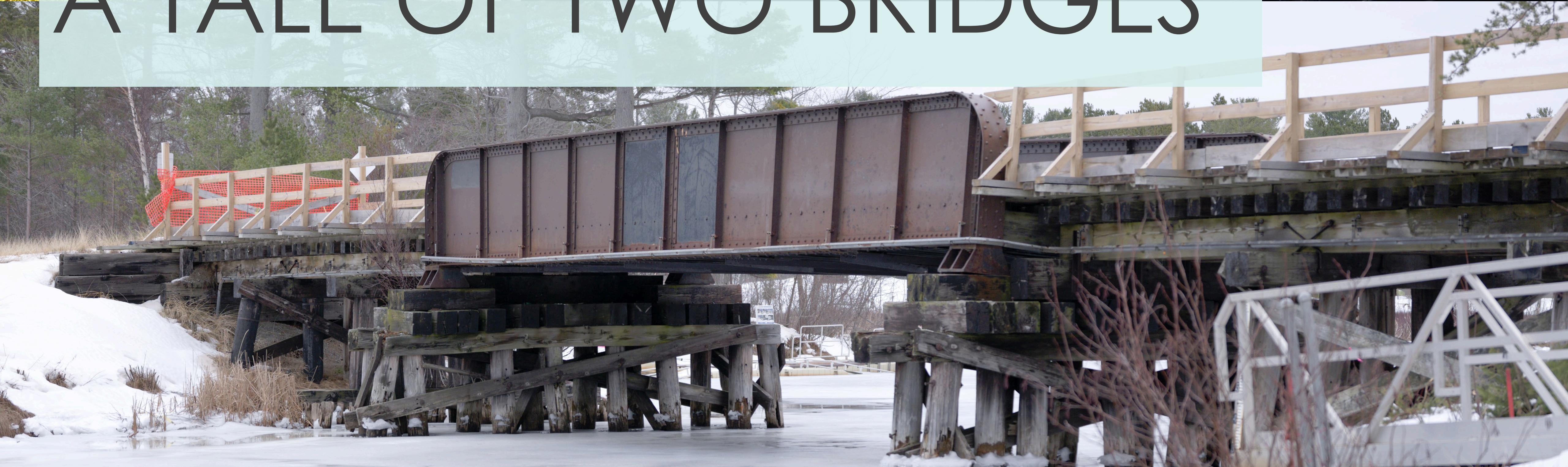
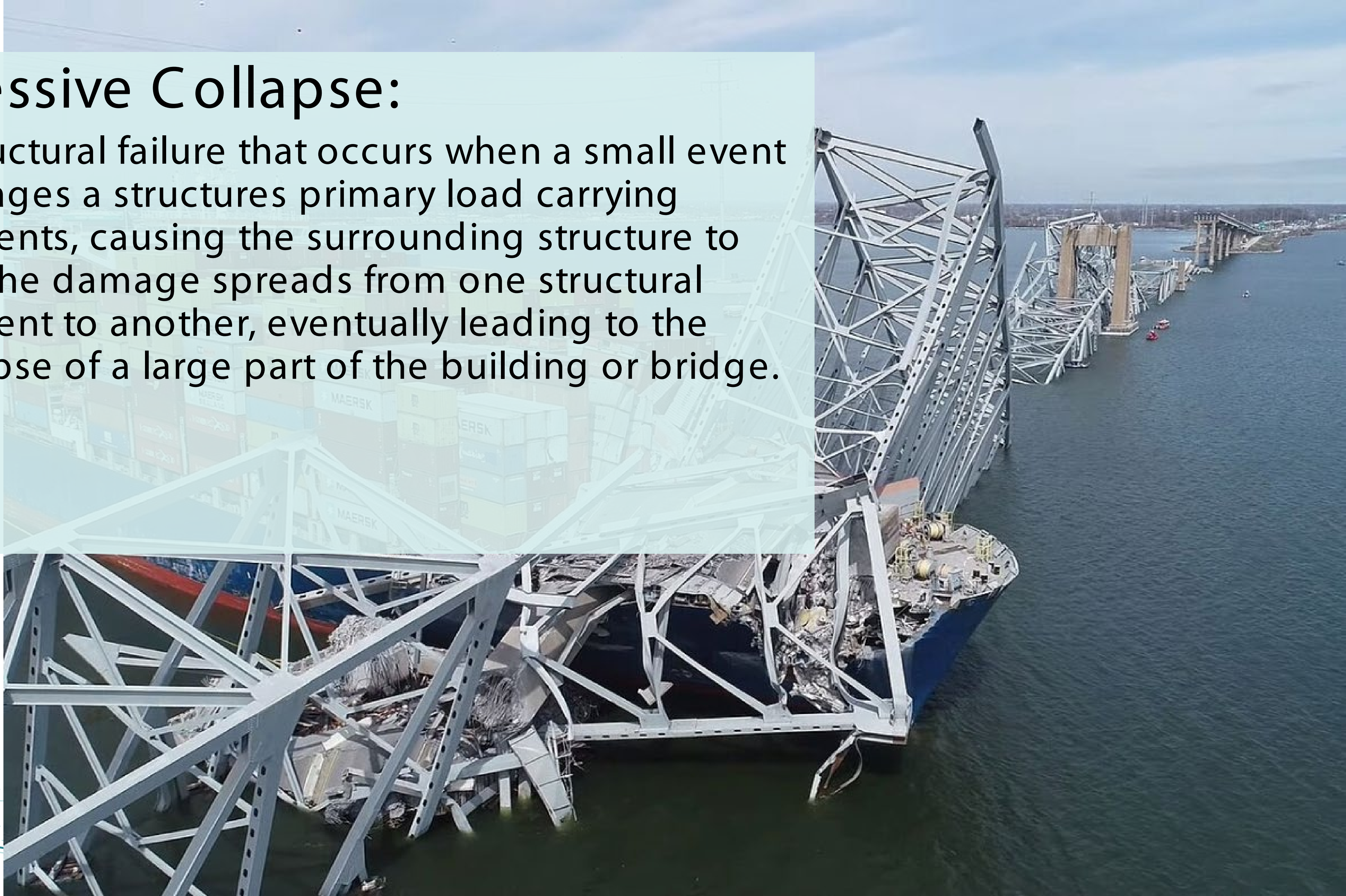
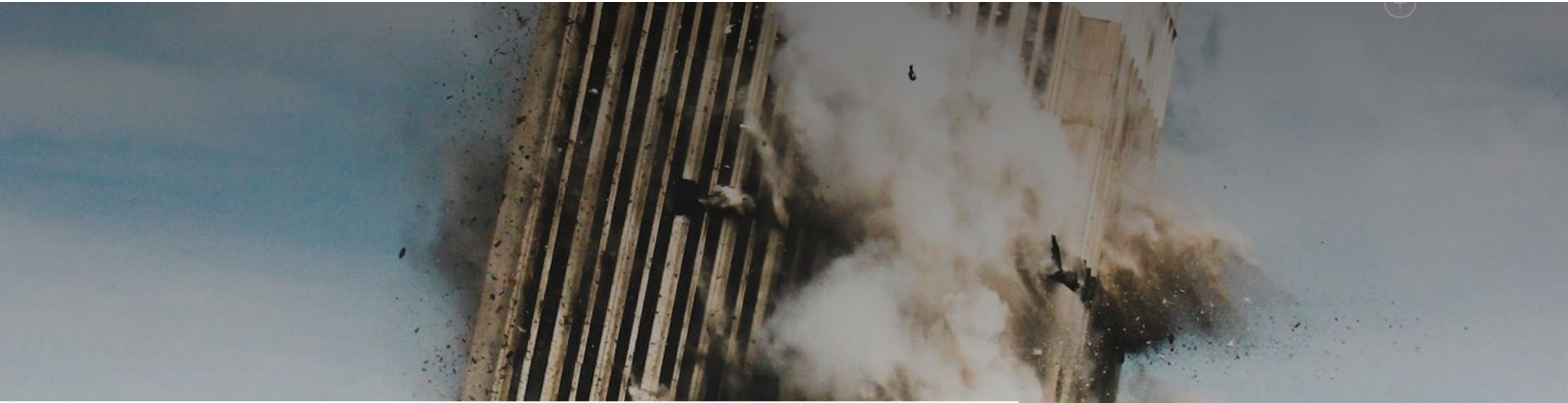
A TALE OF TWO BRIDGES



Progressive Collapse:

A structural failure that occurs when a small event damages a structures primary load carrying elements, causing the surrounding structure to fail. The damage spreads from one structural element to another, eventually leading to the collapse of a large part of the building or bridge.





Progressive Collapse of structures

Words by BSBG Media Team, Thursday 19 July 2018

Share

The latest BSBG blog is written by Structural Engineer Mark Juinio, who provides a study of progressive collapse in structures, which begins with a minor failure, and results in widespread collapse. BSBG's Engineering division has delivered some of Dubai's most prestigious developments, and is currently working on a variety of projects under construction, including Bluewaters Island Residential, Jebel Ali Park Hotel, E15, Festival Plaza, Banyan Tree Residences - Hillside Dubai and BLVD Crescent.

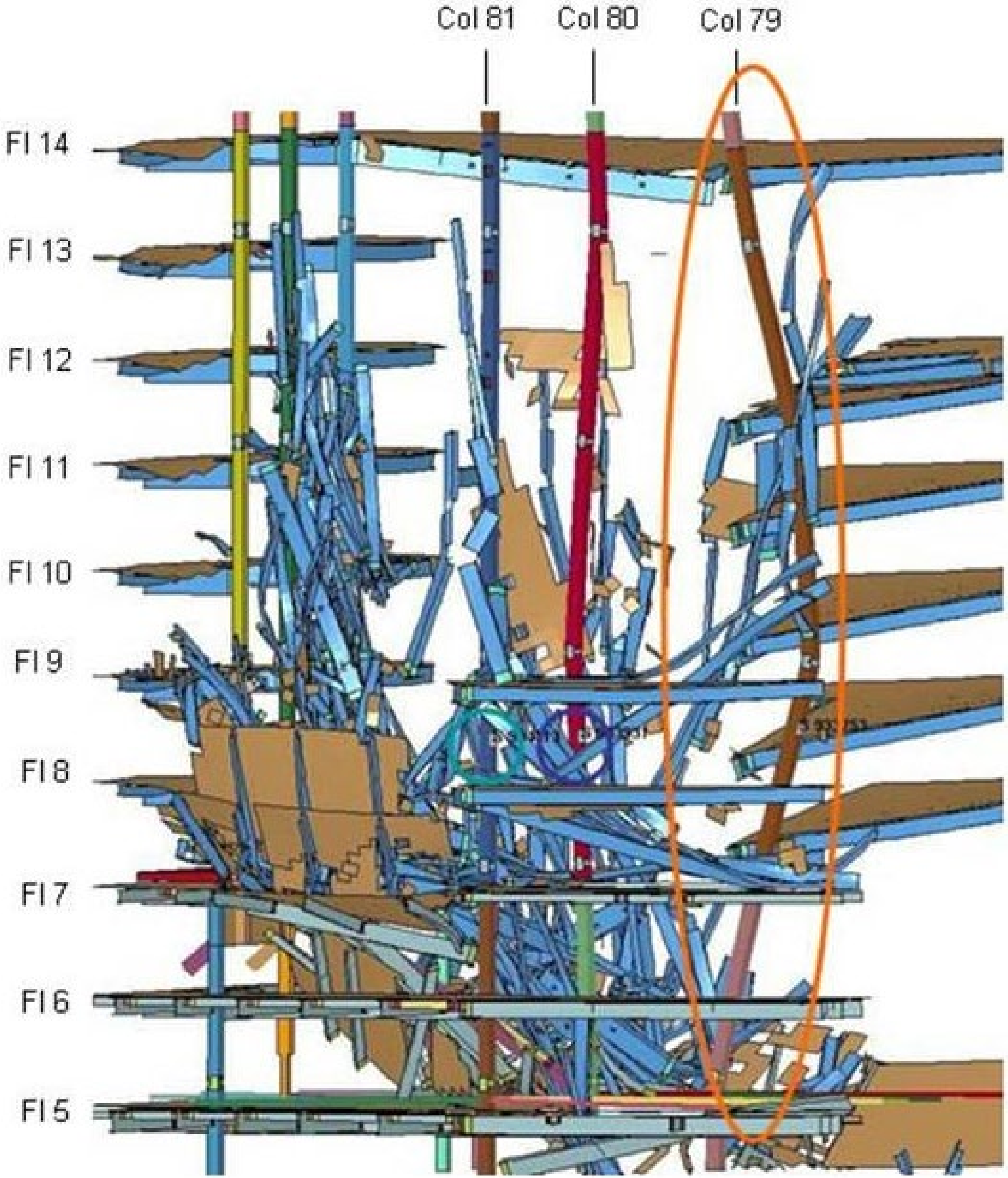
Do you find falling dominoes satisfying to watch? When you line up a series of rectangular blocks in any form you want – a simple line, a curve, a circle - once everything is arranged, it only takes one gentle push on the first block to start the thrilling chain reaction.

