Moose Creek Bridge

Bridge No. 2223

Project No. 58013/NH-0A1-5(25)

Bridge Type Study Report

Prepared For: Alaska Department of Transportation and Public Facilities

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April 12, 2013

Executive Summary

This purpose of this report is to provide the Alaska Department of Transportation and Public Facilities with a determination of the most feasible bridge type for the Moose Creek Bridge Project. For this project, AK DOT&PF is planning to realign the Glenn Highway between MP 53 and MP 56.

A bridge type study was conducted, and the following design alternatives were evaluated: (1) precast, prestressed concrete decked bulb-tee, (2) steel plate girder with concrete deck, (3) trapezoidal steel box girder with concrete deck, and (4) cast-in-place, post-tensioned concrete box girder. Each alternative was evaluated for estimated cost, design criteria satisfaction, and construction methodology to determine the most feasible bridge type for this project.

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1 Introduction

The Moose Creek Bridge Project aims to provide an improved alignment from MP 53 to MP 56 on the Glenn Highway by constructing a bridge that will cross a canyon containing Moose Creek. Project site is located on the Glenn Highway at 61˚ 40’ N, -149˚ 2’ W, about seven miles outside of Palmer, Alaska as shown in Figure 1 in white. This section of the highway circumvents a canyon and utilizes a lengthy route with grades that reach 8% and horizontal curves. Constructed in 1951, the existing bridge has been deemed structurally deficient. Due to these considerations, the Alaska Department of Transportation and Public Facilities is considering a bridge that would cross the Moose Creek canyon in a more manageable alignment and grade. This would allow a higher speed limit to be maintained as well as increase driver safety.

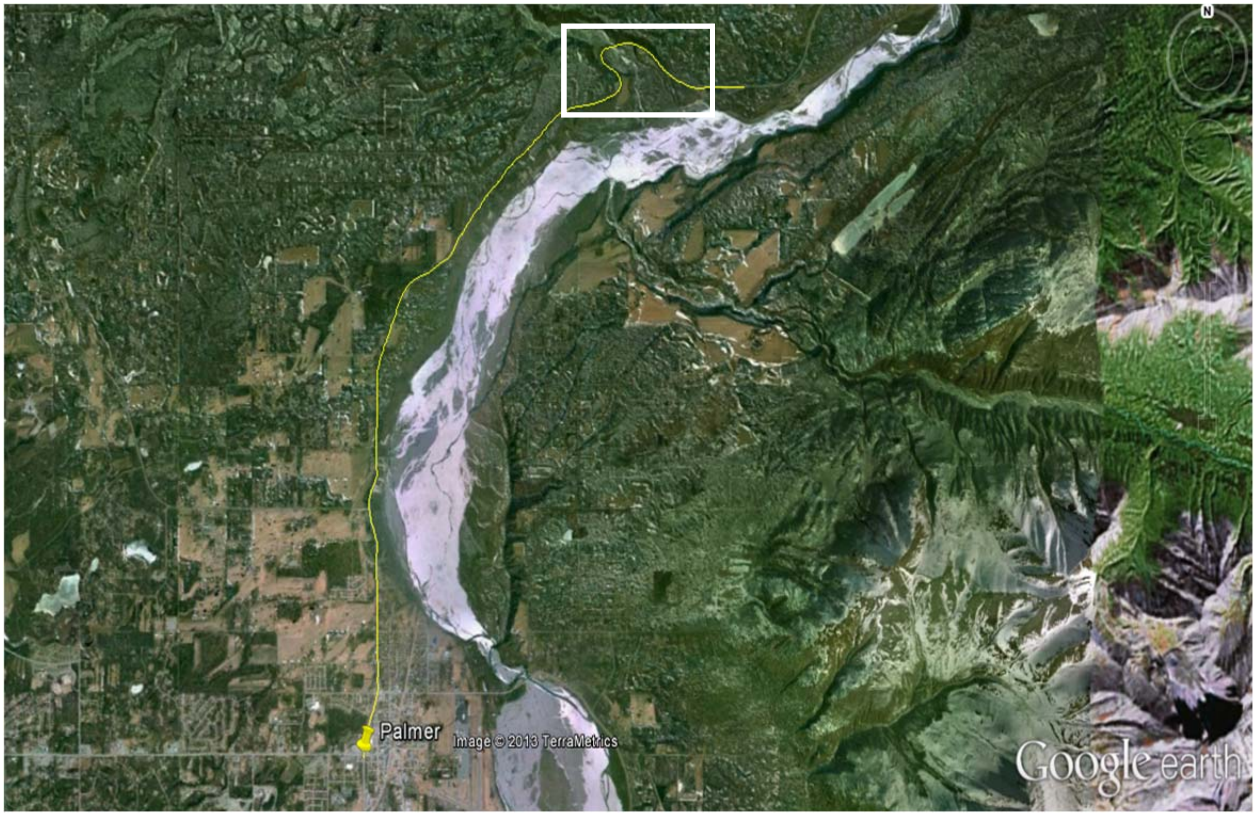

Figure 1-1 Project Site Location, Glen Hwy. MP 53 - MP 56

The new bridge will cut straight across Moose Creek in order to bypass the existing, circuitous road. However, this alignment produces several challenges including Moose Creek, canyon depth, and an Alaska Railroad owned Right of Way. Taking these factors into account, the most ideal bridge was conservatively assumed to be about 800 feet long. This dimension was adapted into each of the alternatives for consideration.

1.1 Background

The AK DOT&PF provides many engineering services for the state of Alaska. These range from airports to highways and bridges.

1.2 Project Objectives

As part of the Senior Design class for the Civil Engineering program at the University of Alaska Anchorage and working with the AK DOT&PF, a bridge type was selected from four alternatives. The alternatives considered were: (1) precast, prestressed concrete decked bulb-tee, (2) steel plate girder with concrete deck, (3) trapezoidal steel box girder with concrete deck, and (4) cast-in-place, post-tensioned concrete box girder.

2 Existing Conditions

Figure 2-1 Moose Creek canyon

The canyon that will be traversed by the bridge is about 1700 feet long and contains a good amount of vegetation in addition to the creek and railroad right of way. There are clearings in the trees for power lines that give a decent estimation of the proposed alignment for the bridge.

3 Design Criteria

One of the design criteria considered was bridge height. According to drawings from AK DOT&PF, the proposed vertical alignment is such that the highway is 80 feet above the valley floor. This is illustrated in Figure 3-1. The second design criterion for this proposed alignment is the spanning of the 40-foot wide creek as well as the 105-foot wide railroad right of way. And the third design criterion is the proposed horizontal alignment from AK DOT&PF, which crosses the canyon and ties back into the Glenn Hwy. at MP 55.5 as shown in Figure 3-2.

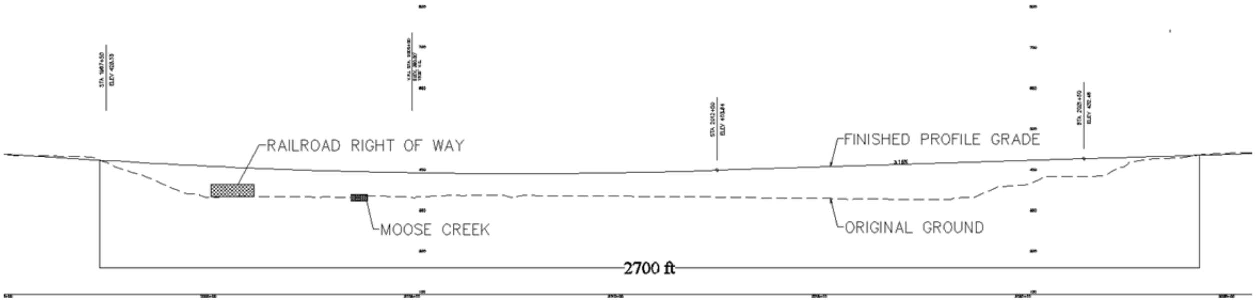

Figure 3-1 Moose Creek Project Vertical Alignment

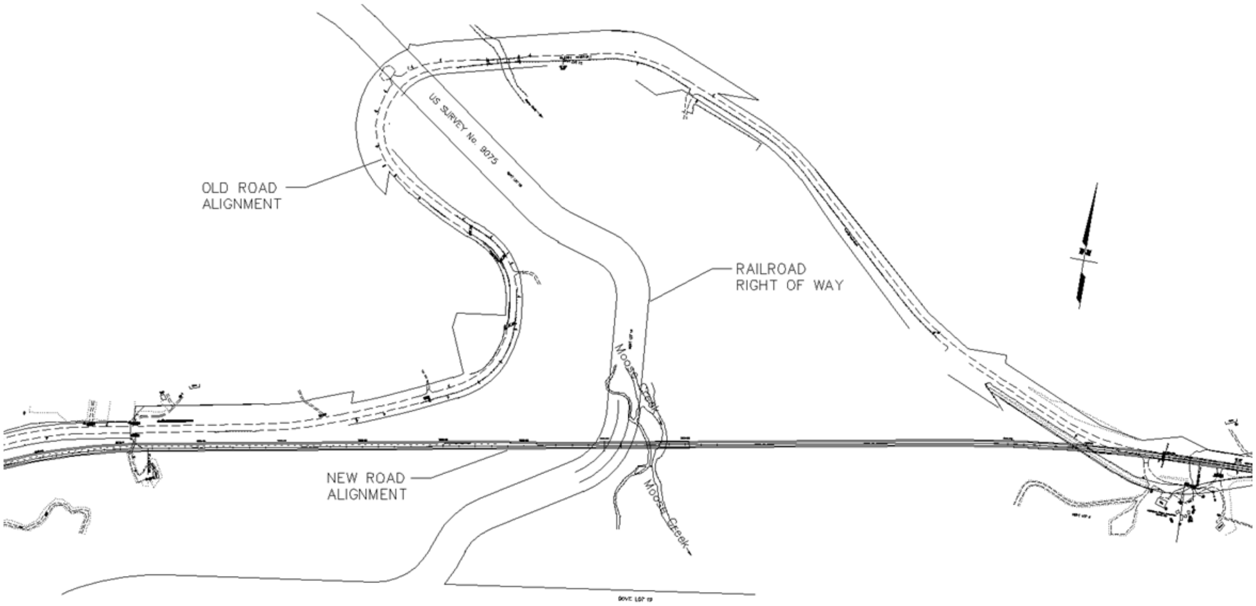

Figure 3-2 Moose Creek Project Horizontal Alignment

4 Bridge Type Alternatives

This section goes over the advantages and disadvantages as well as construction methods for each of the different design alternatives.

BRIDGE TYPE SUMMARY

Alt.	Superstructure Type	No. of Spans	Total Length	Span Configuration	Total No. of Girders
1	Precast Concrete Bulb-Tee	6	800’	2x120’ 4x140’	48
2	Steel Plate I-Girder w/ CIP deck	4	800’	2x180’ 2x220’	4
3	Trapezoidal Steel Box Girder w/ CIP deck	4	800’	2x180’ 2x220’	2
4	CIP Concrete Box Girder	3	800’	2x260’ 1x280’	-

Table 1 Summary of Bridge Alternatives

4.1 Pre-stressed Concrete Bulb-Tee

4.1.1 Advantages:

Precast Concrete offers many advantages, especially in Alaska. Girders are cast and cured in a controlled environment allowing manufacturers to make high strength concrete of up to 12,000 psi. In the bulb tee design option, the top flange doubles as the bridge deck. This removes the need for formwork, a time consuming and expensive process. As a result, precast girders are a favorable option for contractors who have to deal with the short construction season in the Alaskan summer.

Pre-stressing the concrete accounts for its lack of tensile strength and puts the girder in a constant state on compression reducing the likelihood of crack formation. This is important as cracks in concrete allow moisture to enter the concrete, which will corrode the steel reinforcing and drastically reduce the strength of the bridge over time. For this reason, concrete Bulb-Tee girders have very low maintenance costs and a long life expectancy.

Bulb Tee bridges are very common in Alaska. This means that it is likely that a contractor will have experience in constructing this bridge type, and as a result, construction and maintenance costs are well understood.

4.1.2 Disadvantages:

In Alaska, the maximum span length for decked bulb tee girders is 148ft. It is possible to ship longer girders from outside of Alaska, but costs would increase astronomically. This limit on span length results in more tall and expensive supports in the overall structure for this project compared to the longer spans of the steel girder bridge options. Also, concrete bulb tee bridges are usually only built with simply supported spans. Continuous bulb tee beams are not typically constructed because of the large tensile forces that develop as a result of the moment over the supports.

4.1.3 Construction:

Two cranes will be positioned on a temporary track on the valley floor, which will require earth work. These cranes will lift the girders from either end and move them into position. A temporary work bridge will be required for the cranes in order to place the girders on the east side of the gorge into place. The other option is to use a beam launcher which will move the girders in place starting from the east side of the gorge, moving to the west. The use of the beam launcher means that cranes will not be needed for superstructure construction. However, considering that the cranes will already be used to place the piers and drilled shafts, it is more likely that construction will not involve beam launchers.

4.2 Steel Plate Girder

4.2.1 Advantages

Steel Plate Girders are formed by welding three steel plates together forming an I-girder. The adjacent girders are braced together increasing their torsional stiffness. Steel plate girder bridges can accommodate large continuous spans. Due to the large cost of piers and drilled shafts, it is advantageous to use longer spans to reduce the number of piers that are required.

4.2.2 Disadvantages

Steel Girders are quite expensive compared to concrete options. The bracing required to provide torsional restraint is only put in place after the girder has been moved into position. Before it is braced, the beam is more susceptible to lateral torsional buckling, which must be taken into consideration during construction of this type of bridge.

Maintenance costs are relatively high because of the measures taken to prevent steel from corroding. Steel is prone to corrosion when directly exposed to weather. Repainting to minimize these effects of weathering is important but can be costly. It is estimated that over the bridges lifetime, the repainting costs can be as high at 25% of the initial cost.

4.2.3 Construction

Two cranes would be utilized in placing the girders due to availability; they are already on site as they were used for the substructure construction. The differences in placement methods from the concrete bulb tee option arise in the continuous nature of the steel bridge. To optimize the design, different cross sections are used to handle the areas where the superstructure develops a maximum negative or maximum positive moment. Larger cross sections are required to carry the larger negative moment over the supports. Figure 4 shows the configuration of these two different beams. Given that the negative moment carrying beam is directly supported by the pier, two cranes will be needed to put the girder in place. Once these girders are secured, the positive moment carrying beam can be lifted into place and secured with large bolted flanges. Once the girders are in place, then the deck can be laid. Construction costs are high for steel girder bridges as contractors in Alaska do not have as much experience with steel bridges as opposed to the Precast Concrete Bulb-Tee bridges.

4.3 Cast-in-place Concrete Box Girder

Concrete box girders are unique in that they are typically built in short sections. In considering this option, it was decided that the segments be cast-in-place instead of precast because the associated costs with formwork and precasting were too high. Construction and casting procedures of the segments are explained below in the construction section.

4.3.1 Advantages

This type of bridge is as widely used in the lower 48 as the bulb-tee option is used in Alaska. Maximum span lengths are larger for this option compared to the bulb-tee option, so this reduces the number of piers needed for the structure. Casting the segments in place removes the need for a casting yard, specialized forms, and transportation of the girders. Cost is somewhat reduced because of this.

4.3.2 Disadvantages

Given that this bridge will be supported 80 feet high, there will be significant challenges with setting up formwork or even getting big enough cranes to the site. However, this may actually turn out to be an advantage given that the bridge will span the creek, and this will reduce the need for machinery at ground level. Finally, these types of bridges are not commonly built in Alaska. Bulb Tee technology has been widely adopted in Alaska as the precast bridge girder option rather than the segmental concrete. There is a low chance that local contractors will be familiar with this option. As a result, contractors and labor may have to be brought in from out of state, which would dramatically increase costs.

4.3.3 Construction

This type of bridge becomes more economical as bridge length increases. Less than 100 segments will be used for a bridge of this length, so the other bridge type alternatives become more attractive cost-wise. However, construction methodology for this option was still evaluated.

The segmental concrete bridge varies drastically in construction from the bulb tee and steel plate girder bridges. Starting from the pier, the sections are cantilevered out in both directions to keep the structure balanced. Once the segment has been added the cables are tensioned pulling the bridge into compression. The process continues until the two ends of the bridge meet.

Casting the segments in place removes the cost of setting up a temporary casting yard. A slip form type system can be used which means that each section is cast in its final position then post tensioned. The forms then move forward to cast the next section. Casting the segments in place removes the need to make a casting yard or transport the segments out from Anchorage. However, the downside is that construction is more time consuming and expensive, and the slip form system must be brought up from the contiguous United States.

The other construction method for this option that was considered was precasting the segments and trucking them out to the site. In this method, forms will still need to be barged up from a manufacturer in the contiguous United States. However, it may be effective in environmentally sensitive situations where it is difficult to set up formwork for a cast in place concrete box girder or even if it is too difficult to use a crane to put the girders in place. However, construction method is anticipated to be very expensive. The other construction method that was determined as a possibility for the precast segmental bridge type is a span by span option, which uses a temporary truss. Segments roll along the truss until they are in position, then the next segment is put in place. Despite the fact that the truss can be quite expensive, this option will often be more cost effective.

4.4 Steel Box Girder

Steel Box Girders are very similar in design to that of the steel plate girder. The difference is that the two webs share a common bottom flange. The webs are also angled, primarily for aesthetic reasons. When the box is closed, either through a bracing system across the top flanges or by using a permanent steel formwork for the deck, torsional resistance of the girder is radically increased.

4.4.1 Advantages

Steel box girders are lightweight compared to the other options, and longer spans are viable. Costs may be reduced as fewer piers are needed to support the highway. Due to its high torsional stiffness, steel box girder bridges handle curvature well and are ideal for horizontally curved bridges. Lastly, steel box girders are considered to be aesthetically pleasing because of their smooth, clean lines.

4.4.2 Disadvantages

As was the case with the steel plate girder bridge, steel corrosion and maintenance costs will become a large cost throughout the lifetime of the bridge. Another factor in maintenance is accessibility within the confined space within the steel box for labor. These girders are also expensive because of the reduced scope for automated fabrication.

4.4.3 Construction

Construction for the steel box girder is very similar to the steel plate girder. Two cranes will be placed on the ground and lift one girder at a time into place. The first girders to be moved into place will be the ones that sit over the top of the piers. Once these are secured then the cranes will lower the mid span girder into place and the flanges will be bolted together.

5 Substructure

5.1 Abutments

The abutments utilize steel H-piles, driven into the ground in order to provide end supports to the bridge. The fill that is utilized underneath the abutments must be compacted to a much higher percentage than normal fill in order to provide the support that the abutment requires. In addition to providing support for the superstructure, the abutment also acts as a retaining wall for the fill in order to prevent failure in the soil. Because of this use, the fill must not be too steep and a slope ratio of 2:1 is often used.

A typical abutment will be utilized in this project.

5.2 Piers

The piers are designed to transfer the loads from the bridge decking into the foundations. The piers must withstand not only the axial forces, caused by the deadweight of the concrete and the live loads of traffic, but also the lateral loads that are caused by wind and seismic forces. The piers transfer these forces into the ground through the drilled shaft or driven pile below each pier that extends into the subsurface. The piers and drilled shafts are reinforced with circular or spiral steel in order to provide tensile strength to the concrete in all directions.

It was determined that single 10-foot diameter reinforced concrete columns will serve as the supports for the bridge. Through conservative assumptions, the foundations under these columns were determined to be 12-foot diameter drilled shafts at a depth of 140 feet.

6 Recommended Alternative

With the information available, the Precast Concrete Bulb-Tee Girder option has been determined to be the preferred type of bridge for this project.

To be considered feasible, a bridge type needs to meet the design criteria and the needs of the project. Although all of the bridge types satisfy the design criteria, some alternatives fulfill the conditions better than others. For this reason, bridge types are judged based on how well they meet the conditions of constructability, maintenance, and cost effectiveness. And the preferred bridge option is selected based on these criteria.

6.1 Constructability

Description: Given the construction difficulties on the project site, there will be significant challenges with all proposed bridge types. Analysis will include how well Alaskan contractors are able to utilize the construction techniques that are required of each type of bridge as well as the cost incurred.

All of the design the same amount of soil excavation. To be able to utilize the cut to fill the gorge, a temporary bridge would need to be constructed across Moose Creek in order to prevent haul units from employing the highway. This would minimize traffic disruption and increase efficiency. In addition, a temporary bridge can accommodate larger haul units. This bridge would need to be able to withstand a 4-year storm as well as support 40-50 ton haul units that will transport the soil across the creek. Temporary construction easements will be required to make a route for the haul trucks to cross the gorge. This land will not be used directly for the bridge, only for ease in transport of the fill during the construction phase.

Environmental regulations can affect the construction process in a number on ways. Vegetation clearing will have to be undertaken to make room for the bridge and transportation to and from site. Vegetation clearing can only be done between May 1st and July 15th. If eagles nests are in the trees then it would be necessary to get a permit to continue work. Construction near waterways also has the following stipulations; preserving riparian habitat as much as possible, avoid in-stream work, maintaining natural stream morphology, avoiding channelization of the creek, avoiding placement of fill, piers or other structures below the ordinary high water line.

The Alaska Department of Fish & Game, U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, and Environmental Protection Agency are a few of the resource agencies that will be coordinated with to acquire permits and determine restrictions on project design and construction activities.

Evaluation: The simple span nature of the bulb tee girders simplifies girder placement making it easier to construct than the steel or concrete bridges. Also, contractor experience and familiarity in Alaska make the Precast Concrete Bulb-Tee Girder bridge the preferred option.

6.2 Maintenance

Description: For the maintenance aspect, concrete bridges are often cheaper to maintain over long periods. Steel bridges require maintenance in order to prevent corrosion from forming, whereas concrete does not require painting and has extremely low costs associated with maintenance.

Evaluation: The Pre-cast Concrete Bulb-Tee bridge alternative was determined to be the easiest and cheapest in terms of maintenance.

6.3 Cost

Description: Cost is evaluated in terms of material and construction costs. Bridge alternatives that are not typically constructed in Alaska, such as the concrete segmental box girder option, will have an additional cost factored in to account for shipping, specialized labor, etc. In addition to construction cost, a contingency factor of 30% was added to all bridge alternatives. Cost estimates seem to be higher than typical bridge costs mainly due to the lack of geotechnical data and resulting cost of the substructure. Cut and fill costs were neglected in the estimates because the focus of the project was solely on the bridge structure.

Evaluation: The precast concrete bulb-tee girder option has been determined to be the most cost effective.

COST ESTIMATION SUMMARY

Alt.	Superstructure Type	Estimated Cost	Notes
1	Precast Concrete Bulb-Tee	$24.9M	$724/ft.
2	Steel Plate I-Girder w/ CIP deck	$29.5M	$857/ft.
3	Trapezoidal Steel Box Girder w/ CIP deck	$34.0M	$990/ft.
4	CIP Concrete Box Girder	$43.6M	$1,270/ft.

