Comprehensive Study

Merrimac Railroad Bridge No. 334

Wisconsin & Southern Railroad Company

January 2010

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AYRES ASSOCIATES

MERRIMAC RAILROAD BRIDGE NO. 334

COMPREHENSIVE STUDY

Prepared for:

WISCONSIN & SOUTHERN RAILROAD CO.
1890 E. Johnson Street
Madison, Wisconsin 53704

January 2010

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COMPREHENSIVE STUDY

Prepared by:

____
Steven Laney
Supervisor – Structural Engineering

____
Ivan Gualtero
Structural Engineer

AYRES ASSOCIATES
3433 Oakwood Hills Parkway
Eau Claire, WI 54701-7698
715.834.3161 • Fax: 715.831.7500
www.AyresAssociates.com

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Executive Summary

Wisconsin & Southern Railroad Co. (WSOR) contracted with Ayres Associates (Ayres) to provide a comprehensive engineering study to evaluate and load rate its existing bridge over the Wisconsin River and Lake Wisconsin near Merrimac, Wisconsin. The study was funded by WSOR with financial participation from Sauk County. The steel deck truss and deck girder bridge, Structure 334, was inspected, analyzed, and evaluated to determine the current conditions against the original design and for future considerations using heavier Cooper loadings to verify the feasibility of possible improvements.

In addition to verifying the structural capacity of the bridge for present and future loadings, Ayres also provided a survey of the site and determined the existing river bed channel profiles to compare with the original profiles for a scour assessment of the substructure units. An evaluation for ice vulnerability was included to determine if any improvements were needed, and an evaluation of converting to welded rail from the present rail sections is provided.

The results of the structural analysis indicate the existing bridge is at or just below the Cooper E40 rating level as shown in Table 1. Although the current condition shows that the Cooper rating is below E40, the bridge is considered to be in good condition, and with the implementation of repairs to the piers and chords with pack rust, the rating of the bridge can be adjusted to the as-designed level for Cooper E40.

Table 1 Bridge Rating Summary.

Span	AS DESIGNED AND CONSTRUCTED		CURRENT CONDITION	
	Normal Load Rating	Maximum Load Rating	Normal Load Rating	Maximum Load Rating
A	E45	E79	E45	E79
B & C	E47	E78	E42	E70
D to M	E46	E68	E44	E65
N	E92	E159	E87	E151
O	E40	E65	E36	E59
P	E56	E88	E53	E84
Q	E40	E66	E38	E63
R	E41	E69	E37	E62
S	E53	E82	E50	E78
T	E40	E53	E38	E50
U	E40	E53	E36	E48
Minimum	**E40**	**E53**	**E36**	**E48**

Preliminary engineering costs for repairs, rehabilitation, or replacement were also evaluated to provide WSOR a summary of possible funding needs. Table 2 is the cost summary that was developed.

Table 2 Cost Summary.

Repair Method	Estimated Cost	Estimate Remaining Service Life
Routine Repairs	$4,200,000	25 years
3rd Line of Girders/Trusses	$24,000,000	30 years
New Structure	$35,000,000 to $65,300,000	75 years

To upgrade to normal Cooper E80 loading, replacement of the existing bridge is the recommended alternative that will also provide for the expected service life for a new structure. Although different replacement alternatives were analyzed and further study would be needed, the preferred design would include 16 spans with a typical section of 110' deck plate girders supported by drilled shafts. This would meet the proper clearance and loading requirements. The preferred location is just south of the existing structure and would allow the line to remain open until the traffic can be switched. Preliminary review of environmental impacts does not appear to show that there would be issues to preclude the new alignment. However, additional right-of-way would be necessary.

The existing bridge was also evaluated for vulnerability to ice pressures and ice flow, particularly the spans and bearings closer to the water surface. In review of the supporting data and referring to the inspection results, it was found that the concrete piers are being abraded by the flow of ice or the impact of ice sheets, and the masonry piers appear to be losing some stones due to freeze-thaw cycles. In both cases, repairs to the piers are recommended and will be sufficient to counteract the effects of ice.