World Trade Center Building 7

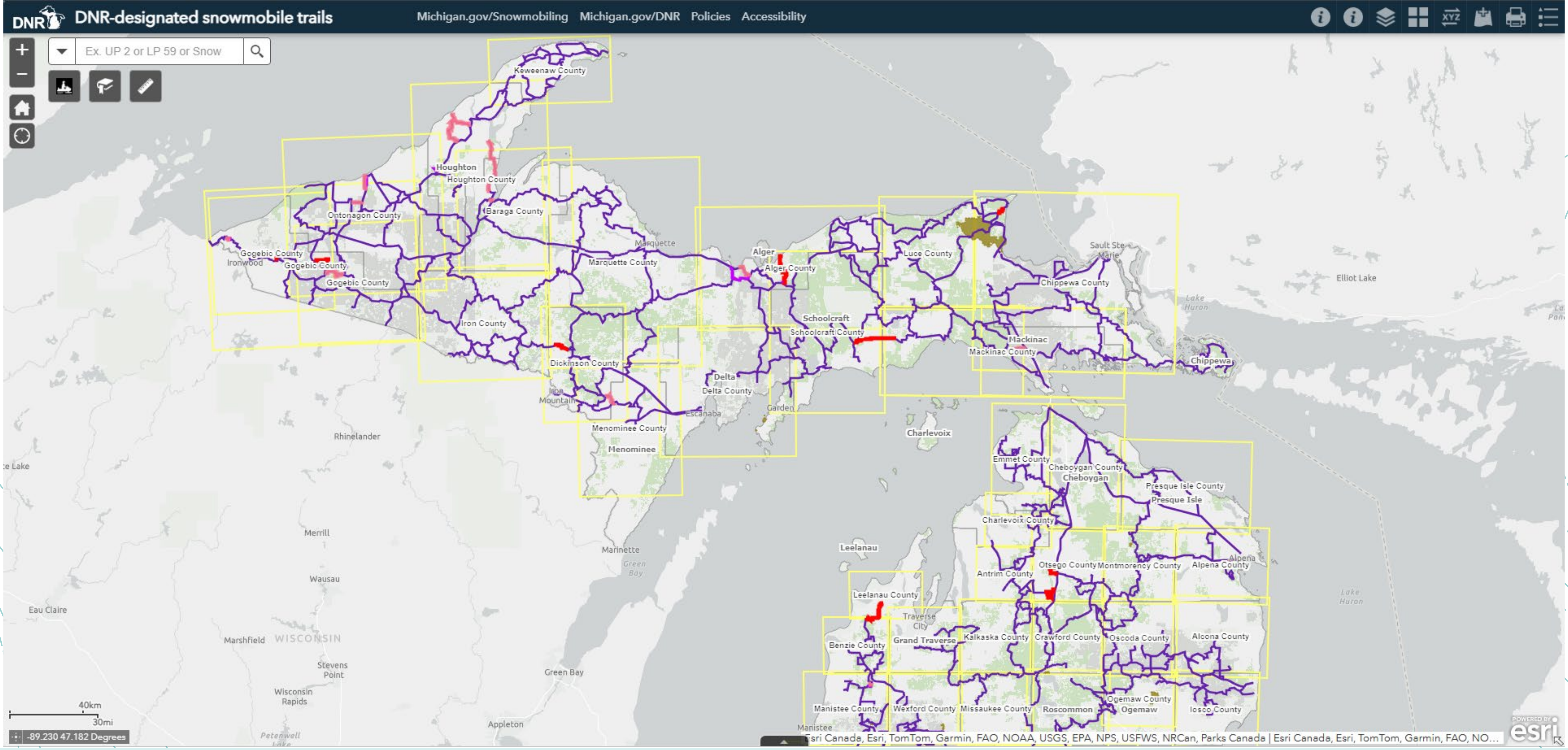
From International Fire & Safety Journal

World Trade Center Building 7

World Trade Center Building 7



Leaning Barn
M-32 Vienna Corners



NORTHERN MICHIGAN SNOWMOBILE TRAILS

NORTHERN MICHIGAN SNOWMOBILE TRAILS

Michigan Department of Natural Resources

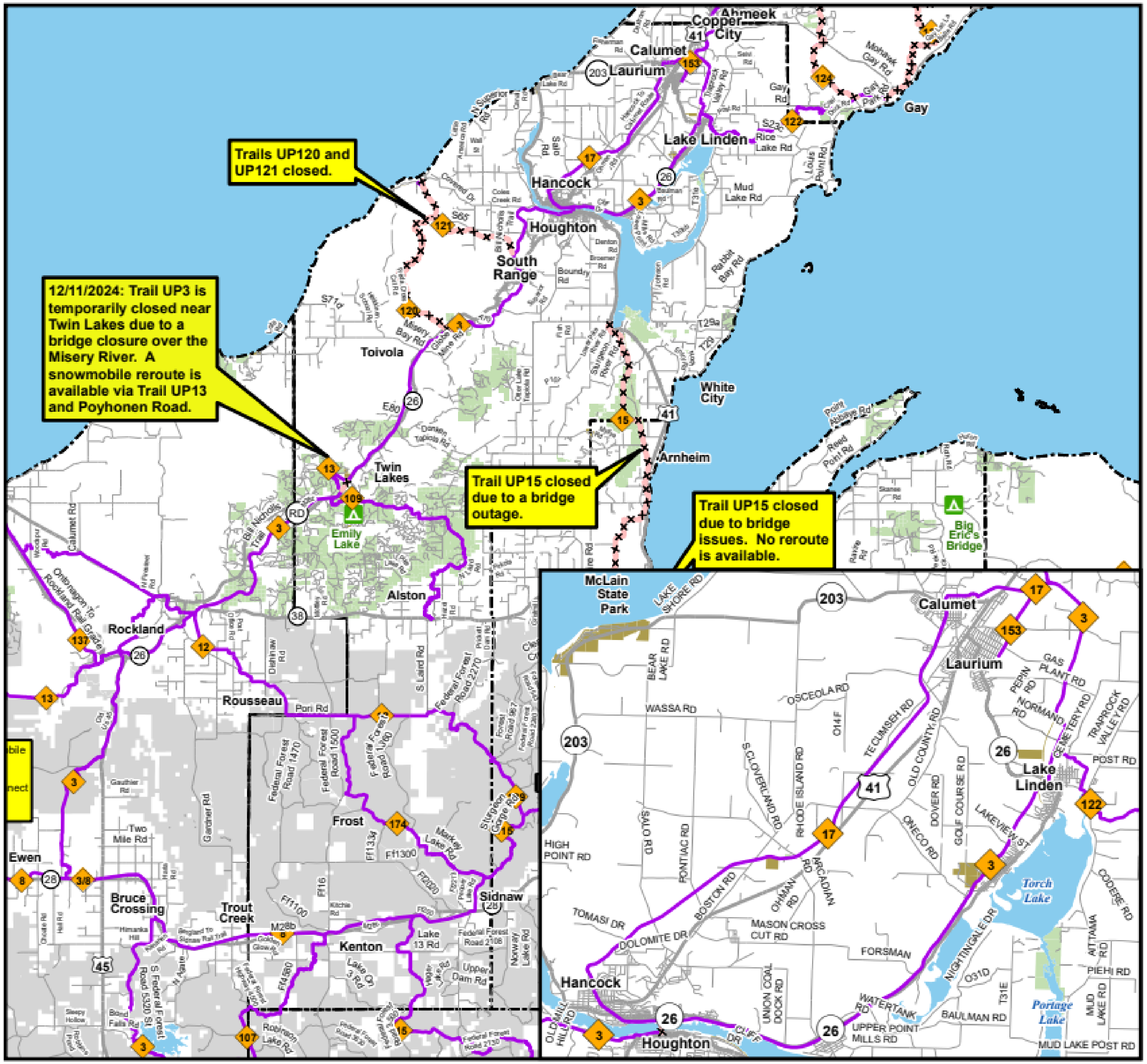
Snowmobile Trails
Houghton County

Updated: 12/11/2024

0 1 2
Miles

- Parking lot
- State forest campground
- Temporarily closed trail
- Snowmobile trail
- Highway
- Paved road
- Gravel or dirt road
- State forest land
- State park
- Federal land
- Lakes and rivers
- County boundary

Parking
Parking lot information is currently unavailable.



RIDE RIGHT

Michigan.gov/Snowmobiling

NORTHERN MICHIGAN SNOWMOBILE TRAILS

Michigan
Department of
Natural Resources

Snowmobile Trails
Dickinson County

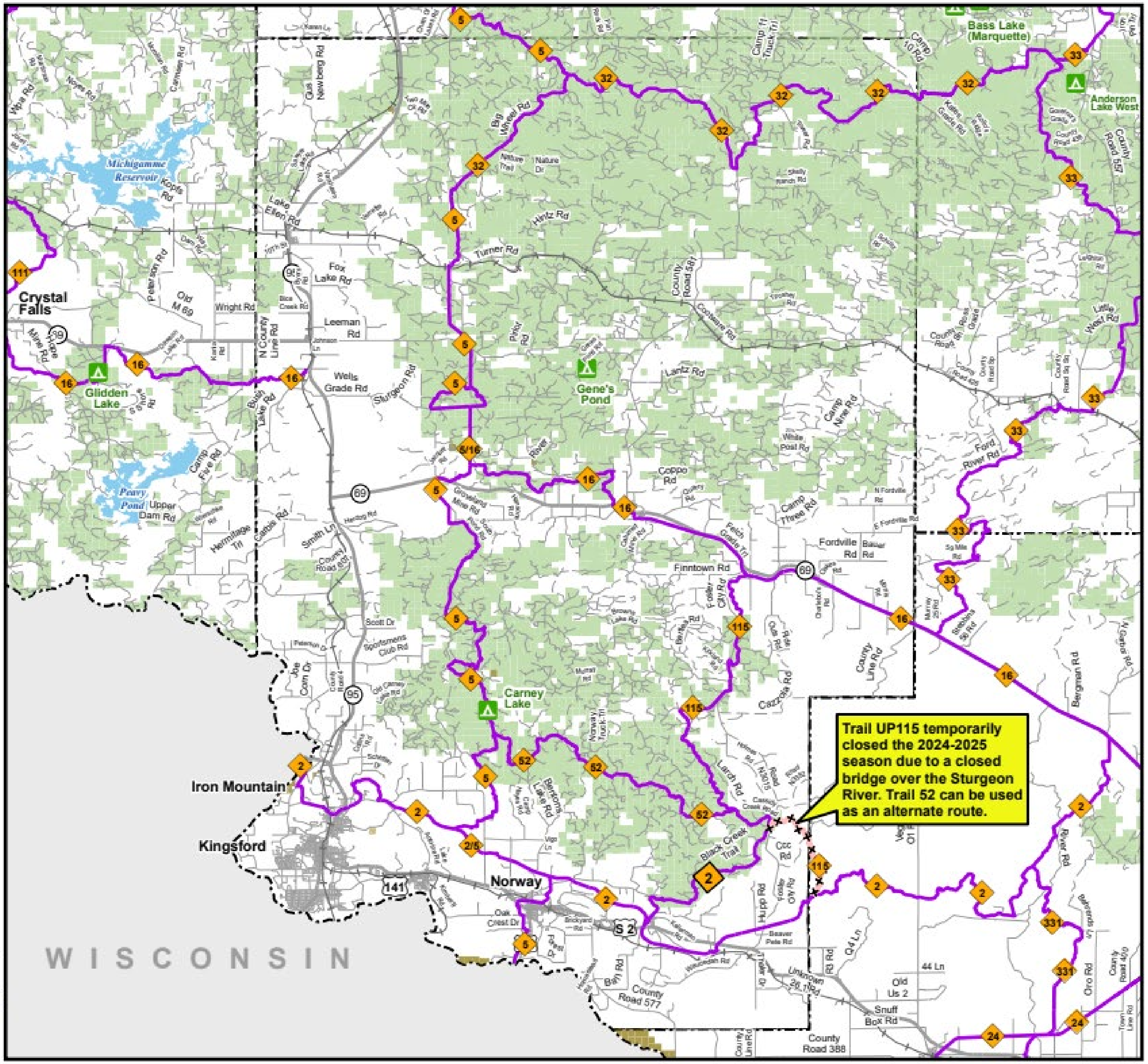
Updated: 11/27/2024

N

0 1 2
Miles

- State forest campground
- Temporarily closed trail
- Snowmobile trail
- Highway
- Paved road
- Gravel or dirt road
- Railroad
- State forest land
- State park
- Lakes and rivers

Parking
Parking lot information is currently unavailable.



RIDE RIGHT

Michigan.gov/Snowmobiling

NORTHERN MICHIGAN SNOWMOBILE TRAILS

Michigan Department of Natural Resources

Snowmobile Trails
Alger County

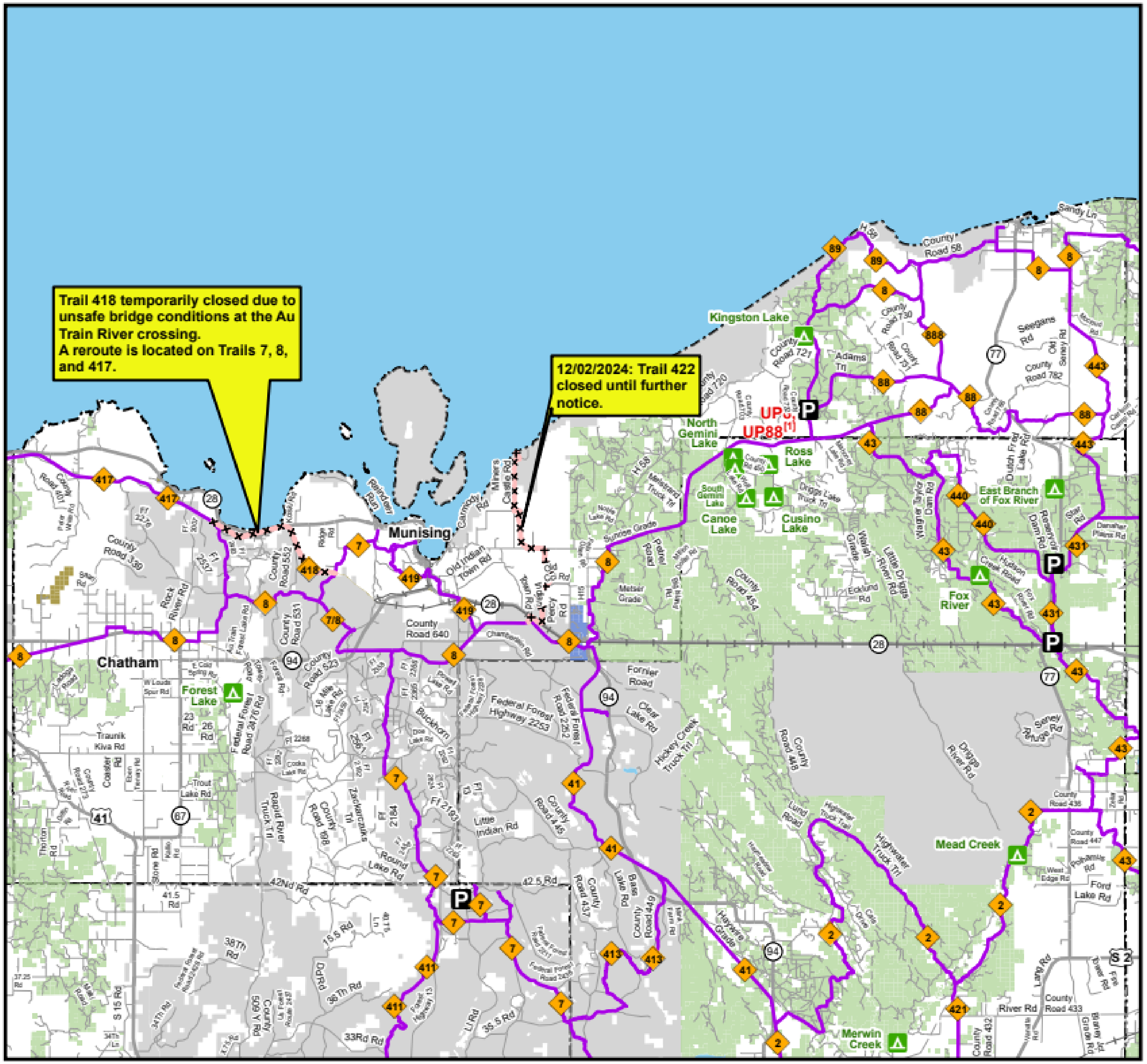
Updated: 12/03/2024

0 1 2 Miles

N

- P Parking lot
- State forest campground
- Temporarily closed trail
- Snowmobile trail
- Highway
- Paved road
- Gravel or dirt road
- Railroad
- State forest land
- State park
- State wildlife land
- Federal land
- Lakes and rivers