Table 2 Summary of Moose Creek Bridge Project Cost Estimations

7 Acknowledgements

The authors would like to acknowledge the following people for their support in this project:

- Alaska Department of Transportation & Public Facilities
    - **Elmer E. Marx** – Bridge Design Engineer
    - **Sean M. Baski** – Project Manager
    - **Thomas Dougherty** – Construction Management
    - **Matt Dietrick** – Environmental Section
- University of Alaska Anchorage
    - **Scott E. Hamel** – Structural Mentor
- Figg Engineering
    - **Michael Keller** – Concrete Box Mentor
- Agg Pro Concrete
    - **Don Brucehaber** – Bulb Tee Contractor

8 References

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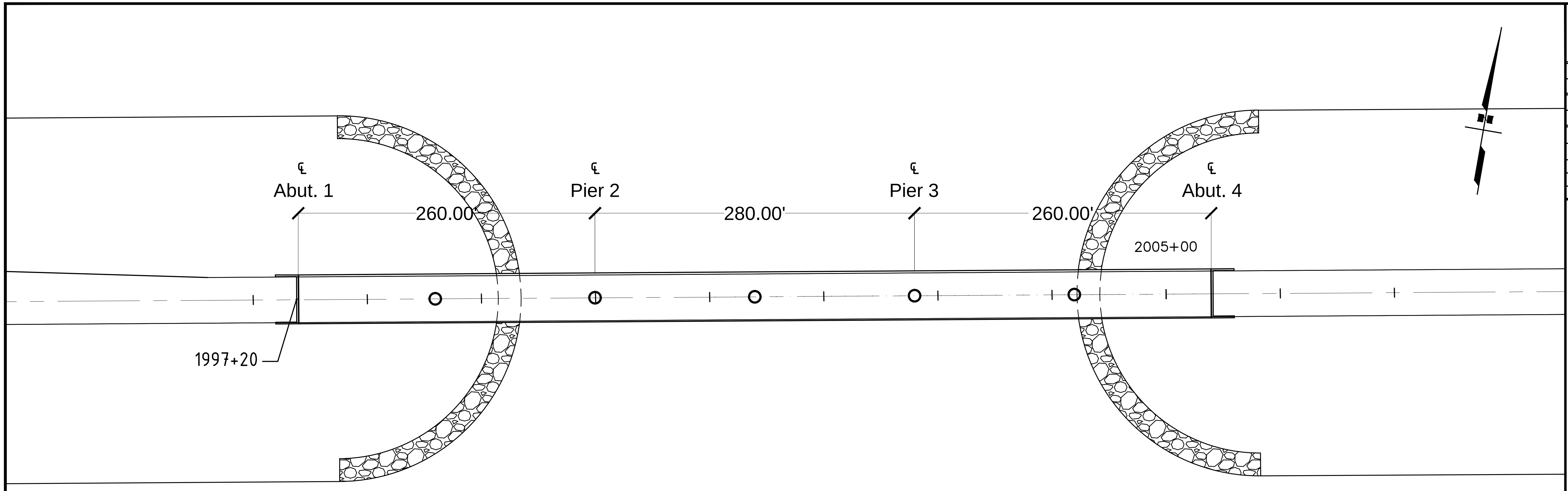
APPENDIX A: GENERAL LAYOUT DRAWINGS



PLAN VIEW

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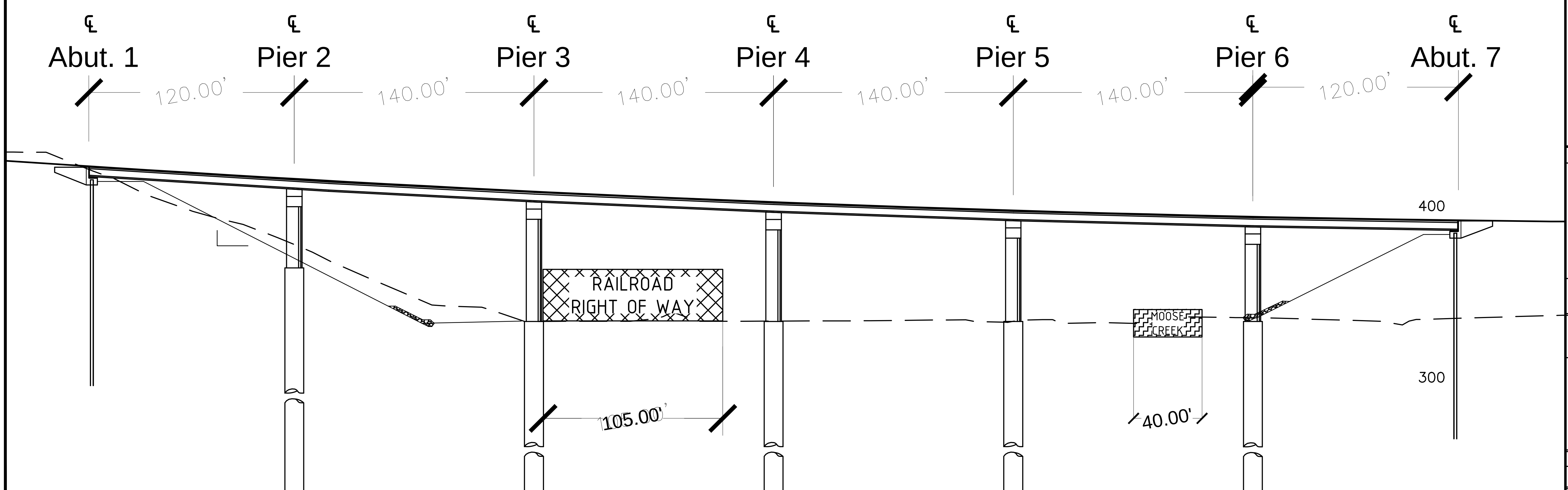

PROFILE VIEW

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GENERAL LAYOUT
DECK BULB TEE
GIRDERS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
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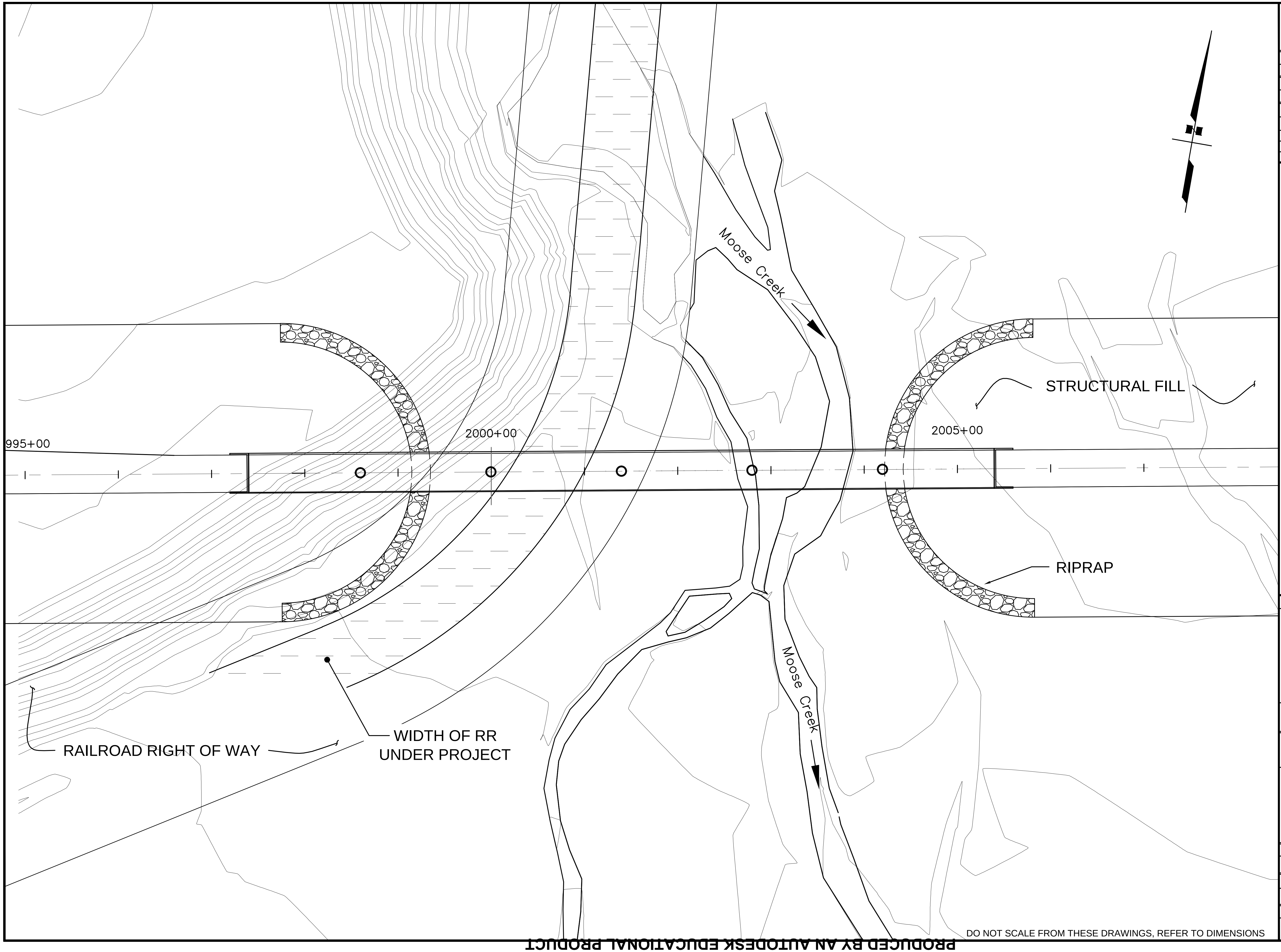

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SITE PLAN
DECK BULB TEE
GIRDERS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
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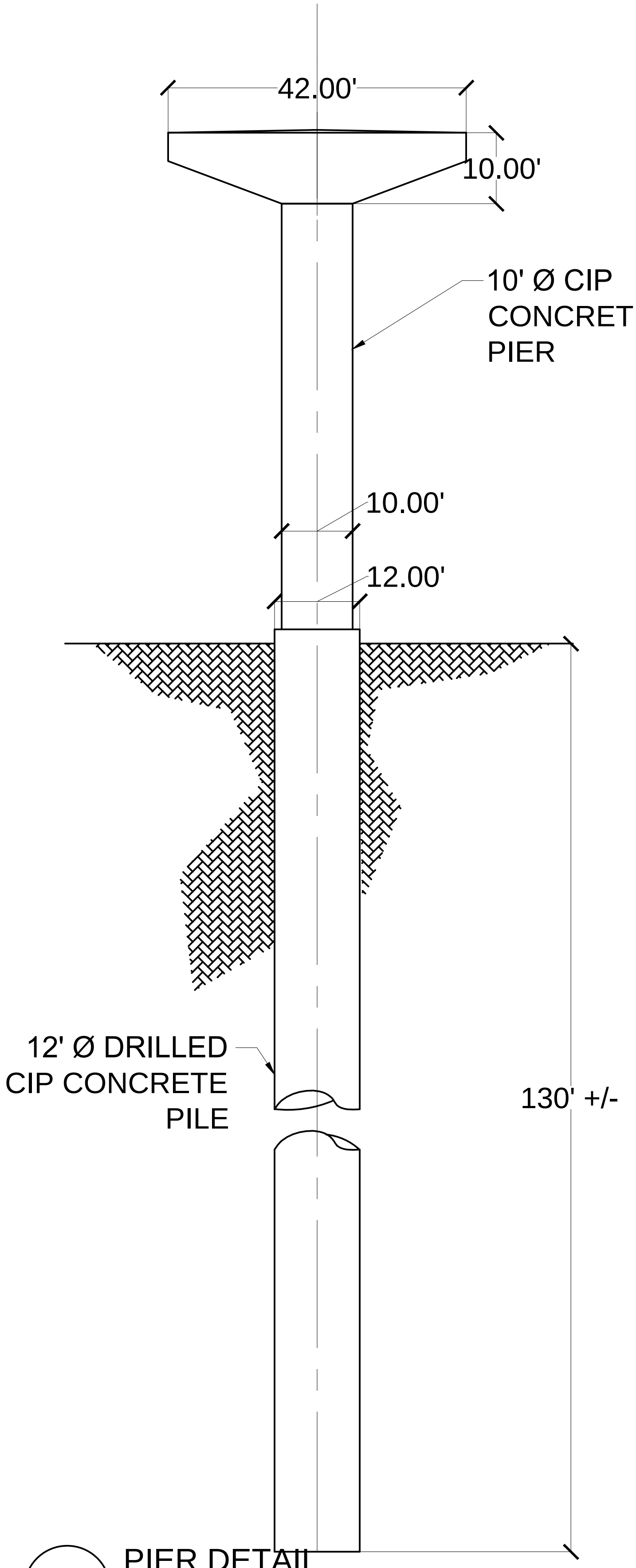

PIER DETAIL

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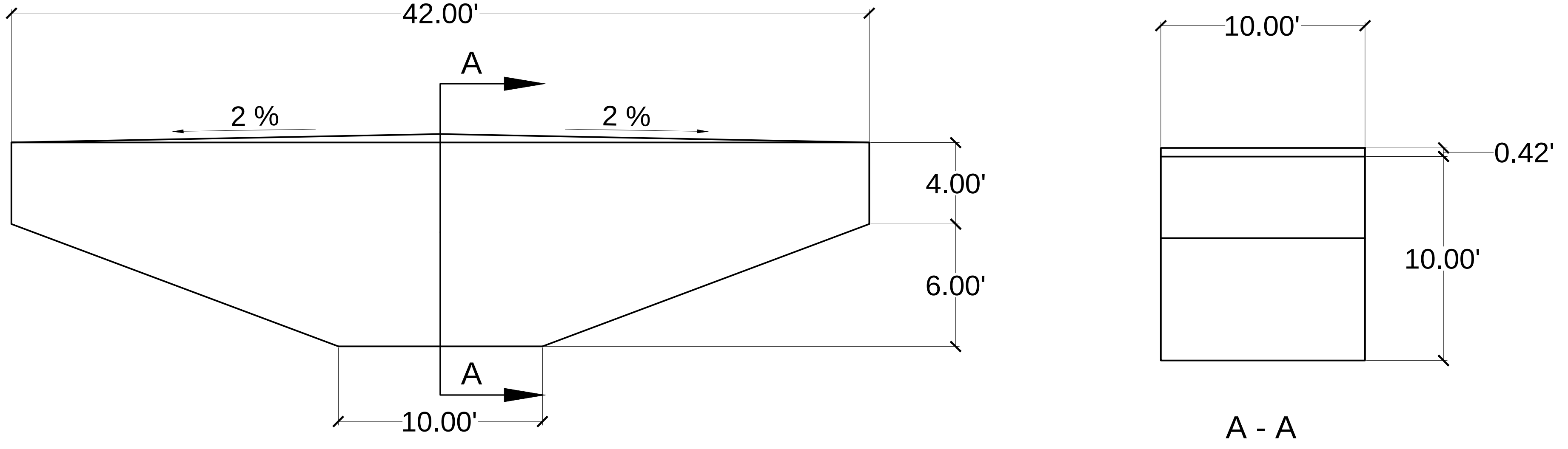

HAMMER HEAD DETAIL

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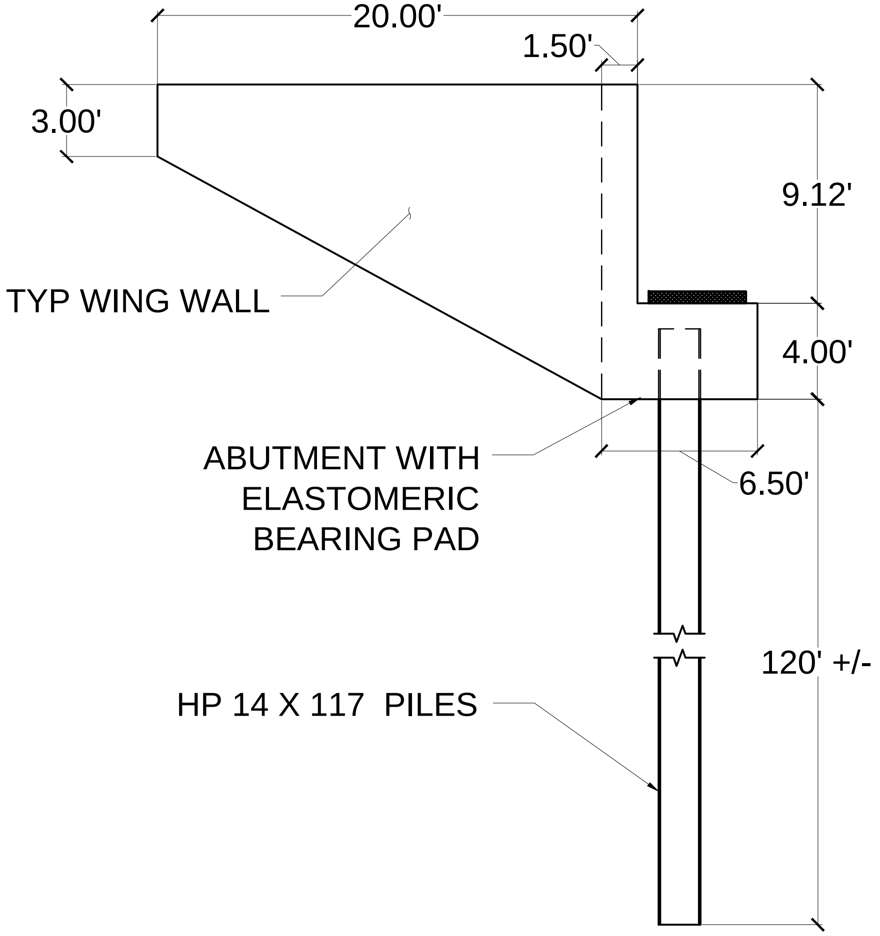

TYPICAL ABUTMENT PROFILE

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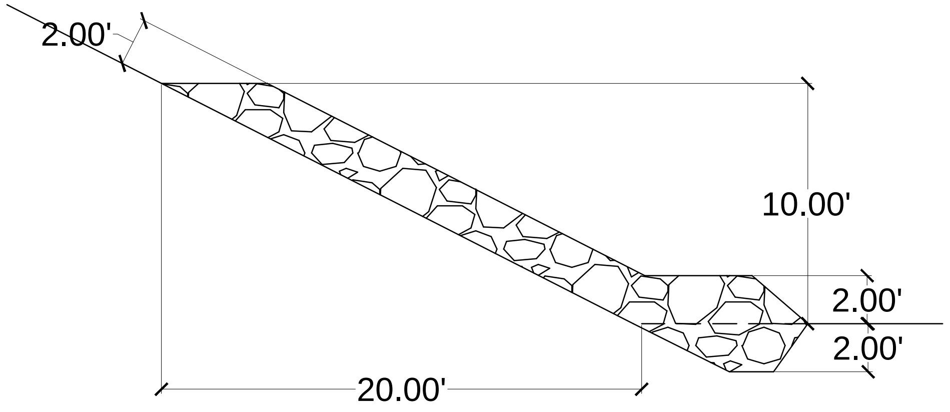

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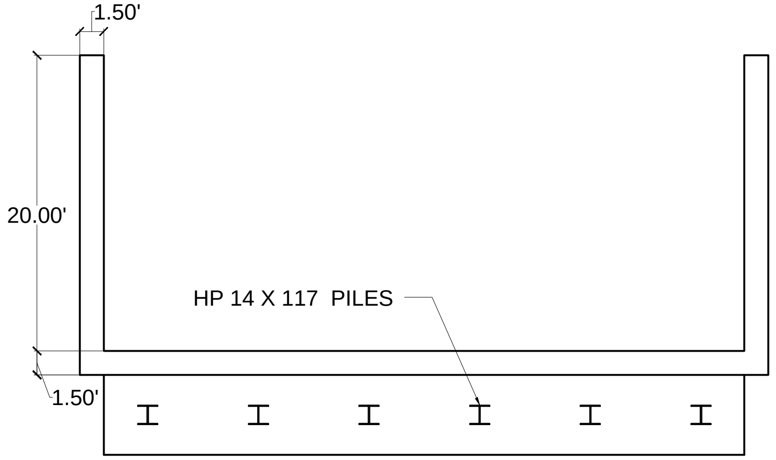

TYPICAL ABUTMENT PLAN

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SUBSTRUCTURE DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**
SHEET NUMBER	TOTAL SHEETS
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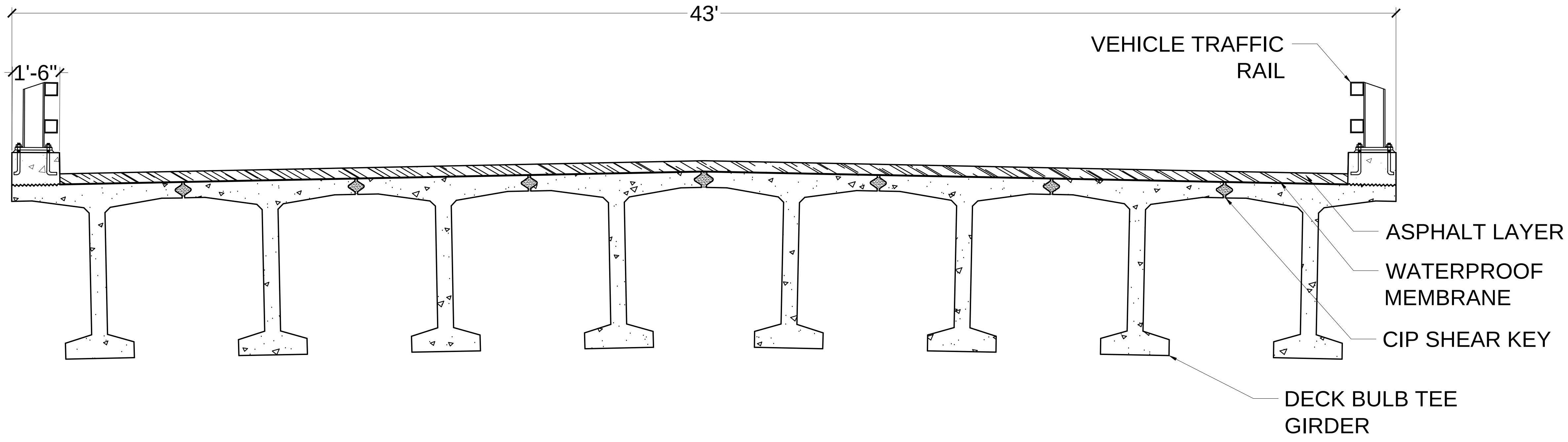

SUPERSTRUCTURE CROSS SECTION

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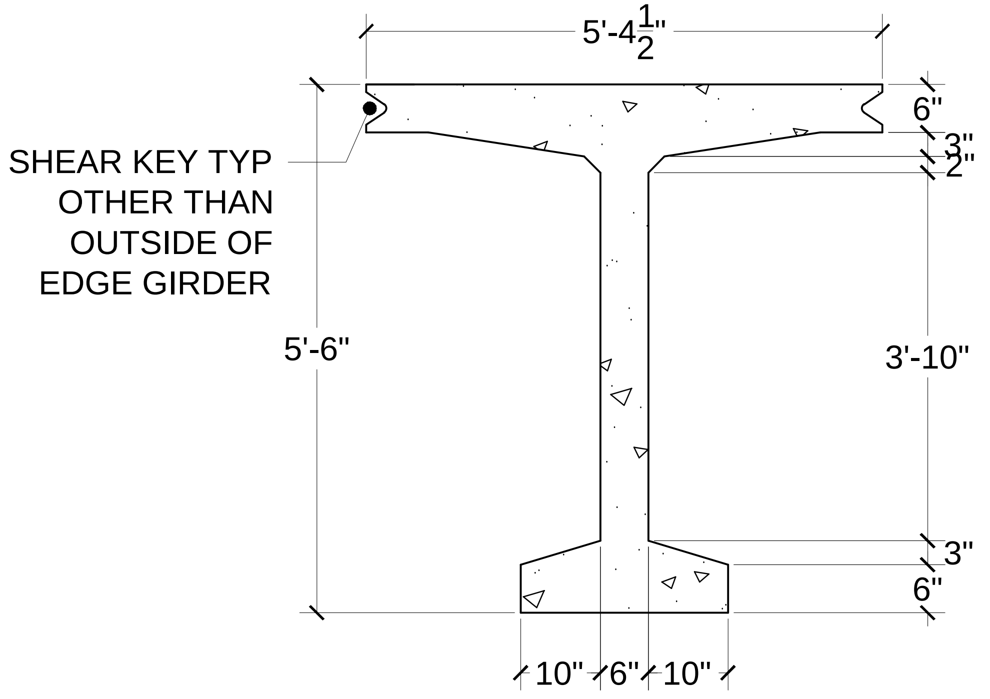

DECK BULB TEE CROSS SECTION

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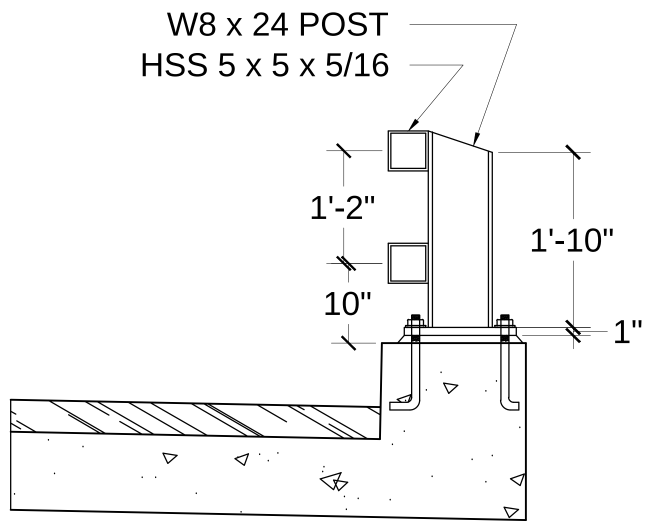

