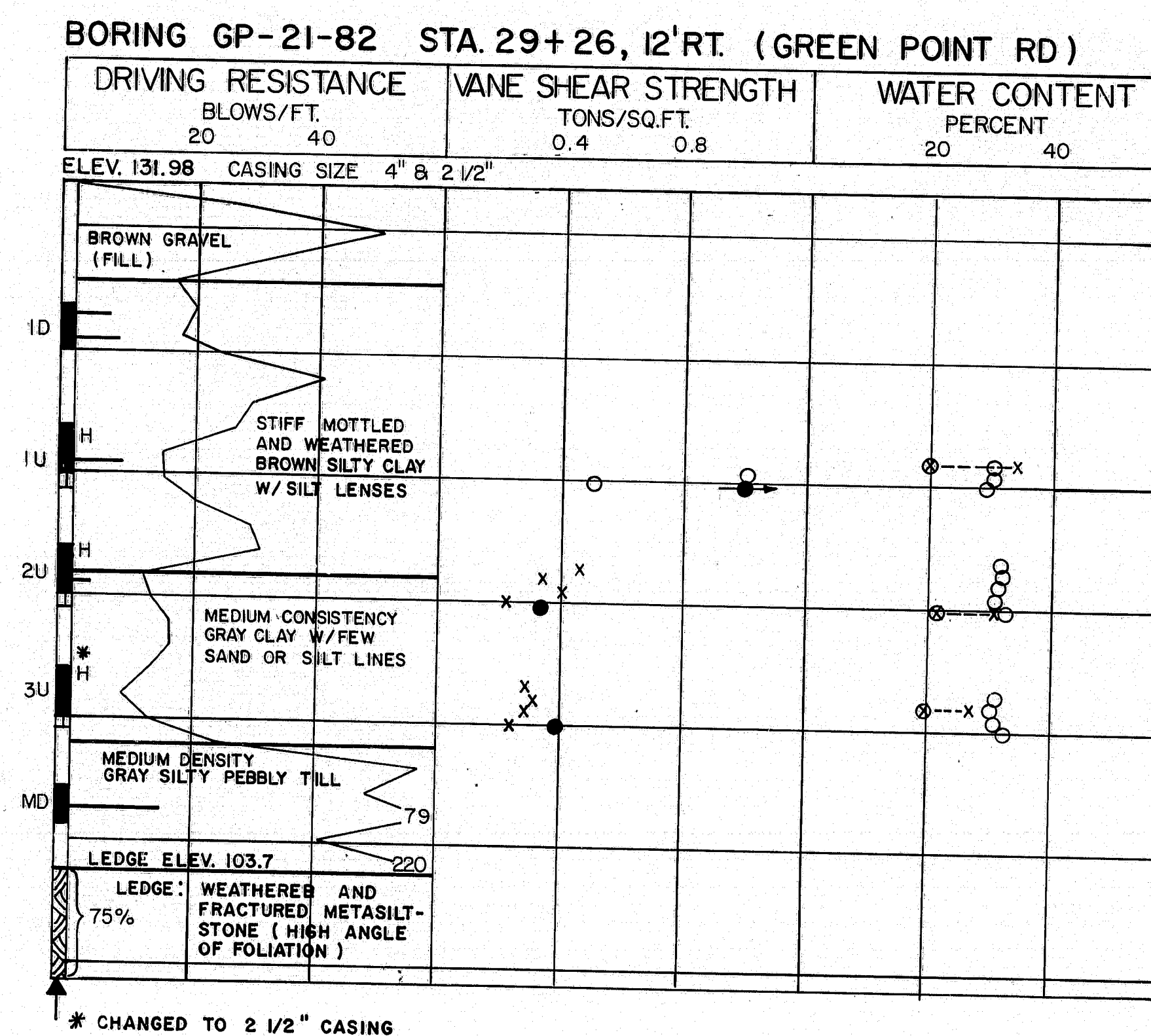
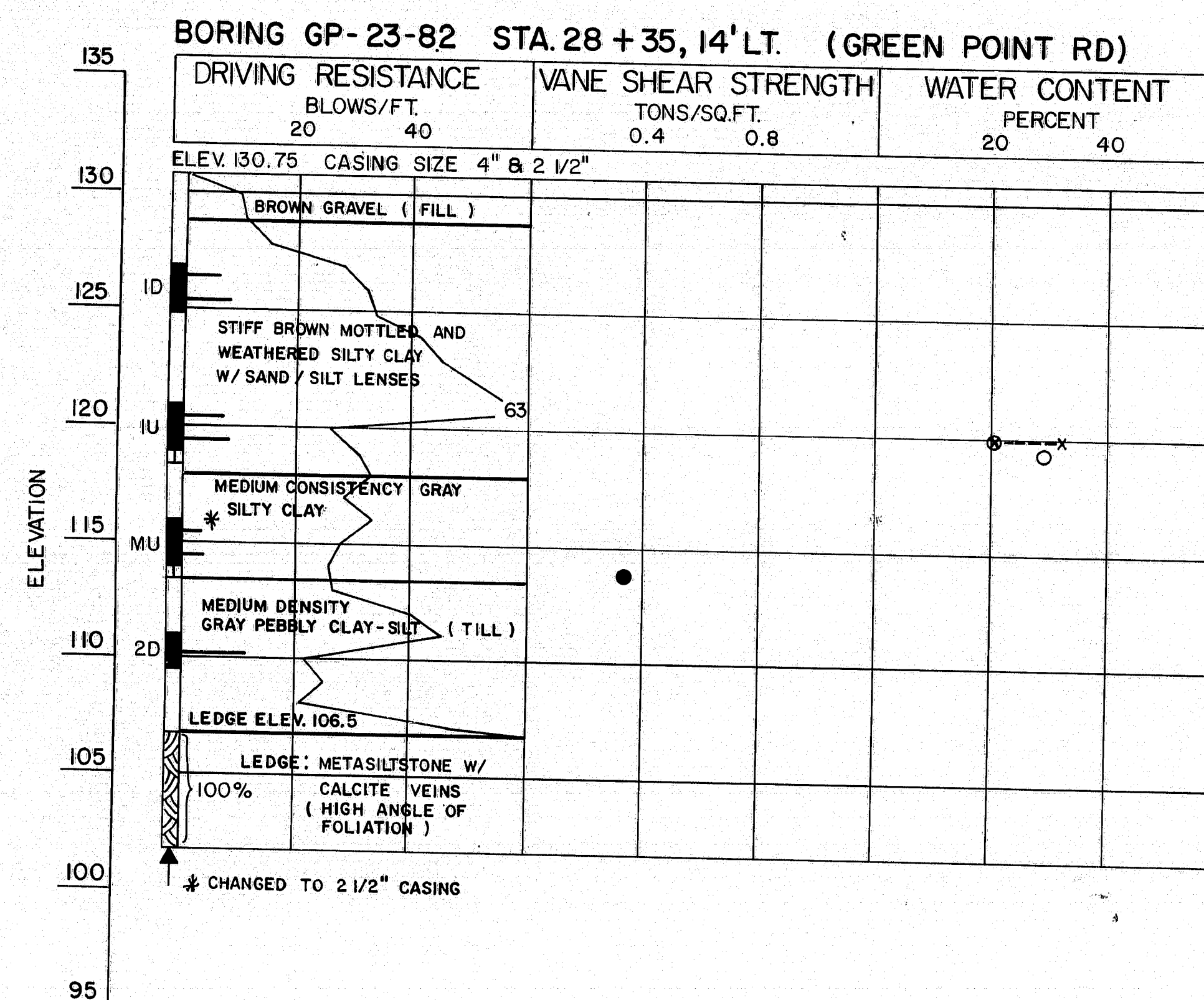
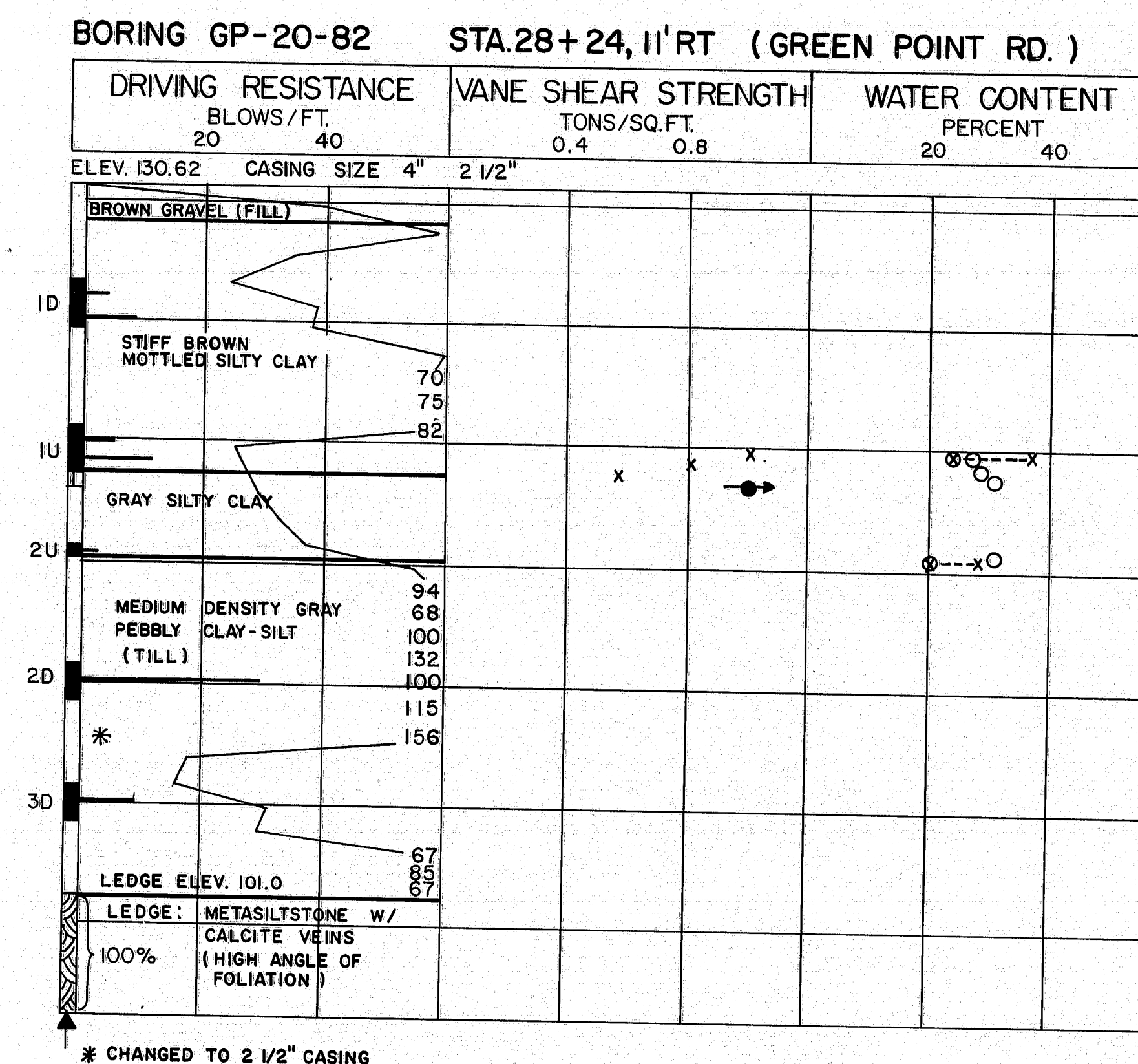
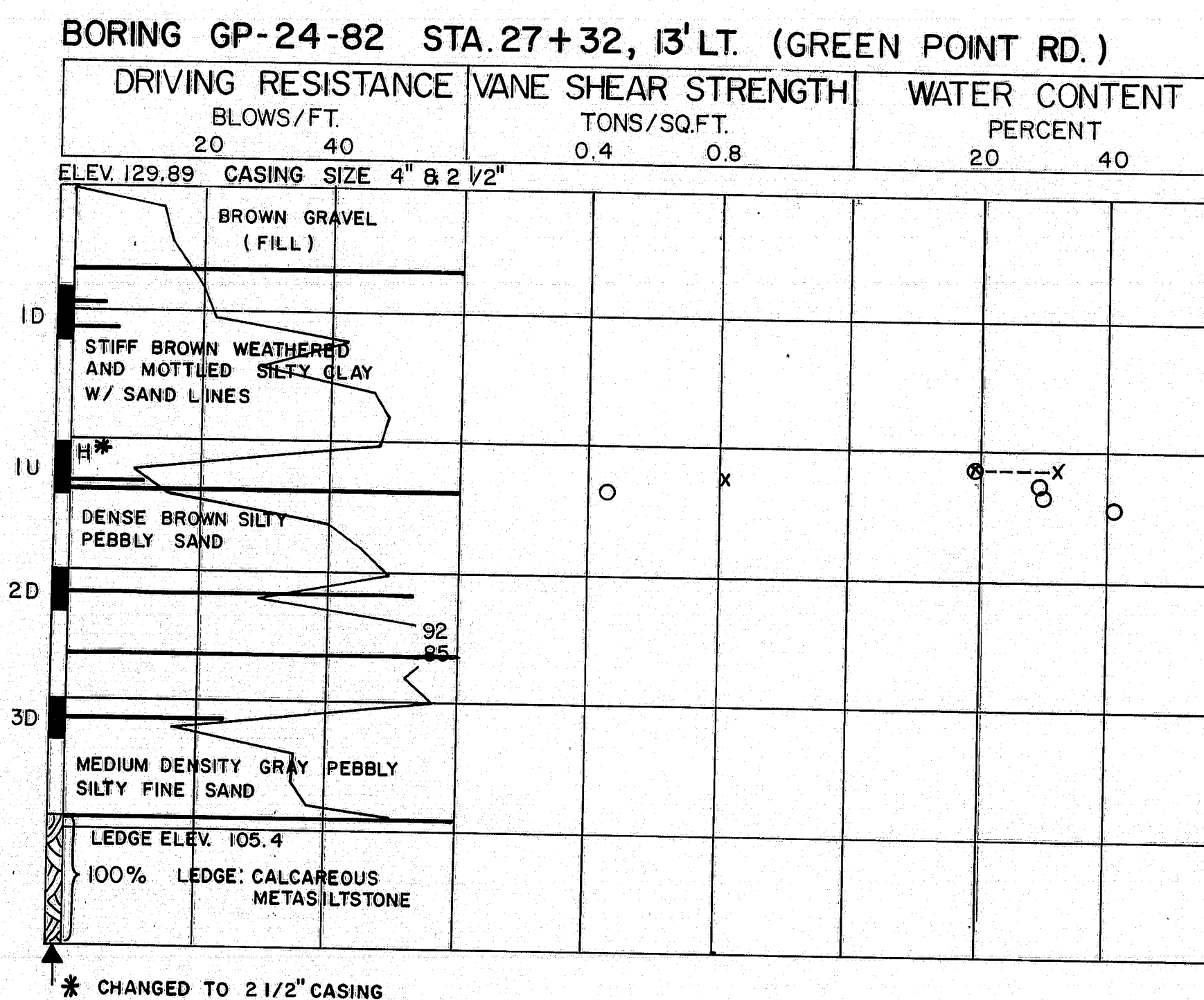
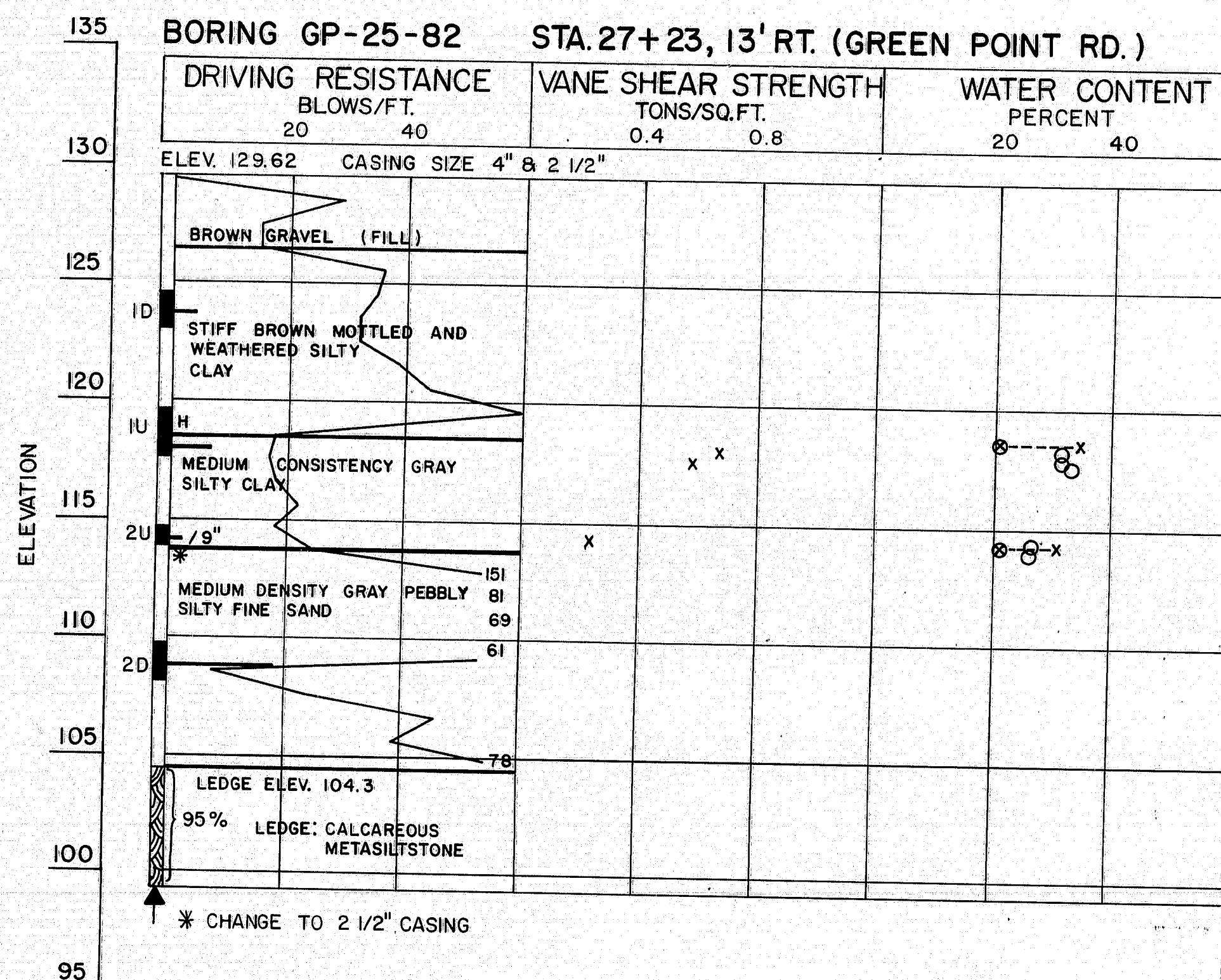
GREEN POINT ROAD  
 OVER  
 I-395  
 IN THE TOWN OF  
 BREWER  
 PENOBSCOT COUNTY  
 FOUNDATION SURVEY

183-161

SHEET OF AUGUSTA, MAINE



F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8 (79)	67	84



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

BORING 44-132-45710

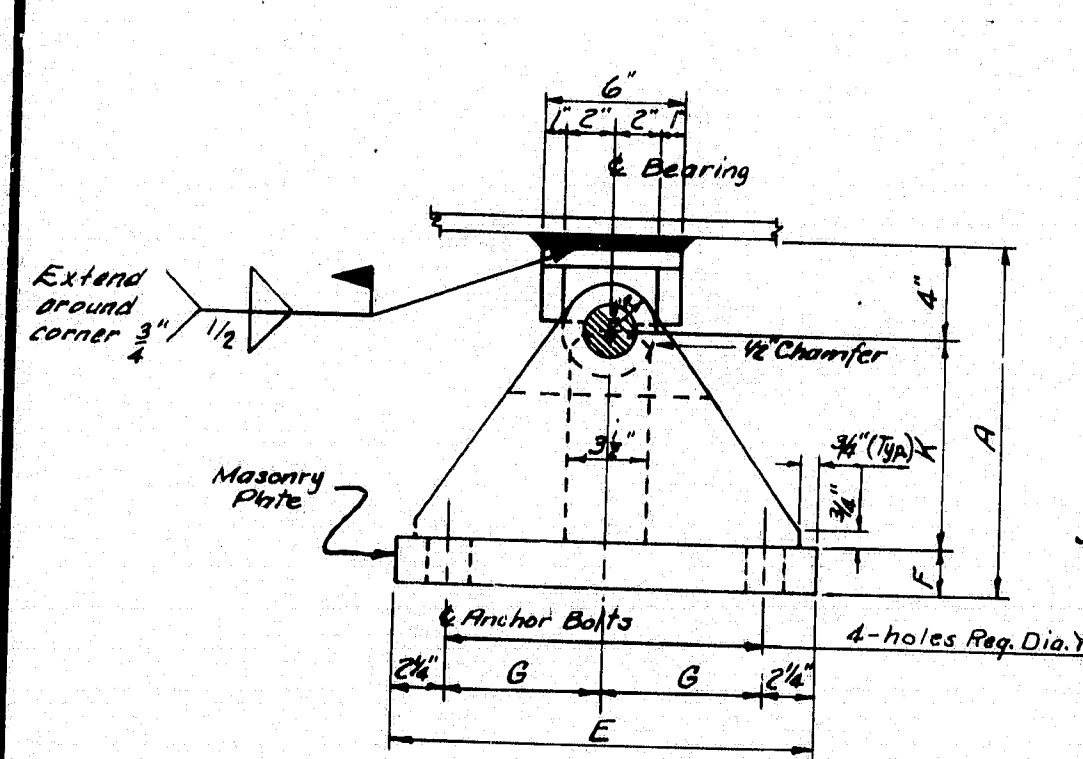
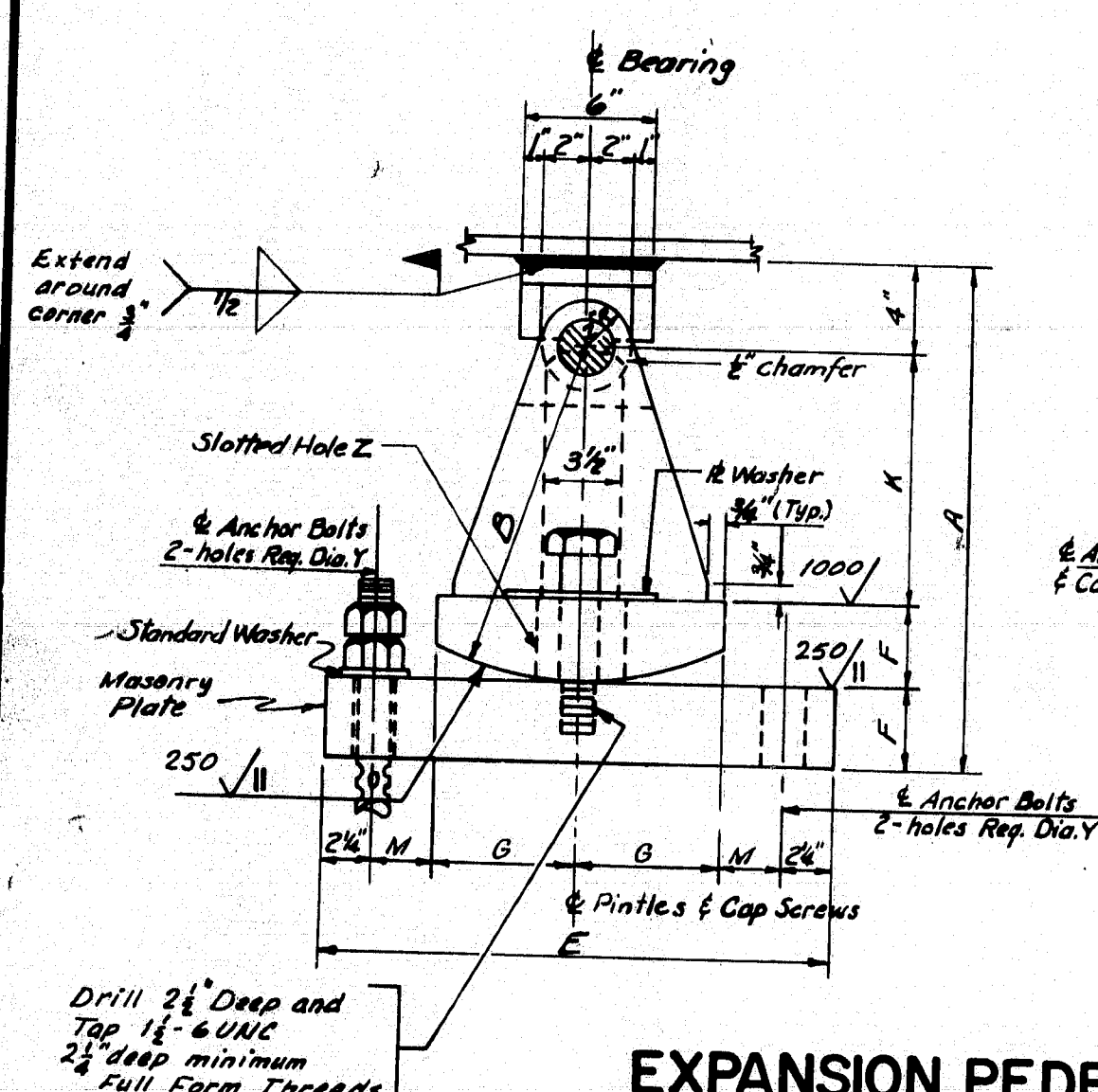
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD  
OVER  
1-395  
IN THE TOWN OF  
BREWER  
PENOBSCOT COUNTY

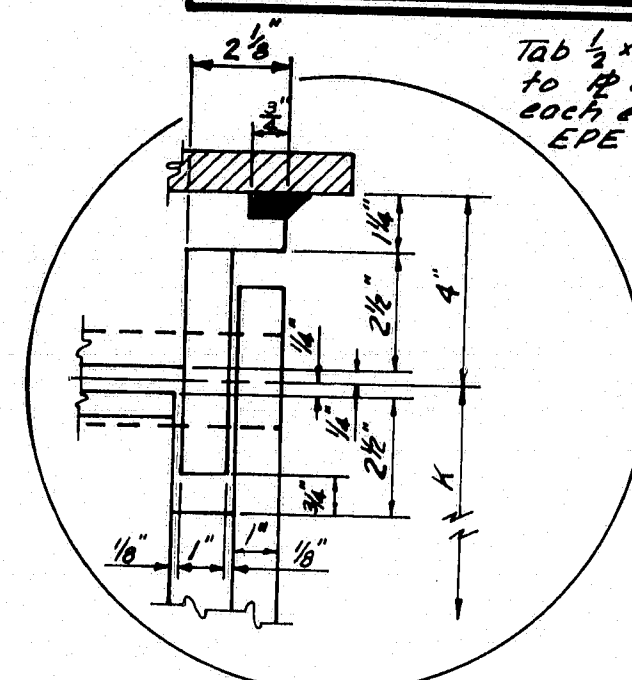
BORING DETAILS  
SHEET OF AUGUSTA, MAINE

183-162

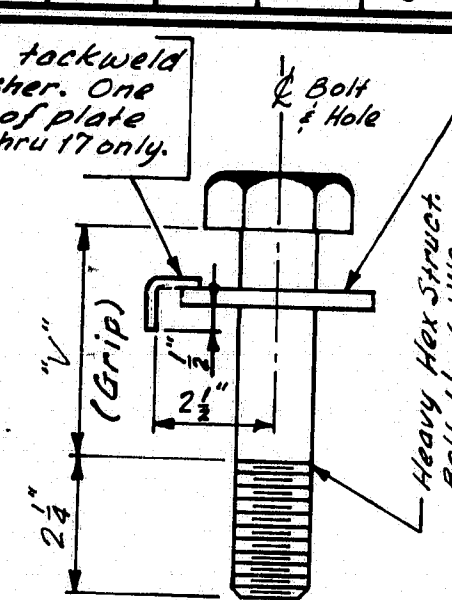


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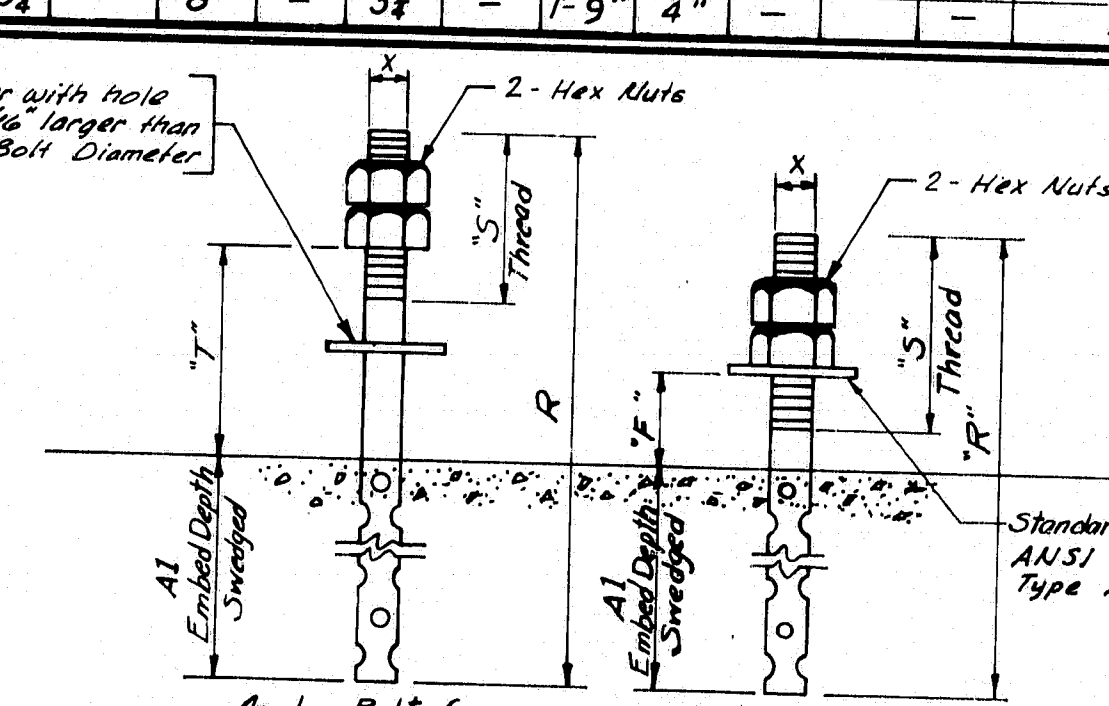
MARK		LOAD	A	B	C	D	E	F	G	H	J	K	M	P	Q	R	S	T	V	X-Anchor Bolt Diameter	Y-Masonry Plate Hole Size	Number Anchor Bolts Required	Z-Slotted Hole for Anchor Bolts or Cap Screws	Washer Size for Anchor Bolts or Cap Screws	41 Embedment Depth	MARK
EPD-1	100 <sup>K</sup>	1'-2 3/4"	9"	8"	1'-6"	8"	1 1/2"	3 1/2"	4"	2 1/2"	7"	1 1/2"	-	3"	1'-4 1/2"	3"	4 1/2"	-	-	1"	1 1/8"	2	3" x 1 1/8"	3" x 5" x 1/2"	10"	EPD-1
EPD-2	100 <sup>K</sup>	1'-2 3/4"	9"	8"	1'-6"	9"	1 1/2"	4"	4"	2 1/2"	7"	1 1/2"	-	3"	1'-4 1/2"	3"	4 1/2"	-	-	1"	1 1/8"	2	3" x 1 1/8"	3" x 5" x 1/2"	10"	EPD-2
EPD-3	100 <sup>K</sup>	1'-2 3/4"	9"	8"	1'-6"	10"	1 1/2"	4 1/2"	4"	2 1/2"	7"	1 1/2"	-	3"	1'-4 1/2"	3"	4 1/2"	-	-	1"	1 1/8"	2	3" x 1 1/8"	3" x 5" x 1/2"	10"	EPD-3
EPD-4	200 <sup>K</sup>	1'-3 1/2"	10"	8"	1'-6"	11"	1 1/2"	5"	4"	2 1/2"	10"	1 1/2"	-	3"	1'-5"	3"	4 1/2"	-	-	1"	1 1/8"	2	3" x 1 1/8"	3" x 5" x 1/2"	10"	EPD-4
EPD-5	100 <sup>K</sup>	1'-9 1/2"	1'-3"	10"	1'-8"	1'-0"	2 1/2"	5 1/2"	4"	3 1/2"	1'-0 1/2"	1 1/2"	-	4"	2'-0 1/2"	4"	6 1/2"	-	-	1 1/2"	1 1/8"	2	4" x 1 1/8"	4" x 7" x 1/2"	1'-3"	EPD-5
EPD-6	200 <sup>K</sup>	1'-9 1/2"	1'-3"	10"	1'-8"	1'-1"	2 1/2"	6"	4"	3 1/2"	1'-0 1/2"	1 1/2"	-	4"	2'-1"	4"	6 1/2"	-	-	1 1/2"	1 1/8"	2	4" x 1 1/8"	4" x 7" x 1/2"	1'-3"	EPD-6
EPD-7	200 <sup>K</sup>	1'-9 1/2"	1'-3"	10"	1'-8"	1'-2"	2 1/2"	6 1/2"	4"	3 1/2"	1'-0 1/2"	1 1/2"	-	4"	2'-1"	4"	6 1/2"	-	-	1 1/2"	1 1/8"	2	4" x 1 1/8"	4" x 7" x 1/2"	1'-3"	EPD-7
EPD-8	200 <sup>K</sup>	1'-9 1/2"	1'-3"	10"	1'-8"	1'-3"	2 1/2"	7"	4"	3 1/2"	1'-0 1/2"	1 1/2"	-	4"	2'-1"	4"	6 1/2"	-	-	1 1/2"	1 1/8"	2	4" x 1 1/8"	4" x 7" x 1/2"	1'-3"	EPD-8
EPD-9	300 <sup>K</sup>	1'-10"	1'-3"	1'-2"	2'-0"	1'-4"	3"	7 1/2"	5"	4 1/2"	1 1/2"	1 1/2"	-	6"	2'-2 1/2"	4"	8"	-	-	1 1/2"	1 1/8"	2	5" x 1 1/8"	4" x 8" x 1/2"	1'-3"	EPD-9
EPD-10	400 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-6"	2'-4"	1'-6"	3 1/2"	8 1/2"	6"	5 1/2"	1 1/2"	1 1/2"	8 1/2"	3 1/2"	2'-3"	4"	8 1/2"	-	-	1 1/2"	1 1/8"	2	5" x 1 1/8"	4" x 8" x 1/2"	1'-3"	EPD-10
EPE-1	200 <sup>K</sup>	1'-10"	1'-3"	10"	1'-7"	1'-6"	3"	4"	4"	3 1/2"	1'-0"	2 1/2"	-	4"	1'-10"	4 1/2"	-	-	3 1/2"	1 1/8"	4	3" x 1 1/8"	3 1/2" x 4" x 1/2"	1'-3"	EPE-1	
EPE-2	200 <sup>K</sup>	1'-10"	1'-3"	11"	1'-8"	1'-9"	3"	5 1/2"	4 1/2"	3 1/2"	1'-0"	2 1/2"	-	4 1/2"	1'-10"	4 1/2"	-	-	4"	1 1/8"	4	3 1/2" x 1 1/8"	3 1/2" x 5" x 1/2"	1'-3"	EPE-2	
EPE-3	200 <sup>K</sup>	1'-10"	1'-3"	11"	1'-8"	1'-10"	3"	6"	4 1/2"	3 1/2"	1'-0"	2 1/2"	-	4 1/2"	1'-10"	4 1/2"	-	-	4 1/2"	1 1/8"	4	3 1/2" x 1 1/8"	3 1/2" x 5 1/2" x 1/2"	1'-3"	EPE-3	
EPE-4	200 <sup>K</sup>	1'-10"	1'-3"	11"	1'-8"	1'-11"	3"	6 1/2"	4 1/2"	3 1/2"	1'-0"	2 1/2"	-	4 1/2"	1'-10"	4 1/2"	-	-	4 1/2"	1 1/8"	4	4" x 1 1/8"	3 1/2" x 6" x 1/2"	1'-3"	EPE-4	
EPE-5	200 <sup>K</sup>	1'-10"	1'-3"	11"	1'-8"	2'-0"	3"	7"	4 1/2"	3 1/2"	1'-0"	2 1/2"	-	4 1/2"	1'-10"	4 1/2"	-	-	4 1/2"	1 1/8"	4	4" x 1 1/8"	3 1/2" x 6" x 1/2"	1'-3"	EPE-5	
EPE-6	300 <sup>K</sup>	1'-10"	1'-3"	1'-2"	1'-11"	1'-6"	3"	4"	4 1/2"	3 1/2"	1'-0"	2 1/2"	-	6"	1'-10"	4 1/2"	-	-	4 1/2"	1 1/8"	4	4" x 1 1/8"	3 1/2" x 4" x 1/2"	1'-3"	EPE-6	
EPE-7	300 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-2"	1'-11"	1'-8"	3 1/2"	5"	5"	4 1/2"	1'-0"	2 1/2"	-	6"	1'-10"	4 1/2"	-	-	3 1/2"	1 1/8"	4	3" x 1 1/8"	3 1/2" x 4" x 1/2"	1'-3"	EPE-7	
EPE-8	300 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-2"	1'-11"	1'-10"	3 1/2"	6"	5"	4 1/2"	1 1/2"	2 1/2"	-	6"	1'-10 1/2"	4 1/2"	-	-	4 1/2"	1 1/8"	4	3" x 1 1/8"	3 1/2" x 4" x 1/2"	1'-3"	EPE-8	
EPE-9	300 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-2"	1'-11"	2'-0"	3 1/2"	7"	5"	4 1/2"	1 1/2"	2 1/2"	-	6"	1'-10 1/2"	4 1/2"	-	-	4 1/2"	1 1/8"	4	3" x 1 1/8"	3 1/2" x 4 1/2" x 1/2"	1'-3"	EPE-9	
EPE-10	300 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-2"	1'-11"	2'-1"	3 1/2"	8"	5"	4 1/2"	1 1/2"	3 1/2"	-	6"	1'-10 1/2"	4 1/2"	-	-	5"	1 1/8"	4	4 1/2" x 1 1/8"	3 1/2" x 4 1/2" x 1/2"	1'-3"	EPE-10	
EPE-11	400 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-7"	2'-4"	1'-7"	3 1/2"	4 1/2"	5"	6 1/2"	1 1/2"	2 1/2"	9"	4"	1'-10 1/2"	4 1/2"	-	-	6"	1 1/8"	4	4 1/2" x 1 1/8"	3 1/2" x 6" x 1/2"	1'-3"	EPE-11	
EPE-12	400 <sup>K</sup>	1'-10 1/2"	1'-3"	1'-7"	2'-4"	1'-11"	3 1/2"	6 1/2"	5"	6 1/2"	1 1/2"	2 1/2"	8 1/2"	4"	1'-10 1/2"	4 1/2"	-	-	4"	1 1/8"	4	5 1/2" x 1 1/8"	3 1/2" x 8 1/2" x 1/2"	1'-3"	EPE-12	
EPE-13	400 <sup>K</sup>	1'-11"	1'-3"	1'-7"	2'-4"	2'-4"	4"	8 1/2"	5"	6 1/2"	11"	3 1/2"	8 1/2"	4"	1'-11"	4 1/2"	-	-	5"	1 1/8"	4	4 1/2" x 1 1/8"	3 1/2" x 4" x 1/2"	1'-3"	EPE-13	
EPE-14	600 <sup>K</sup>	2'-1 1/2"	1'-6"	1'-11"	3'-0"	1'-10"	3 1/2"	6"	7"	8 1/2"	1'-2 1/2"	2 1/2"	1 1/2"	5"	1'-11"	4 1/2"	-	-	6 1/2"	1 1/8"	4	4 1/2" x 1 1/8"	3 1/2" x 6 1/2" x 1/2"	1'-3"	EPE-14	
EPE-15	600 <sup>K</sup>	2'-2 1/2"	1'-6"	1'-11"	3'-0"	2'-3"	4 1/2"	9"	7"	8 1/2"	1'-2 1/2"	2 1/2"	1 1/2"	5"	1'-11"	4 1/2"	-	-	6 1/2"	1 1/8"	4	4" x 1 1/8"	4" x 5 1/2" x 1/2"	1'-3"	EPE-15	
EPE-16	800 <sup>K</sup>	2'-2"	1'-6"	2'-6"	3'-10"	1'-11"	4"	6 1/2"	10"	10 1/2"	1'-2"	2 1/2"	1 1/2"	6 1/2"	1'-11"	4 1/2"	-	-	6 1/2"	1 1/8"	4	6" x 1 1/8"	4" x 9 1/2" x 1/2"	1'-3"	EPE-16	
EPE-17	800 <sup>K</sup>	2'-2 1/2"	1'-6"	2'-6"	3'-10"	2'-3"	4 1/2"	9"	10"	10 1/2"	1'-1 1/2"	3 1/2"	10 1/2"	6 1/2"	1'-11 1/2"	4 1/2"	-	-	5 1/2"	1 1/8"	4	6" x 1 1/8"	4" x 9 1/2" x 1/2"	1'-3"	EPE-17	
FPD-1	100 <sup>K</sup>	1'-0"	-	8"	1'-6"	9"	2"	2 1/2"	6 1/2"	-	6"	-	-	-	1'-3"	3 1/2"	-	-	1"	1 1/8"	4	-	-	Standard	10"	FPD-1
FPD-2	200 <sup>K</sup>	1'-0"	-	10"	1'-8"	1'-2"	2"	4 1/2"	7 1/2"	-	6"	-	-	-	1'-5"	4"	-	-	1"	1 1/8"	4	-	-	Standard	1'-3"	FPD-2
FPD-3	300 <sup>K</sup>	1'-0"	-	1'-2"	2'-0"	1'-4"	2"	5 1/2"	9 1/2"	-	6"	-	-	-	1'-8"	4"	-	-	1"	1 1/8"	4	-	-	Standard	1'-3"	FPD-3
FPD-4	400 <sup>K</sup>	1'-3"	-	1'-6"	2'-4"	2"	6 1/2"	11 1/2"	-	9"	-	6 1/2"	-	-	1'-9"	4"	-	-	1"	1 1/8"	4	-	-	Standard	1'-3"	FPD-4
FPD-5	600 <sup>K</sup>	1'-3"	-	1'-11"	3'-0"	1'-10"	3"	8 1/2"	1'-3 1/2"	-	8"	-	3 1/2"	-	1'-9"	4"	-	-	1"	1 1/8"	4	-	-	Standard	1'-3"	FPD-5
FPD-6	800 <sup>K</sup>	1'-3"	-	2'-6"	3'-10"	1'-11"	3"	9 1/2"	1'-8 1/2"	-	8"	-	3 1/2"	-	1'-9"	4"	-	-	1"	1 1/8"	4	-	-	Standard	1'-3"	FPD-6
2 1/2"		Tab 1/2 x 3/4, tackweld																								



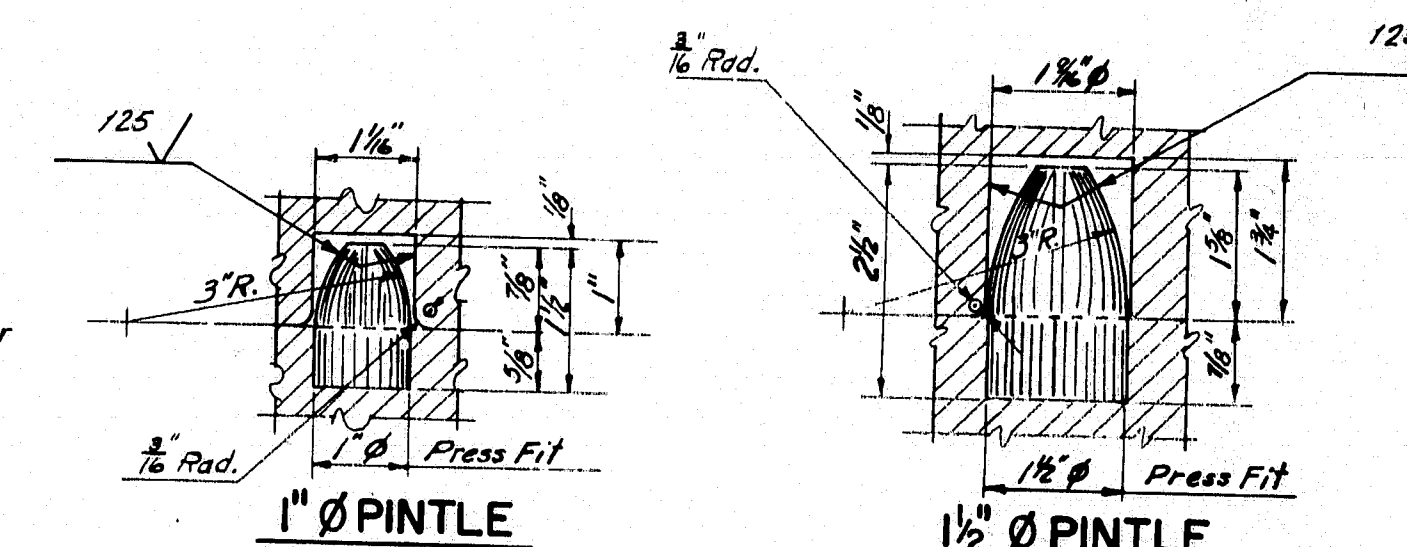
### CAP SCREW DETAIL



*EFD Series*      *EPE & FPD Series*  
**ANCHOR BOLT DETAILS**



## PINTLE DETAILS



## GENERAL NOTES

41. The location of bearing pedestals the concrete bridge seats shall be dressed one inch larger all around the size of masonry plates and to exact elevations shown on the plans. If dressed areas are below the surface of the surrounding bridge seat a small channel shall be cut/bledge of the bridge seat for drainage where required by the Engineer. Channels shall have a min. width of 2" and a min. slope of 4 inch per foot. No separate payment for this work will be made as it shall be considered incidental to contract items.

Fabricate pedestals with  $\frac{1}{4}$ " fillet welds. The diameter of the pin hole shall not exceed that of the pin by more than 10 inch.

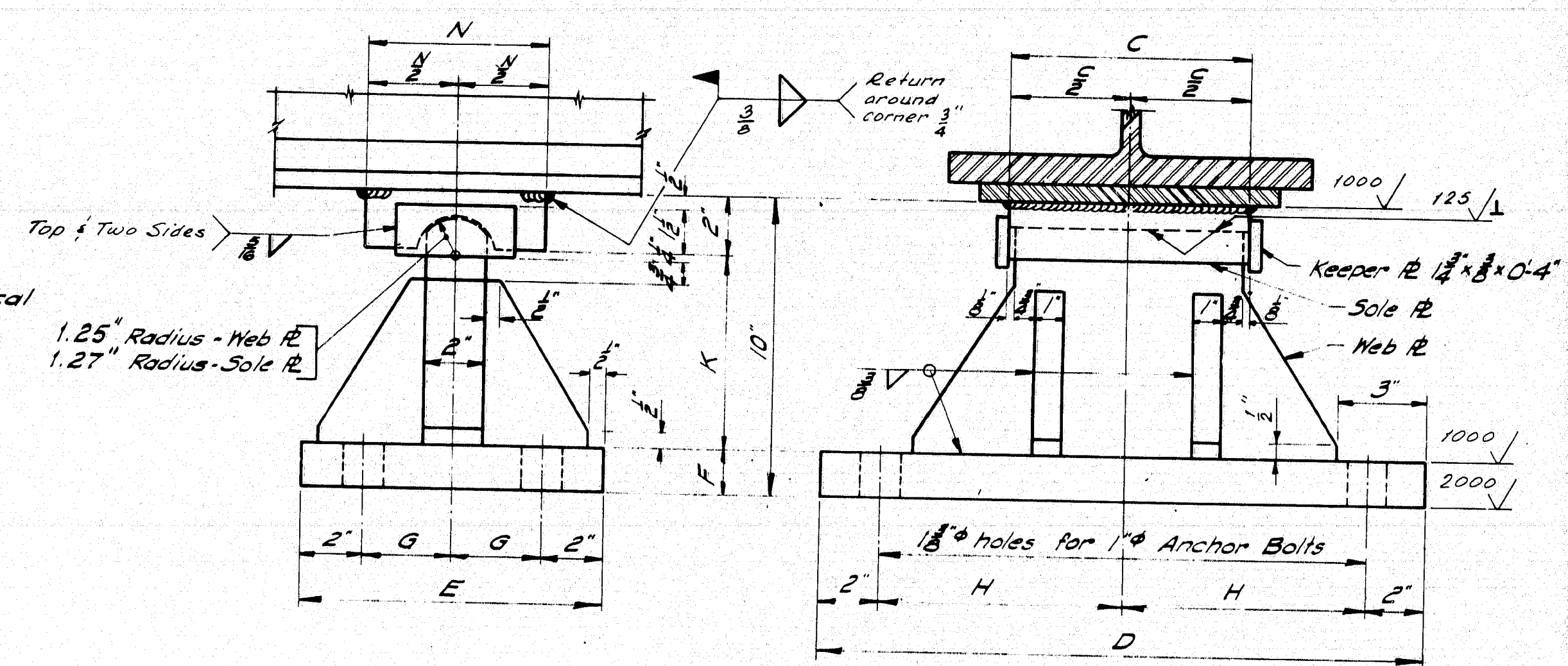
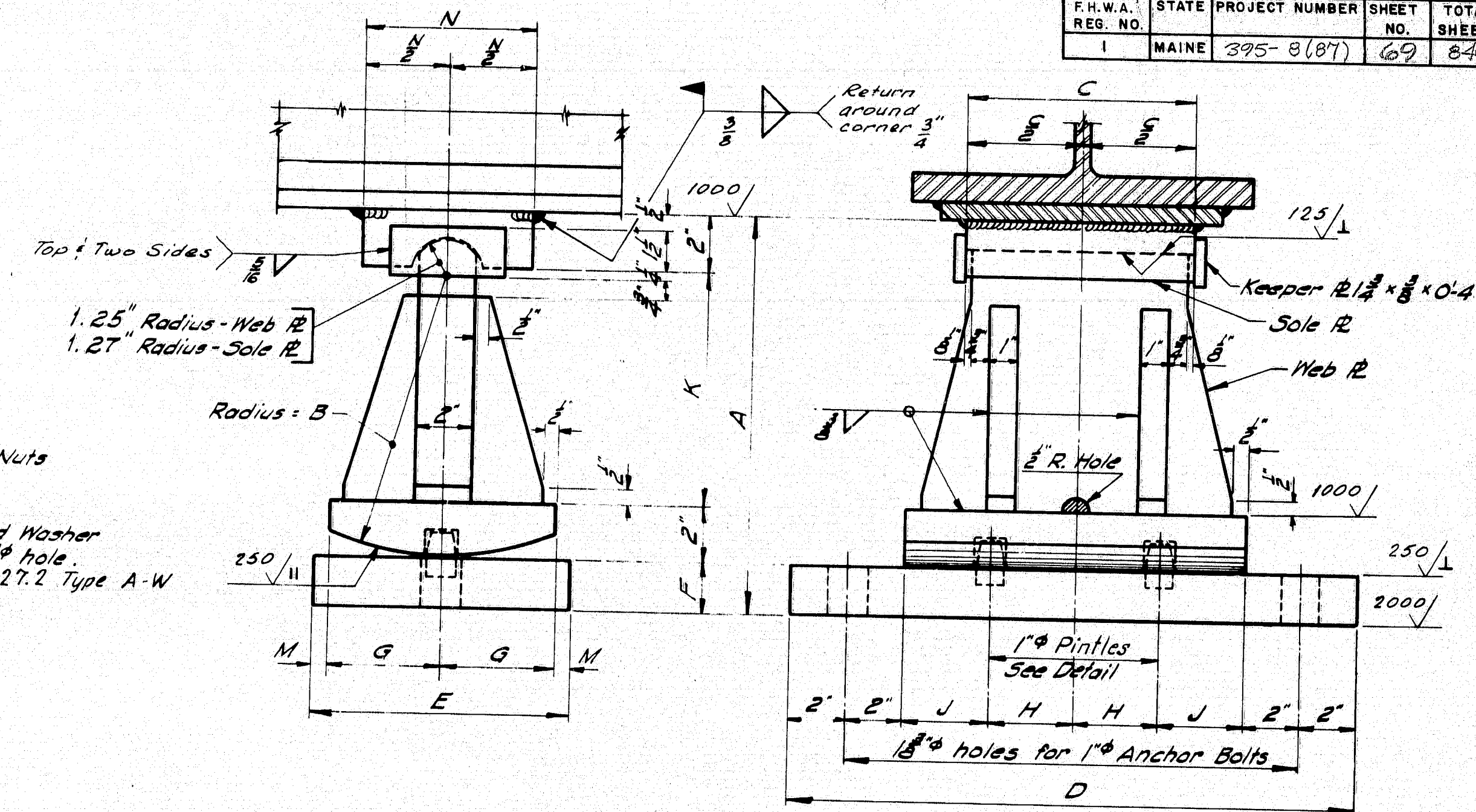
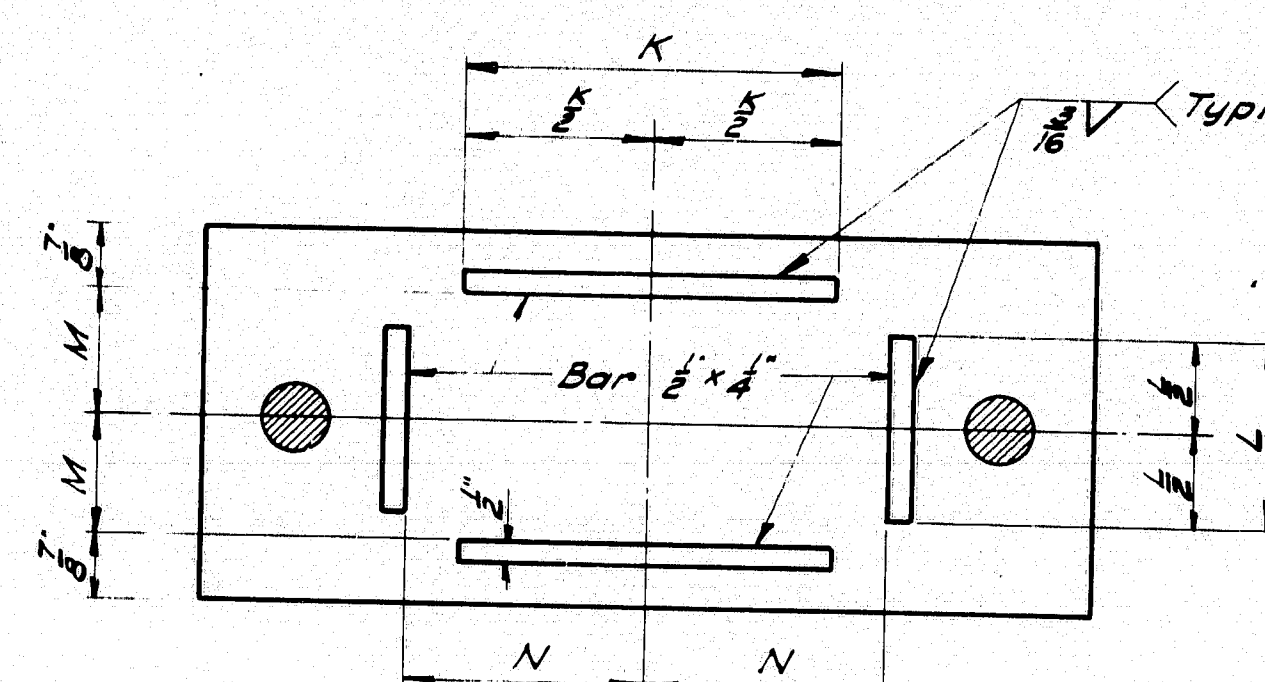
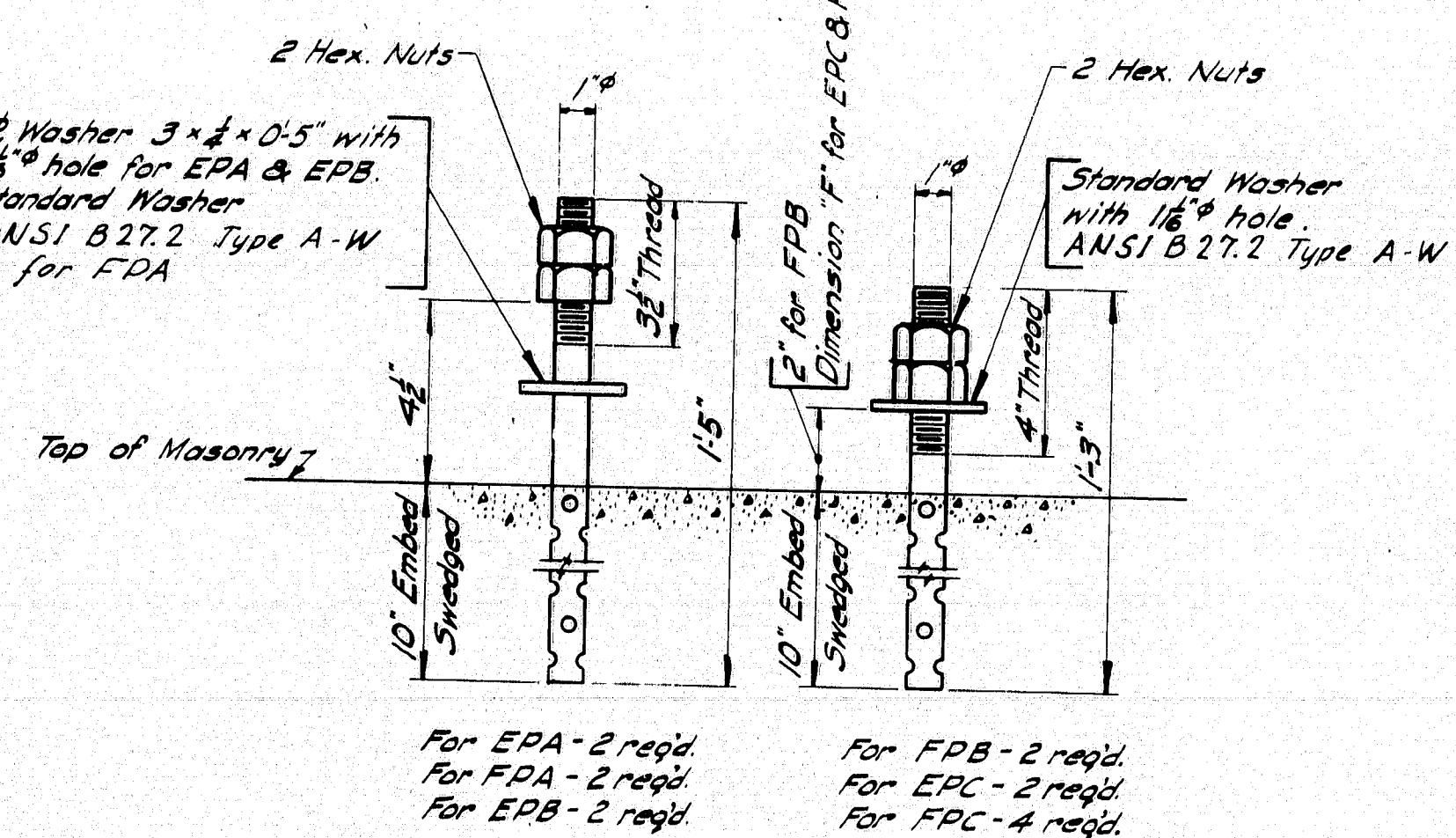
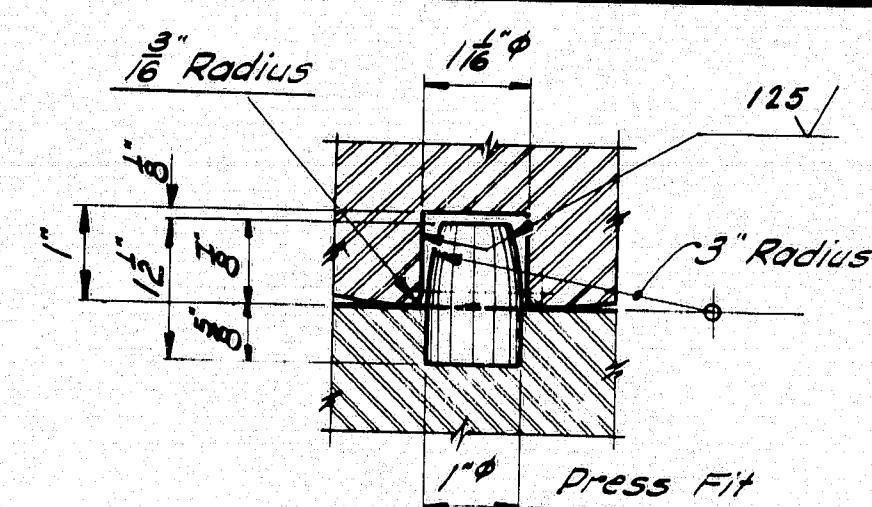
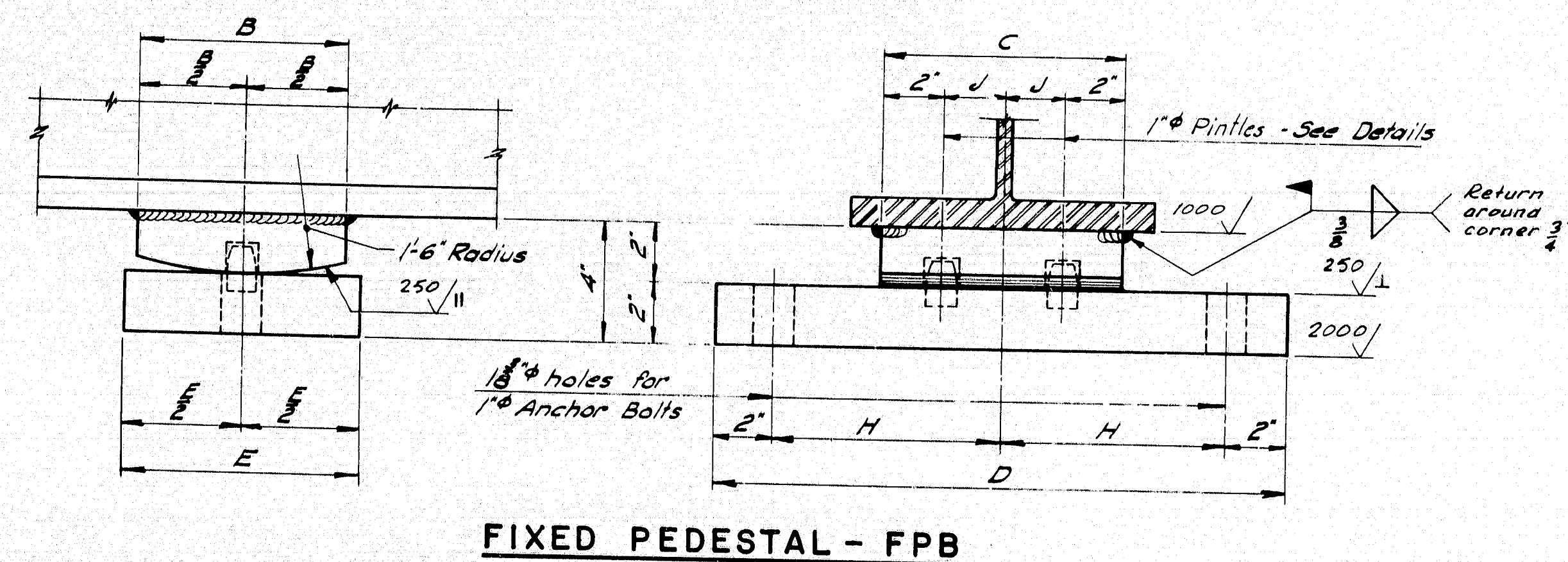
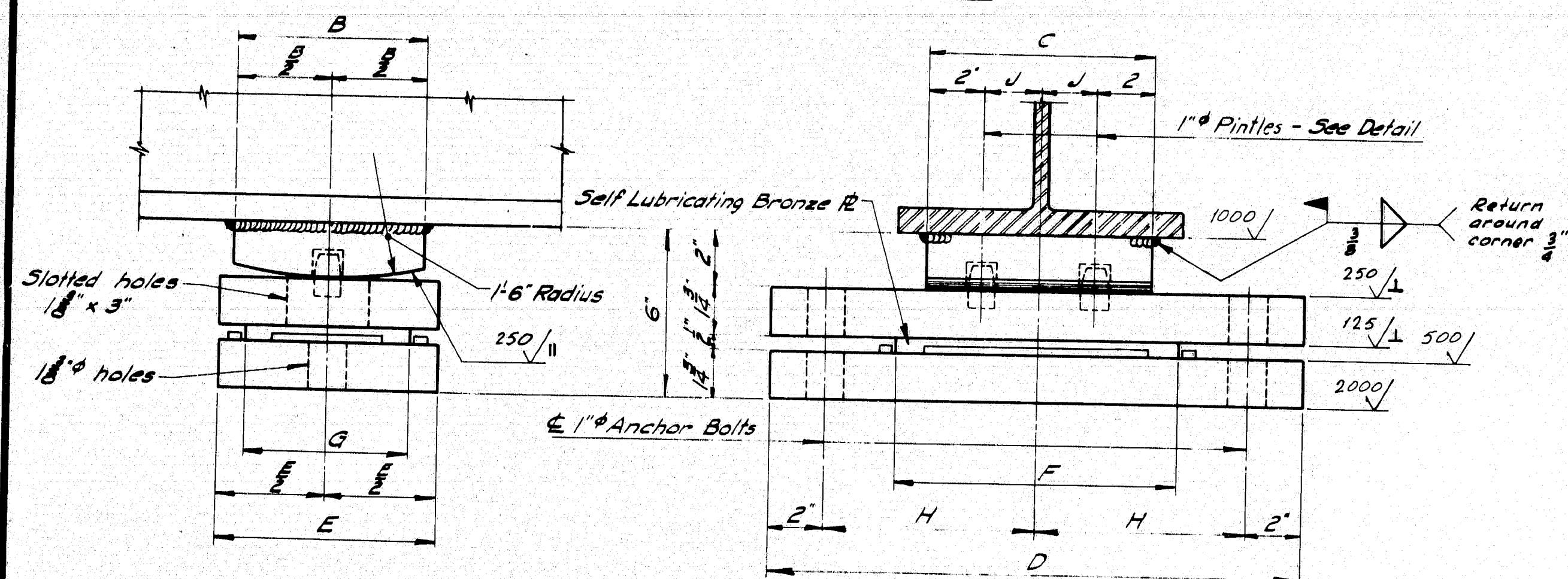
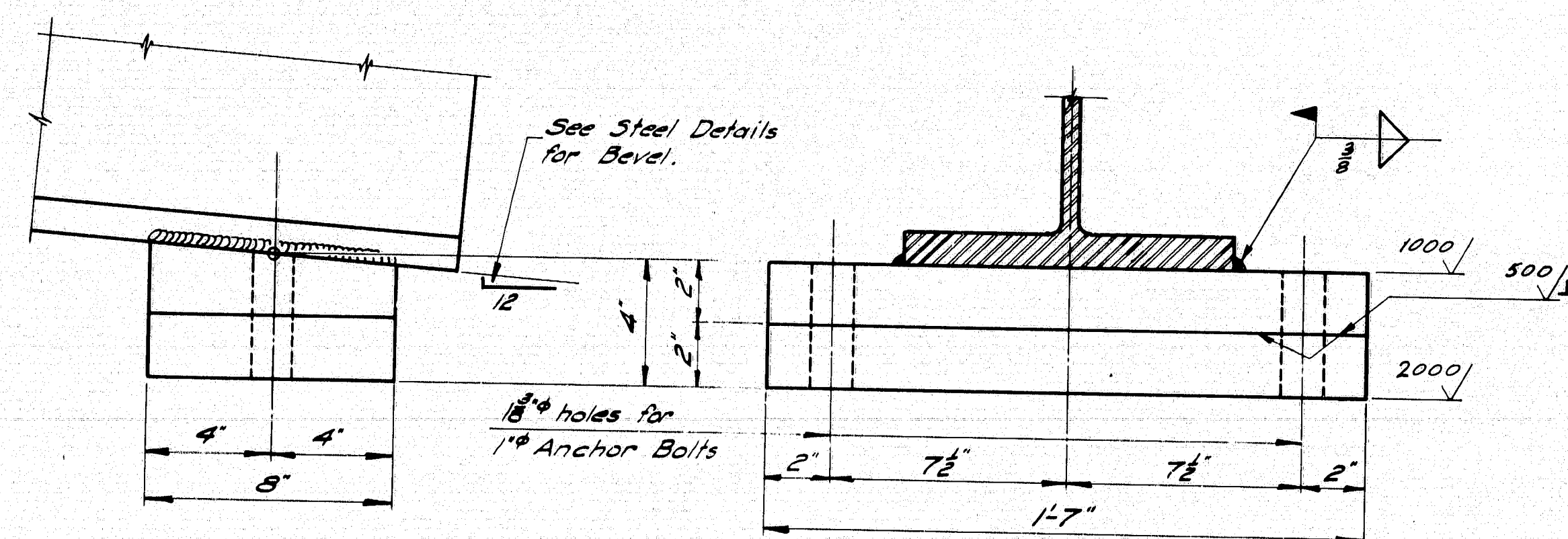
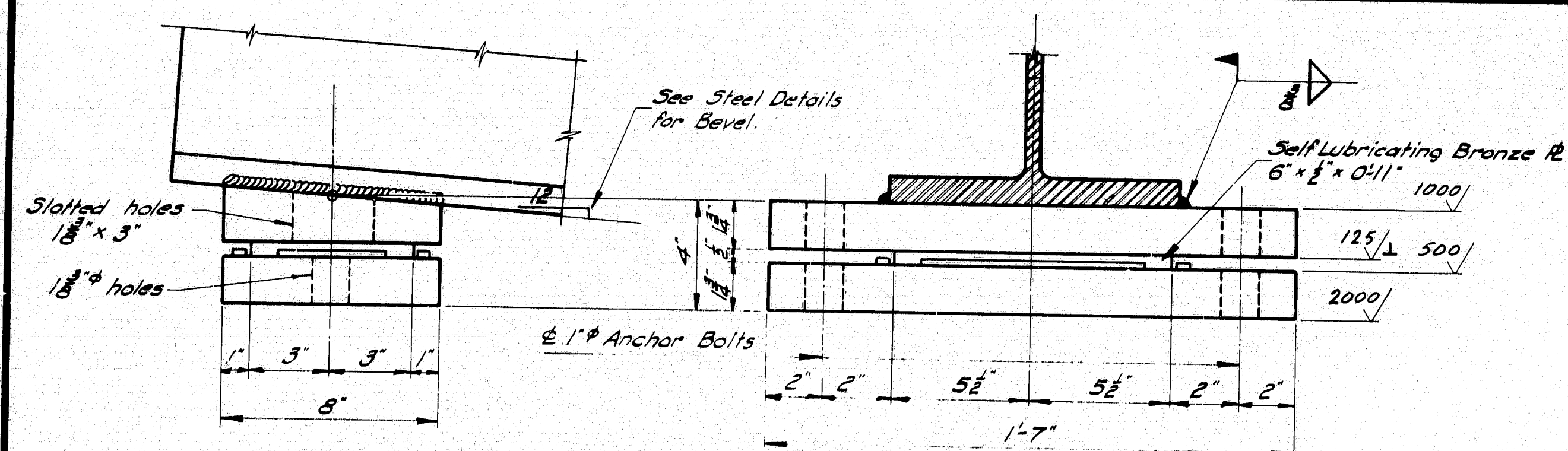
Pedestals EPD and EPE without center stiffeners have only one drainage hole. Pedestals FPD have no drainage holes.