Parking
UP8, UP88[1] Northside of Adams Trail East of Int with H 58



RIDE RIGHT

Michigan.gov/Snowmobiling

Au Train Bridge

1. Seven Span
   a. 133′ length
   b. 50′ Main Span, Steel Plate Through Girder
   c. Timber Side Spans
2. Timber Pile Piers & Abutments
3. Age Unknown, Est. ~100 Years Old
4. Originally Designed for Rail Loads
5. Second Bridge Built at This Site

Au Train Bridge

Rail vs. Highway

1. Cooper E80 live loads are almost 16x HL-93 loads
2. Dead load weight of center span = 60 Tons

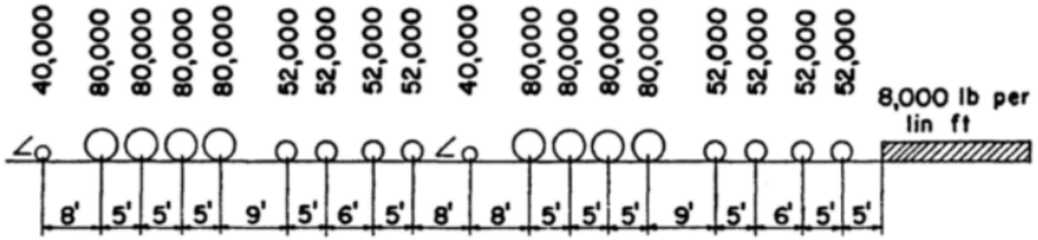


Figure 15-1-2. Cooper E 80 Load

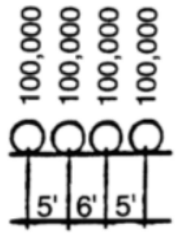
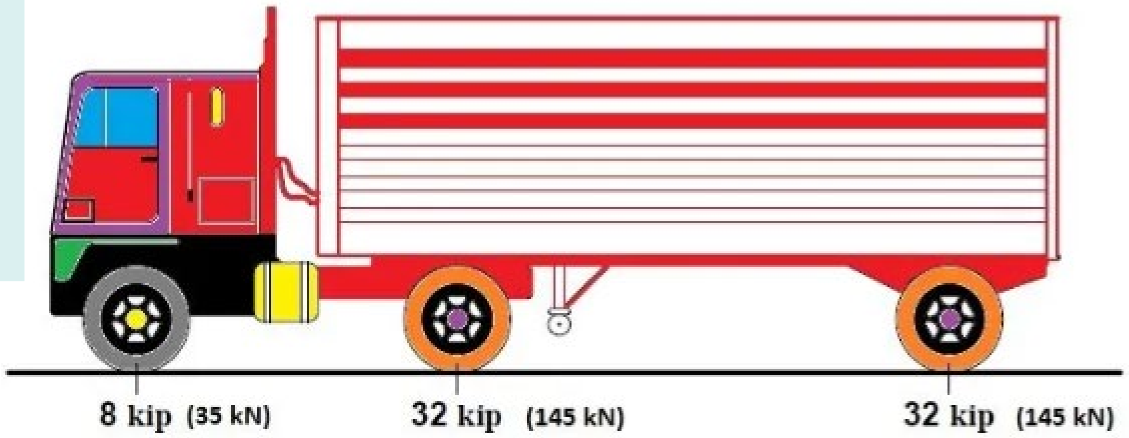


Figure 15-1-3. Alternate Live Load on 4 Axles



HL-93 Design Truck AASHTO

1. August 2024: 8″ of settlement
2. December 2024: 12″ of settlement

Au Train Bridge

Au Train Bridge

Au Train Bridge

Au Train Bridge

Au Train Bridge

Au Train Bridge

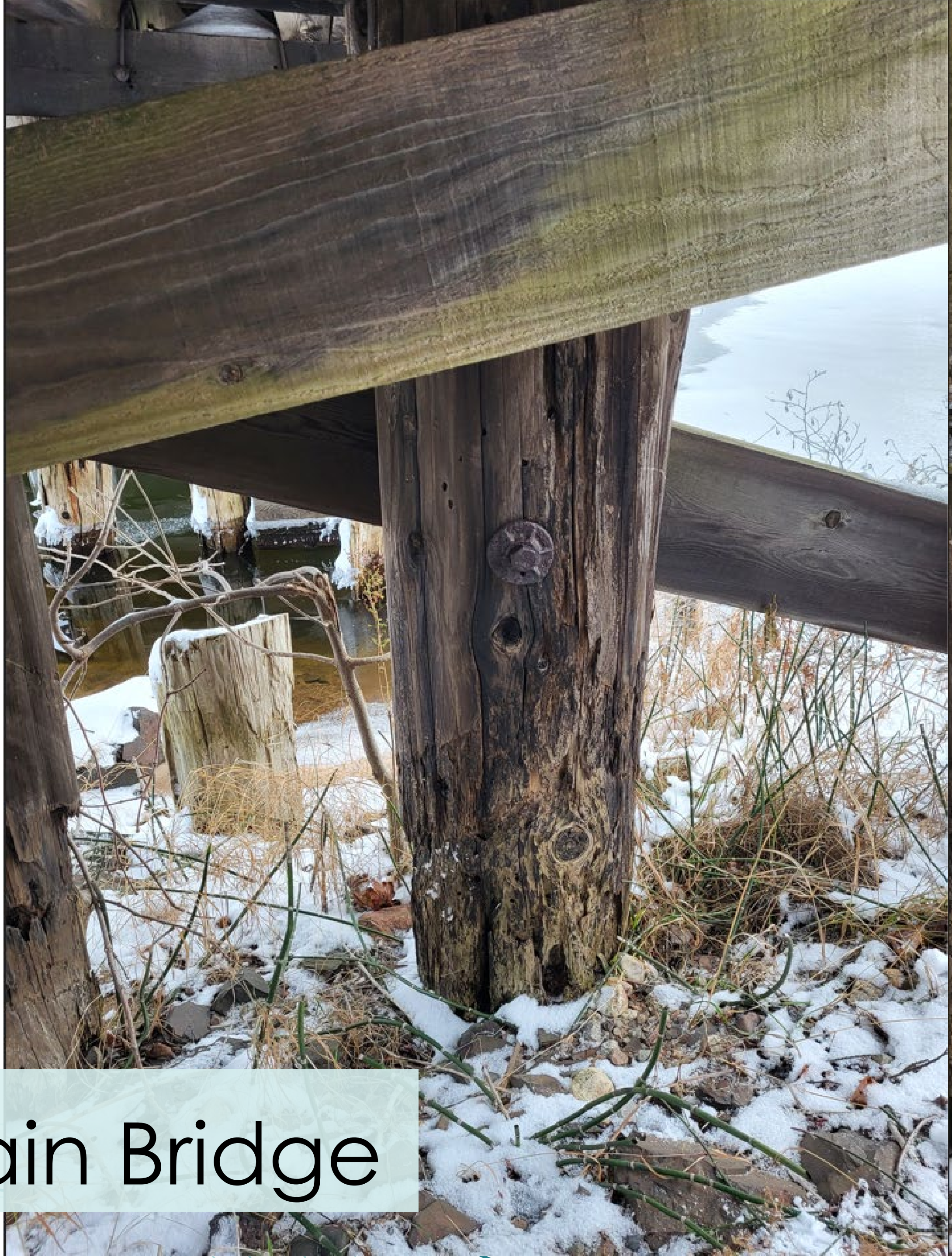
Au Train Bridge

Au Train Bridge

Au Train Bridge

Au Train Bridge

Au Train Bridge



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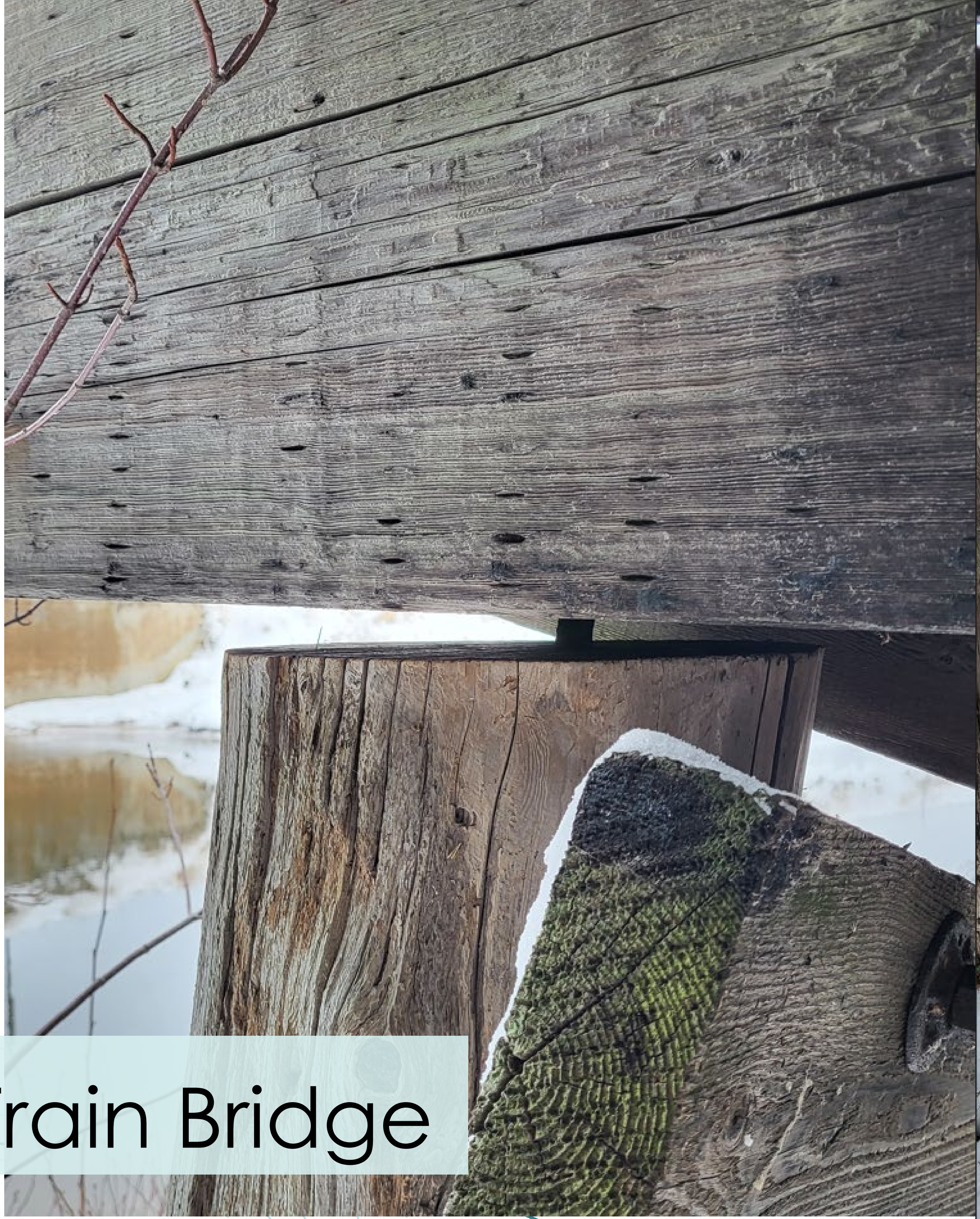

Au Train Bridge

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Au Train Bridge

Au Train Bridge - Perspective is Everything

Au Train Bridge

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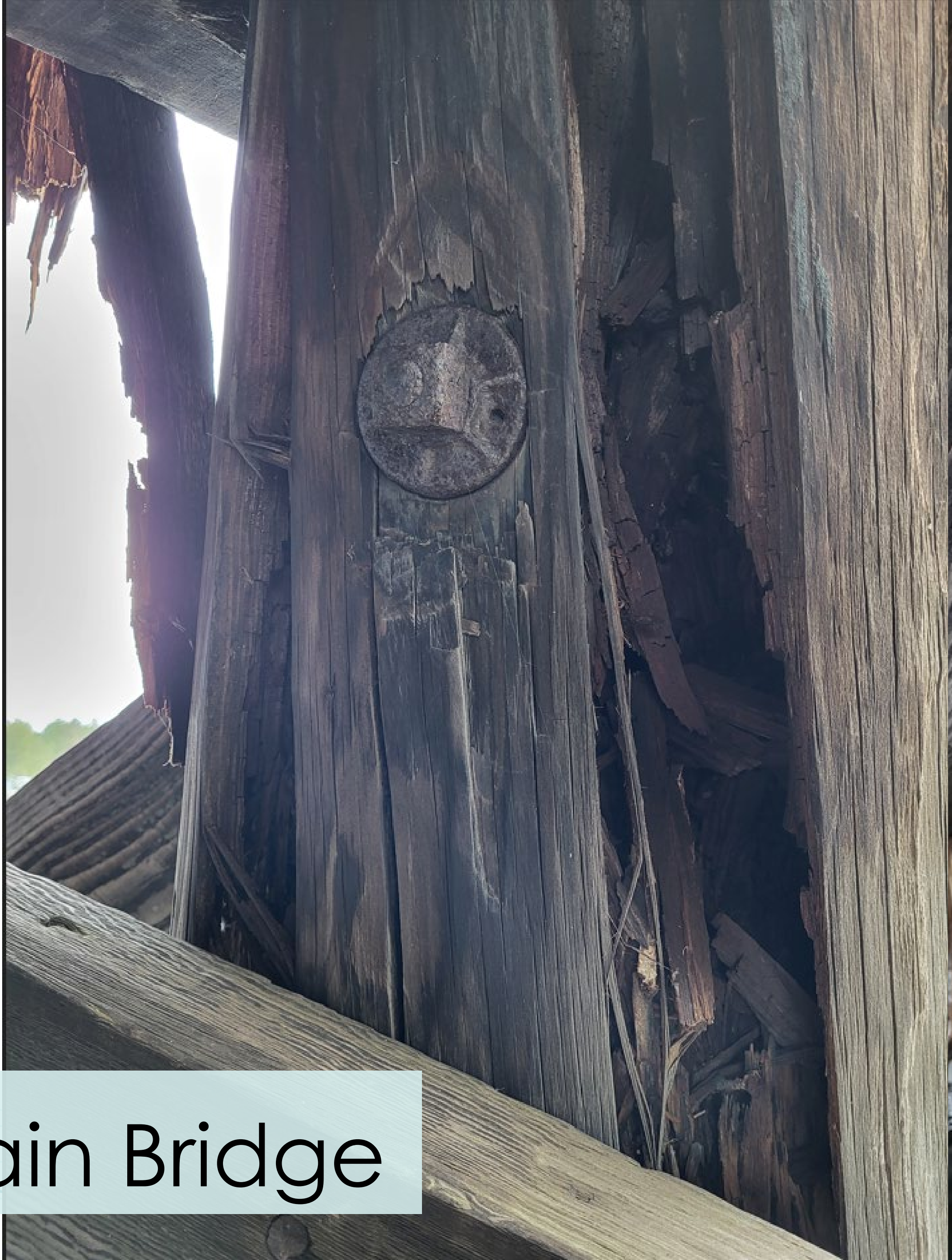
No noticeable distress at the track level up to this point.