BRIDGE RAIL DETAIL

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DECK BULB TEE GIRDER DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**
SHEET NUMBER	TOTAL SHEETS
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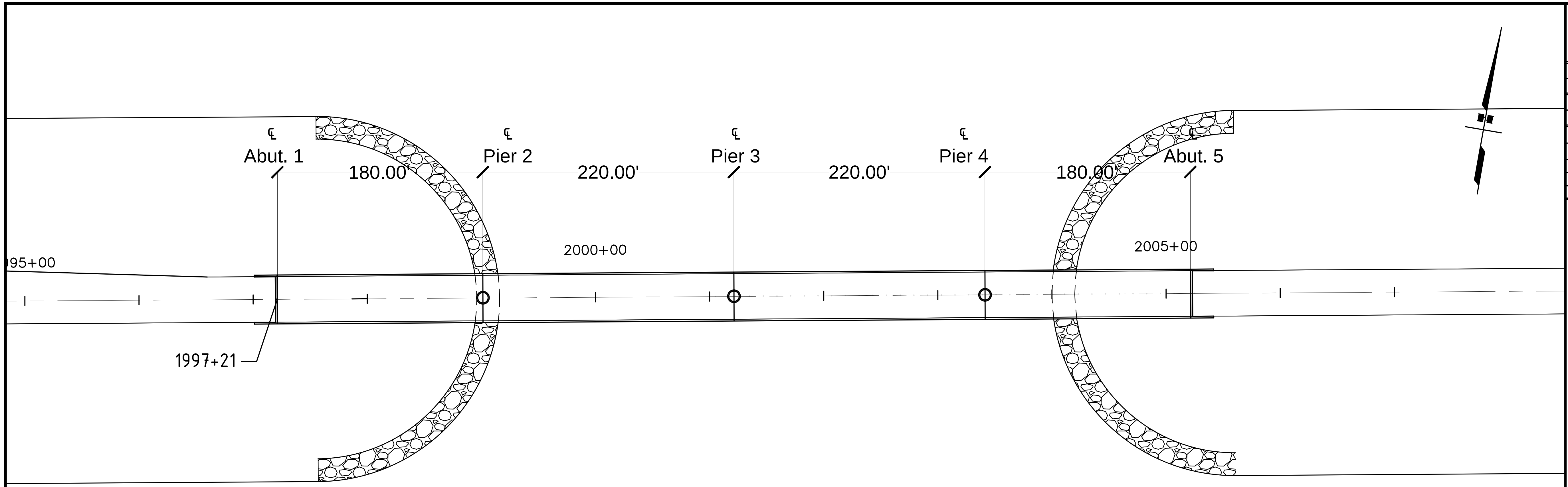

PLAN VIEW

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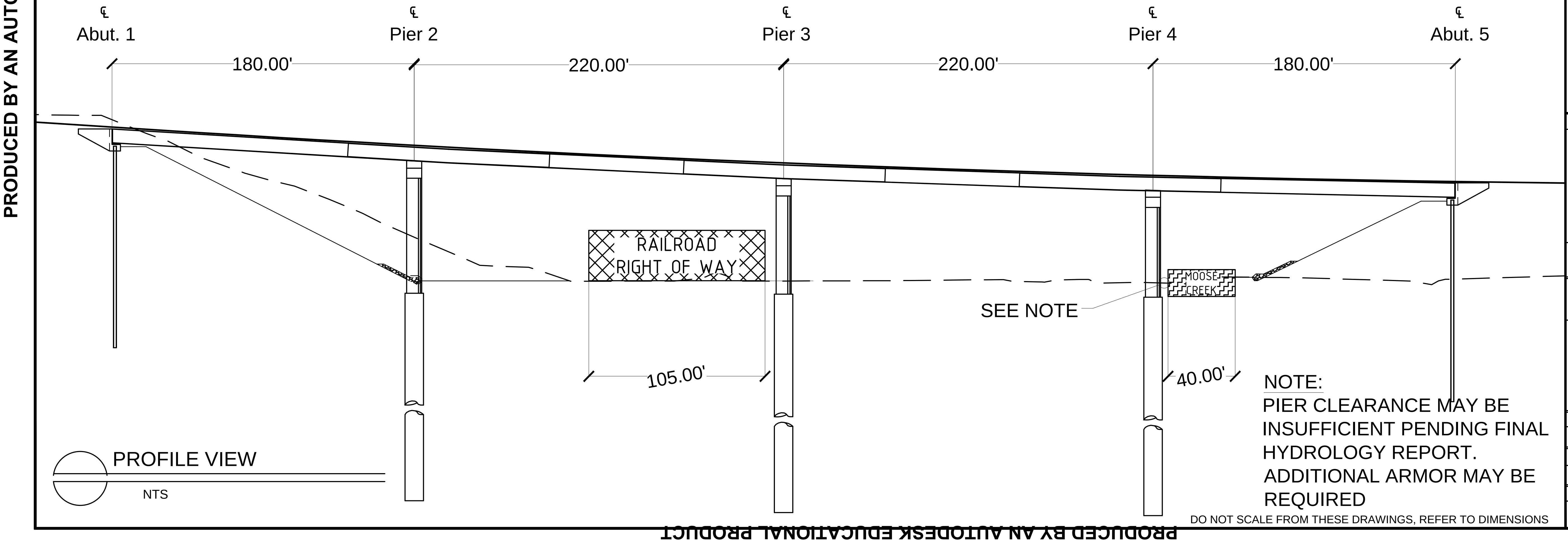

NOTE:
PIER CLEARANCE MAY BE INSUFFICIENT PENDING FINAL HYDROLOGY REPORT. ADDITIONAL ARMOR MAY BE REQUIRED

PROFILE VIEW

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SENIOR DESIGN

GENERAL LAYOUT
STEEL I GIRDER

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
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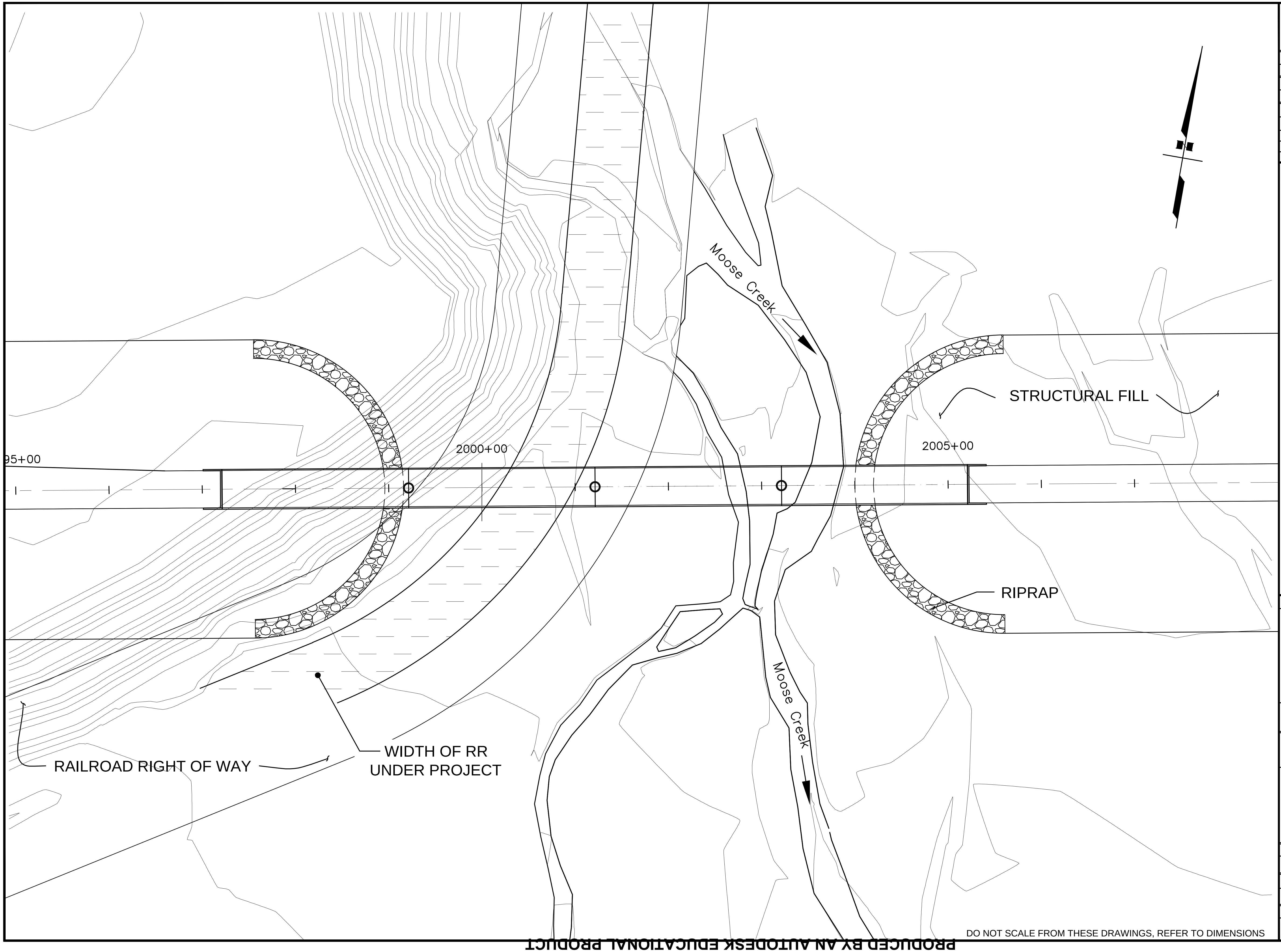

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SENIOR DESIGN

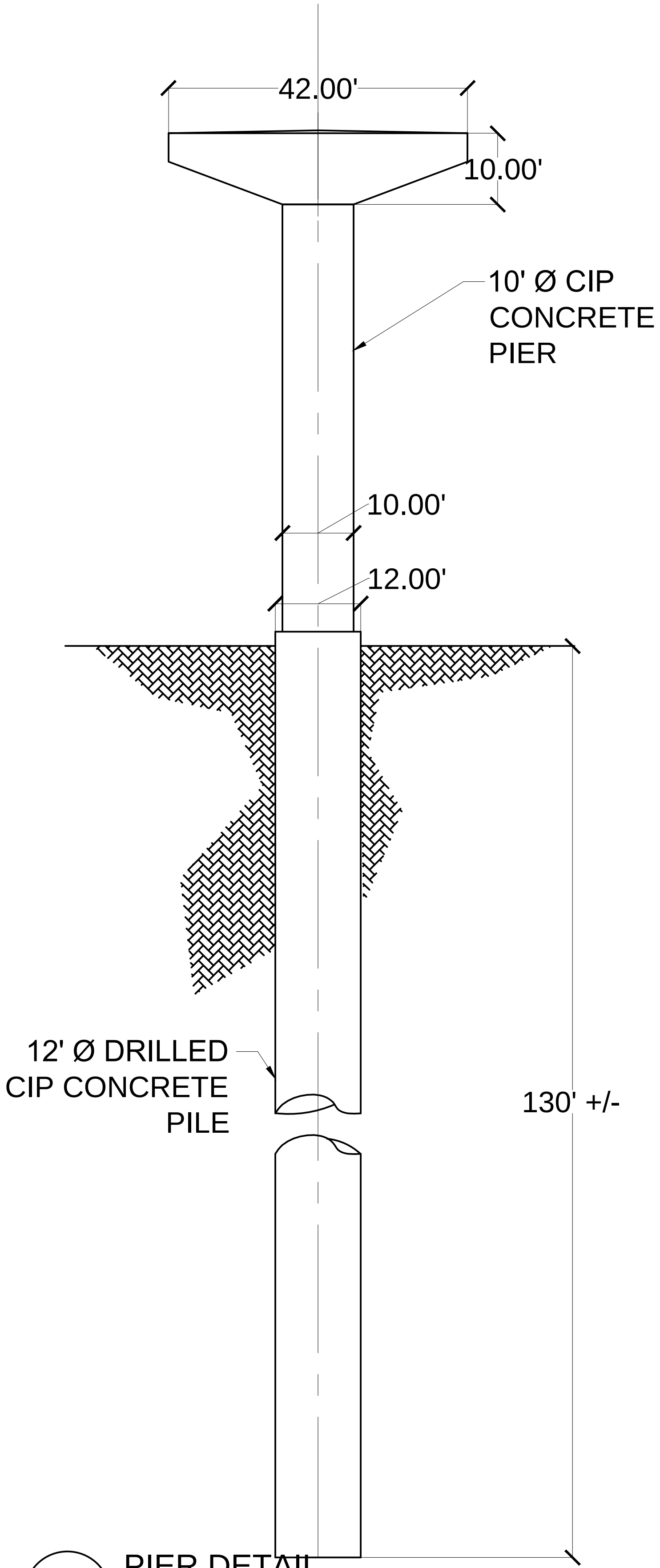
SITE PLAN

PROJECT DESIGNATION

MOOSE CREEK BRIDGE

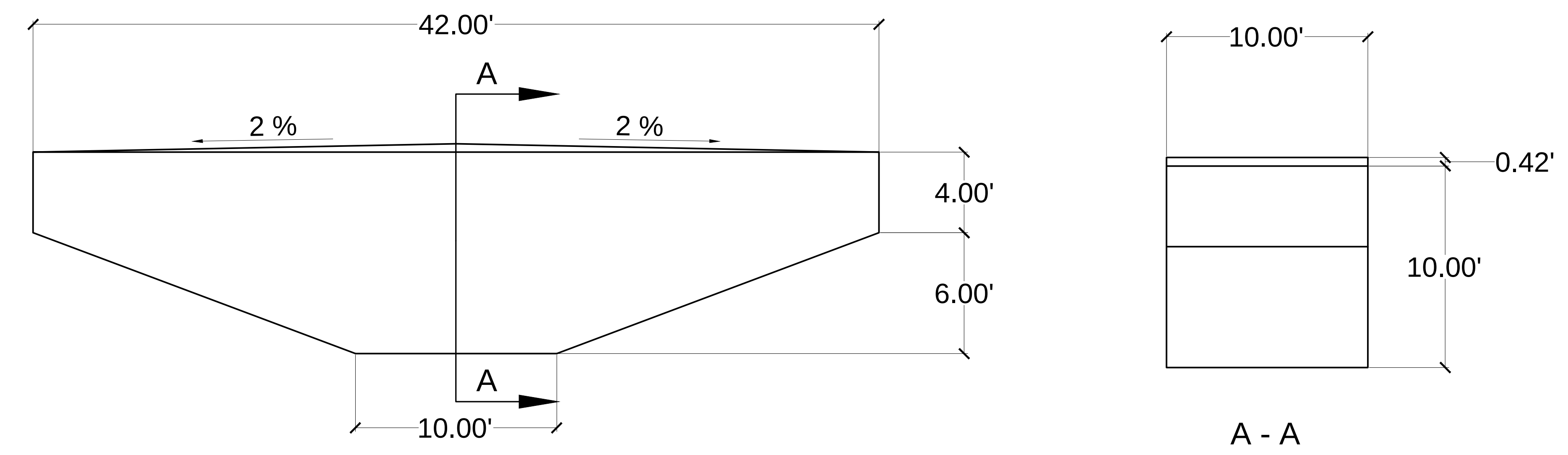
STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
2	**4**



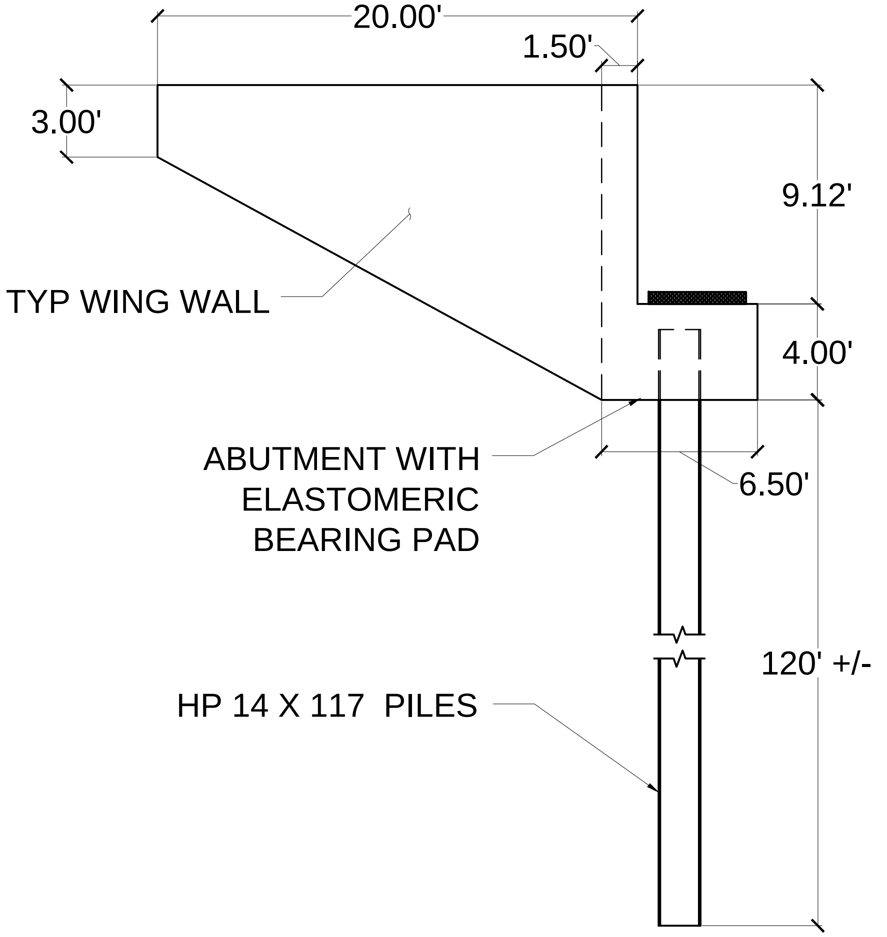
PIER DETAIL

NTS



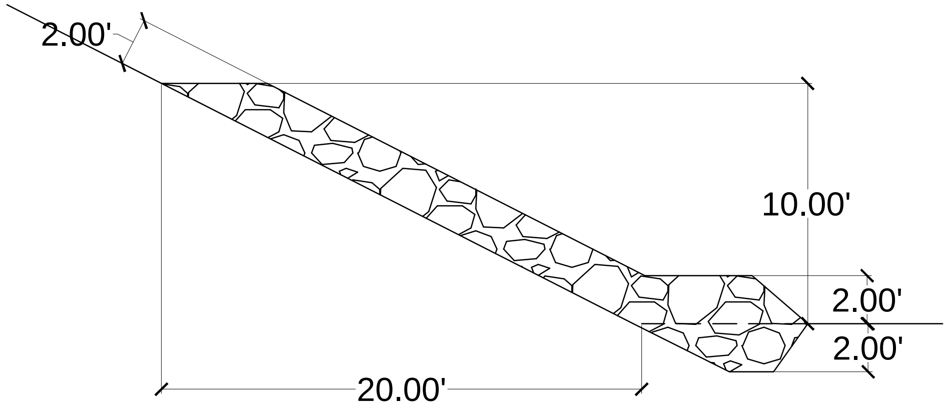
HAMMER HEAD DETAIL

NTS



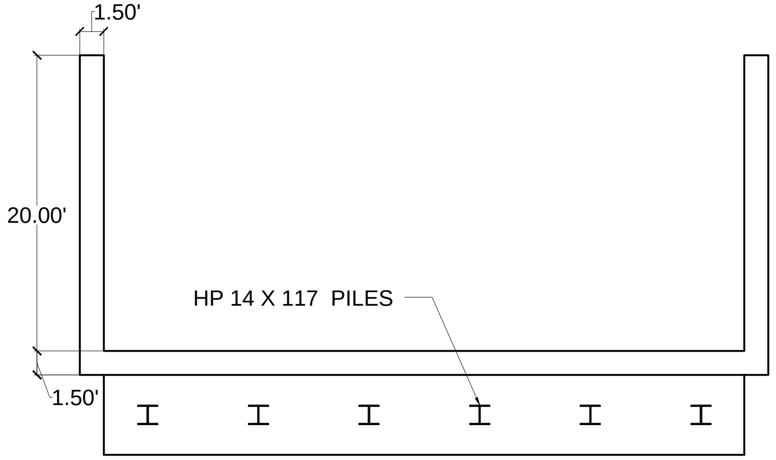
TYPICAL ABUTMENT PROFILE

NTS



RIPRAP DETAIL

NTS



TYPICAL ABUTMENT PLAN

NTS

DO NOT SCALE FROM THESE DRAWINGS, REFER TO DIMENSIONS

FILE Path:

ADDENDUM NUMBER

ATTACHMENT NUMBER

RECORD OF REVISIONS

No.	DATE	DESCRIPTION

CHECKED BY:
Liam Gibcus
Hunter Seibold
Newel Pangulayan

DRAWN BY: Mike Beauvais

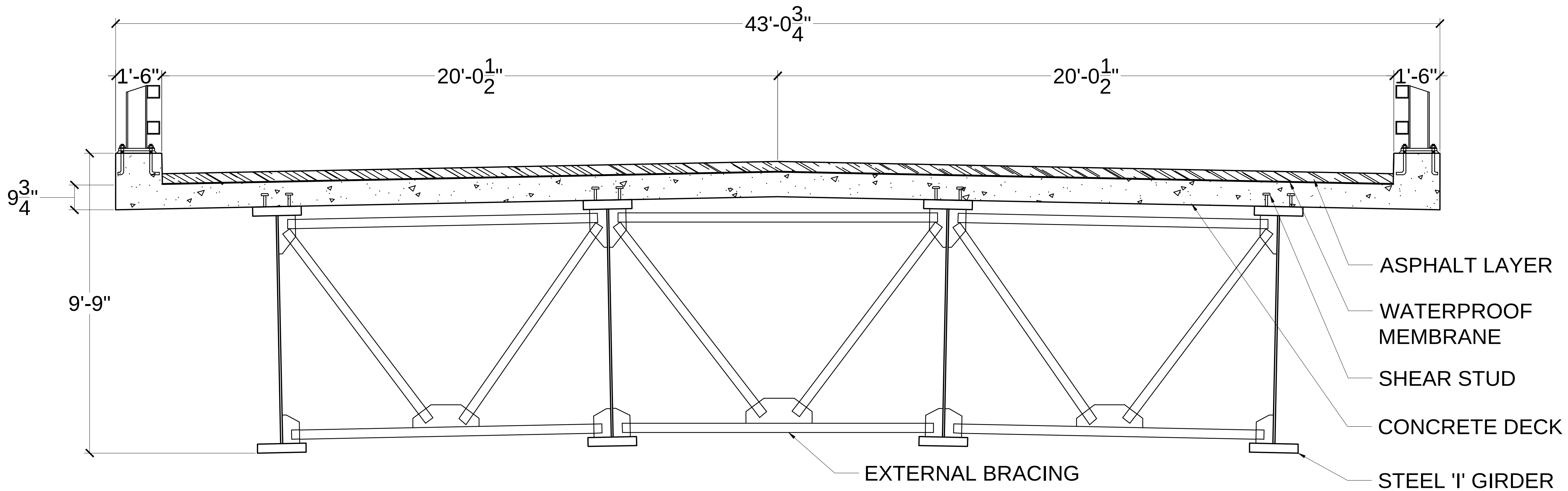
UNIVERSITY OF ALASKA
ANCHORAGE
CE-438
SENIOR DESIGN

SUBSTRUCTURE DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

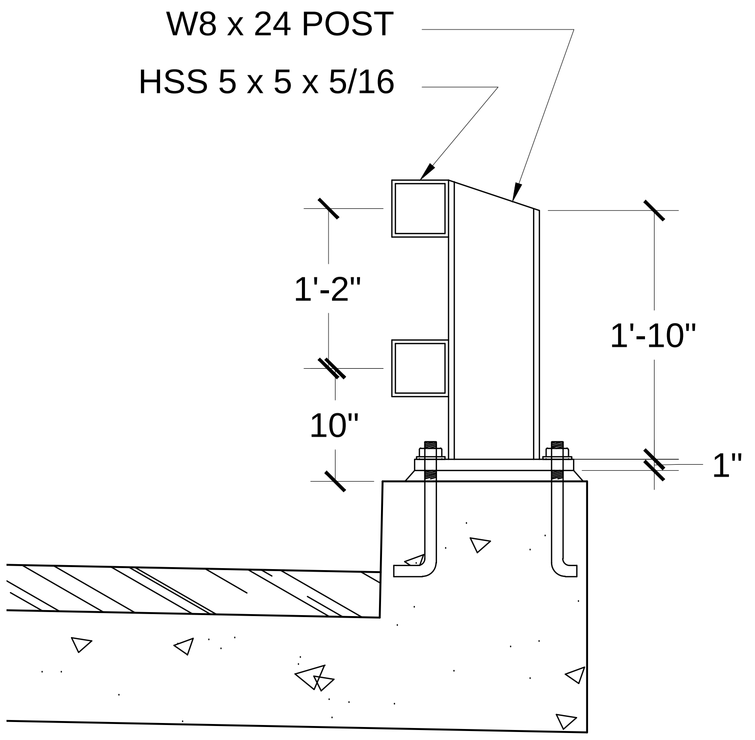
STATE	YEAR
ALASKA	**2013**

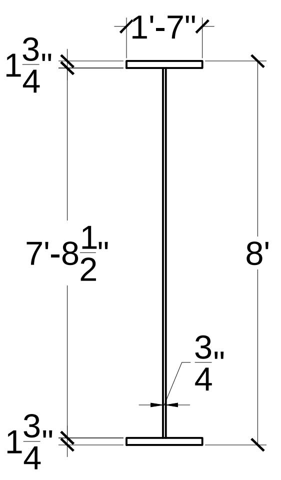
SHEET NUMBER	TOTAL SHEETS
3	4



SUPERSTRUCTURE CROSS SECTION

NTS





POSITIVE MOMENT SECTION

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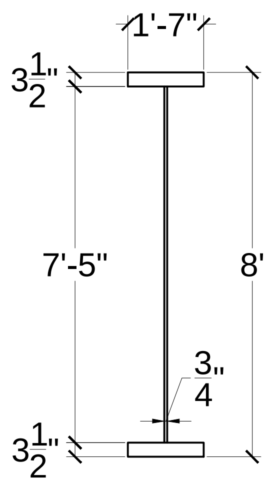