### MINIMUM STEEL CLASSIFICATION

1. Chordy V-notch tests are not required for steel in bearing pedestals.
2. When structural steel is specified to be unpainted, all steel including anchor bolts and 2"  $\phi$  pins shall be A588 unpainted, except cap screws for EPE pedestals shall be A.S.T.M. A325, Type 3.
3. When structural steel is specified to be painted, all steel including anchor bolts shall be A36, except the following: 2"  $\phi$  pin - A36, A668, Class D or A108, Grade 1016 - 1030 inclusive; cap screws for EPE pedestals shall be A.S.T.M. A325, Type 1.

<b>REVISIONS</b>		DATE	
STATE OF MAINE DEPARTMENT OF TRANSPORTATION			
<b>STANDARD DETAILS</b> ( BD 100-81 )			
<b><u>BEARING PEDESTALS</u></b>			





PEDESTALS — ALLOWABLE LOADS & DIMENSIONS														
<i>Pedestal</i>	<i>Load</i>	A	B	C	D	E	F	G	H	J	K	L	M	N
EPA	132 <sup>K</sup>	—	—	—	—	—	—	—	—	—	8"	4"	3 $\frac{1}{2}$ "	5 $\frac{1}{2}$ "
FPA	130 <sup>K</sup>	—	—	—	—	—	—	—	—	—	—	—	—	—
EPB-1	120 <sup>K</sup>	—	6"	8"	1 $\frac{1}{2}$ "	8"	10"	6"	7 $\frac{1}{2}$ "	2"	8"	4"	3 $\frac{1}{2}$ "	5 $\frac{1}{2}$ "
EPB-2	165 <sup>K</sup>	—	7"	10"	1 $\frac{1}{8}$ "	9"	1 $\frac{1}{2}$ "	7"	8"	3"	10"	5"	3 $\frac{1}{2}$ "	6 $\frac{1}{2}$ "
EPA-3	224 <sup>K</sup>	—	8"	1 $\frac{1}{2}$ "	2 $\frac{1}{2}$ "	10"	1 $\frac{1}{4}$ "	8"	10"	4 $\frac{1}{2}$ "	1 $\frac{1}{2}$ "	5"	4 $\frac{1}{2}$ "	6 $\frac{1}{2}$ "
FPB-1	120 <sup>K</sup>	—	6"	8"	1 $\frac{1}{2}$ "	8"	—	—	7 $\frac{1}{2}$ "	2"	—	—	—	—
FPB-2	165 <sup>K</sup>	—	7"	10"	1 $\frac{1}{8}$ "	9"	—	—	8"	3"	—	—	—	—
FPB-3	224 <sup>K</sup>	—	8"	1 $\frac{1}{2}$ "	2 $\frac{1}{2}$ "	10"	—	—	10"	5"	—	—	—	—
EPC-1	70 <sup>K</sup>	2 $\frac{1}{2}$ "	6"	8"	1 $\frac{1}{8}$ "	8"	1 $\frac{1}{2}$ "	3 $\frac{1}{2}$ "	3"	3"	4 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	6"
EPC-2	100 <sup>K</sup>	1 $\frac{1}{8}$ "	8"	8"	1 $\frac{1}{8}$ "	8"	1 $\frac{1}{2}$ "	3 $\frac{1}{2}$ "	3"	3"	6 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	6"
EPC-3	130 <sup>K</sup>	1 $\frac{1}{2}$ "	10"	8"	1 $\frac{1}{8}$ "	9"	1 $\frac{1}{2}$ "	4"	3"	3"	8 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	7"
EPC-4	160 <sup>K</sup>	1 $\frac{1}{2}$ "	10"	8"	1 $\frac{1}{8}$ "	9"	1 $\frac{1}{2}$ "	4"	4"	3"	8 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	7"
EPC-5	190 <sup>K</sup>	1 $\frac{1}{2}$ "	10"	9"	2 $\frac{1}{2}$ "	10"	2"	4 $\frac{1}{2}$ "	5"	3"	8 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	7"
EPC-6	220 <sup>K</sup>	1 $\frac{1}{4}$ "	1 $\frac{1}{2}$ "	10"	2 $\frac{1}{2}$ "	10"	2 $\frac{1}{2}$ "	5"	3"	3"	10 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	8"
EPC-7	250 <sup>K</sup>	1 $\frac{1}{4}$ "	1 $\frac{1}{2}$ "	1 $\frac{1}{2}$ "	2 $\frac{1}{2}$ "	10"	2 $\frac{1}{2}$ "	5"	3"	4"	10 $\frac{1}{2}$ "	—	1 $\frac{1}{2}$ "	8"
FPC-1	100 <sup>K</sup>	—	—	8"	1 $\frac{1}{8}$ "	9"	1 $\frac{1}{2}$ "	2 $\frac{1}{2}$ "	8"	—	6 $\frac{1}{2}$ "	—	—	6"
FPC-2	160 <sup>K</sup>	—	—	8"	1 $\frac{1}{8}$ "	9"	1 $\frac{1}{2}$ "	3"	8"	—	6 $\frac{1}{2}$ "	—	—	7"
FPC-3	180 <sup>K</sup>	—	—	9"	2 $\frac{1}{2}$ "	10"	1 $\frac{1}{2}$ "	3"	10"	—	6 $\frac{1}{2}$ "	—	—	8"
FPC-4	220 <sup>K</sup>	—	—	10"	2 $\frac{1}{2}$ "	10"	1 $\frac{1}{2}$ "	3"	10"	—	6 $\frac{1}{2}$ "	—	—	8"
FPC-5	250 <sup>K</sup>	—	—	10"	2 $\frac{1}{2}$ "	10"	2"	4"	10"	—	6 $\frac{1}{2}$ "	—	—	8"

NOTE: At the location of bearing pedestals the concrete bridge seats shall be dressed one inch larger all around than size of masonry plates and to exact conditions shown on the plans. If dressing areas are to be on the surface of the surrounding bridge seat a small channel shall be cut to the edge of the bridge seat for drainage where required by the Engineer. Channels shall have a min. width of 2" and min. slope of 1 inch per foot. No separate payment for this work will be made as it shall be considered incidental to contract items.

### A.S.T.M. STEEL CLASSIFICATION

1. Charpy V-Notch tests are not required for steel used in bearing pedestals.
2. When structural steel is specified to be unpainted, all steel including anchor bolts shall be A588 unpainted.
3. When structural steel is specified to be painted, all steel including anchor bolts shall be A36.

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

# STANDARD DETAILS

( BD 101 - 81 )

## BEARING PEDESTALS

### REVISIONS

DATE \_\_\_\_\_

SHEET OF AUGUSTA, MAINE JUNE 1981

**183-164**



Hand-drawn technical drawing of a steel beam splice. The drawing shows a top view and a side view. The top view shows a central splice area with dimensions: 1 1/2" spacing, 3" splice, 3" splice, 3" splice, 1 1/2" spacing. The side view shows a beam with dimensions: 2" top flange, 5" web, 1 1/2" bottom flange, and 6" total height. The beam is labeled "W27 x 84, 94, 102, 114".

[illegible]

Hand-drawn structural drawings of a reinforced concrete slab and column. The top drawing is a plan view of a slab with dimensions: 1 1/2" end offsets, 3" spacing between reinforcement bars, and 7" total width. It shows a central column area with a splice. The bottom drawing is a cross-section of the slab and column, showing reinforcement bars (2-#8, 2-#7, 2-#8) and dimensions: 1 1/2" end offsets, 3" spacing between bars, and 7" total width. It also shows a splice.

The image contains two hand-drawn technical drawings of a reinforced concrete beam section, showing the layout of reinforcement bars (rebar) and their spacing.

**Top Drawing:** This drawing shows a cross-section of a beam with a central splice. The total width of the beam is 12 inches. The spacing of the reinforcement bars is indicated as "D" spaces @ 3". The distance from the centerline to the edge of the beam is 1 1/2 inches. The total length of the beam is 34 3/4 inches. The spacing of the bars is 3 inches, with a 1/2 inch gap at the ends. The drawing shows a central splice area with a width of 3 inches and a length of 3 inches. The bars are arranged in a grid pattern, with 7 bars in the top and 7 bars in the bottom.

**Bottom Drawing:** This drawing shows a cross-section of a beam with a central splice. The total width of the beam is 12 inches. The spacing of the reinforcement bars is indicated as "D" spaces @ 3". The distance from the centerline to the edge of the beam is 1 1/2 inches. The total length of the beam is 34 3/4 inches. The spacing of the bars is 3 inches, with a 1/2 inch gap at the ends. The drawing shows a central splice area with a width of 3 inches and a length of 3 inches. The bars are arranged in a grid pattern, with 7 bars in the top and 7 bars in the bottom.

Hand-drawn structural drawings of a beam splice. The top drawing is a side elevation showing a central splice region with reinforcement bars. Dimensions include 1 1/2" end offsets, 3" spacing for bars, and a 7" total height. The bottom drawing is a top-down view of the splice, showing a 9" wide section with 3" spacing between bars. Reinforcement is labeled as 2-#4, 2-#5, and 2-#6 bars. A 1'-0" dimension is shown for the overall width.

The diagram illustrates the reinforcement layout for a concrete slab. It includes the following details:

- Top View:** Shows a rectangular grid of reinforcement bars. The overall dimensions are 7'9" by 3'0". The spacing between the main reinforcement bars is 12" D-spaces @ 3". A central splice is shown with dimensions 2'6", 3'0", and 1'6".
- Side Elevation:** Shows the vertical arrangement of the reinforcement bars. The total height is 3'0". The bars are arranged in three layers, each labeled "2-#8". The vertical spacing between the layers is 1'6", 3'0", and 1'6".

SPlice PLATES AND FLANGE HOLES				
BEAM	PLATE "A"	PLATE "B"	PLATE "C"	"D"
W 27 x 84	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$10 \times \frac{1}{2}$	3
*94	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$10 \times \frac{1}{2}$	3
*102	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$10 \times \frac{1}{2}$	3
*114	$12\frac{1}{2} \times \frac{1}{8}$	$4 \times \frac{1}{8}$	$10 \times \frac{1}{2}$	3
W30 x 99	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$10 \times \frac{1}{2}$	4
*108	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$10 \times \frac{1}{2}$	3
*116	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$10 \times \frac{1}{2}$	3
*124	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{4}$	$10 \times \frac{1}{2}$	4
*132	$12\frac{1}{2} \times \frac{1}{8}$	$4 \times \frac{1}{8}$	$10 \times \frac{1}{2}$	4
W33 x 118	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$11 \times \frac{1}{2}$	3
*130	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$11 \times \frac{1}{2}$	4
*141	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$11 \times \frac{1}{2}$	4
*152	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$11 \times \frac{1}{2}$	5
W36 x 133	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$11 \times \frac{1}{2}$	4
*150	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{8}$	$11 \times \frac{1}{2}$	5
*160	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$11 \times \frac{1}{2}$	5
*170	$12\frac{1}{2} \times \frac{1}{2}$	$4 \times \frac{1}{2}$	$11 \times \frac{1}{2}$	6
*182	$16\frac{1}{2} \times \frac{1}{8}$	$4 \times 1$	$11 \times \frac{1}{8}$	6
*194	$16\frac{1}{2} \times \frac{1}{8}$	$4 \times 1\frac{1}{8}$	$11 \times \frac{1}{8}$	6
*210	$16\frac{1}{2} \times \frac{1}{2}$	$4 \times 1\frac{1}{2}$	$11 \times \frac{1}{2}$	7
*230	$16\frac{1}{2} \times \frac{1}{2}$	$6 \times 1$	$16 \times \frac{1}{2}$	9
*245	$16\frac{1}{2} \times \frac{1}{8}$	$6 \times 1$	$16 \times \frac{1}{8}$	9
*260	$16\frac{1}{2} \times \frac{1}{8}$	$6 \times 1\frac{1}{8}$	$16 \times \frac{1}{8}$	11
*280	$16\frac{1}{2} \times \frac{1}{2}$	$6 \times 1\frac{1}{2}$	$16 \times \frac{1}{2}$	11
*300	$16\frac{1}{2} \times \frac{1}{8}$	$6 \times 1\frac{1}{2}$	$16 \times \frac{1}{8}$	13

- 1.) Splice connections shall be made with  $\frac{7}{8}" \phi$  ASTM A325 high tensile strength bolts. Holes shall be  $\frac{15}{16}" \phi$ .
- 2.) Web and flange filler plates shall be used as required when splicing beams of different sizes. Filler plates of  $\frac{1}{16}"$  or less in thickness are not required.
- 3.) If beams of different sizes are to be spliced, use splice details shown for the smaller of the beams being spliced unless otherwise directed by design drawings.
- 4.) For material specifications and details not shown, refer to design drawings.

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



F.H.W.A. SHEET NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(87)	77	84

### FABRICATION NOTES

- 1.) All bolts shall be  $\frac{7}{8}$ " H.S. Bolts. Hole sizes for bolts shall conform to Section 504.23 of the Standard Specifications, and edge distances shall be  $\frac{1}{2}$ " min. unless otherwise shown.
- 2.) Connection Plates and gusset plates shall have a minimum thickness of  $\frac{3}{8}$ " and shall have sufficient width to provide erection clearances. For bearing stiffeners or intermediate stiffeners and for bent connection plates the plate size will be given on the design drawings.
- 3.) Connection Plates shall be fastened to web plates by fillet welds as shown. All fillet welds shall be the minimum size as specified in A.A.S.H.T.O. Standard Specifications for Highway Bridges, Art. 1.7.21, unless otherwise shown on design drawings.
- 4.) Connection Plates shall be  $\frac{3}{8}$ " clear from flanges, except as indicated by notes 5 & 6.
- 5.) Connection Plates on welded beams and girders shall extend to the top flange in areas where the top flange is always in compression.
- 6.) Connection Plates shall extend to the bottom flange at points where lateral bracing is attached and on welded beams and girders in areas where the bottom flange is always in compression.
- 7.) When a connection plate is extended to a flange it shall fit within  $\frac{1}{16}$ " except if the design drawings show it is to be welded.
- 8.) Bearing Stiffeners at end bearings shall extend to both top and bottom flanges and shall be welded to both flanges. Weld at bottom flange shall be a full penetration weld. Weld at top flange shall be a fillet weld both sides (see Note 3).
- 9.) Bearing Stiffeners at other than end bearings shall extend to both top and bottom flanges, shall be welded to the bottom flange with a full penetration weld and shall fit within  $\frac{1}{16}$ " at top flange.
- 10.) Intermediate Stiffeners shall extend to both top and bottom flanges, shall be welded to the compression flange with a fillet weld on both sides (see Note 3) and shall fit within  $\frac{1}{16}$ " at the tension flange.
- 11.) Use only those items called for on the design drawings. In case of conflict between these standard details and design drawings, the design drawings shall be followed.
- 12.) All dimensions shown as " - ± 1" are variable in order to allow a series of crossframes to have the same slopes and/or dimensions.
- 13.) All connection plates and stiffeners that are extended to a flange shall be clipped  $\frac{3}{8}$ ", except as indicated by note 14.
- 14.) Bearing stiffeners at end bearings shall be clipped  $\frac{1}{2}$ " at top and bottom. Bearing stiffeners at all other bearings shall be clipped  $\frac{1}{2}$ " at the compression flange.
- 15.) For unpainted applications all steel for diaphragms and crossframes shall be A.S.T.M. - A588. For bridges specified to be painted the steel for diaphragms and connection plates shall be A.S.T.M. - A36, except other steel classifications may be used subject to the approval of the Engineer.

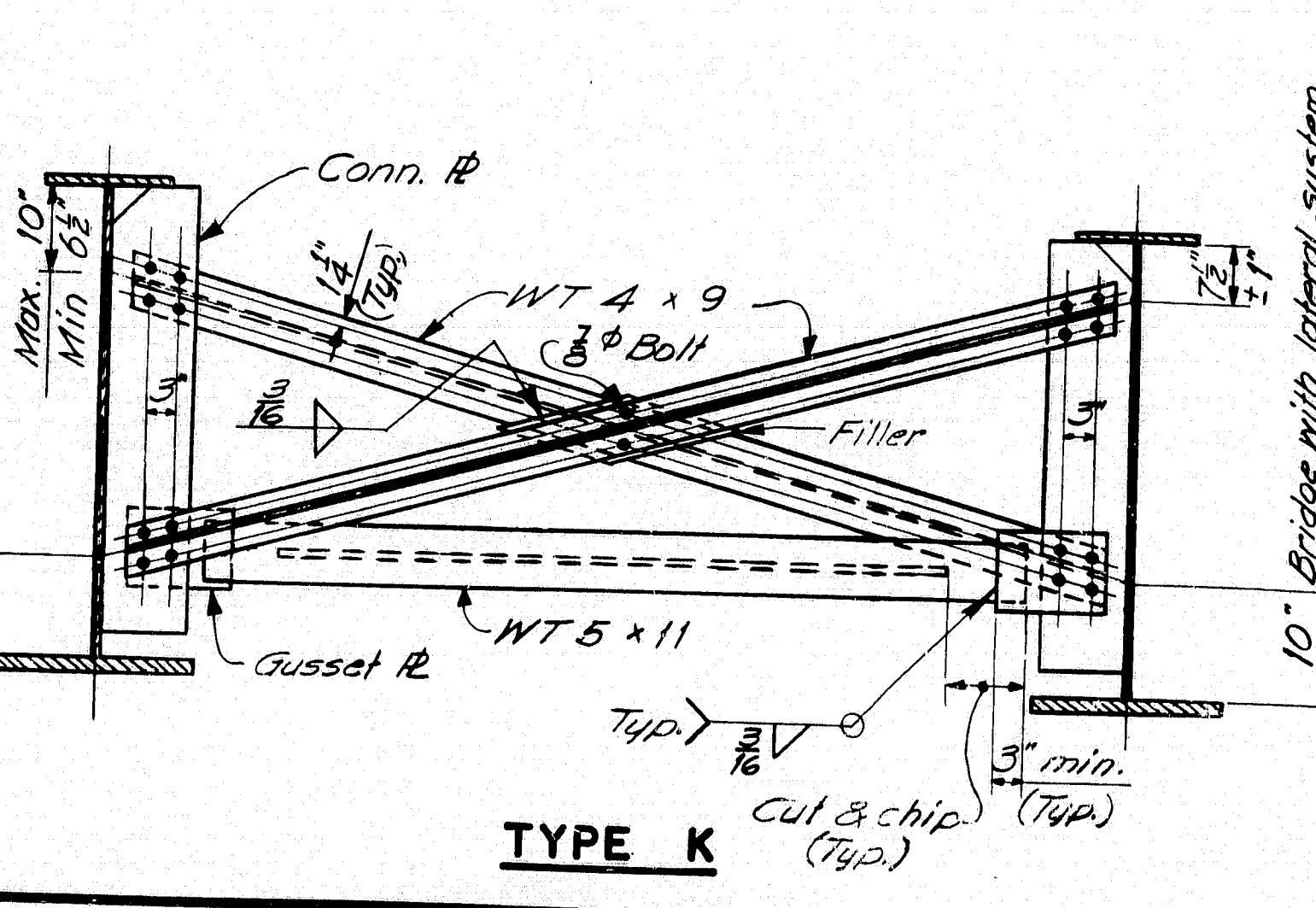
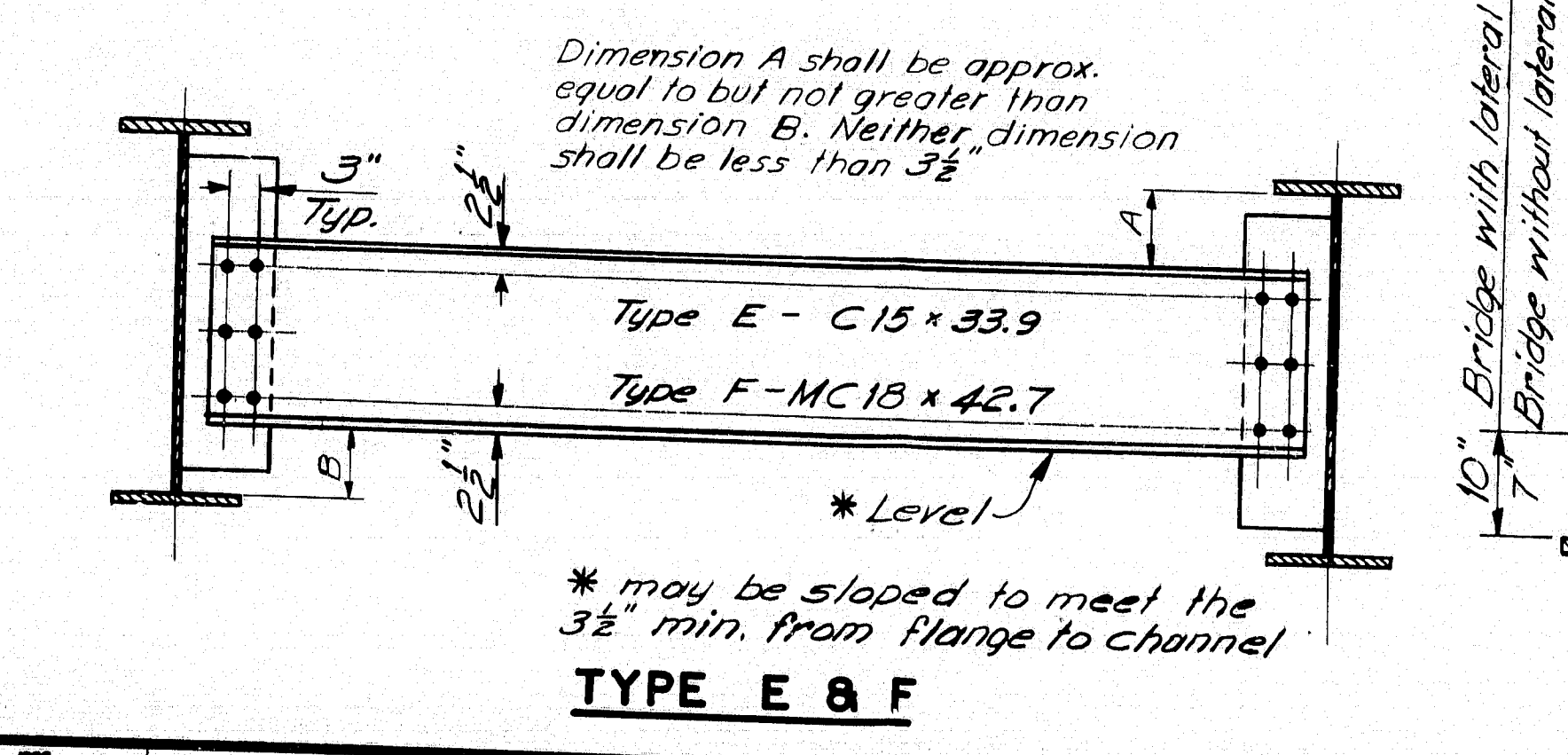
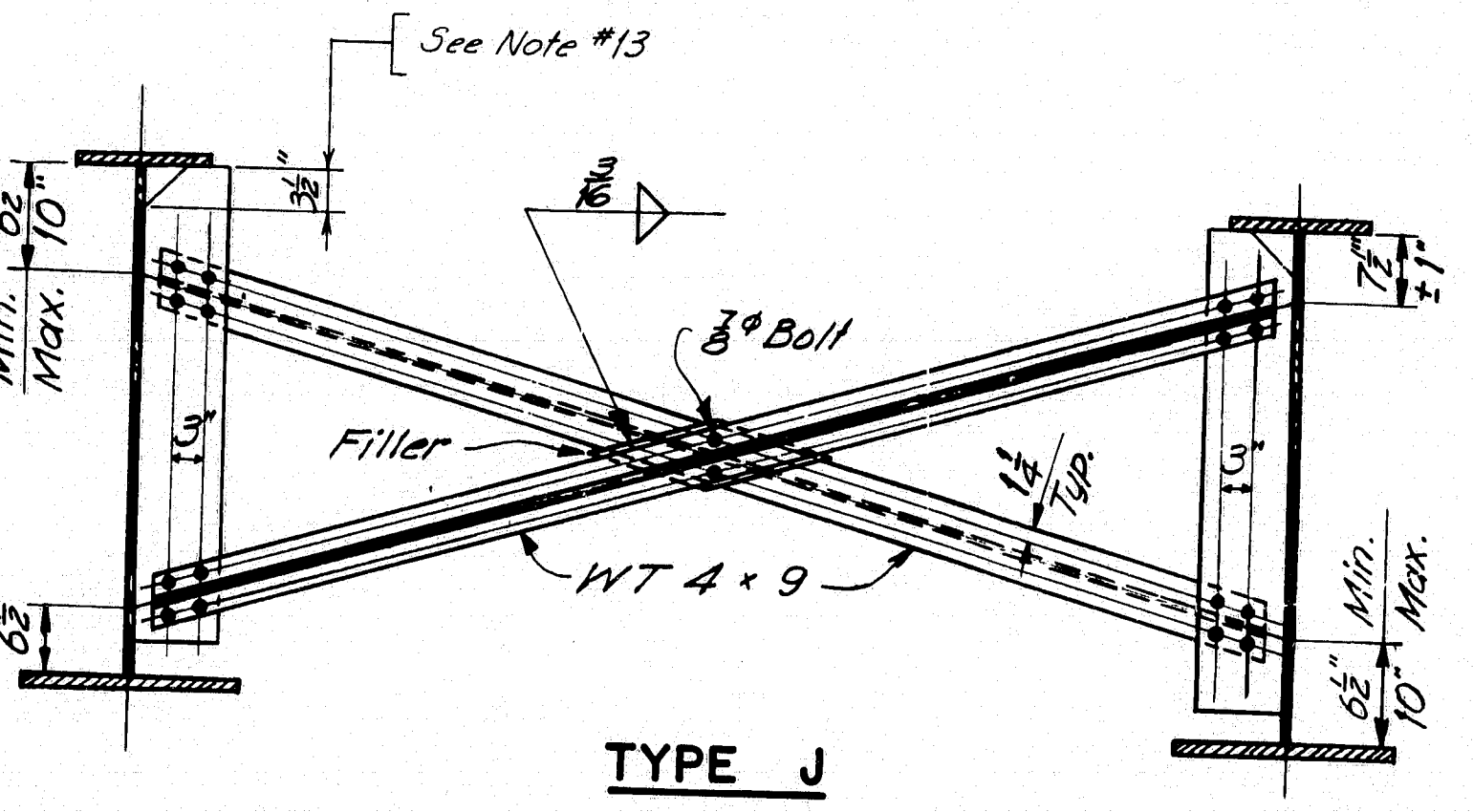
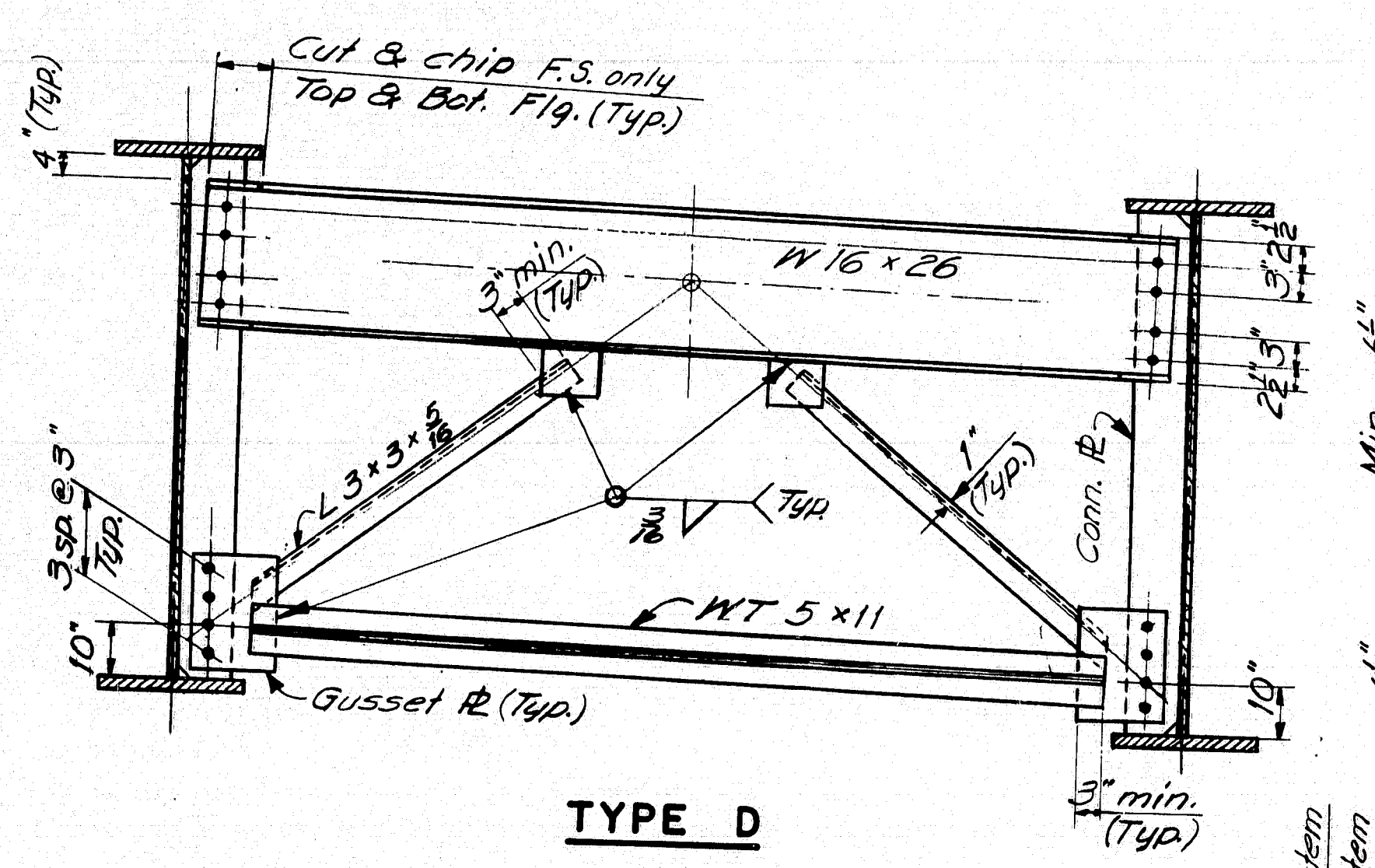
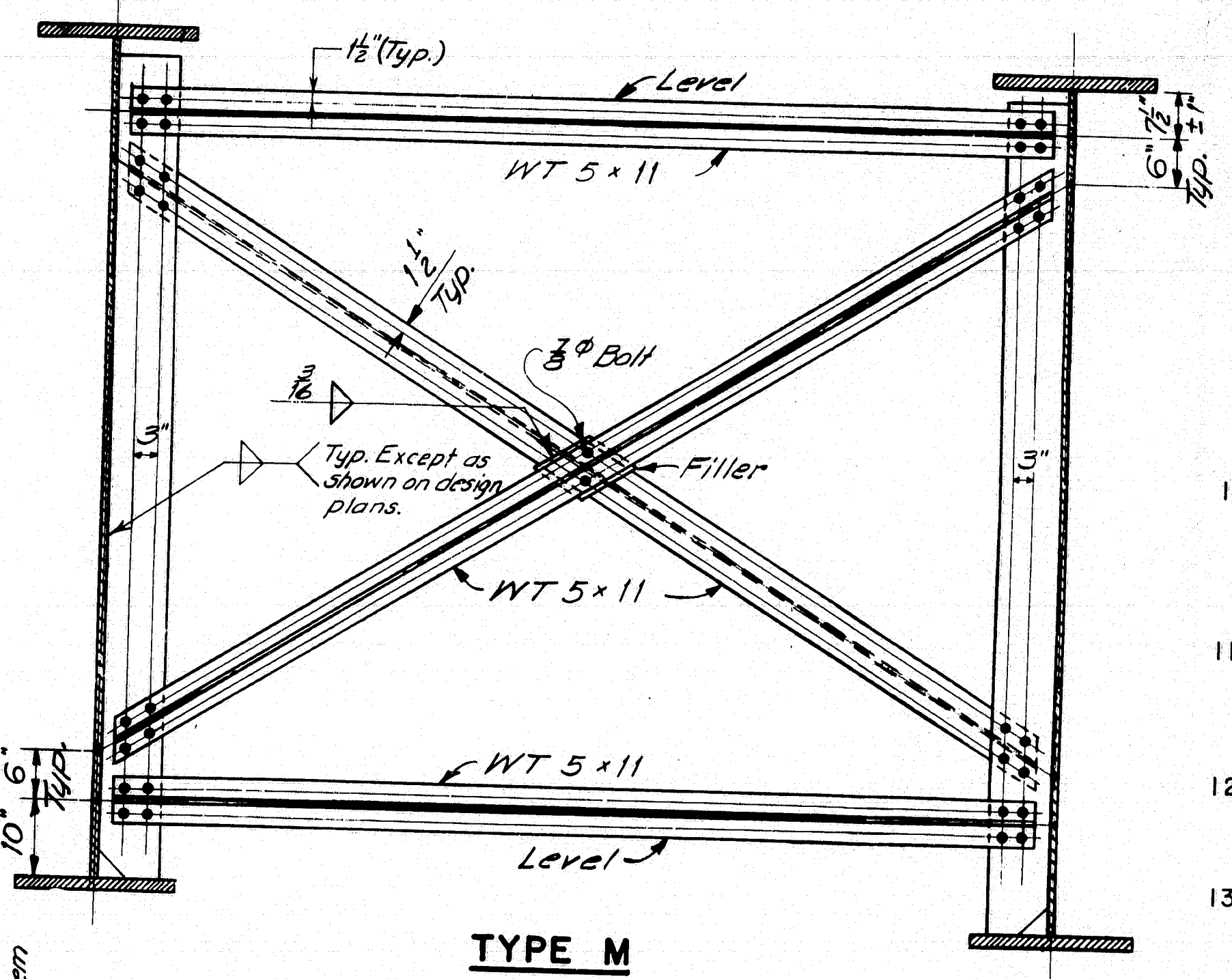
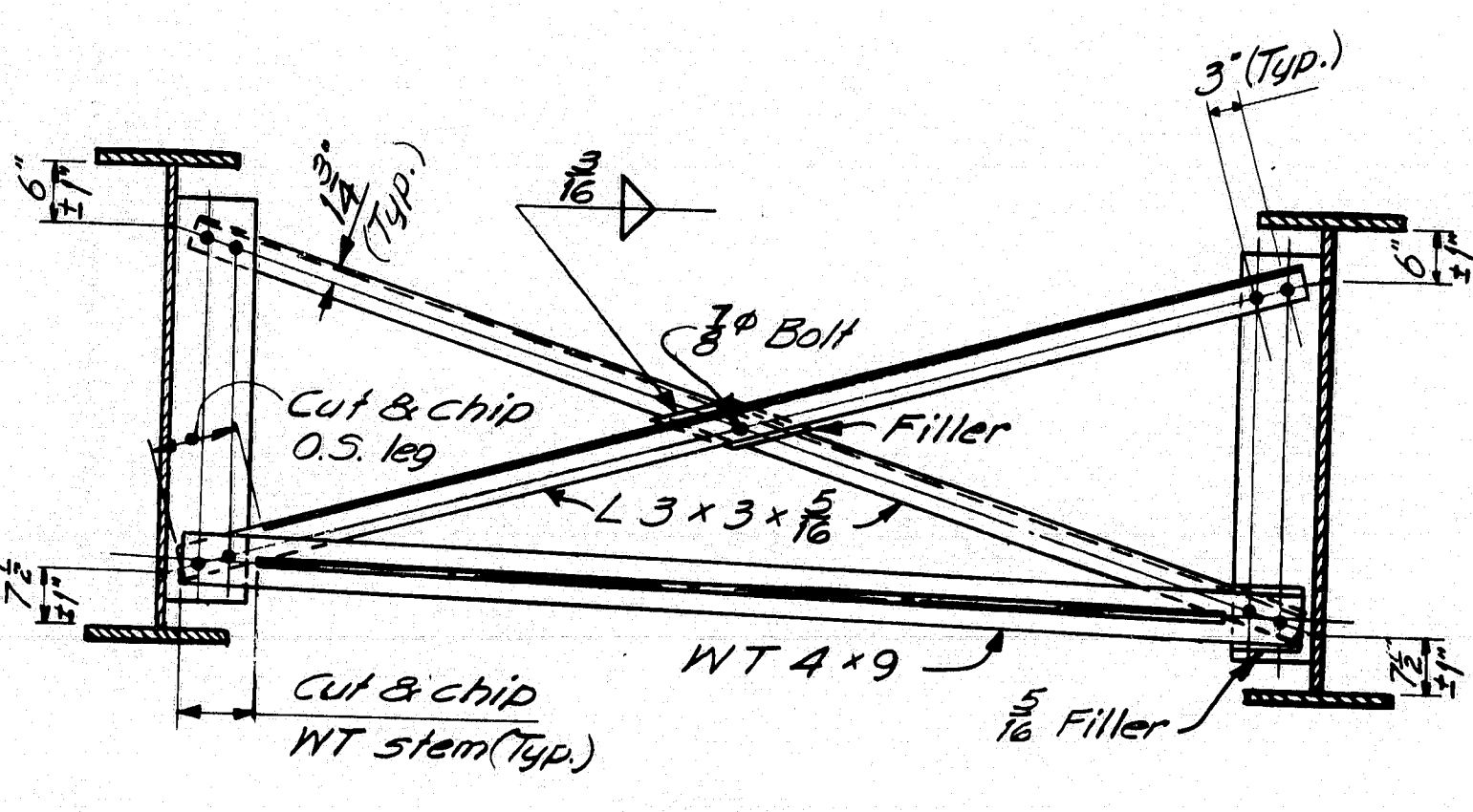
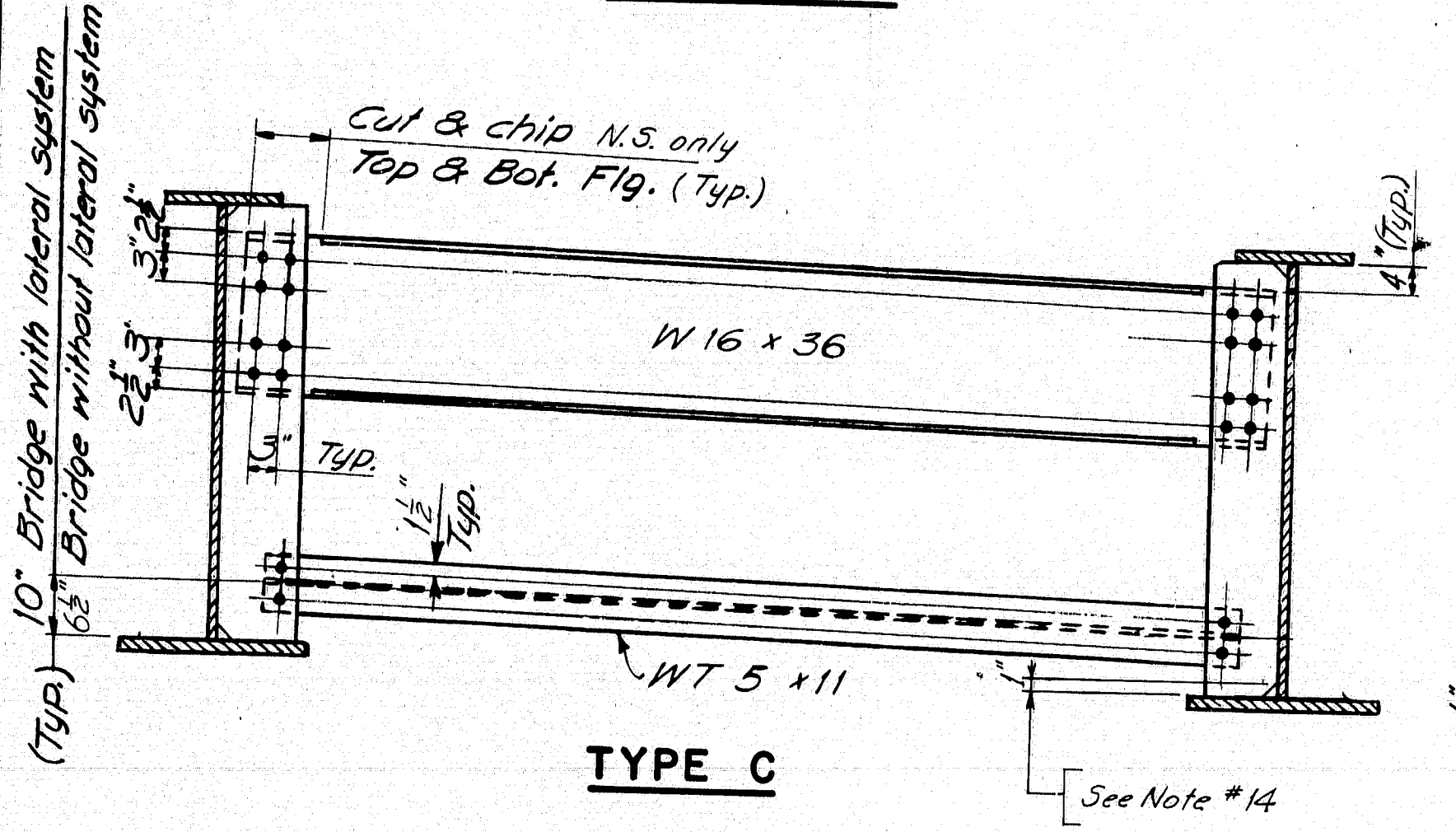
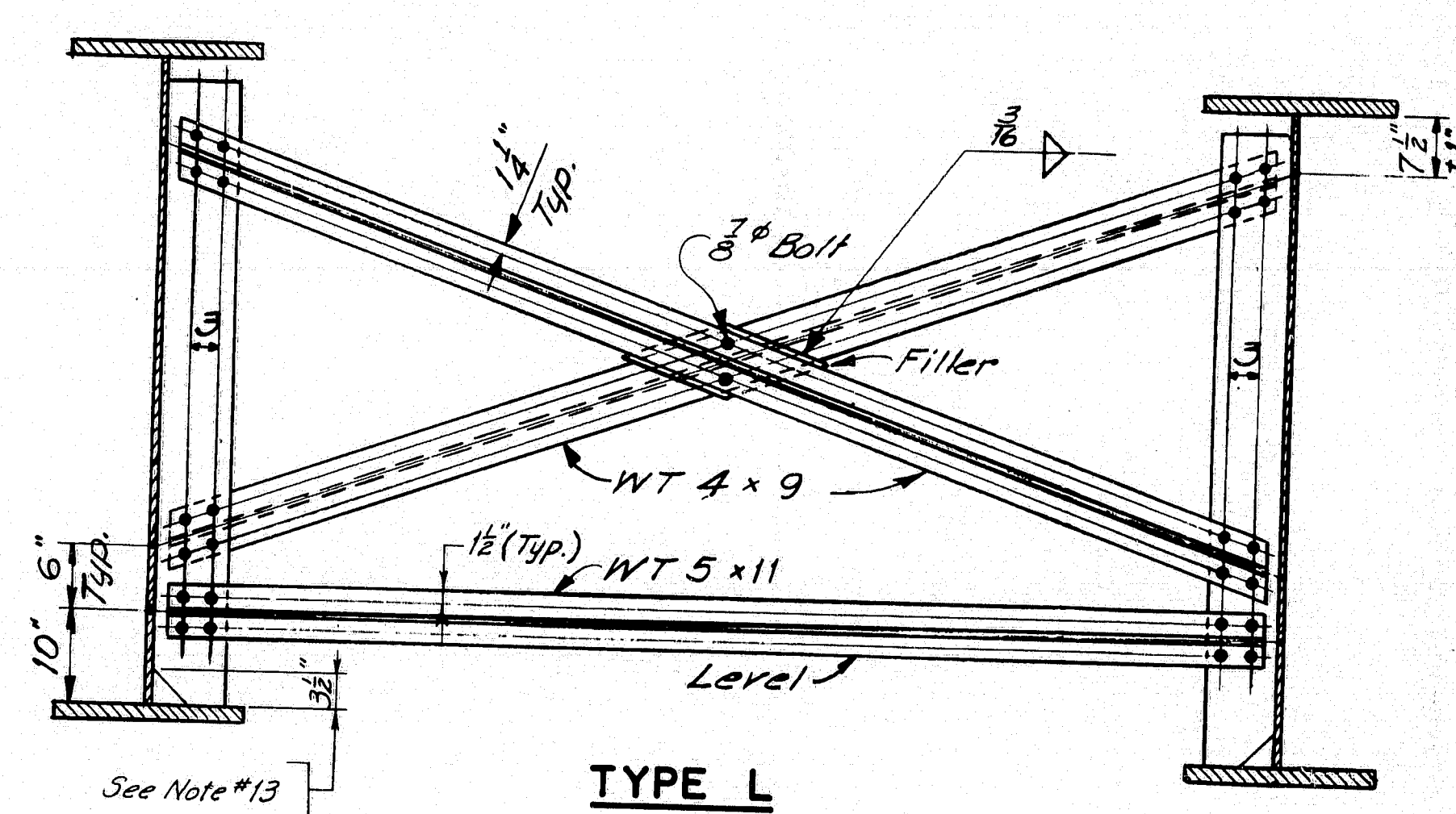
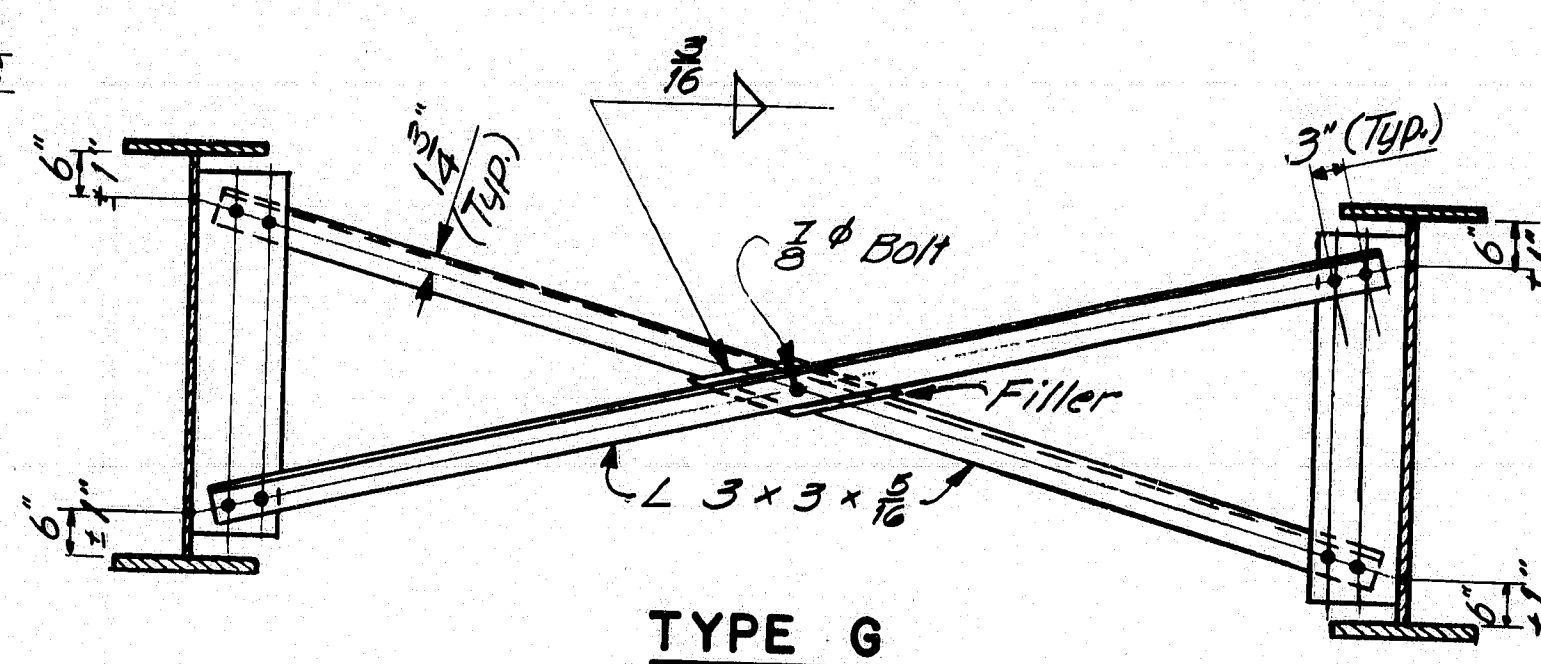
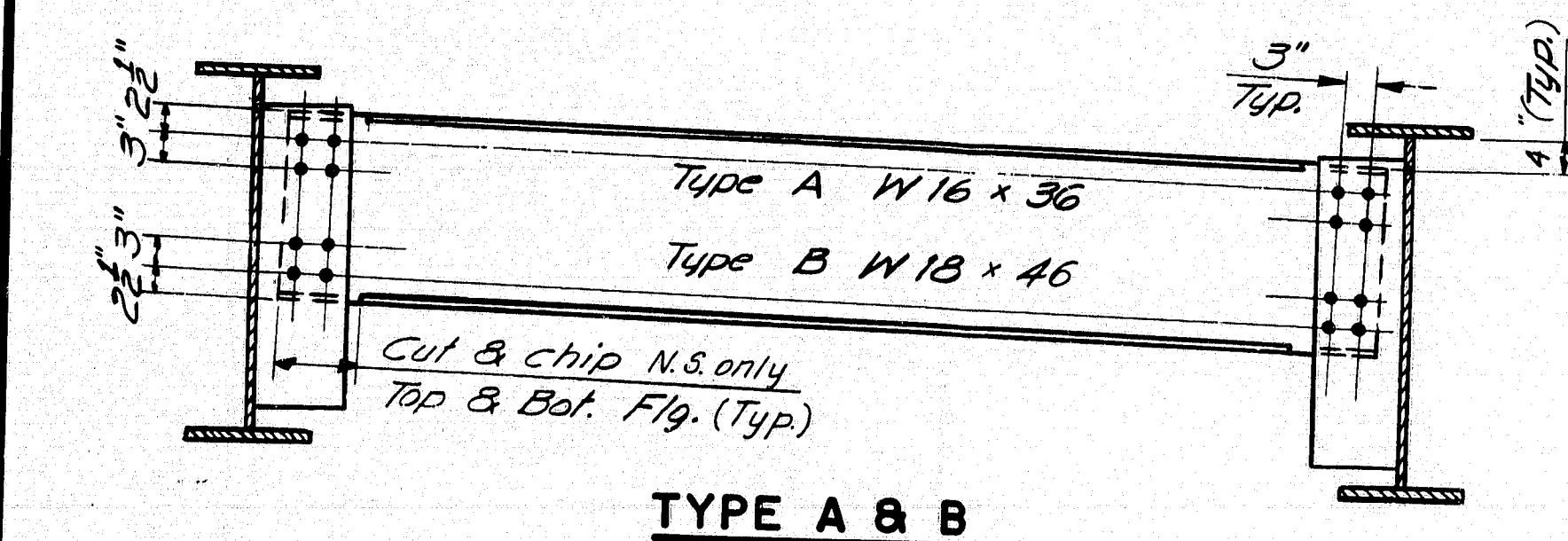
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

### STANDARD DETAILS (BD 113-81) DIAPHRAGMS & CROSSFRAMES

Revised notes 2, 3, 7, & 11	1-63
REVISIONS	DATE

103-166

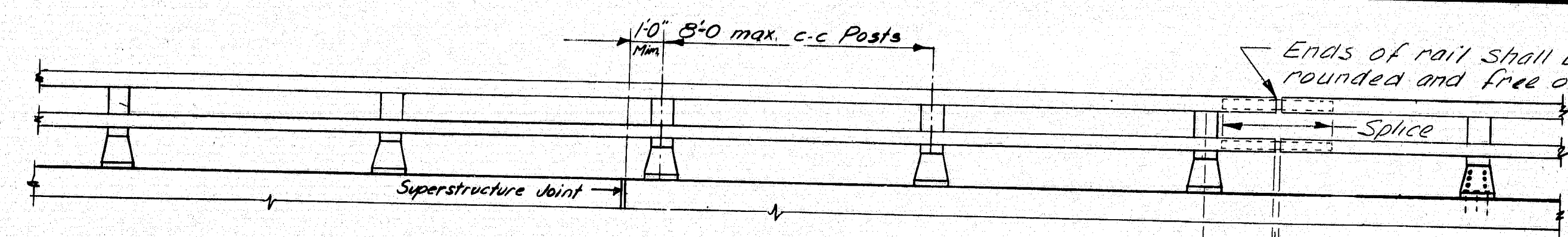
SHEET OF AUGUSTA, MAINE JUNE 1981



DESIGN - DETAILED	DATE
CHECKED	BY
REVISIONS	
FIELD CHANGES	
PLANS	

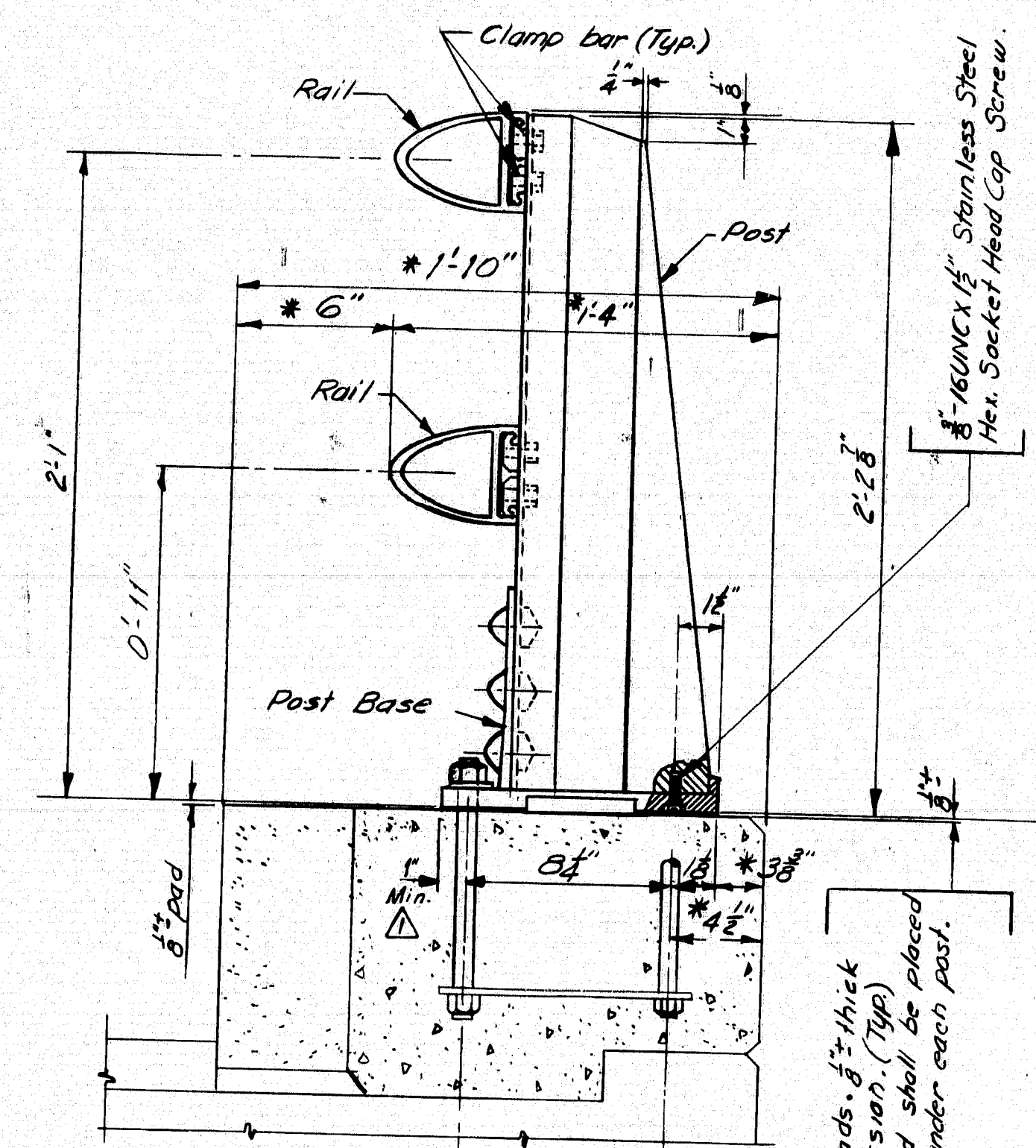


F.R.W. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(87)	72	84



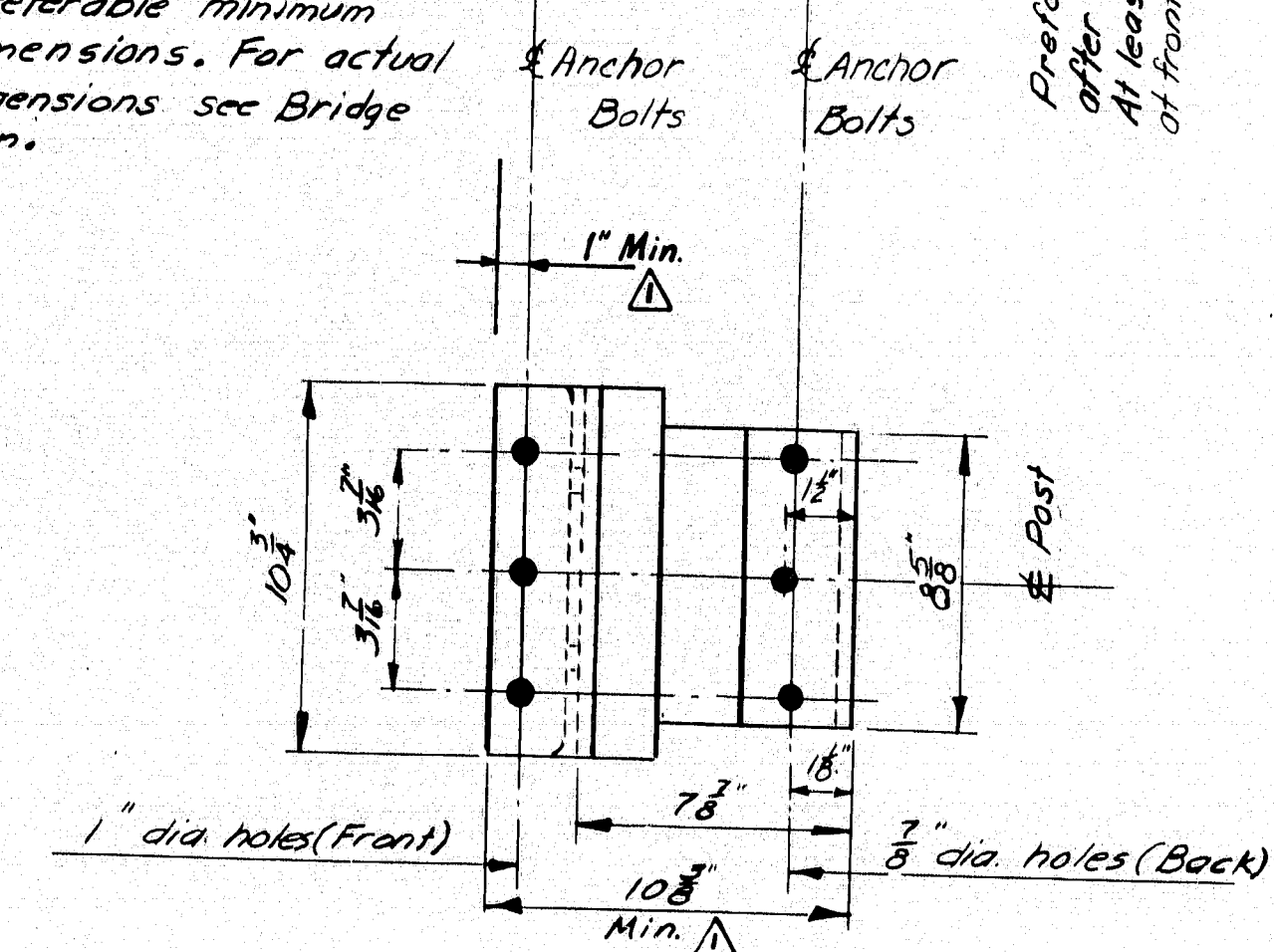
**RAILING - ELEVATION**

Lengths of rail shall be attached to a minimum of four (4) rail posts wherever possible, and in any case never less than two (2). Rail posts are to be set normal to grade unless otherwise shown on the Bridge Plans.