Au Train Bridge

Au Train Bridge



Au Train Bridge

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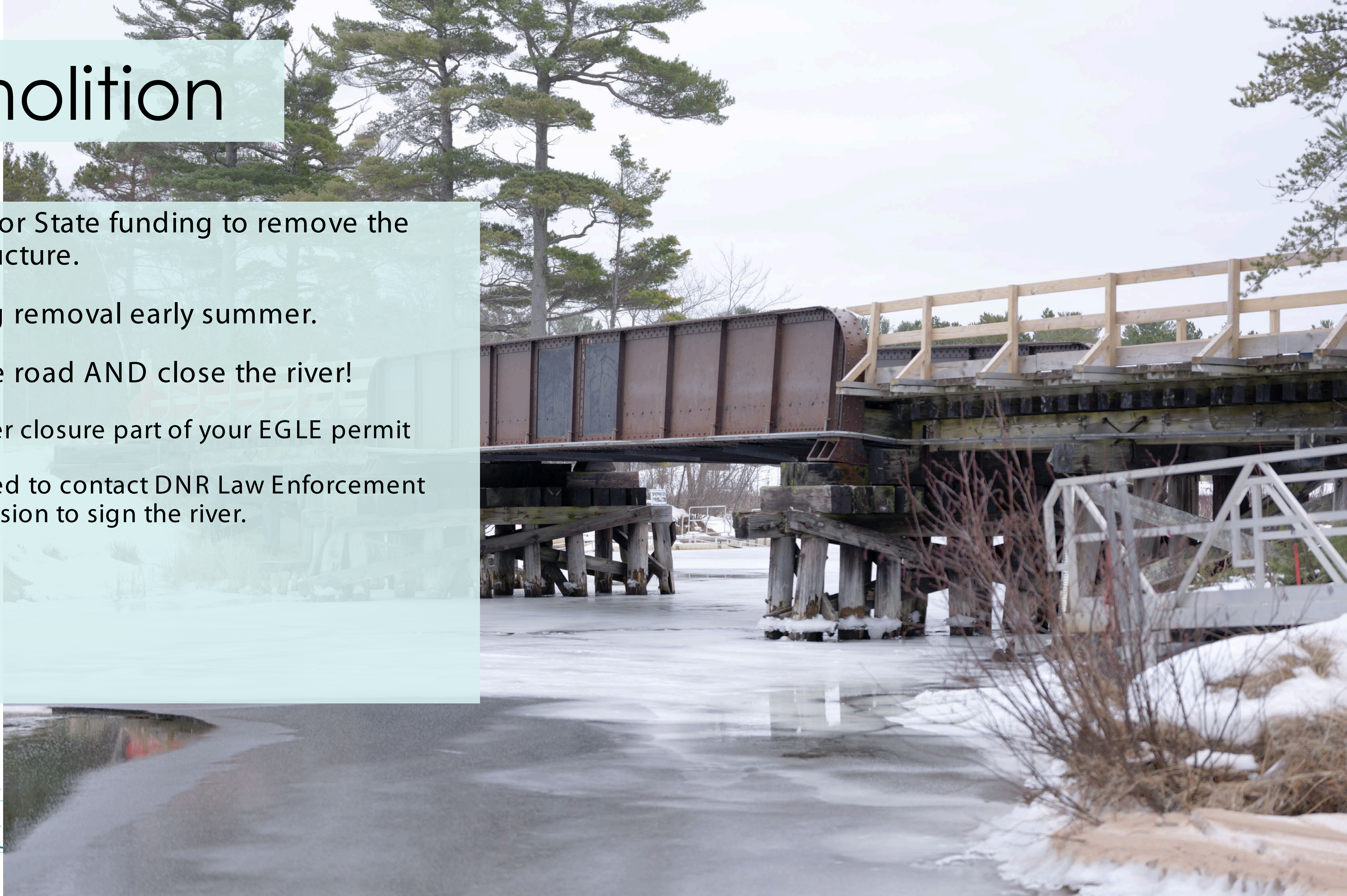
Au Tro

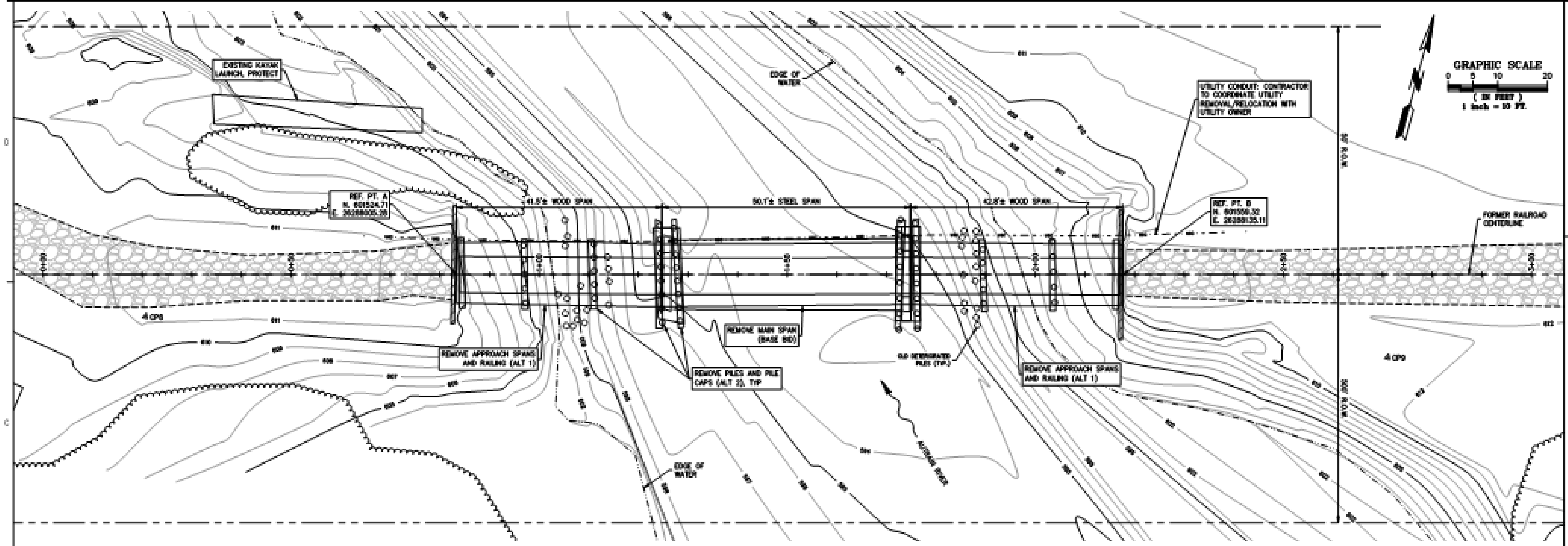
Au Train Bridge

Demolition

1. Waiting for State funding to remove the superstructure.
2. Targeting removal early summer.
3. Close the road AND close the river!
    1. River closure part of your EGLE permit
    2. Need to contact DNR Law Enforcement Division to sign the river.



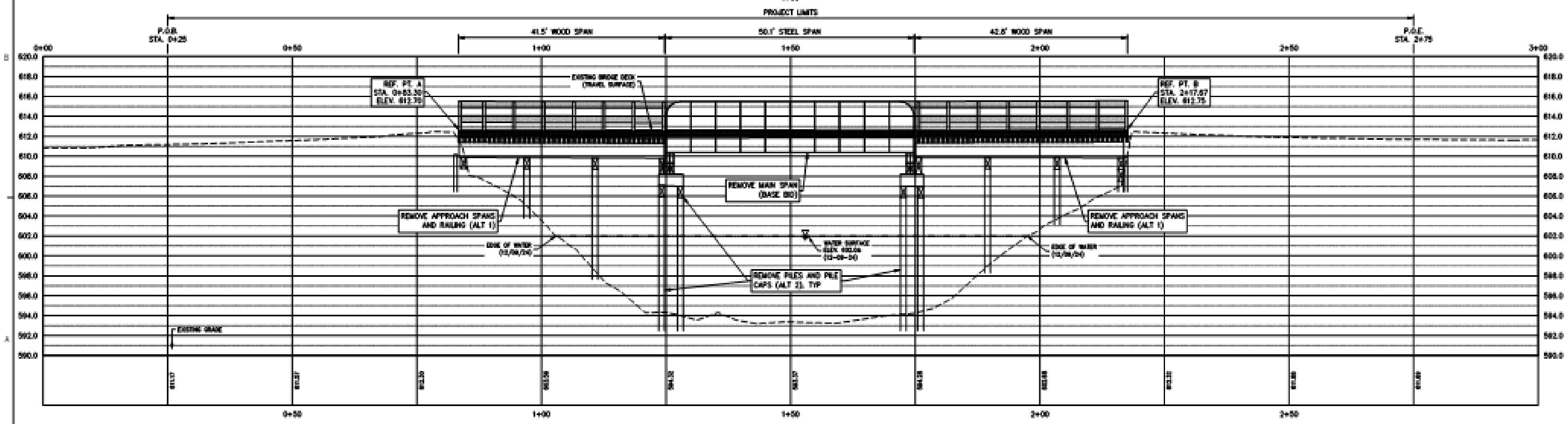

Pinewood Lodge Vacation Rental
Au Train Beach Campground
28
AuTrain River Bridge
AuTrain beach
Sandy Public Beach
28
Au Train
Drag the slider or click the button
Autrain Forest Lake Rd
Autrain Forest Lake Rd
Autrain Forest Lake Rd
Northwoods Resort



C3 BRIDGE PLAN
SCALE: 1" = 10'

BRIDGE DEMOLITION

1. EXISTING MAIN SPAN WEIGHT APPROXIMATELY 60 TONS. THE TIMBER DECK WEIGHS APPROXIMATELY 30 TONS.
2. CONTRACTOR MAY ELECT TO REMOVE THE BRIDGE IN ONE PIECE OR MULTIPLE PIECES. PROVIDE DEMOLITION PLAN WITH PROPOSED EQUIPMENT LIST.
3. DECK AND RAILING TIMBERS SHALL BE SALVAGED BY REMOVING EXISTING SCREWS/FASTENERS. CONTRACTOR SHALL DELIVER LUMBER TO ____________.



A3 CENTERLINE BRIDGE PROFILE
SCALE: HORZ. 1" = 10' VERT. 1" = 5'

HORZ. SCALE
(IN FEET)
1 inch = 10 FT.

VERT. SCALE
(IN FEET)
1 inch = 5 FT.

UPEA ENGINEERS & ARCHITECTS
www.upea.com

PROPOSED AU TRAIN BRIDGE REHABILITATION
MDNR - DTMB
ALGER COUNTY, MICHIGAN

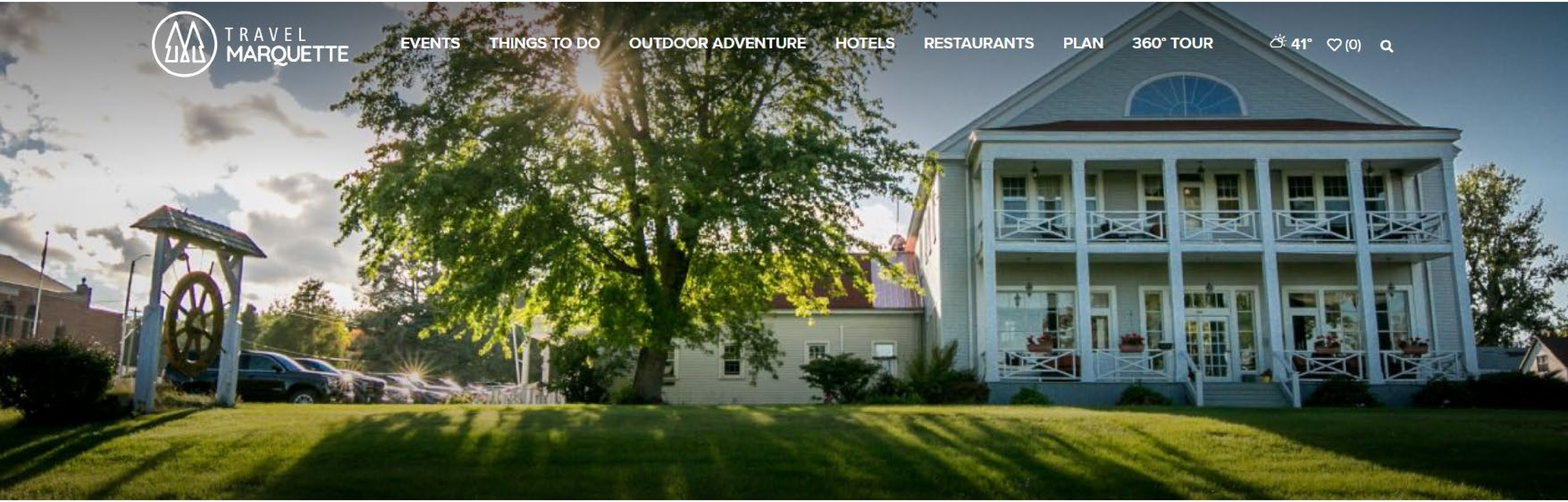
DATE:
ISSUED FOR:

PROPOSED AU TRAIN BRIDGE REHABILITATION
MDNR - DTMB
PROJECT NO. M12-06126
DESIGNED BY: ML
DRAWN BY: LGH/DJC
CHECKED: SW
APPROVED: SW

GENERAL PLAN OF STRUCTURE

5

The Bridge in the Middle of Nowhere



← Back

BIG BAY: HENRY FORD'S UP GETAWAY

Sep. 02, 2020

The Ford Bungalow: Keweenaw Bay's Secret Oasis

BY TIA OSWEILER DECEMBER 1, 2016 12:00 AM

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The Bridge in the Middle of Nowhere

Iron Range and Huron Bay Railroad

Contents hide

From Wikipedia, the free encyclopedia

The **Iron Range & Huron Bay Railroad (IR&HB)** is a defunct railroad constructed to haul iron ore in Michigan's Upper Peninsula during the 1890s. Financial and engineering problems prevented the railroad's operation; it remains an unusual example of a railroad which was completed but never used.