NEGATIVE MOMENT SECTION

NTS

BRIDGE RAIL DETAIL

NTS

FILE Path:

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CE-438
SENIOR DESIGN

SUPERSTRUCTURE DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
4	**4**

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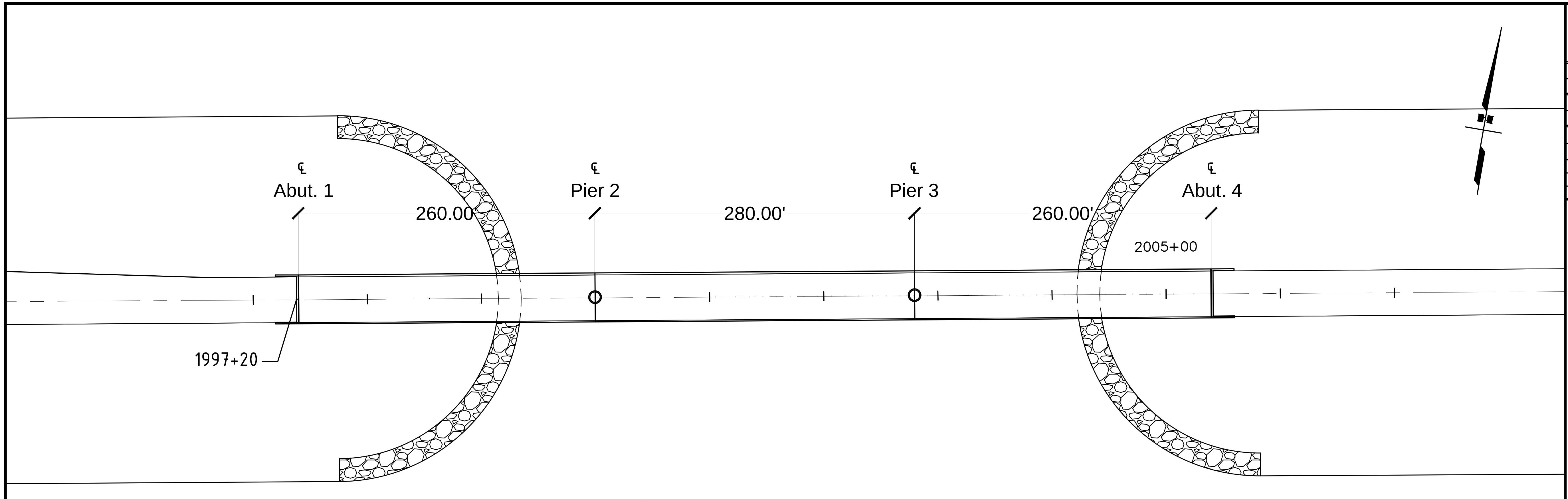
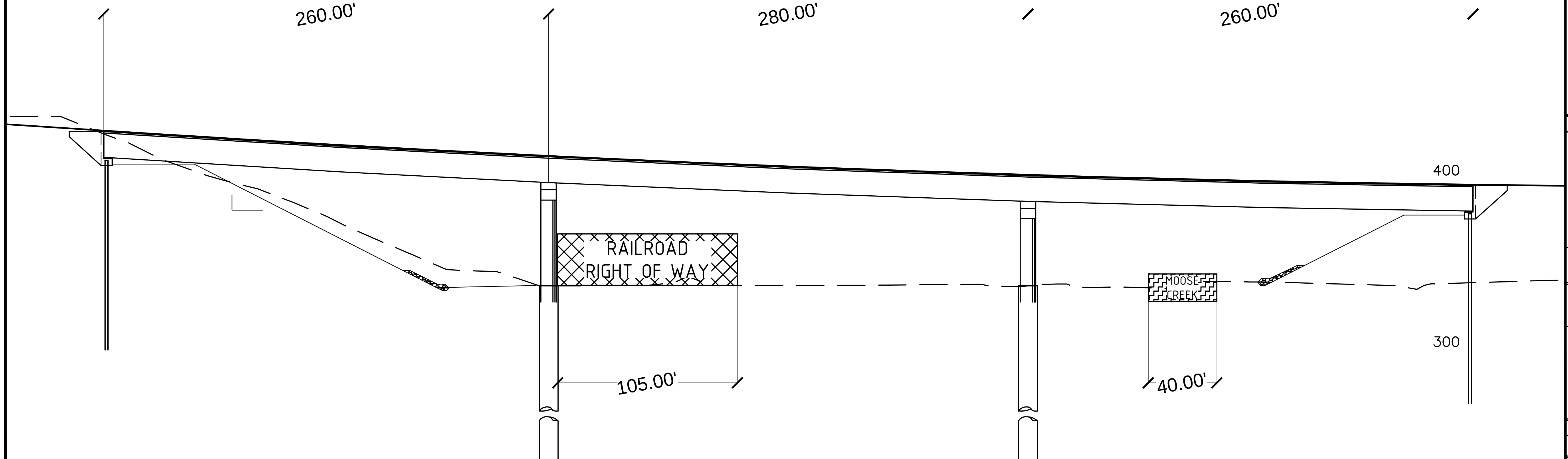


PLAN VIEW

NTS



PROFILE VIEW

NTS

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UNIVERSITY OF ALASKA
ANCHORAGE
CE-438
SENIOR DESIGN

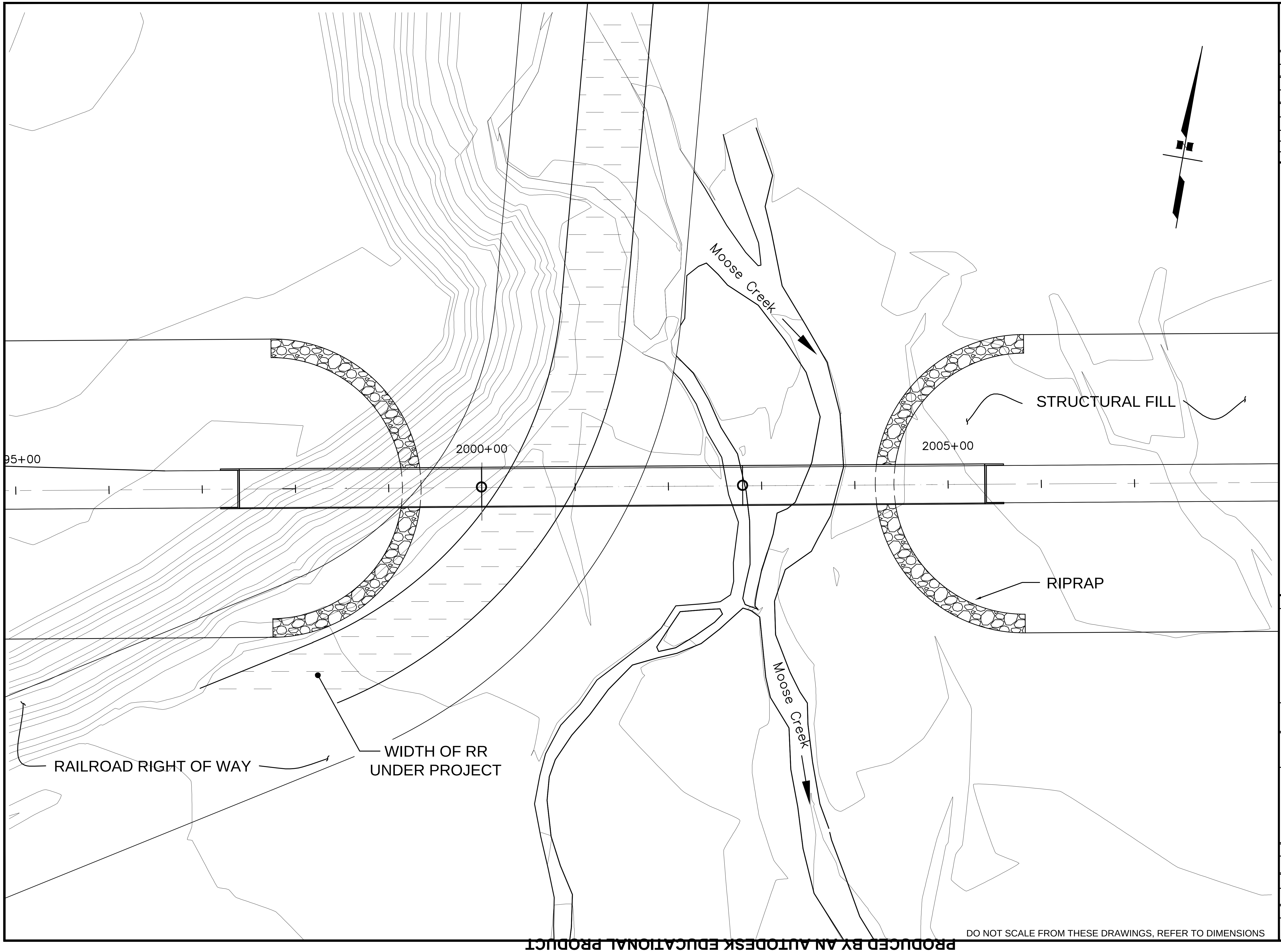
**GENERAL LAYOUT
CONCRETE BOX
GIRDER**

PROJECT DESIGNATION

MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
1	4



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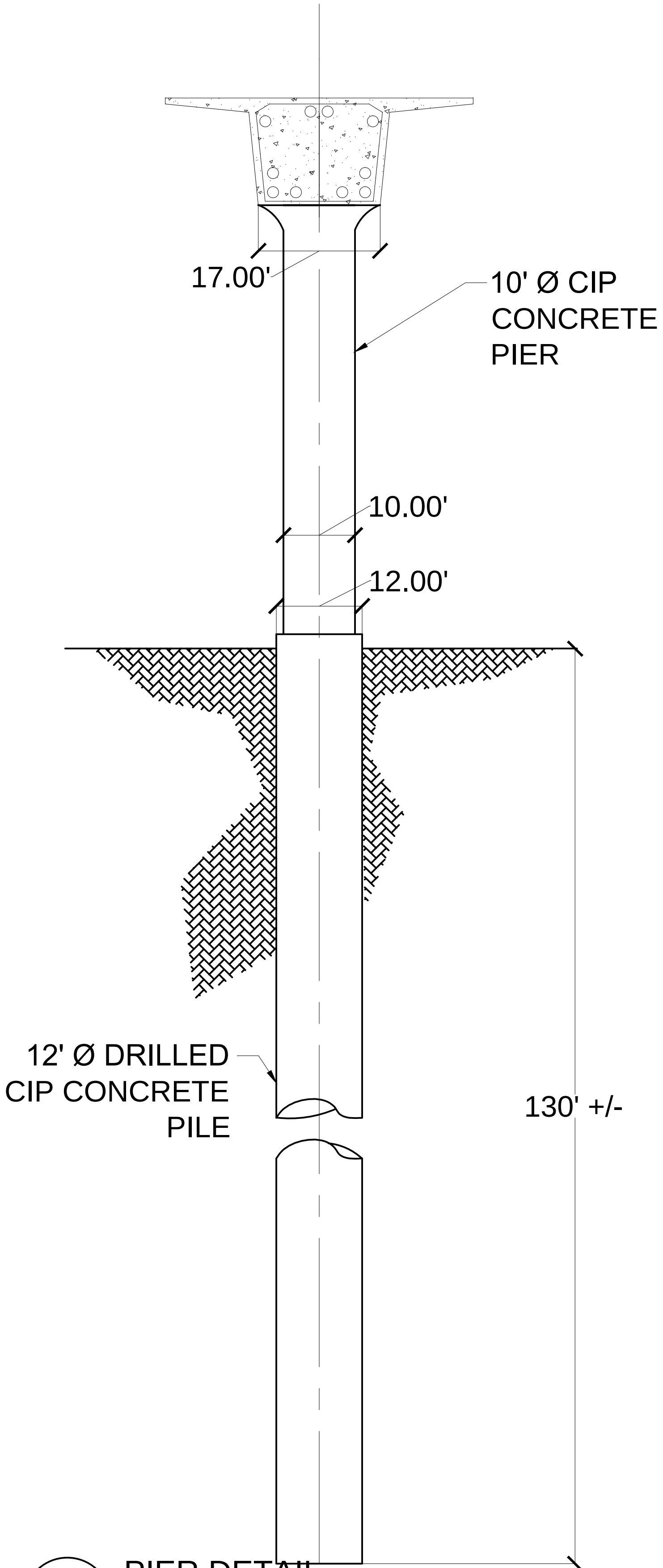
ANCHORAGE
CE-438
SENIOR DESIGN

SITE PLAN

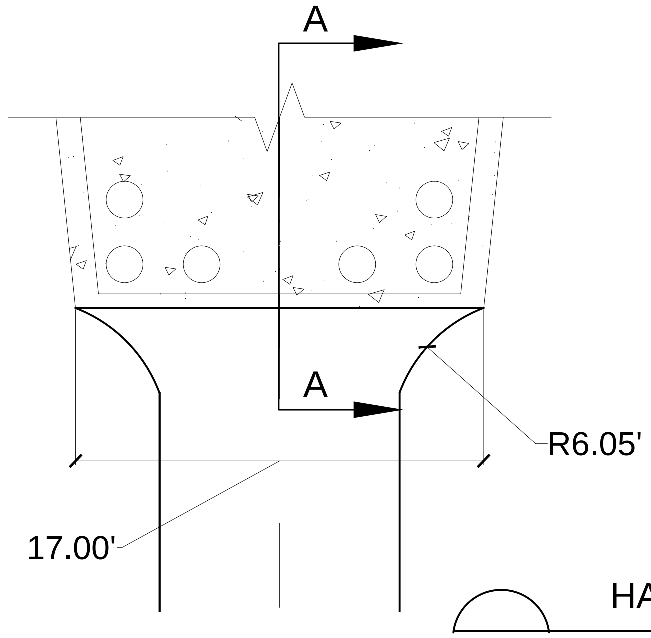
PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**
SHEET NUMBER	TOTAL SHEETS
2	**4**



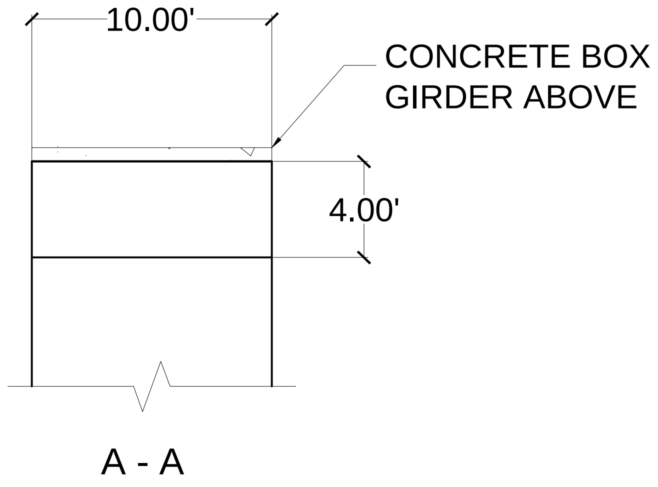
PIER DETAIL

NTS



HAMMER HEAD DETAIL

NTS



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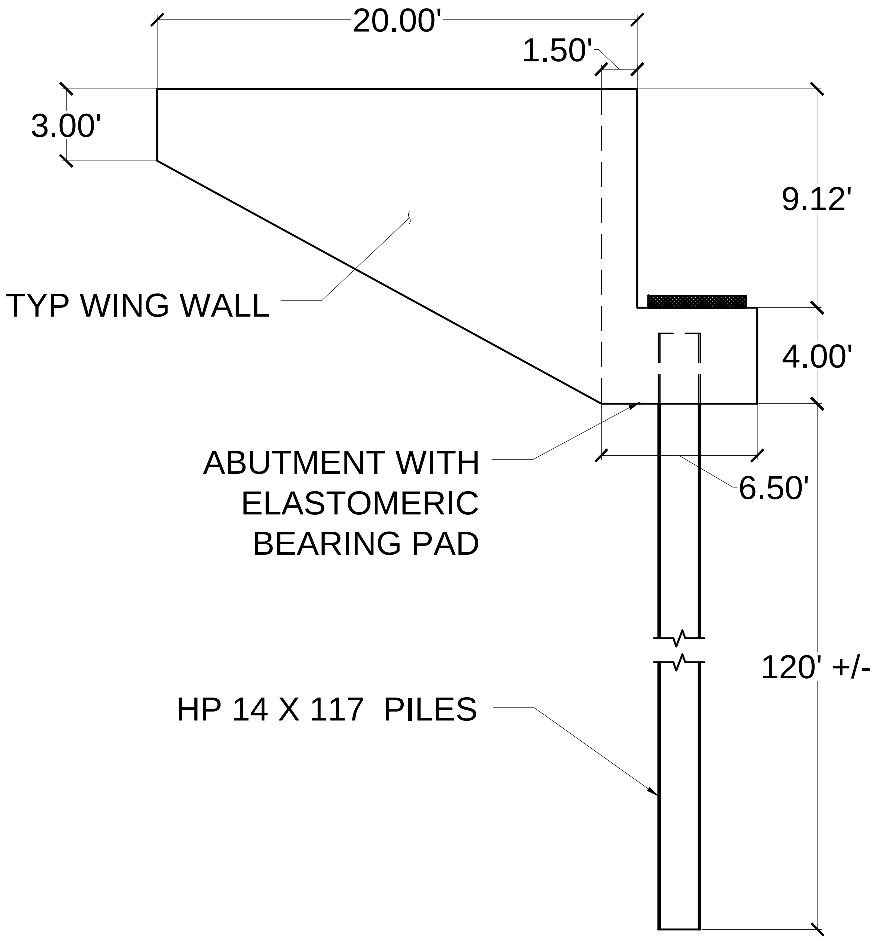
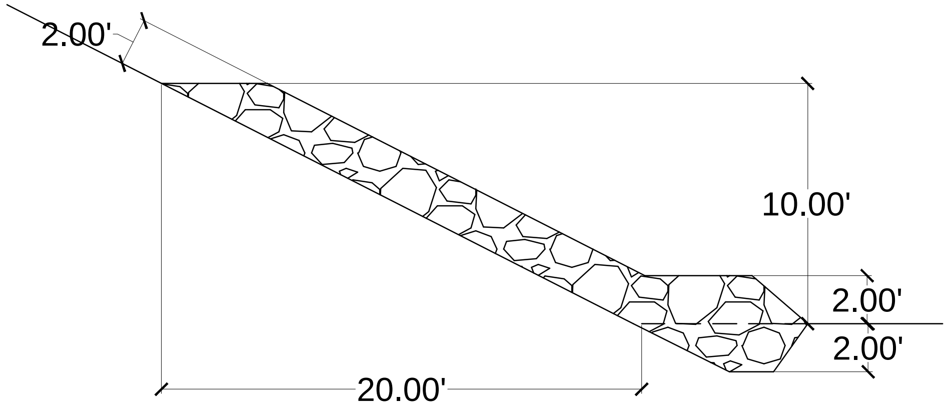


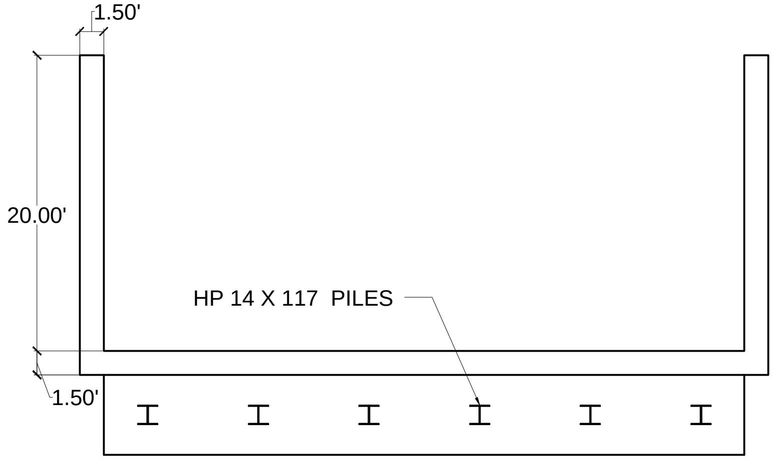
TYPICAL ABUTMENT PROFILE

NTS



RIPRAP DETAIL

NTS



TYPICAL ABUTMENT PLAN

NTS

FILE Path:

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CE-438
SENIOR DESIGN

SUBSTRUCTURE DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
3	4

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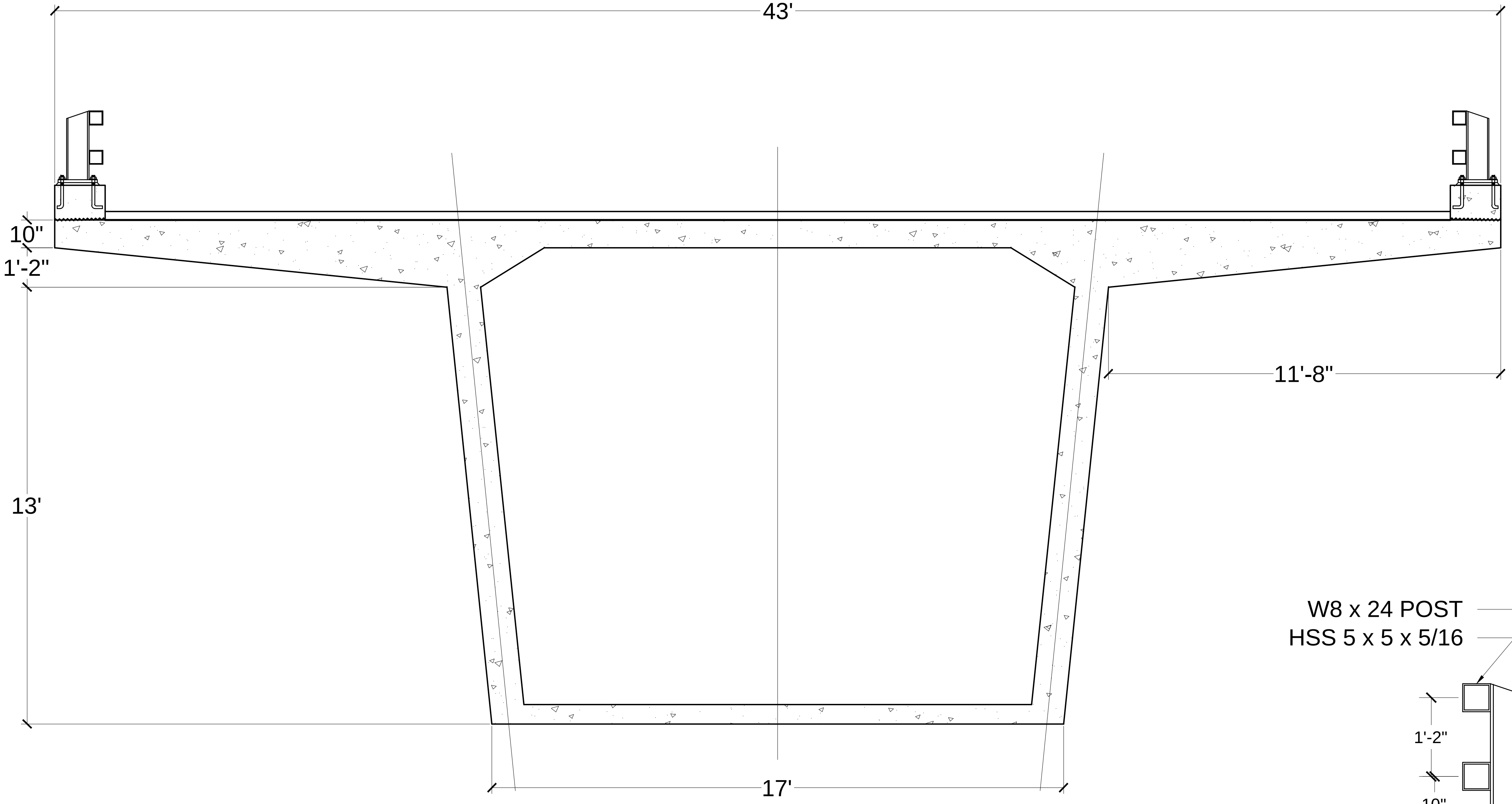
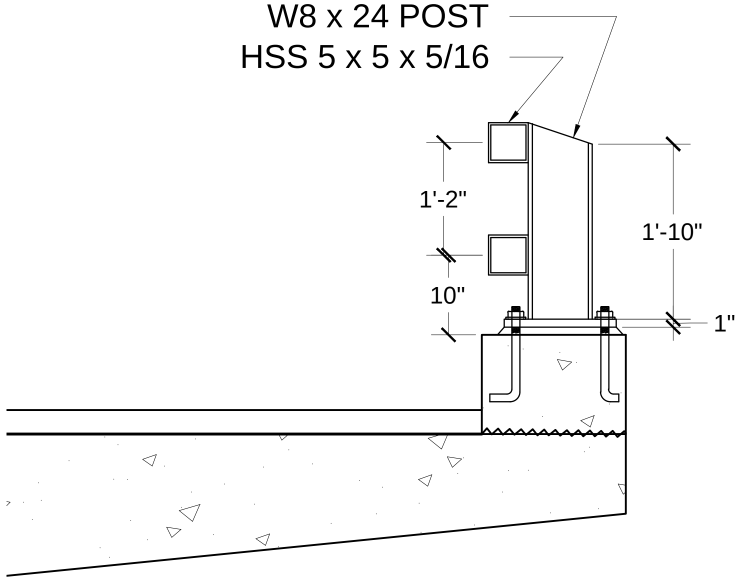