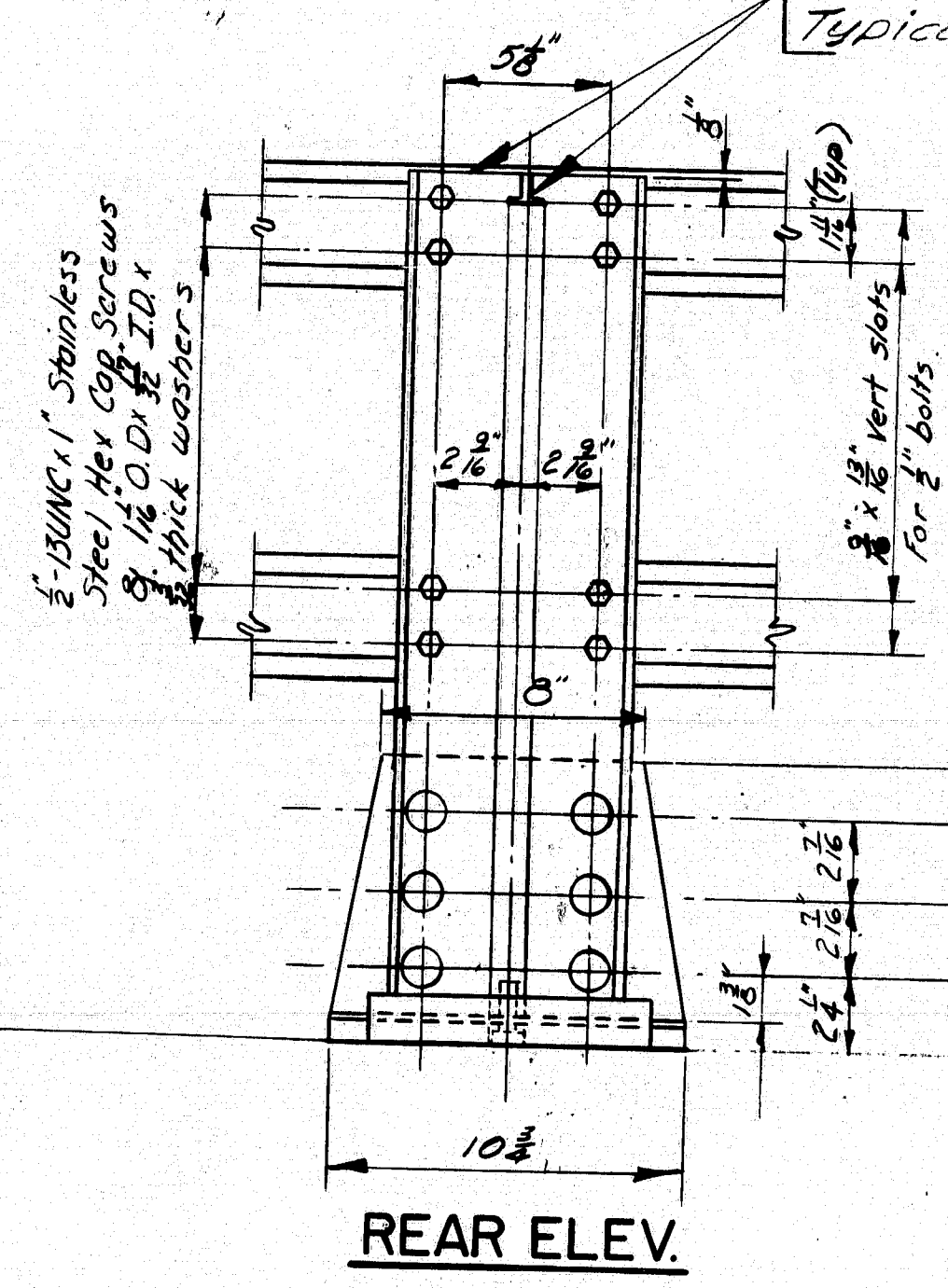


**BRIDGE RAILING (Assembly)**

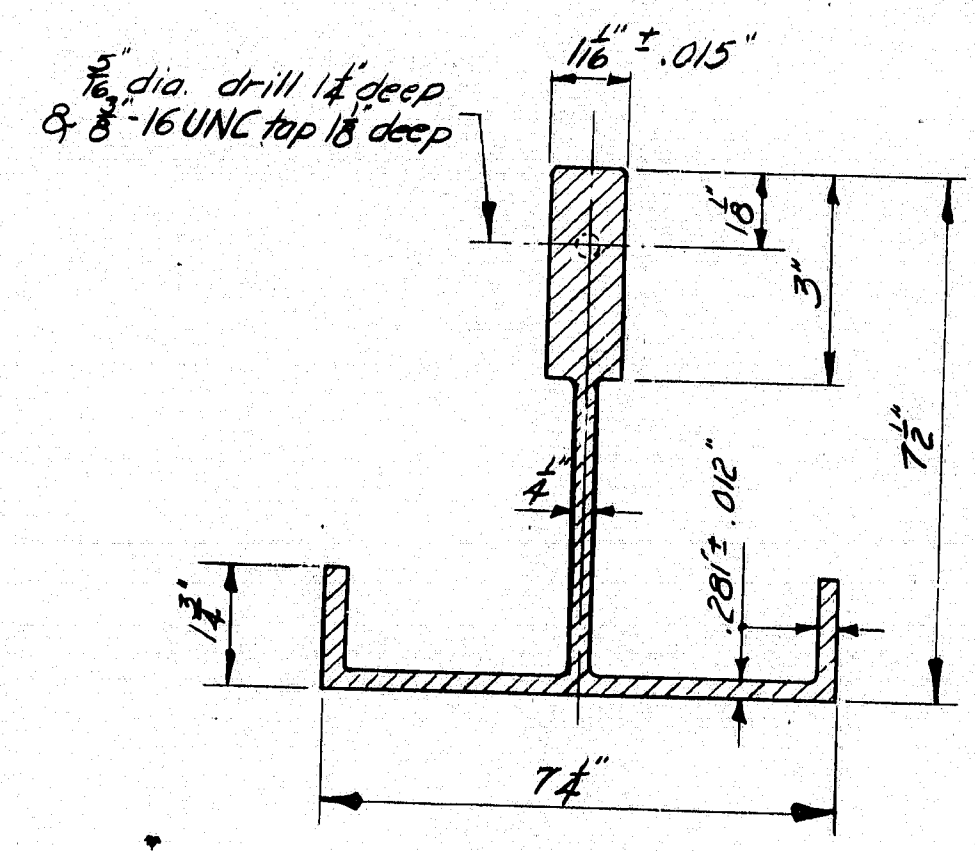
\* Preferable minimum dimensions. For actual dimensions see Bridge Plan.



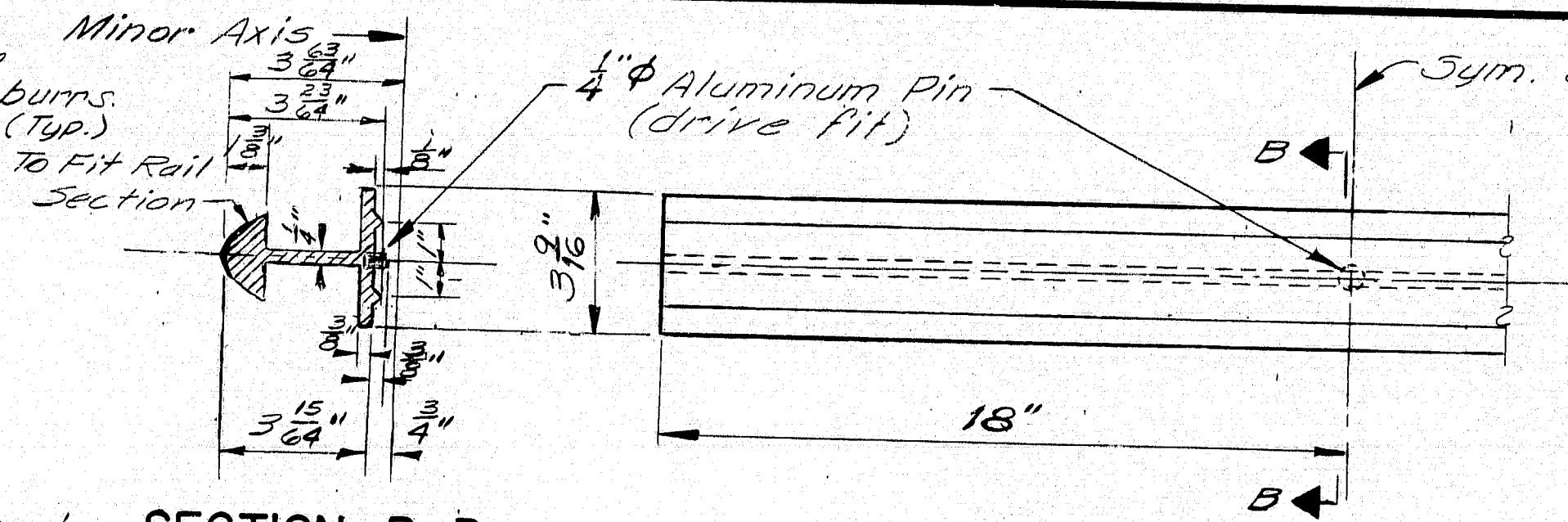
**POST BASE (Bottom View)**



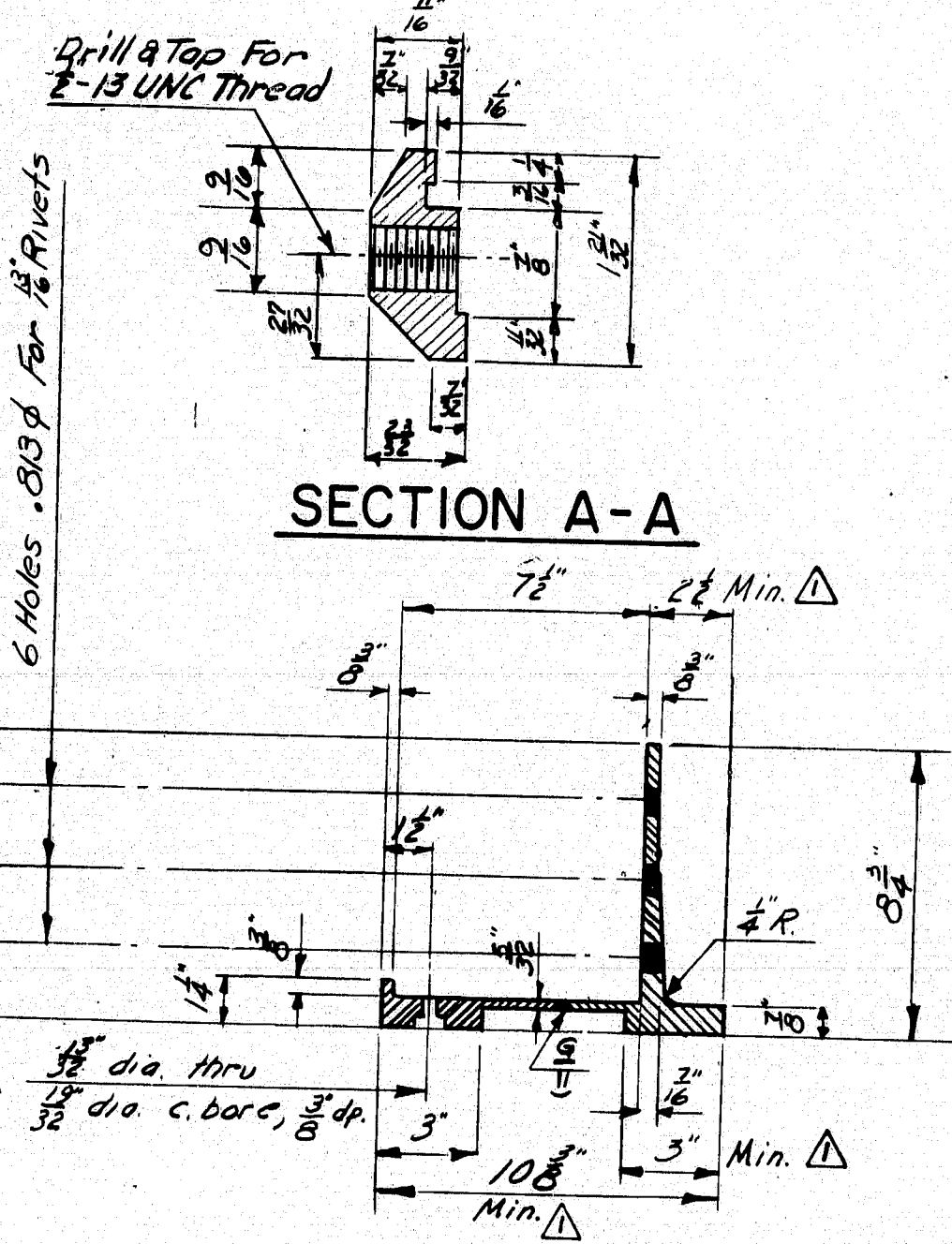
**REAR ELEV.**



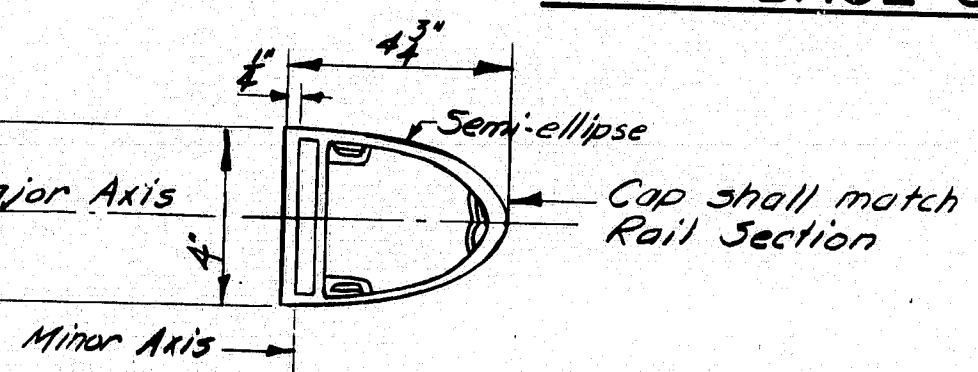
**POST SECTION**



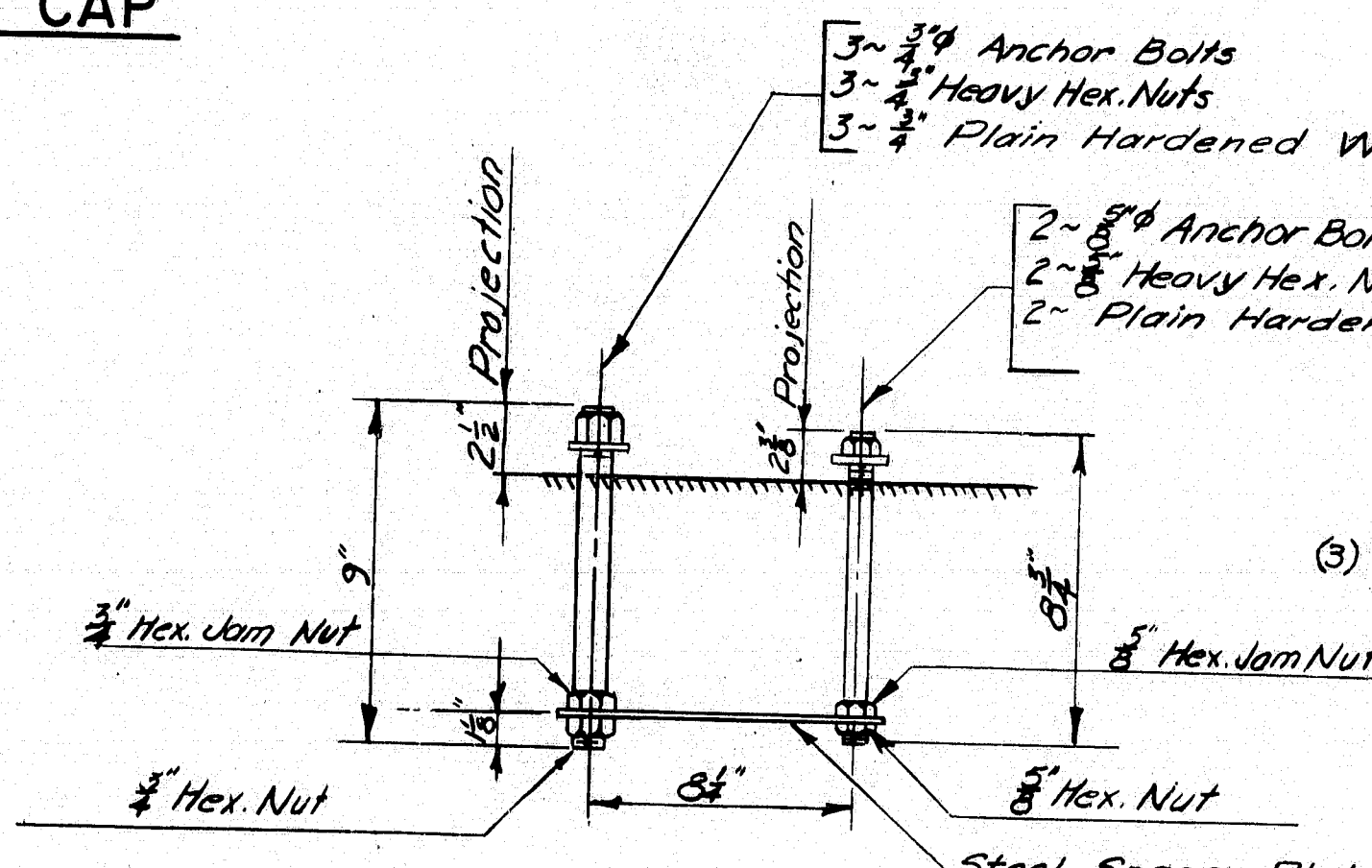
**SECTION B-B**



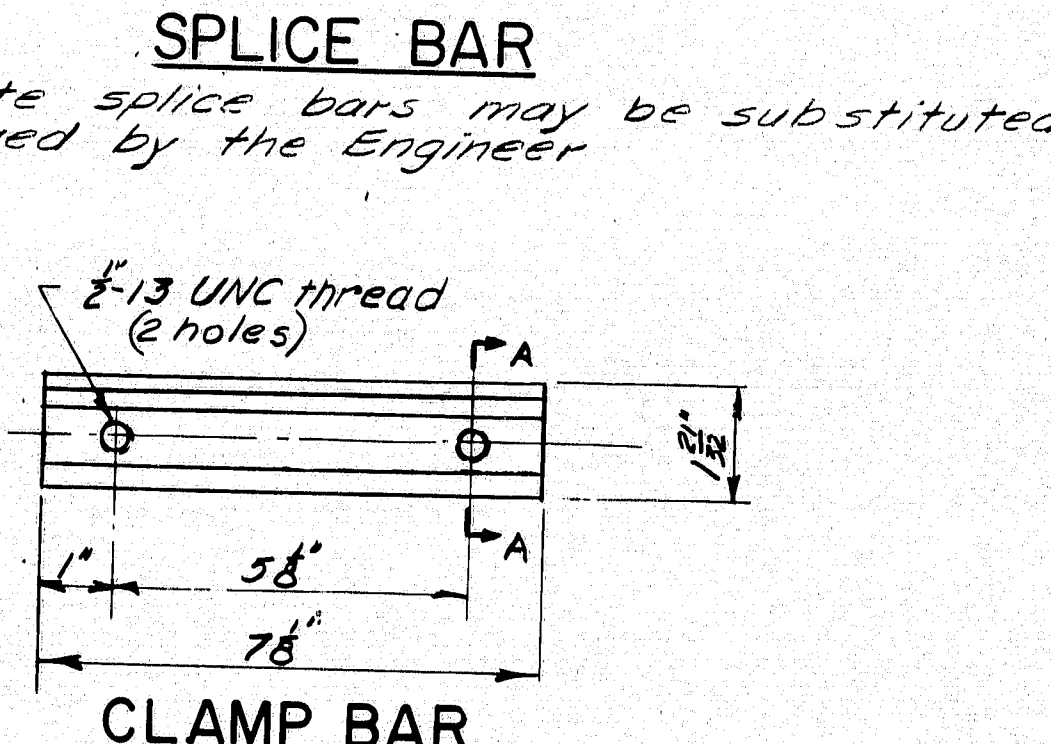
**SECTION A-A**



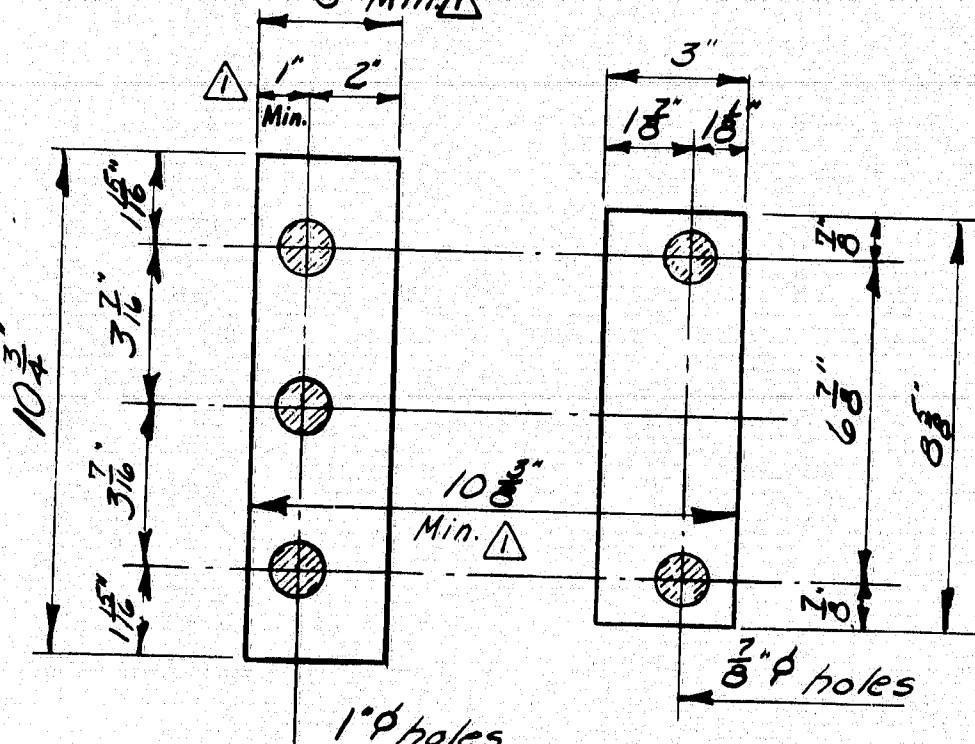
**RAIL CAP**



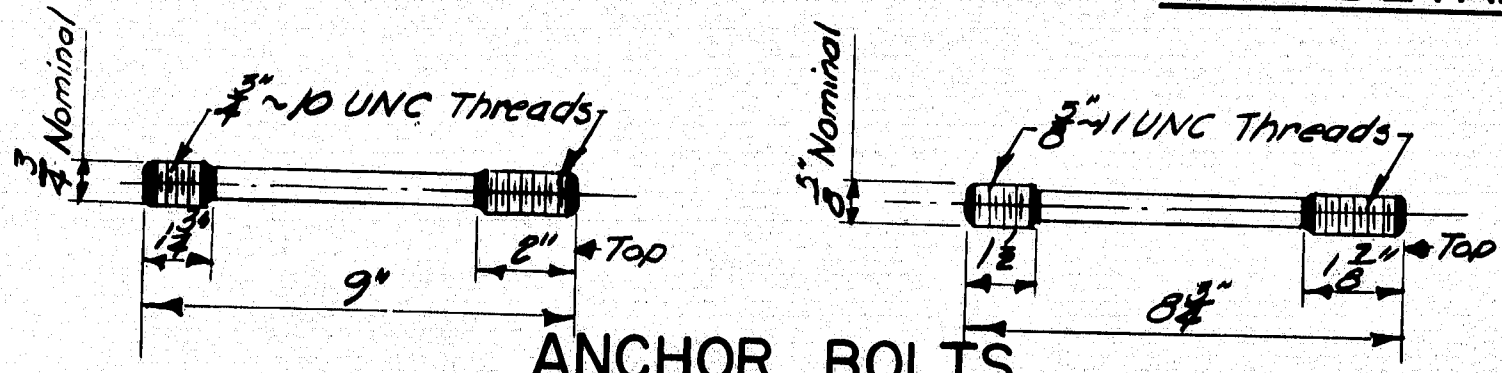
**RAIL POST ANCHORAGE (Assembly)**



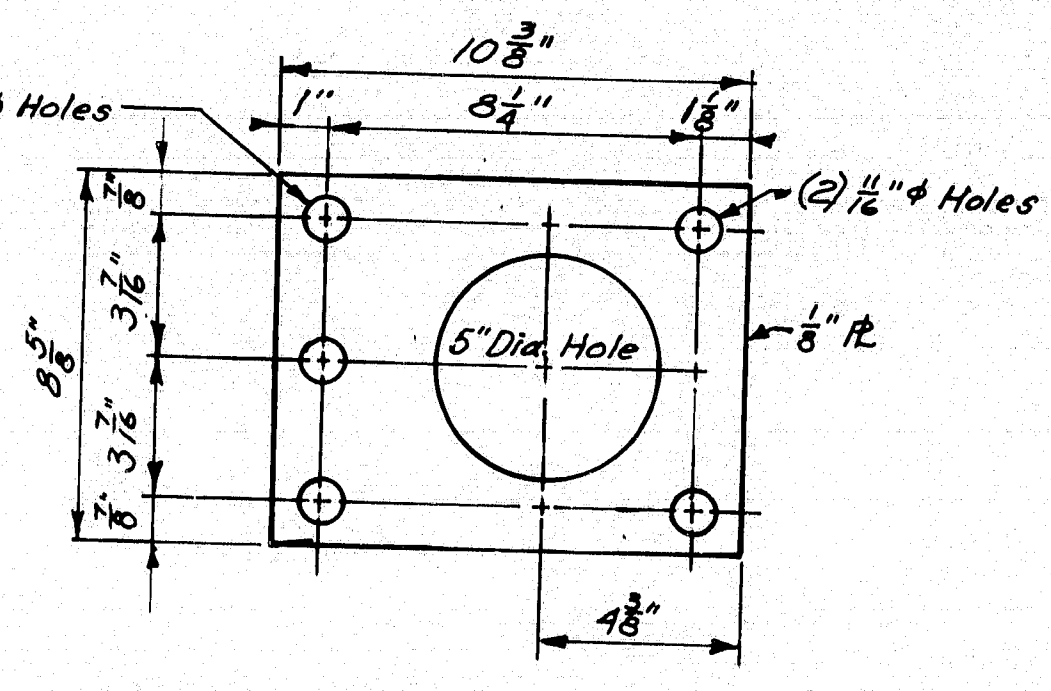
**CLAMP BAR**



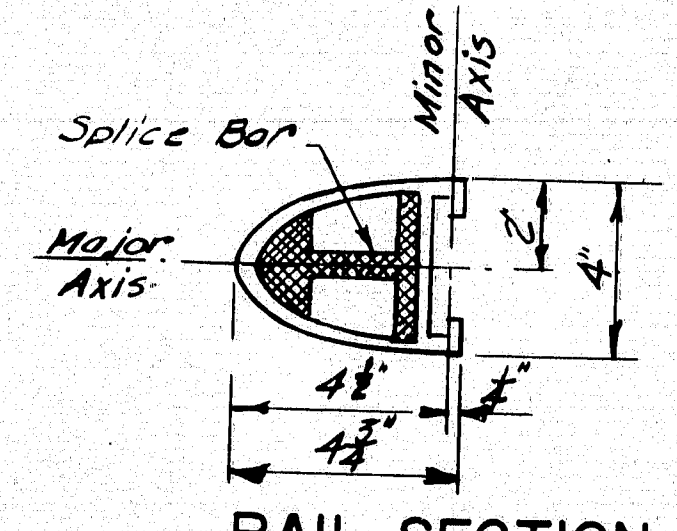
**PREFORMED PADS**



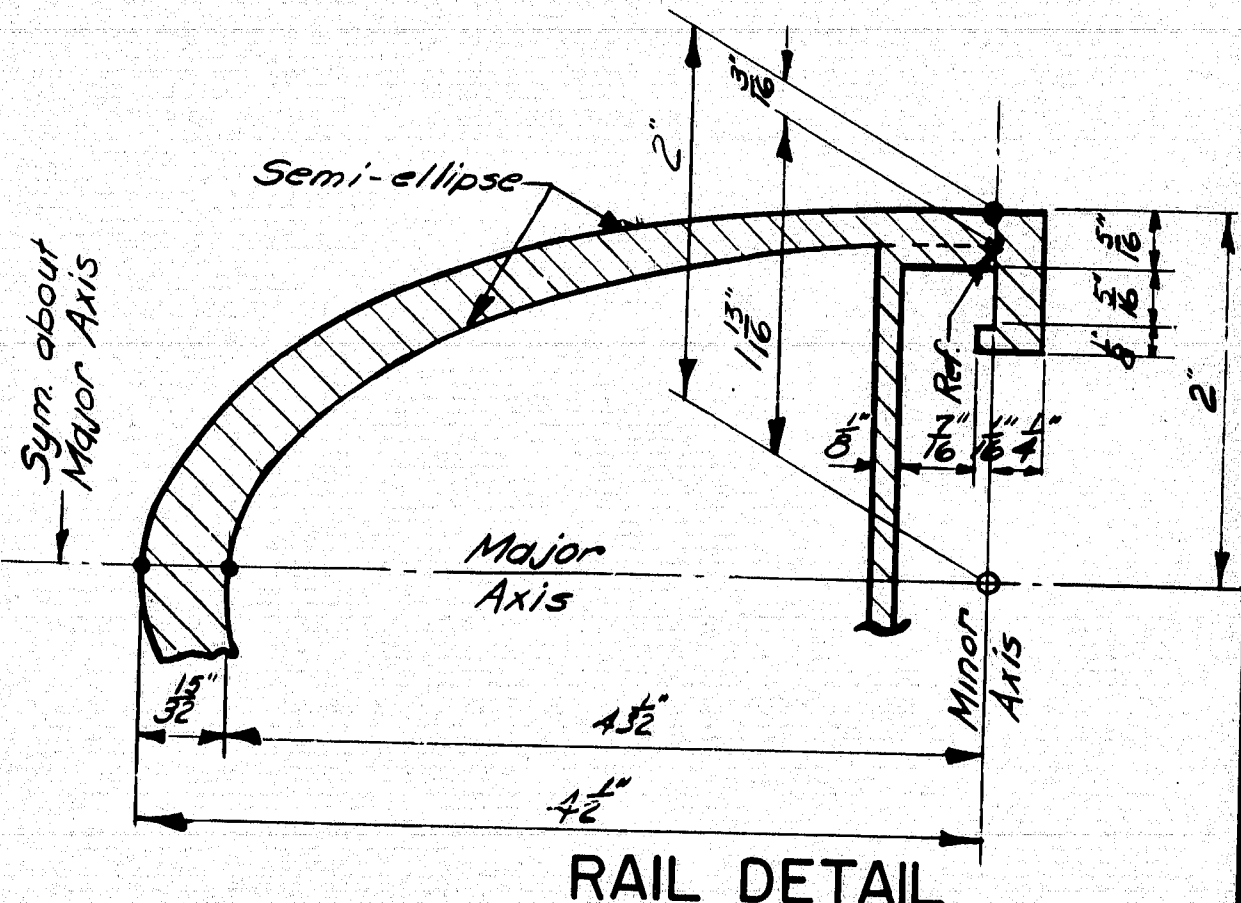
**ANCHOR BOLTS**



**STEEL SPACER PLATE (For Anchorage)**



**RAIL SECTION**  
See "Rail Detail"



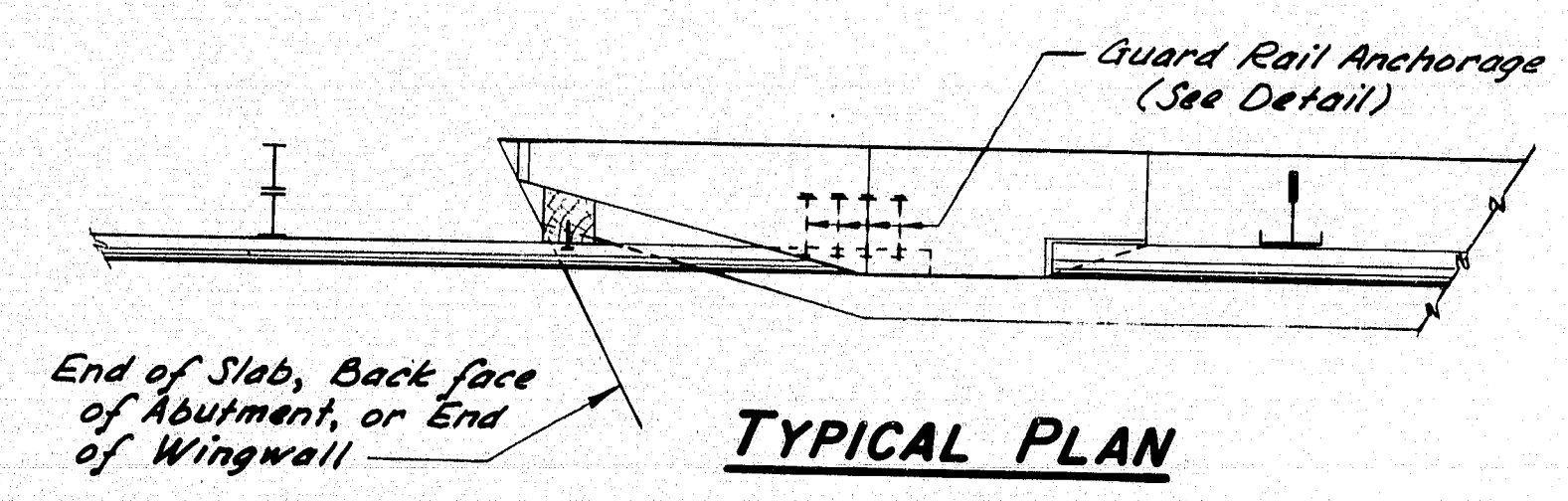
**RAIL DETAIL**

Altered base dimensions	7-83
<b>REVISIONS</b>	DATE
STATE OF MAINE	
DEPARTMENT OF TRANSPORTATION	
<b>STANDARD DETAILS</b>	
(SD 114-81)	
<b>ALUMINUM BRIDGE RAILING</b>	
2 - BAR (SEMI-ELLIPSE)	
<b>103-167</b>	
SHEET OF	AUGUSTA, MAINE JUNE 1981

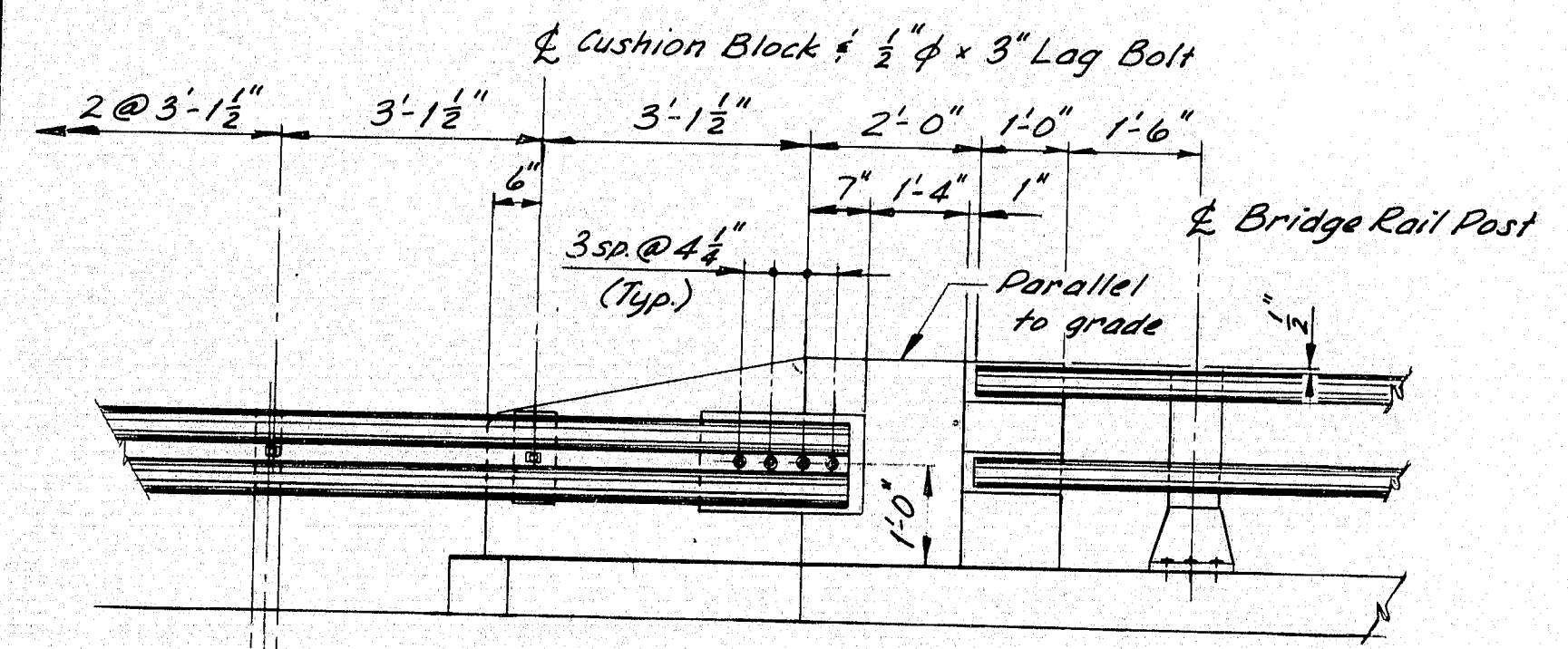
DESIGN - DETAILED	BY	DATE
CHECKED	K. Leach	Jan. 1979
REVISIONS		
FIELD CHANGES		
<b>PLANS</b>		



F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(87)	73	84

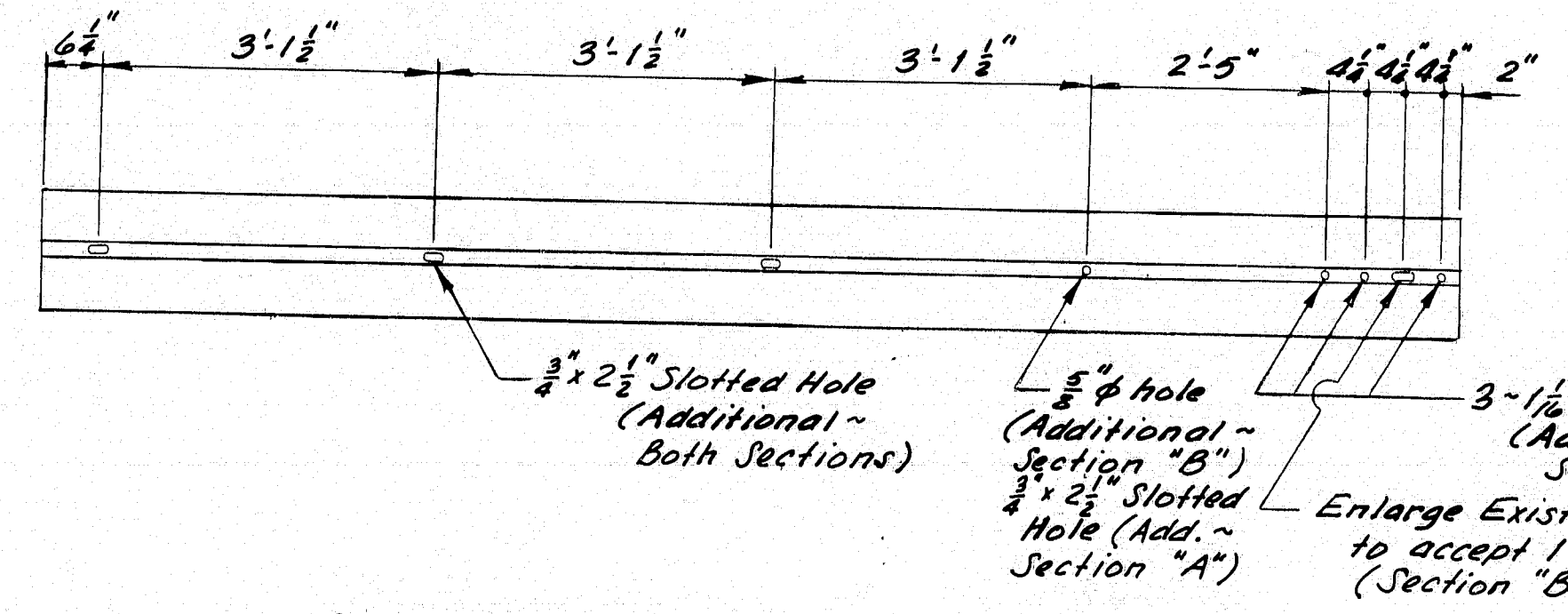


**TYPICAL PLAN**

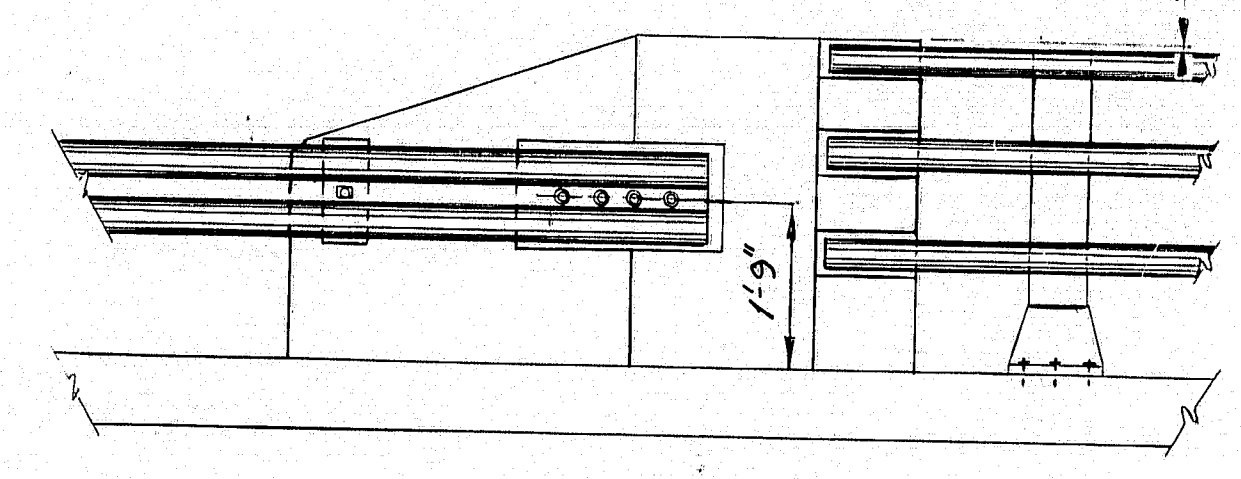


**ELEVATION**

2-Bar Bridge Rail (Aluminum or Steel)

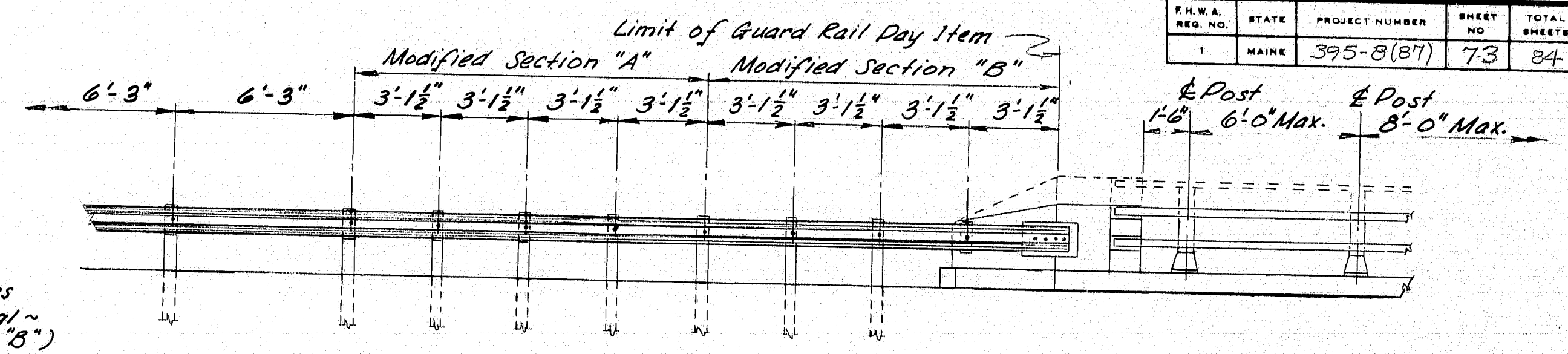


**MODIFIED GUARD RAIL SECTIONS**  
(See Note #6)

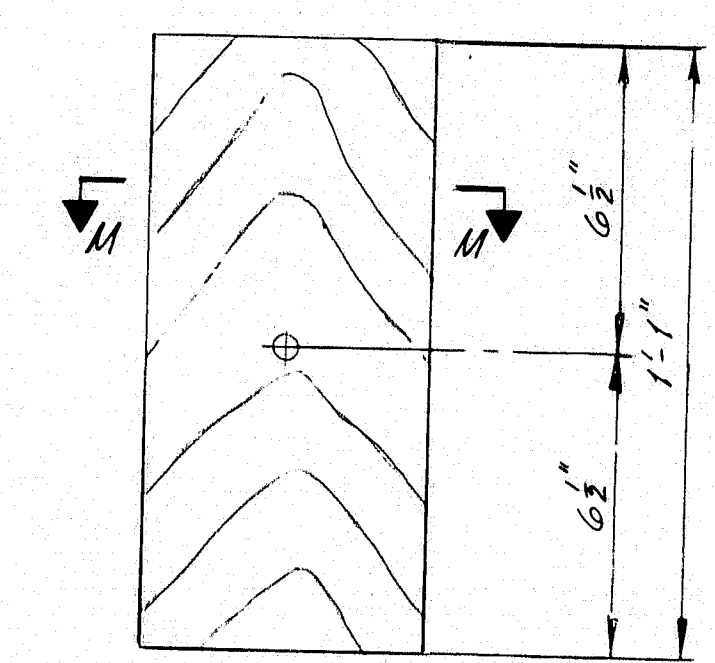


**ELEVATION**

3-Bar Bridge Rail (Aluminum or Steel)

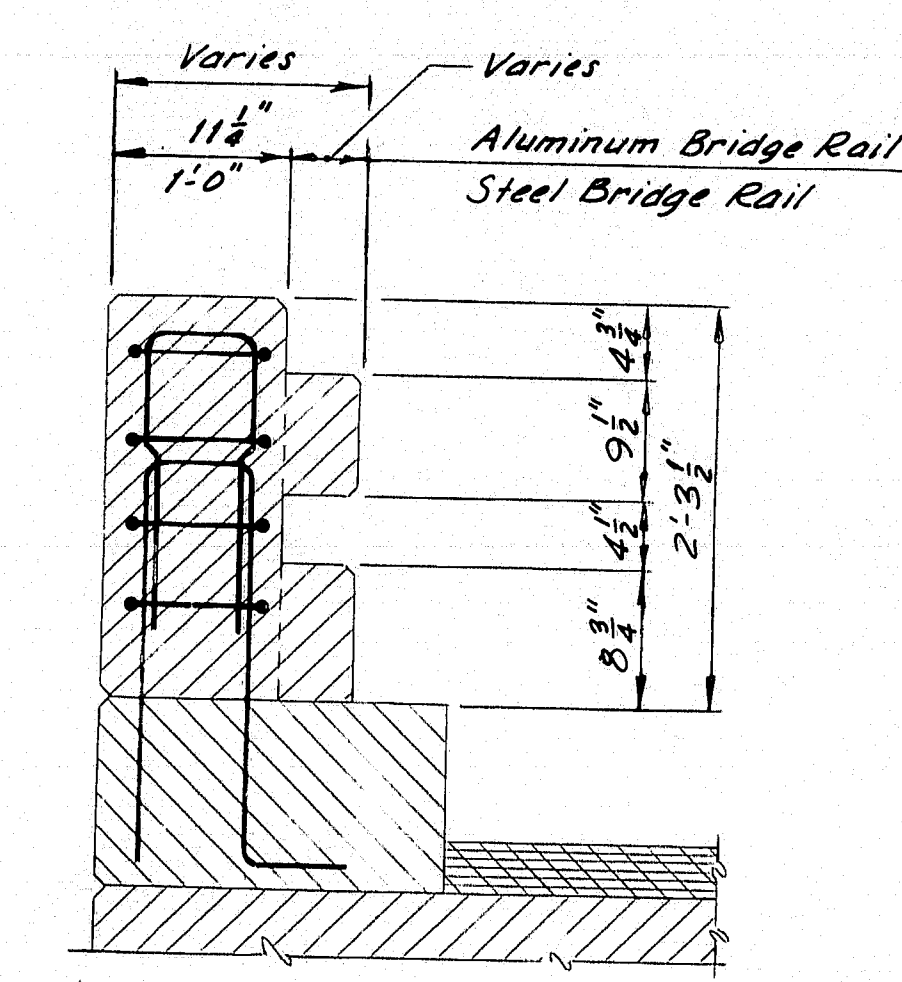


**RAILING - ELEVATION**



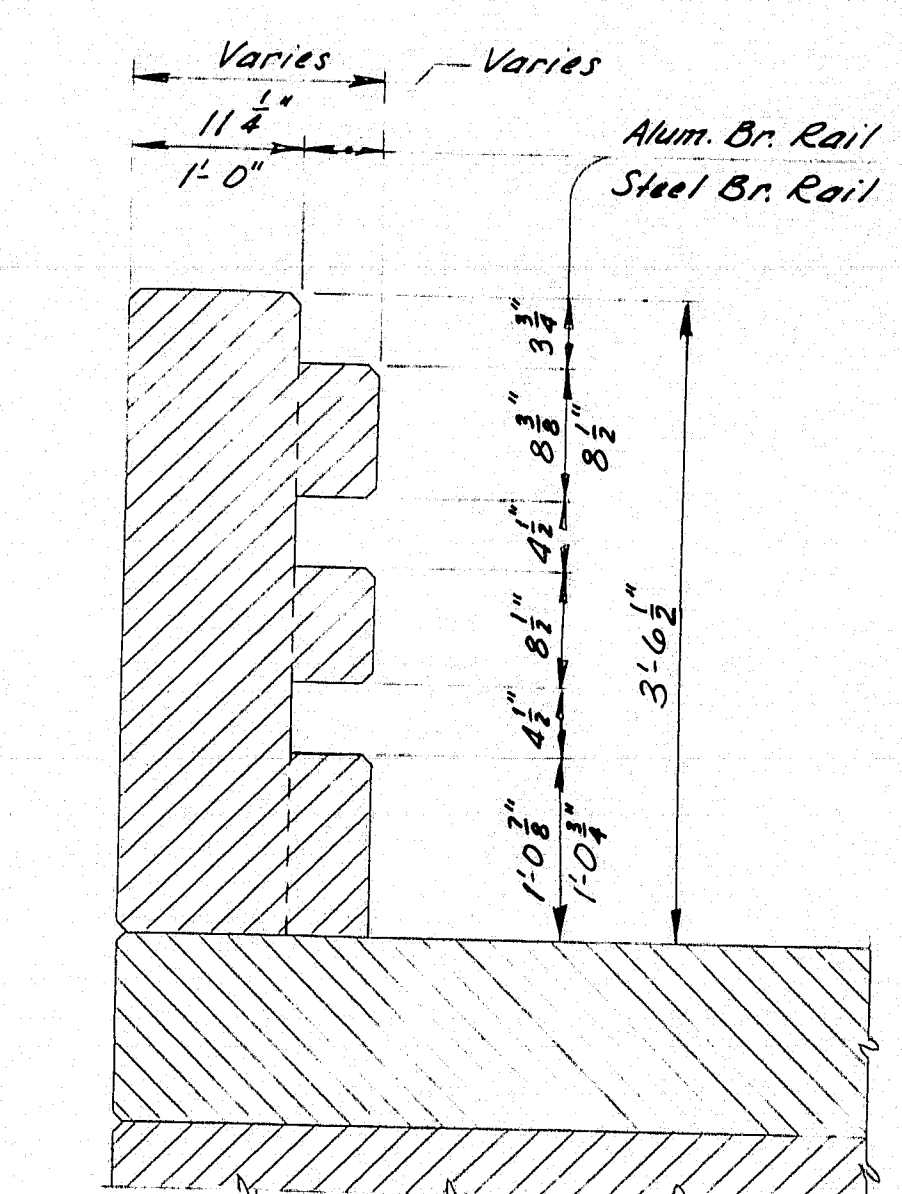
**SECTION M-M**

**CUSHION BLOCK**  
(See Note #7)



**SECTION B-B**

2-Bar Bridge Rail (Aluminum or Steel)



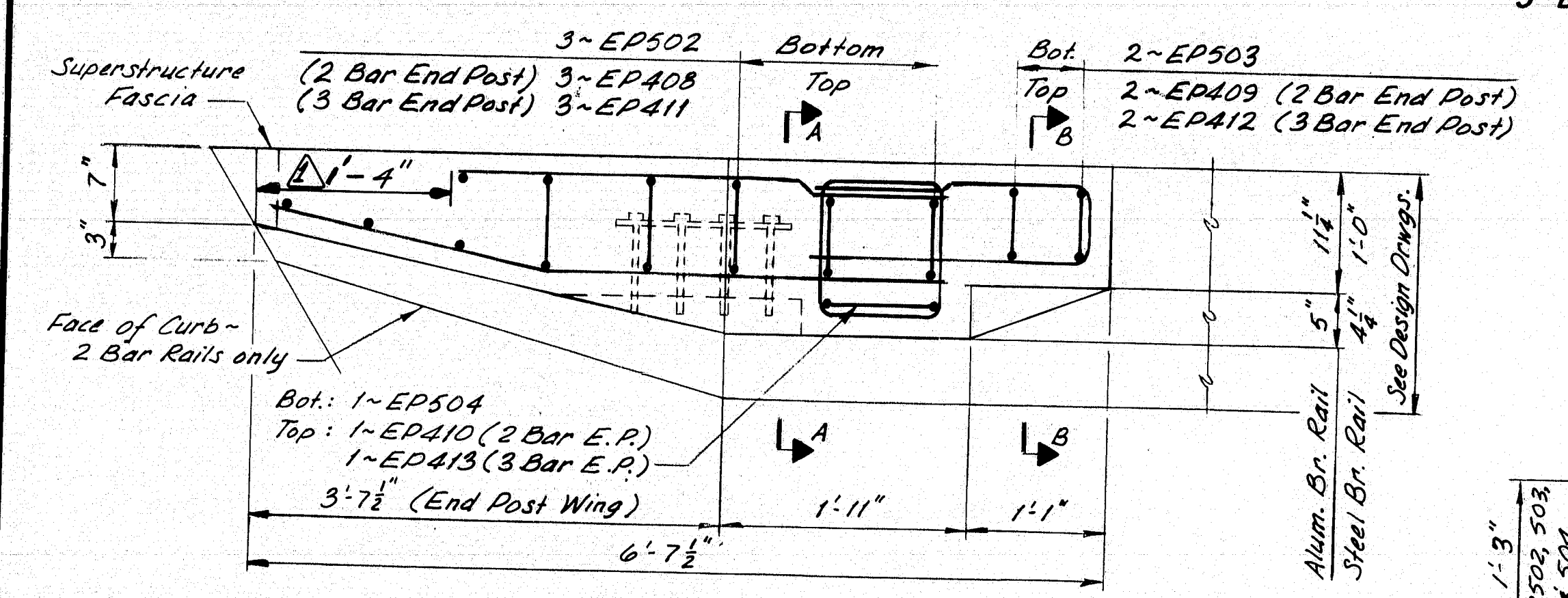
**SECTION B-B**

3-Bar Bridge Rail (Aluminum or Steel)

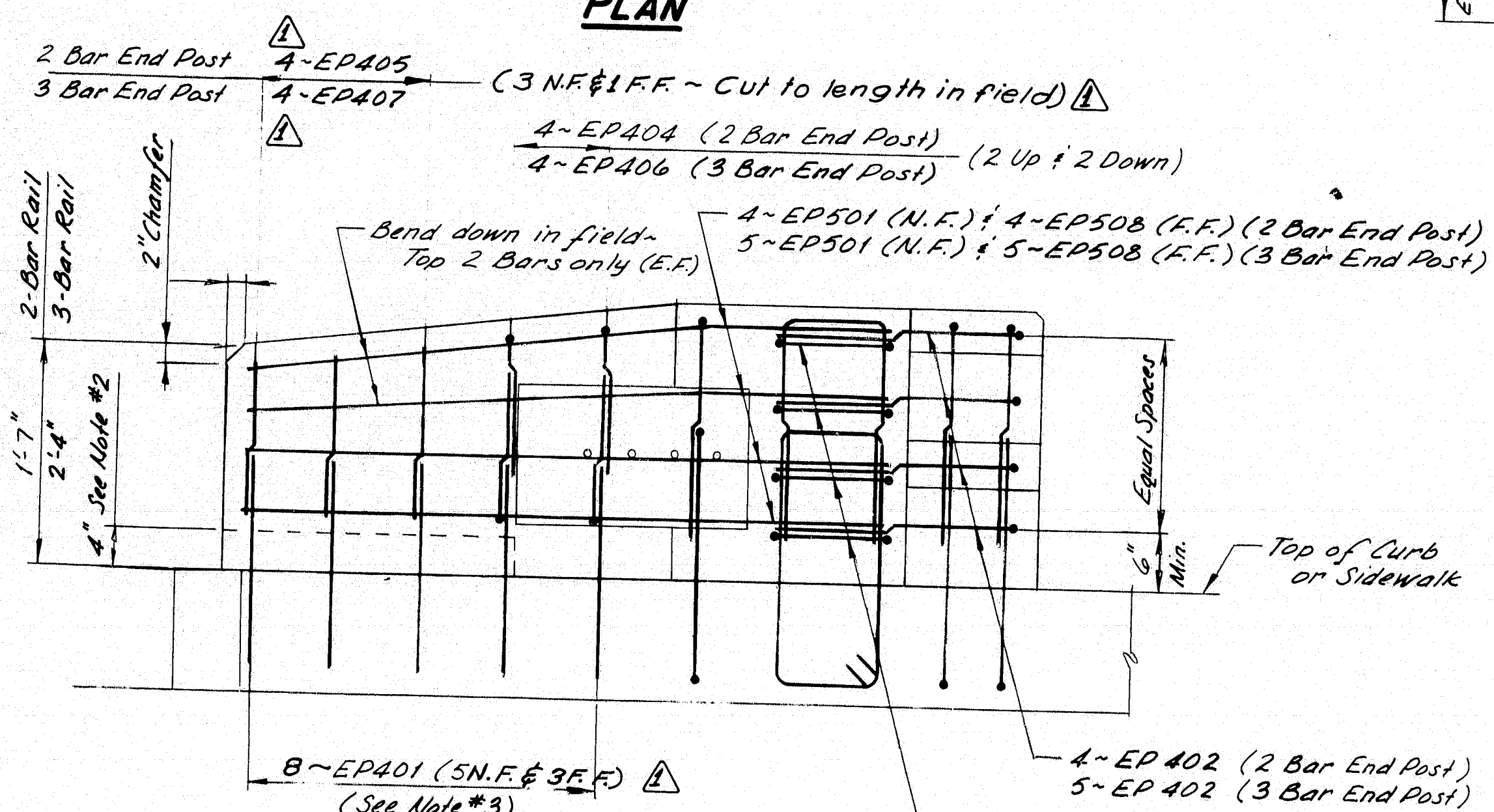
**LEGEND**

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

- 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  $\frac{3}{4}$ " anchor bolts shall be incidental to Guard Rail Pay Items. Nuts shall conform to A.S.T.M. A563, 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.

