Appendix 2

Frank J. Wood Preliminary Design Report

Preliminary Design Report

Frank J. Wood #2016
over
Androscoggin River

Brunswick-Topsham, Maine

STP-2260(300)X
WIN 22603.00

Maine Department of Transportation
Bridge Program

August 4, 2017

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BACKGROUND INFORMATION

TOWN	Brunswick-Topsham	**WIN**	22603.00	**BRIDGE NO.**	2016
BRIDGE	Frank J. Wood			**STATE ROUTE**	201/24

FUNDING: Federal/State

PROGRAM SCOPE: Bridge Improvement

PROGRAM DESCRIPTION: Frank J. Wood Bridge (#2016) over Androscoggin River. Located at the Brunswick – Topsham town line.

PROJECT BACKGROUND: This bridge was constructed in 1931 and was rehabilitated in 1985 and 2006. It is currently in poor condition and has safety and mobility limitations. Preconstruction engineering was funded in the 15/16/17 Work Plan with partial construction funding added in the 16/17/18 Work Plan.

		JURISDICTION	State Highway	**NHS**	No
		FUNCTIONAL CLASSIFICATION	Minor Arterial	**CORRIDOR PRIORITY**	3
		URBAN/RURAL	Urban	**FHWA SUFFICIENCY RATING**	25.4
		LOAD POSTING	25 tons	**POSTED SPEED**	25 mph
TRAFFIC:	2015	**AADT**	18,860	**ACCIDENT DATA, CRF**	1.93
	2035	**AADT**	22,630	**DHV**	2263

EXISTING BRIDGE

YEAR BUILT 1931 **SPAN LENGTHS** 310’-310’-175’=805’ **CURB TO CURB WIDTH** 30’

TYPE OF SUPERSTRUCTURE: Three-span painted, riveted steel through truss with a concrete filled steel grid deck and bituminous wearing surface supported on steel crossbeams, steel stringers, and steel floor beams. 2’ each side of roadway remains open grid for drainage. There is a 5’ sidewalk cantilevered off the upstream truss.

GENERAL CONDITION: Steel members are in poor condition with significant section loss and pack rust evident along with extensive failing paint. Concrete filled steel grid deck is in poor condition with rust staining the underside. Bridge joints were recently replaced in 2015.

TYPE OF SUBSTRUCTURE: Cantilevered concrete abutments on ledge. Mass concrete piers on ledge.

GENERAL CONDITION: The substructures, having been rehabilitated in 2006, are in satisfactory condition. The south abutment has a 2’ long horizontal crack about 4’ above the bridge seat near the wing wall. A portion of this abutment sits on stone masonry that shows signs of shifting stones.

LOAD RATINGS:

	OPERATING	**INVENTORY**
HL-93 Truck	23.7 Tons	18.4 Tons
Rating Factor	0.66	0.51

	LEGAL LOADS
Controlling Configuration: 6	25 Tons
Rating Factor	0.65
Controlling Member:	Span1 & 2 floor beam 7 in shear See Appendix C for updated load rating in 2016 Inspection Report

STRUCTURALLY DEFICIENT Yes **FUNCTIONALLY OBSOLETE** N/A

MAINTENANCE PROBLEMS: Open grid at curb lines allows salt laden water to run on supporting steel truss members of floor framing system and bottom chord of truss.

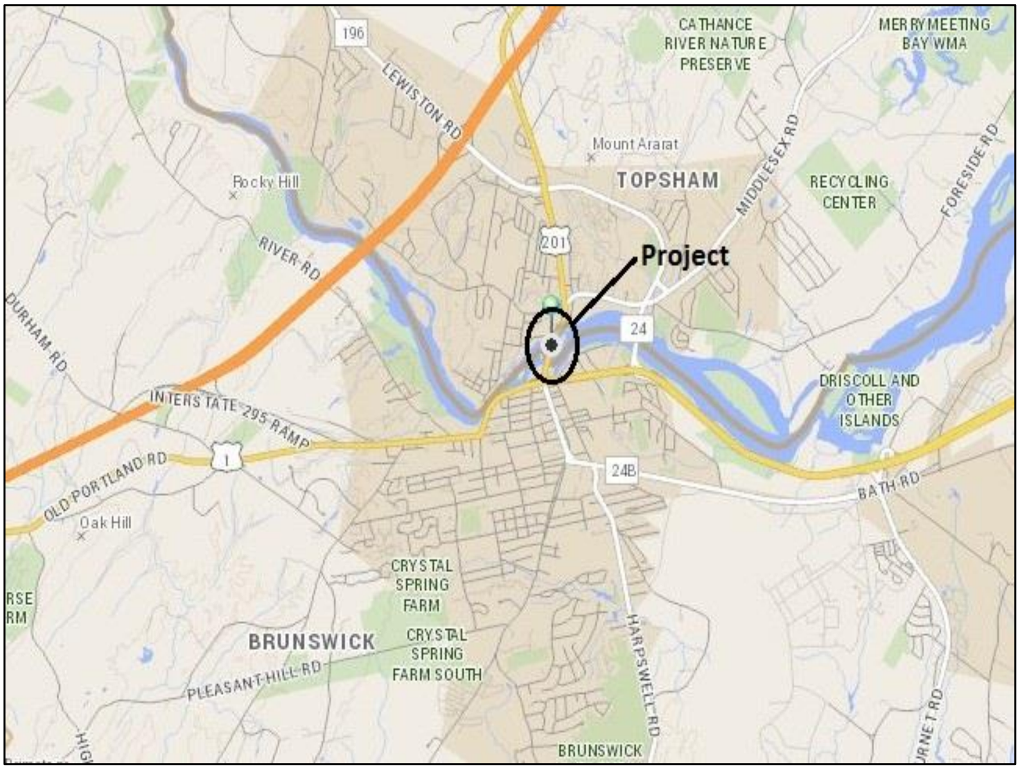
MAINTENANCE WORK: NA

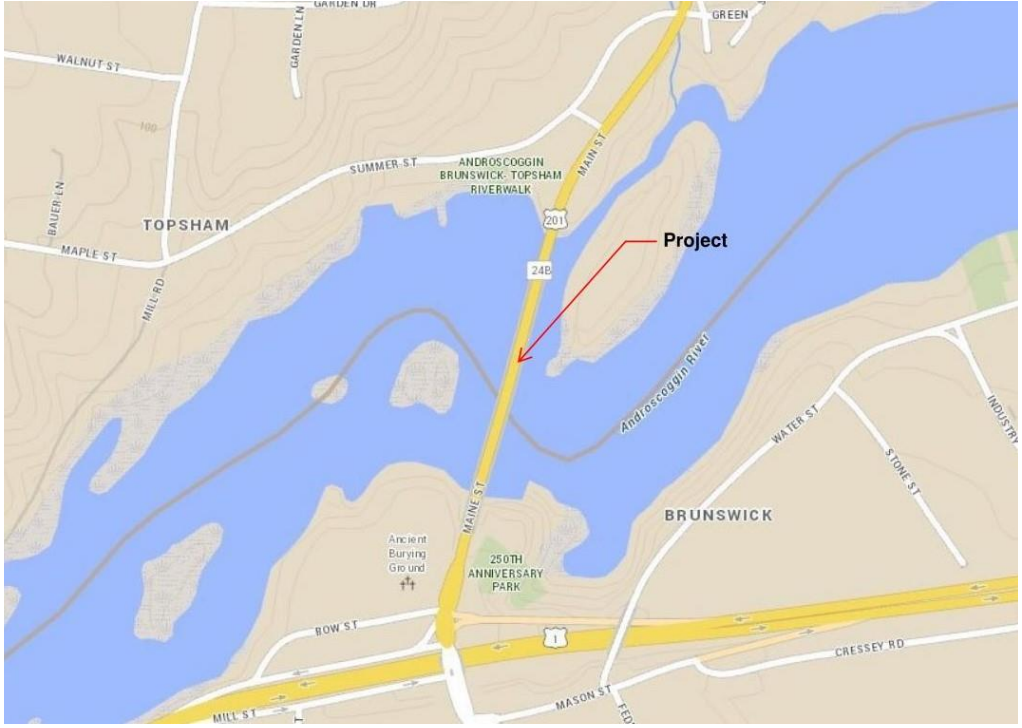
PREVIOUS STRUCTURE: A timber covered bridge on granite abutments.

OTHER COMMENTS: The bridge is not individually eligible for listing on the National Register of Historic Places, but is a contributing element to the Brunswick-Topsham Historic District.

LOCATION MAP

Brunswick-Topsham, Frank J. Wood #2016, WIN 22603.00
Route 201/24 over Androscoggin River





Latitude: 43° 55' 14.27" N, Longitude: 69° 57' 57.46" W

BRIDGE RECOMMENDATION FORM

TOWN	Brunswick-Topsham	**BRIDGE**	Frank J. Wood	**BRIDGE NO.**	2016
DESIGNED BY	TY Lin Intern.	**DATE**	8/4/2017	**WIN**	22603.00
APPROVED BY	JSF	**DATE**	9-6-2017		
APPROVED BY	WLF	**DATE**	9-6-2017		

PROJECT: Bridge replacement with 760′ of approaches, including transitions.

ALIGNMENT DESCRIPTION: Bridge on a 1200′ radius horizontal curve matching into existing approaches with an 800′ radius curve in Brunswick and a tangent in Topsham. Vertical grade is 0.90% tangent on bridge matching into existing approaches with a combination 90′ crest and 200′ sag vertical curves in Brunswick and a combination 100′ crest and 90′ sag vertical curves in Topsham. The finished grade is approximately 2.5′ higher than existing bridge. The new centerline is located about 120′ upstream (west) of existing bridge centerline at its greatest distance.

APPROACH SECTION: Two 11′ lanes with 5′ shoulders and 5′ sidewalks each side. 1:2 sideslopes with standard steel guardrail and 1:4 sideslopes without guardrail.

SPANS	260′-205′-205′-145' = 815′	**SKEW**	0° ahead Radial except as noted
LOADING	HL-93 modified for Strength 1	**DESIGN SPEED**	25 mph

SUPERSTRUCTURE: 4-span, continuous steel I girder composite bridge with an 8 1/2″ CIP concrete deck and 3″ bituminous wearing surface with 1/4″ high performance membrane waterproofing. 32′ curb-to-curb roadway with a 2% normal crown and 5′ sidewalks each side. Bridge rail is a TL-2 compliant, traffic/pedestrian bridge railing. Final cross section and aesthetic details will be developed through collaboration with the Towns′ Design Advisory Committee and the Section 106 Consulting Parties.

ABUTMENTS: Deep cantilevered concrete abutment on the Brunswick side and stubbed cantilevered concrete abutment on the Topsham side all supported on concrete subfootings founded on ledge.

PIERS: Reinforced concrete solid shaft piers supported on concrete seals founded on ledge. Pier 3 skewed 35° ahead on left for improved hydraulics.

OPENING AND CLEARANCE	**EXISTING**	**PROPOSED**
TOTAL OPENING	23,750 SF	23,400 SF
TOTAL OPENING AT ELEVATION * FT	* SF	* SF
FREEBOARD CLEARANCE AT Q50 ELEVATION	* FT	* FT

*Refer to detailed hydraulic analysis data included in Appendix E.

AVAILABLE SOILS INFORMATION: Existing plans and survey show ledge to be present and exposed throughout this site. Exact ledge locations to be determined with field borings.

ADDITIONAL DESIGN FEATURES: Begin transition @ STA 00+70, begin project @ STA 1+00, end project @ STA 14+50, end transition @ STA 15+75. Variable height retaining walls will be constructed between STA 2+19+/- and 3+04+/- 22’ left at the Brunswick approach and between Sta 12+55 to Sta 13+95 +/- 24.25’ left at the Topsham approach. Bridge will be lighted both sides of roadway. Add 10’ long by 5’ wide overlook platforms to each side of new superstructure. Existing brick paved approach sidewalks will be matched and continued to the new bridge. Amenities and aesthetics on the bridge and impacted approaches will be further reviewed and discussed with established Design Advisory Committee representing Brunswick and Topsham.

MAINTENANCE OF TRAFFIC: Maintain two-way traffic on existing bridge.

CONSTRUCTION SCHEDULE: Two construction seasons with removal of the existing bridge the following winter.

ADVERTISING DATE: August, 2018

		Program Amount	Available Funding	Estimated Project Cost	Shortfall/ Surplus
	Preliminary Engineering	$1,225,000	$1,200,000	$1,200,000	*$0*
	Right-of-Way	$50,000	$50,000	$50,000	*$0*
Construction [	**Structure**	$13,000,000	$13,000,000	$12,455,000	*$0*
	Approaches			$545,000	*$0*
	Construction Engineering	$650,000	$650,000	$750,000	*-$100,000*
	Total	**$14,925,000**	**$14,900,000**	**$15,000,000**	**-$100,000**

ADDITIONAL BORINGS REQUIRED? No

ADDITIONAL GEOTECHNICAL EVALUATIONS REQUIRED? Yes

APPROVED DESIGN EXCEPTIONS: Design variance needed from the Program for opening in bridge rail to accommodate the overlook platforms.

COMMENTS BY ENGINEER OF DESIGN:

SUMMARY OF EXPECTED IMPACTS

RIGHT OF WAY Number of: Property Owners 4
Buildings to Be Taken 0

Type of Acquisitions: ☒ Fee Simple ☒ Easement
☒ Temporary Rights ☐ Temporary Road

UTILITIES: On Existing Bridge – Brunswick-Topsham Water District, GWI Communication, Fairpoint Communication, OTT Communication; On Approaches – Maine Natural Gas, CMP, Brunswick Sewer, Topsham Sewer

COAST GUARD PERMIT NEEDED? Exception Request Required **FAA PERMIT NEEDED?** No

ENVIRONMENTAL COORDINATION
Team Member: Kristen Chamberlain

NEPA	The FHWA and the MaineDOT initially proposed to prepare a Categorical Exclusion for this project under 23 CFR 771.117(d)(3). However, due to the presence of several environmental resources within the project area such as historic properties and districts, and threatened and endangered species and critical habitat, in addition to substantial public interest and controversy, the FHWA and the MaineDOT decided in the spring of 2017 to prepare an Environmental Assessment.
STIP	PE, ROW, ADVERTISE/CONSTRUCTION: 4/4/17
Section 106	The Section 106 process determined that the upstream replacement alternative would have adverse effects to three historic resources: the Cabot Mill, Pejepscot Paper Company, and the Brunswick Topsham Historic District resulting from the removal of the Frank J. Wood Bridge. The bridge is the last element of the setting of the two mills that was constructed during the period of significance of the mills. Removal of the Frank J. Wood Bridge will diminish the Cabot Mill's and the Pejepscot Paper Company's integrity of setting, feeling, and association. Section 106 requires mitigation of adverse effects if they cannot be avoided. Mitigation will be finalized with input from Section 106 consulting parties as design of the proposed alternative proceeds.
Section 4(f)	The Town of Brunswick Park on the southeast corner of the bridge is a 4f resource. In addition, the Section 106 resources listed above are also 4(f) resources. Adverse Effects to historic transportation structures under Section 106 of the National Historic Preservation Act are considered a "use" under Section 4(f) of the U.S. Department of Transportation Act of 1966. The upstream alternative will result in a use of Section 4(f) properties. Final evaluation of impacts to Section 4(f) resources and approval of the use will be completed by FHWA.

Endangered Species	Shortnose sturgeon and Atlantic sturgeon are known to use the project area for staging and spawning. MaineDMR has provided data collected to date about species use of the area. Consultation with National Marine Fisheries Service under Section 7 of the Endangered Species Act will be required. Pre-coordination with NMFS prior to initiation of consultation is on-going.
Essential Fish Habitat	Project is located within Essential Fish Habitat for Atlantic salmon. Other NOAA Trust Resources present include alewives, American shad, and blueback herring. Permanent and temporary impacts to EFH need to be avoided and minimized.
Fish Passage	Will be provided during and post-construction. Impacts to Brookfield fish way need to be considered, minimized, and included in Section 7 consultation.
In-Stream Window	AVOID APRIL 7-AUGUST 30 to minimize impacts to Sturgeon, alewives, American shad and blueback herring.
Hazardous Material	Initial site assessments have indicated a property on the northwest Topsham approach that was a former gas station. The data suggests the alternative would not directly impact the site with the initial limits of cuts, fills and property acquisition, but will require additional borings and coordination through final design to ensure compliance.
Dredge Material	Material excavated from below OHW/HAT is considered dredge and must be managed as special waste. Amount of dredge and disposal options TBD.
Stormwater/MS4	N/A
DEP/LUPC	Permit-by-Rule Section 11
ACOE	Individual Permit

Avoidance & Minimization:

Avoidance and Minimization of impacts to Natural Resources, Cultural Resources, and Endangered Species will continue during Final Design in accordance with applicable State and Federal Laws.

River Impacts: 1.75H: 1V riprapped slopes used at the abutments. A 2H: 1V side slope used at the southwest approach corner and a retaining wall located at the back side of the sidewalk used at the northwest approach corner. State standard bridge width used.

SUMMARY OF PRELIMINARY DESIGN

BACKGROUND

The Frank J. Wood Bridge is a critical link spanning the Androscoggin River between the Towns of Brunswick and Topsham, carrying US 201 and ME 24 and about 19,000 vehicles a day. Just 500 feet upriver of the bridge is a power generation dam harnessing the power of Brunswick Falls. On the southern, Brunswick side of the bridge sits the 250th Anniversary Park on the east and the bustling Fort Andross Mill Complex (originally the Cabot Mill) on the west. The Topsham approach adjoins a bank on the west side, and a dentist office and the Bowdoin Mill Complex (originally the Pejepscot Paper Company) on the east side. Both the Fort Andross and the Bowdoin mill complexes house a variety of shops, businesses, and restaurants, and the Frank J. Wood Bridge is a key pedestrian connection between the two of them and between the larger business districts and communities on each side. The bridge links the hearts of the two communities across the Androscoggin River, connecting Brunswick and Topsham.

Figure 1: The Frank J. Wood Bridge spanning the Androscoggin River between Brunswick and Topsham

The Frank J. Wood is the central of three vehicular crossings of the Androscoggin River between Brunswick and Topsham. About 2 miles upstream, I-295 crosses the river; it has interchanges with U.S. 1 on the Brunswick side and ME 196 on the Topsham side. Less than 1 mile downstream, ME 196 (also known as the Coastal Connector) crosses the river. In addition to these vehicular crossings, the historic Swinging Bridge is a pedestrian crossing of the river about ½ mile upstream of the Frank J. Wood Bridge. Figure 2 shows all of these crossings.

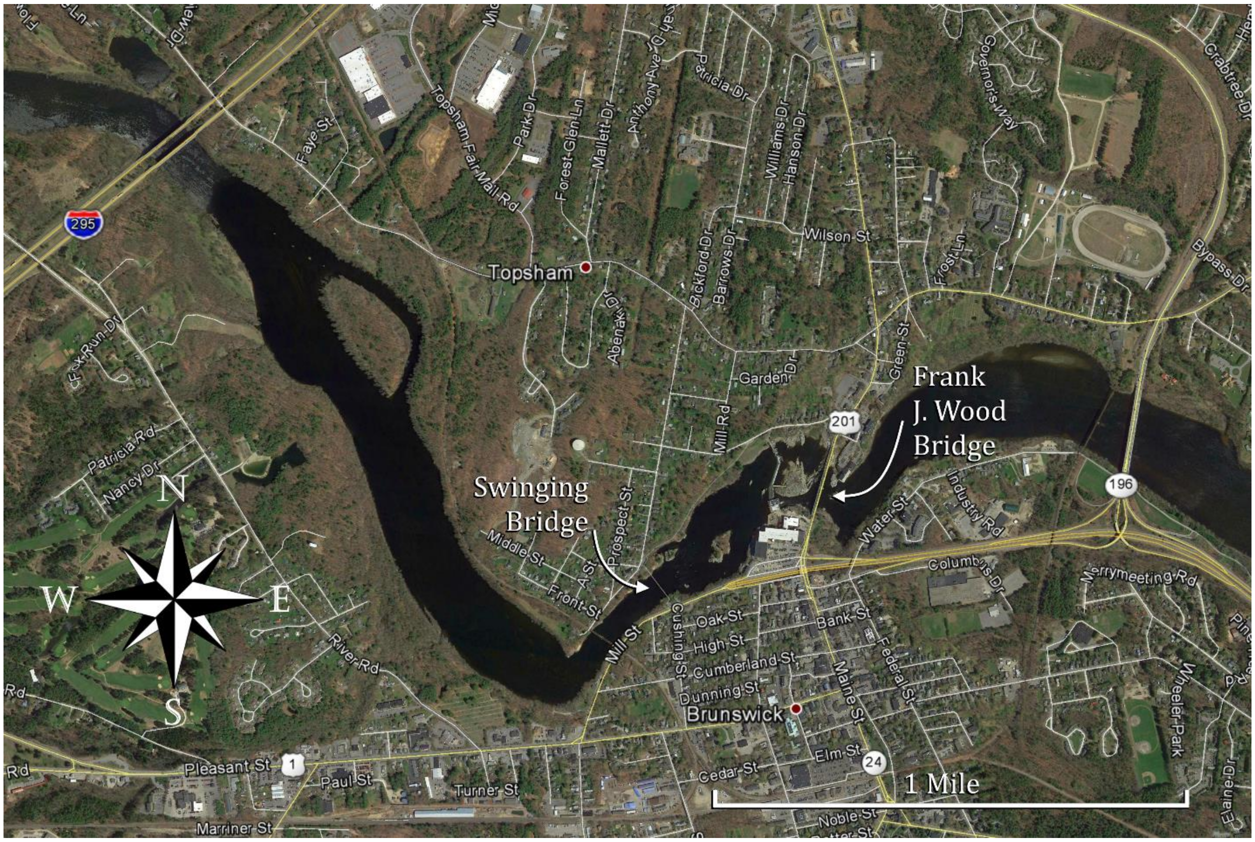


Figure 2: Androscoggin River crossings between Brunswick and Topsham

The Frank J. Wood Bridge is an 85-year-old, 805-foot-long, three span steel through-truss bridge with spans of 310’-310’-175’. It was rehabilitated most recently in 1985, 2006, and 2015. It is a “fracture critical” structure, indicating it is vulnerable to sudden collapse if certain components fail, in this case the truss diagonal and bottom chord members and their connections and the floor beams. Because of this designation, more detailed inspections are required. Detailed inspections by MaineDOT in 2012, June 2016 and August 2016 found many deteriorated areas. A load rating done by MaineDOT in 2013 and updated in August 2016 found some floor system members are no longer adequate for Maine’s legal loads. The bridge is now posted for 25 tons. There is corrosion and section loss in the steel floor system supporting the deck (the transverse cross beams, longitudinal stringers, and transverse floor beams). The floor system, bottom chords, and the concrete deck are currently in poor condition, and the bridge has a FHWA Sufficiency Rating of 25.4. Corrosion at the deteriorated areas is continuing and accelerating, and will do so until the bridge is rehabilitated comprehensively. Refer to Appendix C for sections of the reports listed.

Because of the ongoing deterioration of the structural steel, MaineDOT has completed temporary repairs to address the worst issues so the bridge can maintain its current load rating for up to five years. Steel was added to the worst sections of the floor system beneath the deck and missing and deteriorated rivets were repaired or replaced. Refer to Appendix C for a Summary Report of this temporary work. These temporary repairs were needed to keep the 25 ton weight limit from being reduced more. As maintenance, this 5-year repair was funded separately from the longer-term “capital improvement” project. However, a long-term solution needs to be implemented within the 5 year timeframe or sooner. There is no guarantee that this temporary repair will eliminate additional emergency work. The rate of deterioration evidenced within the August 2016 Inspection Report referenced above identifies the urgency of implementing a long-term improvement solution. This report examines what the alternatives are for the long-term solution.

The travelway through the truss is 30 ft wide, with two 11 ft travel lanes and 4 ft shoulders. Though there are sidewalks on both sides of the road within a few hundred feet of the bridge, the existing bridge carries a single sidewalk on the west side of the bridge. Because the outer 2 feet of each shoulder is an open steel grid for drainage, the usable shoulder width for bicycle travel is reduced to just 2 ft.

The bridge is not individually eligible for listing on the National Register of Historic Places, but is a contributing element to the Brunswick-Topsham Historic District. It is also adjacent to the National Register-Listed Pejepscot Paper Company, National Register-Eligible Cabot Mill and National Register-Eligible Summer Street Historic District.

Accident data from 2009-2013 shows 27 accidents at the intersection of Maine Street and Bow/Cabot Street in Brunswick and 11 accidents at Summer Street and Main Street in Topsham. Also, there were 24 accidents just off the bridge on the Brunswick approach. The accident reports show that these accidents were primarily caused by driver inattention and distraction or by following too closely. In general, these accidents do not appear to be influenced by the bridge. Refer to Appendix G for traffic and accident data.

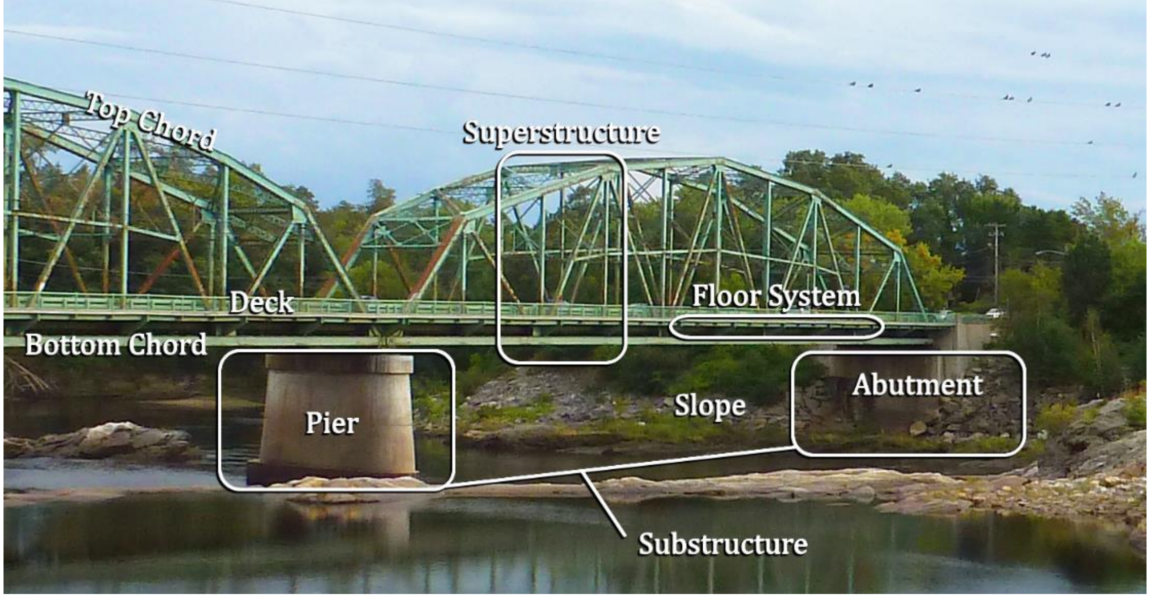


Figure 3: This report uses technical terms to describe various parts of the bridge. The superstructure is what many think of as a "bridge", including the steel floor system or girders below the deck, while the substructure is what supports the superstructure. The deck (what cars drive on) rests on the floor system, which is made up of floor beams, stringers, and sometimes crossbeams. The floor system carries load from the deck to the truss bottom chord.

PURPOSE AND NEED

The purpose of the project is to address poor structural conditions and load capacity issues on the Frank J. Wood Bridge and to address pedestrian and bicycle mobility and safety concerns.

Bridge improvements are needed to improve the condition ratings of the superstructure and deck from a rating of 4 (poor condition) to 7 (good condition). Because of the age of the bridge, 85 years old, and the considerable number of heavy loading cycles it has already experienced, steel fatigue concerns on critical tension members need to be addressed to continue to carry heavy truck traffic on the existing truss. Additionally, the floor beams and stringers need improvements to bring their load rating factors to a 1.0 for all MaineDOT legal loads.

This bridge is classified by the Federal Highway Administration (FHWA) as structurally deficient with superstructure and deck condition ratings of 4 out of 9 (poor condition). The 3 truss spans are fracture critical, meaning that failure of certain steel tension members could cause any of the 3 spans to collapse. Some of the steel truss bridge components are fatigue sensitive, susceptible to cracking and fracture as a result of heavy cyclic loading. The floor beams and stringers within the truss spans do not meet current design load or MaineDOT legal load standards.

Pedestrians on the east side of Routes 201/24 cannot cross the river without crossing the highway, and the existing mid-block pedestrian crossings are considered dangerous. Bicycle traffic is seriously limited by the narrow, 2 ft, paved shoulder.

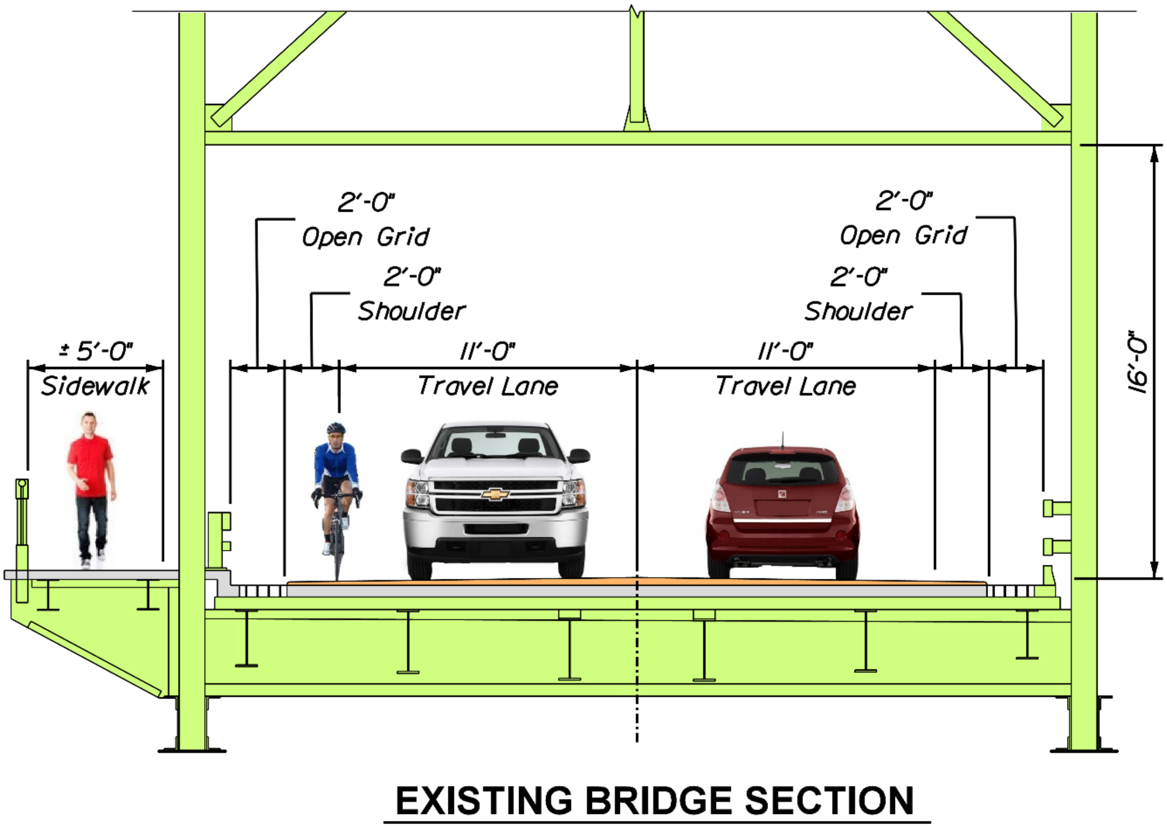


Figure 4: The existing truss bridge cross section

SUMMARY OF ALTERNATIVES

The following alternatives were considered:

1. New 800 ft bridge on the existing alignment.
2. New 835 ft bridge on a curved alignment upstream of the existing bridge.
3. Rehabilitation of the existing steel truss bridge.
4. Rehabilitation of the existing steel truss bridge, including the addition of a new east-side sidewalk.
5. New 800 ft bridge on a parallel alignment downstream of the existing bridge.

A No Build alternative was also considered.

On Point Construction Services, a private consultant firm specializing in construction scheduling and estimating, joined the Project Team to review the constructability of the proposed alternatives, to develop construction schedules, and to estimate temporary bridge costs.

All of the alternatives were compared based on hydraulic requirements; environmental, historical, right of way, and utility impacts; maintenance of traffic, constructability, maintainability, and geotechnical site conditions; and construction, life cycle, and user costs.

NO BUILD ALTERNATIVE

The No Build Alternative serves as a benchmark for the other alternatives. Basic maintenance, such as the 5-year repairs listed in the August 2016 inspection report, is included.

The urgent repairs needed to keep the Frank J. Wood Bridge in place for the next few years are only a temporary solution. The structural steel will continue to deteriorate at an increasing pace unless a comprehensive repair and paint project takes place.

A No Build Alternative does not fulfill the purpose and need for this project.

REPLACEMENT ALTERNATIVES

Alternatives 1, 2, and 5 would provide a new bridge. Many characteristics of the new bridge would be the same for each of the replacement alternatives; these will be discussed below before the specifics of each alternative are presented.

A new bridge would be a multi-span steel girder bridge, with 4 or 5 spans. A steel girder bridge is considerably less expensive in Maine than alternative bridge types for this range of spans. To increase the life span of the new structure, the concrete deck would likely be reinforced with corrosion-resistant rebar and the steel girders would be metalized. Metallization of the girders will protect them from corrosion due to spray from the turbulent river beneath the bridge.

Figure 5: Artist's rendering of a steel girder bridge

The new bridge would have concrete wall abutments and solid shaft piers, all founded on the shallow bedrock at this site. New concrete bridge decks with high-performance membrane waterproofing and corrosion-resistant reinforcing bar are expected to last the service life of the new bridge or 100 years.

A replacement structure of this type will have low maintenance costs. The primary anticipated maintenance would be to mill and resurface the asphalt wearing surface at regular intervals and to paint the girders. Biannual inspections of a bridge of this type can be completed relatively quickly and at low cost.

Any new bridge will include 11 foot lanes, 5 foot shoulders, and 5 foot sidewalks on each side. Having sidewalks on both sides of the bridge will connect the existing sidewalks on the approaches and will improve safety by reducing the need for pedestrians to cross the road. On the Brunswick approach, the new east sidewalk will tie into the sidewalk that runs along the Town's 250th Anniversary Park. On the Topsham approach, the east sidewalk will continue with a crosswalk through the commercial entrance to the Sea Dog parking, and a new curb-cut will be constructed to access the sidewalk that runs along the dentist office. This will provide continuity of pedestrian passage on both sides of Route 201. Additionally, the MaineDOT will work with the Towns to determine crosswalk locations, needs, and enhancements that provide for safer passage across Route 201 than what exist currently.

5 foot wide shoulders with no adjacent bridge railing or truss verticals will improve the bridge for bicyclists. The available "riding" width will increase by 3 feet which will be enhanced further with the removal of the truss verticals. The verticals act as obstacles that tend to force bicyclists towards the travelway to avoid contact.

For new bridges on this site, the contractor would need a work trestle for access to construct the cofferdams and piers, to erect the structural steel superstructure, to place deck concrete, and to remove the existing bridge. A cost premium of $1 million is included in the estimate for each new bridge to account for the added expense of a work trestle on this challenging site. Installation of a work trestle at this site is unique due to the exposed and highly variable bedrock, exposure to high velocity flows, and proximity to the upstream dam.

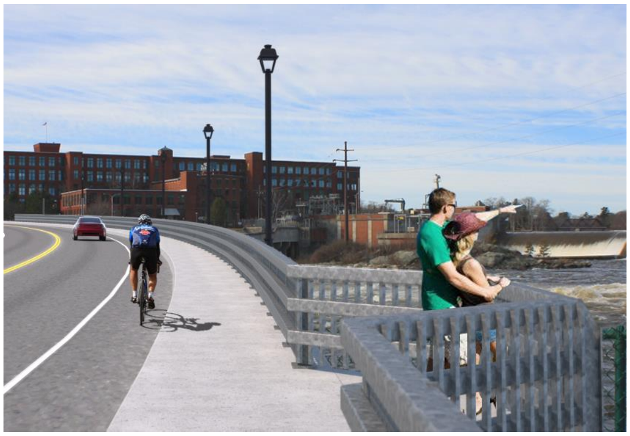


Figure 6: Rendering of a Possible New Bridge

Railings for a new bridge would meet all standards for vehicle and pedestrian safety. Railings go through stringent testing programs to ensure appropriate safety in a variety of situations. Only those railings that meet appropriate criteria can be used on a new bridge, based on the specific constraints of this site. MaineDOT's standard 4-bar steel pedestrian and traffic rail (which meets a TL-4 performance level) is recommended for this bridge, but input from the Towns of Brunswick and Topsham and the Section 106 Consulting Parties would be considered for the final selection of the rail type. A TL-2 performance level bridge rail system would meet standards for this site.

During meetings with Officials from both Towns, requests were made to enhance the “River Walk Loop” that exists over the existing bridge and continues to the pedestrian bridge upstream of the dam. A new bridge at this site would include deck overlooks, where the sidewalk widens out to provide viewpoints of the river upstream and downstream. In addition, the bridge would be lighted and lamp posts and fixtures would be ornamental and closely match the street lighting in the approaches. The MaineDOT would consider input from the Towns of Brunswick and Topsham and the Section 106 Consulting Parties for the final selection of the bridge lighting during final design.

Alternative 1: New 800 ft Bridge on Existing Alignment

Alternative 1 is a new 800 ft, five span, steel girder bridge on the existing alignment. The new bridge would have the characteristics discussed above that are similar for any replacement bridge on this site.

Because the new bridge would be constructed on the existing alignment, the existing truss bridge would have to be removed completely before new construction could begin. The limitations on in-water work add to the construction duration. Without a temporary bridge, this alternative would have a traffic disruption period of over 2 years.

Given the large user costs (see the Maintenance of Traffic Section) and other impacts such a disruption would cause, a temporary bridge is required for this alternative. This adds another year to the construction duration, bringing the total construction time to 3.5 years. This also increases the river impacts—this alternative would need a work trestle and a temporary bridge beyond the impacts of the new structure itself. Permanent environmental impacts would include the wetland footprint impact of 4 piers and riprap protected abutment slopes within the river channel. Two of the piers would be located near the edges of the Brunswick side powerhouse outfall channel.

The construction cost of this alternative is estimated at $16,000,000 (including the cost of a temporary bridge). Refer to Appendix H for detailed cost estimates.

Alternative 1 Summary:

- New 800 ft bridge on the existing alignment
- 11 ft travel lanes with 5 ft shoulders and 5 ft sidewalks each side
- Construction Cost: $16 million
- Life Cycle Cost: $16.7 million
- Service Life Cost: $20.3 million
- Construction Duration: approximately 3.5 years
- Maintenance of Traffic: on-site temporary detour
- In-Water Impacts: temporary work trestle, temporary bridge, new piers, new slopes at abutments
- Right-of-Way Impacts: minimal
- Utility Impacts: existing utilities relocated to new bridge
- Historic Impacts: existing truss bridge removed
- Brookfield Dam and Brunswick fish way: no permanent effects
- Meets Purpose and Need

Alternative 2: New 835 ft Bridge on Curved Upstream Alignment

Alternative 2 is a new 835 ft, five span, steel girder bridge on a curved upstream alignment. A curved bridge reduces the length of approach roadway construction and reduces right of way impacts to abutting properties when compared to using a straight bridge in the same upstream location. This bridge alternative would have a short southern span to better align the spans to bridge the Brookfield power station outflow channel with a minimum of impact. The remaining four spans would be continuous haunched steel girder spans with a concrete deck. The span arrangement and number of piers would be selected to minimize footprint impact within the channel and within the FERC Boundary and to maximize the efficiency of steel girder superstructure. In addition, the existing hydraulic clearance over the river would be maintained as a minimum. To avoid an unacceptable rise in flood elevations along the Topsham bank of the river adjacent to the Bowdoin Mill area, Pier 3 would be skewed at 35 degrees to better align with the flow and the North Pier of the existing bridge would be retained.

The estimated construction duration for this alternative is approximately 2.5 years. No temporary bridge is required since traffic could be maintained on the existing bridge during construction. A short term (about 2 month) single lane northbound road closure and detour as described in the “Maintenance of Traffic” section for the New Alignment maintenance of traffic option would be needed during the final tie-in.

Figure 7: A Possible Curved Upstream Bridge

The four piers and the abutment slopes would be permanent wetland environmental impacts. Two of the piers would be located near the edges of the Brunswick side powerhouse outfall channel. Temporary environmental impacts would include the construction of a work trestle from the Topsham bank of the river out to the proposed Pier 2 location.

The construction cost of this alternative is estimated to be $13,000,000.

The life cycle construction cost of this alternative (Alternative 2 – Replacement Bridge on Curved Upstream Alignment) is estimated to be $13,700,000. The life cycle cost includes costs for future inspection and maintenance (painting and wearing surface replacement) anticipated to be needed out to 100 years. Refer to Appendix H for detailed cost estimates.

Alternative 2 Summary:

- 835 ft replacement bridge on a curved, upstream alignment
- 11 ft travel lanes with 5 ft shoulders and 5 ft sidewalks each side
- Construction Cost: $13 million
- Life Cycle Cost: $13.7 million
- Service Life Cost: $17.3 million
- Construction Duration: approximately 2.5 years
- Maintenance of Traffic: on existing bridge
- In-Water Impacts: temporary work trestle, new piers, new slopes at abutments
- Right-of-Way Impacts: impacts to 4 properties
- Utility Impacts: existing utilities relocated to new bridge
- Historic Impacts: existing truss bridge removed
- Brookfield Dam and Brunswick fish way: potential effects to be determined
- Meets Purpose and Need

Alternative 5: New 800 ft Bridge on Parallel Downstream Alignment

Alternative 5 is listed here, since like Alternatives 1 and 2 it is a new bridge. It would be a new 800 ft, five span steel girder bridge located downstream of the existing bridge on a straight alignment, between the current bridge and the Bowdoin Mill Complex parking lot. For all of the bridge alternatives, a hydraulic analysis was run to estimate how the river would behave with new piers added in the river. This analysis showed that a downstream replacement bridge will raise water levels at the Bowdoin Mill Complex, particularly the end of the mill building where the Sea Dog Brewing Company is located. The models suggested that during the design flood, floodwaters would rise more than 6 feet higher than existing conditions near the deck area of the Sea Dog. No reasonable approach to reduce that water rise could be found, so Alternate 5 was rejected.

REHABILITATION ALTERNATIVES

Alternative 3 and Alternative 4 are both rehabilitation options, where the existing truss bridge is repaired. Detailed inspections of the bridge were done by MaineDOT in 2012, June 2016 and August 2016, and a load rating was done by MaineDOT in 2013 and updated in August 2016. These reports outline what needs to be done to bring the existing truss bridge up to the standards established as the "Purpose & Need" for this project, which were described above.

These repair needs will be described here, and the differences between the two rehabilitation alternatives will be discussed later. The needs are:


Figure 8: Deteriorated crossbeams & deck

1. Replace the existing bridge deck (including crossbeams) with a new reinforced concrete bridge deck. The existing concrete-filled steel grid deck is in poor condition and the supporting transverse crossbeams are badly deteriorated (See Figure 8).
2. Repair the top of steel sidewalk support brackets. The top of each bracket is non-existent now due to corrosion or other past modifications. This requires replacing the sidewalk concrete deck as well.
3. Replace the bridge joints. Although these were replaced in 2015, replacement of the existing deck will require these to be replaced.
4. Replace the entire steel floor system, including the longitudinal stringer beams and transverse floor beams. The new floor system would be composite with the new deck. The floor system is heavily deteriorated and is below load carrying standards (see Figures 9 and 10).


Figure 9: Hole in floor beam


Figure 10: Deteriorated floor beam

5. Replace the bottom flange angles of the bottom chord of the main trusses due to corrosion and distortion from pack rust, as seen in Figure 11. It is anticipated this work could be done one angle at a time without shoring while the deck is off the bridge.

6. Replace the lattice plates of the bottom chord, which are severely bowed due to pack rust. See Figure 12.

7. Remove the welded steel plates attached to truss vertical members. These are fatigue sensitive details on fracture critical members. Remediate these locations using cover plates. See Figure 13.

Figure 11: Bottom chord bottom flange corrosion

Figure 13: Bottom chord lattice plate bowing

Figure 13: Plate welded to truss vertical

8. Paint the entire steel superstructure, including all above and below deck components. Doing a comprehensive paint job on this structure is expected to cost about $4,000,000.

9. Replace all existing utility brackets that support the conduit and water lines on the bridge. See Figure 14.

10. Remove and reuse the existing pedestrian sidewalk rail and bridge traffic rails. They will have to be removed to replace the deck and floor system. The traffic rail on the sidewalk side meets current standards, while the rail on the other side does not. However, that rail is considered acceptable on this structure and adding a new traffic rail would reduce the travelway width further.

11. Replace the abutment back walls due to the overall poor condition of these elements.

Figure 14: Utility brackets

12. Repair areas of stone masonry with missing and loose stones at the south abutment by encasing the masonry in concrete. See Figure 15.

13. Replace cracked concrete bearing pedestals at Pier 2 supporting the east side truss of Span 3 near the Topsham end of the bridge. This work will also include removal, refurbishing, and resetting of the truss bearing at this support. See Figure 16.

Figure 15: Abutment masonry

Figure 16: Damaged concrete pedestals

Once all of the listed repairs are completed, the structure will meet all current design strength requirements. All repairs would be completed using modern design standards and construction practices to help them last as long as possible.

The existing bridge deck is a lightweight, concrete-filled steel grid deck. To keep from adding more weight to the truss, a new bare concrete bridge deck without a paved surface will be required (additional research may show a conventional paved deck would be acceptable). To improve durability of the new deck, it would be reinforced with corrosion-resistant rebar. A drainage system that discharges below the bottom chord of the truss would be added to limit moisture and salt on the floor system and lower parts of the truss. The existing deck has open drainage which lets debris, salt and water from the roadway above drop right onto the steel.

The existing 30 ft available travelway matches the existing approaches and would provide two 11 ft travel lanes with 4 ft shoulders bound by rails located along the inside of the trusses. Using 10 ft travel lanes with 5 ft shoulders was considered but is not recommended. The Department considers 10 ft travel lanes as less safe than 11 ft lanes given the high traffic volume, almost 19,000 vehicles per day that this bridge has.

A full road closure is needed to complete all major structural steel rehabilitation activities except painting. The construction and traffic disruption duration for this alternative without a temporary bridge would be approximately 20 months. User costs (see the Maintenance of Traffic section) and other impacts indicate a temporary bridge is needed for this alternative. When the temporary bridge is added in, construction duration for this alternative is approximately 3 years. The bridge would also be painted while the temporary bridge is in place.

Rehabilitating the existing bridge would preserve the existing river flow conditions. It would also have No Adverse Effect to the three Historic Districts. However, construction of a temporary bridge will still have temporary environmental impacts. Utilities on the bridge will have to be temporarily relocated on the bridge during the rehab process.

A bridge rehabilitation will require significant future maintenance. To get 75 more years of life, the bridge will need approximately 3 future paintings, 1 deck replacement, and 2 substructure rehabilitations, beyond the current project. All of these activities will disrupt traffic to varying degrees. Painting will disrupt traffic for approximately 8 months, and the deck replacement will disrupt traffic for approximately 6 months.

Based on past performance of the modern paint systems used by MaineDOT on similar truss bridges, the structural steel will need to be painted approximately every 20 years. The current paint systems used today perform very well, replacing the previous lead-based paint systems. The estimated current cost for painting this bridge is $4,000,000.

Built-up members (where multiple pieces of steel are riveted together to make a larger member) present a special challenge for paint systems. Over time, pack rust develops in the crevices between pieces of steel and gradually expands, pushing the plates apart (See Figure 17). It is impossible to remove this corrosion using the normal cleaning processes used when painting the steel. The best way to remove pack rust is to disassemble the member, clean the pieces, and then reassemble. That approach is time-consuming and expensive.

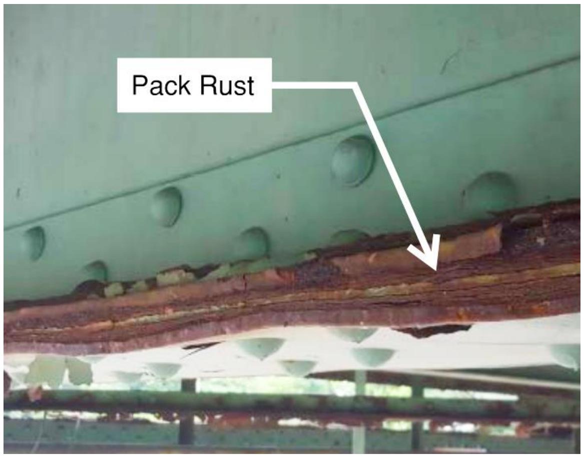


Figure 17: Pack rust is corrosion in the crevice between two plates of steel that are bolted or riveted together. As the rust progresses, it gradually pushes the pieces of steel apart, bending them and sometimes breaking bolts or rivets. To truly fix pack rust, the members need to be taken apart and thoroughly cleaned, which is a complex and expensive effort.

Alternative ways of removing pack rust have been tried, but none have been fully validated. One experimental methodology, based on heating the area and hammering the buckled section to drive out the rust, was investigated further. Mr. Vern Mesler, who developed the method, was consulted. He noted that more research was needed on the effects of this method on the steel itself. It is possible that it will make the steel brittle, a serious concern for fracture-critical members, but one that has not been studied.

On the Frank J. Wood Bridge, the rehabilitation options would remove or replace all of the members that have developed significant pack rust so far. However, many built-up members will remain on the bridge that are susceptible to pack rust. Many of those likely already have some level of initiated pack rust. New paint spans the seams of the built-up steel members and prevents water and air from getting to the steel. However, once the paint cracks at all, existing pack rust will reactivate. To slow the advance of pack rust and other corrosion, future paint jobs will have to be budgeted for and done on a regular cycle of approximately 20 years.

Use of corrosion-resistant reinforcement would extend the life of a bare concrete deck, but without a high performance membrane and paved wearing surface that can be regularly replaced, 50 years of life is a good estimate. Based on the historic performance of similar aged bridges (currently 85 years old) and the age of the most recent major substructure rehabilitation (2006), additional substructure rehabilitations would be expected at years 20 and 50 following this current project.

Besides these major future maintenance efforts, there will be more frequent smaller repair efforts needed on the steel, bridge joints, and the aging substructure. This bridge will also require Fracture Critical Bridge Inspections, including fatigue detail inspections, costing about $60,000 every two years. These inspections will also disrupt traffic, requiring a single lane closure for 1 to 2 weeks. If cracks in fatigue sensitive or fracture critical members are found in these inspections, more frequent inspections and immediate repairs would be required. A conservative value of $40,000 a year to repair fatigue cracks was used in the life cycle and service life cost estimates. Refer to Appendix H for more detailed cost estimates.

Alternative 3: Rehabilitation of Existing Steel Truss Bridge:

Alternative 3 would rehabilitate the existing truss bridge as outlined above. It would still have only one sidewalk, so pedestrian mobility and safety would not be improved. The open grid decking along the outside of the existing shoulders would be replaced with a solid concrete deck, improving the situation for bicyclists (though not fully to current standards). This alternative meets Purpose and Need for this project but does not address the pedestrian mobility and safety concerns.

The construction cost of this alternative is estimated at $15,000,000. This cost includes a 15 percent contingency above the repair work that has already been identified. Rehabilitation projects nearly always discover issues not previously found in inspections, causing budget overruns.

The overall life cycle construction cost of this alternative, including estimates for all future maintenance on the bridge out to 75 years of life, is projected to be $20,800,000.

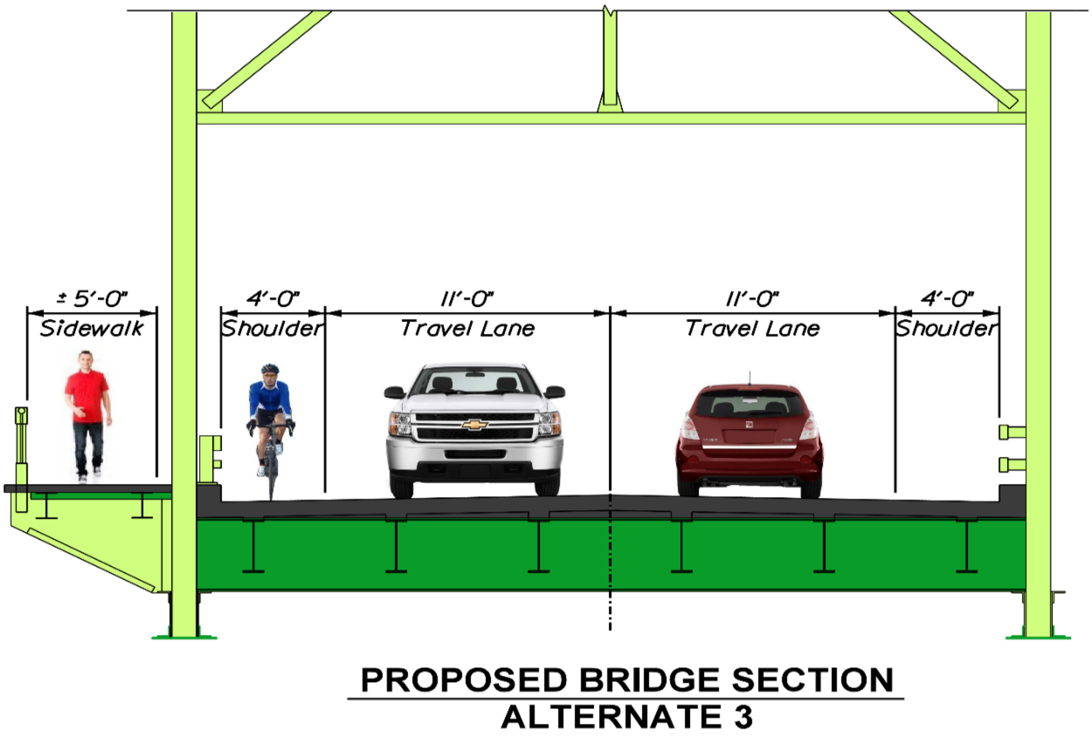


Figure 18: Alternative 3 cross section

Early in the investigation of alternatives at this site prior to the August 2016 inspection, this alternative was examined as a 30 year rehabilitation and either maintaining one lane of traffic on the bridge or allowing a 5 to 7 month bridge closure. The initial estimate for this improvement was $8 million, which was less expensive than the least-cost replacement alternative and potentially a cost-effective improvement solution if its life-cycle cost was competitive. It was anticipated that a deck replacement with minor steel rehabilitation with a full painting now would yield a bridge that would not need significant capital improvements for 30 years. A complete replacement would then be needed. A replacement after 30 years would yield the lowest life cycle cost of any rehabilitation alternative because expensive capital improvements such as repainting and substructure repairs would be avoided. Also, 45 years of costly inspection and maintenance would be avoided. However, the August 2016 inspection recommended a complete floor system replacement with extensive repairs to the bottom chords of the truss. Maintaining one lane of traffic would not be possible. Given changes to the rehabilitation scope and the associated user costs for maintenance of traffic (see Maintenance of Traffic Section), the initial cost of this alternative now includes a temporary bridge and a full floor system replacement. The originally estimated construction cost of $8 million to rehabilitate the bridge now is $15 million after adding a full floor system replacement and an on-site temporary bridge detour. Refer to Appendix H for detailed cost estimates.

Summary of Alternative 3:

- Rehabilitation of existing steel truss bridge
- 11 ft travel lanes with 4 ft shoulders each side and a 5 ft sidewalk on the west side
- Construction Cost: $15 million
- Life Cycle Cost: $20.8 million
- Service Life Cost: $35.2 million
- Construction Duration: approximately 3 years
- Maintenance of Traffic: on-site temporary detour
- In-Water Impacts: temporary bridge, Abutment 1 repair work
- Right-of-Way Impacts: minimal
- Utility Impacts: temporary support or relocations
- Historic Impacts: none
- Brookfield Dam and Brunswick fish way: no permanent effects
- Meets Purpose and Need (but does not address pedestrian mobility and safety)

Alternative 4: Rehabilitation of Existing Steel Truss Bridge with Added East Sidewalk

Alternative 4 is also a rehabilitation of the existing truss bridge, but with a second 5 foot sidewalk added on the opposite side of the bridge. This fully addresses the pedestrian issues at this site. Like Alternative 3, bicyclists would have 4 foot shoulders with adjacent traffic rails—not ideal, but better than the current condition for bicyclists. Alternative 4 adequately meets the Purpose and Need for this project.

To maintain the existing loading on the trusses while adding a new second sidewalk, weight will need to be taken off the truss elsewhere. Various lightweight concrete deck systems such as lightweight concrete, sandwich steel plate systems, and composite deck systems were considered, but a new lightweight concrete-filled Exodermic bridge deck would be recommended for this alternate. An Exodermic deck system can be as much as fifty percent lighter than a conventional concrete deck of the same span, is more durable than a lightweight concrete deck, and is more cost-effective than other lightweight systems. This alternative includes the addition of new structural steel framing, concrete deck, and pedestrian rail for the added 5 ft wide sidewalk on the east side of the bridge. Between the more expensive deck and the new sidewalk and framing, this option will have a construction cost about $2,000,000 more than Alternative 3.

The estimated construction duration for this alternative is approximately 3 years (similar to Alternative 3).

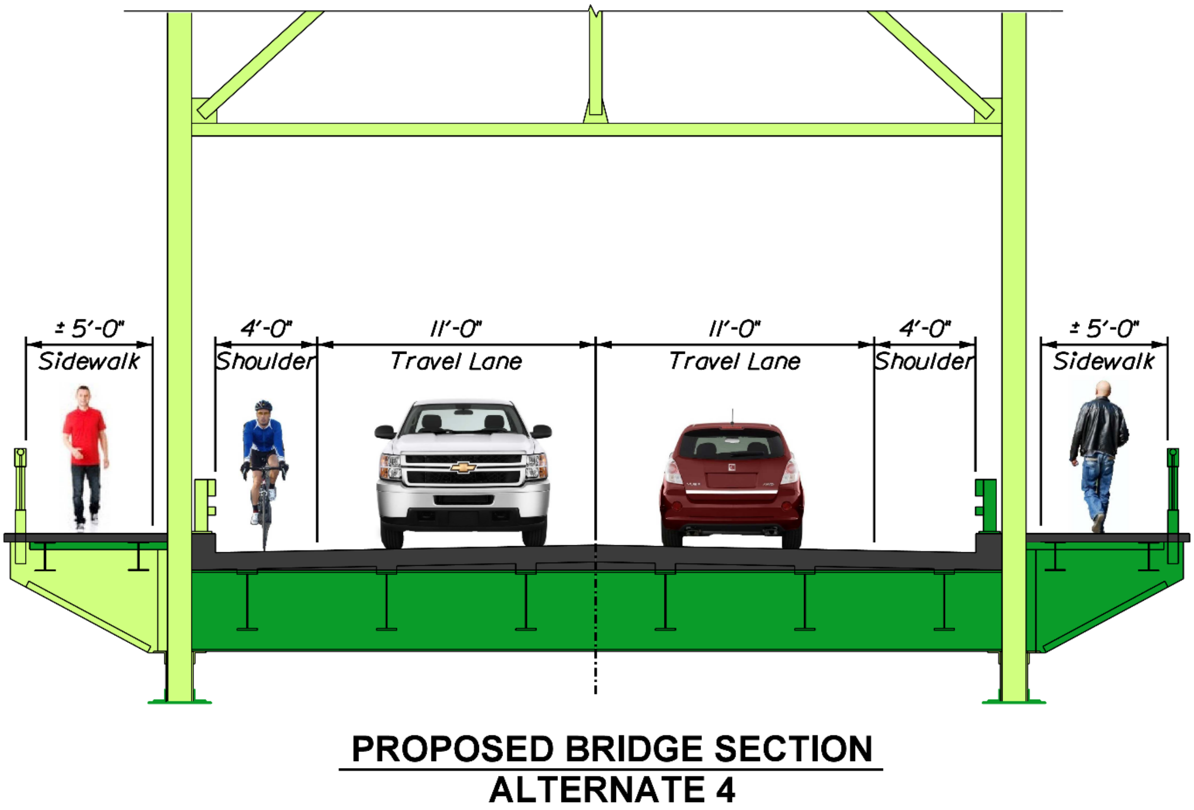


Figure 19: Alternative 4 cross section

The construction cost of this alternative is estimated at $17,000,000. The life cycle cost of this alternative, including estimates for all future maintenance on the bridge out to 75 years of life, is estimated to be $23,200,000. Refer to Appendix H for detailed cost estimates.

Summary of Alternative 4:

- Rehabilitation of existing steel truss bridge with added east side sidewalk
- 11 ft travel lanes with 4 ft shoulders and 5 ft sidewalks each side
- Construction Cost: $17 million
- Life Cycle Cost: $23.2 million
- Service Life Cost: $38.2 million
- Construction Duration: approximately 3 years
- Maintenance of Traffic: on-site temporary detour
- In-Water Impacts: temporary bridge, Abutment 1 repair work
- Right-of-Way Impacts: minimal

- Utility Impacts: temporary support or relocations
- Historic Impacts: minimal
- Brookfield Dam and Brunswick fish way: no permanent effects
- Meets Purpose and Need

Repurpose Existing Bridge and Build a New Replacement Bridge

An additional alternative suggested by the public was to '*Restore and repurpose the historic bridge for pedestrian and bicycle use, and as a public historic park. Build a new bridge on alternative alignment.'* This is a combination of two alternatives discussed above, Alternatives 2 and 3. All work to preserve the existing bridge under Alternative 3 would still be required, except possibly rehabilitating the sidewalk. Conservatively, the construction cost of this rehabilitation could be reduced to $9.5 million (with the removal of the sidewalk), and there would be no need for a temporary bridge. This alternative would also require the cost of a new replacement bridge, Alternative 2, at $13 million, for a total construction cost of $22.5 million. The question of future ownership and maintenance responsibility for the bridge would have to be addressed. Also, the effect on river water levels from having more piers permanently in the river channel would need investigation.

MAINTENANCE OF TRAFFIC

Four options were investigated to maintain traffic at this site during construction. They are not all feasible for all of the bridge improvement alternatives. Specifics for each alternative, along with estimated traffic disruption durations and user costs, were discussed with the each alternative.

1. <u>Complete road closure with a detour</u>. Detour all traffic along U.S. Route 1, State Route 196. The total detour distance is approximately 2.5 miles for through traffic and 3.7 miles end to end (see Figure 20).
2. <u>Single lane closure with staged construction</u>. One way, southbound traffic will be carried across the bridge on a 12-foot travelway and all northbound traffic will be detoured. This option can only work for certain construction activities, like painting. This traffic control method has been used successfully in the past on the Frank J. Wood Bridge.

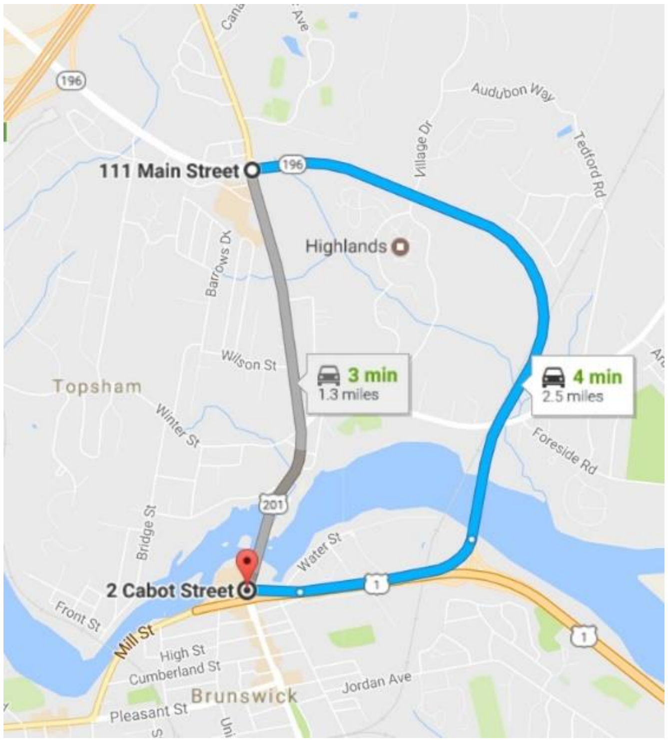


Figure 20: Traffic detour

3. <u>On-site detour on temporary bridge</u>. Construct a 2 lane temporary bridge parallel to the existing bridge and

detour all traffic onto it. Traffic would only be disrupted during the construction of tie-ins to the existing roadway and to the new roadway upon conclusion of the project. These disruptions could be limited by requiring work be done during off-peak hours. Construction and removal of the temporary bridge would likely extend the total construction duration by about 1½ years (1 construction season for construction of the temporary bridge and half a season for its removal). The cost for a temporary bridge is estimated to be about $4 million.

4. New alignment. If a new bridge is constructed on a new alignment, the existing bridge could be used to maintain traffic during construction. Traffic would primarily be disrupted during construction of the final tie-in. Again, this could be mitigated by requiring work during off-peak hours. This option would result in the least traffic disruption.

Staged construction maintaining two-way traffic is not feasible due to the existing structure type and needed rehabilitation repairs. Alternating one-way traffic is not feasible because of the traffic volume and proximity of signalized intersections.

As mentioned earlier in this report, traffic disruption results in indirect costs to the users of the bridge and to the surrounding businesses. A way to quantify the cost of delays to the traveling public is to develop "user costs." The average delay for vehicles is estimated and a fixed cost per hour is applied. The average delay was estimated at between 3 and 4 minutes, with delays at peak times higher and at off times lower. Based on these delays, the added length of the detour, and the number of vehicles traveling, the user cost for a complete road closure is estimated at approximately $22,000 per day and the user cost for a northbound lane closure is estimated at approximately $10,000 per day. The indirect costs to the surrounding businesses, which are not easily quantified, would add to those costs of traffic disruption further. These costs may then be compared to the costs of temporary bridges or other methods of mitigating traffic disruption. Refer to Appendix F for user costs information.

UTILITIES

A hydropower dam operated by Brookfield Renewable Energy Partners (Brookfield) is located about 500 ft upstream of the existing bridge crossing. No impacts to the power generation facility are anticipated for any of the bridge improvement alternatives investigated.

Brookfield operates a fish way at the dam. No direct impact to the fish way is proposed. An assessment of potential indirect effects to the fish way and options to avoid and minimize effects is underway.

Overhead utilities and a water main are carried by the existing bridge. Temporary support or relocation of these facilities within the limits of the existing bridge would be needed during a bridge rehabilitation.

With a bridge replacement, these facilities would need to be relocated. Some of the utility poles in the approaches would also need to be relocated. The overhead utilities would need to transition to underground in the approaches close to the replacement bridge ends. The overhead utilities and the waterline would be carried on the bridge below the bridge deck, between girders, out of sight.

RIGHT OF WAY

A bridge rehabilitation or bridge replacement on the existing alignment would not require permanent property impacts. However, temporary property rights would be needed for any temporary bridge.

Construction of a replacement bridge on a new upstream alignment would require permanent property acquisitions of parts of two properties on the west side of the south approach and one property on each side of the north approach. The south approach property impacts would include reconstruction of a retaining wall between the drive entrances to the small Fort Andross parking lot and the Brookfield hydroelectric station at the dam. The 250th Anniversary Park located at the southeast corner of the bridge is a Brunswick town park constructed on land leased from Brookfield. At this location, permanent structures and fill slopes would be within the existing State-owned right-of-way. The north approach would have a new 130-ft-long retaining wall along the northwest approach to limit impacts to the property and parking area. Reconstruction of the drive entrance to the Bowdoin Mill complex will require impacts beyond the existing MaineDOT right of way.

Temporary property rights would be needed to construct work access platforms like work trestles. These rights would be similar to temporary rights needed for a temporary bridge.

Additionally, for an upstream bridge replacement alternative, the abutments and three of the four bridge piers would be located within the limits of the Federal Energy Regulatory Commission (FERC) Boundary of the dam. Temporary property rights would be needed for construction access along the north side of the approaches and within the FERC Boundary.

ENVIRONMENTAL

Protected species such as the shortnose sturgeon, Atlantic sturgeon, and Atlantic salmon are present in the project area, and all alternatives will likely adversely affect them. A formal Section 7 consultation with the National Marine Fisheries Service will be required. This project is within Essential Fish Habitat and permanent and temporary impacts need to be avoided or minimized. In-water work must be avoided during crucial migrating periods. This restriction is in place from April 7 to August 30, and will be a significant constraint on construction durations.

Any impacts to the Brunswick fish way at the Brookfield dam will be carefully considered.

The bridge is not individually eligible for listing on the National Register of Historic Places, but is a contributing element to the Brunswick-Topsham Historic District. It is also abutting the National Register-Listed Pejepscot Paper Company and the National Register-Eligible Cabot Mill. Removal of the bridge would be an adverse effect to those protected resources.

If a temporary bridge is used to maintain traffic for either a bridge rehabilitation or bridge replacement, then temporary environmental impacts would be needed within the existing river channel to support the temporary bridge.

Figure 21: Two types of temporary impacts

Construction of a new replacement bridge would have environmental impacts that would need to be minimized or mitigated. Permanent impacts would include the piers and pier foundations within the channel. Foundation locations should avoid the Brunswick side powerhouse outfall river channel that leads to the dam fish way by taking advantage of ledge outcrops where possible. Also, if a temporary work trestle is needed for the construction of a new replacement bridge, temporary environmental impacts would need to be addressed.

Impact avoidance and minimization strategies will be determined through the ongoing Section 106, 4(f) and NEPA processes. FHWA and MaineDOT are preparing an Environmental Assessment that will discuss Environmental Impacts in more detail.

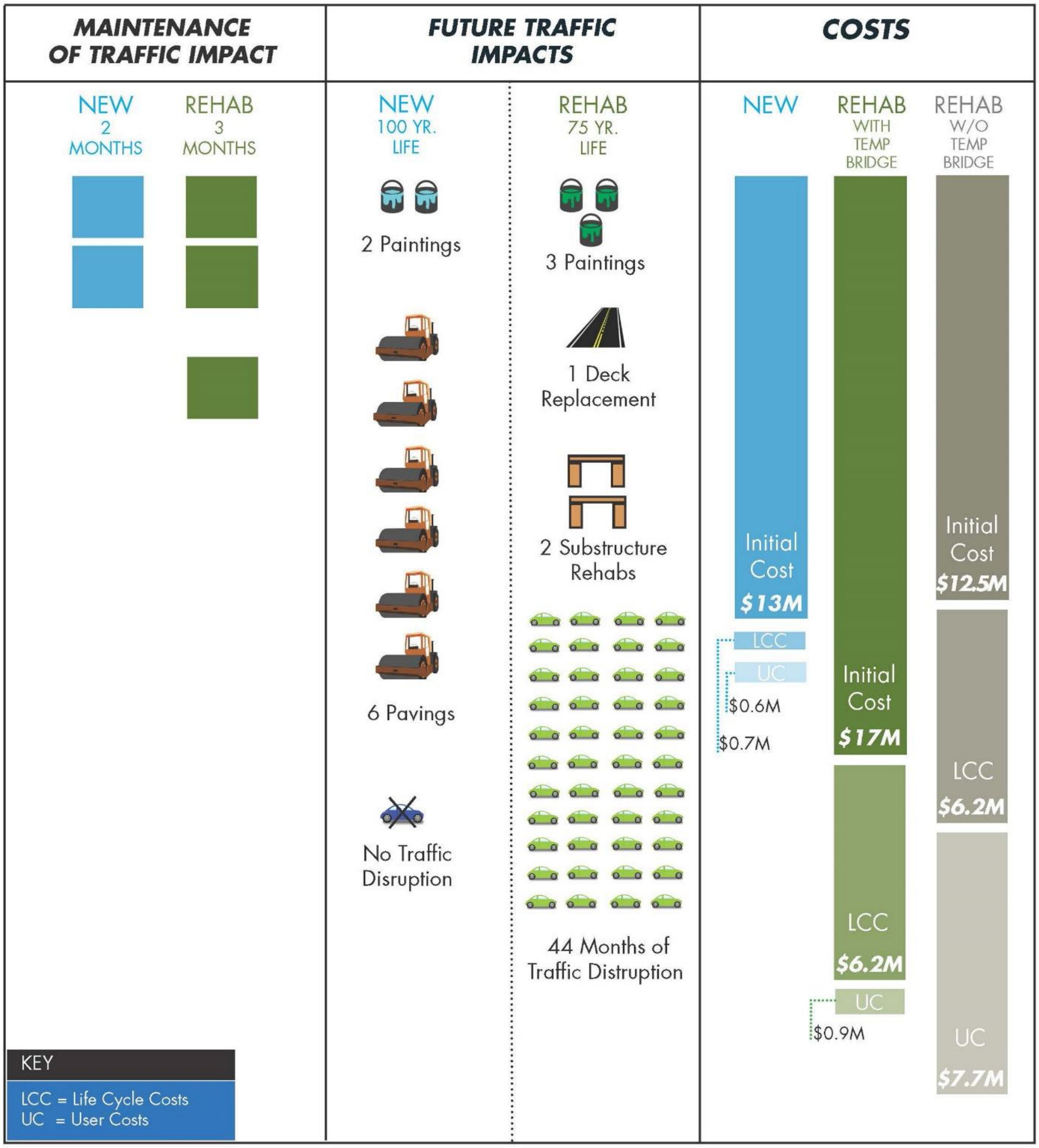
LIFE CYCLE COST and SERVICE LIFE COST

Life cycle costs are considered in the comparison of bridge improvement alternatives. A life cycle cost estimate (LCCE) totals all estimated bridge costs throughout the life of each bridge improvement alternative and translates them to current dollar equivalents. The LCCE accounts for estimated construction cost on the current project and the translated present value of anticipated future inspection, maintenance, and rehabilitation. It also accounts for anticipated future bridge replacement dates for each alternative.

Service life costs are also used when comparing bridge improvement alternatives. A service life cost estimate (SLCE) similarly totals all the estimated bridge costs throughout the life of each alternative but does not translate or discount these costs to current dollar equivalents. The SLCE is a running total of initial construction cost and all anticipated future inspection, maintenance, and rehabilitation costs associated with each alternative.

GRAPHIC COMPARISON

The graphic below compares Alternative 2 (the low cost replacement or new option) and Alternative 4 (the rehab option with two sidewalks). Three main areas are contrasted: maintenance of traffic during construction, future rehabilitation and maintenance, and costs (initial costs and life cycle costs). Refer to Appendix H for detailed life cycle cost estimates.



PROPOSED ALTERNATIVE

The recommended alternative for improvements to the Frank J. Wood Bridge is a modified version of Alternative 2, a replacement structure on a curved alignment upstream of the existing bridge. The modification is to the length of the bridge, number of piers, and to the southerly span arrangement. This recommended alternative provides:

Less Traffic Disruption. The upstream replacement alternative allows traffic during construction to stay on the existing bridge. The other viable alternatives require a temporary detour bridge. Traffic will also be less affected during future inspections and maintenance of this alternative.

Improved bicycle mobility. This alternative provides wider shoulders for bicycle passage with no adjacent vertical restrictions. Multiple comments were received indicating some people will not bicycle across the existing truss due to safety concerns regarding widths. The Bicycle Coalition of Maine recommends a replacement bridge with 5 ft shoulders to better accommodate and encourage bicycle travel.

Improved pedestrian mobility. This alternative provides sidewalks on both sides of the roadway, connecting the approach sidewalks. It also eliminates the restrictions of the existing truss that made maintenance difficult in the winter.

Minor Right of way Impacts. Right of way impacts, though higher than those of a rehabilitated alternative, are considered minor with no relocations required.

Minimal Impacts to dam operation and fish way. An upstream curved replacement alternative has no measureable effects on river hydraulics to affect dam operation. The effects of moving the bridge closer to the fish way will be further reviewed through final design. Shadow effects have been examined and reflect a slight increase from the existing conditions.

Effects to Section 106 Resources. The Section 106 process determined that the upstream replacement alternative would have adverse effects to three historic resources: the Cabot Mill, Pejepscot Paper Company, and the Brunswick Topsham Historic District resulting from the removal of the Frank J. Wood Bridge. The bridge is the last element of the setting of the two mills that was constructed during the period of significance of the mills. Removal of the Frank J. Wood Bridge will diminish the Cabot Mill's and the Pejepscot Paper Company's integrity of setting, feeling, and association. Section 106 requires mitigation of adverse effects if they cannot be avoided. Mitigation will be finalized with input from Section 106 consulting parties as design of the proposed alternative proceeds.

Impacts to Section 4(f) Properties. Adverse Effects to historic transportation structures under Section 106 of the National Historic Preservation Act are considered a "use" under Section 4(f) of the U.S. Department of Transportation Act of 1966. The upstream alternative will result in a use of Section 4(f) properties. Selection of the upstream alternative is the result of many months of analysis, public comment, and review of a variety of rehabilitation and replacement options. In determining the preferred alternative, FHWA and MaineDOT considered the environmental, cultural, social, economic impacts, and transportation needs (i.e., vehicular, bicycle and pedestrian), in addition to considering the engineering, cost, constructability, traffic, utilities, maintenance, and public input. Final evaluation of impacts to Section 4(f) resources and approval of the use will be completed by FHWA.

Lowest cost. The upstream replacement alternative has the lowest initial cost and significantly lower long-term cost than a rehabilitation. The proposed alternative is projected to cost half as much as rehabilitation options over the life of the alternatives.

Lower risk. This alternative has much lower long-term risk of unanticipated costs or issues with the bridge than a rehabilitation of the existing truss bridge.

Easy maintenance and inspection. The replacement alternative provides a new structure that is much less complex than the existing bridge, is made with modern materials, and has no major ongoing maintenance risks. It will require significantly less effort to maintain and inspect throughout its life.

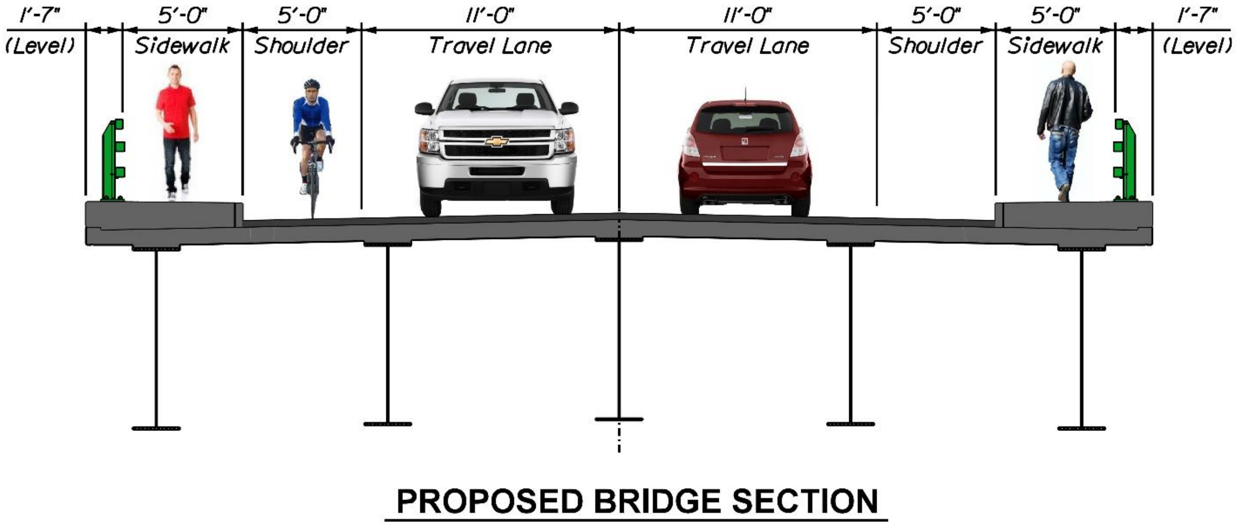


Figure 22: Proposed Alternative Cross Section

Final cross section and aesthetic details will be developed through collaboration with the Towns' Design Advisory Committee (DAC) and the Section 106 Consulting Parties.

A request by Brookfield to move the originally proposed Pier 1 as far away from the fish way as possible prompted a reexamination of the span arrangements on the Brunswick end of the bridge. Two additional span arrangements were investigated to see how moving or eliminating Pier 1 might affect the water flow near the fish way and through the main channel. Span arrangement 1 increased the first span by 35 ft by moving Pier 1 to the north. Span arrangement 2 eliminated Pier 1, but moved Abutment 1 20 ft north reducing the overall bridge length. This modification eliminates Pier 1 and increases the length of the first span to 260 ft.

Both these span arrangements had similar river flow characteristics to the existing bridge and the Alternative 2 described earlier.

The cost estimates for these additional span arrangements is not expected to change measurably from Alternative 2. Span arrangement 1 is essentially the same as Alternative 2, with a longer first span but an equally shorter second span. Span arrangement 2 has a considerably longer first continuous span, which will increase the girder size and weight and require a longer temporary trestle to aid in its construction, but it will also eliminate one pier and shorten the overall bridge length by 20 ft. The preferred option is span arrangement 2, an 815 ft, four-span continuous bridge.

The recommended base alternative is as follows:

- 815 ft, four-span continuous steel girder bridge on a curved, upstream alignment, 260 ft-205 ft-205 ft-145 ft.
- The superstructure will be metalized steel girders supporting an 8½ inch thick composite concrete deck using corrosion-resistant reinforcement.
- The bridge cross-section will have two 11 ft lanes with 5 ft shoulders and sidewalks on each side. The total bridge width is 45 ft-2 in, including an appropriate railing on each side.
- A 3 inch bituminous wearing surface with a high performance membrane waterproofing will add additional protection to the concrete deck.
- The Department will work with the DAC and the Section 106 Consulting Parties to develop details to be utilized in the new bridge that support the historic setting.
- Corrosion resistant, bicycle friendly box drains are recommended to protect further the concrete deck and to enhance bicycle use through this corridor.
- A five-foot-wide by ten-foot-long overlook is anticipated on each side of the bridge. The exact size and location of these overlooks will be determined in final design. The Department will be considering input from the DAC and the Section 106 Consulting Parties on this.
- It is anticipated that the bridge will be lighted using light fixtures that closely match existing street lighting on the approaches. The Department will work closely with the DAC and the Section 106 Consulting Parties on this feature as well.
- The abutments will be cantilevered reinforced concrete wall structures supported directly on shallow bedrock.
- The piers will be reinforced concrete solid shaft piers supported on concrete seals to bedrock.

HYDROLOGY REPORT

Design flows for the Q1.1, Q2, Q10, Q25, Q50, Q100 and Q500 discharges were provided by the Hydraulics Section of the Maine DOT. The discharge rates recommended for design were developed using the USGS program PeakFQ and USGS gage Systematic Record data. The bridge site estimates were calculated from the gage estimates by area scaling using the form of the USGS regression equation.

The Flood of Record on the Androscoggin River occurred in March 1936, and the discharge for this event is estimated to be 143,000 ft^3 / s (from historical record paper “The Floods of March 1936, Part 1. New England Rivers” by the U.S. Department of Interior, Geological Survey).

The entry distribution of the discharge into the 2-D hydraulic model is based on historic dam flow distribution and dam operation data provided by the upstream dam operator (Brookfield Renewable Energy). Distributions account for discharge through the powerhouse (located near the Brunswick side of the channel), through the tainter gates (located near the Topsham side of the channel), and over the spillway (spanning between the powerhouse and the tainter gates).

Refer to Appendix E for hydrology data and computations provided by MaineDOT’s Hydraulics Section.

SUMMARY

Drainage Area	3435	mi^2
Q1.1	27,486	ft^3 / s
Q2	41,755	ft^3 / s
Q10	66,203	ft^3 / s
Q25	79,255	ft^3 / s
Q50	89,321	ft^3 / s
Q100	99,671	ft^3 / s

Reported by: Charles Hebson
Date: February 24, 2014

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

HYDRAULIC REPORT

A standard 1D hydraulic modeling approach is not appropriate for the Frank J. Wood Bridge, and would not provide adequate water surface and discharge velocity results. A standard 1D model only accounts for variation in water surface and velocity in one direction (along the stream). A 2D hydraulic model will capture the significant variations in water surface elevation and velocity distribution in two directions (along the stream and across the streams) known to exist at this site. The upstream split flow conditions at the dam, highly varying topography of the river channel, and highly varying horizontal limits of the river channel banks require the site to be modeled with a 2D hydraulic analysis method to obtain reasonable results that capture the variability in water surface elevations and velocity distributions within the river channel. The flow through the reach of interest is known to be turbulent and highly variable, and 2D modeling will capture these influences and provide an appropriate water surface that can be used for hydraulic evaluations and design of the existing and proposed bridge conditions.

The water surface elevations were developed for steady flow at the peak discharge for the following design flows using SRH-2D (Sedimentation and River Hydraulics – Two Dimensional) modeling software, developed by the U.S. Bureau of Reclamation, and SMS (Surface-water Modeling System) software available from Aquaveo:

Q50, design discharge used to evaluate hydraulic clearance

Q100, check discharge used to evaluate hydraulic clearance and scour

QFOR, estimated discharge for the Flood of Record

Water surface profiles for other discharges needed for development of the contract documents and hydraulic impacts and limitations resulting from temporary access structures as required for FERC review will be developed during final design. These additional discharges are anticipated to include:

Q1.1, ordinary high water

Q10

Q25, used to evaluate cofferdam needs and temporary detour bridge hydraulic clearance

Q500, super flood discharge used to evaluate scour

Topographic data for the river channel and floodplain used in the hydraulic model was provided by the Maine DOT Survey Section. Geometric data for the existing bridge was taken from the existing bridge plans. All elevations were referenced to the project datum (NAVD 1988). Geometric data for the proposed bridge was taken from the preliminary design and plans (see Appendix A). Other input data was obtained through research conducted for the preliminary design.

The Frank J. Wood Bridge is located approximately 500 ft downstream from a hydro-electric dam owned by Brookfield Renewable Power. The water surface elevation for all design flows is highly influenced by the flow distribution through the dam. The flow distributions through this upstream boundary were considered in the hydraulic modeling, and are based on historic dam flow distribution (at powerhouse, tainter gates, and spillway) and dam operation data provided by Brookfield Renewable Energy.

Existing Bridge:

The existing truss bridge was evaluated for the Q100 and Q50 discharges. The low chord of the proposed bridge varies due to profile grade and variation in structure depth between Span 1 and 2 (south and middle) trusses and the Span 3 (north) truss. The lowest elevations for each span occur at the support locations.

In general, the average water surface elevation (WSE) below truss Span 3 is much higher than the WSE below truss Spans 1 and 2, and the average WSE below Spans 1 and 2 only vary slightly. The variations are primarily due to the topography of the river channel and poor alignment of Pier 2 with the flow.

The summary of Q100 WSEs and hydraulic clearance data is for the existing bridge is included in the table below:

Location	Low Chord El. (ft)	Q_{100} WSE (ft)	Hydraulic Clearance (ft)
South Abutment	37.1	20.4	16.7
Pier 1	34.1	21.9	12.2
Pier 2 south	31.1	24.9	6.2
Pier 2 north	32.1	**30.0**	2.1
North Abutment	**30.4**	28.9	**1.5**
Span 1	34.1 (min)	20.2 (avg)	13.9 (min)
Span 2	31.1 (min)	21.1 (avg)	10.0 (min)
Span 3	30.4 (min)	28.8 (avg)	1.6 (min)

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

The minimum Q100 hydraulic clearance under the existing bridge occurs at the location of the existing North Abutment below Span 3. The minimum clearance to the low chord is 1.5 ft at this location. This measure is also close to the minimum clearance based on a measure of the average WSE below Span 3 of 1.6 ft. The peak WSE below the existing truss bridge occurs below

Span 3 near Pier 2 where the water surface rises steeply along Pier 2 due to the poor alignment of this pier to the flow.

Discharge velocities for the Q100 event are variable near the location of the existing bridge and reach a maximum of about 30 ft/sec near Pier 2 below Span 2. At this location the riverbed topography drops dramatically approximately 8 to 10 ft.

Preliminary modeling runs conducted early in the design development phase showed that the WSE at the North Abutment for the Q50 discharge event was approximately 2.3 ft lower than the Q100 discharge event. The existing truss bridge minimum clearance to the Q50 WSE is approximately 3.8 ft. Additionally, the model shows that the WSE for a Q100 discharge event at the Sea Dogs Restaurant would be almost 1 ft above the patio deck surface and restaurant floor or about El 24.7. Any changes to the hydraulic conditions need to account for this and look to, at the very least, maintaining this. Improving this potential flooding condition should be examined.

The existing bridge was also evaluated for the Flood of Record discharge. The maximum WSE for this event generally follows the same gradient across the river at the bridge location, except the variations are not as dramatic. The difference in WSE between the lower south side of the channel and higher north side of the channel is approximately 6 ft. The maximum WSE occurs near the North Abutment at El. 32.2, or 1.8 ft above the minimum low chord elevation. This modeled flow proved to be in close agreement with historical record photos of the 1936 flood showing the WSE along the Bowdoin Mill complex.

Refer to Appendix F for additional hydraulic modeling data including resultant water surface elevation plan plots, velocity distribution plan plots, and river section plots for the Existing Bridge.

Recommended Replacement Five Span Bridge on Upstream Alignment:

The replacement bridge was evaluated for the Q100 discharge. The low chord of the proposed bridge varies, due to the profile grade and use of haunched girders. The lowest elevations for each span occur at the support locations.

Similar to the existing bridge conditions, the average water surface elevation (WSE) below the northerly spans of the proposed bridge replacement is much higher than the WSE below the southerly spans. In general, the WSE below the bridge matches the existing WSE, and variations are localized near the proposed piers. The WSE generally rises from the south side of the channel to the north side of the channel. For both the existing and proposed conditions, this is primarily due to the topography of the river channel. There is a dramatic drop in WSE at Pier 3. This is because Pier 3 is located at the edge of a steep drop in the river channel bed. The topography difference at this location is approximately 8.6 ft. The water surface difference on each side of the pier is 5.5 ft.

The summary of Q100 WSEs and hydraulic clearance data is for the replacement bridge is included in the table below:

Location	Low Chord El. (ft)	Q_{100} WSE (ft)	Hydraulic Clearance (ft)
Abutment 1	41.40	19.6	21.8
Pier 1 south	40.68	20.5	20.2
Pier 1 north	36.42	20.5	15.9
Pier 2	33.04*	20.8	12.2
Pier 3	31.20*	23.8	7.4
Pier 4	**30.35***	**29.3**	**1.0**
Abutment 2	31.63	26.8	4.8
Span1	40.68 (min)	19.7 (avg)	21.0 (min)
Span 2	33.04 (min)*	20.5 (avg)	12.5 (min)
Span 3	31.20 (min)*	20 (avg)	11.2 (min)
Span 4	30.35 (min)*	25 (avg)	5.4 (min)
Span 5	30.35 (min)*	27.4 (avg)	3.0 (min)

Note: All elevations based on North American Vertical Datum (NAVD) of 1988.

The minimum Q100 hydraulic clearance under the replacement bridge occurs at Pier 4 where the beam is haunched. The minimum clearance to the low chord is 1.0 ft at this isolated location. The peak WSE below the replacement bridge occurs near Pier 4 where the water surface rises steeply adjacent the pier. This rise is partly attributed to the riverbed topography which rises steeply and to the resulting high velocities immediately upstream of Pier 4, which is well aligned with the flow at this location. This situation is isolated and far less than the minimum clearance to the water surface below Spans 4 or 5 when measured to the average WSE under those spans. The average WSE yield minimum clearances of 5.4 ft (Span 4) and 3.0 ft (Span 5) to the bottom of the haunched girder section. The replacement bridge provides significantly more clearance than the existing bridge to the water surface below the bridge for the majority of these controlling spans.

Discharge velocities for the Q100 event below the replacement bridge are significantly lower than the velocities below the existing bridge. At the replacement bridge, the Q100 event velocities reach up to 23 ft/sec. The velocities reach up to 30 ft/sec in other upstream and downstream areas and are similar to the existing conditions.

Water surface elevations upstream and downstream of the proposed bridge closely match the existing conditions, and there is no rise in the regulatory flow water surface elevation along either bank of the river or within the powerhouse outfall channel.

Several different geometric configurations were evaluated as part of the hydraulic modeling. The modeling initially showed that the water surface would rise significantly (up to 1.6 ft on average) in an isolated area downstream of the existing truss bridge along the north riverbank, adjacent to the Bowdoin Mill complex. Several combinations of measures were investigated to alleviate this issue including rotating the bridge piers to better align with the flow or to help redirect flow back toward the center of the channel, retaining the North Pier of the existing bridge (which helps to redirect flow back to the center of the channel and away from the Bowdoin Mill complex), and removing a significant portion of the large rock outcrops within the channel near the end of the Bowdoin Mill complex (that act to block the passage of high flows). Although all of these measures helped to improve the situation, no one solution alone would resolve the issue. Also, the environmental impacts and cost of the removal of the ledge outcrops made this configuration less desirable than the others. The analysis determined that the combination of skewing Pier 3 35 degrees to better align with the flow and retaining the North Pier of the existing bridge was sufficient to achieve the desired result of maintaining the existing WSE along the Sea Dogs Restaurant. This recommendation can be further refined during final design.

Scour evaluation was not conducted for the PDR study due to the anticipation that all foundations will bear on exposed or shallow bedrock. The need for scour evaluations may be further considered during later phases of the project development once borings are obtained and geotechnical recommendations are made.

Refer to Appendix F for additional hydraulic modeling data including resultant water surface elevation plan plots, velocity distribution plan plots, and river section plots for the Replacement Bridge Option.

Reported By: Rick Hebert (T.Y. Lin International)
Date: May 27, 2016

APPENDIX A

Preliminary Plans

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

SPECIFICATIONS

Design: Load and Resistance Factor Design per AASHTO LRFD Bridge Design Specifications, 7th Edition 2014 and interim specifications through 2016.

DESIGN LOADING

Live Load .... HL - 93 Modified for Strength I (Truck only increased 25%)

TRAFFIC DATA

Current (2015) AADT	18860
Future (2035) AADT	22630
DHV - % of AADT	10%
Design Hour Volume	2263
Heavy Trucks (% of AADT)	3%
Heavy Trucks (% of DHV)	3%
Directional Distribution (% of DHV)	50%
18 Kip Equivalent P 2.0	189
18 Kip Equivalent P 2.5	181
Design Speed	25 mph

HYDROLOGIC DATA

Drainage Area	3435 sq mi
Ordinary High Water Discharge (Q1.1)	27,500 cfs
Design Discharge (Q50)	89,300 cfs
Check Discharge (Q100	99,700 cfs

Headwater Elevation &
Discharge Velocities
Vary Due to nearby Dam, and
Riverbed Topography

MATERIALS

Concrete:

Sidewalk and End Posts	Class "LP"
Seals	Class "S" (Unless Noted)
All Other	Class "A"

Reinforcing Steel	ASTM A 615/A 615M, Grade 60
Glass Fiber Reinforced Polymer (GFRP)	CSA 5807-10, ACI 440.1R-06

Structural Steel:

All Material (except as noted)	ASTM A 709, Grade 50 Metalized
High Strength Bolts	ASTM A 325, Type I, Galvanized

BASIC DESIGN STRESSES

Concrete (Class A)	f 'c = 4350 psi
Concrete (Class LP)	5000 psi
Concrete (Class S)	3000 psi
Reinforcing Steel	f y = 60,000 psi

Glass Fiber Reinforced Polymer (GFRP)

Minimum Tensile Strength (Straight)	100,000 psi
Minimum Tensile Strength (Bent)	50,000 psi
Minimum Elastic Modulus	5,850,000 psi

Structural Steel:

ASTM A 709, Grade 50	F y = 50,000 psi
ASTM A 325	F µ = 120,000 psi

BRUNSWICK - TOPSHAM
CUMBERLAND & SAGADAHOC COUNTIES
FRANK J. WOOD BRIDGE
OVER
ANDROSCOGGIN RIVER
ROUTE 201/24
PROJECT NO. STP-2260(300)X
PROJECT LENGTH 0.3 mi.
BRIDGE NO. 2016

MAINTENANCE OF TRAFFIC

Maintain two lanes of traffic (one each direction) over existing bridge and approaches during construction.