One of the two 4-8-0 locomotives owned by the IR&HB. They were later sold to the Algoma Central Railway.

Background [edit]

See also: Upper Peninsula of Michigan § History

Rich iron ore deposits were first discovered in the Upper Peninsula in the 1840s, and remain a significant source of wealth for the state. By the 1890s Michigan was the largest supplier of iron ore in the United States. Railroads would haul ore from the mines to great ore docks on the Great Lakes in places such as Escanaba and Marquette, where it would be loaded on ore freighters and transported to the rest of the country. The Huron Mountains west of Marquette were known to be rich in ore deposits, particularly around Lake Michigamme (near Michigamme, Michigan), and were believed to contain marble, granite, silver, gold, lead, graphite, asbestos, and silica.[1]

History [edit]

The IR&HB was formed on June 27, 1890, by seven businessmen,[2] all from Michigan's Lower Peninsula and all but one from Detroit. Milo H. Davis,[3][4] also of

THe Ore Dock in Skanee. 1890's


Big Erick's Falls
Big Eric's Bridge State Forest Campground
Erick Rd
Erick Rd
East Branch Huron River
Image © 2025 Airbus

E. Branch Huron River Spring

E. Branch Huron River Fall

1. Third bridge at this site, built in 1992 through a competitive grant from the Timber Bridge Information Resource Center

2. Three Span
   a. 73' length
   b. 32' Main Span, Post-Tensioned Box Girder
   c. 20' Post-Tensioned Timber Deck Side Spans

3. Concrete Piers & Abutments Reused

Photo 1: Big Erick's Bridge Around the 1930's

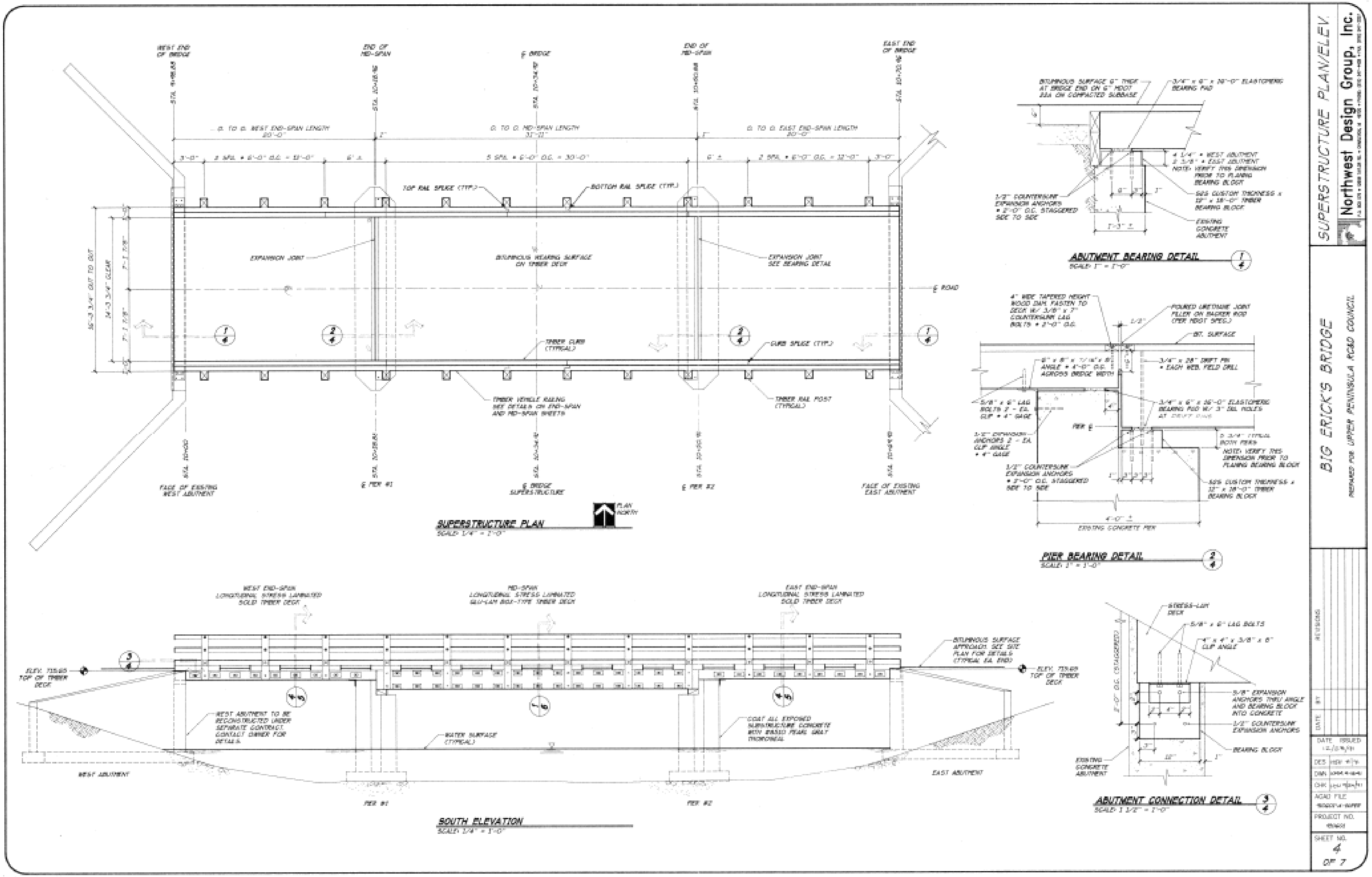
Bridge Facts

Photo 2: Big Erick's Bridge, 1957

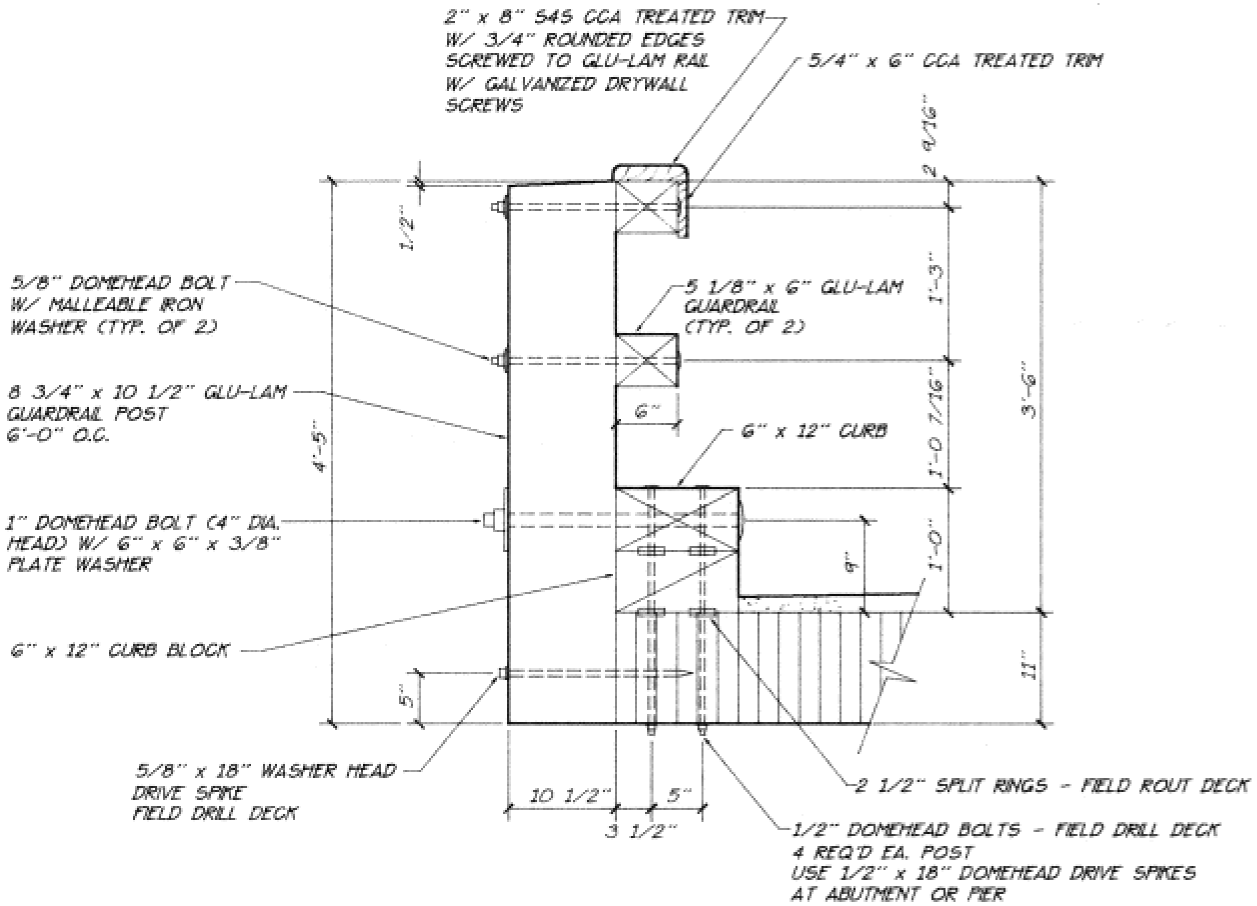
1. Post Tensioned Timber Bridge
2. Center Span is a Stress Laminated Box-Beam
3. Eastern Hemlock wood used

Unique Bridge

Nail-Laminated vs. Post Tensioned

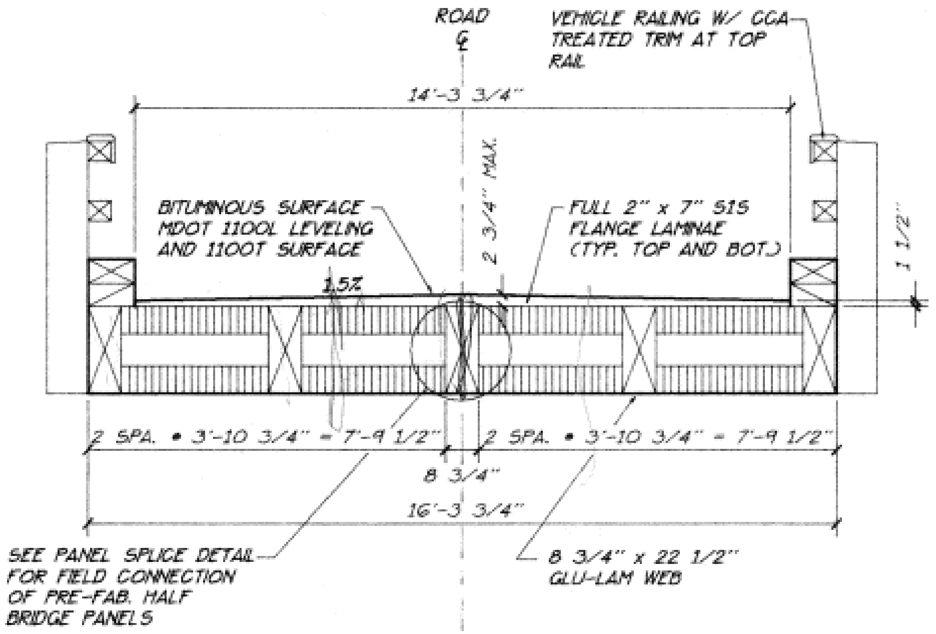


WEST END OF BRIDGE
END OF MID-SPAN
℄ BRIDGE
END OF MID-SPAN
EAST END OF BRIDGE
℄ TO ℄ WEST END-SPAN LENGTH 20'-0"
℄ TO ℄ MID-SPAN LENGTH 31'-11"
℄ TO ℄ EAST END-SPAN LENGTH 20'-0"
TOP RAIL SPLICE (TYP.)
BOTTOM RAIL SPLICE (TYP.)
EXPANSION JOINT
BITUMINOUS WEARING SURFACE ON TIMBER DECK
EXPANSION JOINT SEE BEARING DETAIL
℄ ROAD
TIMBER CURB (TYPICAL)
CURB SPLICE (TYP.)
TIMBER VEHICLE RAILING SEE DETAILS ON END-SPAN AND MID-SPAN SHEETS
TIMBER RAIL POST (TYPICAL)
FACE OF EXISTING WEST ABUTMENT
℄ PIER #1
℄ BRIDGE SUPERSTRUCTURE
℄ PIER #2
FACE OF EXISTING EAST ABUTMENT
PLAN NORTH
SUPERSTRUCTURE PLAN
SCALE: 1/4" = 1'-0"
WEST END-SPAN LONGITUDINAL STRESS LAMINATED SOLID TIMBER DECK
MID-SPAN LONGITUDINAL STRESS LAMINATED GLU-LAM BOX-TYPE TIMBER DECK
EAST END-SPAN LONGITUDINAL STRESS LAMINATED SOLID TIMBER DECK
TOP OF TIMBER DECK
BITUMINOUS SURFACE APPROACH SEE SITE PLAN FOR DETAILS (TYPICAL EA. END)
WEST ABUTMENT TO BE RECONSTRUCTED UNDER SEPARATE CONTRACT. CONTACT OWNER FOR DETAILS.
WATER SURFACE (TYPICAL)
COAT ALL EXPOSED SUBSTRUCTURE CONCRETE WITH SIKA PEARL GRAY THOROSEAL
WEST ABUTMENT
PIER #1
PIER #2
EAST ABUTMENT
SOUTH ELEVATION
SCALE: 1/4" = 1'-0"
ABUTMENT BEARING DETAIL
SCALE: 1" = 1'-0"
PIER BEARING DETAIL
SCALE: 1" = 1'-0"
EXISTING CONCRETE PIER
ABUTMENT CONNECTION DETAIL
SCALE: 1 1/2" = 1'-0"
STRESS-LAM DECK
5/8" x 6" LAG BOLTS
BEARING BLOCK
EXISTING CONCRETE ABUTMENT
SUPERSTRUCTURE PLAN/ELEV.
Northwest Design Group, Inc.
BIG ERICK'S BRIDGE
PREPARED FOR UPPER PENINSULA ROAD COUNCIL
REVISIONS
DATE BY
DATE ISSUED
DES
DWN
CHK
ACAD FILE
PROJECT NO.
SHEET NO. 4 OF 7