SUPERSTRUCTURE CROSS SECTION

NTS



BRIDGE RAIL DETAIL

NTS

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ADDENDUM NUMBER

ATTACHMENT NUMBER

RECORD OF REVISIONS

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UNIVERSITY OF ALASKA
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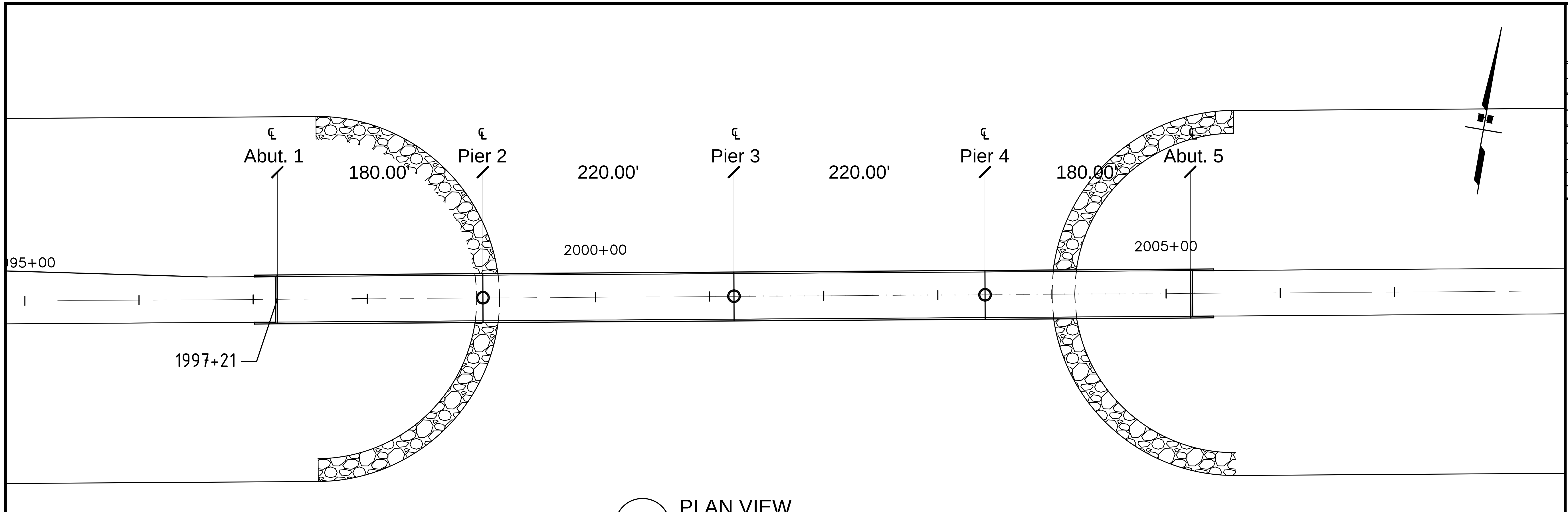
CE-438
SENIOR DESIGN

CONCRETE BOX GIRDER DETAILS

PROJECT DESIGNATION

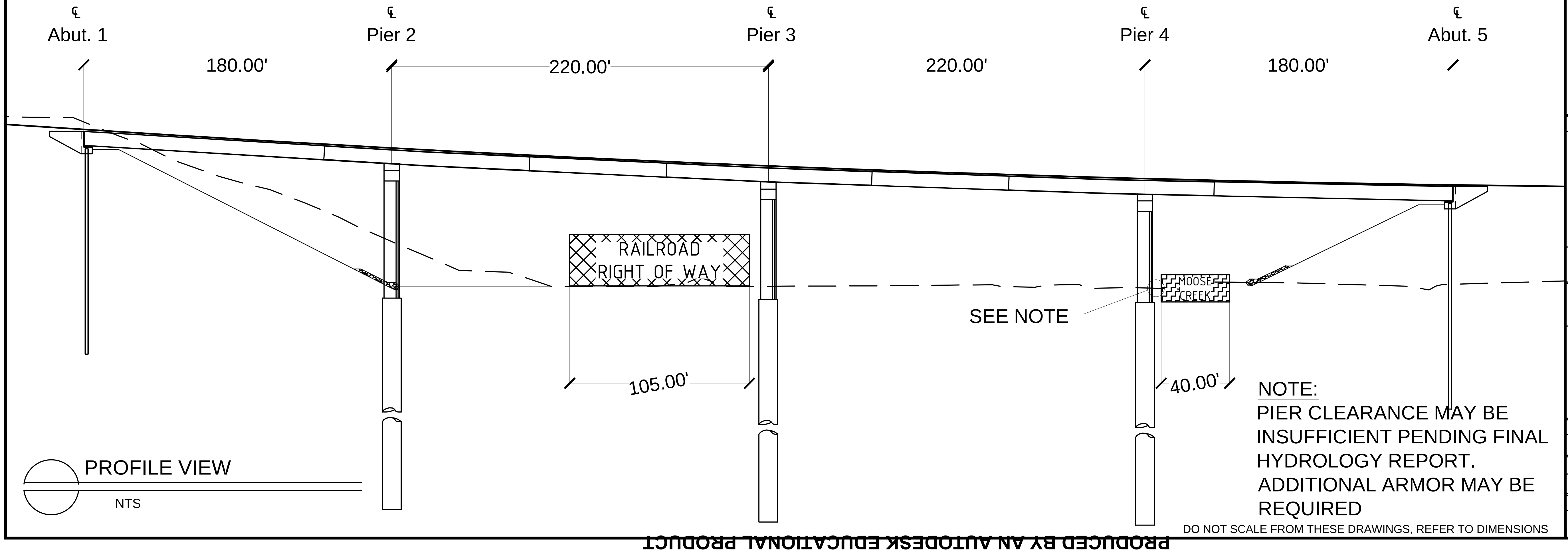
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**
SHEET NUMBER	TOTAL SHEETS
4	**4**



PLAN VIEW

NTS



PROFILE VIEW

NTS

NOTE:
PIER CLEARANCE MAY BE INSUFFICIENT PENDING FINAL HYDROLOGY REPORT.
ADDITIONAL ARMOR MAY BE REQUIRED

DO NOT SCALE FROM THESE DRAWINGS, REFER TO DIMENSIONS

FILE Path:

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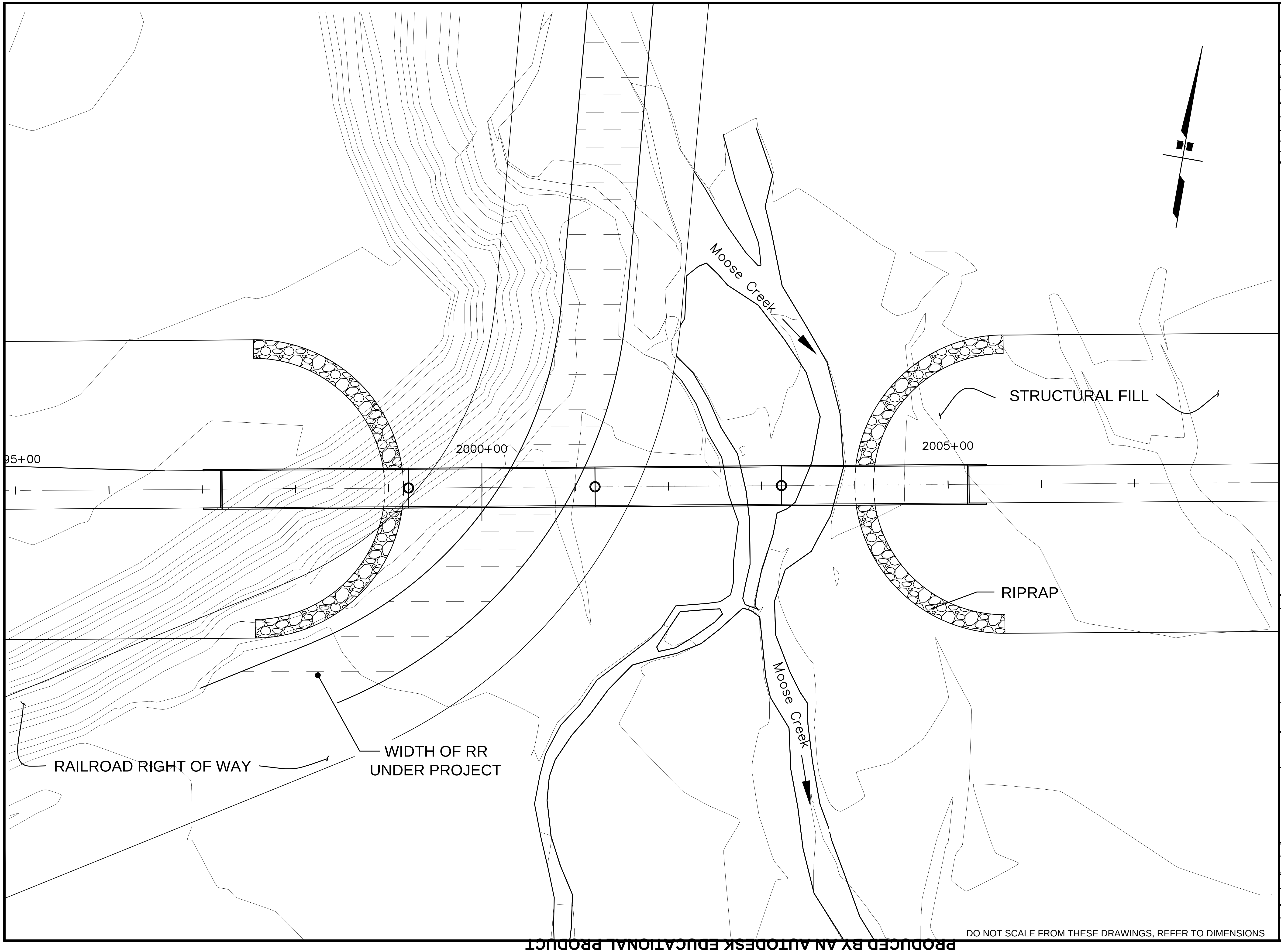
UNIVERSITY OF ALASKA
ANCHORAGE
CE-438
SENIOR DESIGN

GENERAL LAYOUT
STEEL BOX GIRDER

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**
SHEET NUMBER	TOTAL SHEETS
1	4



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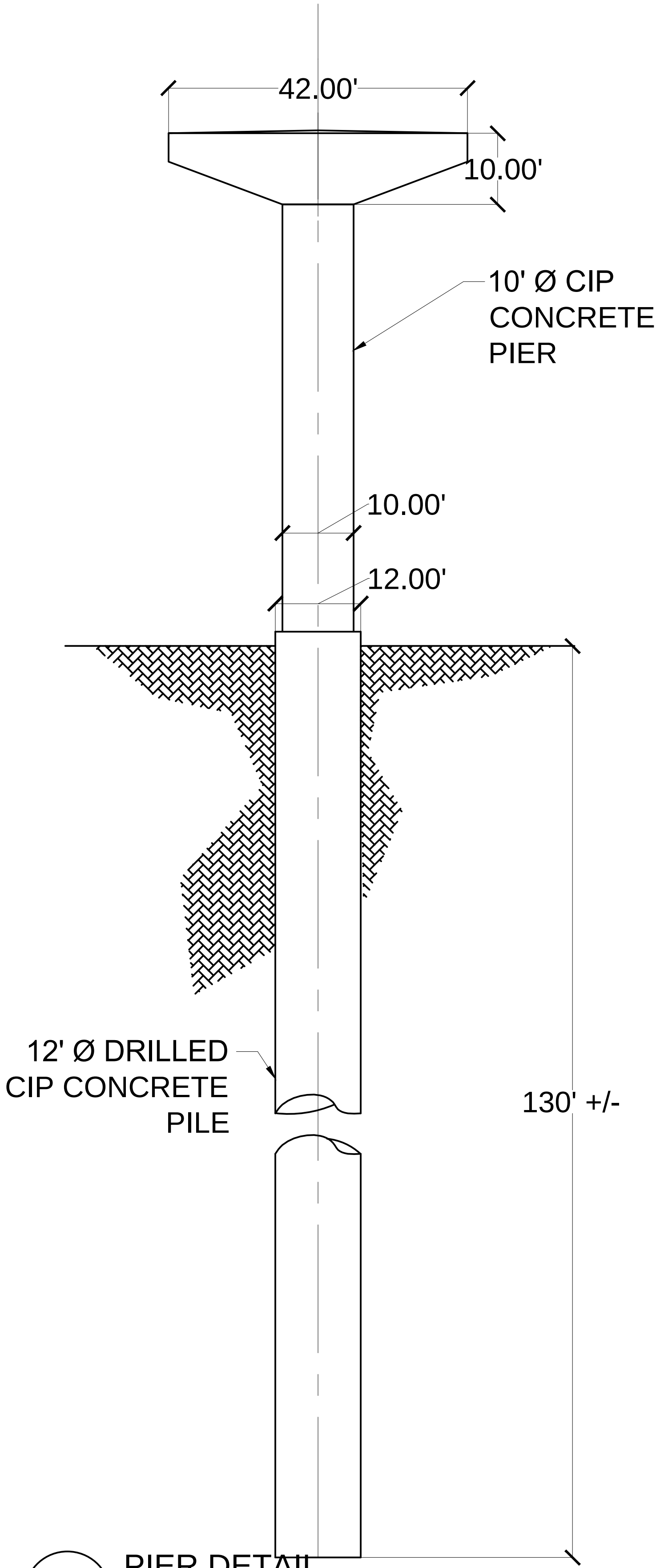
SENIOR DESIGN

SITE PLAN
STEEL BOS GIRDER

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

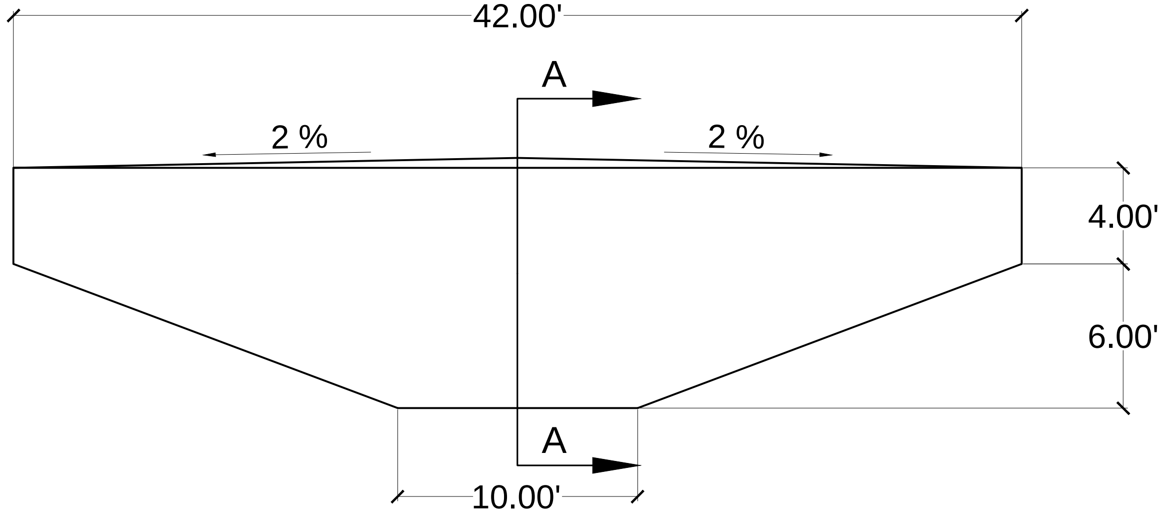
STATE	YEAR
ALASKA	**2013**

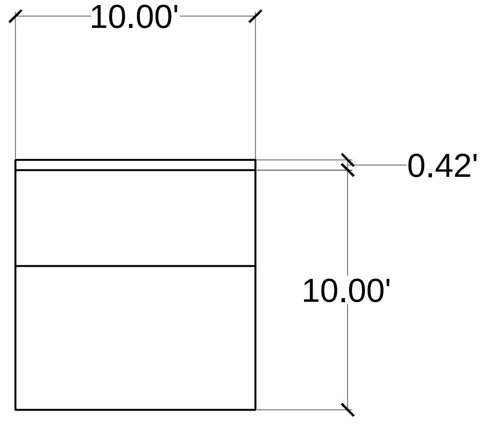
SHEET NUMBER	TOTAL SHEETS
2	4



PIER DETAIL

NTS

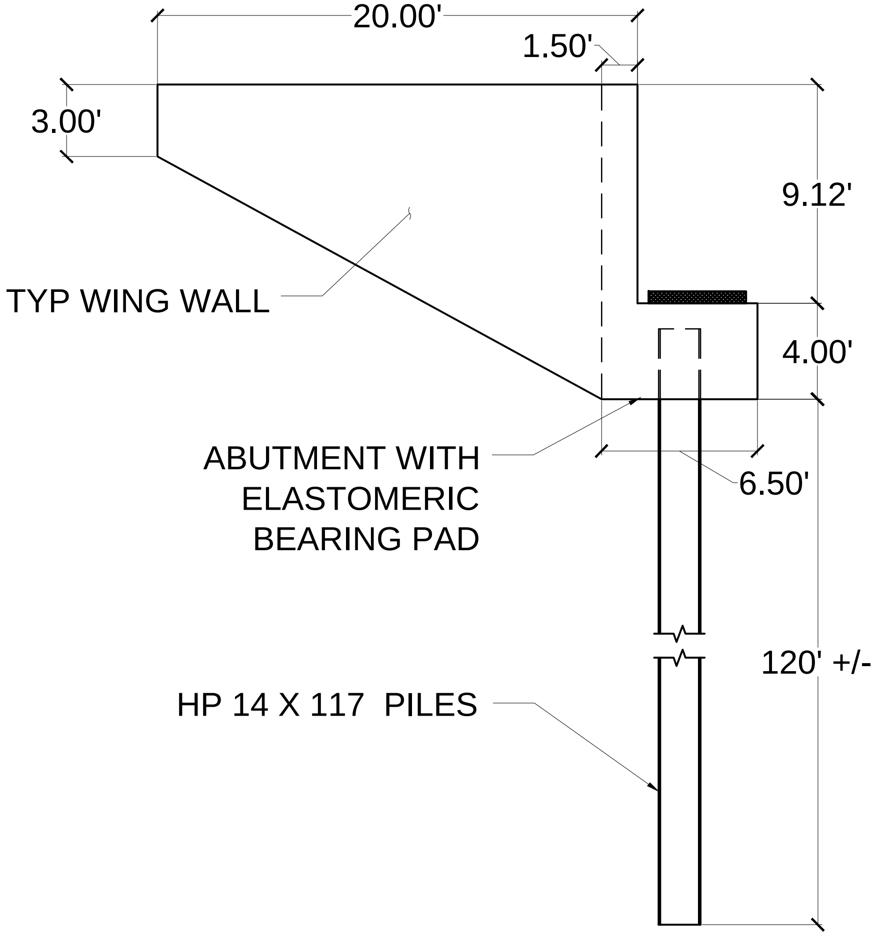




A - A

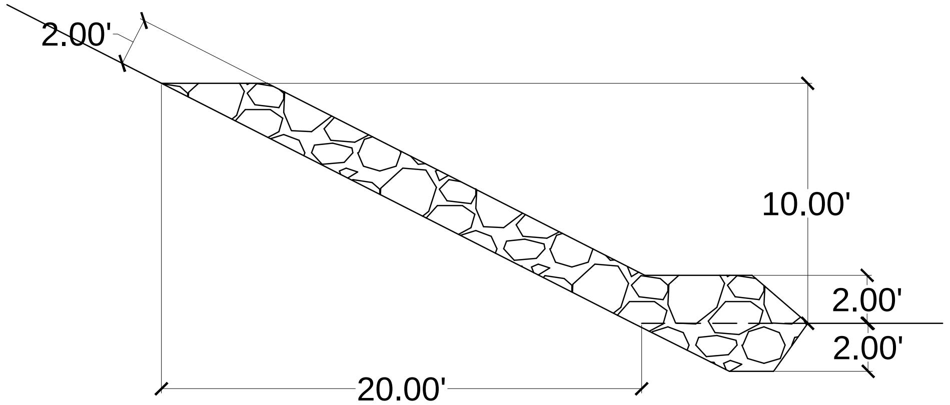
HAMMER HEAD DETAIL

NTS



TYPICAL ABUTMENT PROFILE

NTS



RIPRAP DETAIL

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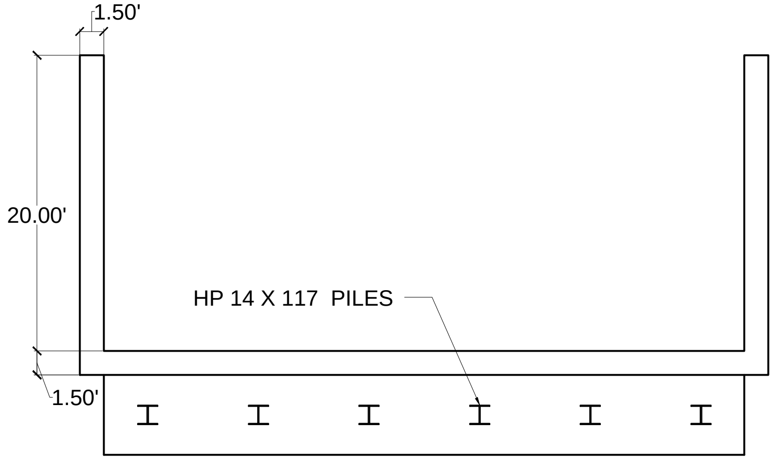

TYPICAL ABUTMENT PLAN

NTS

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Hunter Seibold
Newel Pangulayan

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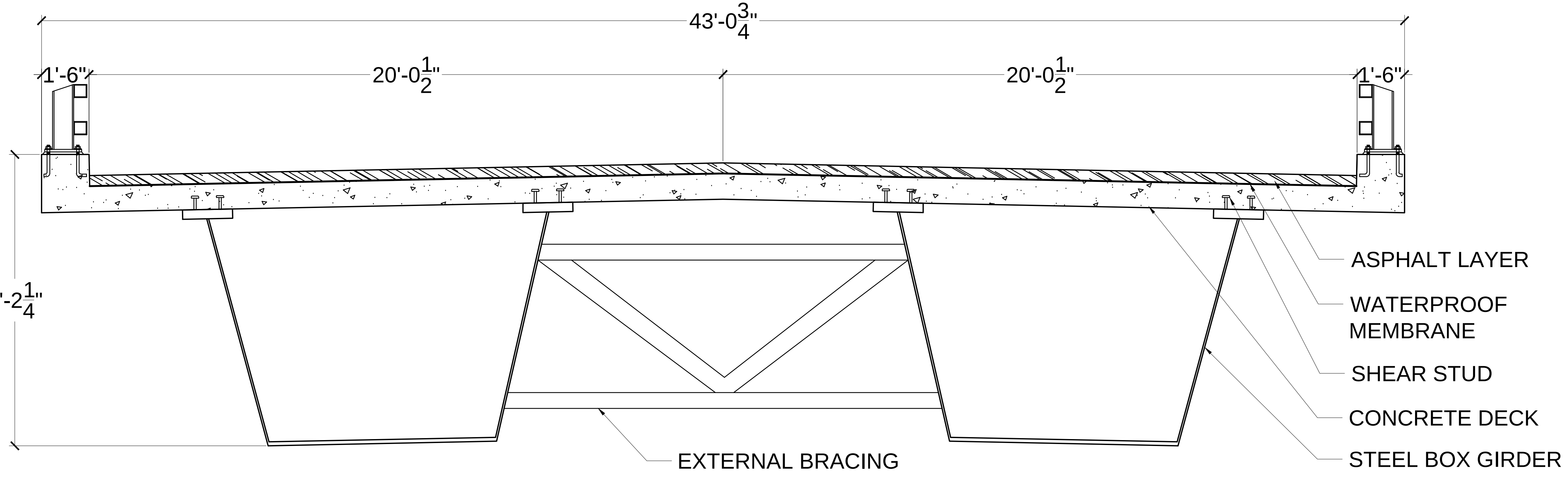
UNIVERSITY OF ALASKA
ANCHORAGE
CE-438
SENIOR DESIGN