PROJECT LOCATION:	Frank J. Wood Bridge # 2016 on the Brunswick-Topsham TL which carries Route 201/24 over the Androscoggin River. Latitude 43°55'14.27N" Longitude 69° 57'57.46W"
PROGRAM AREA:	Bridge Program
OUTLINE OF WORK:	Bridge Replacement

WIN 22603.00

STP-2260(300)X

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	
CHIEF ENGINEER:	

PROJECT INFORMATION

PROGRAM	BRIDGE	SIGNATURE
PROJECT MANAGER	J. KILTREDGE	.
DESIGNER	R. M. HEBERT	P.E. NUMBER
CONSULTANT	T. Y. LIN	.
PROJECT RESIDENT		DATE
CONTRACTOR		
PROJECT COMPLETION DATE		

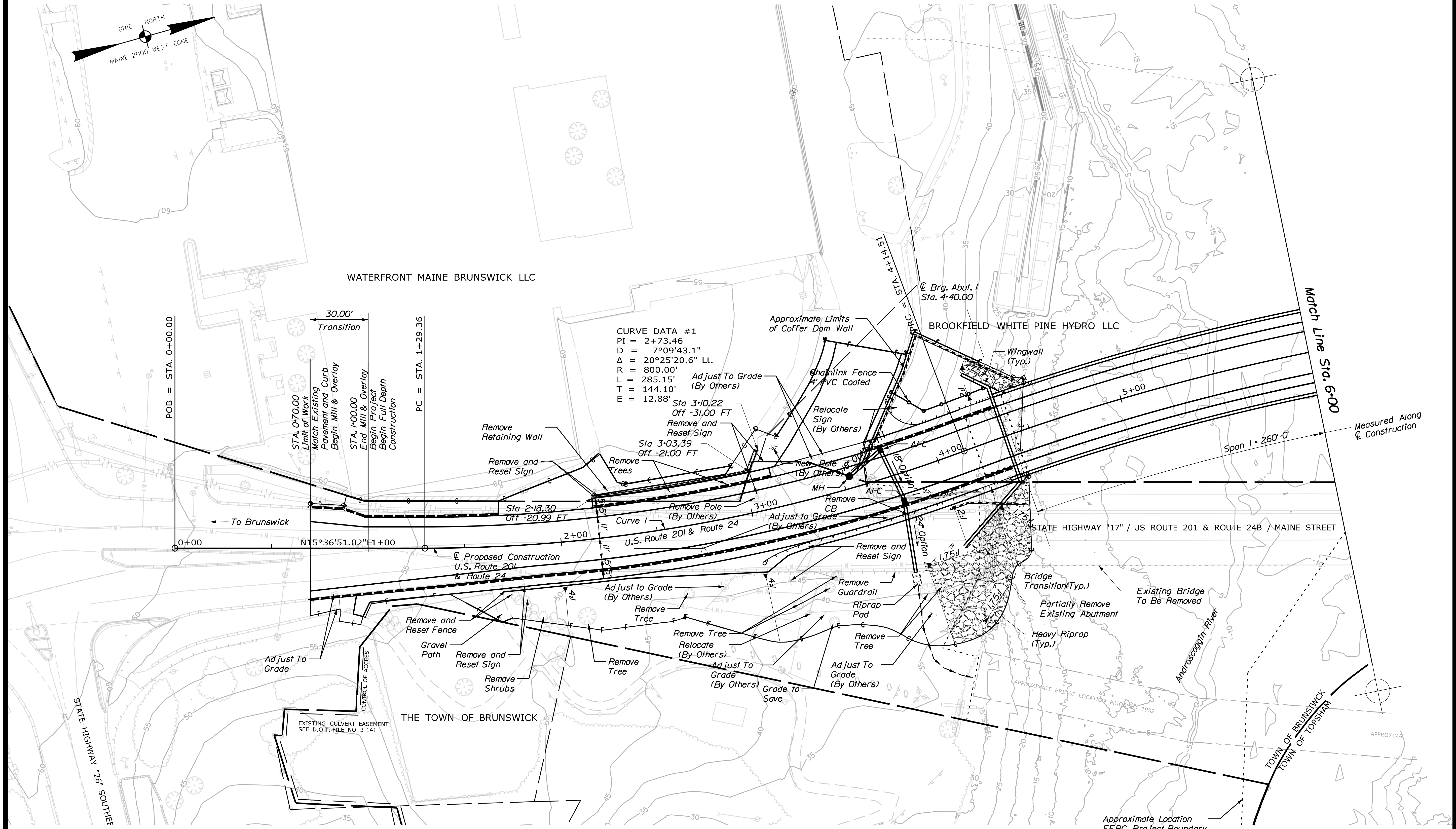
TOPSHAM - BRUNSWICK
FRANK J. WOOD BRIDGE

TITLE SHEET

SHEET NUMBER
1
OF 9

Date:8/4/2017 Username: Division: HIGHWAY Filename:\00\BRIDGE\MSTA\001_Title.dgn

Filename: ...\HIGHWAY\MSTA000_HDPlan_01.DGN Division: HIGHWAY Username: Date:8/4/2017



PLAN

25 0 25 50
Scale of Feet

PRELIMINARY

T·Y·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016
WIN 22603.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. Baker		
DESIGN-DETAILED	D. Burhans	D. Burhans	06/17
CHECKED-REVIEWED	R. Hebert	K. Ducharme	06/17
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PLAN 1 OF 3

SHEET NUMBER

2

OF 9

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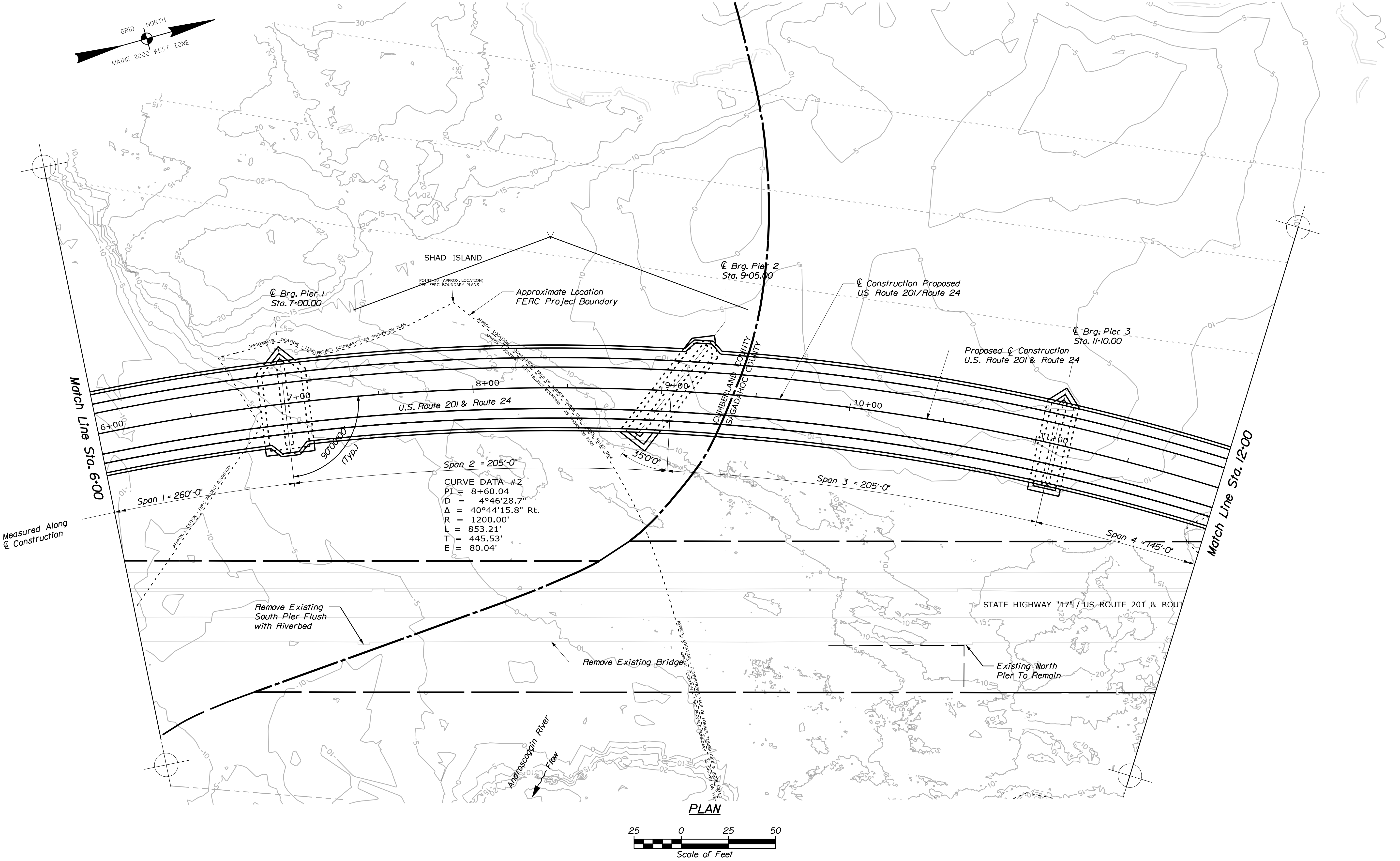

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L = 853.21'
T = 445.53'
E = 80.04'

PLAN

PRELIMINARY

T·Y·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016
WIN
22603.00
BRIDGE PLANS

PROJ. MANAGER	N. Baker	BY	DATE
DESIGN-DETAILED	D. Burhans	D. Burhans	04/16
CHECKED-REVIEWED	R. Hebert	R. Hebert	04/16
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PLAN 2 OF 3

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OF 9

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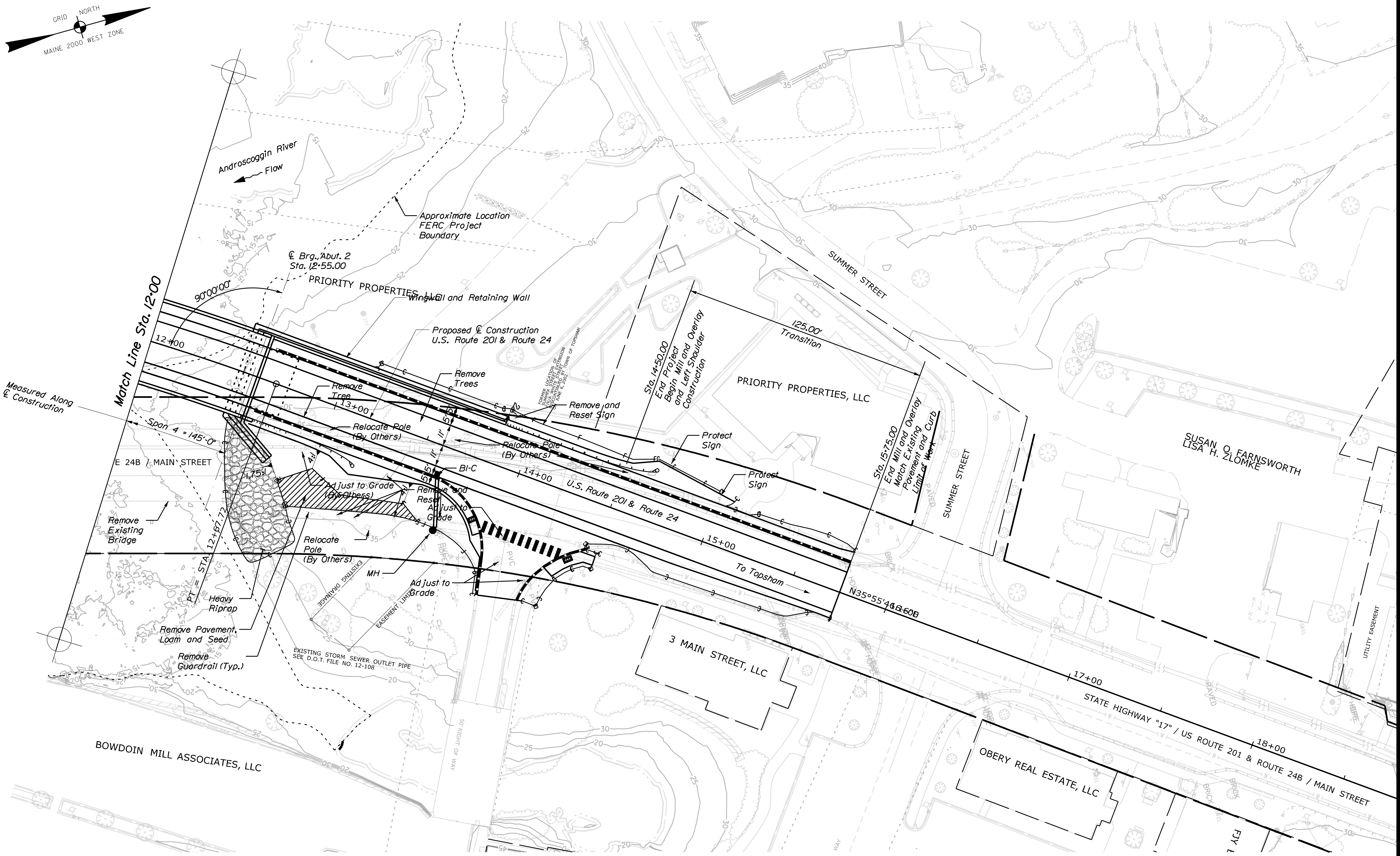

PLAN

PRELIMINARY

T·Y·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016
WIN
22603.00
BRIDGE PLANS

PROJ. MANAGER	N. Baker	BY	DATE
DESIGN-DETAILED	D. Burhans	D. Burhans	06 / 17
CHECKED-REVIEWED	R. Hebert	K. Ducharme	06 / 17
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PLAN 3 OF 3

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OF 9

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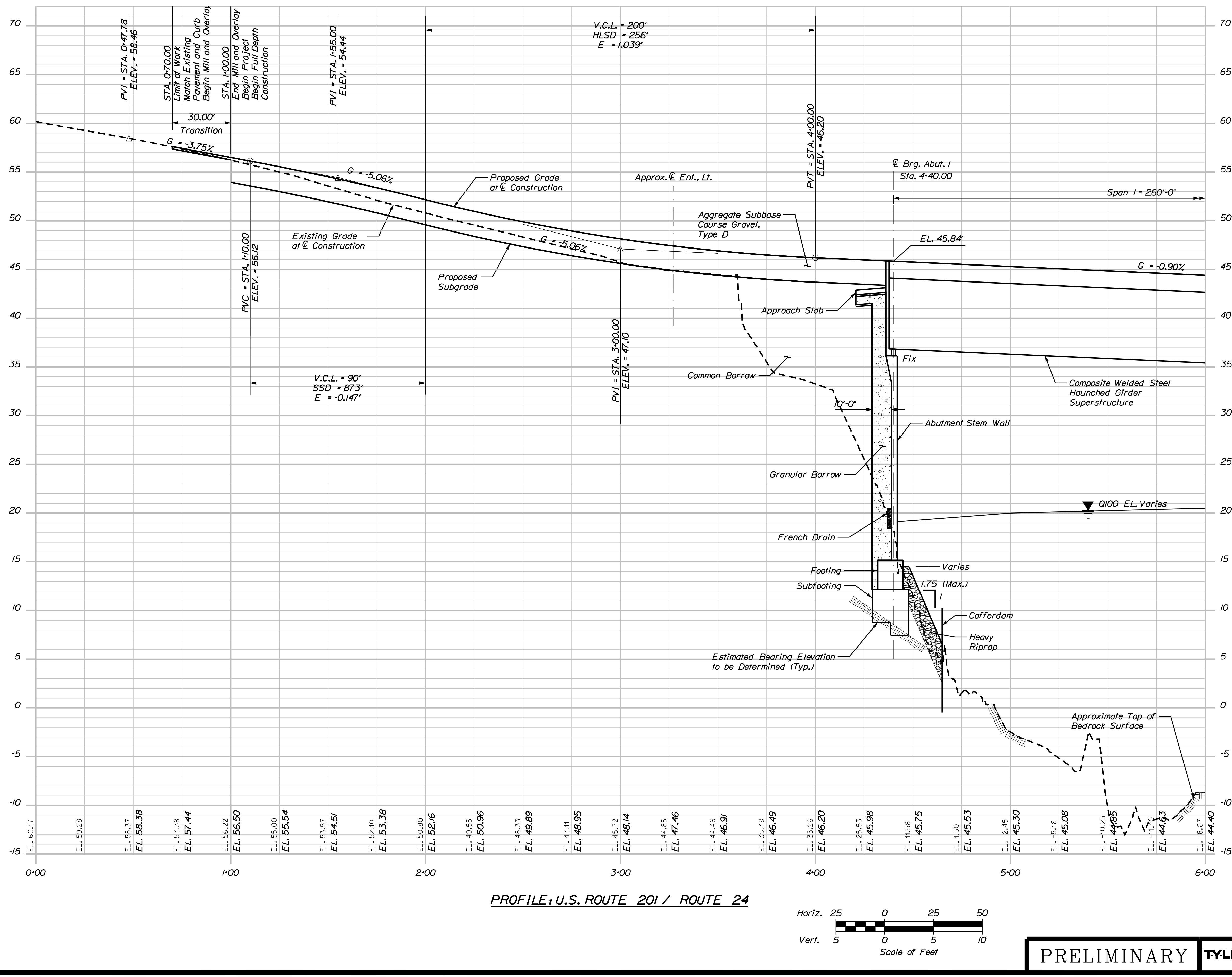

PROFILE: U.S. ROUTE 201 / ROUTE 24

PRELIMINARY

T·Y·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE PLANS

WIN
22603.00

BRIDGE NO. 2016

SIGNATURE

P.E. NUMBER

DATE

PROJ. MANAGER	N. Baker	BY	DATE
DESIGN-DETAILED	D. MEYERS	D. BURHANS	04 / 16
CHECKED-REVIEWED	R. HEBERT	R. HEBERT	04 / 16
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DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
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FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PROFILE 1 OF 3

SHEET NUMBER

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OF 9

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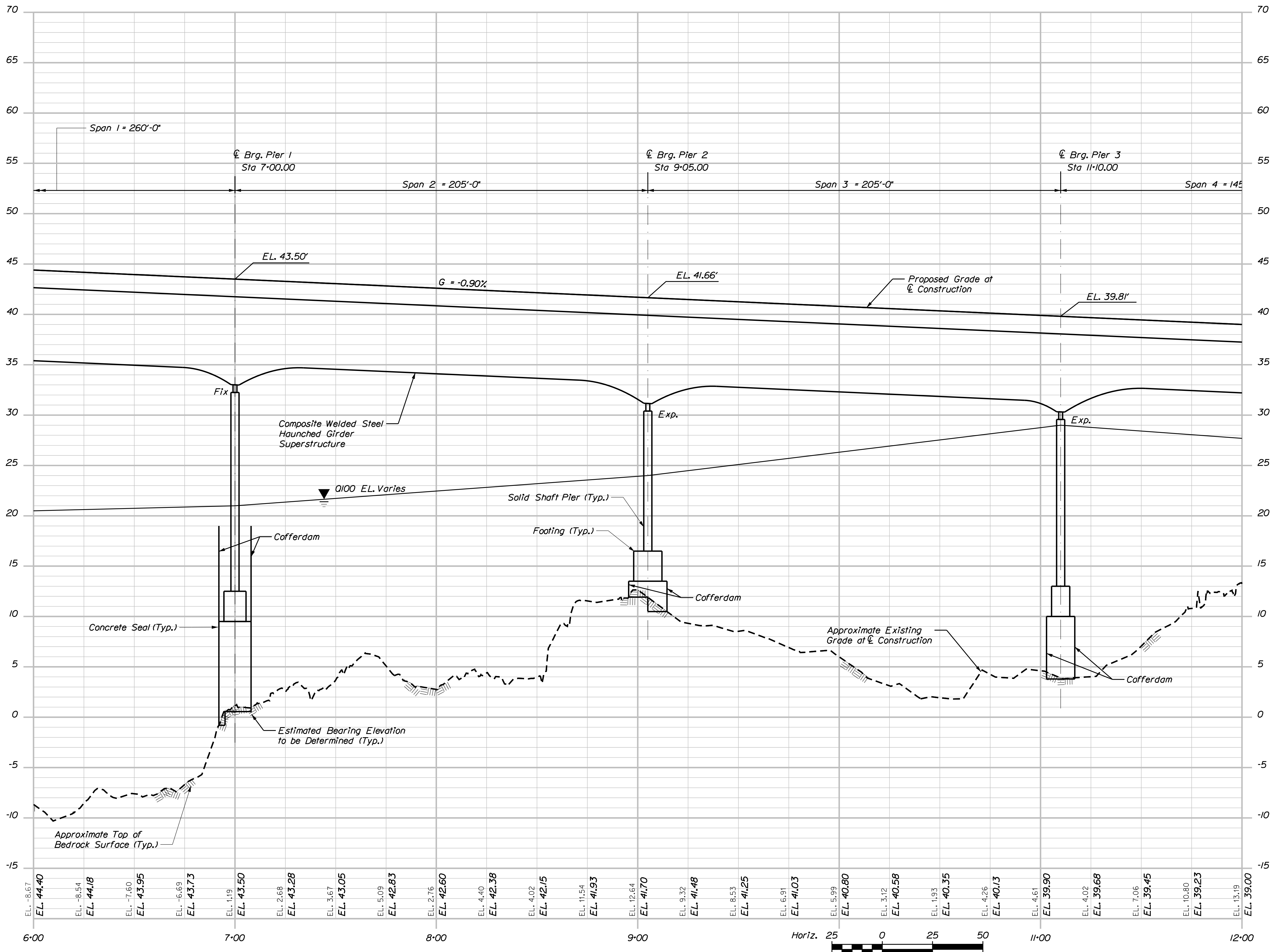

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Vert. 5 0 5 10
Scale of Feet

PRELIMINARY

T·Y·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE PLANS

WIN
22603.00

BRIDGE NO. 2016

		BY	DATE
PROJ. MANAGER	N. Baker		
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CHECKED-REVIEWED	R. HEBERT	R. HEBERT	04 / 16
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REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PROFILE 2 OF 3

SHEET NUMBER
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OF 9

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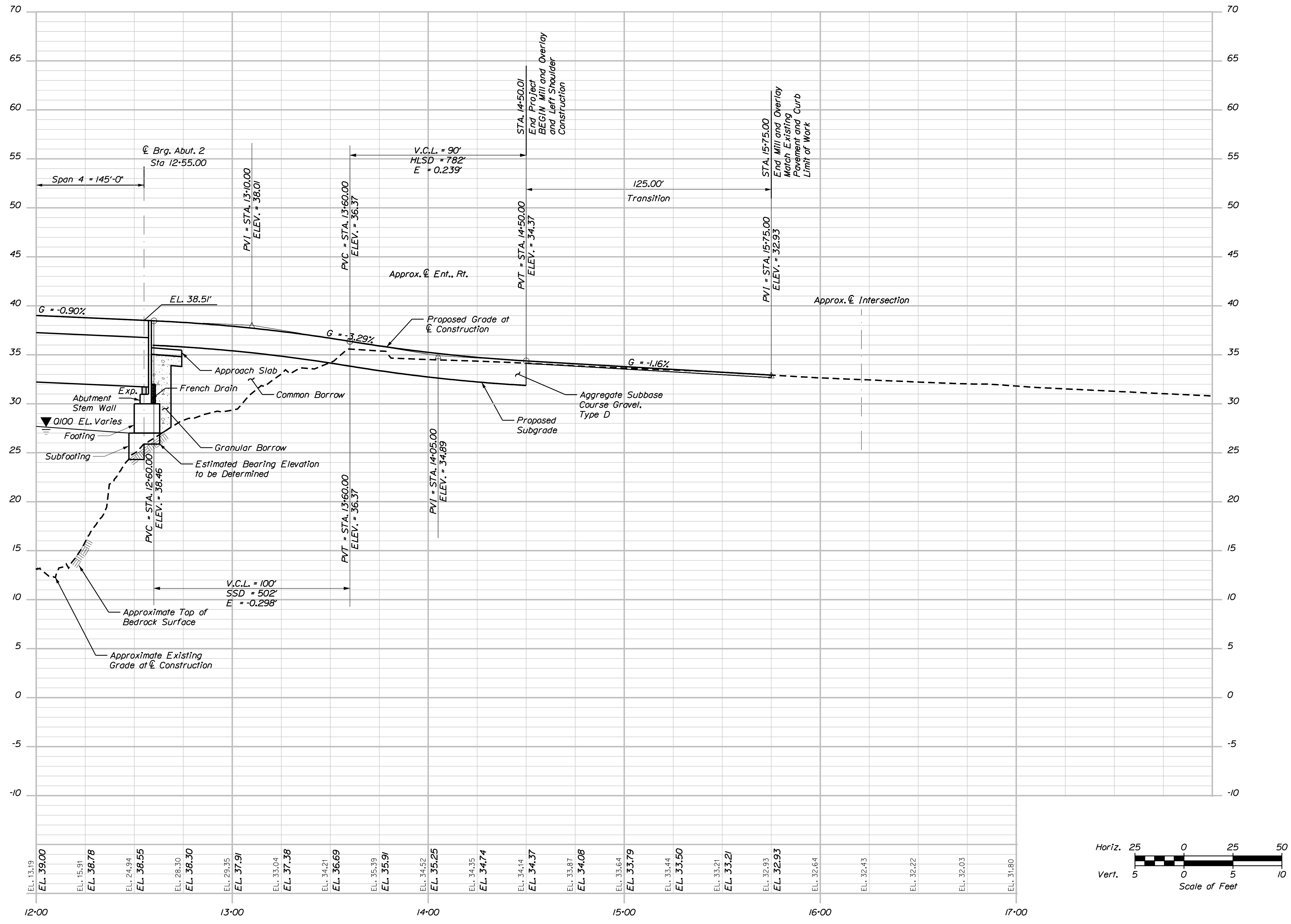

PROFILE: U.S. ROUTE 201 / ROUTE 24

PRELIMINARY

TYLININTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016

WIN
22603.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. Baker		
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CHECKED-REVIEWED	R. HEBERT	R. HEBERT	04 / 16
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DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

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FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

PROFILE 3 OF 3

SHEET NUMBER

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OF 9

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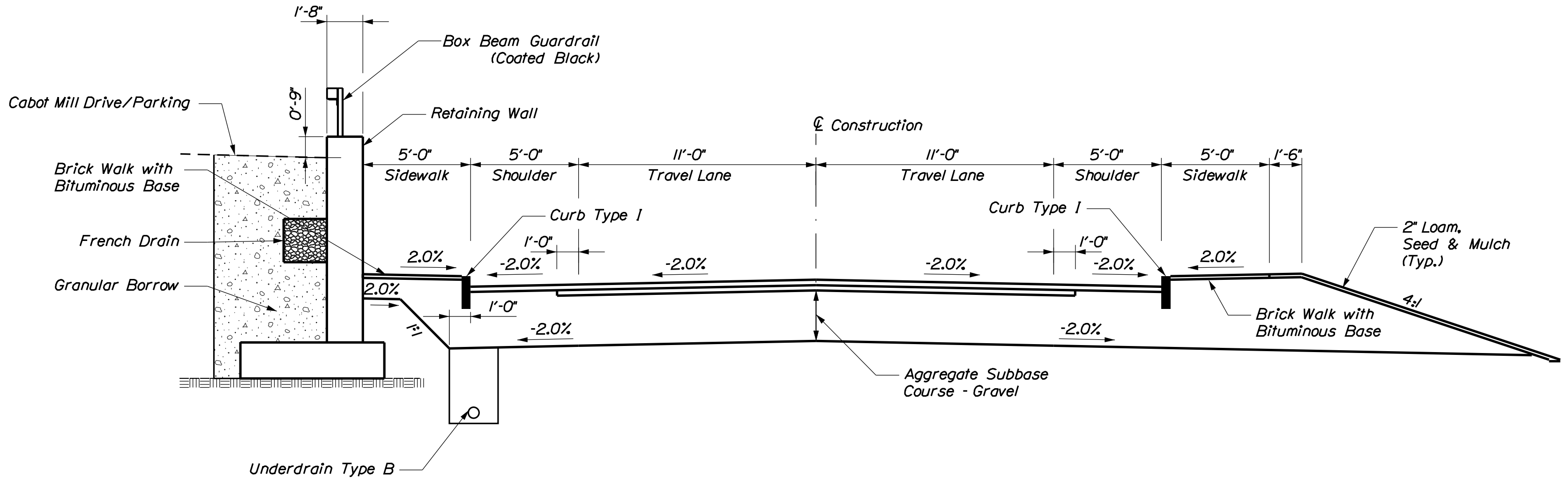

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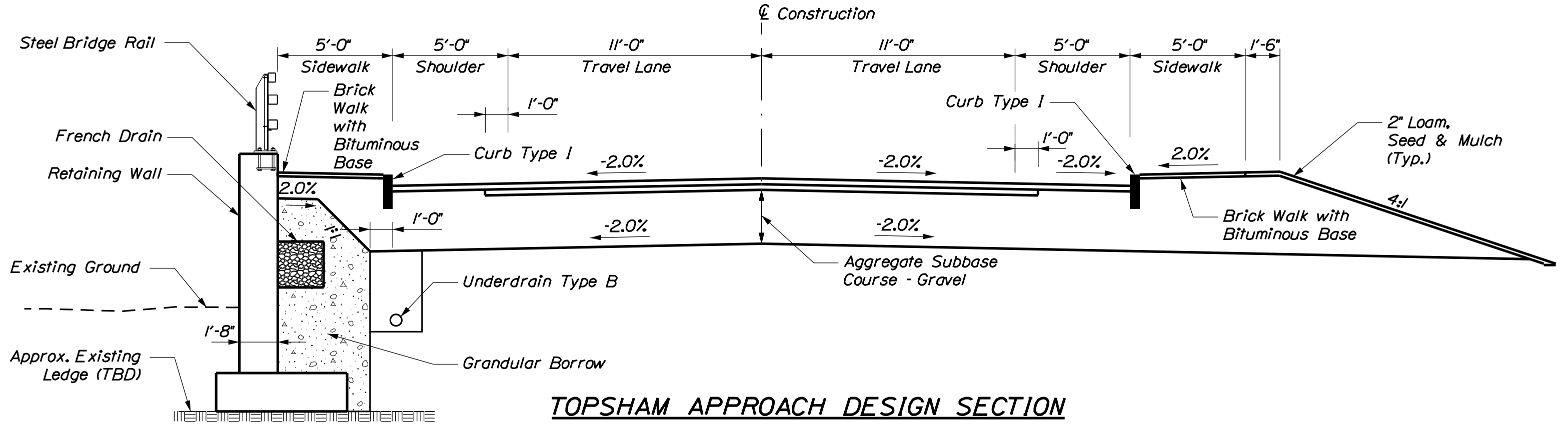

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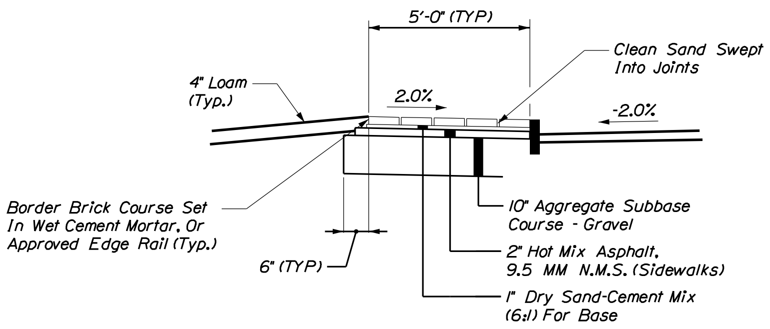
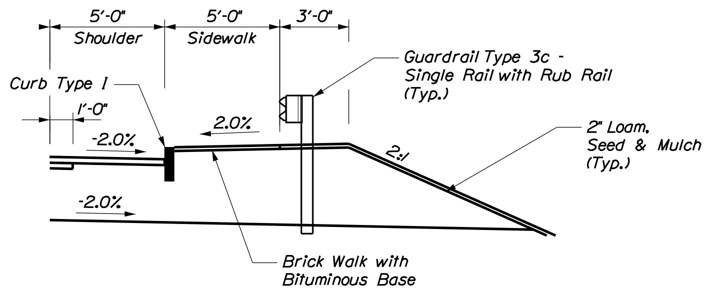


Bricks To Be Used:

Bricks Shall Meet Maine Department Of Transportation Standard Specifications Section 704.02 And Shall Match The Existing Bricks As Closely As Possible. Final Brick Selection To Be Approved By The Resident.

ITEM 608.15 BRICK WALK WITH BITUMINOUS BASE



APPROACH SECTION WITH GUARDRAIL (TYP.)

PRELIMINARY

TYLININTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016

WIN
22603.00

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. Baker		
DESIGN-DETAILED	D. Burhans	D. Burhans	04 / 16
CHECKED-REVIEWED	R. Hebert	R. Hebert	04 / 16
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DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

TYPICAL SECTIONS

SHEET NUMBER

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OF 9

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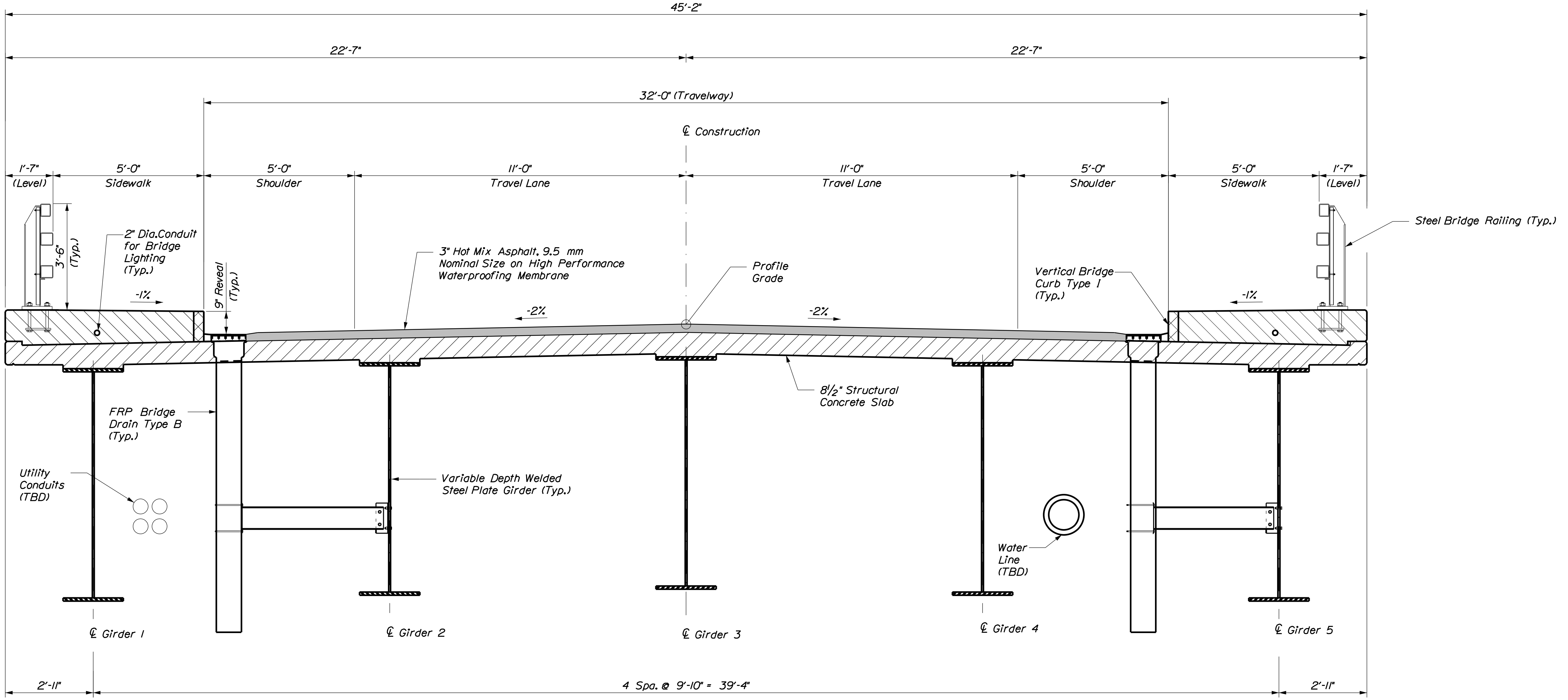


TRANSVERSE SECTION

PRELIMINARY

TY·LIN INTERNATIONAL

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2260(300)X

BRIDGE NO. 2016
WIN
22603.00
BRIDGE PLANS

PROJ. MANAGER	N. Baker	BY	DATE
DESIGN-DETAILED	R. Hebert	D. Burhans	06 / 17
CHECKED-REVIEWED	N. Baker	R. Hebert	06 / 17
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK-TOPSHAM CUMBERLAND

TRANSVERSE SECTION

SHEET NUMBER
9
OF 9

APPENDIX B

Photographs

- Photos 1 – 5, October 31, 2014
- Photos 6 – 8, March 13, 2016

Photo #1: Bridge View looking north from Brunswick

Photo #2: Bridge View looking south from Topsham

Photo #3: Topsham Approach looking north from bridge

Photo #4: Fort Andross Driveway Entrance

Photo #5: Brookfield Dam & Fish Ladder

Photo #6: Topsham Approach looking south

Photo #7: Brunswick Approach looking north

Photo #8: Fort Andross Parking Lot Wall

APPENDIX C

Inspection Reports

Summary of Frank J. Wood Bridge Repair Contract – Executed April/May 2017

Topsham/Brunswick Route 201/SR 24, WIN 023602.00

William Doukas, PE – Sr. Structural Engineer, Bridge Maintenance, MaineDOT

Contractor: Stetson and Watson. Subcontractor: BMB Construction

Resident: Glenn Philbrook, P.E.

Inspector: Dana Weisner

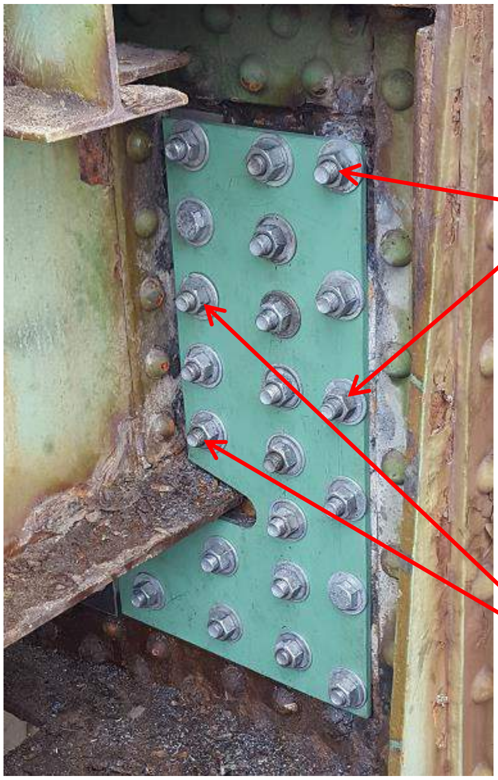
Additional Technical Inspector & Designer: William Doukas, P.E.

Winning Bid Amount: $188,733

Scope of Contract: Apply Traffic Control Plan per contract. Add steel over-plating to Floorbeams 2, 5 and 7 in Span 1 (closest to Brunswick) per plan sheets 3 & 4. This is along the roadside truss near the connections of Stringer 8. Improve the connection between Floorbeam 2 and Stringer 8 (Brunswick side) per plan sheet 3 & 5. Apply Fluid Film to "Hotspots" on entire bridge per sheet 8. These Hotspots are principally the lower third of all the Floorbeams, all around the connections of Stringers 8 and 3 to each of the Floorbeams. The length of stringers requirement is min of 3'. See sheet 8 of plans. All parts and hardware to be designed and fabricated by the Department.

Modifications of Contract: Steel beneath the deck is to be blown off with approximately 140 +/- psi air. (River is at high flow.) This effort includes blowing off level surfaces of Floorbeam flanges, Stringer flanges, Horizontal Gusset plates, accessible Chord areas utilizing an Under Bridge Truck. The interior Stringer 4 thru 7 connections to all the Floorbeams were included, Chord Gusset plates and as much as reasonably possible on the roadside truss chord as well as some of the more deteriorated cross (or needle) beams just under the steel grating.

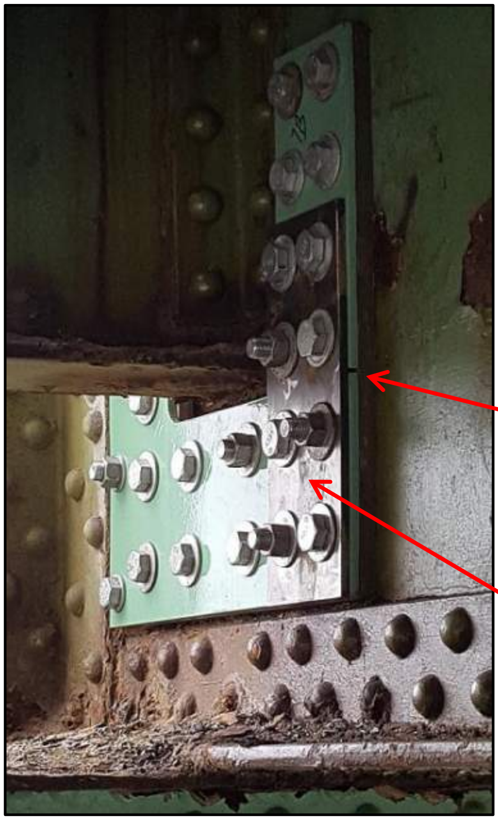
Technical Work Performed: Floorbeams 2, 5 and 7 were repaired identically. The existing specified rivets were removed and replaced with temporary high strength bolts after the Traffic Control Plan was in place. The "J" shaped cover plates needed to be modified as they would not fit on to the web around the stringer connection as detailed. Bill Doukas, Designer, indicated the coverplate could be cut and then tied back in with a small 8 fastener splice plate. BMB cut the plate where suggested and MaineDOT Fleet produced the additional splice plates and supplied longer high strength bolts.



Floorbeam 2 (5 & 7 identical repair).

This column of 6 high strength fasteners has 5 plies of steel engaged: 2 cover plates, 2 filler plates and the web of the floorbeam.

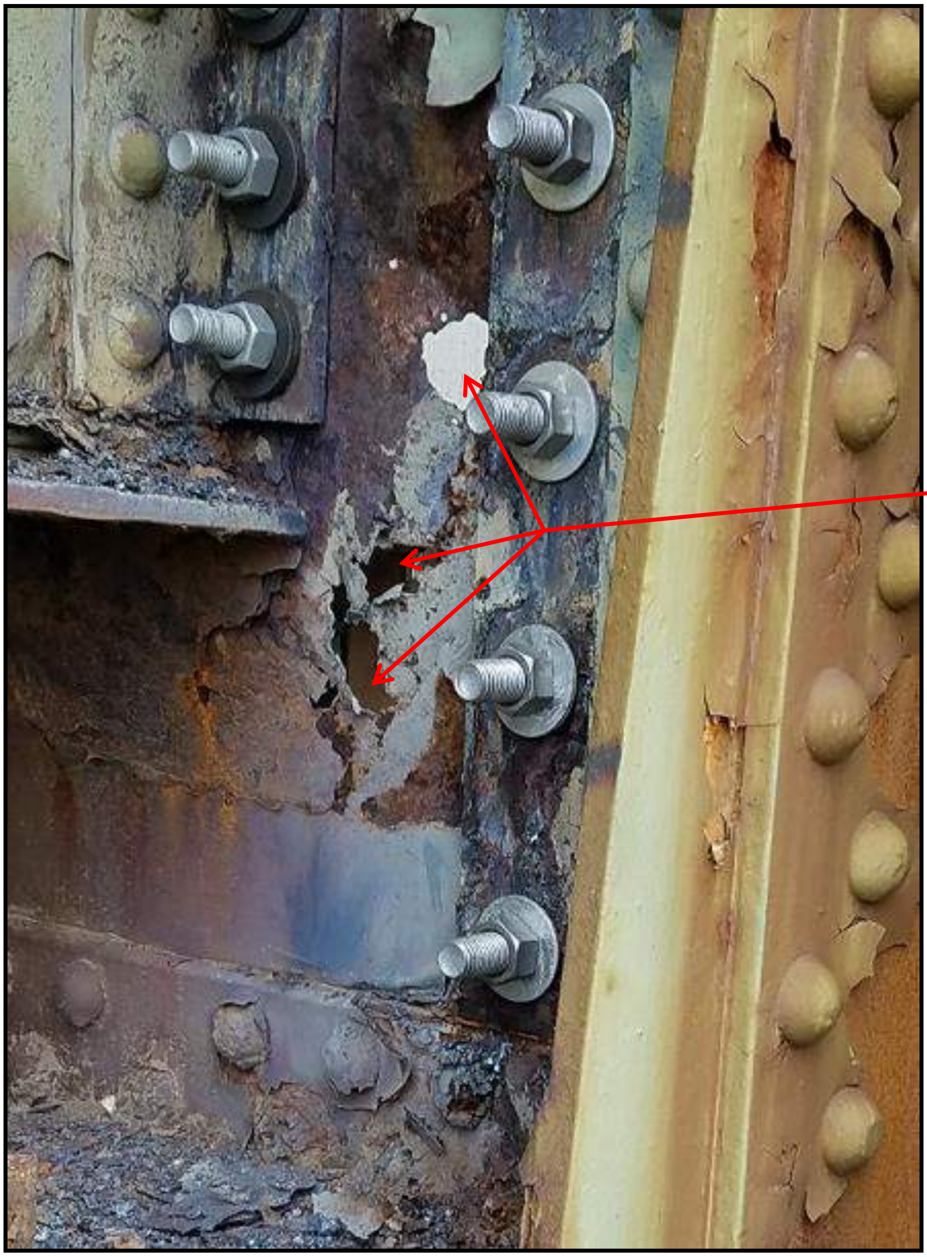
This column of 5 high strength fasteners joins the cover plates to the 5/16” thick angles that fasten the Stringer webs to the Floorbeam webs. This forms a bridge over the thinning area of the Floorbeams.



The backside of the cover plate needed to be cut and then spliced back together in order to fit around the Stringer.

Neat cut by contractor.

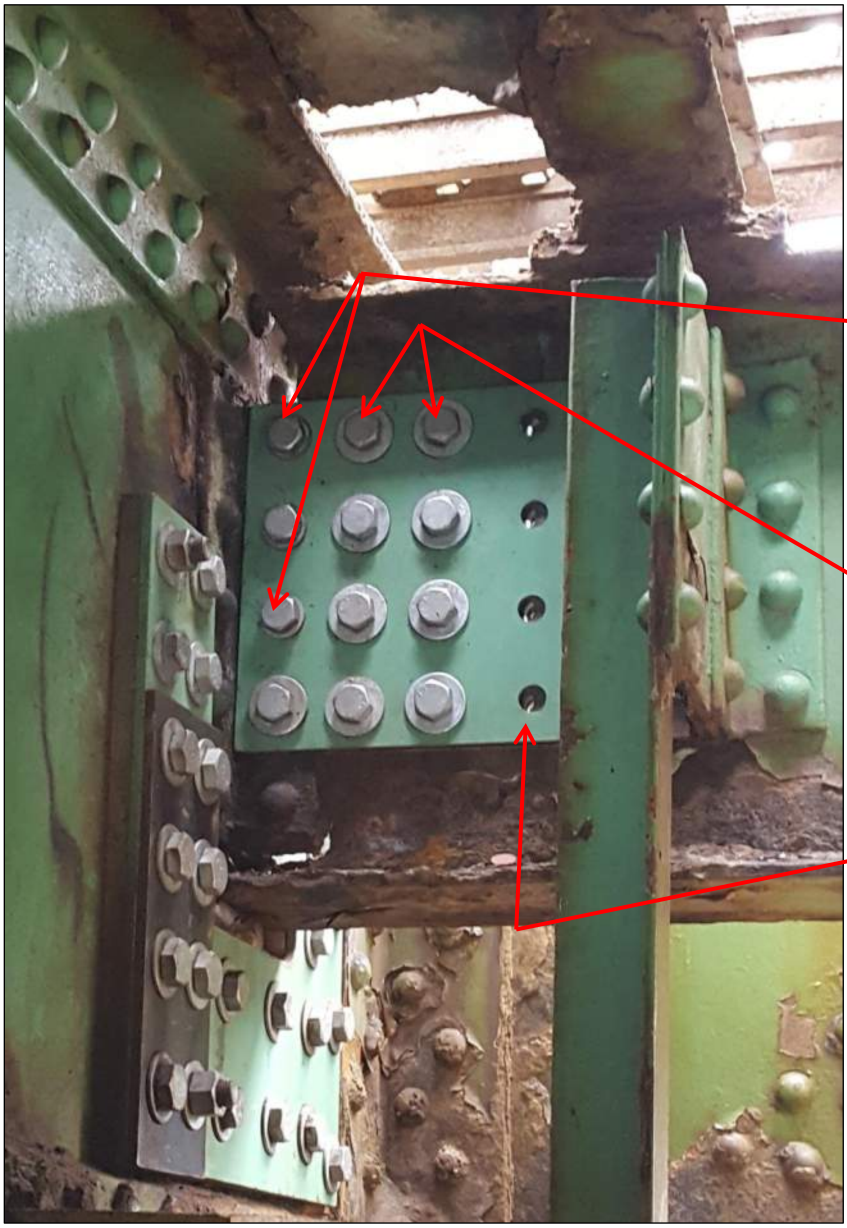
Additional splice plates supplied by Fleet.



Floorbeam 7 was in the worst condition.

Holes and thinning obviously apparent in this area of the Floorbeam.

Another view of Floorbeam 7 deterioration prior to repair.



Repair of Stringer 8 to Floorbeam 2 (Brunswick side).

This column of 4 high strength fasteners joins the cover plates to the 5/16" thick angle fastened to the Floorbeam web. 5 plies of steel involved.

These 2 columns of high strength bolts (8) pull together cover plates, shims and the Stringer web.

This column of holes was unable to be used. Contractor inadvertently flipped plate backwards after drilling 3 required holes in cover plate. Using the cover plate as a template, the holes drilled through the Stringer web were incorrectly located. The result is the last column of holes could not be used. Rather than torch cut, or re-fabricate all the parts, Designer William Doukas, PE, accepted the 8 fasteners in place through the web alone as adequate to support the load. See computations at end.

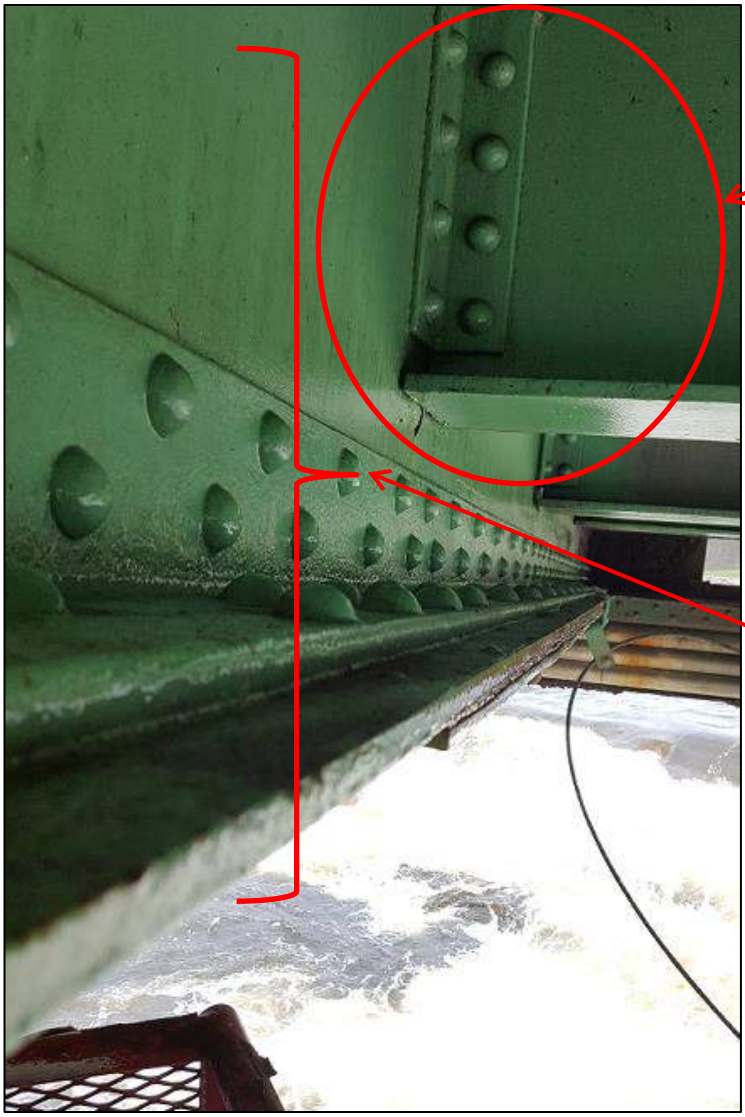
Mis-drilled holes – The usual piece of equipment to field drill holes is a mag-drill. It will not drill a hole in steel overlapping another existing hole. (Bit shatters.) The offer to torch cut was rejected because it may create a heat-affected-zone around the hole perimeters, and possibly, a brittleness. The additional joining was not needed and had potential of additional harm.

Contract Modification Work – Blowing Off Material and Additional Application of Fluid Film:

BMB, the Subcontractor, utilized an Underbridge Truck to blow off sand and dirt from level surfaces on components under the bridge deck. This occupied 2 workers for at least 6-7 working days. One operated the truck while the other 2 hoses: 1 for fluid film (with air) application and the 2nd for high pressure air for cleaning.

This is important to the Department because Bridge Maintenance is scheduling a Routine and Fracture Critical Inspection in the Fall of this year (2017). Clearing the steel will enable to inspectors to examine components more thoroughly and efficiently.

During August of 2016 Special Inspection it was noted a significant buildup of sand/dirt made visual clues more difficult.



Although the contract indicated to apply fluid film to the connection of Stringers 3 & 8 (exterior stringers), the contract was modified to include the interior stringers as well. Blown and fluid film applied 3 foot min along their lengths.

As part of this repair contract the lower 1/3 all Floorbeams on all spans were blow clean and coated with fluid film.

The apparent most susceptible area of the Floorbeam to deterioration were blown clean and coated with fluid film where reasonably accessible with a UBIT.

Noted last August was significant build-up of sand, dirt & debris on horizontal members including gusset plates.

The modification of this contract resulted in most of these areas successfully blown clean with 145 psi of air.

Specific Recommendations for Fall Inspection Crew: Following the completion of steel repairs, blowing steel clean and applying fluid film, a list of suggestions for the upcoming Routine and Fracture Critical Bridge Inspection are included.

1. **Conduct a close inspection of the sidewalk chord rivet condition along the bottom angles, particularly span 3. (Contractor UBIT could not extend far enough for analysis.)**

These rivets should be more exposed after blowing sand and dirt off.

In August of 2016 a typical rivet was located in the sand/debris was struck until rusted portion was removed. A smaller head resulted.

2. **Check closely the web thickness of Floorbeams 5 and 7 in Span 1 on the Sidewalk Truss side. (Repairs have been completed on the Roadside Truss side.) It looked as though the deterioration was more significant in these 2 Floorbeams.**

This area in particular.

3. **Inspect the condition of the rivets and cover plates that appear rusty or excessive wave shape:** These components appear to contain significant pack rust but I have not observed any missing or popped off rivets. (Most of these in Span 2 but could also be located in Span 1. None in Span 3.)

4. **Inspect the web area above and below pin hole in the Floorbeam web shown below. This would be a good spot check for a series of D-meter readings.** This spot check will serve as good general information for the Department.

Repair Location at Floorbeam 5, Roadside Truss – Stringer 8, note hole through web to the left of cover plate.

Suggest collecting D-meter readings along this vertical length for thickness of web. (Original thickness = 0.5”.)

Location of web hole.

5. **Review full length of water pipe hangers for significant loss of continuous support.** A determination should be made on how many compromised water pipe hangers is acceptable. It currently appears that the system is at a stage where more than one in a row may become ineffective. Some steel hangers have deteriorated steel components.

This block appears to be "rolling over".

Custom blocks of wood missing.

If an immediate concern for the Water Pipe structural integrity is evident, contact:

Craig W. Douglas, P.E., Assistant General Manager, Brunswick-Topsham Water District.

Telephone: 207-729-9956

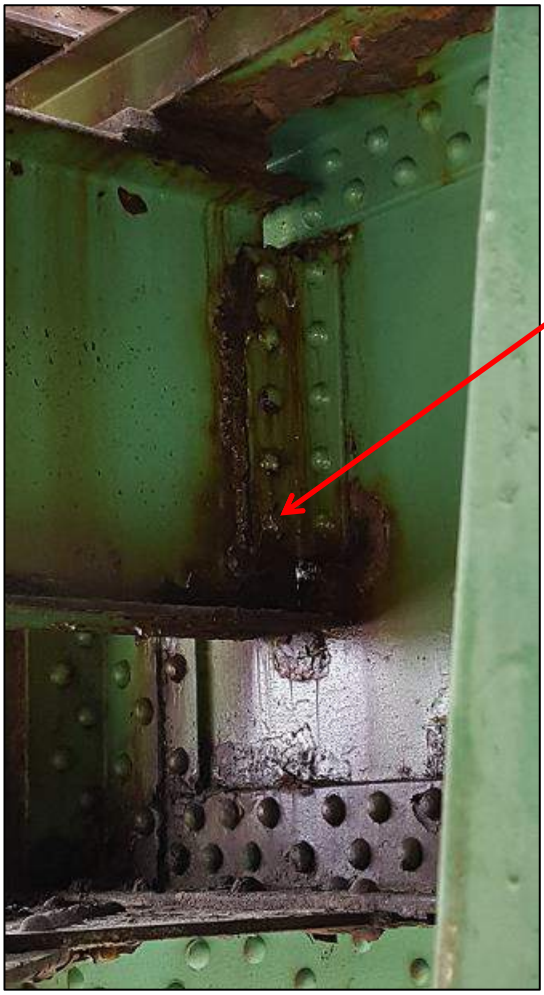
Cell: 207-798-0467

cwdouglas@btwater.org

6. **Look for any distress topside particularly where severe cross-beam deterioration underneath has occurred.** (Between Stringers 7 & 8 and 3 & 4, full length.)

Area of high stress and Cross-beam deterioration.

7. **Continue to review/inspect Stringer ends. Significant numbers in Spans 1 and 3 are deteriorating.** If any rivets become suspect, identify which one(s).



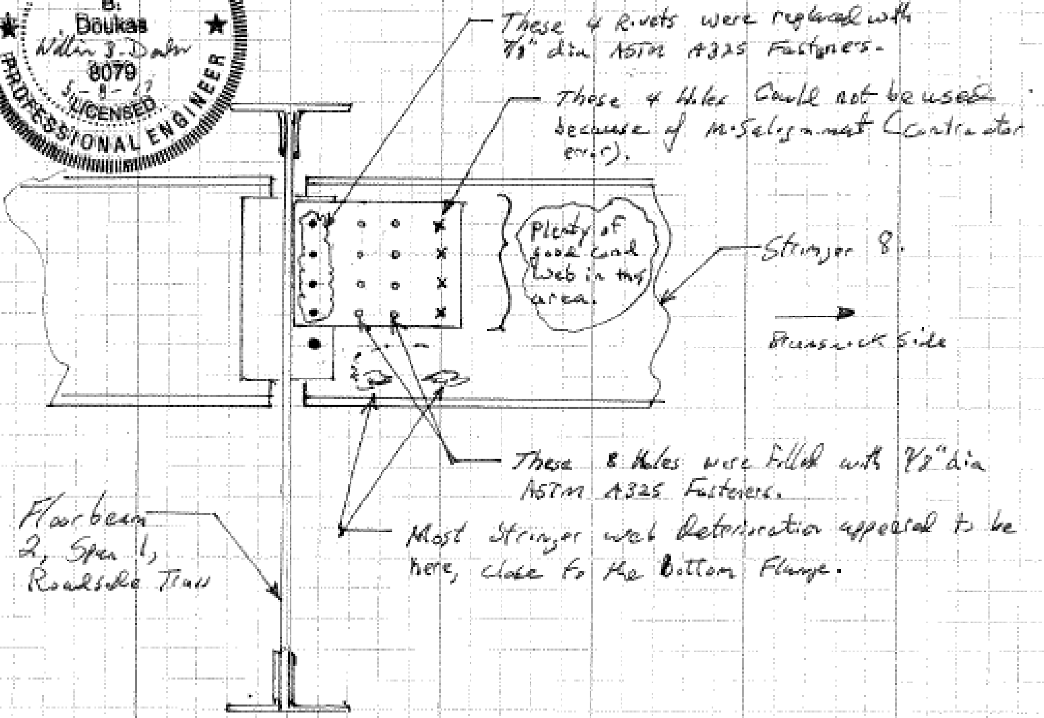
The Department has a standard uniform fix for stringer end connection. The bottom rivet remains to keep the stringer in place while plating is added.

PRELIM. BY W. Doukas, PE DATE 5/17 PROJ. NO. FILE NO. OF
FINAL CHK. BY DATE LOCATION SHEET NO. 1 OF 1
ITEM NO. SUBJECT Frank J Wood, Stringer 8, FB 2, Span 1

Repair of Stringer 8 to Floorbeam 2 in Span 1
Permitting Column of Fastener Holes to Not be Used

4 Bolts + 1 Rivet hold one side of joint.
8 Bolts hold the other side of stringer joint.

William B. Boukas
8079
5-8-17



- Rivets - 7/8" dia - are assigned 18 Ksi (before 1936).
- Rivet area = $0.60\ \text{in}^2$, ∴ $18\ \text{Ksi} \times 0.6 = 10.8^{K}$ per Rivet.
- ASTM A325 - 7/8" dia are assigned 92 Ksi. min Yield Strength.
  Steel Manual - Double Shear, 25.3 kips/Bolt. in Shear.

<u>Left side of Joint</u>		<u>Right Side of Joint</u>
$25.3^{\text{kips}} \times 4 + 10.8^{K} = 112^{\text{kips}}$	$<$	$8 \times 25.3^{\text{kips}} = 202.4^{\text{kips}}$

Right Side has adequate strength. Ok to leave of Column of 4 empty as shown.

Inspection Report
8/1/16 – 8/2/16

Frank J. Wood Bridge #2016
over
Androscoggin River

Brunswick - Topsham, Maine

STP-2260(300)
WIN 022603.00

STATE OF MAINE
William
B.
Doukas
8079
LICENSED
PROFESSIONAL ENGINEER
William B. Doukas

STATE OF MAINE
Devan
Comee
Eaton
13725
LICENSED
PROFESSIONAL ENGINEER

Maine Department of Transportation
Bridge Program

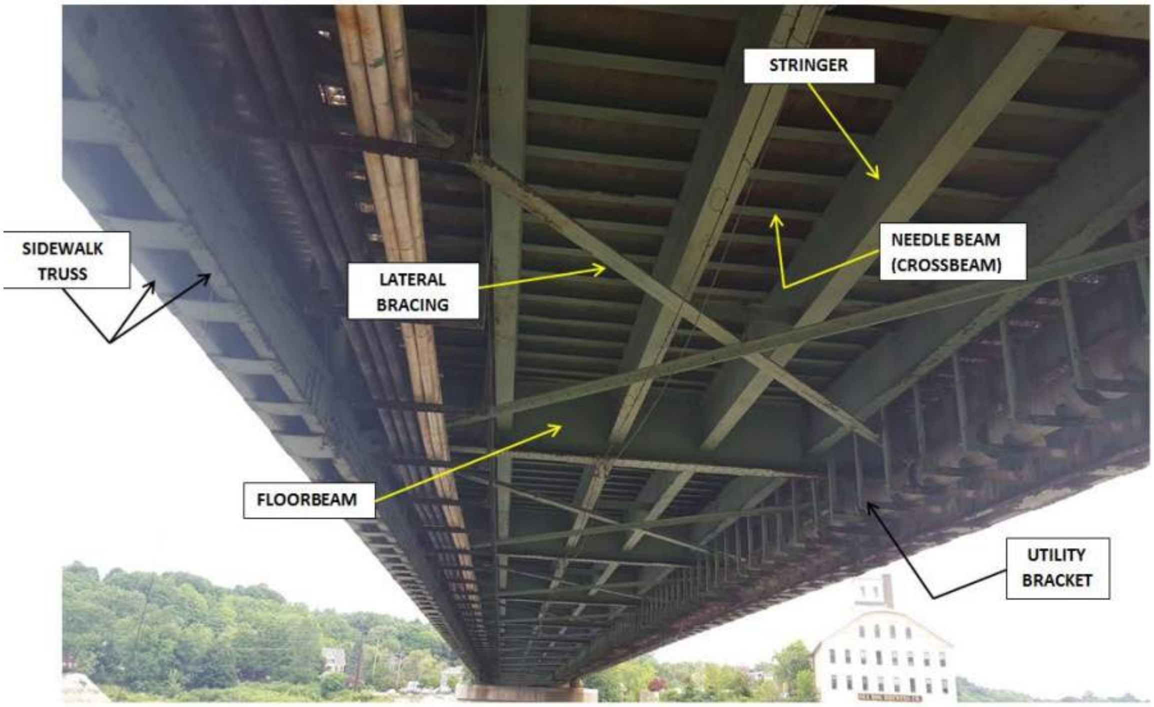
TABLE OF CONTENTS

BACKGROUND AND GENERAL NOMENCLATURE

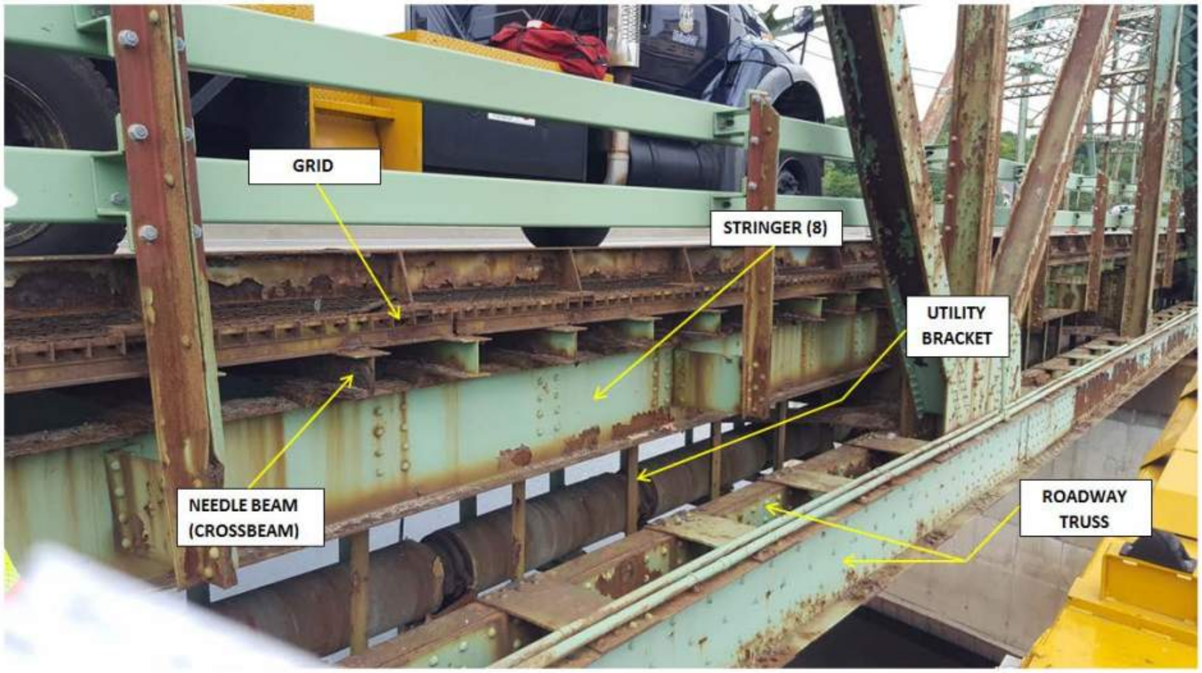
BACKGROUND

A joint field inspection was performed from 8/1/16 to 8/2/16 by representatives from the Bureau of Project Development and Maintenance & Operations, MaineDOT, in an effort to provide additional insight on the condition of Frank J Wood, Bridge #2016. This special inspection was designed to summarize and confirm advanced deterioration, target expected repairs over the next 5 years as well as carefully review issues involving rehabilitation alternatives of 30 - 75 years. Utilizing an earlier results of NBI Bridge Inspection Conducted 6/16/2016 by the Department, Bridge Load Rating results prepared by Parsons Brinckerhoff – March 2013, results of a Routine And Fracture Critical Bridge Inspection Report by Parson Brinkerhoff – 8/20/2012 and interim Department inspections, the team was able to focus on component/attachment conditions of interest effectively. This most recent NBI Bridge Inspection dropped the superstructure rating from 5 to 4 prompting this field work and report.

GENERAL NOMENCLATURE



Typ. nomenclature of Frank J Wood Bridge – looking north from Abut. 1 to Pier 1, in Span 1.



Typical Nomenclature of Frank J Wood Bridge – Looking Northwest midspan of Span 2.

PART A - 5 YEAR REPAIR

EXECUTIVE SUMMARY

Part A of this report focuses on possible repairs within the next 5 years on the MaineDOT Bridge #2016 (Frank J Wood) carrying Routes 201 and 24 over the Androscoggin River in Brunswick and Topsham, Maine. The inspection revealed & confirmed deterioration of this 3 span riveted through truss essentially in Floorbeams, Stringer connections and Truss rivet issues.

- Span 1 – Floorbeam Section Loss in Shear Area, Stringer Connections.
- Span 2 – Floorbeam Cover Plate Distress in Moment Area.
- Span 3 – Stringer Connections, Truss Lower Chord Rivet Deterioration.

There may also be eventual repair work needed in the deck structure (Needle Beam & Grid) as well as utility brackets in order to reach the 5 year mark.

The most significant finding is the loss of section (holes rusted through) in the shear area of 3 Floorbeams (7, 5 & 2). A recalculation of the Load Rating for Floorbeam 7 resulted in a drop from 0.66 to 0.51 or 19% in 4 years. This contributed to the Posting Committee re-evaluating the structure and posting the Frank J Wood Bridge to 25 Tons.

The rate of deterioration is difficult to quantify. This report briefly compares the specific deterioration in Floorbeams 7 & 5 (Span1) using the Routine And Fracture Critical Bridge Inspection Report by Parson Brinkerhoff – 8/20/2012 to the current conditions (4 years later). The result is concerning.

An approximate estimate to repair the listed possible current deterioration within the next 5 years is $805K. Previous successful repair methods on other bridges could be applied to this steel superstructure. There is a possibility not all the listed deteriorated components will reach repair status.

FLOORBEAMS

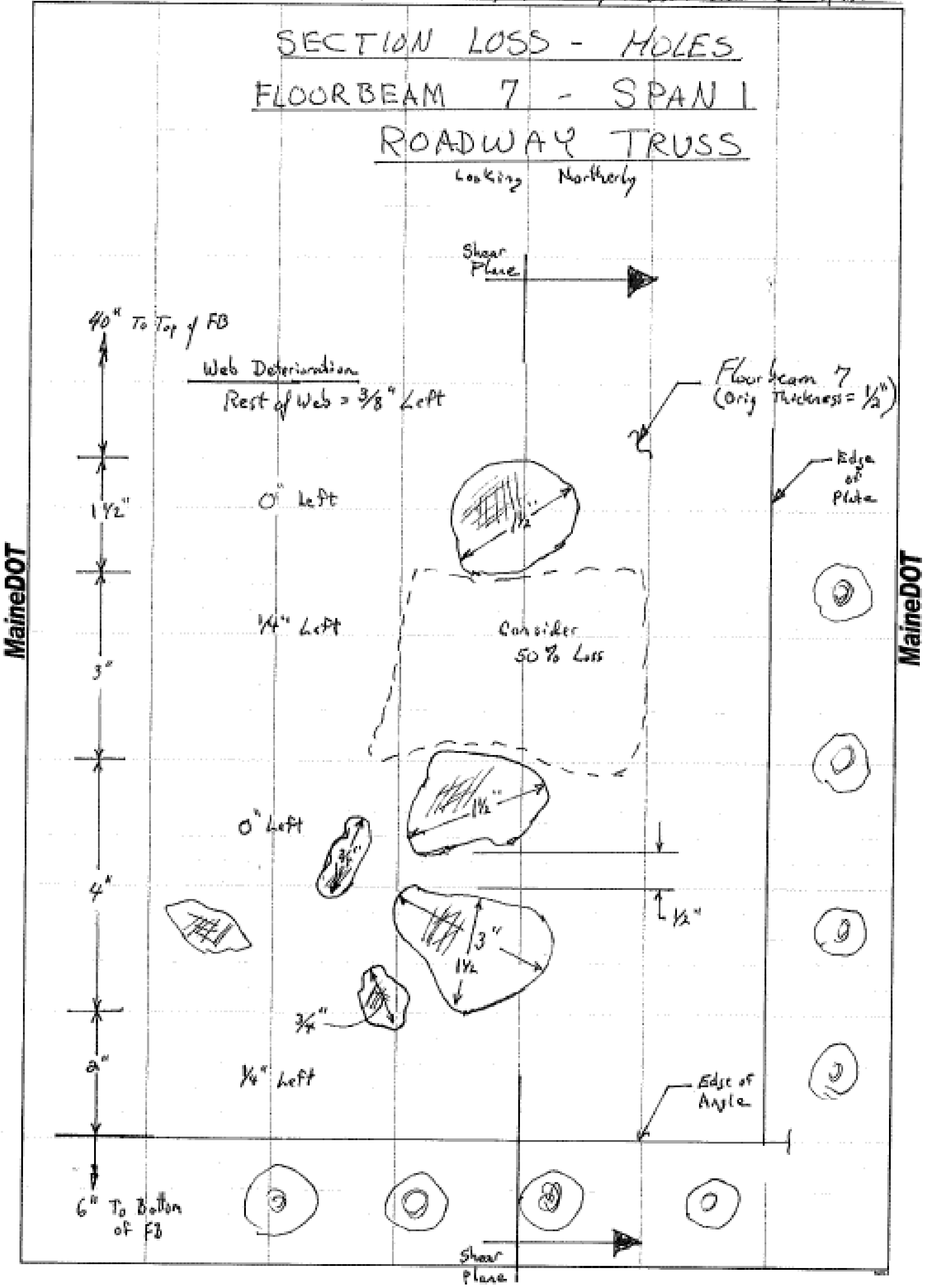
Span 1

Floorbeams 7, 5 and 2 (in order of most deterioration) had holes representing significant section loss to the web.   See table and photos below.

TABLE 1 – SUMMARY OF FLOORBEAM REPAIR – WITHIN 5 YEARS SPAN 1		
Floorbeam #	**Repair Location**	**Closest Stringer**
FB7	Near Conn to Roadside Truss	S8
FB5	Near Conn to Roadside Truss	S8
FB2	Near Conn to Roadside Truss	S8

Photo A1 – Floorbeam 7, Roadway Truss @ Stringer 8 (Exterior)

PRELIM. BY Dowks DATE 8/5/16 PROJ. NO. FILE NO. OF
FINAL CHK. BY DATE LOCATION SHEET NO. OF
ITEM NO. SUBJECT Frank J Wood, Br 2016, Field Sketch FB7 - Span 1



Sketch 1 – Section Loss in Floorbeam 7 Shear Plane. See Photo A1.

Photo A2 – Floorbeam 5, Roadway Truss @ Stringer 8 (Exterior)

Photo A3 – Floorbeam 2, Roadway Truss @ Stringer 8 (Exterior)

Floorbeam 2, bottom flange cover plate is exhibiting advanced corrosion, pack rust, significant loss of rivet head section loss reaching enough concern to warrant a repair in the next 5 years.

Photo A4 – Floorbeam 2 coverplate particularly between Stringer 7 & 6 is an area of concern which will be closely monitored.

Span 2

Span 2 of the bridge is exhibiting more concerns with the riveted bottom flange cover plates at Floorbeams 5, 6 and 8. The other Floorbeams have noted corrosion problems as well, but likely may remain adequate over the next 5 years.

TABLE 2 – SUMMARY OF FLOORBEAM REPAIR – WITHIN 5 YEARS
SPAN 2

Floorbeam #	Repair Location	Closest Stringers
FB5	Cover Plate	S6 – S7
FB6	Cover Plate	S6 – S7
FB8	Cover Plate	S2 – S6

Photo A5 – Floorbeam 5 coverplate has enough deterioration issues to expect a repair within 5 years.

Photo A6 – Floorbeam 6 exhibiting excessive pack rust and rivet deterioration.

Photo A7 – Floorbeam 8 bottom flange midspan plies showing pack rust and deformation which may lead to repair activity within the next 5 years.

Span 3

Span 3 did not appear to have any Floorbeam issues likely to need repair within the next 5 years.

TRUSSES – SIDEWALK & ROADWAY

Spans 1 & 2 - there are no expectations of Truss Repairs within the next 5 years at this time.

Span 3

Span 3 exhibited a rivet deterioration issue which could reach a necessary repair status within the next 5 years. Due to the open grid at the gutter lines of this bridge, a good amount of sand, salts and debris are left covering portions of primary members. The difference of Span 3 and the others is the Sidewalk Truss has continuous rivet head deterioration in the lower tension chord. The panels of Span 3 are 22' in length. Along the lower chord, from Floorbeam 5 to 8 (3 panels @ 22' = 66'), the rivets fastening the lower angle to the plate will need to be monitored. Additionally, at least another portion of this same chord had this same condition from Floorbeam 1 to 2. See table and photos below.

TABLE 3 – SUMMARY OF TRUSS REPAIR – WITHIN 5 YEARS - SPAN 3

Truss	Repair Location	Deteriorated Component
Sidewalk	FB 5 thru 8 & FB 1 to 2	Rivets fastening bottom inside angle of the lower tension chord.

Photo A8 - View of the inside of Sidewalk Truss's lower angle depicts continuous corroding rivet heads and sand, salts and debris occupying the angle flange.

Using a claw hammer the loose corrosion was pounded away from a typical rivet head just described to reveal what remained. The result was a significantly smaller head size. See Photos A9 and A10. Further investigation and computations may be necessary before determining precise remedial action.

Photo A9 - Typical rivet head condition along Sidewalk Truss Lower Chord of Span 3 for at least 100 ' length of angle to plate connection.

Photo A10 – This is the same rivet as above after briefly pounding loose corrosion away. The rivet head size is significantly smaller than the diameter and height of a new rivet head.

Spans 1 & 2

Although Spans 1 & 2 had noted corrosion and accumulated debris, the trusses did not appear to require an immediate repair or one within 5 years at this time.

Photo A11 – This is the general condition view of the inside lower chord truss member collecting rust and debris on the lacing bars.

STRINGERS

Span 1

At Floorbeam 2, Stringer 8 has a corroded hole in its web close to the bottom flange. See Photo A12. It is likely this will develop into a large enough issue to warrant a reattachment repair for this end of the stringer. At the other end of Stringer 8, the connection at Floorbeam 3 is exhibiting enough deterioration issues to expect a reattachment repair within 5 years as well. See Photo A13. Repair of Stringer 8 connection to Floorbeam 5 is also likely. See Photo-A3. Over the next 5 years – Stringer 8 connection to Floorbeam 6, Stringer 3 connection to Floorbeam 6 and Stringer 3 connection to Floorbeam 7 are likely. In all, Span 1 Stringer connection repairs total 6.

TABLE 4 – SUMMARY OF STRINGER CONNECTION REPAIR – WITHIN 5 YEARS
SPAN 1

Stringer #	Connection Component	Closest Truss
S8	Floorbeam 2	Roadway
S8	Floorbeam 2	Roadway
S8	Floorbeam 5	Roadway
S8	Floorbeam 6	Roadway
S3	Floorbeam 6	Sidewalk
S3	Floorbeam 7	Sidewalk

Photo A12 – Stringer 8, Roadway Truss @ Floorbeam 2 has section loss revealing a web hole.

Photo A13 – Looking at the condition of north end of Stringer 8 attached to Floorbeam 3.

Photo A14 – Stringer 8 connection to Floorbeam 6, Roadside Truss, may warrant a reattachment repair within 5 years. Similar connection conditions @ Stringer 3 to Floorbeams 6 and 7.

Span 2

There are no expectations of a Stringer Connection Repairs within the next 5 years at this time.

Span 3

In all, there appears to be approximately 10 Stringer to Floorbeam connections that will likely become a repair item.

TABLE 5 – SUMMARY OF STRINGER CONNECTION REPAIR – WITHIN 5 YEARS
SPAN 3

Stringer #	Connection Component	Closest Truss
S8	Floorbeam 0	Roadway
S8	Floorbeam 1	Roadway
S3	Floorbeam 1	Sidewalk
S8	Floorbeam 2	Roadway
S7	Floorbeam 3	Roadway
S8	Floorbeam 5	Roadway
S3	Floorbeam 5	Sidewalk
S8	Floorbeam 6	Roadway
S3	Floorbeam 6	Sidewalk
S3	Floorbeam 7	Sidewalk

Photo A15 – Stringer 8 connection to Floorbeam 1 showing active corrosion.

Photo A16 – Stringer 8 connection to Floorbeam 2 will likely need repair within 5 years.

Photo A17 – Stringer 8 connection to Floorbeam 5 shows active corrosion.

Photo A18 - Stringer 8 connection to Floorbeam 6. Stringer 3 connection similar condition.

Photo A19 – Stringer 8 connection to Floorbeam 6 will likely require repairs within 5 years.

NEEDLE BEAMS (CROSSBEAMS)

In particularly poor condition are areas in Span 1, Floorbeam 0 to 1 on the Roadway side of the truss and Floorbeams 3 to 4 on the Sidewalk side of the truss. Additionally, areas near Floorbeam 2 at Stringers S7 and S8 are in similar condition. Area between Floorbeams 3 and 4 on the Sidewalk Truss side have multiple Needle Beams with major section loss.

The grid is supported by these needle beams at every 2', distributing the loads effectively. The grid is adequate condition filled with concrete with the exception of 1 foot gutter area (open). The deterioration of the Needle Beams is widespread to the point where the Department will carefully monitor the deck for distress over the next 5 years. Should there be a compromise of the deck, one repair method would be to cut out area topside, lay in similar grid with a steel plate and then fill with asphalt as a temporary repair until a full deck is replaced or a new bridge is built.

Photo A20 – Deteriorated Needle Beams shown here in Span1, Floorbeam 0 to 1, are evident throughout all 3 spans. These beams are 2'-0" on center supporting a grid deck mostly filled with concrete supporting an 1 ½ " to 2" asphalt wearing surface.

Photo A21 – View showing Needle Beams between Stringers 7 and 8. Area between stringers 3 and 4 are similar.

Photo A22 – An open grid approximately 1 foot wide runs parallel to Stringers.

UTILITY BRACKETS

The utility brackets supporting an insulated steel pipe are in generally poor condition. There is a possibility of strengthening a percentage of them within the next 5 years - and will be monitored.

Photo A23 – View of steel insulated utility pipe extending across the bridge on the Roadside Truss side.

Photo A24 – The Utility Brackets are in poor condition with extensive active corrosion.

CANTILEVERED SIDEWALK

The Routine And Fracture Critical Bridge Inspection Report, 8/2012, from Parsons Brinckerhoff considered the top flange of these cantilevered sections to be severely corroded in all 3 spans. It is likely the top flange of the angle was likely removed roughly with a torch when the sidewalk was updgraded/installed. This component can be monitored rather than repaired immediately.

Photo A25 – View of the top half of cantilevered brackets that support the sidewalk.

LATERAL BRACING

Lateral bracing has pack rust issues throughout the structure. Considered essentially for construction forces and loads, these components will be monitored rather than be targeted for repair.

Photo A26 – In general, a good amount of deterioration exists in the lateral bracing system.

RATE OF DETERIORATION

Precisely determining the “rate” of steel bridge deterioration is challenging. NBI ratings provide a good overall consideration. The methodology of comparing earlier recorded conditions to current also shows reasonably good insight. Consider the comparisons below regarding Span 1, Floorbeam 7 conditions changes from 2012 to 2016.

Excerpt from Routine & Fracture Critical Bridge Inspection Report

At Span 1 FB7 below S8 both faces of the web exhibit active corrosion with section loss of 1/8 inch depth over a height of 8 inches and length of 12 inches (Photo 44).

Photo 44 – At Span 1 FB7 below S8 both faces of the web exhibit active corrosion with section loss of 1/8 inch depth over a height of 8 inches and length of 12 inches.

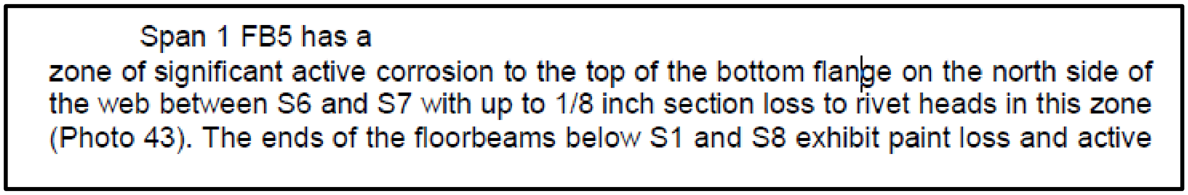
4 YEARS LATER -

Results of Special Bridge Inspection conducted 8/1/2016

Photo A1 – Floorbeam 7, Roadway Truss @ Stringer 8 (Exterior)

12 inches at the bottom of this 42" Floorbeam has nearly 100% section loss.

Excerpt from <u>Routine & Fracture Critical Bridge Inspection Report</u>



Span 1 FB5 has a
zone of significant active corrosion to the top of the bottom flange on the north side of the web between S6 and S7 with up to 1/8 inch section loss to rivet heads in this zone (Photo 43). The ends of the floorbeams below S1 and S8 exhibit paint loss and active

4 YEARS LATER –

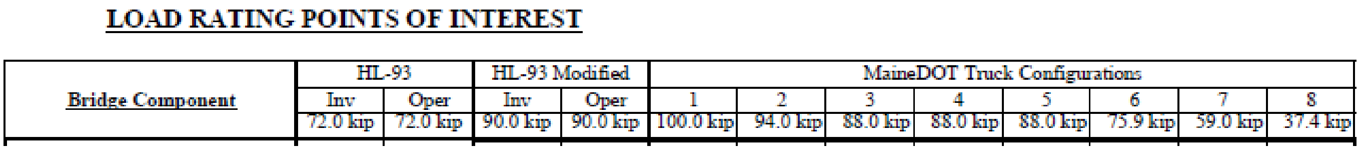
Results of Special Bridge
Inspection conducted 8/1/2016

Photo A2 – Floorbeam 5, Roadway Truss @ Stringer 8 (Exterior)

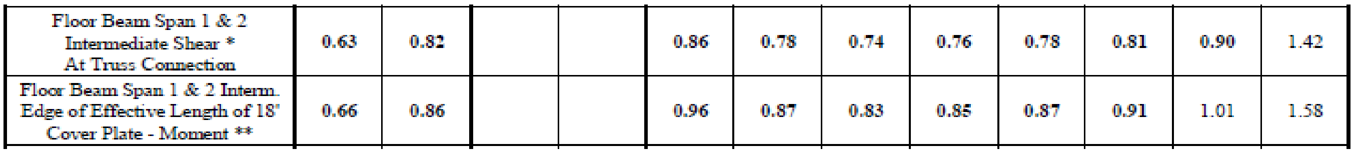
This area of 100% section loss was not targeted as a corrosion problem 4 years ago.

LOAD RATINGS

Listed in the Parsons Brinckerhoff Load Rating conducted March of 2013 is the "Breakdown of Truss Bridge Rating" highlighting the various Bridge Component ratings. Of particular interest are several Floorbeam ratings in Spans 1 and 2.



LOAD RATING POINTS OF INTEREST



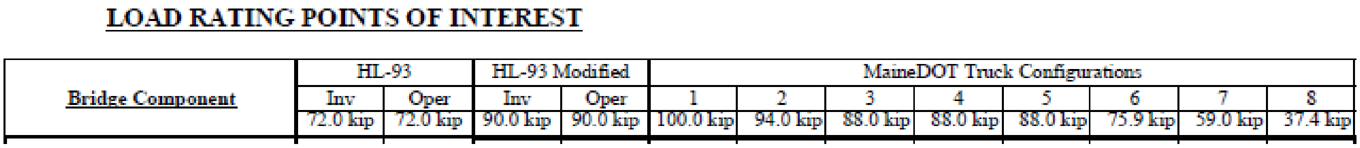
Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Floor Beam Span 1 & 2 Intermediate Shear * At Truss Connection	**0.63**	**0.82**			**0.86**	**0.78**	**0.74**	**0.76**	**0.78**	**0.81**	**0.90**	1.42
Floor Beam Span 1 & 2 Interm. Edge of Effective Length of 18' Cover Plate - Moment **	**0.66**	**0.86**			**0.96**	**0.87**	**0.83**	**0.85**	**0.87**	**0.91**	1.01	1.58

Currently 3 Intermediate Floorbeams in Span 1 have significant section loss in the shear area near the truss connection. The above 2013 table lists 0.63 for HL-93 Inventory Loading. (1.0 is the successful rating target.) In August of 2016 the Department recalculated this Load Rating component with the discovered section loss (Floorbeam 7, see Sketch) resulting in a rating of 0.51. This is a 19% reduction over the past 4 years.

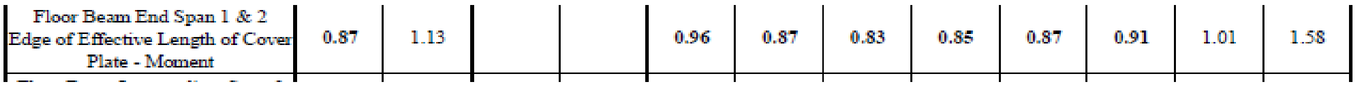
Additional Load Ratings of the Legal Load Configurations were conducted which resulted in a Rating Tonnage of 25 Tons (Configurations 6, 7 &8).

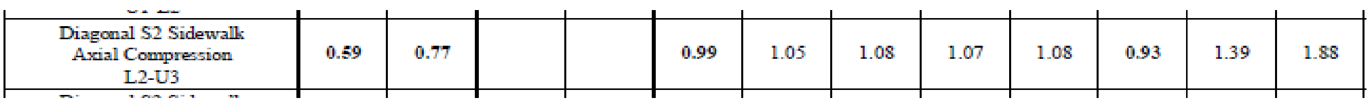
A second area of interest is the pack rust and related rivet stress midspan of 3 Floorbeam Lower Flanges. The 2013 Load Rating above depicts 0.66 for HL-93 Inventory Loading. This portion of the Floorbeam is considered Fracture Critical and is currently targeted for possible repair on 3 Floorbeams within the next 5 years.

There are other load rated components below 1.0 for HL-93 Loading (and scored below 1.0 under the Legal Configurations) in the 2013 Report which our team was alerted to review component current conditions.



LOAD RATING POINTS OF INTEREST





Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Floor Beam End Span 1 & 2 Edge of Effective Length of Cover Plate - Moment	**0.87**	1.13			**0.96**	**0.87**	**0.83**	**0.85**	**0.87**	**0.91**	1.01	1.58
Diagonal S2 Sidewalk Axial Compression L2-U3	**0.59**	**0.77**			**0.99**	1.05	1.08	1.07	1.08	**0.93**	1.39	1.88

REPAIR COST ESTIMATES – 5 YEARS

A repair cost estimate was developed for Floorbeam Repairs in Span 1- Floorbeam 7 and 5. The total for both accomplished during the same time frame was $65K. Using this as a template for other repair costs for Floorbeam midspan strengthening and Stringer end Connections are calculated. These are approximate costs accomplished by M & O Forces and do not include public user costs.

Floorbeams

Floorbeam	Repair Location	Cost per Location
7 – Span 1	Shear Section – End of FB	$35K
5 – Span 1	"	$35K
2 – Span 1	"	$35K
5 – Span 2	Midspan Pos Moment	$100K
6 – Span 2	"	$100K
8 – Span 2	"	$100K
Total		**$405K**

Stringers

No. of Stringer Locations	Span Location	Cost per Span
6	1	$75K
10	3	$130K
Total		**$205**

Truss

Sidewalk Truss Rivets	Allowance
Investigation Of Rivet Condition*	$20K
Repair Place Holder*	$100K
Total	**$120K**

Unsure at this time the extent of rivet deterioration and appropriate retrofit.*

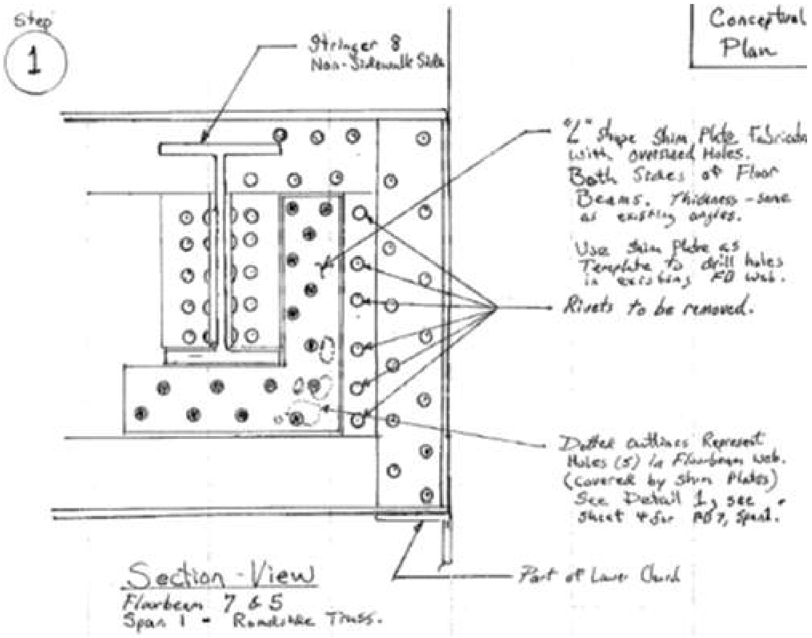
Needle Beams (Crossbeams)/Utility Brackets/Misc

Component	Allowance
Needle Beam (Topside Repair)	$25K
Utility Brackets	$25K
Misc	$25K
Total	**$75K**

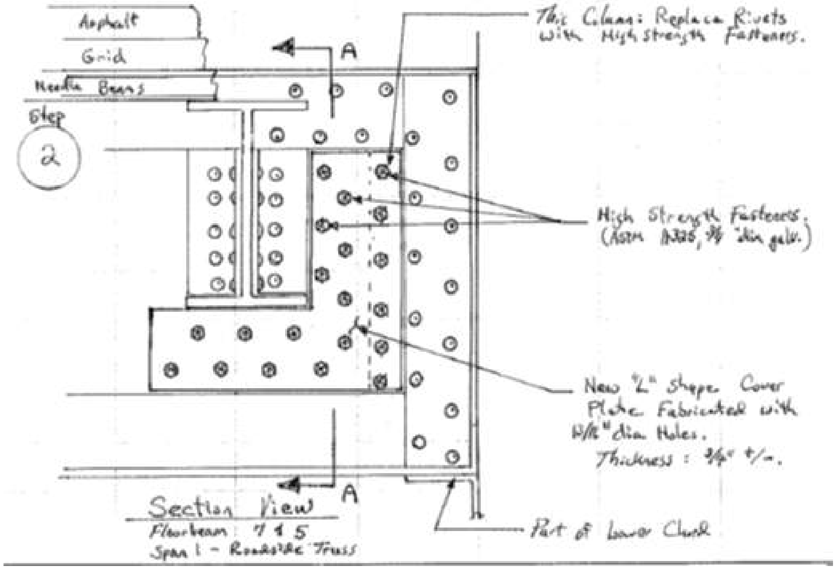
GRAND TOTAL ALL REPAIRS AND ALLOWANCES $805K

REPAIRS

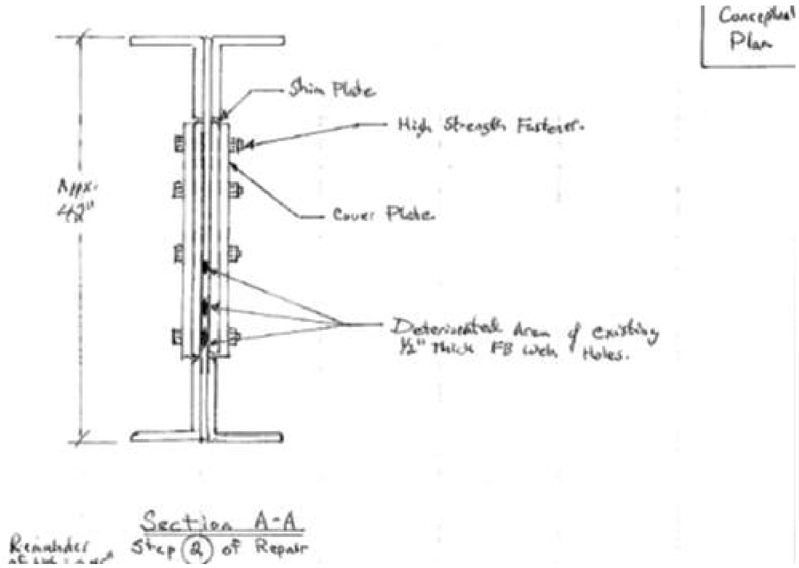
A conceptual design repair was developed for Floorbeam 7 & 5 (and eventually could be applied to Floorbeam 2 and possibly others. Essentially the section loss in the web would be restored by bolting steel plates (shim and cover) on both sides by high strength bolts.



Sketch 2 – Step 1 of Floorbeam Repair.



Sketch 3 – Step 2 of Floorbeam Repair.



Sketch 4 – Step 2, Section View of Floorbeam Repair.

Photo A26 – End result of similar repair to Floorbeam.

Midspan of a distressed Floorbeam caused by pack rust could be temporarily repaired by replacing the rivets with bolts and adding a new steel coverplate. This could possibly be accomplished using a “split coverplate” to accommodate a 2 phase installation rather than removing all of the rivets at one time.

Photo A27 – End result of repair to built-up Floorbeam bottom flange.

The repair method of Stringers vary depending on what deteriorates. Essentially adding steel plates, to both sides of the webs, extending to areas that are in reasonably good condition serves as an acceptable temporary repair. A shim the same thickness of the attaching angle is placed under the cover plate.

Photo A28 – End result of a typical Stringer to Floorbeam connection.

PART B - 30-75 YEAR REHABILITATION

EXECUTIVE SUMMARY

Part B of this report focuses on the rehabilitation necessary to provide an additional 30-75 years of life for MaineDOT Bridge #2016 (Frank J Wood) carrying Routes 201 and 24 over the Androscoggin River in Brunswick and Topsham, Maine. The inspection revealed & confirmed deterioration in all of the critical members of the three (3) span structure. This portion of the report ignores any repairs that may be done as the result of Part A of this report as those are short term fixes that will not last the desired 30-75 year range.

The overall recommendation of this portion of the report is that the entire floor system (roadway deck, floorbeams, stringers, cross (needle) beams, etc.) needs to be replaced, portions of the truss members need to be rehabilitated or replaced, and the entire structure is repainted in order to extend the life of the existing structure 30-75 years. This recommendation is based upon the existing condition and the assumed continued deterioration of the individual elements. The projected rate of deterioration is difficult to quantify, however when comparing the 8/1/16 to 8/2/16 inspection to the Routine And Fracture Critical Bridge Inspection Report by Parson Brinkerhoff from 8/20/2012, the level of deterioration seen in a time span of only 4 years, leads any assumed continued deterioration to result in the failure of multiple critical structural members and connections.

A breakdown of the existing conditions of each element group, as well as an explanation of the rehabilitation recommendations is provided in the following sections.

ELEMENT INSPECTION SUMMARY

FLOORBEAMS (FB)

A total of thirty one (31) floorbeams, eleven (11) in spans 1 & 2 and nine (9) in span 3, make up the transverse supports for the deck and stringers. The floorbeams span between trusses at all vertical members, supporting the six roadway stringers on each side of the floorbeam for a total of twelve (12) stringers supported per floorbeam, with the end floor beams supporting just six (6) stringers. The floorbeams are attached to the trusses by angles that are riveted through the floorbeam web and through the inside gusset plate of the truss. The floorbeams are fracture critical members, as they are not redundant in the transverse direction. The floorbeams were defined as FB0 through FB11 in spans 1 & 2, and as FB0 through FB8 in Span 3.

The floorbeams are in overall poor condition due to the severe corrosion, pack rust and section loss of the beams at their connection to the interior gusset plates and near mid-span.

The ends of every floorbeam, at the connection to the interior truss gusset plates, are exposed to the elements as they are not below the concrete deck. The beam ends are also partially located below the open grid decking in the roadway shoulder, allowing roadway salts and debris to spray and collect on and around the floorbeams. This level of exposure has generally resulted in significant paint failure at the beam ends, with varying degrees of corrosion, pack rust and section loss along the length of the floorbeam, at all floorbeam locations.

Span 1 had the highest level of floorbeam deterioration as several floorbeams have areas of complete section loss. Holes through the web were present in FB2, FB5 & FB7, with the condition of the web on FB7 being significantly worse than the other two. During the 2016 routine inspection mentioned above, the inspector was able to punch holes through the web of FB7 when struck with a hammer on the non-sidewalk truss end of beam. Holes in the web were naturally formed in FB2 & FB5, however the steel near the holes appeared sound when struck during the 8/1/16-8/2/16 inspection. These three floorbeams were the only beams in the structure that had holes through the webs, although the condition of the all of the remaining floorbeams could result in similar section loss in the near future. See Appendix A, photos 12 through 17 for images of the holes through the floor beam webs.

Paint failure and varying degrees of corrosion, pack rust and section loss was present near midspan of all of the floorbeams in all three spans of the bridge. Several floorbeams in span 2 have the worst deterioration, as there is severe corrosion and section loss to the bottom flange and rivets, as well as pack rust between the cover plate and the angles making up the bottom flange of the floorbeams at FB5, FB6 and FB8, (see Appendix A, photos 18 through 20 for images of the midpsan corrosion). These floorbeams had the worst deterioration in the cover plated area of the beams near midspan, however most, if not all of the floorbeams showed corrosion at this location.

The horizontal connection plates, joining the floorbeams and lateral bracing at the floorbeam ends, are overall in poor condition. Roadway salts and debris from the open grid decking in the roadway shoulder collects on these plates resulting in varying degrees of corrosion, section loss and pack rust. The rivets connecting the elements to this plate are in poor condition as a result of the debris collection.

<u>STRINGERS (S1 – S8)</u>

A total of eight (8) stringers make up the longitudinal support of the needle beams and bridge deck. Six (6) stringers are attached to the floorbeams to support the vehicular travel way, while the remaining two (2) attach to the cantilevered sidewalk brackets and support the sidewalk. The stringer connections are comprised of an angle on either side of the stringer web, connected with rivets to both the stringer and their respective supporting members. The stringers are located 3 ½” below the top flange of the floorbeams, and 2” below the top of the sidewalk bracket. The stringers are labeled S1 through S8 starting on the sidewalk side of the structure, with S1 & S2 below the sidewalk and S3 through S8 below the roadway.

The stringers are in overall fair condition based on the current levels of corrosion to the stringers themselves and their connections to their supporting members. The sidewalk stringers (S1 &S2) are in good condition as they are protected by the stay in place formwork used to cast the sidewalk concrete. There are some areas of paint distress and failure, but corrosion is minimal. The roadway stringers (S3-S8) are in overall poor condition, as paint failure with varying degrees of corrosion, pack rust and section loss is present on all roadway stringers, and stringer connection areas.

The exterior roadway stringers in all three spans were consistently in the worst condition due to their location below the open grid decking in the roadway shoulders which allows roadway salts and debris to drop directly onto S3 & S8. As a result, there is varying degrees of corrosion, pack rust and section loss along the length of these stringers and at the stringer to floorbeam connections areas. The worst deterioration was a hole through the web of S8 near its connection to FB2 (see Appendix A, photo 26). Pack rust between the connection angle and the stringer/floorbeam webs has begun to twist the connection angles at several locations. The top and bottom flanges have varying degrees of deterioration, section loss and pack rust.

The interior stringers (S4 to S7) are in better condition than the exterior stringers, however corrosion is still a concern. Generally, most of the interior stringers and stringer connections are in fair condition, however varying degrees of paint failure has occurred at all interior stringer to floorbeam connections, which has resulted in varying degrees of corrosion, pack rust and section loss to the flanges, as well as pack rust between the connection angles and the stringer web.

Please see Appendix A, (photos 23 through 29) for photos of the stringers.

TRUSS MEMEBERS & COMPONENTS

The truss members and components are in overall fair condition, however the general paint failure on the structure has resulted in varying degrees of corrosion, section loss and pack rust. The vertical and overhead members were in good condition, with some collision damage and general corrosion. The horizontal members at the deck level and below are in far worse condition, bringing the overall condition rating down. The bottom chords of both the sidewalk and non-sidewalk trusses are in poor condition with varying degrees of corrosion, pack rust and section loss to the angles, plates and rivets comprising the bottom chords.

The lattice plates connecting the two vertical components of the truss bottom chord are in poor condition as all of the bottom lattice plates have “bowed” due to general corrosion to the plates and pack rust between the connection of the plates and the vertical components. There are multiple top lattice plates that have a lesser degree of “bow” to them, due to general corrosion of the plates and pack rust at the plate to vertical component connection. (See Appendix A, (photos 30 through 34) for images of the “bowed” lattice plates).

The bottom angle of the vertical component of the bottom chord is overall in poor condition. Paint failure has resulted in varying degrees of corrosion, section loss and pack rust to the bottom flange as roadway salts and debris has collected on top of the bottom flange and on the lattice plates. Span 3 has the worst deterioration as a large portion of the bottom chord on the sidewalk truss has severe deterioration, section loss and pack rust to the angles, cover plates and rivets. (See Appendix A, photos 35 through 39 for images of the corroded bottom chord).

The gusset plates are in fair condition, with minimal corrosion and section loss as the paint on the gusset plates has not failed to the levels of other areas of the structure. The largest issue affecting the gusset plates themselves is any pack rust that has formed at the connection between the floorbeams and the interior gusset plates. The condition of a small portion of the rivets through the gusset plates may be suspect, but overall are in fair condition.

The sidewalk brackets that cantilever from the exterior gusset plates to support the sidewalk on the western side of the bridge are in overall good condition. Paint failure is present on all brackets, however the sidewalk has provided protection from weathering, and from roadway salts & debris. The sidewalk is in overall good condition and has experienced far less corrosion than other portions of the structure.

Please see Appendix A, photos 30 through 41 for Truss Members & Component photos

CROSS BEAMS (NEEDLE BEAMS)

The W6x15.5 cross beams (needle beams) run the full width of the deck and are supported by six (6) stringers, connected by welds on each side of the stringer top flange. The needle beams were generally in poor condition based on the level of deterioration to the ends of the cross beams over the exterior stringer bays. There was moderate to severe corrosion and section loss to every needle beam in these exterior bays as water, roadway salts & sands drop through the open grid decking above these bays directly onto the cross beams ends. Knifing of the top and bottom flanges is present on essentially every beam. Measurable section loss in the webs is consistently present in the end bays, with complete web section loss present at several locations. The weld between the needle beams and stringers has deteriorated to the point that many have cracked, and at several locations along the bridge, completely failed.

Please see Appendix A for photos of the cross beams (photos 42 through 47).

LATERAL BRACING (LB)

4”x4”x5/8” angles comprise the lateral bracing spanning diagonally between floor beams (FB) in all three spans. The lateral bracing in spans 1 & 2 consist of two (2) of these angles riveted together to form a “T” shape, while the lateral bracing in span 3 is only one (1) angle. The lateral bracing was generally in poor condition in all three spans of the structure due to the level of corrosion and section loss to the angles and rivets. In spans 1 & 2, pack rust has formed between the angles and has distorted and in some cases completely separated the two angles. In all three spans, there are areas of complete section loss to portions of the angles. Collection of debris and salt spray from the roadway above has resulted in heavy corrosion, pack rust and section loss to the ends of the bracing at their connection to the horizontal connection plate near the ends of the floor beams. The amount of corrosion to these members compromises their effectiveness to the overall structure. The location of the lateral bracing will also limit access to several key structural members of the bridge during any rehabilitation efforts.

Please see Appendix A for photos of the lateral bracing members (photos 48 through 55).

UTILITY HANGERS

The utility hangers are in overall poor condition. Paint failure has resulted in varying degrees of corrosion, section loss and pack rust between the angles comprising the brackets and their connection to the stringers. There is severe deterioration to the rivets connecting these angles. Pack rust and section loss to the angle supporting the conduit and water pipe greatly reduces the capacity of each individual hanger.

Please see Appendix A, for photos of the utility hangers (photos 6 through 8).

SUBSTRUCTURE

The substructure units are in overall fair condition and appear sound. The abutment backwalls have areas of spalling and concrete section loss, and minor cracks. The piers are in good condition as they were rehabilitated in 2006. The bearing pedestals were in fair shape, although the non-sidewalk side bearing at pier 2, for span 3 had shoring in place due to the condition of the adjacent bearing pedestal & bearing.

Please see Appendix A for photos of the shoring at pier 2 (photos 57 and 58).

SUMMARY OF INSPECTION & RECOMMENDATIONS

In order to maintain the existing structure for the next 30-75 years, a great deal of rehabilitation work would need to be done. General paint failure across the entire structure has resulted in the varying degrees of deterioration to multiple key structural, and most importantly fracture critical elements. Below is a summary of the rehabilitation work suggested to provide a minimum of 30 years of additional life for the elements mentioned above. As standard practice, all rivets removed for any work done on the structure are to be replaced by high strength bolts and heavy hex nuts.

GENERAL

The following is recommended as general rehabilitation work to be done:

- Replace the existing concrete filled grid deck with a standard 8” composite reinforced concrete deck
- Repaint the all components of the bridge every 25 to 30 years, once as part of a potential rehabilitation now, and at least once more over the next 30-75 years
- Deck replacement of the rehabilitated structure after 50 years

FLOORBEAMS

The floorbeams are the most important fracture critical elements on the bridge and are in overall poor condition due to the areas of severe corrosion, section loss and pack rust as previously described. This deterioration (holes through webs, section loss to flanges (angles), deteriorated rivets, etc) to such a key component of the structure warrants complete replacement of all the floorbeams and their connections.

Recommendation: Replace all floorbeams and floor beam connections

STRINGERS

The overall condition of the stringers as a whole is currently fair, however the existing state of corrosion to all of the stringer ends and connections would not last 30+ years. The majority of the stringer ends have at a minimum, the early stages of corrosion and pack rust at their connection to the floorbeams and are assumed to not last 30+ years due to the continued presence of chlorides in the steel, thus warranting the replacement of the stringer to floorbeam connections. The exterior roadway stringers (S3 & S8) which have areas of severe corrosion, section loss and pack rust along their entire length warrant replacement based on condition. The interior roadway stringers may be able to be blast cleaned and repainted to last 30+ years, however since the recommendation is to replace all of the existing floorbeams, all of the stringers should be replaced as well. Replacing the floorbeams means either having to temporarily support the twelve (12) stringers connected to each interior floorbeam, or remove and reset the stringers to avoid installing expensive and time consuming temporary structural supports. Once the existing stringers have been detached from the floorbeams it is recommended that they are replaced, as reinstalling 80+ year old steel with varying levels of existing corrosion to new floorbeams would not good engineering judgment/practice. Pre-existing corroded beams would need rehabilitation much sooner than any new steel, which would add an additional maintenance operation to the structure life.

Recommendation: Replace all stringers and stringer connections

TRUSS MEMEBERS & COMPONENTS

The general condition of the existing bottom chord does not bode well for lasting 30-75 years based on the conditions described above. The top and bottom lattice plates should be replaced due to the “bow” observed in every plate. These plates are already under distress and should not be counted on as part of any long term rehabilitation. Replace the bottom flanges of the bottom chords for the full length of the bridge as the flanges are already in poor condition with reduced structural capacity.

Recommendation:

- Replace all steel lattice plates (top and bottom) on both trusses
- Replace the bottom flange components on the bottom chord of the truss
- Replace all deteriorated rivets with high strength bolts and hex nuts

CROSS BEAMS (NEEDLE BEAMS)

The overall condition of the needle beams is too poor to warrant any rehabilitation. A proposed 30-75 year rehabilitation to the existing bridge will include a new composite structural concrete deck to replace the existing concrete filled steel grid deck. The concrete filled steel grid deck is welded to the needle beams, so removing the needle beams as part of the deck removal will aid in the constructability of a rehabilitation project. A new deck would be a composite concrete deck, meaning shear studs would be installed on the floorbeams and stringers, so the needle beams will no longer be necessary.

Recommendation: Remove all cross beams (needle beams)

LATERAL BRACING

The overall condition of the lateral bracing is too poor to warrant any rehabilitation. The levels of section loss and pack rust between angles comprising the lateral bracing would require replacing multiple bays of bracing. Once the act of removing and replacing bracing is required for the structure, it should be carried through the remaining bays as the bracing will likely not last 30+ years, and the location of the bracing in the bays limits construction access to more important structural components of the bridge. A new composite concrete deck would provide additional lateral support to the structure, so lateral bracing would not be necessary as part of a rehabilitation.

Recommendation: Remove all lateral bracing members

UTILITY HANGERS

The utility hangers that support the conduit on the western side, and the water pipe on the eastern side of the structure are in overall poor condition. These hangers are extremely redundant, as they appear to be spaced every 3-4' along the structure, however their condition will not last 30-75 years. These hangers are currently supported by the first two (2) roadway stringers on each side of the structure (S3-S4 & 37-S8), and since the recommendation is to replace all of the stringers, the utilities will need to be temporarily supported during the rehabilitation work. This may not be at a direct cost to MaineDOT, but it should still be noted that it will affect the constructability of any rehabilitation project.

Recommendation: Replace all utility hangers

SUBSTRUCTURE

The substructure is in overall fair condition as there was a substructure rehabilitation done in 2006. A 30-75 year fix is going to require a minimum of one (1) or two (2) additional rehabilitations to the substructure units, with the first major rehabilitation likely required roughly 10+ years into the rehabilitated life of the structure. The abutment backwalls should be replaced during a proposed deck replacement, as this will be the best time for construction access and the top 18" will need to be replaced as part of a new joint installation anyways. The bearing and bearing pedestal at pier 2 for span 3 should be rehabilitated such that the shoring is no longer necessary. This will likely require casting a new bearing pedestal as well as removing, refurbishing, and resetting the bearing in question.

Recommendation:

- Reconstruct the abutment backwalls during the deck replacement work
- Replace the bearing pedestal at pier 2 for span 3
- Remove, refurbish and reset (or replace if not able to refurbish) the non-sidewalk truss bearing at pier 2 for span 3.