Continuous welded rail was structurally analyzed for the existing bridge and found to cause an increase in stresses, thus reducing the rating capacity in some of the truss members to an unacceptable level. However, additional expansion joints can be installed and would be recommended throughout the existing bridge while the rail is secured using sliding joints.

The effects of the Prairie du Sac Dam on the bridge were also evaluated. The dam was constructed in 1907 approximately 7 miles downstream from the bridge, causing the backwater to rise at the bridge and placing some components closer to the water surface. Corrosion was evaluated and does not appear to have been increased significantly.

If the priorities of WSOR are to maintain current rail traffic while allowing additional traffic not to exceed the structural rating, implementing the repairs are recommended to maintain the structure at its current Cooper E40 normal operating level as long as locomotive and car equipment are available that meet the lower ratings. If the priorities are to increase the traffic level and use of the line where standard locomotives and cars are expected, then a new structure is recommended to upgrade to the Cooper E80 rating levels.

1. Introduction

Wisconsin & Southern Railroad Co. (WSOR) contracted with Ayres Associates (Ayres) to provide a comprehensive engineering study to evaluate and load rate its existing bridge over the Wisconsin River and Lake Wisconsin near Merrimac, Wisconsin. The study was funded by WSOR with financial participation from Sauk County. The steel deck truss and deck girder bridge, Structure 334, was inspected, analyzed, and evaluated to determine the current conditions against the original design and for future considerations using heavier Cooper loadings to verify the feasibility of possible improvements. The line is used for freight service, but WSOR may want to include commuter rail service.

In addition to verifying the structural capacity of the bridge for present and future loadings, Ayres also provided a survey of the site and determined the existing river bed channel profiles to compare with the original profiles for a scour assessment of the substructure units. An evaluation for ice vulnerability was included to determine if any improvements were needed, and an evaluation of converting to welded rail from the present rail sections is provided.

Repairs and improvements were also evaluated with costs associated and are provided along with final recommendations.

The structure has 22 spans and is 1,729 feet long. It consists of eight (8) deck truss spans and fourteen (14) deck girder spans. Structure 334 was originally constructed in 1895 and rehabilitated in 1903 and 1930. Also, the steel towers for Piers 4 to 14 were encased with concrete after the original construction in 1911 and 1915. During the 1903 rehabilitation, Spans A, B and O to U were replaced, but the existing deck girder spans from Spans C to N were left in place. The significant span modifications in 1930 included the following:

1. Pier 3 was removed and replaced 40’-9” closer to the east end from its original position. Span B truss span was shortened by removing the westernmost panels for the north and south trusses but overhangs the new Pier 3. Span B was modified from a 144’-7¼” span to a 103’-10” span. For Span C, the original 48’-2 7/8” deck girder span was removed, and a longer deck girder span was installed as a pin type connected span to the truss where Span C is now a combination span with truss and girder superstructure types with a new length of 89’-0”.

2. Piers 15 and 16 were removed and replaced 29’-9” closer to the west end from their original position. Span O truss was shifted with the new piers. Span P truss was shortened by removing the two easternmost panels of the north and south trusses and does not overhang the new Pier 16. Span P was modified from 107’-2¼” to 77’-5”. For Span N, the original 49’-1” deck girder span was removed, and a longer deck girder span, 78’-11”, was installed.

Structure 334 is located at Mile Post 164.15 and crosses the Sauk County-Columbia County line near Merrimac. Refer to Figure 1-1. The segment of track lies between the cities of Merrimac and Lodi and is part of the main track from Madison to Reedsburg. The stationing and numbering or labeling of the bridge is from east to west. Abutment 1 is on the east side of Wisconsin River in Columbia County.

The spans are labeled from Span A to Span U on the original plans, and this labeling will be used for this report as well. Spans A and C to N are the deck girder spans and Spans B and O to U are the deck truss spans. Span S is the movable swing span presently not in operation. Refer to the plan and elevation view in Appendix A.

Although the bridge does not appear to be on the national or state historical registers, the bridge has historical significance due to the double intersecting Warren type truss systems that are no longer commonly used today. Truss systems constructed without vertical members are rare.