SUBSTRUCTURE DETAILS

PROJECT DESIGNATION
MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
3	4



SUPERSTRUCTURE CROSS SECTION

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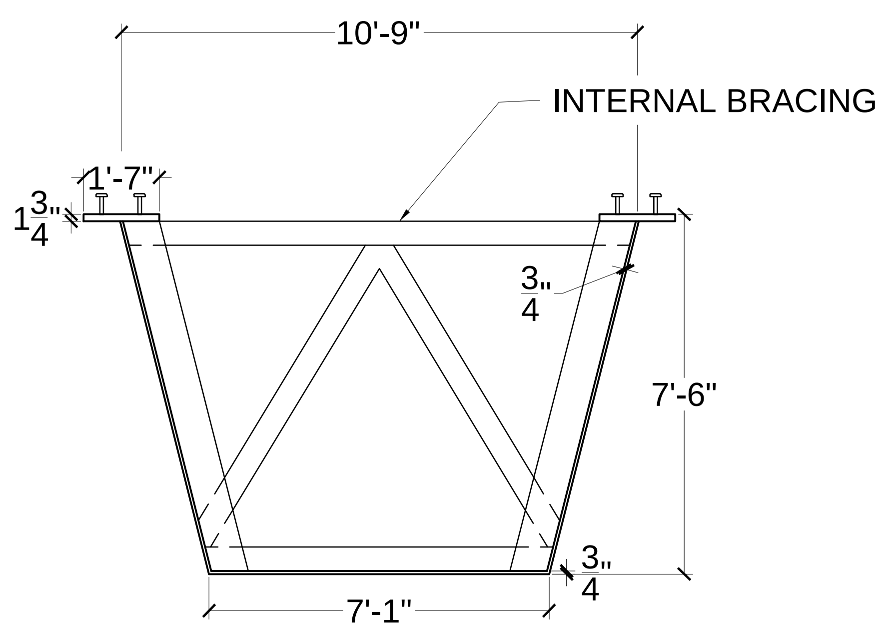

POSITIVE MOMENT SECTION

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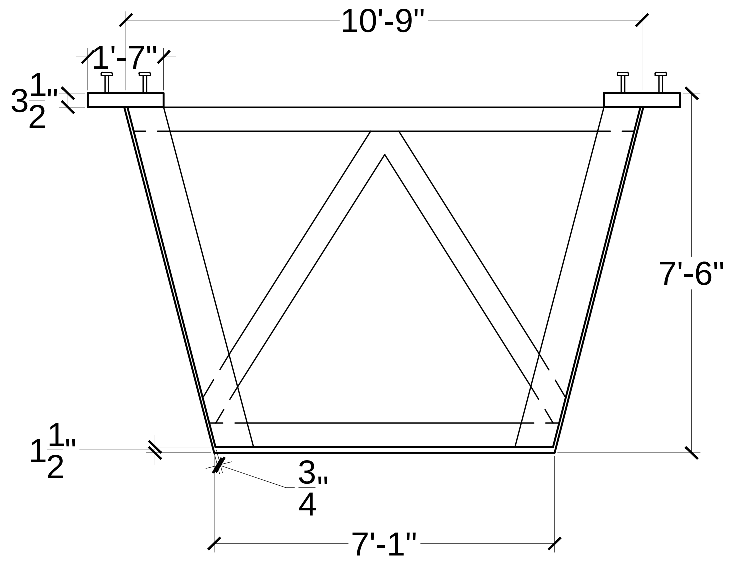

NEGATIVE MOMENT SECTION

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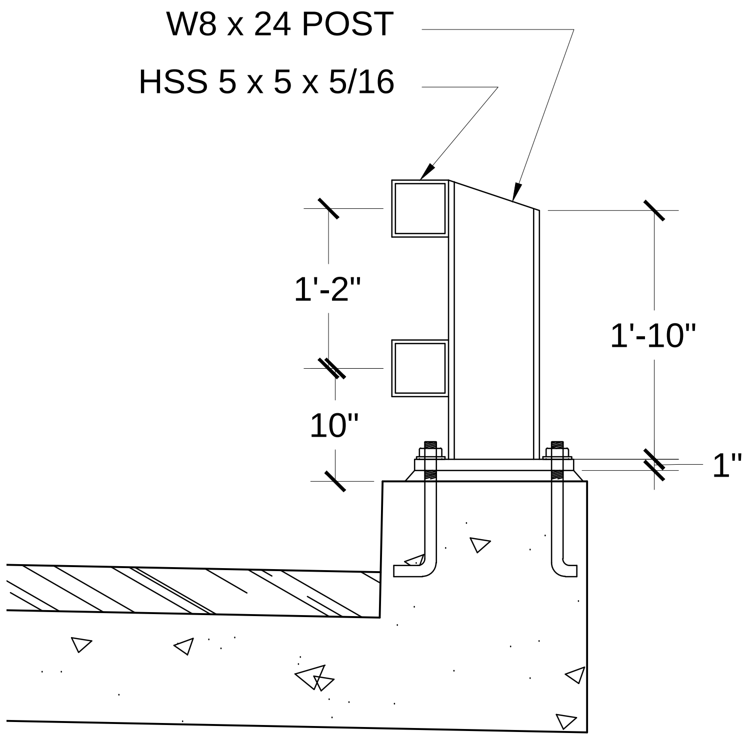

BRIDGE RAIL DETAIL

NTS

TEXT

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Liam Gibcus
Hunter Seibold
Newel Pangulayan

DRAWN BY: Mike Beauvais

UNIVERSITY OF ALASKA
ANCHORAGE
CE-438
SENIOR DESIGN

STEEL BOX GIRDER DETAILS

PROJECT DESIGNATION

MOOSE CREEK BRIDGE

STATE	YEAR
ALASKA	**2013**

SHEET NUMBER	TOTAL SHEETS
4	4

APPENDIX B: ITEMIZED COST ESTIMATES

University of Alaska Anchorage
Seawolf Engineering
Computations

Bridge No.: 2223
Calc by: NTP
For: Moose Creek Bridge - Preliminary
Date: 4/11/2013

Estimate of Quantities - Precast Concrete Bulb-Tee Bridge

Alignment: N/A
Bridge Length: 800 ft
Spans: 2 x 120'
4 x 140'
Deck Width: 43 ft
Superstructure: 8 Girder Decked Bulb tee
Substructure: Single Column, Drilled Shaft

Comment

ALASKA F!CTOR - Used lowest unit costs, account for familiar labor & locally available materials

Item No.	Description	Unit	Unit Cost	Quantity	Cost	Comment
205(5)	Structural Fill	CY	$ 35.00	1600	$ 56,000.00	
501(1)	Class A Concrete	CY	$ 1,400.00	1960	$ 2,744,000.00	
501(X)	Class DS Concrete	CY	$ 1,500.00	2950	$ 4,425,000.00	
501(7)	Precast Concrete Members (Girders)	EA	$ 94,000.00	48	$ 4,512,000.00	
503(1)	Reinforcing Steel	LB	$ 1.50	318000	$ 477,000.00	*Used 4% of Class A Conc.*
503(2)	Epoxy-Coated Reinforcing Steel	LB	$ 2.25	159000	$ 357,750.00	*Used 2% of Class A Conc.*
505(5)	Furnish Structural Steel Piles	LF	$ 75.00	1600	$ 120,000.00	*HP 14X117 under the abutments, one for each girder*
505(6)	Drive Structural Steel Piles	EA	$ 5,000.00	16	$ 80,000.00	
507(1)	Steel Bridge Railing	LF	$ 175.00	1600	$ 280,000.00	*Two-Tube railing*
515(1)	Drilled Shaft	LS	$ 1,000,000.00	1	$ 1,000,000.00	*Based on no. of piers*
606(12)	Bridge Rail Connection	EA	$ 2,500.00	4	$ 10,000.00	
611(1)	Riprap, Class II	CY	$ 50.00	1000	$ 50,000.00	
515(2)	Unclassified Shaft Excavation	CY	$ 25.00	2950	$ 73,750.00	
515(4)	Shaft Casing	LB	$ 0.75	955000	$ 716,250.00	
515(5)	Shaft Instrumentation and Data Collection	LS	$ 100,000.00	1	$ 100,000.00	
	Subtotal				$ 15,001,750.00	
640(1)	Demobilization & Mobilization	LS	11%		$ 1,666,861.11	
	Subtotal				$ 16,668,611.11	
	Contingency	LS	30%		$ 5,000,583.33	
	Subtotal				$ 21,669,194.44	
	Construction Engineering	LS	15%		$ 3,250,379.17	
	Subtotal				$ 24,919,573.61	
	ICAP	LS	4.75%		$ 1,183,679.75	
	Total Cost				$ 24,919,573.61	$ 724.41 /ft

University of Alaska Anchorage
Seawolf Engineering
Computations

Bridge No.: 2223
Calc by: NTP
For: Moose Creek Bridge - Preliminary
Date: 4/11/2013

Estimate of Quantities - Steel Plate I-Girder Bridge

Alignment: N/A
Bridge Length: 800 ft
Spans: 2 x 180'
2 x 220'
Deck Width: 43 ft
Superstructure: 4 I-Girder Steel Plate
Substructure: Single Column, Drilled Shaft

Comment

ALASKA FACTOR - Added 10% to max unit costs, account for special labor

Item No.	Description	Unit	Unit Cost	Quantity	Cost	Comment
205(5)	Structural Fill	CY	$ 110.00	2080	$ 228,800.00	
501(1)	Class A Concrete	CY	$ 2,640.00	2235	$ 5,900,400.00	*Includes slab*
501(X)	Class DS Concrete	CY	$ 2,750.00	1770	$ 4,867,500.00	
504(1)	Structural Steel	LB	$ 3.50	546000	$ 1,911,000.00	*Girders + Bracing Steel*
503(1)	Reinforcing Steel	LB	$ 2.50	362000	$ 905,000.00	*Used 4% of Class A Conc.*
503(2)	Epoxy-Coated Reinforcing Steel	LB	$ 3.00	453000	$ 1,359,000.00	*Used 5% of Class A Conc.*
505(5)	Furnish Structural Steel Piles	LF	$ 115.00	960	$ 110,400.00	*HP 14X117 under the abutments, one for each girder*
505(6)	Drive Structural Steel Piles	EA	$ 8,250.00	8	$ 66,000.00	
507(2)	Steel Bridge Railing	LF	$ 275.00	1600	$ 440,000.00	*Two-Tube railing*
515(1)	Drilled Shaft	LS	$ 660,000.00	1	$ 660,000.00	*Based on no. of piers*
606(12)	Bridge Rail Connection	EA	$ 3,850.00	4	$ 15,400.00	
611(1)	Riprap, Class II	CY	$ 110.00	1000	$ 110,000.00	
515(2)	Unclassified Shaft Excavation	CY	$ 110.00	1770	$ 194,700.00	
515(4)	Shaft Casing	LB	$ 1.50	573000	$ 859,500.00	
515(5)	Shaft Instrumentation and Data Collection	LS	$ 110,000.00	1	$ 110,000.00	
	Subtotal				$ 17,737,700.00	
640(1)	Demobilization & Mobilization	LS	11%		$ 1,970,855.56	
	Subtotal				$ 19,708,555.56	
	Contingency	LS	30%		$ 5,912,566.67	
	Subtotal				$ 25,621,122.22	
	Construction Engineering	LS	15%		$ 3,843,168.33	
	Subtotal				$ 29,464,290.56	
	ICAP	LS	4.75%		$ 1,399,553.80	
	Total Cost				$ 29,464,290.56	$ 856.52 /ft

University of Alaska Anchorage
Seawolf Engineering
Computations

Bridge No.: 2223
Calc by: NTP
Date: 4/11/2013

For: Moose Creek Bridge - Preliminary

Estimate of Quantities - Trapezoidal Steel Box Girder Bridge

Alignment: N/A
Bridge Length: 800 ft
Spans: 2 x 260'
1 x 280'
Deck Width: 43 ft
Superstructure: 2 Girder Trapezoidal Steel Box
Substructure: Single Column, Drilled Shaft

Comment

ALASKA FACTOR - Added 25% to max unit costs, account for special labor, fabrication, shipping, etc.

Item No.	Description	Unit	Unit Cost	Quantity	Cost	Comment
205(5)	Structural Fill	CY	$ 125.00	2080	$ 260,000.00	
501(1)	Class A Concrete	CY	$ 3,000.00	2235	$ 6,705,000.00	*Includes slab*
501(X)	Class DS Concrete	CY	$ 3,125.00	1770	$ 5,531,250.00	
504(1)	Structural Steel	LB	$ 4.00	593000	$ 2,372,000.00	*Girders + Bracing Steel*
503(1)	Reinforcing Steel	LB	$ 2.75	362000	$ 995,500.00	*Used 4% of Class A Conc.*
503(2)	Epoxy-Coated Reinforcing Steel	LB	$ 3.50	453000	$ 1,585,500.00	*Used 5% of Class A Conc.*
505(5)	Furnish Structural Steel Piles	LF	$ 130.00	1440	$ 187,200.00	*6 HP 14X117 under the abutments*
505(6)	Drive Structural Steel Piles	EA	$ 9,375.00	12	$ 112,500.00	
507(2)	Steel Bridge Railing	LF	$ 315.00	1600	$ 504,000.00	*Two-Tube railing*
515(1)	Drilled Shaft	LS	$ 750,000.00	1	$ 750,000.00	*Based on no. of piers*
606(12)	Bridge Rail Connection	EA	$ 4,375.00	4	$ 17,500.00	
611(1)	Riprap, Class II	CY	$ 125.00	1000	$ 125,000.00	
515(2)	Unclassified Shaft Excavation	CY	$ 125.00	1770	$ 221,250.00	
515(4)	Shaft Casing	LB	$ 1.75	573000	$ 1,002,750.00	
515(5)	Shaft Instrumentation and Data Collection	LS	$ 125,000.00	1	$ 125,000.00	
	Subtotal				$ 20,494,450.00	
640(1)	Demobilization & Mobilization	LS	11%		$ 2,277,161.11	
	Subtotal				$ 22,771,611.11	
	Contingency	LS	30%		$ 6,831,483.33	
	Subtotal				$ 29,603,094.44	
	Construction Engineering	LS	15%		$ 4,440,464.17	
	Subtotal				$ 34,043,558.61	
	ICAP	LS	4.75%		$ 1,617,069.03	
	Total Cost				$ 34,043,558.61	$ 989.64 /ft

University of Alaska Anchorage
Seawolf Engineering
Computations

Bridge No.: 2223
Calc by: NTP
Date: 4/11/2013

For: Moose Creek Bridge - Preliminary

Estimate of Quantities - CIP Concrete Box Bridge

Alignment: N/A
Spans: 4 x 200'
Bridge Length: 800 ft
Deck Width: 43 ft

Superstructure: Post Tensioned Segmental CIP Concrete Box Girder
Substructure: Single Column, Drilled Shaft

Comment

ALASKA FACTOR - Added 50% to max unit costs, account for special labor, fabrication, shipping, forms, etc.

Item No.	Description	Unit	Unit Cost	Quantity	Cost	Comment
205(5)	Structural Fill	CY	$ 150.00	2080	$ 312,000.00	
501(1)	Class A Concrete	CY	$ 3,600.00	3725	$ 13,410,000.00	*Includes superstr & 8% conc. for deck*
501(X)	Class DS Concrete	CY	$ 3,750.00	1180	$ 4,425,000.00	
503(1)	Reinforcing Steel	LB	$ 3.50	603000	$ 2,110,500.00	*Used 4% of Class A Conc.*
503(2)	Epoxy-Coated Reinforcing Steel	LB	$ 4.25	754000	$ 3,204,500.00	*Used 5% of Class A Conc.*
505(5)	Furnish Structural Steel Piles	LF	$ 160.00	1200	$ 192,000.00	*6 HP 14X117 under the abutments*
505(6)	Drive Structural Steel Piles	EA	$ 11,250.00	12	$ 135,000.00	
507(2)	Steel Bridge Railing	LF	$ 375.00	1600	$ 600,000.00	*Two-Tube railing*
515(1)	Drilled Shaft	LS	$ 600,000.00	1	$ 600,000.00	*Based on no. of piers*
606(12)	Bridge Rail Connection	EA	$ 5,250.00	4	$ 21,000.00	
611(1)	Riprap, Class II	CY	$ 150.00	1000	$ 150,000.00	
502(X)	Post-Tensioning	CY	$ 30.00	340	$ 10,200.00	*Used total amount of reinforcing steel*
515(2)	Unclassified Shaft Excavation	CY	$ 150.00	1180	$ 177,000.00	
515(4)	Shaft Casing	LB	$ 2.00	382000	$ 764,000.00	
515(5)	Shaft Instrumentation and Data Collection	LS	$ 150,000.00	1	$ 150,000.00	
	Subtotal				$ 26,261,200.00	
640(1)	Demobilization & Mobilization	LS	11%		$ 2,917,911.11	
	Subtotal				$ 29,179,111.11	
	Contingency	LS	30%		$ 8,753,733.33	
	Subtotal				$ 37,932,844.44	
	Construction Engineering	LS	15%		$ 5,689,926.67	
	Subtotal				$ 43,622,771.11	
	ICAP	LS	4.75%		$ 2,072,081.63	
	Total Cost				$ 43,622,771.11	$ 1,268.10 /ft

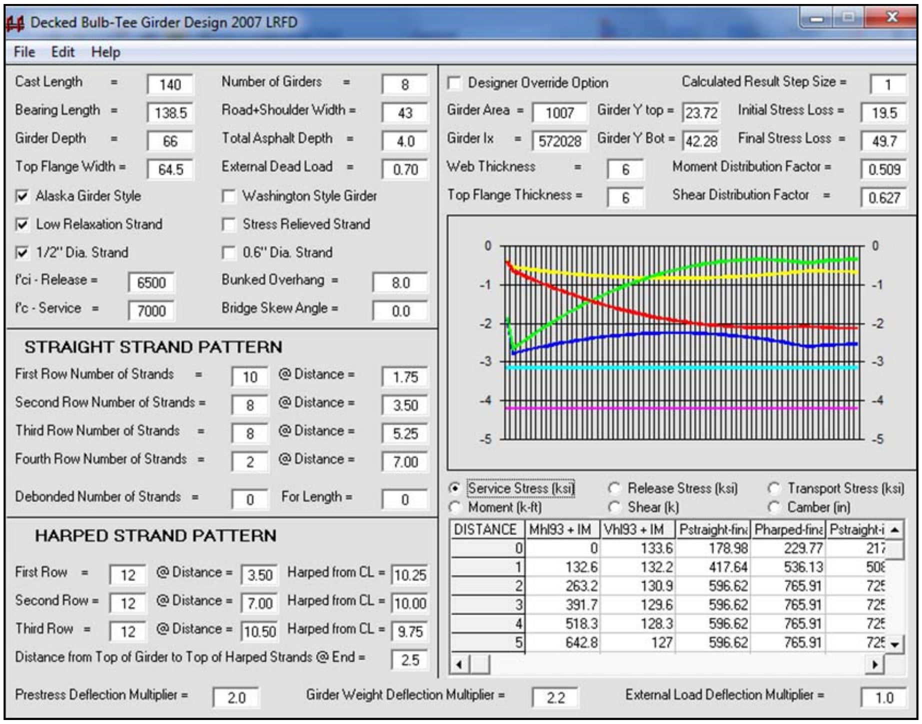
APPENDIX C: SUPERSTRUCTURE OPTIMIZATION ANALYSIS

Pre-stressed Concrete Decked Bulb Tee Optimization

Pre-stressed concrete decked bulb tees are only designed for simple spans. As discussed in the report, these types of girders cannot handle the internal forces that develop with a continuous span. Girder design was primarily done using a program that had been specifically developed for these types of girders.

It was known before any analysis was done that precast bulb tee girders can be made up to 148 feet long. Knowing that an 800-foot bridge must be built to span the railroad right-of-way and Moose Creek, four 140-foot spans and two 120-foot spans were designed. The two 120-foot spans are at both ends of the bridge. These span lengths were chosen for three reasons: (1) to fit in with the geometry of the site, (2) to maximize the span length while still satisfying criterion 1 (this is why 140-foot spans were chosen as opposed to 148-foot), and (3) for symmetrical, aesthetic reasons.

Figure C - 1 is a screenshot from the Bulb Tee program. The values that have been inputted in this analysis are for a 140-foot girder that has eight girders per span that are each 66 inches deep. The strand pattern which is the maximum number of strands possible in these girders is the same for both the 140-foot and 120-foot girders.



Cast Length = 140
Bearing Length = 138.5
Girder Depth = 66
Top Flange Width = 64.5
☑ Alaska Girder Style
☑ Low Relaxation Strand
☑ 1/2" Dia. Strand
f'ci - Release = 6500
f'c - Service = 7000

Number of Girders = 8
Road+Shoulder Width = 43
Total Asphalt Depth = 4.0
External Dead Load = 0.70
☐ Washington Style Girder
☐ Stress Relieved Strand
☐ 0.6" Dia. Strand
Bunked Overhang = 8.0
Bridge Skew Angle = 0.0

☐ Designer Override Option
Calculated Result Step Size = 1
Girder Area = 1007
Girder Y top = 23.72
Initial Stress Loss = 19.5
Girder Ix = 572028
Girder Y Bot = 42.28
Final Stress Loss = 49.7
Web Thickness = 6
Moment Distribution Factor = 0.509
Top Flange Thickness = 6
Shear Distribution Factor = 0.627

STRAIGHT STRAND PATTERN

First Row Number of Strands = 10 @ Distance = 1.75
Second Row Number of Strands = 8 @ Distance = 3.50
Third Row Number of Strands = 8 @ Distance = 5.25
Fourth Row Number of Strands = 2 @ Distance = 7.00
Debonded Number of Strands = 0 For Length = 0

HARPED STRAND PATTERN

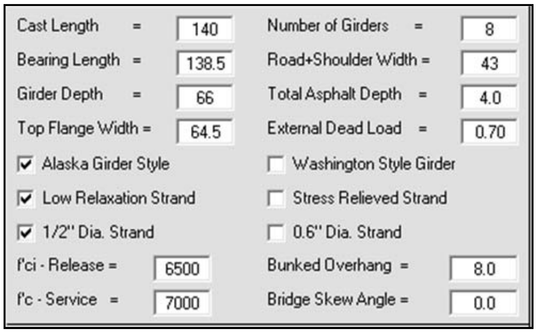
First Row = 12 @ Distance = 3.50 Harped from CL = 10.25
Second Row = 12 @ Distance = 7.00 Harped from CL = 10.00
Third Row = 12 @ Distance = 10.50 Harped from CL = 9.75
Distance from Top of Girder to Top of Harped Strands @ End = 2.5

● Service Stress (ksi) ○ Release Stress (ksi) ○ Transport Stress (ksi)
○ Moment (k-ft) ○ Shear (k) ○ Camber (in)

DISTANCE	Mhl93 + IM	Vhl93 + IM	Pstraight-fin	Pharped-fin	Pstraight-i
0	0	133.6	178.98	229.77	217
1	132.6	132.2	417.64	536.13	508
2	263.2	130.9	596.62	765.91	725
3	391.7	129.6	596.62	765.91	725
4	518.3	128.3	596.62	765.91	725
5	642.8	127	596.62	765.91	725

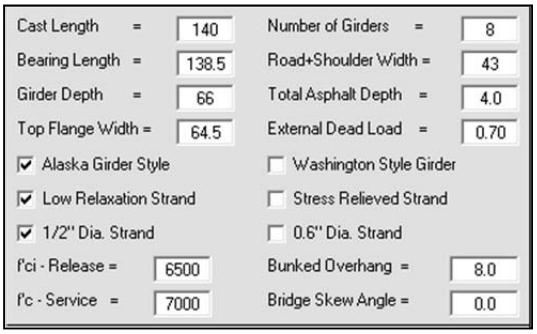
Prestress Deflection Multiplier = 2.0
Girder Weight Deflection Multiplier = 2.2
External Load Deflection Multiplier = 1.0

Figure C - 1: Bulb Tee design program screenshot



Cast Length = 140
Bearing Length = 138.5
Girder Depth = 66
Top Flange Width = 64.5
☑ Alaska Girder Style
☑ Low Relaxation Strand
☑ 1/2" Dia. Strand
f'ci - Release = 6500
f'c - Service = 7000
Number of Girders = 8
Road+Shoulder Width = 43
Total Asphalt Depth = 4.0
External Dead Load = 0.70
☐ Washington Style Girder
☐ Stress Relieved Strand
☐ 0.6" Dia. Strand
Bunked Overhang = 8.0
Bridge Skew Angle = 0.0

Figure C - 2 shows the parts of the program that were primarily used for the girder design. Seven, eight, nine and ten girder options were all analyzed and their results can be seen the spreadsheet below. The more girders that were added, the more the loading is distributed amongst them, requiring narrower girders. Also, these girders didn't have to be as strong. This meant that less concrete was needed for each girder. Although this was the case, having ten girders as opposed to seven still required more concrete and steel overall. This trend can be seen in Figures C - 4 (a), (b) and (c). For this reason, it is clear that it was best to keep the number of girders per span to a minimum.