Photographs

General Bridge Photos

Photo #1: General view of the bridge (from the Brunswick looking north) (8/1/16)

Photo #2: General view of the underside of the non-sidewalk side of the bridge (eastern side, Span 1 looking north) (8/1/16)

Photo #3: General view of the underside of the sidewalk side of the bridge (western side, Span 2 looking south) (8/1/16)

Photo #4: General view of the superstructure section (8/1/16)

Photo #5: General view of the Floor Beam, Stringer & Vertical Truss member connection (8/1/16)

Photo #6: General view utility conduit supported by a hanger system attached to the stringers (western side of structure looking south) (8/1/16)

Photo #7: General view of utility pipe supported by a hanger system attached to the stringers (eastern side of structure looking north) (8/1/16)

Photo # 8: General view of paint loss and corrosion to bays below the open grid decking in the travel way shoulders on the non-sidewalk side (eastern) of the bridge (8/1/16)

Photo # 9: General view of paint loss and corrosion to bays below the open grid decking in the travel way shoulders on the sidewalk side (western) of the bridge (8//1/16)

Floor Beams (FB)

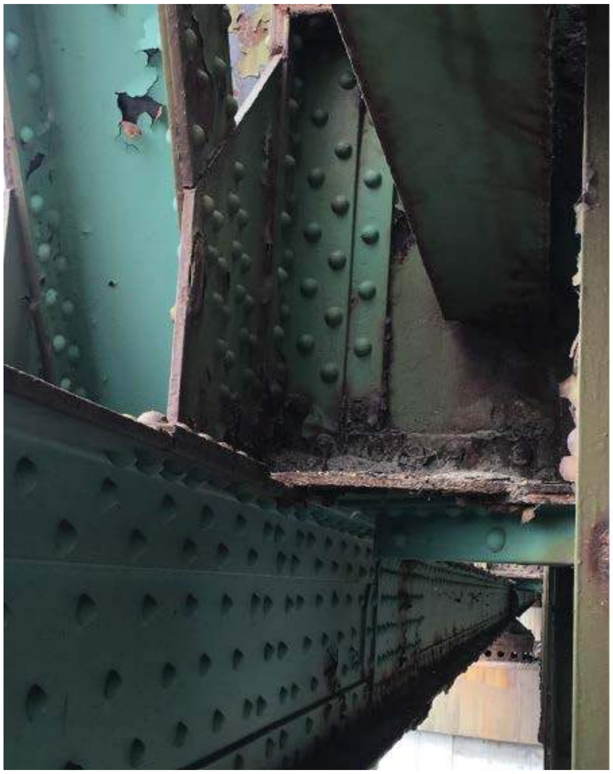


Photo # 10: General view of floor beam to truss gusset plate connection (8/1/16)

Photo # 11: General view of typical paint loss & debris collection at floor beam ends (8/1/16)

Photo # 12: General view of corrosion & hole through the web on the non-sidewalk (eastern) end of FB2 in span 1 (8/1/16)

Photo # 13: Closer image of hole through the web of FB2 in span 1 (8/1/16)

Photo # 14: General view of corrosion and hole through the web of the non-sidewalk (eastern) end of FB5 in span 1 (8/1/16)

Photo # 15: General view of corrosion and holes through the web of non-sidewalk (eastern) end of FB7 in span 1 (8/1/16)

Photo # 16: Closer view of holes through the web of FB7 (8/1/16)

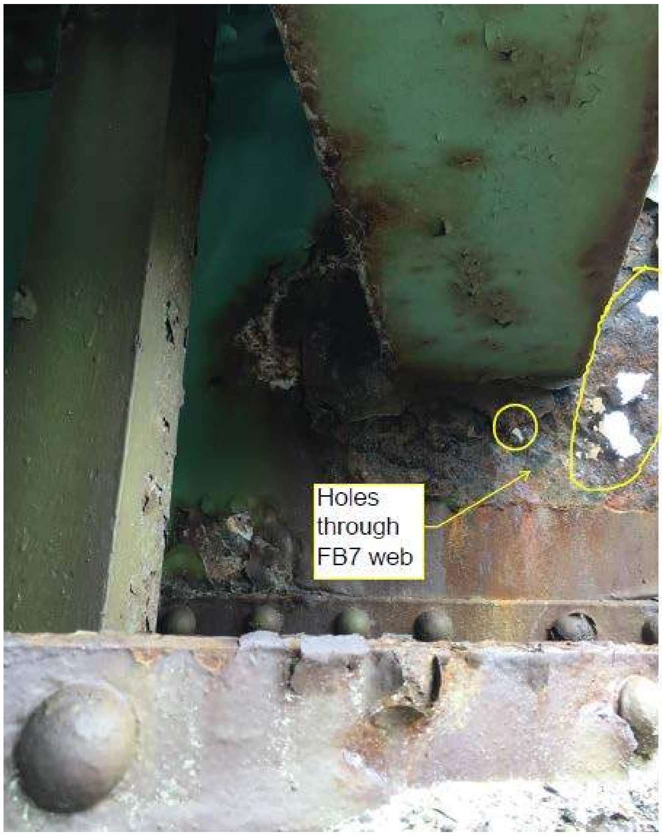


Photo # 17: Closer view of holes through the web of FB7 (8/1/16)

Photo # 18: Severe corrosion & pack rust between the bottom flange of the floor beam and the cover plate near midspan of FB5 in span 2, (span 2 FB4, FB6 & FB7 similar) (8/2/16)

Photo # 19: Severe corrosion to the rivets as well as corrosion & pack rust between the bottom flange of the floor beam and the cover plate near midspan of FB5 in span 2 (span 2 FB4, FB6 & FB7 similar) (8/2/16)

Photo # 20: Advanced deterioration of rivets near midspan of FB7 in span 2 (8/2/16)

Photo # 21: General view of floor beam end in span 3 (8/2/16)

Photo # 22: Top flange corrosion & section loss of FB 5 in span 3 (8/2/16)

Stringers (defined as S1 through S8)

Photo #23: Typical view of the sidewalk stringers (S1 & S2) on the upstream side of the bridge (western side, looking north) (8/1/16)

Photo #24: Typical exterior stringer (S3 & S8) to floor beam connection (Typ. all spans) (8/1/16)

Photo #25: Typical view of bottom flange of S3 & S8 in spans 1 & 2 (8/1/16)

Photo #26: Hole through the web of S8 at FB2 in span 1 (8/1/16)

Photo #27: Moderate corrosion to S3 in span 3 near FB4 (8/2/16)

Photo #28: Moderate corrosion with pack rust between the connection angles and floor beam, at S8 in span 3 near FB5 (8/2/16)

Photo #29: Typical interior stringer (S4 through S7) to floor beam connection (Typ. all spans) (8/1/16)

Truss Members & Components

Photo #30: Typical corrosion of the bottom chord of the truss members, showing “bowing” of the Lattice plates due to pack rust (8/2/12)

Photo #31: Corrosion of lattice plate on bottom chord of non-sidewalk truss in span 1 (8/1/16)

Photo #32: “Bowing” of lattice plates on the bottom chord of the sidewalk truss in span 3 (8/2/16)

Photo #33: “Bowing” of lattice plates on the top of the bottom chords of the sidewalk truss in span 3 (8//16)

Photo #34: Pack rust between lattice plate and bottom angle on the the bottom chord of the sidewalk truss in span 3 (8//16)

Photo #35: Bottom chord of non-sidewalk (eastern) truss, in span 1 near FB 2 (8/1/16)

Photo #36: Corrosion to the bottom flange of the bottom chord of the sidewalk side truss in span 3 (western side of the bridge looking north) (8/2/16)

Photo #37: Pack rust between plates on the bottom chord of the non-sidewalk truss in span 3 (eastern side of the bridge) (8/2/16)

Photo #38: Severely deteriorated rivet heads on the bottom chord of the sidewalk side truss, span 3 (8/2/16)

Photo #39: Remnants of rivet heads along the top of the bottom flange of the bottom chord of the sidewalk truss (western) in span 3 (8/2/16)

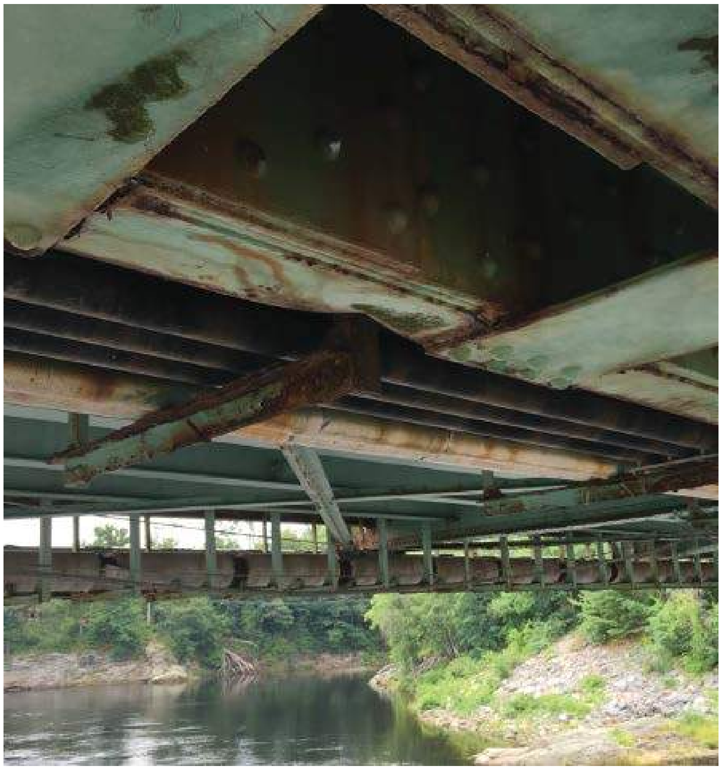


Photo #40: “Dent” to the interior member making up the bottom chord on the sidewalk (western) truss, located near midspan of span 1, likely due to impact of debris during extreme high flood waters (8/1/16)

Photo #41: “Dent” to the exterior member making up the bottom chord on the sidewalk (western) truss at the same location of the previous photo (8/1/16)

Cross Beams (Needle Beams)

Photo #42: Typical view of cross beams (needle beams) in the exterior bay between S3 & S4, S7 & S8 similar (8/1/16)

Photo #43: Typical view of cross beams (needle beams) in the first interior bays from S4 through S7 (8/1/16)

Photo #44: Moderate corrosion to cross beams (needle beam) (Typ.) (8/1/16)

Photo #45: Severe corrosion to a cross beam (needle beam) near FB 1 in span 1 of non-sidewalk (eastern) truss (8/1/16)

Photo #46: Severe corrosion to a cross beam (needle beam) on the sidewalk (western) truss (8/1/16)

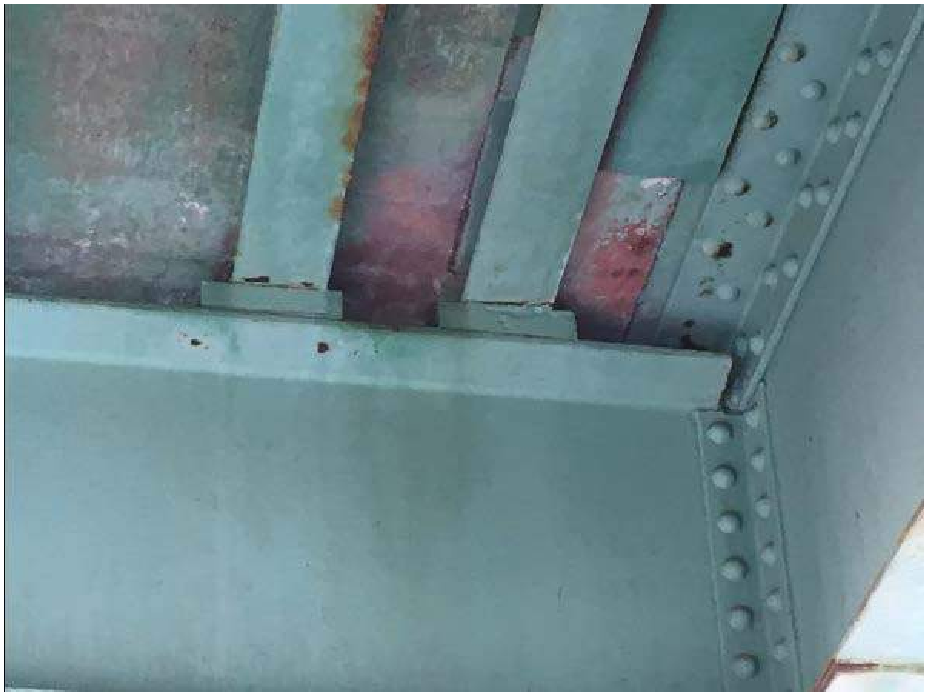


Photo #47: Failed weld between needle beam and an interior stringer in span 1 (8/1/16)

Lateral Bracing (LB)

Photo #48: General view of lateral bracing spanning between utility supports in span 1 & 2 (8/1/16)

Photo #49: Typical view of lateral bracing connection to bottom horizontal gusset plate in spans 1 & 2 (8/1/16)

Photo #50: Severely corroded lateral bracing member view from top of double angle in spans 1 & 2 (8/1/12)

Photo #51: Severely corroded lateral bracing member, view from the bottom of the double angle connections in Spans 1 & 2 (8/1/12)

Photo #52: General top view of lateral bracing span (inverted) from non-sidewalk truss to sidewalk truss in spans 1 & 2 (8/2/16)

Photo #53: General top view of lateral bracing span from sidewalk truss to non-sidewalk truss in spans 1 & 2 (8/2/16)

Photo #54: Corroded lateral bracing end at its connection to the horizontal plate at FB8 span 2 (8/2/16)

Photo #55: Pack rust and section loss to the lateral bracing near FB6 in span 1 (8/1/16)

Bearings

Photo #56: Typical view of the bearings at pier 1 (8/1/16)

Photo #57: Shoring of non-sidewalk bearing, pier 2, span 3 (looking southwest) (8/2/16)

Photo #58: Shoring of non-sidewalk bearing, pier 2, span 3 (looking south) (8/2/16)

ROUTINE AND FRACTURE CRITICAL BRIDGE INSPECTION REPORT

Maine DOT CONTRACT NO. 20111026000000002063

BRIDGE # 2016-FRANK J. WOOD BRIDGE
US 201 & RT. 24 OVER THE ANDROSCOGGIN RIVER
BRUNSWICK, CUMBERLAND COUNTY
MAINE DOT REGION 1 (SOUTHERN)

Start Date of Inspection: 05/18/2012
Finish Date of Inspection: 08/20/2012

Prepared For
Maine Department of Transportation
Bridge Maintenance Division

MaineDOT

Prepared By:

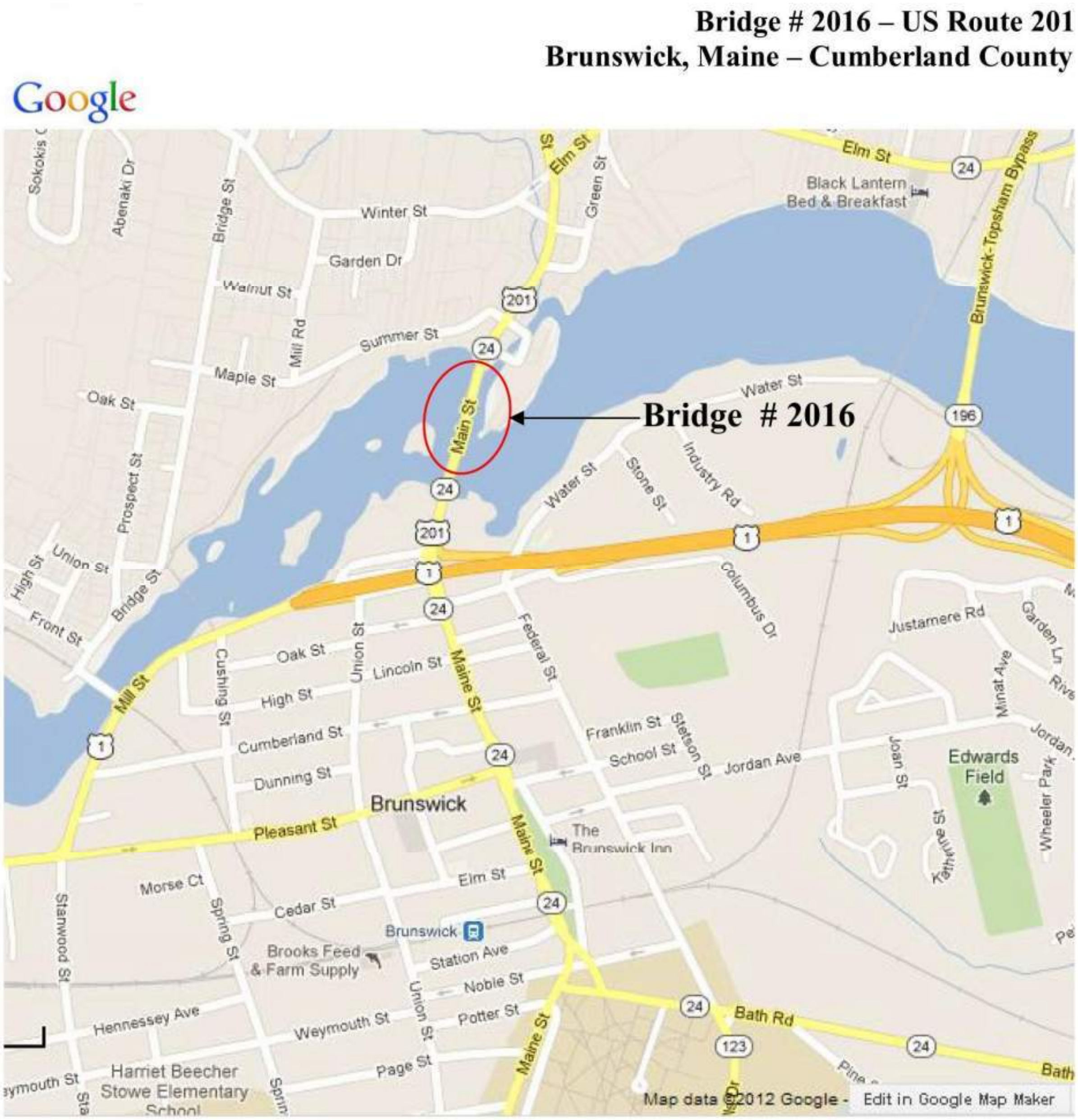
PARSONS BRINCKERHOFF

Parsons Brinckerhoff, Inc.
650 Elm Street
Manchester, NH 03101

Table of Contents

LOCATION MAP

(Located: Brunswick, Maine)



Bridge # 2016 – US Route 201
Brunswick, Maine – Cumberland County

BRUNSWICK, CUMBERLAND COUNTY, MAINE

I. Executive Summary

The focus of this report is the presentation of the routine and fracture critical inspection findings for Maine DOT Bridge No. 2016 (Frank J. Wood Bridge) carrying US 201 and Maine Rt. 24 over the Androscoggin River in Brunswick, Cumberland County, Maine.

The bridge inspection was commenced May 18 to May 20 and was completed on August 20, 2012. No underwater inspection was performed. The routine inspection included examination of the gusset plates to determine any section loss and check the straightness of plates. Since the gusset plate thicknesses and plate dimensions for Bridge No. 2016 are not included in the original construction and/or shop drawings made available by Maine DOT, detailed field measurements of the gusset plates were required to be collected as part of the inspection in order to perform a live load rating analysis.

A. Significant Findings

1. All four deck joints have previous bituminous patch repairs and more recent concrete patch repairs (Photos 7-10). The south abutment joint has required emergency repairs on two separate occasions in the last two years. The transverse cross-beam at the south abutment exhibits severe advanced corrosion with section loss and perforation below the northbound lane. Between stringers S6 and S8, there are 2 locations of extensive section loss. A three inch width of the bottom flange south edge is missing over a 4 foot length between S7 and S8 (Photo 59). Within this same zone there is also section loss to the opposite north bottom flange edge with loss of up to 2 inches in width over a length of 12 to 18 inches. The lower two inches of the web was also perforated in a portion of this 4 foot length. A similar condition was observed at a separate location along the same member located between S6 and S7.

   Recommendation: Replace the four deck joints by performing full depth deck removal and replacement of a four foot strip centered at each joint, including reconstruction of the top of the abutment backwalls at each end of the bridge. Replace the severely corroded and perforated south abutment cross-beam in conjunction with the deck joint reconstruction.

2. The paint condition is poor throughout the superstructure with measurable section loss to six main truss members and four floorbeams (See Table 1 for locations). Active corrosion is present in all of the different types of superstructure elements (Photos 13, 23-26, & 42) and is especially significant in the floor system members, including the cross-beams (Photos 60-63). Distortion/out of plane deformation of upper chord lacing bars along with bowing of lower chord tie plates was observed, indicating the effects of pack rust are becoming a concern. Heavy debris accumulations up to four inches deep were observed on the top of the lower chord bottom plate and up to two inches deep were observed on the top of the lower chord outboard channel where electrical conduits and an electrical junction box combine to trap and collect dirt droppings (Photo 20). A similar condition was observed in the Span 1 roadway truss L4 joint where minor exfoliated rusting was

noted over the lower three inches for the full length of the outboard gusset plate due to the moisture and dirt accumulation caused by the electrical conduits (Photo 21). Tightly packed debris accumulations 1 ½ inches in depth were also observed between an electrical conduit and the top face of the roadway truss lower chord in Span 3.

Recommendation: In order to correct the on-going active corrosion of the truss members and other steel superstructure components and extend the service life of the bridge, the bridge superstructure should be cleaned and painted by removing the existing coatings down to bare metal and applying a coating system in accordance with Maine DOT standard specifications. Of particular concern are the corroding lower truss chords, floorbeams, and deck support members. The cleaning operations prior to painting should include cleaning and caulking at the bowed lower chord top face tie plates to prevent future water intrusion and advancement of the bowing deformation.

3. The sidewalk truss verticals and diagonals have welded plates added at the sidewalk level which are intended to partially block the openings at the sidewalk penetrations (Photos 37 & 38). These welded plates represent fatigue-sensitive details in FCM’s wherever the base members are FCM’s. Even though some of these plates are no longer in place the weld remnant is still present, so the location remains as an FSD in a FCM.

   Recommendation: Re-inspect all welded sidewalk penetration plates in the sidewalk truss vertical and diagonal FCM’s during future biennial bridge inspection cycles. Check for any indications of fatigue cracking initiation as part of the regularly scheduled bridge inspection cycles.

4. The end floorbeams have partial height welded stiffeners and full height jacking stiffeners which represent fatigue-sensitive details in FCM’s. In addition the north face of the Span 1 end floorbeam FB0 has a welded plate stringer bearing stiffener below S3 which constitutes a fatigue-sensitive detail in a FCM (Photo 53).

   Recommendation: Re-inspect all end floorbeam welded stiffener fatigue-sensitive details in the FCM’s during future biennial bridge inspection cycles. Check for any indications of fatigue cracking initiation as part of the regularly scheduled bridge inspection cycles.

5. The top flange angles of the floorbeam sidewalk cantilever brackets exhibit severe section loss at numerous locations throughout all three spans. In the most severe locations, this section loss amounts to 100% of the area of the horizontal legs of the top flange angles on both sides of the web between S1 and S2 (Photo 54).

   Recommendation: Perform repairs to all severely corroded floorbeam sidewalk cantilever brackets in all three spans to restore the lost top flange area between S1 and S2. Remove the existing top flange angles on both sides of the web and install bolted angles fastened through the existing holes in the web with a flange area equivalent to the original top flange angles.

6. At the north abutment the second cross-beam from the deck joint has severe section loss to the bottom flange. Section loss, perforation, and delamination to

two adjacent grid deck cross-beams was observed in Span 1 near FB2 between S7 and S8 where the underside of the bottom flange has up to 3/16 inches of material lost over two inches of the edge of the bottom flange along a length of 16 inches (Photo 60). Perforations at the flange edges and in the bottom flange of the cross-beams was observed in Span 3 near L6 in the ends of the cross beams (Photo 63). Two cross-beams have perforations of the outer one inch along the flange edges in this vicinity.

Recommendation: Restore the section loss to the five specified cross-beams by making bolted plate repairs to the bottom flange.

7. The roadway truss bearing at the south abutment has a loose anchor bolt which was removed, photographed and loosely re-inserted by the inspection crew (Photo 73). The expansion slots in the base of the upper shoe are completely filled with debris accumulations which preclude normal thermal movement (Photo 74).

   Recommendation: Drill and grout a new anchor bolt at the south abutment roadway truss bearing. Clean the dirt and debris accumulations from the south abutment bearings expansion slots and lubricate the bearings to restore uninhibited thermal movement. Evaluate and if necessary repair the bent anchor bolts at the south abutment roadway truss bearing (See Photo 75).

8. The Span 2 sidewalk truss fixed bearing at Pier 2 is missing a nut on the southwest anchor bolt (Photo 76).

   Recommendation: Install a nut on the southwest anchor bolt at the Pier 2 Span 2 sidewalk truss fixed bearing.

9. The south abutment backwall has a wide 2 foot long horizontal crack near the wingwall corner at approximately 4 feet above the bridge seat (Photo 77).

   Recommendation: Repair the crack in the south abutment backwall by epoxy-injection.

10. The older stone masonry portions of the south abutment exhibit remnants of missing timber bracing anchors and possible signs of outward shifting of a few of the stones (Photo 78).

    Recommendation: Monitor the south abutment stone masonry every 3 months to determine if there is any shifting of the individual stones.

11. There is a large 3 foot by 3 foot by 3 inch deep spall in the lower portion of the north abutment backwall around the edges of the utility conduit penetration and extending behind the sidewalk truss (Photos 80 & 81).

    Recommendation: Perform concrete spall repair to the one north abutment backwall location by removing and replacing unsound concrete.

12. Pier 2 has a wide diagonal crack in the southwest corner of the sidewalk truss bearing pedestal which extends for the full height of the concrete pedestal (Photo 82).

    Recommendation: Reconstruct the corner of the Pier 2 sidewalk truss bearing pedestal by removing and replacing unsound concrete.

13. Two conduits presumed to supply power for bridge lighting were observed to have a three inch separation in the conduit in Span 1 along the top face of the roadway truss between L9 and L10 which results in exposed electrical cables (Photo 86).

    Recommendation: Repair the separation in the Span 1 roadway truss conduit run between L9 and L10 to seal and protect the electrical cables.

B. Condition Summary

The results of the inspection indicate that the bridge is overall in fair condition.

The **deck** is in fair condition overall, rated a 5. However the deck joints are in poor condition, rated a 4. All four deck joints have previous bituminous patch repairs and more recent concrete patch repairs. The south abutment joint appears to have been retrofitted with an open plate type joint which is continuing to require ongoing maintenance and repairs. In numerous locations the exposed underside of main longitudinal grid deck bearing bars were visible and exhibited active corrosion. This condition is mostly confined to the outer edges of the deck along S3 and S8.

The **superstructure** is in fair condition overall, rated a 5. However the paint is in poor condition, rated a 4. There is measurable section loss to six main truss members and four floorbeams. Previously arrested or active pack rust was observed in several floorbeams between the bottom flange cover plate and the edges of the bottom flange angles. Active corrosion is present in all of the different types of superstructure elements and is especially significant in the floor system members, including the cross-beams. Several of the cross-beams have advanced section loss with perforations in the bottom flange. Distortion/out of plane deformation of upper chord lacing bars along with bowing of lower chord tie plates was observed, indicating the effects of pack rust are becoming a concern. Section loss, perforation, and delamination to two adjacent grid deck cross-beams was observed where the underside of the bottom flange has up to 3/16 inches of material lost over two inches of the edge of the bottom flange along a length of 16 inches. The top flange angles of the floorbeam sidewalk cantilever brackets exhibit severe section loss at numerous locations throughout all three spans. In the most severe locations, this section loss amounts to 100% of the area of the horizontal legs of the top flange angles on both sides of the web between S1 and S2. Heavy debris accumulations up to two inches deep were observed on the top of the lower chord outboard channel at two locations where electrical conduits and/or an electrical junction box combine to trap and collect dirt droppings. Severe pack rust with active corrosion was observed in a few of the lower lateral bracing angles.

The **substructure** is in satisfactory condition, rated a 6. The older stone masonry portions of the south abutment exhibit remnants of missing timber bracing anchors and possible signs of outward shifting of a few of the stones. There is a large 3 foot by 3 foot by 3 inch deep spall in the lower portion of the north abutment backwall around the edges of the utility conduit penetration and extending behind the sidewalk truss.

II. Introduction

The focus of this report is the presentation of the routine and fracture critical inspection findings for Maine DOT Bridge No. 2016 (Frank J. Wood Bridge) carrying US 201 and Maine Rt. 24 over the Androscoggin River in Brunswick, Cumberland County, Maine.

The bridge was inspected on May 18, 19, 21-23 and August 20, 2012. The inspection team over the course of the various dates of inspection was comprised of Roger Stanley, P.E. (TL), Paul Armano, P.E. (ATL), Wen-Shang Liu, P.E. (ATL), Helena Charron (ATL), Ben Holsapple, E.I.T. (ATL), and Adam Stockin, P.E. (ATL).

In addition, as specified in the contract, red line markups of the previous Maine DOT Structure Inventory & Appraisal forms have been provided as separate attachments to this report.

Bridge Description

Bridge No. 2016 is a two-lane three span structure consisting of three riveted steel Parker through truss spans (Photo on front cover of Report). Pier #1 is constructed along a skew of 15 degrees while all other substructure units are square (i.e. perpendicular to the bridge alignment). Spans #1 and #2 are similar in span length and truss depth with variations as required to accommodate the skewed orientation of Pier #1. The sidewalk truss in Span #1 measures 314’-5 1/2” from center to center of bearings. The roadway truss in Span #1 measures 305’-6 1/2” from center to center of bearings. The roadway truss in Span #2 measures 314’-5 1/2” from center to center of bearings. The sidewalk truss in Span #2 measures 305’-6 1/2” from center to center of bearings. Both the sidewalk and roadway trusses for Span #3 measure 174’-6” from center to center of bearings. The bridge was built in 1931 and currently carries two 12 foot wide traffic lanes along with a 3 foot wide outer shoulder in each direction, providing a total roadway width of 30’-0”. There is a 5’-3 5/8” (+/-) clear sidewalk located outboard of the west fascia sidewalk truss along the upstream side. The total out to out width of the bridge roadway and sidewalk measures approximately 41’-0”. The trusses are spaced at 32’- 11” center to center.

Abutment #1 was partially constructed on top of stone masonry abutments from an earlier bridge at this same site. Both abutments and both piers are cast-in-place reinforced concrete. All four substructure units are supported on spread footings presumed to be founded on rock. The bridge runs from south to north, carrying US 201 and Maine Rt. 24 over the Androscoggin River. For purposes of the inspection and Report documentation, plan north has been established to match the original construction plan orientation. The waterway flows downstream to the east.

The SIA Report for the bridge indicates that the substructure and sidewalk were rehabilitated in 2006. However there are no plans available for the work performed at that time. The original construction plans show a solid reinforced concrete deck slab, however the existing deck consists of a concrete filled steel grid deck supported on transverse cross-beams with a bituminous wearing surface. Therefore, it is presumed that the original deck was replaced at an unknown date. No plans are available for the currently existing concrete filled steel grid deck. The original steel plate type deck joints above both abutments and both piers have been replaced at an unknown date with either a closed compression seal type joint at the north abutment or strip seal type joints at the other three remaining joint locations. The south abutment joint appears to have been retrofitted with an open plate type joint which is continuing to require ongoing maintenance and repairs.

Maine DOT maintenance forces performed a number of repairs during the four days in which the bridge inspection was performed. The work included pressure washing and flushing the lower chords in both truss lines through the strip of open grid at each curbline. Patching of the concrete headers at the deck joints was also underway during the inspection. An unforeseen south abutment deck joint repair was required by welding several of the deck joint plates when it was determined that timber temporary shoring would not be a feasible interim repair.

Inspection Access

The hands-on and visual Routine inspection was performed using an Underbridge Inspection Unit (UB-50) to inspect the underside of the deck, floorbeams, and lower chord members. A Ford E350 33-foot aerial lift bucket truck was used to reach the upper portion of the truss (above the deck), sway frames, and top chord lateral bracing members. In addition, the taller portions of the truss which are beyond the maximum reach of the 33 foot lift were inspected with an Elliot L60 aerial lift bucket truck which has a 60 foot working height. The truss upper chords were inspected from the deck as well as from the aerial lift. The abutments and wingwalls were also inspected from the Underbridge Inspection Unit.

Prior to the inspection of the bridge, advanced notification to the appropriate authorities was required before performing any lane closures. Maine DOT provided flaggers to implement temporary single lane closures during this inspection. Maine DOT also provided advance notification to advise motorists of the scheduled bridge restrictions. For this particular bridge, the DOT elected to close the bridge to all northbound traffic which was detoured to the separate downstream crossing between the hours of 8 AM and 8 PM. During these hours, while the inspection equipment utilized one closed lane of the bridge for inspection access, the southbound traffic was maintained in the other available lane. Maine DOT also implemented provisions for maintaining pedestrian access during times when the Underbridge Inspection Unit (UB-50) was operating from the southbound lane and deployed across the sidewalk on the west side of the structure.

Fracture Critical Members on this bridge are the tension members of the non-redundant trusses, such as lower chords and selected verticals and diagonals, along with the bottom flange and portions of the floorbeam web that are in tension.

The general layout, framing and orientation of the bridge may be viewed on the Fracture-

Critical Member (FCM) diagrams located in the following section of this Report. Numbering of truss panel points used for the inspection is in accordance with the FCM diagrams. For Spans 1 & 2, L0 corresponds to the south end of the span and L10 corresponds to the north end. For Span 3, L0 corresponds to the Pier 2 end of the span while L8 corresponds to the Abutment 2 end. Stringer lines designations were assigned looking ahead station from south to north and numbering from left to right. Following this convention, the west fascia stringer is S1, stringers S3 to S8 are located between trusses, and the two sidewalk stringers located outboard of the west sidewalk truss are S1 and S2, respectively.

Additional bridge data can be found in the Maine DOT Structure Inventory and Appraisal Sheet.

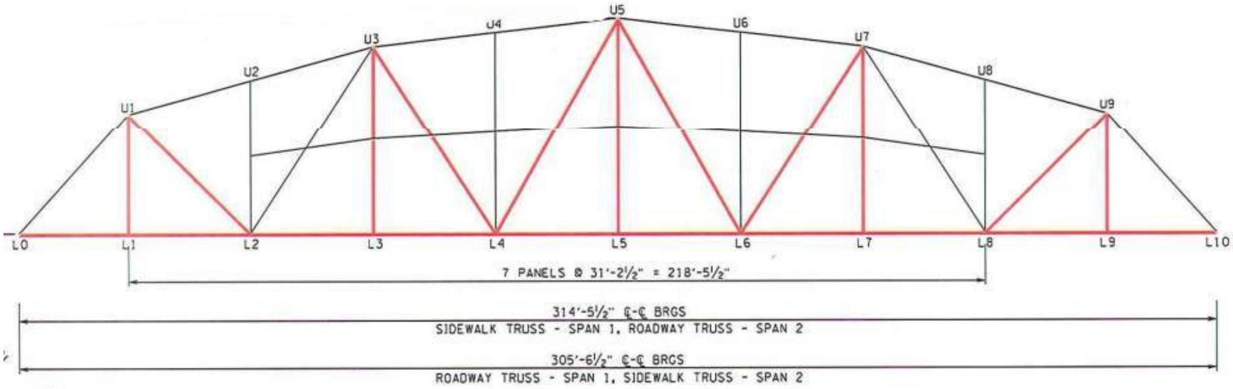
III. Fracture Critical Members and Fatigue-Prone Details

Fracture Critical Members on this bridge are the non-redundant tension members of the trusses along with the floorbeam bottom flange and portions of the floorbeam webs which experience tension loading. The riveted built up truss connections are classified as Fatigue Category D in accordance with AASHTO LRFD Bridge Design Specification, 4th Edition, Table 6.6.1.2.3-1.

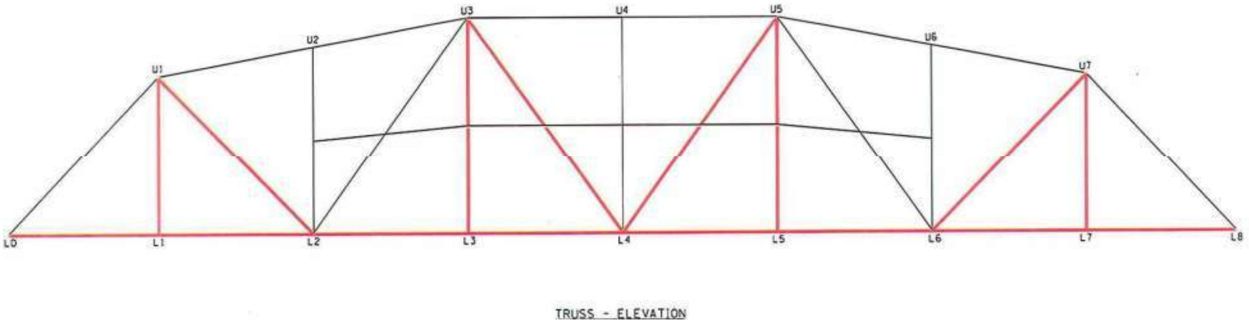
Per the requirements of the National Bridge Inspection Standards found in Title 23 Part 650 Subpart C of the Code of Federal Regulations all FCMs were inspected hands-on from a distance no further than arms-length.

The sidewalk truss verticals and diagonals have welded plates added at the sidewalk level which are intended to partially block the openings at the sidewalk penetrations. These welded plates represent fatigue-sensitive details in FCM's wherever the base members are FCM's. The end floorbeams have partial height welded stiffeners and full height jacking stiffeners which represent fatigue-sensitive details in FCM's. In addition the north face of the Span 1 end floorbeam FB0 has a welded plate stringer bearing stiffener below S3 which constitutes a fatigue-sensitive detail in a FCM.

Diagrams depicting the truss FCM members highlighted in red is included directly below.



Truss diagram showing Panel Point numbering system for Spans 1 & 2



Truss diagram showing Panel Point numbering system for Span 3

Fracture Critical tension Members shown in **Bolded Red** linestyle (per original contract plan design loads)

IV. Inspection Findings

Deck Elements

The deck elements are in fair overall condition.

Deck & Wearing Surface: The concrete filled steel grid deck is covered with a bituminous wearing surface which is in satisfactory condition with few visible defects (Photo 4) except at the deck joints as described below under "Joints".

Underside of Deck: In numerous locations the exposed underside of main longitudinal grid deck bearing bars were visible and exhibited active corrosion (Photo 5). This condition is mostly confined to the outer edges of the deck along S3 and S8. The cross-beams supporting the grid deck are in poor condition at a number of locations as described below under "Cross-beams".

Bridge Railings: There are newer steel bridge railings connected to the inner faces of the truss verticals and diagonals along both truss lines which appear to meet standards (Photo 4). The bicycle/pedestrian railing at the west side of the sidewalk is also newer and is in good condition.

Sidewalk & Curbs: The steel channel curb along the east side of the bridge is in fair condition with no significant visible defects. The concrete curbs and sidewalk on the west side of the bridge are in good condition (Photo 6).

Joints: The south abutment joint appears to have been retrofitted with an open plate type joint which is continuing to require ongoing maintenance and repairs. The south abutment deck joint was previously patched with bituminous material in a 2 foot strip on both sides of the joint for full curb to curb length of the joint and a subsequent 2 foot strip of bituminous material was added along the south approach side of the joint during the inspection (Photo 7). A total of three separate quick setting concrete patches ranging from 4 to 10 SF each in size were added in both lanes along the north side of the joint during the course of the inspection.

The south pier (Pier 1) strip seal type deck joint was previously patched with bituminous material in a 2 foot strip on both sides of the joint for full curb to curb length of the joint and a concrete patch measuring 10 SF was placed in the southbound lane along the north side of the joint during the inspection (Photo 8). This latest concrete patch is covering a 2 foot square pothole observed in the original bituminous patch.

The north pier (Pier 2) strip seal type deck joint has three concrete patch repairs each ranging from 6 to 12 SF in size in the headers on both sides of the joint in the southbound (Photo 9). These patches are covering spalled areas observed prior to the repairs.

The north abutment compression seal type deck joint is filled with moderate debris accumulations full length and the concrete headers in the southbound lane on both sides of the joint were patched with quick-setting concrete during the lane closures implemented for the inspection (Photo 10). The south edge of the 3 foot by 6 foot concrete patched area in the outer wheel path of the southbound lane on the south side was observed to be severely spalled prior to the patch repair. Bituminous patching was also placed in a three foot long strip on the north approach side of the concrete header.

Deck Drainage: Deck drainage is provided through the continuous open grid gutter drainage strips located along each curbline (Photo 11). Several damaged, perforated, and failed secondary distribution bars were observed in the open grid gutter drainage strips (Photo 11).

Superstructure

The superstructure steel elements are in fair condition overall. However the paint condition is poor throughout the superstructure.

Trusses:

Upper Chord: The paint system is failed over more than 40% of the exterior surfaces of the upper chords and there are significant and widespread areas with missing paint and active corrosion (Photo 12 & 13). The interior surfaces of the upper chords exhibit paint failure rates averaging 10% and there are bird nests present on the top of nearly all of the bottom face batten plates (Photo 14). There is active corrosion on all exterior surfaces, including top plate and both the inboard and outboard side channels of the upper chord. Although there is active corrosion within the upper chords, there are no significant section losses noted to the upper chords at this time.

Two mis-drilled rivet holes were observed in the vertical leg of the top angle in the outboard east face of the Span 1 roadway truss at U6. Three additional holes interspersed with the rivet holes were observed at the Span 2 roadway truss west side channel lower angle (Photo 15).

Slight vertical distortion was noted in the bottom flange angles at and/or between the lacing bars at various locations (Photo 16). Minor localized bends in the flange angles with up to 5/8 vertical deflection of the flange tip were observed at a few locations (Photo 17). The double lattice lacings bars in the Span 2 sidewalk truss U7-U8 upper chord have noticeable upward deflection (Photo 18).

Open holes with single missing rivets were observed at approximately one dozen individual locations in two recurring positions, one being the top cover plate and the other being the sidewalk truss inboard face lower side channel at the center of the bottom face upper lateral connection plate.

Lower Chords: Heavy debris accumulations up to four inches deep were observed on the top of the lower chord bottom plate near the Span 1 roadway truss L2 splice region (Photo 19). At the same joint similar packed debris accumulations up to two inches deep were observed on the top of the lower chord outboard channel where electrical conduits and an electrical junction box combine to trap and collect dirt droppings (Photo 20). A similar condition was observed in the Span 1 roadway truss L4 joint where minor exfoliated rusting was noted over the lower three inches for the full length of the outboard gusset plate due to the moisture and dirt accumulation caused by the electrical conduits (Photo 21). Tightly packed debris accumulations 1 ½ inches in depth were also observed between an electrical conduit and the top face of the roadway truss lower chord in Span 3.

In the Span 1 roadway truss at L4 section loss of 1/16 inch depth was observed over a 4 inch long by 8 inch high area on the east face of the L3-L4 chord west side channel cover plate immediately adjacent to the vertical edge of the U4-L4 vertical (Photo 22). Minor negligible section loss to three of five total rivets heads in the lower chord bottom face on the north side of the outboard side channel was observed in the Span 1 roadway truss splice at L4.

The roadway truss lower chord top face exhibits poor paint condition with missing paint and active corrosion typical throughout the bridge length (Photo 23). The overall typical upper chord paint condition is characterized as 30-40% paint loss. The interior vertical faces of the roadway truss lower chord also exhibits poor paint condition with missing paint and active corrosion typical throughout the bridge length (Photo 24). The east face of the Span 1 roadway truss between L8 and L9 exhibits missing paint and active corrosion over a 15 foot length (Photo 25). In Span 3 the entire length of the east face of the roadway truss has severe paint failure with active corrosion (Photo 26).

The Span 2 sidewalk truss lower chord east face has debris accumulations up to an inch deep over the top of the bottom flange of the side channel for at least half of the span length (Photo 27). The Span 2 sidewalk truss lower chord L8-L9 splice located just north of L8 has up to 50% section loss to one rivet head of 5 in the group along the lowest position of the side channels on the west face.

The Span 2 sidewalk truss lower chord bottom face tie plates were observed to be bowed downward due to pack rust at the interfaces with the bottom of the lower chord side channels (Photo 28). This condition is typical throughout the span. The top face tie plates are similarly bowed in the upward direction due to pack rust at the interfaces with the top of the lower chord side channels (Photo 29). At one location in Span 2 near L1 a top face tie plate was observed to be bent in double curvature rather than the usual single curvature pattern. The most severe instance of bowing of the top face lower chord tie plates was observed at Span 3 sidewalk truss at the third tie plate north of L0 (Photo 30).

A minor one inch bend in the underside of the L7-L8 Span 1 sidewalk truss west face was observed which could be attributable to either impact damage during shipping and handling prior to initial erection or possible impact from floating debris during past extreme high water incidents.

There is a localized zone of 1/8 inch deep by 3 inch wide section loss to underside of the Span 3 sidewalk truss lower chord outboard side channel located at the corner of the first tie plate south of the center splice in the L4-L5 (Photo 31). There is similar section loss to both side channels of the Span 3 roadway truss lower chord at the second tie plate north of L6. At this location the east side channel has ¼ inch of section loss over a 4 inch width at the edge of the tie plate and the west side channel has 1/8 inch section loss over a 4 inch width (Photo 32). The interior vertical side channel faces of the Span 3 roadway truss L3-L4 exhibit large areas of failed paint with active corrosion along with raised expansive delamination/exfoliation similar to pack rust (Photo 33).

Vertical and Diagonal Members: Active corrosion with delamination and rust exfoliation was observed at the Span 1 roadway truss U1-L1 where there is estimated section loss of 1/16 inch to the east face of the vertical west flange over the half of the flange width to the south of the vertical web (Photo 34). The section loss extends over a height of about 3 feet and is roughly centered on the top of the lower chord.

At approximately 5 feet above the deck in the Span 3 roadway truss U1-L2 there is a zone of 1/8 inch section loss to the west face of the flange over an area measuring approximately 5 inches square (Photo 35). The Span 3 roadway truss U4-L4 vertical has a 7/16 diameter hole in the west flange located approximately 3’-6” above the deck level.

Impact damage to the Span 3 sidewalk truss L0-U1end post was observed which has caused two bends of ¾ to 1 inch to the northeast corner angle (Photo 36). Similar impact damage was observed to the Span 3 roadway truss U7-L8 end post at approximately 8 feet above the deck.

In Span 3 the verticals and diagonals of the roadway truss have severe paint failure with active corrosion over the lower half of the truss depth throughout the span (Photo 26). Similar conditions were noted along the Span 2 roadway truss verticals and diagonals.

The sidewalk truss verticals and diagonals have welded plates added at the sidewalk level which are intended to partially block the openings at the sidewalk penetrations (Photos 37). These welded plates represent fatigue-sensitive details in FCM’s wherever the base members are FCM’s. Even though some of these plates are no longer in place the weld remnant is still present, so the location remains as an FSD in a FCM.

Floorbeams: Previously arrested pack rust was observed in the Span 1 FB1 floorbeam between the bottom flange cover plate and the edges of the bottom flange angles (Photo 39). This area appeared to have been previously caulked with some sort of material. A similar but active pack rust condition was noted in the Span 2 FB5 floorbeam between S5 and S6 (Photo 40).

Span 1 FB2 has heavy debris accumulation from 4 to 6 inches deep on the top of the floorbeam flange (Photo 41). Span 1 FB3 has moderate active corrosion to the bottom flange on the south side of the web between S6 and S8 (Photo 42). Span 1 FB5 has a zone of significant active corrosion to the top of the bottom flange on the north side of the web between S6 and S7 with up to 1/8 inch section loss to rivet heads in this zone (Photo 43). The ends of the floorbeams below S1 and S8 exhibit paint loss and active corrosion to the web and bottom flanges at most locations throughout the bridge length. At Span 1 FB7 below S8 both faces of the web exhibit active corrosion with section loss of 1/8 inch depth over a height of 8 inches and length of 12 inches (Photo 44).

Span 2 FB8 has pack rust and active corrosion over the full web height on the south face just inboard of S8. Span 2 FB8 also has a zone of significant active corrosion to the top of the bottom flange on the north side of the web between S6 and S8. Delamination to the top and edge of the bottom flange angle horizontal leg with up to 1/8 inch section loss was observed in this zone in the portion of the floorbeam outside the partial length cover plate (Photo 45). Flakes of material measuring 3 inches square were removed with minimal effort from this location. Span 2 FB6 has a zone of significant active corrosion and 1/8 inch section loss to the top of the bottom flange on the south side of the web between S3 and S4 (Photo 46). Span 2 FB4 has a 4 inch wide by 8 inch long zone of significant active corrosion and delamination with 1/8 inch section loss to the top of the bottom flange on the north side of the web at 2 feet west of S4 (Photo 47).

Span 3 FB6 has a zone of active corrosion and exfoliation on the top of the bottom flange on the north side of the web between S3 and S4. Span 3 FB4 has a zone of active corrosion and poor paint condition on the top of the bottom flange on the south side of the web over the eastern end 10 foot length.

The end floorbeams over Pier 1 (FB10 in Span 1 and FB0 in Span 2) were observed to have partial height welded stiffeners on the face of the web below the deck joint for the transverse deck joint support member (Photo 48), which constitutes a number of fatigue-sensitive details in FCM’s (for the portion of the weld below the web mid-height neutral axis). This identical condition was also observed at Pier 2 for the Span 2 and 3 end floorbeams (Photo 49). The end floorbeams at both abutments also have a similar condition with partial height welded stiffeners on the face of the web below the deck joint. The end floorbeam at the south abutment also has full height welded jacking stiffeners on the face opposite the deck joint at the sidewalk and roadway truss ends. In addition, Span 2 FB0 and Span 3 FB0 each have full height welded jacking stiffeners which also represent fatigue-sensitive details in FCM’s (Photos 49 & 50). Full height welded jacking stiffeners were also noted on the north face of the Span 2 and 3 FB0 webs (on the face opposite the deck joint) at the sidewalk and roadway truss ends (Photo 51). The sidewalk cantilever bracket clip angles were observed to be welded full height to the gusset plate and to the web fill plate on the bracket web at Span 1 L0 (Photo 52).

The north face of the Span 1 end floorbeam FB0 has a welded plate stringer bearing stiffener below S3 which constitutes a fatigue-sensitive detail in a FCM (Photo 53).

The top flange angles of the floorbeam sidewalk cantilever brackets exhibit severe section loss at numerous locations throughout all three spans. In the most severe locations, this section loss amounts to 100% of the area of the horizontal legs of the top flange angles on both sides of the web between S1 and S2 (Photo 54). A pair of welded lugs was observed on the north face of the Span 2 FB0 sidewalk cantilever bracket web, which appear to be in the lower half of the web and are therefore non-FCM's.

Stringers: Stringer S8 in Span 1 has heavy debris accumulations along the top flange at each of the grid deck cross beams between L2 and L3 (Photo 55). Stringer S8 exhibits poor paint condition on the east face of the web and the bottom flange with missing paint and active corrosion typical throughout the bridge length (Photo 23).

Stringer S3 has pack rust on the bottom flange and at bottom of the web at the connection to the north face of Span 3 FB3 (Photo 56). Similar conditions were noted at three other locations in the S3 stringer ends.

Sidewalk stringer S1 has been previously repaired at numerous panels by stitch welding a pair of angles to the top of the web, presumably to restore section loss to the original top flange (Photo 57). This typical previous repair was left prime- painted only on the inboard face of the stringer, leaving the repair angle with no intermediate or finish coats. Similar repairs were also observed on sidewalk stringer S2 where the top flange angle repair was left prime- painted only on both sides of the stringer (Photo 58).

Cross-beams: The transverse cross-beam at the south abutment exhibits severe advanced corrosion with section loss and perforation below the northbound lane. Between stringers S6 and S8, there are 2 locations of extensive section loss. A three inch width of the bottom flange south edge is missing over a 4 foot length between S7 and S8 (Photo 59). Within this same zone there is also section loss to the opposite north bottom flange edge with loss of up to 2 inches in width over a length of 12 to 18 inches. The lower two inches of the web was also perforated in a portion of this 4 foot length. A similar condition was observed at a separate location along the same member located between S6 and S7. This member is located below a deck joint which has required on-going maintenance repairs prior to and during the most recent inspection as described elsewhere in this report.

At the north abutment the second cross-beam from the deck joint has severe section loss to the bottom flange which could not be properly accessed for complete documentation due to the utility conduits obstructing this location.

Section loss, perforation, and delamination to two adjacent grid deck cross-beams was observed in Span 1 near FB2 between S7 and S8 where the underside of the bottom flange has up to 3/16 inches of material lost over two inches of the edge of the bottom flange along a length of 16 inches (Photo 60). The typical paint condition on grid deck cross-beams between S7 and S8 is poor throughout the bridge length with active corrosion on most cross-beams in this stringer bay (Photo 61).

The east end of a grid deck cross-beam above S8 between L7 and L8 was observed to be twisted around the cross-beam axis due to pack rust at the S8 connection (Photo

62).

Perforations at the flange edges and in the bottom flange of the cross-beams was observed in Span 3 near L6 in the ends of the cross beams (Photo 63). Two cross-beams have perforations of the outer one inch along the flange edges in this vicinity.

Secondary Members: Upper Chord Lateral Bracing- The paint system is failed over more than 30% of the surfaces of the top chord lateral bracing (Photo 64 & 65). The ends of the top laterals at numerous locations have intentional crimping-related distortion to accommodate the member slopes at the upper panel points (Photo 66). Many of the upper chord lateral bracing connection plates have paint failures and active corrosion (Photo 67). The mid-lateral sway frame bracing connection plates at U3 to U7 typically have peeling or failed paint (Photo 68). At a few locations slightly kinked lacing bars were observed in the upper chord lateral bracing transverse top struts. The Span 3 U5 upper chord lateral bracing transverse top strut has a ¾ inch upward bend in the south side lower angle edge at one foot east of the roadway centerline (Photo 69).

The Span 3 U6 sway frame lower strut has sustained impact damage in a 15 inch long zone which has deflected the north horizontal leg downward by 1 ¼ inches over the northbound lane just east of the bridge centerline (Photo 70).

Lower Chord Lateral Bracing- Severe pack rust with active corrosion was observed over a four foot length of the lower lateral bracing angle connecting Span 1 L3 roadway truss and L4 sidewalk truss at the L3 end of the member (Photo 71). A similar condition was observed over a four foot length of the lower lateral bracing angle connecting Span 2 L4 sidewalk truss and L3 roadway truss at the L4 end of the member. Severe pack rust with active corrosion was also observed between the back to back vertical angles over a ten foot length of the lower lateral bracing angle connecting Span 1 L5 roadway truss and L6 sidewalk truss at the L5 end of the member (Photo 72).

Bearings: The south abutment expansion bearings exhibit active corrosion and poor overall paint condition. The roadway truss bearing at the south abutment has a loose anchor bolt which was removed, photographed and loosely re-inserted by the inspection crew (Photo 73). The expansion slots in the base of the upper shoe are completely filled with debris accumulations which preclude normal thermal movement (Photo 74). The two anchor bolts on the east side of the bearing were noted to be bent off plumb in opposite directions (Photo 75).

The Span 2 sidewalk truss fixed bearing at Pier 2 is missing a nut on the southwest anchor bolt (Photo 76).

Corrosion Losses for Primary Members: Table 1 contains a listing of specific locations where measurable section loss was observed in primary superstructure members, including primary truss members, floorbeams, and stringers. For cases in which section loss was documented in the primary superstructure components, the live load rating computations were prepared to consider the actual remaining section. (The upper and lower lateral bracing and associated connection plates are secondary members which serve to carry wind and lateral loads but do not participate directly in resisting traffic live loads.)

Member	Location	Description
Span 1 roadway truss U1-L1	The section loss extends over a height of about 3 feet and is roughly centered on the top of the lower chord within the height of the gusset plate	Estimated section loss of 1/16 inch to the east face of the vertical west flange over the half of the flange width to the south of the vertical web (Photo 34).
Span 1 roadway truss L3-L4	East face of the L3-L4 chord west side channel cover plate immediately adjacent to the vertical edge of the U4-L4 vertical	Section loss of 1/16 inch depth was observed over a 4 inch long by 8 inch high area (Photo 22).
Span 3 sidewalk truss L4-L5	Corner of the first tie plate south of the center splice	Localized zone of 1/8 inch deep by 3 inch wide section loss to underside of lower chord outboard side channel (Photo 31).
Span 3 roadway truss U1-L2	At approximately 5 feet above the deck	Zone of 1/8 inch section loss to the west face of the flange over an area measuring approximately 5 inches square (Photo 35).
Span 3 roadway truss U4-L4	Approximately 3'-6" above the deck	The vertical has a 7/16 diameter hole in the west flange
Span 3 roadway truss L6-L7	Second tie plate north of L6	Underside of east side channel has ¼ inch of section loss over a 4 inch width at the edge of the tie plate and the west side channel has 1/8 inch section loss over a 4 inch width (Photo 32).
Span 1 FB7	Below S8	Both faces of web exhibit active corrosion with section loss of 1/8" depth over a height of 8" and length of 12" (Photo 44)
Span 2 FB8	Between S6 and S8 top of the bottom flange on the north side of the web	Delamination to top and edge of bottom flange angle horizontal leg with up to 1/8 inch section loss was observed in the portion of the floorbeam outside the partial length cover plate (Photo 45).
Span 2 FB6	Between S3 and S4 south half of bottom flange only	Zone of 1/8 inch section loss to the top of the bottom flange on south side of web (Photo 46).
Span 2 FB4	Bottom flange on north side of web at 2 feet west of S4	4" wide by 8" long zone of 1/8 inch section loss to top of bottom flange on north side of the web at 2 feet west of S4 (Photo 47).
Spans 1-3, (all flrbm. cantilever brackets	Floorbeam sidewalk cantilever brackets between S1 and S2	Top flange angles exhibit severe section loss throughout. In most severe locations, 100% of horizontal legs of top flange angles on both sides of web are lost (Photo 54)

TABLE 1- Locations of Measurable Section Loss in Primary Members

Substructure

The visible substructure elements are in satisfactory condition.

Abutments: The south abutment backwall has a wide 2 foot long horizontal crack near the wingwall corner at approximately 4 feet above the bridge seat (Photo 77). The older stone masonry portions of the south abutment exhibit remnants of missing timber bracing anchors and possible signs of outward shifting of a few of the stones (Photo 78).

The north abutment backwall has numerous medium diagonal cracks in the portion below the sidewalk (Photo 79). A previous large spall repair was observed in the upper and middle portion of the north abutment backwall located roughly in line with the sidewalk truss. There is a large 3 foot by 3 foot by 3 inch deep spall in the lower portion of the backwall around the edges of the utility conduit penetration and extending behind the sidewalk truss (Photos 80 & 81).

Piers: Both piers are in satisfactory condition with no significant visible defects (Photos 82 and 83). Pier 2 has a wide diagonal crack in the southwest corner of the sidewalk truss bearing pedestal which extends for the full height of the concrete pedestal (Photo 84).

Wingwalls: The wingwalls are in satisfactory condition with no major visible defects.

Channel

There were no visible deficiencies in the channel. There is a dam located approximately 300 feet upstream of the bridge.

Miscellaneous

Approach Pavement: Both approach pavements are in fair condition with several transverse and longitudinal cracks clustered in the wheel paths (Photo 85). The south approach has visible settlement and depression within the wheel paths of both lanes.

Approach Curb: There is concrete or granite curb at all four corners of the bridge. No significant defects were noted in the approach curbs.

Approach Guiderails: There are approach guiderails at the southeast, northeast, and northwest corners which appear to meet standards. The existing guiderails are stiffened in the transition zones at these three bridge corner connections to the bridge railings (Photo 85). No guiderail is present at the southwest bridge corner because there is a section of railing attached to the U-shaped retaining wall/wingwall extension at this location (Photo 86). The approach guiderail at the northwest corner of the bridge is aligned with the outboard edge of the sidewalk and connected to the west pylon (Photo 87). The end of the southeast corner approach guiderail has impact damage (Photo 86). The end terminal on the northeast guiderail also has minor impact damage.

Load Posting: The bridge is not currently posted for live load.

Signage: No vertical clearance signs are posted on the approach roadways or on the structure. There are bridge ID markers located at both of the leading end corners of the structure, one at each approach.

Conduits: Two conduits presumed to supply power for bridge lighting were observed to have a three inch separation in the conduit in Span 1 along the top face of the roadway truss between L9 and L10 which results in exposed electrical cables (Photo 88).

Utilities: There is a major large diameter insulated under-deck utility located just inboard of the east roadway truss which is suspended from utility support brackets which are attached to the S7 and S8 stringers. This utility has missing blocking at several locations (Photo 89) and the insulation jacket is missing over a 12 foot length in Span 1 at FB5 (Photo 72).

There are two separate banks of six conduits each located just inboard of the east roadway truss which are suspended from utility support brackets which are attached to the S3 and S4 stringers. In Span 2 between L9 and L10 both conduit banks were observed to have open one inch gaps in the conduits along with two consecutive twisted support hangers (Photo 90). In Span 2 between L2 and L3 a repair was observed consisting of additional angles installed due to perforations in the horizontal leg of the original utility support hanger. In Span 2 at FB0 the western bank of conduits have an open gap, are mis-aligned, and the vertical hanger angle is missing. It appears this bank may be empty as there were no exposed contents visible at the gap. Two small quarter size perforations were noticed in the horizontal lower angle of the Pier 1 utility conduit hanger support.

Miscellaneous: Remnants of an abandoned wire cable attached to the floorbeams were observed dangling at several locations (Photo 919).

V. Conclusions and Recommendations

The results of the inspection indicate that the bridge is overall in fair condition.

- The **deck** is in fair condition overall, rated a 5. However the deck joints are in poor condition, rated a 4. All four deck joints have previous bituminous patch repairs and more recent concrete patch repairs. The south abutment joint appears to have been retrofitted with an open plate type joint which is continuing to require ongoing maintenance and repairs. In numerous locations the exposed underside of main longitudinal grid deck bearing bars were visible and exhibited active corrosion. This condition is mostly confined to the outer edges of the deck along S3 and S8.

- The **superstructure** is in fair condition overall, rated a 5. However the paint is in poor condition, rated a 4. There is measurable section loss to six main truss members and four floorbeams. Previously arrested or active pack rust was observed in several floorbeams between the bottom flange cover plate and the edges of the bottom flange angles. Active corrosion is present in all of the different types of superstructure elements and is especially significant in the floor system members, including the cross-beams. Several of the cross-beams have advanced section loss with perforations in the bottom flange. Distortion/out of plane deformation of upper chord lacing bars along with bowing of lower chord tie plates was observed, indicating the effects of pack rust are becoming a concern. Section loss, perforation, and delamination to two adjacent grid deck cross-beams was observed where the underside of the bottom flange has up to 3/16 inches of material lost over two inches of the edge of the bottom flange along a length of 16 inches. The top flange angles of the floorbeam sidewalk cantilever brackets exhibit severe section loss at numerous locations throughout all three spans. In the most severe locations, this section loss amounts to 100% of the area of the horizontal legs of the top flange angles on both sides of the web between S1 and S2. Heavy debris accumulations up to two inches deep were observed on the top of the lower chord outboard channel at two locations where electrical conduits and/or an electrical junction box combine to trap and collect dirt droppings. Severe pack rust with active corrosion was observed in a few of the lower lateral bracing angles.
- The **substructure** is in satisfactory condition, rated a 6. The older stone masonry portions of the south abutment exhibit remnants of missing timber bracing anchors and possible signs of outward shifting of a few of the stones. There is a large 3 foot by 3 foot by 3 inch deep spall in the lower portion of the north abutment backwall around the edges of the utility conduit penetration and extending behind the sidewalk truss.

Recommendations:

We recommend that the following safety improvements, repairs or rehabilitation, and/or monitoring should be made to retard further deterioration, preserve the structural integrity of the bridge, and extend its useful life:

1. Replace the four deck joints by performing full depth deck removal and replacement of a four foot strip centered at each joint, including reconstruction of the top of the abutment backwalls at each end of the bridge. Replace the severely corroded and perforated south abutment cross-beam in conjunction with the deck joint reconstruction.
2. In order to correct the on-going active corrosion of the truss members and other steel superstructure components and extend the service life of the bridge, the bridge superstructure should be cleaned and painted by removing the existing coatings down to bare metal and applying a coating system in accordance with Maine DOT standard specifications. Of particular concern are the corroding lower truss chords, floorbeams, and deck support members. The cleaning operations prior to painting should include cleaning and caulking at the bowed lower chord top face tie plates to prevent future water intrusion and advancement of the bowing deformation.
3. Re-inspect all welded sidewalk penetration plates in the sidewalk truss vertical and diagonal FCM’s during future biennial bridge inspection cycles. Check for any indications of fatigue cracking initiation as part of the regularly scheduled bridge inspection cycles.
4. Re-inspect all end floorbeam welded stiffener fatigue-sensitive details in the FCM’s during future biennial bridge inspection cycles. Check for any indications of fatigue cracking initiation as part of the regularly scheduled bridge inspection cycles.
5. Perform repairs to all severely corroded floorbeam sidewalk cantilever brackets in all three spans to restore the lost top flange area between S1 and S2. Remove the existing top flange angles on both sides of the web and install bolted angles fastened through the existing holes in the web with a flange area equivalent to the original top flange angles.
6. Restore the section loss to the five specified cross-beams by making bolted plate repairs to the bottom flange.
7. Drill and grout a new anchor bolt at the south abutment roadway truss bearing. Clean the dirt and debris accumulations from the south abutment bearings expansion slots and lubricate the bearings to restore uninhibited thermal movement. Evaluate and if necessary repair the bent anchor bolts at the south abutment roadway truss bearing (See Photo 75).
8. Install a nut on the southwest anchor bolt at the Pier 2 Span 2 sidewalk truss fixed bearing.
9. Repair the crack in the south abutment backwall by epoxy-injection.
10. Monitor the south abutment stone masonry every 3 months to determine if there is any shifting of the individual stones.
11. Perform concrete spall repair to the one north abutment backwall location by removing and replacing unsound concrete.

12. Reconstruct the corner of the Pier 2 sidewalk truss bearing pedestal by removing and replacing unsound concrete.
13. Repair the separation in the Span 1 roadway truss conduit run between L9 and L10 to seal and protect the electrical cables.

Inspection Photographs

Photo 1 – Partial upstream elevation looking southeast with Span 1 at right. (Note- See Report cover for similar photo).

Photo 2 – View of bridge looking north from the south approach roadway. Note minor impact damage to southeast corner guiderail at right.

Photo 3 – View of bridge looking south from the north approach roadway.

Photo 4 – General top of deck view in Spans 2 & 3, looking south from Span 3 at approximately 25 feet north of Pier 2.

Photo 5 –Along outer edges of deck at S3 and S8 exposed main longitudinal grid deck bars were visible and exhibited active corrosion. Looking north in Span 1 along S3.

Photo 6 – The concrete curbs and sidewalk on the west side of the bridge are in good condition having been rehabbed relatively recently. Looking south from north abutment.

Photo 7 – South abutment open plate deck joint was previously patched with bituminous material; additional bituminous and concrete patching and other repairs were being performed during the inspection.

Photo 8 – The south pier strip seal type deck joint was previously patched with bituminous material and a concrete patch was placed during the inspection.

Photo 9 – The north pier strip seal type deck joint has three quick-setting concrete patch repairs placed during the lane closures implemented for the inspection. Looking east.

Photo 10 – The north abutment compression seal type deck joint is filled with debris and the concrete headers were patched with quick-setting concrete during the inspection.

Photo 11 – Several damaged, perforated, and failed secondary distribution bars were observed in open grid gutter drainage strips. Looking north along west curb near Pier 2.

Photo 12 – Paint system is failed over more than 40% of exterior surfaces of upper chords with significant and widespread areas of missing paint and active corrosion.

Photo 13 – Paint system is failed over more than 40% of exterior surfaces of upper chords with significant and widespread areas of missing paint and active corrosion.

Photo 14 – The interior surfaces of upper chords exhibit paint failure rates averaging 10% and there are bird nests present on top of nearly all of bottom face batten plates.

Photo 15 – Three additional mis-drilled rivet holes interspersed with the rivet holes were observed at the Span 2 roadway truss U9 west side channel lower angle. Looking north.

Photo 16 – Slight vertical distortion was noted in the bottom flange angles at and/or between the lacing bars at various locations. View at Span 1 U7-U8 sidewalk truss.

Photo 17 – Minor localized bends in flange angles with up to 5/8 inch vertical deflection of flange tip were observed at a few locations. View at Span 1 U7-U8 sidewalk truss.

Photo 18 – The double lattice lacings bars in the Span 2 sidewalk truss U7-U8 upper chord have noticeable upward deflection. View is looking west.

Photo 19 – Heavy debris accumulations up to four inches deep were observed on the top of the lower chord bottom plate near the Span 1 roadway truss L2 splice region.

Photo 20 – Packed debris up to two inches deep on top of Span 1 roadway truss L2 lower chord outboard channel where electrical conduits and junction box trap and collect dirt.

Photo 21 –In Span 1 roadway truss L4 joint minor exfoliated rusting was noted over 3” of outboard gusset plate due to moisture and dirt accumulation caused by the conduits.

Photo 22 – In Span 1 roadway truss at L4 section loss of 1/16 inch depth was observed over a 4” long by 8 “ high area on east face of L3-L4 chord west side channel cover plate.

Photo 23 – The roadway truss lower chord top face exhibits poor paint condition with missing paint and active corrosion typical throughout bridge. View of Span 1 L5-L6.

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Photo 24 – The interior vertical faces of the roadway truss lower chord also exhibit poor paint condition with missing paint and active corrosion typical throughout. Span 1, L6-L7.

Photo 25 – The east face of the Span 1 roadway truss between L8 and L9 exhibits missing paint and active corrosion over a 15 foot length.

Photo 26 – In Span 3 the entire length of the east face of the roadway truss has severe paint failure with active corrosion. Looking north toward north abutment.

Photo 27 – Span 2 sidewalk truss lower chord east face has debris accumulations up to an inch deep over top of bottom flange of side channel for at least half of the span length.

Photo 28 – Span 2 sidewalk truss lower chord bottom face tie plates bowed downward due to pack rust at the interfaces with the bottom of the lower chord side channels.

Photo 29– Top face tie plates are similarly bowed in the upward direction due to pack rust at the interfaces with the top of the lower chord side channels.

Photo 30 – The most severe instance of bowing of the top face lower chord tie plates was observed at Span 3 sidewalk truss at the third tie plate north of L0.

Photo 31– Zone of 1/8 inch deep by 3 inch wide section loss to Span 3 L4-L5 sidewalk truss outboard side channel located at corner of first tie plate south of center splice.

Photo 32 – Span 3 roadway truss L6-L7 has ¼ inch of section loss over 4 inch width on east side channel and 1/8 inch section loss over a 4 inch width to west side channel.

Photo 33– Interior vertical side channels of Span 3 roadway truss L3-L4 exhibit areas of failed paint, active corrosion, and raised expansive exfoliation similar to pack rust.

Photo 34 – Span 1 roadway truss U1-L1 has section loss of 1/16 inch to the east face of the vertical west flange over the half of the flange width south of the vertical web.

Photo 35– At 5 feet above the deck in Span 3 roadway truss U1-L2 there is a zone of 1/8 inch section loss to west face of flange over an area approximately 5 inches square.

Photo 36 – Impact damage to the Span 3 sidewalk truss L0-U1 end post was observed which has caused two bends of ¾ to 1 inch to the northeast corner angle.

Photo 37– Sidewalk truss verticals and diagonals have welded plates at sidewalk level openings. These plates represent FSD's in FCM's (wherever base members are FCM's).

Photo 38 – Sidewalk truss verticals and diagonals have welded plates at sidewalk level openings. These plates represent FSD's in FCM's (wherever base members are FCM's).

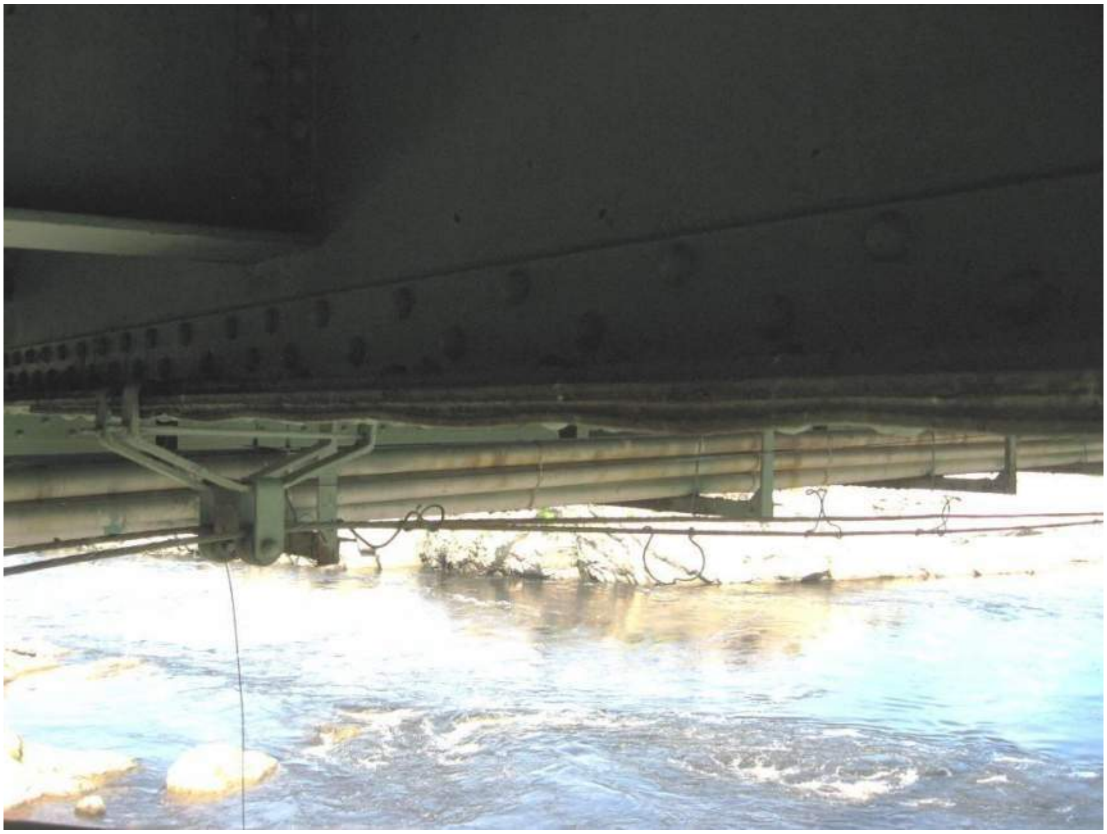

Photo 39– Previously arrested pack rust was observed in the Span 1 FB1 floorbeam between the bottom flange cover plate and the edges of the bottom flange angles.

Photo 40 – An active pack rust condition was noted in the Span 2 FB5 floorbeam between S5 and S6.

Photo 41– Span 1 FB2 has heavy debris accumulation from 4 to 6 inches deep on the top of the floorbeam flange. View looking west at roadway truss.

Photo 42 – Span 1 FB3 has moderate active corrosion to the bottom flange on the south side of the web between S6 and S8. View looking east.

Photo 43– Span 1 FB5 has a zone of significant active corrosion to top of bottom flange on north side of web between S6 and S7 with up to 1/8 inch section loss to rivet heads.

Photo 44 – At Span 1 FB7 below S8 both faces of the web exhibit active corrosion with section loss of 1/8 inch depth over a height of 8 inches and length of 12 inches.

Photo 45– Span 2 FB8 has significant active corrosion and delamination to top of bottom flange angle on north side of web between S6 and S8 with up to 1/8 inch section loss.

Photo 46 – Span 2 FB6 has a zone of significant active corrosion and 1/8 inch section loss to the top of the bottom flange on the south side of the web between S3 and S4.

Photo 47– Span 2 FB4 has a 4” wide by 8” long zone of active corrosion/ delamination with 1/8” section loss to top of bottom flange on north side of web at 2 feet west of S4.

Photo 48 – The end floorbeams over Pier 1 have partial height welded stiffeners on face of web below deck joint, which constitutes a number of fatigue-sensitive details in FCM’s.

Photo 49– Partial height welded stiffeners on face of web below deck joint also observed at Pier 2 for the Span 2 and 3 end floorbeams (fatigue-sensitive details in FCM's).

Photo 50 – Span 2 FB0 on right also has full height welded jacking stiffeners at both ends (FSD's in FCM's). Jacking stiffeners are also present on opposite face of web.

Photo 51– Full height welded jacking stiffeners were also noted on north face of Span 2 and 3 FB0 webs (on face opposite deck joint) at sidewalk and roadway truss ends.

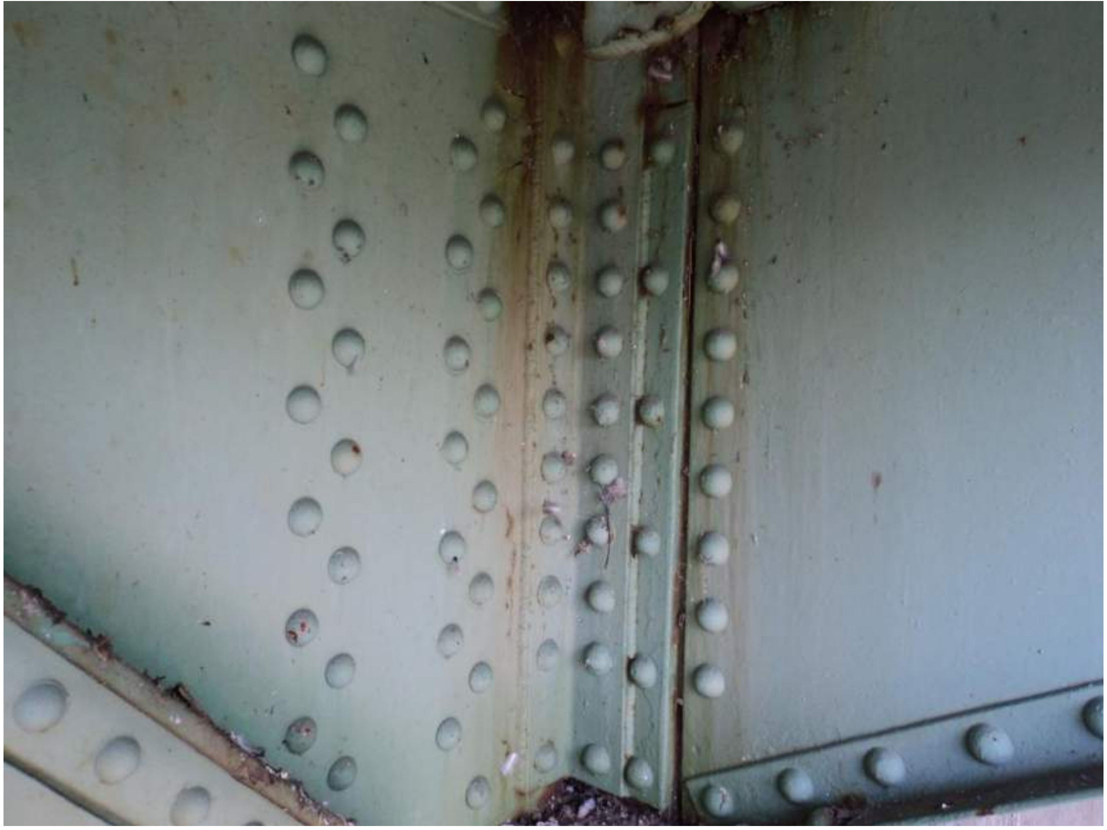

Photo 52 – Sidewalk cantilever bracket clip angles were observed to be welded full height to the gusset plate and to the web fill plate on the bracket web at Span 1 L0.

Photo 53– The north face of the Span 1 end floorbeam FB0 has a welded plate stringer bearing stiffener below S3 which constitutes a fatigue-sensitive detail in a FCM.

Photo 54 – Top flange angles of floorbeam cantilever brackets exhibit severe section loss, amounting to 100% of horizontal legs on both sides of the web between S1 and S2.

Photo 55– Stringer S8 in Span 1 has heavy debris accumulations along the top flange at each of the grid deck cross beams between L2 and L3.

Photo 56 – Stringer S3 has pack rust on bottom flange and bottom of web at connection to north face of Span 3 FB3. Similar conditions noted at three other S3 stringer ends.

Photo 57– Sidewalk stringer S1 has been previously repaired at numerous panels by stitch welding a pair of angles to top of web, to restore section loss to original top flange.

Photo 58 – Similar repairs were also observed on sidewalk stringer S2 where the top flange angle repair was left prime- painted only on both sides of the stringer.

Photo 59– A three inch width of the bottom flange south edge is missing from the south abutment transverse cross-beam over a 4 foot length between S7 and S8.

Photo 60 – Section loss and perforation to two cross-beams in Span 1 near FB2 between S7 and S8 where bottom flange has up to 3/16” section loss over 2 inches along edge.

Photo 61– The typical paint condition on grid deck cross-beams between S7 and S8 is poor throughout the bridge length with active corrosion on most cross-beams this bay.

Photo 62 – The east end of a grid deck cross-beam above S8 between L7 and L8 was observed to be twisted around cross-beam axis due to pack rust at the S8 connection.

Photo 63– Perforations at the flange edges and in the bottom flange of the cross-beams was observed in Span 3 near L6 in the ends of the cross beams.

Photo 64 – The paint system is failed over more than 30% of the surfaces of the top chord lateral bracing. View looking northwest in Span 1 toward U9 sidewalk truss.

Photo 65– The paint system is failed over more than 30% of the surfaces of the top chord lateral bracing. Looking north toward Span 2 portal.

Photo 66 – The ends of the top laterals at numerous locations have intentional crimping-related distortion to accommodate the member slopes at the upper panel points.

Photo 67– Many of the upper chord lateral bracing connection plates have paint failures and active corrosion. View at Span 1 roadway truss U8.

Photo 68 – The mid-lateral sway frame bracing connection plates at U3 to U7 typically have peeling or failed paint. View at Span 2 roadway truss U3-L3 mid-lateral.

Photo 69– The Span 3 U5 upper chord lateral bracing transverse top strut has a ¾ inch upward bend in the south side lower angle edge at one foot east of roadway centerline.

Photo 70 – Span 3 U6 sway frame strut has impact damage which has deflected the north horizontal leg downward by 1 ¼" over northbound lane just east of bridge centerline.

Photo 71– Severe pack rust with active corrosion over a 4’ length of lower lateral bracing angle connecting Span 1 L3 roadway truss and L4 sidewalk truss at the L3 end.

Photo 72 – Severe pack rust with active corrosion between back to back angles of lower lateral bracing connecting Span 1 L5 roadway truss and L6 sidewalk truss at L5 end.

Photo 73– The roadway truss bearing at the south abutment has a loose anchor bolt which was removed, photographed and loosely re-inserted by the inspection crew.

Photo 74 – Expansion slots in the south abutment roadway truss bearing upper shoe are completely filled with debris accumulations which preclude normal thermal movement.

Photo 75– The two anchor bolts on the east side of the south abutment roadway truss bearing were noted to be bent off plumb in opposite directions.

Photo 76 – The Span 2 sidewalk truss fixed bearing at Pier 2 is missing a nut on the southwest anchor bolt. View looking northeast.

Photo 77– The south abutment backwall has a wide 2 foot long horizontal crack near the wingwall corner at approximately 4 feet above the bridge seat.

Photo 78 – Older stone masonry portions of south abutment exhibit remnants of missing timber bracing anchors and possible signs of outward shifting of a few of the stones.

Photo 79– The north abutment backwall has numerous medium diagonal cracks in the portion below the sidewalk. View looking north along sidewalk truss.

Photo 80 – There is a large 3 foot by 3 foot by 3 inch deep spall in lower portion of backwall around edges of utility conduit penetration and extending behind sidewalk truss.

Photo 81– There is a large 3 foot by 3 foot by 3 inch deep spall in lower portion of backwall around edges of utility conduit penetration and extending behind sidewalk truss.

Photo 82– Piers are in satisfactory condition with no significant visible defects. View looking south at north face of Pier 1.

Photo 83– Piers are in satisfactory condition with no significant visible defects. View looking north at south face of Pier 2.

Photo 84 – Pier 2 has a wide diagonal crack in the southwest corner of the sidewalk truss bearing pedestal which extends for the full height of the concrete pedestal.

Photo 85– Both approach pavements are in fair condition with several transverse and longitudinal cracks clustered in the wheel paths. Looking southeast at northeast corner.

Photo 86 – No guiderail is present at southwest bridge corner (at left) because there is a section of railing attached to the U-shaped wingwall. Also note guiderail impact damage.

Photo 87– The approach guiderail at the northwest corner of the bridge is aligned with the outboard edge of the sidewalk and connected to the west pylon.

Photo 88 – Two conduits for bridge lighting have a 3” separation in one conduit in Span 1 along top face of roadway truss between L9 and L10 with exposed electrical cables.

Photo 89–Large insulated under-deck utility located just inboard of roadway truss has missing blocking at several locations. View looking south In Span 1 at south abutment.

Photo 90 – In Span 2 between L9 and L10 both conduit banks have open one inch gaps in the conduits along with two consecutive twisted support hangers.

Photo 91– Remnants of an abandoned wire cable attached to the floorbeams were observed dangling at several locations. View looking south in Span

Bridge Load Rating

Prepared for

Maine Department of Transportation

Bridge No. 2016

BRUNSWICK, CUMBERLAND COUNTY

US 201 & RT. 24

OVER

THE ANDROSCOGGIN RIVER

Date of Inspection: MAY 18, 2012

Date of Rating: March 2013

Prepared By:

PARSONS BRINCKERHOFF
75 ARLINGTON STREET
BOSTON, MA 02116

Checked By:

G.
Keith
Donington
D6329

Bridge No: 2016
Town/City: Brunswick
Route Carried: US 201 & Rt. 24
Crosses: The Androscoggin River

Owner: State Highway Agency
Maintainer: State Highway Agency
Year Built 1931
Year(s) Rebuilt/Rehab: 1985

SUMMARY OF TRUSS BRIDGE RATING

VEHICLE TYPE		RF	RT (TONS)	POSTING LOAD (TONS)
HL-93	INVENTORY	***0.53***	19.08	
	OPERATING	***0.69***	24.84	
HL-93 modified	INVENTORY	***0.00***	0.00	
	OPERATING	***0.00***	0.00	
CONFIGURATION 1		***0.86***	43.00	40.00
CONFIGURATION 2		***0.78***	36.66	32.23
CONFIGURATION 3		***0.74***	32.56	27.66
CONFIGURATION 4		***0.76***	33.44	28.91
CONFIGURATION 5		***0.78***	34.32	30.17
CONFIGURATION 6		***0.81***	30.74	27.65
CONFIGURATION 7		***0.90***	26.55	25.29
CONFIGURATION 8		***1.42***	26.55	OK

Group 1 Posting Analysis (Configuration 1)
Governing Posting: 40.00
Governing Load Model: CONFIGURATION 1

Group 2 Posting Analysis (Configurations 2 - 5)
Governing Posting: 27.66
Governing Load Model: CONFIGURATION 2

Group 3 Posting Analysis (Configurations 6 - 8)
Governing Posting: 25.29
Governing Load Model: CONFIGURATION 6

LRFR Evaluation Factors:

Live Load Distribution Factor: ____________
Live Load DF Routine Commercial: ____________
Live Load DF Special Hauling: ____________
Impact Factor: 33%
Governing Condition Factor, ϕ_c: 0.95
System Factor, ϕ_s: 0.9
ADTT (one-way): 534

Please check all the boxes that apply:

☐ Bridge load rating is governed by substructure rating
☐ Connections control the load rating
☐ Exterior girder controls load rating
☐ As-built load rating
■ As-inspected load rating
☐ One Lane Loaded
☐ Advanced Analysis Used
☐ Actual Measurements Taken
☐ Finite Fatigue Life ________ years

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Date: 3/25/2013
Checked by:AMC
Date:4/1/13

BREAKDOWN OF TRUSS BRIDGE RATING

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LOAD RATING POINTS OF INTEREST

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Stringer 3 Span 2 Positive Moment Midspan	**0.60**	**0.77**										
Stringer 3 Span 2 Shear At Floor Beam Connection	2.40	3.11										
Stringer 3 Skewed Span 2 Positive Moment Midspan	**0.54**	**0.70**										
Stringer 3 Skewed Span 2 Shear At Floor Beam Connection	2.05	2.66										
Stringer 4 Span 2 Positive Moment Midspan	**0.83**	1.07										
Stringer 4 Span 2 Shear At Floor Beam Connection	2.06	2.67										
Stringer 4 Skewed Span 2 Positive Moment Midspan	**0.82**	1.06										
Stringer 4 Skewed Span 2 Shear At Floor Beam Connection	2.17	2.81										
Stringers 5 and 6 Span 2 Positive Moment Midspan	1.06	1.37										
Stringers 5 and 6 Span 2 Shear At Floor Beam Connection	2.46	3.19										
Stringer 5 Skewed Span 2 Positive Moment Midspan	1.09	1.42										
Stringer 5 Skewed Span 2 Shear At Floor Beam Connection	2.50	3.23										
Stringer 6 Skewed Span 2 Positive Moment Midspan	**0.74**	**0.96**										
Stringer 6 Skewed Span 2 Shear At Floor Beam Connection	2.60	3.37										

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Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Stringer 7 Span 2 Positive Moment Midspan	**0.82**	1.06										
Stringer 7 Span 2 Shear At Floor Beam Connection	2.06	2.67										
Stringer 7 Skewed Span 2 Positive Moment Midspan	**0.72**	**0.93**			1.47	1.33	1.39	1.44	1.44	1.23	1.34	2.09
Stringer 7 Skewed Span 2 Shear At Floor Beam Connection	1.97	2.55			3.79	3.52	3.41	3.38	3.44	3.31	3.58	5.46
Stringer 8 Span 2 Positive Moment Midspan	**0.53**	**0.69**			1.51	1.42	1.47	1.51	1.49	1.28	1.39	2.16
Stringer 8 Span 2 Shear At Floor Beam Connection	2.08	2.69			5.47	4.91	4.81	4.77	4.87	4.59	4.95	7.52
Stringer 8 Skewed Span 2 Positive Moment Midspan	**0.66**	**0.86**										
Stringer 8 Skewed Span 2 Shear At Floor Beam Connection	2.64	3.43										
Stringer 3 Span 3 Positive Moment Midspan	**0.73**	**0.95**										
Stringer 3 Span 3 Shear At Floor Beam Connection	2.24	2.91										
Stringer 4 Span 3 Positive Moment Midspan	**0.80**	1.04										
Stringer 4 Span 3 Shear At Floor Beam Connection	1.61	2.08										
Stringers 5 and 6 Span 3 Positive Moment Midspan	**0.95**	1.23										
Stringers 5 and 6 Span 3 Shear At Floor Beam Connection	1.99	2.57										

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Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Stringer 7 Span 3 Positive Moment Midspan	**0.79**	1.03			1.48	1.48	1.48	1.69	1.52	1.30	1.45	2.15
Stringer 7 Span 3 Shear At Floor Beam Connection	1.60	2.07			3.07	2.85	2.94	2.92	2.88	2.57	2.69	4.00
Stringer 8 Span 3 Positive Moment Midspan	**0.56**	**0.72**			1.60	1.60	1.60	1.83	1.64	1.41	1.56	2.34
Stringer 8 Span 3 Shear At Floor Beam Connection	1.88	2.43			4.82	4.49	4.62	4.58	4.52	4.04	4.23	6.29
Floor Beam Intermediate Span 1 & 2 Positive Moment Midspan	1.07	1.38										
Floor Beam Span 1 & 2 Intermediate Shear * At Truss Connection	**0.63**	**0.82**			**0.86**	**0.78**	**0.74**	**0.76**	**0.78**	**0.81**	**0.90**	1.42
Floor Beam Span 1 & 2 Interm. Edge of Effective Length of 18' Cover Plate - Moment **	**0.66**	**0.86**			**0.96**	**0.87**	**0.83**	**0.85**	**0.87**	**0.91**	1.01	1.58
Floor Beam Span 1 & 2 Interm. Edge of Effective Length of 10' Cover Plate - Moment	**0.89**	1.15										
Floor Beam End Span 1 & 2 Positive Moment Midspan	**0.99**	1.28										
Floor Beam End Span 1 & 2 Shear At Truss Connection	1.38	1.79										
Floor Beam End Span 1 & 2 Edge of Effective Length of Cover Plate - Moment	**0.87**	1.13			**0.96**	**0.87**	**0.83**	**0.85**	**0.87**	**0.91**	1.01	1.58
Floor Beam Intermediate Span 3 Positive Moment Midspan	**0.87**	1.12			1.31	1.06	1.08	1.12	1.15	1.06	1.16	1.82
Floor Beam Intermediate Span 3 Shear At Truss Connection	1.89	2.45			2.87	2.33	2.35	2.43	2.52	2.31	2.52	3.98
Floor Beam End Span 3 Positive Moment Midspan	**0.85**	1.10			1.26	1.22	1.22	1.26	1.16	1.07	1.08	1.57
Floor Beam End Span 3 Shear At Truss Connection	2.21	2.86			3.30	3.20	3.20	3.30	3.02	2.78	2.82	4.12

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Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Lower Chord S2 Roadway Axial Tension L0-L2	1.58	2.04			2.81	2.97	3.07	3.04	3.07	2.69	3.91	5.14
Lower Chord S2 Roadway Axial Tension L2-L4	1.62	2.10			2.90	3.02	3.14	3.14	3.14	2.77	4.02	5.27
Lower Chord S2 Roadway Axial Tension L4-L6	1.52	1.97			2.74	2.82	2.93	2.93	2.94	2.58	3.75	4.92
Lower Chord S2 Roadway Axial Tension L6-L8	1.71	2.22			3.06	3.19	3.31	3.31	3.31	2.92	4.24	5.56
Lower Chord S2 Roadway Axial Tension L8-L10	1.57	2.03			2.80	2.95	3.05	3.03	3.05	2.68	3.89	5.11
Upper Chord S2 Roadway Axial Compression U1-U2	**0.92**	1.19			1.64	1.71	1.78	1.77	1.78	1.56	2.27	2.98
Upper Chord S2 Roadway Axial Compression U2-U3	**0.94**	1.22			1.68	1.75	1.82	1.82	1.82	1.60	2.33	3.06
Upper Chord S2 Roadway Axial Compression U3-U5	**0.87**	1.13			1.56	1.62	1.68	1.68	1.68	1.48	2.15	2.82
Upper Chord S2 Roadway Axial Compression U5-U7	**0.90**	1.17			1.61	1.67	1.73	1.74	1.74	1.53	2.23	2.92
Upper Chord S2 Roadway Axial Compression U7-U9	**0.97**	1.26			1.74	1.82	1.89	1.88	1.89	1.66	2.41	3.17
Verticals S2 Roadway Axial Tension U1-L1	1.61	2.09			2.69	2.57	2.47	2.53	2.53	1.86	2.99	4.39
Verticals S2 Roadway Axial Tension U3-L3	1.71	2.21			2.93	2.71	2.60	2.65	2.69	1.93	3.10	4.59

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Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Diagonal S2 Roadway Axial Compression L0-U1	2.86	3.71			5.10	5.39	5.57	5.53	5.57	4.89	7.10	9.33
Diagonal S2 Roadway Axial Tension U1-L2	1.20	1.55			2.06	2.19	2.26	2.24	2.26	1.96	2.89	3.86
Diagonal S2 Roadway Axial Compression L2-U3	1.16	1.50			1.93	2.05	2.11	2.09	2.11	1.82	2.72	3.70
Diagonal S2 Roadway Axial Tension U3-L4	1.67	2.17			2.72	2.90	2.98	2.94	2.98	2.54	3.84	5.27
Diagonal S2 Roadway Axial Tension L4-U5	1.96	2.54			3.06	3.26	3.35	3.28	3.34	2.77	4.28	6.02
Diagonal S2 Roadway Axial Tension U5-L6	2.18	2.83			3.39	3.61	3.71	3.63	3.70	3.05	4.73	6.67
Diagonal S2 Roadway Axial Tension L6-U7	1.46	1.89			2.39	2.54	2.62	2.58	2.61	2.23	3.37	4.62
Diagonal S2 Roadway Axial Compression U7-L8	**0.85**	1.10			1.42	1.51	1.56	1.54	1.56	1.35	2.01	2.72
Diagonal S2 Roadway Axial Tension L8-U9	1.11	1.44			1.92	2.03	2.10	2.08	2.10	1.83	2.69	3.59
Diagonal S2 Roadway Axial Compression U9-L10	2.86	3.70			5.09	5.37	5.56	5.51	5.55	4.88	7.08	9.30
Lower Chord S2 Sidewalk Axial Tension L0-L2	1.65	2.14			2.93	3.10	3.20	3.17	3.20	2.81	4.08	5.37
Lower Chord S2 Sidewalk Axial Tension L2-L4	1.87	2.42			3.33	3.46	3.60	3.60	3.60	3.17	4.61	6.06
Lower Chord S2 Sidewalk Axial Tension L4-L6	1.71	2.22			3.08	3.18	3.29	3.30	3.30	2.90	4.22	5.56
Lower Chord S2 Sidewalk Axial Tension L6-L8	1.90	2.46			3.38	3.51	3.65	3.65	3.65	3.22	4.68	6.15
Lower Chord S2 Sidewalk Axial Tension L8-L10	1.69	2.19			2.99	3.16	3.27	3.24	3.27	2.87	4.17	5.49

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Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Upper Chord S2 Sidewalk Axial Compression U1-U2	1.19	1.54			2.11	2.20	2.29	2.28	2.29	2.01	2.93	3.85
Upper Chord S2 Sidewalk Axial Compression U2-U3	1.18	1.53			2.09	2.19	2.27	2.27	2.28	2.00	2.91	3.82
Upper Chord S2 Sidewalk Axial Compression U3-U5	1.09	1.41			1.95	2.02	2.09	2.10	2.10	1.84	2.69	3.53
Upper Chord S2 Sidewalk Axial Compression U5-U7	1.06	1.37			1.89	1.95	2.03	2.04	2.03	1.79	2.60	3.42
Upper Chord S2 Sidewalk Axial Compression U7-U9	1.13	1.46			2.01	2.09	2.18	2.17	2.18	1.92	2.79	3.67
Verticals S2 Sidewalk Axial Tension U1-L1	1.66	2.15			2.85	2.63	2.53	2.61	2.61	1.85	2.99	4.40
Verticals S2 Sidewalk Axial Tension U3-L9	1.60	2.07			2.75	2.54	2.44	2.49	2.52	1.81	2.90	4.30
Diagonal S2 Sidewalk Axial Compression L0-U1	2.57	3.33			4.55	4.81	4.97	4.93	4.97	4.36	6.34	8.35
Diagonal S2 Sidewalk Axial Tension U1-L2	1.45	1.88			2.48	2.63	2.71	2.69	2.71	2.36	3.48	4.65
Diagonal S2 Sidewalk Axial Compression L2-U3	**0.59**	**0.77**			**0.99**	1.05	1.08	1.07	1.08	**0.93**	1.39	1.88
Diagonal S2 Sidewalk Axial Tension U3-L4	1.31	1.70			2.13	2.27	2.34	2.31	2.33	1.99	3.01	4.13
Diagonal S2 Sidewalk Axial Tension L4-U5	2.23	2.89			3.45	3.68	3.78	3.69	3.77	3.10	4.81	6.80
Diagonal S2 Sidewalk Axial Tension U5-L6	2.00	2.59			3.11	3.31	3.40	3.33	3.40	2.80	4.34	6.12
Diagonal S2 Sidewalk Axial Tension L6-U7	1.51	1.96			2.45	2.62	2.69	2.65	2.69	2.29	3.47	4.76
Diagonal S2 Sidewalk Axial Compression U7-L8	**0.86**	1.12			1.43	1.52	1.57	1.55	1.57	1.35	2.02	2.75

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	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Diagonal S2 Sidewalk Axial Tension L8-U9	**0.92**	1.19			1.58	1.67	1.72	1.71	1.72	1.50	2.21	2.96
Diagonal S2 Sidewalk Axial Compression U9-L10	**0.96**	1.25			1.71	1.80	1.86	1.85	1.86	1.63	2.38	3.13
Lower Chord S3 Roadway Axial Tension L0-L2	1.83	2.37			2.62	2.83	2.93	2.87	2.92	3.03	4.01	6.33
Lower Chord S3 Roadway Axial Tension L2-L4	1.41	1.83			2.05	2.13	2.21	2.23	2.23	2.32	3.10	4.86
Lower Chord S3 Roadway Axial Tension L6-L7	1.67	2.17			2.39	2.58	2.67	2.62	2.67	2.76	3.66	5.77
Upper Chord S3 Roadway Axial Compression U1-U3	1.15	1.48			1.65	1.73	1.81	1.80	1.81	1.89	2.51	3.94
Upper Chord S3 Roadway Axial Compression U3-U4	1.21	1.56			1.78	1.83	1.89	1.90	1.91	1.98	2.63	4.16
Verticals S3 Roadway Axial Tension U1-L1	1.23	1.60			1.84	1.51	1.54	1.59	1.63	1.41	1.64	2.56
Verticals S3 Roadway Axial Tension U3-L3	**0.89**	1.15			1.33	1.09	1.10	1.14	1.17	1.02	1.19	1.86
Diagonal S3 Roadway Axial Compression L0-U1	2.31	3.00			3.31	3.57	3.70	3.62	3.69	3.83	5.07	7.99
Diagonal S3 Roadway Axial Tension U1-L2	1.12	1.46			1.53	1.68	1.71	1.67	1.71	1.79	2.31	3.63
Diagonal S3 Roadway Axial Compression L2-U3	1.06	1.37			1.39	1.53	1.54	1.49	1.53	1.62	2.04	3.20
Diagonal S3 Roadway Axial Tension U3-L4	1.31	1.70			1.68	1.83	1.85	1.78	1.84	1.92	2.37	3.71
Diagonal S3 Roadway Axial Tension L6-U7	1.17	1.52			1.60	1.75	1.78	1.74	1.78	1.86	2.41	3.79

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	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Lower Chord S3 Sidewalk Axial Tension L0-L2	1.78	2.31										
Lower Chord S3 Sidewalk Axial Tension L2-L4	1.86	2.42										
Upper Chord S3 Sidewalk Axial Compression U1-U3	**0.95**	1.24										
Upper Chord S3 Sidewalk Axial Compression U3-U4	1.01	1.31										
Verticals S3 Sidewalk Axial Tension U1-L1	1.15	1.49										
Verticals S3 Sidewalk Axial Tension U3-L3	1.12	1.45										
Diagonal S3 Sidewalk Axial Compression L0-U1	2.01	2.60										
Diagonal S3 Sidewalk Axial Tension U1-L2	1.01	1.31										
Diagonal S3 Sidewalk Axial Compression L2-U3	1.35	1.75										
Diagonal S3 Sidewalk Axial Tension U3-L4	2.14	2.77										
Gusset Plate S2 Roadway Bottom Chord L1	**0.92**	1.20			1.54	1.47	1.41	1.45	1.45	1.20	1.71	2.52
Gusset Plate S2 Roadway Bottom Chord L2	**0.99**	1.28										
Bottom Chord L4 L0	1.54	2.00										
Gusset Plate S2 Roadway Bottom Chord L5	**0.95**	1.23										

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	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Gusset Plate S2 Roadway Bottom Chord L6	1.34	1.74										
Gusset Plate S2 Roadway Bottom Chord L8	1.11	1.44										
Gusset Plate S2 Roadway Bottom Chord L9	**0.99**	1.29										
Gusset Plate S2 Roadway Bottom Chord L10	**0.94**	**1.22**			1.68	1.78	1.84	1.82	1.84	1.82	2.34	3.08
Gusset Plate S2 Roadway Upper Chord U1	**0.86**	1.11			1.48	1.57	1.55	1.59	1.59	1.32	1.88	2.76
Gusset Plate S2 Roadway Upper Chord U3	1.04	1.35										
Gusset Plate S2 Roadway Upper Chord U5	1.04	1.35										
Gusset Plate S2 Roadway Upper Chord U7	1.04	1.35										
Gusset Plate S2 Roadway Upper Chord U9	1.09	1.41										
Gusset Plate S2 Sidewalk Bottom Chord L0	**0.67**	**0.87**			1.19	1.26	1.30	1.29	1.30	1.28	1.66	2.18
Gusset Plate S2 Sidewalk Bottom Chord L2	1.05	1.36										
Bottom Chord L4 L0	1.19	1.54										
Gusset Plate S2 Sidewalk Bottom Chord L5	2.42	3.13										
Gusset Plate S2 Sidewalk Bottom Chord L6	1.39	1.80										

BREAKDOWN OF TRUSS BRIDGE RATING

Town/City: Brunswick
Bridge No: 2016
Route Carried: US 201 & Rt. 24
Crosses: The Androscoggin River