The construction of the bridge used riveted connections typical for the period of construction. Today, if repairs are needed, bolts would be installed in place of the rivets or used for new members.

For the structural evaluation, an in-depth above water and underwater inspection was provided to verify the existing conditions of the structural members and locate areas of deficiencies. The deficiencies were incorporated into the analyses of the spans to determine the normal and maximum capacity ratings.

This corridor is considered to be a vital and significant link, and this report is provided to describe the current conditions and capacity status of Structure 334, along with recommendations for repairs and improvements to maintain, upgrade, or replace the bridge.

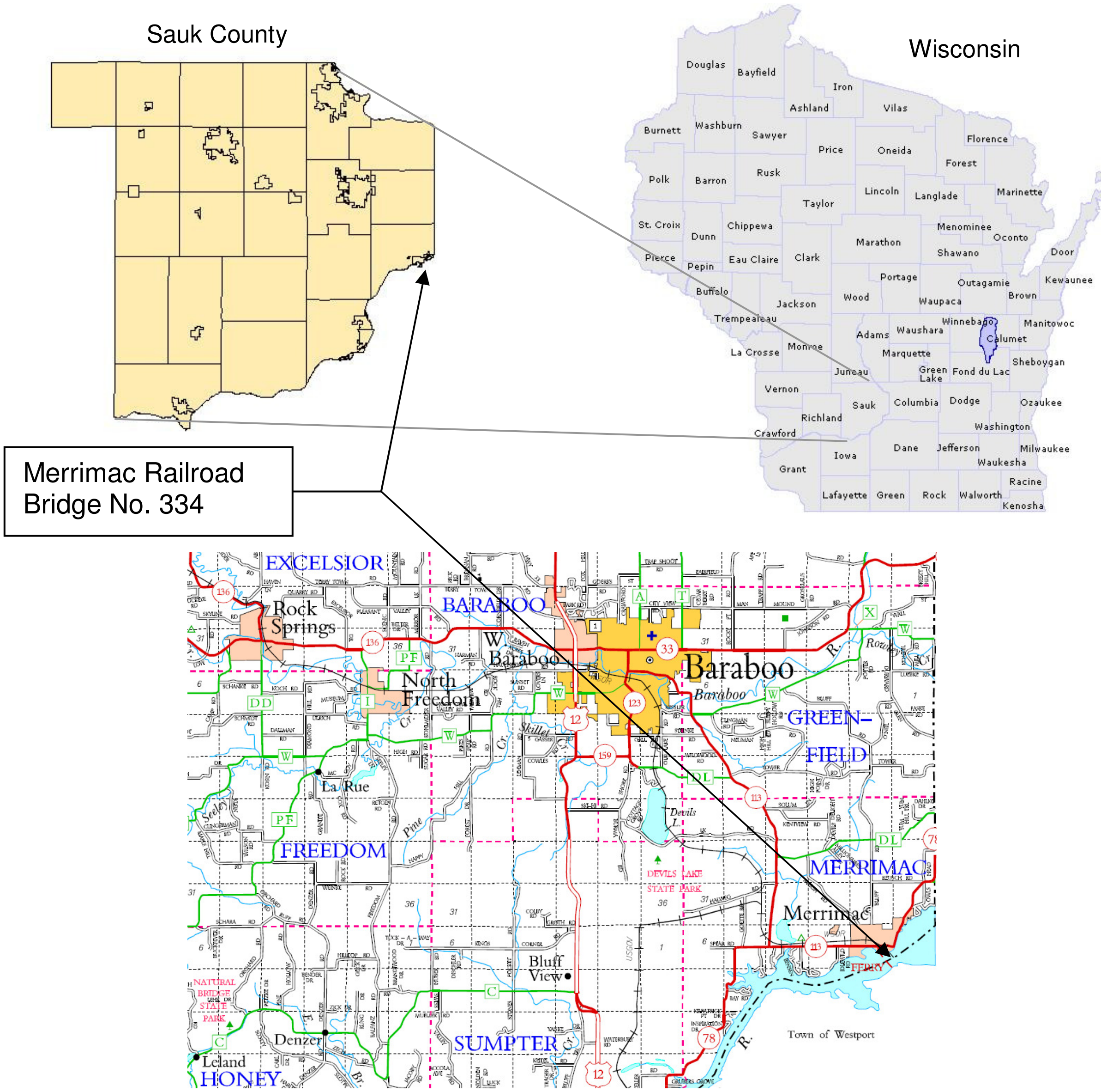


Figure 1-1 Location Map.