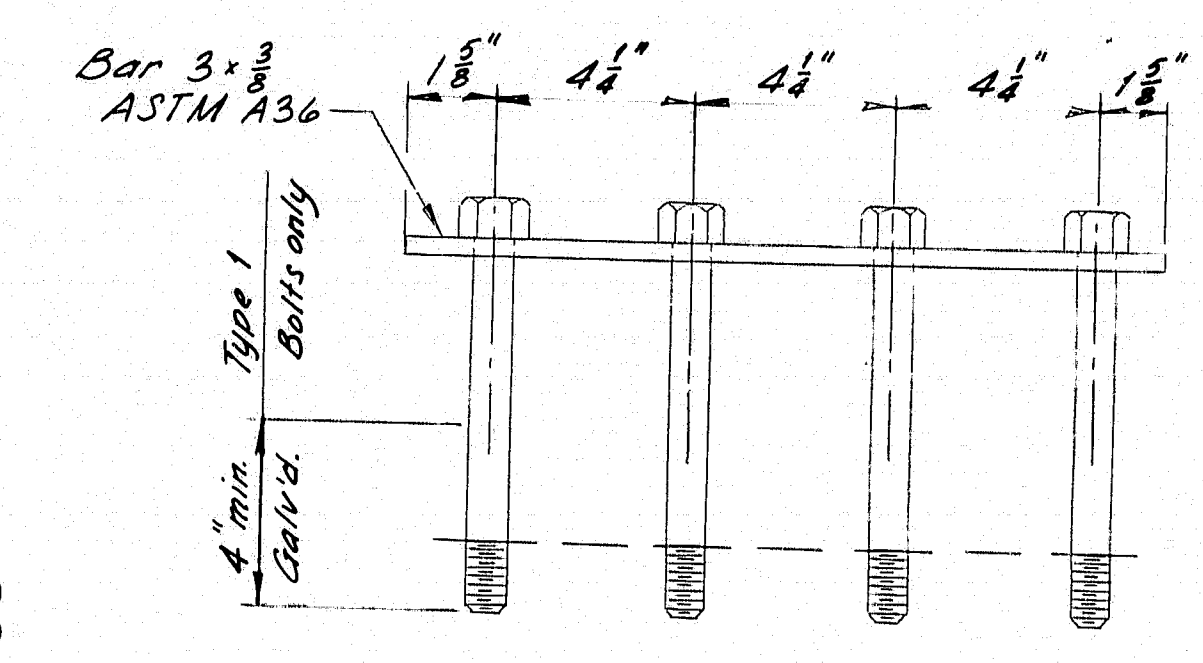


**PLAN**

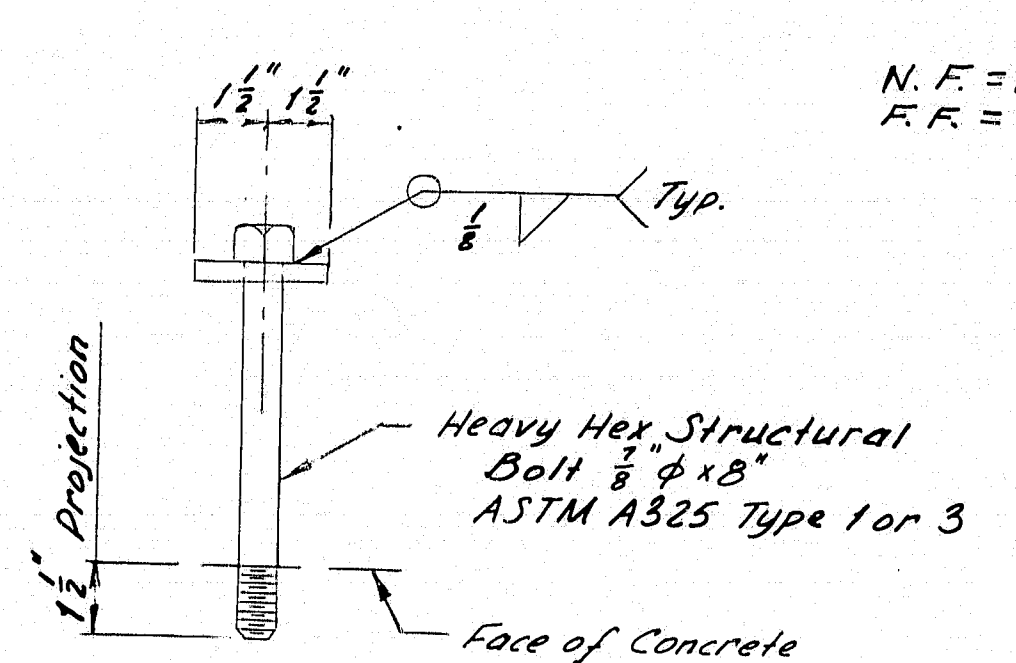


**ELEVATION**

**SECTION A-A**



**GUARD RAIL ANCHORAGE**



**VIEW N-N**

PROJECT DESIGN ENGINEER	DATE
BY	12/28/87
DESIGN - DETAIL	
CHECKED	
REVISIONS	
FIELD CHANGES	

REVISIONS	DATE
General Revisions 1-83	

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

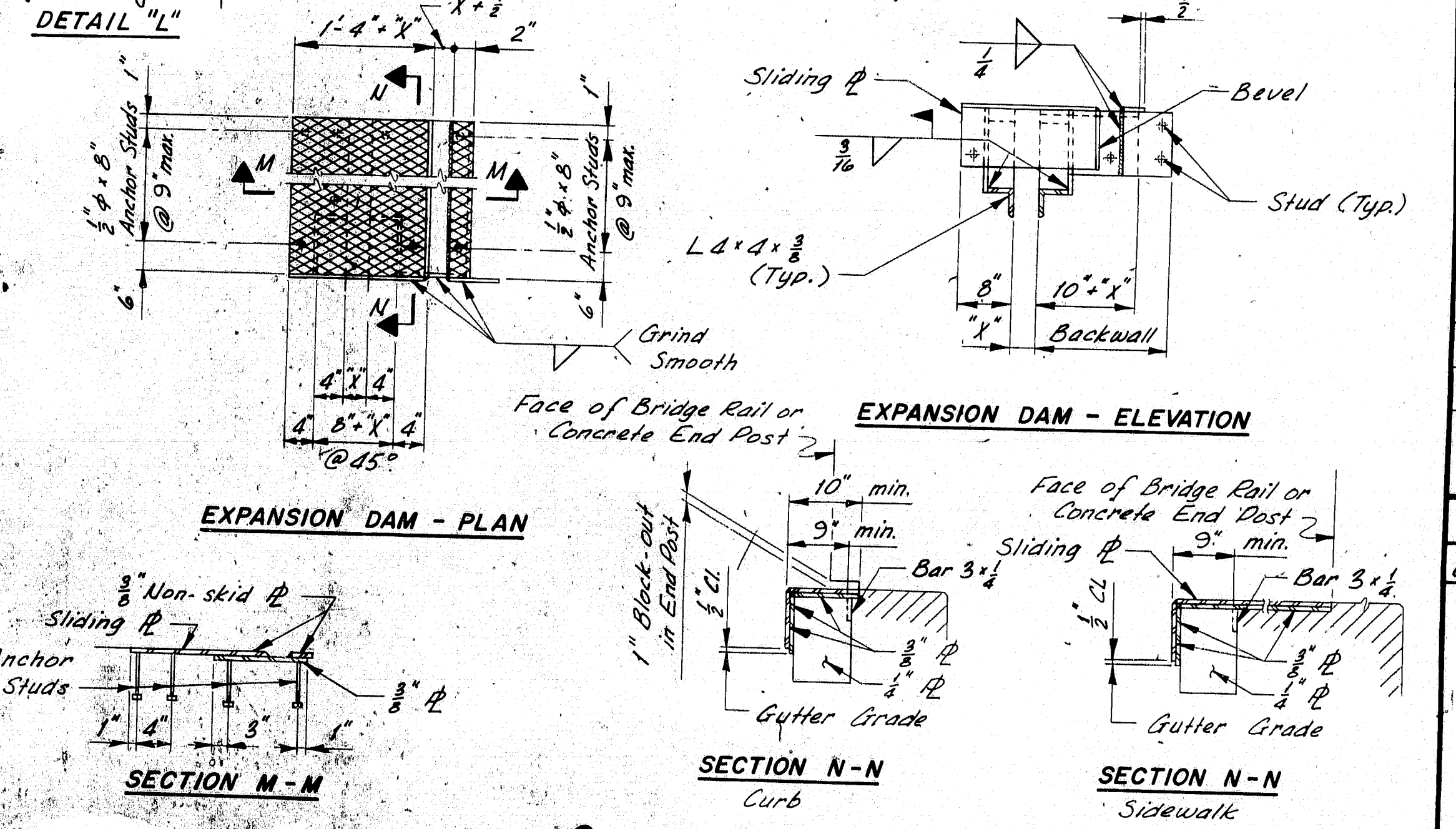
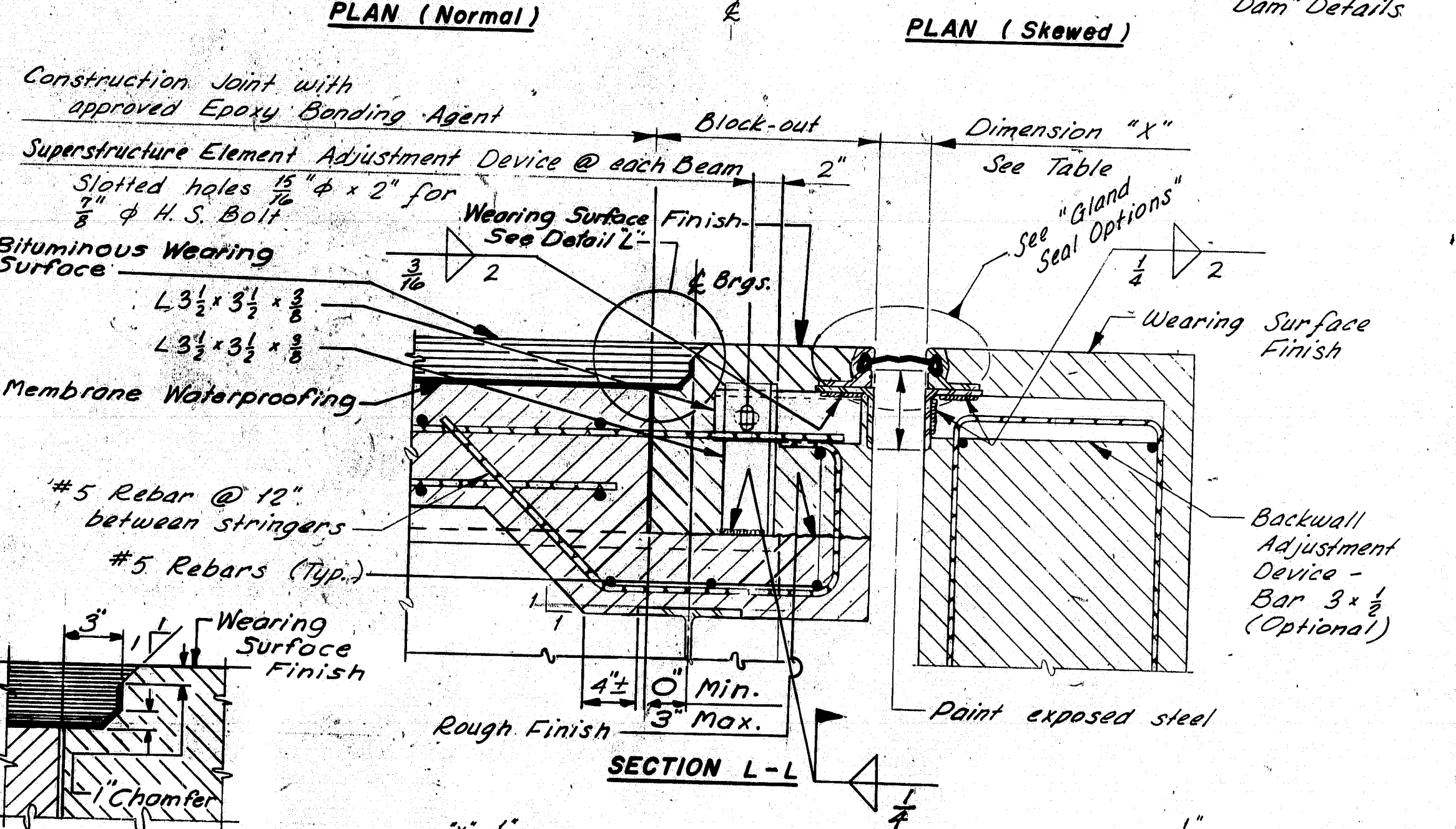
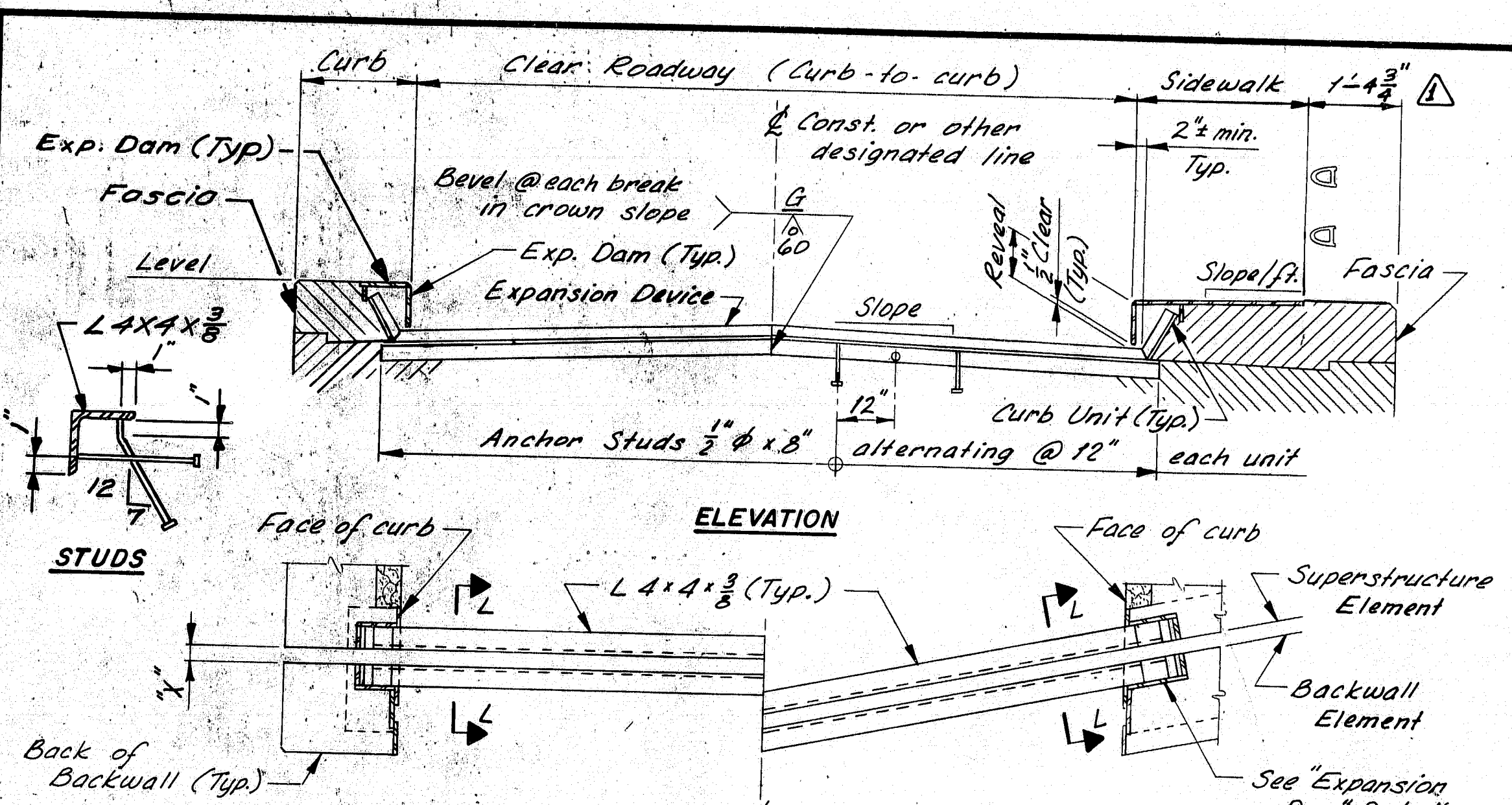
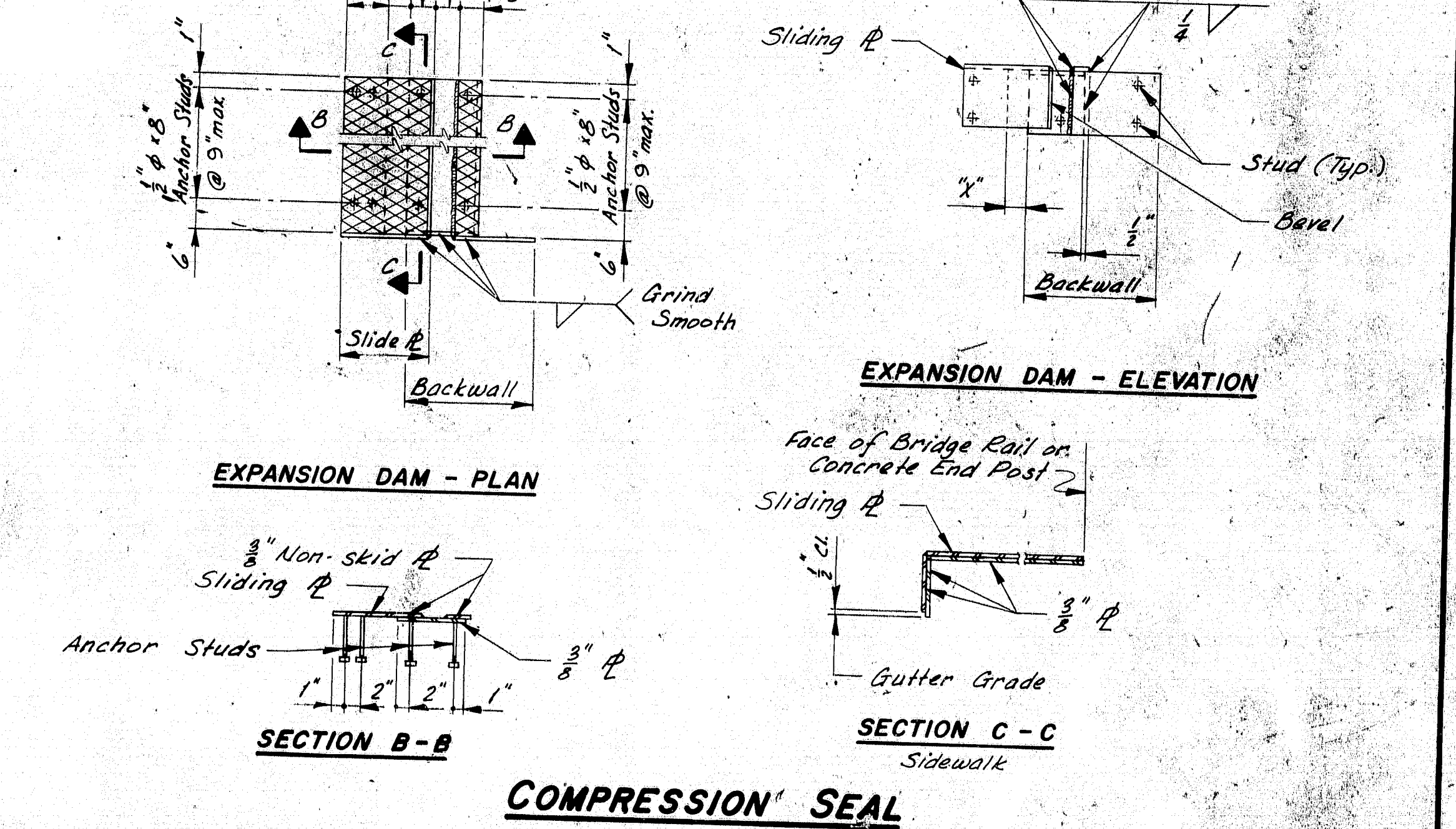
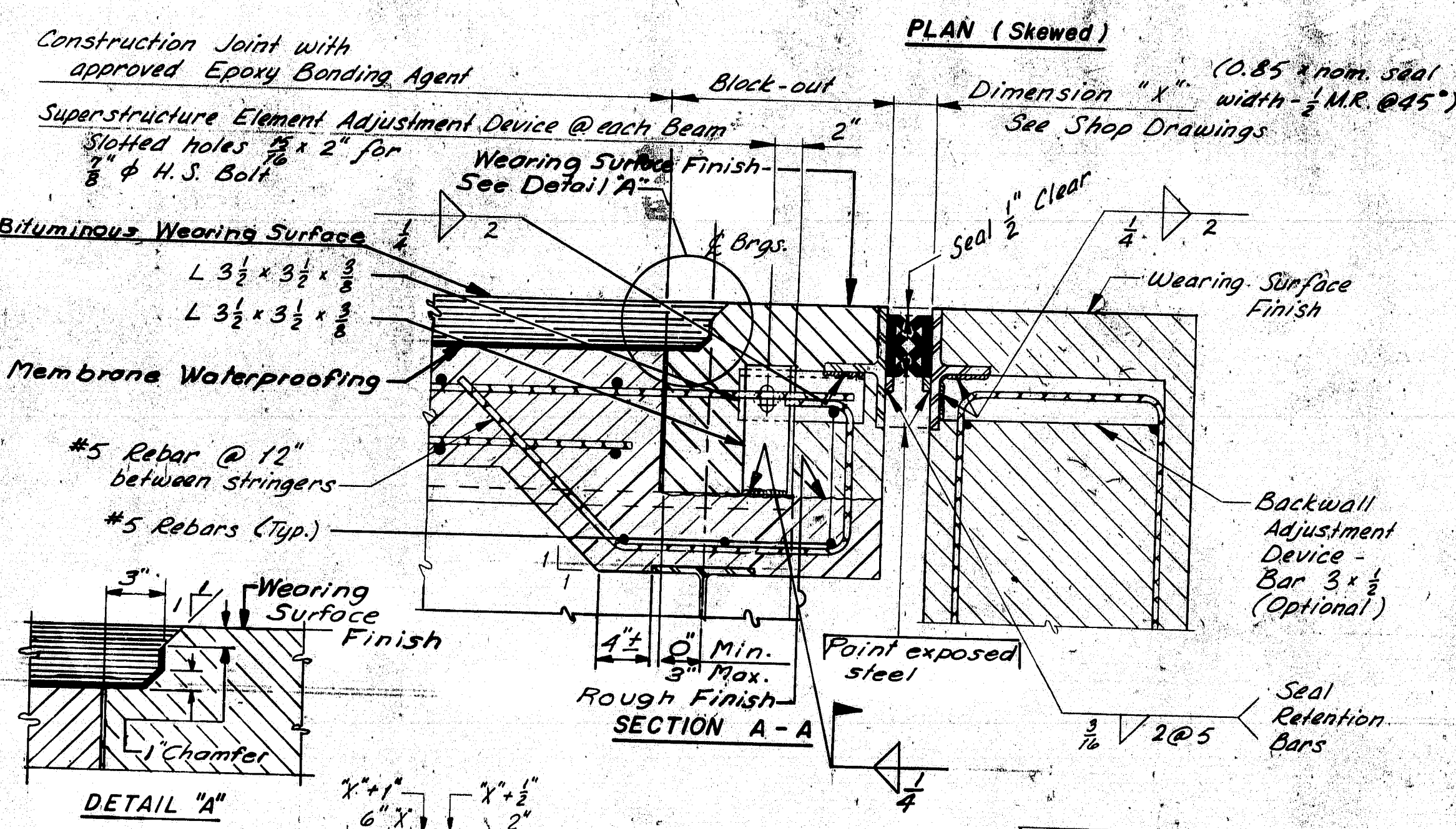
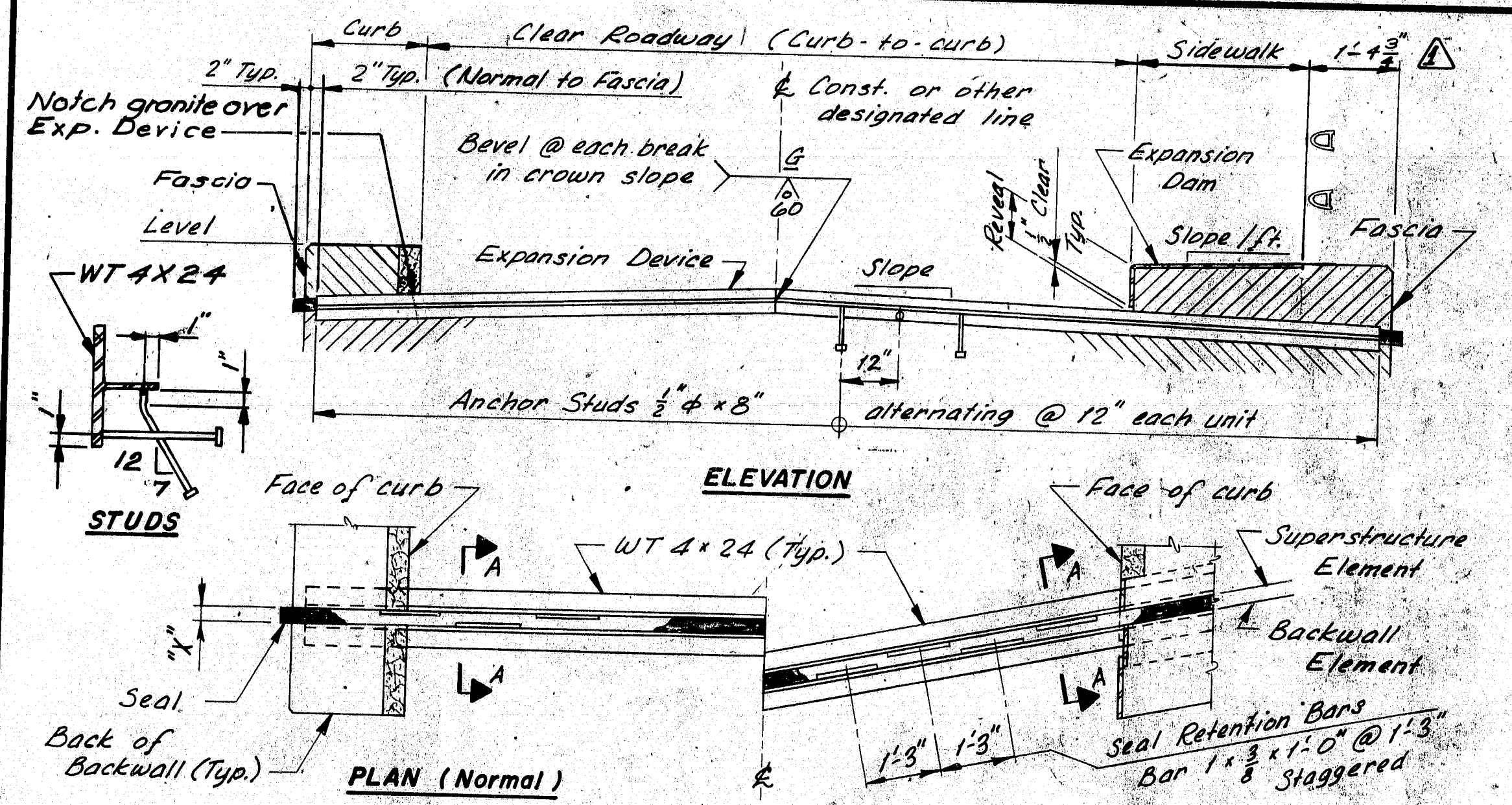
**STANDARD DETAILS**  
(BD 120 - 81)

**CONCRETE END POSTS**  
**183-168**

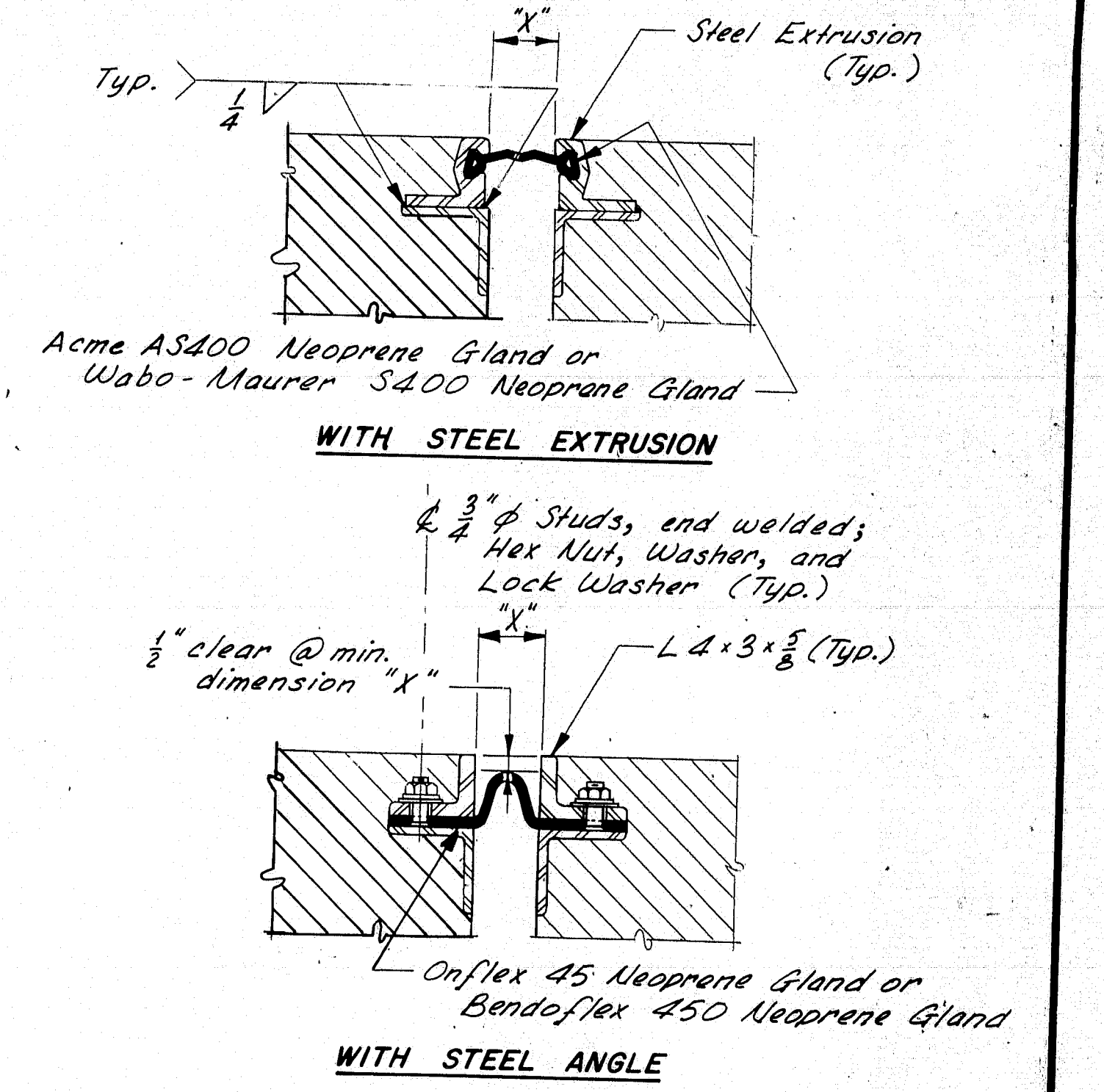
SHEET OF AUGUSTA, MAINE JUNE 1981



PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	12/1/81
CHECKED	12/1/81
REVISIONS	
FIELD CHANGES	



- NOTES:**
- Each Expansion Device Unit consists of one pair of matching Elements and Expansion Dams as required. At joints over Piers, two Superstructure Elements shall be used.
  - Welding to reinforcing steel will be allowed in the top 1'-6" of the Abutment backwall.
  - See Design Drawings for dimensions, slopes, skew, and all other information necessary to fabricate and install the units. Expansion Devices shall be installed normal to grade.
  - The concrete in the Superstructure Adjustment Device Block-out may be placed with the Sidewalk, and Curb Concrete.



GLAND SEAL SETTING TABLE												
Total Movement Required *	Dim. "X" (Measured parallel to & of Roadway)											
	TEMPERATURE (°F)											
	120°	105°	90°	75°	60°	45°	30°	15°	0°	-15°	-30°	
1 1/2"	1"	1 1/8"	1 1/4"	1 1/2"	1 3/4"	1 7/8"	1 5/8"	2"	2 1/8"	2 1/4"	2 3/8"	2 1/2"
2"	1 1/4"	1 3/8"	1 1/2"	1 3/4"	1 7/8"	2"	2 1/8"	2 1/4"	2 3/8"	2 1/2"	2 3/4"	2 5/8"
2 1/2"	1 3/4"	1 7/8"	1 3/4"	1 7/8"	2"	2 1/8"	2 1/4"	2 3/8"	2 1/2"	2 3/4"	2 5/8"	2 3/4"
3"	2"	2 1/8"	2 1/4"	2 3/8"	2 1/2"	2 3/4"	2 3/8"	2 3/4"	2 3/8"	2 3/4"	2 3/8"	2 3/4"

\* Multiply expanding length of Superstructure, in feet, by .0125 in./ft.

Max. Dimension "X" allowed = 3 1/2" @ -30°F

REVISIONS	DATE	STATE OF MAINE
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REVISIONS	DATE
General Revisions	1-83

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

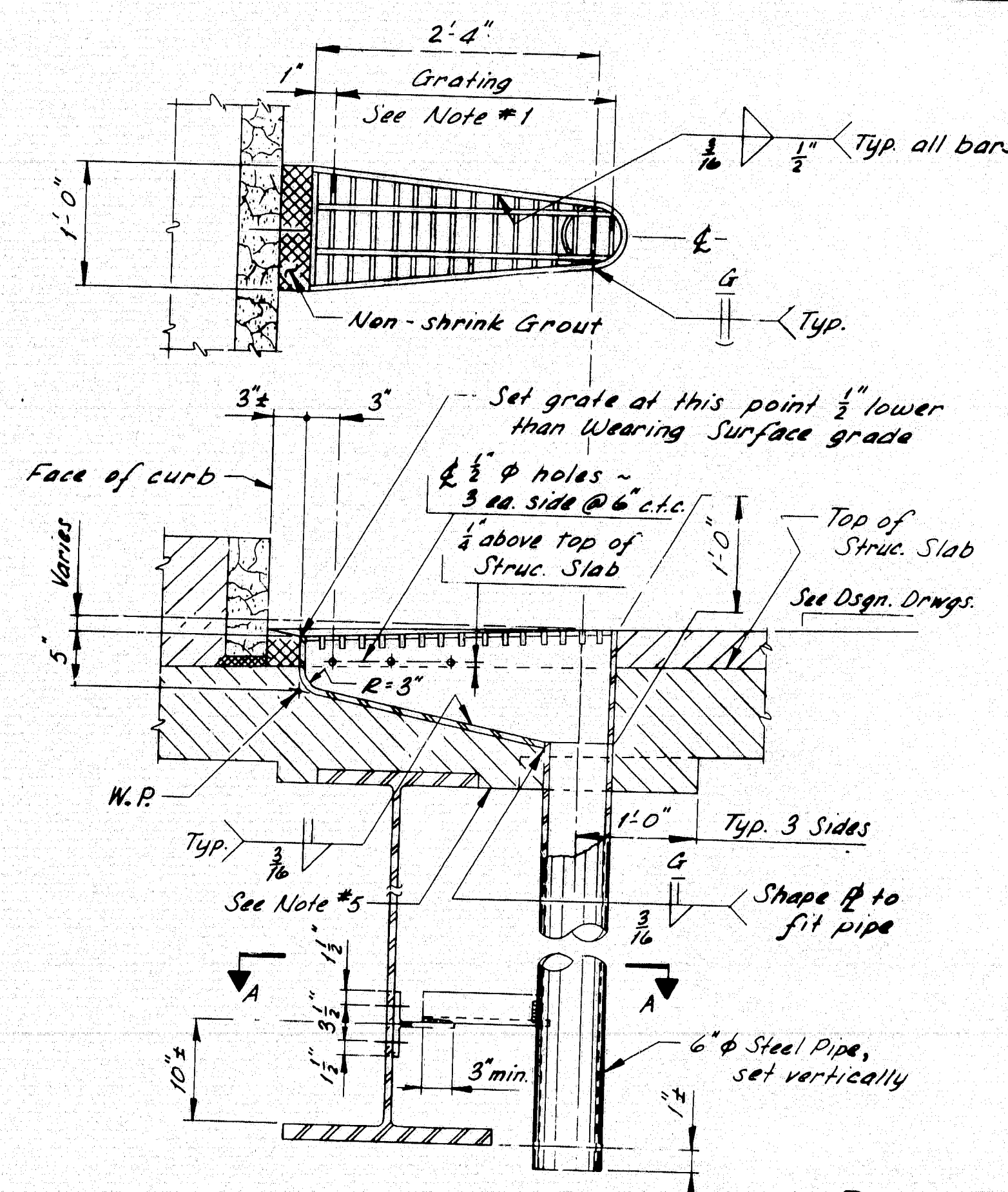
**STANDARD DETAILS**  
(BD 125 - 82)  
(FOR USE WITH BITUMINOUS WEARING SURFACE)

**EXPANSION DEVICE**  
COMPRESSION SEAL  
GLAND SEAL

103-169  
SHEET OF AUGUSTA, MAINE AUGUST 1982



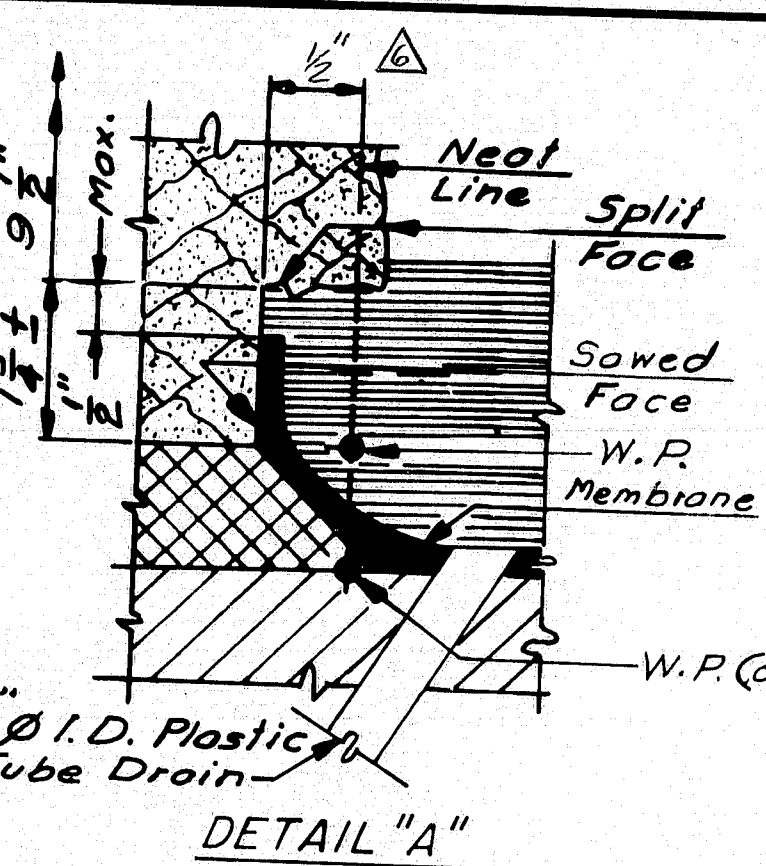
PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	10/1/81
CHECKED	10/1/81
REVISIONS	
FIELD CHANGES	
PLANS	



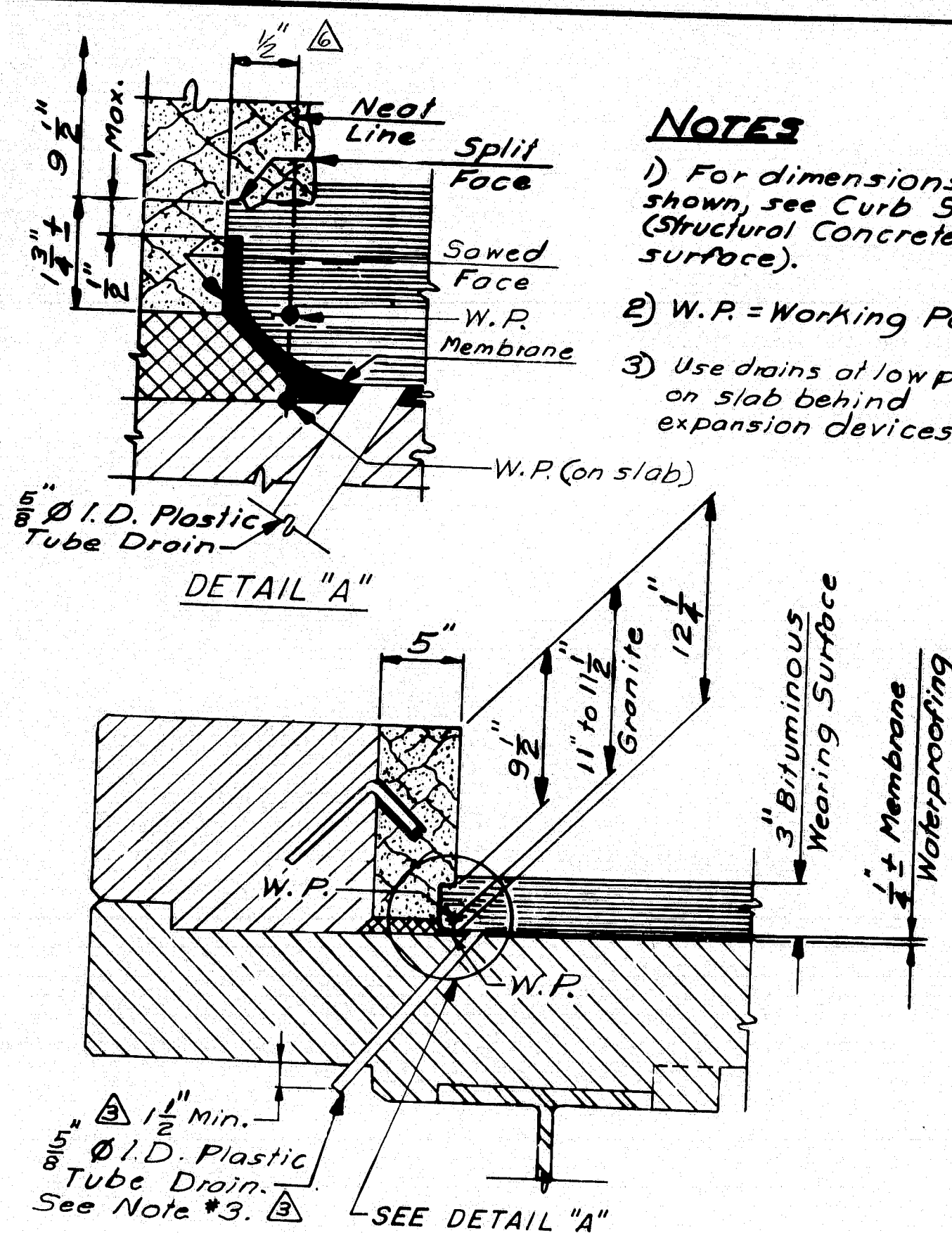
#### NOTES:

- Grating shall be a commercial heavy-duty grating with 1 1/2" x 3" bearing bars spaced at 2 3/8" c.t.c., and 3/4" x 3/4" cross bars spaced at 4" c.t.c.
- Plates shall be A.S.T.M. A36, 1/4" thick.
- WT 6x13 shall be of the same material as the beam web.
- At the option of the Contractor, the Bridge Drain may be modified to allow the use of TS 6x6x1/4 conforming to A.S.T.M. A501 or A.S.T.M. A500, Gr. "A", in place of the 6" steel pipe.
- If the minimum thickness of concrete below the Drain is 2" or less, the haunch shall be extended as shown.
- Painting will not be required when the structural steel is specified to be unpainted.
- Payment for Bridge Drain shall be as specified under subsection 502.19 of the Standard Specifications.

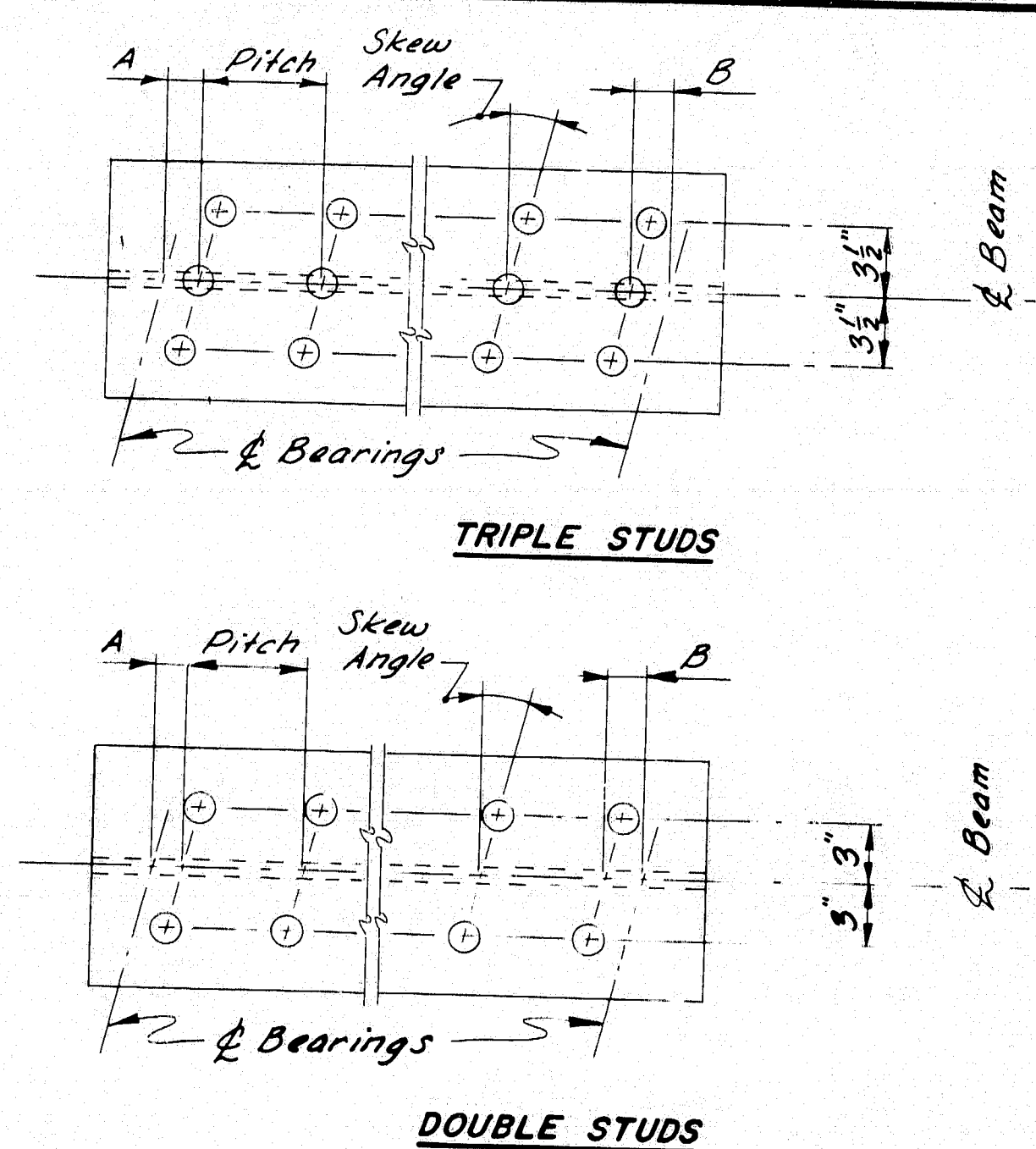
#### BRIDGE DRAIN



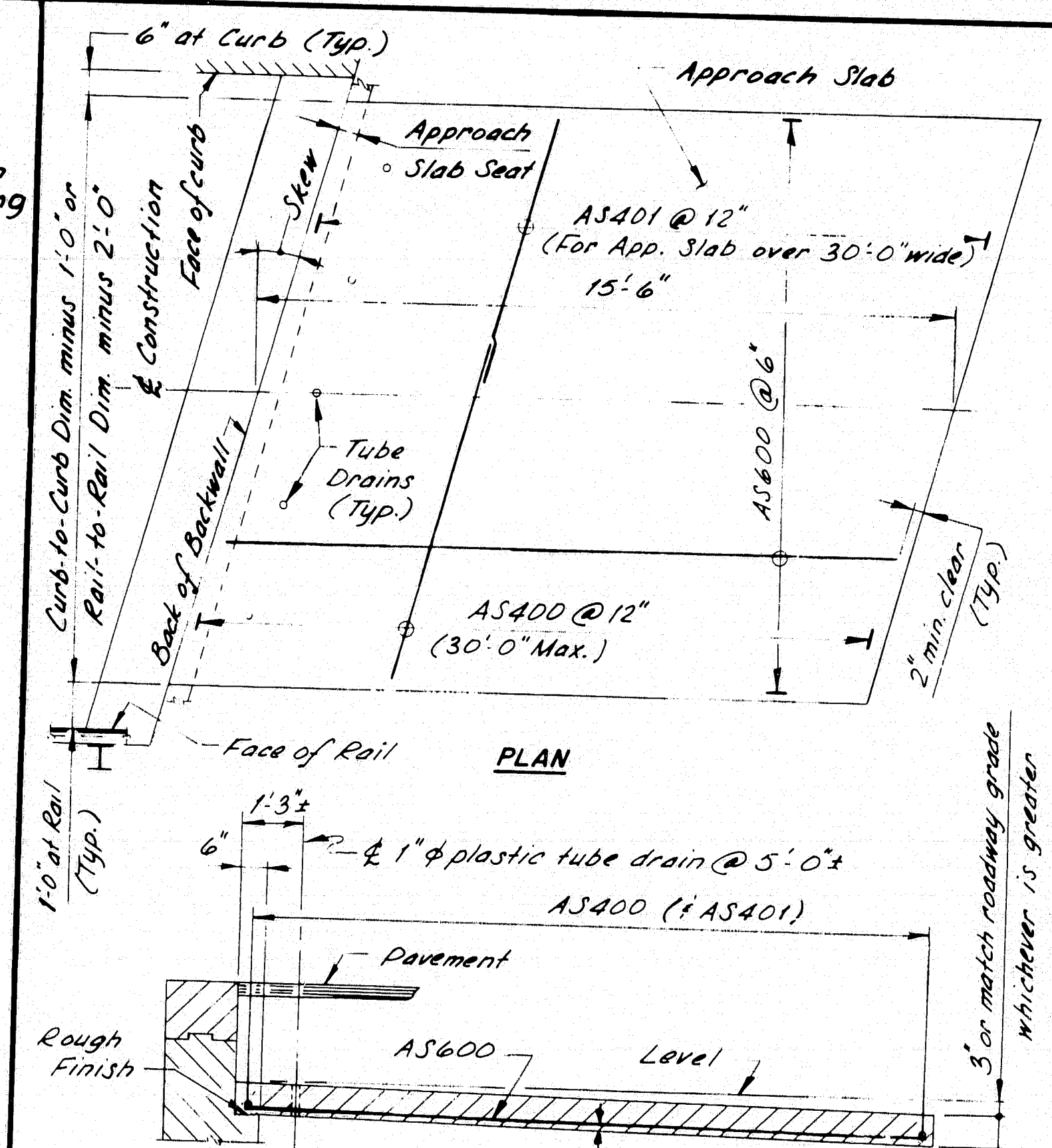
#### CURB SECTION TYPE 1A (STRUCTURAL CONCRETE WEARING SURFACE)



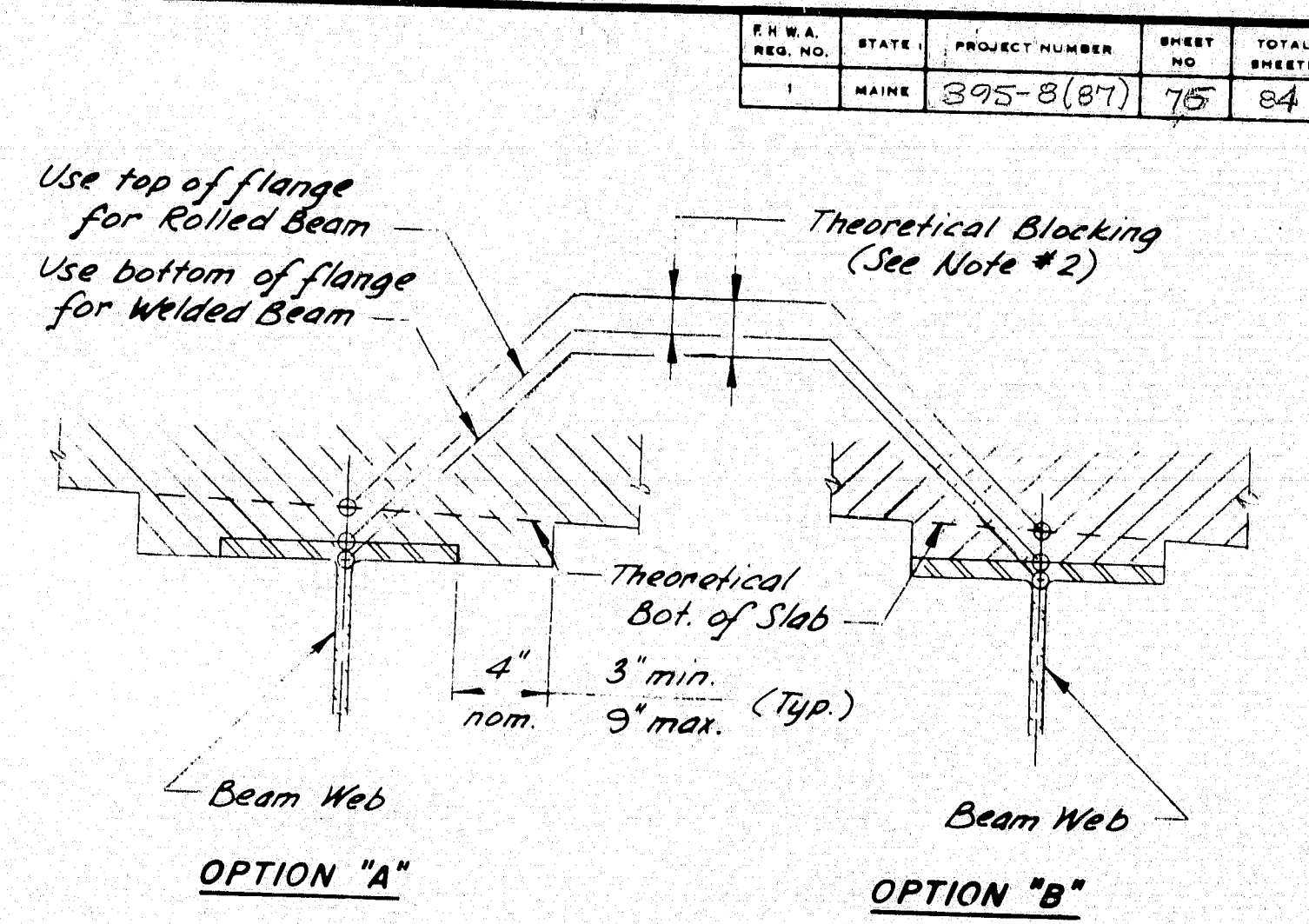
#### CURB SECTION TYPE 1B (BITUMINOUS WEARING SURFACE)



#### SHEAR CONNECTORS



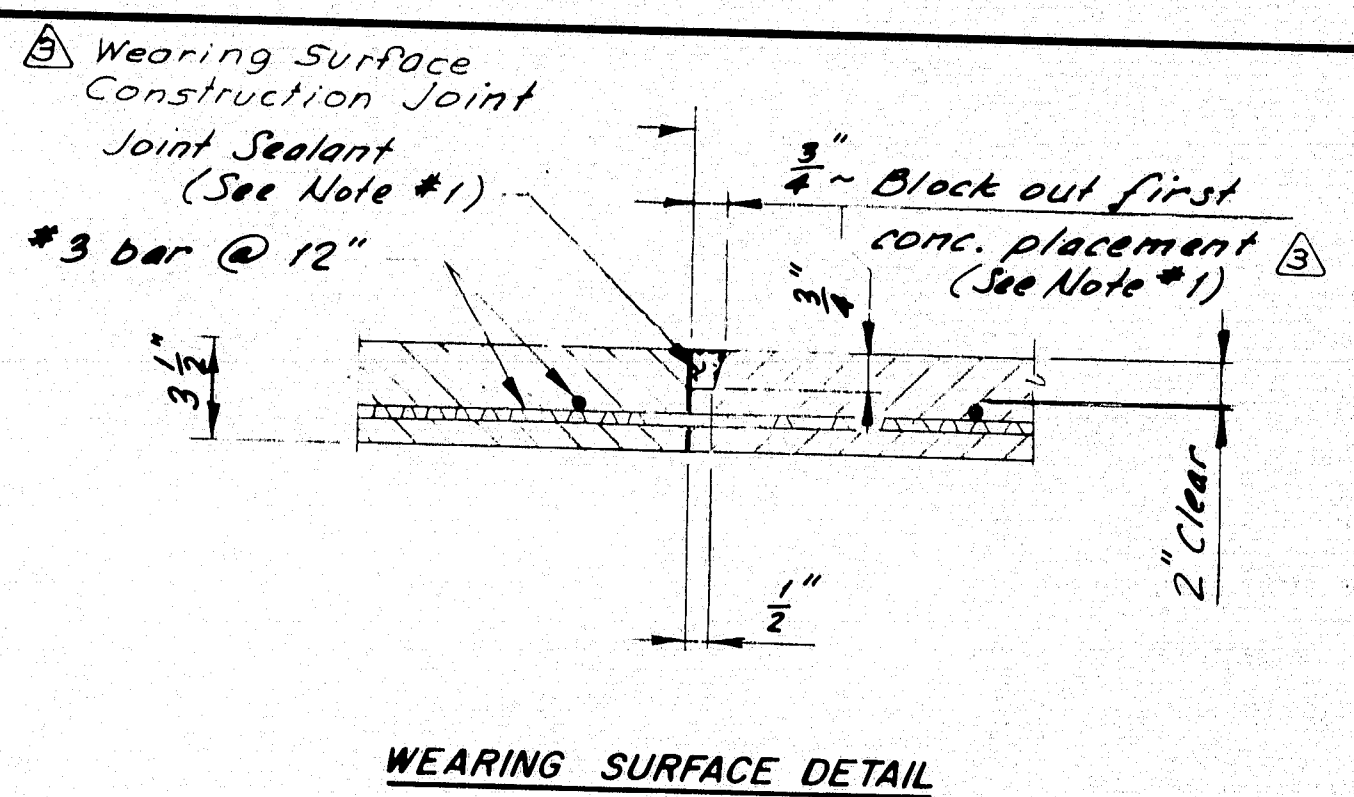
#### APPROACH SLAB



#### NOTE:

- Haunch Option "A" or Option "B" may be used at the Contractor's discretion. Only one option shall be used on each structure, except that Option "A" must always be used on the fascia side of all fascia beams and on beams designed without shear connectors.
- Theoretical Blocking shall be as indicated on Design Drawings.
- Do not use Theoretical Blocking for setting of form-work.

#### HAUNCH DETAILS



#### NOTE:

- Use Block-out and Sealant only at Wearing Surface Construction Joints over Structural Slab Construction Joints. At all other joints, brush joint with neat cement paste before making adjacent concrete placement.

#### STRUCTURAL CONCRETE WEARING SURFACE

REVISIONS	DATE
Revised Stud Detail	3-82
Added Curb Section	7-82
Added Plastic Tube Drain & modified Structural Concrete Wearing Surface	11-82
Revise Curb Anchorage	2-83
Revise Curb Title	6-83
Revise Curb Type 1B	11-83

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

#### STANDARD DETAILS (BD 126 - 81)

#### MISCELLANEOUS DETAILS

BRIDGE DRAIN - SHEAR CONNECTORS  
STRUC. CONC. WEAR. SURFACE  
CURB SECTION - APPROACH SLAB  
HAUNCH DETAILS

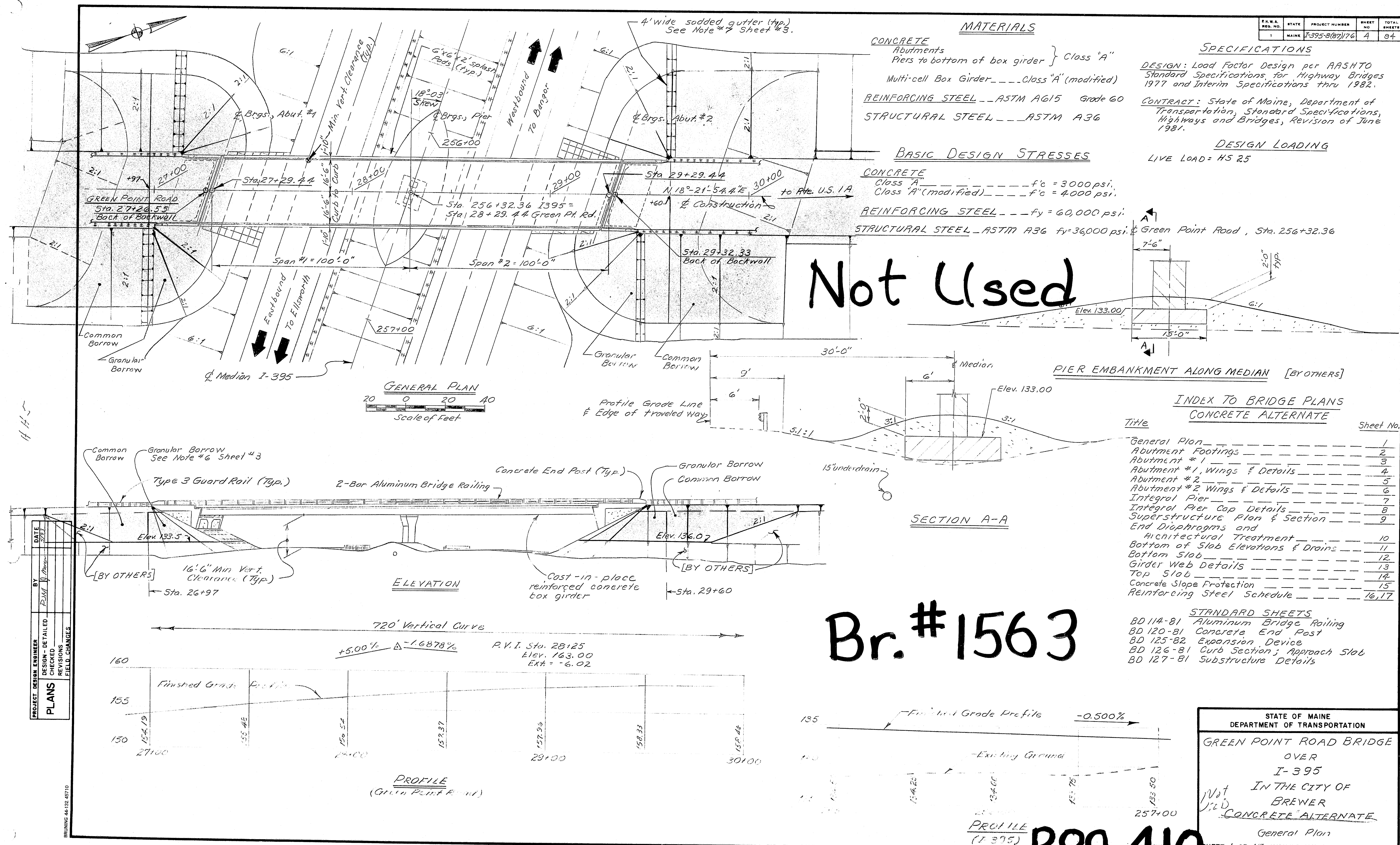
SHEET OF AUGUSTA, MAINE JUNE 1981

183-170



183-171





**MATERIALS**

CONCRETE  
Abutments  
Piers to bottom of box girder } Class "A"  
Multi-cell Box Girder --- Class "A" (modified)

REINFORCING STEEL --- ASTM A615 Grade 60  
STRUCTURAL STEEL --- ASTM A36

**SPECIFICATIONS**

DESIGN: Load Factor Design per AASHTO Standard Specifications for Highway Bridges 1977 and Interim Specifications thru 1982.  
CONTRACT: State of Maine, Department of Transportation, Standard Specifications, Highways and Bridges, Revision of June 1981.