Cast Length = 140
Bearing Length = 138.5
Girder Depth = 66
Top Flange Width = 64.5
☑ Alaska Girder Style
☑ Low Relaxation Strand
☑ 1/2" Dia. Strand
f'ci - Release = 6500
f'c - Service = 7000
Number of Girders = 8
Road+Shoulder Width = 43
Total Asphalt Depth = 4.0
External Dead Load = 0.70
☐ Washington Style Girder
☐ Stress Relieved Strand
☐ 0.6" Dia. Strand
Bunked Overhang = 8.0
Bridge Skew Angle = 0.0

Figure C - 2: Data input for 140ft span, eight girder option

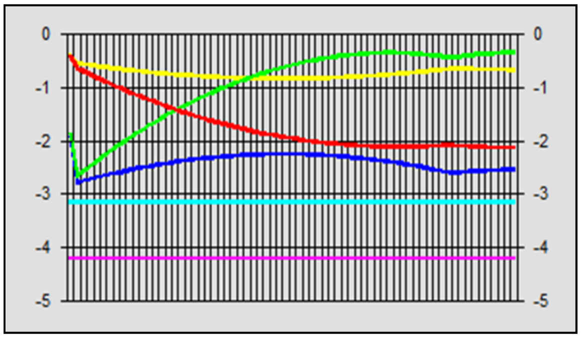


Figure C - 3: Service stress graph from Bulb Tee program

It was not possible to have any less than seven girders as the top flange becomes too large and it was not possible to have any more than ten because the girder become too narrow to fit all the steel strands in. An eight girder option ended up being chosen as the roadway required a downward gradient from the center for water runoff. If seven girders were used, the center girder would sit flat in the middle of the bridge. Choosing eight girders meant that there was a join on the centerline of the bridge making the downward gradient in each direction from the centerline possible.

The primary concerns when using this program to design bulb tee girders was to make sure that are no point would the girder enter a state on compression. Precast pre-stressed concrete girders must always be in a state of compression as concrete has no tensile properties. Figure C – 3 shows an output graph from the program. On the y-axis is the stress and the x-axis is the position alone the girder. The graph only plots from one end of the girder to center span as it is simply supported the forces are mirrored on the other side of the girder. The graph shows that the highest stresses develop at mid span and this is the value that must remain below zero.

Bulb Tee Optimization

(Data from Bulb Tee Program)

Constants	
Road + Shoulder Width (feet)	43
Total Asphalt Depth (inches)	4
Bunked Overhang (feet)	8
External Dead Load	0.7
Alaska Girder Style	Yes
Low Relaxation Strand	Yes
1/2" Dia. Strand	Yes

Input Value	🟧
Spreadsheet Derived Value	🟪
Value Computed through Program	🟩

*Number of Strands is always the maximum

Input Variables								
Number of Girders	7	8	9	10	7	8	9	10
Span Length (feet)	120	120	120	120	140	140	140	140
Bering Distance (inches)	9	9	9	9	9	9	9	9
Depth/Span	0.0458	0.0458	0.0458	0.0458	0.0393	0.0393	0.0393	0.0393
Total Width (feet)	43	43	43	43	43	43	43	43
Weight (pcf)	160	160	160	160	160	160	160	160
Bulb Tee Cost ($/lb)	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Optimization								
Concrete Strength								
Release, f'ci (psi)	7000	5000	5000	5000	6500	6500	6500	6500
Service, f'c (psi)	8000	7000	6000	6000	7500	7000	7000	7000
Resultant Variables								
Bearing Length (feet)	118.5	118.5	118.5	118.5	138.5	138.5	138.5	138.5
Girder Depth (inches)	66	66	66	66	66	66	66	66
Top Flange Width (inches)	73.71	64.50	57.33	51.60	73.71	64.50	57.33	51.60
Results								
Girder Area (in^2)	1062.2	1007	963.9	929.6	1062.2	1007	963.9	929.6
Girder Weight (lbs)	141626.7	134266.7	128520.0	123946.7	165231.1	156644.4	149940.0	144604.4

Girders per span	Weight of Girder		Price per Girder		Number of Girders		Cost ($)		Total cost (S)
	120ft	140ft	120 ft	140ft	120ft	140ft	120ft	140ft	
7	141626.7	165231.1	84,976.00	99,138.67	14	28	1,189,664.00	2,775,882.67	3,965,546.67
8	134266.7	156644.4	80,560.00	93,986.67	16	32	1,288,960.00	3,007,573.33	4,296,533.33
9	128520.0	149940.0	77,112.00	89,964.00	18	36	1,388,016.00	3,238,704.00	4,626,720.00
10	123946.7	144604.4	74,368.00	86,762.67	20	40	1,487,360.00	3,470,506.67	4,957,866.67

Table C - 1: Summary of Bulb Tee weight and costs

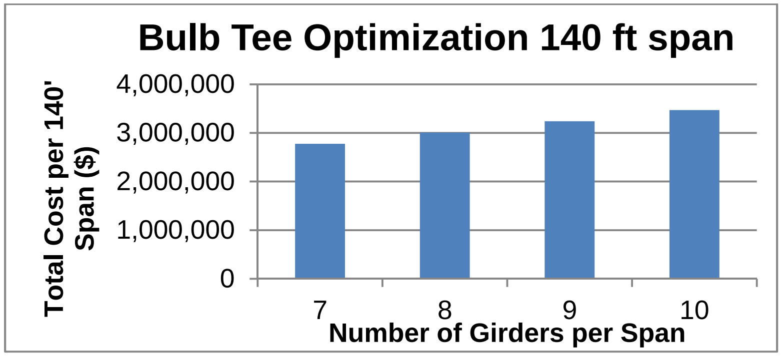


Figure C - 4 (a) Cost for 140 ft spans

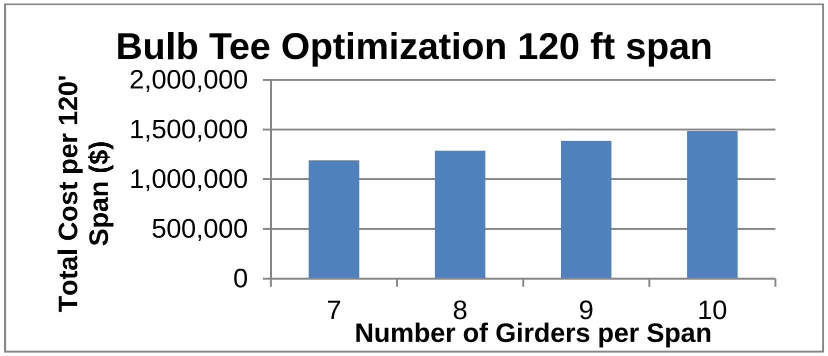


Figure C - 4 (b) Costs for 120 ft spans

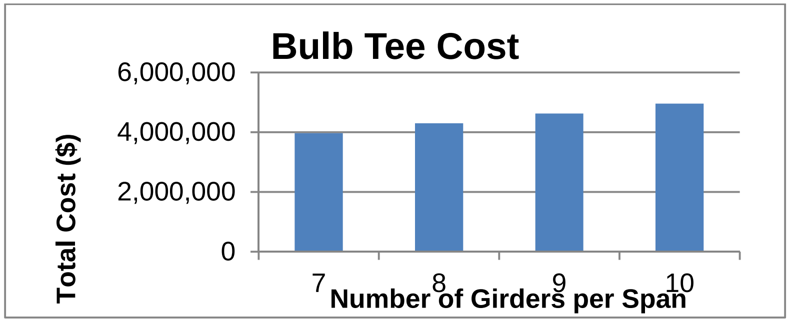


Figure C - 4 (c) Bulb Tee Cost

Steel I Girder Optimization

The steel spans were analyzed as multi-span continuous beams. Steel is relatively easy to splice on site making it economical to use the advantages of a continuous span. By designing the super structure as one continuous girder the moment diagram is shifted down reducing the maximum moment in the girder as can be seen in figures C – 5 and C - 6. This poses other design issues for which the negative moment must be considered however the extra effort in design proves cost effective in the end.

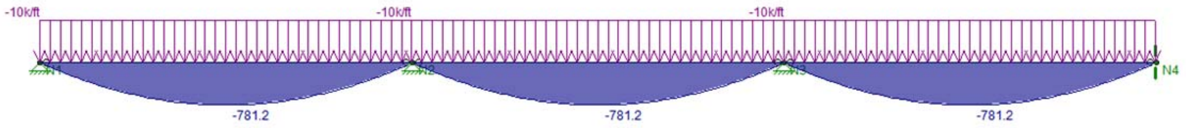


Figure C – 5 Simple Span Moment Diagram

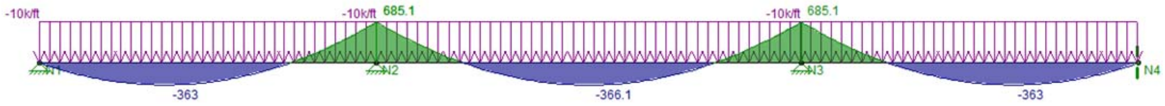
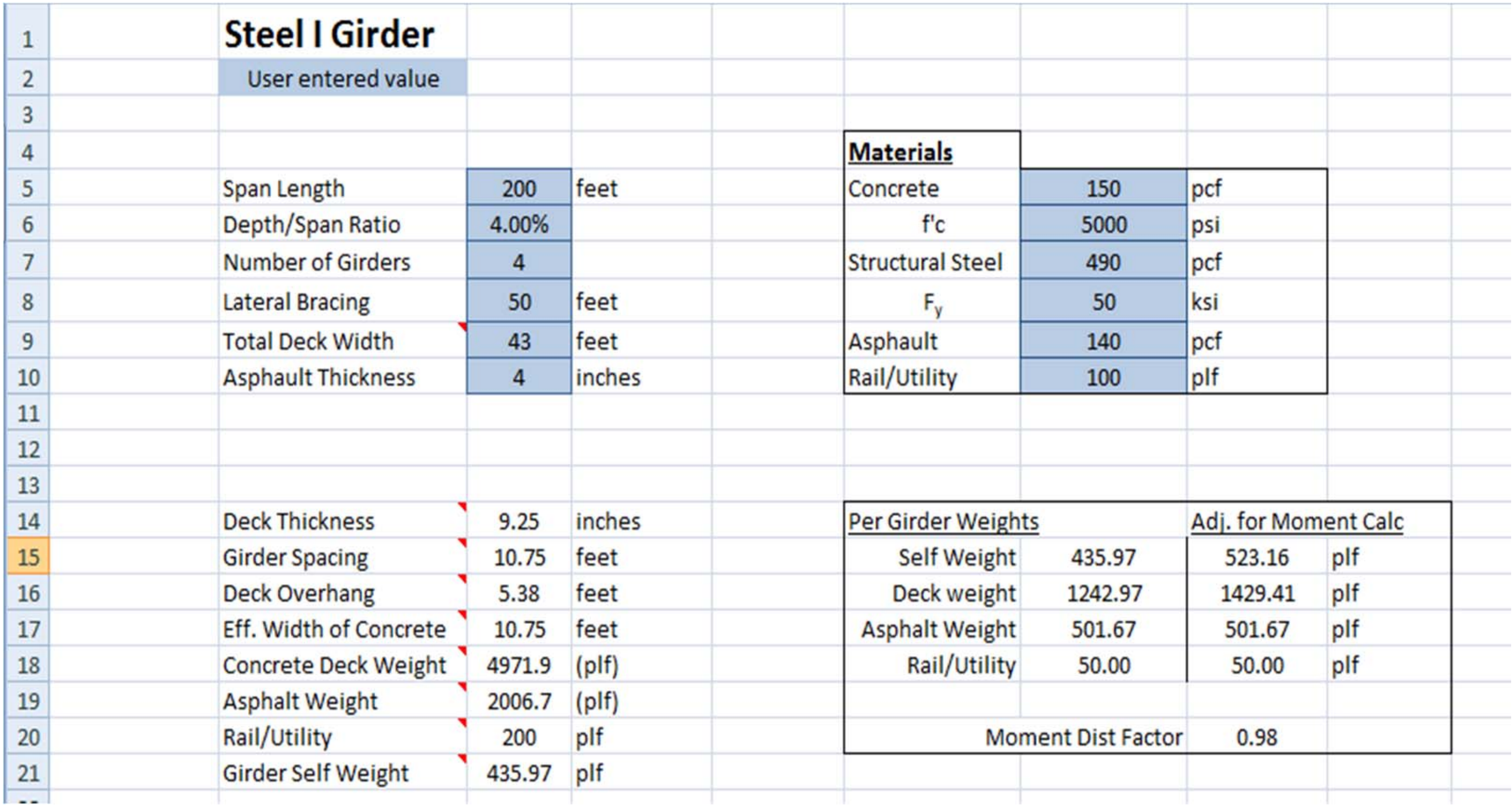


Figure C – 6 Continuous Span Moment Diagram

Figure 1 shows a 75 foot structure with three equal 25 foot spans. From the RISA output we can see a maximum positive moment of 781 k-ft. By making the span continuous as in Figure 2 and making the outer spans 70~75 % the length of the central spans the moment can be redistributed and reduced to a maximum of 685 k-ft and over a much shorter girder length. This shorter length of maximum moment is significant as composite action between the steel girder and concrete deck cannot be used in these areas of negative moment (fig. C - 6) and therefore a larger steel cross section must be used for this section of girder.

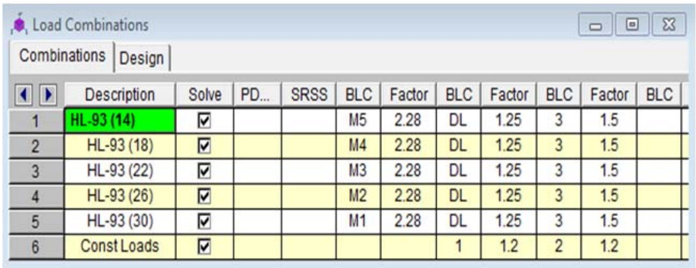
For the Moose Creek Bridge the structure length was determined by the configuration of obstructions that needed to be spanned (i.e. railroad and creek). Once a structure length was determined it was then divided into span lengths based off of research, site limitations and collaboration with experienced bridge engineers. An Excel spreadsheet was used to aid in the girder optimization and is shown in figure 8-2



1	**Steel I Girder**						
2	User entered value						
3							
4				**Materials**			
5	Span Length	200	feet	Concrete	150	pcf	
6	Depth/Span Ratio	4.00%		f'c	5000	psi	
7	Number of Girders	4		Structural Steel	490	pcf	
8	Lateral Bracing	50	feet	F_y	50	ksi	
9	Total Deck Width	43	feet	Asphault	140	pcf	
10	Asphault Thickness	4	inches	Rail/Utility	100	plf	
11							
12							
13							
14	Deck Thickness	9.25	inches	Per Girder Weights		Adj. for Moment Calc	
15	Girder Spacing	10.75	feet	Self Weight	435.97	523.16	plf
16	Deck Overhang	5.38	feet	Deck weight	1242.97	1429.41	plf
17	Eff. Width of Concrete	10.75	feet	Asphalt Weight	501.67	501.67	plf
18	Concrete Deck Weight	4971.9	(plf)	Rail/Utility	50.00	50.00	plf
19	Asphalt Weight	2006.7	(plf)				
20	Rail/Utility	200	plf		Moment Dist Factor	0.98	
21	Girder Self Weight	435.97	plf				

Figure C – 7 Calculating 'Per Girder' Weight

With the required dimensions entered into the spreadsheet the thickness of the road deck is calculated as a function of the number of girders in the cross section. With the dimensions of the superstructure components determined the weight of the structure components or dead load (DL) is calculated and displayed "Per Girder" for use with RISA computer software to determine the ultimate moments on the structure. In addition to the dead loads, an HL-93 design truck as specified by the ASSHTO LRFD Bridge Design Specifications section 3.6 was used for calculating the live loads (LL) on the structure. As part of the live load analysis a moment distribution factor is calculated by the spreadsheet as a function of the number of girders, this value was incorporated into the load factors when entered into RISA.



	Description	Solve	PD...	SRSS	BLC	Factor	BLC	Factor	BLC	Factor	BLC
1	HL-93 (14)	☑			M5	2.28	DL	1.25	3	1.5	
2	HL-93 (18)	☑			M4	2.28	DL	1.25	3	1.5	
3	HL-93 (22)	☑			M3	2.28	DL	1.25	3	1.5	
4	HL-93 (26)	☑			M2	2.28	DL	1.25	3	1.5	
5	HL-93 (30)	☑			M1	2.28	DL	1.25	3	1.5	
6	Const Loads	☑					1	1.2	2	1.2	

Figure C – 8 RISA Load Combination Input (4 Girder)

With the dead and live load values determined per girder a RISA model could be completed for the loading. RISA software was used so that a large number of calculations could be completed quickly since much iteration was necessary to optimize the cross section. RISA was programmed for five different design trucks with the rear axle dimensions ranging from 14 - 30 feet in 4 foot increments. The moving load was then turned around and run the opposite direction. The software made roughly two thousand calculations for each girder configuration producing more accurate results than could be accomplished by hand. Once the ultimate moments were determined the second portion of the spreadsheet was used to quickly optimize the design of the positive moment sections (using composite action) and the negative moment sections (without composite action). With an optimized cross section to resist the positive and negative moment if four girders are used the process was repeated for six, eight, ten and twelve girder options. Examples of the spreadsheets can be seen in Figures C – 10 & C – 11.

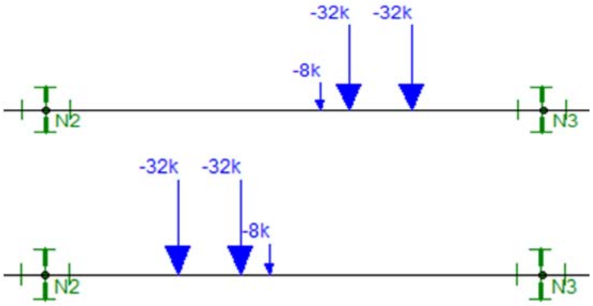


Figure C – 9 Moving load in RISA

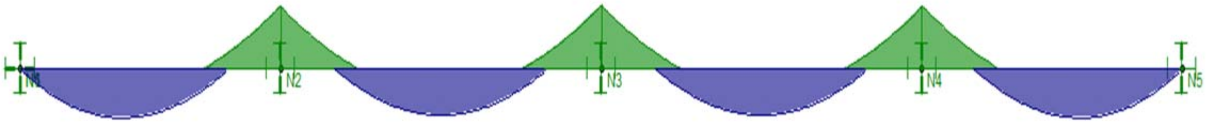
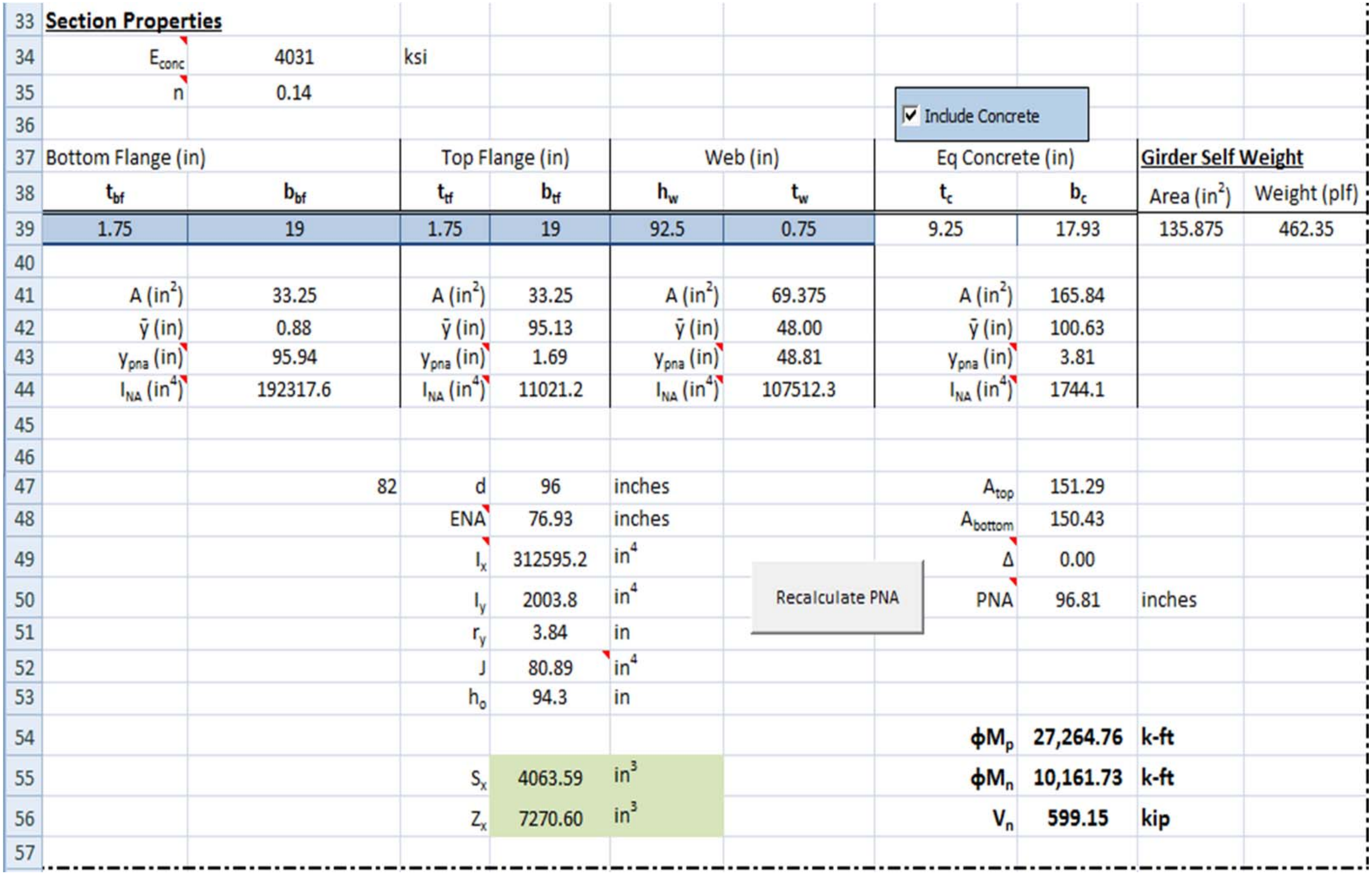


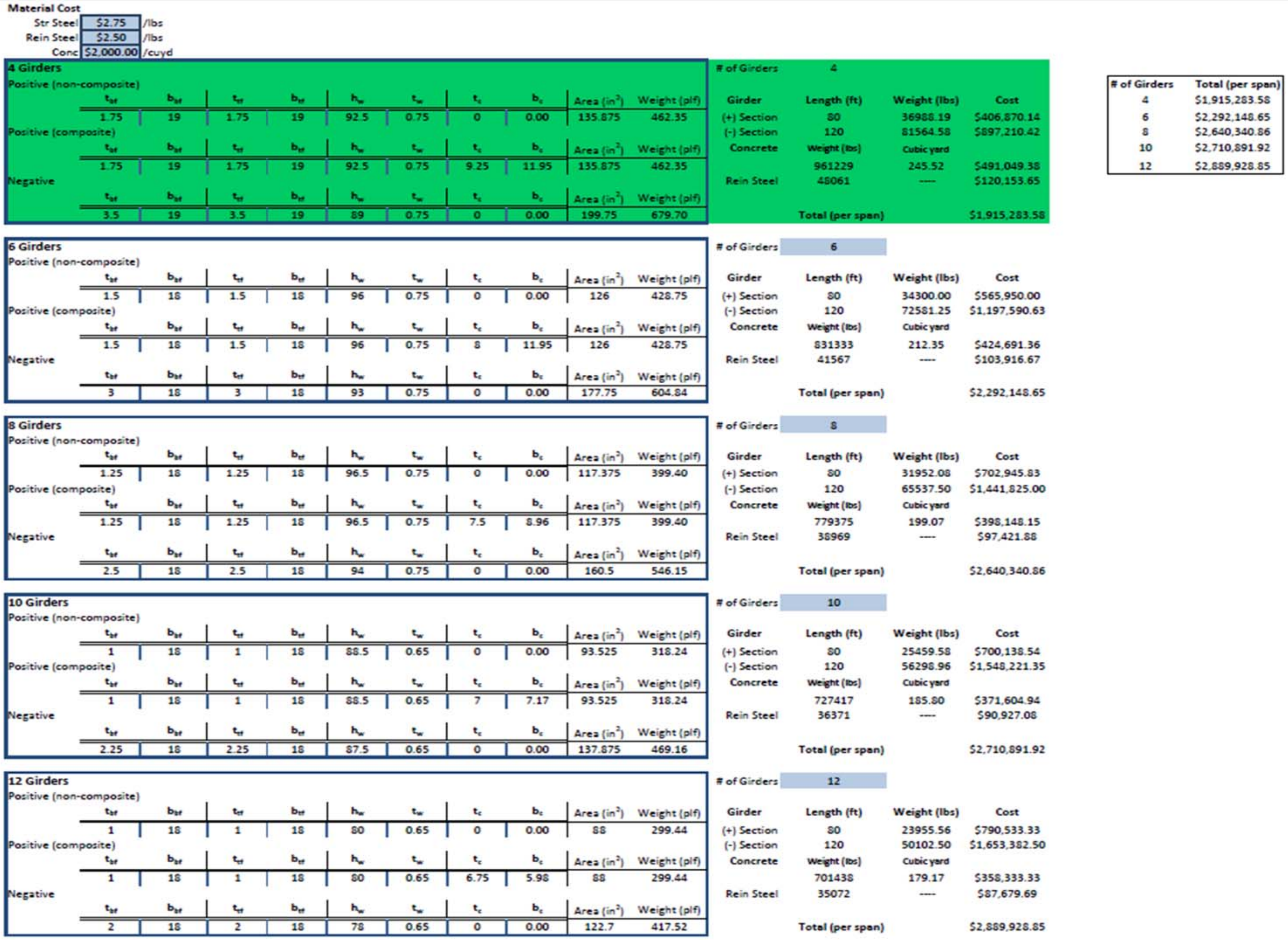
Figure C – 10 Moment Envelope from RISA (4 Girder)

The cross sectional dimensions were compiled on a separate worksheet from which a plot of the cost (steel and concrete) vs. number of girders could be constructed.