LOAD RATING POINTS OF INTEREST

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Gusset Plate S2 Sidewalk Bottom Chord L8	1.01	1.30										
Gusset Plate S2 Sidewalk Bottom Chord L10	**0.71**	**0.92**			1.25	1.33	1.37	1.36	1.37	1.35	1.75	2.30
Gusset Plate S2 Sidewalk Upper Chord U1	1.02	1.33										
Gusset Plate S2 Sidewalk Upper Chord U3	**0.94**	1.22			1.61	1.49	1.43	1.46	1.48	1.20	1.70	2.52
Gusset Plate S2 Sidewalk Upper Chord U5	**0.93**	1.21			1.60	1.48	1.42	1.45	1.47	1.19	1.69	2.51
Gusset Plate S2 Sidewalk Upper Chord U7	**0.94**	1.21			1.61	1.48	1.43	1.45	1.47	1.19	1.70	2.51
Gusset Plate S2 Sidewalk Upper Chord U9	**0.98**	1.27										
Gusset Plate S3 Roadway Bottom Chord L0	**0.91**	1.18			1.30	1.40	1.45	1.42	1.45	1.50	1.99	3.14
Gusset Plate S3 Roadway Bottom Chord L1	**0.88**	1.14			1.31	1.07	1.10	1.14	1.16	1.00	1.17	1.83
Bottom Chord L2 L0	1.02	1.32										
Gusset Plate S3 Roadway Bottom Chord L3	**0.85**	1.11			1.28	1.04	1.06	1.10	1.13	**0.98**	1.14	1.79
Gusset Plate S3 Roadway Bottom Chord L4	1.60	2.07										
Gusset Plate S3 Roadway Upper Chord U1	1.10	1.43										
Gusset Plate S3 Roadway Upper Chord U3	1.08	1.40										

Made by: MSD
Date: 3/25/2013
Checked by:AMC
Date:4/1/13

BREAKDOWN OF TRUSS BRIDGE RATING

Town/City: Brunswick
Bridge No: 2016

Route Carried: US 201 & Rt. 24
Crosses: The Androscoggin River

LOAD RATING POINTS OF INTEREST

Bridge Component	HL-93		HL-93 Modified		MaineDOT Truck Configurations							
	Inv	Oper	Inv	Oper	1	2	3	4	5	6	7	8
	72.0 kip	72.0 kip	90.0 kip	90.0 kip	100.0 kip	94.0 kip	88.0 kip	88.0 kip	88.0 kip	75.9 kip	59.0 kip	37.4 kip
Gusset Plate S3 Sidewalk Bottom Chord L0	**0.72**	**0.94**			1.04	1.12	1.16	1.13	1.15	1.19	1.59	2.50
Gusset Plate S3 Sidewalk Bottom Chord L1	2.15	2.79										
Bottom Chord L2 L0	**0.94**	1.22			1.29	1.41	1.43	1.40	1.43	1.50	1.94	3.05
Gusset Plate S3 Sidewalk Bottom Chord L3	2.11	2.74										
Gusset Plate S3 Sidewalk Bottom Chord L4	1.54	1.99										
Gusset Plate S3 Sidewalk Upper Chord U1	**0.94**	1.22			1.29	1.25	1.28	1.32	1.35	1.17	1.36	2.13
Gusset Plate S3 Sidewalk Upper Chord U3	**0.99**	1.28										
CONTROLLING RATING FACTORS	**0.53**	**0.69**	**0.00**	**0.00**	**0.86**	**0.78**	**0.74**	**0.76**	**0.78**	**0.81**	**0.90**	1.42

* Includes Shear Section Losses. Rates at 0.90 Inv / 1.16 Op without losses

** Includes Flexural Section Losses. Rates at 0.69 Inv / 0.89 Op without losses

APPENDIX D

Existing Plans

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BRUNSWICK - TOPSHAM
CUMBERLAND COUNTY
FRANK J. WOOD BRIDGE
OVER
ANDROSCOGGIN RIVER
PROJECT NO. 20467.00

BRIDGE JOINT REPLACEMENT

BRIDGE NO. 2016

LOCATION: 43° 55' 14.73" N 69° 57' 57.30" W

SIGNATURE

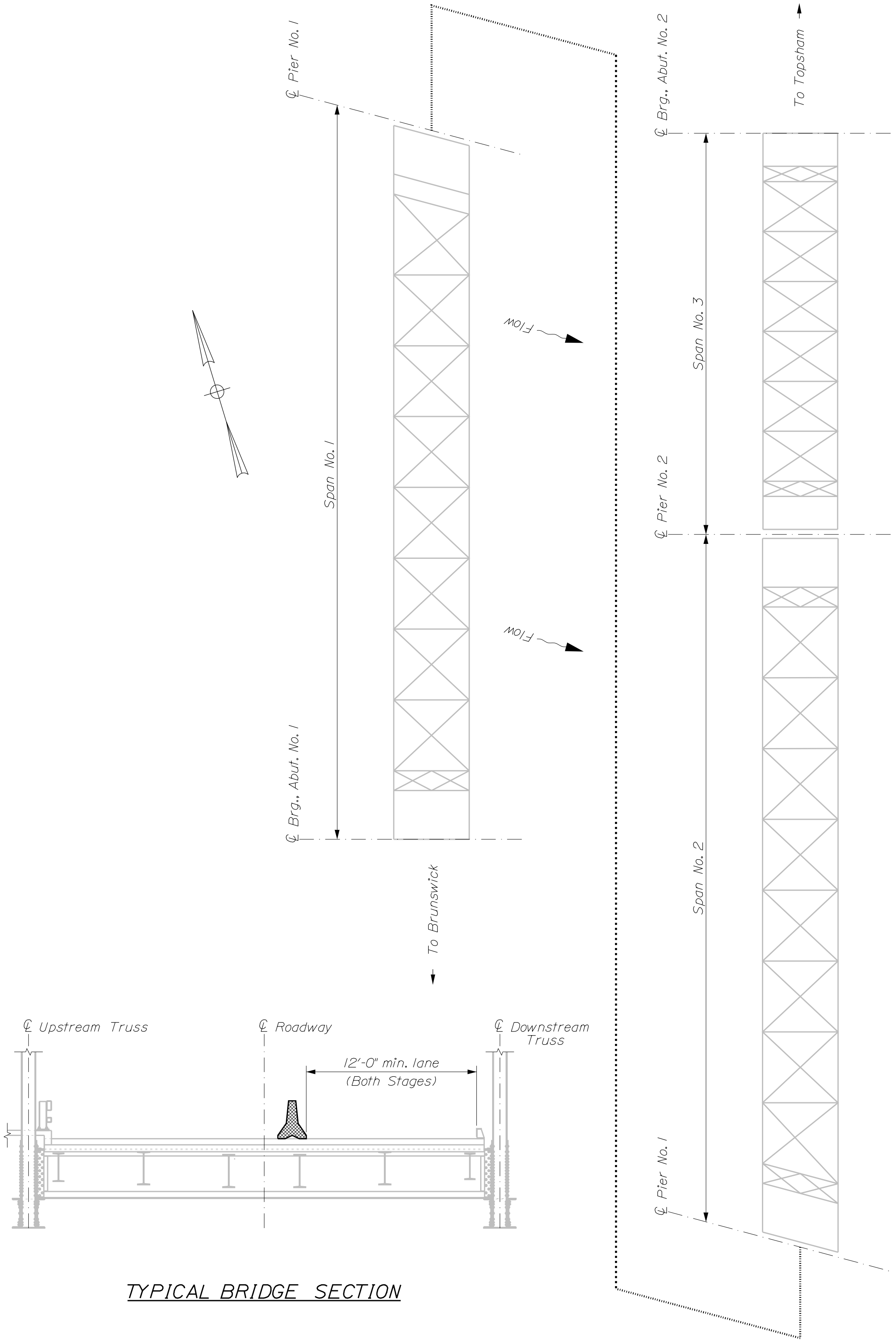
P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	
CHIEF ENGINEER:	

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	BRUNSWICK - TOPSHAM CUMBERLAND COUNTY	SHEET NUMBER 1 OF 9
20467.00	TITLE SHEET	



TYPICAL BRIDGE SECTION

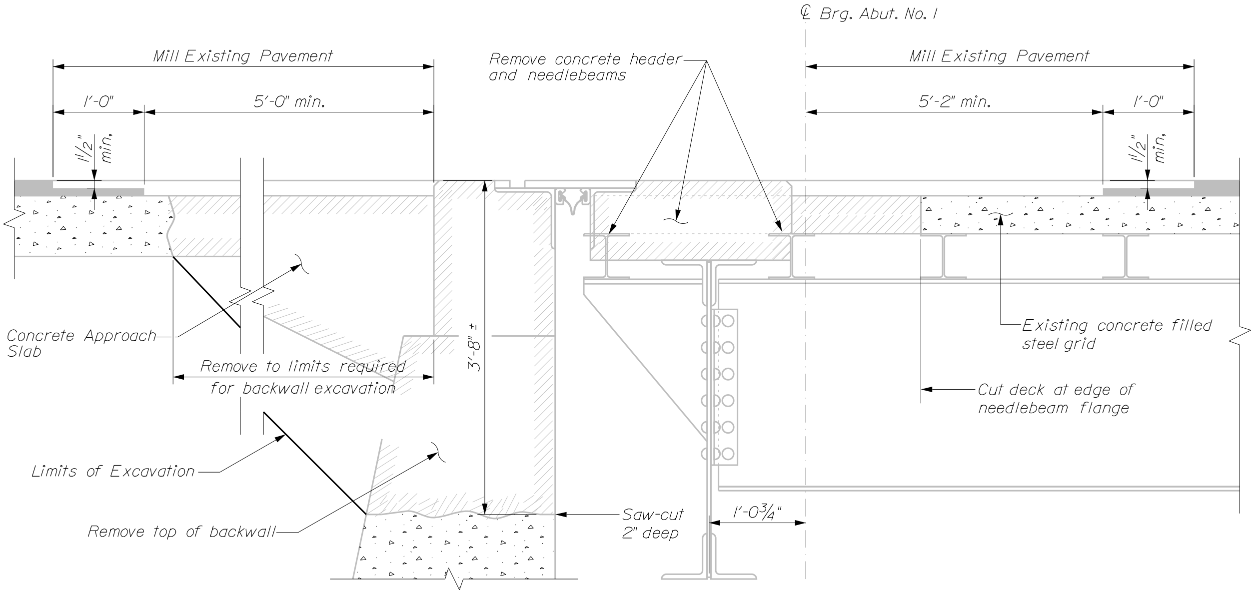
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER
2
OF 9

STP-2046(700)
20467.00 BRIDGE NO. 2016

GENERAL PLAN



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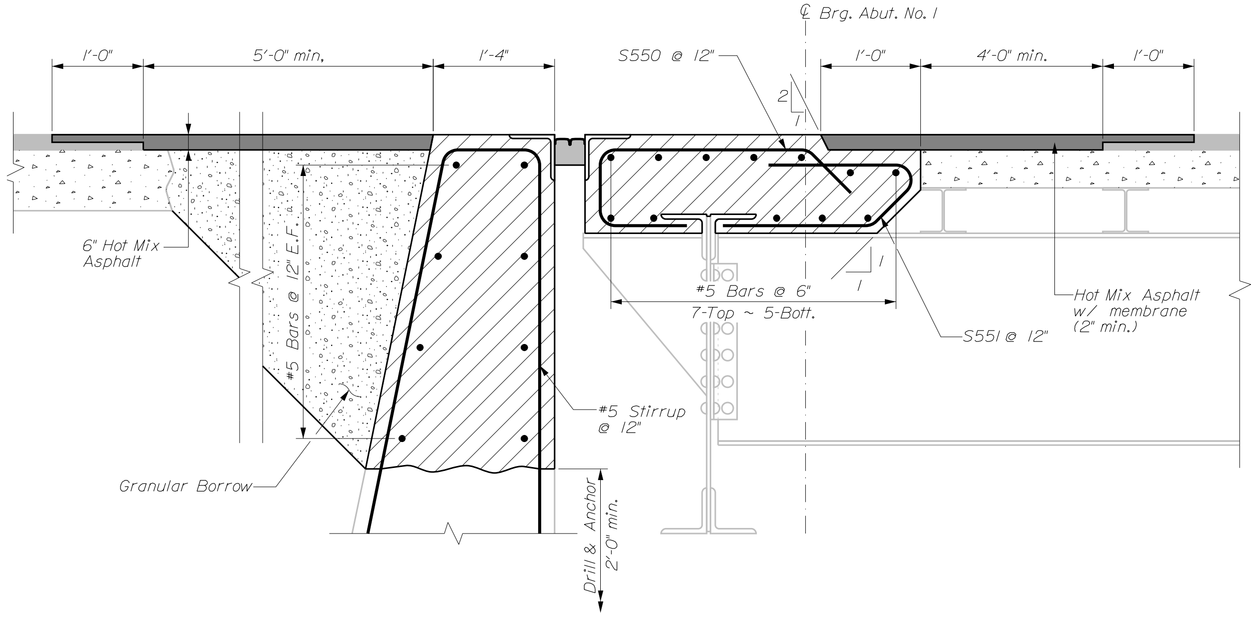


ABUTMENT NO. 1 JOINT MODIFICATION

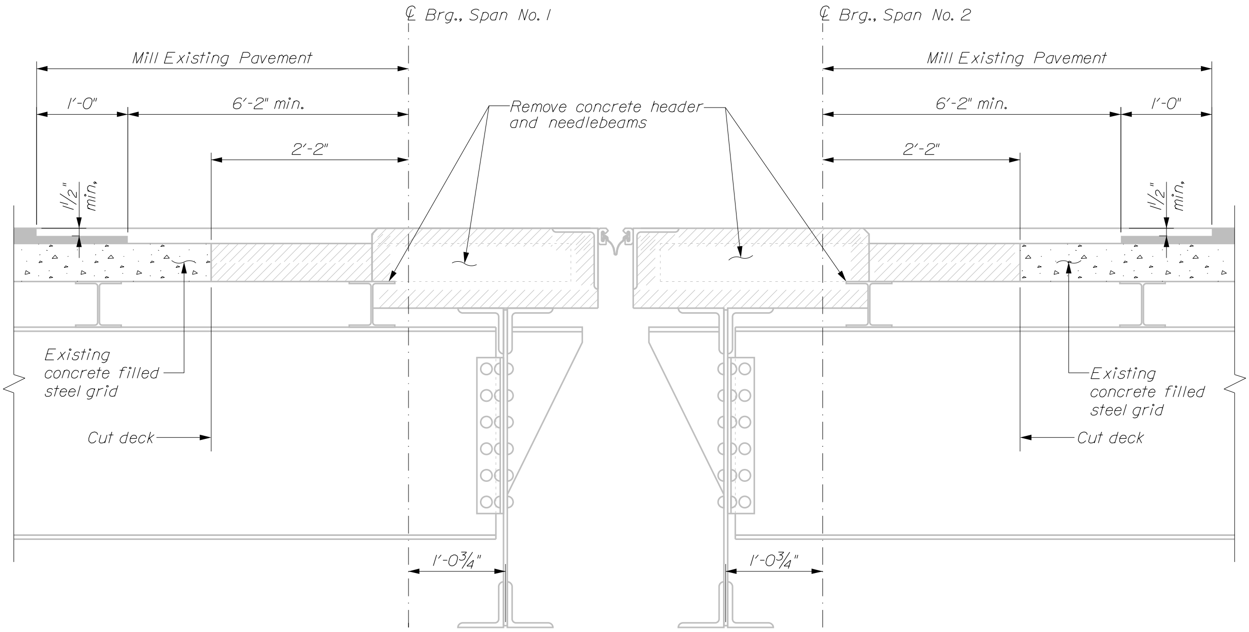
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER
3
OF 9

STP-2046(700)
20467.00 BRIDGE NO. 2016

ABUTMENT NO. 1



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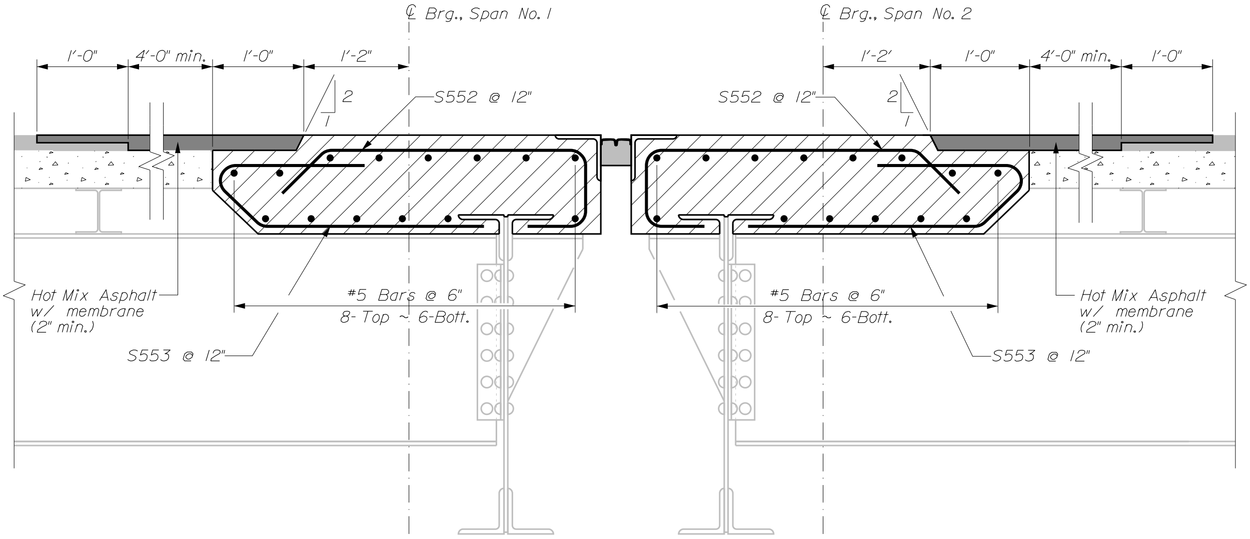
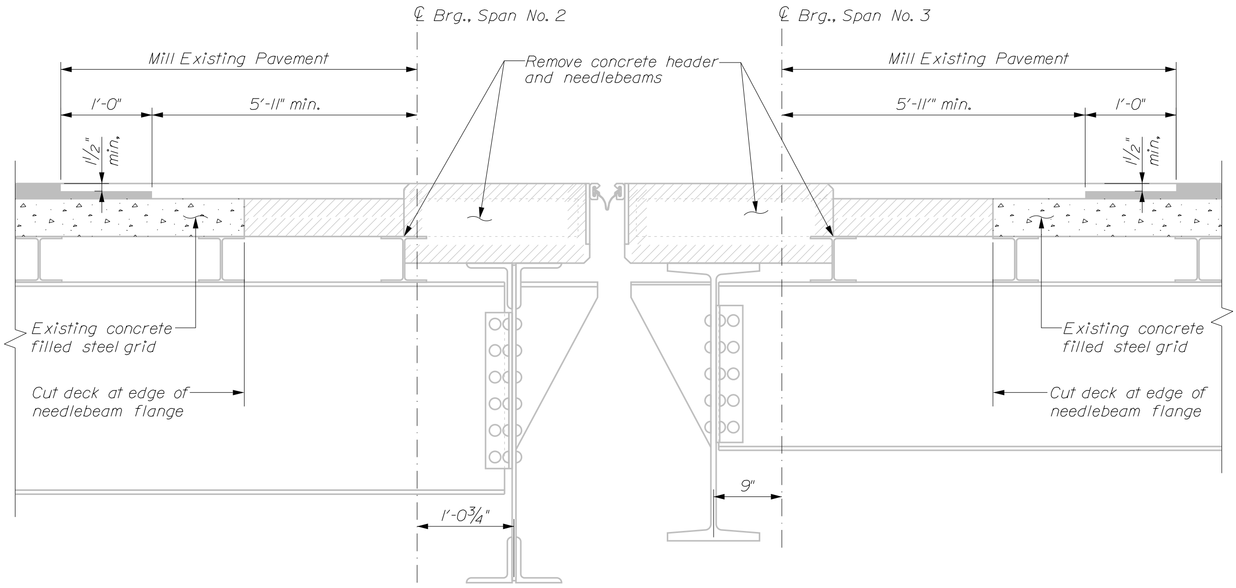


PIER NO. 1 JOINT MODIFICATION

STATE OF MAINE DEPARTMENT OF TRANSPORTATION	FRANK J. WOOD BRIDGE ANDROSCOGGIN RIVER BRUNSWICK - TOPSHAM CUMBERLAND COUNTY	SHEET NUMBER 4
STP-2046(700) 20467.00 BRIDGE NO. 2016	PIER NO. 1	OF 9



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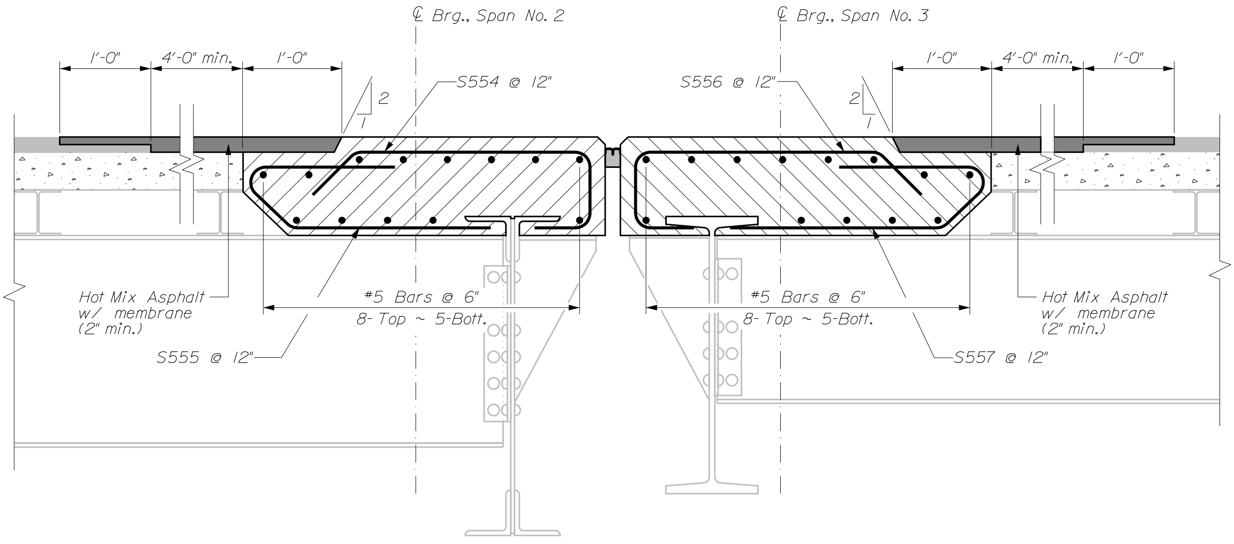


PIER NO. 2 JOINT MODIFICATION

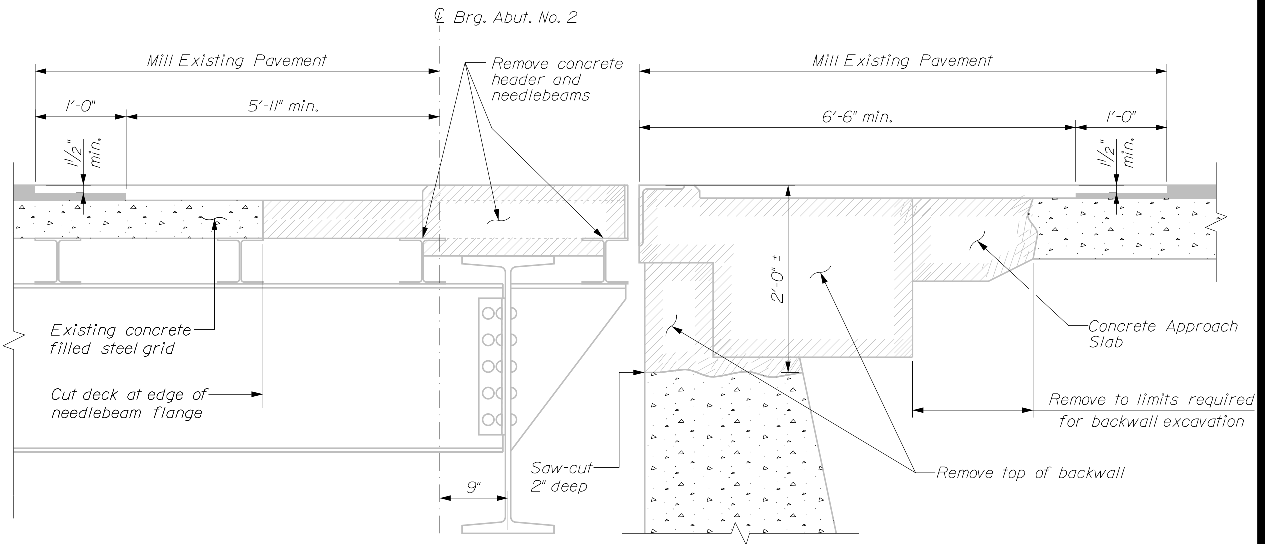
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER
5
OF 9

STP-2046(700)
20467.00 BRIDGE NO. 2016

PIER NO. 2



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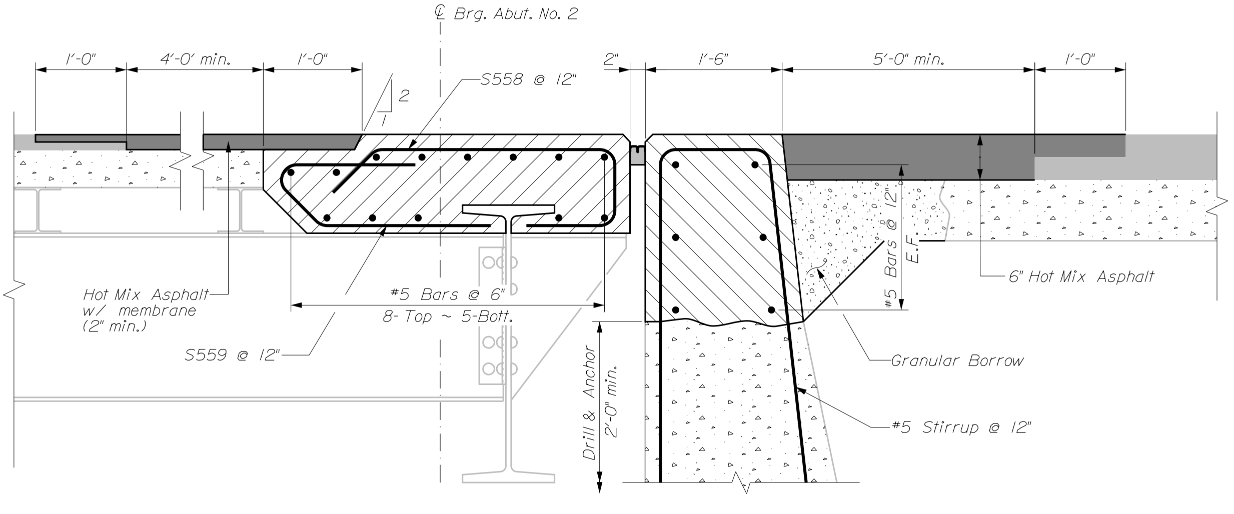


ABUTMENT NO. 2 JOINT MODIFICATION

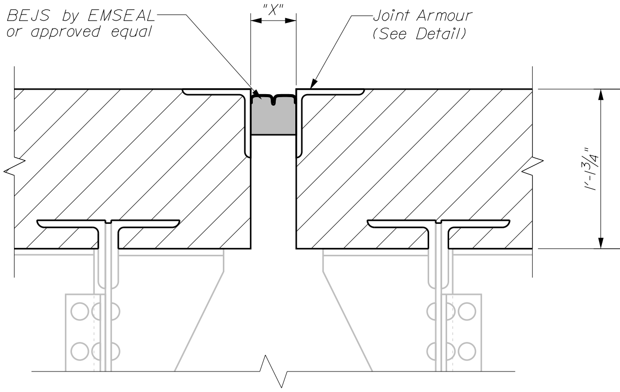
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

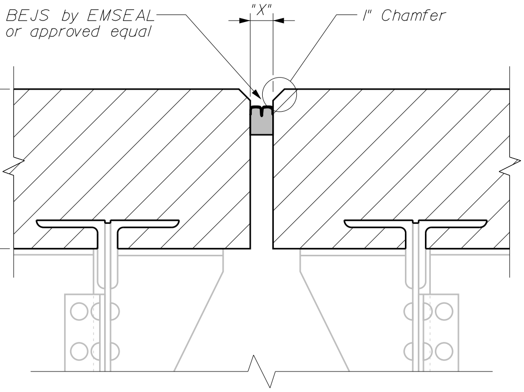
SHEET NUMBER
6
OF 9

STP-2046(700)
20467.00 BRIDGE NO. 2016

ABUTMENT NO. 2



ABUTMENT NO. 1 & PIER NO. 1 JOINT DETAIL



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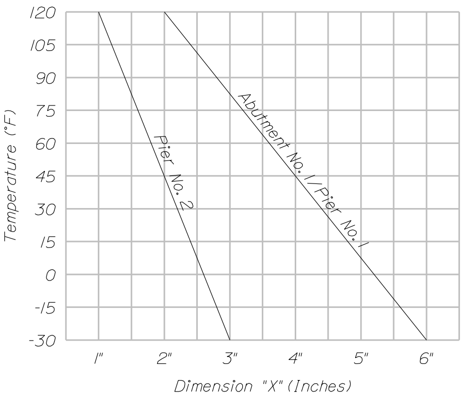
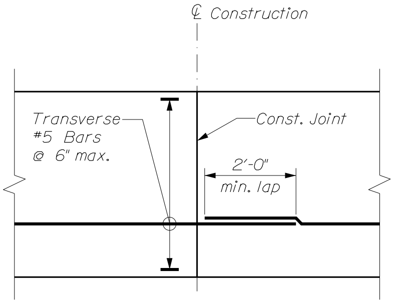
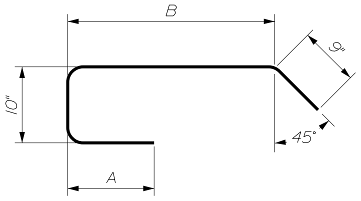


EXPANSION JOINT ADJUSTMENT CHART



CONSTRUCTION JOINT DETAIL

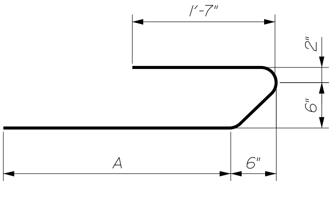
The Contractor may substitute mechanical couplers for the lap splices. No additional payment will be made.



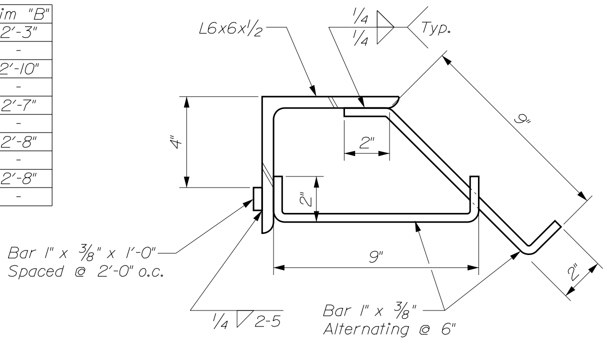
BEND TYPE "A"

BENT BARS

Bar	Qty	Type	Dim "A"	Dim "B"
S550	32	A	0'-11"	2'-3"
S551	32	B	1'-8"	-
S552	64	A	0'-8"	2'-10"
S553	64	B	2'-5"	-
S554	32	A	0'-8"	2'-7"
S555	32	B	2'-2"	-
S556	32	A	0'-8"	2'-8"
S557	32	B	2'-4"	-
S558	32	A	1'-0"	2'-8"
S559	32	B	2'-0"	-



BEND TYPE "B"



ARMOR DETAIL

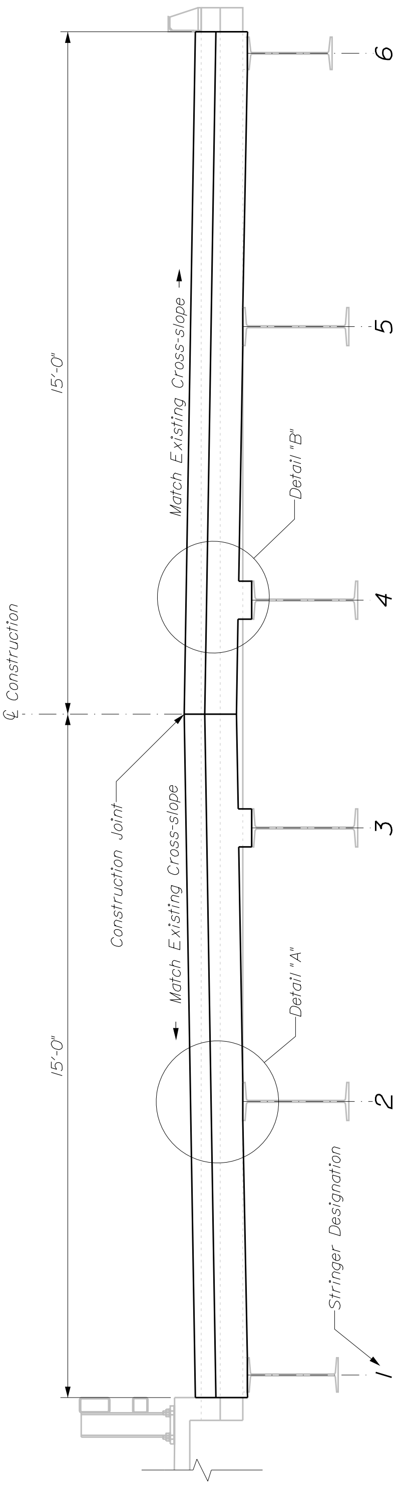
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER
7
OF 9

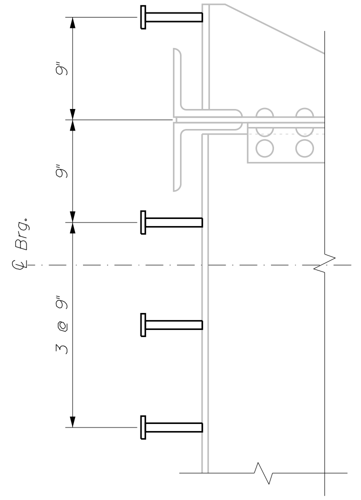
STP-2046(700)
20467.00 BRIDGE NO. 2016

JOINT DETAILS

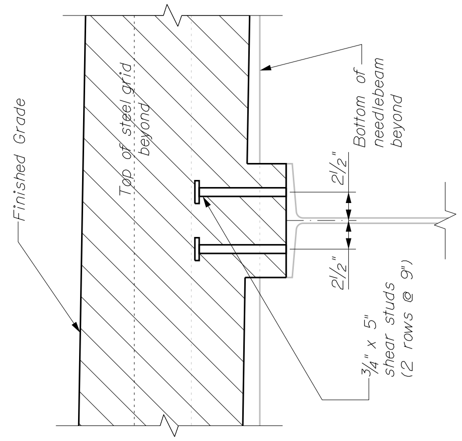


EXPANSION JOINT ELEVATION

Flow

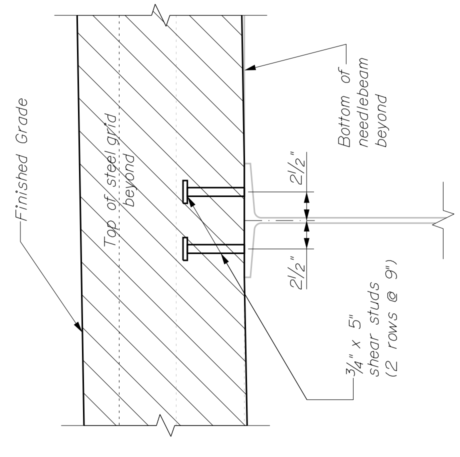


SHEAR STUD LAYOUT



DETAIL "B"

Stringers 3,4



DETAIL "A"

Stringers 1,2,5,6

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER

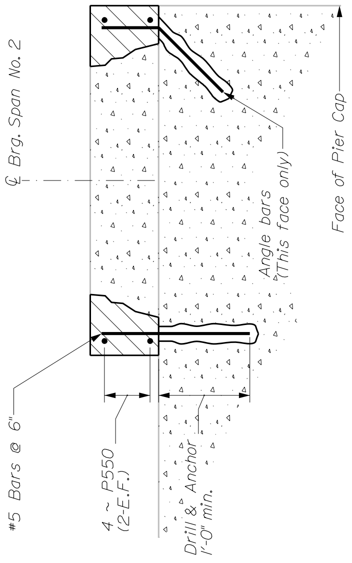
8

OF 9

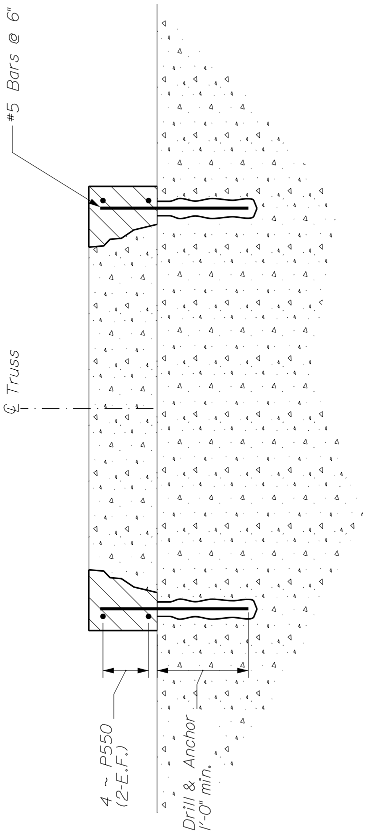
STP-2046(700)

20467.00 BRIDGE NO. 2016

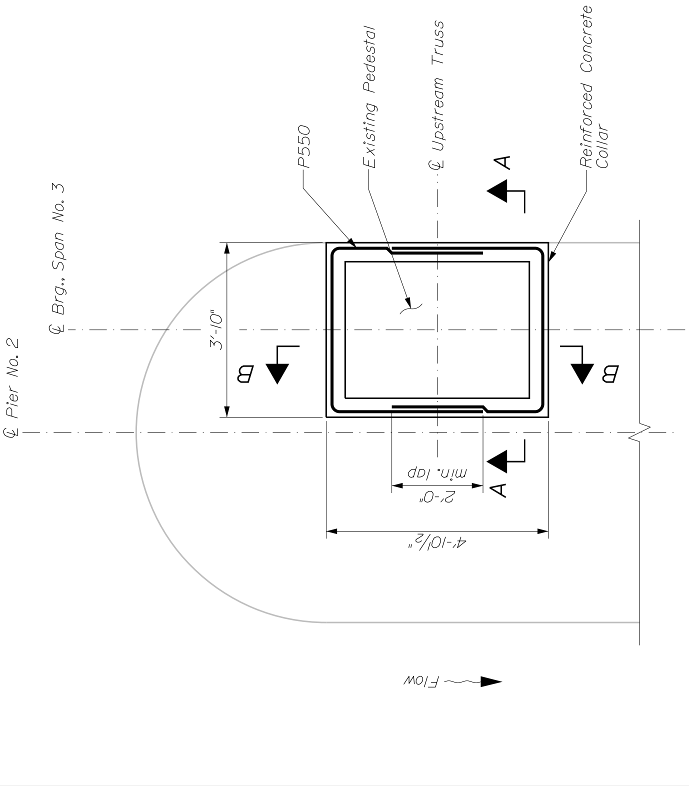
JOINT DETAILS



SECTION "A-A"



SECTION "B-B"



PEDESTAL PLAN

Upstream pedestal show - downstream pedestal similar

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK - TOPSHAM CUMBERLAND COUNTY

SHEET NUMBER
9
OF 9

STP-2046(700)
20467.00 BRIDGE NO. 2016

PIER NO. 2 BEARING PEDESTALS

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

SPECIFICATIONS

DESIGN: AASHTO LRFD Bridge Design Specifications, Third Edition 2004.

DESIGN LOADING

Live Load	HL - 93 Modified

TRAFFIC DATA

Current (2003) AADT	17,800
Future (2023) AADT	21,360
DHV - % of AADT	10
Design Hour Volume	2136
% Heavy Trucks (AADT)	3
% Heavy Trucks (DHV)	2
Directional Distribution (DHV)	55
18 kip Equivalent P 2.0	234
18 kip Equivalent P 2.5	232
Design Speed (mph)	25

MATERIALS

Concrete (Unless noted otherwise)	Class "A"
Concrete (Sidewalk)	Class "LP"
Reinforcing Steel	ASTM A615/A615M, Grade 60
Structural Steel:	
All Material (except as noted)	ASTM A709/A709M, Grade 36
High Strength Bolts	ASTM A325, Type 3

BASIC DESIGN STRESSES

Concrete	f 'c = 4,000 psi
Reinforcing Steel	f y = 60,000 psi
Structural Steel:	
ASTM A 709/A 709M, Grade 36	F y = 50,000 psi
ASTM A 325	F y = 36,000 psi

UTILITIES

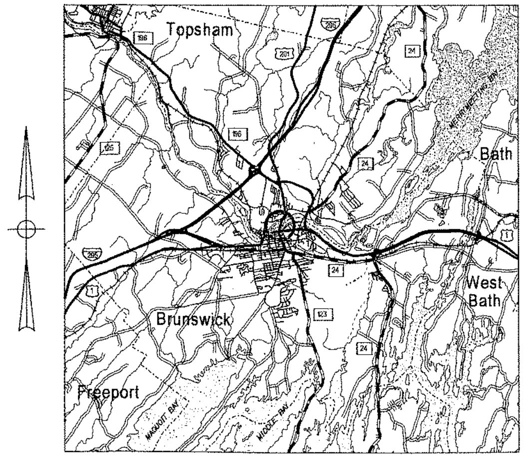
Brunswick - Topsham Water District
Central Maine Power Company
FPL Energy Maine
Maine Natural Gas
Susquehanna Communications
Verizon

MAINTENANCE OF TRAFFIC

One two - week full bridge closure will be allowed.
Maintain two 11'-0" wide traffic lanes during construction.
Maintain pedestrian traffic at all times except during closure.

BRUNSWICK, CUMBERLAND CTY.
TOPSHAM, SAGADAHOC CTY.
FRANK J. WOOD BRIDGE
OVER
ANDROSCOGGIN RIVER
U. S. ROUTE 201 / STATE ROUTE 24

PROJECT NO. BH-1108(900)X
PROJECT LENGTH: 0.154 mi.
BRIDGE REHABILITATION
BRIDGE NO. 2016



LOCATION MAP

2 0 2 4
Scale in Miles

LIST OF DRAWINGS

141-77

Filename: ...\00\BRIDGE\MSTA\001_Title.dgn Division: BRIDGE Username: asif.iqbal Date:3/21/2006

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

APPROVED	DATE
COMMISSIONER:	3/27/06
CHIEF ENGINEER:	03/27/06

M. Asif Iqbal 11030

M. Asif Iqbal
SIGNATURE
11030
P.E. NUMBER
3/21/06
DATE

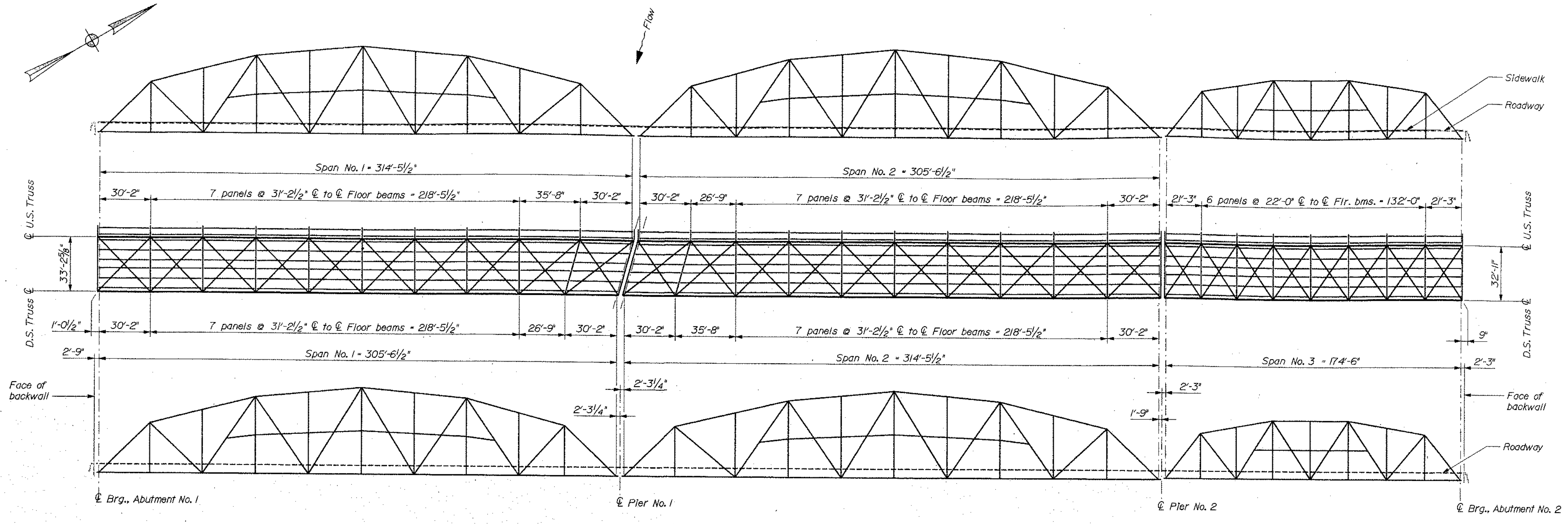
PROJECT INFORMATION	
PROGRAM	BRIDGE PROGRAM
PROJECT MANAGER	DEVIN ANDERSON
DESIGNER	M. ASIF IQBAL
CONSULTANT	N/A
PROJECT RESIDENT	PETER BROWN
CONTRACTOR	
PROJECT COMPLETION DATE	

BH-1108(900)X PIN 11089.00

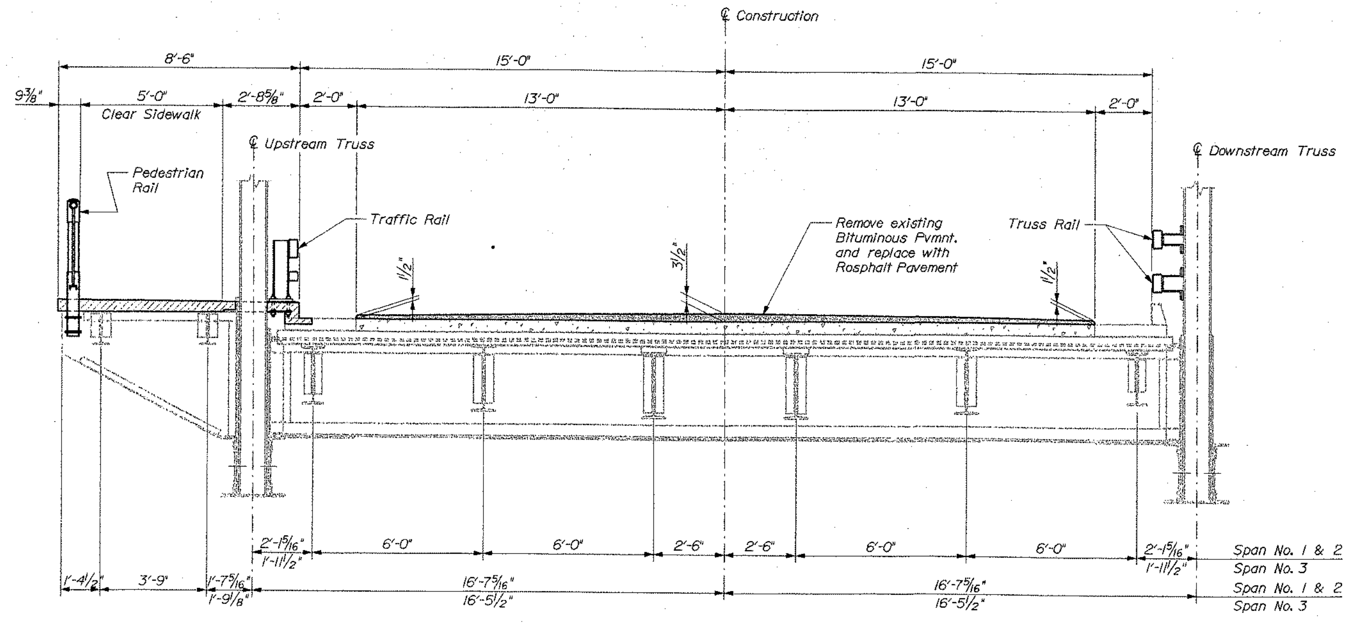
BRUNSWICK & TOPSHAM
FRANK J. WOOD BRIDGE

TITLE SHEET

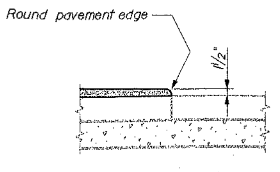
SHEET NUMBER
1
OF 21



GENERAL PLAN & ELEVATION



TRANSVERSE SECTION



PAVEMENT EDGE DETAIL

141-79

Filename: ...\msta\003_GeneralPlan.dgn   Division: BRIDGE   Username: dana.damren   Date:3/29/2006

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

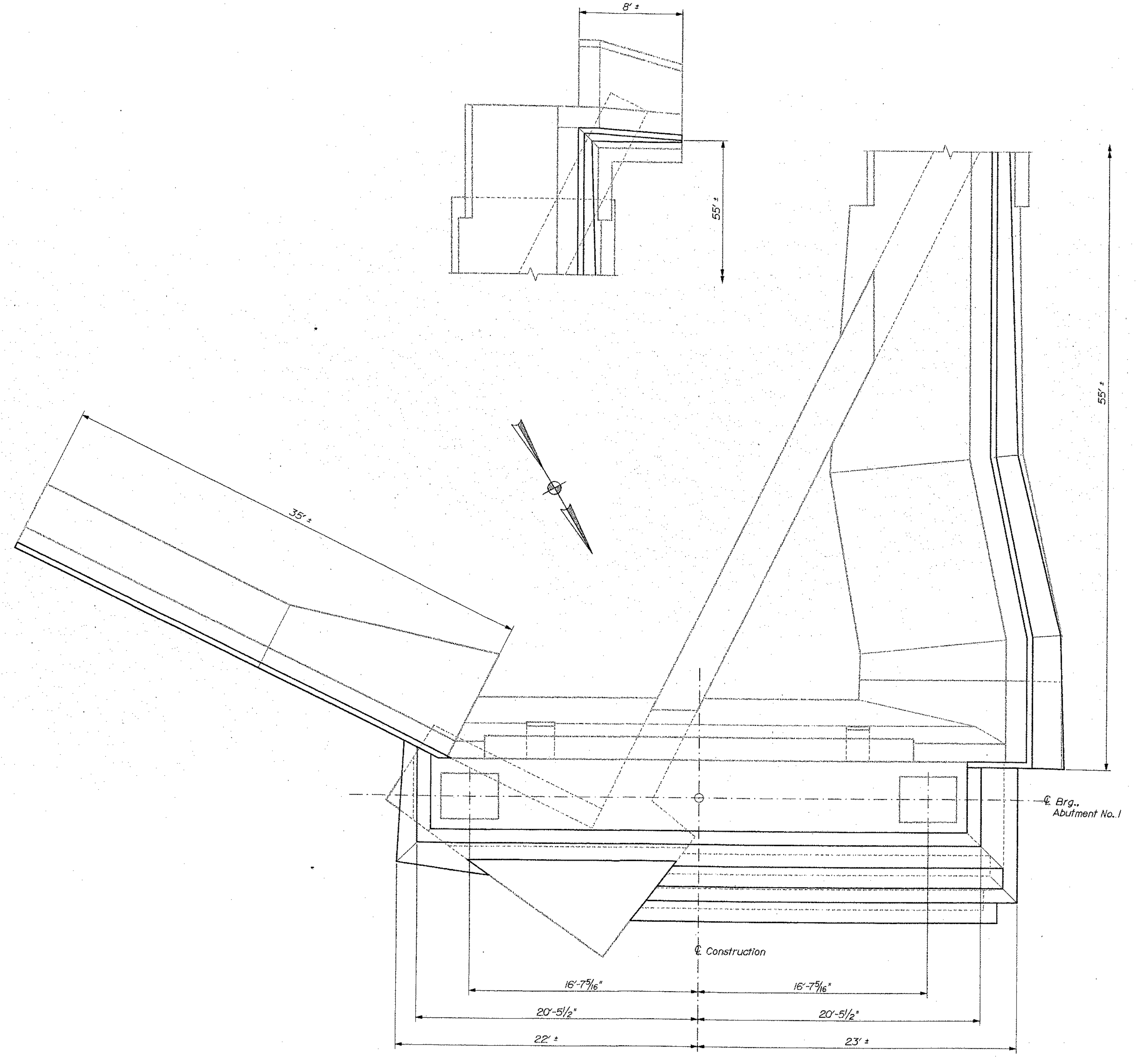
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

GENERAL PLAN

SHEET NUMBER
3
OF 21

Filename: ...\004_Abutment_No. 1 Plan.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



ABUTMENT NO. 1 PLAN

SUBSTRUCTURE NOTES

1. All substructure concrete repair, with the exception of the seal concrete for Pier No. 1, will be paid for under Item No. 518.61, Repair of Vertical Surfaces (> 7.9 inches), regardless of thickness.

2. Reinforcing steel shall have 3" minimum cover unless otherwise noted.

3. Reinforcing steel schedules shall be provided by the Contractor in accordance with Subsection 503.03 of the Standard Specifications. Payment for reinforcing steel will be considered incidental to Item No. 518.61, Repair of Vertical Surfaces (> 7.9 inches).

4. Drilling and anchoring of dowels into sound concrete shall be in accordance with Subsection 503.06 of the Standard Specifications. Payment for dilling and anchoring will be considered incidental to Item No. 518.61, Repair of Vertical Surfaces (> 7.9 inches).

5. Payment for filling voids under Pier No. 2 will be considered incidental to the concrete repair item.

6. Payment for excavation and replacement of material at the bases of abutment jackets will be made under equipment rental items.

CONCRETE ANCHOR NOTES

1. Concrete anchor design capacity is 67 kips per anchor.

2. Concrete anchors shall be 1 3/8" ϕ epoxy - grouted or resin - grouted threadbar anchors as manufactured by Dywidag Systems International, or Williams Form Engineering Corporation, or approved equal.

3. Concrete anchors shall be manufactured of steel having a yield strength equal to or greater than 150 ksi, conforming to ASTM A722.

4. Concrete anchors shall be installed in pre-drilled 1 3/4" ϕ holes.

5. The annular space between the threadbar and the drill hole shall be filled with a two - part epoxy or resin. The minimum seven day compressive strength of the mixed and cured epoxy/resin shall be 10,000 psi, when tested in accordance with ASTM C 39.

6. The epoxy/resin bond system with concrete shall have minimum two day bond strength of 1,100 psi and a seven day bond strength of 1,600 psi per ASTM C 882.

7. Epoxy/resin concrete anchor materials, construction, installation and acceptance shall be in conformance with Special Provision Section 504, Concrete Anchors.

8. The sequence of installation shall be as follows:
   Drill and clean hole.
   Insert resin cartridges or discharge epoxy into drill hole.
   Insert threadbar while spinning bar at about 100 rpm. The time for advancing the bar should be less than the epoxy/resin gel time.
   Mount bearing plate and secure anchor nut.

9. Allowance should be made for a minimum of 15% wastage of epoxy when estimating.

10. Epoxy/resin shall be warmed prior to use at temperatures below 70 °F.

11. The Contractor shall take all measures necessary to protect the existing approach pavement structure and existing bridge from damage throughout the concrete anchor installation procedures. All damage to the existing structure caused by the Contractor's operations shall be repaired at his expense.

12. The Contractor shall coordinate the spacing of concrete anchors to avoid interference with utilities.

141-80

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

PIN
11089.00

BRIDGE NO. 2016

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

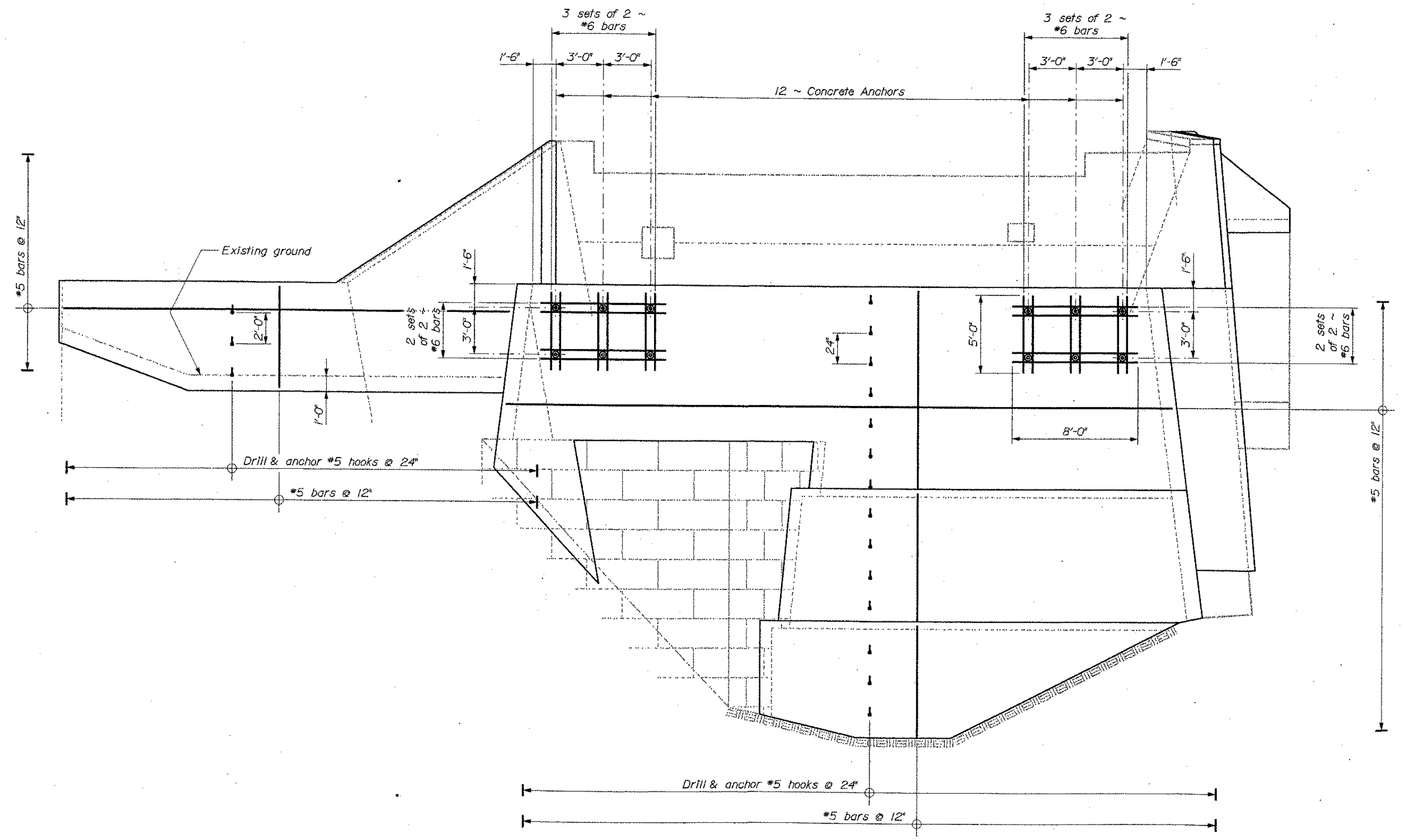
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

ABUT. NO. 1 PLAN & NOTES

SHEET NUMBER
4
OF 21

Filename: ...\005_Abutment_No. 1 Elevation.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



ABUTMENT NO. 1 ELEVATION

141-81

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

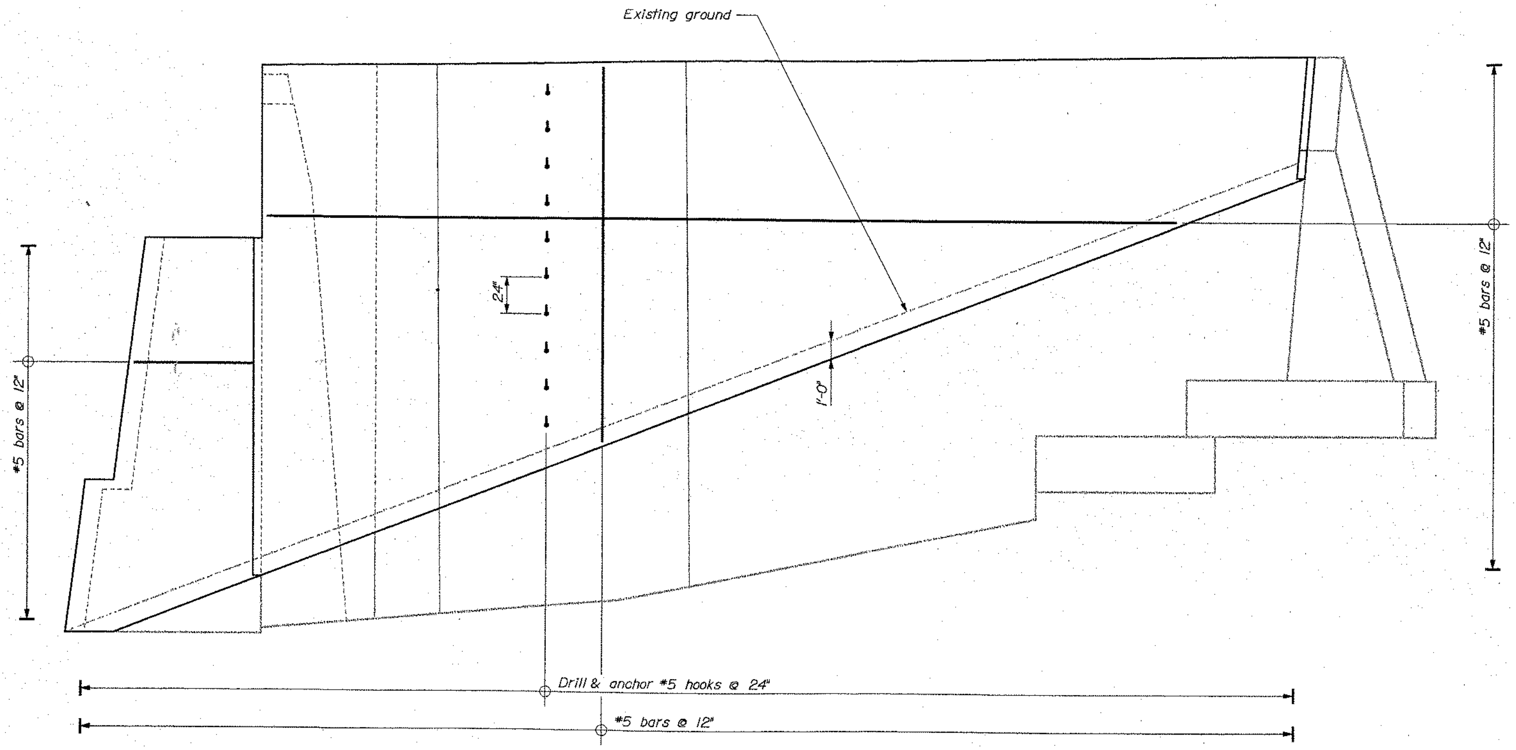
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

ABUTMENT NO. 1 ELEVATION

SHEET NUMBER
5
OF 21

Filename: ...\006_Abutment_No. 1 Wingwall.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



ABUTMENT NO. I WEST WINGWALL ELEVATION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

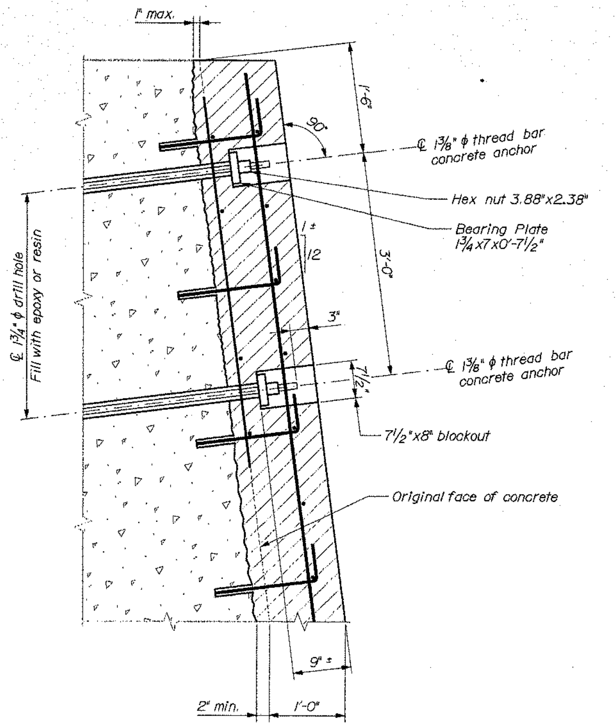
ABUTMENT NO. 1 WINGWALL

SHEET NUMBER
6
OF 21

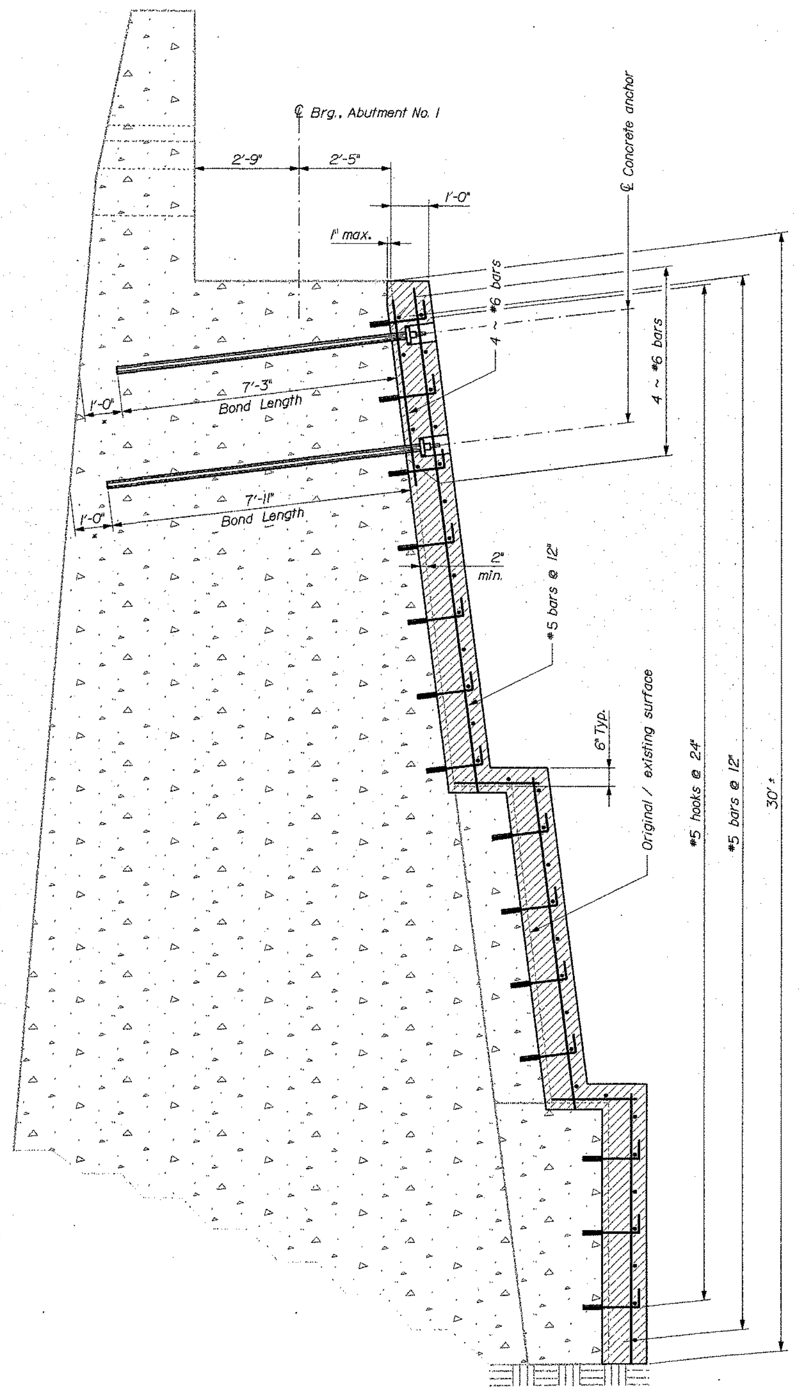
141-82

Filename: ...\007_Abutment_No. 1 Sections.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006

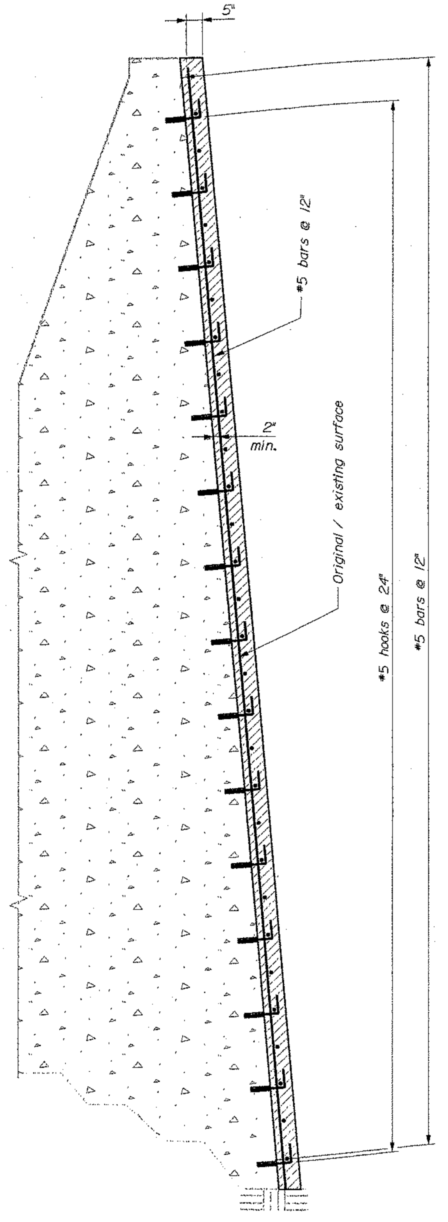
* 1'-0" clearance and bond length determined from dimensions shown on original construction drawings



CONCRETE ANCHOR DETAIL



BREASTWALL SECTION



WINGWALL SECTION

SYMBOLS

░ Existing concrete to remain

▨ New concrete (in section)

141-83

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

ABUTMENT NO. 1 SECTIONS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

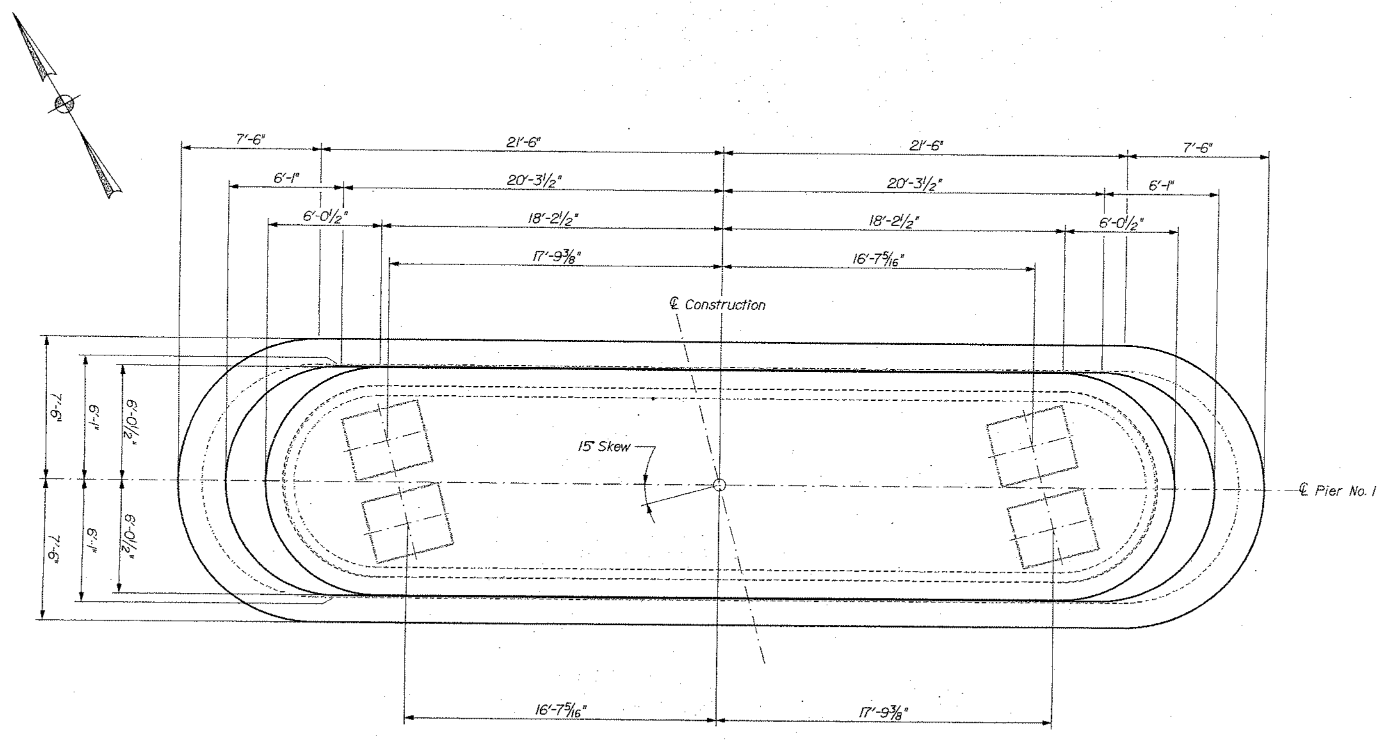
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

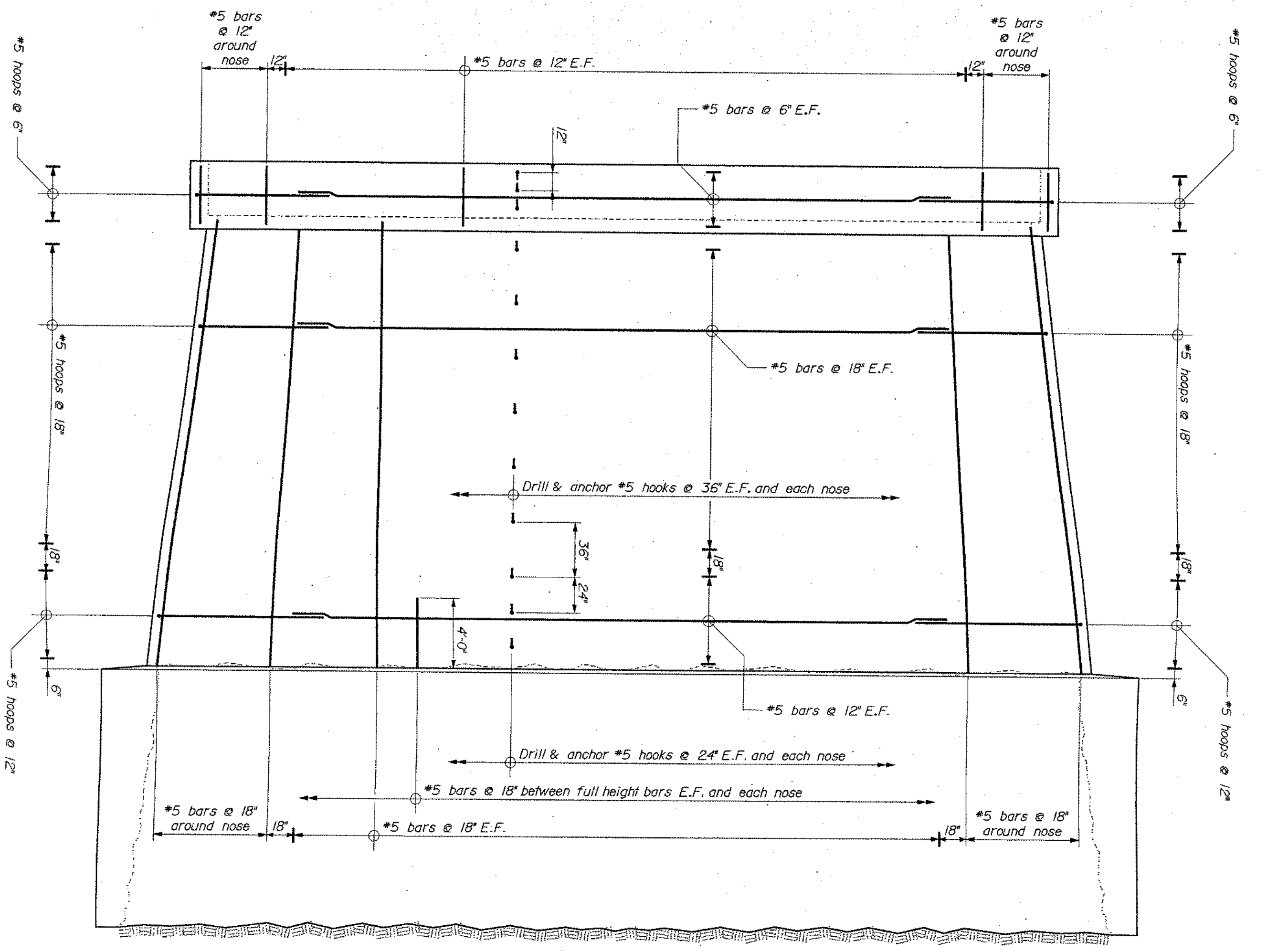
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

SHEET NUMBER
7
OF 21

Filename: ...\00\bridge\msta\008_Pier 1.dgn   Division: BRIDGE   Username: dana.damren   Date:3/29/2006



PIER NO. 1 PLAN



PIER NO. 1 ELEVATION

141-84

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

PIER NO. 1 PLAN & ELEVATION

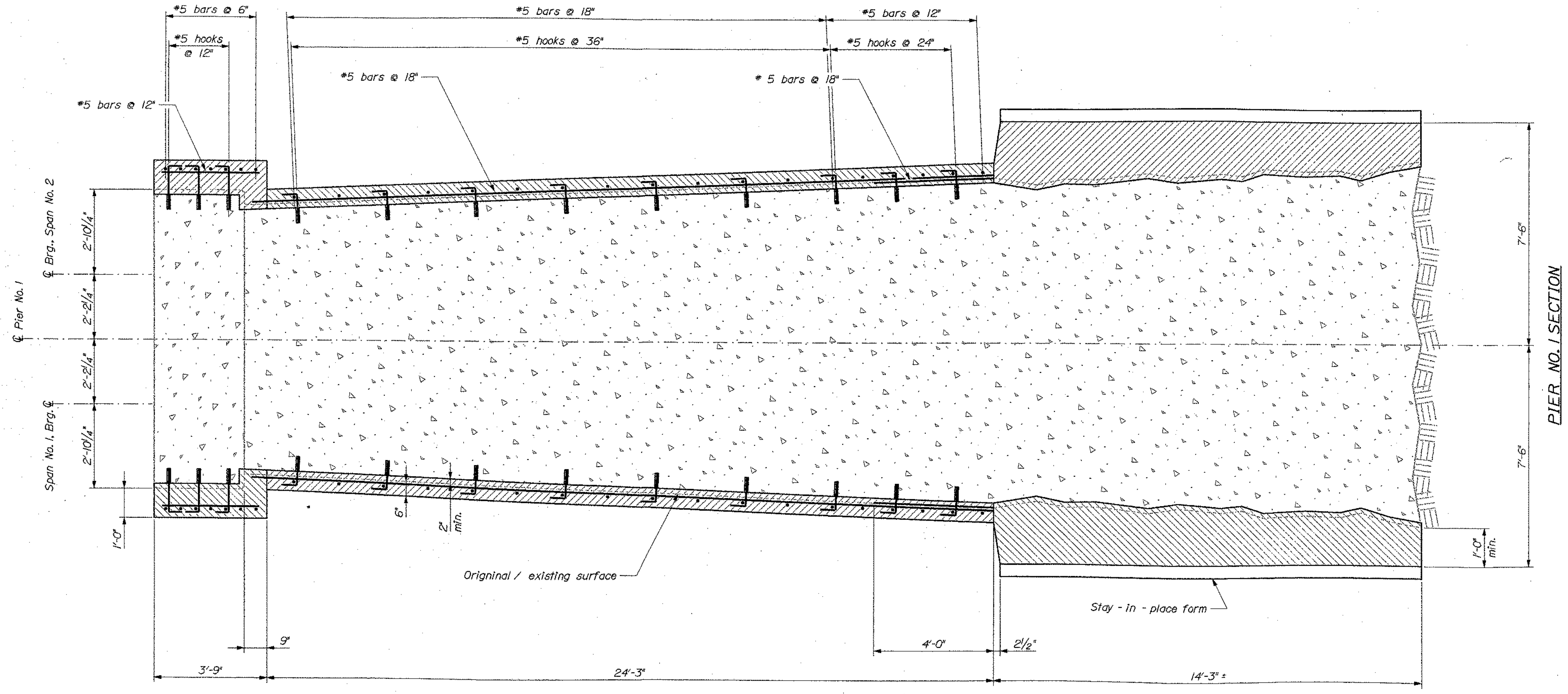
		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

SHEET NUMBER
8
OF 21

Filename: ...\00\bridge\msta\009_Pier 1.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



SYMBOLS

░ Existing concrete to remain

▨ New concrete (in section)

141-85

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016 | PIN 11089.00 | BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

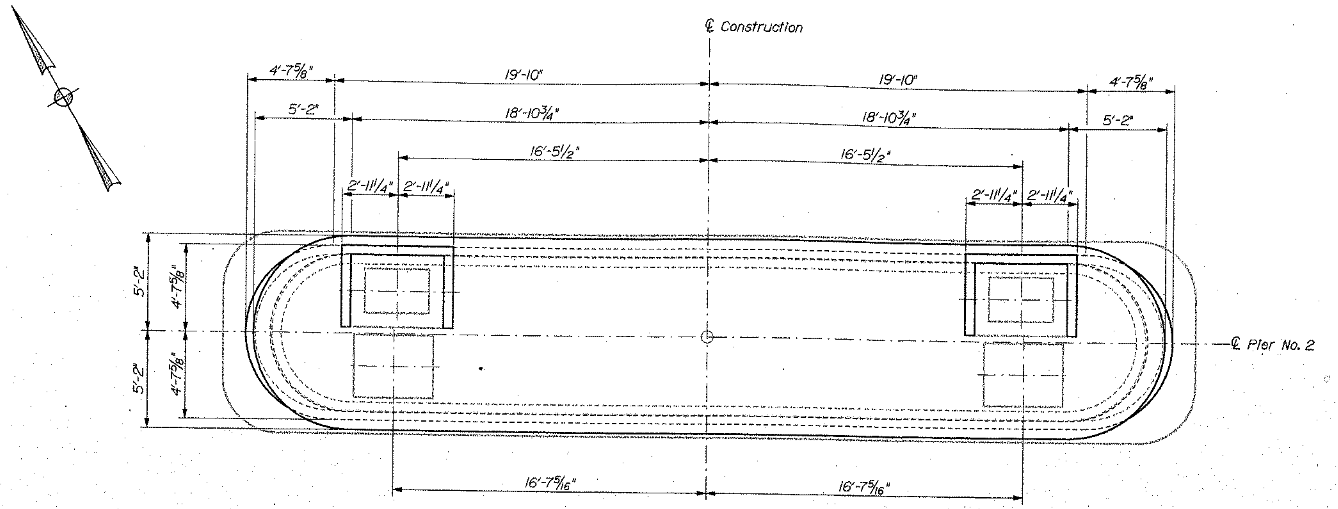
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

PIER NO. 1 SECTION

SHEET NUMBER
9
OF 21

Filename: ...\00\bridge\msta\010_Pier 2.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006

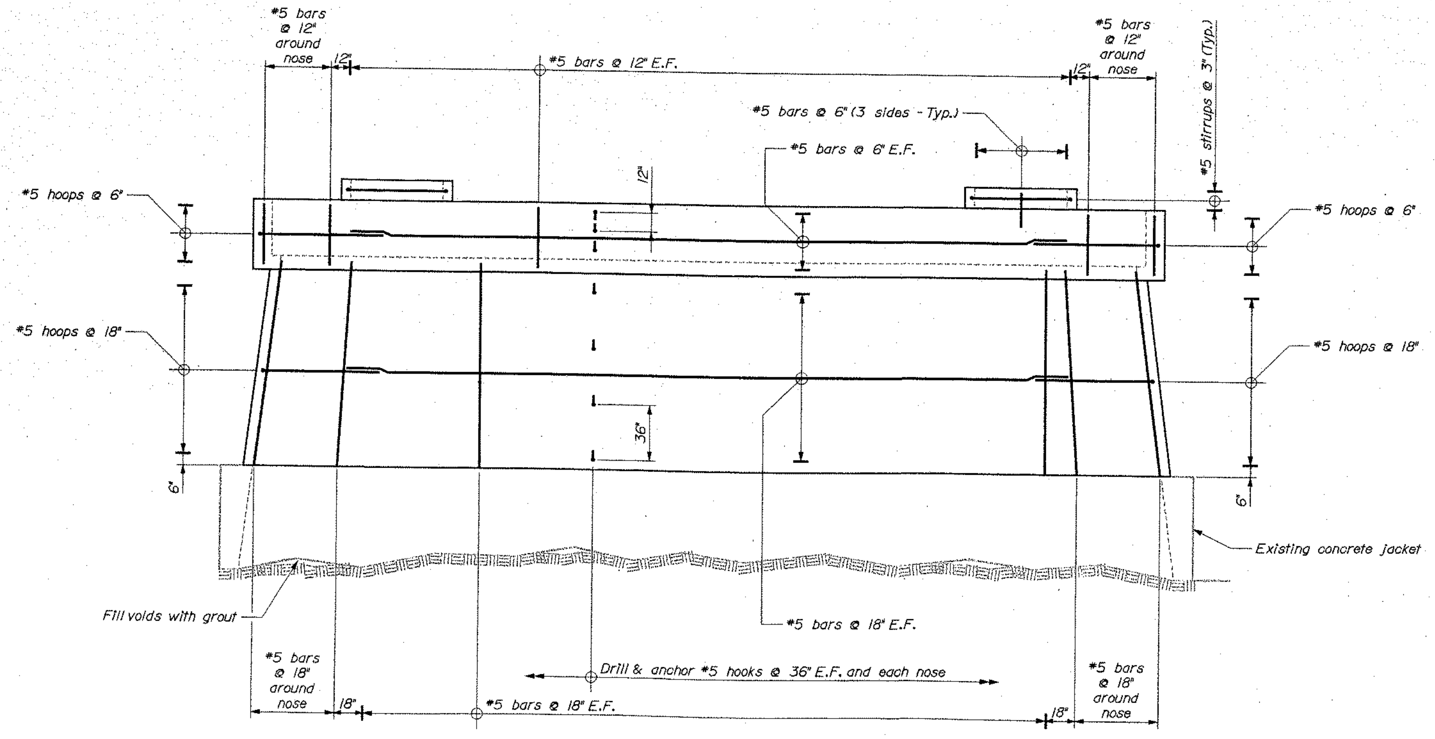


PIER NO. 2 PLAN

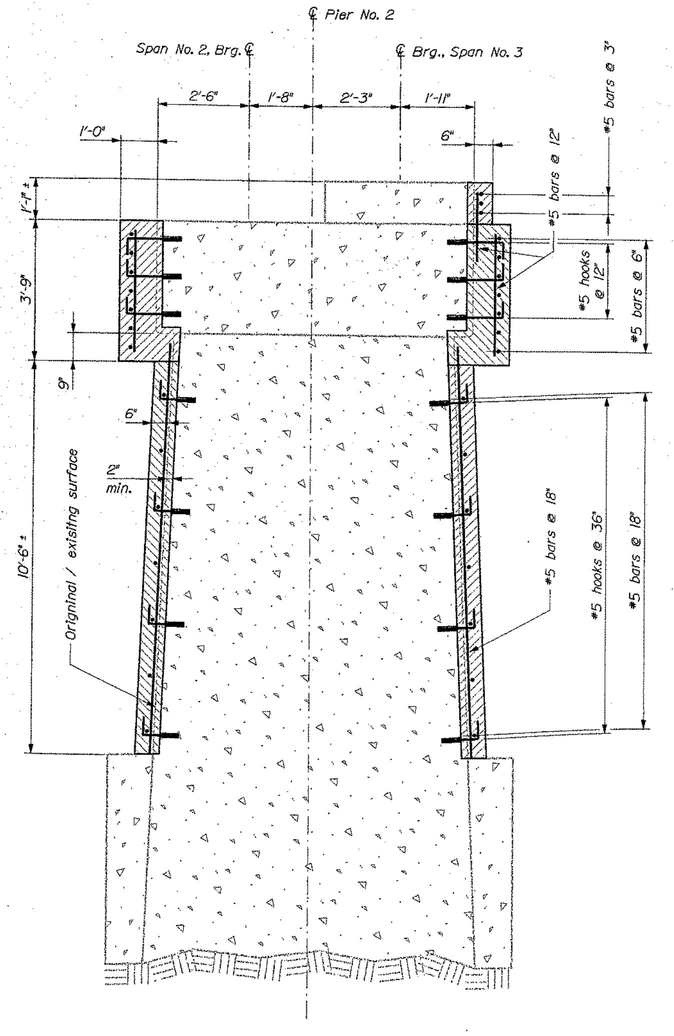
SYMBOLS

▨ Existing concrete to remain

▨ New concrete (in section)



PIER NO. 2 ELEVATION



PIER NO. 2 SECTION

141-86

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Igbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

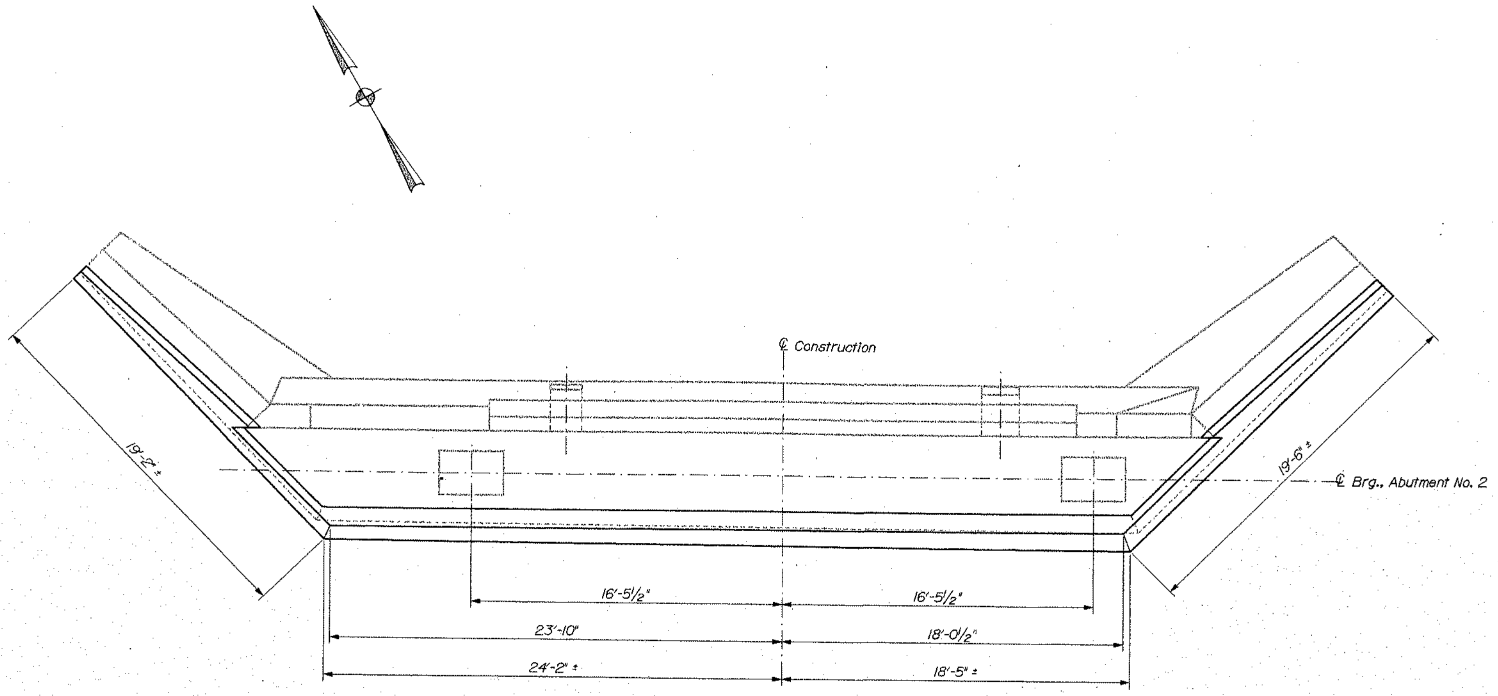
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

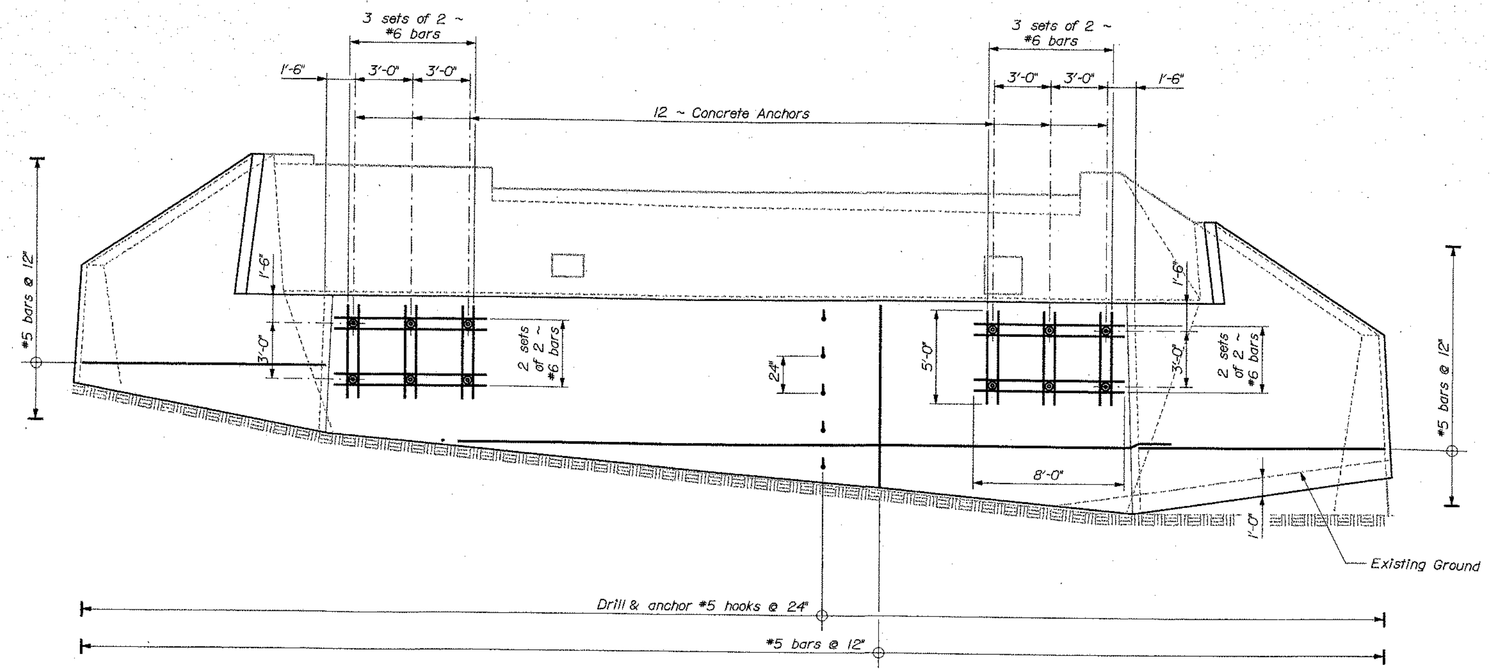
PIER NO. 2

SHEET NUMBER
10
OF 21

Filename: ...\011_Abutment_No. 2 Plan & Elev.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



ABUTMENT NO. 2 PLAN



ABUTMENT NO. 2 ELEVATION

141-87

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

SIGNATURE

P.E. NUMBER

DATE

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

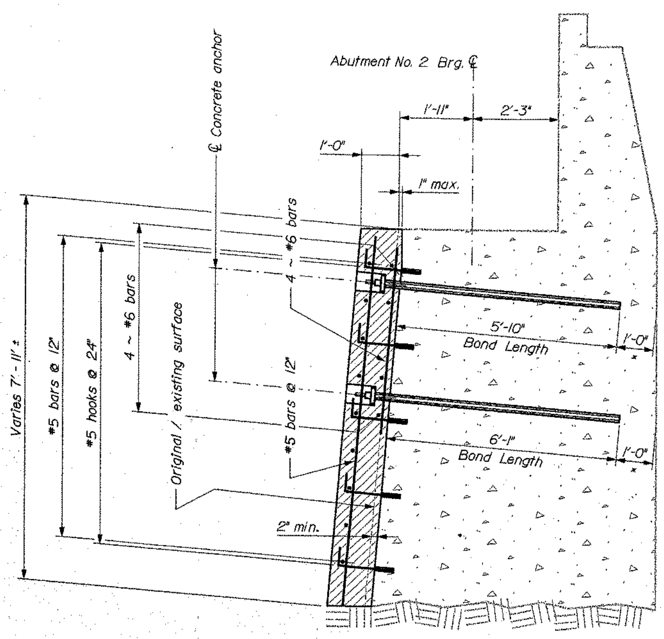
FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

ABUT. NO. 2 PLAN & ELEV.

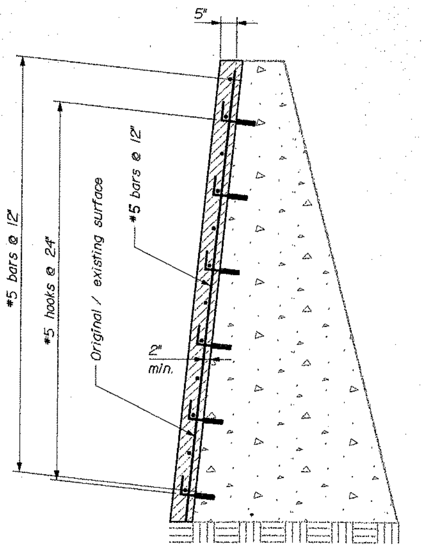
SHEET NUMBER
11
OF 21

Filename: ...\012_Abutment_No. 2 Sections.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006

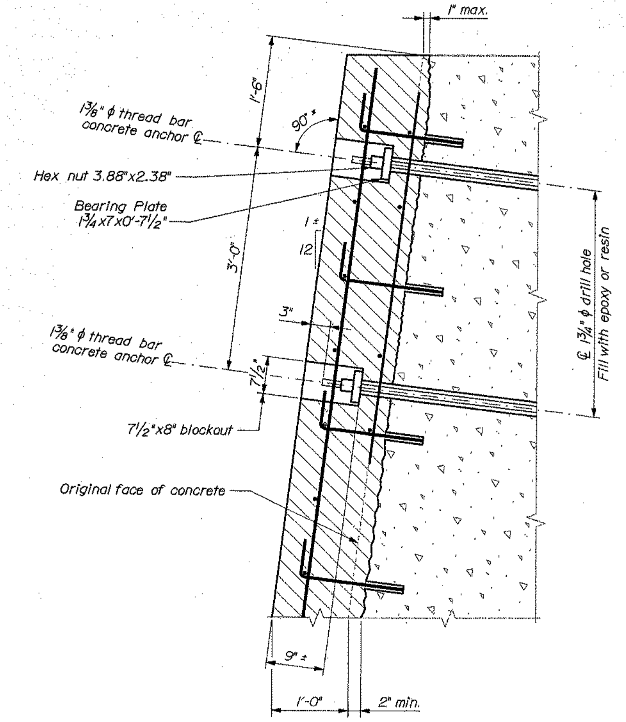
* 1'-0" clearance and bond length determined from dimensions shown on original construction drawings



BREASTWALL SECTION



WINGWALL SECTION



CONCRETE ANCHOR DETAIL

SYMBOLS

░ Existing concrete to remain

▨ New concrete (in section)

141-88

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

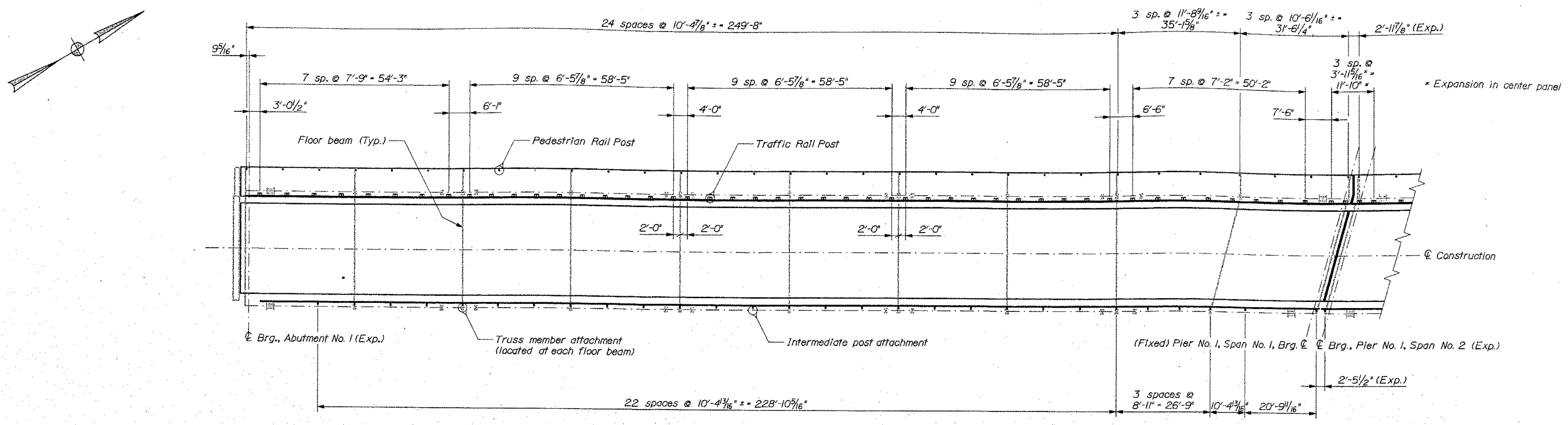
P.E. NUMBER

DATE

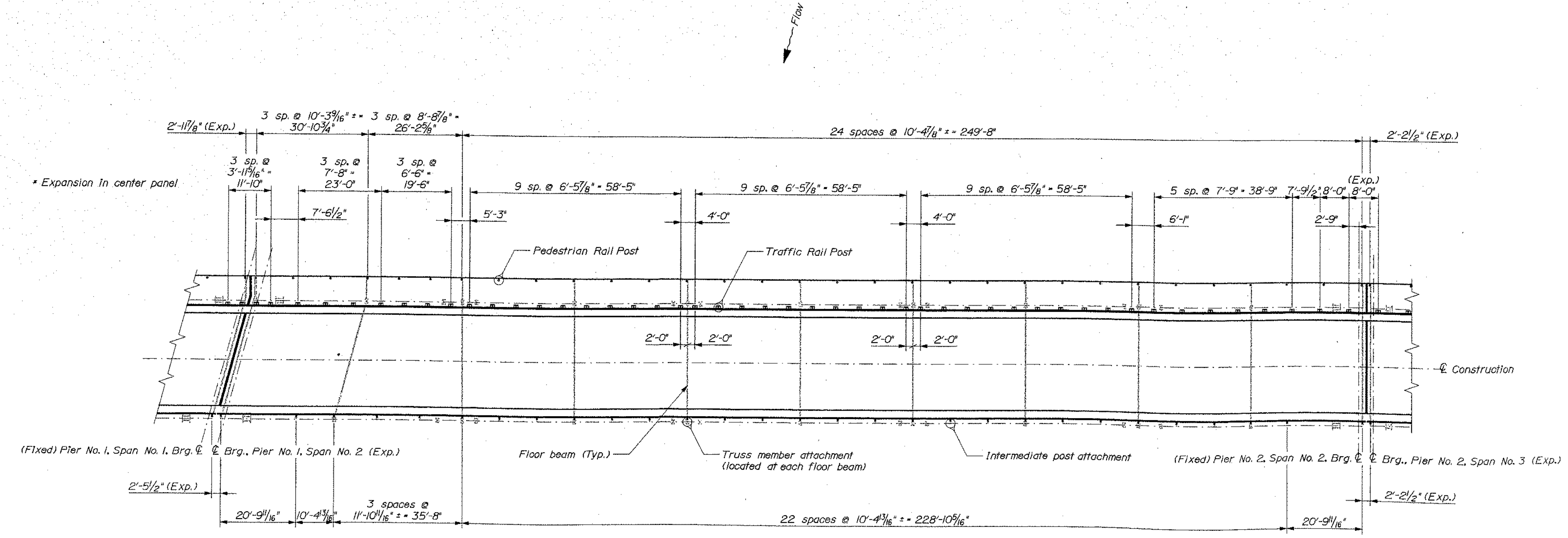
FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

ABUTMENT NO. 2 SECTIONS

SHEET NUMBER
12
OF 21



SPAN NO. 1 PLAN



SPAN NO. 2 PLAN

141-89

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

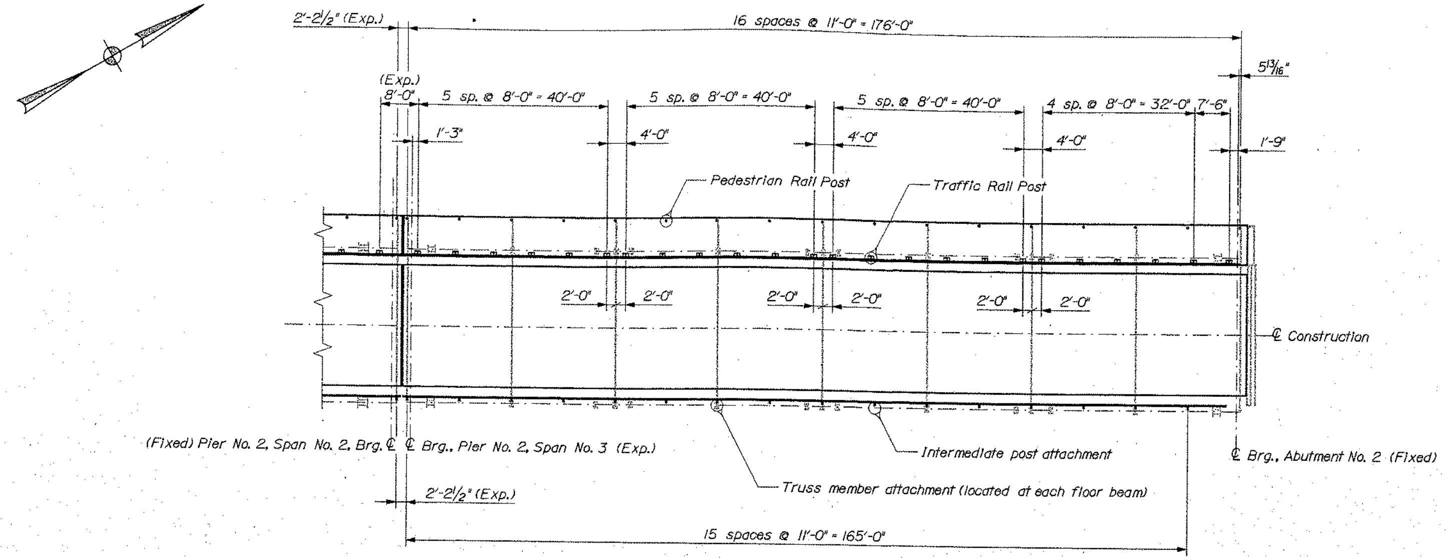
SIGNATURE
P.E. NUMBER
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

SUPERSTRUCTURE PLAN

SHEET NUMBER
13
OF 21

Filename: ...\014_Superstructure_Plan.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



SPAN NO. 3 PLAN

SUPERSTRUCTURE NOTES

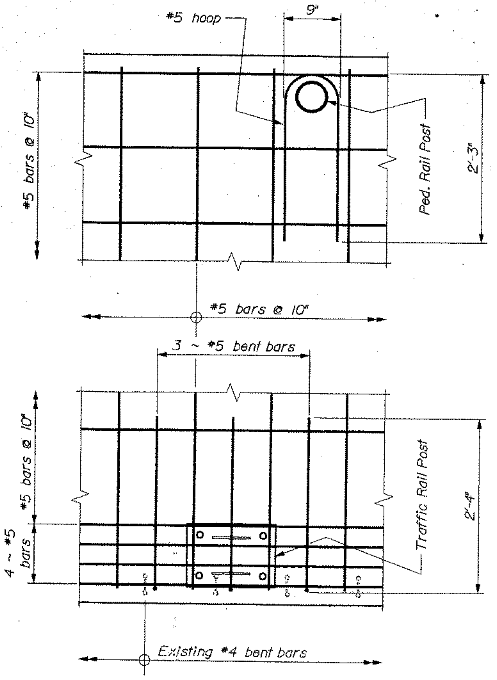
1. Reinforcing steel shall have two inches minimum cover unless otherwise noted.