2. Inspection and Condition Summary

2.1 General Description

The components of the deck, superstructure, and substructure were visually inspected and the superstructure rated to provide the current conditions of the elements in accordance to AREMA, AASHTO, and WisDOT guidelines. The conditions of the elements inspected provide the information to evaluate the current structural capacity of the structure. Repair recommendations are also provided based on the results of the inspection to restore the structure to the original design capacity of Cooper E40 loading and to evaluate improvements that could increase the capacity to Cooper E80 loading. In addition, groundline measurements and the underwater inspection provide the conditions of the river bed and scour conditions around the piers and bents. This data is compared with the original profile from the plans to determine if any significant scour has occurred.

Repair recommendations will be further evaluated in Section 5.0, Improvement Alternatives, and Section 8.0, Engineering Cost Analysis.

As previously noted, Spans A and C to N are the deck girder spans and Spans B and O to U are the deck truss spans. Span S is the nonfunctioning swing span. The conditions of the elements will be discussed in more detail in the following sections, and the data for the full above water and below water inspections are provided in Appendix B.

2.2 Deck

The deck for both the deck girder spans and the deck truss spans is an open type deck system. The deck elements inspected include the steel rail track, tie plates, spikes, timber ties and timber or steel guards. Refer to Figure 2-1 for a typical view of the deck system.

The deck elements are not part the capacity analysis, but they were found to be generally in good condition with the timber components showing typical cracking and weathering. No significant deficiencies were noted with the rail or tie plates throughout the bridge. The deck system appears to be under a good routine maintenance program as there are no repair recommendations. Although the timber tie deck system is in good condition, the life span is typically about 30 years where the last 10 includes a rapid acceleration of deterioration. The existing deck system is about 20 years old, and replacement will be expected within the next 10 years. Costs for replacement will be included in the estimate.

2.3 Superstructure

Spans A and C through N are the deck girder spans and Spans B and O through U are the truss spans. Refer to Figure 2-2 for a typical view of a deck girder system and refer to Figure 2-3 for a typical view of a deck truss girder system for the bridge.

Figure 2-1 Typical View of Open Deck

The deck girder spans consist of two (2) main steel girders with floor beams, stringers, and lateral bracing where the main girders and floor beams are riveted built-up members and the stringers and lateral bracing are rolled beams and angles, respectively. The floor beam locations also include an angle strut attached to the lower flange of the main girders as part of the cross frame system.

The deck truss spans include 2 main trusses that consist of built-up riveted steel box top and bottom chords with plate and latticed webs, end verticals and compression diagonals that are built-up riveted steel box members with plate webs, and tension diagonals that are built-up riveted steel T sections connected with intermittent plate diaphragms. The end diaphragms or cross frames are combinations of built-up riveted steel beams and steel angle bracing, and the compression diagonal sway bracing consists of steel angles. The stringers are rolled beam members, and the top and bottom lateral bracing are steel angles.

2.3.1 Deck Girder Spans

The conditions of the deck girder span elements are generally in good condition throughout where the members typically have minor surface corrosion without section loss. The main girders were noted to have a small area of pitting up to 1/8" deep in the interior of the webs adjacent to the bottom flanges of the floor beams; however, no other significant deficiencies were located in the girders. The webs of the floor beams typically have small areas of section loss up to 1/8" deep adjacent to the stringer supports. The section loss is up to 1/4" deep for the floor beams in Span J. Floor Beam F-2 has a 2" x 3" area with up to 50% section loss with a 1/8" diameter hole. Also, the lower strut at Floor Beam 4 is bent 6 inches out of plane 4 feet from the south girder. No other significant deficiencies were noted in the stringers or lateral bracing, and the rivets were also in good condition with no loose or deteriorated rivets encountered. Refer to Appendix B for photos of typical deficiencies.