**DESIGN LOADING**  
LIVE LOAD = HS 25

**BASIC DESIGN STRESSES**

CONCRETE  
Class "A" ---  $f'_c = 3000$  psi.  
Class "A" (modified) ---  $f'_c = 4000$  psi.

REINFORCING STEEL ---  $f_y = 60,000$  psi.  
STRUCTURAL STEEL --- ASTM A36  $f_y = 36,000$  psi.

Green Point Road, Sta. 256+32.36

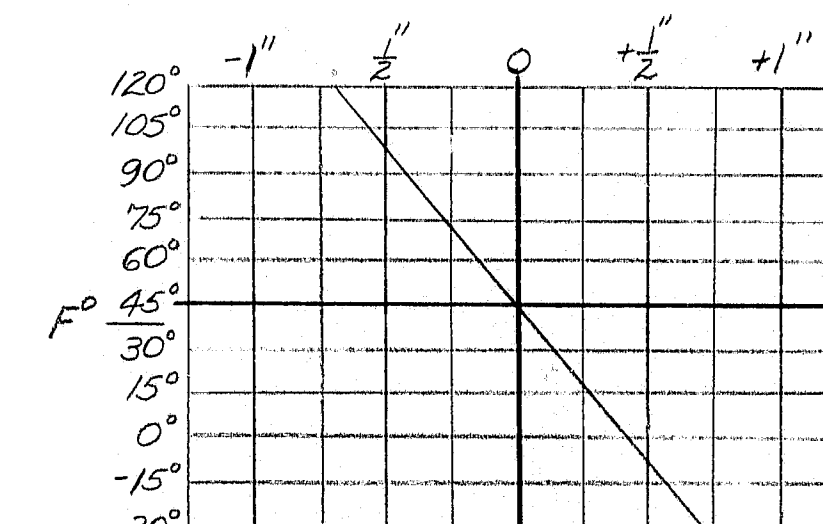
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
General Plan  
SHEET 1 OF 17 AUGUSTA, MAINE



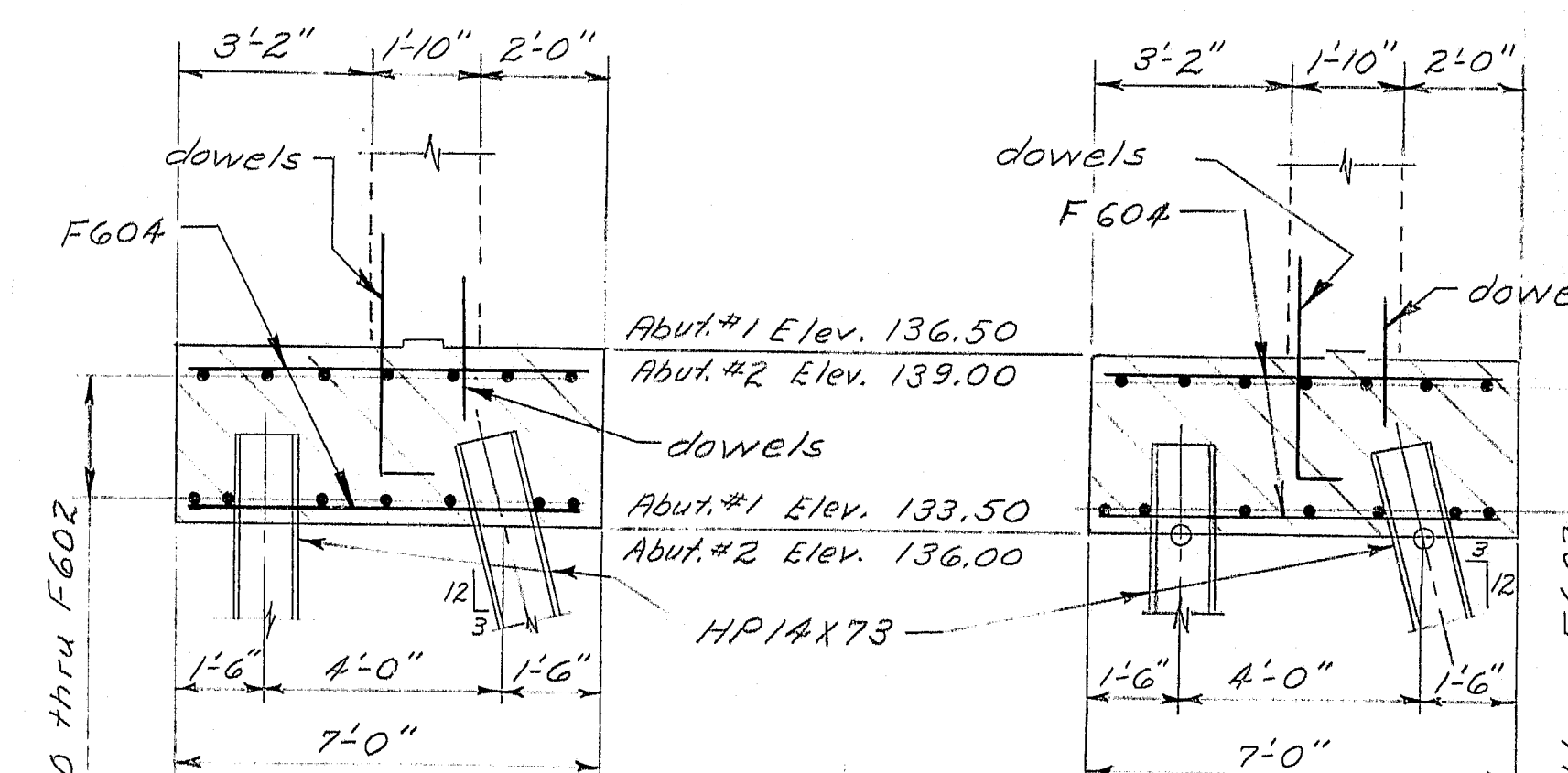
PLANS	PROJECT DESIGN ENGINEER	BY	DATE
	DESIGN - DETAILED	<i>RJM</i>	<i>5/83</i>
	CHECKED		
	REVISIONS		
	FIELD CHANGES		

# PILE NOTES

- 1) Piles marked thus  $\Rightarrow$  shall be battered 3 inch per foot in the direction of the arrow.
- 2) Maximum calculated pile load: 99.4 tons.
- 3) Estimate of piles required:  
Abutment #1 — 21-HP14X73 @ 30 feet  
Abutment #2 — 21-HP14X73 @ 32 feet



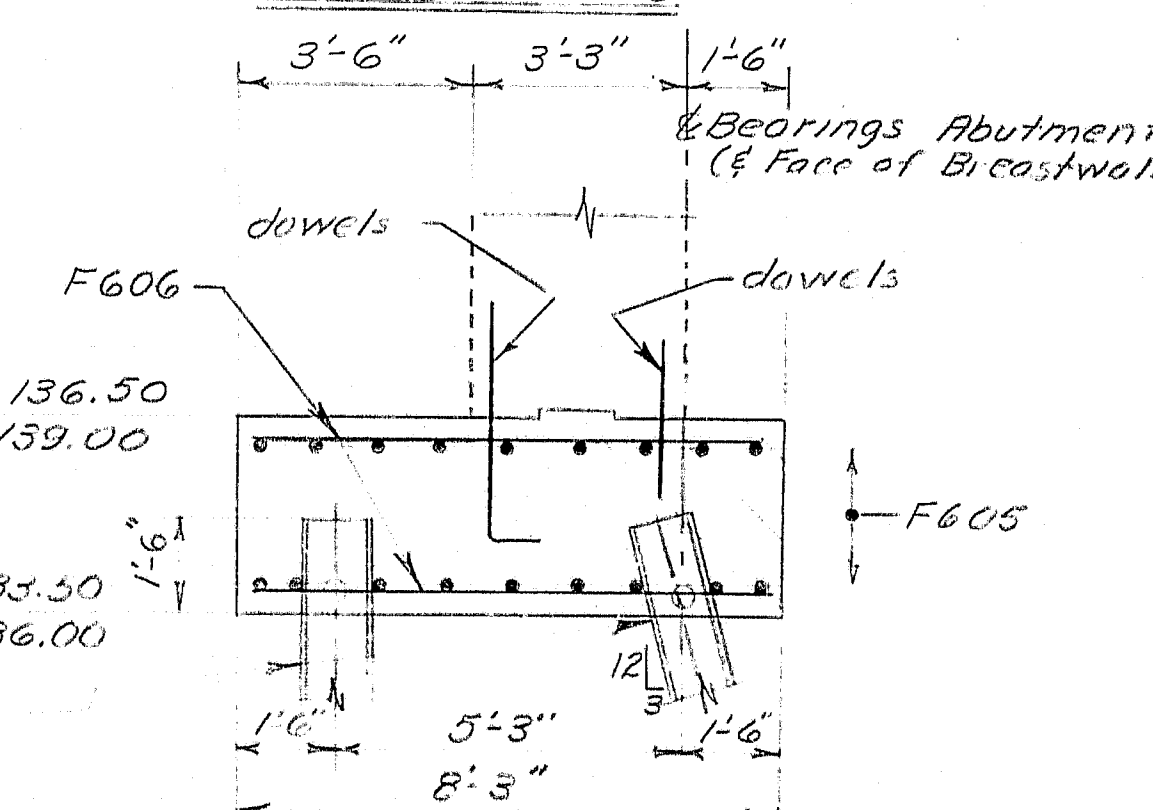
COMPRESSION SEAL ADJUSTMENT CHART



SECTION B-B

SECTION C-C

NOTE : For down locations see the Abutment sheets.



SECTION A-

Abut #1 Elev. 136.50  
Abut #2 Elev. 139.00

Abut #1 Elev. 133.50  
Abut. #2 Elev. 136.00

HP 14x73

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

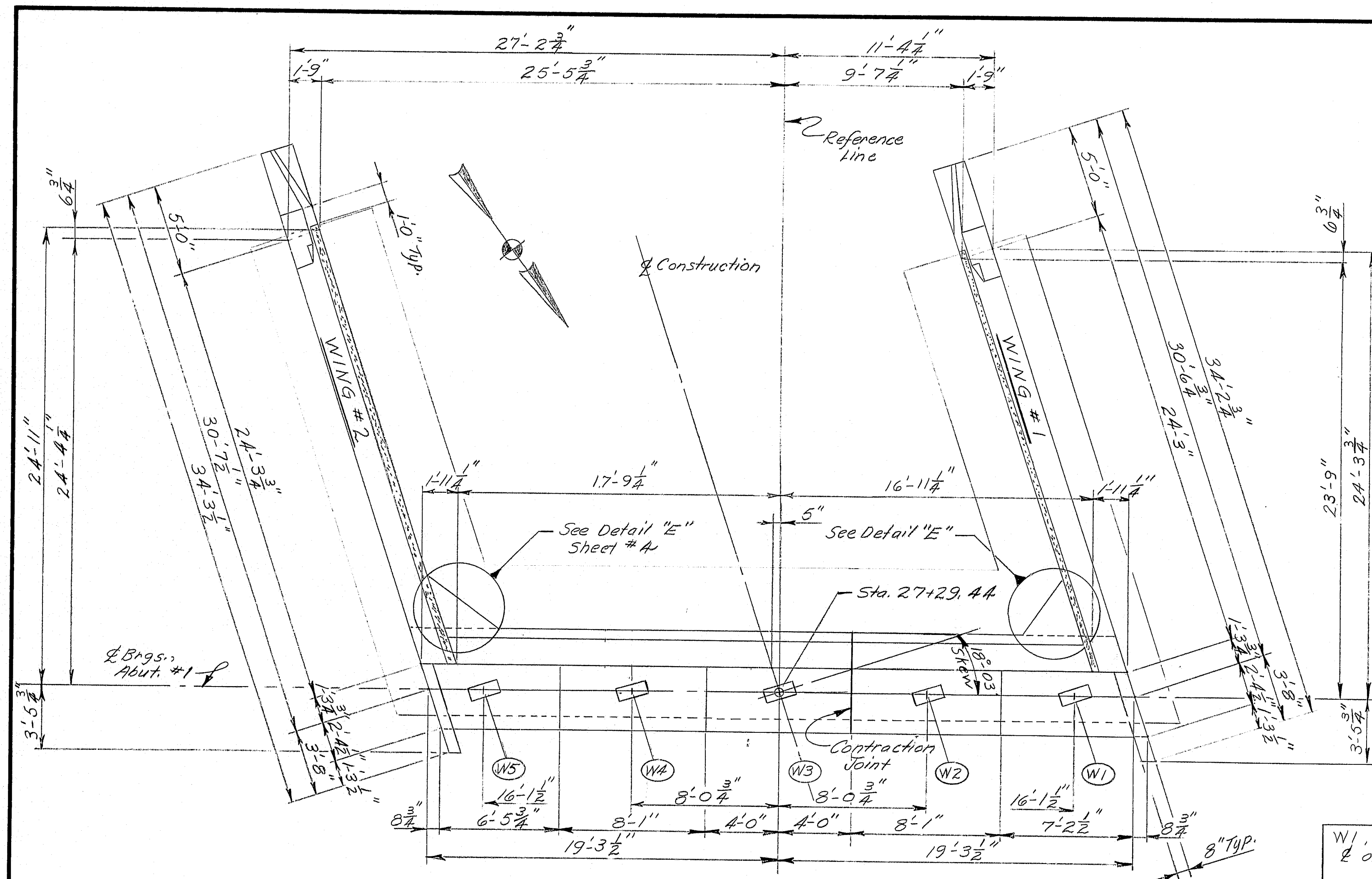
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWSTER  
CONCRETE ALTERNATE  
ABUTMENT FOOTINGS

SHEET 2 OF 17 AUGUSTA, MAINE

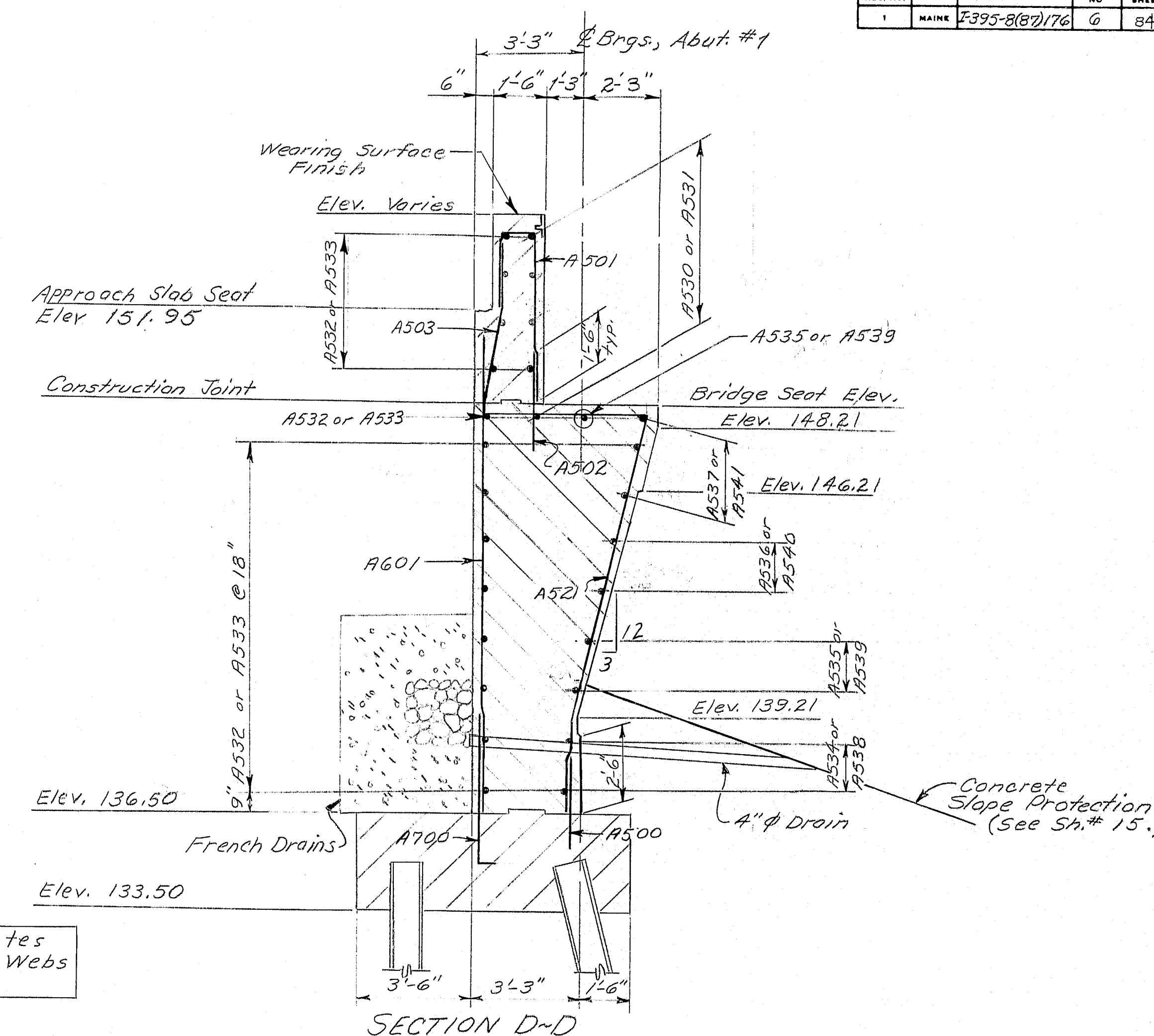
**R89-411**



F.R.A.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	395-8(87)176	6	84



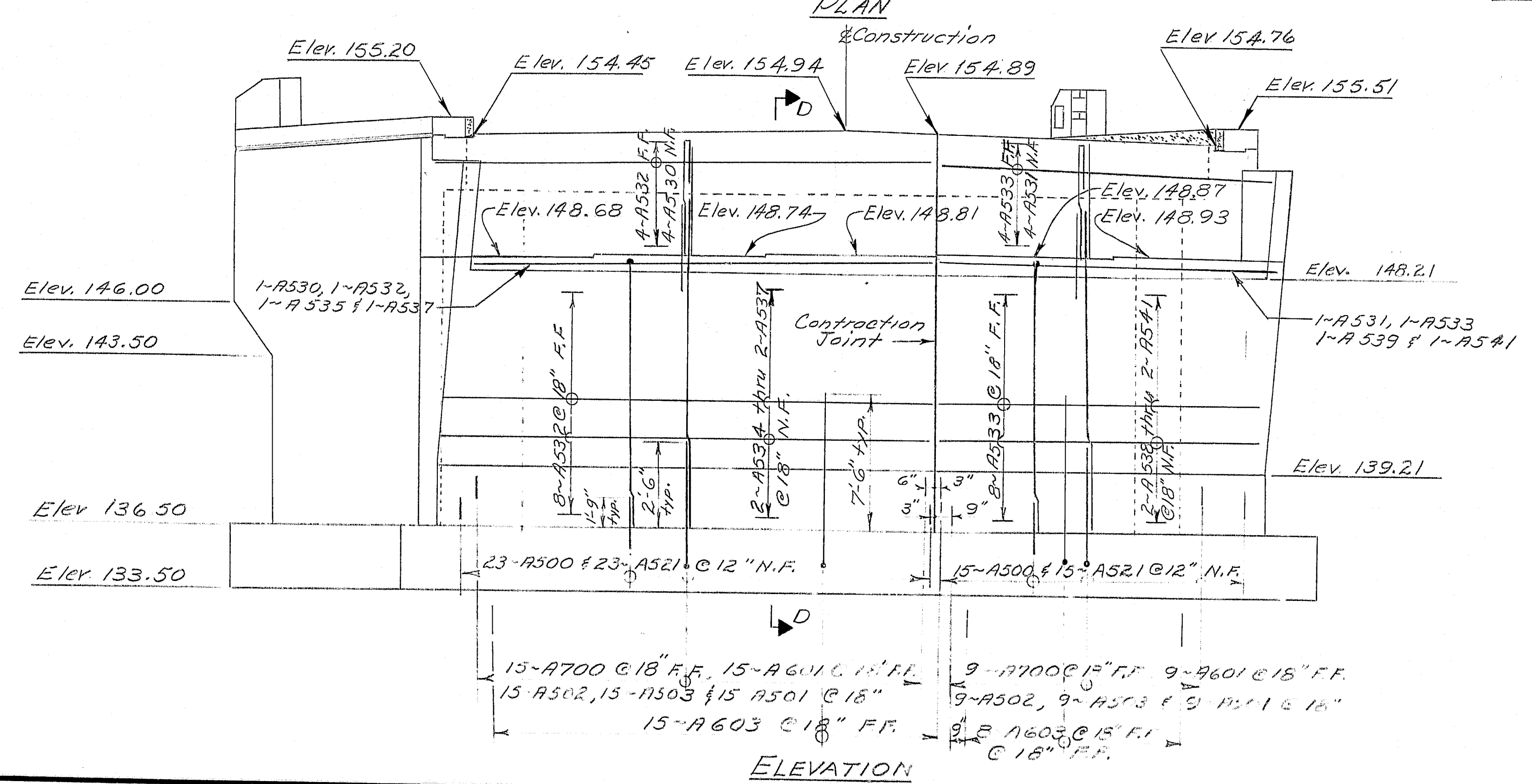
W1, etc. indicates  
2 of Girder Webs  
typical



### ABUTMENT NOTES

1. Reinforcing Steel shall have 2" minimum cover unless otherwise noted.
2. Protective Coating for Concrete Surfaces shall be applied to tops of concrete curbs, top of Abutment backwalls and one foot below top of back-wall on the back side, and all exposed surfaces of concrete end posts.
3. Place 4" φ drains in Breastwall and Wings at 20'-0" maximum spacing. Exact location to be determined in the field by the Engineer.
4. Payment for concrete end posts will be made under Item 502.21.
5. Abutment reinforcing steel splices shall be as follows unless otherwise shown on the plans:

Bar Size	Minimum splice
#5	1'-9"
#6	2'-3"
#7	3'-0"
#8	3'-11"
6. Granular borrow shall meet the requirements of subsection 703.19, material for underwater backfill.
7. Sodded gutters shall be constructed after paving and shoulder work is completed, where it is apparent that runoff will cause continual erosion.



ELEVATION

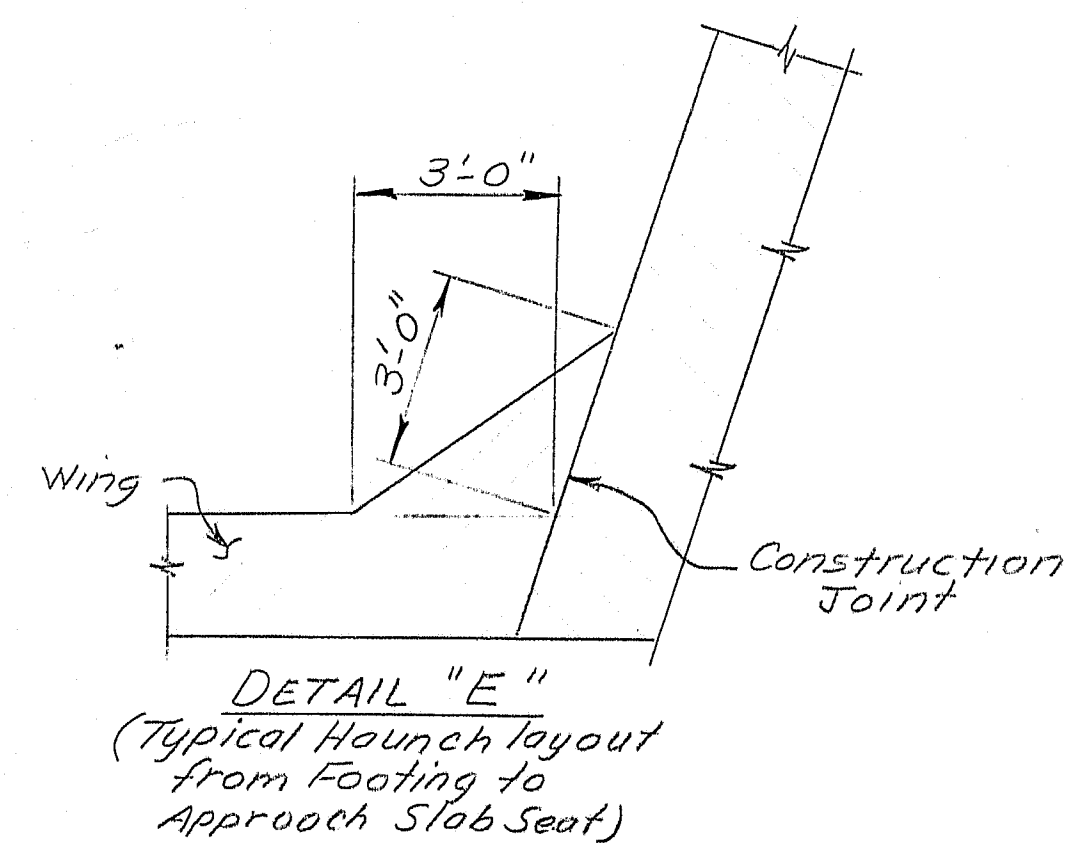
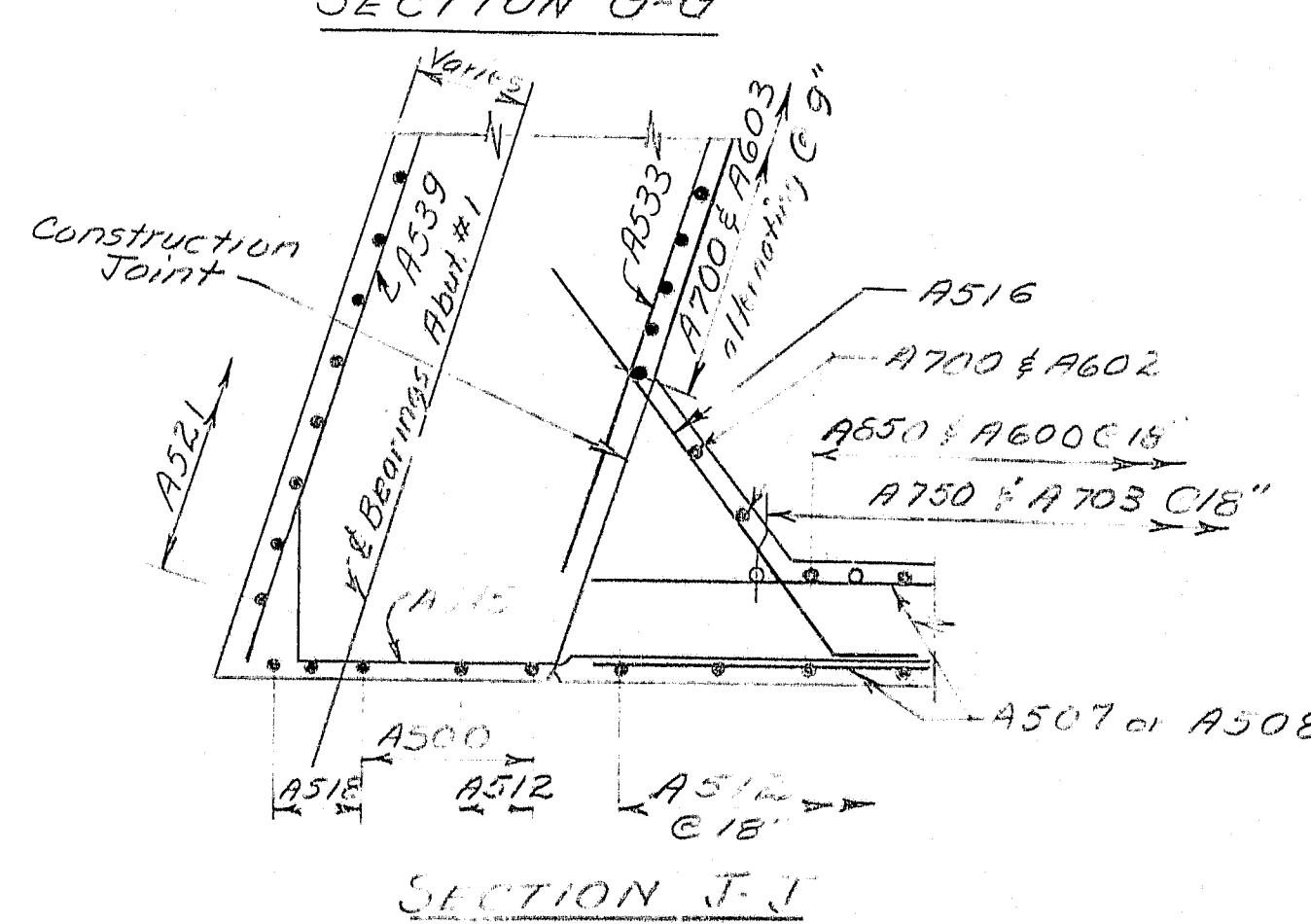
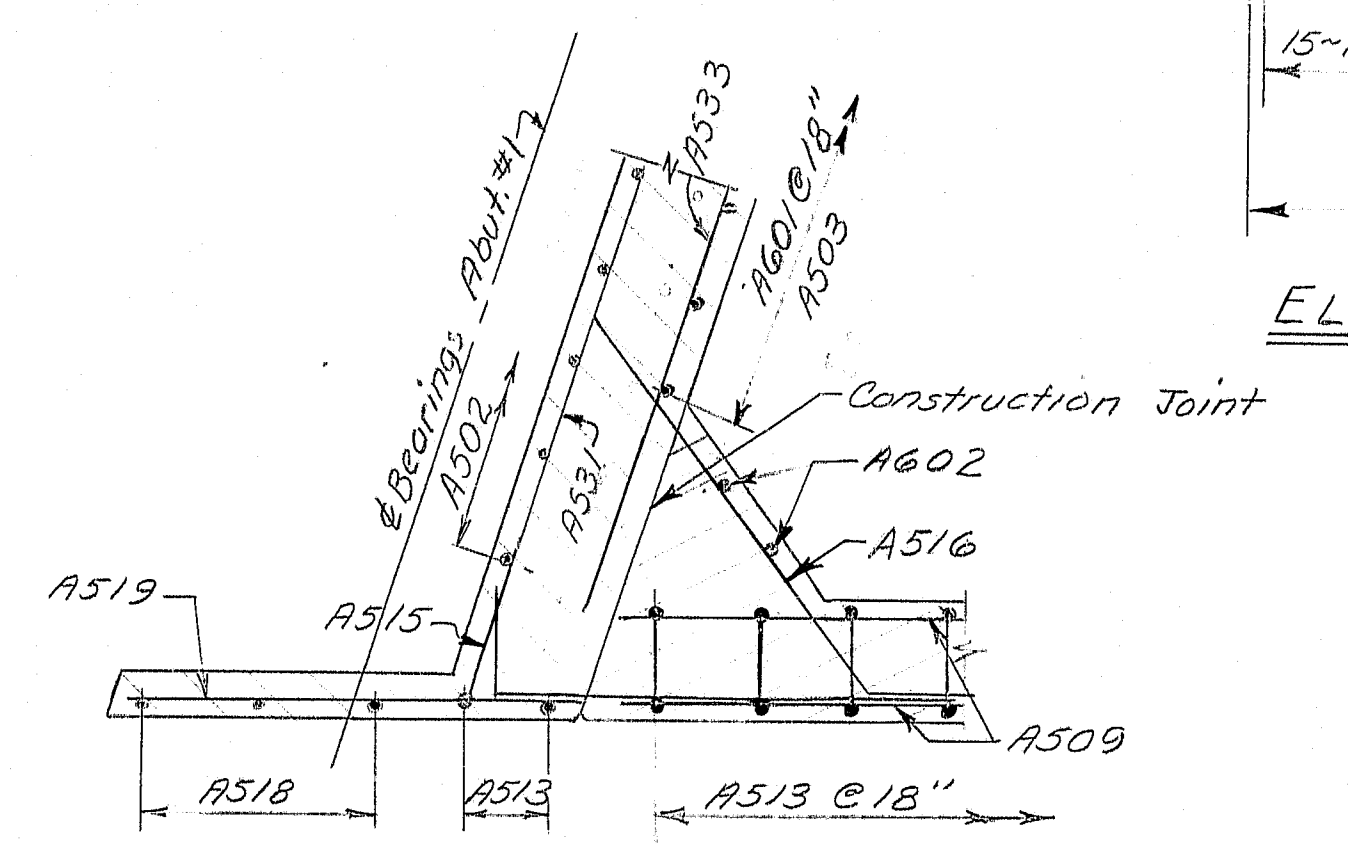
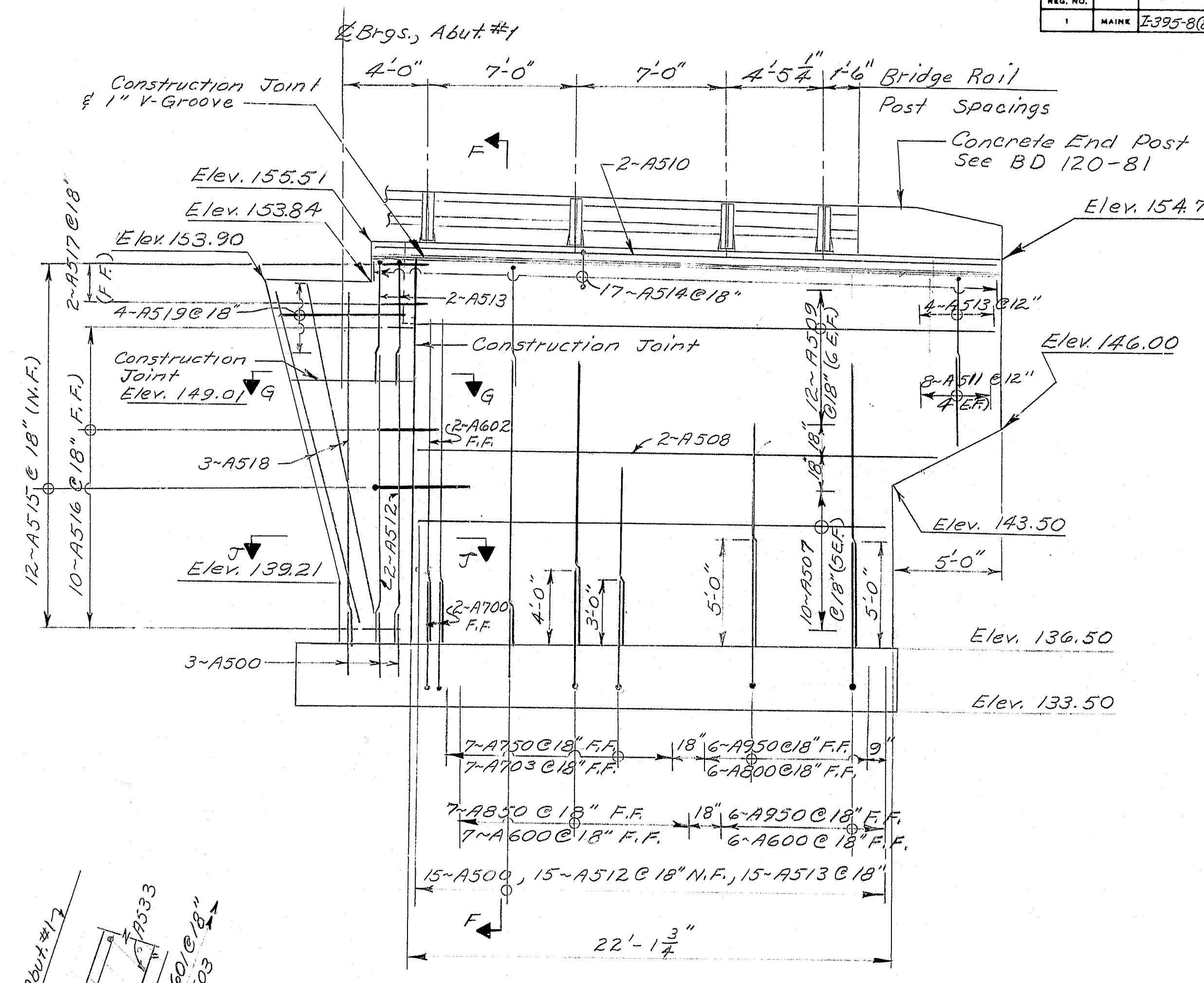
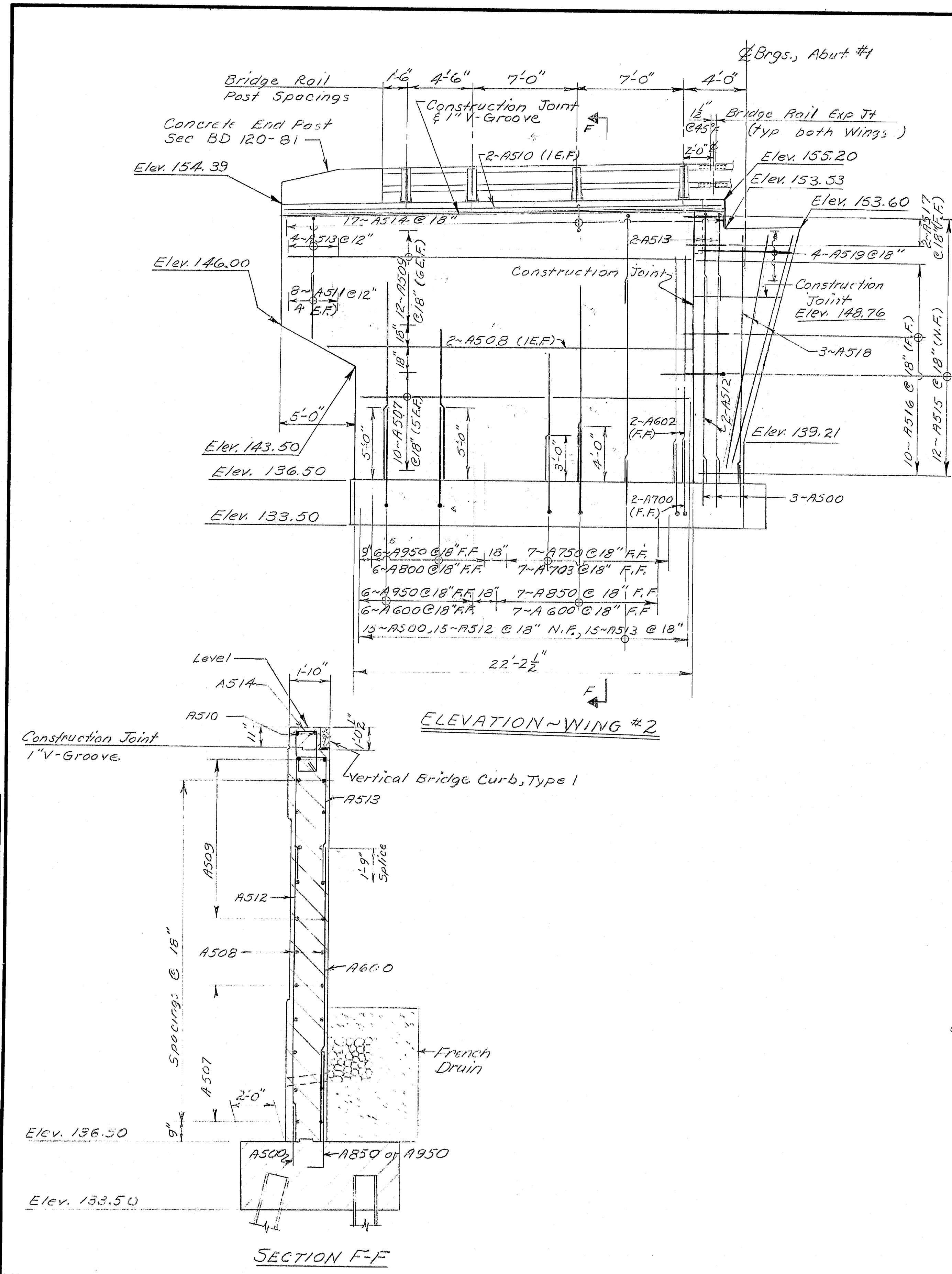
PROJECT DESIGN ENGINEER	DATE
BY	5/83
DESIGN - DETAILED	RM
CHECKED	
REVISIONS	
FIELD CHANGES	

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
JUNCTION IN THE CITY OF  
BREWSTER  
CONCRETE ALTERNATE  
ABUTMENT No. 1  
SHEET 3 OF 17 AUGUSTA, MAINE

R89-412



F.R.D.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	7-395-8(87)176	7	84



PROJECT DESIGN ENGINEER	DATE
BY	5/13
DESIGN - DETAILED	RJM
CHECKED	5/13
REVISIONS	
DATE	
BY	
DESCRIPTION	

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
ABUTMENT No. 1 WINGS AND DETAILS

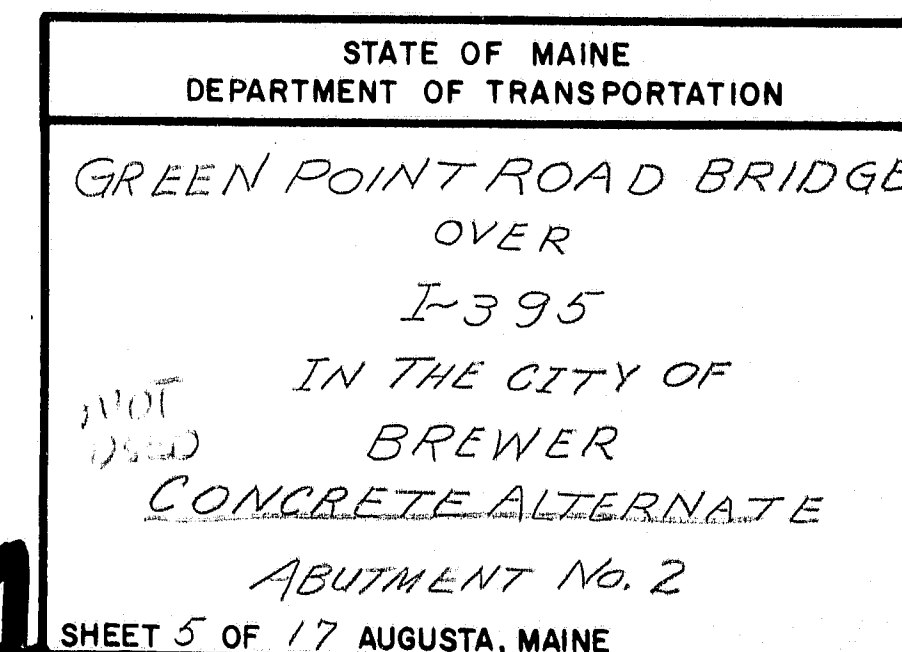
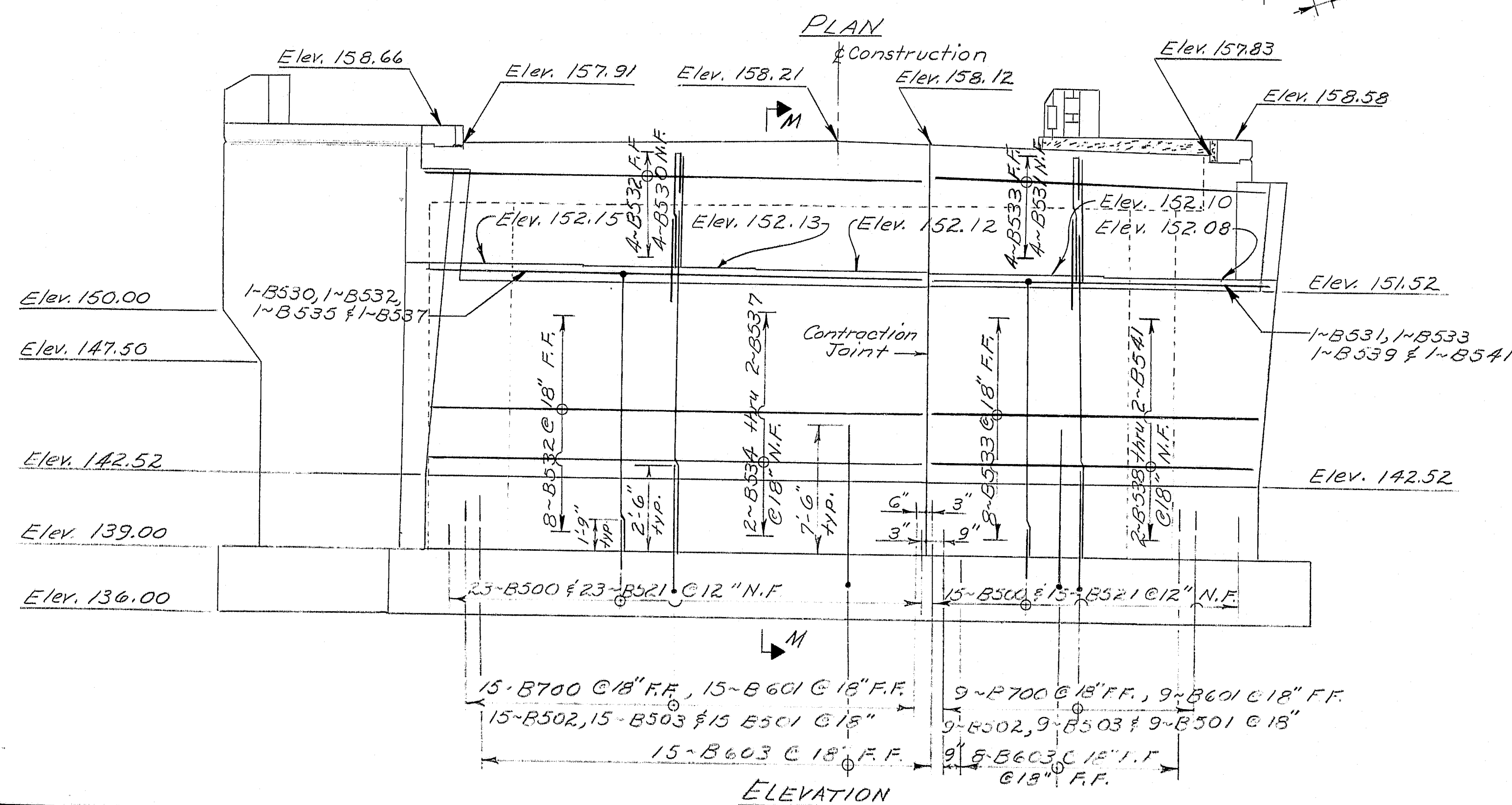
SHEET 4 OF 17 AUGUSTA, MAINE

R89-413



PROJECT DESIGN ENGINEER	BY	DATE
DESIGN - DETAILED	<i>PLM</i>	<i>9 March 5/83</i>
CHECKED		
REVISIONS		
FIELD CHANGES		

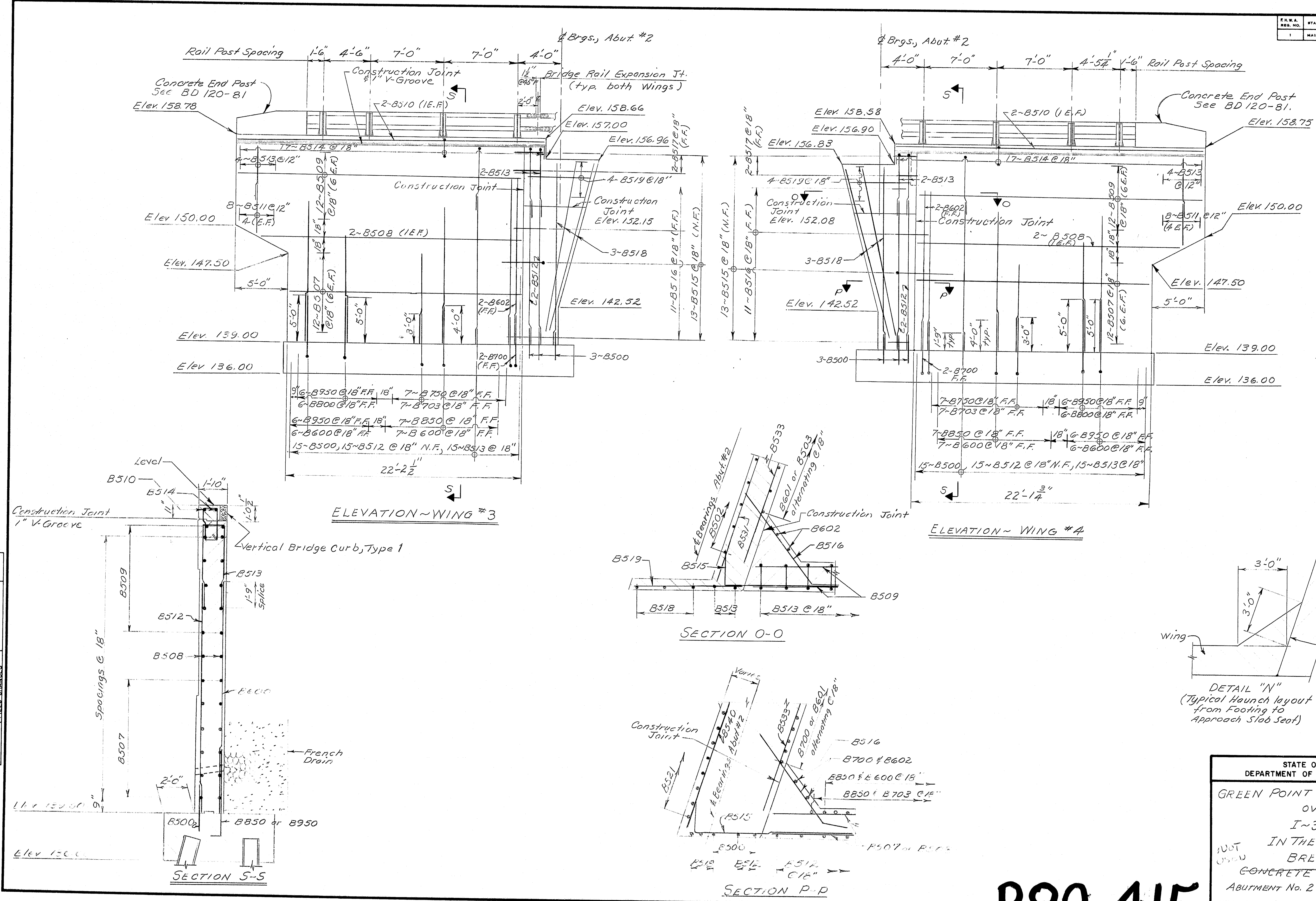
**PLANS**



~~R89-414~~



PARA.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	F-395-B(87)176	9	84



PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAILED	5/1/83
CHECKED	
REVISIONS	
FIELD CHANGES	

PLANS


STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
ABUTMENT No. 2 WINGS AND DETAILS  
SHEET 6 OF 17 AUGUSTA, MAINE

R89-415



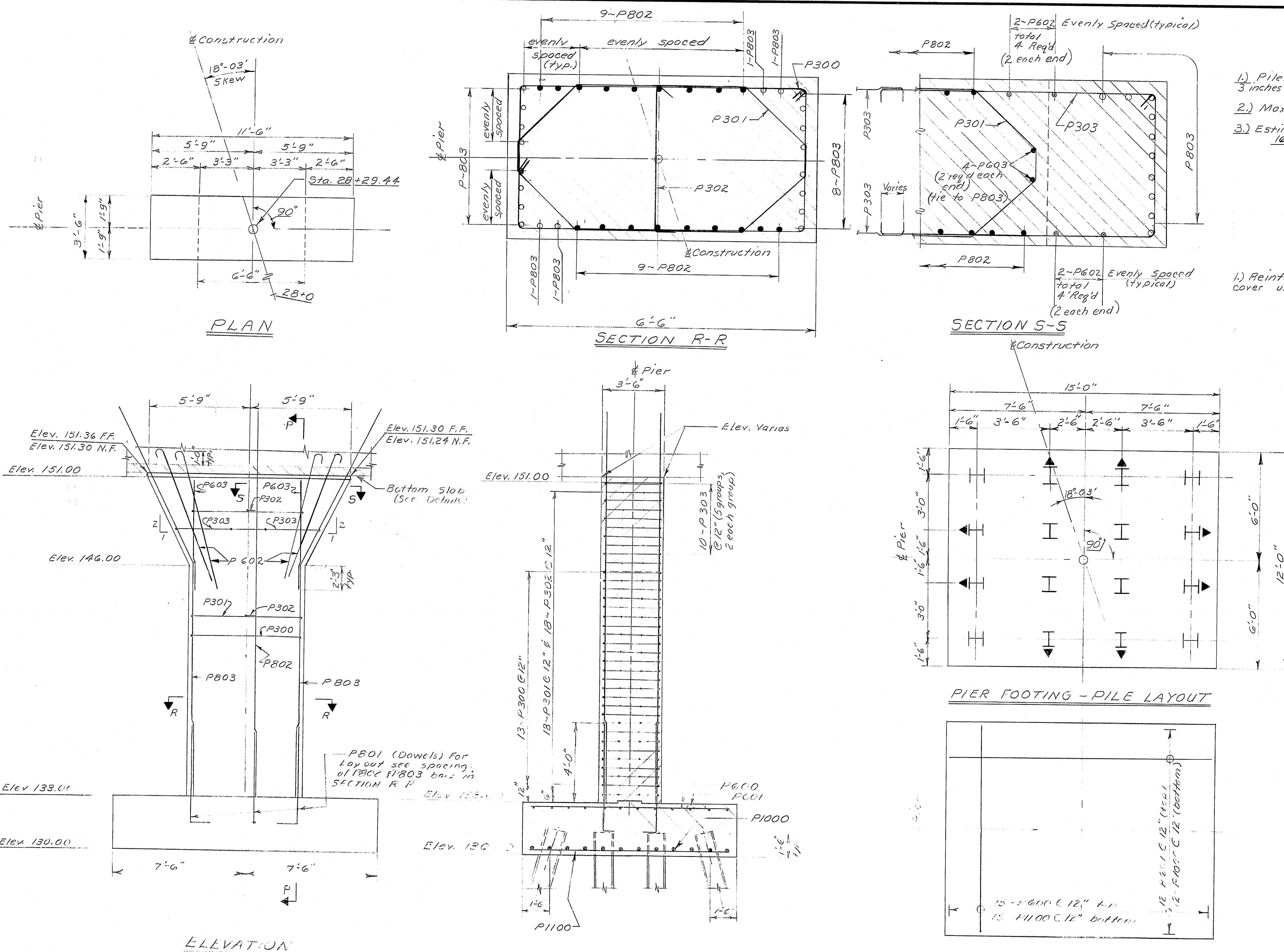
F.R.A. REQ. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	E395-B (87)176	10	84

### PILE NOTES

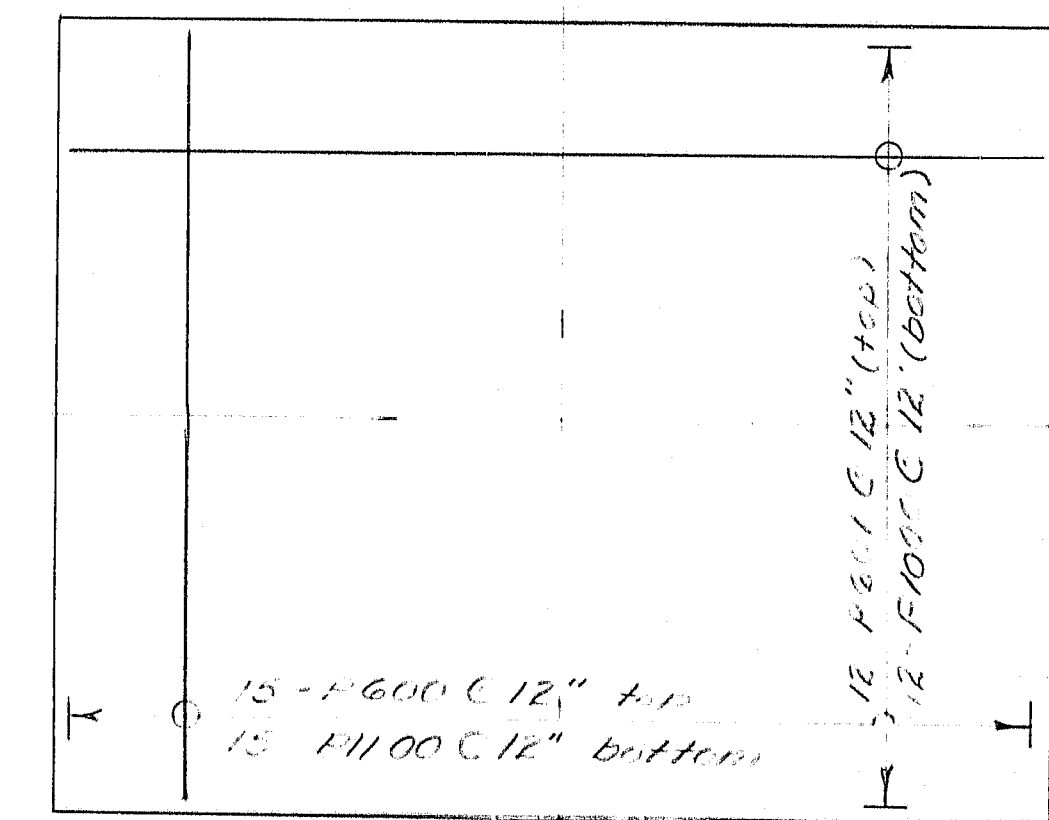
- 1.) Piles marked thus  shall be battered 3 inches per foot in the direction of the arrow.
- 2.) Maximum calculated pile loads: 131 tons.
- 3.) Estimate of piles required at pier: HP14X89  
16 required @ 30 feet long = 480 L.F. TOTAL

### PIER NOTES

- 1.) Reinforcing Steel shall have 3 inches minimum cover unless otherwise indicated.