2. Reinforcing steel schedules shall be provided by the Contractor in accordance with Subsection 503.03 of the Standard Specifications. Payment for reinforcing steel will be considered incidental to Item No. 502.26, Structural Concrete Roadway and Sidewalk Slabs on Steel Bridges.

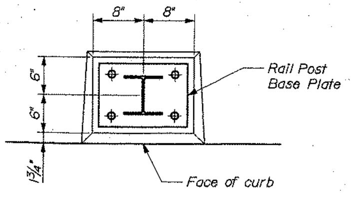
3. Form a one inch blockout in the sidewalk around all truss members. The material used for forming shall be completely removed.

4. Stay - in - place forms may be used to construct the sidewalk provided that they are galvanized and that no corrugations or irregularities are visible on the fascia. Stay - in - place forms shall be installed so as not to trap drainage around truss members.

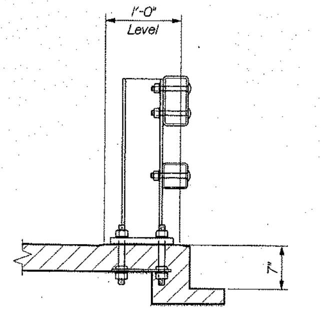
5. Payment for filling the grid deck over the piers with concrete will be made under Item No. 502.26, Structural Concrete Roadway and Sidewalk Slabs on Steel Bridges.



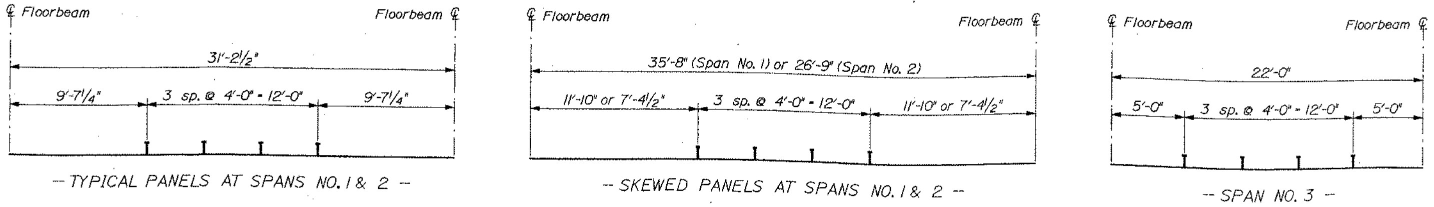
SIDEWALK REINFORCING PLAN



TRAFFIC RAIL POST LEVEL AREA



SECTION AT TRAFFIC RAIL POST



-- TYPICAL PANELS AT SPANS NO. 1 & 2 --

-- SKEWED PANELS AT SPANS NO. 1 & 2 --

-- SPAN NO. 3 --

STUD SPACING AT SIDEWALK STRINGERS

141-90

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

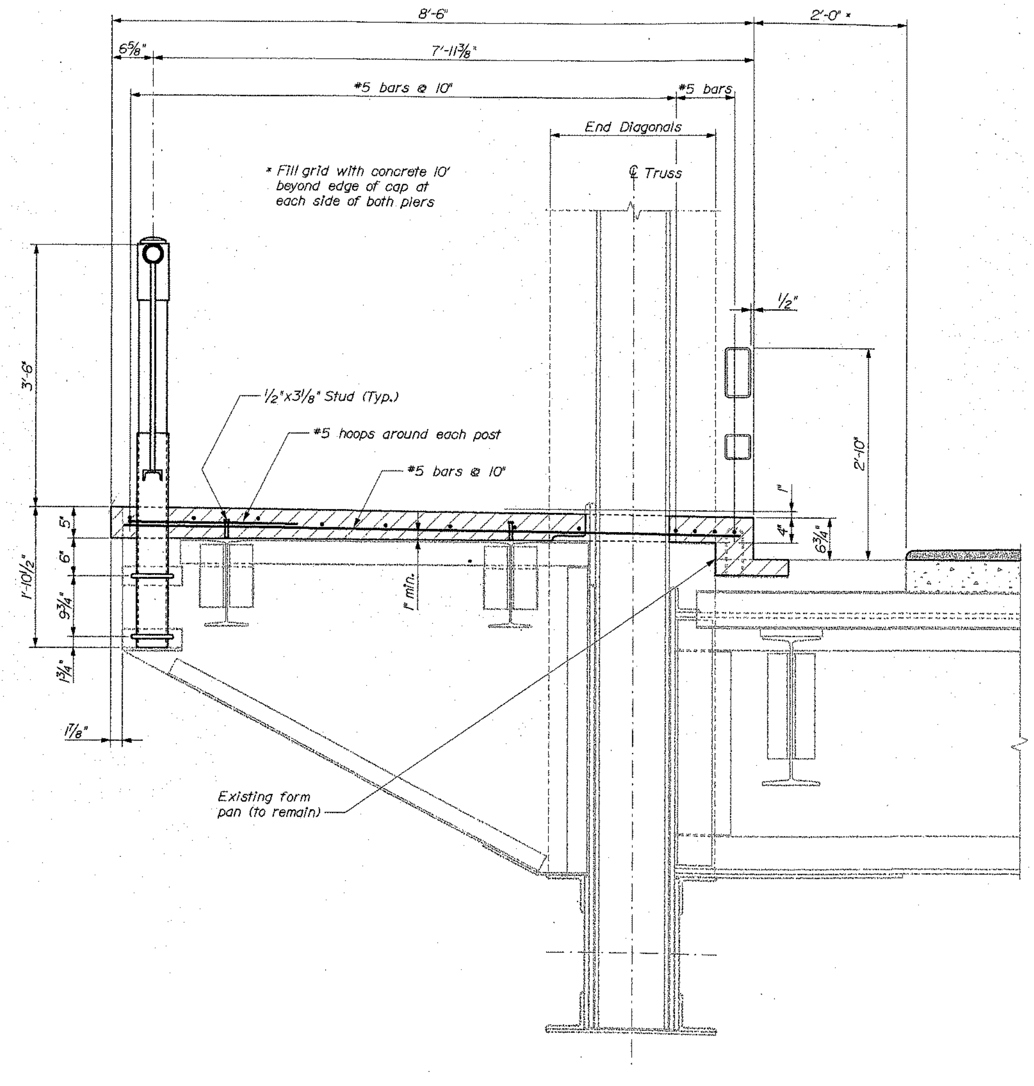
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

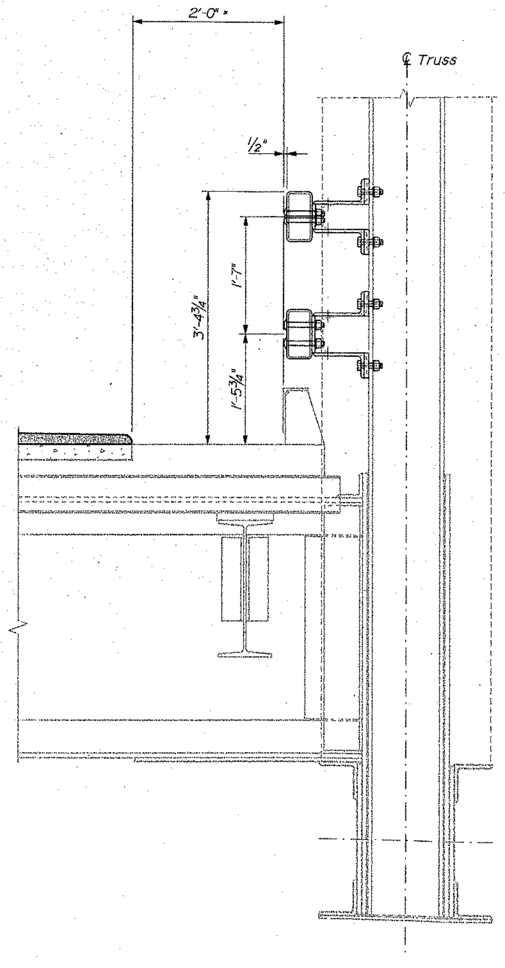
SUPERSTRUCTURE PLAN

SHEET NUMBER
14
OF 21

Filename: ...\015_Superstructure_Sections.dgn   Division: BRIDGE   Username: dana.damren   Date:3/29/2006



SIDEWALK SECTION AT TRUSS MEMBER



CURB SECTION AT TRUSS MEMBER

SYMBOLS

░	Existing concrete to remain
▨	New concrete (in section)

141-91

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016   PIN 11089.00   BRIDGE PLANS

PROJ. MANAGER	D. Anderson	
	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

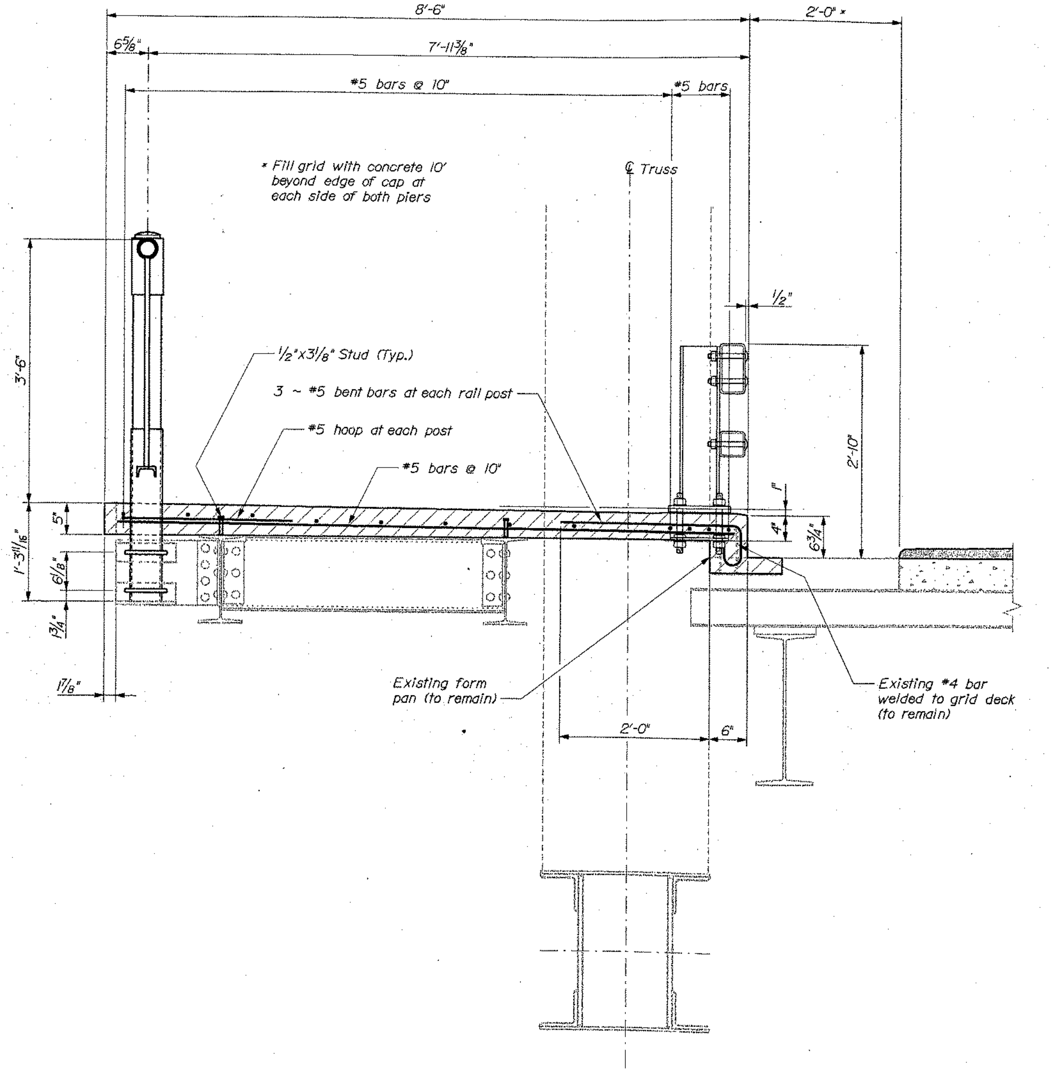
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

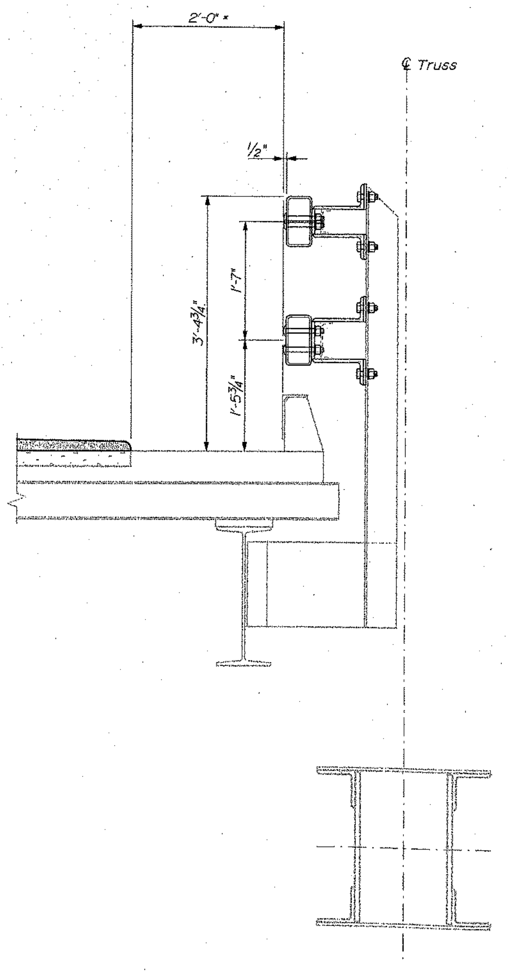
SUPERSTRUCTURE SECTIONS

SHEET NUMBER
15
OF 21

Filename: ...\016_Superstructure_Sections.dgn Division: BRIDGE Username: dana.domren Date:3/29/2006



SIDEWALK SECTION BETWEEN TRUSS MEMBERS



CURB SECTION AT INTERMEDIATE RAIL POST

SYMBOLS

- ▦ Existing concrete to remain
- ▨ New concrete (in section)

141-92

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Domren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

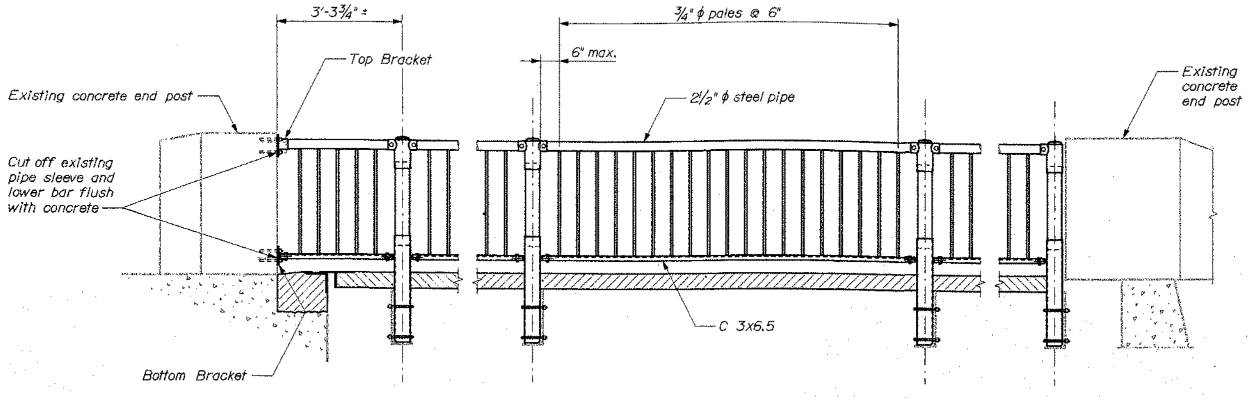
P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

SUPERSTRUCTURE SECTIONS

SHEET NUMBER
16
OF 21



PEDESTRIAN RAIL ELEVATION

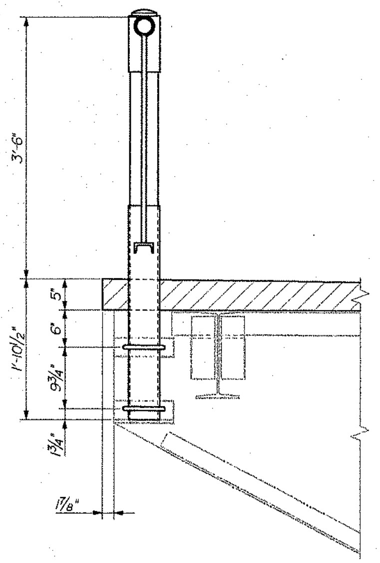
PEDESTRIAN RAIL NOTES

1. Payment for cutting the exisitng pipe and bar and for providing and installing the new brackets at the existing Abutment No. 1 concrete end post will be considered incidental to Item No. 507.0841, Steel Pipe Hand Railing.

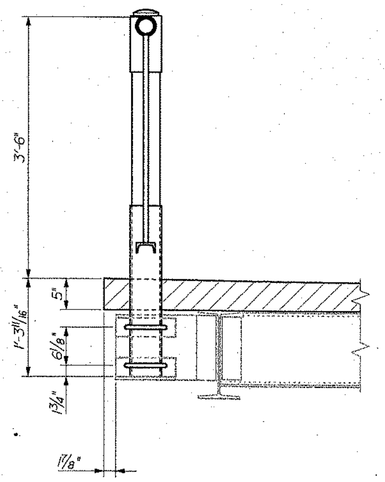
2. Dimensions shown are based on existing plans dated 1931. The Contractor shall field measure the existing Pedestrian Rail steel sections and fittings prior to ordering new materials to ensure exact match.

3. It is anticipated that due to warpage, approximately 12 existing angles at the bottom of the posts will need repalcement in-kind and this work will be considered incedental to Steel Pipe Hand Railing item.

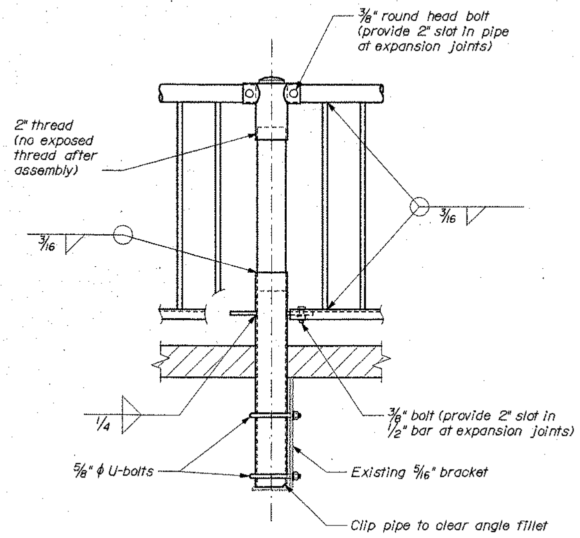
4. The Center-to-Center spacing of Pales shall not exceed 6 inches.



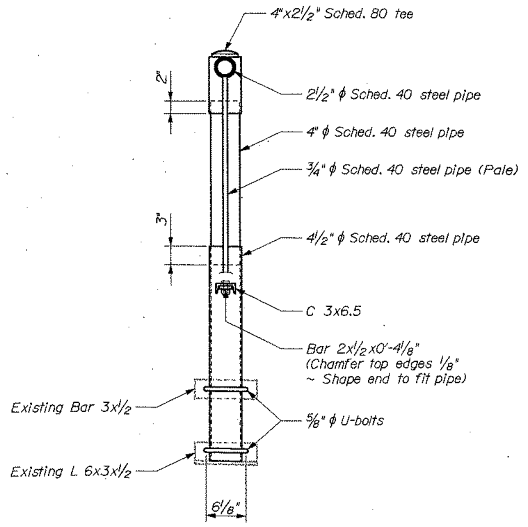
SECTION AT FLOORBEAM



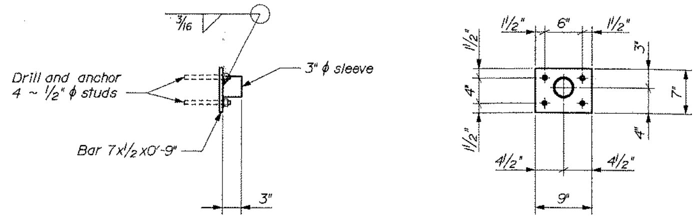
SECTION BETWEEN FLOORBEAMS



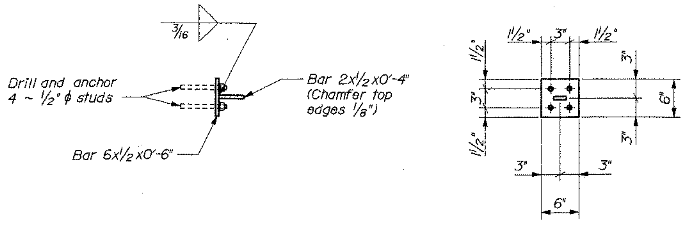
PEDESTRIAN RAIL DETAILS



PEDESTRIAN RAIL SECTION



TOP BRACKET DETAIL



BOTTOM BRACKET DETAIL

SYMBOLS

▨ Existing concrete to remain

▨ New concrete (in section)

141-93

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

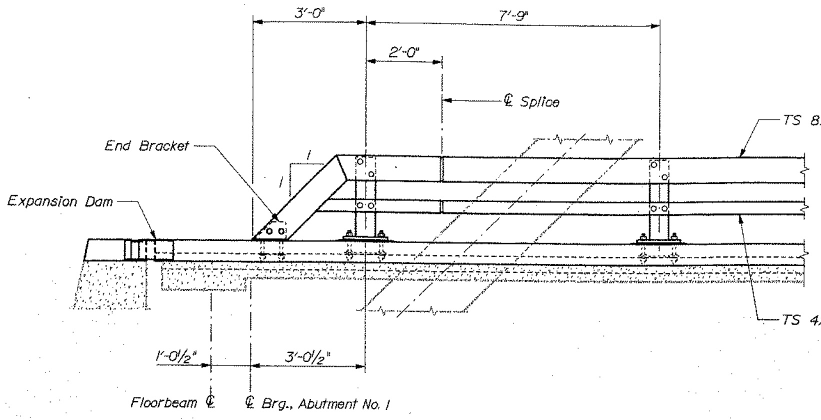
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

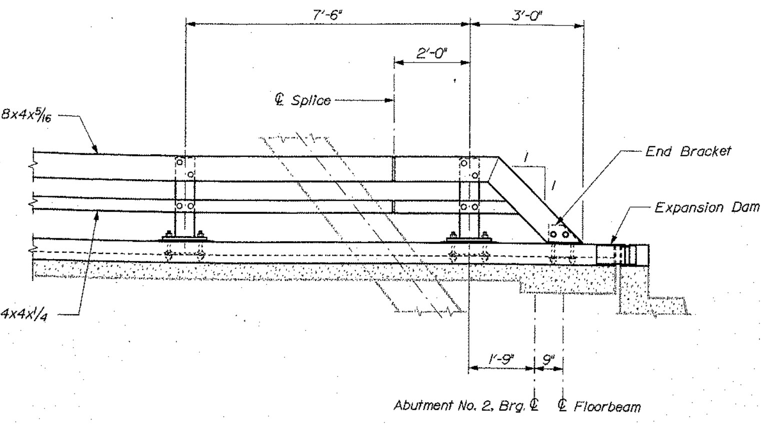
PEDESTRIAN RAIL

SHEET NUMBER
17
OF 21

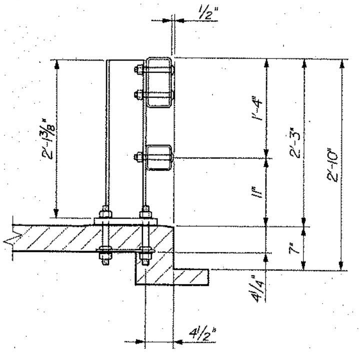
Filename: ...\msta\018_Rail_Traffic.dgn   Division: BRIDGE   Username: dana.damren   Date:3/29/2006



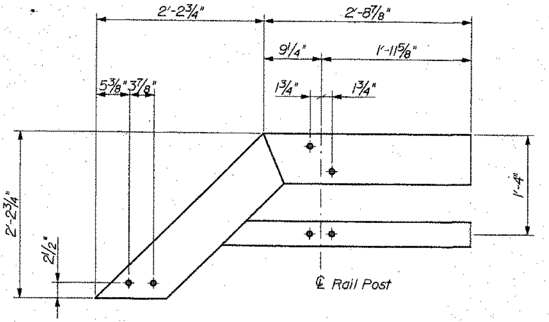
TRAFFIC RAIL ELEVATION AT ABUTMENT NO. 1



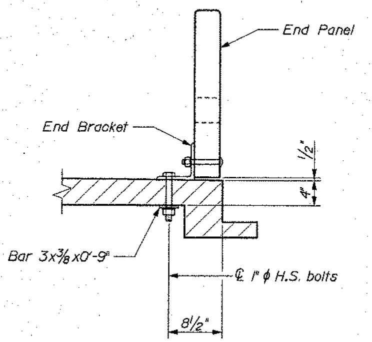
TRAFFIC RAIL ELEVATION AT ABUTMENT NO. 2



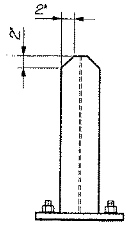
TRAFFIC RAIL SECTION



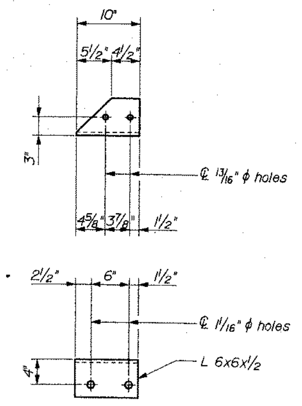
TRAFFIC RAIL END PANEL
See Std. Detail 507(25) for weld details



END PANEL SECTION



TRAFFIC RAIL POST
(Rear Elevation)



TYPICAL END BRACKET

SYMBOLS

▨ Existing concrete to remain

▨ New concrete (in section)

141-94

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
BH-1108(900)X
BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

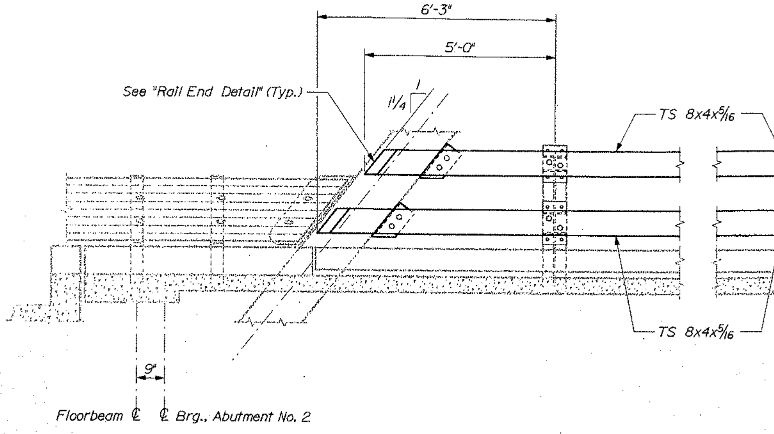
PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE
P.E. NUMBER
DATE

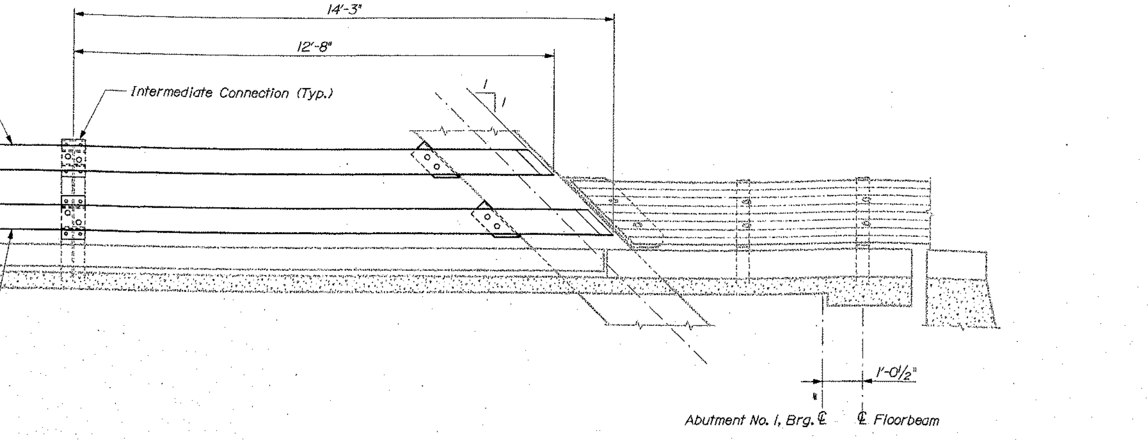
FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

TRAFFIC RAIL

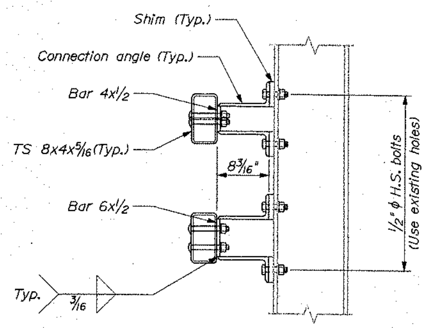
SHEET NUMBER
18
OF 21



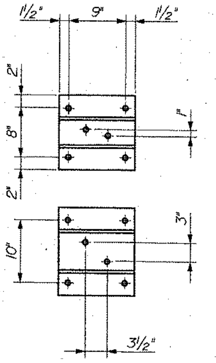
TRUSS RAIL ELEVATION AT ABUTMENT NO. 2



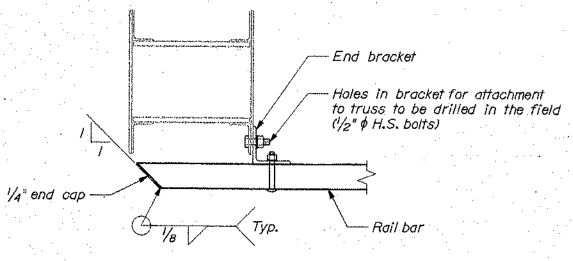
TRUSS RAIL ELEVATION AT ABUTMENT NO. 1



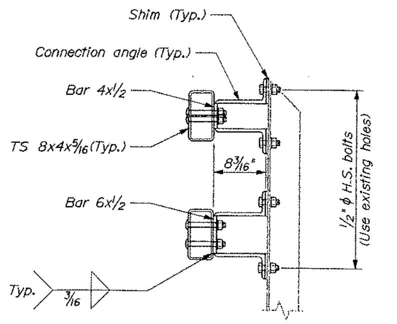
TRUSS ATTACHMENT DETAIL



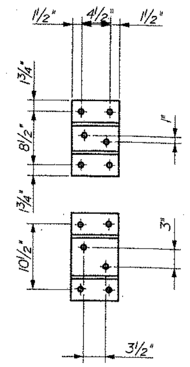
TRUSS ATTACHMENT



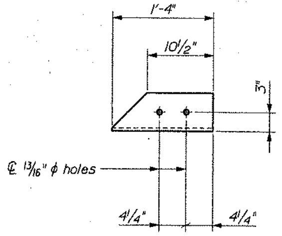
RAIL END DETAIL



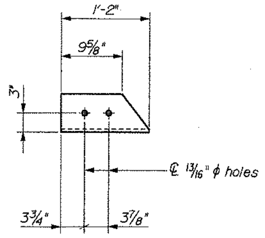
INTERMEDIATE ATTACHMENT DETAIL



INTERMEDIATE ATTACHMENT



SPAN NO. 1 END BRACKET



SPAN NO. 3 END BRACKET

TRUSS RAIL ATTACHMENT		
Location	Connection Angle	Shim
Span No. 1 & 2 Truss	L 8x4x½	Bar 4x¾
Span No. 1 & 2 Int.	L 8x4x½	Bar 4x⅜
Span No. 3 Truss	L 8x4x½	None
Span No. 3 Int.	L 7x4x½	Bar 4x⅜

SYMBOLS

░ Existing concrete to remain

▨ New concrete (in section)

141-95

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN
11089.00
BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	D. Anderson		
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

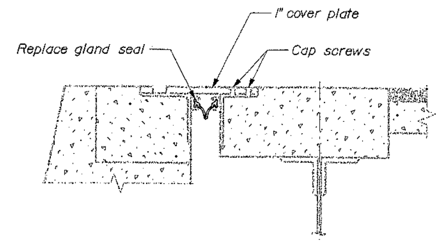
FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

TRUSS RAIL

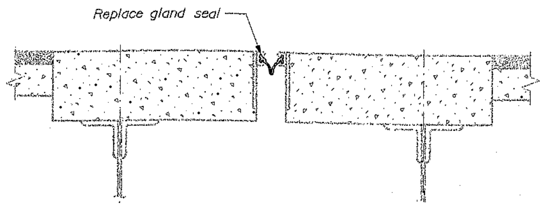
SHEET NUMBER
19
OF 21

Filename: ...\bridge\msta\019_Rail_Truss.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006

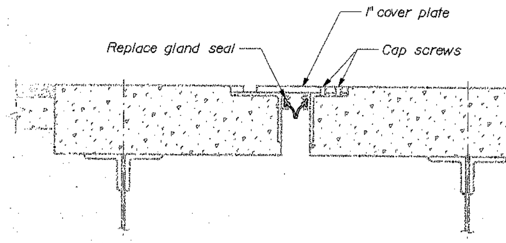
Filename: ...\msta\020_Expansion Joints.dgn Division: BRIDGE Username: dana.damren Date:3/29/2006



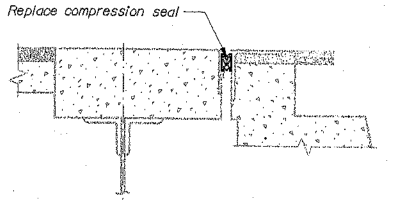
ROADWAY EXPANSION JOINT
(Abutment No. 1)



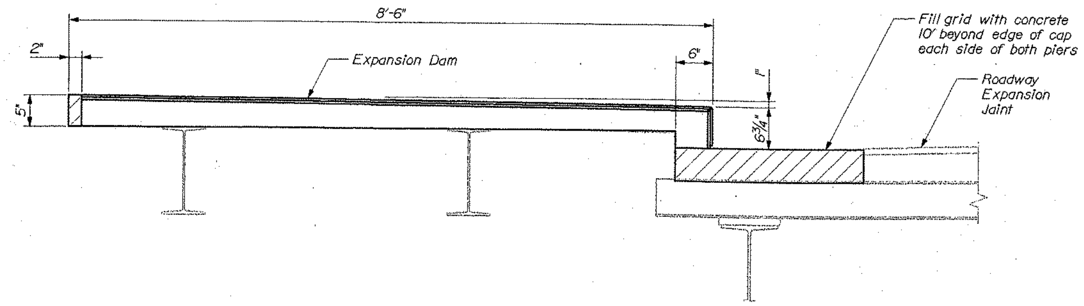
ROADWAY EXPANSION JOINT
(Pier No. 2)



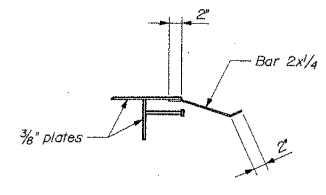
ROADWAY EXPANSION JOINT
(Pier No. 1)



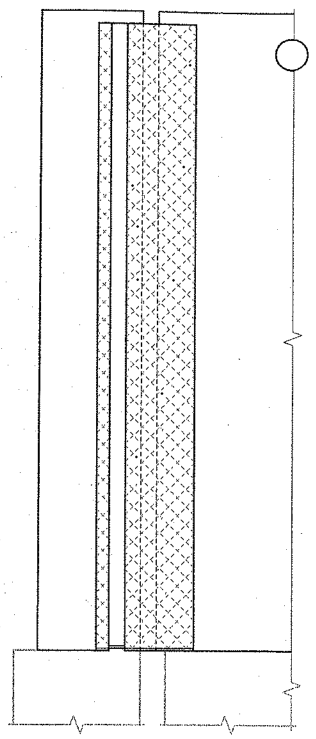
ROADWAY EXPANSION JOINT
(Abutment No. 2)



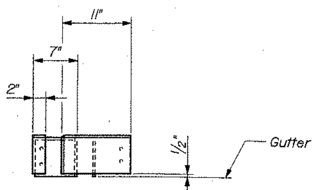
TRANSVERSE SECTION AT EXPANSION JOINT



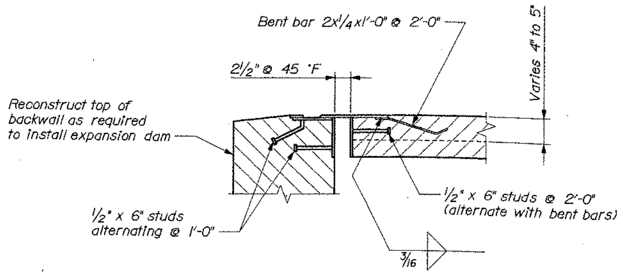
ANCHOR BAR DETAIL



-- PLAN --



-- ELEVATION --



-- SECTION --

ABUTMENT NO. 1 EXPANSION DAM

EXPANSION JOINT NOTES

1. Refer to Standard Details Section 520 for welding details and other expansion dam information not shown.

2. Payment for the new expansion dams and seal replacement at Abutment No. 1 and Pier No. 1 will be made under Item No. 520.2401, Bridge Joint Modification Type 1.

3. Payment for the new expansion dam and seal replacement at Pier No. 2 will be made under Item No. 520.2402, Bridge Joint Modification Type 2.

3. Payment for the new expansion dam and seal.replacement at Abutment No. 2 will be made under Item No. 520.2403, Bridge Joint Modification Type 3.

4. Payment for removal and replacement of abutment backwall concrete required to install the new sidewalk expansion dams will be considered incidental to Item No.'s 202.128, Removing Concrete Curbs and Sidewalks and 502.26, Structural Concrete Roadway and Sidewalk Slabs on Steel Bridges, respectively.

5. Replacement seals shall be furnished as follows:

Abument No. 1: Wabo Stripseal SE-400
Pier No. 1: Wabo Stripseal SE-400
Pier No. 2: Wabo Stripseal SE-400
Abutment No. 2: Wabo Compression Seal WA-250

The installation width for the WA-250 seal is 1½". The existing joint opening at Abutment No. 2 shall be verified prior to ordering the seal.

SYMBOLS

⁘	Existing concrete to remain
▨	New concrete (in section)

141-96

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

PIN
11089.00

BRIDGE NO. 2016

BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

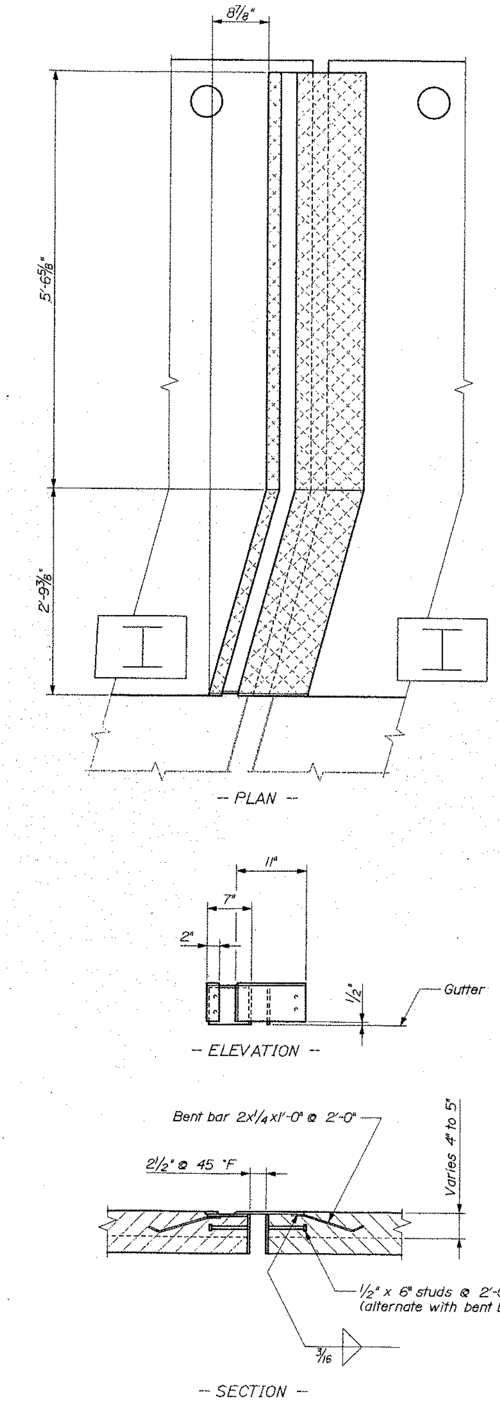
DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

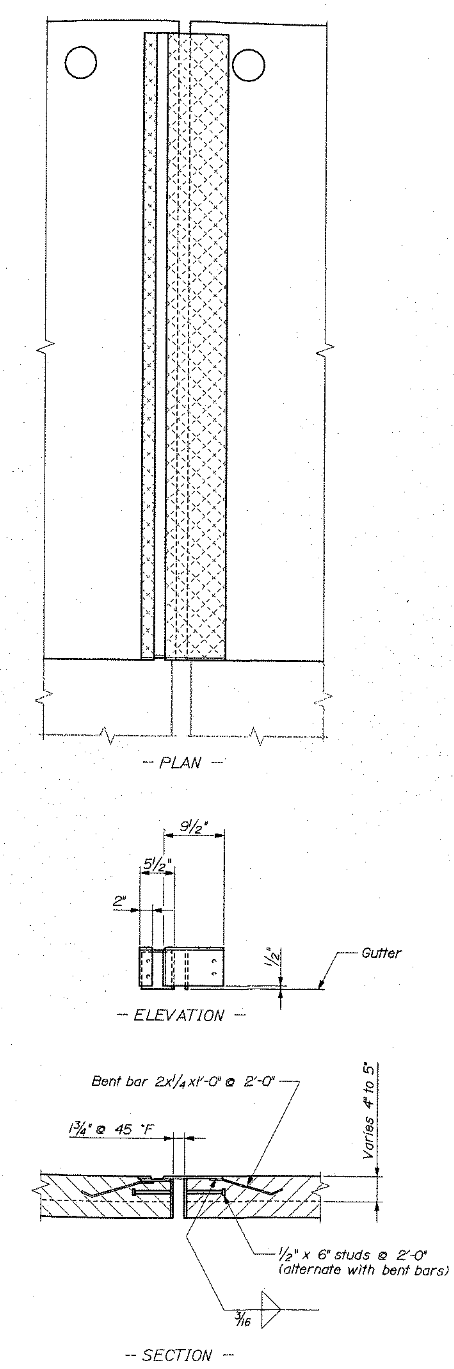
EXPANSION JOINTS

SHEET NUMBER
20
OF 21

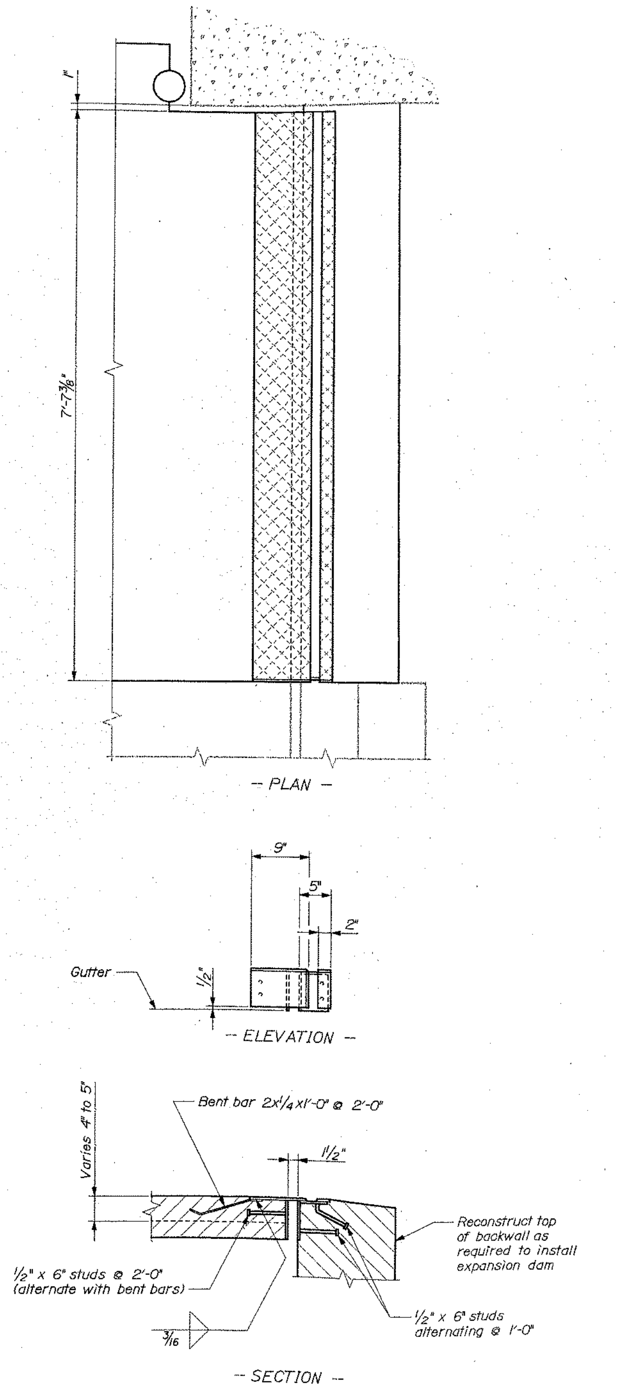
Filename: ...\msta\021_Expansion Joints.dgn   Division: BRIDGE   Username: dana.damren   Date:3/29/2006



PIER NO. 1 EXPANSION DAM



PIER NO. 2 EXPANSION DAM



ABUTMENT NO. 2 EXPANSION DAM

SYMBOLS

░	Existing concrete to remain
▨	New concrete (in section)

141-97

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1108(900)X

BRIDGE NO. 2016
PIN 11089.00
BRIDGE PLANS

PROJ. MANAGER	D. Anderson	BY	DATE
DESIGN-DETAILED	M. A. Iqbal	D. Damren	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FRANK J. WOOD BRIDGE
ANDROSCOGGIN RIVER
BRUNSWICK & TOPSHAM

EXPANSION JOINTS

SHEET NUMBER
21
OF 21

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	F-017-1(53)	1	7

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

PLANS

BUREAU OF HIGHWAYS
REHABILITATION
OF THE BRIDGE DECK
ON THE
ANDROSCOGGIN RIVER BRIDGE
OVER
ANDROSCOGGIN RIVER
BETWEEN THE TOWNS OF
BRUNSWICK-CUMBERLAND COUNTY
TOPSHAM - SAGADAHOC COUNTY
PROJECT NO. F-017-1(53)
PROJECT LENGTH 0.162 MILES

PROJECT COMPLETED
1986

CONVENTIONAL SIGNS

COUNTY LINES
TOWN LINES
PROPERTY LINES
R/W LINES - EXISTING
R/W LINES - NEW - ACCESS CONTROL
R/W LINES - NEW - NO ACCESS CONTROL
CULVERT - EXISTING
CULVERT - PROPOSED
CURBING - EXISTING
CURBING - PROPOSED
TRAVELLED WAY - EXISTING
TRAVELLED WAY - PROPOSED
UNDERGROUND UTILITIES - EXISTING
UNDERGROUND UTILITIES - PROPOSED
RAILROAD - SINGLE TRACK
RAILROAD - DOUBLE TRACK
UTILITY POLE - EXISTING
UTILITY POLE - JOINT OCCUPANCY J
PROPOSED UTILITY POLE - TEMPORARY x
PROPOSED UTILITY POLE - PERMANENT
TREES hardwood softwood
WOODS

INDEX OF SHEETS

DESCRIPTION	SHEET NO.
TITLE SHEET	1
FLOOR MODIFICATION	2 & 3
BEARING DETAILS	4
MAINTENANCE OF TRAFFIC	5 - 7

ESTIMATED QUANTITIES

ITEM No.	DESCRIPTION	QUANT.	UNIT
403.08	Hot Bituminous Pavement, Grading C	270	Tons
410.15	Emulsified Asphalt, Applied	250	Gal
502.39	Concrete Fill for Steel Grid	369	C.Y.
506.142	Field Painting Existing Structural Steel	1	L.S.
506.16	Surface Preparation of Existing Structural Steel	1000	M.H.
514.06	Curing Box for Concrete Cylinders	1	Each
520.25	Bridge Joint Modifications	1	LS
523.25	Expansion Bearing Modifications	1	LS
526.301	Temporary Concrete Barrier, Type I	1	LS
527.31	Energy Absorbing Unit - Temporary	4	Each
627.61	4 Inch Solid White Pavement Marking Line	1650	LF
627.63	4 Inch Solid Yellow Pavement Marking Line	1650	LF
627.68	Temp 4 Inch Painted Pavement Marking Line, Yellow or White	5000	LF
629.05	Hand Labor, Straight Time	40	MH
631.10	Air Compressor (Including Operator)	210	HR
631.11	Air Tool (Including Operator)	10	HR
631.30	Cutting Torch (Including Operator)	20	MH
631.33	Welding Machine (Including Operator)	200	MH
631.34	Grinder (Including Operator)	50	MH
631.35	Sand Blaster (Including Operator)	200	MH
631.19	Field Office Type B	1	Each
652.31	Type I Barricade	25	Each
652.33	Drum	35	Each
652.34	Cone	30	Each
652.35	Construction Signs	250	S.F.
652.361	Maintenance of Traffic Control Devices	1	L.S.
652.37	Warning Lights	2	GRP.
652.38	Flagger	700	M.H.
659.10	Mobilization	1	L.S.

SPECIFICATIONS

DESIGN: LOAD FACTOR DESIGN PER AASHTO STANDARD SPECIFICATIONS FOR HIGHWAYS AND BRIDGES 1983

CONTRACT: STATE OF MAINE, DEPARTMENT OF TRANSPORTATION STANDARD SPECIFICATIONS, REVISION OF JANUARY 1984.

MATERIALS

STRUCTURAL STEEL: .ASTM A36
CONCRETE: . . . . .CLASS AA(H.E.)

BASIC ALLOWABLE STRESS

STRUCTURAL STEEL. F_y 36,000psi
CONCRETE . . . . $f'c$ 4000psi

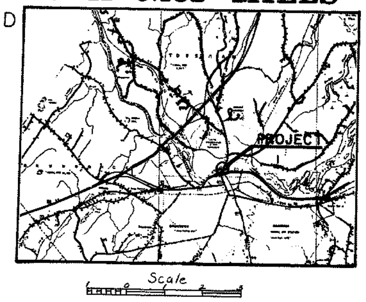
DESIGN LOADING

LIVE LOAD . . . . .HS 20

Plans of the existing bridge are available for the Contractor's reference at the Bridge Design Office in Augusta. The plans are reproductions of original drawings as prepared for the construction of the bridge and it is very unlikely that the plans will show any construction field changes or any alterations which may have been made to the bridge during its life span.

TRAFFIC DATA

A.D.T. 1984 = 23450
A.D.T. 2004 = 32980
D.H.V. 3298
T. (%) 7
D. (%) 54
V.
P.S.D. (%)
18 KIPS P2.5 422



LOCATION MAP

NOTE

All work contemplated under this contract to be governed by and in conformity with the STANDARD SPECIFICATIONS (revision of January 1984) and supplementals thereto, except as modified on the plans and in the special provisions.

RICHARD A. COLEMAN

APPROVED:

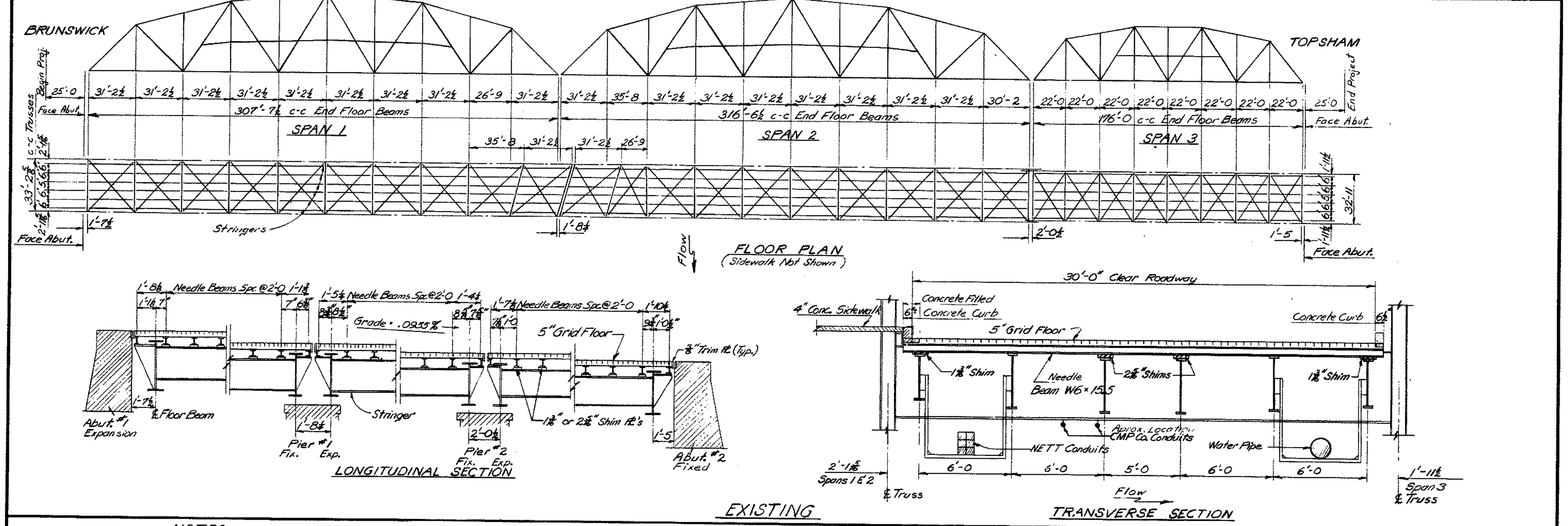
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

	DATE
COMMISSIONER	5-22-85
Richard A Coleman CHIEF ENGINEER	5-22-85

MEB
REVISED AS BUILTS 3-1-90

UNITED STATES
DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
REGION 1
APPROVED: 99-394
DIVISION ADMINISTRATOR DATE

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	F-017-1(53)	2	7



FLOOR PLAN
(Sidewalk Not Shown)

LONGITUDINAL SECTION

EXISTING

TRANSVERSE SECTION

NOTES

1. — Two lanes of traffic shall be maintained on a minimum width of 26' between 6 a.m. and 6 p.m. except on weekends when the minimum width shall be 20' between 6 a.m. and 11 a.m. if necessary for curing concrete. Two lanes of traffic shall be maintained on a minimum width of 20' between 6 p.m. and 6 a.m., except one lane will be allowed on a minimum width of 12' between 6 p.m. and 6 a.m. while placing concrete or the bituminous wearing surface, or installing the bridge joints.

2. — The field painting of existing Structural Steel, Item 506.142 will include the painting of all existing Structural Steel below the top of the existing steel grid except for the utility support hungers

3. — Payment for painting of new structural Steel will be incidental to Item 506.142

4. — Any temporary ramps required for the maintenance of traffic, including ramps for Joint modifications, will be as approved by the Engineer, and incidental to Contract Items.

5. - All utility facilities shall be adjusted by the respective utility.

6. - Portions of the existing grid floor will be repaired prior to the placing of concrete as directed by the Engineer. Payment for all work and materials will be paid for under the applicable labor and equipment rental pay items.

7. - The surface preparation of the existing Structural Steel, Item 506.16 will include the preparation of all existing structural steel below the top of the existing steel grid except for the utility support hangers.

8. - The steel grid shall be cleaned by sand blasting within 48 hours prior to the placement of concrete or as directed by the Engineer. Payment for cleaning will be paid for under Item 631.35.

9. - Remove existing concrete from the steel grid over the abutments. Payment for the removal of existing concrete, including the removal of concrete for the bridge joint modifications at the abutments, will be considered incidental to related contract items.

10. - Shim 25 feet of approaches at each end of the bridge to transition the existing roadway to the new bridge template as directed by the Engineer. Payment for all work and materials will be paid for under Items 403.08 and 410.15.

11. - Both expansion bearings of a span must be in their final position before the expansion joints are installed.

12. - An alternate construction sequence may be used if previously approved by the Engineer.

13. - Dimensions shown shall be verified in the field prior to any fabrication of structural steel.

CONSTRUCTION SEQUENCE

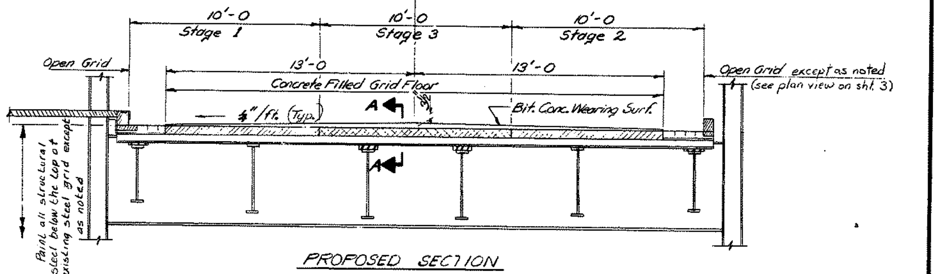
STAGE 1 — Close a 10 foot lane (6 p.m. to 6 a.m.)
Modify the expansion bearings
Modify the bridge joints
Repairing and cleaning the grid floor, erecting forms and placing concrete can be done before, after or concurrently with the bearing and joint work.

STAGE 2 — Close the second 10 foot lane and proceed as in Stage 1.

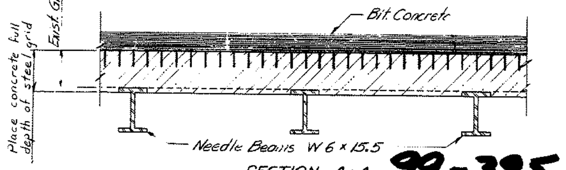
STAGE 3 — Close the remaining 10 foot lane and proceed as in Stage 1.

STAGE 4 — Install gland seals and the compression seal, and pave the 26 foot roadway.

Utilities: Central Maine Power Co,
New England Telephone
Brunswick and Topsham Water Co.
Casco Cable T.V.



PROPOSED SECTION



SECTION A-A

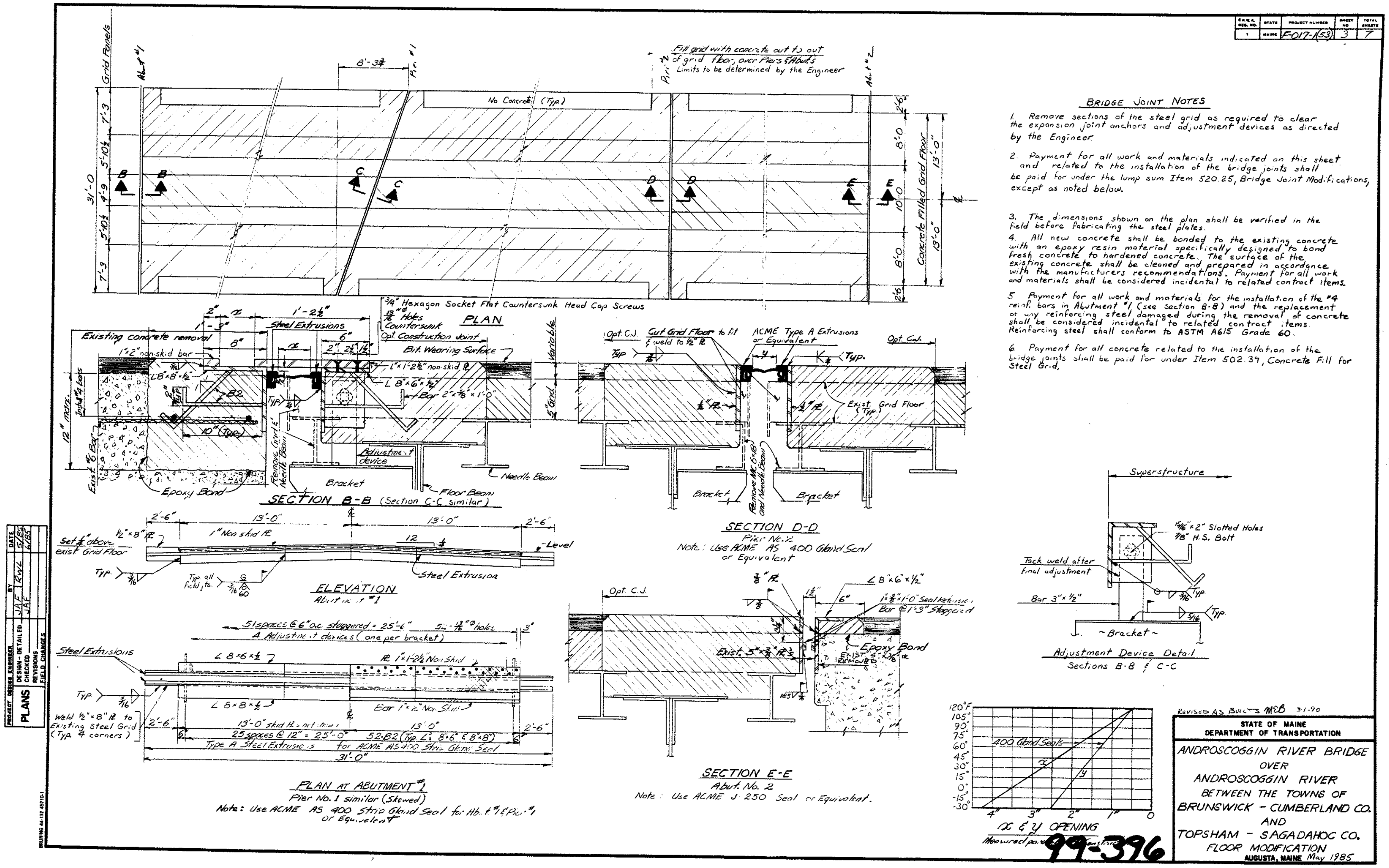
99-395

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

ANDROSCOGGIN RIVER BRIDGE
OVER
ANDROSCOGGIN RIVER
BETWEEN THE TOWNS OF
BRUNSWICK - CUMBERLAND CO.
AND
TOPSHAM - SAGADAHOC CO.
FLOOR MODIFICATION
AUGUSTA, MAINE May 1985

PLANS		BY	DATE
	PROJECT DESIGN ENGINEER		
	DESIGN - DETAILED	JAF	RWL 5/85
	CHECKED	DMD	
	REVISIONS		
	FIELD CHANGES		

BRUNING 44-132 45710-1



F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	F-017-1(53)	3	7

BRIDGE JOINT NOTES

1. Remove sections of the steel grid as required to clear the expansion joint anchors and adjustment devices as directed by the Engineer.

2. Payment for all work and materials indicated on this sheet and related to the installation of the bridge joints shall be paid for under the lump sum Item 520.25, Bridge Joint Modifications, except as noted below.

3. The dimensions shown on the plan shall be verified in the field before fabricating the steel plates.

4. All new concrete shall be bonded to the existing concrete with an epoxy resin material specifically designed to bond fresh concrete to hardened concrete. The surface of the existing concrete shall be cleaned and prepared in accordance with the manufacturers recommendations. Payment for all work and materials shall be considered incidental to related contract items.

5. Payment for all work and materials for the installation of the #4 reinf. bars in Abutment #1 (see Section B-B) and the replacement of any reinforcing steel damaged during the removal of concrete shall be considered incidental to related contract items. Reinforcing steel shall conform to ASTM A615 Grade 60.

6. Payment for all concrete related to the installation of the bridge joints shall be paid for under Item 502.39, Concrete Fill for Steel Grid.

REVISED AS BUILTS MEB 3-1-90

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

ANDROSCOGGIN RIVER BRIDGE
OVER
ANDROSCOGGIN RIVER
BETWEEN THE TOWNS OF
BRUNSWICK - CUMBERLAND CO.
AND
TOPSHAM - SAGADAHOC CO.
FLOOR MODIFICATION
AUGUSTA, MAINE May 1985

99-396

[redacted]

F.H.W.A. REG. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	F-017-1(53)	4	7

BEARING NOTES

1. A jacking proposal shall be submitted to the Engineer for approval before any jacking is done. A temporary stiffening system shall be designed to adequately distribute the stresses introduced into the truss members due to jacking. Jacking shall be limited to a maximum of ½" unless otherwise directed by the Engineer.

2. Remove rollers and the lower bearing castings and clean castings by sandblasting. Machine the top bearing surface of the lower bearing castings and shim the upper bearing castings to assure the proper alignment of the new plates with the castings as directed by the Engineer. At the contractors option, the lower bearing castings may be replaced with new steel plates or reinforced concrete pedestals. The method of replacing the lower bearing castings shall be submitted to the Engineer for approval before any jacking is done.

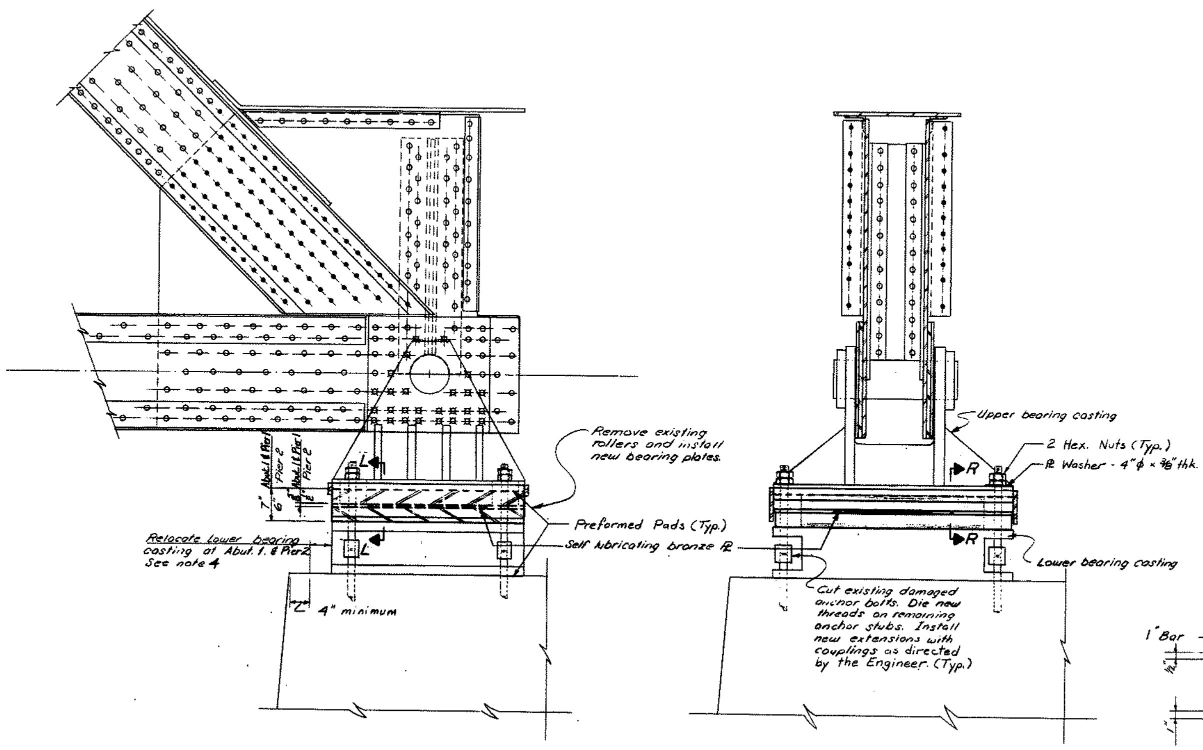
3. The concrete bearing area will be checked prior to reassembly of the bearings and an approved levelling epoxy grout shall be used to dress the area, if required, as determined by the Engineer.

4. The lower bearing castings on Abut. 1 & Pier 2 shall be moved so that the respective ℄ Bearings of the upper and lower bearing castings are aligned at 45° F. Use the chart on sheet 3 to determine the proper relative adjustment between the castings. In no case shall the lower casting be set closer than 4" to the face of the abutment.

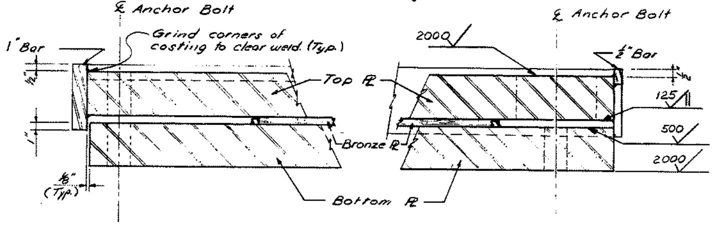
5. Drill and grout new anchor bolts for the relocated bearings at abutment 1 & pier 2. Supply 1½"Φ anchor bolts, hex. nuts and washers and provide 1'-3" of embedment.

6. Payment for all work and materials indicated on this sheet shall be paid for under the lump sum Item 523.25, Expansion Bearing Modifications.

7. The dimensions shown on the plan shall be verified in the field before fabricating the top and bottom plates to ensure a proper fit with the existing castings.

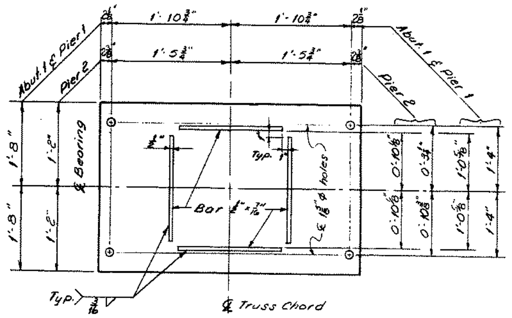


BEARING REHABILITATION DETAILS



SECTION L-L

SECTION R-R

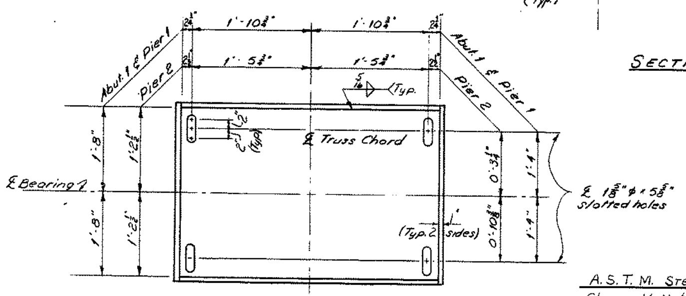


BOTTOM PLATE

Abut. 1 & Pier 1 - 4 req. 3¼" ℞ Thickness

Pier 2 - 2 req. 2⅜" ℞ Thickness

Self Lubricating Bronze ℞s
6 required
4 - 25" x 25" x ⅝" Abut. 1 & Pier 1
2 - 20" x 20" x ½" Pier 2



TOP PLATE

Abut. 1 & Pier 1 - 4 req. 3⅛" ℞ Thickness

Pier 2 - 2 req. 2⅝" ℞ Thickness

A. S. T. M. STEEL CLASSIFICATION

Charpy V-Notch tests are not required for steel used in bearing pedestals.

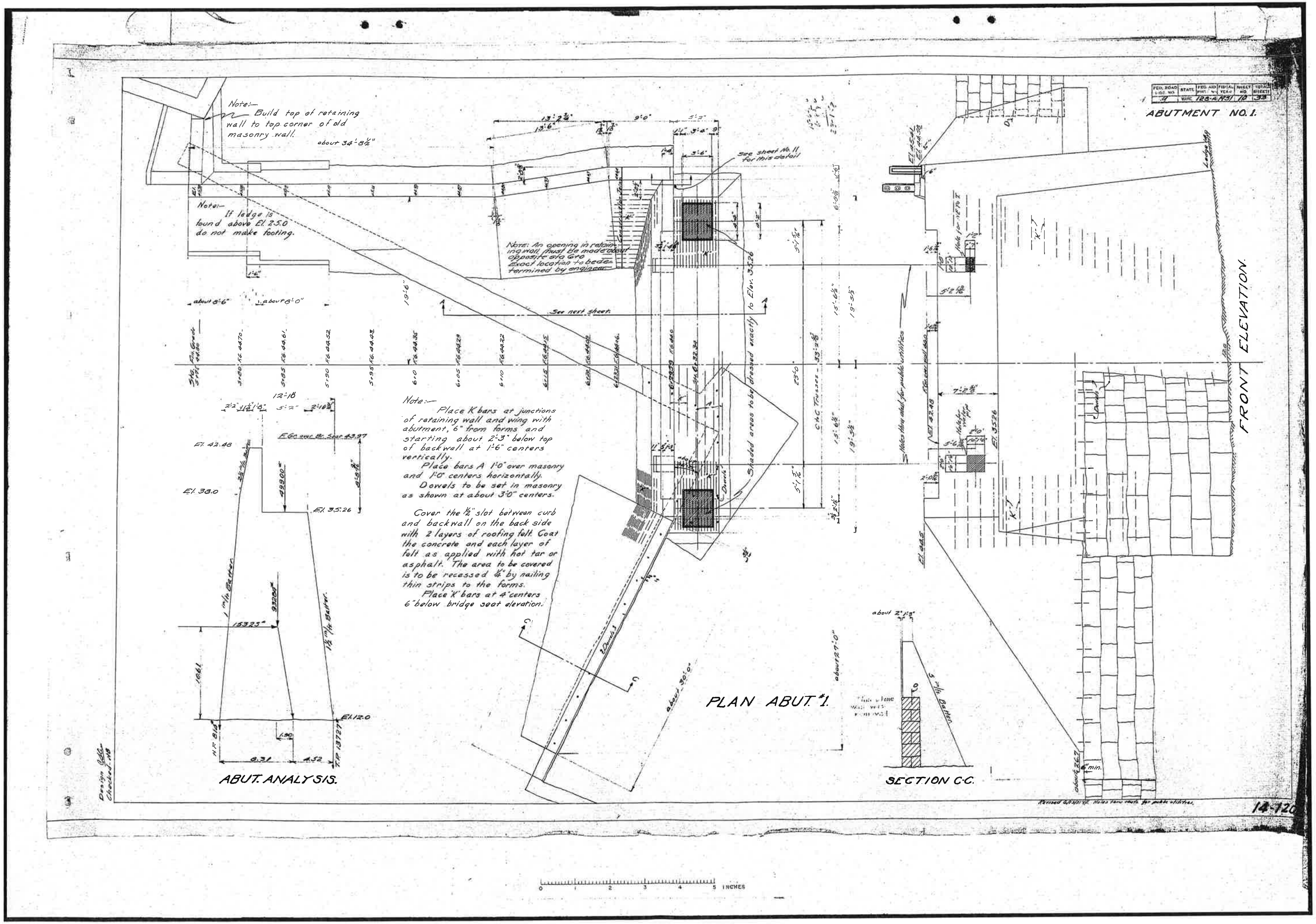
99-397

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

ANDROSCOGGIN RIVER BRIDGE
OVER THE
ANDROSCOGGIN RIVER
BETWEEN THE TOWNS OF
BRUNSWICK - CUMBERLAND CO.
AND
TOPSHAM - SAGADAHOC CO.
BEARING DETAILS
AUGUSTA, MAINE May 1985

	BY	DATE
PROJECT DESIGN ENGINEER		
PLANS DESIGN - DETAILED	JAF DMD	
CHECKED	JAF	
REVISIONS		
FIELD CHANGES		

BRUNING 44-132 45710-1



FED. ROAD DIST. NO.	STATE	FED. AID PROJ. No.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
3	MAINE	128-A	1931	10	33

ABUTMENT NO. 1.

Note:— Build top of retaining wall to top corner of old masonry wall.

Note:— If ledge is found above El. 25.0 do not make footing.

Note: An opening in retaining wall must be made about opposite sta 6+0 Exact location to be determined by engineer

Note:— Place K bars at junctions of retaining wall and wing with abutment, 6" from forms and starting about 2'-3" below top of backwall at 1'-6" centers vertically.

Place bars A 1'0" over masonry and 1'0" centers horizontally.

Dowels to be set in masonry as shown at about 3'0" centers.

Cover the ½" slot between curb and backwall on the back side with 2 layers of roofing felt. Coat the concrete and each layer of felt as applied with hot tar or asphalt. The area to be covered is to be recessed ¼" by nailing thin strips to the forms.

Place 'K' bars at 4" centers 6" below bridge seat elevation.

Shaded areas to be dressed exactly to Elev. 35.26

See sheet No. 11 for this detail

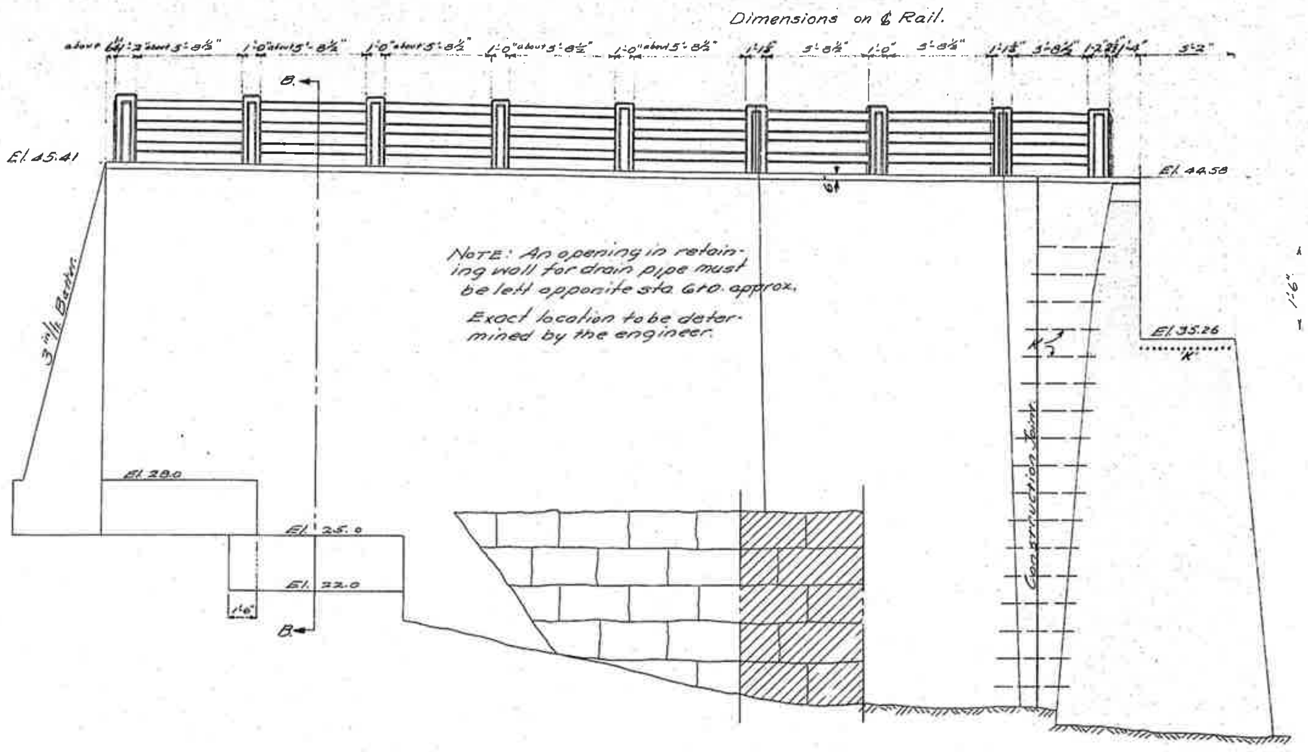
Holes thru wall for public utilities

This stone wall was removed

Revised 6/1931 Holes thru walls for public utilities.

FED. ROAD DIST. NO.	STATE	FED. AID PROJECT NO.	SHEET NO.	TOTAL SHEETS
9	[illegible]	128-A 1131	11	33

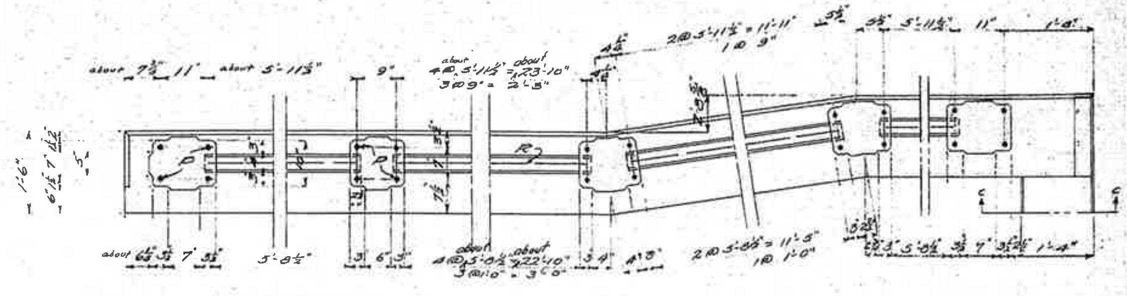
ABUTMENT NO.1.



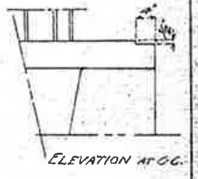
NOTE: An opening in retaining wall for drain pipe must be left opposite sta 670 approx.

Exact location to be determined by the engineer.

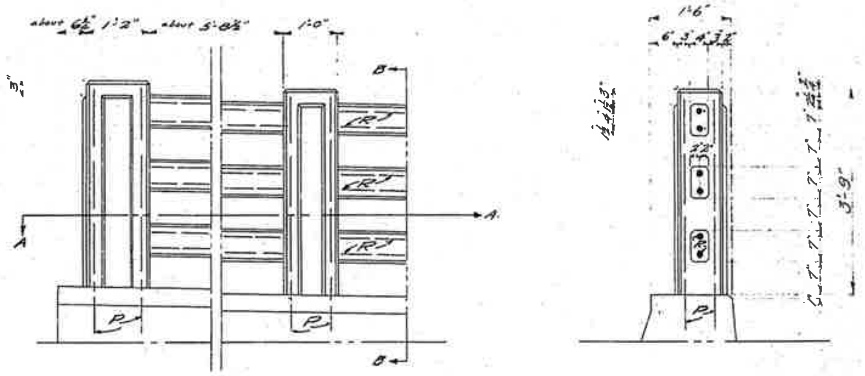
SIDE ELEVATION & SECTION A-A, ABUTMENT #1.



SECTION A-A.



ELEVATION AT C.C.

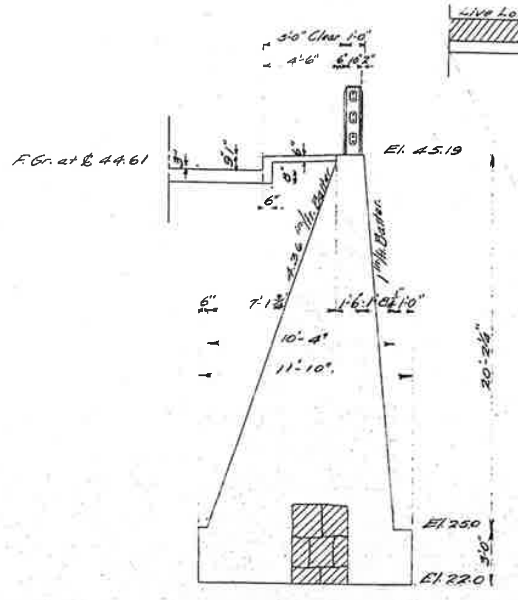


SIDE ELEVATION.

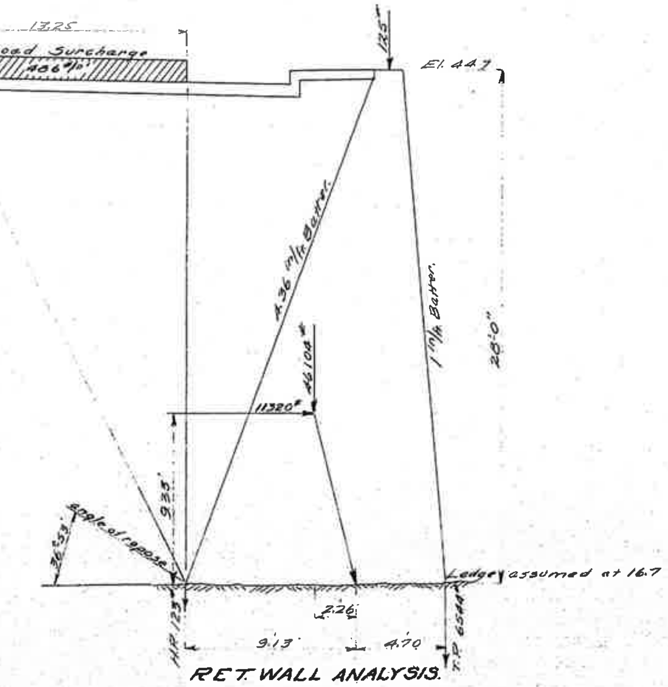
SECTION B-B.

RAIL DETAIL.

Steel for posts to be set in retaining wall. Precast rail bars in lengths as set in field. Place rail bars in position with ends projecting into post forms 2½". Wrap ends 6" with two thicknesses of roofing felt. Fold in ends, and, when post forms are removed, cut away all exposed felt. Panels on posts to be ⅞" thick. Chamfer all exposed edges of concrete ½" unless otherwise indicated.



SECTION B-B.



RET. WALL ANALYSIS.

STEEL SCHEDULE, ABUT. #1.

MARK	SIZE	NO. RQD.	LENGTH	LOCATION.
A	¾" ⌀	4	12'-0"	Abut. over Masonry
D	"	32	1'-6"	Dowels to "
K	"	70	8'-0"	Ret. Wall, Wings & Br. St.
P	"	36	4'-9"	Posts.
R.	⅝" ⌀	48	6'-6"	Rail, Cut in field.

All steel to be plain round bars of structural grade.

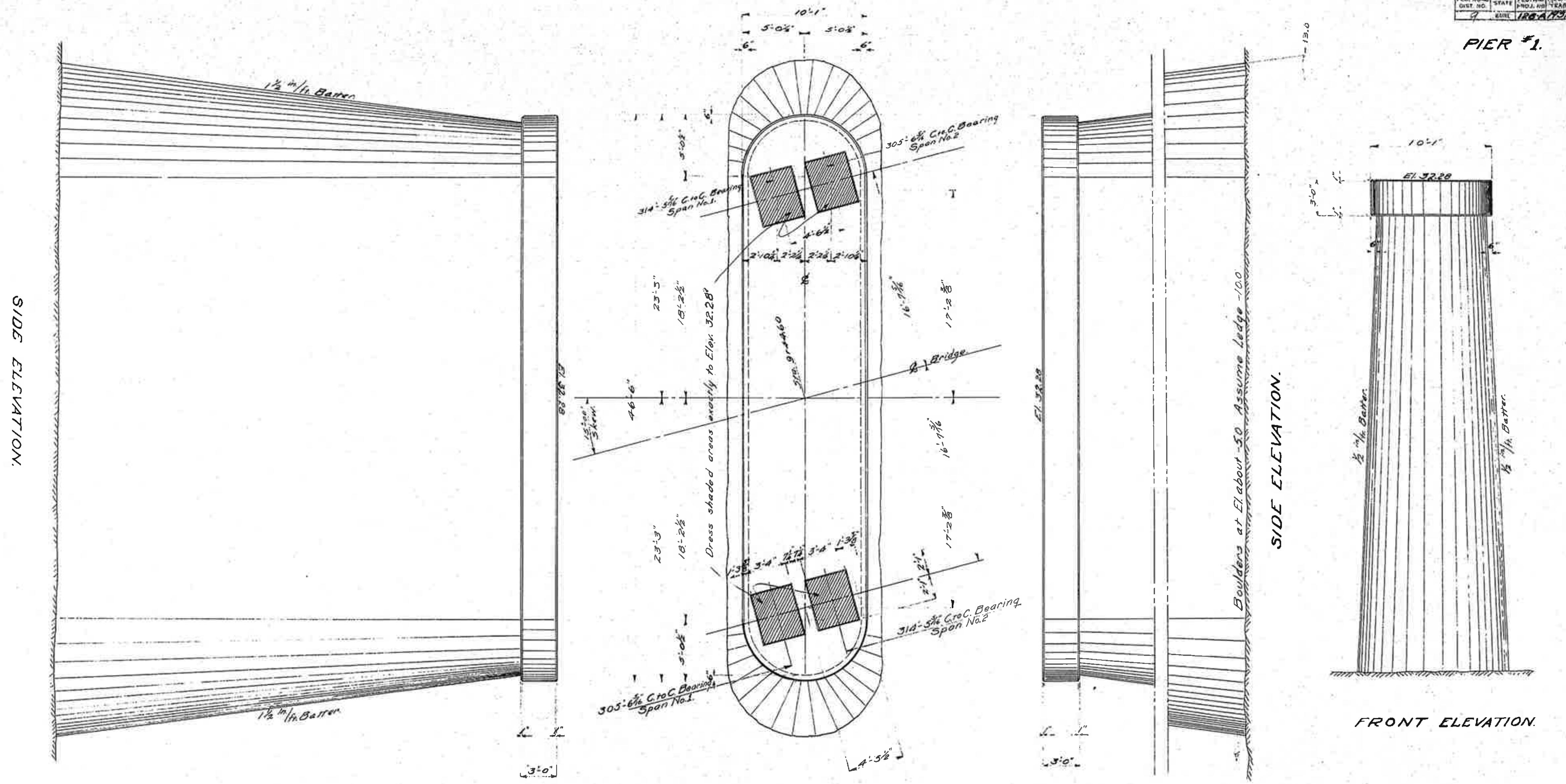
Design - O.H.
Checked - W.B.

14-121

0 1 2 3 4 5 INCHES

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	126-A	1931	72	93

PIER #1.



PLAN PIER #1.

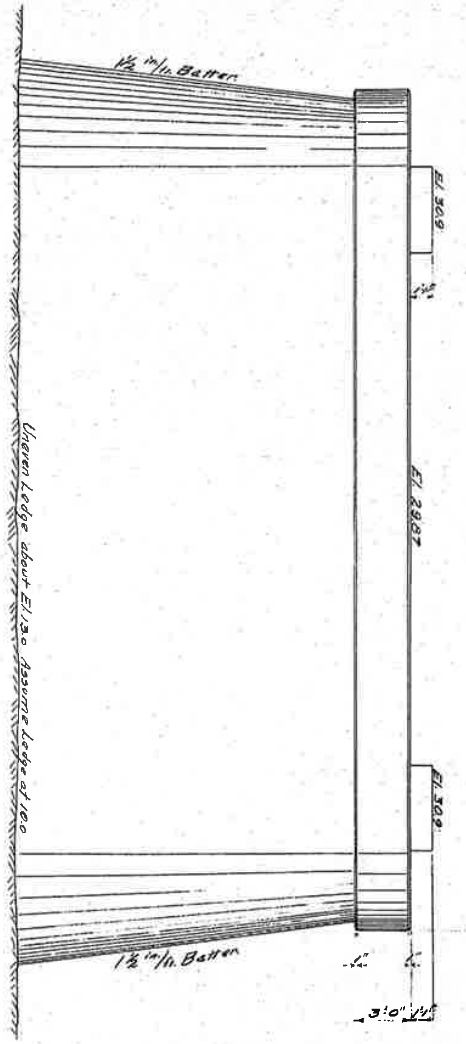
Design - G.H.G.
Checked - H.B.

14-122

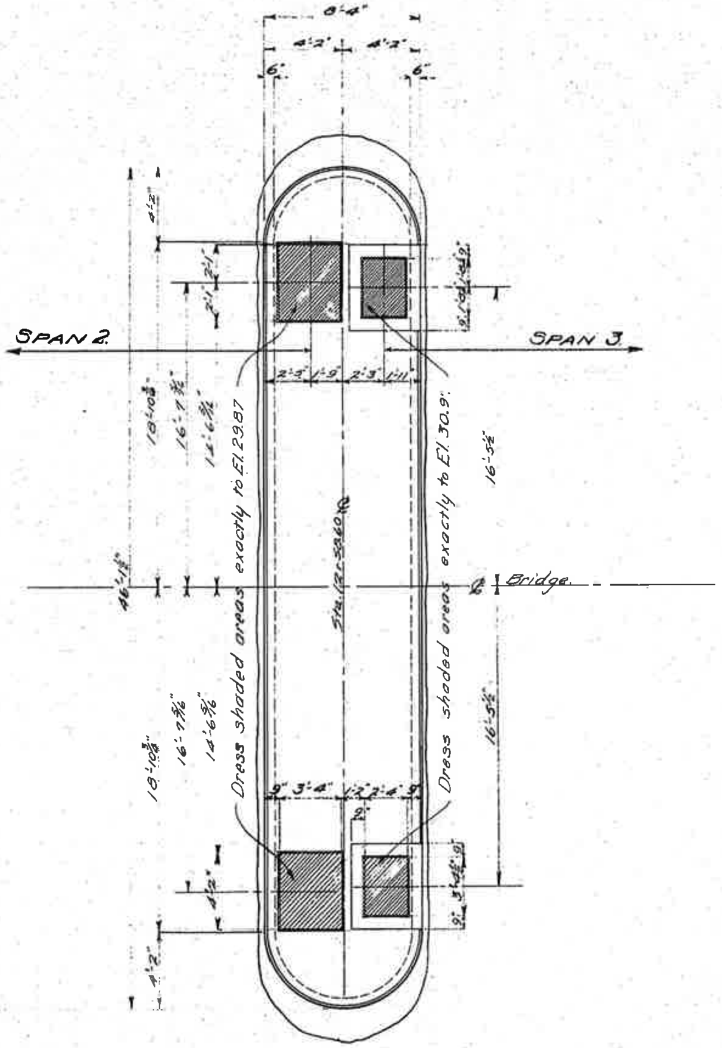
0 1 2 3 4 5 INCHES

FED ROAD DIST NO	STATE	FED AID PROJ NO	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	[illegible]	128-A	1931	13	33

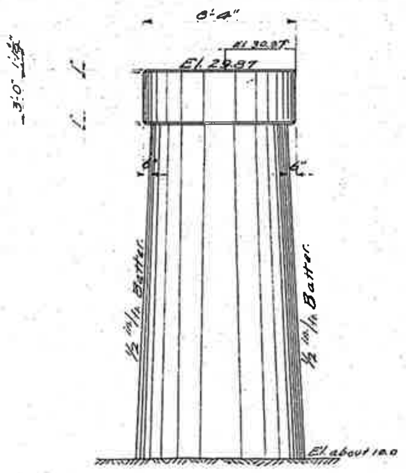
PIER #2.



SIDE ELEVATION.



PLAN PIER #2.

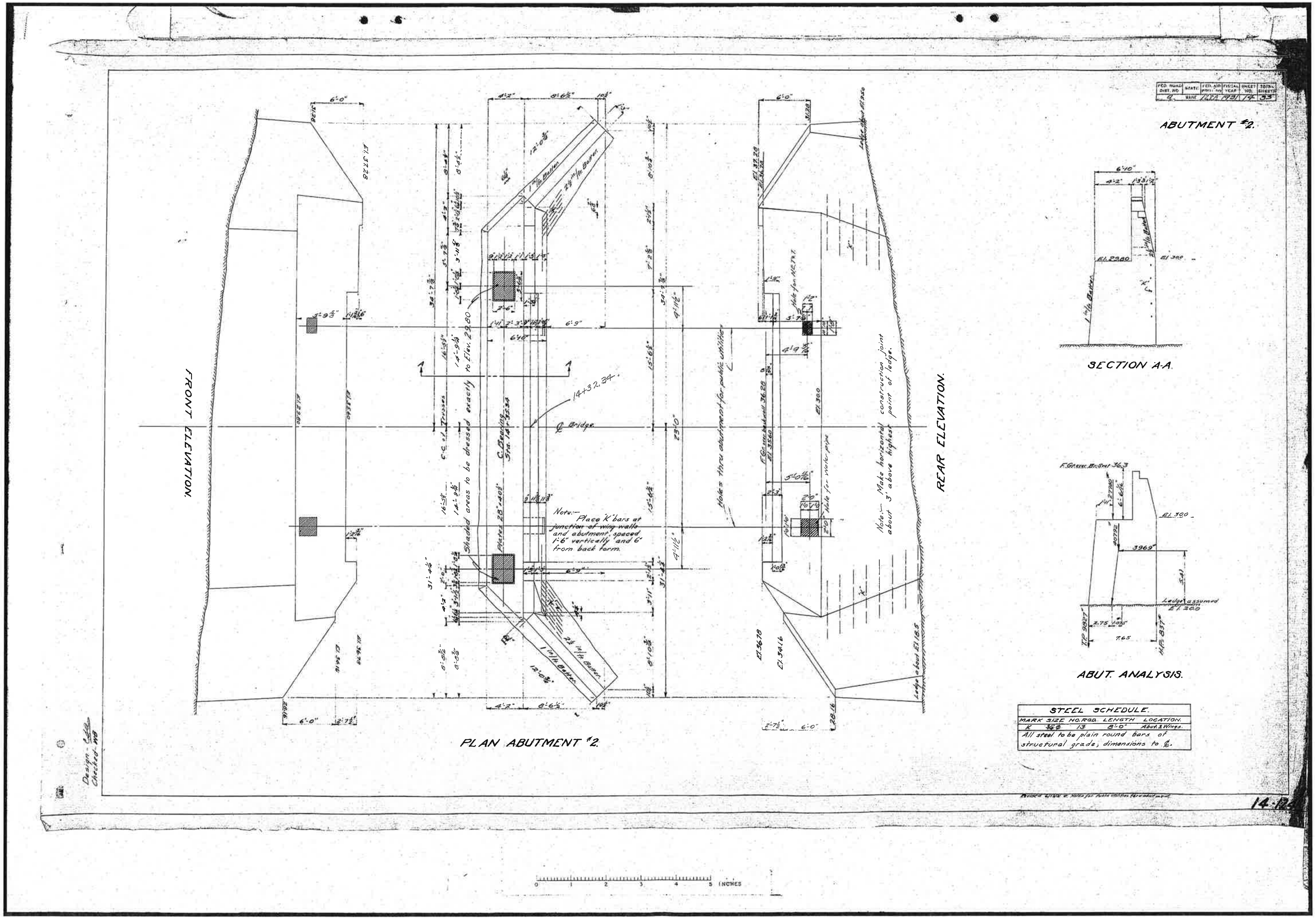


FRONT ELEVATION.

Design - C.J.
Checked - W.B.

14-12

0 1 2 3 4 5 INCHES



STEEL SCHEDULE.				
MARK	SIZE	NO. RQD.	LENGTH	LOCATION.
K	¾"⌀	13	8'-0"	Abut. & Wings.

All steel to be plain round bars of structural grade; dimensions to ℄.

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	128 A	1931	15	33

LOADING.

Live Load-
Floor system and hangers, two 15-ton trucks and one electric train.
Trusses - Total moving uniform load of 469 lbs. per linear foot; 13,200 lbs. concentration for moment, 19,100 lbs. concentration for shear and electric car loading.

Impact coefficient of live load stress.
$I = \frac{50}{L+125}$ where L = loaded length of span in feet.

Sidewalk Loading.
Stringers and brackets - 100 lbs per sq. foot.
Sidewalk truss - $P = (40 + \frac{3000}{L})(\frac{55-W}{50})$
where P = live load in lbs. per sq. ft. of Sidewalk area, but not to exceed 100 lbs. per sq. ft.
L = loaded length of sidewalk in feet.
W = width of sidewalk in feet.

Wind Loading.
50 lbs. per sq. ft. on 1½ times area of windward truss, including floor stringers, etc., and considered as a moving load.
For electric car loading see sheet no. 17

Dead Panel Loads—

Span No. 1.

Sidewalk Truss.

Panels 1 to 7 incl.	148,800 lbs.
Panel 8	154,500 "
" 9	136,300 "

Roadway Truss.

Panels 1 to 7 incl.	124,700 "
Panel 8	119,900 "
" 9	120,900 "

Span No. 2.

Sidewalk Truss.

Panel 1.	146,900 "
" 2.	146,200 "
Panels 3 to 9 incl.	148,000 "

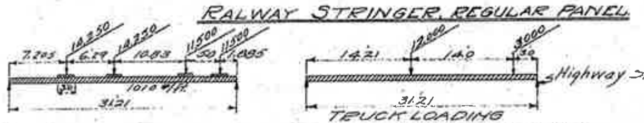
Roadway Truss.

Panel 1.	127,500 "
" 2	126,400 "
Panels 3 to 9 incl.	125,400 "

200 lb. per lin. ft. added to each truss for public utilities.

RAILWAY STRINGERS.

RALWAY STRINGER, REGULAR PANEL.



Dead Load-
rail = 30 lbs.
surf & slab = 853 "
additional wgt. 23 "
stringer = 104
1010 lb. per ft.

$RL = (3000 \times \frac{3}{31.21} + 12,000 \times \frac{17}{31.21}) = 6,824$ lbs.
L.L.M. = 6,824 × 14.21 × 12 × 1/6 = 193,900 in. lbs.
100% overload = 193,900 " "
Impact M. = .321 = 124,300 " "
512,300 " "

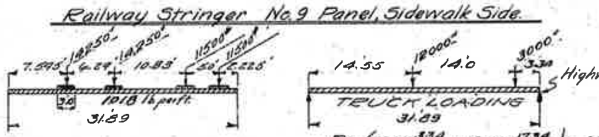
$RL\text{-live} = \frac{11,500 \times 8.77 + 14,250 \times 91.72}{31.21} = 22,280$

L.L.M. = (22280 × 13.995 − 14,250 × 6.29 − 7125 × .75) 1.5 × 12 = 3,702,500 in. lbs.
Impact M. = .321 = 1,118,500 " "
D.L.M. = 1.5 × 1010 × $\overline{31.21}^2$ = 1475700 " "
H 15 Moment + I = 512,300 " "
Total Mom. = 6,809,000 " "

$S = \frac{6809,000}{24,000} = 283.7$

Use B 28 - 28" × 104 lb. S = 284.7

Railway Stringer No. 9 Panel, Sidewalk Side.



$RL = (3000 \times \frac{3.34}{31.89} + 12000 \times \frac{17.34}{31.89}) = 6839$ lbs.