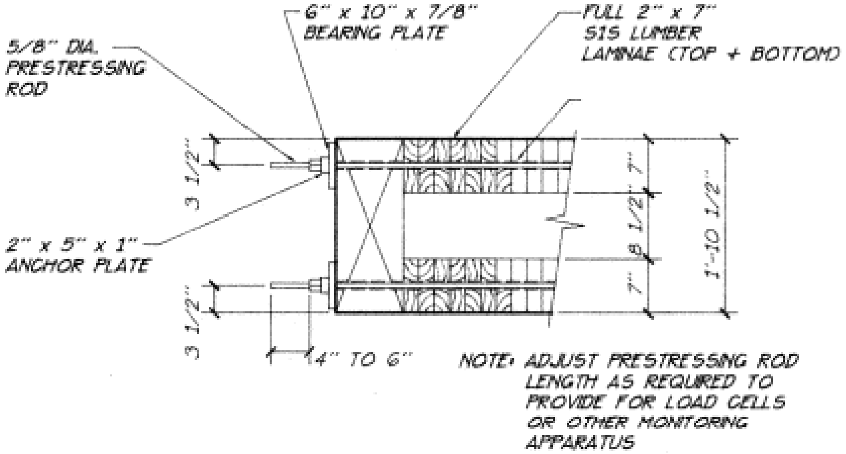
GUARDRAIL SECTION 2/5

SCALE: 1" = 1'-0"



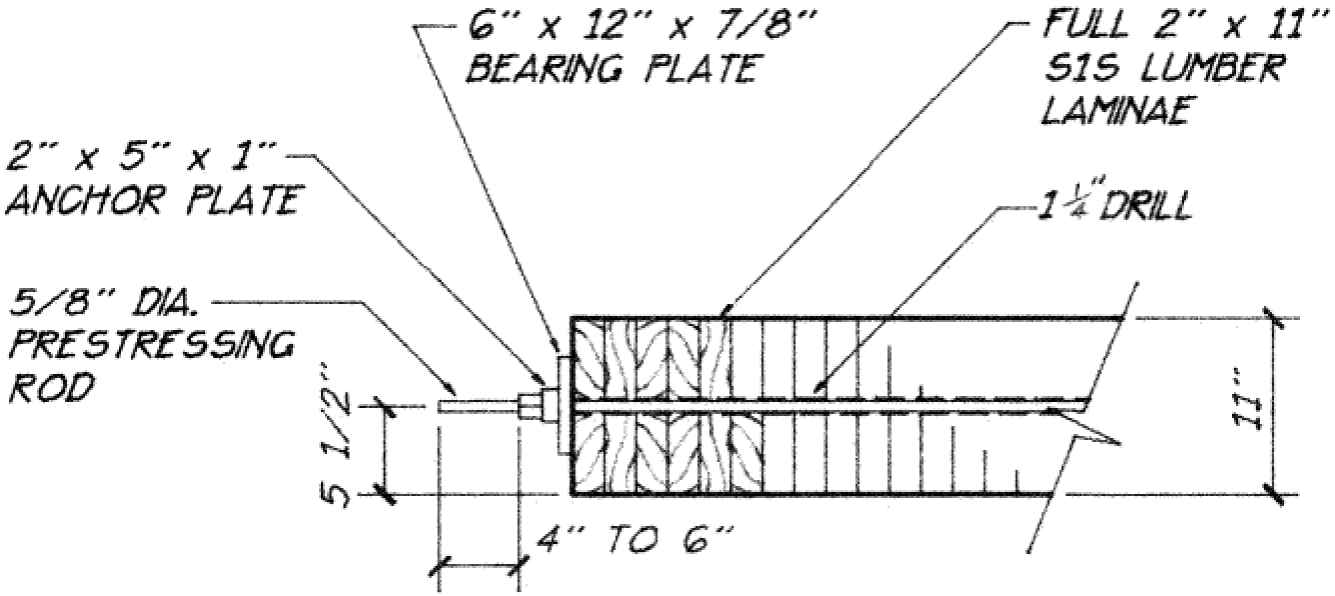
MID-SPAN SECTION 1/6

SCALE: 3/8" = 1'-0"



PRESTRESSED DECK ANCHORAGE DETAIL 3/6

SCALE: 3/4" = 1'-0"



NOTE: ADJUST PRESTRESSING ROD LENGTH AS REQUIRED TO PROVIDE FOR LOAD CELLS OR OTHER MONITORING APPARATUS

PRESTRESSED DECK ANCHORAGE DETAIL

SCALE: 3/4" = 1'-0"

1/5

Big Ericks Bridge

Big Ericks Bridge

Big Ericks Bridge

Big Ericks Bridge

1. Eastern Hemlock wood is softer than Doug Fir or Southern Pine
2. Timber bridges lose post-tensioning over time

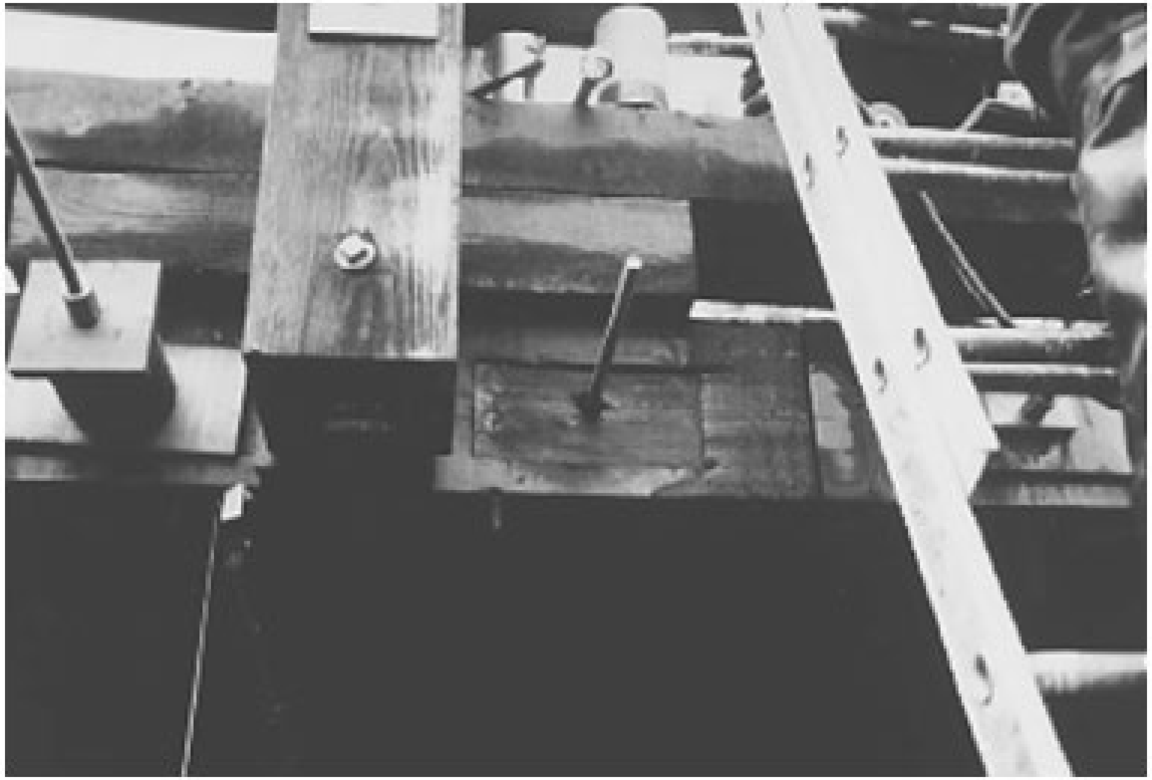


Photo 3: Visible crushing of approach span, Big Erick's Bridge

Bridge Issues

1. Timber box beam superstructures are rare and there are no nationally approved design specifications
2. Flange Lumber Flexing

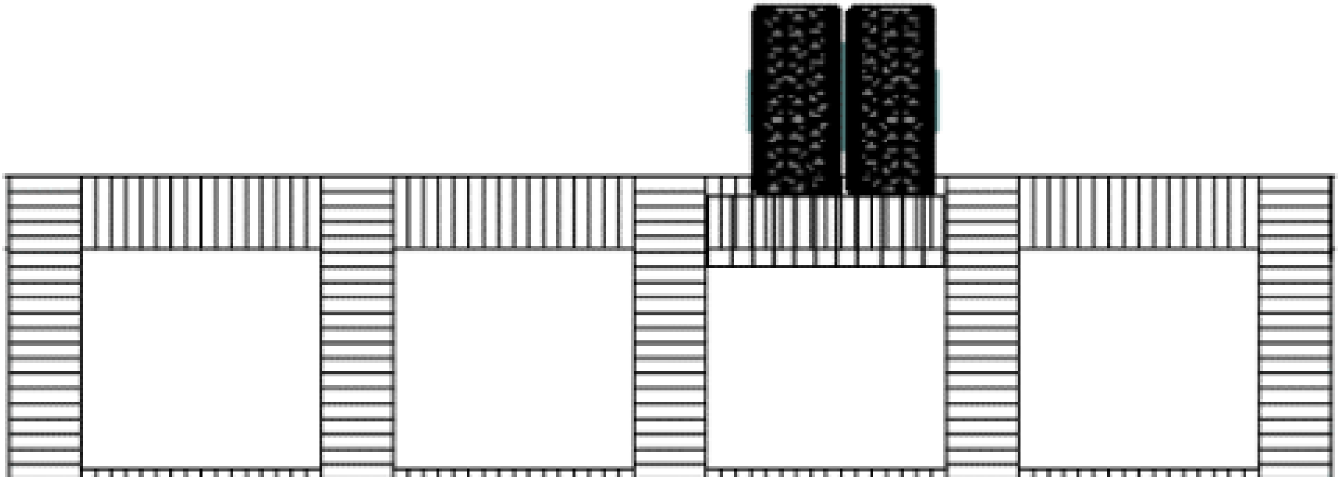


Figure 3: Flange Lumber Flexing

Bridge Issues

United States
Department of
Agriculture

Forest Service

Forest
Products
Laboratory

Timber Bridge
Information
Resource Center

Research
Paper
FPL–RP–552

Field Performance of Timber Bridges

9. Big Erick's Stress-Laminated Deck Bridge

James A. Kainz
James P. Wacker
Martin Nelson

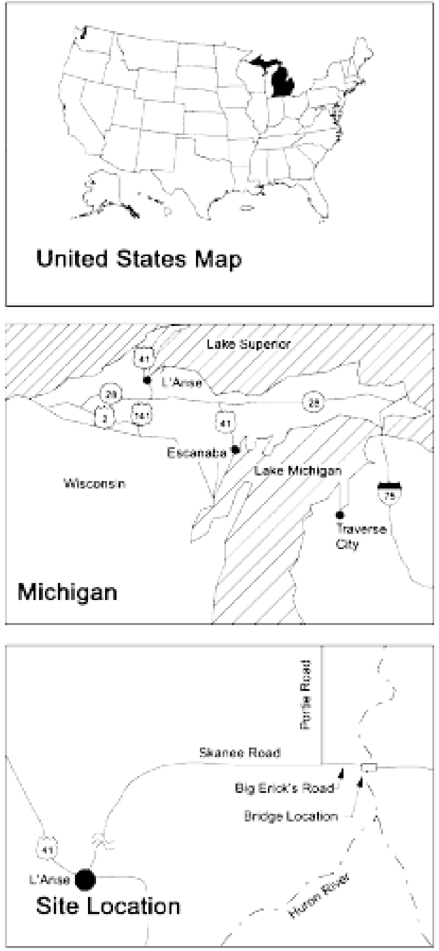


Figure 1—Location maps for the Big Erick's bridge.

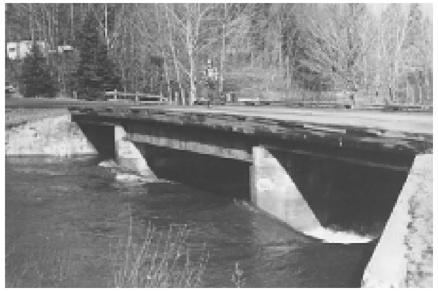


Figure 2—Big Erick's bridge prior to replacement (1980).

The original Big Erick's bridge was constructed in 1957 by the MI–DNR Fire Division to provide access for administrative vehicles, fire equipment, and logging trucks. This bridge was 20 ft wide and consisted of two 20-ft-long approach spans and a 32-ft-long center span (Fig. 2). Beneath two layers of timber plank deck, the approach spans were supported by untreated sawn lumber stringers and the center span was supported by steel stringers. Failure of the deck material in 1980 alerted MI–DNR officials to structural problems with the bridge. After the decking material was repaired, an annual inspection in 1989 revealed cracked and decayed wood stringers on the approach spans. Several options were examined to repair or replace the structure. As a result of uncertainty regarding the extent of rehabilitation required to restore full-load capacity, replacement of the bridge was deemed the most feasible solution.