PIER FOOTING - PILE LAYOUT



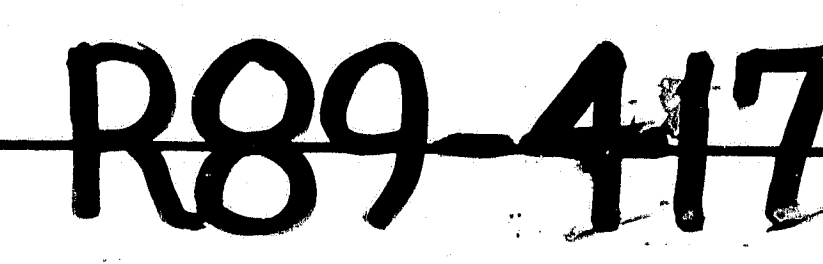
PIER FOOTING - REINFORCING STEEL

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
IN 395  
IN THE CITY OF  
BREWSTER  
CONCRETE ALTERNATE  
Integral Pier  
SHEET 7 OF 17 AUGUSTA, MAINE

R89-416



<b>PLANS</b>	PROJECT DESIGN ENGINEER	BY	DATE
	DESIGN - DETAILED	<i>RIM</i>	<i>J. M. ... 5/83</i>
	CHECKED		
	REVISIONS		
	FIELD CHANGES		



STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

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GREEN POINT ROAD BRIDGE

OVER  
I-395

IN THE CITY OF  
BREWER

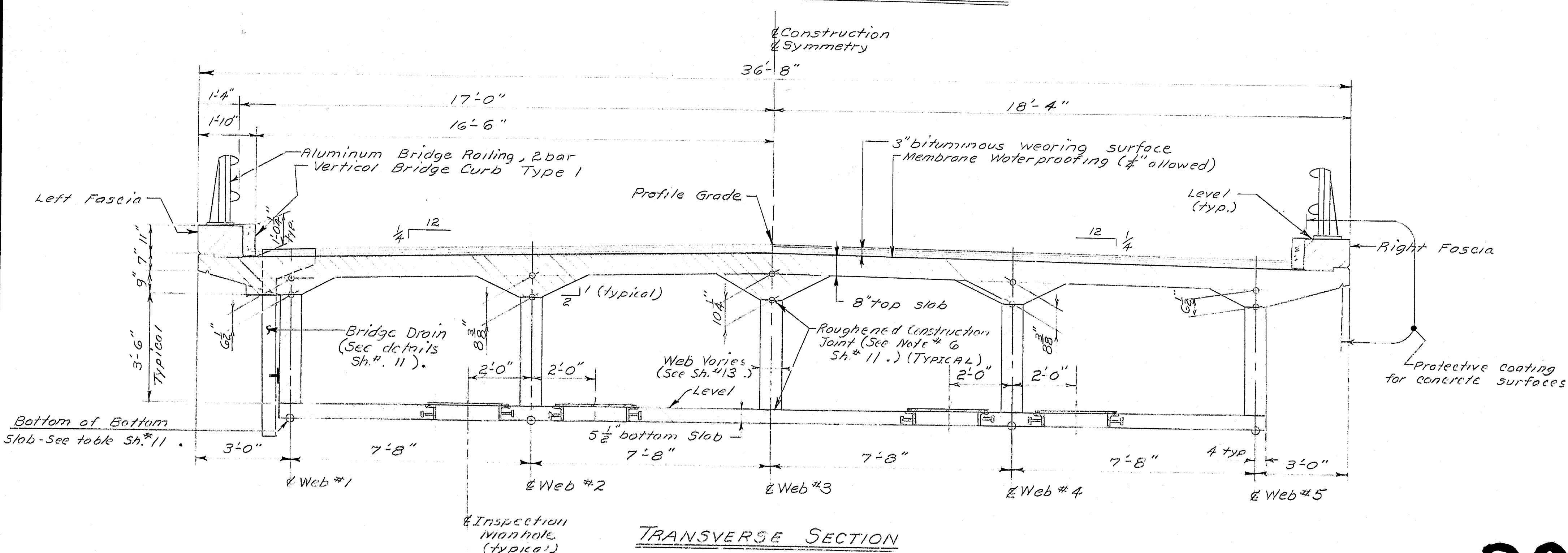
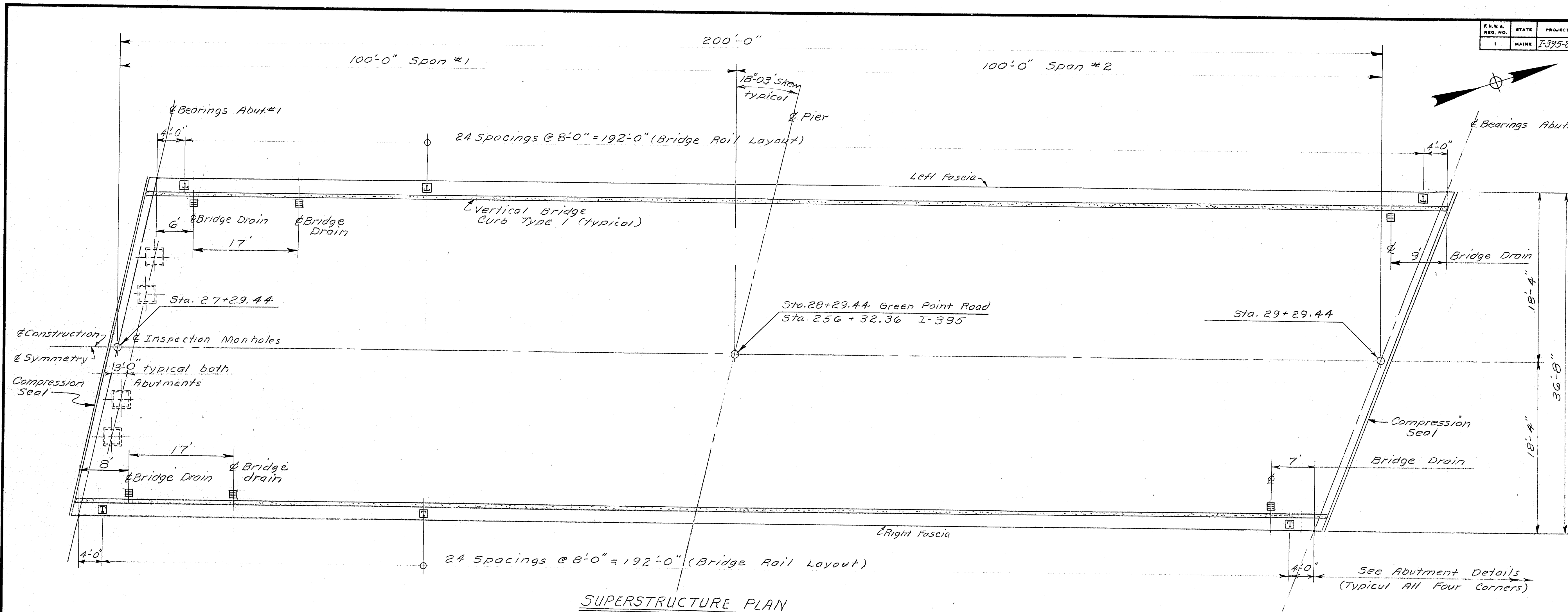
CONCRETE ALTERNATE

Integral Pier Cap Details

SHEET 8 OF 17 AUGUSTA, MAINE



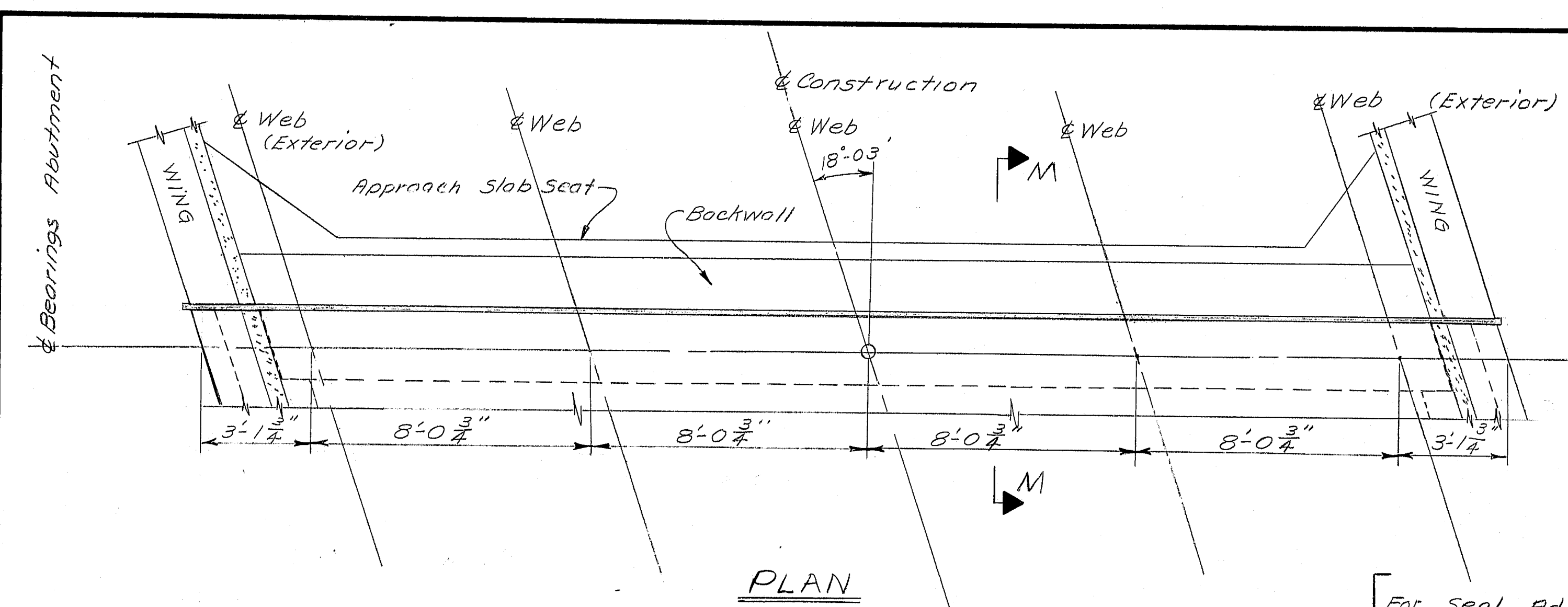
F.R.W.A.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-395-8(82)176	12	84



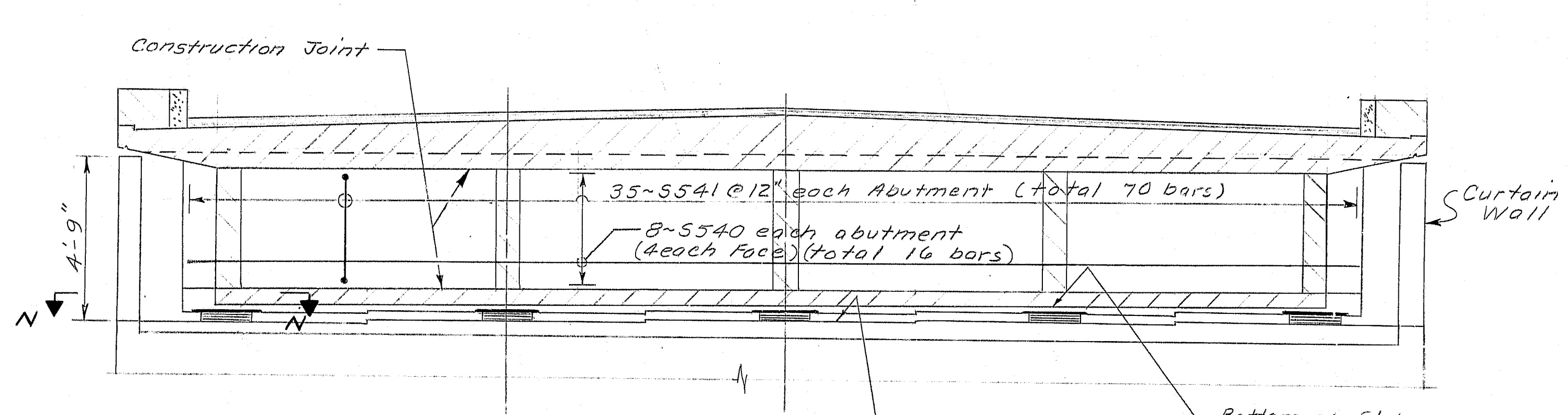
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
Superstructure Plan & Section  
SHEET 9 OF 17 AUGUSTA, MAINE

R89-418

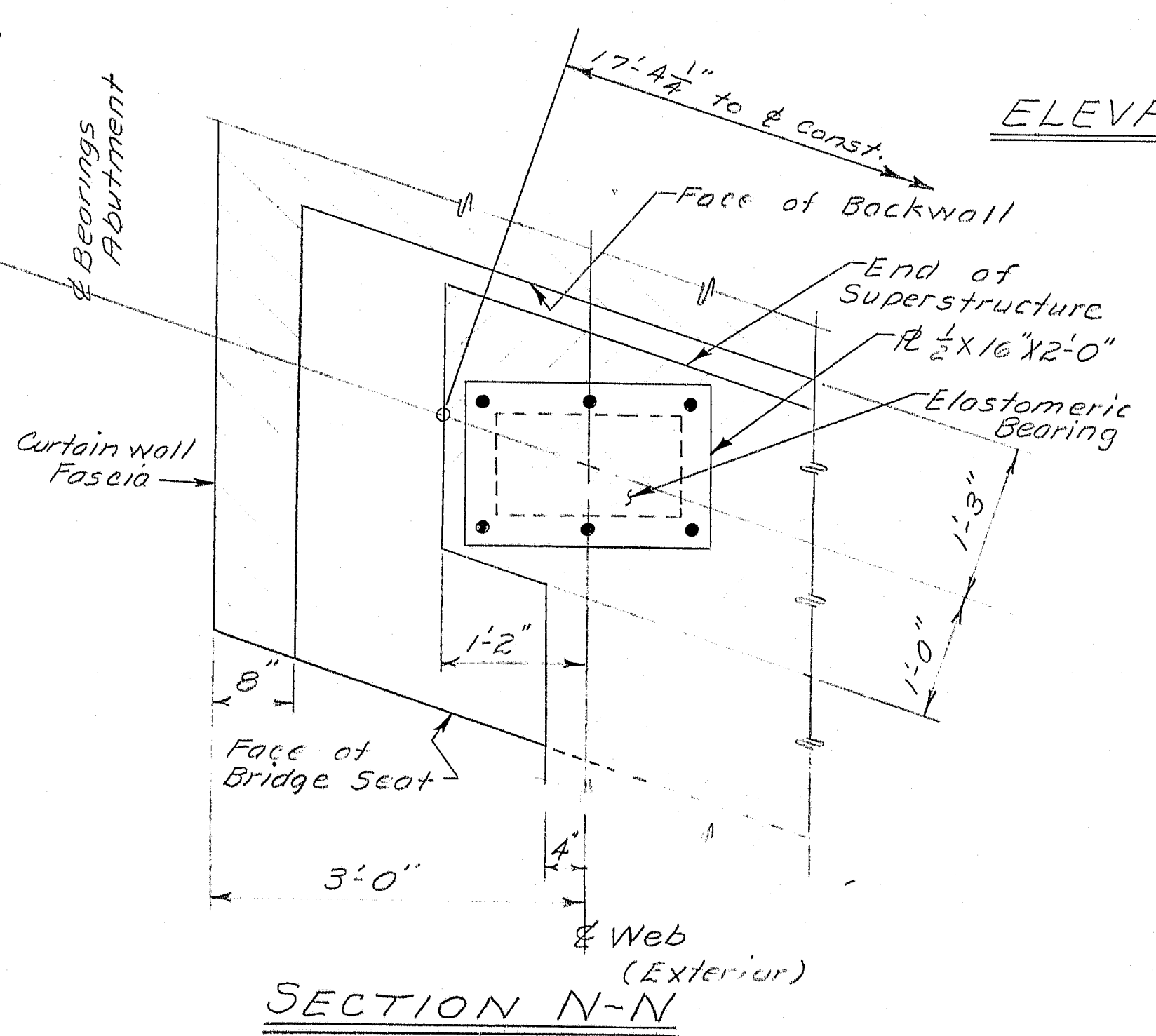




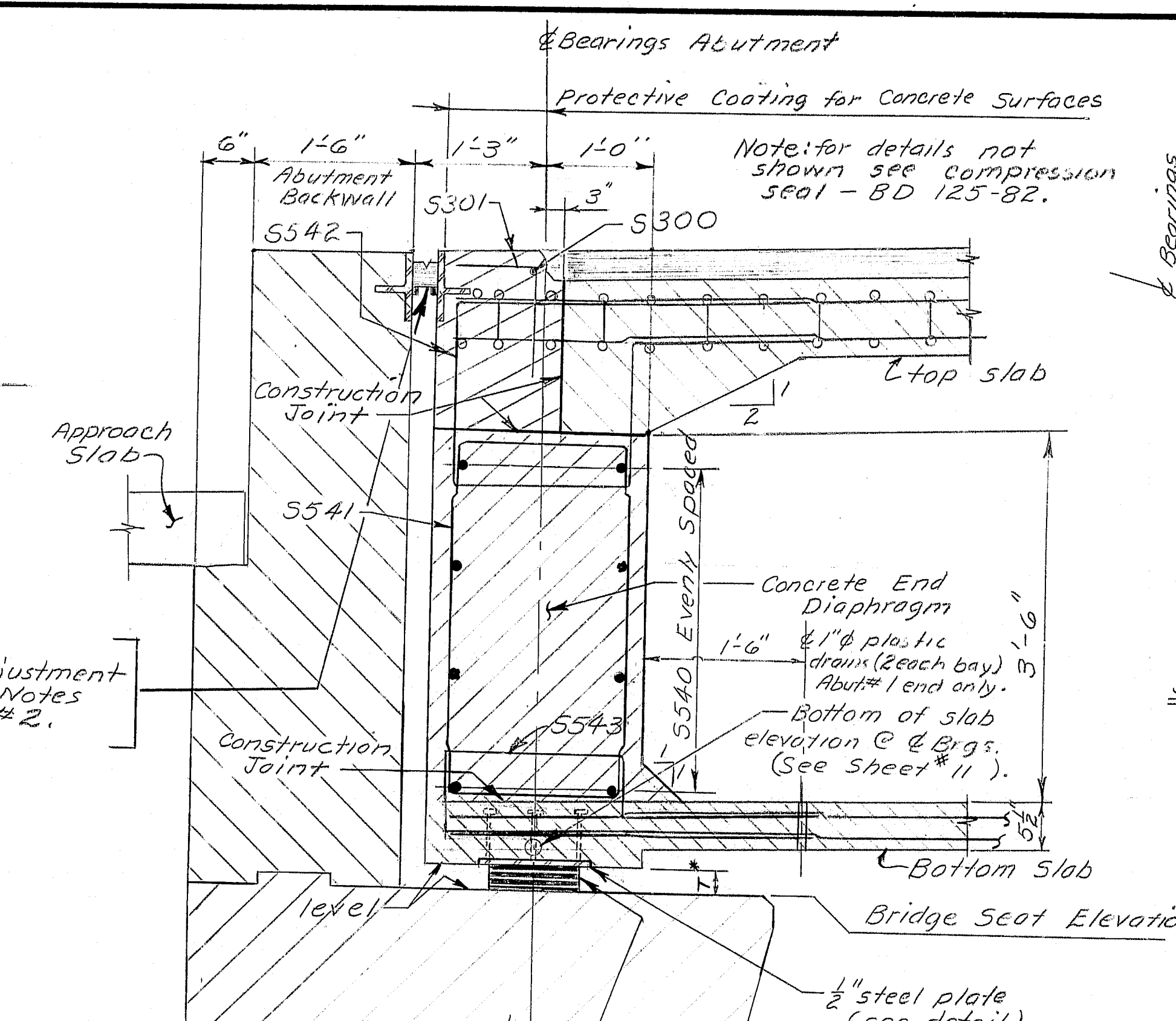
PLAN



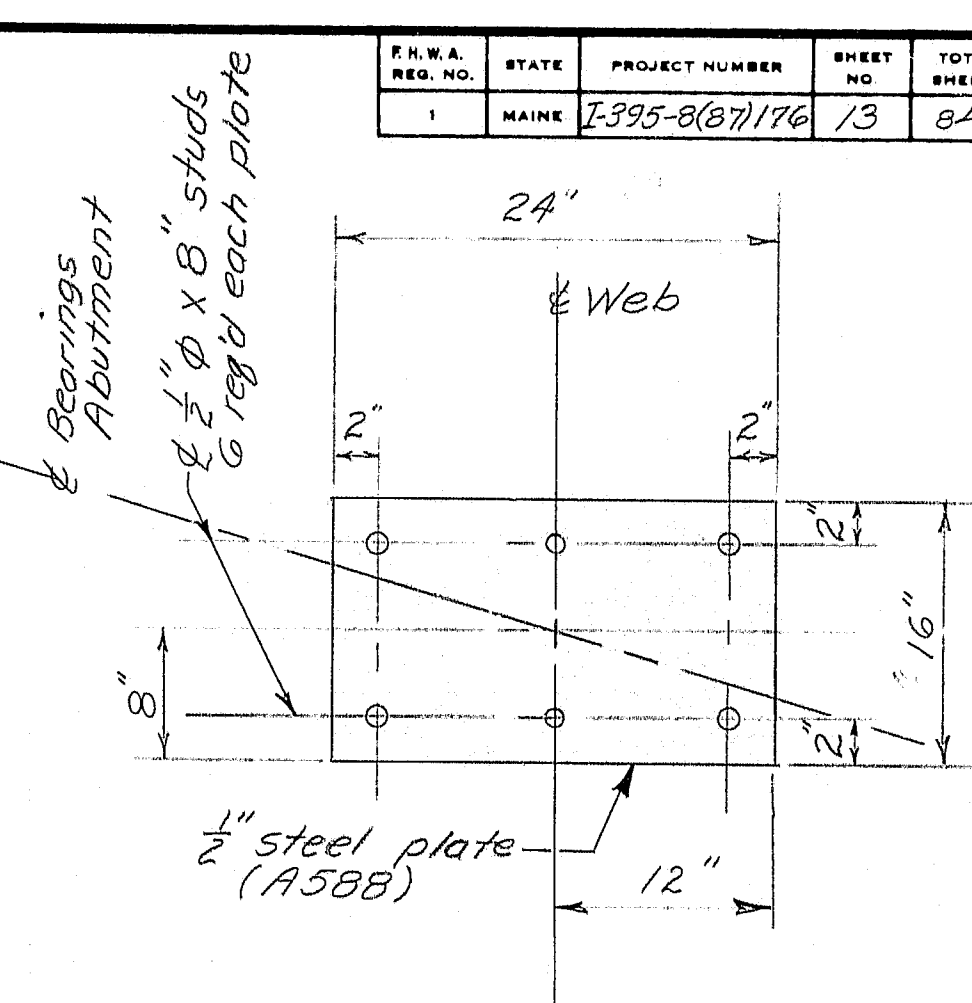
ELEVATION



SECTION N-N



SECTION M-M

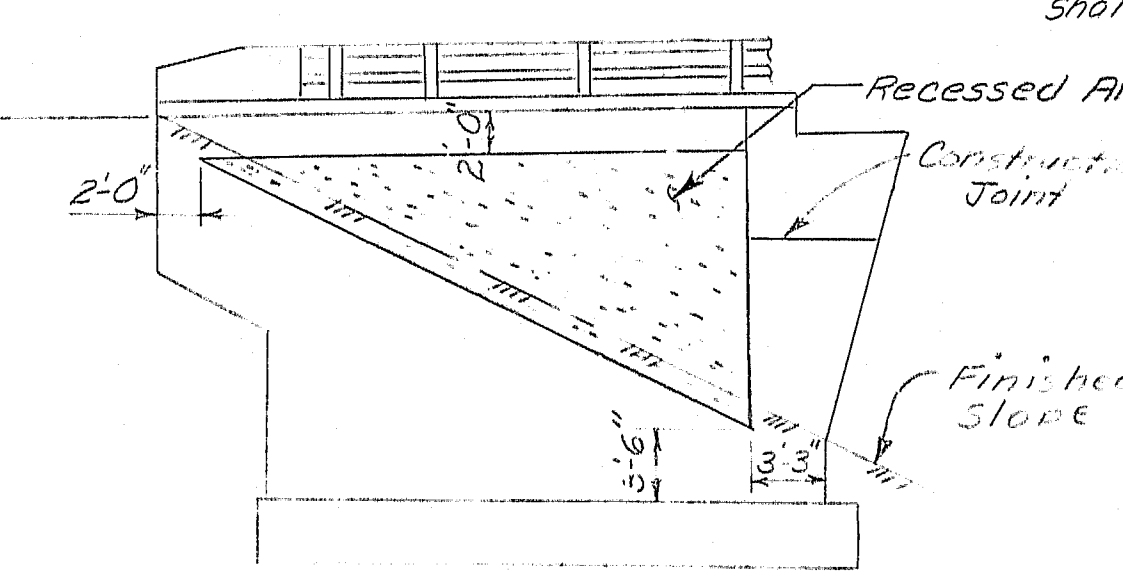


STEEL PLATE DETAIL

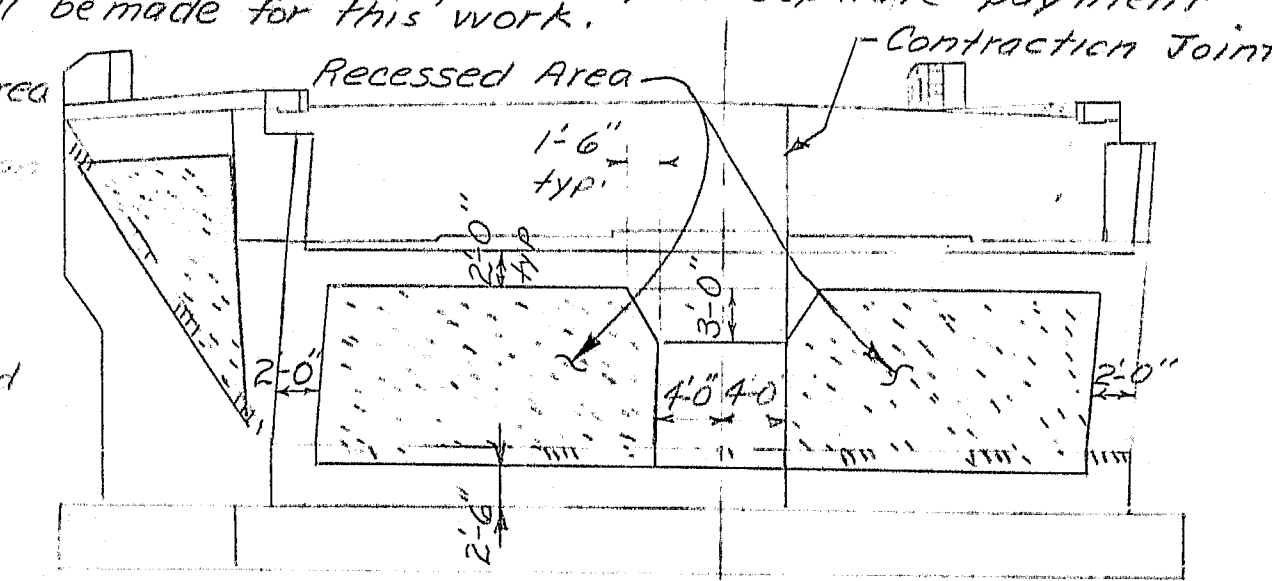
T\* - The full thickness of the elastomeric bearing chosen. (2 1/2" maximum)

Note: Elastomeric bearing dimensions are minimum. Laminated bearings of other dimensions may be used subject to the approval of the Engineer.

At the location of the elastomeric bearings the concrete bridge seats shall be dressed one inch larger all around than the elastomeric bearing. If dressed areas are below the surface of the surrounding bridge seat a small channel shall be cut to the edge of the bridge seat where required by the Engineer. The channel shall be 2" wide (min) and have a slope of 1/4" ft (min). No separate payment shall be made for this work.

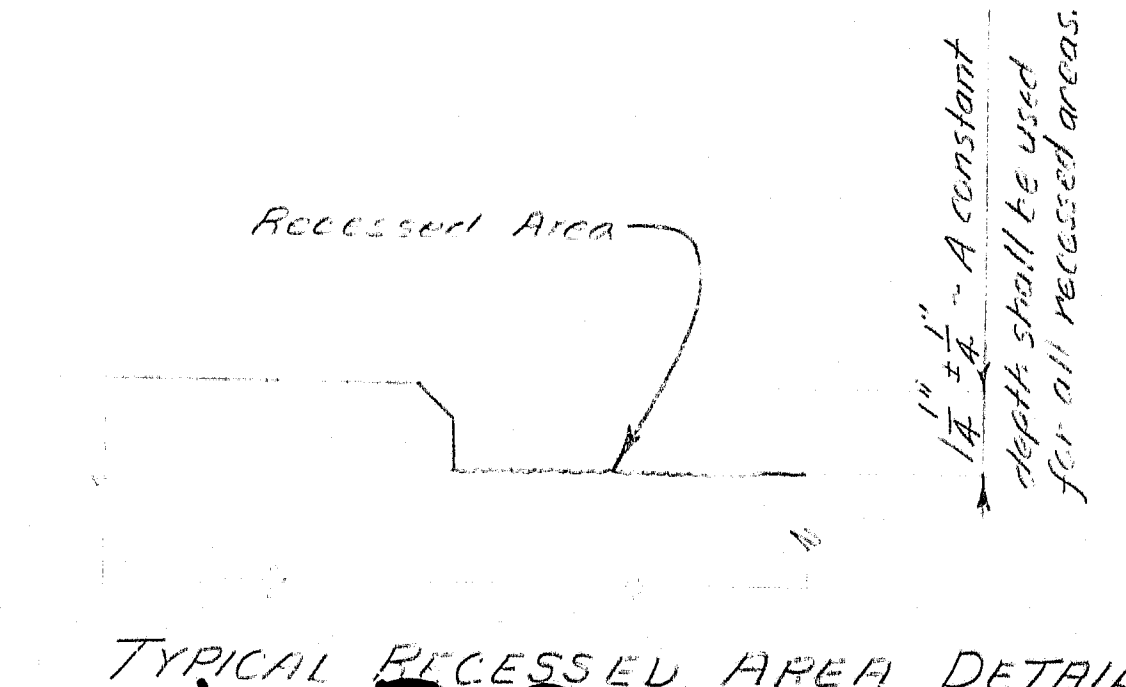


TYPICAL WING ELEVATION



TYPICAL ABUTMENT ELEVATION

ARCHITECTURAL TREATMENT - ABUTMENTS



TYPICAL RECESSED AREA DETAIL

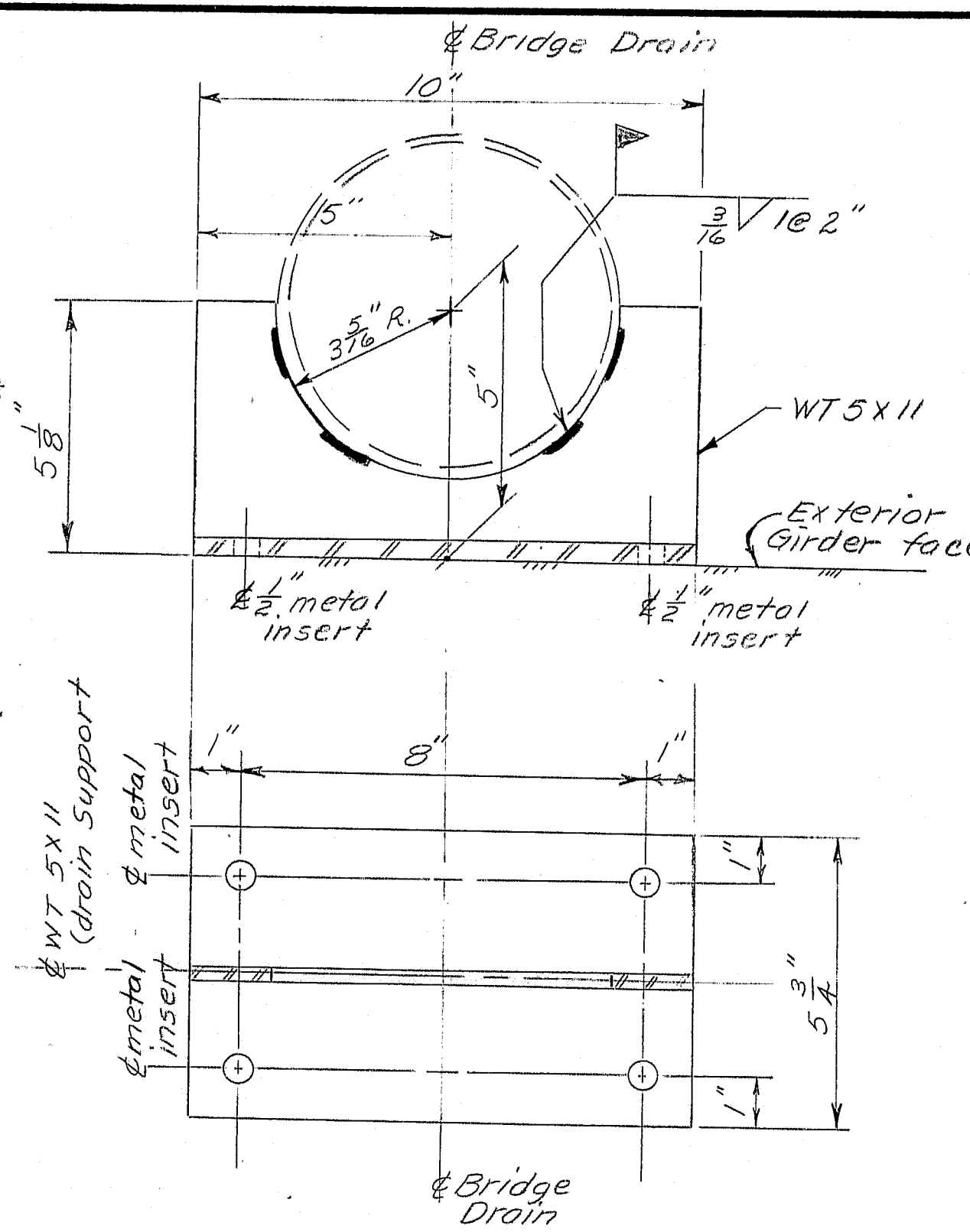
PROJECT DESIGN ENGINEER	DATE
DESIGN - DETAIL	BY
REVISIONS	BY
FIELD CHANGES	BY
PLANS	

BRUNING 44-132 42710

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
GREEN POINT ROAD BRIDGE
OVER
I-395
IN THE CITY OF
BREWER
CONCRETE ALTERNATE
End Diaphragms &
Architectural Treatment
SHEET 10 OF 17 AUGUSTA, MAINE

R89-419





F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-395-8(87)/176	14	84

- The Contractor shall submit plans of the supporting forms and falsework to the Engineer for approval prior to their construction.

9) In removing the supporting falsework the Contractor shall take special care that the dead loading of the concrete will be done gradually, using sand boxes or other methods as specified under Section 502.10(d). The method used by the Contractor shall be subject to the approval of the Engineer.

19	48rgs Abut <sup>2</sup>	10.) At the Contractor's option the bottom slab may be placed on a crown to match the top slab, as approved by the Engineer. Details for this option will be furnished by the Department at the Contractor's request.
----	----------------------------	---

152.34	152.48	
152.52	152.46	11) Payment for superstructure concrete will be made to the bottom of bottom slab under Item 502.37. Such payment will include the integral pier cap and the concrete end diaphragms.
152.50	152.45	

152.43	152.43	12.) The 1/2 inch steel plate and stud anchors at the abutment ends will be considered incidental to Item 502.37.
152.48	152.43	

152.4.6 152.4.1 13.) The superstructure concrete shall be constructed in placements as follows:  
the bottom slab; the webs and integral pier cap;  
the top slab and haunches; the end diaphragms;  
the concrete curbs. Each placement shall be made in one continuous operation, and the concrete shall be kept plastic until the entire placement has been completed.

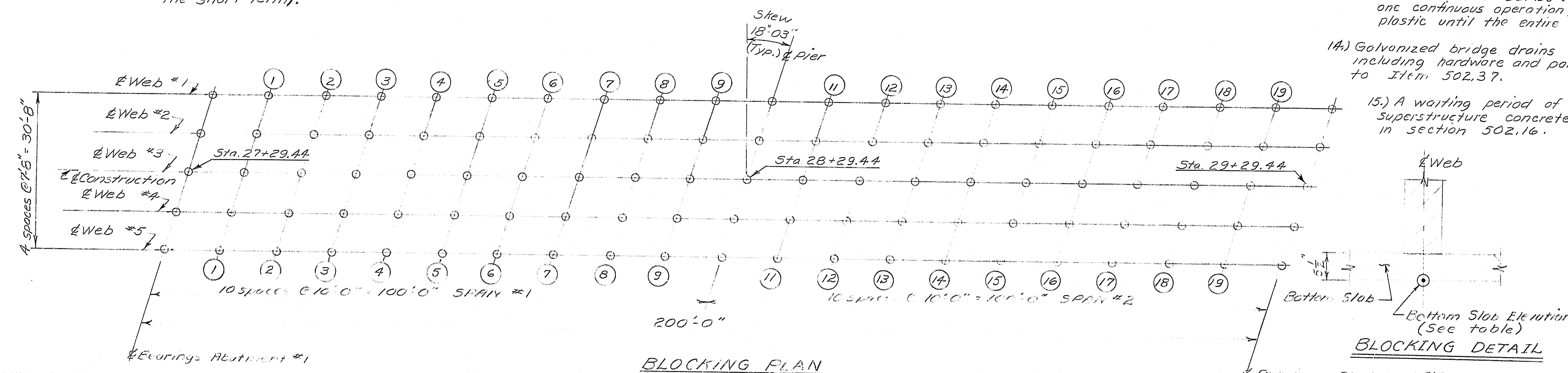
14) Galvanized bridge drains and the inspection manholes, including hardware and painting, will be considered incidental to Item 502.32.

15.) A waiting period of 7 days shall elapse between superstructure concrete placements and as specified in section 502.16.

		<u>DRAIN SUPPORT DETAIL</u>																				of the	
	¢ Brgs Abut. #1	1	2	3	4	5	6	7	8	9	¢ Pier	11	12	13	14	15	16	17	18	19	¢ Brgs Abut. #2		
Web #1	149.34	149.72	150.06	150.36	150.62	150.82	150.97	151.08	151.17	151.27	151.38	151.58	151.80	152.03	152.22	152.38	152.50	152.56	152.56	152.54	152.46		
Web #2	149.28	149.66	150.00	150.31	150.56	150.77	150.92	151.04	151.13	151.22	151.34	151.54	151.77	151.99	152.19	152.36	152.47	152.53	152.54	152.52	152.4		
Web #3	149.22	149.60	149.94	150.25	150.51	150.71	150.87	150.99	151.09	151.18	151.30	151.50	151.73	151.96	152.16	152.33	152.45	152.51	152.52	152.50	152.4		
Web #4	149.15	149.54	149.88	150.19	150.46	150.66	150.82	150.94	151.04	151.14	151.26	151.47	151.70	151.93	152.13	152.30	152.42	152.49	152.50	152.48	152.4		
Web #5	149.09	149.47	149.82	150.14	150.40	150.61	150.77	150.90	150.99	151.09	151.21	151.43	151.66	151.89	152.10	152.27	152.39	152.46	152.48	152.46	152.4		

NOTE: The bottom of slab elevations, as shown, have been adjusted to compensate for dead load deflections, both immediate and long term shrinkage and creep. (long term taken as twice the short term).

BOTTOM OF SLAB ELEVATIONS



BLOCKING DETAIL

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD BRIDGE  
OVER  
J-395  
IN THE CITY OF  
BREWER  
CONCRETE-ALTERNATE  
Bottom of slab Elevation: 6 107.00

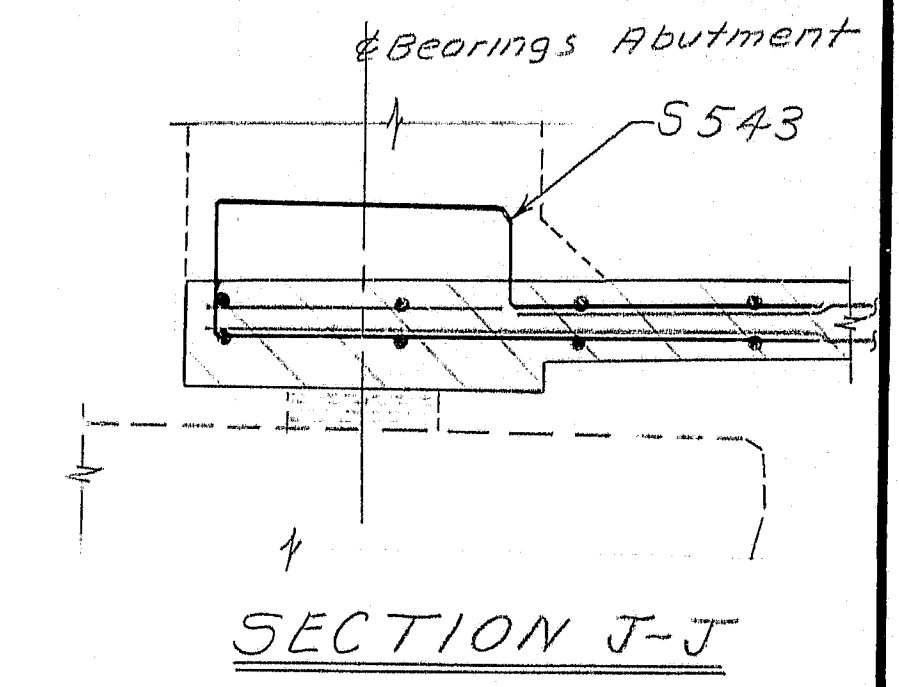
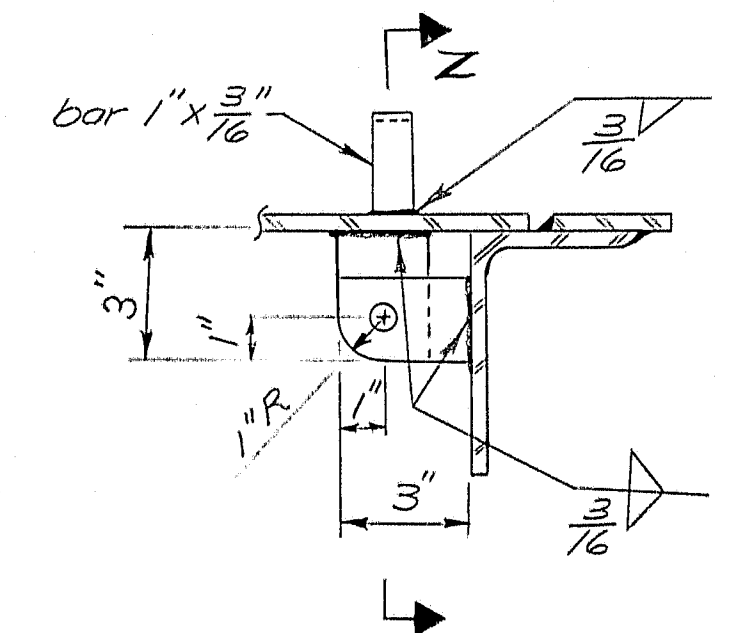
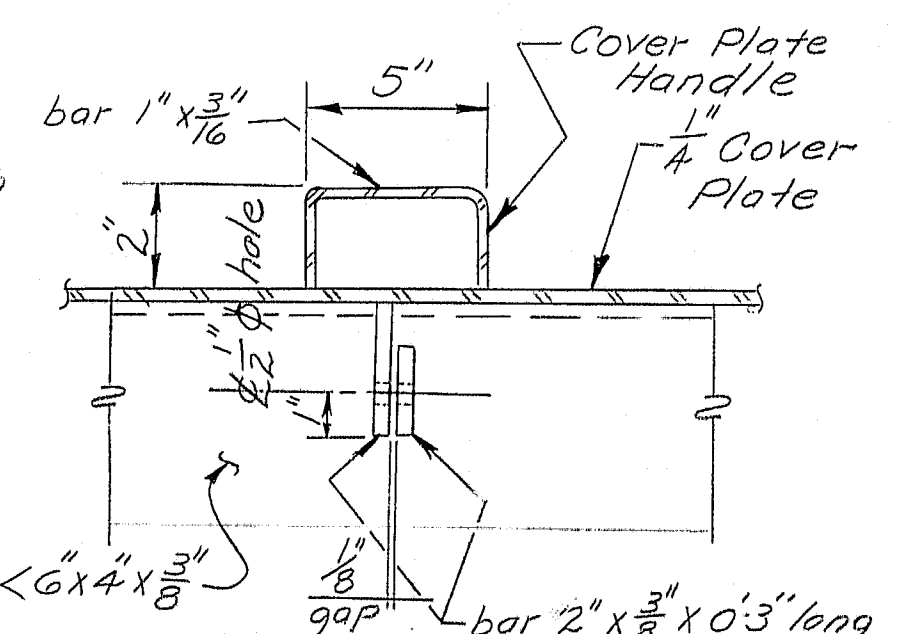
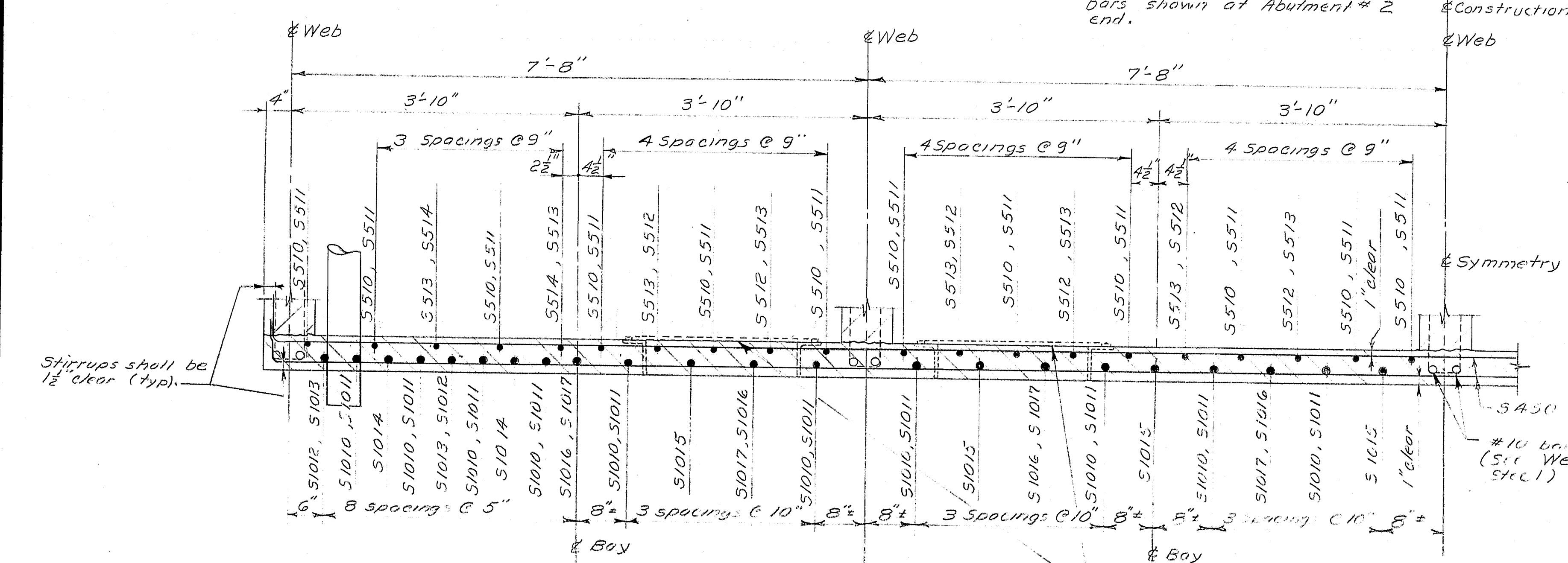
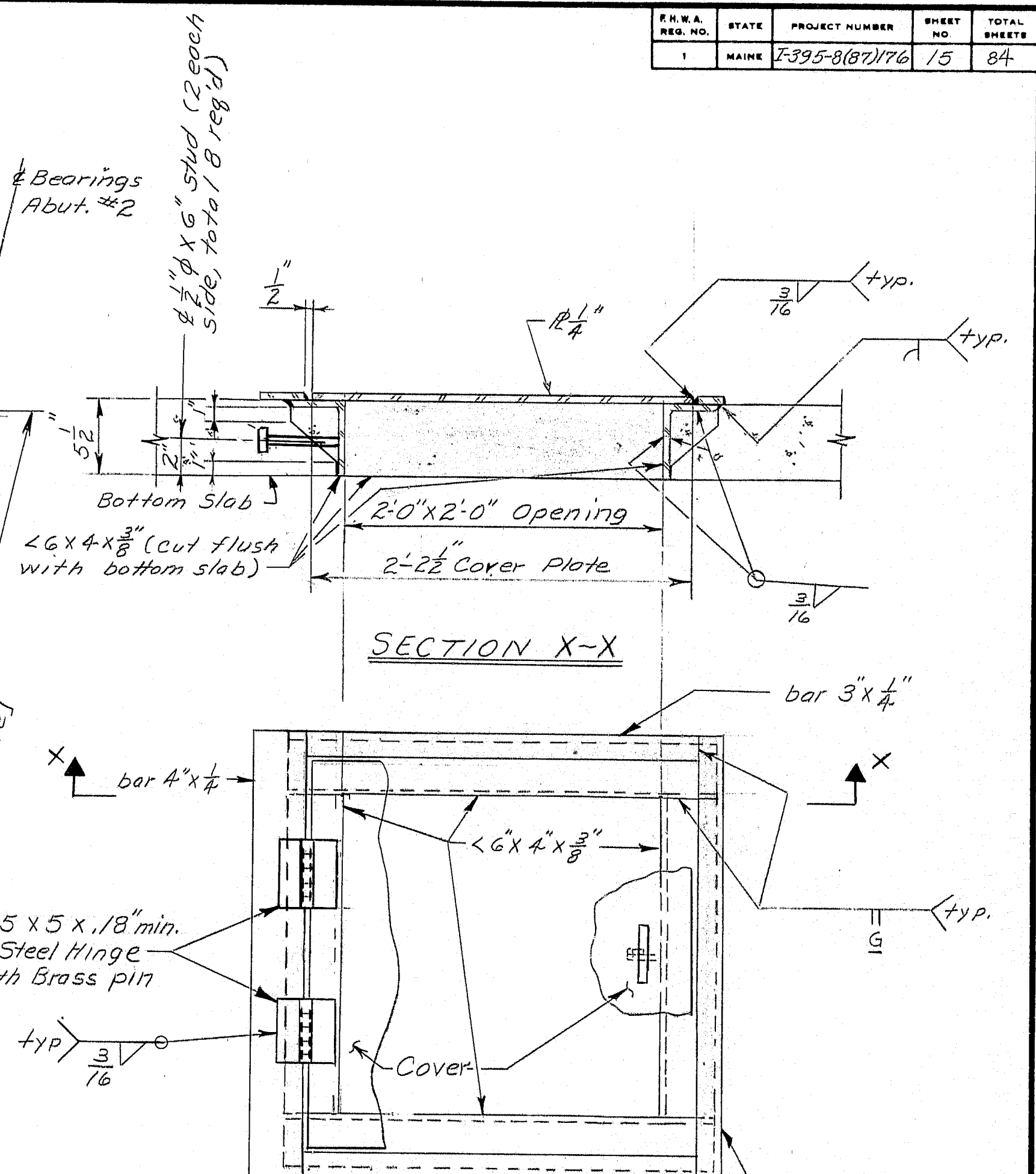
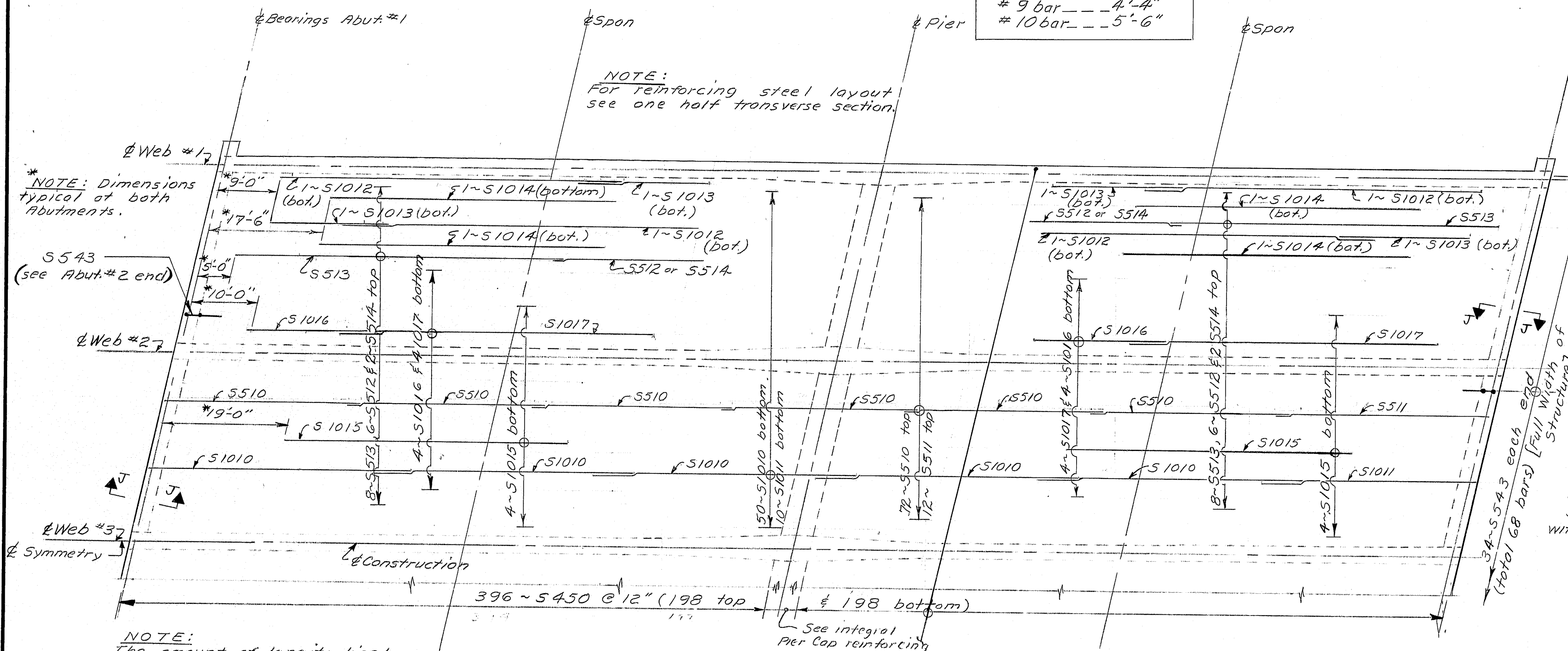
SHEET 11 OF 17 AUGUSTA, MAINE

**R89-420**



F.R.M.A.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-395-8(B7)176	15	84

TABLE OF SPLICES	
(For $f'_c = 4000 \text{ psi}$ )	
#4 bar	1'-3"
#5 bar	1'-6"
#9 bar	4'-4"
#10 bar	5'-6"



PROJECT DESIGN ENGINEER	DATE
BY	7/23
CHECKED	
REVISIONS	
FIELD CHANGES	

BRIDGE 44132 25710

R89-421

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
Bottom Slab  
SHEET 12 OF 17 AUGUSTA, MAINE







STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
MAINE	I-395-B(81)176	17	84

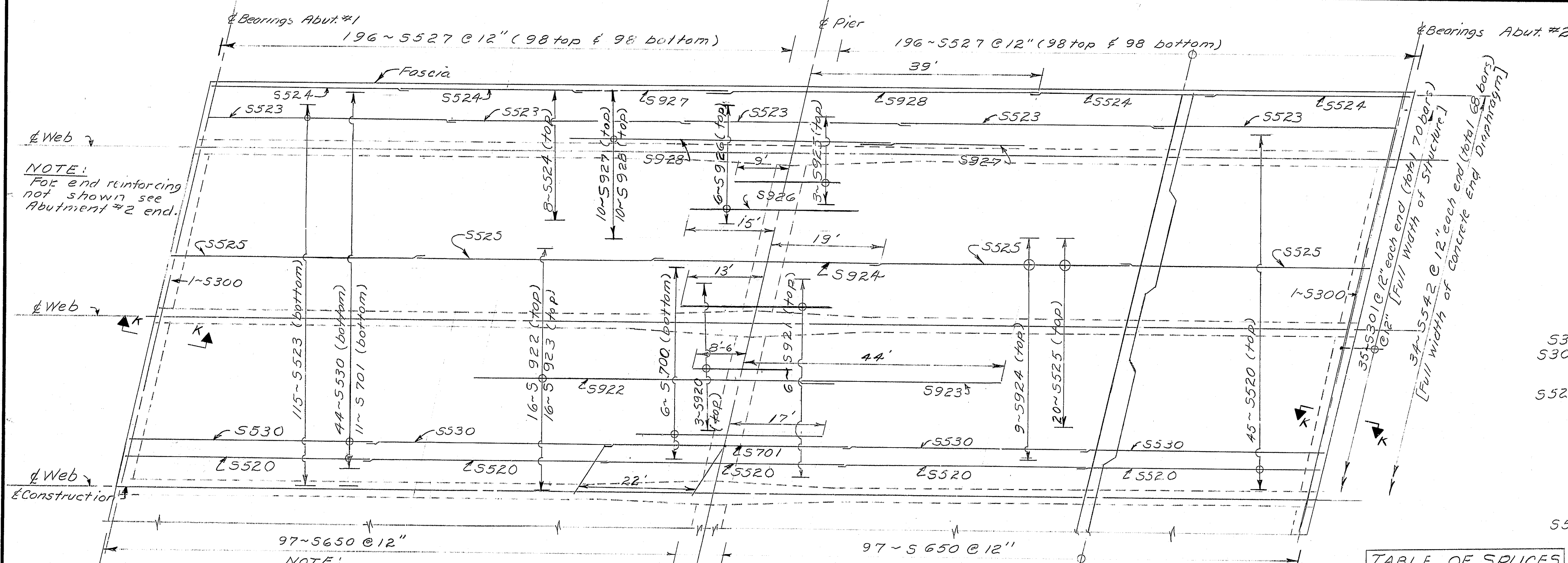
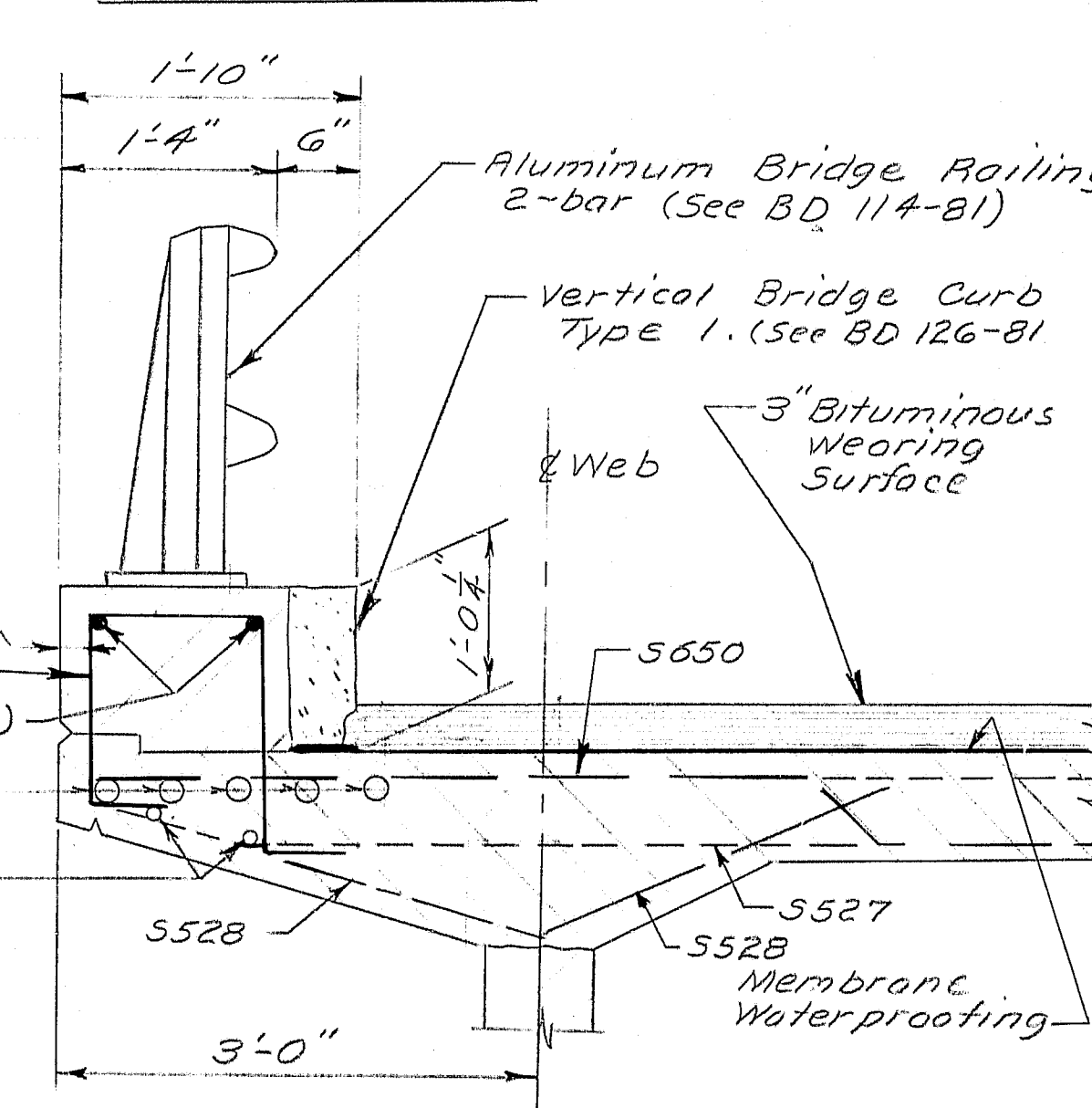
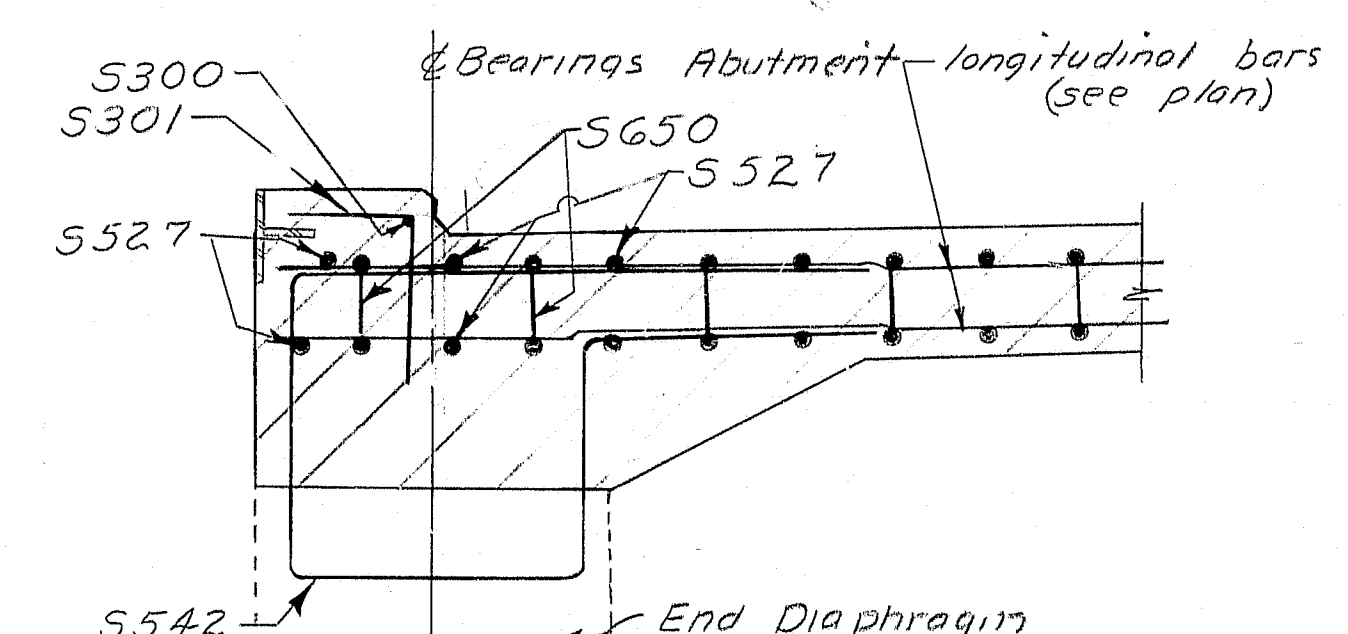
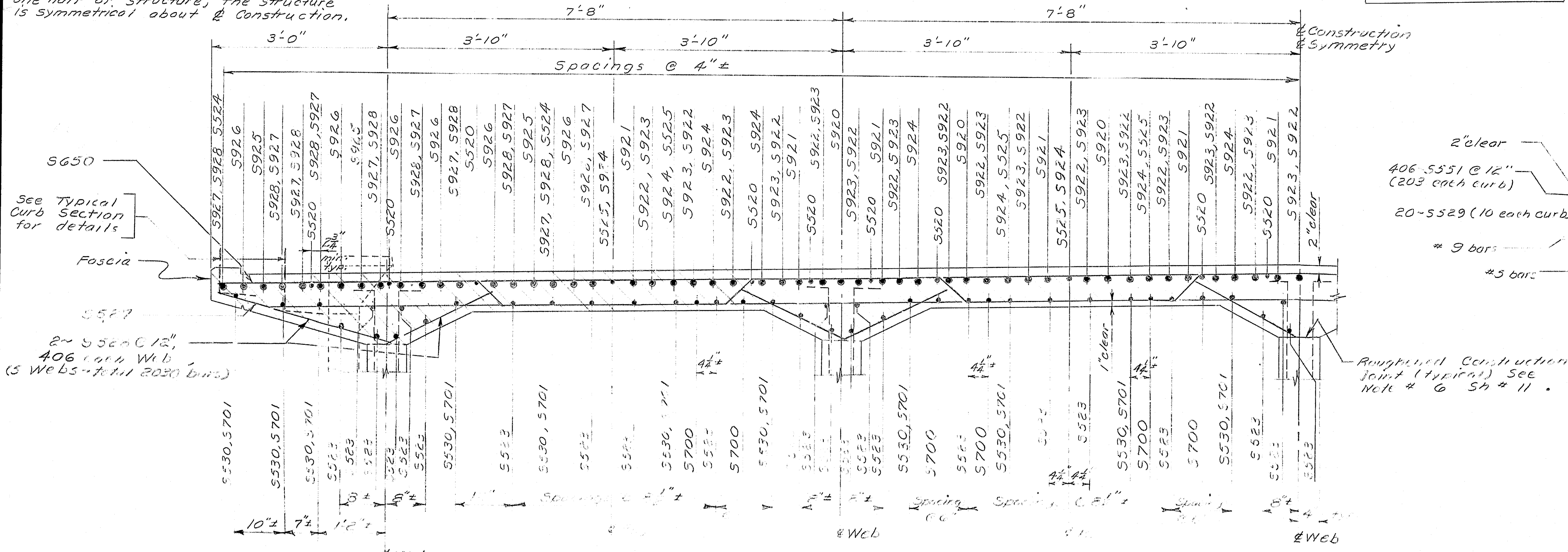


TABLE OF SPLICES  
(for  $f'_c = 4000 \text{ psi}$ )

#5 bar	1'-6"
#9 bar	4'-4"