33	**Section Properties**									
34	E_{conc}	4031	ksi							
35	n	0.14					☑ Include Concrete			
36										
37	Bottom Flange (in)		Top Flange (in)		Web (in)		Eq Concrete (in)		**Girder Self Weight**	
38	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
39	1.75	19	1.75	19	92.5	0.75	9.25	17.93	135.875	462.35
40										
41	A (in^2)	33.25	A (in^2)	33.25	A (in^2)	69.375	A (in^2)	165.84		
42	$\bar{y}$ (in)	0.88	$\bar{y}$ (in)	95.13	$\bar{y}$ (in)	48.00	$\bar{y}$ (in)	100.63		
43	y_{pna} (in)	95.94	y_{pna} (in)	1.69	y_{pna} (in)	48.81	y_{pna} (in)	3.81		
44	I_{NA} (in^4)	192317.6	I_{NA} (in^4)	11021.2	I_{NA} (in^4)	107512.3	I_{NA} (in^4)	1744.1		
45										
46										
47		82	d	96	inches		A_{top}	151.29		
48			ENA	76.93	inches		A_{bottom}	150.43		
49			I_x	312595.2	in^4		Δ	0.00		
50			I_y	2003.8	in^4	Recalculate PNA	PNA	96.81	inches	
51			r_y	3.84	in					
52			J	80.89	in^4					
53			h_o	94.3	in					
54							**ϕM_p**	**27,264.76**	**k-ft**	
55			S_x	4063.59	in^3		**ϕM_n**	**10,161.73**	**k-ft**	
56			Z_x	7270.60	in^3		**V_n**	**599.15**	**kip**	
57										

Figure C – 10 Cross Section Optimization



Material Cost

Str Steel	$2.75	/lbs
Rein Steel	$2.50	/lbs
Conc	$2,000.00	/cuyd

4 Girders

	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
Positive (non-composite)	1.75	19	1.75	19	92.5	0.75	0	0.00	135.875	462.35
Positive (composite)	1.75	19	1.75	19	92.5	0.75	9.25	11.95	135.875	462.35
Negative	3.5	19	3.5	19	89	0.75	0	0.00	199.75	679.70

# of Girders	4		
Girder	Length (ft)	Weight (lbs)	Cost
(+) Section	80	36988.19	$406,870.14
(-) Section	120	81564.58	$897,210.42
Concrete	Weight (lbs)	Cubic yard	
	961229	245.52	$491,049.38
Rein Steel	48061	----	$120,153.65
	Total (per span)		$1,915,283.58

6 Girders

	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
Positive (non-composite)	1.5	18	1.5	18	96	0.75	0	0.00	126	428.75
Positive (composite)	1.5	18	1.5	18	96	0.75	8	11.95	126	428.75
Negative	3	18	3	18	93	0.75	0	0.00	177.75	604.84

# of Girders	6		
Girder	Length (ft)	Weight (lbs)	Cost
(+) Section	80	34300.00	$565,950.00
(-) Section	120	72581.25	$1,197,590.63
Concrete	Weight (lbs)	Cubic yard	
	831333	212.35	$424,691.36
Rein Steel	41567	----	$103,916.67
	Total (per span)		$2,292,148.65

8 Girders

	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
Positive (non-composite)	1.25	18	1.25	18	96.5	0.75	0	0.00	117.375	399.40
Positive (composite)	1.25	18	1.25	18	96.5	0.75	7.5	8.96	117.375	399.40
Negative	2.5	18	2.5	18	94	0.75	0	0.00	160.5	546.15

# of Girders	8		
Girder	Length (ft)	Weight (lbs)	Cost
(+) Section	80	31952.08	$702,945.83
(-) Section	120	65537.50	$1,441,825.00
Concrete	Weight (lbs)	Cubic yard	
	779375	199.07	$398,148.15
Rein Steel	38969	----	$97,421.88
	Total (per span)		$2,640,340.86

10 Girders

	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
Positive (non-composite)	1	18	1	18	88.5	0.65	0	0.00	93.525	318.24
Positive (composite)	1	18	1	18	88.5	0.65	7	7.17	93.525	318.24
Negative	2.25	18	2.25	18	87.5	0.65	0	0.00	137.875	469.16

# of Girders	10		
Girder	Length (ft)	Weight (lbs)	Cost
(+) Section	80	25459.58	$700,138.54
(-) Section	120	56298.96	$1,548,221.35
Concrete	Weight (lbs)	Cubic yard	
	727417	185.80	$371,604.94
Rein Steel	36371	----	$90,927.08
	Total (per span)		$2,710,891.92

12 Girders

	t_{bf}	b_{bf}	t_{tf}	b_{tf}	h_w	t_w	t_c	b_c	Area (in^2)	Weight (plf)
Positive (non-composite)	1	18	1	18	80	0.65	0	0.00	88	299.44
Positive (composite)	1	18	1	18	80	0.65	6.75	5.98	88	299.44
Negative	2	18	2	18	78	0.65	0	0.00	122.7	417.52

# of Girders	12		
Girder	Length (ft)	Weight (lbs)	Cost
(+) Section	80	23955.56	$790,533.33
(-) Section	120	50102.50	$1,653,382.50
Concrete	Weight (lbs)	Cubic yard	
	701438	179.17	$358,333.33
Rein Steel	35072	----	$87,679.69
	Total (per span)		$2,889,928.85

# of Girders	Total (per span)
4	$1,915,283.58
6	$2,292,148.65
8	$2,640,340.86
10	$2,710,891.92
12	$2,889,928.85

Figure C – 11 Steel Girder Comparison

The results of the analysis show that increasing the number of girders will increase the cost of the structural components of the superstructure. This is unexpected when compared to similar bridge type studies. Some possible reasons for the unexpected results could be the relative short width of this structure as compared to others, an error in the price items included when changing the number of girders or just plain inexperience. From the analysis preformed 4 girders were chosen to reduce cost but retain some redundancy in the event of a structural failure. The optimal cross sections for positive and negative moments are shown in Figure C - 13 & C - 14 respectively.

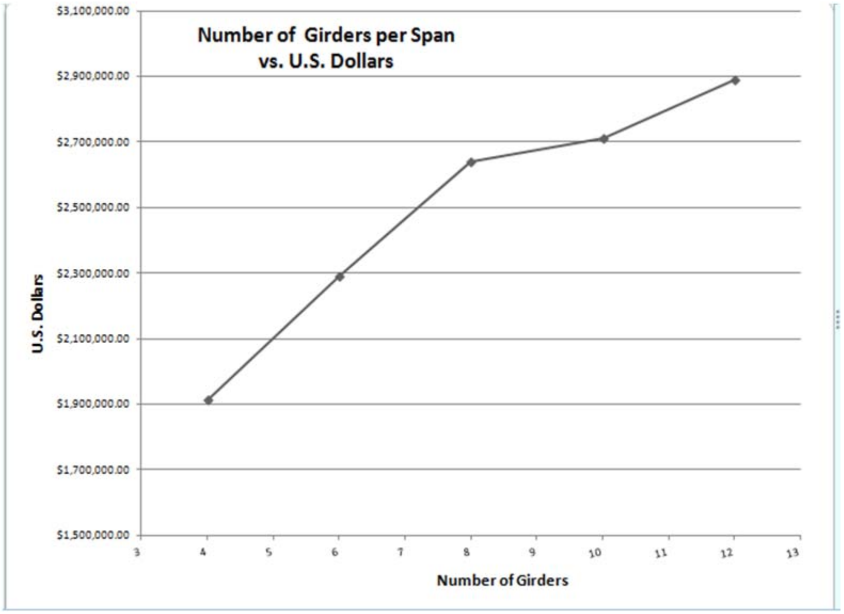


Figure C – 12 No. of Girders Cost Comparison

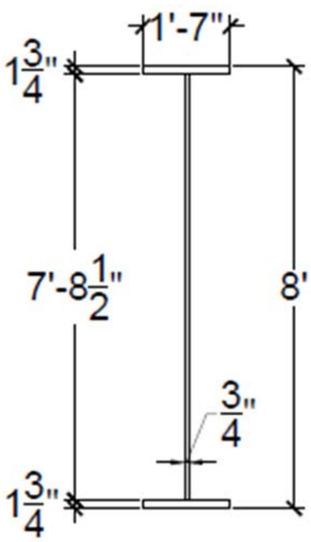


Figure C – 13 Positive Moment Section, I-Girder

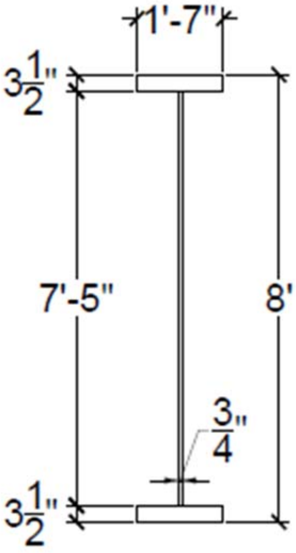


Figure C – 14 Negative Moment Section, I-Girder

Steel Box Girder Optimization

Steel box girders are very similar to I girders working in pairs with regard to moment and shear resistance. The advantage of the steel box girder is the closed form which increases the girders torsional resistance. This added resistance is not 'free' however, while the structural steel dimensions are roughly equal for both I and box girders the box girder requires considerably more bracing increasing the fabrication costs.

Figure C – 14

For bridges that are strait in plan torsion is generally not an issue and the added cost is not warranted. It is when a bridge span is curved like that in figure C - 14 that torsion becomes a design concern. When a span is required to curve like that of figure C – 14, the forces applied to the girder are no longer applied parallel to the pier-to-pier centerline (to the left of the red line on girder 2). This eccentricity causes a torsional force that must be resisted by the bridge girders.

For the Moose Creek Bridge (#2223) the horizontal alignment of the roadway is strait and therefore torsion is not a concern in the superstructure design. Due to the scope and schedule of this project a more detailed analysis of steel box girders was not possible, the following approximations were used for sizing. The steel box girders were sized as two I girders analyzed previously. An additional cost for internal and external bracing steel was added into the cost of the steel box girders for comparison. It was obvious that the steel box girder option would be more expensive than the I-girder by using this process and this was expected from the research completed for this project. As discussed earlier steel box girders have advantages that make them a very competitive option for curved and sometimes long span bridges; however those advantages come at a cost deemed unnecessary for the Moose Creek Bridge.

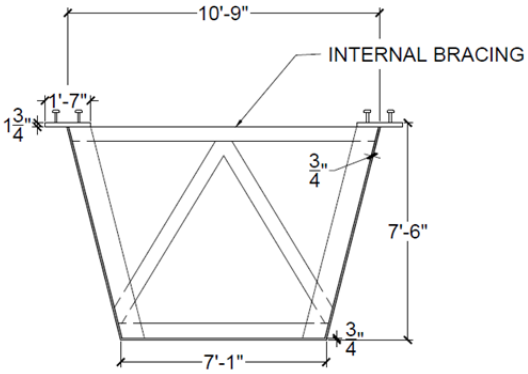


Figure C – 15

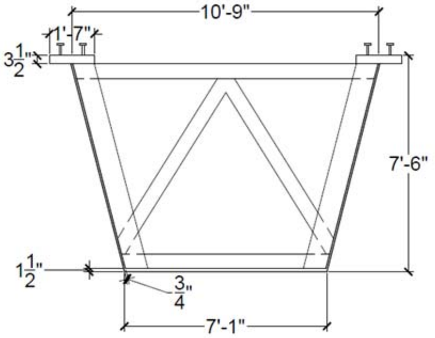


Figure C – 16

Concrete Box Girder Optimization

Due to both the scope and schedule of this project as well as the complexity of designing large post-tensioned elements like the sections of a concrete box girder (Fig. C - 17) the team reached out to an experienced concrete bridge designer for help in dimensioning this superstructure option. A video conference was arranged with Mike Keller PhD, P.E., S.E. of FIGG Engineering Group in Denver, CO to discuss the reasons why a concrete box girder would or would not be a viable option for the Moose Creek Project. The team discussed how to determine the cross sectional dimensions of the girder as well as the approximate amount of reinforcing steel and pre-stressing tendons that would be required so that an estimated cost could be developed for this superstructure option. The meeting lasted for approximately two hours and included a discussion about challenges in constructing this style of bridge that are discussed in construction methods. The rules for dimensioning the concrete box cross section are summarized in Table C - 1. The cross section used in this study is shown in Figure C - 17.

Table C – 1 Sizing Rules (Concrete Box Girder)

Span / Depth Ratio	18 ~ 20
Width of Box / Width of Web	18 ~ 20
Top Slab Thickness	10"
Bottom Web Thickness	7"
Diaphram Concrete	8% of Total Weight
Reinforcing Steel in Diaph.	16% of Diaph. Weight

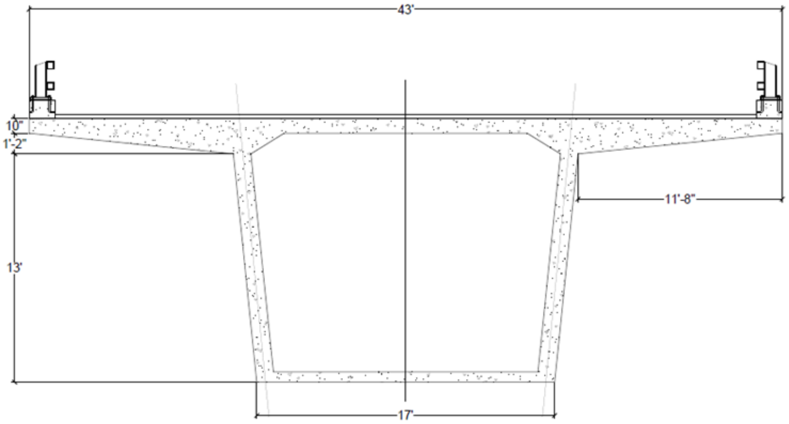


Figure C – 17

APPENDIX D: COST ESTIMATION HAND CALCULATIONS

University of Alaska Anchorage
Seawolf Engineering
Cost Estimation Hand Calculations

Date: 03/15/2013
Project No.: 2223
Project Name: GLENN 53-56
Calc. By: NTP

For: MOOSE CREEK COST ESTIMATION

Comments

STRUCTURAL FILL

W = width of bridge = 43ft

ℓ = 50ft

h = height of girders + 4ft:

Bulb tee: h = 66in ≈ 6ft

Steel: $t_f + h_w = 1.625in \times 2 + 100in \approx 9ft$

Conc. Box: 9ft

$V_{BULBTEE} = 2 \times [50ft \times 43ft \times (6ft + 4ft)] \approx \underline{\underline{1600 yd^3}}$

$V_{STEEL} = 2 \times [50ft \times 43ft \times (9ft + 4ft)] \approx \underline{\underline{2080 yd^3}}$

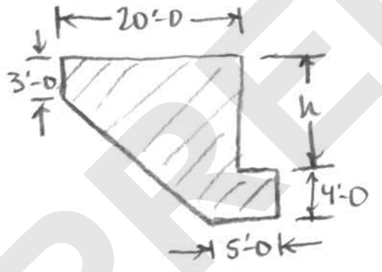
$V_{conc. BOX} = 2 \times [50ft \times 43ft \times (9ft + 4ft)] \approx \underline{\underline{2080 yd^3}}$

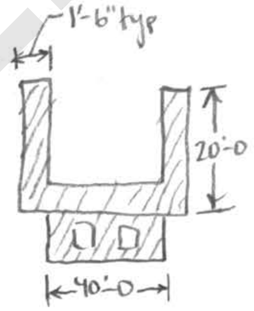
— ℓ from unit cost data, DOT 2012

— steel plate and steel box girders identical in height

CLASS A. CONCRETE

Abutments:





- BULB TEE
  - wingwalls: $1.5ft \times [3ft \times 20ft + \frac{1}{2} 20ft \times (6ft + 4ft - 3ft)] \times 2 = 390ft^3$
  - back wall: $1.5ft \times (6ft + 4ft) \times 40ft = 600ft^3$
  - bearing surface: $5ft \times 4ft \times 40ft = 800ft^3$

TOTAL: $2 \times (390ft^3 + 600ft^3 + 800ft^3) \approx 135yd^3$

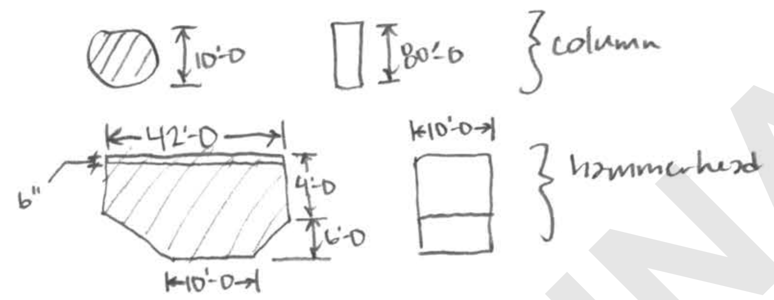
- STEEL:
  - wingwalls: $2\times1.5ft\times[3ft\times20ft+\frac{1}{2}20ft\times(9ft+4ft-3ft)]$
    $=480ft^3$
  - backwall: $1.5ft\times(9ft\times4ft)\times40ft=780ft^3$
  - bearing surface: $800ft^3$

    TOTAL: $2\times(480ft^3+780ft^3+800ft^3)\approx155yd^3$

- Concrete Box:

  TOTAL: $155yd^3$

Piers:



dimensions from drawings

$V_{column}=\frac{\pi}{4}(10ft)^2\times80ft=6283ft^3$

$V_{hammerhead}=10ft\times[42ft\times4.5ft+\frac{1}{2}6ft\times(42ft+10ft)]$
$=3450ft^3$

$V_{PIER}=6283ft^3+3450ft^3\approx365yd^3$

- BULB TEE

  TOTAL: $365yd^3\times5=1825yd^3$

- STEEL PLATE
- STEEL BOX

  TOTAL: $365yd^3\times3=1095yd^3$

- CONCRETE BOX

  TOTAL: $365yd^3\times2=730yd^3$

CLASS A TOTALS, SUBSTRUCTURE:

- BULB TEE: $1825yd^3+135yd^3=\underline{1960yd^3}$
- STEEL PLATE
- STEEL BOX

  $1095yd^3+155yd^3=1250yd^3$
- CONCRETE BOX: $730yd^3+155yd^3=885yd^3$

Additional Class A Concrete:

- deck for steel plate/box girders:

  $t = 9.25in$

  $V = 9.25in \frac{ft}{12in} \times 43ft \times 800ft \approx 985yd^3$

  TOTAL: $1250yd^3 + 985yd^3 = \underline{2235yd^3}$

- Girders for Concrete Box

  $A = 88.76ft^2$ — from drawings

  $V = 1.08(88.76ft^2 \times 800ft) \approx 2840yd^3$ — 8% additional for deck, MIKE KELLER FIGG ENGINEERING

  TOTAL: $2840yd^3 + 885yd^3 = \underline{3725yd^3}$

CLASS DS CONCRETE

$V_{EACH} = \frac{\pi}{4}(12ft)^2 \times 140ft \approx 590yd^3$

- BULB TEE

  TOTAL: $590yd^3 \times 5 = \underline{2950yd^3}$

- STEEL PLATE
- STEEL BOX

  } TOTAL: $590yd^3 \times 3 = \underline{1770yd^3}$

- CONCRETE BOX

  TOTAL: $590yd^3 \times 2 = \underline{1180yd^3}$

REINFORCING STEEL

- uncoated steel — assumed $\gamma_{conc.} = 150lb/ft^3$

  -BULB TEE: $4\% \times 1960yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3} = \underline{318,000lbs.}$

  -STEEL PLATE
  -STEEL BOX
  } $4\% \times 2235yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3} = \underline{362,000lbs.}$ — 4% of class A conc. ELMER MARX, AK DOT&PF

  -CONCRETE BOX: $4\% \times 3725yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3}$

  $= \underline{603,000lbs}$

- epoxy coated steel — coated steel: bulb tee - 2% others - 5% ELMER MARX, AK DOT & PF

  -BULB TEE: $2\% \times 1960yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3} = \underline{159,000lbs.}$

  -STEEL PLATE
  -STEEL BOX
  } $5\% \times 2235yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3} = \underline{453,000lbs.}$

  -CONCRETE BOX: $5\% \times 3725yd^3 \times \frac{150lb/ft^3}{yd^3/ft^3}$

  $= \underline{754,000lbs.}$

STRUCTURAL STEEL

W = 36988 lbs/span + 81565 lbs/span = 118553 lbs./span

TOTAL: 118553 lbs/span × 4 spans = 474212 lbs.

- Bracing Steel Added:
  - STEEL PLATE: 1.15 × 474212 lbs ≃ 546,000 lbs.
  - STEEL BOX: 1.25 × 474212 lbs. ≃ 593,000 lbs.

- I-Girder Optimization Spreadsheet. Michael Beauvais.
- steel plate: 15%. steel box: 25%. ELMER MARX. AK DOT & PF

FURNISH STRUCTURAL STEEL PILES

l = 100ft ~ 120ft

TOTALS:

- BULB TEE: 2 × (100ft × 8) = 1600ft
- STEEL PLATE: 2 × (120ft × 4) = 960ft
- STEEL BOX: 2 × (120ft × 6) = 1440ft
- CONCRETE BOX: 2 × (100ft × 6) = 1200ft

- one H-Pile per girder
  6 H-Piles for box girders
- used 100' for conc. and 120' for steel due to greater load

DRIVE STRUCTURAL STEEL PILES

- BULB TEE: n = 8×2 = 16
- STEEL PLATE: n = 4×2 = 8
- STEEL BOX: } n = 6×2 = 12
- CONCRETE BOX: }

BRIDGE RAILING

2 × length of bridge: 2 × 800ft = 1600 ft

DRILLED SHAFT LUMP SUM

take as ratio of max. no. of piers

- $1,000,000/5 piers = $200,000/pier

- BULB TEE: $200,000/pier × 5 piers = $1,000,000
- STEEL PLATE: } $200,000/pier × 3 piers = $600,000
- STEEL BOX: }
- CONCRETE BOX: $200,000/pier × 4 piers = $800,000

- SEAN BASKI, AK. DOT & PF

SHAFT CASING

assume Steel is 8% of concrete weight in shaft — DOT unit cost data, 2012

$W_{ea\,ch} = 0.08 \times 590yd^3 \times \frac{150lb/ft^3}{yd^3/27ft^3} = 191,000lbs.$

TOTALS:

- BULB TEE: 191,000lbs × 5 = 955,000lbs.
- STEEL PLATE: } 191,000 lbs × 3 = 573,000lbs.
- STEEL BOX: }
- CONCRETE BOX: 191,000lbs × 2 = 382,000lbs.

SHAFT EXCAVATION

depth of excavation = 140ft each

TOTALS:

- BULB TEE: 140ft × 5 = 700ft
- STEEL PLATE: } 140ft × 3 = 420ft
- STEEL BOX: }
- CONCRETE BOX: 140ft × 2 = 280ft