$RL = \frac{11500 \times 9.46 + 14250 \times 92.40}{31.89} = 22,354$

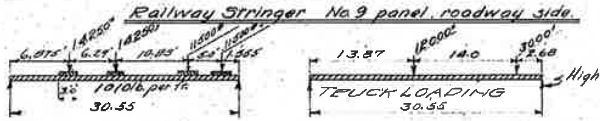
Dead Load-
rail = 30 lb.
surf & slab = 853 "
additional wgt. 23 "
stringer = 112
1018 lb. per ft.

L.L.M. car = (22354 × 13.835 − 14250 × 6.29 − 7125 × .75) 1.5 × 12 = 3,857,200 in. lb.
Impact M. = .319 = 1230400 " "
L.L.M. H15 = 6839 × 14.55 × 12 × 1/6 = 199,000 " "
+100% overload = 199,000 " "
Impact = .319 = 127000 " "
D.L.M. = 1.5 × 1018 × $\overline{31.89}^2$ = 1552900 " "
7,165,500 " " Total M.

$S = \frac{7165500}{24000} = 298.6$

Use B 28 - 28¼" × 112 lb. S = 306.4

Railway Stringer No. 9 panel, roadway side.



Dead Load-
rail = 30 lb.
surf & slab = 853 "
additional wgt. 23 "
stringer = 104 "
1010 lb. per ft.

$RL = (12000 \times \frac{16.68}{30.55} + 3000 \times \frac{2.68}{30.55}) = 6815$ lb.

$RL = \frac{11500 \times 8.11 \times 14250 \times 41.06}{30.55} = 22,205$

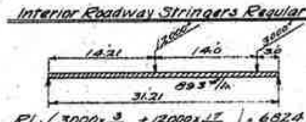
L.L.M. = (22205 × 13.165 − 14250 × 6.29 − 7125 × .75) 1.5 × 12 = 3,552,300 in. lb.
Impact = .322 = 1,143,800 " "
L.L.M. H15 = 6815 × 13.87 × 12 × 1/6 = 189,000 " "
+100% overload = 189,000 " "
Impact = .322 = 121,700 " "
D.L.M. = 1.5 × 1010 × $\overline{30.55}^2$ = 1414,000 " "
6,609,800 " " Total M.

$S = \frac{6609,800}{24000} = 275.4$

Use B 28 - 28¼" × 104 lb. S = 284.7

ROADWAY STRINGERS.

Interior Roadway Stringers Regular Panel.



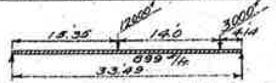
Dead Load-
slab 712 lbs.
surface 90 "
stringer 91 "
893 #/ft.

$RL = (3000 \times \frac{3}{31.21} + 12000 \times \frac{17}{31.21}) = 6824$ lbs.
L.L.M. = 6824 × 14.21 × 12 × 6/4.5 = 1,551,500 in lbs.
+100% overload 1,551,500 " "
Impact M. = .321 996,100 " "
D.L.M. = 1.5 × 893 × $\overline{31.21}^2$ 1,304,800 " "
Total M. = 5,403,900 " "

$S = \frac{5403900}{24000} = 225.2$

Use B 28 - 27⅝" × 91 lb. - S = 246.9

Interior Roadway Stringer No. 9 panel, Sidewalk Side.



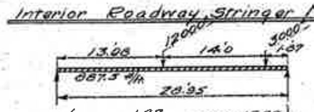
Dead Load.
slab 712 lbs.
surface 90 "
stringer 97 "
899 #/ft.

$RL = (3000 \times \frac{4.14}{33.49} + 12000 \times \frac{18.14}{33.49}) = 6871$ lbs.
L.L.M. = 6871 × 15.35 × 12 × 6/4.5 = 1,687,500 in. lbs.
+100% overload = 1,687,500 " "
I = .316 = 1,066,500 " "
D.L.M. = 1.5 × 899 × $\overline{33.49}^2$ = 1,512,450 " "
Total M. = 5,933,950 " "

$S = \frac{5933950}{24000} = 248.1$

Use B 28 - 28" × 97 lb. - S = 265.1

Interior Roadway Stringer No. 9 panel Roadway side.



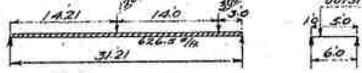
Dead Load.
slab = 712 lbs.
surface = 90 "
stringer = 85.5 "
887.5 lbs per ft.

$RL = (3000 \times \frac{1.87}{28.95} + 12000 \times \frac{15.87}{28.95}) = 6772$ lbs.
L.L.M. = 6772 × 13.08 × 12 × 6/4.5 = 1,417,250 in. lbs.
+100% overload = 1,417,250 " "
Impact = .325 = 921,200 " "
D.L.M. = 1.5 × 887.5 × $\overline{28.95}^2$ = 1,115,700 " "
Total M. = 4,871,400 " "

$S = \frac{4871,400}{24000} = 203.0$

Use B 28 - 27¾" × 85 lb. S = 222.1

Exterior stringer, Roadway side ordinary panel.



Dead Load
slab = 450 lbs.
curb = 56 "
surface = 53 "
stringer = 67.5 "
626.5 lb. per ft.

$RL = (3000 \times \frac{3}{31.21} + 12000 \times \frac{17}{31.21}) = 6,824$ lbs.
L.L.M. = 6,824 × 14.21 × 12 × 5/6 = 969,700 in. lbs.
+100% overload 969,700 " "
Impact = .321 622,500 " "
D.L.M. = 1.5 × 626.5 × $\overline{31.21}^2$ = 915,400 " "
3,477,300 " " Total M.

$S = \frac{3477,300}{24000} = 144.9$

Use B 22 - 22⅛" × 67.5 lb.
S = 148.1

Exterior stringer sidewalk side ordinary panel.

Same live loading as above

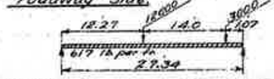
From above-
L.L.M. = 969,700 in. lbs.
+100% overload 969,700 " "
Impact 622,500 " "
D.L.M. = 1.5 × 877 × $\overline{31.21}^2$ = 1,281,400 " "
3,843,300 " " Total M.

$S = \frac{3843,300}{24000} = 160.1$

Dead Load.
slab = 450 lbs.
curb = 56 "
S.W. slab = 43 "
S.W. live = 206 "
surface = 52 "
stringer = 70 "
877 lb. per ft.

Use - B 24 - 23⅝" × 70 lb.
S = 163.7

Exterior Stringer No. 9 Panel Span No. 1. roadway side.



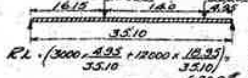
Dead Load-
slab = 450 lb.
curb = 56 "
surface = 33 "
stringer = 38 "
617 = total.

$RL = (3000 \times \frac{1.07}{27.34} + 12000 \times \frac{15.07}{27.34}) = 6731$ lb.
L.L.M. = 6731 × 12.27 × 12 × 5/6 = 825,900 in. lb.
+100% overload = 825,900 " "
Impact = .328 = 541,800 " "
D.L.M. = 1.5 × 617 × $\overline{27.34}^2$ = 691,800 " "
2,885,400 " " Total M.

$S = \frac{2,885,400}{24000} = 120.2$

Use B 22 - 21⅝" × 56 lb. S = 124.7

Exterior Stringer No. 9 Panel Span No. 1. sidewalk side.



$RL = (3000 \times \frac{4.95}{35.10} + 12000 \times \frac{18.95}{35.10}) = 6902$ #

Dead Load.
slab = 450 lb.
curb = 56 "
S.W. slab = 43 "
S.W. live = 206 "
surface = 52 "
stringer = 81 "
888 total.

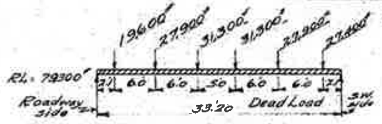
L.L.M. = 6902 × 16.15 × 12 × 5/6 = 1,114,700 in. lbs.
+100% overload = 1,114,700 " "
Impact = .313 = 697,800 " "
D.L.M. = 1.5 × 888 × $\overline{35.1}^2$ = 1,641,000 " "
4,568,200 " " Total M.

$S = \frac{4,568200}{24000} = 190.2$

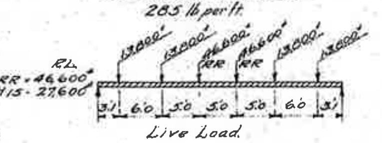
Use - B 26 - 25¾" × Pl. # S = 201.7

FLOORBEAMS.

Intermediate.



Intermediate Floorbeam
285 lb. per ft.



Live Load.

Use plate girder
Web 42" × ½"
Flange, 4 Ls 6" × 6" × ¾"
Cov. Pls. 2 pls. 16" × ½" × 18'-0"
2 pls. 16" × 7/16" × 10'-0"
S = 1122

STRESSES SPANS 1 & 2.

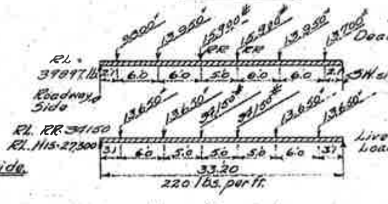
$RL = (27,400 \times \frac{2.1}{33.2} + 27,900 + 31,300 + 19600 \times \frac{31.1}{33.2}) = 79,300$ lbs.

L.L.M. Car = 46,600 × 14.1 = 657,100 ft. lbs.
+50% overload = 328,500 " "
Impact = .267 = 263,200 " "
L.L.M. H15 = 27,600 × 14.1 − 13,800 × 16 = 168,300 " "
+100% overload 168,300 " "
Impact = .267 = 89,900 " "
Total L.L.M. = 1,675,300 " "
−10% = 167,500 " "
1,507,800 " "

D.L.M. conc. = 79,300 × 14.1 − 19600 × 12 − 27,900 × 6 = 715500 ft. lbs.
D.L.M. uniform = $\frac{285 \times \overline{33.2}^2}{8}$ = 39,300 " "
754800 " "
1,507,800 " "
Total M. = 2,262,600 " "

$S = \frac{2262,600 \times 12}{24000} = 1131.3$

End Floorbeam.



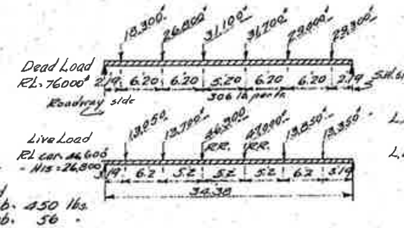
$RL = (13,700 \times \frac{2.1}{33.2} + 13,950 + 15,900 + 9800 \times \frac{31.1}{33.2}) = 39,897$ lbs.

L.L.M. Car = 34,150 × 14.1 = 481,500 ft. lbs.
+50% overload 240,800 " "
Impact = .321 = 231,900 " "
L.L.M. H15 = 27,300 × 14.1 − 13650 × 16 = 166,500 " "
+100% overload 166,500 " "
Impact = .321 = 106,900 " "
Total L.L.M. = 1,394,100 " "
−10% = 139,400 " "
1254700 " "
D.L.M. conc. = 39,600 × 14.1 − 9800 × 12 − 13950 × 6 = 357,100 " "
D.L.M. uniform = $\frac{220 \times \overline{33.2}^2}{8}$ = 30,300 " "
Total M. = 1642,100 " "

$S = \frac{1642,100 \times 12}{24000} = 821.0$

Use plate girder
Web 42" × ½"
Flange - 4 Ls 6 × 6 × ⅝
Cov. Pls. - 2 pls. 14½" × ½" × 16'-0"
S = 802.4

Skewed Floorbeam at Panel Point No. 9.



$RL, dead = 29,300 \times \frac{2.19}{34.38} + 29,000 \times \frac{8.39}{34.38} + 31,700 \times \frac{14.59}{34.38} + 31,100 \times \frac{19.79}{34.38} + 26,800 \times \frac{25.99}{34.38} + 18,200 \times \frac{32.19}{34.38} = 77,600$ lbs.

$RL, car = 47,000 \times \frac{14.59}{34.38} + 46,300 \times \frac{19.79}{34.38} = 46,600$ lbs.

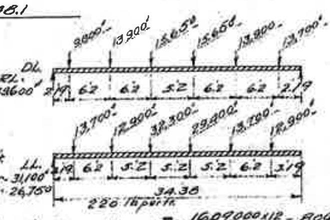
$RL. H15 = 13,350 \times \frac{3.19}{34.38} + 13,850 \times \frac{9.39}{34.38} + 13,700 \times \frac{24.99}{34.38} + 13,050 \times \frac{31.19}{34.38} = 26,800$ lbs.

L.L.M. car = 46,600 × 14.59 = 679,900 ft. lbs.
+50% overload = 339,950 " "
Impact = .267 = 272,300 " "
L.L.M. H15 = 26,800 × 14.59 − 13,050 × 11.9 + 13,700 × 5.2 = 171,000 " "
+100% overload = 171,000 " "
Impact = .267 = 91,300 " "
1,725,450 " " L.L.M.
−10% 172,550 " "
1,552,900 " "
D.L.M. conc. = 77600 × 14.59 − 18,300 × 12.4 − 26800 × 6.2 = 739100 " "
D.L.M. uniform = $\frac{306 \times \overline{34.38}^2}{8}$ = 45,200 " "
2,337,200 " " Total M.

$S = \frac{2,337,200 \times 12}{24000} = 1168.6$

Use-
Web 42" × ⅝"
4 Ls 6" × 6" × ¾"
2 pls. 16" × ½" × 15'-0"
2 pls. 16" × 7/16" × 20'-0"
S = 1188.6

Skewed End Floorbeam.



$RL, dead = 13,700 \times \frac{2.19}{34.38} + 13,900 \times \frac{8.39}{34.38} + 15,650 + 13,900 \times \frac{25.99}{34.38} + 9800 \times \frac{32.19}{34.38} = 39,600$ lbs.

$RL, Car = 29,400 \times \frac{14.59}{34.38} + 32,300 \times \frac{19.79}{34.38} = 31,100$ lbs.

$RL, H15 = 12,900 \times \frac{3.19}{34.38} + 13,700 \times \frac{9.39}{34.38} + 12,900 \times \frac{24.99}{34.38} + 13,700 \times \frac{31.19}{34.38} = 26,750$ lbs.

L.L.M. car = 31,100 × 14.59 = 453,800 ft. lbs.
+50% overload = 226,900 " "
Impact = .321 = 218,500 " "
L.L.M. H15 = 26,750 × 14.59 − 13,700 × 11.9 − 12,900 × 5.2 = 167,000 " "
+100% overload 167,000 " "
Impact = .321 = 107200 " "
1,340,400 " "
−10% 139,000 " "
1,206,400 " " L.L.M.
D.L.M. conc. = 39,600 × 14.59 − 9800 × 12.4 + 13,900 × 6.2 = 370,100 " "
D.L.M. uniform = $\frac{220 \times \overline{34.38}^2}{8}$ = 32,500 " "
1,609,000 " " Total M.

$S = \frac{1609,000 \times 12}{24000} = 804.5$

Use-
Web 42" × ½"
4 Ls 6 × 6 × ⅝"
cov. pl. 2 pls. 14½" × ½" × 18'-0"
S = 802.4

Sidewalk Stringers.

Interior Stringer
Loading-
slab = 187 lb.
live = 558 "
stringer 33 "
778 "

$M = \frac{778 \times \overline{31.21}^2}{8} = 94700$ #

$S = \frac{94700 \times 12}{24000} = 47.3$

Use B 14 - 14" × 33 #
S = 47.8

Exterior Stringer
Loading - slab = 160 lb.
live = 393 "
stringer = 30 "
rail = 30 "
615

$M = \frac{615 \times \overline{31.21}^2}{8} = 74900$ in. lbs.

$S = \frac{74900 \times 12}{24000} = 37.4$

Use B 14 - 13⅞" × 30 #
S = 42.6

Interior Stringer
No. 9 panel Span no. 1
Loading-
slab = 187 lb.
live = 558 "
stringer = 40 "
785 "

$M = \frac{785 \times \overline{35.67}^2}{8} = 124850$

$S = \frac{124850 \times 12}{24000} = 62.4$

Use - B 16 - 16" × 40 # S = 65.8

Exterior Stringer
Loading-
slab = 160 lb.
live = 393 "
stringer = 35 "
rail = 30 "
618

$M = \frac{618 \times \overline{35.67}^2}{8} = 98300$ #

$S = \frac{98300 \times 12}{24000} = 49.2$

Use - B 16 - 15⅞" × 35 #
S = 55.1

Design - V.B.E.
Traced - C.A.C.
Checked - 2/13

Revised Apr. 3, 1931. by G.B.

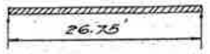
14-125

Fed. Road Div. No.	State	Fed. Aid Proj. No.	Fiscal Year	Sheet No.	Total Sheets
9	[illegible]	128-A	1931	16	[illegible]

DETAILS

SIDEWALK STRINGERS CONTINUED

No. 2 span Panel No. 2



Interior Stringer

Load- Slab= 187 lb
Live x 1.5 = 558 "
Stringer= 28 "
773 lb per ft.

$M=\frac{773\times26.75^2}{8}=69141$ ft. lb.
Use B12-28#
S=33.6

$S=\frac{69141\times12}{24000}=34.6$

Exterior Stringer

Load- Slab= 160 lb.
Live x 1.5= 393 "
rail = 30 "
Stringer= 25 "
608 "

$M=\frac{608\times26.75^2}{8}=54383$ ft. lb.

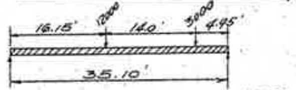
$S=\frac{54383\times12}{24000}=27.2$

Use B12 x 25#
S=31.2

ROADWAY STRINGERS CONTINUED

No. 2 span Roadway Stringers Panel No. 2.

Exterior Stringer Roadway Side



D.L= Slab = 450 lb. per ft.
Curb.= 56 " " "
Surface = 53 " " "
Stringer = 74 " " "
633 " " "

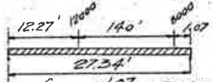
$RL=(3000\ \frac{4.95}{35.10}+12000\times\frac{18.95}{35.10}=6902$#

$LLM=6902\times16.15\times12\times\frac{5}{6}=1114700$ in. lb.
+ 100 % overload= 1114700 " "
Impact = 313 = 697800 " "
$DLM=1.5\times633\times35.1^2=$ 1169800 " "
4,097,000 in. lb. total M.

$S=\frac{4097000}{24000}=170.7$

Use B24 - 24" x 73.5 lb. S = 175.7

Exterior Stringer Sidewalk Side



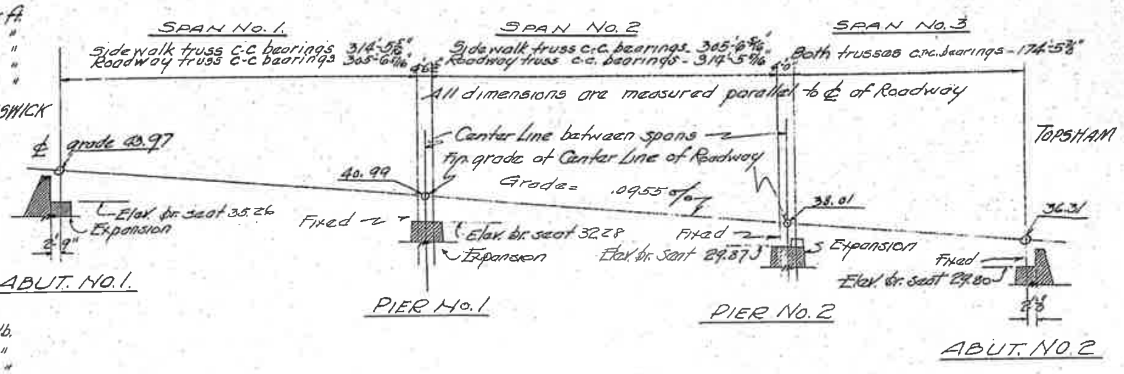
D.L. = slab = 450 lb.
Curb= 56 "
S.W. Slab= 43 "
S.W. live = 206 "
Surface= 52 "
Stringer= 63
870 lb. per ft.

$RL=\left(3000\times\frac{1.07}{27.34}+12000\times\frac{15.07}{27.34}\right)=6731$ lb.

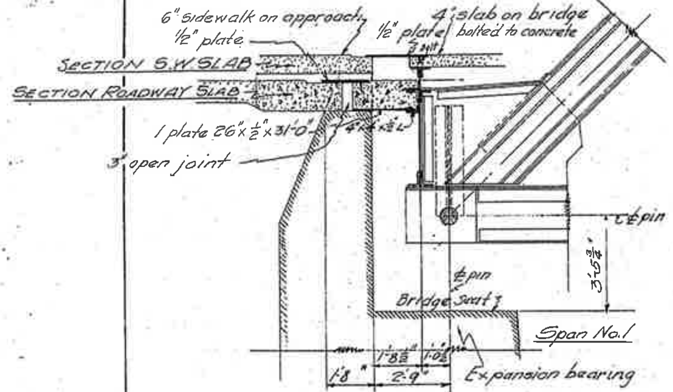
$LL\ M=6731\times12.27\times12\times\frac{5}{6}=$ 825900 in. lb.
+ 100 % overload= 825900 " "
Impact= .328 541800 " "
$DLM=1.5\times870\times27.34^2=$ 975500 " "
3,169,100 in. lb. total M.

$S=\frac{3169100}{24000}=132.0$

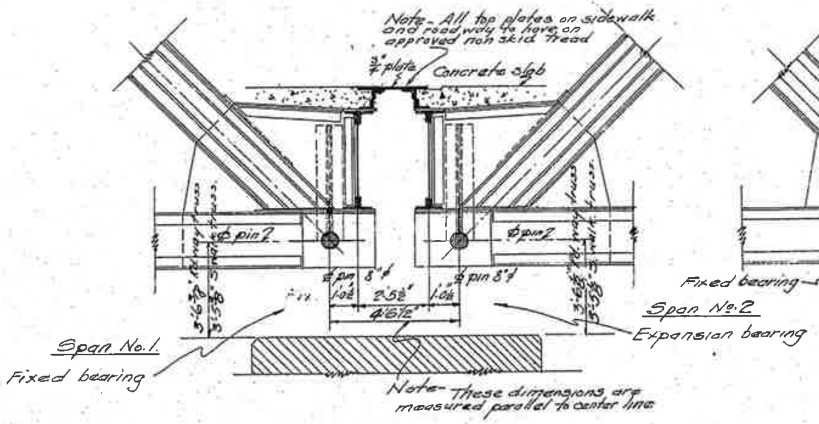
Use B22 X 62.5#
S = 135.9



GENERAL ELEVATION



ABUTMENT No. 1

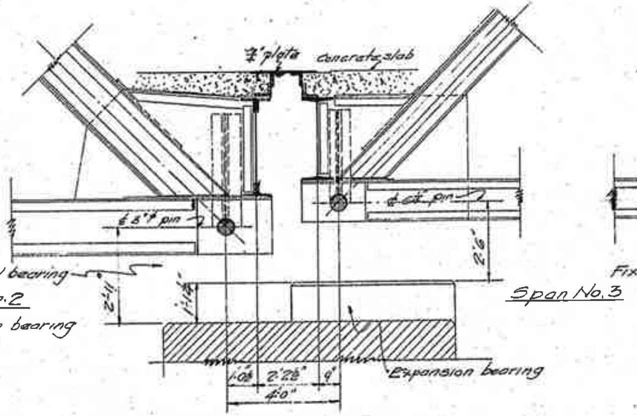


Note - All top plates on sidewalk and roadway to have an approved non skid tread

Note - These dimensions are measured parallel to center line

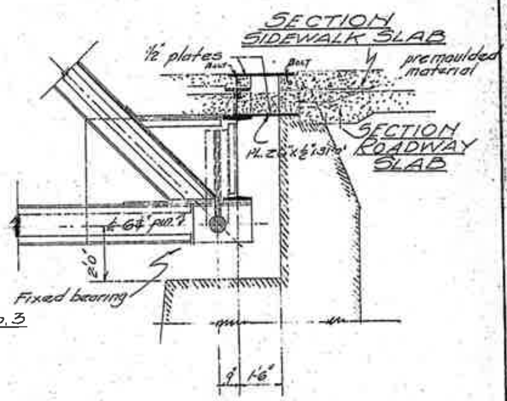
PIER No. 1

Skewed end panels over this pier

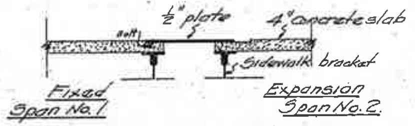


PIER No. 2

Note - Method of expansion for sidewalk Same as at pier No. 1. between Spans 1 and 2.



ABUTMENT No. 2



SIDEWALK DETAIL AT PIER No. 1

DRAWN BY V.A.E.
TRACED BY L.A.K.
CHECKED BY G.G.

14-

0 1 2 3 4 5 INCHES

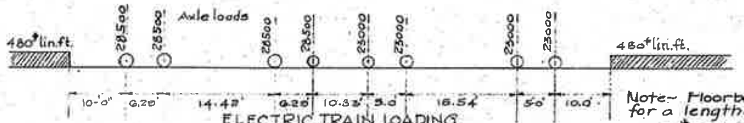
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	[illegible]	123-A	1931	17	33

CROSS SECTION AND FLOOR SYSTEM SPAN NO. 3.

-RIVETS-
Sway Frames and struts 3/4"⌀
Truss connections 7/8"⌀

LOADING

Floor System and Hangers, Two 15 Ton Trucks and Electric Train



ELECTRIC TRAIN LOADING

Note– Floorbeams figured for a length of 32'-8"

DEAD LOAD = 423# per lin. ft. of S.W. Truss.
" 3465 " " " " Roadway Truss.

TRUSSES:–
Total moving uniform load of 469# per linear ft. 13200# concentration for moment and 19100# concentration for shear

IMPACT COEFFICIENT OF LIVE LOAD STRESS

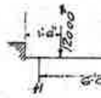
$I = \frac{50}{L+125}$ Where L = Load length of Span in feet

SIDEWALK LOADING
Stringers and brackets – 100# per sq. ft.
Sidewalk Truss $p = (40 + \frac{3000}{L})(\frac{55 - W}{50})$

P = Live Load in lbs. per sq. ft of Sidewalk area but not to exceed 100# per sq. ft.
L = Loaded length of Sidewalk in feet
W = Width of Sidewalk in feet

WIND LOADING
30# per sq. ft. on 1½ times the projected area of Windward Truss including floor, stringers, etc. and considered as a moving load.

STRINGER NO. 1



Proportion of truck concentration to Stringer No 1 = 5/6

D.L. Mom. = 1/8 × 22² × 611 × 12 = 443 600'#
L. L. Mom. = 5/6 × 792,000 660,000'#
100% Live Load Mom. = 660000'#
Impact on Live Load @ .34 = 448,800'#
2,212,400'#

Slab =	450
Curb =	56
future Surface	53
Stringer	52
	611#

S = 2212400 / 24000 = 92.2"³
Use 18" B.I. @ 52# S = 94.3"³

STRINGER NO. 2

Proportion of truck concentration to Stringer No 2 = 1
D.L. Moment = 1/8 × 22² × 869.5 × 12 = 631,300
L.L. Mom. = 792000 × 6/4.5 = 1,056,000
100% Live Load = 1,056,000
Impact on Live Load @ .34 = 718,100
3,461,400

Slab =	712
Future Surface	90
Stringer	67.5
	869.5

S = 3,461400 / 24000 = 144.2"³
Use 22" B.I. @ 67.5# S = 148.1

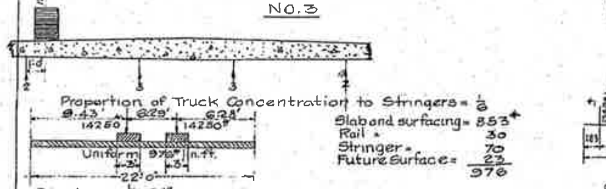
STRINGER NO. 4

Proportion of truck concentration to Stringer No. 4 = 5/6
D.L. Moment = 1/8 × 862 × 22² × 12 = 625,800
L.L. Moment = 5/6 × 792000 = 660,000
100% L. L. Moment = 660,000
Impact on L.L. @ 34 = 448,800
2,394,600

Slab =	450#
Curb =	56
S.W. Slab =	43
1½ × S.W. L.L. =	206
Future Surface	52
Stringer	55
	862

S = 2,394,600 / 24,000 = 99.8
Use 22" B.I. @ 54.5# S = 113.3"³

RAILWAY STRINGERS NO. 3



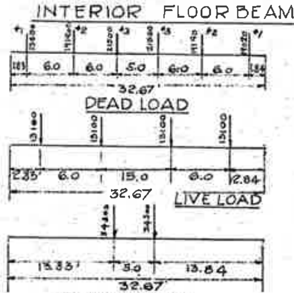
Proportion of Truck Concentration to Stringers = 1/3

Slab and surfacing =	853#
Rail =	30
Stringer =	70
Future Surface =	23
	976

D.L. Mom. = 1/8 × 22² × 976 × 12 = 708,600
Rail Mom. = 12210 × 10.5 – 4730 × 2.37 × 1.28.5 × 12 = 1,350,200
L. L. Mom = 1/6 × 792000 132,000
100% Truck Moment = 132,000
50% Rail Moment = 675,100
Impact on Live Load @ .34 = 778,400
3,776,300'#

S = 3776,300 / 24000 = 157.3"³
Use 24" B.I. @ 70# S = 163.7"³

INTERIOR FLOOR BEAMS



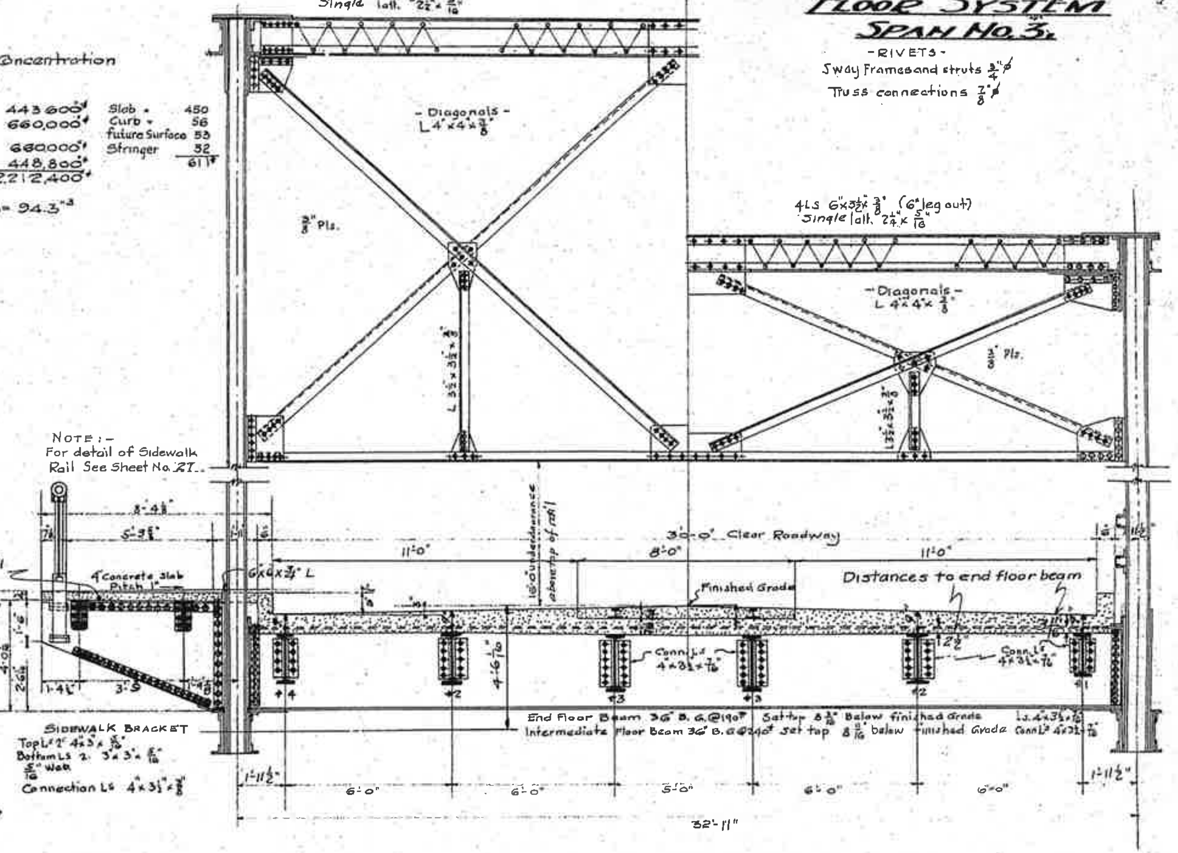
RAILWAY LOAD

MOMENTS AT STRINGER 3
D.L.M = 54500 × 13.83 – 19190 × 6 = 18490 × 12 = 476700'#
Moment of Beam = 240 × 16.33 × 13.83 – 240 × 13.83 × 6.92 = 31234'#
Live Load Mom. 26200 × 13.83 – 13100 × 16 = 152,700
Rail Mom. = 34300 × 13.83 = 474 400#

SUMMARY

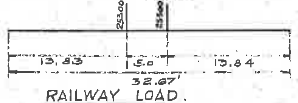
Dead Load Moment =	476,700
Live Load Moment =	152,700
100% Live Load Moment =	152,700
Rail Moment =	474,400
50% Rail Moment =	237,200
Impact on Rail and L.L. @ .295 =	300,000
Dead Load Mom. of Beam	31,200
	1,824,900
Reduce by 10% of Combined Live Load 1,317,000 =	131,700
	1,693,200'# = 20,318,400"#

S = 20318400 / 24000 = 846.6"³
Use 36" B.G. @ 240# S = 872.0



END FLOOR BEAMS

Dead load Moment = ½ of that for interior Beams
Live Load is the same as that for interior Beams for H 15



RAILWAY LOAD.

Rail Moment 25300 × 13.83 = 349900
D.L. Mom. of Beam = 190 × 16.33 × 13.83 – 190 × 13.83 × 6.92 = 24700

SUMMARY

Dead Load Moment =	238400
Live Load Moment =	152700
100% Live Load Moment =	152700
Rail Moment =	349900
50% Rail Moment =	175000
Impact on Rail and Live Load .34	282300
Dead Load Moment of Beams	24700
	1,375,700'#
Reduce by 10% of Combined Rail and Live Load 1,112,000	111300
	1,264,400'# = 15,172,800"#

S = 15172 800 / 24000 = 632.2"³
Use 36" B G @ 190# S = 659.9"³

INTERIOR S.W. STRINGER

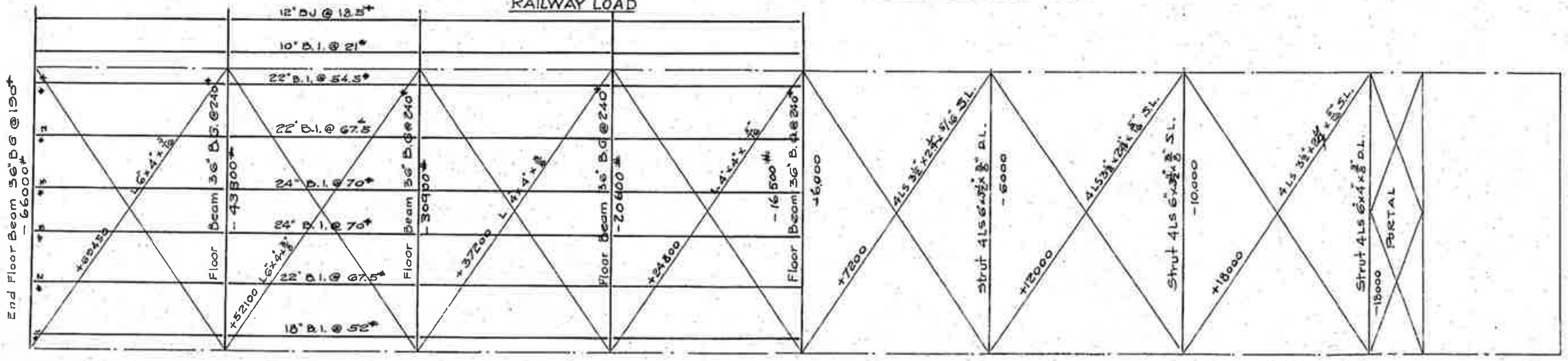
Slab	160#
1½ L L	485
Stringer	21
	666

M = 1/8 × 666 × 22² × 12 = 483800"#
S = 483,800 / 24 000 = 20.1"³
Use 10" B.I. @ 21# S = 21.8

EXTERIOR S.W. STRINGER

Slab =	160.
1½ L.L.	393.
Stringer	18.5
	571.5

M = 1/8 × 571.5 × 22² × 12 = 414 900"#
S = 414900 / 24000 = 17.3"³
Use 12" B.I. @ 18.5# S = 20.2"



HALF PLAN - LOWER LATERAL SYSTEM

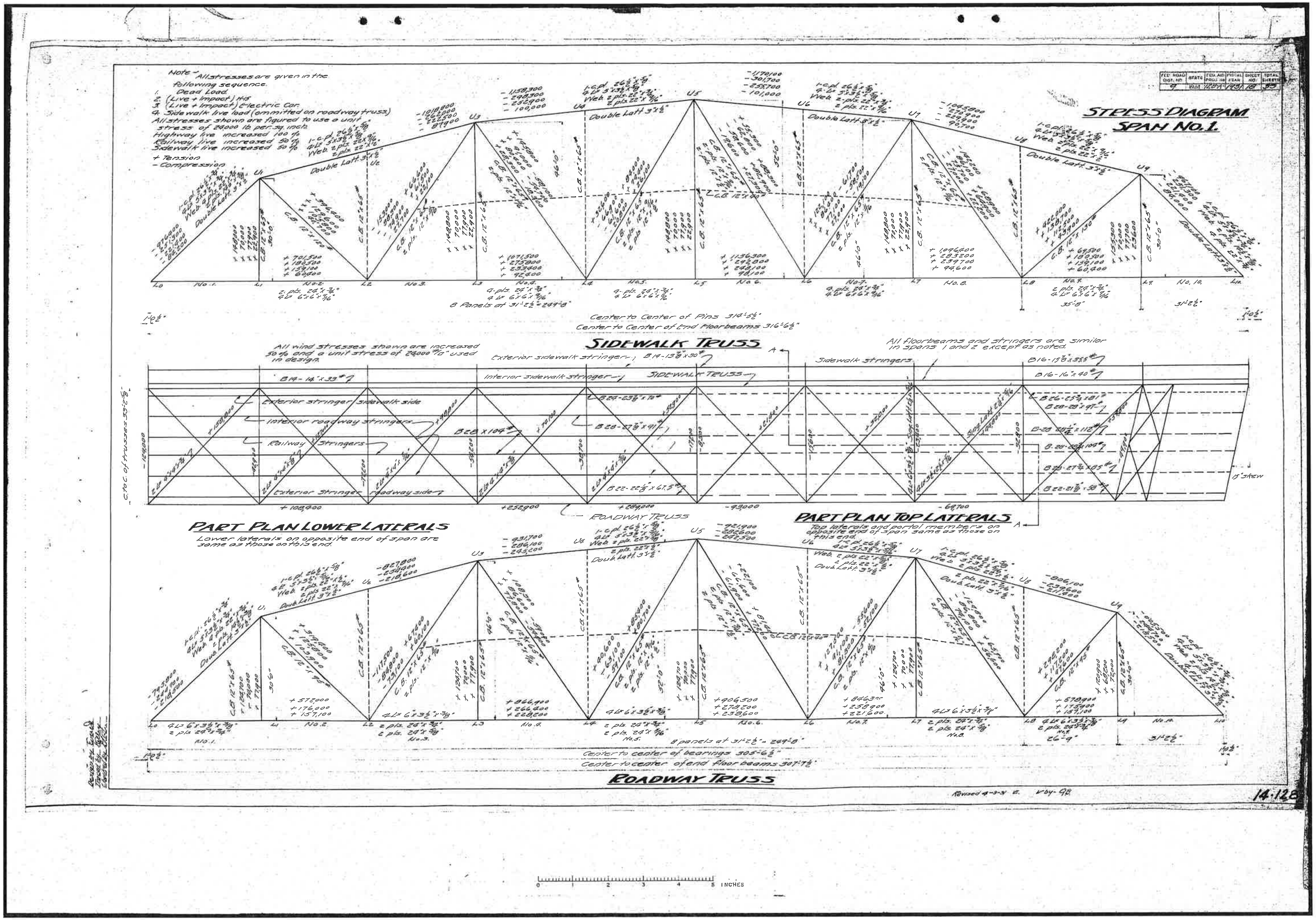
HALF PLAN - UPPER LATERAL SYSTEM

All Wind Stresses increased 30%

0 1 2 3 4 5 INCHES

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	WASH	128-A	1931	18	35

STRESS DIAGRAM SPAN NO. 1.



Note – All stresses are given in the following sequence.
1. Dead Load.
2. (Live + Impact) H15
3. (Live + Impact) Electric Car.
4. Sidewalk live load (ommitted on roadway truss)

All stresses shown are figured to use a unit stress of 24000 lb. per. sq. inch.
Highway live increased 100 %
Railway live increased 50 %
Sidewalk live increased 50 %

+ Tension
– Compression

8 Panels at 31'-2½" = 249'-8"

Center to Center of Pins 314'-5½"
Center to Center of End Floorbeams 316'-6½"

SIDEWALK TRUSS

All wind stresses shown are increased 50% and a unit stress of 24000 #/□" used in design.

All floorbeams and stringers are similar in spans 1 and 2 except as noted.

PART PLAN LOWER LATERALS

Lower laterals on opposite end of span are same as those on this end.

PART PLAN TOP LATERALS

Top laterals and portal members on opposite end of span same as those on this end.

8 panels at 31'-2½" = 249'-8"

Center to center of bearings 305'-6½"
Center to center of end floor beams 307'-7½"

ROADWAY TRUSS

Revised 4-3-31 E. Vby-98

14-128

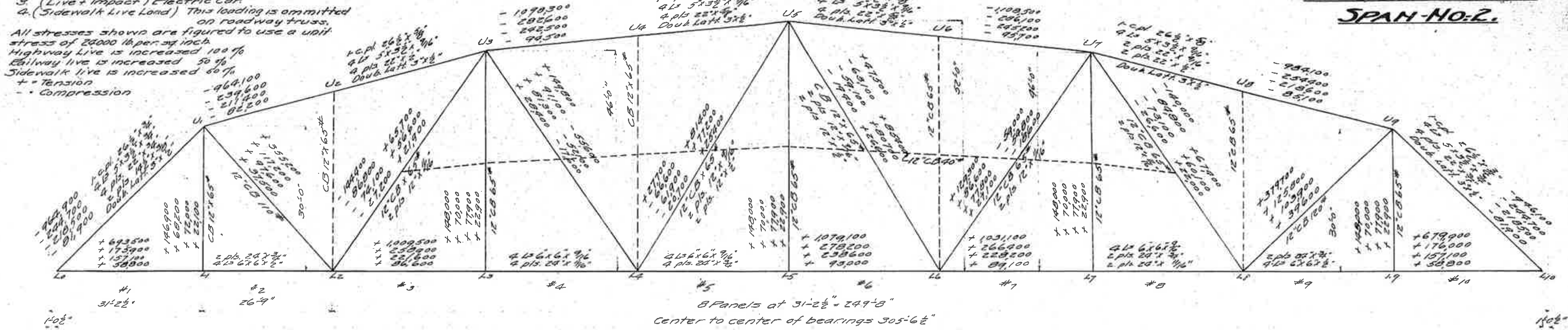
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	128-A	[illegible]	14	33

STRESS DIAGRAM
SPAN-NO-2.

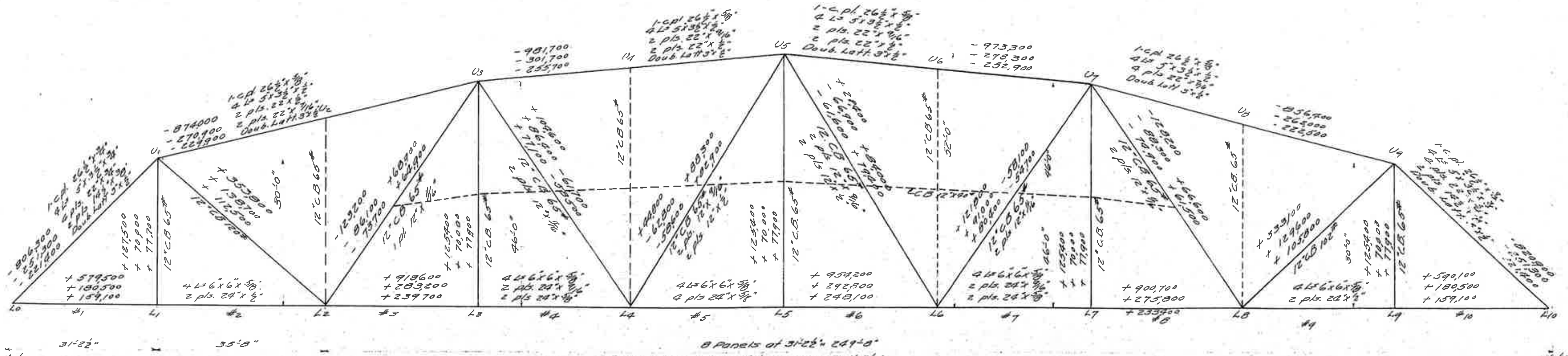
Note— All stresses are given in the following sequence:

1. Dead Load
2. (Live + Impact) H-15.
3. (Live + Impact) Electric Car.
4. (Sidewalk Live Load) This loading is ommitted on roadway truss.

All stresses shown are figured to use a unit stress of 24000 lb. per sq. inch.
Highway Live is increased 100%
Railway live is increased 50%
Sidewalk live is increased 60%
+ = Tension
- = Compression



SIDEWALK TRUSS



ROADWAY TRUSS

Designed by MBC
Traced by [illegible]
Checked by TBB

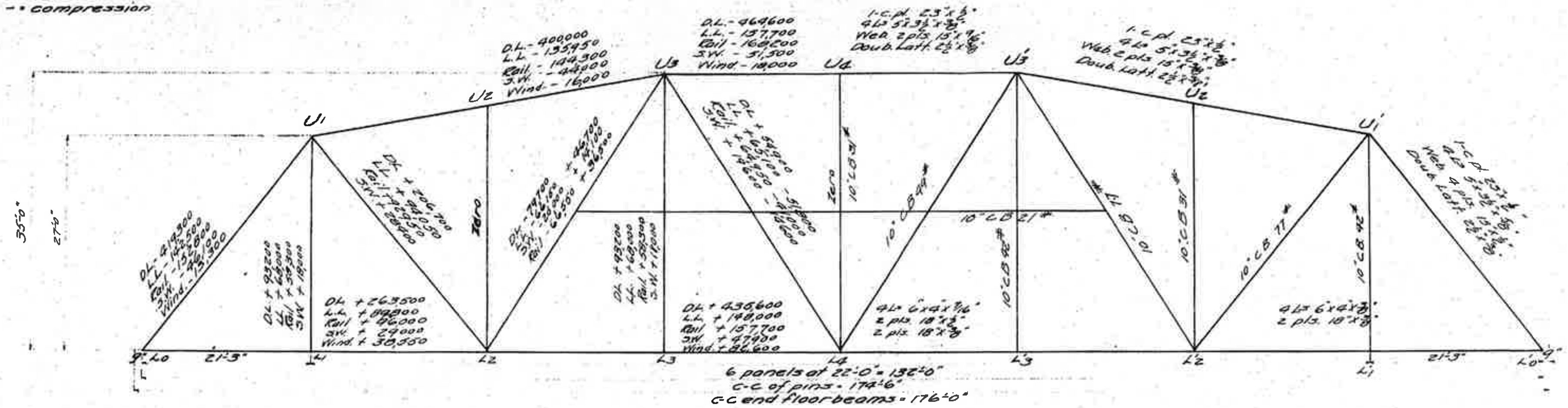
0 1 2 3 4 5 INCHES

14-129

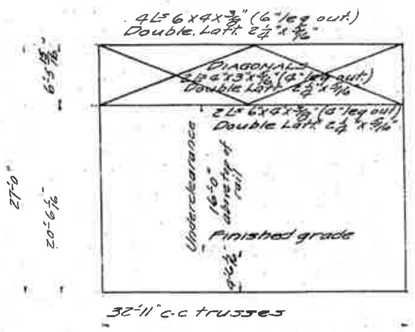
FED. ROAD DIST. NO.	STATE	FED AID PROJ. NO	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	[illegible]	128A	1931	20	33

STRESS DIAGRAM
SPAN NO. 3

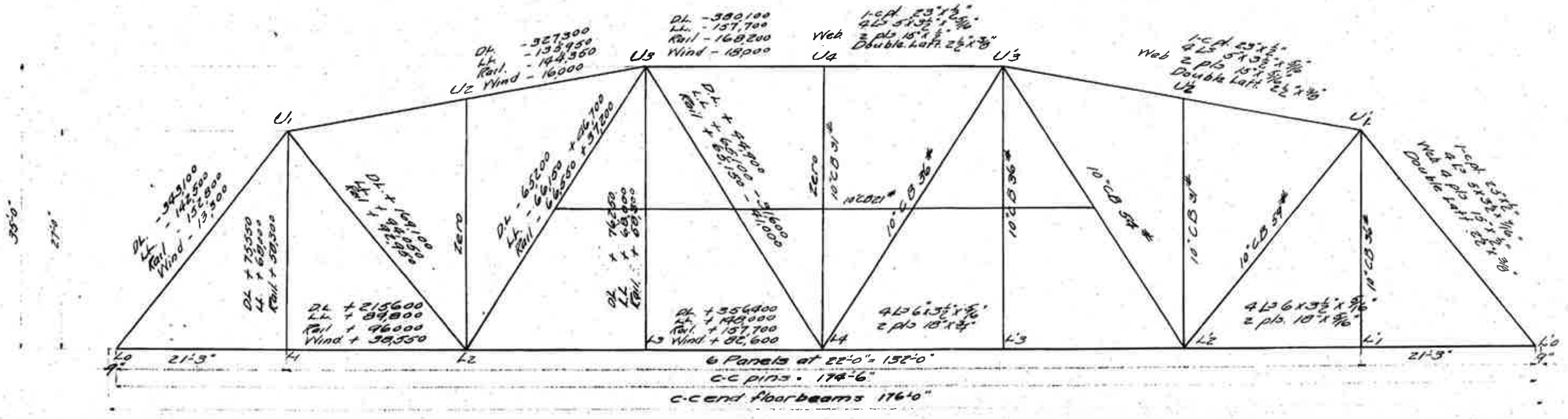
Note-
Live load stresses shown include impact
Highway live is increased 100%
Railway live is increased 50%
Wind is increased 50%
Stresses shown are figured for a unit
stress of 24000 lb. per. sq. inch
+ = tension
- = compression



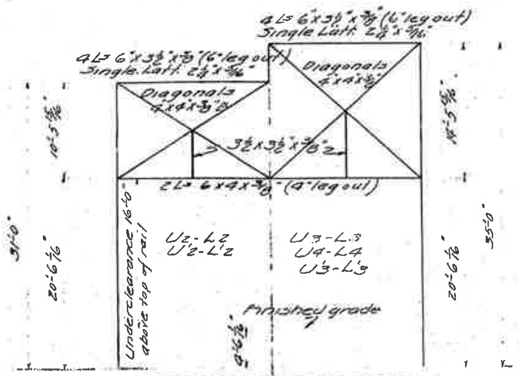
SIDEWALK TRUSS



PORTAL



ROADWAY TRUSS

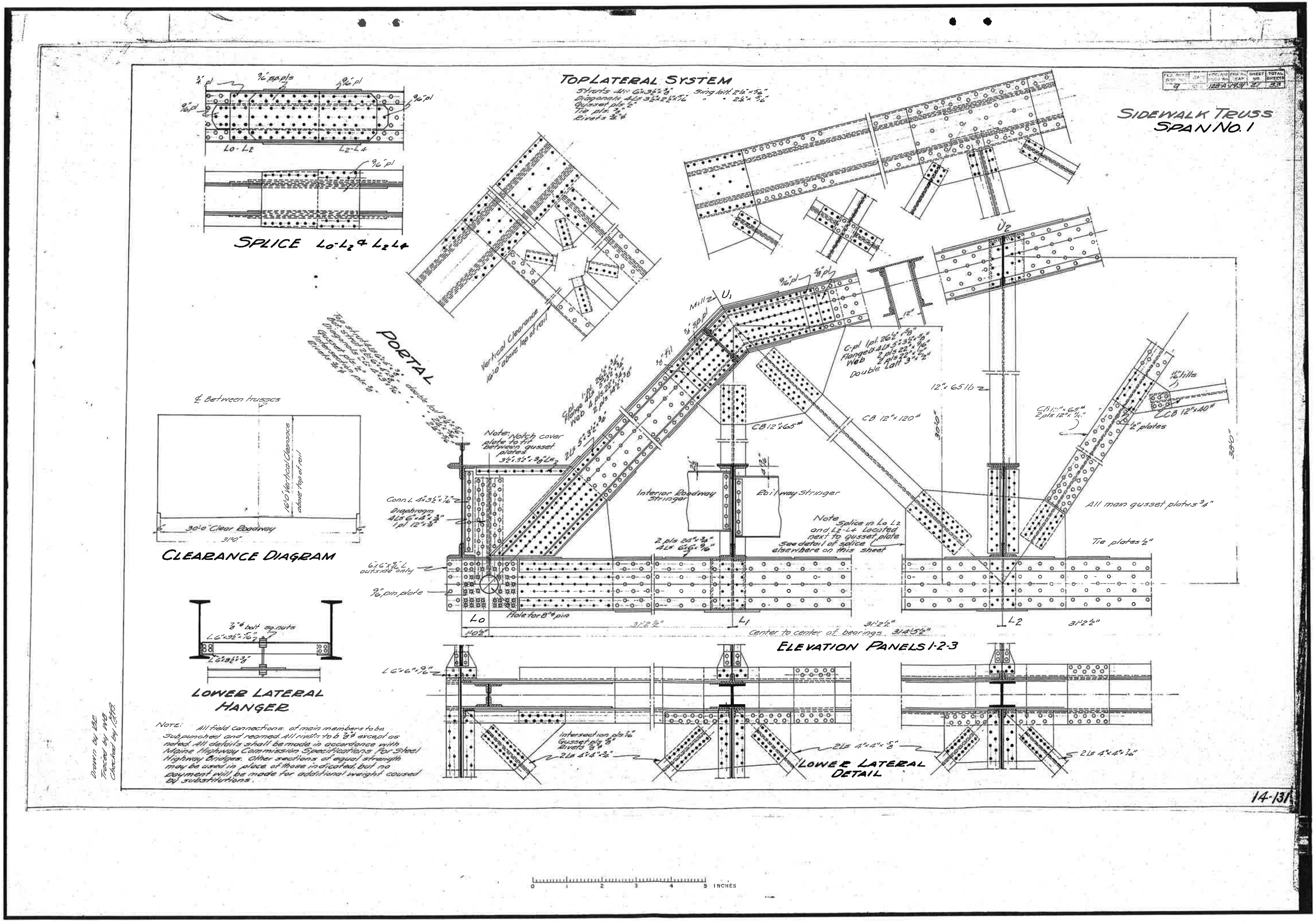


SWAY FRAMES

Designed by Burg...
Traced by J.B. & V.L.
Checked by V.L.

14-130

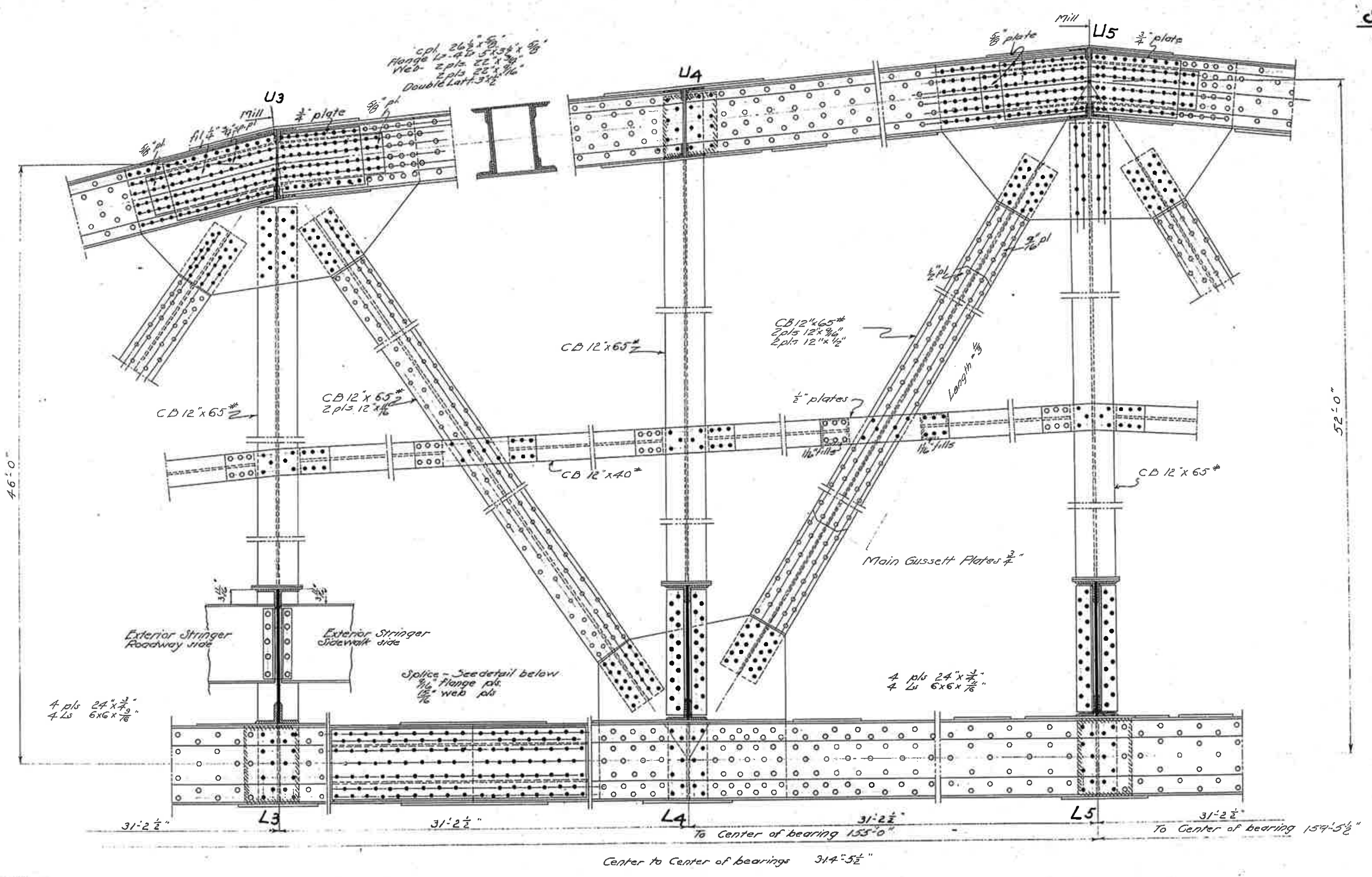
0 1 2 3 4 5 INCHES



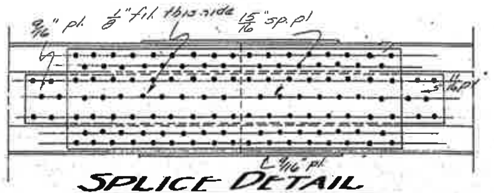
NOTE: All field connections of main members to be Subpunched and reamed. All rivets to b ⅞"⌀ except as noted. All details shall be made in accordance with Maine Highway Commission Specifications for Steel Highway Bridges. Other sections of equal strength may be used in place of those indicated, but no payment will be made for additional weight caused by substitutions.

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	(2FA)	1931	22	33

SIDEWALK TRUSS
SPAN No-1



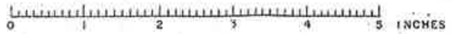
PANELS NO 3-4-5



SPLICE DETAIL

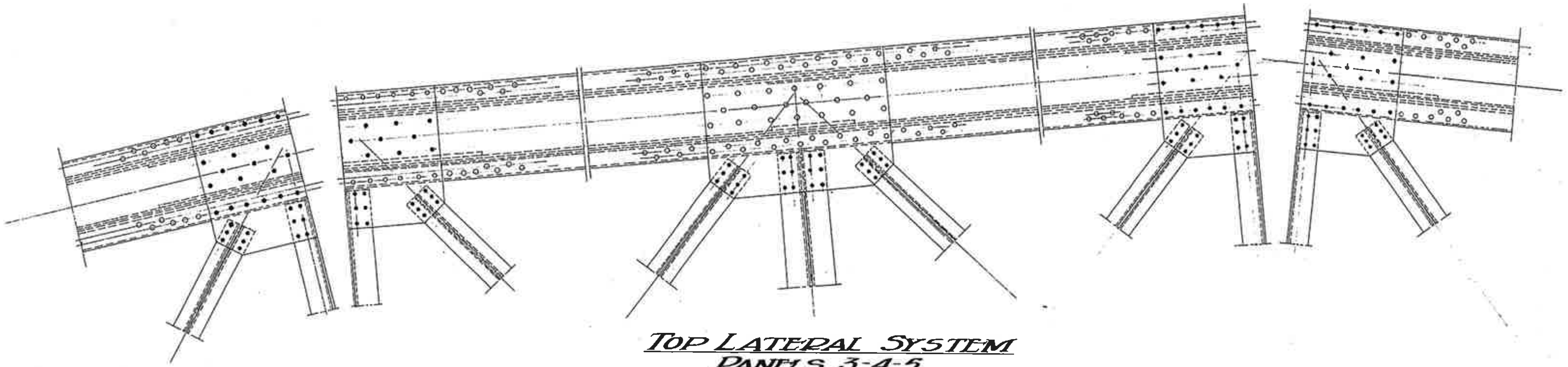
Design ~ V.D.E.
Traced ~ Hamilton
Checked ~ G.T.D.

14-132



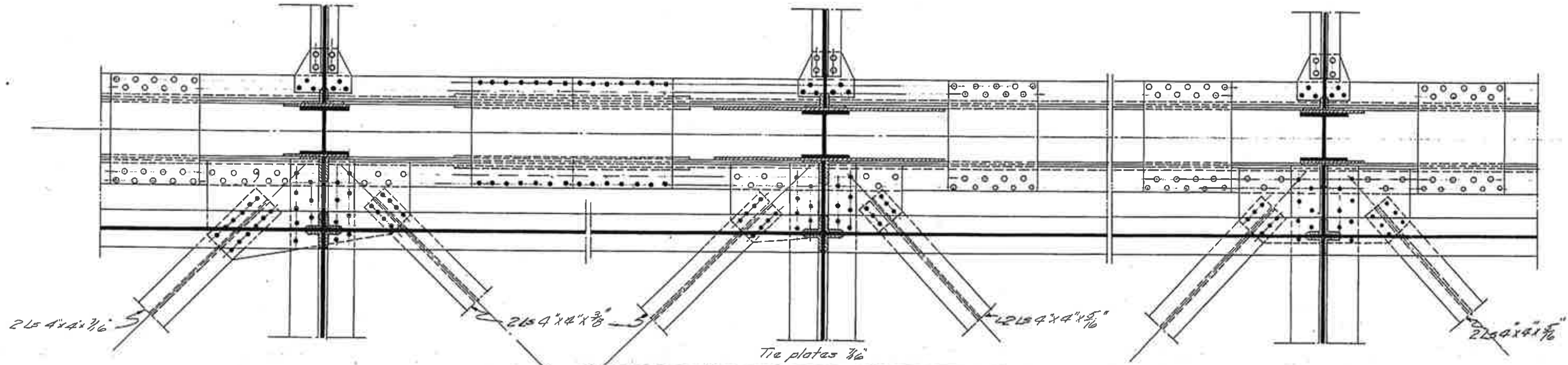
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	12PA	1931	23	33

SIDEWALK TRUSS
SPAN No.1



TOP LATERAL SYSTEM
PANELS 3-4-5

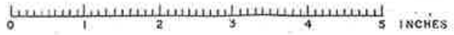
Struts - 4 Ls 6"x3½"x 3/8" Single Latt. 2¼"x 5/16"
Diagonals - 4 Ls 3½"x2½"x 5/16" Single Latt 2¼"x 5/16"
Gussett Plates ½"
Tie plates 3/8"
Rivets ¾"⌀



LOWER LATERAL SYSTEM
PANELS 3-4-5

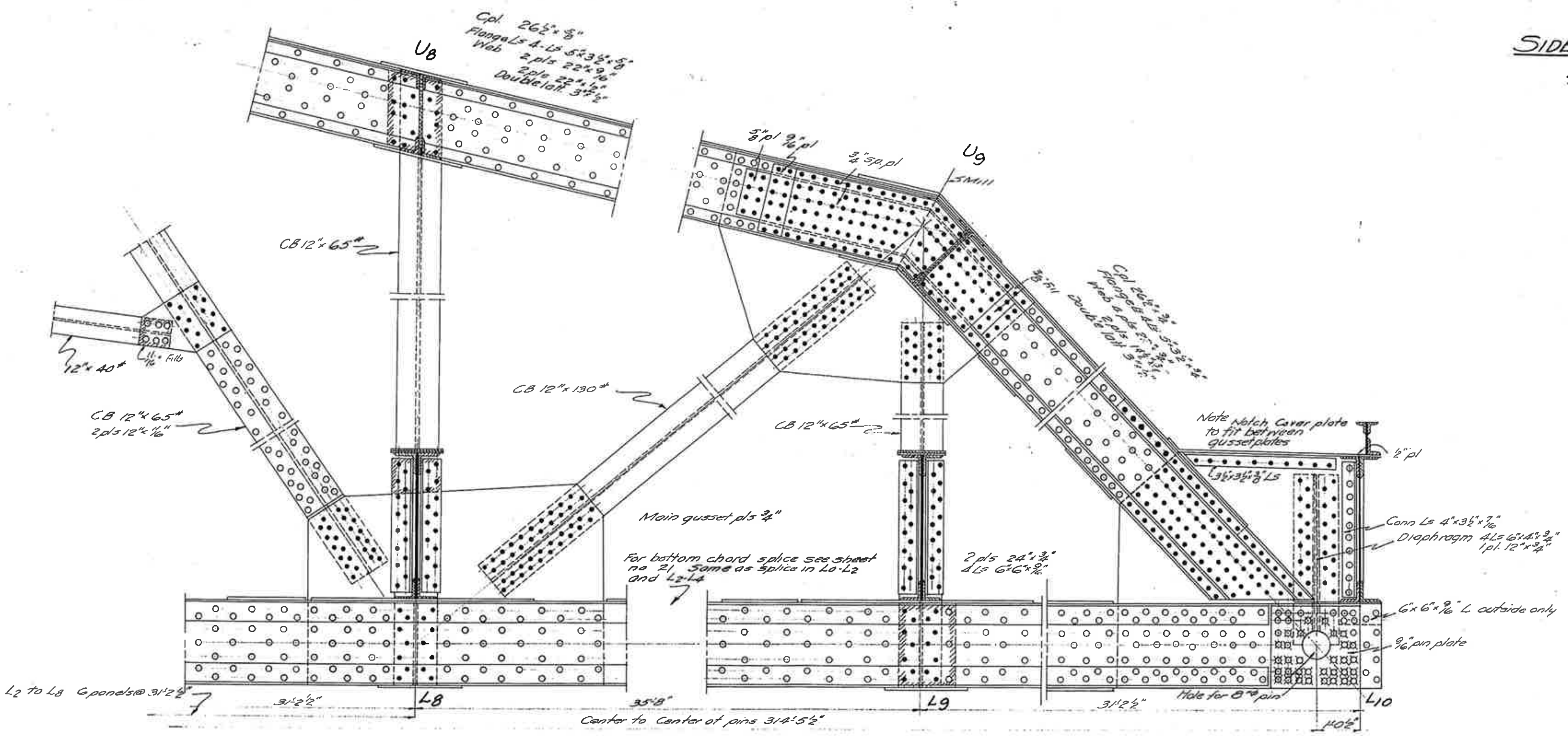
Detailed by KBE
Traced by WHG
✓ Bragdon

14-13

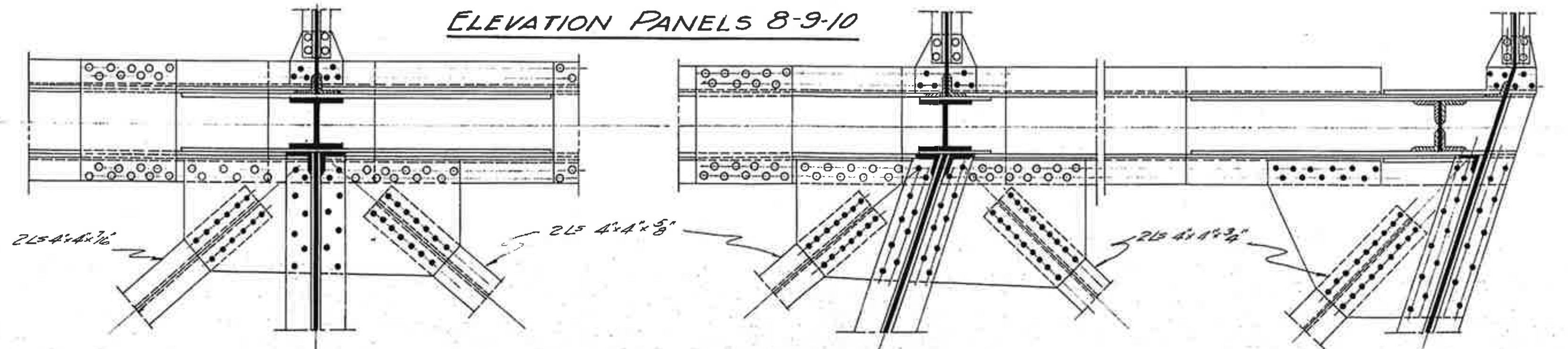


FED. ROAD DIST. NO	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	1264	1931	24	33

SIDEWALK TRUSS
SPAN No. 1



ELEVATION PANELS 8-9-10

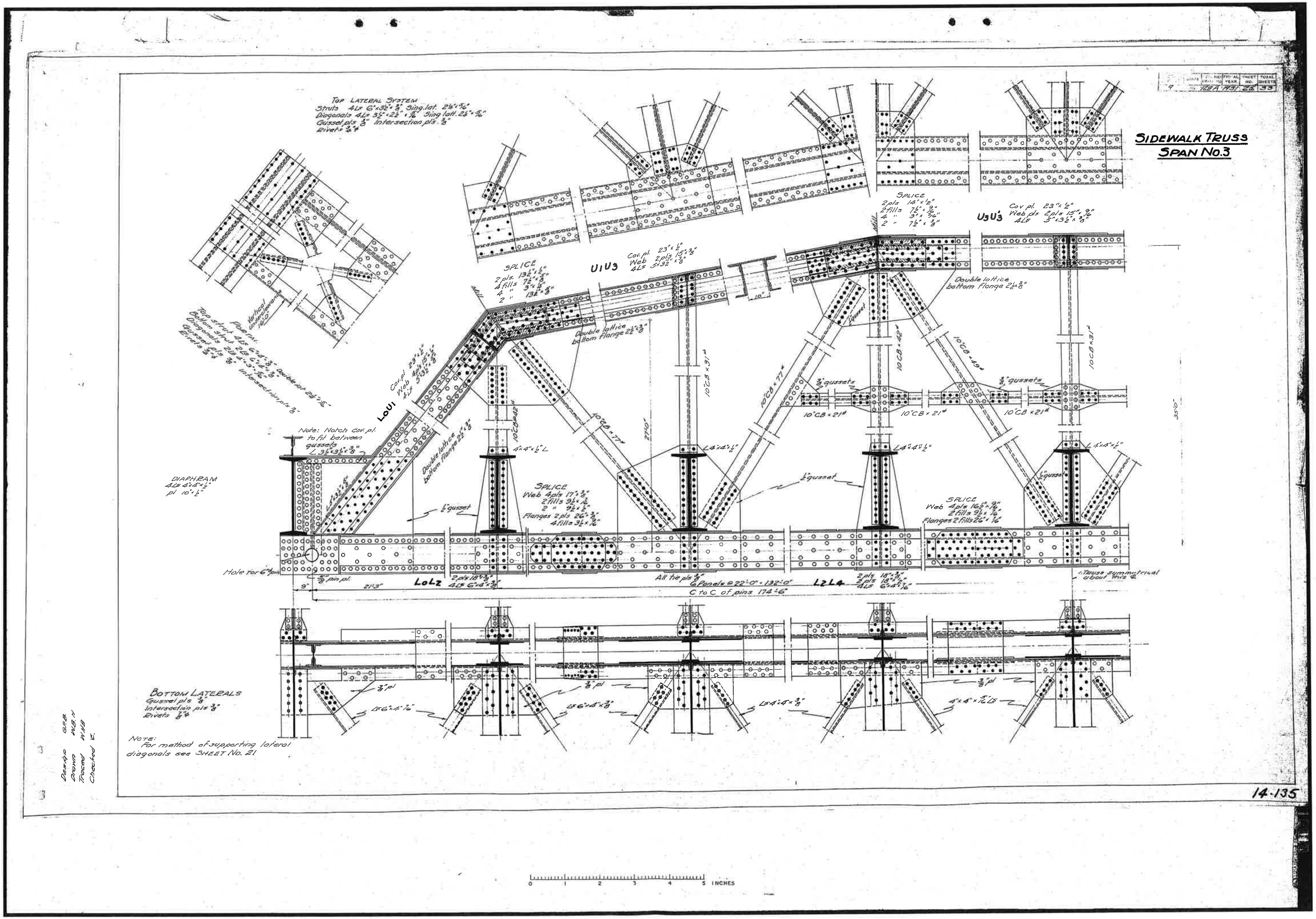


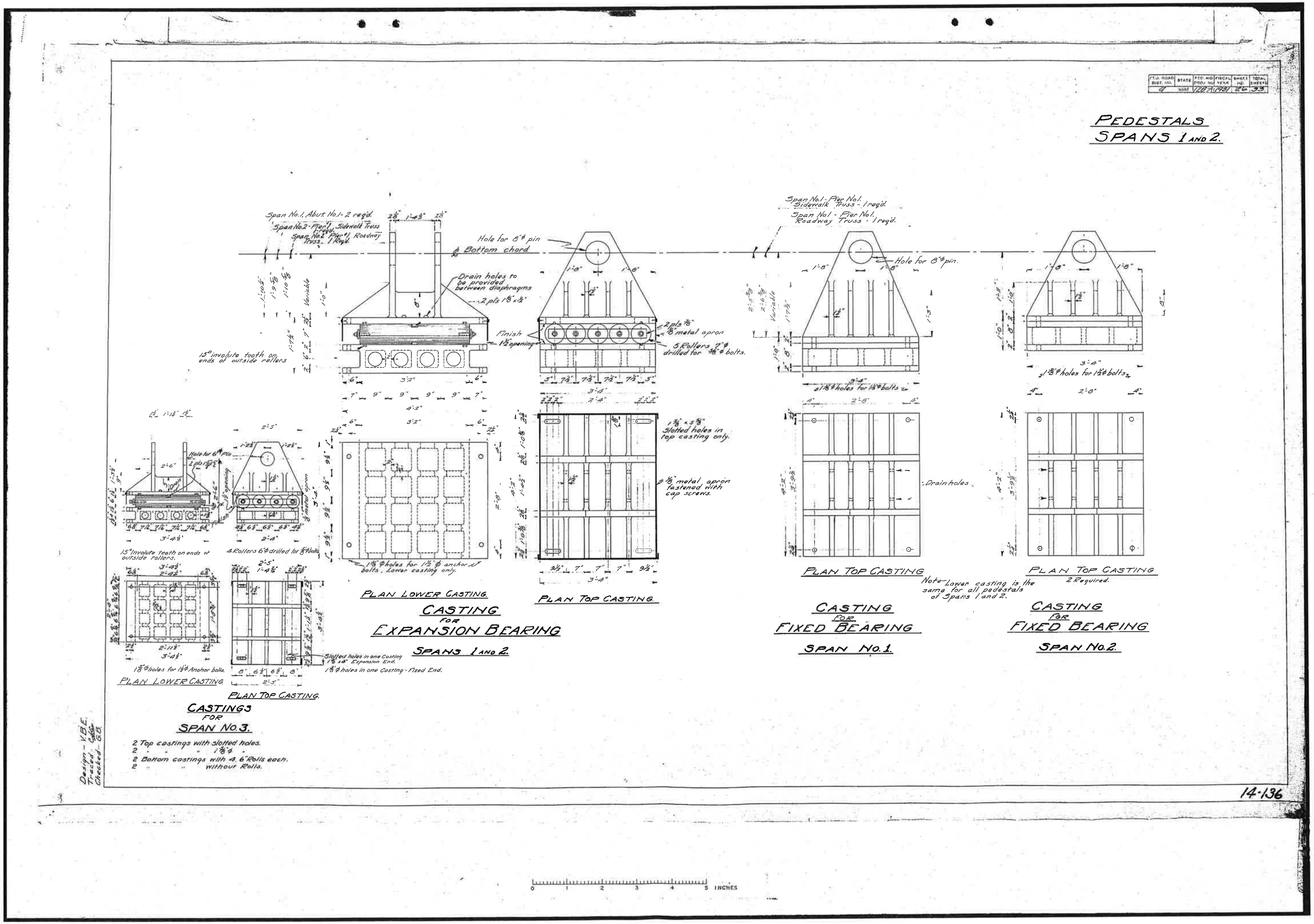
LOWER LATERAL DETAIL

Drawn by WBE
Traced by MB
Checked by G.D.

14-134

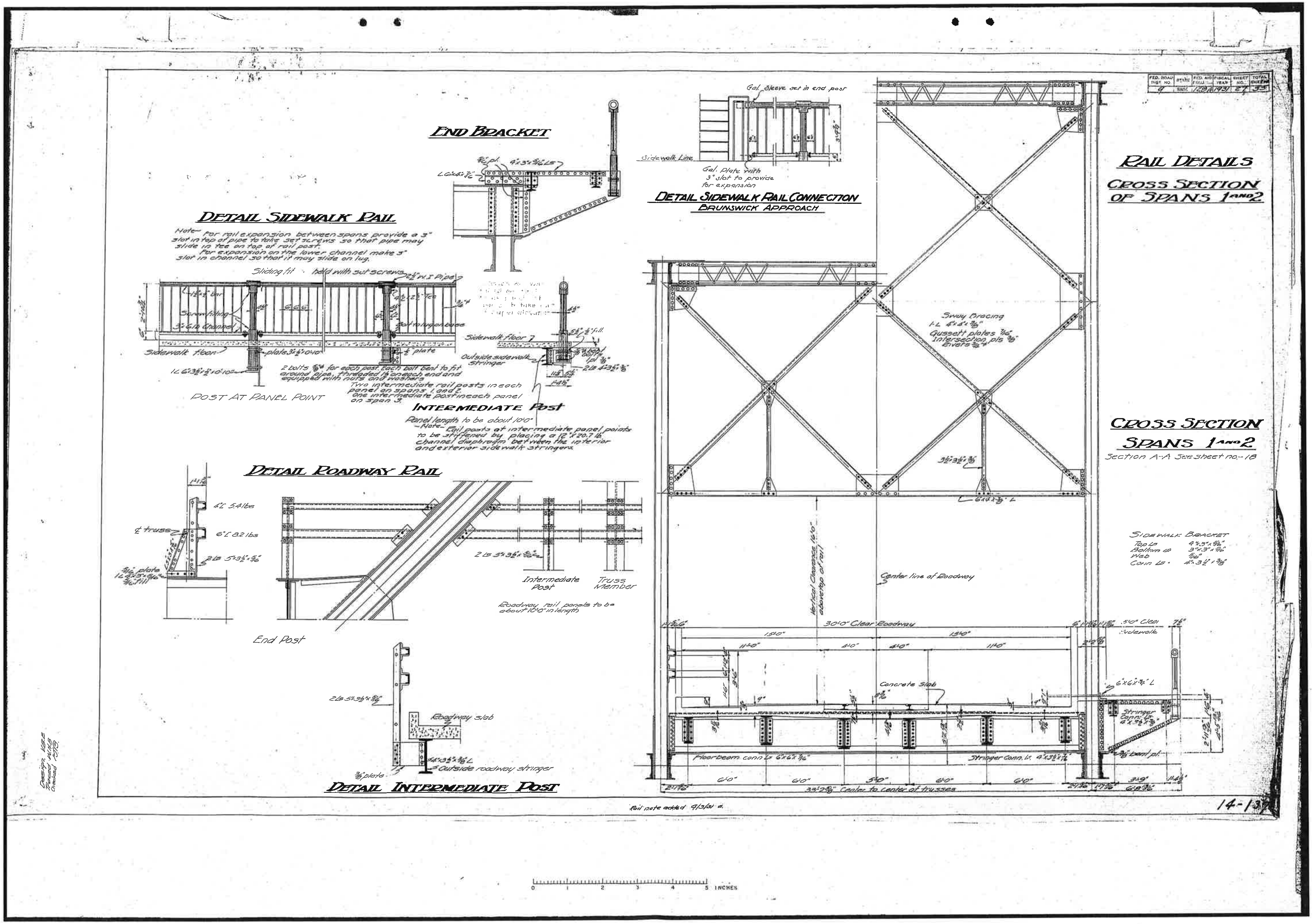
0 1 2 3 4 5 INCHES


SIDEWALK TRUSS
SPAN No.3
9
128A 1931 26 33
TOP LATERAL SYSTEM
Struts 4Ls 6"×3½"×3/8" Sing.lat. 2¼"×5/16"
Diagonals 4Ls 3½"×2½"×5/16" Sing latt. 2¼"×5/16"
Gusset pls 3/8" Intersection pls 3/8"
Rivets ¾"φ
Portal
Top strut 4Ls 6"×4"×3/8" Double lat 2¼"×5/16"
Bottom strut 2Ls 6"×4"×3/8"
Diagonals 2Ls 4"×3"×5/16"
Gusset pls 3/8" Intersection pls 3/8"
Rivets ¾"φ
Vertical underclearance 16'0"
SPLICE
2 pls. 13½"×5/8"
4 fills 7½"×3/8"
4 " 3"×¼"
2 " 13½"×5/8"
U1U3
Cov.pl. 23"×½"
Web 2pls 15"×3/8"
4Ls 5"×3½"×3/8"
Double lattice bottom flange 2½"×3/8"
SPLICE
2pls 14"×½"
2fills 7½"×9/16"
4 " 3"×9/16"
2 " 7½"×3/8"
U3U'3
Cov pl. 23"×½"
Web pls 2pls 15"×9/16"
4Ls 5"×3½"×3/8"
Double lattice bottom flange 2½"×3/8"
gusset
L0U1
Cov.pl 23"×½"
Web 4pls 15"×3/8"
4Ls 5"×3½"×3/8"
Note: Notch cov.pl. to fit between gussets
∠3½"×3½"×3/8"
∠5"×3½"×5/8"
Double lattice bottom flange 2½"×3/8"
DIAPHRAM
4Ls 4"×4"×½"
pl 10"×½"
10"CB×42#
10"CB×77#
10"CB×31#
10"CB×77#
10"CB×42#
10"CB×49#
10"CB×31#
27'10"
4"×4"×½"L
L4"×4"×½"
L4"×4"×½"
4"×4"×½"
3/8" gussets
3/8" gussets
10"CB×21#
10"CB×21#
10"CB×21#
35'0"
½" gusset
½" gusset
½" gusset
½" gusset
SPLICE
Web 4pls 17"×3/8"
2 fills 9½×¼
2 " 9½×½
Flanges 2 pls 26"×3/8"
4 fills 3½×7/16"
SPLICE
Web 4pls 16½"×9/16"
2 fills 9½×7/16"
Flanges 2 fills 26"×7/16"
Hole for 6"φ pin
5/8" pin pl.
9"
21'3"
L0L2
2 pls 18"×3/8"
4Ls 6"×4"×3/8"
All tie pls 3/8"
6 Panels @ 22'-0" = 132'-0"
L2L4
2 pls 18"×3/8"
2 pls 18"×½"
4Ls 6"×4"×7/16"
Truss symmetrical about this ℄
C to C of pins 174'-6"
3/8" pl
3/8" pl
3/8" pl
Ls 6"×4"×7/16"
Ls 6"×4"×3/8"
Ls 4"×4"×3/8"
4"×4"×7/16" Ls
BOTTOM LATERALS
Gusset pls 3/8"
Intersection pls 3/8"
Rivets 7/8"φ
NOTE:
For method of supporting lateral diagonals see SHEET No. 21
Design G.F.B.
Drawn W.B.H.
Traced W.H.B.
Checked V.
14-135
0 1 2 3 4 5 INCHES


FED. ROAD DIST. NO.
STATE
FED. AID PROJ. NO.
FISCAL YEAR
SHEET NO.
TOTAL SHEETS
4
MAINE
128 A
1931
26
33
PEDESTALS
SPANS 1 AND 2.
Span No.1, Abut. No.1 - 2 req'd.
Span No.2 - Pier 1, Sidewalk Truss 1 req'd.
Span No.2 Pier 1, Roadway Truss, 1 Req'd.
Hole for 8" ϕ pin
℄ Bottom chord
Drain holes to be provided between diaphragms
2 pls 1 3/8" x 1/2"
Variable
Finish
1 1/2" opening
15° involute tooth on ends of outside rollers
2 pls 3/8" 1/8" metal apron
5 Rollers 7" ϕ drilled for 7/8" ϕ bolts.
1 7/8" x 3 3/8" Slotted holes in top casting only.
1/8" metal apron fastened with cap screws.
1 7/8" ϕ holes for 1 1/2" ϕ anchor bolts. Lower casting only.
PLAN LOWER CASTING.
PLAN TOP CASTING.
CASTING FOR EXPANSION BEARING
SPANS 1 AND 2.
Span No.1 - Pier No.1 Sidewalk Truss - 1 req'd.
Span No.1 - Pier No.1 Roadway Truss - 1 req'd.
Hole for 8" ϕ pin.
1 7/8" ϕ holes for 1 1/2" ϕ bolts
Drain holes
PLAN TOP CASTING
Note - Lower casting is the same for all pedestals of Spans 1 and 2.
CASTING FOR FIXED BEARING
SPAN NO.1.
PLAN TOP CASTING
2 Required.
CASTING FOR FIXED BEARING
SPAN NO.2.
Hole for 6" ϕ Pin
1 pl 8" x 3/8"
1/8" metal apron
15° involute teeth on ends of outside rollers.
4 Rollers 6" ϕ drilled for 5/8" ϕ bolts.
1 5/8" ϕ holes for 1 1/2" ϕ Anchor bolts.
PLAN LOWER CASTING
Slotted holes in one Casting 1 5/8" x 8" Expansion End.
1 5/8" ϕ holes in one Casting - Fixed End.
PLAN TOP CASTING.
CASTINGS FOR SPAN NO.3.
2 Top castings with slotted holes.
2 " " " 1 5/8" ϕ "
2 Bottom castings with 4, 6" Rolls each.
2 " " without Rolls.
Design - V.B.E.
Traced -
Checked - G.B.
14-136
0 1 2 3 4 5 INCHES

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	128A	1931	27	33

RAIL DETAILS
CROSS SECTION OF SPANS 1 AND 2



Note— For rail expansion between spans provide a 3" slot in top of pipe to take set screws so that pipe may slide in tee on top of rail post.
For expansion on the lower channel make 3" slot in channel so that it may slide on lug.

2 bolts 5/8" for each post. Each bolt bent to fit around pipe, threaded 1 1/2" on each end and equipped with nuts and washers.

Two intermediate rail posts in each panel on spans 1 and 2. One intermediate post in each panel on span 3.

Panel length to be about 10'-0"

—Note— Rail posts at intermediate panel points to be stiffened by placing a 12"x20.7 lb. channel diaphragm between the interior and exterior sidewalk stringers.

CROSS SECTION SPANS 1 AND 2

Section A-A See sheet no.-18

SIDEWALK BRACKET

Top Ls	4"x3"x5/16"
Bottom Ls	3"x3"x5/16"
Web	5/16"
Conn Ls	4"x3 1/2"x3/8"

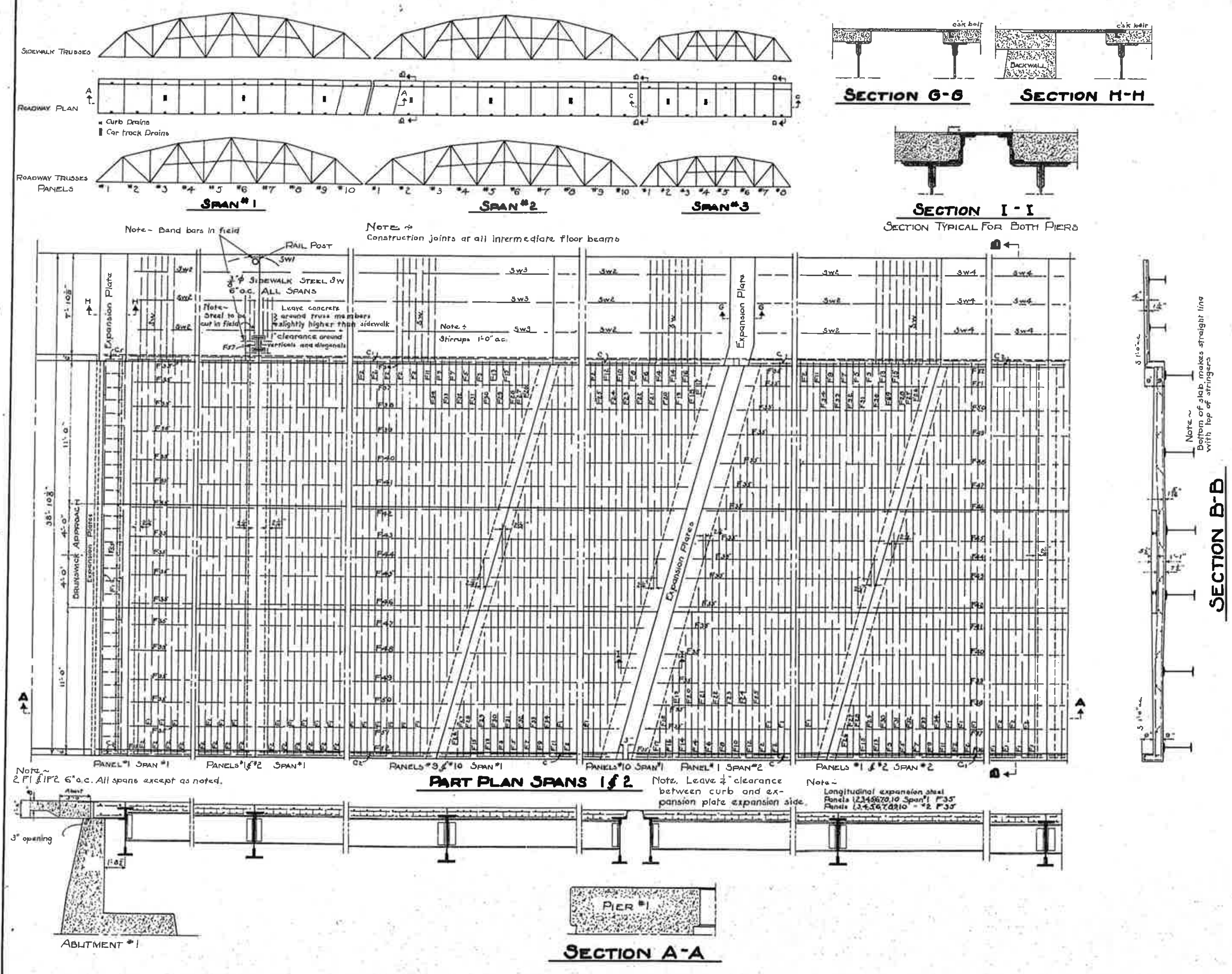
Design. V.B.E
Traced W.H.B
Checked R.T.B.

Rail note added 9/3/31 a.

14-137

FED. ROAD DIST. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MN	128-A	1931	28	35

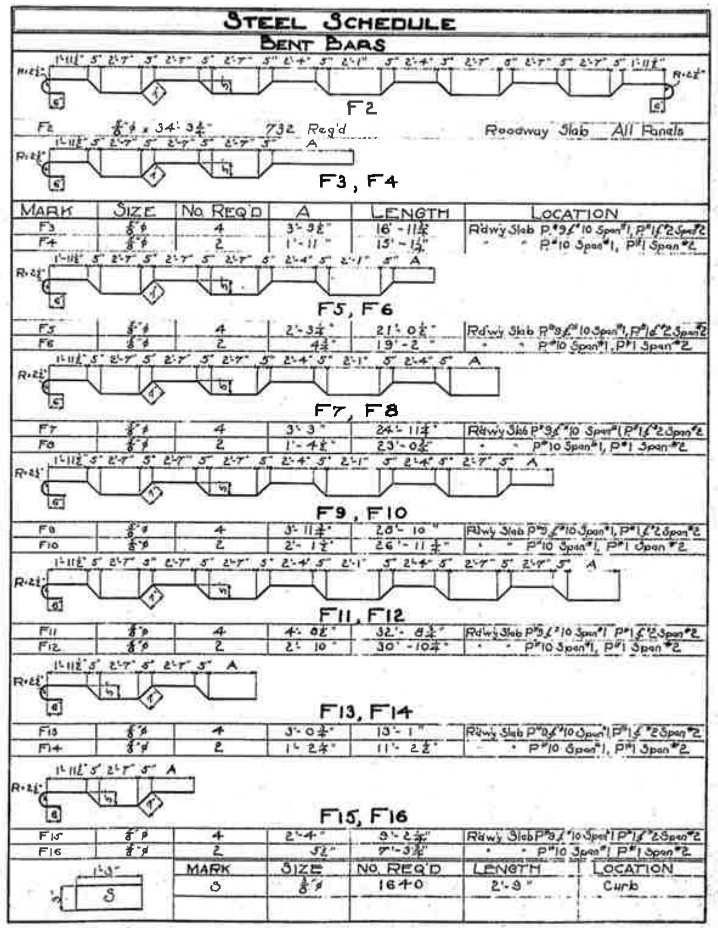
FLOOR PLAN SPANS 1 & 2



Plan - Hamilton
Traced - "
Checked - H.G.H.

STEEL SCHEDULE

BENT BARS



MARK	SIZE	NO. REQ'D	A	LENGTH	LOCATION
F2	⅝"ϕ x 34'-3¼"	732 Req'd			Roadway Slab All Panels
F3	⅝"ϕ	4	3'-3¾"	16'-11¾"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F4	⅝"ϕ	2	1'-11"	15'-1¾"	" " P.#10 Span #1, P.#1 Span #2
F5	⅝"ϕ	4	2'-3¾"	21'-0¾"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F6	⅝"ϕ	2	4¾"	19'-2"	" " P.#10 Span #1, P.#1 Span #2
F7	⅝"ϕ	4	3'-3"	24'-11¾"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F8	⅝"ϕ	2	1'-4¾"	23'-0¾"	" " P.#10 Span #1, P.#1 Span #2
F9	⅝"ϕ	4	3'-11¾"	28'-10"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F10	⅝"ϕ	2	2'-1½"	26'-11¾"	" " P.#10 Span #1, P.#1 Span #2
F11	⅝"ϕ	4	4'-8¾"	32'-8¾"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F12	⅝"ϕ	2	2'-10"	30'-10¾"	" " P.#10 Span #1, P.#1 Span #2
F13	⅝"ϕ	4	3'-0¾"	13'-1"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F14	⅝"ϕ	2	1'-2¾"	11'-2½"	" " P.#10 Span #1, P.#1 Span #2
F15	⅝"ϕ	4	2'-4"	9'-2¾"	Rd'wy Slab P.#9 & #10 Span #1, P.#1 & #2 Span #2
F16	⅝"ϕ	2	5½"	7'-3¾"	" " P.#10 Span #1, P.#1 Span #2

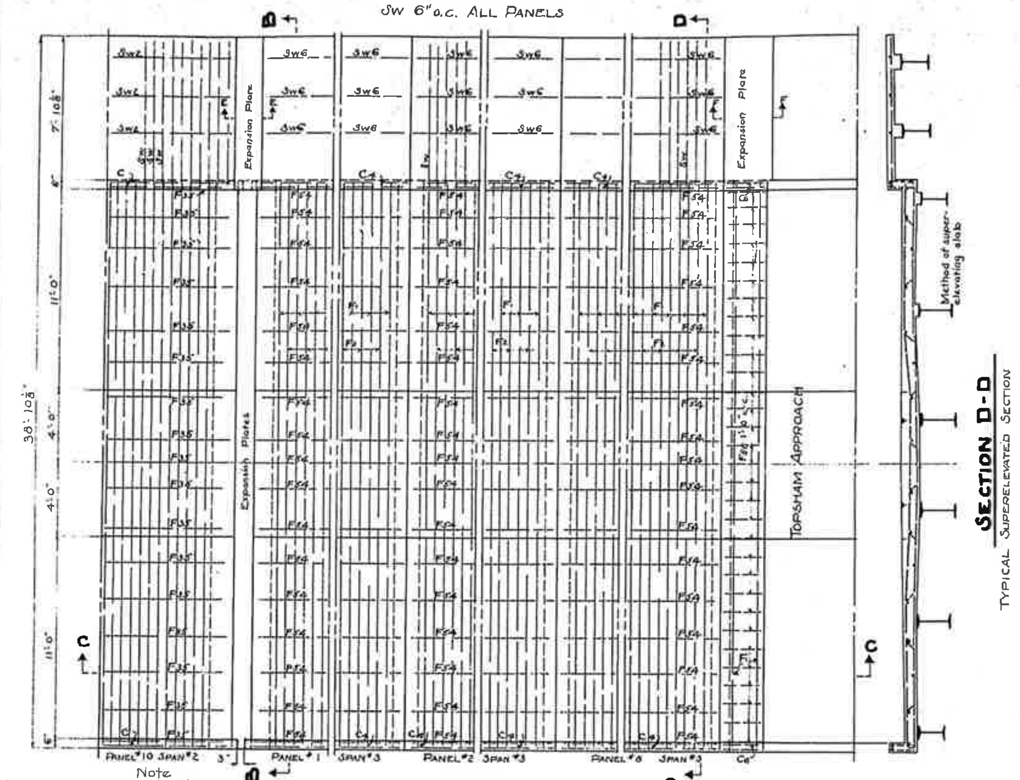
MARK	SIZE	NO. REQ'D	LENGTH	LOCATION
S	⅜"ϕ	1640	2'-9"	Curb

14-138

0 1 2 3 4 5 INCHES

FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	[illegible]	128-A	1931	29	33

FLOOR PLAN SPANS 2 & 3

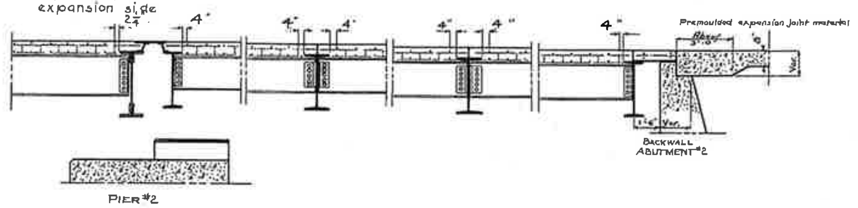


Note ~
2 F1 & F2 6" o.c.

Note
Leave 1/4" clearance between curb and expansion plate on expansion side

PART PLAN SPANS #2 & #3

Note ~
Longitudinal expansion steel F54 Panels #1 to #8 incl.



SECTION C-C

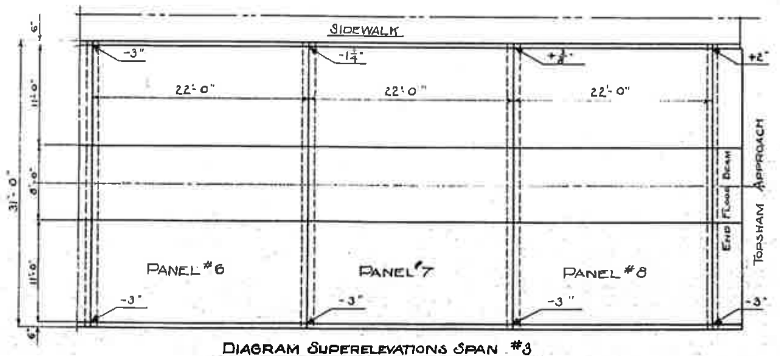
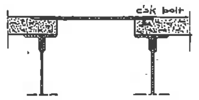
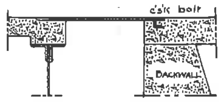


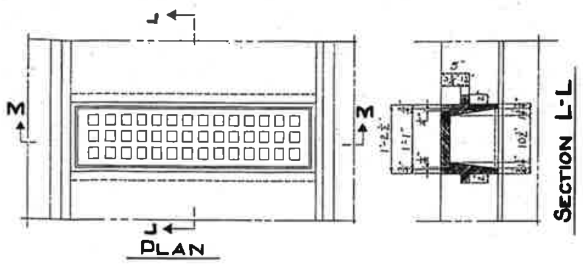
DIAGRAM SUPERELEVATIONS SPAN #3



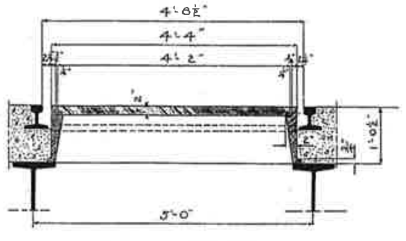
SECTION E-E



SECTION F-F



PLAN



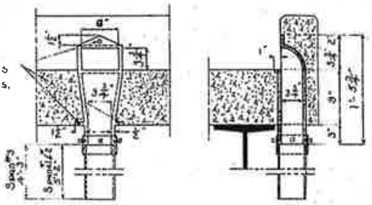
SECTION M-M

Note ~
Grate 12 1/2" x 50" x 2" 42 holes 2 1/2" x 2"
Frame - casting as shown.

Slab steel to be cut to allow placing of drains.

DRAIN DETAILS FOR RAILWAY

8 Required



1/2" cored holes for form bolts.

DRAIN DETAILS

49 Required
49 castings as shown
49 couplings for 4 1/2" dia. pipe.
One-half coupling threaded.
Four threaded holes in each coupling for set screws.
Four holes to be drilled 1/4" deep in each casting to hold ends of set screws.
196 - set screws 1/2" x 3/4"
40 - 4 1/2" x 5'-2" std pipes } threaded one end.
9 - 4 1/2" x 4'-3" " " } threaded one end.

STEEL SCHEDULE

STRAIGHT BARS

MARK	SIZE	No REQ'D	LENGTH	LOCATION	MARK	SIZE	No REQ'D	LENGTH	LOCATION
F1	5/8"⌀	1492	30'-9"	Roadway Slab All Panels	F46	1/2"⌀	2	30'-0"	[illegible]
F17	5/8"⌀	4	2'-3 1/2"	Panel #10 Span #1 & 2	F47	1/2"⌀	2	29'-6 3/4"	"
F18	5/8"⌀	4	4'-2 1/4"	"	F48	1/2"⌀	2	29'-1"	"
F19	5/8"⌀	4	7'-11"	"	F49	1/2"⌀	2	28'-4 1/2"	"
F20	5/8"⌀	4	11'-7"	"	F50	1/2"⌀	2	27'-11"	"
F21	5/8"⌀	4	15'-4 1/2"	"	F51	1/2"⌀	2	27'-5 1/2"	"
F22	5/8"⌀	4	19'-1 1/4"	"	F52	1/2"⌀	2	26'-11"	"
F23	5/8"⌀	4	22'-10"	"					
F24	5/8"⌀	4	26'-6 3/4"	"	F54	1/2"⌀	136	21'-8"	All Panels Span #3
F25	5/8"⌀	4	30'-3 1/2"	"	C	3/8"⌀	72	31'-0"	[illegible]
F26	5/8"⌀	8	2'-3 1/2"	[illegible]	C1	3/8"⌀	4	35'-4"	[illegible]
F27	5/8"⌀	8	4'-2"	"	C2	3/8"⌀	4	26'-5"	"
F28	5/8"⌀	8	6'-0 1/2"	"					
F29	5/8"⌀	8	9'-3 3/4"	"	C4	3/8"⌀	32	21'-8"	Span #3
F30	5/8"⌀	8	13'-6"	"	C5	3/8"⌀	4	2'-0"	Curb as shown
F31	5/8"⌀	8	17'-2 3/4"	"	C6	3/8"⌀	4	2'-0"	" " "
F32	5/8"⌀	8	20'-11 1/2"	"	Sw	3/8"⌀	1632	7'-6"	All Panels
F33	5/8"⌀	8	24'-8 1/4"	"	Sw1	3/8"⌀	79	3'-0"	" All Panels
F34	5/8"⌀	8	28'-5"	"	Sw2	3/8"⌀	54	31'-0"	[illegible]
F35	1/2"⌀	306	31'-0"	[illegible]	Sw3	3/8"⌀	3	35'-4"	Panels #3 Span #1
F36	1/2"⌀	2	35'-1"	[illegible]	Sw4	3/8"⌀	3	26'-5"	Panels #2 Span #2
F37	1/2"⌀	2	34'-6 1/2"	"					
F38	1/2"⌀	2	34'-1"	"	Sw6	3/8"⌀	24	21'-8"	All panels Span #3
F39	1/2"⌀	2	33'-7 1/2"	"	F55	3/8"⌀	62	2'-0"	As shown Abut. 1
F40	1/2"⌀	2	32'-11"	"	F56	3/8"⌀	31	2'-0"	" " " 2
F41	1/2"⌀	2	32'-5 1/2"	"	F57	3/8"⌀	106	3'-0"	[illegible]
F42	1/2"⌀	2	32'-0"	"					
F43	1/2"⌀	2	31'-4 1/2"	"					
F44	1/2"⌀	2	31'-0"	"					
F45	1/2"⌀	2	30'-7 1/2"	"					

Note ~
All steel to be plain round bars of structural grade.
All dimensions to center lines of bars.