As a result of the high volume of public recreation in the area, the MI–DNR chose a timber bridge as a replacement to blend with the natural aesthetics of the area. The Upper Peninsula Resource Conservation and Development Council was contacted to submit a proposal for partial funding for a replacement demonstration timber bridge under the Forest Service demonstration program. As part of this proposal, a preliminary bridge design was developed, consisting of two stress-laminated decks and one stress-laminated box. Eastern Hemlock was selected as a primary material for the bridge because it is an underutilized, locally grown species in the Upper Peninsula of Michigan. The proposal was submitted in January 1991 to the TBIRC for inclusion under the Timber Bridge Initiative (USDA 1995). After review by a selection panel, funds were awarded and final design of the replacement bridge was initiated. During the final design phase of the project, FPL was contacted to monitor the new Big Erick's timber bridge. As a result, FPL and MI–DNR developed a mutually agreeable monitoring plan that was initiated at installation.

Figure 5—Eastern Hemlock lumber for the Big Erick's bridge awaiting preservative treatment.

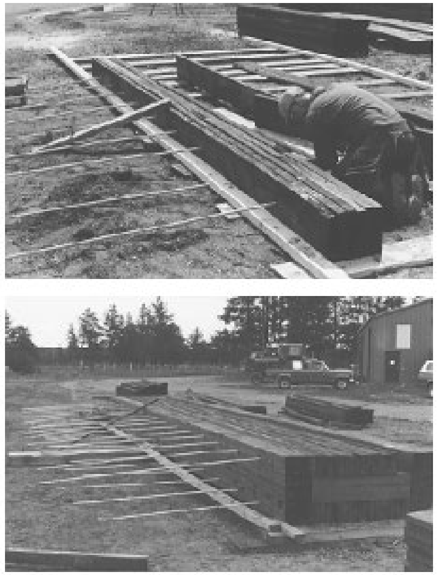


Figure 6—Prefabrication of the stress-laminated deck (top) and stress-laminated box (bottom) in half-width panels at an outdoor work site.

The stressing was accomplished with a hydraulic jacking system, which consisted of a hydraulic pump, a single hollow core jack, and a stressing chair (Fig. 7) (Wacker and Ritter 1992). After all panels were assembled and stressed, a second design load stressing was completed August 26, 1992, approximately 1 month after the initial stressing. The panels were then loaded on two flatbed trailers and transported to the bridge site in preparation for bridge installation (Fig. 8).

Figure 7—High strength steel bars were tensioned using a single hydraulic jack, electric pump, and steel chair.

Figure 8—Prefabricated half-width panels arriving at the bridge site prior to installation.

Installation

After demolition and removal of the existing bridge superstructure, an assessment of the reinforced concrete substructure revealed that the west abutment had sustained freeze-thaw damage. To remedy the problem, the damaged part of the west abutment was removed, and the remaining portion of the abutment was capped with new concrete. The remainder of the substructure was structurally adequate, and the complete substructure was coated with a sealer prior to the new bridge installation. Wood sleeper blocks, measuring 12.5 by 5.75 in., were added at the center span piers to raise the new bridge to the existing roadway grade. After the addition of neoprene bearing pads, additional modifications were not made to the abutments or piers.

When the substructure work was completed, the assembled half-width panels were lifted into place with a large overhead crane (Fig. 9). After the half-width panels were placed, each panel was connected to the adjacent panel with high strength steel couplers on the stressing bars (Fig. 10). After all bars were coupled, stressing bars were partially tensioned to bring the half-width panels into contact. Full-design bar force was then introduced into the bridge by utilizing two separate sets

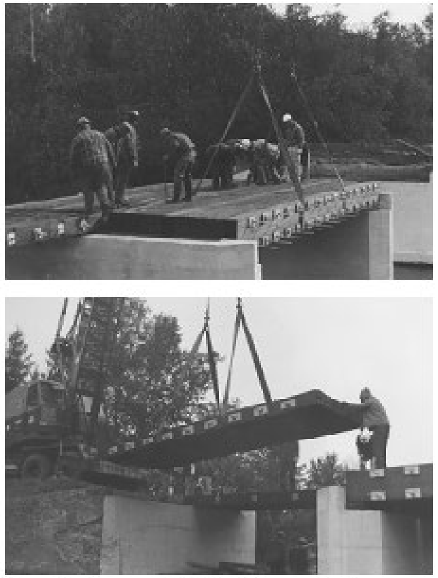


Figure 9—Placement of half-width panels with an overhead crane.

of hydraulic equipment. The MI–DNR crew began at opposite ends of each span and stressed each bar along the length of that span (Fig. 11). On the center span, each set of bars (top and bottom) was tensioned before moving along the span length to the next set of bars. After all bars were fully tensioned, the stressing process was repeated to ensure that the stress level was uniform and at the required design level.

After bar stressing, the bridge was attached to the substructure. The two approach spans were connected by steel angles, concrete anchors, and lag bolts located in the corners, at the abutments, and underneath the deck at the piers. The center span was connected by drift pins driven through the end of each glulam web to the sleeper block on the concrete pier.

Following substructure attachment, the timber curb and rail system was installed. As a result of expected pedestrian traffic from the adjoining campground, CCA-treated lumber facia boards were installed over the creosote rail members and a roofing sealer was applied to the curb. The asphalt wearing surface was applied approximately 2 weeks after bridge installation. The completed bridge is shown in Figure 12.

Figure 10—Connection betwen half-width panels was achieved by placing couplers on adjacent bars and stressing the entire deck together.

Figure 11—Two sets of deck stressing equipment were used to tension the stressing bars after deck placement.

Figure 12—Completed Big Erick's bridge: side view (top), end view (bottom).

Table 2—Final costs of bridge superstructure

Span	Deck area (ft^2)	Cost (US$) Materials	Labor	Total	Per ft^2
Approach[a]	640	12,390	15,391	27,781	43
Center	512	24,909	13,467	38,376	75

[a]Costs are based on total costs for west and east spans.

Cost

Total cost of the Big Erick's bridge superstructure was $66,157 and included fabrication, materials, labor, and construction. As a result of the different design methodologies used in this three-span bridge, costs for each span type were tabulated separately and are summarized in Table 2. The cost per square foot is based on the total cost of the span divided by the total deck area of that span. The $75/ft^2 for the center span was almost twice the $43/ft^2 for the approach spans. The increased center span cost is attributable to the greater quantity of material and longer installation time. The increased material cost of the center span is the result of using glulam timber for the webs; two rows of plates, bars, and nuts; and two layers of flange decking material.

Evaluation Methodology

To evaluate the structural and serviceability performance of the Big Erick's bridge, MI–DNR contacted FPL for assistance. Through mutual agreement, a 3-year bridge monitoring plan was developed by the FPL and implemented through a Cooperative Research and Development Agreement with MI–DNR. The plan included performance monitoring of the deck moisture content, stressing bar force, static-load test behavior, and general bridge condition. During the installation of the bridge, FPL representatives visited the bridge site to install instrumentation and train MI–DNR personnel in the data collection process for moisture content and bar force measurements. Static-load tests and general bridge condition assessments were conducted by FPL representatives during site visits. The evaluation methodology employed procedures and equipment developed by FPL and used on similar structures (Ritter and others 1991).

Moisture Content

To characterize changes in moisture content, an electrical-resistance moisture meter was used to obtain wood moisture content readings on a monthly basis. Moisture meter measurements were taken by MI–DNR personnel from the sawn lumber in the west approach span and were assumed to be representative of overall bridge moisture content. The west span was chosen because of safety concerns in accessing other bridge locations. Measurements were obtained in accordance with ASTM D4444–84 (ASTM 1990) by driving the moisture pins into the underside of the deck at depths of 2 to 3 in., recording the moisture content value from the unit, then adjusting the moisture content value for temperature and wood species, if necessary.

Figure 13—One of six load cells installed on stressing bars to measure changes in bar force.

Bar Force

To monitor stressing bar force, two calibrated load cells were placed on each span of the Big Erick's bridge at the time of bridge installation. The cells were placed on the stressing bar, between the bearing and anchorage plates, to monitor bar forces based on the strain variations in the load cell (Fig. 13). Load cell measurements were obtained by MI–DNR personnel with a portable strain indicator on a biweekly basis for the first year, and monthly thereafter. The measurements were then converted to force levels, based on laboratory load cell calibrations, to determine the tensile force in the bar. Approximately midway through the monitoring period, the load cells were unloaded and adjusted for zero balance shift. At the conclusion of the monitoring period, the load cells were removed, adjusted for zero balance shift, and recalibrated in the laboratory. In addition, hydraulic stressing equipment was used at the site visits during the monitoring period to verify bar force levels obtained from the load cells.

Load Test Behavior

To determine load behavior of the Big Erick's bridge, two static-load tests were conducted during the monitoring period. The first load test was completed on two separate occasions; the approach spans were tested 2 months after bridge installation and the center span was tested 11 months after installation. A second and final load test was completed 35 months after bridge installation and included tests on all three spans.

The static-load test consisted of positioning a fully loaded truck separately on each span and measuring the resulting deflections along the transverse midspan of the loaded span. For both load tests, the truck was positioned for three transverse load cases on each span (Fig. 14). The first load case centered the vehicle over the longitudinal centerline; the second load case located the vehicle adjacent to the upstream curb; the third load case located the vehicle adjacent to the

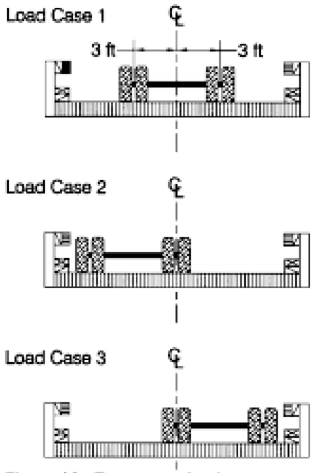


Figure 14—Transverse load cases (looking west) used for all static-load tests on the Big Erick's bridge.

downstream curb (Fig. 15). Longitudinal vehicle placement differed, depending upon the span length and vehicle configuration. Measurements of bridge deflections were taken prior to testing (unloaded), for each load case (loaded), and at the conclusion of testing (unloaded). Measurements of bridge deflections from an unloaded to loaded condition were obtained by hanging calibrated rules on the underside of the deck and reading values with a surveyor's level. The accuracy of this method for repetitive readings is estimated to be ±0.04 in.

Load Test 1A

The first load test was November 19, 1992, and involved approach span testing only. The test vehicle was a loaded, three-axle dump truck with a gross vehicle weight of 54,760 lb (Fig. 16). The vehicle was positioned longitudinally, with the tandem rear axles bisecting the midspan with the front axle off the bridge. The truck faced west when testing the west span and east when testing the east span. Data points were transversely positioned along the midspan of each approach span at an interval of 2 ft, beginning at the longitudinal bridge centerline. It should be noted that the data points were not directly below the vehicle wheel lines during load case 1.

Load Test 1B

Load test 1B involved the center span only and was completed August 23, 1993,—9 months after the sawn lumber approach spans were tested. The test vehicle was a loaded, three-axle dump truck with a gross vehicle weight of

Figure 15—Transverse load test cases used for all spans: (top) load case 1, centered on roadway; (middle) load case 2, adjacent to upstream curb; (bottom) load case 3, adjacent to downstream curb.

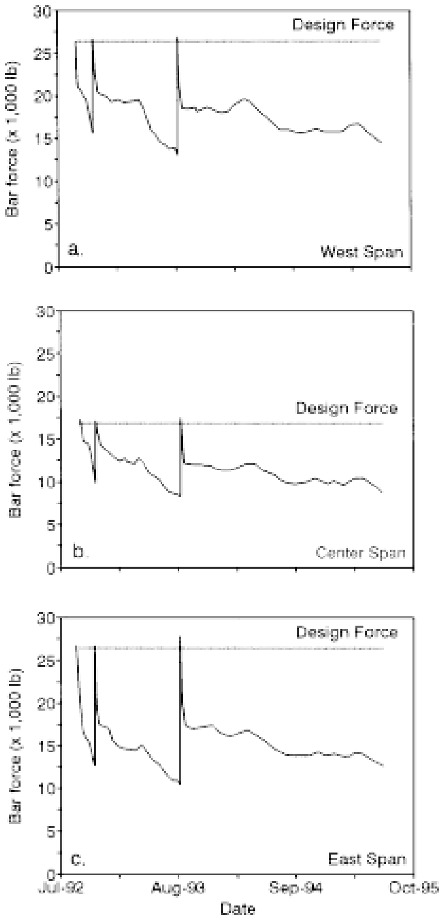


Figure 18—Average trend in bar force, starting at the third bar tensioning for (a) west, (b) center, and (c) east spans.

for the west span, 8,700 lb for the center span, and 12,500 lb for the east span, which is 45 to 55 percent of the design bar force. With each bar tensioning, the rate of bar force loss decreased, although the overall amount of bar force lost between stressings remained relatively equal.

The bar force loss was probably the result of three factors: stress relaxation, prefabrication methodology, and crushing of the timber laminations. The majority of bar force loss was the result of stress relaxation, a time-dependent phenomenon caused by the long-term compressive force of the steel bars acting on the wood microstructure. The rate of stress relaxation was greater on the Big Erick's bridge when compared with other stress-laminated bridges and was probably the result of the high moisture content in the timber laminations. An additional factor affecting bar force loss was the prefabrication methodology of completing two of the recommended bar tensionings on partial-width sections. This methodology diminished the positive effects of repetitive tensionings, because the bars were not full width until the third tensioning. Another plausible cause of bar force loss was crushing of the exterior laminations on the approach spans. Crushing was observed under the bearing plates during construction and the monitoring period. The rate of crushing and the corresponding bar force loss decreased with each subsequent bar tensioning. This is discussed in detail in the Condition Assessment section.

The observed bar force on the center and approach spans behaved similarly, as displayed in Figure 18. The center and approach spans have different bar force design values; therefore, percentage of bar force loss was used for comparison. All spans exhibited similar bar force losses of 40 to 50 percent before the fourth stressing, 50 to 60 percent prior to the fifth stressing, and 45 to 55 percent at the end of monitoring. These similarities between the approach spans and the center span suggest that stress-laminated decks and boxes, built from comparable materials, perform alike when subjected to the same environmental conditions.