PROJECT DESIGN ENGINEER	DATE
BY [Signature]	5/7/23
DESIGN - CHECKED	REVISIONS
PLANS	FIELD CHANGES

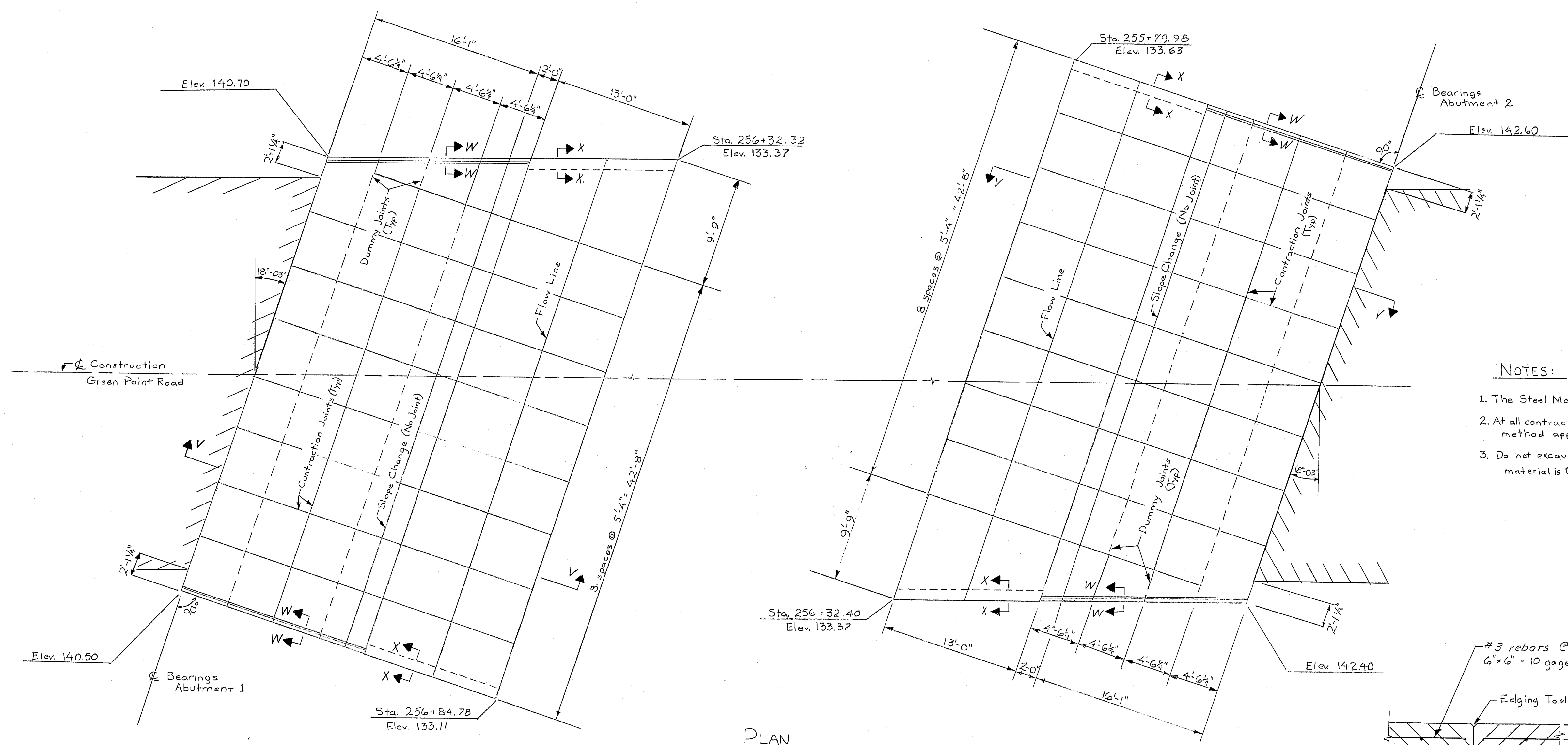


STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
I-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
Top 5106  
SHEET 14 OF 17 AUGUSTA, MAINE

R89-423



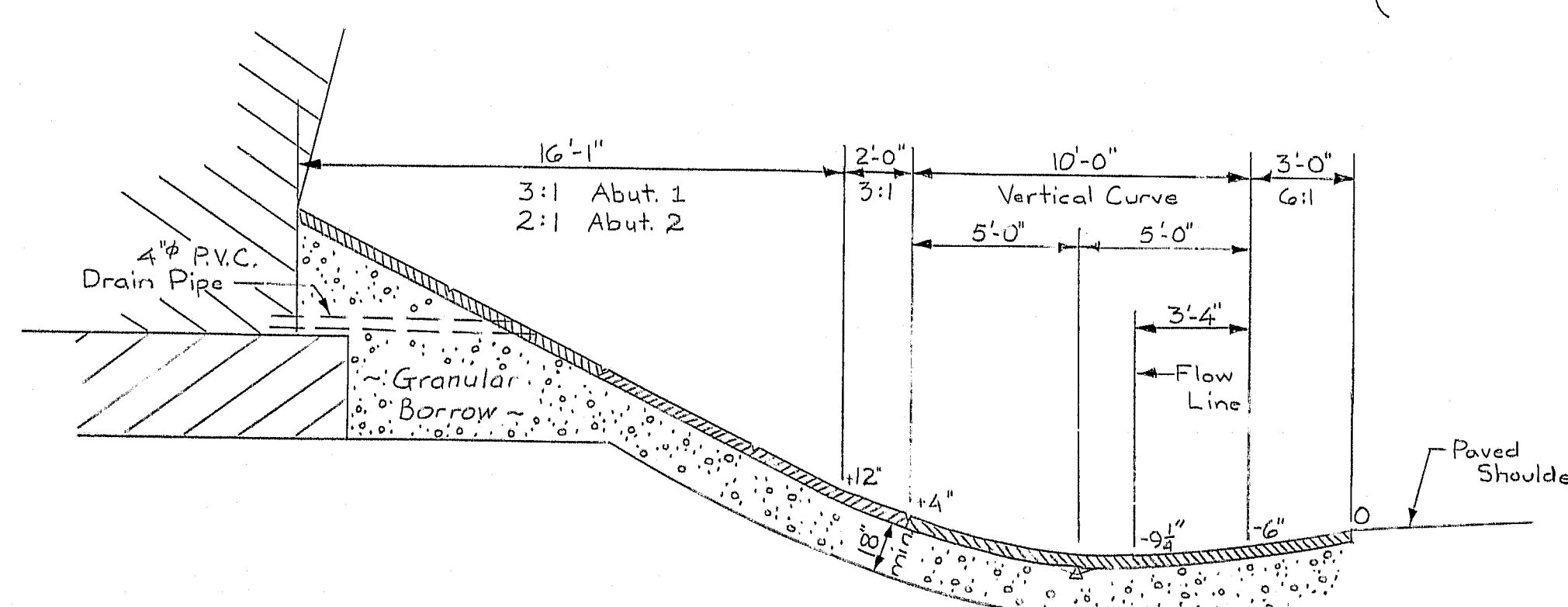
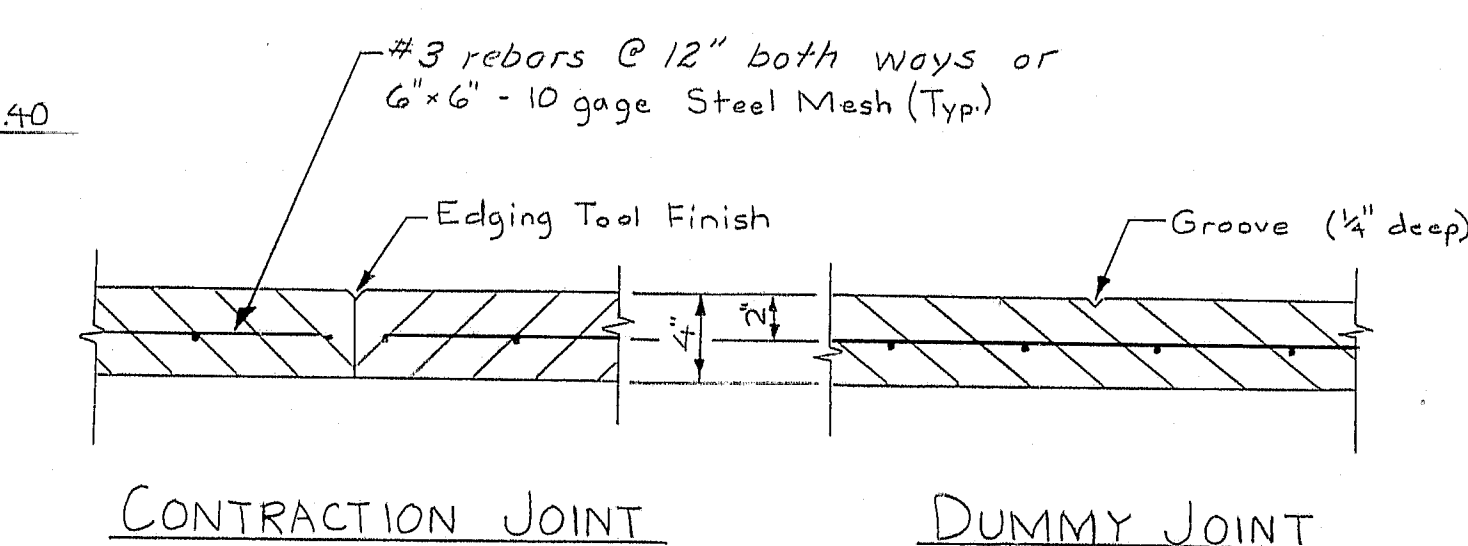
F.H.W.A. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	1395-8(22)76	18	84



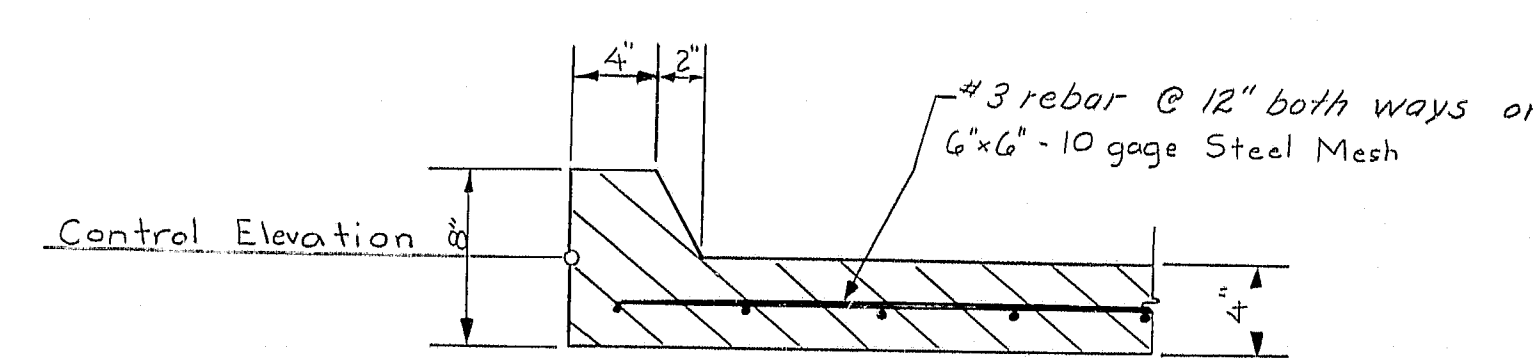
PLAN  
(Dimensions are Horizontal)

NOTES:

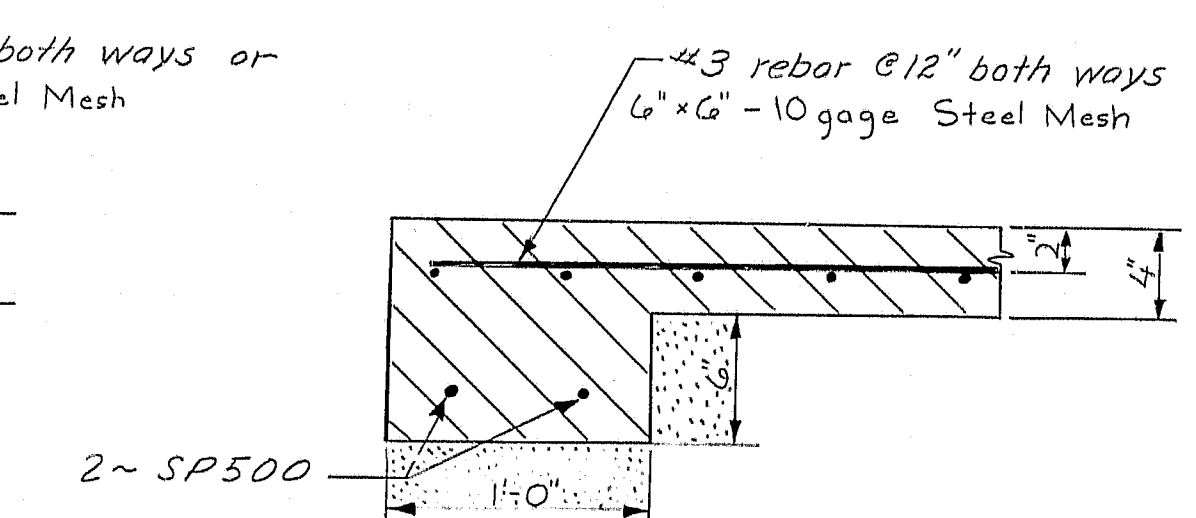
1. The Steel Mesh shall not pass through any contraction joints
2. At all contraction joints, break bond between sections by a method approved by the engineer.
3. Do not excavate for granular borrow where the existing material is found suitable in the opinion of the engineer.



SECTION V-V



SECTION W-W



SECTION X-X

PROJECT DESIGN ENGINEER	BY	DATE
DESIGN-DETAILED	K.D.P.	7/83
CHECKED		
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710-1

R89-424

STATE OF MAINE DEPARTMENT OF TRANSPORTATION
GREEN POINT ROAD OVER I-395 IN THE CITY OF BREWER CONCRETE ALTERNATE CONCRETE SLOPE PROTECTION
SHEET 15 OF 17 AUGUSTA, MAINE

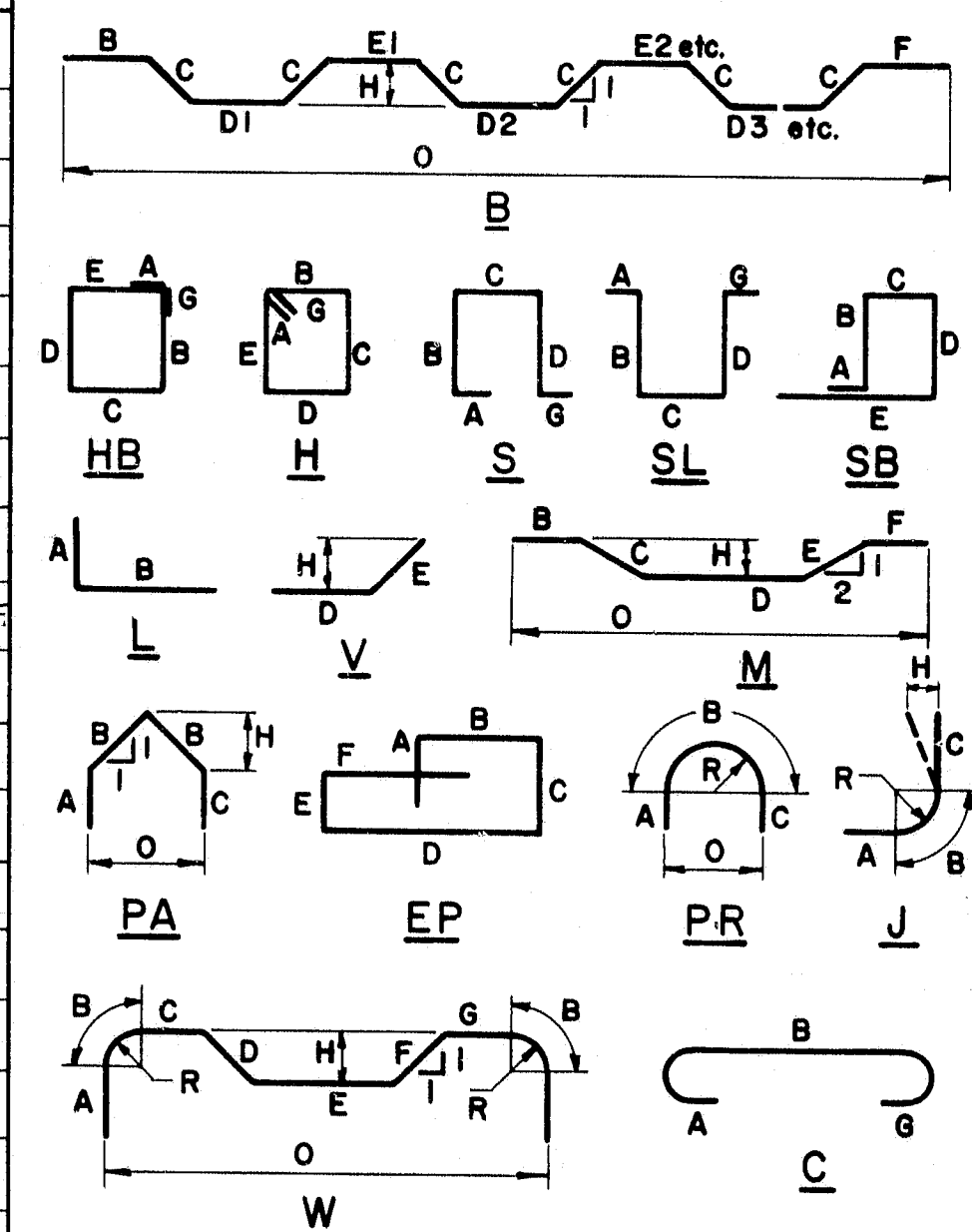


# REINFORCING STEEL SCHEDULE

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							
ABUTMENT FOOTINGS				A600	26	14'-3"	Vert. Wings F.F.									ABUTMENT #1																	
				A601	24	14'-6"	Vert. backwall F.F.					A501	24	9'-0"	S		2'-4	1'-2"	5'-6"								Vert., backwall						
F600	12	25'-3	longi., wings	A602	4	15'-3"	Vert. Haunch F.F.					A503	24	5'-0"	V				2'-0"	3'-0"			0'-6"				Vert. backwall F.F.						
F601	16	26'-3	longi., wings																														
F602	16	27'-3	longi., wings	A703	14	8'-6"	Wings F.F. Vert					A513	42	11'-4"	S	—	5'-0"	1'-4"	5'-0"			—					Vertical Wings						
				A800	12	10'-0"	Wings F.F. Vert					A514	34	7'-6"	H	0'-5"	1'-1"	2'-3"	1'-1"	2'-3"		0'-5"					Vertical Wings						
F603	12	25'-0"	longi. wings	ABUTMENT #2								A515	24	7'-0"	L	1'-3"	5'-9"										Horiz. Backwall						
F604	156	6'-6"	transv. wings									A516	20	8'-6"	V				7'-0"	1'-6"			1'-1"				Horiz. bk wall, haunch						
F605	36	42'-3"	longi.									A521	38	17'-2"	A	5'-0"	9'-4"	2'-11"					2'-3"				Vert. breastwall						
F606	124	7'-9"	transv.	B500	74	3'-0"	dowels					A603	23	9'-6"	L	1'-0"	8'-6"										dowel, backwall, F.F.						
				B502	24	2'-9"	Vert., backwall, N.F.	APPROACH SLABS								A700	24	4'-9"	L	1'-2"	3'-7"							dowel, backwall F.F.					
								A5400	32	20'-0"	transverse	A750	14	5'-3"	L	1'-2"	4'-1"										dowel, wings, F.F.						
ABUTMENT #1				B507	24	22'-0"	Horiz., Wings, E.F.	B5401	32	15'-0"	"	A850	14	6'-7"	L	1'-4"	5'-3"										dowel, wings, F.F.						
				B508	4	24'-6"	"					A950	24	8'-3"	L	1'-7"	6'-8"										dowel, wings, F.F.						
A500	74	3'-0"	dowels	B509	24	27'-0"	"	A5600	128	15'-0"	longitudinal	ABUTMENT #2																					
A502	24	2'-9"	Vert. backwall N.F.	B510	4	29'-0"	Horiz., Wings, top					B501	24	9'-0"	S		2'-4	1'-2"	5'-6"								Vert., backwall						
												B503	24	5'-0"	V				2'-0"	3'-0"			0'-6"				Vert. backwall F.F.						
				B511	16	7'-0"	Vert., Wings, butterfly																										
A507	20	22'-0"	Horiz. Wings E.F.	B512	34	15'-0"	Vert., Wings N.F.					B513	42	11'-4"	S	—	5'-0"	1'-4"	5'-0"			—					Vert. Wings						
A508	4	24'-6"	"									B514	34	7'-6"	H	0'-5"	1'-1"	2'-3"	1'-1"	2'-3"		0'-5"					Vert. Wings						
A509	24	27'-0"	"									B515	26	7'-0"	L	1'-3"	5'-9"										Horiz. backwall						
				B517	4	4'-0"	Horiz., Wings, F.F.					B516	22	8'-6"	V				7'-0"	1'-6"			1'-1"				Horiz. bk wall, haunch						
A510	4	29'-0"	Horiz., Wings, top	B518	6	17'-8"	Horiz. Curtain wall					B521	38	17'-10"	A	5'-0"	9'-2"	3'-8"					2'-3"				Vert. breastwall N.F.						
A511	16	7'-0"	Vert., Wings, butterfly	B519	8	6'-0"	Horiz. Curtain wall					B603	23	9'-6"	L	1'-0"	8'-6"										dowel, backwall, F.F.						
A512	34	14'-3"	Vert., wings, N.F.									B700	24	4'-9"	L	1'-2"	3'-7"										dowel, backwall, F.F.						
				B530	5	23'-4"	Horiz., backwall, etc.																										
				B531	5	14'-6"	Horiz., bridge seat					B750	14	5'-3"	L	1'-2"	4'-1"										dowel, wings, F.F.						
A517	4	4'-0"	Horiz., Wings, F.F.	B532	13	23'-9"	Horiz., backwall, F.F.					B850	14	6'-7"	L	1'-4"	5'-3"										dowel, wings, F.F.						
A518	6	16'-9"	Vert., Curtain wall	B533	13	14'-1"	Horiz., backwall F.F.					B950	24	8'-3"	L	1'-7"	6'-8"										dowel, wings, F.F.						
A519	8	6'-0"	Horiz., curtain wall																														
				B534	2	23'-0"	Horiz., breastwall, N.F.	END POSTS								END POSTS																	
				B535	3	22'-10"	"					EP402	16	4'-9"	S		2'-1"	0'-7"	2'-1"								End Post						
A530	5	23'-4"	Horiz. backwall N.F.	B536	2	22'-6"	"	EP401	32	1'-10"	End Post Dowels	EP403	16	4'-9"	H	0'-4"	1'-0"	1'-0"	1'-0"	1'-0"		0'-4"					"						
A531	5	14'-6"	Horiz. bridge seat	B537	3	22'-3"	"	EP405	16	1'-5"	End Post	EP404	16	3'-1"	S		1'-3"	0'-7"	1'-3"								"						
A532	13	21'-9"	Horiz. breastwall, etc.																														
				B538	2	15'-0"	Horiz., breastwall, N.F.	EP308	16	4'-0"	End Post	EP408	12	4'-3"	S		1'-10"	0'-7"	1'-10"								End Post						
A533	13	13'-0"	Horiz., backwall, F.F.	B539	3	15'-2"	"					EP409	8	4'-2"	S		1'-10"	0'-6"	1'-10"								"						
A534	2	23'-0"	Horiz., breastwall, N.F.	B540	2	15'-6"	"					EP410	4	4'-6"	S		1'-10"	0'-10"	1'-10"								"						
A535	3	22'-10"	"	B541	3	15'-9"	"																										
A536	2	22'-6"	Horiz., breastwall, N.F.									EP501	16	5'-3"	V				3'-0"	2'-3"			0'-4"				End Post						
A537	3	22'-3"	"	B600	26	15'-3"	Vert., Wings, F.F.					EP502	12	4'-11"	S		1'-11"	0'-7"	1'-11"			0'-6"				"							
A538	2	15'-0"	"	B601	24	15'-6"	Vert., backwall, F.F.					EP503	8	4'-10"	S		1'-11"	0'-6"	1'-11"			0'-6"				"							
				B602	4	15'-10"	Vert., Haunch F.F.					EP504	4	6'-5"	H	0'-5"	1'-11"	0'-10"	1'-11"	0'-10"		0'-5"				"							
A539	3	15'-2"	Horiz., breastwall N.F.																														
A540	2	15'-6"	"	B703	14	8'-6"	Wings, F.F. Vert																										
A541	3	15'-9"	"	B800	12	10'-0"	Wings, F.F. Vert																										

F.H.W.A. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	I-395-B(20)76	19	84

## TYPE-BENDING DIAGRAMS



All dimensions are out to out of reinf. bar.  
Bending details and hooks shall conform to the recommendations of the current revision of the ACI Standard 318.

Reinforcing Bar: ASTM A615 Grade 60

## GENERAL NOTES

- First digit(s) following the letter of the Mark indicates size of reinf. bar.  
Mark (A502) bar size - #5  
Mark (P1001) bar size - #10  
Mark (S603) bar size - #6
- Letter of Marks A, P & S locates bars of Abutments, Piers, and Superstructure parts respectively.

Revised ACI Standard 5-12-83

STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION

GREEN POINT ROAD BRIDGE

OVER  
I-395

IN THE CITY OF  
BREWER

CONCRETE-ALTERNATE

Reinforcing Steel Schedule  
SHEET 16 OF 17 AUGUSTA, MAINE

R89-425

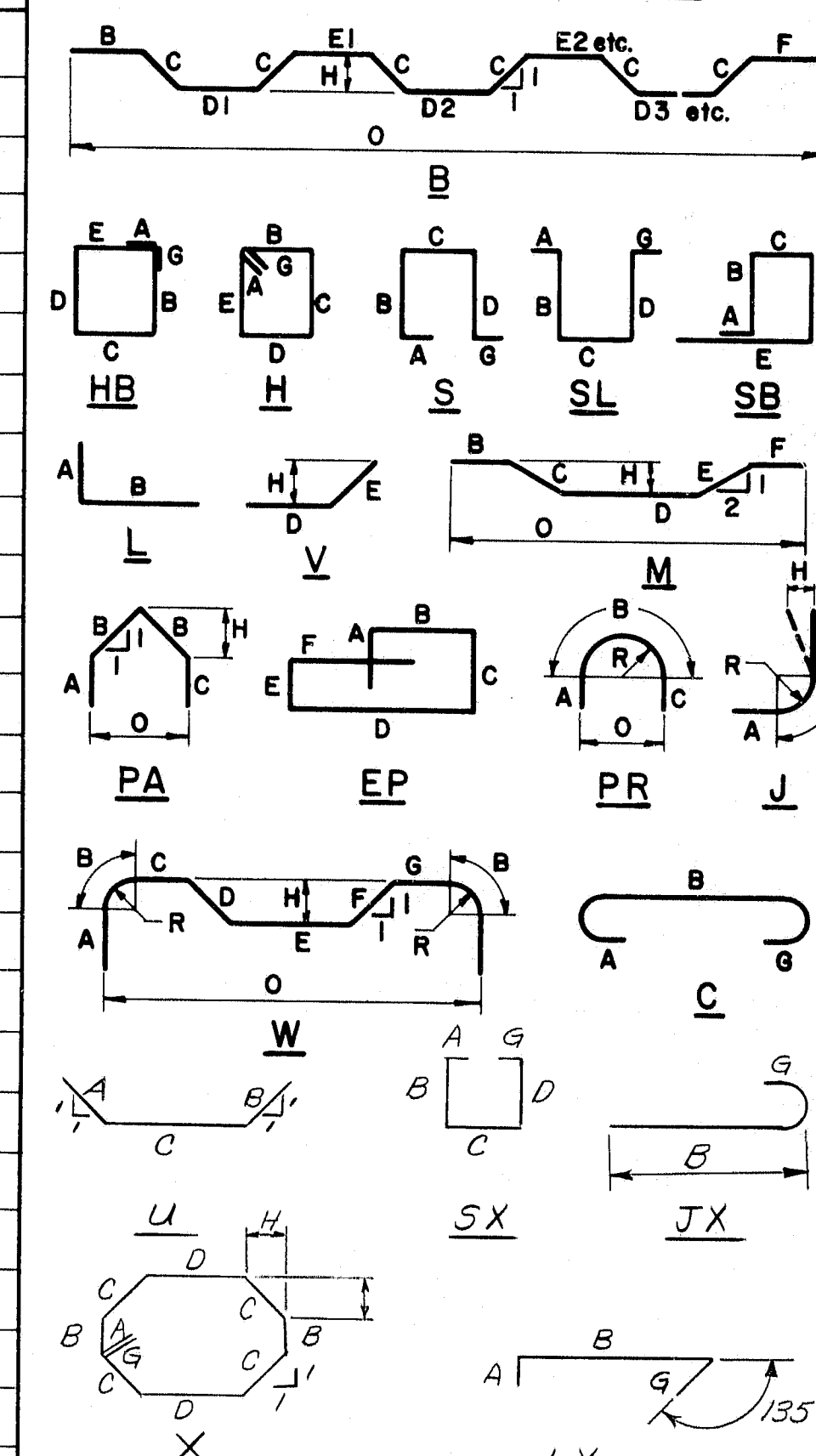


FHWA	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	2-395-3(27)76	20	84

# REINFORCING STEEL SCHEDULE

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	
		PIER				BOTTOM SLAB					TOP SLAB &					PIER											
											END DIAPHRAGM	P300	13	18'-8"	H	0'-4"	3'-0"	6'-0"	3'-0"	6'-0"		0'-4"					horiz. , shaft
P600	15	11'-6"	top footing	5510	144	30'-0"	longi. , top	5300	2	38'-2"	end of slab	P301	18	16'-4"	X	0'-6"	(2X) 0'-8"	(4X) 1'-8"	(2X) 3'-8"				0'-6"	1'-2"			horiz. , shaft
P601	12	14'-6"	top footing	5511	24	31'-6"	longi. , top					P302	18	4'-0"	LX	0'-6"	3'-0"					0'-6"					horiz. , shaft
				5512	24	23'-0"	longi. , top	5520	90	42'-0"	longi. , top	P303	10	16'-4"	SX	0'-8"	6'-0"	3'-0"	6'-0"				0'-8"				horiz. , shaft
P603	4	7'-3"	shaft Vert.					5523	230	42'-0"	longi. , bottom																
				5513	32	50'-0"	longi. , top																				
P802	18	21'-3"	Shaft Vert.	5514	8	40'-0"	longi. , top	5524	16	32'-6"	longi. , top	P602	8	9'-0"	JX		8'-4"					0'-8"					vert. Shaft
								5525	40	43'-6"	longi. , top																
P1000	12	14'-6"	bottom footing	51010	100	40'-0"	longi. , bottom					P801	38	6'-7"	L	1'-4"	5'-3"										dowel
				51011	20	30'-0"	longi. , bottom	5527	392	38'-2"	transv. , top	P803	20	23'-1"	V				13'-1"	10'-0"			4'-6"				vert. , shaft
P1100	15	11'-6"	bottom footing					5528	2030	2'-3"	transv. Web haunch																
				51012	8	60'-0"	longi. , bottom																				
				51013	8	13'-0"	longi. , bottom	5529	20	42'-6"	longi. , curbs																
								5530	88	42'-3"	longi. , bottom	P450	5	34'-7"	SL		1'-0"	32'-7"	1'-0"								longi.
INTEGRAL PIER CAP				51014	8	46'-0"	longi. , bottom	5540	16	34'-3"	end diaphragm	P550	6	34'-7"	SL		1'-0"	32'-7"	1'-0"								longi.
				51015	16	43'-0"	longi. , bottom	5700	12	34'-0"	longi. , bott.	P551	84	12'-4"	S	0'-5"	4'-5"	2'-8"	4'-5"		0'-5"						vert.
P500	4	32'-7"	longitudinal					5701	22	44'-0"	longi. , bott.	P711	8	8'-0"	U	3'-0"	2'-0"	3'-0"									horiz. @ Opening
P501	22	4'-8"	transverse	51016	16	20'-0"	longi. , bottom																				
P502	7	38'-3"	longitudinal	51017	16	48'-0"	longi. , bottom	5920	6	17'-0"	longi. , top																
								5921	12	26'-0"	longi. , top																
P612	15	38'-3"	longitudinal					5922	32	60'-0"	longi. , top																
P700	6	33'-0"	longi.					5923	32	33'-0"	longi. , top																transverse
P701	6	32'-7"	longi.					5924	18	38'-0"	longi. , top																end of slab
								5925	6	18'-0"	longi. , top																
				5400	35	40'-0"	longi. , over pier																				
P900	5	33'-0"	longi.	5500	250	43'-6"	longitudinal	5926	12	30'-0"	longi. , top																
P901	6	23'-0"	longi.	51000	50	44'-10"	longi. bottom slab	5927	20	27'-0"	longi. , top																
P902	6	27'-0"	longi.					5928	20	56'-0"	longi. , top																
												5452	224	10'-9"	SL	0'-5"	4'-9"	0'-5"	4'-9"		0'-5"						vert. , Web #1 & #45
												5453	248	11'-1"	SL	0'-5"	4'-11"	0'-5"	4'-11"		0'-5"						vert. , Web #2 & #44
												5454	124	11'-5"	SL	0'-5"	5'-1"	0'-5"	5'-1"		0'-5"						vert. , Web #3
												5552	268	10'-9"	SL	0'-5"	4'-9"	0'-5"			0'-5"						vert. , Web #1 & #45
												5553	284	11'-1"	SL	0'-5"	4'-11"	0'-5"			0'-5"						vert. , Web #2 & #44
												5554	142	11'-5"	SL	0'-5"	5'-1"	0'-5"			0'-5"						vert. , Web #3
												5555	24	8'-0"	U	3'-6"	1'-0"	3'-6"									Web Opening
																		</									

## TYPE-BENDING DIAGRAMS



All dimensions are out to out of reinf. bar.  
Bending details and hooks shall conform to the recommendations of the current revision of the ACI Standard 318.  
Reinforcing Bar: ASTM A615 Grade 60

## GENERAL NOTES

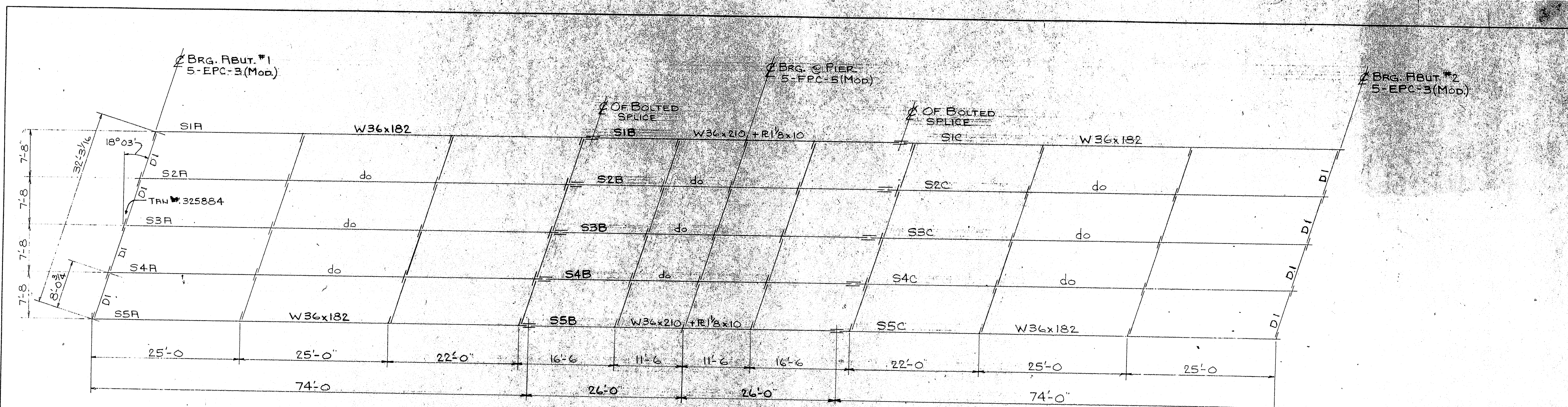
- First digit(s) following the letter of the Mark indicates size of reinf. bar.  
Mark (A502) bar size - #5  
Mark (P1001) bar size - #10  
Mark (S603) bar size - #6
- Letter of Marks A, P & S locates bars of Abutments, Piers, and Superstructure parts respectively.

Revised ACI Standard 5-12-83

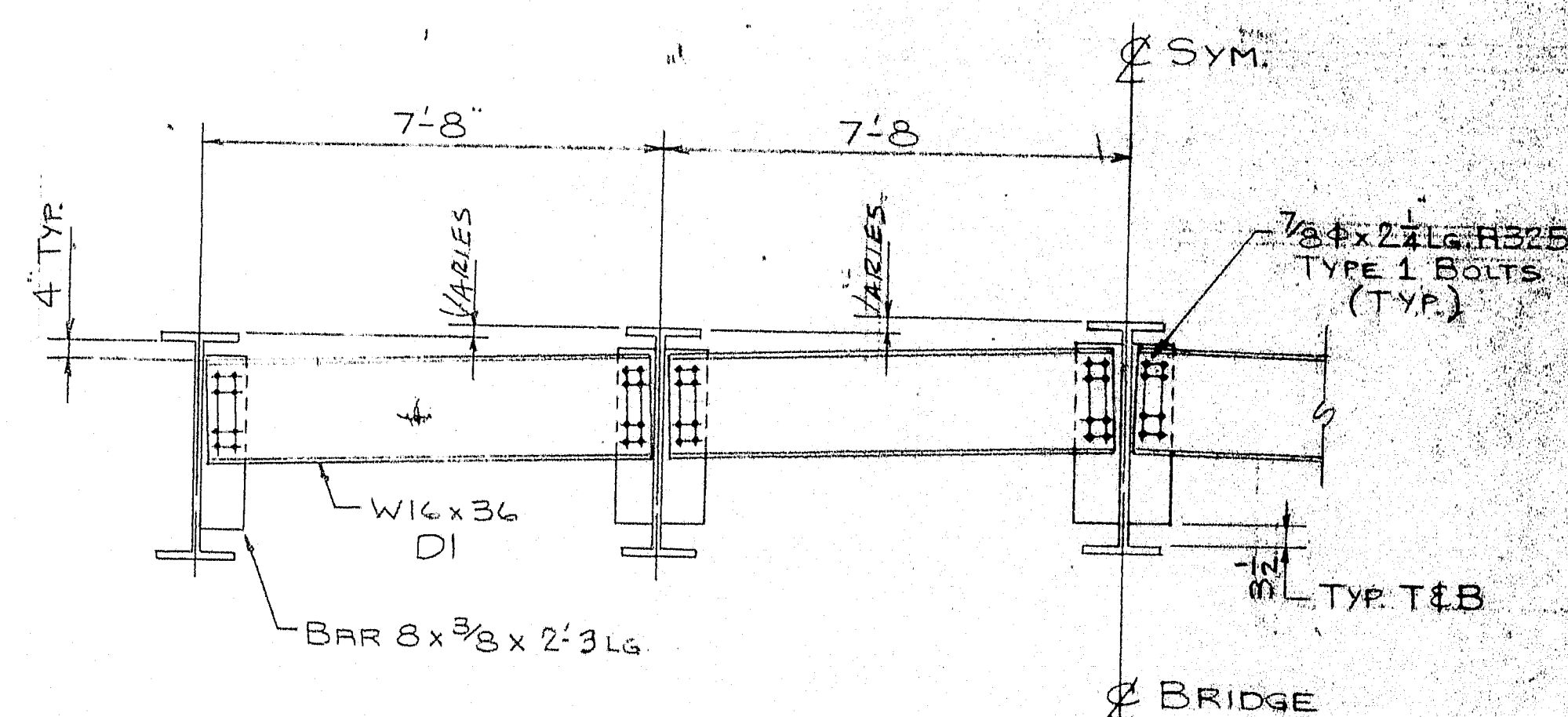
STATE OF MAINE  
DEPARTMENT OF TRANSPORTATION  
GREEN POINT ROAD BRIDGE  
OVER  
IN-395  
IN THE CITY OF  
BREWER  
CONCRETE ALTERNATE  
Reinforcing Steel Schedule  
SHEET 17 OF 17 AUGUSTA, MAINE

R89-426

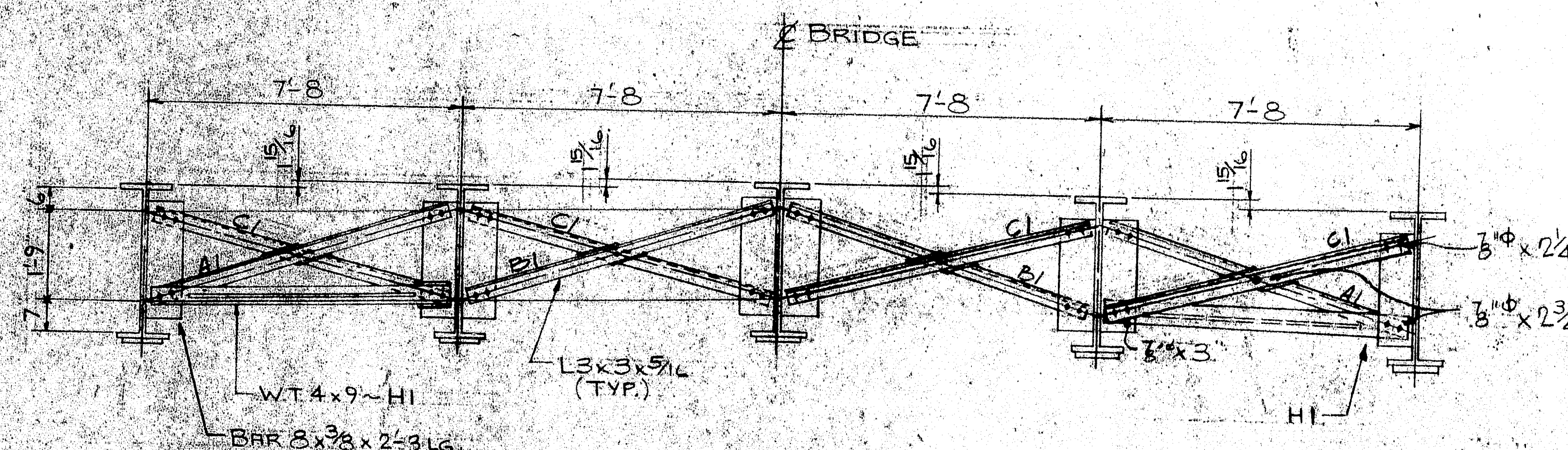




— FRAMING PLAN —



— TYPICAL SECTION AT END DIAPHRAGMS —



— TYPICAL SECTION AT INTERMEDIATE DIAPHRAGMS —

Br.# 1563

FORM BRACKET HOLES ARE TO BE FILLED W/ 1/2" CARTRIDGE BOLTS HELD ON OUTSIDE

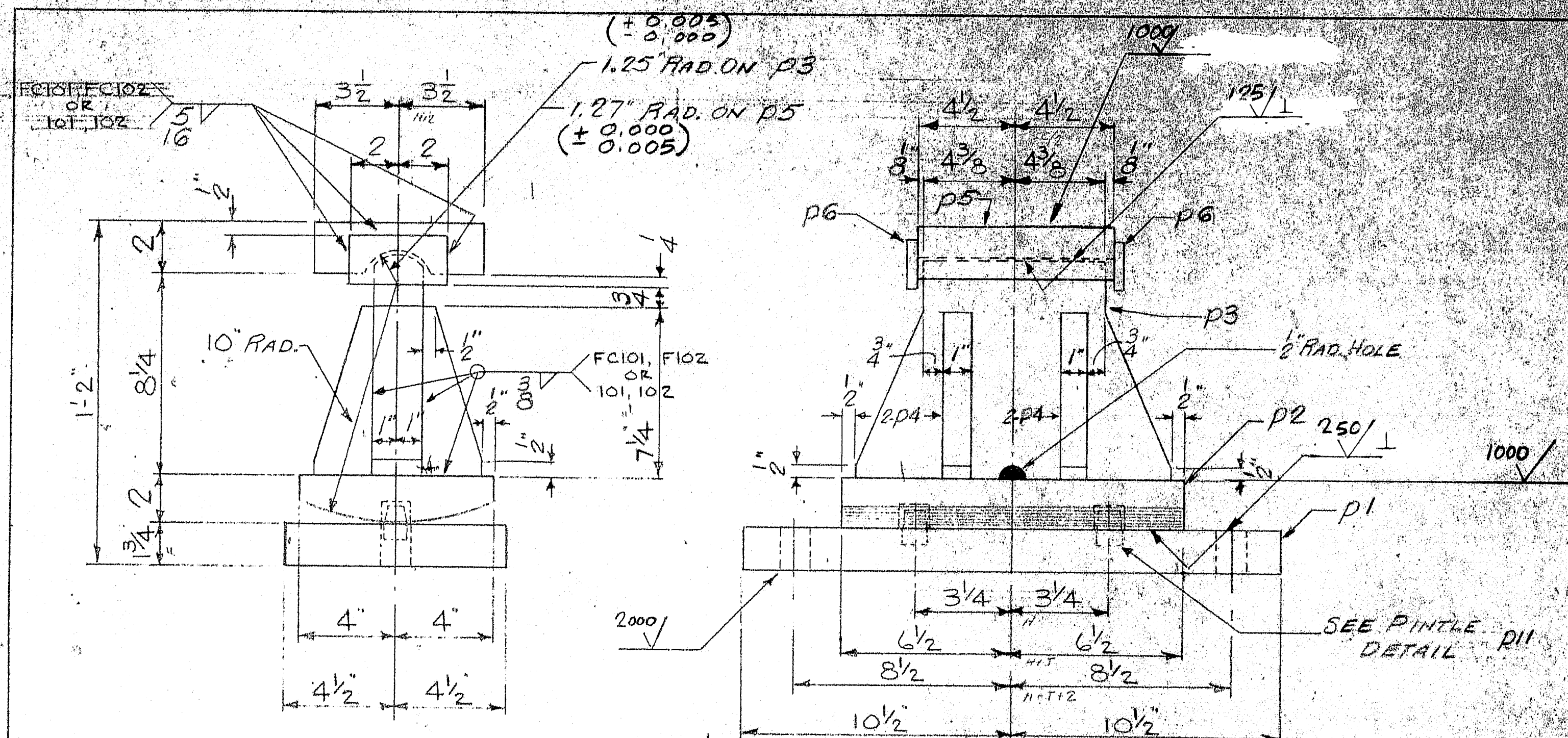
Prov No: I375.8(87)-176

CUSTOMER ORDER NO.	
REFERENCE DRAWINGS	
HOLES	
FIELD CONN.	3" A225-TYPE 1
PAIN	ONE 5/8" BANG LEAD 3/16" CHROMIUM PASTE
FRAMING PLAN	
MEGQUIER & JONES CORP.	
1185 BROADWAY	
SOUTH PORTLAND, MAINE 04106	
GREEN POINT ROAD OVERPASS/TSR	
BREWER	MAINE
CUSTOMER	H.J. SODERSON, INC.
ARCHITECT	MAINE D.O.R.

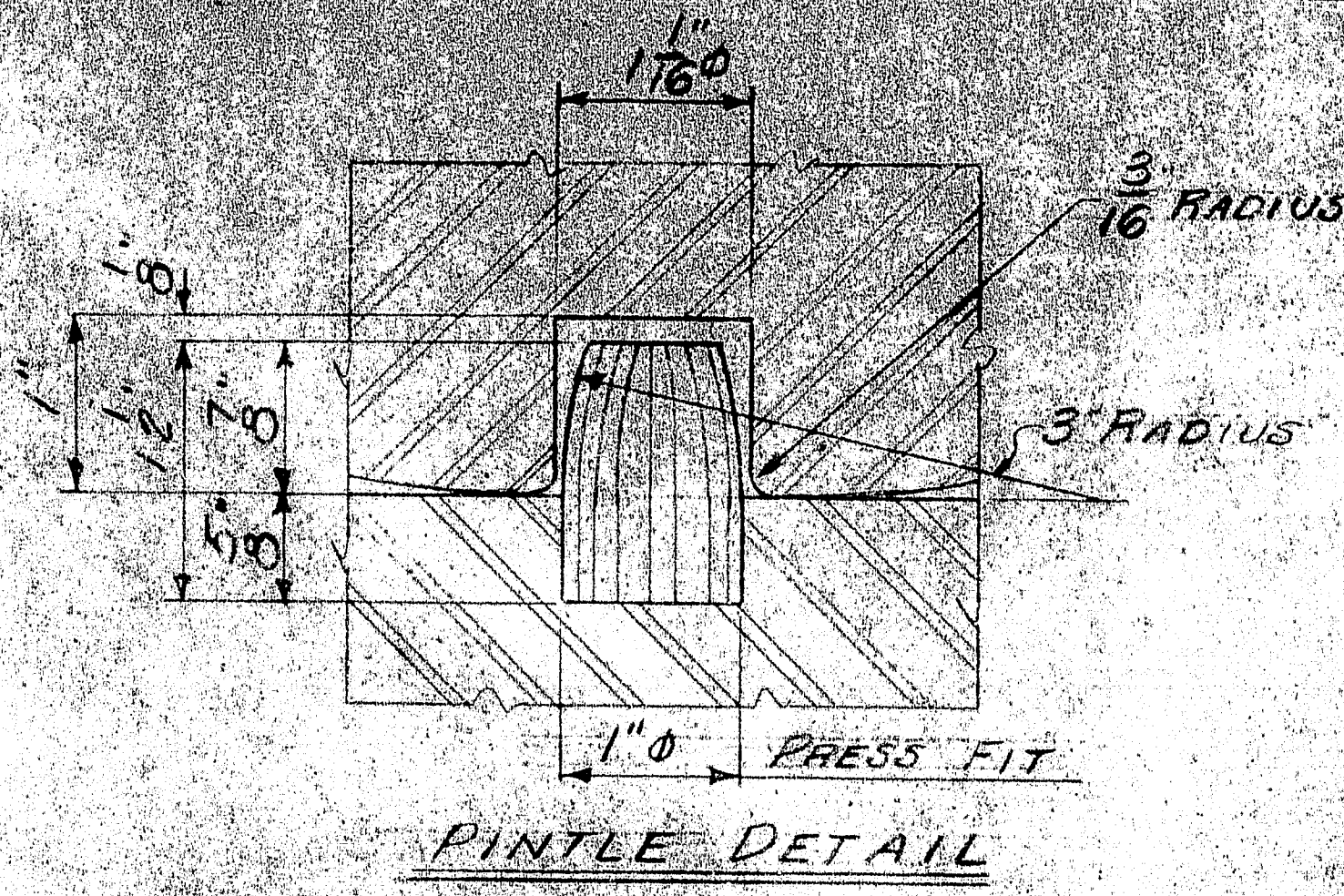
APPR.	5-10-84
APPD.	
SHOP	6/13/84
F.A.O.	6/16/84
DRAWN	FWP
CHECKED	DUPE
REVISION	
REVISION	
REVISION	

R90-404

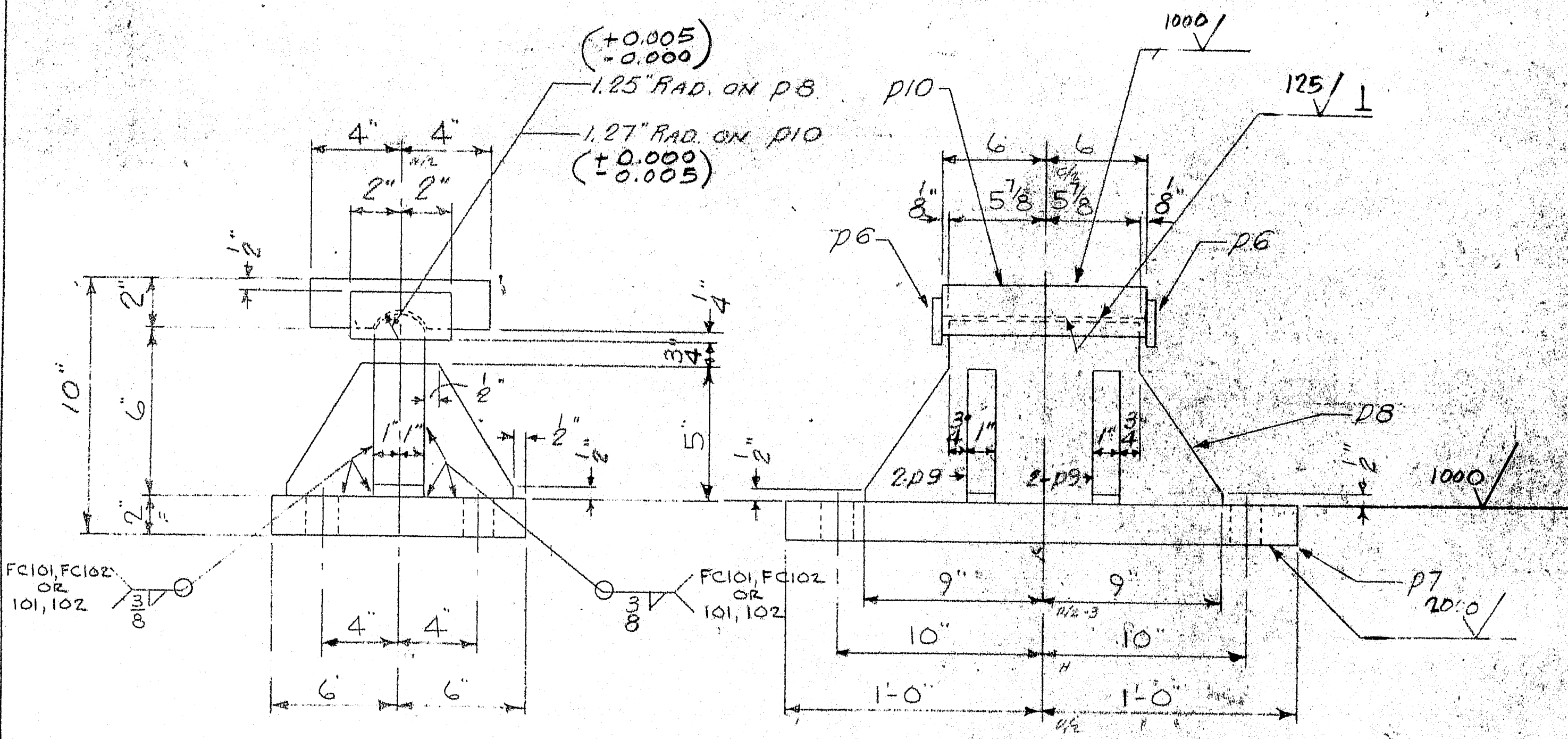




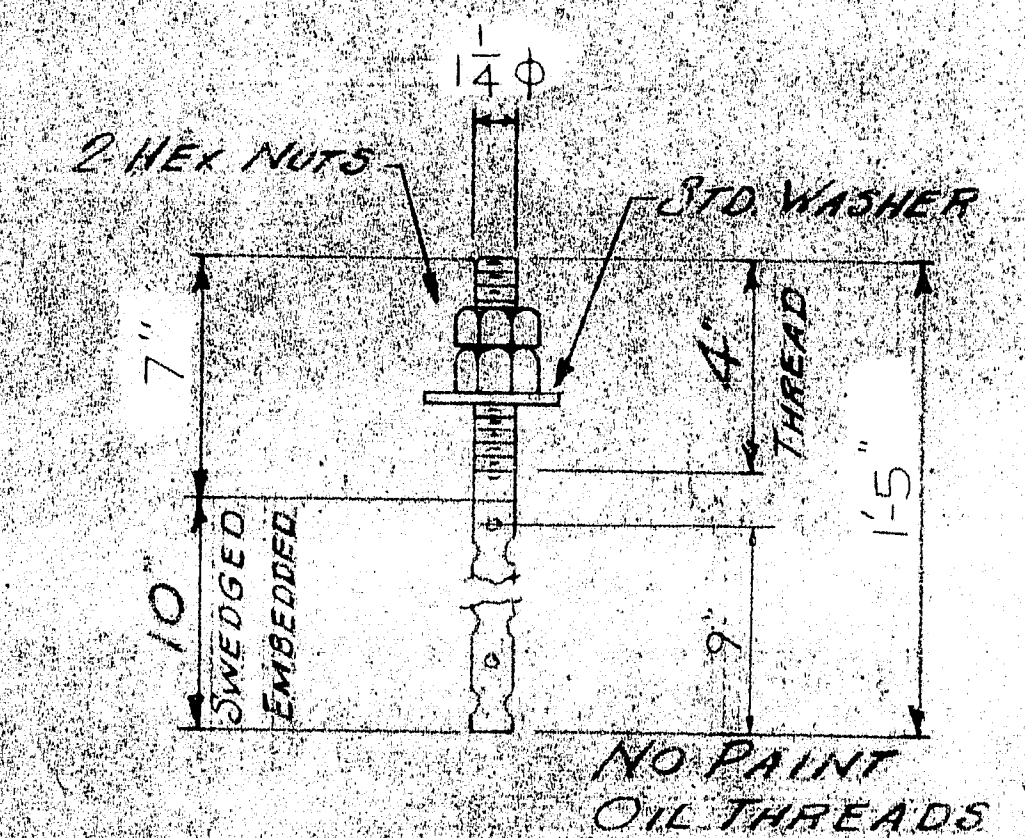
EPC-3 ~ 10 REQ'D



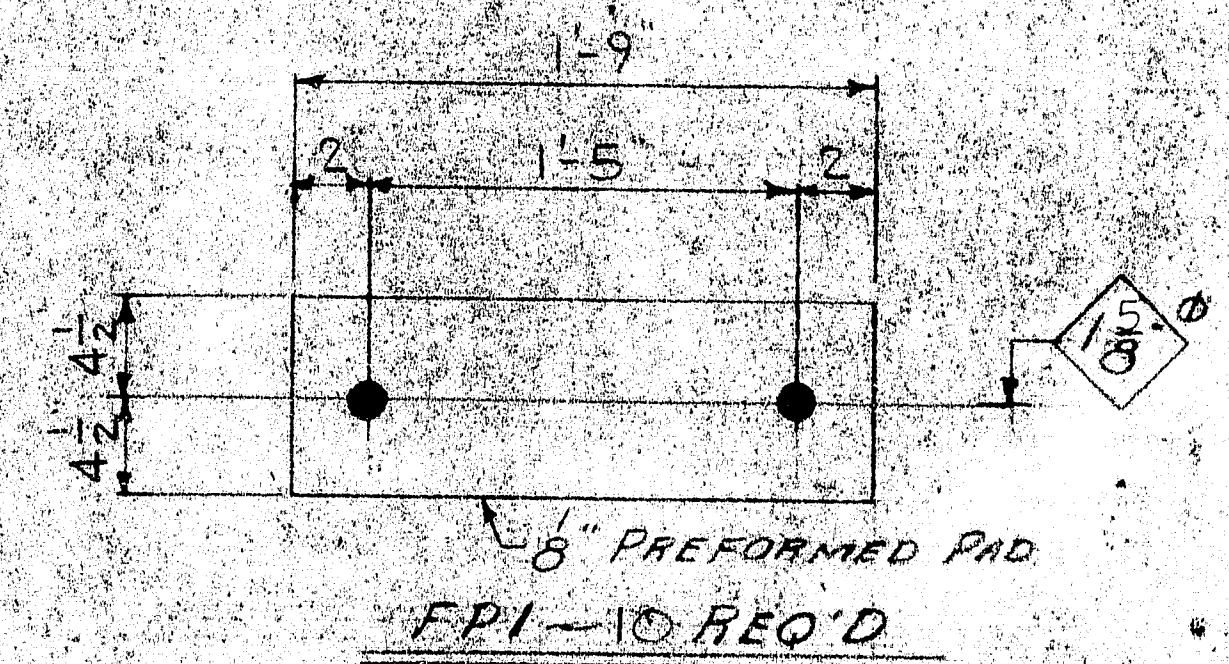
PINTLE DETAIL



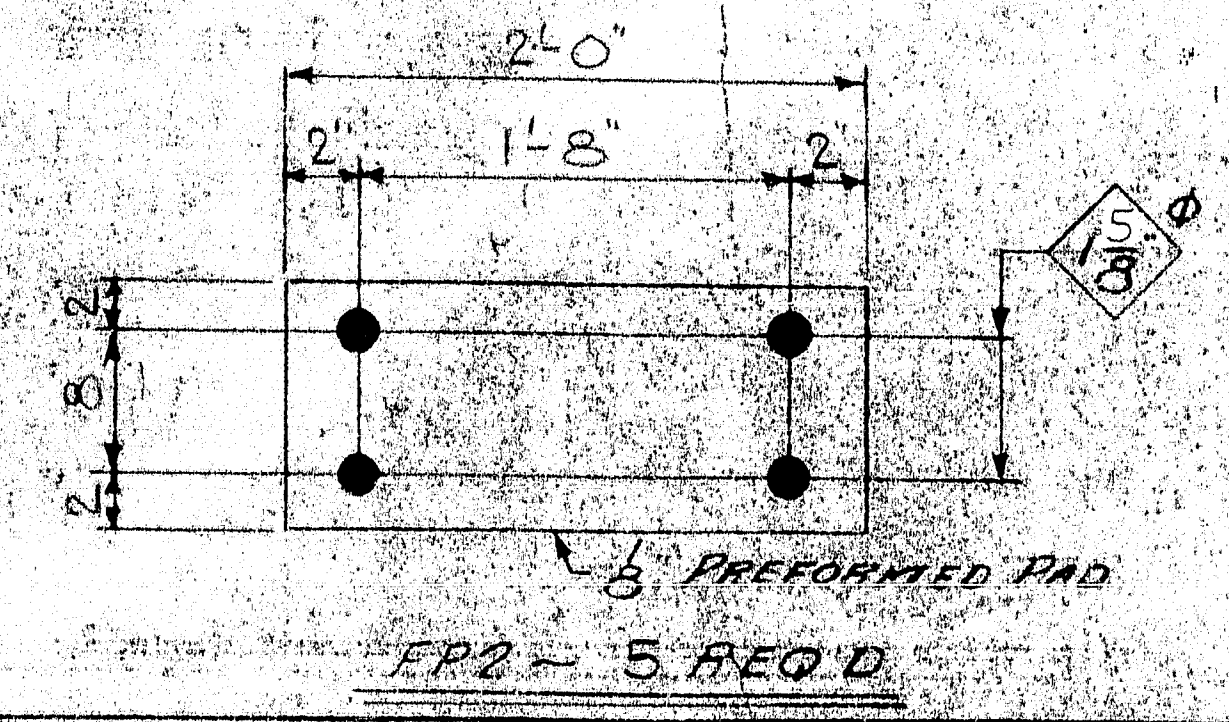
FPC-5 ~ 5 REQ'D



ANCHOR BOLTS AB1  
40 ~ REQ'D



FPI-10 REQ'D



FP2-5 REQ'D

SHIP	BILL OF MATERIAL			JOB NO.	J-45	DWG NO.	1
MARK	NO.	MARK	SHAPE	LENGTH	WT	REMARKS	
EAC3	10		ASSEMBLY				
	20	P11	BAR 1"Ø	0	1 1/2		
	10	P1	R 1 3/4 x 9'	1	9		
	10	P2	R 2 x 8	1	1		
	10	P3	R 2 x 9 1/4	1	0		
	40	P4	R 1 x 2 1/2	0	7 1/4		
	10	P5	R 2 x 7	0	9		
	20	P6	BAR 1 3/8 x 3	0	4		
FPC5	5		ASSEMBLY				
	5	P7	R 2 x 12	2	0		
	5	P8	R 2 x 7	1	6		
	20	P9	R 1 x 5	0	5 1/2		
	5	P10	R 2 x 8	1	0		
	10	P6	BAR 1 3/8 x 3	0	4		
AB1	40		BAR 1 1/2	1	5		
	80		1/2" HEX NUT	---		} A325 TYPE 1	
	40		1/2" STD WASHER	---			

PROJECT NO. I-395-8(87)-176  
 ITEM NO. 504.70 STRUCTURAL STEEL  
 BLAST CLEAN SP-6 (COMMERCIAL BLAST  
 STEEL ASTM A-36)  
 COAT ASA 125 SURFACE WITH "NEVER SEEZE"

PRINT NOTE  
 MASONRY R AND BEARING SURFACES OF ROCKERS  
 TO HAVE (2) COATS OF BASIC LEAD SILICO  
 CHROMATE SHOP PAINT, REMINDER TO HAVE (1)  
 SHOP COAT. (EACH COAT TO BE MIN 4 MILS WET)

CUSTOMER ORDER NO:  
 REFERENCE DRAWINGS: 9 OF 15 & BD-101-81  
 HOLES: 1 1/8" Ø  
 FIELD CONN:  
 PAINT: SEE NOTE ABOVE

EXPANSION PEDESTAL EPC-3 (10)  
 FIXED PEDESTAL FPC-5 (5)