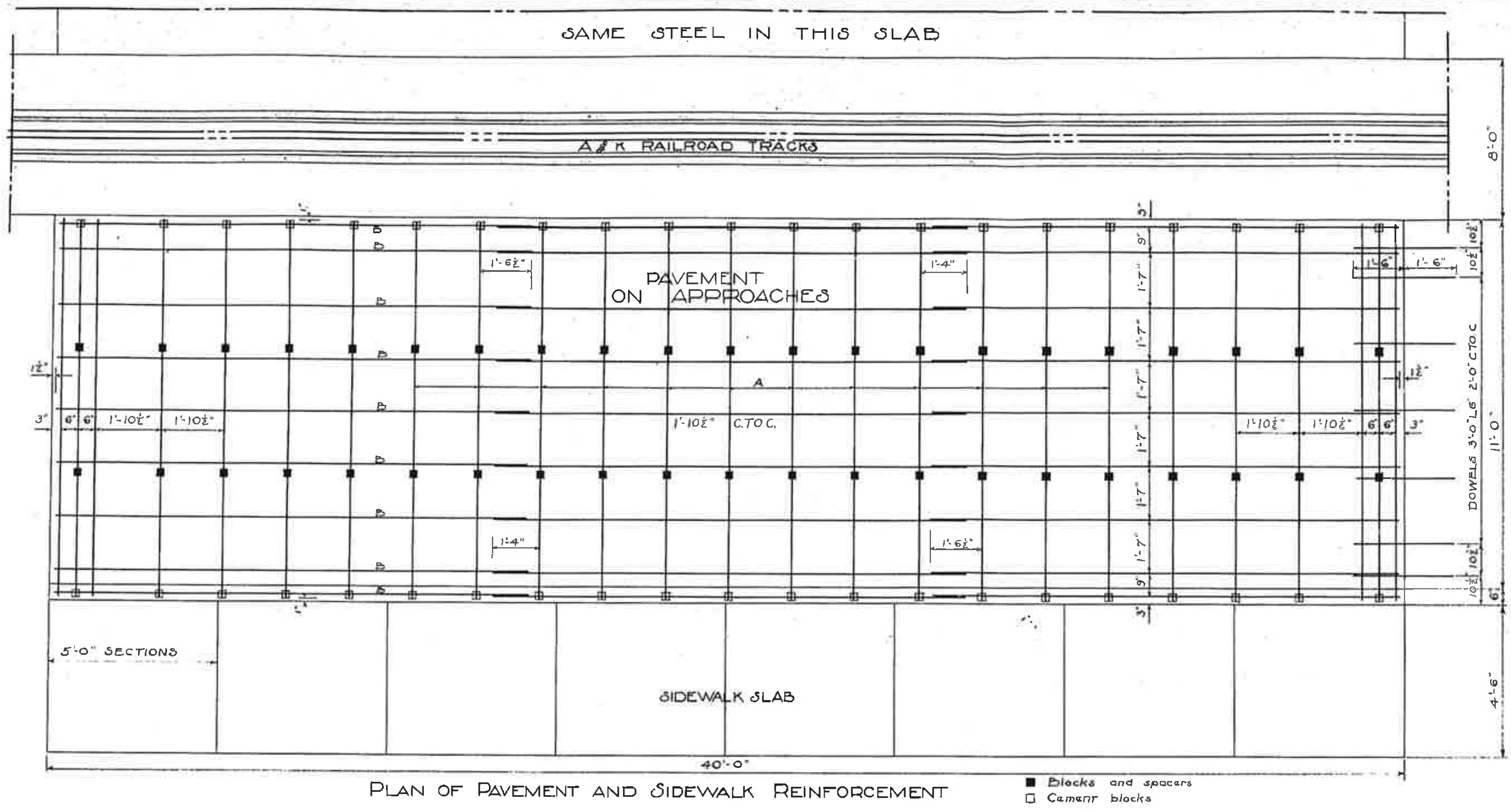
Plan - Hamilton
Traced - "
Checked - H.H.

14-139

0 1 2 3 4 5 INCHES

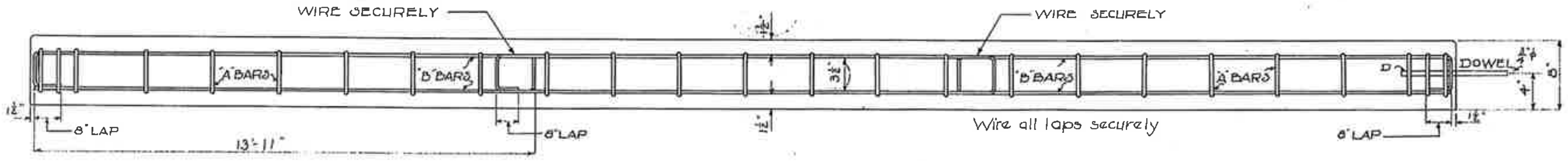
FED. ROAD DIST. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	MAINE	128-A	1931	30	33

PLAN OF PAVEMENT ON APPROACHES

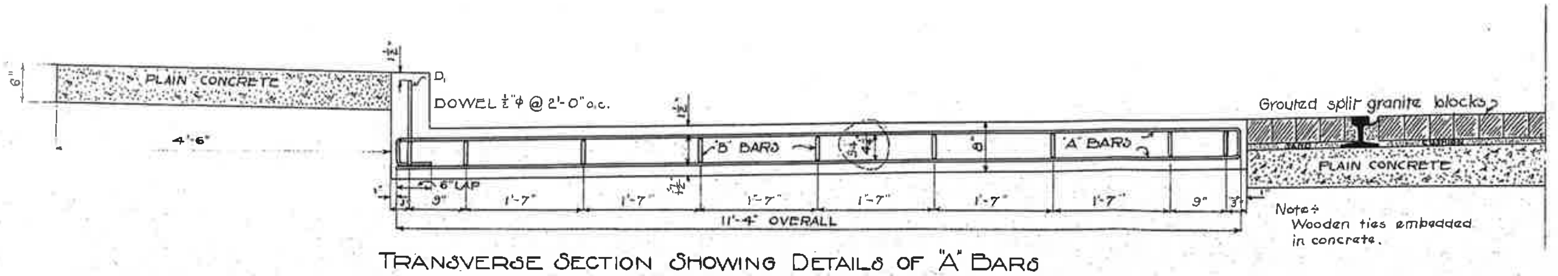


PLAN OF PAVEMENT AND SIDEWALK REINFORCEMENT

■ Blocks and spacers
□ Cement blocks



LONGITUDINAL SECTION SHOWING DETAIL OF "B" BARS



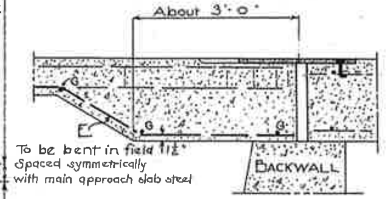
Note:-
Wooden ties embedded in concrete.

TRANSVERSE SECTION SHOWING DETAILS OF "A" BARS

REINFORCING DATA

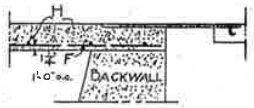
MARK	SIZE	NO REQD	LENGTH	LOCATION
A	⅝"φ	25	23'-10½"	Rdw'y Slab
B	⅝"φ	27	29'-0⅝"	" "
D	¾"φ	7	3'-0"	" "
D,	½"φ	20	1'-2"	Curbs
E	¾"φ	28	6'-0"	App. slabs at Abuts Rdw'y
F	½"φ	14	3'-0"	" sdw'k
G	½"φ	12	11'-3"	" Rdw'y
H	½"φ	6	6'-0"	" sdw'k

Note:-
All steel to be plain rounds of structural grade
Each 11'x6" x 40'-0" slab contains a minimum of 42 cement blocks & 42 blocks and spacers to support steel. See detail.
Each 11'-6"x40'-0" slab and curb contains 11.97 cu.yds. concrete.
8-5'-0"x4'-6" sidewalk slabs contain 3.33 cu.yds concrete.
Each 11'-6"x40'-0" slab and curb contains 566.93# steel
Dowels "D" to be one-half encased in metal tube.



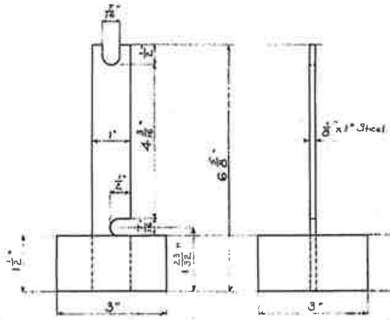
To be bent in field 1½"
Spaced symmetrically with main approach slab steel

Method of reinforcing roadway approach slab at backwall.
Similar at both abutments.



Method of reinforcing sidewalk approach slab at backwall.
Similar at both abutments.

Note:
Sidewalk slab adjacent to bridge on Topsham and Brunswick approaches are similar.



BLOCK AND SPACER

CEMENT BLOCK
3" Square

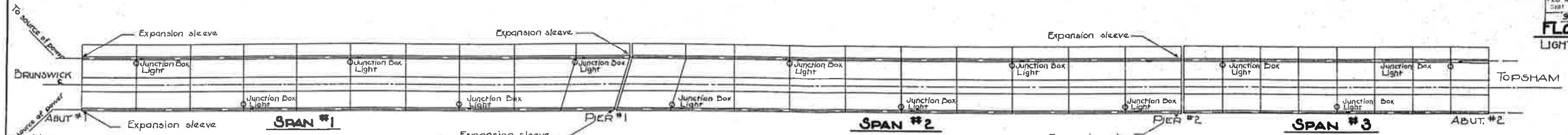
Note:-
Blocks to be made of one part Portland cement and two parts of approved sand and cured under cover for seven days.

Plan - Shamll
Traced - "
Checked - HCH

14-140

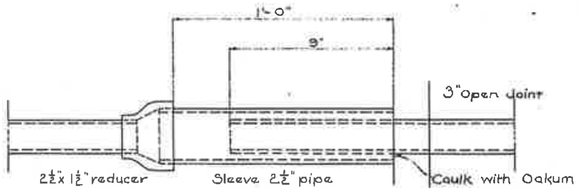
FED. ROAD DIST. NO.	STATE	FED. AID PROJECT NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
9	[illegible]	128-A	1931	32	33

FLOOR PLAN
LIGHTING SYSTEM

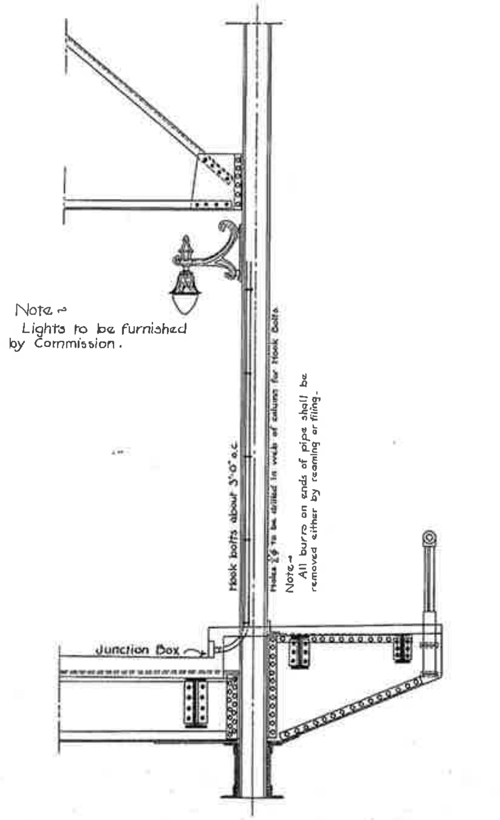


Note ~
Conduits to extend in to curbs of approaches for a few feet.

PLAN
CONDUITS – JUNCTION BOXES – LIGHTS – EXPANSION SLEEVES



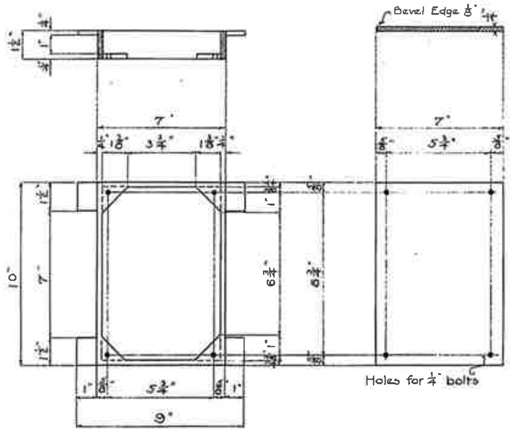
DETAIL OF EXPANSION SLEEVE
8 REQUIRED



Note ~
Lights to be furnished by Commission.

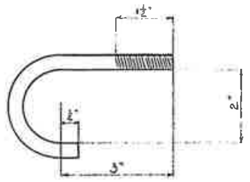
Note ~
All burrs on ends of pipe shall be removed either by reaming or filing.

LOCATION OF LIGHTS – JUNCTION BOXES – CONDUIT



DETAILS OF ZINC JUNCTION BOX CASTING AND COVER
REQUIRED
13 Zinc Junction Box Castings
13 Zinc Covers
52 Brass Bolts ½"x ¼"

Note— Junction boxes to be omitted. For revised lighting details see sheet No. 32A.

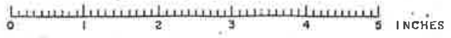


⅜" ∅ HOOK BOLTS
Each bolt to be supplied with nut and check nut.

Note ~
All conduit to be supplied and installed by floor contractor.
Junction boxes will be furnished by the commission
Lights to be furnished by Commission

Plan – Hamilton
Traced – "
Checked – HGH

14-142



APPENDIX E

Hydraulics Data

Environmental Office – Hydrology Section
16 State House Station
Augusta ME 04333-0016
207.557.1052
Charles.Hebson@maine.gov

Maine Department of Transportation

Memo

To: Mark Parlin

From: Charles Hebson

CC:

Date: 24 February 2014

Re: 20467 Brunswick – Frank J Wood Bridge #2016 - Androscoggin River

---

The final recommended design hydrology is summarized in Table 1 and Figure 1 below.

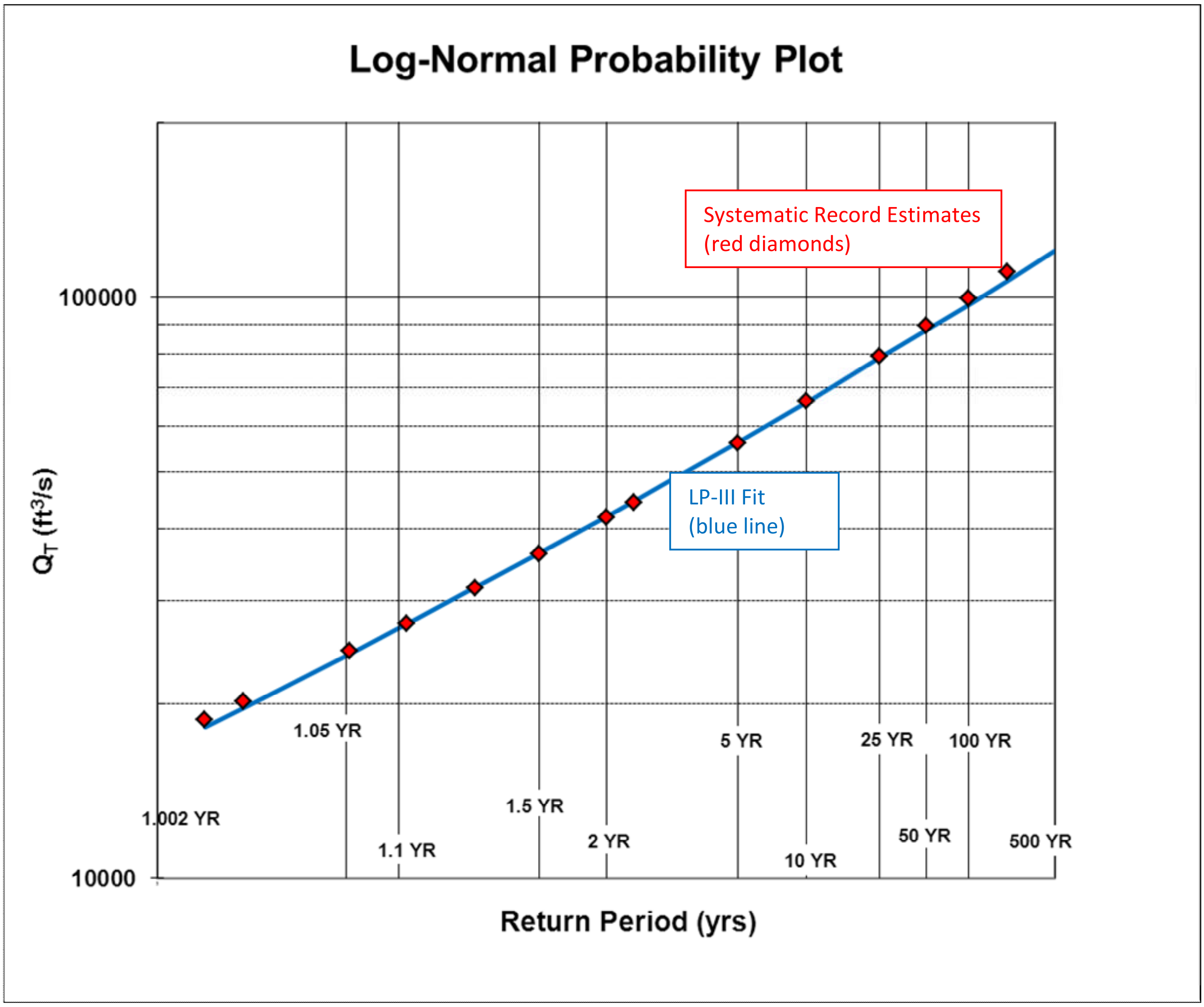
Table 1. Design Hydrology Summary

			Bridge Site	**Gage Site Ann Maxima**	
		Area (mi^2)	3435	3263	
Return Period T (yrs)	**Exceedance Prob P_{ex}**	**Area exponent "a"**	***Final Recommended Q_T (ft^3/s)***	**USGS gage Bull. 17B Est.**	**USGS gage Systematic Record**
1.005	0.995	0.855	***18778***	17440	17970
1.01	0.990	0.855	***20147***	18830	19280
1.05	0.952	0.852	***24594***	23310	23540
1.1	0.900	0.850	***27486***	26200	26310
1.25	0.800	0.843	***31580***	30270	30240
1.5	0.667	0.836	***36110***	34730	34590
2	0.500	0.825	***41755***	40230	40020
2.33	0.429	0.819	***44401***	42800	42570
5	0.200	0.797	***56228***	54080	53970
10	0.100	0.783	***66203***	63410	63590
25	0.040	0.767	***79255***	75400	76190
50	0.020	0.757	***89321***	84480	85910
100	0.010	0.748	***99671***	93710	95910
200	0.005	0.739	***110418***	103100	106300
500	0.002	0.729	***125311***	116000	120700

Notes: Q_T at project = $(A_{ws}/A_{gage)})^a$ x $Q_{T\text{-}gage}$, using "systematic record" results at gage

USGS Gage #01059000, "Androscoggin River near Auburn, Maine"

Figure 1. Annual Maximum Probability Plot – Androscoggin River at Frank J Wood Bridge #2016



Discussion

MaineDOT design hydrology for larger structures is ordinarily calculated with statewide peak flow regression equations (Hodgkins, 1999). However, this is not recommended for the Frank J. Wood Bridge location because the Andoscoggin River is heavily regulated with numerous dams upstream of the project site, whereas the statewide equations are intended for undeveloped, unregulated watersheds.

Fortunately, there is a USGS gage (01059000, "Androscoggin River near Auburn, Maine") about 20 miles of the bridge. The watershed area at the gage A_g (3263 mi^2) is just slightly less the ungaged watershed area A_u (3425 mi^2) at the bridge with the ratio (A^u/A^g) = 1.05, and therefore area scaling of gage peak flows will provide good estimates for peak flows at the bridge.

The standard site regression equations Q_r are of the form

$$Q_r = cA^a10^{wW}$$

where the parameters c, a, and w vary according to return period (Hodgkins, Table 3); A is watershed area and W is the percentage of watershed area that is mapped as NWI wetlands. The watershed area A at the bridge was determined in ArcGIS from available watershed delineations; the watershed map is shown in Figure 2.

Using this form of regression equation, site estimates are calculated from a gage estimates Q_g by area scaling:

$$Q_u = (A_u/A_g)^aQ_g$$

where "a" is the same area exponent in Q_r above; "u" corresponds to the ungaged project site and "g" corresponds to the gaged watershed. Values of "a" are listed in Table 1 above; they are also shown graphically in Figure 3. Hodgkins does not give "a" values for all return period (T) values, so the missing values have been interpolated/extrapolated as needed.

The peak flow estimates Q_g at the Auburn gage were calculated from the gage data using the USGS program PeakFQ (Flynn et al, 2006). Program output is reproduced in Appendix A. This program produces estimates according to the standard "Bulletin 17b" procedures, fitting the annual maximum data to the Log-Pearson III (LP-III) probability distribution. A generalized statewide skew value of 0.029 with standard error = 0.297 was used (Hodgkins, 1999). This skew value is so small that the LP-III distribution is closely approximated by the simpler log-Normal (LN) distribution, as evidenced by a straight-line plot on LN-probability sale. PeakFQ also produces estimates using plotting positions applied to the systematic record. These site-specific, distribution-free "systematic record" estimates were ultimately chosen as the basis for project design hydrology, although the 17B LP-III distribution closely fits the data.

References:

Hodgkins, 1999. Estimating the Magnitude of Peak Flows for Stream in Maine for Selected Recurrence Intervals, US Geological Survey, *WRIR 99-4408*.

Flynn, K., W.H. Kirby, & P.R. Hummel, 2006. User's Manual for Program PeakFQ, Annual Flood Frequency Analysis Using Bulletin 17B Guidelines. US Geological Survey, *Techniques & Methods 4-B4*.

Figure 2. Androscoggin River Watershed at Frank J Wood Bridge

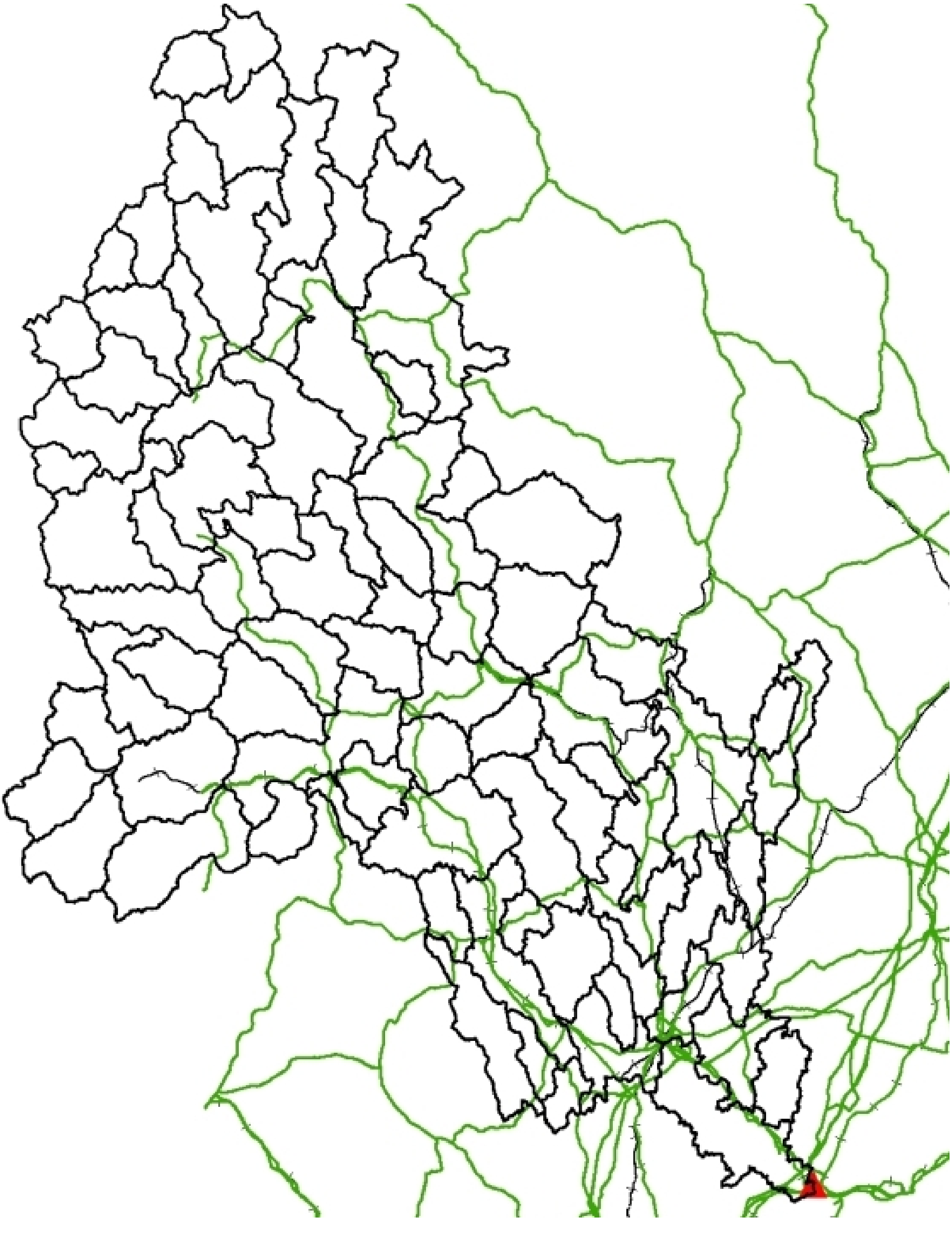
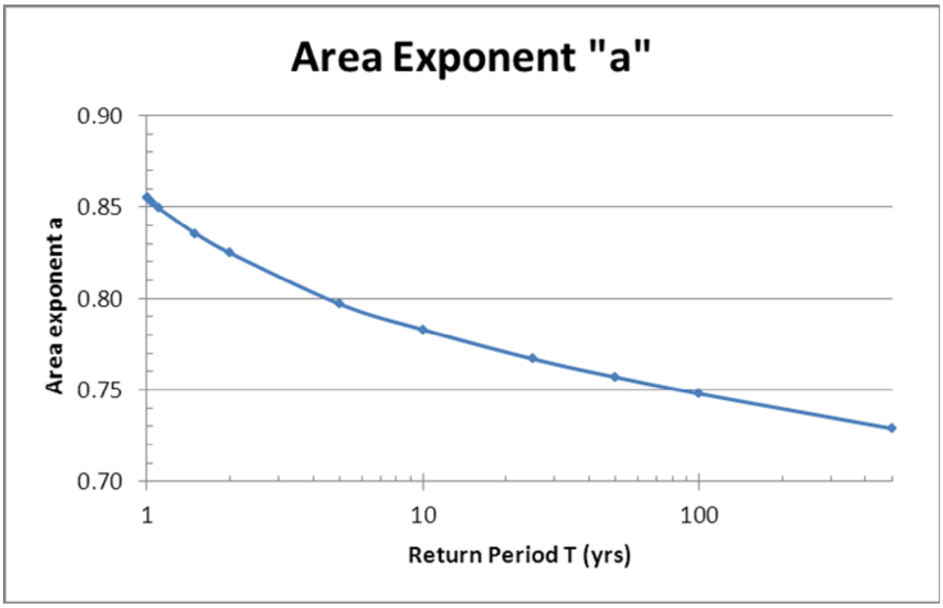


Figure 3. Area Exponent “a” for Watershed Scaling of Peak Flow Estimates



Appendix:

Output for Gage on Androscoggin River near Auburn, Maine

from

USGS Program PeakFQ

```
Program PeakFq           U. S. GEOLOGICAL SURVEY            Seq.000.000
 Ver. 5.2            Annual peak flow frequency analysis     Run Date / Time
 11/01/2007          following Bulletin 17-B Guidelines      02/19/2014 15:18

                        --- PROCESSING OPTIONS ---

                     Plot option         = Graphics device
                     Basin char output   = None
                     Print option        = Yes
                     Debug print         = No
                     Input peaks listing = Long
                     Input peaks format  = WATSTORE peak file

                     Input files used:
                        peaks (ascii)  - D:\PROGFILS\PEAKFQ\TEST\DATA_IN\BRUNS.TXT
                        specifications - PKFQWPSF.TMP
                     Output file(s):
                        main - D:\PROGFILS\PEAKFQ\TEST\DATA_IN\BRUNS.PRT

          Station - 01059000  Androscoggin River near Auburn, Maine


                      I N P U T   D A T A   S U M M A R Y

               Number of peaks in record           =       84
               Peaks not used in analysis          =        0
               Systematic peaks in analysis        =       84
               Historic peaks in analysis          =        0
               Years of historic record            =        0
               Generalized skew                    =    0.029
                    Standard error                 =    0.297
                    Mean Square error              =    0.088
               Skew option                         =   WEIGHTED
               Gage base discharge                 =      0.0
               User supplied high outlier threshold =   --
               User supplied low outlier criterion  =   --
               Plotting position parameter         =     0.40


 *********  NOTICE  --  Preliminary machine computations.        *********
 *********  User responsible for assessment and interpretation.  *********

   WCF134I-NO SYSTEMATIC PEAKS WERE BELOW GAGE BASE.                      0.0
   WCF195I-NO LOW OUTLIERS WERE DETECTED BELOW CRITERION.             14621.3
   WCF162I-SYSTEMATIC PEAKS EXCEEDED HIGH-OUTLIER CRITERION.   1  112490.0
```

```
 Program PeakFq           U. S. GEOLOGICAL SURVEY             Seq.001.002
 Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
 11/01/2007          following Bulletin 17-B Guidelines       02/19/2014 15:18

          Station - 01059000  Androscoggin River near Auburn, Maine


          ANNUAL FREQUENCY CURVE PARAMETERS -- LOG-PEARSON TYPE III

                       FLOOD BASE                   LOGARITHMIC
                 ----------------------  -------------------------------
                            EXCEEDANCE                STANDARD
                  DISCHARGE PROBABILITY     MEAN     DEVIATION     SKEW
                 -------------------------------------------------------
SYSTEMATIC RECORD       0.0     1.0000     4.6080       0.1498      0.231
BULL.17B ESTIMATE       0.0     1.0000     4.6080       0.1498      0.138


   ANNUAL FREQUENCY CURVE -- DISCHARGES AT SELECTED EXCEEDANCE PROBABILITIES

     ANNUAL                             'EXPECTED   95-PCT CONFIDENCE LIMITS
  EXCEEDANCE     BULL.17B    SYSTEMATIC PROBABILITY'  FOR BULL. 17B ESTIMATES
  PROBABILITY    ESTIMATE       RECORD     ESTIMATE         LOWER         UPPER

     0.9950      17440.0      17970.0      17040.0      15160.0      19510.0
     0.9900      18830.0      19280.0      18480.0      16530.0      20910.0
     0.9500      23310.0      23540.0      23100.0      21000.0      25410.0
     0.9000      26200.0      26310.0      26050.0      23910.0      28310.0
     0.8000      30270.0      30240.0      30180.0      28000.0      32410.0
     0.6667      34730.0      34590.0      34690.0      32440.0      36990.0
     0.5000      40230.0      40020.0      40230.0      37790.0      42820.0
     0.4292      42800.0      42570.0      42820.0      40230.0      45610.0
     0.2000      54080.0      53970.0      54260.0      50530.0      58440.0
     0.1000      63410.0      63590.0      63830.0      58660.0      69560.0
     0.0400      75400.0      76190.0      76320.0      68800.0      84350.0
     0.0200      84480.0      85910.0      85960.0      76310.0      95840.0
     0.0100      93710.0      95910.0      95900.0      83820.0     107700.0
     0.0050     103100.0     106300.0     106200.0      91400.0     120100.0
     0.0020     116000.0     120700.0     120700.0     101600.0     137200.0
```

```
Program PeakFq           U. S. GEOLOGICAL SURVEY             Seq.001.003
Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
11/01/2007          following Bulletin 17-B Guidelines       02/19/2014 15:18

         Station - 01059000  Androscoggin River near Auburn, Maine


                     I N P U T   D A T A   L I S T I N G


   WATER YEAR    DISCHARGE   CODES      WATER YEAR    DISCHARGE   CODES

      1929        40100.0       K          1971        41500.0       K
      1930        38900.0       K          1972        35600.0       K
      1931        26700.0       K          1973        45800.0       K
      1932        36000.0       K          1974        60200.0       K
      1933        45400.0       K          1975        28800.0       K
      1934        45000.0       K          1976        47700.0       K
      1935        28200.0       K          1977        46800.0       K
      1936       135000.0       K          1978        39200.0       K
      1937        40700.0       K          1979        53300.0       K
      1938        36600.0       K          1980        45800.0       K
      1939        38400.0       K          1981        38700.0       K
      1940        46300.0       K          1982        35100.0       K
      1941        20100.0       K          1983        40800.0       K
      1942        41500.0       K          1984        62500.0       K
      1943        31200.0       K          1985        17300.0       K
      1944        38900.0       K          1986        59200.0       K
      1945        39300.0       K          1987       103000.0       K
      1946        29700.0       K          1988        28700.0       K
      1947        34700.0       K          1989        63400.0       K
      1948        29700.0       K          1990        35200.0       K
      1949        34800.0       K          1991        35500.0       K
      1950        50800.0       K          1992        38700.0       K
      1951        52900.0       K          1993        53000.0       K
      1952        37500.0       K          1994        40100.0       K
      1953        95800.0       K          1995        17800.0       K
      1954        49600.0       K          1996        42600.0       K
      1955        33000.0       K          1997        40900.0       K
      1956        26200.0       K          1998        56200.0       K
      1957        19400.0       K          1999        47200.0       K
      1958        46700.0       K          2000        42800.0       K
      1959        31000.0       K          2001        43600.0       K
      1960        51500.0       K          2002        42000.0       K
      1961        24300.0       K          2003        28300.0       K
      1962        31000.0       K          2004        48000.0       K
      1963        39200.0       K          2005        58500.0       K
      1964        52600.0       K          2006        38000.0       K
      1965        19600.0       K          2007        45100.0       K
      1966        25800.0       K          2008        45800.0       K
      1967        45000.0       K          2009        43900.0       K
      1968        45000.0       K          2010        42300.0       K
      1969        48100.0       K          2011        50300.0       K
      1970        51400.0       K          2012        60100.0       K
```

```
 Explanation of peak discharge qualification codes

PeakFQ    NWIS
 CODE     CODE   DEFINITION

   D        3    Dam failure, non-recurrent flow anomaly
   G        8    Discharge greater than stated value
   X       3+8   Both of the above
   L        4    Discharge less than stated value
   K     6 OR C  Known effect of regulation or urbanization
   H        7    Historic peak

   -  Minus-flagged discharge -- Not used in computation
         -8888.0 -- No discharge value given
   -  Minus-flagged water year -- Historic peak used in computation
```

```
Program PeakFq           U. S. GEOLOGICAL SURVEY             Seq.001.004
Ver. 5.2            Annual peak flow frequency analysis      Run Date / Time
11/01/2007          following Bulletin 17-B Guidelines       02/19/2014 15:18

         Station - 01059000  Androscoggin River near Auburn, Maine


 EMPIRICAL FREQUENCY CURVES -- WEIBXXX PLOTTING POSITIONS
 *** WEIBA = 0.400 ***

    WATER         RANKED       SYSTEMATIC      BULL.17B
     YEAR       DISCHARGE        RECORD        ESTIMATE

     1936       135000.0         0.0071         0.0071
     1987       103000.0         0.0190         0.0190
     1953        95800.0         0.0309         0.0309
     1989        63400.0         0.0428         0.0428
     1984        62500.0         0.0546         0.0546
     1974        60200.0         0.0665         0.0665
     2012        60100.0         0.0784         0.0784
     1986        59200.0         0.0903         0.0903
     2005        58500.0         0.1021         0.1021
     1998        56200.0         0.1140         0.1140
     1979        53300.0         0.1259         0.1259
     1993        53000.0         0.1378         0.1378
     1951        52900.0         0.1496         0.1496
     1964        52600.0         0.1615         0.1615
     1960        51500.0         0.1734         0.1734
     1970        51400.0         0.1853         0.1853
     1950        50800.0         0.1971         0.1971
     2011        50300.0         0.2090         0.2090
     1954        49600.0         0.2209         0.2209
     1969        48100.0         0.2328         0.2328
     2004        48000.0         0.2447         0.2447
     1976        47700.0         0.2565         0.2565
     1999        47200.0         0.2684         0.2684
     1977        46800.0         0.2803         0.2803
     1958        46700.0         0.2922         0.2922
     1940        46300.0         0.3040         0.3040
     1973        45800.0         0.3159         0.3159
     1980        45800.0         0.3278         0.3278
     2008        45800.0         0.3397         0.3397
     1933        45400.0         0.3515         0.3515
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Data records may have been ignored for the stations listed below.
(Card type must be Y, Z, N, H, I, 2, 3, 4,  or *.)
(2, 4, and * records are ignored.)

 For the station below, the following records were ignored:
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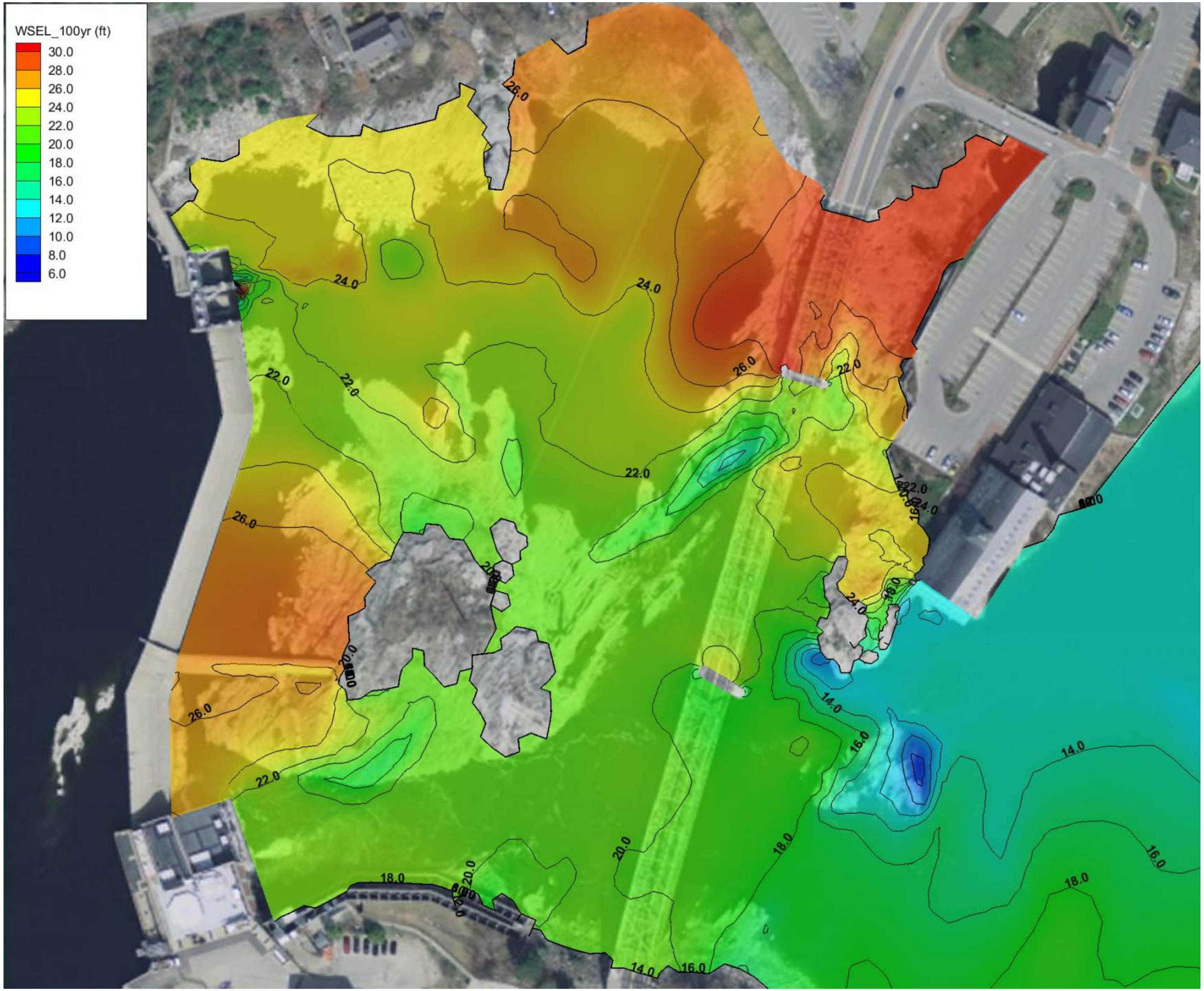

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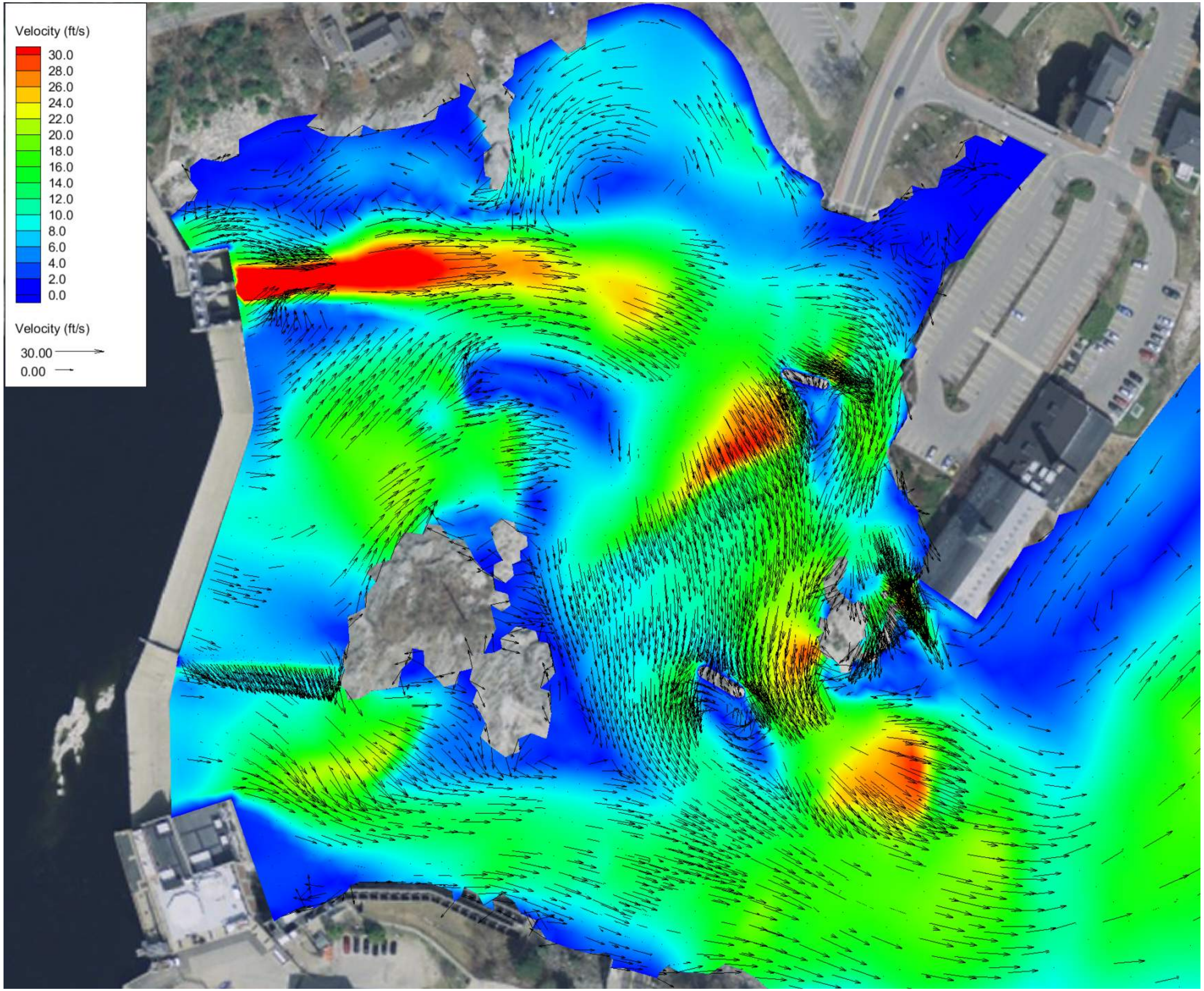

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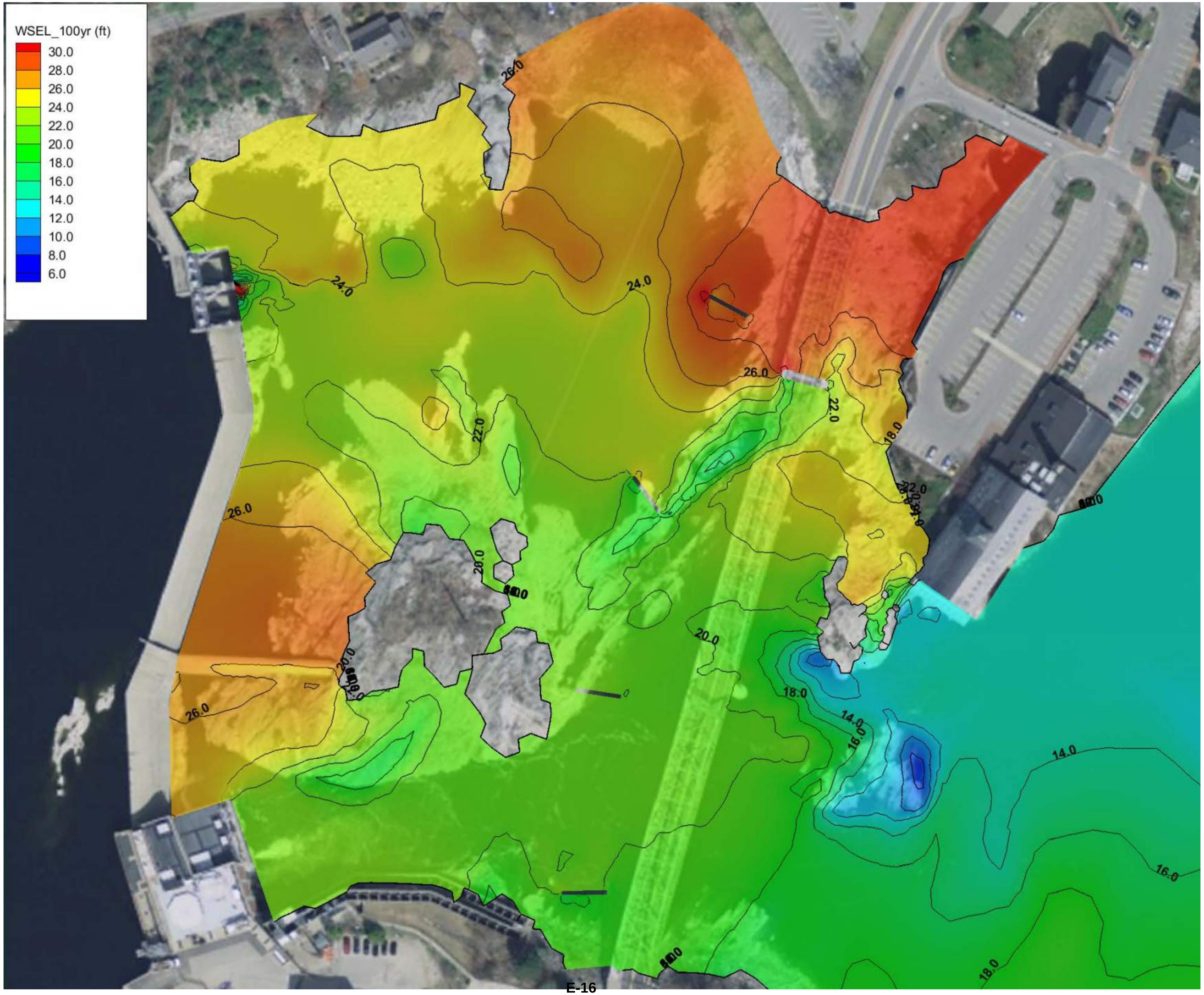

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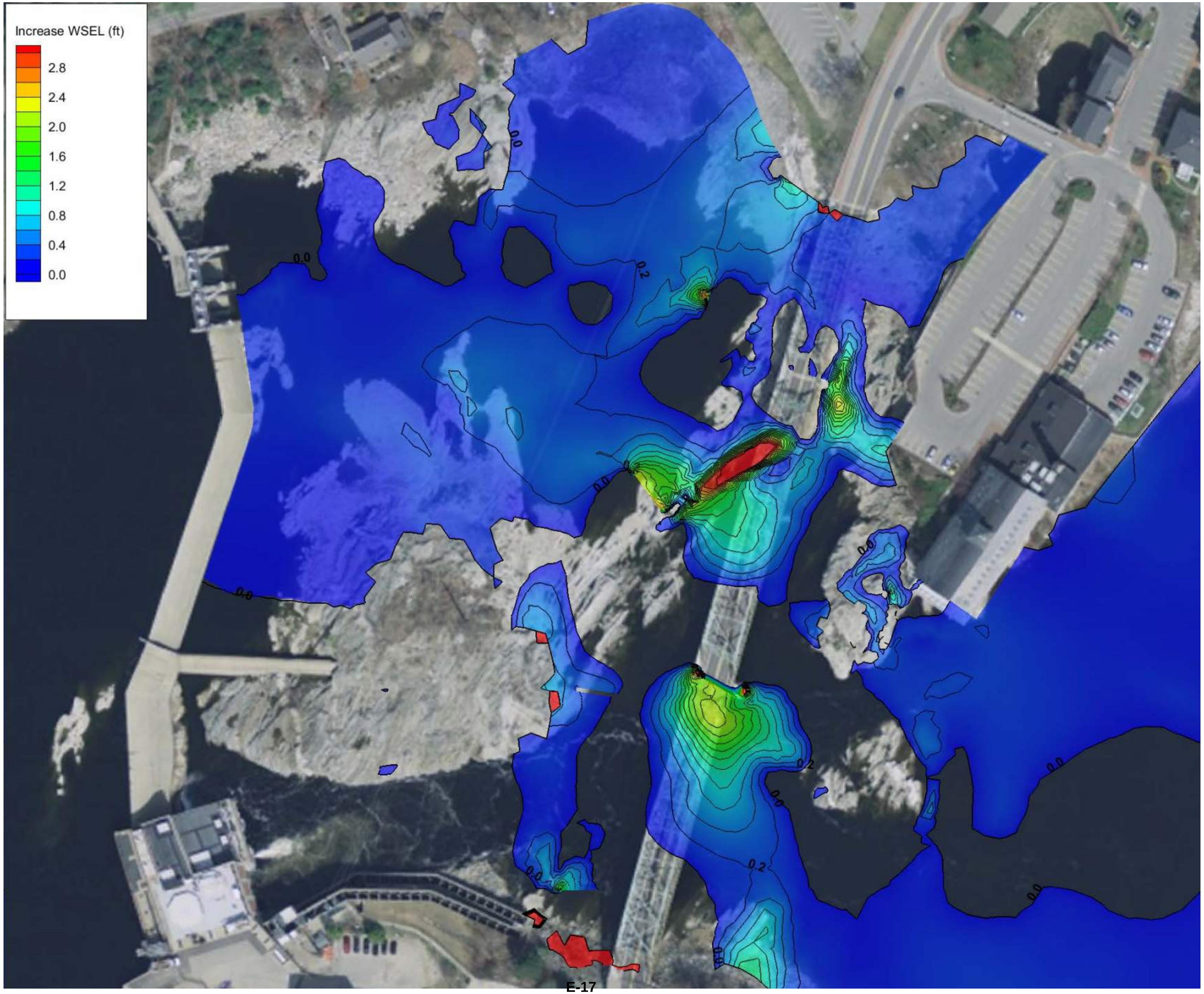

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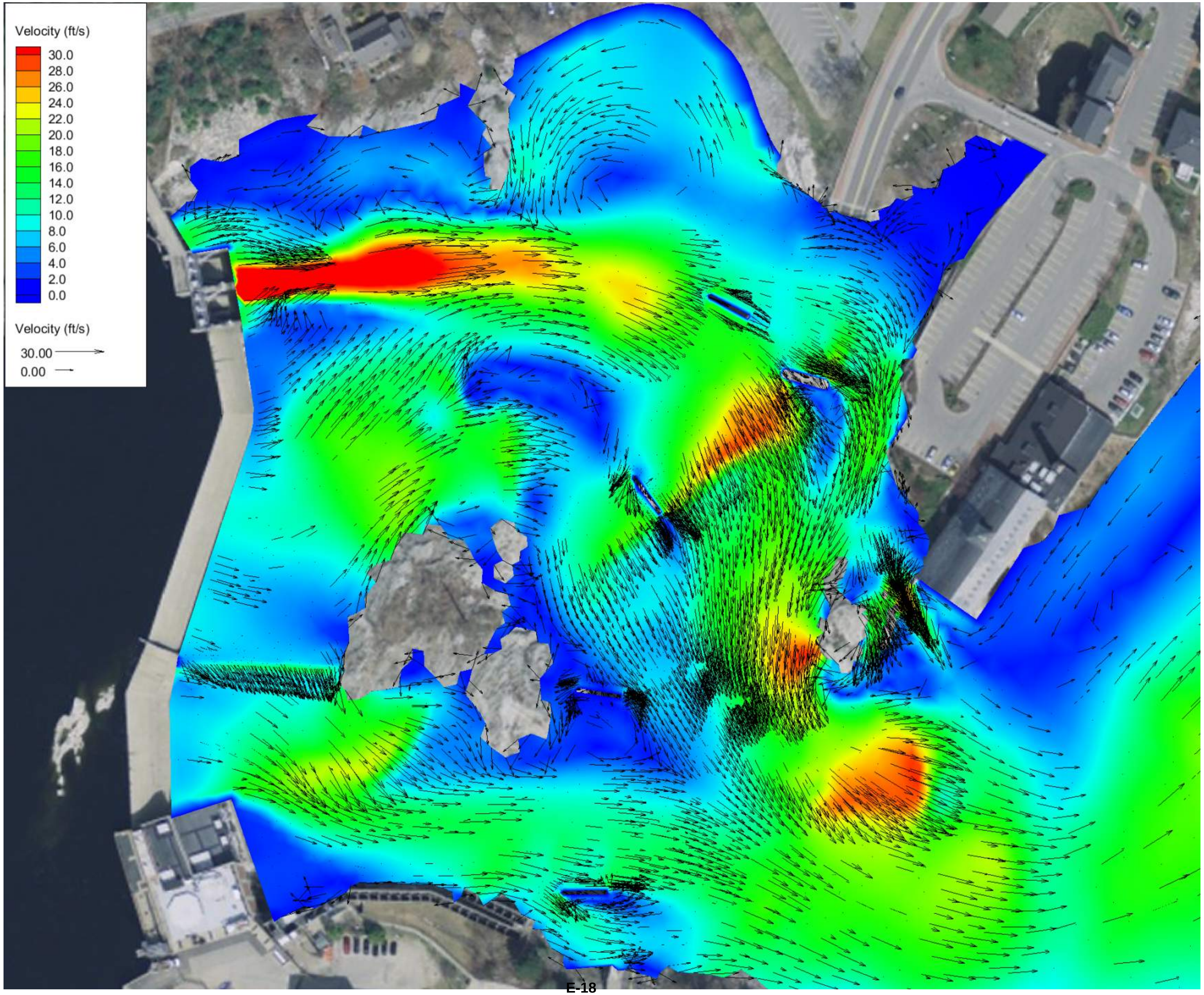
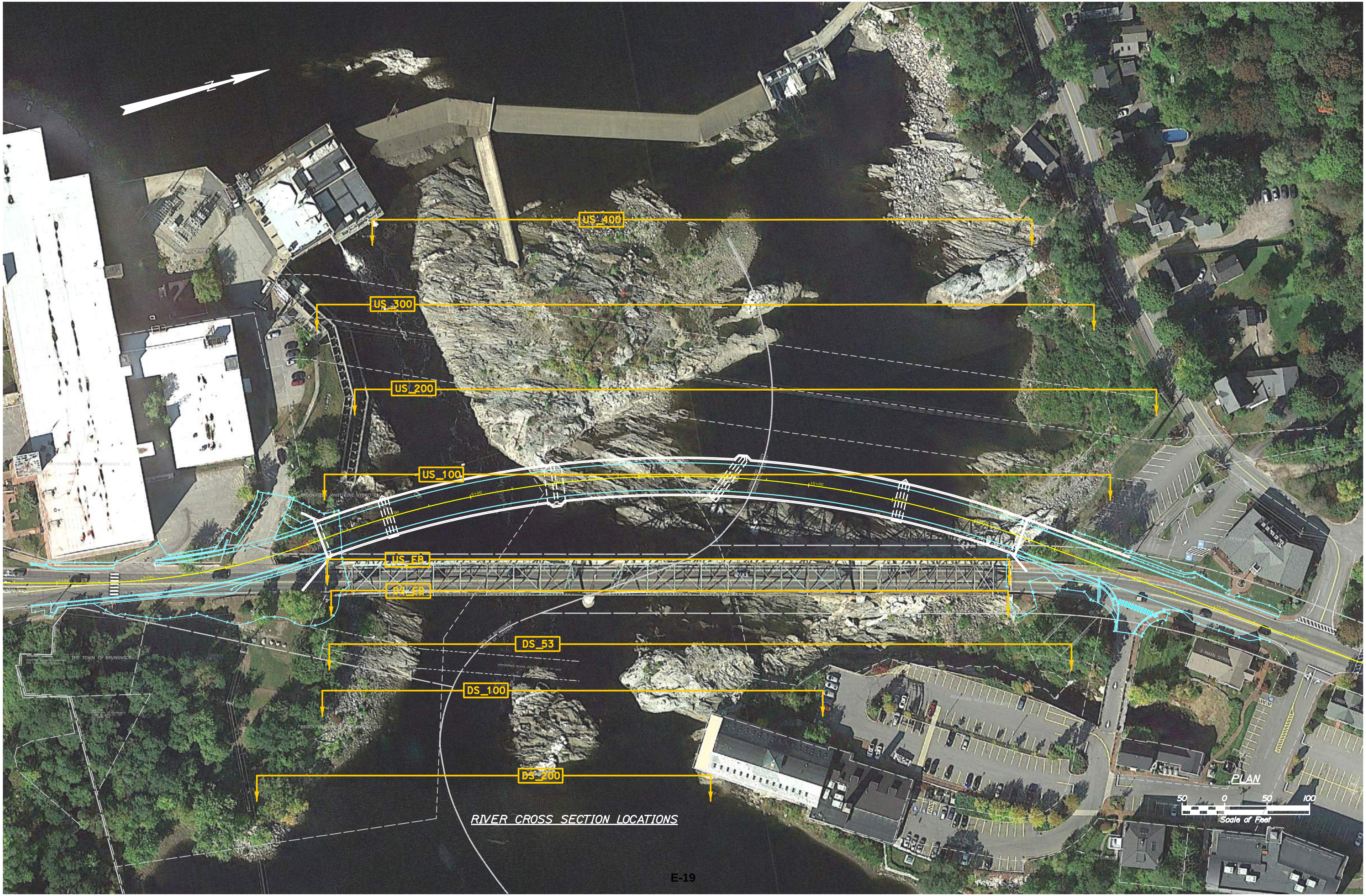
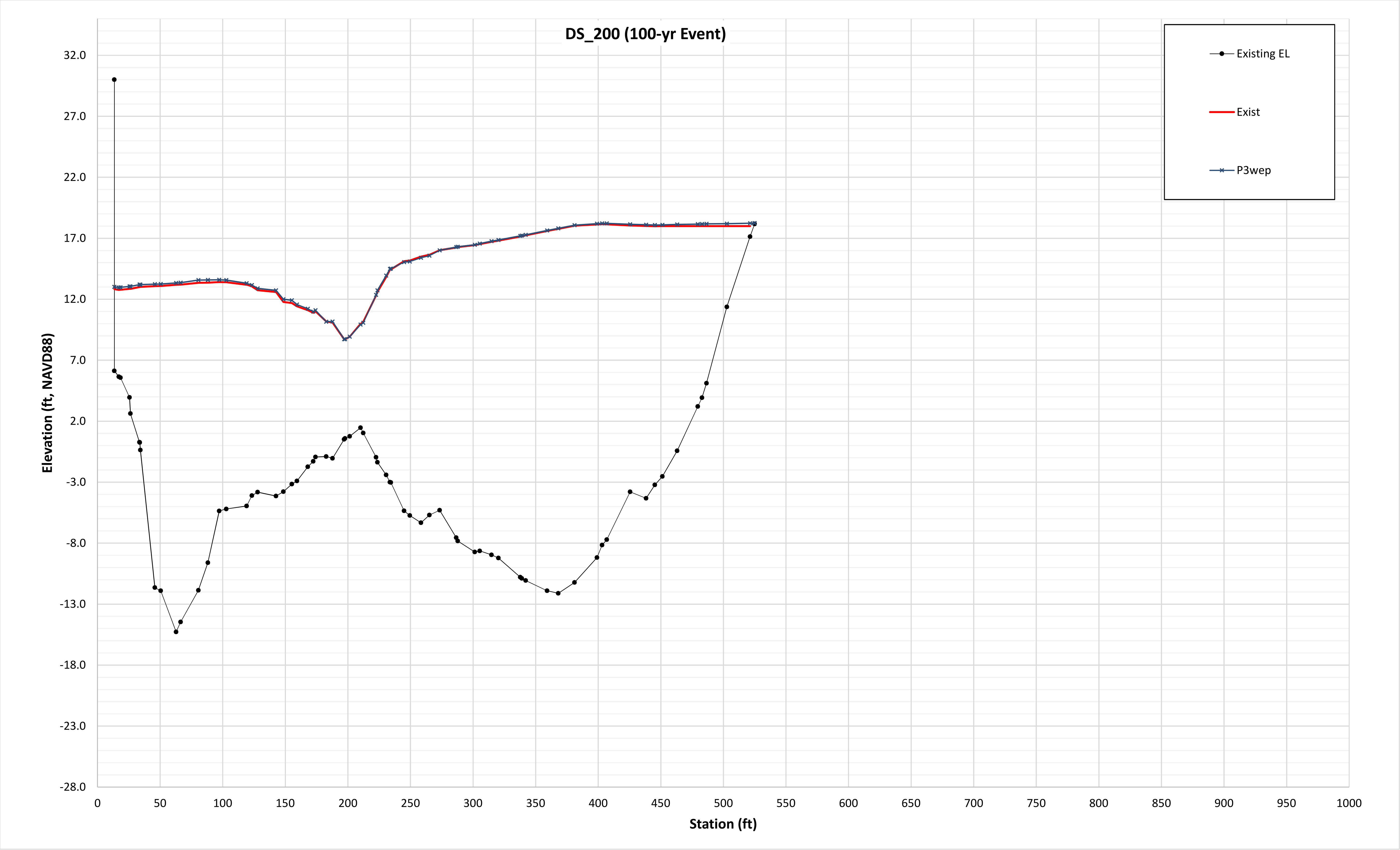




RIVER CROSS SECTION LOCATIONS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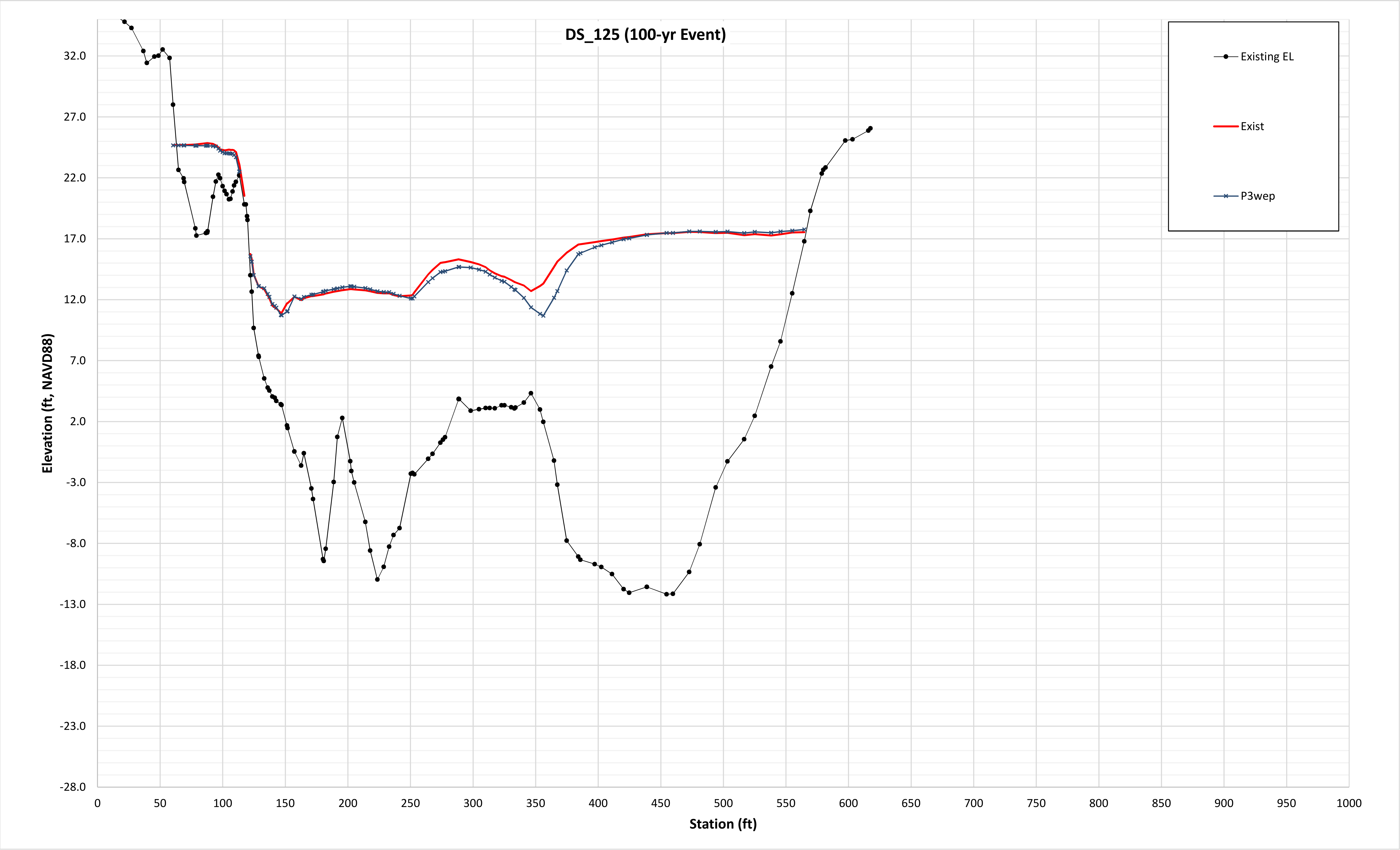
Station (ft)



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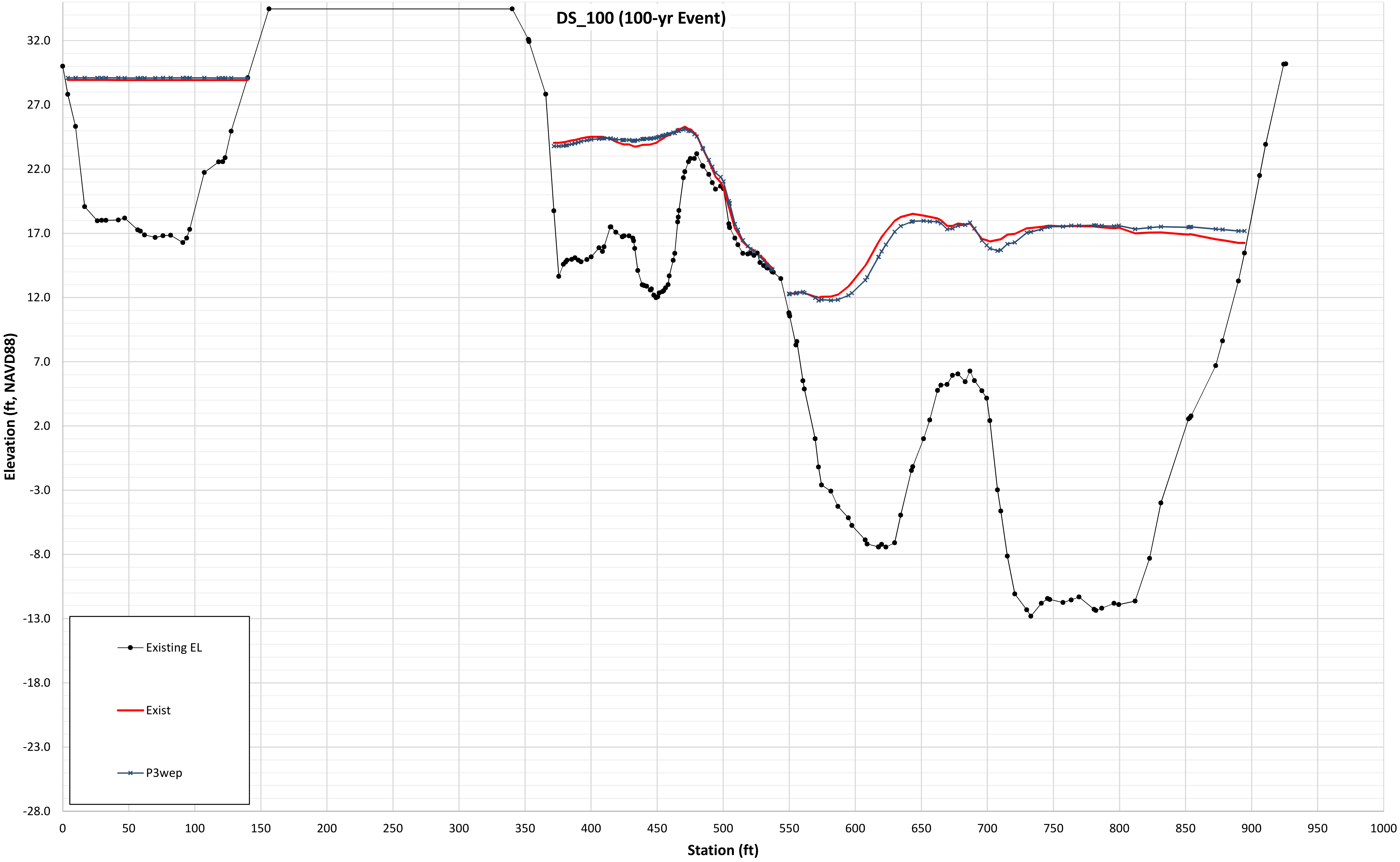
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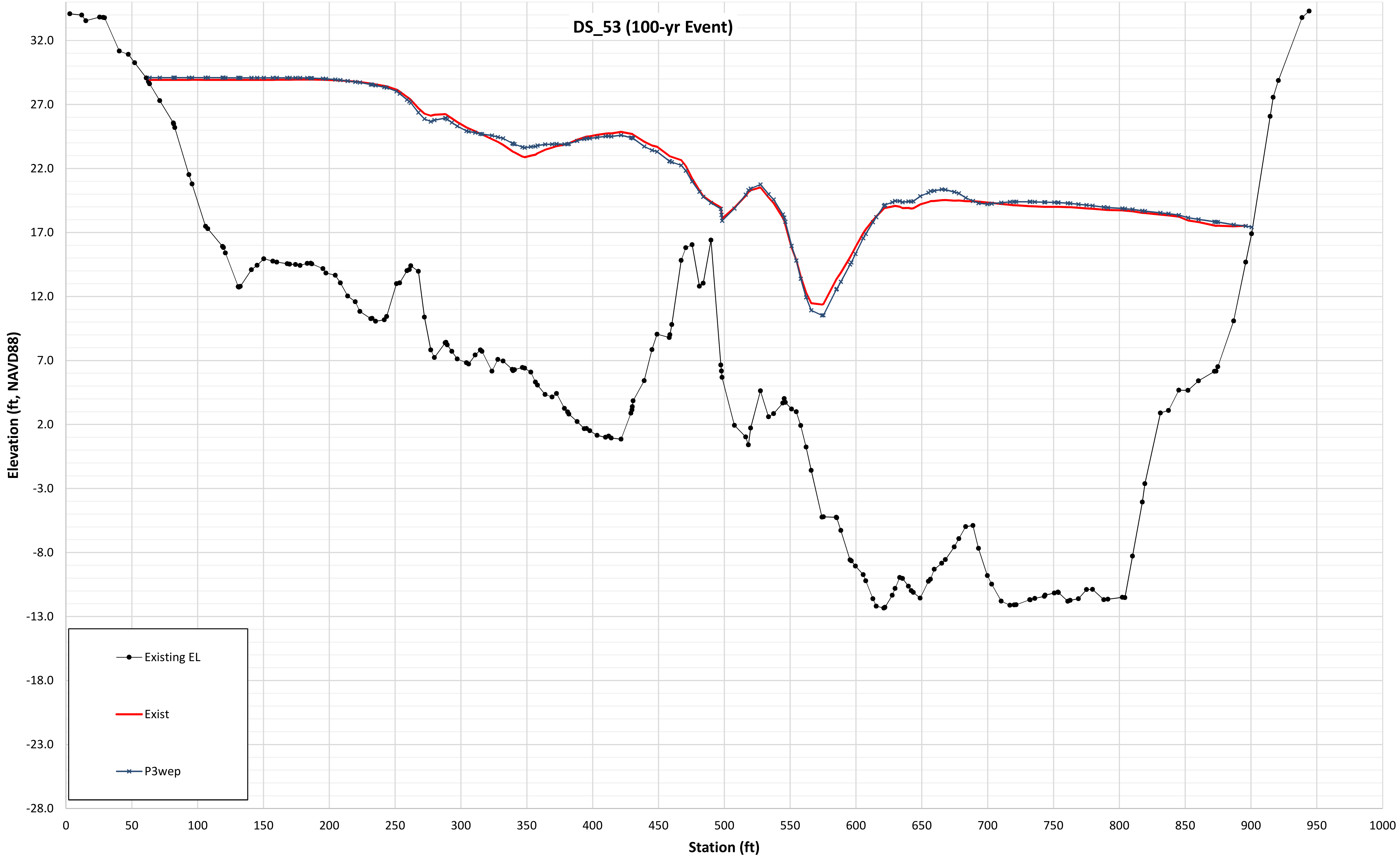
P3wep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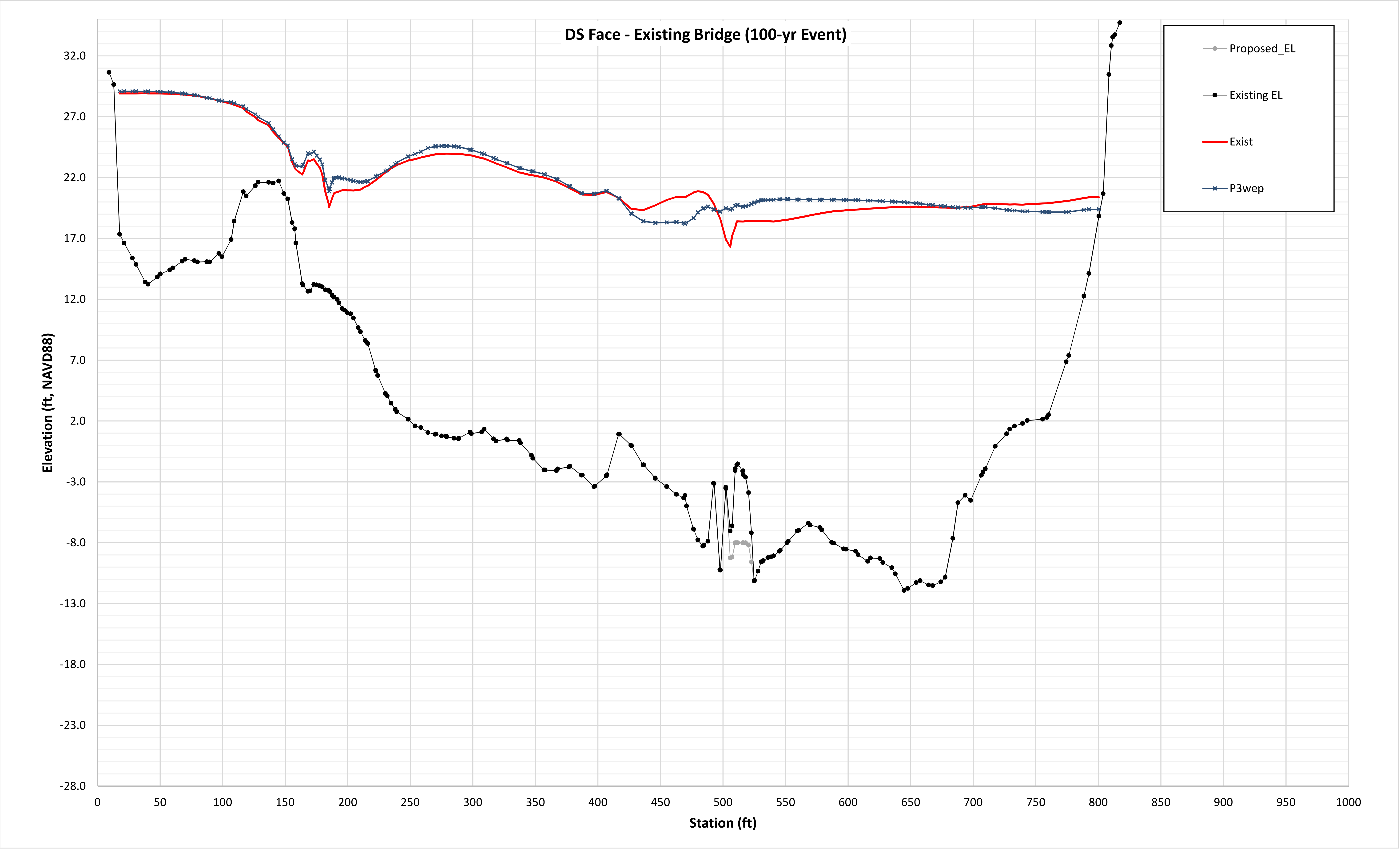
Existing EL
Exist
P3wep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Proposed_EL
Existing EL
Exist
P3wep
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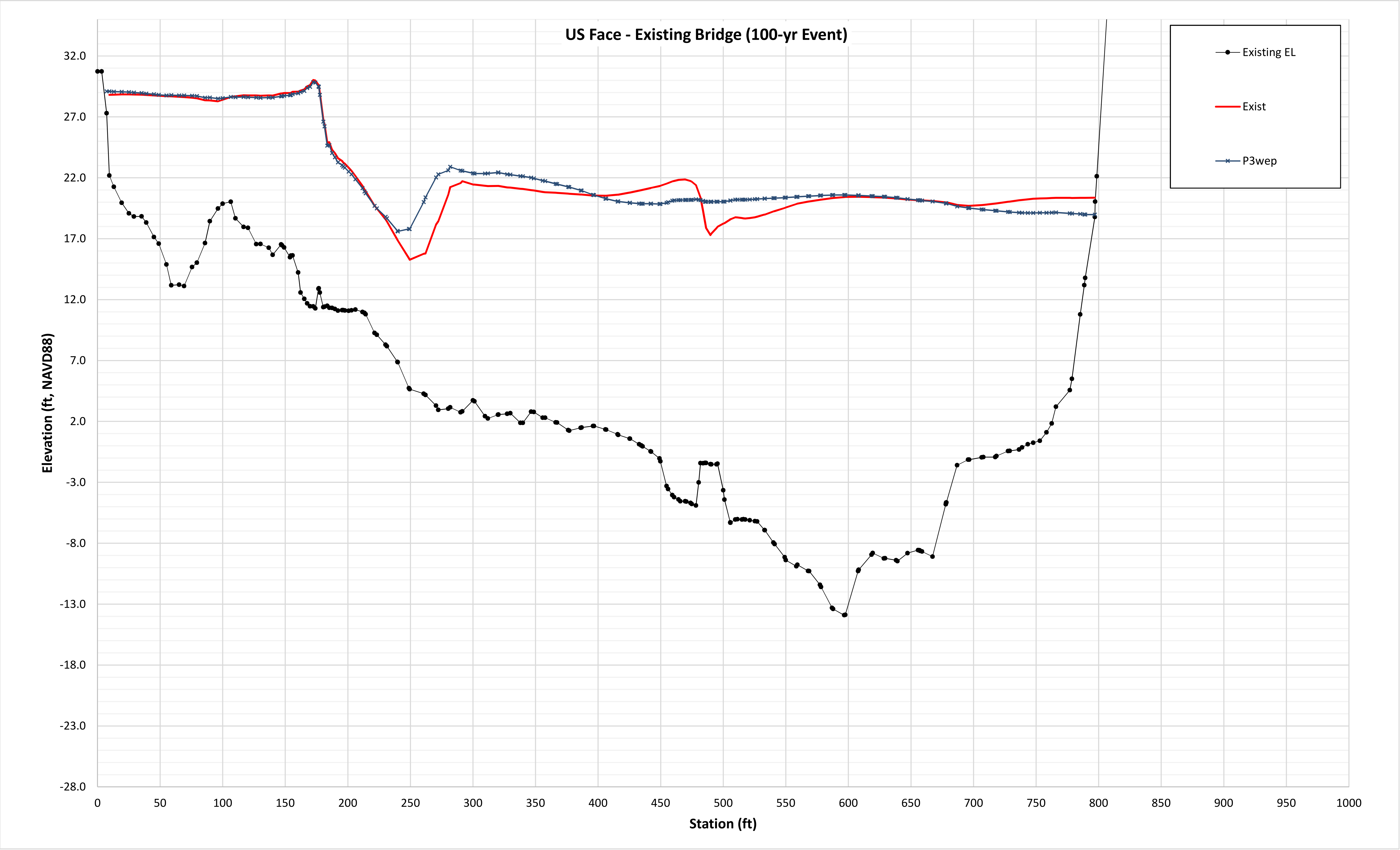
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P3wep
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Station (ft)


US_100 / At Replacement Bridge (100-yr Event)
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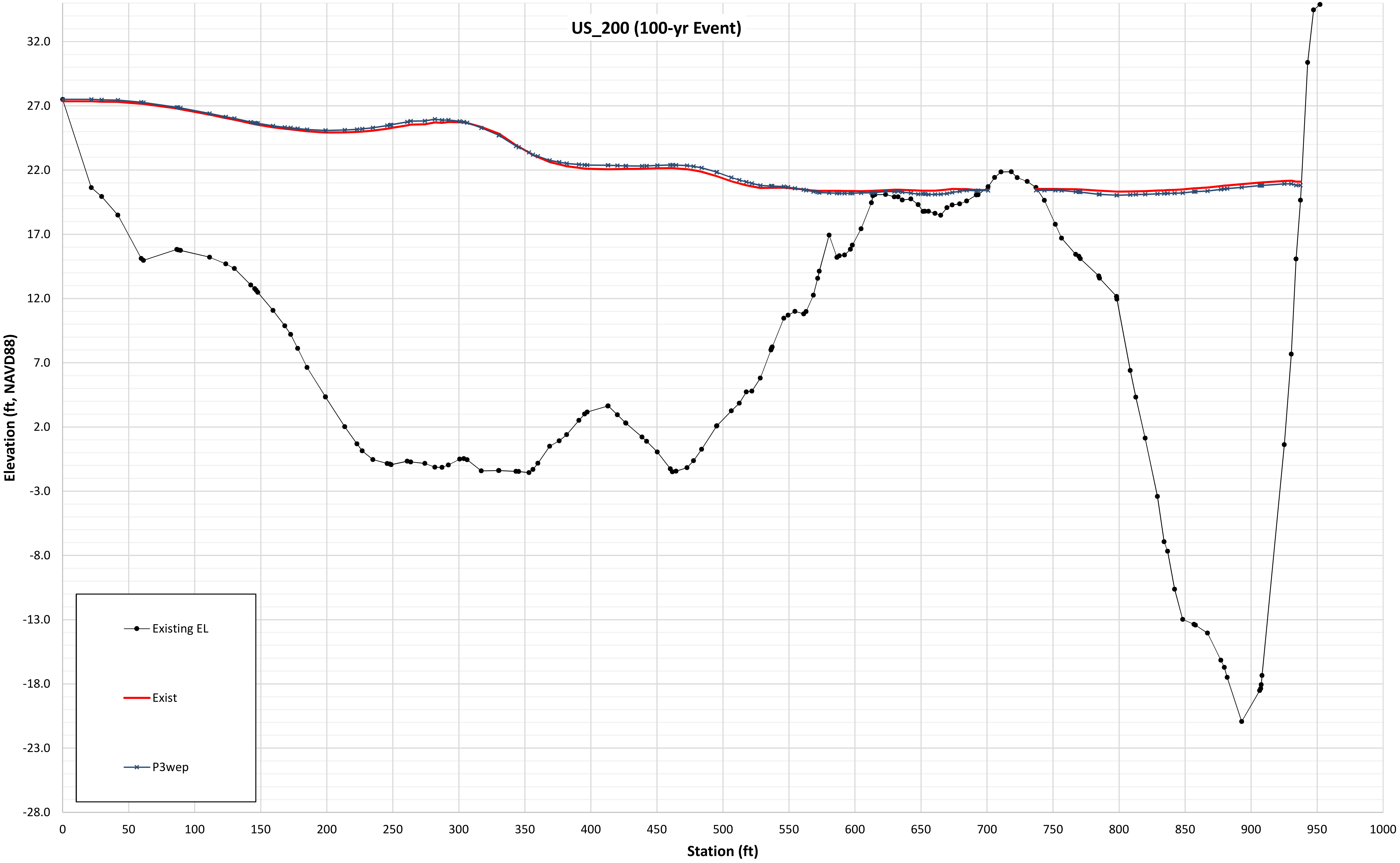
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Station (ft)
Existing EL
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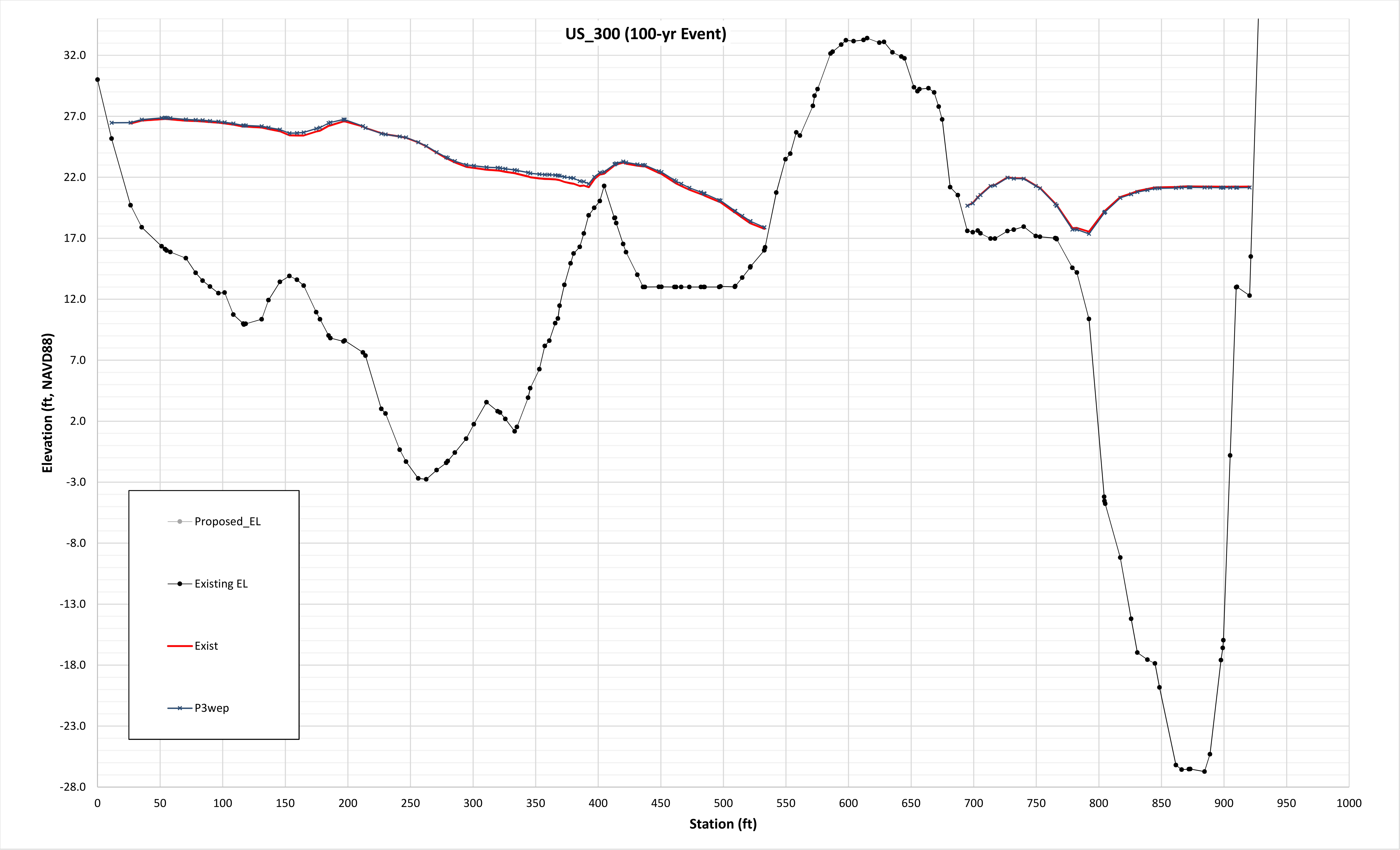
P3wep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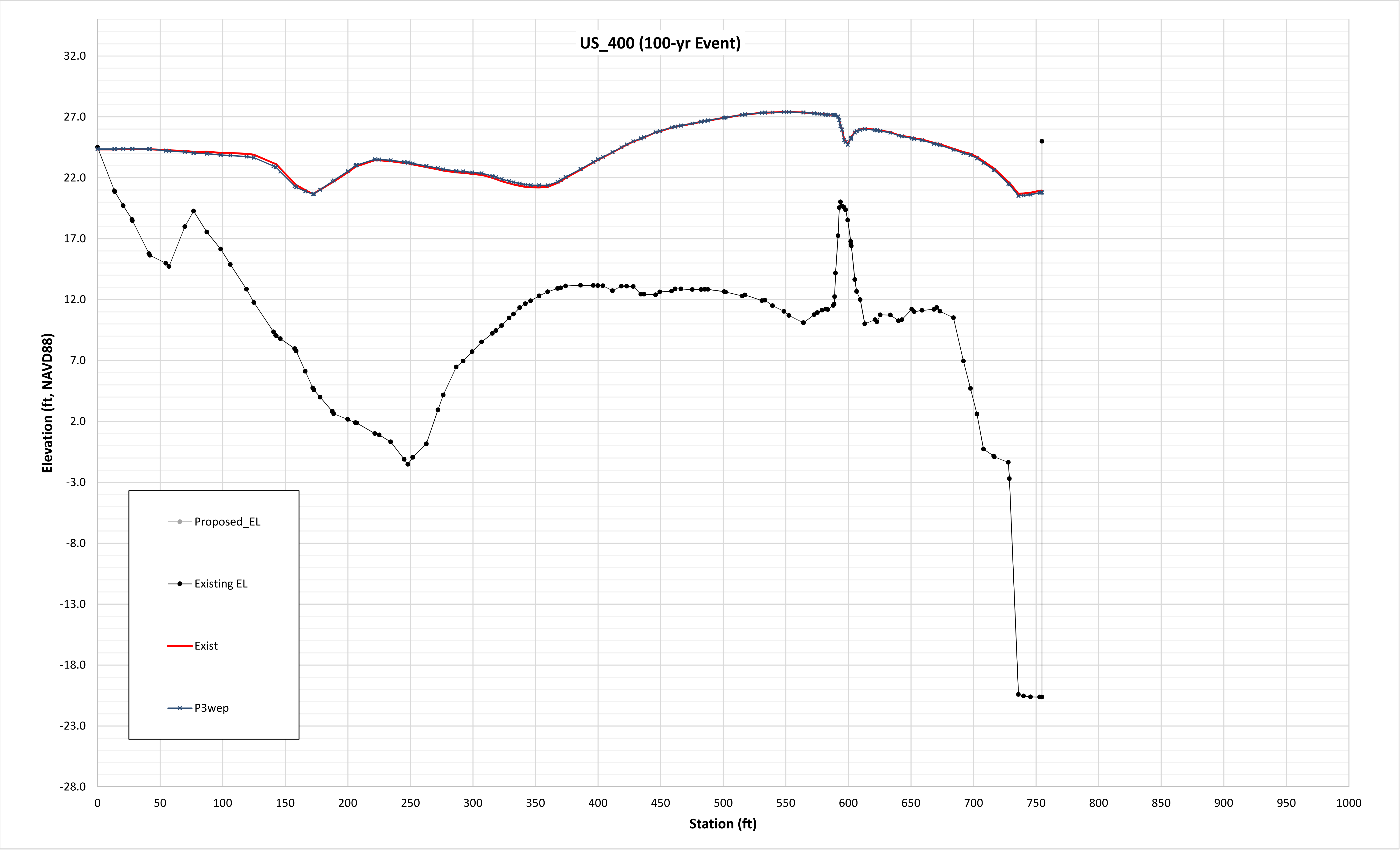
Station (ft)
Proposed_EL
Existing EL
Exist
P3wep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Station (ft)
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Proposed_EL
Existing EL
Exist
P3wep

APPENDIX F

Miscellaneous Information

Brunswick-Topsham

Frank J. Wood meeting dates:

- 6-1-17 Brookfield Coord. Meeting (shadow modeling) *Minutes available @ MaineDOT*
- 4-5-17 Public Meeting (Section 106) *Minutes available @ MaineDOT*
- 3-8-17 Design Advisory Committee Meeting[1]
- 2-8-17 Design Advisory Committee Meeting[1]
- 1-11-17 Design Advisory Committee Meeting[1]
- 12-7-16 Design Advisory Committee Meeting[1]
- 11-9-16 Design Advisory Committee Meeting[1]
- 10-27-17 Section 106 Consulting Parties Meeting[2]
- 10-19-16 Design Advisory Committee Meeting[1]
- 9-14-16 Design Advisory Committee Meeting[1]
- 8-22-16 Design Advisory Committee Meeting[1]
- 7-11-16 Section 106 Consulting Parties Kickoff Meeting[2]
- 6-6-16 Brunswick Town Council meeting[3]
- 6-2-16 Topsham Select Board meeting[4]
- 5-12-16 State and federal environmental agencies
- 4/27/2016 Formal Public Meeting *Minutes Available @ MaineDOT*
  - Combined Towns of Topsham and Brunswick
- 4-25-16 Stakeholder meeting *Minutes Available @ MaineDOT*
  - Combined Towns of Topsham and Brunswick
  - **BRUNSWICK**
    1. Downtown Brunswick and Outer Pleasant Street Master Plan Implementation Committee
    2. Village Review Board
    3. Planning Board
    4. Brunswick Bicycle and Pedestrian Advisory Committee Meeting
    5. Merrymeeting Wheelers (Greater Brunswick-based)
    6. Brunswick Downtown Association
    7. Brunswick Public Art
    8. Androscoggin Riverwalk Committee (Topsham and Brunswick-based)
  - TOPSHAM
    1. Lower Village Development Committee
    2. Historic District Commission
    3. Planning Board
    4. Topsham Community Fund
    5. Greater Topsham Trails Alliance
    6. Topsham Development, Inc.
    7. Brunswick-Topsham Land Trust
    8. Androscoggin Riverwalk Committee
- 4-19-16 Brunswick Town Council meeting[3]
- 3-24-16 Topsham and Brunswick town managers, economic development managers, business and building owner abutters. *Minutes included in Appendix.*
- 2-16-16 Topsham and Brunswick managers and staff---widths, peds, bikes, trails, schedules, costs, etc. *Minutes included in Appendix*
- 1-28-16 Brookfield, FERC discussion *Minutes included in Appendix*
- 9-30-15 Topsham and Brunswick managers and staff---widths, peds, bikes, trails, schedules, costs, etc. *Minutes included in Appendix*
- 6-29-15 Brookfield Coordination Meeting *Minutes included in Appendix*
- 2-25-15 Preliminary Public Meeting *Minutes available @ MaineDOT*
- 2-5-15 Project Kickoff meeting *Minutes included in Appendix*

[1] -Minutes available @ http://www.topshammaine.com/

[2] -Information available @ http://www.maine.gov/mdot/env/frankjwood/

[3] -Minutes available @ http://www.brunswickme.org/

[4] -Minutes available @ http://www.topshammaine.com/

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MEMORANDUM

Date: March 24, 2016

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Abutters Meeting

CC: File, All Attendees

This memo serves to summarize the discussion at the Brunswick-Topsham, Frank J. Wood Bridge Abutters Meeting between representatives of MaineDOT, T.Y. Lin International (TYLI), Town Officials from Brunswick and Topsham, and representatives of abutting property owners on March 24, 2016.

Attendees:

Maine DOT	Affiliation
Joel Kittredge	MaineDOT
Rick Hebert	TYLI
Ric Quesada	Bowdoin Mill
John Foster	Town of Brunswick
John Eldridge	Town of Brunswick
Dan Jacques	Fort Andros
Anthony Gatti	Fort Andros
Jim Howard	Priority Group
C.J. Dirago	Priority Group
Wes Thames	Priority Group
Peter Quesada	Bowdoin Mill
Linda Smith	Town of Brunswick
Debora King	Brunswick Downtown Association
Rich Roedner	Town of Topsham
John Shattuck	Town of Topsham
Rod Melanson	Town of Topsham

The meeting was opened with introductions by all attendees stating their names and affiliation.

Joel Kittredge opened the meeting presentation with a brief overview of the purpose of the meeting:

- To present a synopsis of project development to date.
- To present the MaineDOT's preferred recommendations, including measures taken to mitigate impacts.
- Answer questions and gather information from attendees on specific concerns.

Joel also noted that when this project started the MaineDOT was not certain which direction the project would take, but the Department was committed to doing a thorough investigation of all bridge improvement alternatives to arrive at a solution that best meets the overall project objectives and satisfies constraints. He then reported much of this work is now complete to where a preferred solution for a bridge replacement has been made and is being recommended by the Department. Joel passed the presentation to Rick Hebert of T.Y. Lin International.

Existing site and bridge conditions:

Rick first presented photos and described the existing approach conditions on each end of the bridge and the condition of the existing bridge structure. Sidewalks exist on both sides of the roadway in both approaches to the bridge. The Fort Andros building is located adjacent the southwest approach corner and there are drives on this side of the road that access a back parking lot for Fort Andros and that provide access to the upstream dam powerhouse owned by Brookfield power. The 250th Anniversary Park is located along the southeast approach. On the Topsham end of the bridge, both sides of the roadway are commercially developed and there are intersecting drives and roadways.

The existing bridge is a three-span painted steel truss bridge with a 30 ft roadway between rails and has a single 5 ft sidewalk located on the west side of the bridge. The existing bridge was built in 1931 and is 85 years old. The current FHWA Sufficiency Rating is 52. Rick explained that this condition rating is something that MaineDOT is required to maintain and report to the Federal Highway Administration, and is used to report the overall health of bridges in its inventory. Bridges within this range are typically considered in need of major rehabilitation or replacement. Rick also reported the bridge has had significant repairs done in 1985, 2006 and 2015.

Rehabilitation Alternative:

Major improvements needed to rehabilitate the existing bridge would include:

- Painting the steel truss superstructure (above and below the deck).
- Replacing the concrete bridge deck.
- Repairing or replacing some deteriorated steel elements.
- Strengthening some members which do not meet current loading standards

The estimated cost of the rehabilitation improvements is $9.7M.

Rick H. reported that the disadvantages of the rehabilitation of the existing bridge are: an added sidewalk would be needed to address public safety and local concerns; traffic would be disrupted for a minimum of two years while work was being completed on the bridge; increased maintenance and inspection needs would add future cost and require future traffic disruptions; that there were significant uncertainties associated with a rehabilitation that include but are not limited to fatigue, maintenance and life. The estimated life cycle cost of the rehabilitation is $14.3M, with a deferred bridge replacement of 30 years.

Replacement Bridge:

Rick presented graphic illustrations of the replacement bridge from several different perspectives and described the major features of the bridge. The replacement bridge would follow a curved alignment upstream of the existing bridge. The existing bridge would be used to maintain traffic during construction. The replacement is a five-span 835 ft long haunched steel girder bridge supported on concrete piers and abutments founded on exposed bedrock. The replacement provides two 11 ft travel lanes with 5 ft shoulders (32 ft curb-to-curb roadway) and includes 5 ft wide sidewalks on both sides of the bridge that would connect to the existing sidewalks in the approaches. Each side of the bridge will include an overlook platform extending approximately 5 ft outside of the sidewalk throughway. Joel stated that the major components of the bridge structure have been thoroughly investigated and the Department is supporting this design based on the engineering design needs, but the Department would entertain input on the more ancillary components of the bridge that are of interest to the Towns such as the size of the overlook platforms and the type and coloration of the bridge rail and lighting. The cost of the replacement bridge is estimated to be $12.9M. Major benefits of the replacement are that the second sidewalk improves pedestrian safety and addresses local concerns, traffic disruptions are minimized during construction (estimated to be approximately 4-6 weeks to complete roadway tie-ins), and the replacement is a low maintenance structure.

Measures Taken to Reduce Right-of-Way Impacts:

Rick reported that the curved alignment reduces impacts along both approaches and minimizes the overall project length. At the Brunswick end, the approach roadway would be raised up to approximately two feet and this will allow the existing drive configurations to be maintained to be in close conformance to their existing conditions. A new retaining wall would replace the existing retaining wall along the back side of the westerly approach sidewalk. A traffic barrier would be mounted to the top of the wall to reduce impacts to the existing paved parking area. Most of the setback area is currently a non-paved grassed area. The impact to the paved parking area would be small with no loss of parking spaces.

At the Topsham approach, a short return retaining wall located along the back side of the sidewalk and extended bridge and guardrail with steepened slopes would be provided to reduce impacts outside of the existing MaineDOT right-of-way. Impacts to the existing paved parking area would be limited to the extreme southeast corner of the lot. The limited impact is expected to impact up to one existing parking space.

Open Discussion and Questions

Following the presentation, the following discussion points and questions were posed by the meeting participants:

Q: Were comparative user costs of lost business during the construction considered in the investigation of the alternatives?
A: Joel reported that they were not, but that if considered they would also favor the replacement since the traffic disruptions associated with a rehabilitation were much longer and more significant.

Q: Are the sidewalks on the bridge without a separation barrier a safety concern?
A: Joel explained that sidewalks on bridges are typically taller than approach roadway curb. The increased height of bridge sidewalks improves safety as the curb is not easily mounted. This same standard, that provides a 9” curb reveal, is used on all new bridges by the Department.

Q: A question was asked by Jim Howard (Priority Properties) that if fill was used along the river and the border of his property for construction access as was done for the 2006 bridge work, if this fill could remain permanent?
A: Joel stated that he was aware of this being a potential request, and reported that this could be an issue related to environmental permitting, but would make sure that those coordinating these efforts for the Department are aware of the request. Jim’s “park” would require public park over private property. Nancy

Randolph has done the park development plans. Park requires a public/municipal agreement after a private/municipal agreement.

C: Joel first commented that the Department was aware of the planned Riverwalk Park improvements planned in Topsham along the riverbank at Priority Properties and stated that the Department will be coordinating further with the Town on these plans.

Q: Joel then asked about coordination needs between the bridge project and planned park project, and if there were any specific improvements that needed to be considered?
A: Several people commented on the coordination of an opening in the extended bridge rail at the top of the retaining wall to meet the trail, and the selection of the rail type. Joel responded that these elements will be coordinated.

C: There was a discussion of strategies for presenting bridge rail and light pole options. The consensus of the group was that just showing the utilitarian option for presentation to the public would not likely be the best approach, but rather showing this solution and a couple of comparative decorative options may be the best approach. The Texas Rail and a combination concrete rail with a top steel handrail were mentioned as possible options. It was also mentioned that decorative light poles are located in the existing approach roadways on both sides of the bridge, and the new bridge light poles should be selected to closely match the existing light poles in the approaches. Also, a view of going through the old "tunnel with a biker in it" structure vs. the wide, open, aesthetic, bike and pedestrian friendly, structure.

Q: A question was asked if renaming of the new bridge had been considered and if it was possible?
A: Joel responded that this may be an option and that he could look into this further.

Reported by: Rick Hebert, P.E.

TYLIN INTERNATIONAL
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MEMORANDUM

Date: February 18, 2016

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Project Update Meeting

CC: File, All Attendees

This memo serves to summarize the discussion at the Brunswick-Topsham, Frank J. Wood Bridge Update Meeting with representatives from the Towns of Brunswick and Topsham, TY Lin International, and MaineDOT on February 16, 2016.

Attendees:

Maine DOT	Affiliation	Phone	Email
Joel Kittredge	MaineDOT	207-624-3550	Joel.kittredge@maine.gov
Norman Baker	TYLI	207-347-4349	Norman.baker@tylin.com
Rich Roedner	Town of Topsham	207-725-5821	rroedner@topshammaine.com
John Shattuck	Town of Topsham	207-650-0012	jshattuck@topshammaine.com
Rod Melanson	Town of Topsham	207-725-1724	rmelanson@topshammaine.com
Carol Eyerman	Town of Topsham	207-725-1724	ceyerman@topshammaine.com
Linda E. Smith	Town of Brunswick	207-721-0292	lsmith@brunswickme.org
John Eldridge	Town of Brunswick	207-725-6659	jeldridge@brunswickme.org
Anna Breinich	Town of Brunswick		

Joel began the meeting by welcoming everyone and thanking them for participating in this update meeting. He then requested Norm to step through the preferred alternative that the Department wants to recommend as a solution to this improvement project.

The preferred alternative is a replacement bridge on a curved alignment immediately upstream of the existing bridge. The roadway would be 32’ curb to curb consisting of two 11’ travel lanes with 5’ shoulders/bikeways. 5’ wide raised sidewalks would be placed on each side of the roadway. These would match into the sidewalks that currently exist today. On the Brunswick approach, the easterly side would tie into the sidewalk that runs along the Town Park. On the Topsham approach, the sidewalk would continue with a crosswalk through the commercial entrance to the SeaDog parking that ends with a new curb-cut accessing the sidewalk that runs along the dentist office. The Department feels that sidewalks on both sides of the roadway are needed along this corridor for safety reasons. There is significant development on both sides of the river and both sides of the roadway that encourages pedestrian traffic. The Department wants to discourage mid-block crosswalks that exist today and having two sidewalks running the entire length of the bridge helps this.

What is the schedule of the project? The Department would like to pursue a 2018 construction start but funding is currently not in place for this. The construction would take 1-2 years to construct the new bridge and another 5-7 months to remove the existing. Traffic be maintained on the existing keeping disruptions to a minimum. We would look towards a 1-2 month closure to tie the new bridge in to the roadways. This will be examined in greater detail before the project is advertised for construction to see if some incentives would help reduce these impacts.

Why was the wider shared-use-path on the upstream side of the bridge discounted? The investigation looked at widening the upstream sidewalk so that it could act as a shared-use-path that would connect to the future River Loop that is being considered. 12’ wide is the desired width of shared-use-paths with a minimum width of 10’. They are generally separated from traffic with a traffic barrier. We investigated this additional width from a cost point of view as well as impacts to the approaches. The Topsham side of the river has very little approach impacts other than needing a little wider taking of property. The Brunswick side results in having to relocate the drive entrance to the mill complex parking to along the drive accessing Brookfield Power. By doing so, at least one third of the current parking would be eliminated and this would also require retaining walls to avoid additional impacts to parking or to the Brookfield Power driveway. These impacts potentially could be damaging enough to require relocation of the businesses within the mill complex. The additional construction cost impact for this was $1.3 million plus associated Right-of-Way costs.

Why not lower the sidewalk to roadway height and consider the sidewalk and shoulder as a shared-use-path? This would require a traffic barrier that effectively would prevent traffic from accessing the shoulder should they break down or need to stop or pull off the roadway in case of an emergency.

Joel asked if providing lookouts along the bridge might be a compromise to the wider sidewalk. There was a general consensus that this would be helpful in many way, providing refuge for walkers to avoid bicyclists on the sidewalk, providing an area to view both the river downstream as well as the dam upstream while not interfering with pedestrian/bicycling access. There was agreement that this should be explored.

Is the only option for bridge railing what is shown in the bridge plan section, a 4 bar steel traffic/pedestrian rail? We can explore other railings but they would all have to meet the crash-worthiness requirements of today. This will be explored and if there are any options available, this will be presented along with any financial implications the Towns may need to consider.

The need for bridge lighting was mentioned and that the Towns would be looking for ornamental lighting used here instead of high mast lighting.

The meeting broke up with Joel thanking everyone for participating and that he hopes to be able to move this forward to a public meeting within the next 3-4 months.

Reported by: Norman Baker, P.E.

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MEMORANDUM

Date: February 3, 2016

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Project Team Meeting

CC: File, All Attendees

This memo serves to summarize the discussion at the Brunswick-Topsham, Frank J. Wood Bridge Team Meeting between representatives of T.Y. Lin International (TYLI), MaineDOT, and Brookfield Power on January 28, 2016.

Attendees:

Maine DOT	Affiliation
Patrick McDonough	Brookfield
Steve Michaud	Brookfield
Jeff Folsom	MaineDOT
Joel Kittredge	MaineDOT
Devin Anderson	MaineDOT
Kristen Chamberlain	MaineDOT
Wayne Frankhauser	MaineDOT
Norman Baker	TYLI

The objectives of this meeting were to present a summary of the preferred improvement alternatives for the Frank J Wood Bridge and discuss issues and concerns Brookfield Power may have with these alternatives.

Norm Baker opened the meeting with a brief presentation:

- Project Need. The bridge is in need of structural improvements due to its deteriorated state. The bridge deck is in poor condition and in need of replacement. There are numerous structural members that are deficient and do not rate out to support legal loads. Also, the bridge is in need of paint to protect it from further deterioration.

  The existing bridge has some functionality and mobility issues that need to be addressed. Both approaches have sidewalks on both sides of the road at or near the bridge with the bridge having one on only the upstream side. Also, the shoulder widths across the bridge are substandard for bicycle use.

- Project History. At the preliminary public meeting held in February, 2015, the attendees expressed the need for sidewalks on both sides of the bridge. They also wanted a wider upstream sidewalk to act as a shared use path as part of their River Walk facility in the development phase. They expressed the need for bicycle lanes on the bridge. Some expressed needs to save the existing bridge as well as others expressed need to replace it.

  In June, 2015, the MaineDOT presented to Brookfield Power 3 alignment alternatives for discussion, upstream, downstream, and on-alignment. Brookfield preferred the rehabilitation of the existing bridge as the preferred alternative or a downstream alignment as a replacement alternative. They also expressed the need to avoid the fish ladder on the Brunswick side just upstream of existing bridge.

In September, 2015, the MaineDOT presented bridge width alternatives to the Town Officials for input. The Towns expressed desires for 2 sidewalks and/or a wider upstream sidewalk.

- Preferred Alternatives Discussion. The preferred alternatives are presented to Brookfield were the Rehabilitation Alternative and the Upstream Replacement Alternative.

  Rehabilitating the existing bridge would require a 5-7 month road closure to replace the roadway deck. It would also cause additional traffic disruptions for a second season to paint the bridge.

  The upstream alignment provides for a smooth fit into existing approaches with no property relocations anticipated. It utilizes ledge outcrops that facilitate easier pier construction. There will be minimal traffic disruptions as the entire structure can be constructed with traffic utilizing the existing bridge.

  The downstream alternative was discounted due to potential impacts to the dental office on the Topsham side of the bridge and to the Brunswick Town Park. Also, due to the bridge foundations moving closer to the SeaDog restaurant and parking facility, water elevations increase significantly just downstream of the new alignment. This increase would severely impact the SeaDog restaurant and likely require taking that property and relocating the business.

- Brookfield concerns about current preferred alternatives.
  - Currently the fish ladder gets variable shading from traffic crossing the existing bridge during early morning traffic. This tends to scare the fish from using the ladder. The new alignment upstream should try to mitigate this.
  - FERC review will be needed with any impacts or disturbances within the FERC Boundary. This is not necessarily insurmountable but will require review time. The sooner plans of the proposed improvements can be sent to FERC for review, the better this is for any scheduled improvements.
  - Brookfield will require 24-7 access to their facility. The Contractor will need to accommodate this.
  - Contractor safety is a concern because the water elevation downstream of the dam can change abruptly. There is little storage above the dam and, if power generation is not needed, the flood gates and spillway must be utilized to avoid flooding upstream of the dam. Water elevation can rise within a few minutes.
  - Assurance will be needed that backwater from the new structure as well as from any temporary condition does not impact power generation.
  - Brookfield would expect the MaineDOT to pay for any studies required by their regulatory agencies.

Joel mentioned that an internal meeting was scheduled for February 2 to discuss the direction of the project and that he would be informing Brookfield of the results of that meeting.

Reported by: Norman Baker, P.E.

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MEMORANDUM

Date: October 2, 2015

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Project Update Meeting

CC: File, All Attendees

This memo serves to summarize the discussion at the Brunswick-Topsham, Frank J. Wood Bridge Update Meeting with representatives from the Towns of Brunswick and Topsham, TY Lin International, and MaineDOT on September 30, 2015.

Attendees:

Maine DOT	Affiliation	Phone	Email
Joel Kittredge	MaineDOT	207-624-3550	Joel.kittredge@maine.gov
Norman Baker	TYLI	207-347-4349	Norman.baker@tylin.com
Rich Roedner	Town of Topsham	207-725-5821	rroedner@topshammaine.com
John Shattuck	Town of Topsham	207-650-0012	jshattuck@topshammaine.com
Rod Melanson	Town of Topsham	207-725-1724	rmelanson@topshammaine.com
Carol Eyerman	Town of Topsham	207-725-1724	ceyerman@topshammaine.com
Linda E. Smith	Town of Brunswick	207-721-0292	lsmith@brunswickme.org
John Eldridge	Town of Brunswick	207-725-6659	jeldridge@brunswickme.org
Anna Breinich (Call-in)	Town of Brunswick		

Joel began the meeting by welcoming everyone and thanking them for participating in this update meeting. He then requested Norm to step through the 3 alternatives that are currently under investigation for improvements to this river crossing. These 3 alternatives are: (1) Rehabilitation of the existing bridge, (2) Replacement upstream of the existing bridge, and (3) Replacement on existing alignment. Presented at the meeting was a survey plan view and an aerial plan view of the upstream alignment with its associated profile. It was explained that a downstream alignment had been explored but was ruled out because of the dramatic affects it had on water elevations immediately downstream of the bridge raising elevations up to 7' higher than current conditions.

Upstream Alignment

This is a curved alignment, 835' long bridge currently showing a total bridge width of 45'+/- with two 12' lanes, two 4' shoulders, and two 5' sidewalks. This bridge width configuration was chosen as an acceptable roadway that meets design criteria, fits in well with the existing roadway, accommodates bicycle usage within the shoulders, and provides pedestrian access on both sides of the roadway. It was explained that this cross-sectional configuration is not set in stone and has not been approved by the MaineDOT. Joel requested comments on this:

- How would the sidewalks tie into the existing conditions, especially the downstream sidewalk which doesn't exist today? Whatever cross-section was approved would tie into the existing approaches. The downstream sidewalk on the Topsham side of the bridge would tie into the sidewalk that currently runs next to the dentist office just north of the bridge and tie into the cross-walk just south of the park on the Brunswick side.

- Are there barriers between the sidewalks and the roadway? No, this would require additional bridge width. Each barrier or rail is rough 1.5’-2’.
- To better address the needs for a multi-use path that would be part of the River Walk Loop, the general consensus was that a wider upstream sidewalk would be preferred.
- The general consensus was a preference for 4’ shoulders and, if the lanes were reduced to 11’, the extra width be added to one of the sidewalks.
- Reductions of less than 5’ sidewalks was not excepted due to ADA concerns.
- The Towns wanted similar configurations as the Veteran’s Memorial Bridge or the Martin’s Point Bridge.

Rehabilitation of Existing Bridge

The rehabilitation alternative is still considered a viable option as its initial project cost is expected to be less than the cost of a replacement alternative. A life cycle analysis will be part of this investigation identifying whether it is the most cost-effective solution. Future inspection and painting needs for this bridge is expected to be quite expensive and these costs should be considered when determining its cost-effectiveness.

Norm explained that, because closure of the bridge would likely be needed to make some of the repairs deemed necessary, a consultant experienced in construction scheduling has been brought on the team, On Point Construction Services. They have been tasked with estimating the closure time needed to make the repairs assuming a normal, 8-12 hour construction day. They have been asked to determine what the premium cost might be to accelerate that timeframe to determine the cost of reducing closure times.

John Shattuck mentioned that a closure might work well with the Town’s desire to construct a roundabout at the Summer Street intersection. A Feasibility Study is currently underway on this.

Because the existing bridge is currently at maximum capacity for loading, all repairs are looking to not increase the existing dead load on the bridge. If a downstream sidewalk were to be added to the existing bridge, the needed repairs would have to reduce the loading on the trusses to accommodate the weight of the added sidewalk. This may be possible with the use of a lighter deck system, however, that comes at a cost premium yet to be determined.

Replacement on Existing Alignment

Norm explained that a replacement on existing alignment would likely require a temporary bridge during the construction phase. The construction of a replacement structure would take a minimum of two years and Joel stated that closure of the bridge for that time would not get much support from the MaineDOT. The Towns agreed with this as well.

Another aspect of an on-alignment replacement is the need for a longer main span over the dam sluiceway channel or main part of the river. The sluiceway widens considerably as it approaches the existing bridge. To avoid placing a bridge support within the limits of the sluiceway will require a longer span than what is proposed for the upstream alignment which tends to increase the overall bridge cost.

On Point has been asked to estimate the cost of constructing a temporary bridge immediately upstream of the existing to be used when improvement comparing alternatives.

General

The general consensus at the meeting was that if a replacement alternative becomes the preferred alternative, renderings should be prepared that show what the bridge will look like from different reference points. Views without the existing bridge is recommended as well as views form the sidewalk(s).

Sidewalks on both sides of the roadway is preferred over just one on the upstream stream. However, the upstream sidewalk should be wider than the 5‘currently shown.

Would like to see a railing between the sidewalk and roadway.

Joel concluded the meeting by thanking everyone again for attending and participating. He suggested that we meet again in the future with bridge cross-sections that show potential roadway/sidewalk configurations and associated costs.

Reported by: Norman Baker, P.E.

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MEMORANDUM

Date: July 1, 2015

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Progress Meeting with Brookfield Renewable Energy Partners

CC: File, All Attendees, Joel Kittredge

This memo serves to summarize the discussions of a meeting on June 29, 2015 between TY Lin International, Maine DOT, and Brookfield Renewable Energy Partners (Brookfield) regarding progress design efforts pertaining to improvements to the Frank J. Wood Bridge over the Androscoggin River in Brunswick-Topsham.

Attendees:

Maine DOT	Affiliation
Joel Kittredge	MaineDOT
Richard Myers	MaineDOT
Jeff Folsom	MaineDOT
Wayne Frankhauser	MaineDOT
Norman Baker	TYLI
Rick Hebert	TYLI
Steve Michaud	Brookfield
Nate Stevens	Brookfield
Patrick McDonough	Brookfield
Dick Cole	Brookfield

Items of Discussion

1. Progress Design Presentation

Norm presented progress design information:

- Noting, the bridge improvement project includes consideration of the rehabilitation and preservation of the existing steel truss bridge and replacement alternatives.
- The existing bridge is an 803 ft (312ft – 314ft – 177') three span steel through truss bridge supported on solid shaft gravity wall piers and gravity abutments founded on exposed bedrock.
- The existing bridge was built in 1931 with rehabilitations in 1985, 2006, and 2015.
- Improvements to the existing bridge will include as a minimum, the replacement of the existing bridge deck, strengthening repair of structurally deficient members, and painting of the steel truss.
- Replacement alternatives considered to date include:
  - 835 ft 5 span bridge on a curved alignment located upstream of the existing bridge (plan and profiles drawings presented)
  - 625 ft 4 span bridge on an offset tangent alignment downstream of the existing bridge (plan and profiles drawings presented)
  - 800 ft 5 span bridge on an offset tangent alignment downstream of the existing bridge
  - 800 ft 5 span bridge on existing alignment

  Discussions for this meeting were focused on the first two replacement options listed above.

- The full evaluation of improvement options will consider hydrology and hydraulic analysis, environmental and right-of-way impacts, maintenance of traffic during construction (including access to abutting properties), span arrangement, structure type, future maintenance, constructability, and cost. To date only progress efforts have been made for each of these criteria.

2. Review of Preliminary Hydraulics

TYLI noted the following regarding the alternatives presented:

- Two-dimensional hydraulic modeling has been conducted by TYLI's specialty sub-consultant (West Consultants) for the existing bridge and for the 625 ft downstream replacement bridge option.
- The distribution of discharge through the powerhouse channel at the Brunswick side of the river, flood gates located on the Topsham side of the channel, and over the mid-channel spillway were considered in the hydraulic analysis. The distribution of flow is based on data provided by Brookfield to the design team.
- The river channel narrows dramatically through the affected reach of the river downstream of the dam, through the bridge, and through to a section of the river located approximately 200 ft downstream of the existing bridge.
- The channel topography is highly variable and significantly influences the flow. In general the Brunswick side of the channel varies between elevation -15 and -5 in the deepest portions of the channel. The Topsham side of the channel is significantly higher and varies between elevation -5 and +5 in the deepest portions of the channel.
- The preliminary hydraulic model includes some extrapolation of survey data within the channel on the upstream side of the bridge for areas that include the Brunswick side of the channel below the powerhouse to a section located approximately 75 ft upstream of the existing bridge and for an approximate 100 ft strip located immediately below the mid-channel spillway and the flood gates located on the Topsham side of the channel.
- It was noted by MaineDOT and Brookfield that survey in this area was being coordinated between the Department and Brookfield. The work was still pending but expected to be completed in the near future during low flow conditions. This data once received will be incorporated into final hydraulic modeling for the alternatives being considered.
- Hydraulic analysis of the 835' upstream bridge option had not yet been conducted, but it was expected that there will be little difference between the existing bridge and this alternative. The piers for this alternative align better with the flow, and more area is available downstream to convey flow
- The water surface elevation (WSE) near the Topsham side of the channel will govern bridge clearance requirements. The WSE at the Brunswick side of the channel is significantly lower for all discharges.
- For both the existing bridge and the downstream replacement option, water on the Topsham side of the channel piles up immediately downstream of the bridges and against the retaining walls of the Seadog parking lot area and building structure. This developed area as well as natural streambed outcrops block conveyance of flow downstream and force the water to turn nearly 90 degrees and flow near parallel to the bridge to be actively conveyed downstream. Both the existing and proposed bridge piers on this side of the channel are not or cannot be well aligned with the flow. The poorly aligned piers further exacerbate the issue.
- Comparing the existing conditions to the downstream option, the following conditions were noted by TYLI:
    - In general the WSE at the channel boundaries do not vary significantly, except for the area located adjacent the proposed Topsham side abutment and extending downstream to the upper side of the Seadog property. In this area the WSE will rise 4 ft to 6 ft in comparison to existing conditions. The maximum WSE across the channel will be no higher than the existing condition at the location of the existing bridge. Along the Seadog property parking lot, the maximum WSE of approximately El. 28 would be well contained within the channel along the existing vertical masonry wall and would be well below the parking lot at El 35. It was noted by MaineDOT that the Seadog building had a floor below the parking lot elevation. Data on this floor elevation is not available in the current survey. MaineDOT will include obtaining this information in the pending survey. **Upon further review by TYLI of site photo's following the meeting, it appears the lowest level of the Seadog building finish floor is well below the finish grade of the parking**

lot, and it is likely the rise in WSE adjacent the structure up from near El 22 to El 28 at this location will be unacceptable (near or possibly above the existing building finish floor elevation). This should be confirmed by survey of the building finish floor elevation. If confirmed, then the downstream option is not a viable option. Further, similar results would be expected for any other downstream option since required bridge piers would similarly be a major obstruction to downstream conveyance causing similar effects.

- Changes in the WSE along the Brunswick side of the channel and within the powerhouse discharge channel would not be significant for up to the 100-year discharge.
- Changes in WSE along the Topsham side of the channel in general would not be significant, except for a section located 100 ft downstream of the dam, where the WSE would rise approximately 1 ft in this local area. Brookfield conveyed that this was not likely a significant issue since the flow is split between the powerhouse channel and the flood gate channel.

3. Brookfield Comments

Brookfield noted the following regarding the alternatives presented:

- Major concerns include changes to the available waterway area and water surface profile immediately downstream of the dam. Water surface elevation changes (specifically an increase) below the powerhouse channel at the Brunswick side of the river are a significant concern. Changes influence available head and power generation. Water surface elevation changes downstream of the flood gates located on the Topsham side of the channel and over the mid-channel spillway were of less of a concern.
- Recent Floods of 1986 and 1987 included significant debris and ice, and asked if this was a consideration for design? MaineDOT responded that the proposed waterway clearance and spans configuration includes these concerns, and that adequate clearance and openings would be provided.
- The final evaluations should include both footprint impacts within the channel and impacts to the waterway conveyance.
- Brookfield's consultation period with FERC is typically a minimum of 90 days consisting of a "30 day" consultation and a "60 day" consultation. The total timeframe could extend to 120 days on a project where there are complexities and follow-up may be needed.
- Brookfield has future plans to replace the existing fishway, but this likely will not happen before the bridge project. Placement of a replacement bridge on the upstream side could limit options for a future fishway and this was a concern for Brookfield. Brookfield suggested this was likely an item that could be worked out, and suggested continued consultation as the project progresses would be advisable.
- Brookfield suggested that fisheries issues would likely be a significant concern, and understood that the Department was responsible for review of these concerns with permitting agencies. Mention was made to keeping the deepest portion of the channel clear of obstructions was preferable, that habitat is likely to be considered critical habitat for Atlantic Salmon and potentially other species, and that impact concerns would likely include not only footprint, but shadow footprint concerns (specifically with respect to the existing fishway).
- Brookfield conveyed that it would generally be preferable to have new impacts located further from the existing dam rather than closer (further toward the edge of the FERC boundary). From this perspective replacement downstream from existing would be considered more favorable. However, Brookfield recognized the Department needs to consider many design issues and potentially competing constraints, and suggested an upstream alignment could be considered, but that the evaluation of all alternatives should be thoroughly investigated and documented.
- Brookfield expressed concern about access to their facility via drive at the SW approach corner of the bridge both during construction and with a replacement. MaineDOT explained that access during construction would be maintained. Post construction access would be maintained as a minimum to match existing conditions or be improved with a replacement option.
- The consultations with FERC will likely need to consider both permanent structures and any temporary structures/impacts needed for contractor access to complete the work. Further consultation between the

Department and Brookfield regarding specific project impacts and review needs should be coordinated before a formal FERC submittal is prepared.

- Specific to the 625 ft downstream alternative and Pier 1, placement further away from the deep section of the powerhouse channel and more towards the Brunswick bank would be preferred.
- Relocating specific piers outside the FERC boundary would not be a specific concern for Brookfield. In any case, new piers would need to be located within the FERC boundary, and the preference would be to locate the piers to minimize influences to hydraulics and specifically hydraulics influencing the powerhouse channel.

Reported by: Rick Hebert, P.E.

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MEMORANDUM

Date: February 24, 2015

Re: Brunswick-Topsham, Frank J. Wood Bridge (WIN 22603.00) Kickoff/Initial Team Meeting

CC: File, All Attendees, Joel Kittredge

This memo serves to summarize the discussion at the Brunswick-Topsham, Frank J. Wood Bridge Kickoff/Initial Team Meeting between TY Lin International, Maine DOT, and representatives from the towns of Brunswick and Topsham on February 5, 2015.

Attendees:

Maine DOT	Affiliation
Joel Kittredge	MaineDOT
Roger Sproul	MaineDOT
Richard Myers	MaineDOT
Jeff Folsom	MaineDOT
Tim Soucy	MaineDOT
Kristen Chamberlain	MaineDOT
Jerry Quirion	MaineDOT
Christopher Knight	MaineDOT
Devin Anderson	MaineDOT
Bill Doukas	MaineDOT
Norman Baker	TYLI
Rick Hebert	TYLI
John Foster	Town of Brunswick
Anna Breinich	Town of Brunswick
John Eldridge	Town of Brunswick
Rich Roedner	Town of Topsham
John Shattuck	Town of Topsham
Marie Brillant	Town of Topsham
Ruth Lyons	Town of Topsham
Steve Michaud	Brookfield Energy

Items of Discussion

1. Design

- ➢ Norm presented existing conditions information:
  - ▪ Location description: U.S. Route 201 & SR 24 over Androscoggin River between the towns of Brunswick and Topsham, 0.2 miles north of the intersection with U.S. Route 1.
  - ▪ Corridor Priority: 3
  - ▪ Year Built: 1931 with rehabilitations in 1985 and 2006
  - ▪ Length & Widths: 815’ three-span (315’-315’-175’), 30’ curb to curb travelway, 32’-11” C.L. to C.L. truss, 38’-4” out to out including upstream side outbound cantilevered sidewalk.

  - Structure: Three span overhead steel truss with concrete filled grid deck on solid shaft piers and gravity wall abutments founded on shallow or exposed bedrock. The south pier is skewed 15º.
  - Federal Ratings: Sufficiency Rating = 52.1 (consider for rehabilitation or replacement), Deck = 5 (fair), Superstructure = 5 (fair), Substructure = 6 (satisfactory).
  - Maintenance Reports: Truss contains a significant number of fracture critical members and connection details requiring special inspections. A few members do not meet current load rating standards and strengthening is required to meet current loading standards. Noted deficiencies include leaking joints, and leakage of roadway drainage onto lower superstructure elements, poor paint system condition below and at the level of the deck, corrosion and section loss of below deck stringers and floor beams, scaling and spauling of abutment backwall concrete, serious crack at northeast pier pedestal, and bridge rail type.
  - In-Water Conditions: Waterway Adequacy rating = 9 (above desirable) and Scour Critical rating = 8 (stable above footing). Most recent underwater inspection confirmed south pier solidly founded on bedrock with minor edge ratholes in the seal.
- Norm presented traffic data:
  - AADT: 18,860 vehicles per day with 5% trucks (from SI&A).
  - Accident data has not yet been made available.
- A representative from one of the towns asked if any bike count data was available. It was noted by Joel Kittredge that bike and pedestrian data would be gathered.

- Action Items:
  - Joel to coordinate with MaineDOT Traffic Section and forward updated traffic and accident data to T.Y. Lin International (TYLI).
  - Joel to coordinate with MaineDOT Traffic Section to gather bike and pedestrian count data and forward to TYLI.

2. Survey

- Chris Knight reported on progress of survey. Ground survey along roadway corridor complete, bathometric survey of underwater sections of the channel above the upstream dam and below the bridge complete. Between the bridge and the upstream dam only partially complete due to strong currents and safety concerns during the initial survey. If additional information is needed within the channel then a request for additional bathometric survey data will need to be made. Outstanding survey data includes topographic survey being developed for aerial mapping which was due to be complete in approximately 2 weeks.
- Chris Knight noted that the FERC boundary has been mapped in the ROW file.
- It was noted that the dam owner may have channel survey data, and Joel, Jerry Quirion, and Chris will get together with to prepare a request for this data from the dam owner.

- Action Items:
  - TY Lin will review existing channel survey and will notify Joel K. if additional bathometric data is needed.
  - Joel, Jerry, and Chris will consult and prepare a request for additional available survey data from the dam owner.
  - MaineDOT to forward updated survey to TYLI in approximately 2 weeks.

3. Utilities

- Aerial: Jerry Quirion reported that there is power, telephone, and cable lines in the approaches and telephone lines carried by the existing bridge.
- Underground: Jerry reported there is a watermain and possibly sewer and gas lines in the approaches and the watermain is attached to the underside of the existing bridge. There is also underground power in the Topsham approach.
- He reported that a review by FERC will be needed since the FERC boundary extends from the upstream dam and below the existing bridge. Jerry noted FERC approval can be lengthy and that adequate time should be developed into the project schedule.

- Jerry also reported that initial discussions with the dam owner (Brookfield Renewable Energy) have taken place. Contacts have been identified for this project and coordination efforts are on-going. Jerry noted that information regarding nearby substations and future infrastructure plans for the dam should be requested from Brookfield Renewable.
- He noted that an off-alignment replacement option would require the need for an easement from Brookfield Renewable.

- Action Items: Joel & Jerry to coordinate requests for nearby substation and future infrastructure plans information for the dam from Brookfield Renewable.

- **Steve Michaud mentioned that an alignment downstream of the existing bridge would be less of an impact to the FERC boundary than an alignment upstream.**

4. Geotechnical

- A representative for MaineDOT's Geotechnical Section was not in attendance. It was noted by Norm Baker that the entire project area consists of shallow or exposed bedrock and that all existing substructures were founded on exposed or shallow bedrock. The same was anticipated for any replacement option.

- No action items.

5. Environmental

- Kristen Chamberlain presented the following information:
  - In-stream restrictions are anticipated to be similar to the "Lisbon-Durham Project", and more restrictive than typical projects. A no in-water/noise restriction between April and July was anticipated. Norm asked what the "no noise" restriction would be. Kristen noted, "pile driving" and similar activities producing significant ground vibrations. It was noted that pile driving was not anticipated.
  - The winter months will be the least restrictive timeframe during construction.
  - There is a 4f property (town park) adjacent the SE approach corner of the existing bridge. John Eldridge noted that the town will be willing to work with MaineDOT on park impact issues and that park plans can be made available to MaineDOT.
  - Kristen noted there were many historic eligible properties in the area adjacent to the project.

- Action Items: The town of Brunswick will provide plans for the existing park to MaineDOT.

6. Right of Way

- Roger Sproul noted the existing ROW is 40 ft to each side of C.L. of the existing roadway at the Topsham approach.
- He also noted the existing ROW is generally 30 ft to each side of C.L. of the existing roadway at the Brunswick approach, except the MaineDOT ROW extends along a wedge at the SE approach corner where the pre-existing bridge approach was located. This area is between the existing roadway and the town owned park property.
- Roger reported additional time will be required in the project schedule for ROW clear due to FERC involvement. A typical timeframe of approximately 8 months from Plan Impact Complete (PIC) was noted. The expectation is an additional 6 months may be required with FERC involvement (a total of approximately 14 months may be required for ROW clear past PIC).
- He also noted project development needs for FERC review are in addition to plan development needs for typical PIC, and this should be accounted for in the project effort and schedule.

- No action items.

7. Maintenance Issues

- Bill Doukas noted confirmation of the maintenance issues reported by Norm, and further noted that special inspections for the existing truss bridge are a significant expense for the Department in comparison to inspections for typical bridges. Rick Hebert requested information pertaining to inspection costs for inclusion in the life cycle cost analysis for rehabilitation option.

- Action Items: MaineDOT will provide truss inspection cost data to TYLI for inclusion in the bridge improvement studies.

8. **Local Information**
   - Rich Roedner noted that the town of Topsham is responsible for snow removal on the bridge sidewalk, and the existing sidewalk provides little clearance for snow removal. It was requested that improvement to the existing condition be considered for the bridge improvement study.
   - Joel mentioned he was aware of future trail extensions/improvements that were planned by the towns leading up to the existing bridge and requested related information from the towns. It was noted that the Town of Brunswick was looking to widen the sidewalk in front of Fort Andros along the existing SW approach.
   - Joel inquired about a timeframe and venue for the initial public meeting with the towns. This is scheduled for February 25th at 6:00 pm at the public library in Topsham.

   - Action Items:
     - Both the towns of Brunswick and Topsham will forward to Joel future planned trail extensions/improvements that lead to the existing bridge.

9. **Schedule & Budget**
   - Joel reported the bridge is only funded for PE through completion of the Preliminary Design Report (PDR) and not funded for the completion of the Preliminary Engineering or construction. $210,000 is available for development of the PDR and that $1,000,000 was anticipated for full PE development.
   - The funding split for the project is 80% Federal / 20% State.
   - The scheduled completion date for the PDR is October 28, 2015.

   - No action items.

10. **Design Options**
    - Norm reviewed design options that are anticipated under this study:
      - Rehabilitation to full replacement of the existing bridge is being considered.
      - Alignments options for a replacement bridge will consider an alignment near existing or a parallel alignment either upstream or downstream adjacent to the existing bridge.
      - Devin Anderson noted that drainage is a major concern of the current deck. The sides of the steel grid deck are not filled in and allow water from the roadway to spill onto lower elements of the existing bridge. He noted a complete replacement of all of the underside cross frame members of the existing truss with a new deck seemed like a reasonable option to consider given the condition of the existing bridge and the lack of drainage control on the existing bridge.
      - Maintenance of traffic will be considered in the engineering studies. The initial thoughts were that two lanes of traffic would need to be maintained for a replacement option, and that single lane alternating traffic or short term night closures could be considered for a rehabilitation option. The towns indicated that they may be open to a road closure option depending on the improvement option and duration of traffic impact.
      - A representative of one of the towns expressed concern regarding the aesthetic appearance of a “conventional replacement bridge” in comparison to the existing truss span.
      - A representative of one of the towns suggested the width of the bridge and accommodations for bikes be considered in the PDR study.

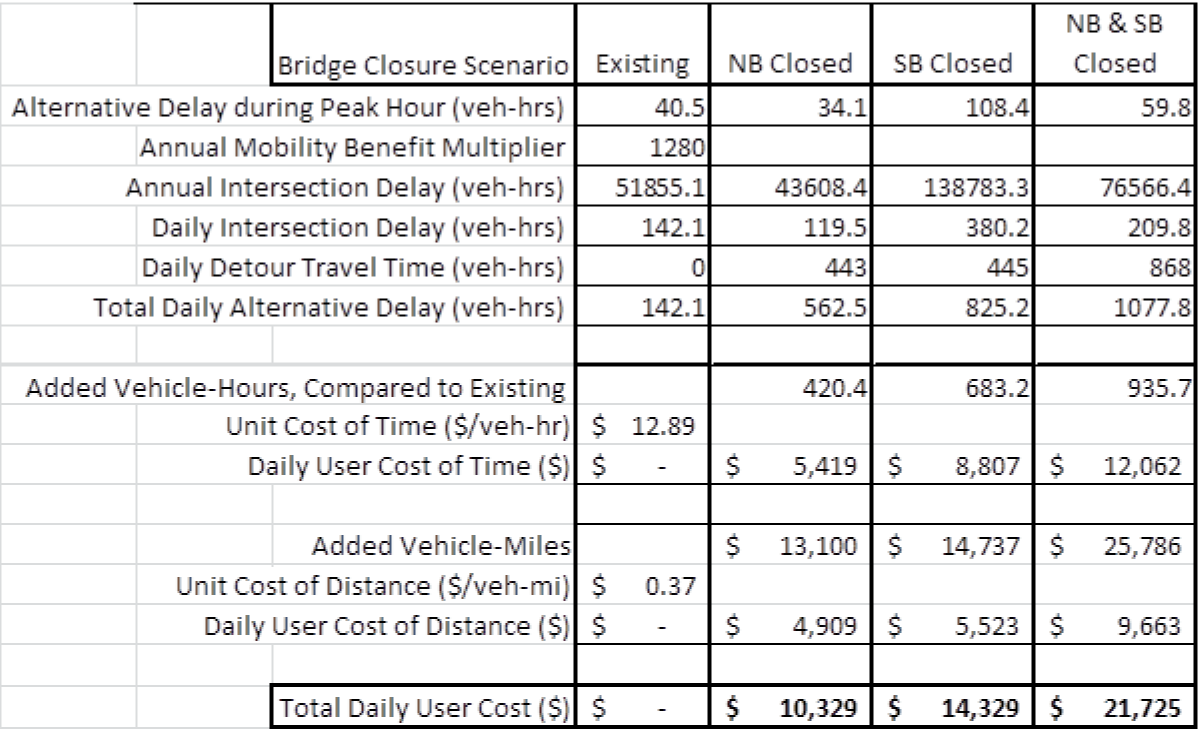
    - No action items.

Reported by: Rick Hebert, P.E.

User Costs

From: Hanscom, Ed
Sent: Friday, July 29, 2016 2:13 PM
To: Kittredge, Joel
Cc: O'Brien, Parker
Subject: RE: Brunswick---Topsham Frank J Wood user costs

Joel,
Below is a summary of the estimated user costs for three types of bridge closures. There are components to the user costs for the Frank J Wood bridge project: added vehicle-miles from detours due to a lane closure, added vehicle-hours due to a lane closure, and changes in intersection delay due to altered traffic patterns. Not surprisingly, the scenario in which both the northbound and southbound lanes are closed would have the highest daily user cost (close to $22,000), mostly due to added travel distance and travel time for detoured traffic. The scenario in which the southbound lane is closed would have the second highest daily user cost (more than $14,000). It also would have the highest intersection delay. The scenario in which the northbound lane is closed would have the lowest daily user cost (more than $10,000). If you need more information, please let me know. --- Ed



Bridge Closure Scenario	Existing	NB Closed	SB Closed	NB & SB Closed
Alternative Delay during Peak Hour (veh-hrs)	40.5	34.1	108.4	59.8
Annual Mobility Benefit Multiplier	1280			
Annual Intersection Delay (veh-hrs)	51855.1	43608.4	138783.3	76566.4
Daily Intersection Delay (veh-hrs)	142.1	119.5	380.2	209.8
Daily Detour Travel Time (veh-hrs)	0	443	445	868
Total Daily Alternative Delay (veh-hrs)	142.1	562.5	825.2	1077.8
Added Vehicle-Hours, Compared to Existing		420.4	683.2	935.7
Unit Cost of Time ($/veh-hr)	$ 12.89			
Daily User Cost of Time ($)	$ -	$ 5,419	$ 8,807	$ 12,062
Added Vehicle-Miles		$ 13,100	$ 14,737	$ 25,786
Unit Cost of Distance ($/veh-mi)	$ 0.37			
Daily User Cost of Distance ($)	$ -	$ 4,909	$ 5,523	$ 9,663
Total Daily User Cost ($)	$ -	**$ 10,329**	**$ 14,329**	**$ 21,725**

MATRIX OF ALTERNATIVES INVESTIGATED

Item/Alternate	**No Build** ***(Maintenance to extend bridge life 5 years. After 5 years, bridge would need to be closed or another alternative chosen.)***	**Alternate 1 - Replacement Bridge**	**Alternate 2 - Replacement Bridge**	**Alternate 3 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. No easterly sidewalk proposed.)***	**Alternate 4 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. Proposed new addition of easterly sidewalk.)***	**Alternate 5 - Replacement Bridge**
Alignment	**Existing alignment**	**Existing alignment**	**Curved upstream alignment**	**Existing alignment**	**Existing alignment**	**Parallel tangent downstream alignment**
Bridge Section	Two 11' lanes with two 4' shoulders and one 5' sidewalk. Shoulders include 2' wide continuous open grating for drainage.	Two 11' lanes with two 5' shoulders and two 5' sidewalks	Two 11' lanes with two 5' shoulders and two 5' sidewalks	a. Two 11' lanes with two 4' shoulders and one 5' sidewalk, OR b. Two 10' lanes with two 5' shoulders (accommodating bicyclists) and one 5' sidewalk	a. Two 11' lanes with two 4' shoulders and two 5' sidewalks, OR b. Two 10' lanes with two 5' shoulders (accommodating bicyclists) and two 5' sidewalks	Two 11' lanes with two 5' shoulders and two 5' sidewalks
Spans	805' three span (310' – 310' – 175')[1]	800' five span (137.5' – 175' – 175' – 175' – 137.5')	835' five span (80' simple span and 200' – 205' – 205' – 145' continuous)	805' three span (310' – 310' – 175')[1]	805' three span (310' – 310' – 175')[1]	800' five span (137.5' – 175' – 175' – 175' – 137.5')
Bridge Superstructure	Existing steel truss rehabilitated as described in August 2016 Inspection Report to get 5 years remaining life	Metalized steel girder with composite concrete deck	Metalized steel girder with composite concrete deck	Existing painted steel truss with composite concrete deck on new structural floor system	Existing painted steel truss with composite concrete deck on new structural floor system	Metalized steel girder with composite concrete deck
Meet Purpose & Need	No	Yes	Yes	Yes	Yes	Yes
Hydraulics	Match existing conditions	Not studied, expected to closely match existing conditions 1 pier located near center of Brunswick side powerhouse channel	Closely match existing conditions 2 piers located near edges of Brunswick side powerhouse channel	Match existing conditions	Match existing conditions	Regulatory Q_{100} water surface elevation > 6 ft above existing conditions along Bowdoin Mill complex and 4 to 5 ft above existing Seadog Restaurant patio deck and finish floor elevation. Unacceptable water surface variance.
Estimated Construction Duration	2 to 3 months	3.5 years (includes removal of existing bridge and construction of new bridge)	2.5 years (includes removal of existing bridge and construction of new bridge)	3 years (includes rehabilitation construction and painting)	3 years (includes rehabilitation construction and painting)	2.5 years (includes removal of existing bridge and construction of new bridge)
Maintenance of Traffic Impacts	2 to 3 months total continuous single NB lane closure.	Maintain two-way traffic with temporary bridge. 3 months total non-continuous single NB lane closure needed for installation and removal of temporary bridge approaches.	Maintain two-way traffic on existing bridge. 2 months total continuous single NB lane closure and detour needed to construct approaches of replacement bridge before shifting traffic onto replacement bridge.	Maintain two-way traffic with temporary bridge. 3 months total non-continuous single NB lane closure needed for installation and removal of temporary bridge approaches.	Maintain two-way traffic with temporary bridge. 3 months total non-continuous single NB lane closure needed for installation and removal of temporary bridge approaches.	Maintain two-way traffic on existing bridge. 2 months total continuous single NB lane closure and detour needed to construct approaches of replacement bridge before shifting traffic onto replacement bridge.
Constructability	Conventional construction means and methods,	Conventional construction means and methods	Conventional construction means and methods,	Conventional construction means and methods,	Conventional construction means and methods,	Conventional construction means and methods

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[1] Total bridge length is 805' between centerline bearings at the abutments. At each of the piers, there is about 5' between the two bearing lines for the individual spans, hence the additional 10' of length.

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Alignment	**Existing alignment**	**Existing alignment**	**Curved upstream alignment**	**Existing alignment**	**Existing alignment**	**Parallel tangent downstream alignment**
	limited access within and below truss		except 2 heavy (250 Ton) cranes needed to erect Span 2 girders	limited access within and below truss	limited access within and below truss	
Impacts under Section 106 of the National Historic Preservation Act *(Protected Resources Present in the Area: the NR-eligible Brunswick-Topsham Industrial Historic District and its contributing properties (including the Frank J. Wood Bridge); the NR-eligible Cabot Mill; the NR-eligible Summer Street Historic District and its contributing properties; and the NR-listed Pejepscot Paper Company.)*	No effect on the Brunswick-Topsham Industrial Historic District, Cabot Mill, Summer Street Historic District, or the Pejepscot Paper Company.	Due to the removal of the Frank J. Wood Bridge, there would be an “adverse effect” on the Brunswick-Topsham Industrial Historic District, the Cabot Mill, and the Pejepscot Paper Company. This alternative would result in a “no effect” to the Summer Street Historic District. Potential presence of archaeological resources is currently under review by the Maine Historic Preservation Commission.	Due to the removal of the Frank J. Wood Bridge, there would be an “adverse effect” on the Brunswick-Topsham Industrial Historic District, the Cabot Mill, and the Pejepscot Paper Company. This alternative would result in a “no adverse effect” to the Summer Street Historic District. Potential presence of archaeological resources is currently under review by the Maine Historic Preservation Commission.	Due to rehabilitation of the Frank J. Wood Bridge with similar in-kind materials, there would be a “no adverse effect” on the Brunswick-Topsham Industrial Historic District. This alternative would result in a “no adverse effect” to the Cabot Mill and the Pejepscot Paper Company. This alternative would result in a “no effect” to the Summer Street Historic District. Potential presence of archaeological resources is currently under review by the Maine Historic Preservation Commission.	Due to rehabilitation of the Frank J. Wood Bridge with similar in-kind materials, there would be a “no adverse effect” on the Brunswick-Topsham Industrial Historic District. This alternative would result in a “no adverse effect” to the Cabot Mill and the Pejepscot Paper Company. This alternative would result in a “no effect” to the Summer Street Historic District. Potential presence of archaeological resources is currently under review by the Maine Historic Preservation Commission.	Due to the removal of the Frank J. Wood Bridge, there would be an “adverse effect” on the Brunswick-Topsham Industrial Historic District, the Cabot Mill, and the Pejepscot Paper Company. This alternative would result in a “no effect” to the Summer Street Historic District. Potential presence of archaeological resources is currently under review by the Maine Historic Preservation Commission.

Item/Alternate	**No Build** ***(Maintenance to extend bridge life 5 years. After 5 years, bridge would need to be closed or another alternative chosen.)***	**Alternate 1 - Replacement Bridge**	**Alternate 2 - Replacement Bridge**	**Alternate 3 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. No easterly sidewalk proposed.)***	**Alternate 4 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. Proposed new addition of easterly sidewalk.)***	**Alternate 5 - Replacement Bridge**
Alignment	**Existing alignment**	**Existing alignment**	**Curved upstream alignment**	**Existing alignment**	**Existing alignment**	**Parallel tangent downstream alignment**
Impacts under Section 4(f) of the U.S. Department of Transportation Act *(Protected Resources Present in the Area: the NR-eligible Brunswick-Topsham Industrial Historic District and its contributing properties (including the Frank J. Wood Bridge); the NR-eligible Cabot Mill; the NR-eligible Summer Street Historic District and its contributing properties; the NR-listed Pejepscot Paper Company Historic District and its contributing properties; and the Town of Brunswick Park.)*	No Section 4(f) use on the Brunswick-Topsham Industrial Historic District, Cabot Mill, Summer Street Historic District, the Pejepscot Paper Company Historic District, or the Town of Brunswick Park.	This alternative would require the use of the Frank J. Wood Bridge, a contributing element to the Brunswick-Topsham Industrial Historic District, due to its removal. Additionally, this alternative would likely require the use of the Section 4(f)-protected Pejepscot Paper Company Historic District, Cabot Mill, and Town of Brunswick Park. This alternative would result in no use to the Summer Street Historic District. In accordance with 23 USC Section 144 (5), MaineDOT is required to offer the historic bridge for alternative use.	This alternative would require the use of the Frank J. Wood Bridge, a contributing element to the Brunswick-Topsham Industrial Historic District, due to its removal. Additionally, this alternative would likely require the use of the Section 4(f)-protected Pejepscot Paper Company Historic District, Cabot Mill, and Town of Brunswick Park. This alternative would result in no use to the Summer Street Historic District. In accordance with 23 USC Section 144 (5), MaineDOT is required to offer the historic bridge for alternative use.	This alternative would likely require the use of the Section 4(f)-protected Brunswick-Topsham Industrial Historic District (but no use on the Frank J. Wood Bridge), Pejepscot Paper Company Historic District, Cabot Mill, and Town of Brunswick Park. This alternative would result in no use to the Summer Street Historic District.	This alternative would likely require the use of the Section 4(f)-protected Brunswick-Topsham Industrial Historic District (but no use on the Frank J. Wood Bridge), Pejepscot Paper Company Historic District, Cabot Mill, and Town of Brunswick Park. This alternative would result in no use to the Summer Street Historic District.	This alternative would require the use of the Frank J. Wood Bridge, a contributing element to the Brunswick-Topsham Industrial Historic District, due to its removal. Additionally, this alternative would likely require the use of the Section 4(f)-protected Pejepscot Paper Company Historic District, Cabot Mill, and Town of Brunswick Park. This alternative would result in no use to the Summer Street Historic District. In accordance with 23 USC Section 144 (5), MaineDOT is required to offer the historic bridge for alternative use.
In-water Impacts	Permanent impacts: None Temporary impacts: None	Permanent impacts: 4 piers Riprap Shading Temporary impacts: Temporary work trestle Temporary bridge Cofferdams Rock removal	Permanent impacts: 4 piers Riprap Shading Temporary impacts: Temporary work trestle Cofferdams Rock removal	Permanent impacts: None Temporary impacts: Temporary bridge Cofferdam (Abutment 1)	Permanent impacts: None Temporary impacts: Temporary bridge Cofferdam (Abutment 1)	Permanent impacts: 4 piers Riprap Shading Temporary impacts: Temporary work trestle Cofferdams Rock removal

Item/Alternate	No Build ***(Maintenance to extend bridge life 5 years. After 5 years, bridge would need to be closed or another alternative chosen.)***	Alternate 1 - Replacement Bridge	Alternate 2 - Replacement Bridge	Alternate 3 - Rehabilitate Existing Truss Bridge ***(Westerly sidewalk remains. No easterly sidewalk proposed.)***	Alternate 4 - Rehabilitate Existing Truss Bridge ***(Westerly sidewalk remains. Proposed new addition of easterly sidewalk.)***	Alternate 5 - Replacement Bridge
Alignment	**Existing alignment**	**Existing alignment**	**Curved upstream alignment**	**Existing alignment**	**Existing alignment**	**Parallel tangent downstream alignment**
Impacts under Section 7 of the Endangered Species Act *(Protected Resources Present in the Action Area under National Marine Fisheries Service (NMFS) jurisdiction: Atlantic sturgeon (ATST), proposed Atlantic sturgeon critical habitat (ATST CH), Atlantic salmon (ATS), Atlantic salmon critical habitat (ATS CH), and Shortnose sturgeon (SNS).)* *(Protected Resources Present in the Action Area under U.S. Fish and Wildlife Service (USFWS) jurisdiction: Northern long-eared bat (NLEB).)*	No effect. No Section 7 consultation with USFWS would be required.	Preliminary determination of impacts to threatened and endangered species and critical habitat: *ATST: LAA *ATST CH: No jeopardy *ATS: LAA *ATS CH: LAA *SNS: LAA Formal Section 7 consultation with NMFS would be required. *NLEB: NLAA Informal Section 7 consultation, under the FHWA Programmatic Consultation, with USFWS would be required.	Preliminary determination of impacts to threatened and endangered species and critical habitat: *ATST: LAA *ATST CH: No jeopardy *ATS: LAA *ATS CH: LAA *SNS: LAA Formal Section 7 consultation with NMFS would be required. *NLEB: NLAA Informal Section 7 consultation, under the FHWA Programmatic Consultation, with USFWS would be required.	Preliminary determination of impacts to threatened and endangered species and critical habitat: *ATST: LAA *ATST CH: No jeopardy *ATS: LAA *ATS CH: LAA *SNS: LAA Formal Section 7 consultation with NMFS would be required. *NLEB: NLAA Informal Section 7 consultation, under the FHWA Programmatic Consultation, with USFWS would be required.	Preliminary determination of impacts to threatened and endangered species and critical habitat: *ATST: LAA *ATST CH: No jeopardy *ATS: LAA *ATS CH: LAA *SNS: LAA Formal Section 7 consultation with NMFS would be required. *NLEB: NLAA Informal Section 7 consultation, under the FHWA Programmatic Consultation, with USFWS would be required.	Preliminary determination of impacts to threatened and endangered species and critical habitat: *ATST: LAA *ATST CH: No jeopardy *ATS: LAA *ATS CH: LAA *SNS: LAA Formal Section 7 consultation with NMFS would be required. *NLEB: NLAA Informal Section 7 consultation, under the FHWA Programmatic Consultation, with USFWS would be required.
Permit Level under Section 404 and Section 10 of the Clean Water Act (U.S. Army Corps of Engineers)	No permit needed.	Individual Permit for jurisdictional in-water work	Individual Permit for jurisdictional in-water work.	Individual Permit for jurisdictional in-water work.	Individual Permit for jurisdictional in-water work.	Individual Permit for jurisdictional in-water work.
Essential Fish Habitat (EFH) Impacts under the Magnuson-Stevens Fishery Conservation and Management Act *(Project is located within designated EFH for Atlantic salmon; Other NOAA Trust Resources Present in the Action Area include Alewives, American shad, Blueback herring, smelt, eels.)*	No effect. No EFH consultation with NMFS would be required.	Due to the temporary and permanent in-water work proposed, this alternative would result in "adverse effects" to EFH. EFH consultation with NMFS would be required.	Due to the temporary and permanent in-water work proposed, this alternative would result in "adverse effects" to EFH. EFH consultation with NMFS would be required.	Due to the temporary in-water work proposed, this alternative would result in "adverse effects" to EFH. EFH consultation with NMFS would be required.	Due to the temporary in-water work proposed, this alternative would result in "adverse effects" to EFH. EFH consultation with NMFS would be required.	Due to the temporary and permanent in-water work proposed, this alternative would result in "adverse effects" to EFH. EFH consultation with NMFS would be required.

Item/Alternate	**No Build** ***(Maintenance to extend bridge life 5 years. After 5 years, bridge would need to be closed or another alternative chosen.)***	**Alternate 1 - Replacement Bridge**	**Alternate 2 - Replacement Bridge**	**Alternate 3 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. No easterly sidewalk proposed.)***	**Alternate 4 - Rehabilitate Existing Truss Bridge** ***(Westerly sidewalk remains. Proposed new addition of easterly sidewalk.)***	**Alternate 5 - Replacement Bridge**
Alignment	**Existing alignment**	**Existing alignment**	**Curved upstream alignment**	**Existing alignment**	**Existing alignment**	**Parallel tangent downstream alignment**
Impacts to the Brookfield Dam and Fish Ladder	No effect.	No permanent effects. Potential temporary affects if temporary bridge or trestles are placed upstream of existing bridge.	Potential permanent and temporary effects to be determined. Currently evaluating noise, vibration and shadow effects.	No permanent effects. Potential temporary effects if temporary bridge or trestles are placed upstream of existing bridge.	No permanent effects. Potential temporary affects if temporary bridge or trestles are placed upstream of existing bridge.	No effect.
Utility Impacts	None	Existing water and communications service may be relocated to new bridge	Existing water and communications service may be relocated to new bridge	Temporary support or relocation of water and communications service within limits of existing bridge required	Temporary support or relocation of water and communications service within limits of existing bridge required	Existing water and communications service may be relocated to new bridge
Right of Way Impacts	No permanent property impacts	No permanent property impacts	Permanent impacts to 2 Brunswick properties and 1 Topsham property	No permanent property impacts	No permanent property impacts	Permanent impacts to 2 Topsham properties
Maintainability	High maintenance. The bridge will no longer function after 5 years.	Low maintenance. 1 future painting and 6 pavings estimated over 100 years with minimal traffic disruption	Low maintenance. 1 future painting and 6 pavings estimated over 100 years with minimal traffic disruption	High maintenance. 3 future paintings, 1 deck replacement, and 2 substructure rehabilitations estimated over 75 years. Estimated 8 months of future traffic disruptions for each painting	High maintenance. 3 future paintings, 1 deck replacement, and 2 substructure rehabilitations estimated over 75 years. Estimated 8 months of future traffic disruptions for each painting	Low maintenance. 1 future painting and 6 pavings estimated over 100 years with minimal traffic disruption
Estimated Initial Construction Cost	$805,000	$16,000,000	$13,000,000	$15,000,000	$17,000,000	Not estimated
Estimated Life Cycle Cost	Not estimated	Not estimated, future inspection and maintenance costs similar to Alternate 2	$13,700,000	$21,000,000	$23,200,000	Not estimated, future inspection and maintenance costs similar to Alternate 2
Estimated Total Cost over Service Life of Bridge	Not estimated	Not estimated, future inspection and maintenance costs similar to Alternate 2	$17,300,000	$35,200,000	$38,200,000	Not estimated, future inspection and maintenance costs similar to Alternate 2

APPENDIX G

Traffic and Accident Data

STATE OF MAINE

FILE: RTE 201

INTERDEPARTMENTAL MEMORANDUM

CC: RTE 24

Date of Request: 2/6/2015 2/9/2015

Latest Date Needed By 2/12/2015

To: **Ed Hanscom**

Dept.: MDOT, Bureau of Planning

From: **Janet Damren**

Dept.: Bridge Program

Subject: **Request for Traffic Information**

Project Manager: **Joel Kittredge**

TOWN(S): Brunswick-Topsham

P.I.N. **22603.00**

Consultant Proj ☐

COUNTY: Cumberland,Sagadahoc

ROUTE: 201

LOCATION/ DESCRIPTION: Frank J. Wood Bridge #2016 carrying Rt. 201 over the Androscoggin River.

	Roadway Changes or Relocation (Attach Sketch)	Turning Movement needed (Provide Locations under Comments)	Other Please Describe Under Comments
Please Check Box if Applicable:	☐	☐	☐

Prep By: MAM	Sec. 1	Sec. 2	Sec. 3	Sec. 4	Sec. 5
Description of Sections	SR 24/US 201 (Maine St.) @ Brunswick-Topsham Town Line				
1 Latest AADT (Year)	18860(2013)				
2 Current **2015** AADT	18860				
3 Future **2025** AADT	20750				
4 Future **2035** AADT	22630				
5 DHV - % of AADT	10%	%	%	%	%
6 Design Hourly Volume	2263				
7 % Heavy Trucks (AADT)	3%	%	%	%	%
8 % Heavy Trucks (DHV)	3%	%	%	%	%
9 Direct.Dist. (DHV)	50%	%	%	%	%
10 18-KIP Equivalent P 2.0	189				
11 18-KIP Equivalent P 2.5	181				

Notes or Remarks: 18-Kip ESALS is based on 20 year life

PLEASE PROVIDE: (1) PIN NUMBER, (2) THE CURRENT & FUTURE YEARS FOR WHICH YOU WANT AADT CALCULATED, AND SEND TO MIKE MORGAN. (A LOCATION MAP IS NO LONGER NEEDED.)

RAFFIC REQUESTS WILL BE FILLED ON A FIRST COME / SERVE BASIS. PLEASE SEND WHEN PROJECT KICKS OFF!!!

Need Only Data Items Numbered ☐

Comments: New project.

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary Report

Report Selections and Input Parameters

REPORT SELECTIONS

☑ Crash Summary I ☑ Section Detail ☑ Crash Summary II ☐ 1320 Public ☐ 1320 Private ☐ 1320 Summary

REPORT DESCRIPTION

WIN 22603 Bridge 2016 area in Topsham and Brunswick

REPORT PARAMETERS

Year 2009, Start Month 1 through Year 2013 End Month: 12

Route: 0201X Start Node: 61018 Start Offset: 0 ☐ Exclude First Node
End Node: 44169 End Offset: 0 ☐ Exclude Last Node

Route: 0201S Start Node: 61017 Start Offset: 0 ☑ Exclude First Node
End Node: 61018 End Offset: 0 ☑ Exclude Last Node

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary I

Nodes

Node	Route - MP	Node Description	U/R	Total Crashes	Injury Crashes K	A	B	C	PD	Percent Injury	Annual M Ent-Veh	Crash Rate	Critical Rate	CRF
P61018	0201X - 0.02	Int of BOW ST MAINE ST US 1 SB OFF RAMP	9	27	0	0	3	7	17	37.0	7.732	0.70	0.97	0.00
												Statewide Crash Rate:	0.65	
61017	0201X - 0.05	Non Int MAINE ST	2	1	0	0	0	1	0	100.0	6.579	0.03	0.29	0.00
												Statewide Crash Rate:	0.14	
44168	0201X - 0.15	TL Brunswick Topsham	2	0	0	0	0	0	0	0.0	3.442	0.00	0.34	0.00
												Statewide Crash Rate:	0.14	
44169	0201X - 0.33	Int of CUT MAIN ST	2	11	0	1	1	2	7	36.4	6.864	0.32	0.29	1.11
												Statewide Crash Rate:	0.14	
Study Years:	5.00	NODE TOTALS:		39	0	1	4	10	24	38.5	24.617	0.32	0.42	0.75

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary I

Sections

Start Node	End Node	Element	Offset Begin - End	Route - MP	Section Length	U/R	Total Crashes	Injury Crashes K	A	B	C	PD	Percent Injury	Annual HMVM	Crash Rate	Critical Rate	CRF
61018 Int of BOW ST	61017 MAINE ST	3129822 US 1 SB OFF RAMP	0 - 0.03	0201X - 0.02 US 201	0.03	2	1	0	0	0	0	1	0.0	0.00094	212.53 Statewide Crash Rate: 186.45	592.97	0.00
61017 Non Int MAINE ST	44168	3122937	0 - 0.10	0201X - 0.05 US 201	0.10	2	24	0	0	2	8	14	41.7	0.00688	697.28 Statewide Crash Rate: 186.45	361.52	1.93
44168 TL	44169 Brunswick	3139863 Topsham	0.06 - 0.18	0201X - 0.15 US 201	0.12	2	11	0	0	1	2	8	27.3	0.00826	266.32 Statewide Crash Rate: 186.45	347.42	0.00
44168 TL	44169 Brunswick	3139863 Topsham	0 - 0.06	0201X - 0.15 US 201	0.06	2	2	0	0	0	1	1	50.0	0.00413	96.84 Statewide Crash Rate: 186.45	407.00	0.00
61017 Non Int MAINE ST	61018	3140105	0 - 0.03	0201S - 2.74 US 201 SB	0.03	2	0	0	0	0	0	0	0.0	0.00112	0.00 Statewide Crash Rate: 186.45	566.67	0.00
Study Years:	5.00			Section Totals:	0.34		38	0	0	3	11	24	36.8	0.02134	356.14	289.45	1.23
				Grand Totals:	0.34		77	0	1	7	21	48	37.7	0.02134	721.65	415.30	1.74

Crash Summary

Section Details														
Start Node	End Node	Element	Offset	Route - MP	Total Crashes		Injury Crashes				Crash Report	Crash Date	Crash Mile Point	Injury Degree
			Begin - End			K	A	B	C	PD				
61018	61017	3129822	0 - 0.03	0201X - 0.02	1	0	0	0	0	1	2012-35555	06/20/2012	0.04	PD
61017	44168	3122937	0 - 0.10	0201X - 0.05	24	0	0	2	8	14	2013-18026	07/24/2013	0.06	C
											2011-7088C	04/06/2011	0.06	C
											2011-6435	07/29/2011	0.06	C
											2010-11429C	06/07/2010	0.06	C
											2009-7561C	03/07/2009	0.06	C
											2011-12335	10/07/2011	0.06	PD
											2011-8327	08/17/2011	0.07	C
											2012-46893	12/06/2012	0.07	PD
											2010-21423C	09/26/2010	0.07	PD
											2013-18904	07/30/2013	0.07	PD
											2009-18572C	08/12/2009	0.08	B
											2011-2925	06/06/2011	0.08	PD
											2011-5231C	03/08/2011	0.09	B
											2013-14100	06/12/2013	0.09	C
											2011-7082C	04/05/2011	0.09	C
											2012-28480	05/14/2012	0.09	PD
											2009-7552C	03/04/2009	0.09	PD
											2012-35679	08/08/2012	0.10	PD
											2011-5238C	03/12/2011	0.10	PD
											2010-24254C	11/03/2010	0.10	PD
											2012-26141	04/07/2012	0.11	PD
											2009-11752C	04/29/2009	0.13	PD
											2009-12378C	05/28/2009	0.14	PD
											2011-2964	06/02/2011	0.14	PD
44168	44169	3139863	0 - 0.06	0201X - 0.15	2	0	0	0	1	1	2010-11414C	05/27/2010	0.17	C
											2013-18233	07/26/2013	0.20	PD

Crash Summary

Section Details

Start Node	End Node	Element	Offset Begin - End	Route - MP	Total Crashes	Injury Crashes K	A	B	C	PD	Crash Report	Crash Date	Crash Mile Point	Injury Degree
44168	44169	3139863	0.06 - 0.18	0201X - 0.15	11	0	0	1	2	8	2011-8298C	05/26/2011	0.23	B
											2009-34056C	10/09/2009	0.25	C
											2012-35874	08/08/2012	0.25	C
											2009-33716C	12/17/2009	0.27	PD
											2009-33621C	06/05/2009	0.29	PD
											2012-1615	01/17/2012	0.29	PD
											2012-24413	03/12/2012	0.30	PD
											2013-19054	08/01/2013	0.30	PD
											2012-22071	02/14/2012	0.31	PD
											2009-33611C	05/15/2009	0.31	PD
											2009-33613C	05/20/2009	0.31	PD
61017	61018	3140105	0 - 0.03	0201S - 2.74	0	0	0	0	0	0				
				Totals:	38	0	0	3	11	24				

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary II - Characteristics

Crashes by Day and Hour

	Hour of Day																									
	AM												PM													
Day Of Week	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	Un	Tot
SUNDAY	0	0	0	0	0	0	0	0	1	0	0	0	0	0	1	1	2	0	0	0	0	0	0	0	0	5
MONDAY	0	0	0	0	0	0	0	0	1	1	1	0	1	2	0	0	0	0	1	0	0	0	0	0	0	7
TUESDAY	0	0	0	0	0	0	0	0	0	2	0	0	1	4	1	2	2	0	0	0	0	0	0	0	0	12
WEDNESDAY	0	0	0	0	0	0	0	1	1	0	1	1	1	1	3	5	1	5	1	0	0	0	0	0	0	21
THURSDAY	0	0	0	0	0	0	0	0	1	0	0	0	2	1	0	2	3	3	0	0	0	0	0	0	0	12
FRIDAY	0	0	0	0	0	0	0	1	0	1	0	2	2	0	1	1	3	1	1	0	0	0	0	0	0	13
SATURDAY	0	0	0	0	0	0	0	0	0	0	0	2	2	0	1	0	0	1	0	0	1	0	0	0	0	7
Totals	0	0	0	0	0	0	0	2	4	4	2	5	9	8	7	11	11	10	3	0	1	0	0	0	0	77

Vehicle Counts by Type

Unit Type	Total	Unit Type	Total
1-Passenger Car	89	23-Bicyclist	2
2-(Sport) Utility Vehicle	31	24-Witness	14
3-Passenger Van	10	25-Other	0
4-Cargo Van (10K lbs or Less)	0	Total	178
5-Pickup	22		
6-Motor Home	0		
7-School Bus	1		
8-Transit Bus	0		
9-Motor Coach	0		
10-Other Bus	0		
11-Motorcycle	4		
12-Moped	0		
13-Low Speed Vehicle	0		
14-Autocycle	0		
15-Experimental	0		
16-Other Light Trucks (10,000 lbs or Less)	0		
17-Medium/Heavy Trucks (More than 10,000 lbs)	3		
18-ATV - (4 wheel)	0		
20-ATV - (2 wheel)	0		
21-Snowmobile	0		
22-Pedestrian	2		

Crash Summary II - Characteristics

Crashes by Driver Action at Time of Crash

Driver Action at Time of Crash	Dr 1	Dr 2	Dr 3	Dr 4	Dr 5	Other	Total
No Contributing Action	15	24	6	0	0	0	45
Ran Off Roadway	0	0	0	0	0	0	0
Failed to Yield Right-of-Way	8	0	0	0	0	0	8
Ran Red Light	2	0	0	0	0	0	2
Ran Stop Sign	0	0	0	0	0	0	0
Disregarded Other Traffic Sign	0	0	0	0	0	0	0
Disregarded Other Road Markings	0	0	0	0	0	0	0
Exceeded Posted Speed Limit	1	0	0	0	0	0	1
Drove Too Fast For Conditions	0	0	0	0	0	0	0
Improper Turn	0	2	0	0	0	0	2
Improper Backing	1	0	0	0	0	0	1
Improper Passing	2	1	0	0	0	0	3
Wrong Way	0	0	0	0	0	0	0
Followed Too Closely	29	14	3	1	0	0	47
Failed to Keep in Proper Lane	0	0	0	0	0	0	0
Operated Motor Vehicle in Erratic, Reckless, Careless, Negligent or Aggressive Manner	0	0	0	0	0	0	0
Swerved or Avoided Due to Wind, Slippery Surface, Motor Vehicle, Object, Non-Motorist in Roadway	0	0	0	0	0	0	0
Over-Correcting/Over-Steering	0	0	0	0	0	0	0
Other Contributing Action	1	1	0	0	0	0	2
Unknown	1	0	0	0	0	0	1
Total	60	42	9	1	0	0	112

Crashes by Apparent Physical Condition And Driver

Apparent Physical Condition	Dr 1	Dr 2	Dr 3	Dr 4	Dr 5	Other	Total
Apparently Normal	76	73	11	1	0	2	163
Physically Impaired or Handicapped	0	0	0	0	0	0	0
Emotional(Depressed, Angry, Disturbed, etc.)	0	0	0	0	0	0	0
Ill (Sick)	0	0	0	0	0	0	0
Asleep or Fatigued	0	0	0	0	0	0	0
Under the Influence of Medications/Drugs/Alcohol	0	0	0	0	0	0	0
Other	0	0	0	0	0	0	0
Total	76	73	11	1	0	2	163

Driver Age by Unit Type

Age	Driver	Bicycle	SnowMobile	Pedestrian	ATV	Total
09-Under	0	0	0	0	0	0
10-14	0	0	0	0	0	0
15-19	13	0	0	0	0	13
20-24	25	0	0	0	0	25
25-29	9	0	0	0	0	9
30-39	23	0	0	0	0	23
40-49	32	0	0	0	0	32
50-59	28	0	0	0	0	28
60-69	19	0	0	0	0	19
70-79	9	0	0	0	0	9
80-Over	2	0	0	0	0	2
Unknown	0	2	0	2	0	4
Total	160	2	0	2	0	164

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary II - Characteristics

Most Harmful Event

Most Harmful Event	Total
1-Overturn / Rollover	0
2-Fire / Explosion	0
3-Immersion	0
4-Jackknife	0
5-Cargo / Equipment Loss Or Shift	0
6-Fell / Jumped from Motor Vehicle	0
7-Thrown or Falling Object	0
8-Other Non-Collision	0
9-Pedestrian	0
10-Pedalcycle	1
11-Railway Vehicle - Train, Engine	0
12-Animal	0
13-Motor Vehicle in Transport	81
14-Parked Motor Vehicle	0
15-Struck by Falling, Shifting Cargo or Anything Set in Motion by Motor Vehicle	0
16-Work Zone / Maintenance Equipment	0
17-Other Non-Fixed Object	0
18-Impact Attenuator / Crash Cushion	0
19-Bridge Overhead Structure	0
20-Bridge Pier or Support	0
21-Bridge Rail	0
22-Cable Barrier	0
23-Culvert	0
24-Curb	0
25-Ditch	0
26-Embankment	0
27-Guardrail Face	0
28-Guardrail End	0
29-Concrete Traffic Barrier	0
30-Other Traffic Barrier	0
31-Tree (Standing)	0
32-Utility Pole / Light Support	0
33-Traffic Sign Support	0
34-Traffic Signal Support	0
35-Fence	0
36-Mailbox	0
37-Other Post Pole or Support	0
38-Other Fixed Object (wall, building, tunnel, etc.)	0
39-Unknown	2
40-Gate or Cable	0
41-Pressure Ridge	0
Total	84

Traffic Control Devices

Traffic Control Device	Total
1-Traffic Signals (Stop & Go)	12
2-Traffic Signals (Flashing)	1
3-Advisory/Warning Sign	0
4-Stop Signs - All Approaches	0
5-Stop Signs - Other	9
6-Yield Sign	12
7-Curve Warning Sign	0
8-Officer, Flagman, School Patrol	1
9-School Bus Stop Arm	0
10-School Zone Sign	0
11-R.R. Crossing Device	0
12-No Passing Zone	0
13-None	38
14-Other	3
Total	76

Injury Data

Severity Code	Injury Crashes	Number Of Injuries
K	0	0
A	1	1
B	7	8
C	21	27
PD	48	0
Total	77	36

Road Character

Road Grade	Total
1-Level	56
2-On Grade	19
3-Top of Hill	1
4-Bottom of Hill	1
5-Other	0
Total	77

Light

Light Condition	Total
1-Daylight	74
2-Dawn	0
3-Dusk	2
4-Dark - Lighted	1
5-Dark - Not Lighted	0
6-Dark - Unknown Lighting	0
7-Unknown	0
Total	77

Crash Summary II - Characteristics

Crashes by Year and Month

Month	2009	2010	2011	2012	2013	Total
JANUARY	1	0	0	1	0	2
FEBRUARY	0	0	0	1	0	1
MARCH	4	1	2	2	1	10
APRIL	1	0	2	1	1	5
MAY	3	2	2	1	2	10
JUNE	3	3	2	2	1	11
JULY	1	1	1	1	4	8
AUGUST	2	0	3	3	3	11
SEPTEMBER	0	3	1	0	2	6
OCTOBER	1	1	1	2	1	6
NOVEMBER	2	2	0	0	0	4
DECEMBER	2	0	0	1	0	3
Total	20	13	14	15	15	77

Report is limited to the last 10 years of data.

Crash Summary II - Characteristics

Crashes by Crash Type and Type of Location

Crash Type	Straight Road	Curved Road	Three Leg Intersection	Four Leg Intersection	Five or More Leg Intersection	Driveways	Bridges	Interchanges	Other	Parking Lot	Private Way	Cross Over	Railroad Crossing	Total
Object in Road	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Rear End / Sideswipe	17	0	8	15	0	10	9	0	0	0	0	0	0	59
Head-on / Sideswipe	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Intersection Movement	0	0	7	1	1	3	0	0	0	0	0	0	0	12
Pedestrians	1	0	0	0	0	1	0	0	0	0	0	0	0	2
Train	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Went Off Road	0	0	1	0	0	0	0	0	0	0	0	0	0	1
All Other Animal	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bicycle	1	0	1	0	0	0	0	0	0	0	0	0	0	2
Other	0	0	1	0	0	0	0	0	0	0	0	0	0	1
Jackknife	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Rollover	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Fire	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Submersion	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Thrown or Falling Object	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Bear	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Deer	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Moose	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Turkey	0	0	0	0	0	0	0	0	0	0	0	0	0	0
Total	19	0	18	16	1	14	9	0	0	0	0	0	0	77

Crash Summary II - Characteristics

Crashes by Weather, Light Condition and Road Surface

Weather Light	Dry	Ice/Frost	Mud, Dirt, Gravel	Oil	Other	Sand	Slush	Snow	Unknown	Water (Standing, Moving)	Wet	Total
Blowing Sand, Soil, Dirt												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Blowing Snow												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Clear												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	50	0	0	0	0	0	0	0	0	0	2	52
Dusk	2	0	0	0	0	0	0	0	0	0	0	2
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Cloudy												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	13	0	0	0	0	0	0	0	0	0	2	15
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary II - Characteristics

Crashes by Weather, Light Condition and Road Surface

Weather Light	Dry	Ice/Frost	Mud, Dirt, Gravel	Oil	Other	Sand	Slush	Snow	Unknown	Water (Standing, Moving)	Wet	Total
Fog, Smog, Smoke												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Other												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Rain												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	1	1
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	6	6
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Severe Crosswinds												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0

Maine Department Of Transportation - Traffic Engineering, Crash Records Section

Crash Summary II - Characteristics

Crashes by Weather, Light Condition and Road Surface

Weather Light	Dry	Ice/Frost	Mud, Dirt, Gravel	Oil	Other	Sand	Slush	Snow	Unknown	Water (Standing, Moving)	Wet	Total
Sleet, Hail (Freezing Rain or Drizzle)												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	0	0	0	0	0	0	0	0	0	0	0
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
Snow												
Dark - Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Not Lighted	0	0	0	0	0	0	0	0	0	0	0	0
Dark - Unknown Lighting	0	0	0	0	0	0	0	0	0	0	0	0
Dawn	0	0	0	0	0	0	0	0	0	0	0	0
Daylight	0	1	0	0	0	0	0	0	0	0	0	1
Dusk	0	0	0	0	0	0	0	0	0	0	0	0
Unknown	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL	65	1	0	0	0	0	0	0	0	0	0	77

Maine DOT Map

Map Generated on Monday, February 09, 2015 02:08:41 PM

Map Scale 1:5224

APPENDIX H

Preliminary Cost Estimates

Preliminary Cost Estimate

PROJECT:	Brunswick-Topsham, F.J. Wood Bridge #2016	**WIN:**	22603.00
Alternative 1:	Bridge Replacement. Five Span (137.5'-175'-175'-175'-137.5') Steel Girder on Existing Alignment. Deck Area: 803’ x 45.33' = 36,400 SF	**ESTIMATED BY:**	RMH

SUPERSTRUCTURE:	36,400	**SF**	**x**	$155.00	=	$5,642,000
ABUTMENTS:	36,400	**SF**	**x**	$17.50	=	$637,000
PIERS:	4	**EA**	**x**	$281,000.00	=	$1,124,000
COFFERDAMS:	3	**EA**	**x**	$200,000.00	=	$600,000
STRUCTURAL EXCAVATION & BORROW:	1,500	**CY**	**x**	$40.00	=	$60,000
RIPRAP:	300	**CY**	**x**	$80.00	=	$24,000
EXISTING BRIDGE REMOVAL:	1	**LS**	**x**	$1,000,000.00	=	$1,000,000
DETOUR AND/OR TEMPORARY BRIDGE:	1	**LS**	**x**	$4,000,000.00	=	$4,000,000
REHABILITATION CONTINGENCIES:				N/A	=	$0
MISCELLANEOUS (TCP'S, FIELD OFFICE, ETC.): (7%+$1M cost premium for work trestle) Excl exist. br. removal & detour				7%	=	$1,567,000
MOBILIZATION:				7%	=	$1,026,000
				STRUCTURE SUBTOTAL	=	**$15,680,000**
APPROACHES:	150	**LF**	**x**	$750.00	=	$113,000
MISCELLANEOUS:				10%	=	$12,000
MOBILIZATION:				10%	=	$13,000
				APPROACHES SUBTOTAL	=	**$140,000**
				TOTAL CONSTRUCTION COST	**=**	**$16,000,000**
PRELIMINARY ENGINEERING:				8%	=	$1,200,000
RIGHT OF WAY:					=	$10,000
CONSTRUCTION ENGINEERING:				6%	=	$1,000,000
OTHER:					=	$0
				TOTAL PROJECT COST	**=**	**$18,500,000**

Preliminary Cost Estimate

PROJECT:	Brunswick-Topsham, F.J. Wood Bridge #2016	**WIN:**	22603.00
Alternative 2:	Bridge Replacement. Five Span (80', 200'-205'-205'-145') Steel Girder on Upstream Curved Alignment. Deck Area: 838’ x 45.33' = 37,990 SF	**ESTIMATED BY:**	RMH

SUPERSTRUCTURE:	37,990	**SF**	x	$170.00	=	$6,459,000
ABUTMENTS:	37,990	**SF**	x	$18.00	=	$684,000
PIERS:	4	**EA**	x	$264,000.00	=	$1,056,000
COFFERDAMS:	2	**EA**	x	$200,000.00	=	$400,000
COFFERDAMS:	2	**EA**	x	$100,000.00	=	$200,000
STRUCTURAL EXCAVATION & BORROW:	1,500	**CY**	x	$40.00	=	$60,000
RIPRAP:	300	**CY**	x	$80.00	=	$24,000
EXISTING BRIDGE REMOVAL:	1	**LS**	x	$1,000,000.00	=	$1,000,000
DETOUR AND/OR TEMPORARY BRIDGE:	0	**LS**	x	$0.00	=	$0
REHABILITATION CONTINGENCIES:				N/A	=	$0
MISCELLANEOUS (TCP'S, FIELD OFFICE, ETC.): (7%+$1M cost premium for work trestle) Excl existing bridge removal				7%	=	$1,622,000
MOBILIZATION:				7%	=	$806,000
				STRUCTURE SUBTOTAL	=	**$12,315,000**
APPROACHES: (inc. add of $175K for walls)	500	**LF**	x	$550.00	=	$450,000
MISCELLANEOUS:				10%	=	$45,000
MOBILIZATION:				10%	=	$50,000
				APPROACHES SUBTOTAL	=	**$545,000**
				TOTAL CONSTRUCTION COST	=	**$13,000,000**
PRELIMINARY ENGINEERING:				9%	=	$1,200,000
RIGHT OF WAY:					=	$50,000
CONSTRUCTION ENGINEERING:				6%	=	$750,000
OTHER:					=	$0
				TOTAL PROJECT COST	=	**$15,000,000**

Preliminary Cost Estimate

PROJECT: Brunswick-Topsham, F.J. Wood Bridge #2016

WIN: 22603.00

Alternative 3: Existing Truss Bridge Rehabilitation. Deck Area: 808’ x 39.9' = 32,240 SF

ESTIMATED BY: RMH

SUPERSTRUCTURE:	32,240	**SF**	**x**	$226.00	**=**	$7,287,000
ABUTMENTS:	32,240	**SF**	**x**	$3.70	**=**	$120,000
PIERS:	1	**EA**	**x**	$100,000.00	**=**	$100,000
COFFERDAMS:	0	**EA**	**x**	$0.00	**=**	$0
STRUCTURAL EXCAVATION & BORROW:	0	**CY**	**x**	$0.00	**=**	$0
RIPRAP:	0	**CY**	**x**	$0.00	**=**	$0
EXISTING BRIDGE DECK REMOVAL:	1	**LS**	**x**	$180,000.00	**=**	$180,000
DETOUR AND/OR TEMPORARY BRIDGE:	1	**LS**	**x**	$4,000,000.00	**=**	$4,000,000
REHABILITATION CONTINGENCIES (w/o Temp Bridge):				15%	**=**	$1,154,000
MISCELLANEOUS (TCP'S, FIELD OFFICE, ETC.): Excl Temp Br.				7%	**=**	$619,000
MOBILIZATION:				10%	**=**	$1,346,000
				STRUCTURE SUBTOTAL	**=**	**$14,815,000**
APPROACHES:	50	**LF**	**x**	$1,000.00	**=**	$50,000
MISCELLANEOUS:				10%	**=**	$5,000
MOBILIZATION:				10%	**=**	$6,000
				APPROACHES SUBTOTAL	**=**	**$65,000**
				TOTAL CONSTRUCTION COST	**=**	**$15,000,000**
PRELIMINARY ENGINEERING:				10%	**=**	$1,450,000
RIGHT OF WAY:					**=**	$10,000
CONSTRUCTION ENGINEERING:				6%	**=**	$900,000
OTHER:					**=**	$0
				TOTAL PROJECT COST	**=**	**$17,500,000**

Preliminary Cost Estimate

PROJECT:	Brunswick-Topsham, F.J. Wood Bridge #2016			**WIN:**		22603.00
Alternative 4:	Existing Truss Bridge Rehabilitation with Exodermic Deck and Added D/S Sidewalk. Deck Area: 808’ x 47' = 37,980 SF			**ESTIMATED BY:**		RMH
SUPERSTRUCTURE:	37,980	**SF**	**x**	$227.00	**=**	$8,622,000
ABUTMENTS:	37,980	**SF**	**x**	$3.15	**=**	$120,000
PIERS:	1	**EA**	**x**	$100,000.00	**=**	$100,000
COFFERDAMS:	0	**EA**	**x**	$0.00	**=**	$0
STRUCTURAL EXCAVATION & BORROW:	0	**CY**	**x**	$0.00	**=**	$0
RIPRAP:	0	**CY**	**x**	$0.00	**=**	$0
EXISTING BRIDGE DECK REMOVAL:	1	**LS**	**x**	$180,000.00	**=**	$180,000
DETOUR AND/OR TEMPORARY BRIDGE:	1	**LS**	**x**	$4,000,000.00	**=**	$4,000,000
REHABILITATION CONTINGENCIES (w/o Temp Bridge):				15%	**=**	$1,354,000
MISCELLANEOUS (TCP'S, FIELD OFFICE, ETC.): Excl Temp Br.				7%	**=**	$727,000
MOBILIZATION:				10%	**=**	$1,511,000
				STRUCTURE SUBTOTAL	**=**	**$16,615,000**
APPROACHES:	100	**LF**	**x**	$1,000.00	**=**	$100,000
MISCELLANEOUS:				10%	**=**	$10,000
MOBILIZATION:				10%	**=**	$11,000
				APPROACHES SUBTOTAL	**=**	**$125,000**
				TOTAL CONSTRUCTION COST	**=**	**$17,000,000**
PRELIMINARY ENGINEERING:				9%	**=**	$1,450,000
RIGHT OF WAY:					**=**	$10,000
CONSTRUCTION ENGINEERING:				5%	**=**	$900,000
OTHER:					**=**	$0
				TOTAL PROJECT COST	**=**	**$19,500,000**

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 1
800 ft 5 Steel Girder 5 Span 137.5'-175'-175'-175'-137.5'

Element	Cost
Steel Girder with CIP Deck	$ 5,642,000
South Abutment & Wingwalls	$ 525,000
North Abutment & Wingwalls	$ 112,000
Solid Shaft Piers	$ 1,124,000
Cofferdam - Piers	$ 600,000
Structural Excavation & Borrow	$ 60,000
Riprap	$ 24,000
Miscellaneous Item Contingency @ 7% + $1M work trestle premium	$ 1,567,000
STRUCTURE SUBTOTAL =	$ 9,654,000
Approaches	$ 113,000
Miscellaneous Approach Item Contingency @ 10%	$ 12,000
APPROACH SUBTOTAL =	$ 125,000
Temporary Bridge	$ 4,000,000
Removal of Existing Bridge Structure	$ 1,000,000
Mobilization/Demobilization @ 7% Bridge, 10% Approaches	$ 1,039,000
CONSTRUCTION TOTAL =	$ 15,818,000
RIGHT OF WAY COST =	$10,000
TOTAL COST =	$15,828,000
SAY	**$16,000,000**

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 1
800 ft 5 Steel Girder 5 Span 137.5'-175'-175'-175'-137.5'
Steel Girder with CIP Deck

Item	Unit	Quantity	Unit Price	Cost
Hot Mix Asphalt, 9.5mm Nominal Maximum Size	Ton	470	$ 200.00	$ 94,000
Structural Concrete Roadway Slab on Steel Bridges	CY	1450	$ 1,230.00	$ 1,783,500
Reinforcing Steel, Fab & Del	LBS	325000	$ 0.88	$ 286,000
Reinforcing Steel, Placing	LBS	325000	$ 0.78	$ 253,500
High Performance Waterproofing Membrane	SY	2860	$ 23.00	$ 65,780
Structural Steel Fab. & Del., Welded	LBS	1374000	$ 1.46	$ 2,006,040
Structural Steel Erection	LBS	1374000	$ 0.36	$ 494,640
Shear Connectors	EA	8050	$ 7.50	$ 60,375
Steel Bridge Railing, 4 Bar	LF	1610	$ 190.00	$ 305,900
Bridge Expansion Joint, Finger; including Fabric Trough	LF	91	$ 1,560.00	$ 141,960
Permanent Concrete Transition Barrier	EA	4	$ 3,400.00	$ 13,600
Pot or Disc Bearings, Expansion or Fixed (including Installation)	EA	30	$ 4,300.00	$ 129,000
			Superstructure Cost Subtotal =	$ 5,634,295

Round up $ 5,635,000

Cost / sf $ 155

Deck Area = 803' x 45'4" = 36403 155 **$ 5,642,000**

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 1

South Abutment & Wingwalls

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete Abutment & Retaining Wall	CY	460	$ 800.00	$ 368,000
Structural Concrete Abutment & Retaining Wall (UW)	CY	135	$ 210.00	$ 28,350
Structural Concrete Approach Slab	CY	16	$ 490.00	$ 7,840
Reinforcing Steel, Fab & Del	LBS	72000	$ 0.88	$ 63,360
Reinforcing Steel, Placing	LBS	72000	$ 0.78	$ 56,160
			South Abutment Cost Total =	$ 523,710

SAY **$ 525,000**

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 1

North Abutment & Wingwalls

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete Abutment & Retaining Wall	CY	88	$ 800.00	$ 70,400
Structural Concrete Abutment & Retaining Wall (UW)	CY	28	$ 210.00	$ 5,880
Structural Concrete Approach Slab	CY	16	$ 490.00	$ 7,840
Reinforcing Steel, Fab & Del	LBS	16000	$ 0.88	$ 14,080
Reinforcing Steel, Placing	LBS	16000	$ 0.78	$ 12,480
			North Abutment Cost Total =	$ 110,680

SAY **$ 112,000**

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 1
800 ft 5 Steel Girder 5 Span 137.5'-175'-175'-175'-137.5'
Solid Shaft Piers (4 Piers)

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete, Piers (Shaft/Wall)	CY	730	$ 940.00	$ 686,200.00
Structural Concrete, Piers (placed under water)	CY	1020	$ 220.00	$ 224,400.00
Reinforcing Steel, Fab & Del	LBS	127500	$ 0.88	$ 112,200.00
Reinforcing Steel, Placing	LBS	127500	$ 0.78	$ 99,450.00
			Pier Cost Total =	$ 1,122,250
			SAY	**$ 1,124,000**
			Average Cost / Pier =	$ 281,000

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 2
835 ft 5 Steel Girder 5 Span 80', 200'-205'-205'-145'

Element	Cost
Steel Girder with CIP Deck	$ 6,459,000
South Abutment & Wingwalls	$ 555,000
North Abutment & Wingwalls	$ 129,000
Solid Shaft Piers	$ 1,056,000
Cofferdam - Piers	$ 600,000
Structural Excavation & Borrow	$ 60,000
Riprap	$ 24,000
Miscellaneous Item Contingency @ 7% + $1M work trestle premium	$ 1,622,000
STRUCTURE SUBTOTAL =	$ 10,505,000
Approaches	$ 450,000
Miscellaneous Approach Item Contingency @ 10%	$ 45,000
APPROACH SUBTOTAL =	$ 495,000
Removal of Existing Bridge Structure	$ 1,000,000
Mobilization/Demobilization @ 7% Bridge, 10% Approaches	$ 855,000
CONSTRUCTION TOTAL =	$ 12,855,000
RIGHT OF WAY COST =	$50,000
TOTAL COST =	$12,905,000
SAY	**$13,000,000**

Prepared by: NLB 6/17
Checked by:RMH 6/17

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 2
835 ft 5 Steel Girder 5 Span 80', 200'-205'-205'-145'
Steel Girder with CIP Deck

Item	Unit	Quantity	Unit Price	Cost
Hot Mix Asphalt, 9.5mm Nominal Maximum Size	Ton	490	$ 200.00	$ 98,000
Structural Concrete Roadway Slab on Steel Bridges	CY	1510	$ 1,230.00	$ 1,857,300
Reinforcing Steel, Fab & Del	LBS	339000	$ 0.88	$ 298,320
Reinforcing Steel, Placing	LBS	339000	$ 0.78	$ 264,420
High Performance Waterproofing Membrane	SY	2980	$ 23.00	$ 68,540
Structural Steel Fab. & Del., Welded	LBS	1753000	$ 1.46	$ 2,559,380
Structural Steel Erection	LBS	1753000	$ 0.36	$ 631,080
Shear Connectors	EA	7200	$ 7.50	$ 54,000
Steel Bridge Railing, 4 Bar	LF	1680	$ 190.00	$ 319,200
Bridge Expansion Joint, Finger; including Fabric Trough	LF	91	$ 1,560.00	$ 141,960
Permanent Concrete Transition Barrier	EA	4	$ 3,400.00	$ 13,600
Pot or Disc Bearings, Expansion or Fixed (including Installation)	EA	35	$ 4,300.00	$ 150,500
			Superstructure Cost Subtotal =	$ 6,456,300

Round up $ 6,457,000

Cost / sf $ 170

Deck Area = 838' x 45'4" = 37990 170 **$ 6,459,000**

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 2

South Abutment & Wingwalls

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete Abutment & Retaining Wall	CY	460	$ 800.00	$ 368,000
Structural Concrete Abutment & Retaining Wall (UW)	CY	280	$ 210.00	$ 58,800
Structural Concrete Approach Slab	CY	16	$ 490.00	$ 7,840
Reinforcing Steel, Fab & Del	LBS	72000	$ 0.88	$ 63,360
Reinforcing Steel, Placing	LBS	72000	$ 0.78	$ 56,160
			South Abutment Cost Total =	$ 554,160
			SAY	**$ 555,000**

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 2

North Abutment & Wingwalls

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete Abutment & Retaining Wall	CY	88	$ 800.00	$ 70,400
Structural Concrete Abutment & Retaining Wall (UW)	CY	110	$ 210.00	$ 23,100
Structural Concrete Approach Slab	CY	16	$ 490.00	$ 7,840
Reinforcing Steel, Fab & Del	LBS	16000	$ 0.88	$ 14,080
Reinforcing Steel, Placing	LBS	16000	$ 0.78	$ 12,480
			North Abutment Cost Total =	$ 127,900

SAY **$ 129,000**

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 2
835 ft 5 Steel Girder 5 Span 80', 200'-205'-205'-145'
Solid Shaft Piers (4 Piers)

Description	Unit	Quantity	Unit Price	Cost
Structural Concrete, Piers (Shaft/Wall)	CY	720	$ 940.00	$ 676,800.00
Structural Concrete, Piers (placed under water)	CY	760	$ 220.00	$ 167,200.00
Reinforcing Steel, Fab & Del	LBS	126000	$ 0.88	$ 110,880.00
Reinforcing Steel, Placing	LBS	126000	$ 0.78	$ 98,280.00
			Pier Cost Total =	$ 1,053,160

SAY **$ 1,056,000**

Average Cost / Pier = $ 264,000

Prepared by: RMH 10/15
Checked by: DSM 10/15

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 3
Rehabilitate Existing 800 ft 3 Span Steel Truss

Element	Cost
Rehabilitate Existing 800 ft Steel Truss	$ 7,287,000
Abutment Backwall inc. Removal	$ 120,000
Pier 2 Span 3 Bearing Pedestal Replacement & Bearing Rehab	$ 100,000
Removal of Existing Slab Full Depth, Inc. Steel Floor Framing	$ 180,000
Rehabilitation Contingencies @ 15%	$ 1,154,000
Miscellaneous @ 7%	$ 619,000
STRUCTURE SUBTOTAL =	$ 9,460,000
Approaches	$ 50,000
Miscellaneous Approach Item Contingency @ 10%	$ 5,000
APPROACH SUBTOTAL =	$ 55,000
Temporary Bridge	$ 4,000,000
Mobilization/Demobilization @ 10%	$ 1,352,000
CONSTRUCTION TOTAL =	$ 14,867,000
RIGHT OF WAY COST =	$10,000
TOTAL COST =	$14,877,000
SAY	**$15,000,000**

Prepared by: NLB 6/17
Checked by: RMH 6/17

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 3
Rehabilitate Existing 800 ft 3 Span Steel Truss
Rehabilitate Existing 800 ft Steel Truss

Item	Unit	Quantity	Unit Price	Cost
Structural Concrete Roadway Slab on Steel Bridges	CY	810	$ 1,230.00	$ 996,300
Reinforcing Steel, Fab & Del	LBS	160000	$ 0.88	$ 140,800
Reinforcing Steel, Placing	LBS	160000	$ 0.78	$ 124,800
Strutural Steel Repairs - Sidewalk Brackets	LS	1	$50,000.00	$ 50,000
Structural Steel, Floor System Fab & Delivered	LBS	444000	$ 2.20	$ 976,800
Structural Steel, Floor System, Erection	LBS	444000	$ 1.80	$ 799,200
Structural Steel, Truss Bottom Chord Flange, Installed	LBS	58000	$ 7.80	$ 452,400
Field Painting	LBS	2700000	$ 0.27	$ 729,000
Surface Prep Existing Str Steel	LBS	2700000	$ 0.46	$ 1,242,000
Containmant and Pollution Control	LBS	2700000	$ 0.46	$ 1,242,000
Disposal of Special Waste Material	LBS	2700000	$ 0.02	$ 54,000
Shear Connectors	EA	27000	$ 7.50	$ 202,500
Steel Bridge Railing, Pedestrian Remove and Reset	LF	810	$ 94.00	$ 76,140
Steel Bridge Railing, 2 Bar Remove and Reset	LF	810	$ 62.00	$ 50,220
Bridge Expansion Joint, Gland Seal	LF	156	$ 840.00	$ 131,040
		Superstructure Cost Subtotal =		$ 7,267,200

Round up $ 7,268,000

Cost / sf $ 226

Deck Area = 808' x 39.9' = 32240 226 **$ 7,287,000**

Prepared by: RMH 8/16
Checked by: DSM 8/16

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 4
Rehabilitate Existing 800 ft 3 Span Steel Truss with New Sidewalk

Element	Cost
Rehabilitate Existing 800 ft Steel Truss with Exodermic Deck and SW	$ 8,622,000
Abutment Backwall inc. Removal	$ 120,000
Pier 2 Span 3 Bearing Pedestal Replacement & Bearing Rehab	$ 100,000
Removal of Existing Slab Full Depth, Inc. Steel Floor Framing	$ 180,000
Rehabilitation Contingencies @ 15%	$ 1,354,000
Miscellaneous @ 7%	$ 727,000
STRUCTURE SUBTOTAL =	$ 11,103,000
Approaches	$ 100,000
Miscellaneous Approach Item Contingency @ 10%	$ 10,000
APPROACH SUBTOTAL =	$ 110,000
Temporary Bridge	$ 4,000,000
Mobilization/Demobilization @ 10%	$ 1,522,000
CONSTRUCTION TOTAL =	$ 16,735,000
RIGHT OF WAY COST =	$10,000
TOTAL COST =	$16,745,000
SAY	**$17,000,000**

Prepared by: NLB 6/17
Checked by: RMH 6/17

Brunswick-Topsham - F.J. Wood Bridge

PDR Structural Cost Estimate
Alternate 4
Rehabilitate Existing 800 ft 3 Span Steel Truss with New Sidewalk
Rehabilitate Existing 800 ft Steel Truss with Exodermic Deck and New Sidewalk

Item	Unit	Quantity	Unit Price	Cost
Structural Concrete Exodermic Deck Infill	CY	655	$ 880.00	$ 576,400
Exodermic Deck, Installed w/o Concrete	SF	25050	$ 55.00	$ 1,377,750
Reinforcing Steel, Fab & Del	LBS	12800	$ 0.88	$ 11,264
Reinforcing Steel, Placing	LBS	12800	$ 0.78	$ 9,984
Strutural Steel Repairs - Sidewalk Brackets	LS	1	$50,000.00	$ 50,000
Structural Steel, Floor System Fab & Delivered	LBS	444000	$ 2.20	$ 976,800
Structural Steel, Floor System, Erection	LBS	444000	$ 1.80	$ 799,200
Structural Steel, Truss Bottom Chord Flange, Installed	LBS	58000	$ 7.80	$ 452,400
Structural Steel, Sidewalk, Installed	LBS	73000	$ 4.30	$ 313,900
Field Painting	LBS	2700000	$ 0.27	$ 729,000
Surface Prep Existing Str Steel	LBS	2700000	$ 0.46	$ 1,242,000
Containmant and Pollution Control	LBS	2700000	$ 0.46	$ 1,242,000
Disposal of Special Waste Material	LBS	2700000	$ 0.02	$ 54,000
Shear Connectors	EA	28600	$ 7.50	$ 214,500
Steel Bridge Railing, Pedestrian Remove and Reset	LF	810	$ 94.00	$ 76,140
Steel Bridge Railing, 2 Bar Remove and Reset	LF	810	$ 62.00	$ 50,220
Steel Bridge Railing, 4 Bar	LF	810	$ 190.00	$ 153,900
Steel Bridge Railing, 2 Bar	LF	810	$ 130.00	$ 105,300
Bridge Expansion Joint, Sidewalk	LF	34	$ 1,040.00	$ 35,360
Bridge Expansion Joint, Gland Seal	LF	156	$ 840.00	$ 131,040
		Superstructure Cost Subtotal =		$ 8,601,158

Round up $ 8,602,000

Cost / sf $ 227

Deck Area = 808' x 47' = 37980 227 **$ 8,622,000**

Prepared by: RMH 8/16
Checked by: DSM 8/16

Brunswick-Topsham, F.J. Wood Bridge
JN: 411813.00

By: RMH
Date: 9/14/2016
Check: NLB
Date: 11/9/2016

Life Cycle Cost Analysis (one or two sidewalk rehab vs. two sidewalk replacement)

Assumptions:

Replacement Bridge Design Life = L = 100 years
Discount Rate = D = 4 %
Year of Future Replacement for Rehabilitated Project = RL = **75 years**

Annual Existing Truss Bridge Expenditures: (+)

Inspection = TI =	$ 30,000
Maintenance (Fatigue & Paint Repairs) = TM =	$ 40,000
Year Specific Existing Truss Bridge Expenditures: (+)	
Deck Replacement (Alt 3 One Sidewalk) = TD1 =	$ 1,000,000
Deck Replacement (Alt 4 Two Sidewalks) = TD2 =	$ 2,000,000
Painting = TP =	$ 4,000,000
Substructure Rehabilitation = TS =	$ 1,000,000

Annual Replacement Bridge Expenditures: (-)

Inspection = RI =	$ 600
Maintenance = RM =	$ 1,000
Year Specific Replacement Bridge Expenditures: (-)	
Wearing Surface Replacement = RW =	$ 100,000
Painting = RP =	$ 1,750,000

Estimated Construction Cost of Alternates:

Bridge Rehabilitation Construction Cost (Alt 3 - One Sidewalk) = R3 =	**$ 15,000,000**
Bridge Rehabilitation Construction Cost w/ added sidewalk (Alt 4 - Two Sidewalks) = R4 =	**$ 17,000,000**
Low Cost Bridge Replacement Construction Cost (Alt 2) = R2 =	**$ 13,000,000**

Present Value of Existing Truss Bridge Expenditures: (+)	@ Year = Y	
Total Inspection = TTI = $(TI)*((1-(1/(1+D/100)^{RL}))/D)$ =	Annual	$ 710,000
Total Maintenance = TTM = $TM*((1-(1/(1+D/100)^{RL}))/D)$ =	Annual	$ 947,000
1st Paint @ Year = TP1 = $TP/(1+D/100)^{Y}$	20	$ 1,826,000
2nd Paint @ Year = TP2 = $TP/(1+D/100)^{Y}$	40	$ 833,000
3rd Paint @ Year = TP3 = $TP/(1+D/100)^{Y}$	60	$ 380,000
Deck Replacement (Alt 3 - One Sidewalk) @ Year = TD = $TD1/(1+D/100)^{Y}$	40	$ 208,000
Deck Replacement (Alt 4 - Two Sidewalks) @ Year = TD = $TD2/(1+D/100)^{Y}$	40	$ 417,000
1st Substructure Rehabilitation @ Year = TS1 = $TS/(1+D/100)^{Y}$	20	$ 456,000
2nd Substructure Rehabilitation @ Year = TS2 = $TS/(1+D/100)^{Y}$	50	$ 141,000
Sum Present Value of Existing Truss Bridge Expenditures (Alt 3 - One Sidewalk)= TBE1 =		**$ 5,501,000**
Sum Present Value of Existing Truss Bridge Expenditures (Alt 4 - Two Sidewalks) = TBE2 =		**$ 5,710,000**

Present Value of Replacement Bridge Expenditures: (-)	@ Year = Y	
Total Inspection = TRI = $(RI)*((1-(1/(1+D/100))^{RL})/D)$ =	Annual	$ 14,000
Total Maintenance = TRM = $(RM)*((1-(1/(1+D/100))^{RL})/D)$ =	Annual	$ 24,000
1st Wearing Surface @ Year = RW1 = $RW/(1+D/100)^{Y}$	15	$ 56,000
2nd Wearing Surface @ Year = RW2 = $RW/(1+D/100)^{Y}$	30	$ 31,000
3rd Wearing Surface @ Year = RW3 = $RW/(1+D/100)^{Y}$	45	$ 17,000
4th Wearing Surface @ Year = RW3 = $RW/(1+D/100)^{Y}$	60	$ 10,000
5th Wearing Surface @ Year = RW3 = $RW/(1+D/100)^{Y}$	75	$ 5,000
6th Wearing Surface @ Year = RW3 = $RW/(1+D/100)^{Y}$	90	$ 3,000
1st Paint @ Year = RP2 = $RP/(1+D/100)^{Y}$	35	$ 443,000
2nd Paint @ Year = RP2 = $RP/(1+D/100)^{Y}$	70	$ 112,000
Sum Present Value of Replacement Bridge Expenditures = RBE =		**$ 715,000**

Present Value of Deferred Replacement Bridge Cost & Residual Value:

Deferred Bridge Replacement Cost (+) DBC = $R2/(1+D/100)^{RL}$	**$ 686,000**
Residual Value of Replacement Bridge at Year 75 (-) = RVR = $R2*(RL/L)/(1+D/100)^{L}$	**$ 193,000**

75 years

Bridge Rehabilitation (Alt 3 - Single Sidewalk) Net Present Value in Comparison to Low Cost Alternate 2 (Replacement):

Net Present Value of Bridge Rehabilitation = R3+TBE1+DBC-RVR = **$ 20,990,000**
% Bridge Replacement Cost = **161%**

Bridge Rehabilitation (Alt 4 - Two Sidewalk) Net Present Value in Comparison to Low Cost Alternate 2 (Replacement):

Net Present Value of Bridge Rehabilitation = R4+TBE2+DBC-RVR = **$ 23,200,000**
% Bridge Replacement Cost = **178%**

Bridge Replacement (Alt 2 - On Parallel Alighment) Net Present Value in Comparison to Low Cost Alternate 2 (Replacement):

Net Present Value of Bridge Rehabilitation = R4+RBE = **$ 13,720,000**
% Bridge Replacement Cost = **106%**

Brunswick-Topsham, Frank J. Wood Bridge: Service Life Costs

	Alternate 2:		**Alternate 3:**		**Alternate 4:**	
	Replacement on Upstream Curved Alignment		Rehabilitation		Rehabilitation with Added Sidewalk	
Cost Items	**Year**	**Cost**	**Year**	**Cost**	**Year**	**Cost**
This Current Project	2019	$ 13,000,000	2019	$ 15,000,000	2019	$ 17,000,000
Inspections	Annual	$ 600	Annual	$ 30,000	Annual	$ 30,000
Routine Maintenance	Annual	$ 1,000	Annual	$ 40,000	Annual	$ 40,000
Paint	35	$ 1,750,000	20	$ 4,000,000	20	$ 4,000,000
	70	$ 1,750,000	40	$ 4,000,000	40	$ 4,000,000
			60	$ 4,000,000	60	$ 4,000,000
Deck Replacement	None		40	$ 1,000,000	40	$ 2,000,000
Substructure Rehab	None		20	$ 1,000,000	20	$ 1,000,000
			50	$ 1,000,000	50	$ 1,000,000
Wearing Surface	15	$ 100,000	None		None	
	30	$ 100,000				
	45	$ 100,000				
	60	$ 100,000				
	75	$ 100,000				
	90	$ 100,000				
Service Life	100 years		75 years		75 years	
Total Cost over Life	$17,300,000		$35,200,000		$38,200,000	