Load Test Behavior

Results of the static-load tests and analytical assessment of the Big Erick's bridge are presented. For each load case, transverse deflection measurements are given at the midspan of each span as viewed from the east end (looking west). No permanent residual deformation was measured at the conclusion of the load testing, and there was no detectable movement at bridge supports. At the time of load tests 1A and 1B, the average bridge prestress was approximately 100 lb/in^2. For load test 2, the bridge prestress was approximately 56 lb/in^2 in the west span, 52 lb/in^2 in the center span, and 47 lb/in^2 in the east span.

Load Test 1A and 1B

Transverse deflections for load test 1A and 1B with the locations and magnitudes of the maximum measured deflections are shown in Figure 19. For each of the three load positions on the west and east approach spans, the deflections are typical of the orthotropic plate behavior observed for other stress-laminated decks (Ritter and others 1990). For the west span, the maximum measured deflection was 0.45 in. for load case 1 and 0.49 in. for load cases 2 and 3. For the center span, the maximum measured deflection was 0.31 in. for load case 1, 0.37 in. for load case 2, and 0.35 in. for load case 3. On the east span, the maximum measured deflection was 0.37 in. for load case 1 and 0.43 in. for load cases 2 and 3. As shown in Figure 19, the approach span maximum measured deflections occurred virtually in the same locations for each load case with minor variations, which are within

1. The last time the bridge was re-tensioned was in 2006.
2. It is recommended that timber superstructures be re-tensioned every 5 to 10 years.
3. If you own a post-tensioned timber bridge, when was the last time you had it re-tensioned?
4. Should re-tensioning affect your load rating?

bearing plates into the Southern Pine glulam exterior laminations of the center span. However, crushing of the Eastern Hemlock laminations on the approach spans was first observed during bridge construction and continued progressively until the end of monitoring (Fig. 29), and seems to have contributed to short-term bar force loss after bar tensioning. This crushing occurred even though the bearing plates were properly designed, based on published values for compression perpendicular to grain adjusted for wet-use conditions. This indicates that the published design values for compression perpendicular to grain may not be representative of the material for use in this application. The use of a treatable hardwood species, such as oak or maple, on the outer laminations has been shown to perform well without crushing problems (Ritter and others 1995).

Conclusions

After 35 months in service, the Big Erick's bridge is performing satisfactorily and should provide many years of acceptable service. Based on extensive bridge monitoring, we make the following observations and recommendations:

- The sawn lumber components of the Big Erick's bridge were initially installed at 33-percent moisture content. The average trend in moisture content indicates that changes are occurring slowly, with an average decrease of 5 percent during the monitoring period. The high moisture content contributed to bar force and camber loss during the monitoring period. It is expected that the moisture content will gradually decrease and lamination dimensional change should be expected. Future bridges constructed of this type should utilize laminations with moisture content at or below 19 percent.
- Following the final design bar tensioning, two additional bar tensionings were required due to rapid loss of bar force. The decrease in bar force was primarily attributable to transverse stress relaxation in the wood laminations with additional effects from crushing of the approach span exterior laminations near the bar anchorages. It is anticipated that the bridge will require retensioning in the near future. In addition, as the moisture content decreases, the bar force will also decrease until an equilibrium moisture content is reached. Therefore, bar force should be checked annually and retensioned as necessary until it reaches a constant level.
- Load testing and analysis indicate that all three spans of the Big Erick's bridge are performing in a linear elastic manner, and the approach spans exhibit orthotropic plate behavior when subjected to truck loading. HS25–44 truck loading conditions produced maximum deflections of 0.46 and 0.39 in. for the west and east approach spans at 100 lb/in^2 and 0.56 in., and 0.49 for the west and east approach spans at 57 and 47 lb/in^2, respectively. These deflections correspond to values of L/490, L/578, L/417, and L/460, based on the center–center of bearing span length.

Figure 27—Vehicle damage to curb and rail system observed during a site visit near the end of the monitoring period.

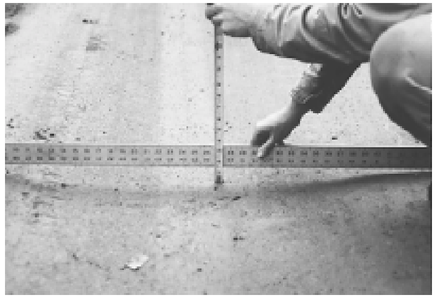


Figure 28—Rutting of asphalt wearing surface measured at the center span during a site visit for load test 2.

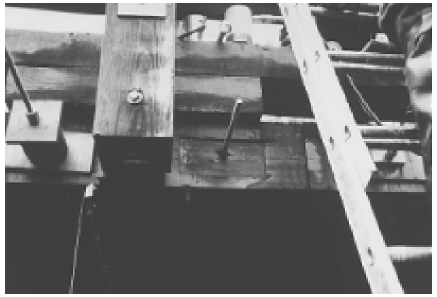


Figure 29—Visible crushing of approach span, Eastern Hemlock lumber lamination, observed after the removal of the bearing plate at the final site visit.

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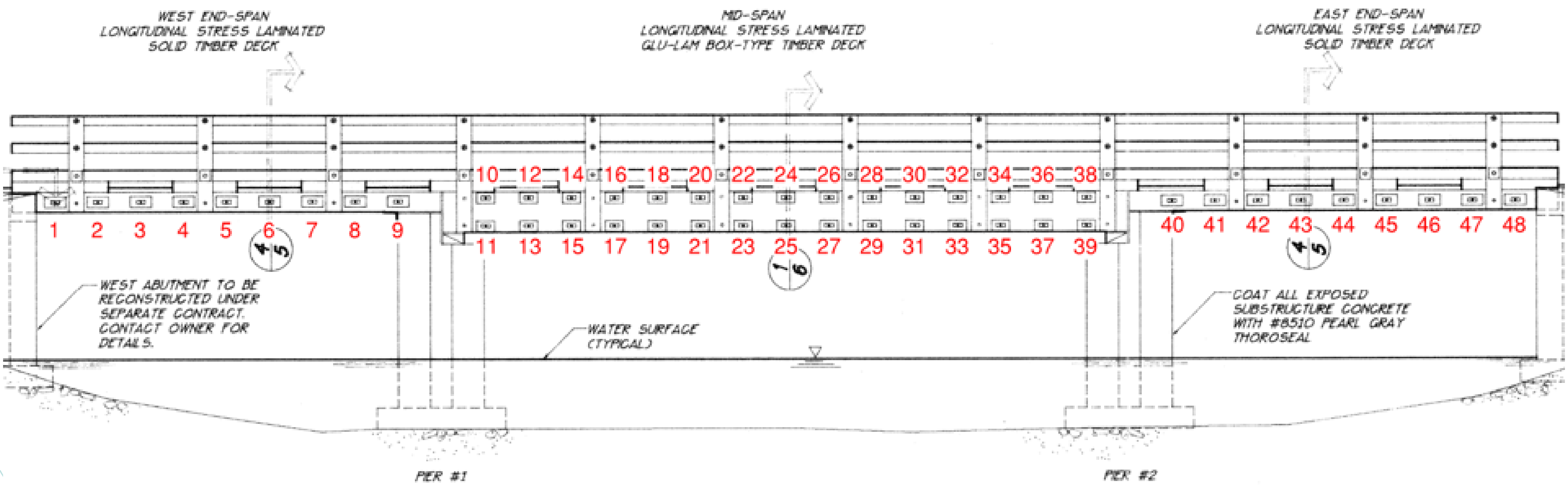
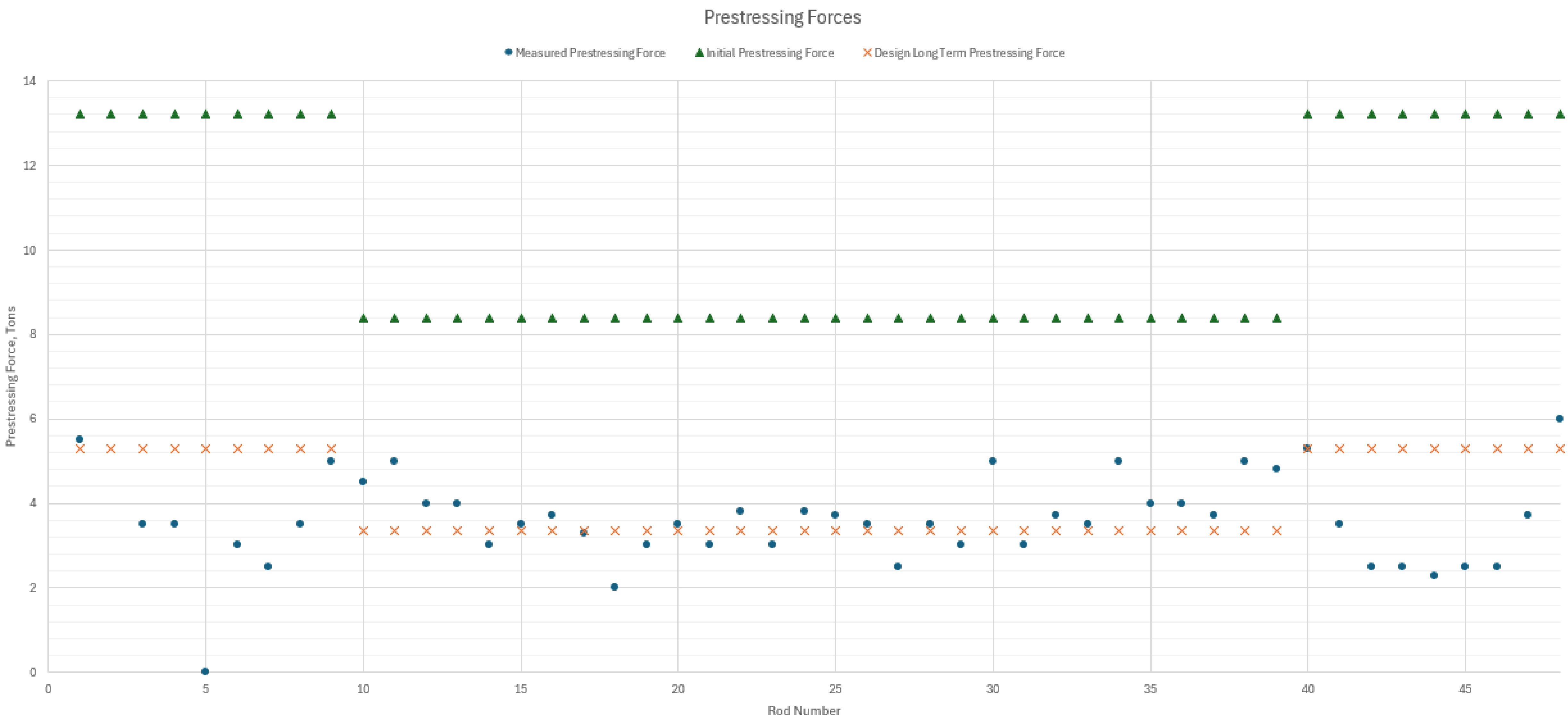


SOUTH ELEVATION

SCALE: 1/4" = 1'-0"



Prestressing Forces
Measured Prestressing Force
Initial Prestressing Force
Design Long Term Prestressing Force
Prestressing Force, Tons
Rod Number
0
2
4
6
8
10
12
14
0
5
10
15
20
25
30
35
40
45

Rod Number	Side Tensioned	Measured Prestressing Force	Design Long Term Prestressing Force	Initial Prestressing Force	Re-tensioning Movement	Bar Force Loss
		Tons	Tons	Tons	in	Percent
1	North	5.5	5.28	13.2	0.5	58%
2	South		5.28	13.2		100%
3	North	3.5	5.28	13.2	0.75	73%
4	South	3.5	5.28	13.2	0.5625	73%
5	North	0	5.28	13.2	0.5	100%
6	South	3	5.28	13.2	0.5625	77%
7	North	2.5	5.28	13.2	0.75	81%
8	South	3.5	5.28	13.2	0.5	73%
9	North	5	5.28	13.2	0.625	62%
10	North	4.5	3.36	8.4	0.125	46%
11	North	5	3.36	8.4	0.1875	40%
12	South	4	3.36	8.4	0.25	52%
13	South	4	3.36	8.4	0.25	52%
14	North	3	3.36	8.4	0.25	64%
15	North	3.5	3.36	8.4	0.25	58%
16	South	3.7	3.36	8.4	0.25	56%
17	South	3.3	3.36	8.4	0.25	61%
18	North	2	3.36	8.4	0.25	76%
19	North	3	3.36	8.4	0.25	64%
20	South	3.5	3.36	8.4		58%
21	South	3	3.36	8.4		64%
22	North	3.8	3.36	8.4	0.375	55%
23	North	3	3.36	8.4	0.25	64%
24	South	3.8	3.36	8.4	0.25	55%
25	South	3.7	3.36	8.4	0.3125	56%
26	North	3.5	3.36	8.4	0.25	58%
27	North	2.5	3.36	8.4	0.25	70%
28	South	3.5	3.36	8.4	0.25	58%
29	South	3	3.36	8.4	0.25	64%
30	North	5	3.36	8.4	0.1875	40%
31	North	3	3.36	8.4	0.25	64%

1. Bridge Owners: Schedule re-tensioning every 5 to 10 years.
2. Inspectors: Look for signs of post-tensioning loss.
3. Load Raters: Consider temporarily reducing load carrying capacity if the last re-tensioning was completed over 10 years ago.
4. Designers: Consider that bridge maintenance is often delayed and sometimes even forgotten about.

32	South	3.7	3.36	8.4	0.25	56%
33	South	3.5	3.36	8.4	0.25	58%
34	North	5	3.36	8.4	0.1875	40%
35	North	4	3.36	8.4	0.125	52%
36	South	4	3.36	8.4	0.1875	52%
37	South	3.7	3.36	8.4	0.1875	56%
38	North	5	3.36	8.4	0.125	40%
39	North	4.8	3.36	8.4	0.125	43%
40	North	5.3	5.28	13.2	0.5	60%
41	South	3.5	5.28	13.2	1	73%
42	North	2.5	5.28	13.2	0.75	81%
43	South	2.5	5.28	13.2	1.375	81%
44	North	2.3	5.28	13.2	1.125	83%
45	South	2.5	5.28	13.2	1.125	81%
46	North	2.5	5.28	13.2	1.125	81%
47	South	3.7	5.28	13.2	1	72%
48	North	6	5.28	13.2	0.625	55%

Lessons Learned

Big Ericks Bridge

Questions