MEGQUIER & JONES CORP.  
 1156 BROADWAY  
 SOUTH PORTLAND, MAINE 04106

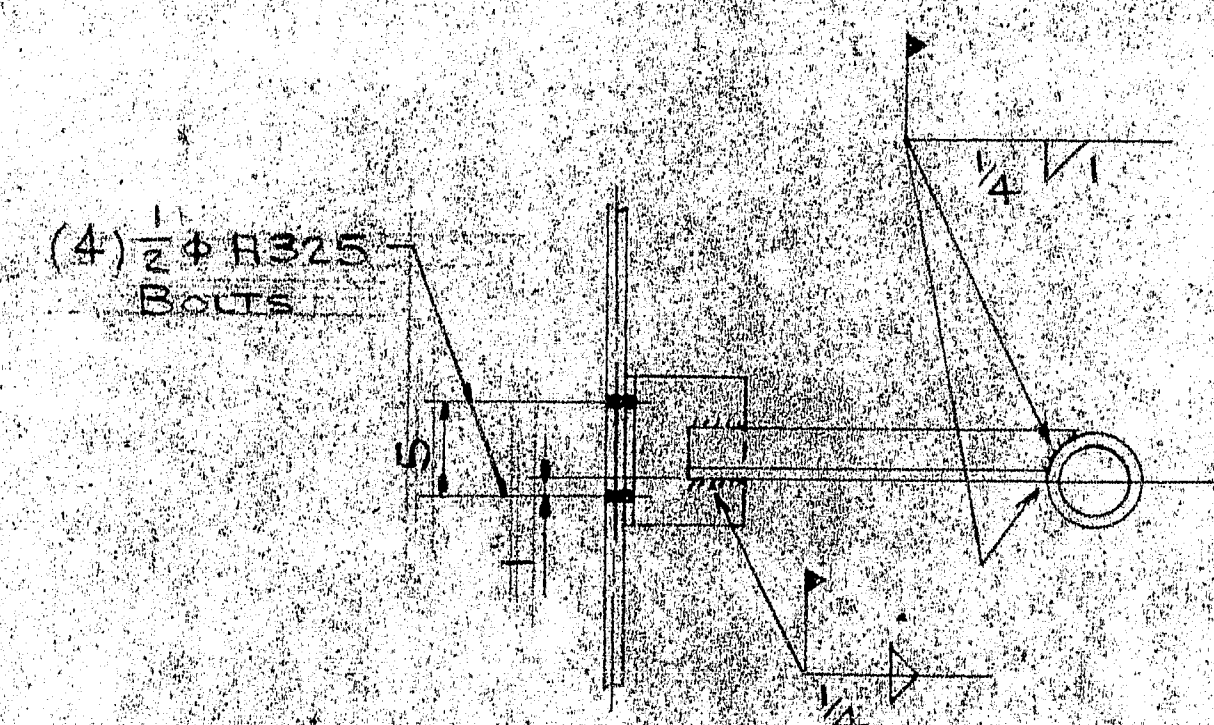
GREEN POINT ROAD OVERPASS/I-395  
 BREWER MAINE

CUSTOMER: H.J. SOCKESSON, INC.  
 ARCHITECT: MAINE DEPT. OF TRANSPORTATION

J-45

R90-405





PCV 406

APPR.	5-10-84	
APPR.	6/13/84	
SHOP	6/13/84	
F. & O.	6/26/84	
DRAWN	ET	3/13/8
CHECKED	JPF	
REVISION		
REVISION		
REVISION		

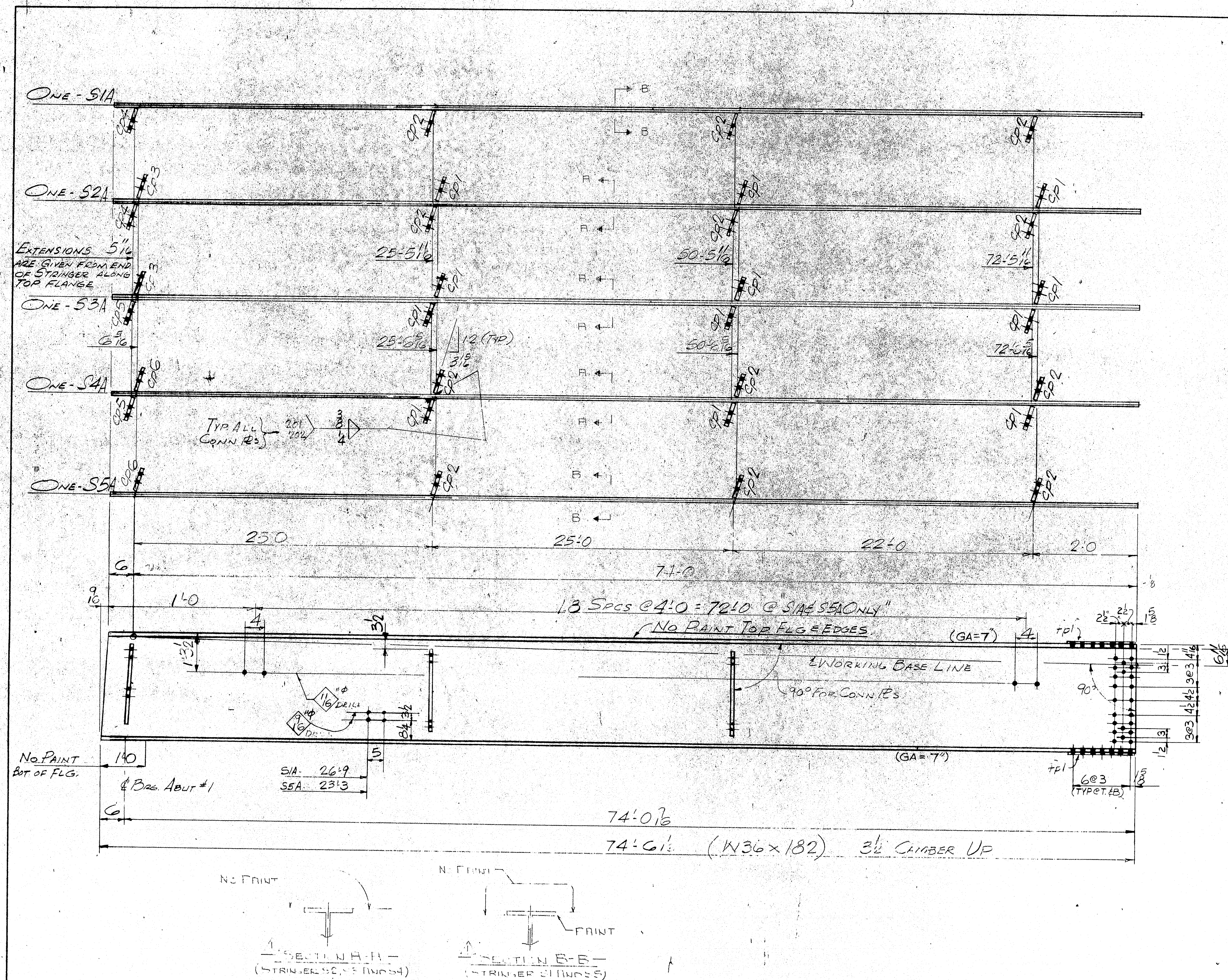
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DWG NO. 4

PCV 406

NO-FOO





SHIP		BILL OF MATERIAL		JOB NO. J-45		DWG. NO. 3	
MARK	NO.	MARK	SHAPE	LENGTH	WT.	REMARKS	
S1A	1		W36x182	74'-0"		A572 CAMBER-3 1/2	
S2A	1			74'-0"		CVN	
S3A	1			74'-0"			
S4A	1			74'-0"			
S5A	1		W36x182	74'-0"		A572 CAMBER-3 1/2	
						GR50	
						10 fpl R3x11	
						A36 WIRE FOR SHIPING	
						12 cp1 R3x8	
						A36	
						12 cp2	
						A36	
						2 cp3	
						A36	
						2 cp4	
						A36	
						2 cp5	
						A36	
						2 cp6 R3x8	
						A36	

Proj No. I-395-B(87)-176  
ITEM NO. 504.70 STRUCTURAL STEEL

BLAST CLEAN ~ SP6 (COMM)

CUSTOMER ORDER NO.  
REFERENCE DRAWINGS:  
HOLES 15/16" UN  
FIELD CONN. A325-TYPE 1

PAINT ONE 1/2" BASIC LEAD SILICO CHROMATE PRIMER  
STRINGER DETAILS

MEGQUIER & JONES CORP.  
1156 BROADWAY  
SOUTH PORTLAND, MAINE 04106

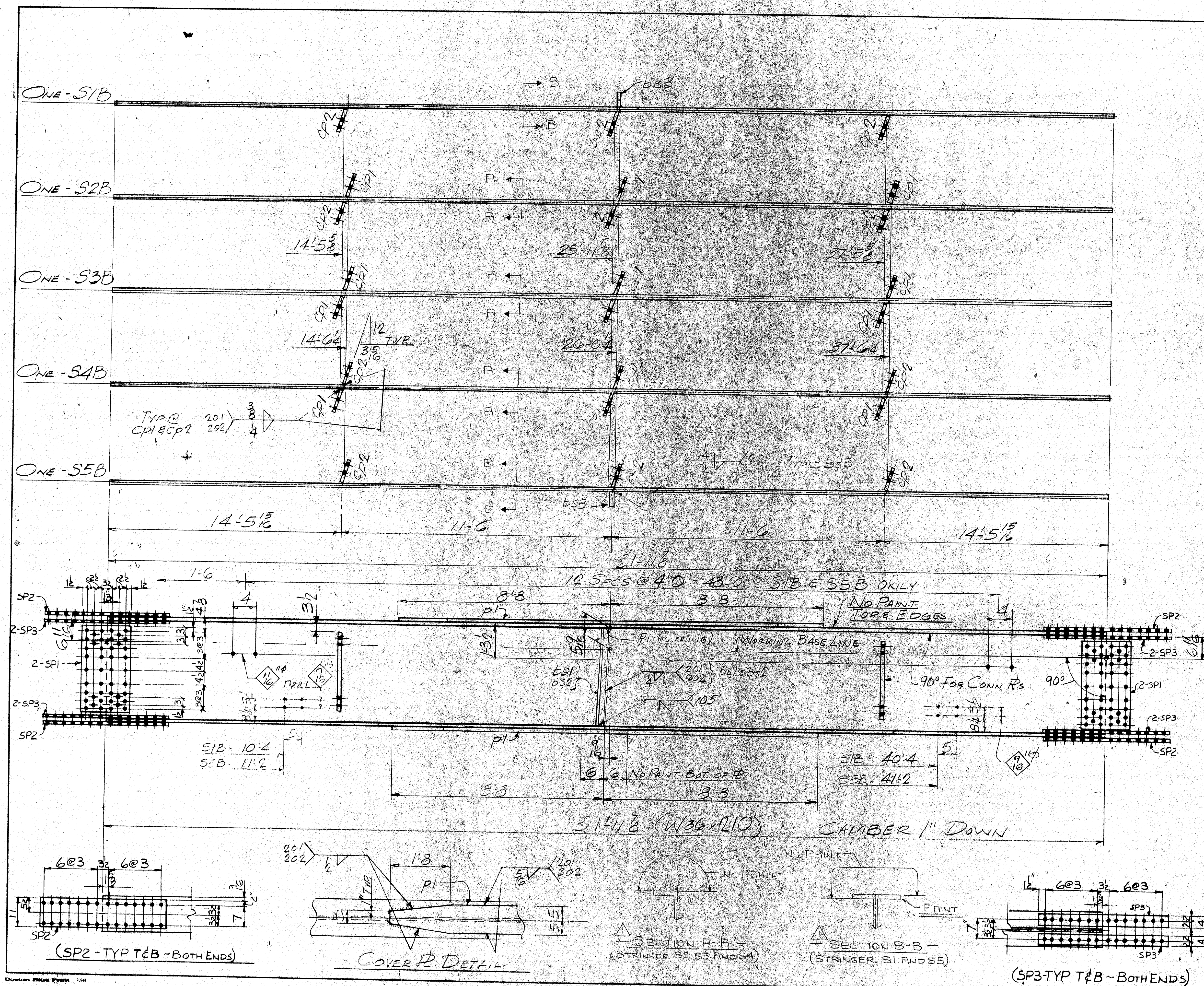
GREEN POINT ROAD OVERPASS/I-395  
BREWER MAINE

CUSTOMER: H. J. SOCKBESON, INC.  
ARCHITECT: MAINE DOT

JOB NO. J-45 DWG. NO. 3

R90-407





SHIP BILL OF MATERIAL						JOB NO.	J-45	DWG. NO.	4
MARK	NO.	MARK	SHAPE	LENGTH	WT.	REMARKS			
S1B	1		W36x210	51' 11 3/4"		A572 CAMBER = 1"			
S2B	1			51' 11 3/4"		CVN			
S3B	1			51' 11 3/4"					
S4B	1			51' 11 3/4"					
S5B	1		W36x210	51' 11 3/4"		CAMBER = 1"			
	10	P1	R 1/8 x 10	17' 4"		A572 GR50			
	8	SP1	R 3/8 x 8	2' 3"		A36			
	8	SP2	R 3/8 x 8	2' 3"		A36			
	4	SP1	R 1/2 x 7 1/2	2' 10"		A572 GR50			
	4	SP2	R 1/2 x 7 1/2	2' 10"					
	2	SP3	R 1/2 x 5	2' 10"					
SPICE PLATES									
	20	SP1	R 5/8 x 16 1/2	2' 6"		A572 GR50			
	20	SP2	R 3/4 x 11	3' 6 1/2"		A572 GR50 CVN			
	40	SP3	R 1 x 4	3' 6 1/2"		A572 GR 50			
SHOP NOTE									
NO PAINT WITHIN 5" OF OPEN HOLES.									

SHOP NOTE  
NO PAINT WITHIN 5"  
OF OPEN HOLES.

PROJ. NO. I-395-B(87)-176  
ITEM NO. 504.70 STRUCTURAL STEEL

BLAST CLEAN ~ SP6 (COMM)

CUSTOMER ORDER NO.  
REFERENCE DRAWINGS:  
HOLES: 5/8" UN.  
FIELD CONN. A325-TYPE I

PAINT: ONE COAT BASIC LEAD SILICO CHROMATE PRIMER  
STRINGER DETAILS

MEGQUIER & JONES CORP.  
1156 BROADWAY  
SOUTH PORTLAND, MAINE 04106  
GREEN POINT ROAD OVERPASS/I-395  
BREWSTER MAINE

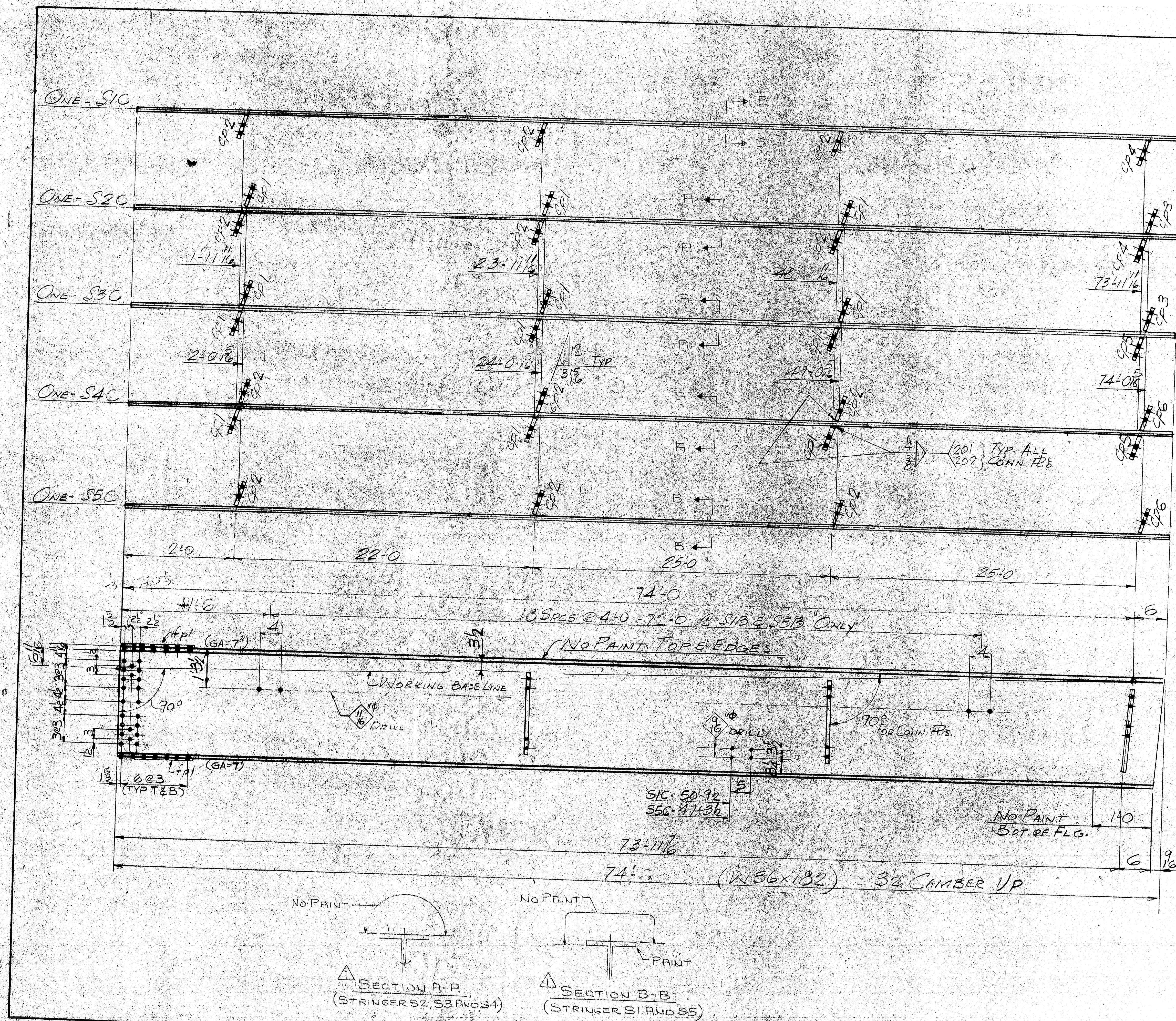
CUSTOMER: H.J. SOCKBESON, INC.  
ARCHITECT: MAINE DOT

APPR.	5-10-84
APPR.	6/13/84
SHOP	6/13/84
F.O.	6/13/84
DRAWN	JPE 8.84
CHECKED	ET
REVISION	ET 7/10/84
REVISION	

JOB NO. J-45 DWG. NO. 4

R90-408





SHIP BILL OF MATERIAL				JOB NO.	J-45	DWG. NO.	5
MARK	NO.	MARK	SHAPE	LENGTH	WT.	REMARKS	
S1C	1		W36x182	74' 0"		A572	CAMBER=3/2
S2C	1			74' 0"			
S3C	1			74' 0"			
S4C	1			74' 0"			
S5C	1		W36x182	74' 0"		A572	CAMBER=3/2
	10	fp1	EP6x11	1' 9"		A36	WIRE FOR SHIPPING
	12	cp1	EP8x8	2' 3"		A36	
	12	cp2		2' 3"			
	2	cp3		2' 3"			
	2	cp4		2' 3"			
	2	cp5		2' 3"			
	2	cp6	EP8x8	2' 3"		A36	

SHIP NOTE

NO PAINT WITHIN 5"  
OF OPEN HOLES.

SHOP NOTE  
NO PAINT WITHIN 5"  
OF OPEN HOLES.

PROJ. NO. I-395-B(87)-176  
ITEM NO. 504.70 STRUCTURAL STEEL

BLAST CLEAN ~ SP6 (COMM)

CUSTOMER ORDER NO.  
REFERENCE DRAWINGS:  
HOLES 15/16" UN.  
FIELD CONN: A325-TYPE 1  
PAINT: ONE % BASIC LEAD SILICO CHROMATE PRIMER  
STRINGER DETAILS

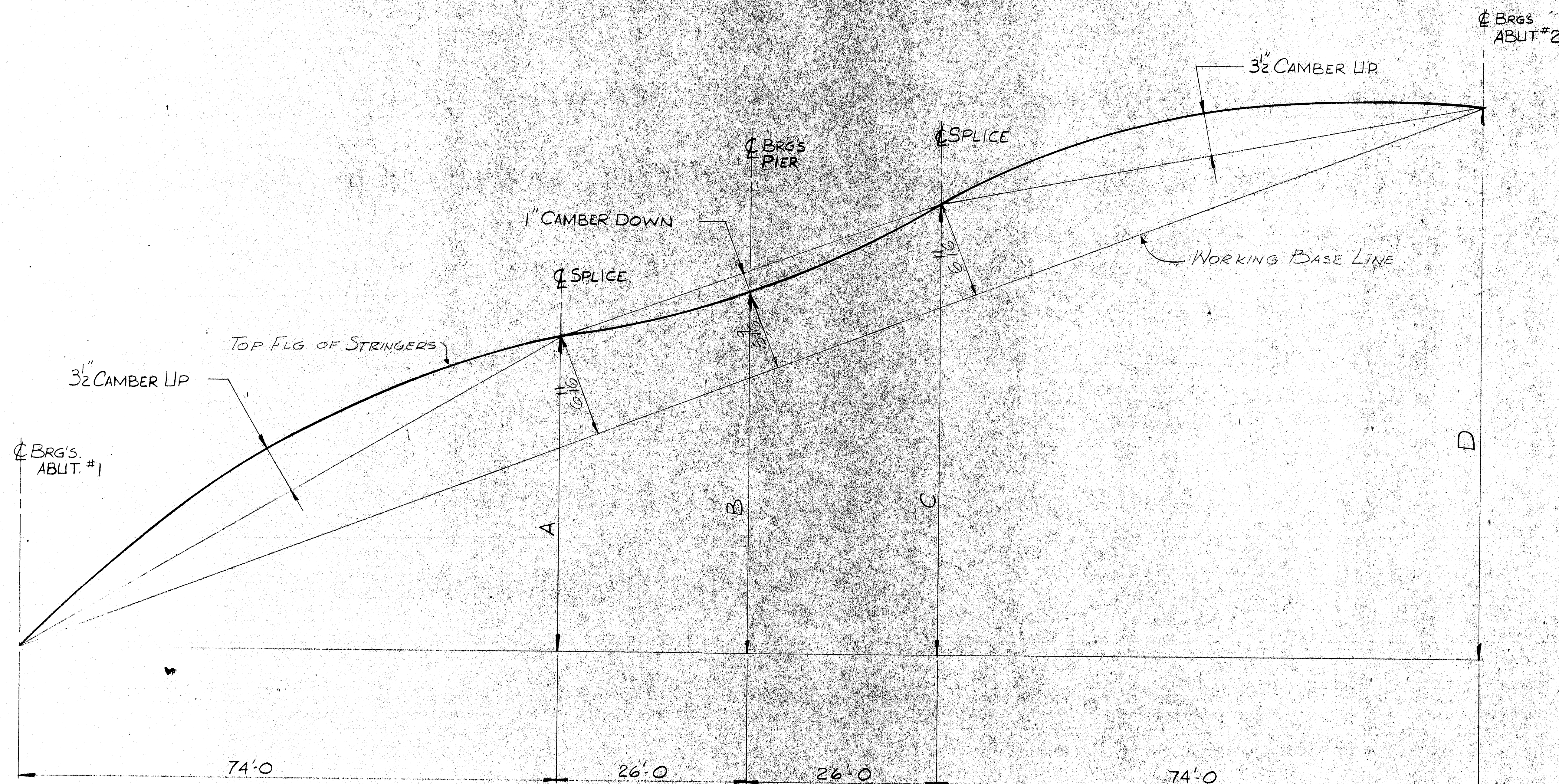
MEGQUIER & JONES CORP.  
1108 BROADWAY  
SOUTH PORTLAND, MAINE 04106  
GREEN POINT ROAD, OYUNEBE, MAINE 04951  
BREWSTER MAINE  
CUSTOMER H.J. SOCKBESON INC.  
ARCHITECT MAINE DOT

APPR.	5-10-84
APPR.	6/13/84
SHOP	6/13/84
F. & O.	6/26/84
DRAWN	JPE 4-84
CHECKED	DEU
REVISION	ET 7/10/84
REVISION	
REVISION	

J-45  
R90-409



	A	B	C	D
S1	1'-8 <sup>5</sup> / <sub>8</sub>	2'-0 <sup>3</sup> / <sub>8</sub>	2'-6 <sup>7</sup> / <sub>16</sub>	3'-1 <sup>5</sup> / <sub>8</sub>
S2	1'-8 <sup>3</sup> / <sub>16</sub>	2'-0 <sup>1</sup> / <sub>16</sub>	2'-6 <sup>3</sup> / <sub>16</sub>	3'-2 <sup>3</sup> / <sub>16</sub>
S3	1'-9 <sup>1</sup> / <sub>16</sub>	2'-0 <sup>5</sup> / <sub>16</sub>	2'-7 <sup>8</sup> / <sub>16</sub>	3'-2 <sup>3</sup> / <sub>16</sub>
S4	1'-9 <sup>1</sup> / <sub>16</sub>	2'-1 <sup>1</sup> / <sub>4</sub>	2'-7 <sup>1</sup> / <sub>16</sub>	3'-3 <sup>5</sup> / <sub>16</sub>
S5	1'-9 <sup>7</sup> / <sub>16</sub>	2'-1 <sup>1</sup> / <sub>2</sub>	2'-7 <sup>13</sup> / <sub>16</sub>	3'-3 <sup>7</sup> / <sub>8</sub>



PROJ. NO I-395-8(87)-176  
ITEM NO. 504-70 STRUCTURAL  
STEEL

CUSTOMER ORDER NO.:  
REFERENCE DRAWINGS:  
HOLES:  
FIELD CONN.:  
PAINT:

CAMBER ASSEMBLY DIAGRAM

MEGQUIER & JONES CORP.

1158 BROADWAY  
SOUTH PORTLAND, MAINE 04106

GREEN POINT ROAD OVERPASS/138  
BREWER MAINE

CUSTOMER: H.J. SOCKEESON, INC.

ARCHITECT: MAINE D.O.T.

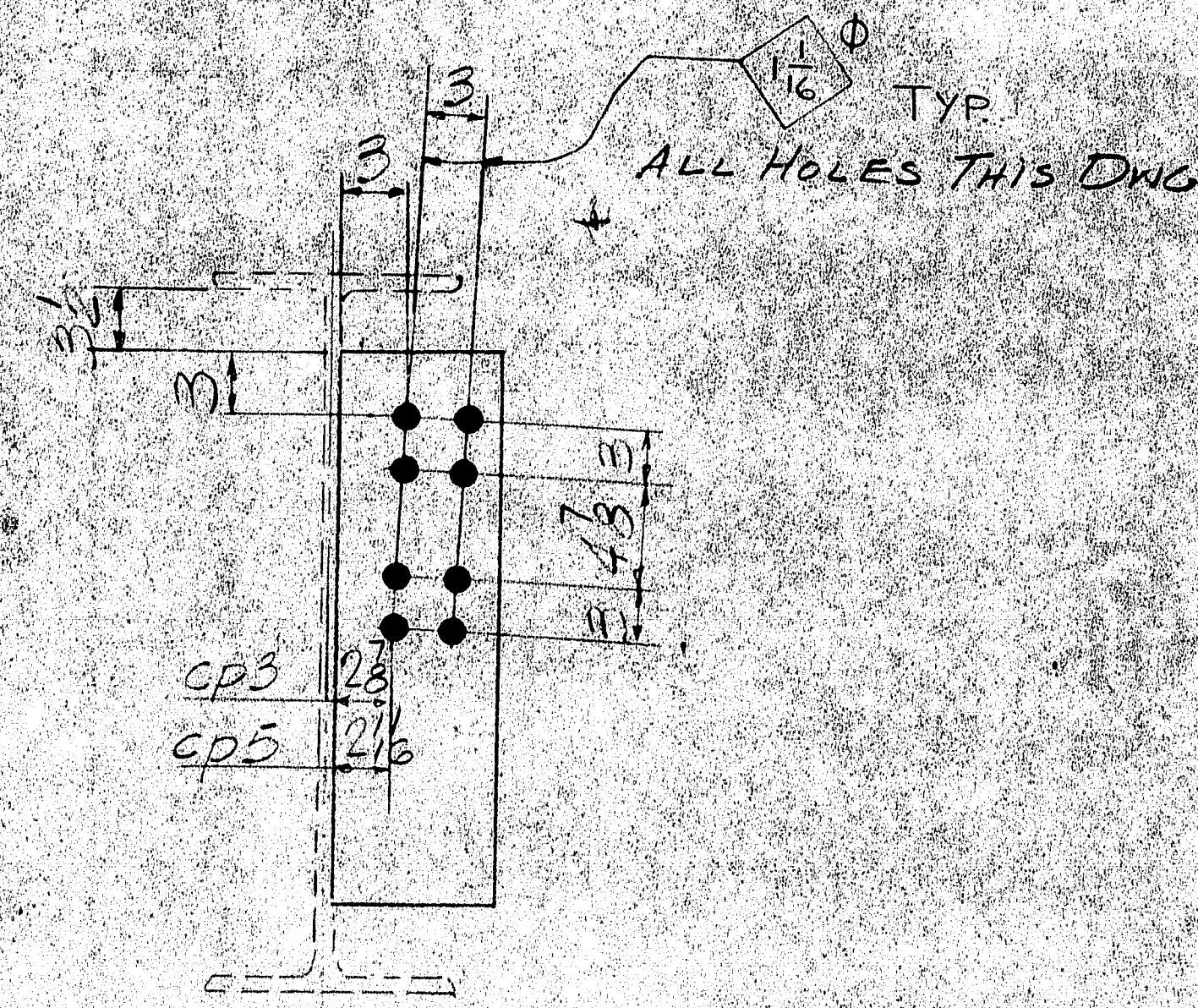
JOB NO. J-45

DWG. NO. 6

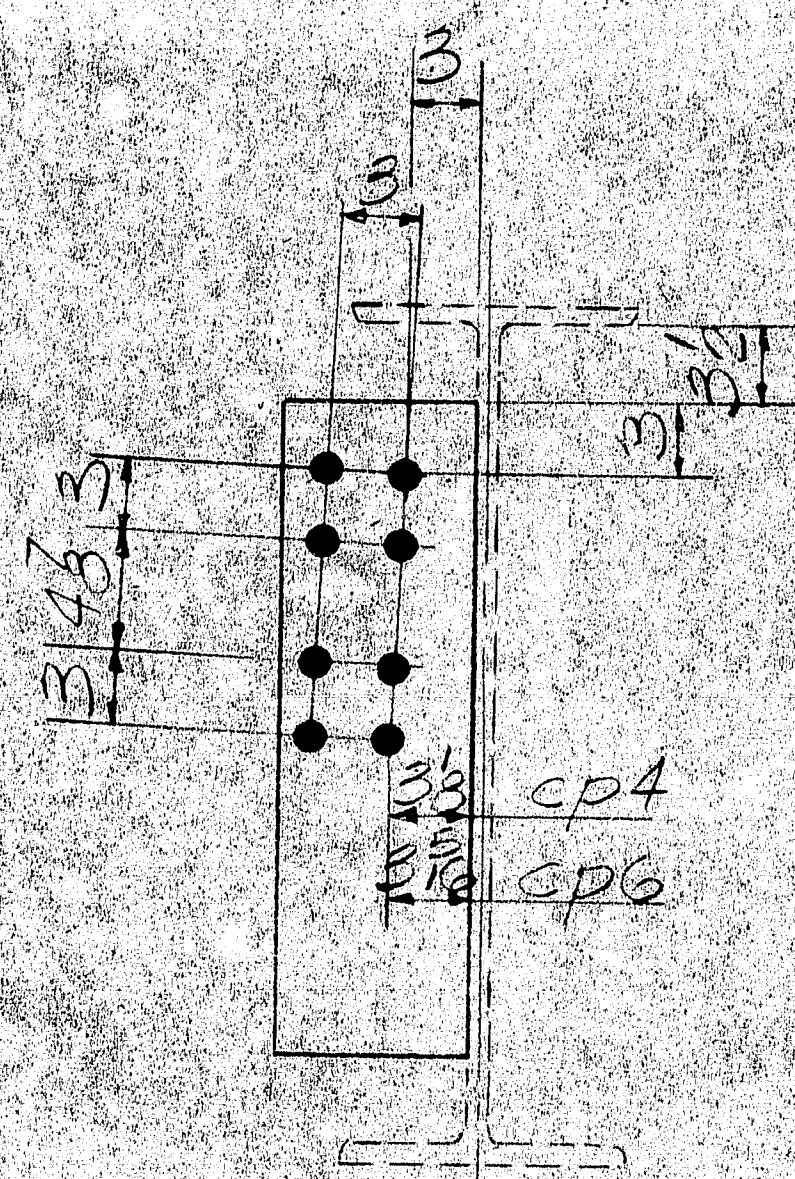
APPR.	5-10-84
SHOP	6/13/84
F.O.	6/24/84
DRAWN	CRT
CHECKED	UPF
REVISION	
REVISION	

R90-410

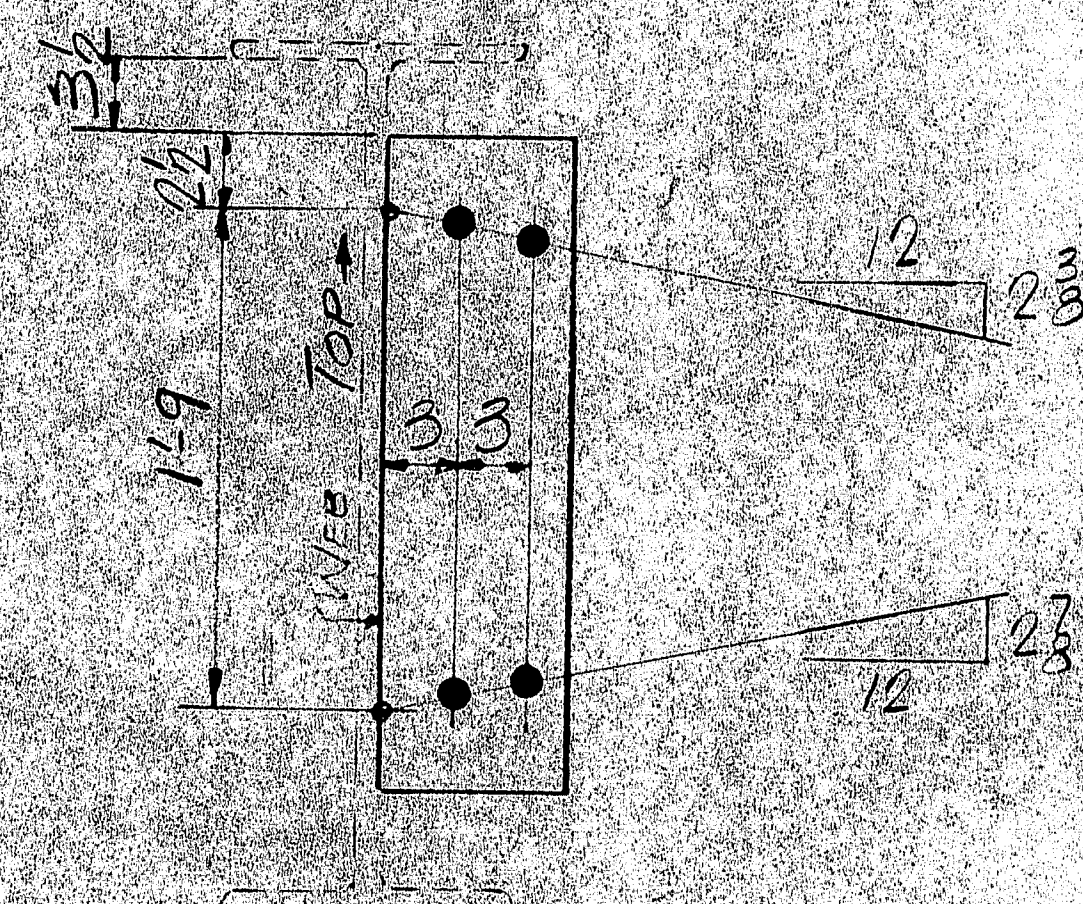




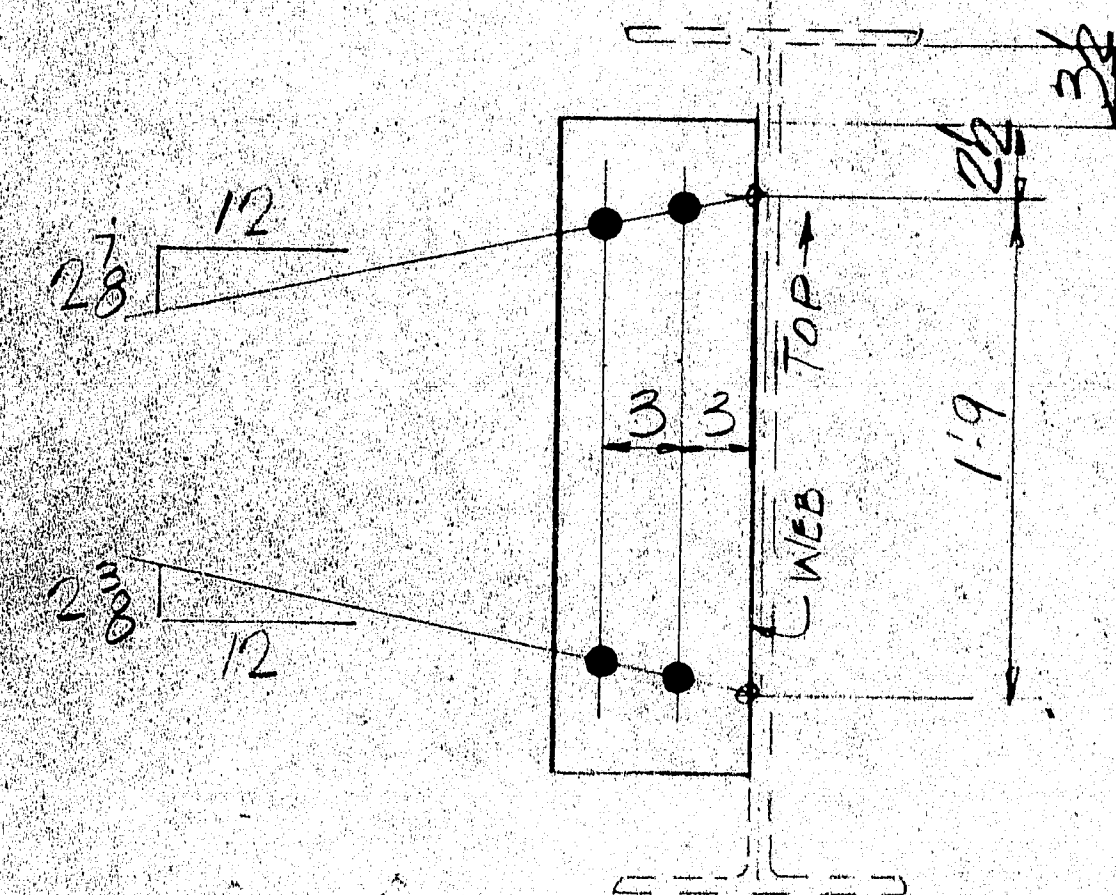
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cp5-4 REQ'D



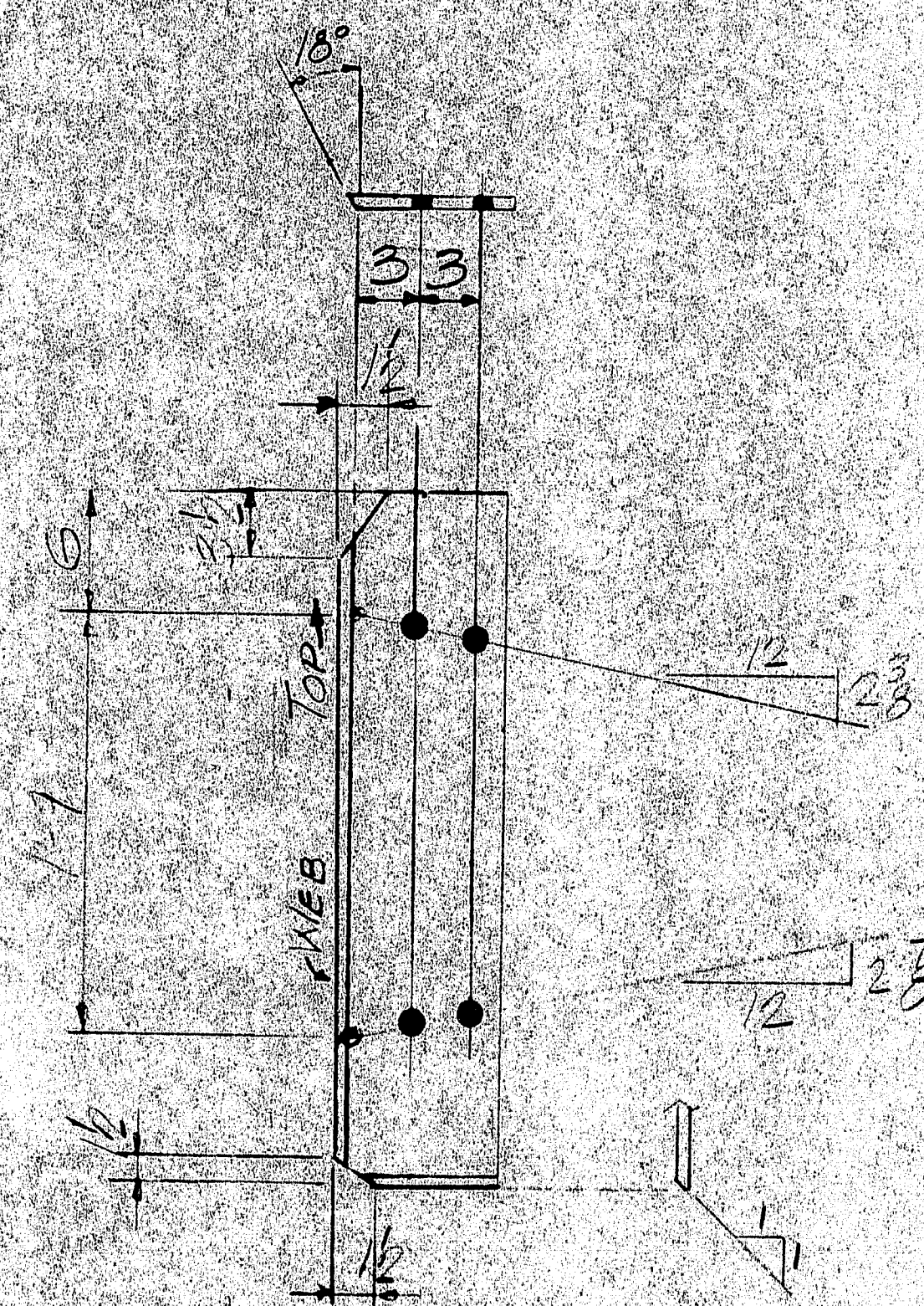
cp4-4 REQ'D  
cp6-4 REQ'D



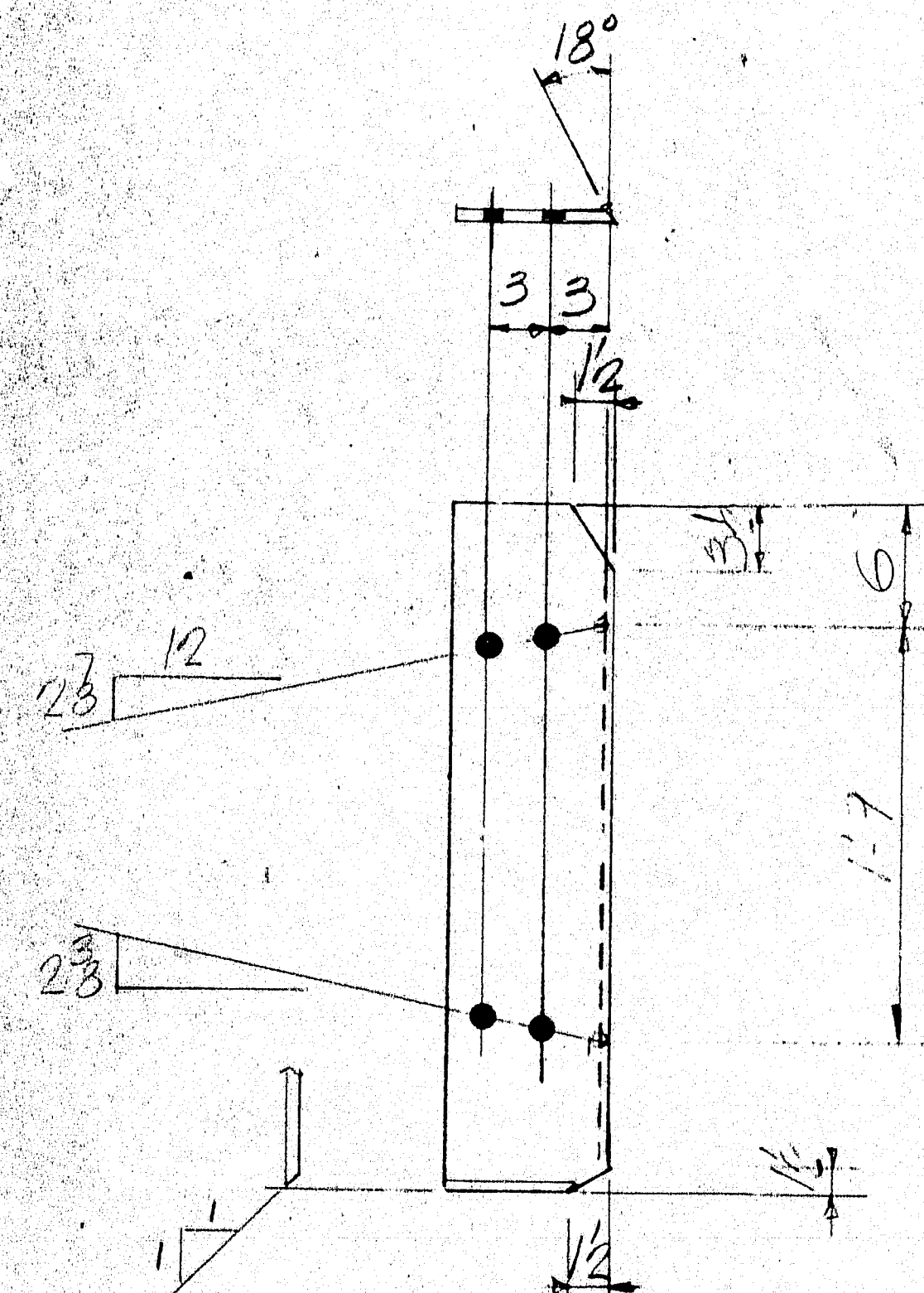
cp2-32 REQ'D



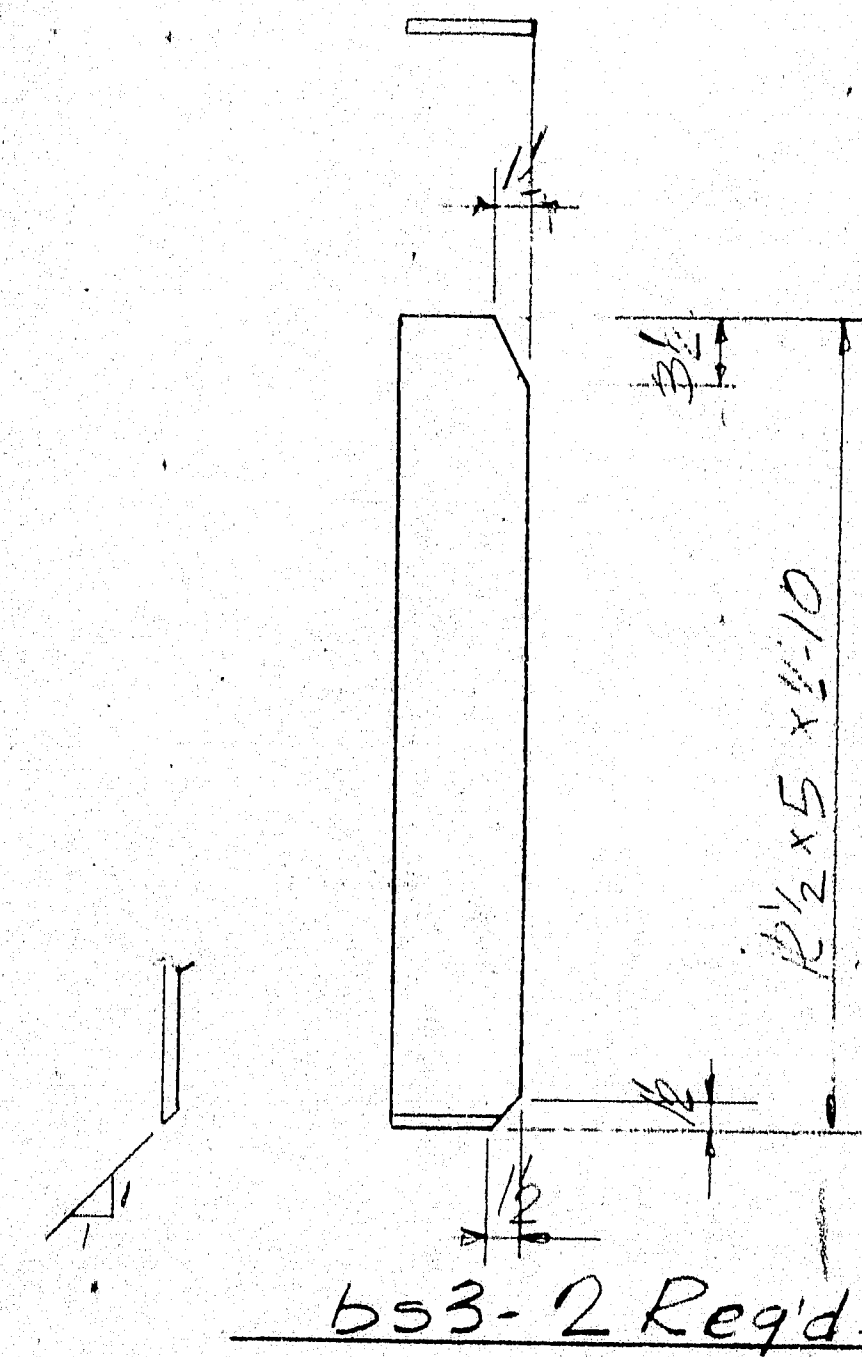
cp1-32 REQ'D



bs3-4 REQ'D



bs1-4 REQ'D



bs3-2 REQ'D

SHOP NOTE  
NO PAINT WITHIN 5"  
OF OPEN HOLES.

CUSTOMER ORDER NO.:  
REFERENCE DRAWINGS:  
HOLES: 1/4"  $\phi$   
FIELD CONN: 8"  $\phi$  A325-TYPE I  
PAINT: ONE 9/16" BASIC LEAD SILICO CHROMATE PRIMER  
DETAILS - CONN RS & BEARING STIFFS

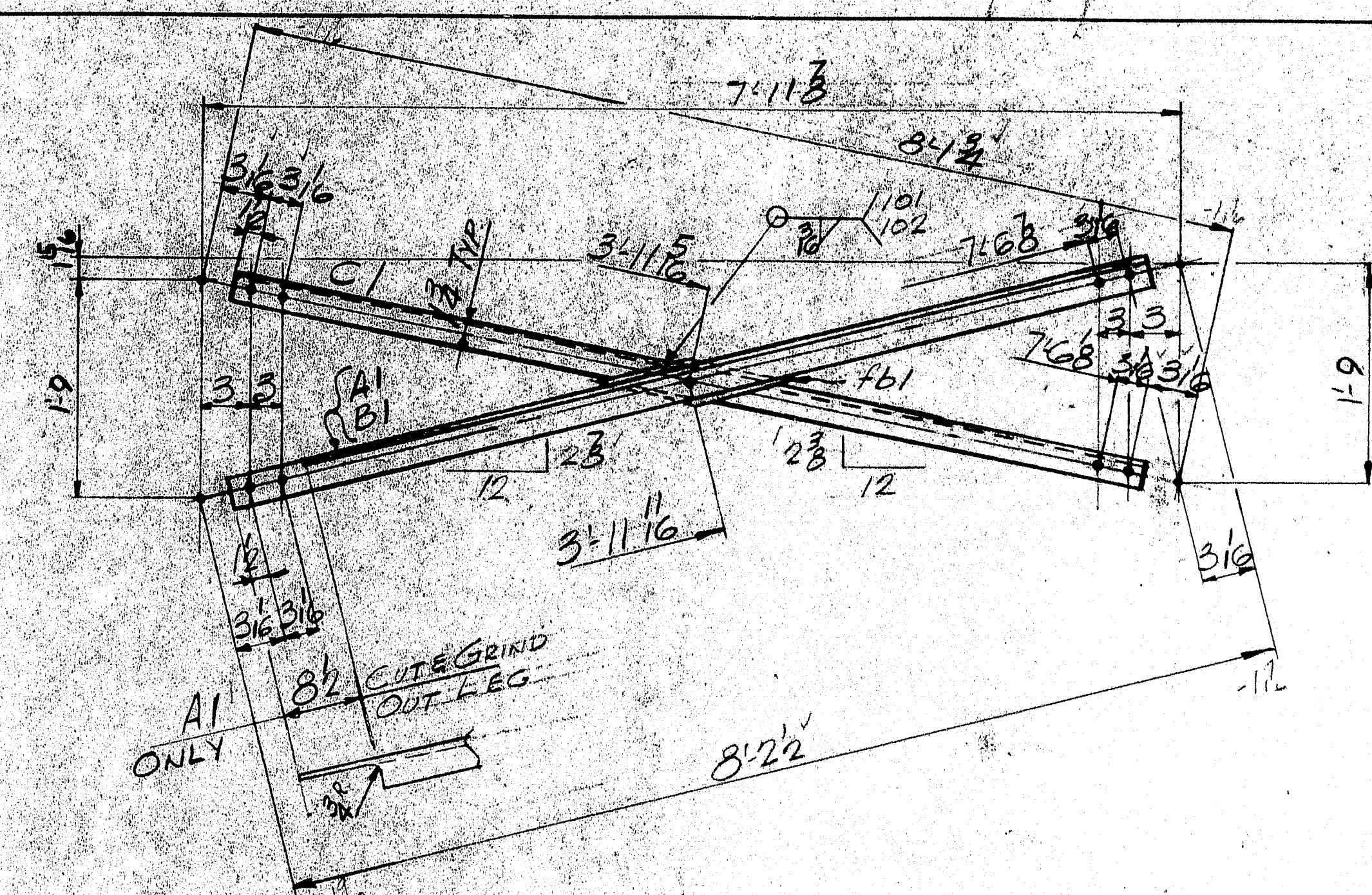
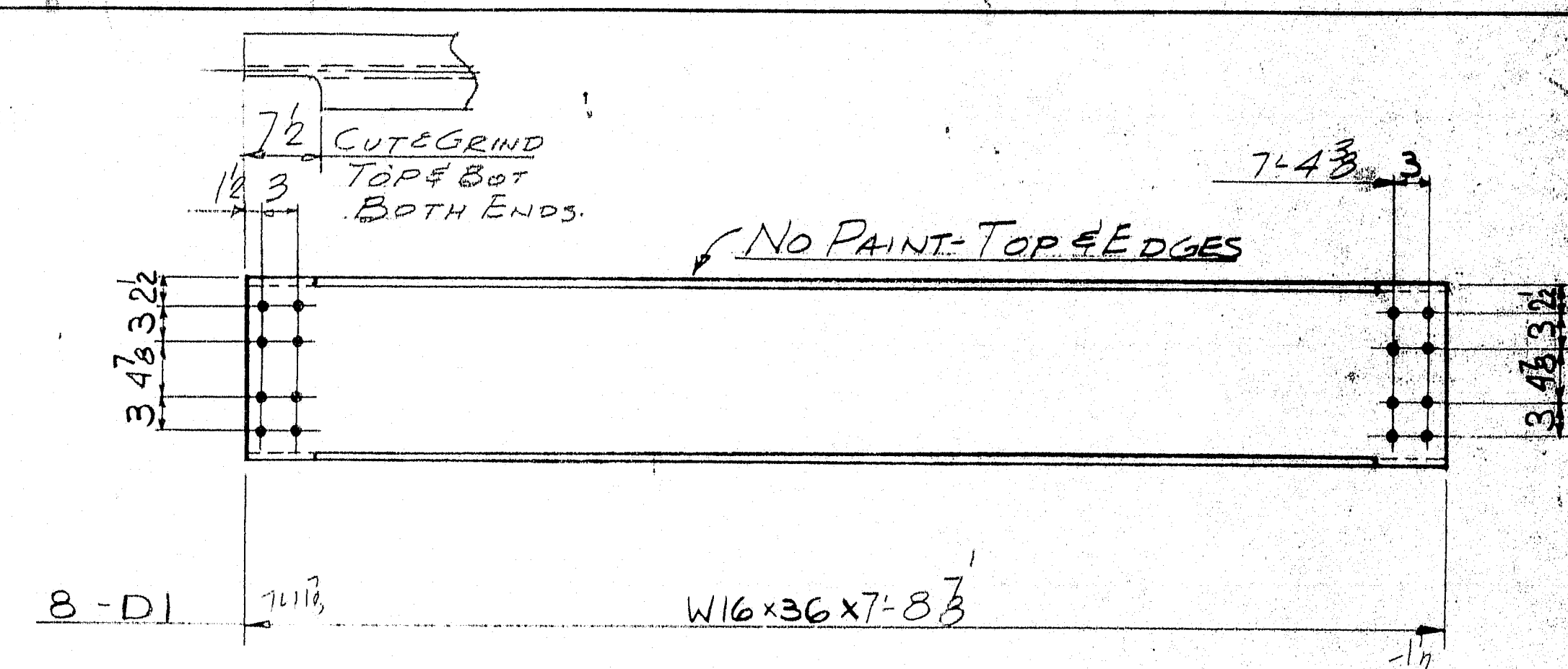
MEGQUIER & JONES CORP.  
1156 BROADWAY  
SOUTH PORTLAND, MAINE 04106  
GREEN POINT ROAD OVERPASS / I-395  
BREWER MAINE  
CUSTOMER: H. J. SOCKBESON, INC.  
ARCHITECT: MAINE D.O.T.  
JOB NO. 4-45  
DWG. NO. 7

APPR.	5-10-84
APPR.	6-13-84
SHOP	6-13-84
P.S.O.	6-26-84
DRAWN	JPF 4.84
CHECKED	EJ
REVISION	
REVISION	

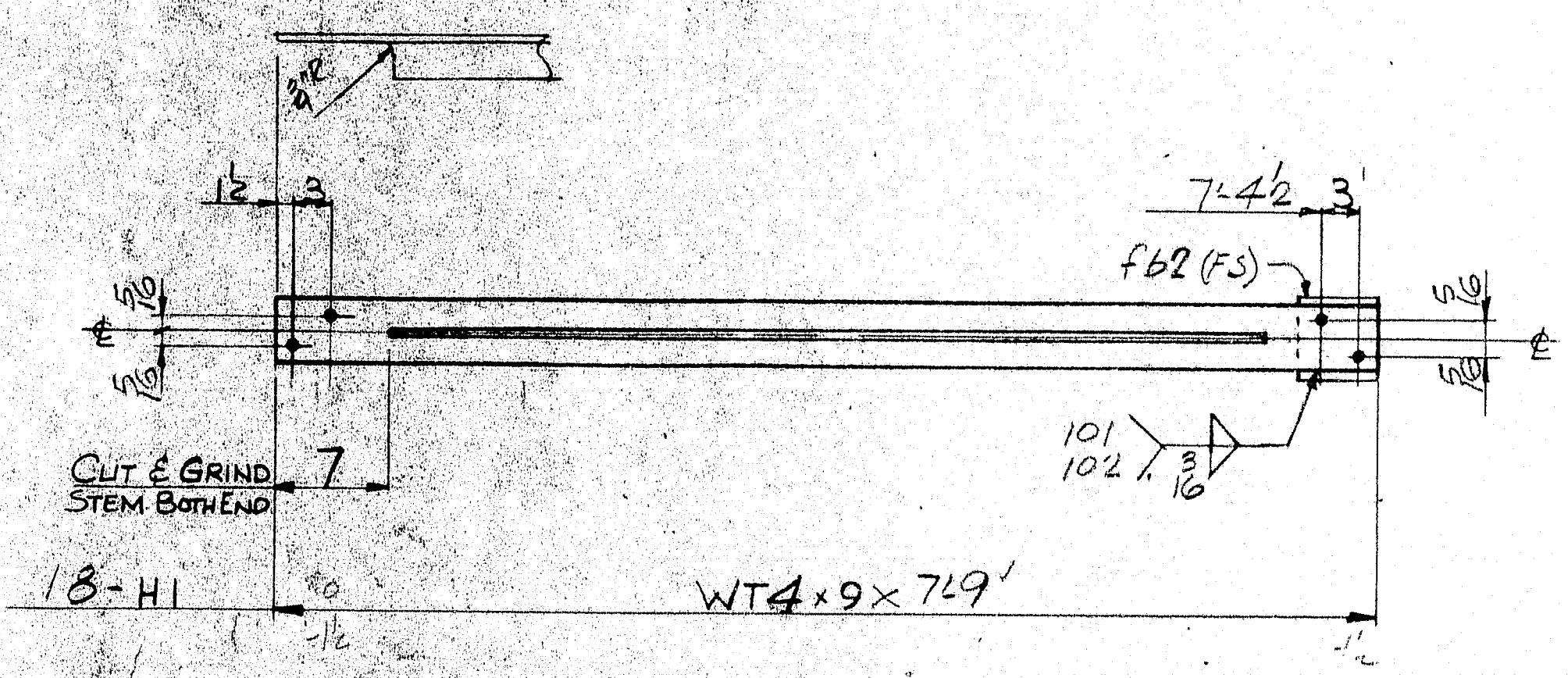
ITEM No. 504.70 - STRUCTURAL STEEL  
PROJECT No. I-395-8 (87) 176

R90-411





A1-18 REQ'D.  
B1-18 REQ'D.  
C1-36 REQ'D.



SHIP		BILL OF MATERIAL			JOB NO. 4-45		DWG. NO. 8	
MARK	NO.	MARK	SHAPE	LENGTH	WT.	REMARKS		
A1	18		2x3x $\frac{5}{16}$	7' 11 $\frac{3}{8}$ "	✓			
B1	18		Do	7' 11 $\frac{3}{8}$ "	✓			
C1	36		Do	7' 10 $\frac{3}{4}$ "	✓			
	36	fb1	Bar 3x3 $\frac{3}{8}$	1' 6"		L.O.		
D1	8		W16x36	7' 8 $\frac{3}{4}$ "	✓			
H1	18		WT4x9	7' 9"	✓			
	18	fb2	Bar 6x $\frac{7}{16}$	6"				

PROJ NO. F395-8(87)-176  
ITEM NO. 504.70 STRUCTURAL STEEL  
BLAST CLEAN ~ SP6 (COMM.)

ALL STEEL A36

CUSTOMER ORDER NO.:  
REFERENCE DRAWINGS:  
HOLES: 15/16" U.N.  
FIELD CONN: 3" A325-TYPE 1  
PAINT: ONE 5/16" BASIC LEAD SILICO CHROMATE PRIMER  
DIAPHRAGM CROSSFRAME DETAILS

MEGQUIER & JONES CORP.  
1156 BROADWAY  
SOUTH PORTLAND, MAINE 04106  
GREEN POINT ROAD OVERPASS/I-395  
BREWER, MAINE  
CUSTOMER: MAINE DOT  
ARCHITECT: MAINE DOT

APPR.	5-10-84
APPR.	6-13-84
SHOP	6-13-84
F.R.O.	6-26-84
DRAWN	CRT
CHECKED	JUP
REVISION	
REVISION	

PROJ NO. 45 DWG. NO. 8  
**R90-412**