Figure 2-2 Typical View of Deck Girder System

2.3.2 Deck Truss Spans

Overall, the deck truss elements are in good condition where the members typically have minor surface corrosion without section loss. A couple of conditions were noted that were typical throughout the deck truss spans. The inside vertical gusset plates on top of the bottom chords attaching the end verticals and diagonals typically have pitting that is up to 25% of the thickness of the plate. Also, the lower transverse lateral bracing angles that span between lower panel points of the bottom chords have pack rust up to 1 inch between the angles.

Except for a few locations of flame cut holes not considered significant, only a few other specific locations have deficiencies to note. Span R between the diagonals and vertical gusset plates at the bottom chords has pack rust up to 3/8 inch wide. Span S north bottom chord has 2 linear feet of pack rust up to ¼ inch wide between the south top flange and south web plate in Bay 4 from the east end. And, Span T bottom batten plates at L7 have a hole up to 4 inches by 2 inches with surrounding pitting. Pack rust is up to 5/8 inch between diagonals and vertical gusset plates at bottom chord. No other significant deficiencies were noted in the stringers or later bracing, and the rivets were also in good condition with no loose or deteriorated rivets encountered. Refer to Appendix B for photos of typical deficiencies. The corrosion and pack rust is likely caused by the accumulation of dirt and debris that can hold moisture against the steel.

The concrete counterweight for Span B over Pier 2 is in good condition. The counterweight was installed for the cantilever.

Figure 2-3 Typical View of Deck Truss System

2.4 Bearings

Each span typically has a movable bearing end and a fixed bearing end except for Span S, where the bearing support is on a ring gear for the old swing span structure. The bearings for the deck girder spans are sliding plates with the movable ends having a slotted bolt hole. The bearings for the deck truss spans are roller and fixed assemblies attached to the trusses with 6" diameter pins.

All bearings are in good condition having minor surface corrosion present except that the fixed bearing assembly for Span U on the north side at Pier 22 has a cracked bottom plate at the northeast bolt possibly due to fatigue or debris under the plate. Also, the truss span bearings typically have a minor accumulation of soil and debris.

2.5 Substructure

The substructure types for Bridge 334 include 2 stone masonry abutments, Abutments 1 and 23; 5 stone masonry pier walls, Pier 2, Piers 18 to 19, and Piers 21 to 22; 4 concrete wall piers, Pier 2 and Piers 15 to 17; 11 concrete encased steel tower piers, Piers 4 through 14; and 1 stone masonry pivot pier for the old swing span, Pier 20. Piers 2 and 15 also include steel frame bent sections for the supports of the deck girder spans adjacent to the deck truss spans that account for the difference in deck depth. The steel bent structures consist of riveted built-up members. In addition, the concrete encased steel towers include towers with riveted built-up members as well. Refer to Photos 2-4, 2-5, and 2-6 for typical views. The masonry piers and abutments are part of the original 1985 construction, and the concrete encased steel tower piers were replaced in 1909 and encased in 1915. The reinforced concrete piers were installed in 1930 when the spans were modified.

Figure 2-4 View of Abutment 1

The masonry piers and abutments are in fair to good condition with minor deficiencies. There is mortar loss up to 1 foot deep from the water surface to 6 feet below. In various locations, the upstream and downstream noses have missing stone blocks at the water surface and other blocks with section loss up to 8 inches deep at the water surface at the downstream and upstream noses. Pier 2 has a missing stone block under the south bearing at the waterline on the west face. Pier 18 has 1 row of blocks missing just below the water surface at the upstream and downstream noses. Pier 19 has 1 row of blocks missing at the water surface at the upstream nose. Pier 20 has 1 block missing at the downstream nose.

Figure 2-5 Pier 22, Typical View of Masonry Piers

Figure 2-6 Typical View of Concrete Encased Tower Pier

Pier 20 also has a timber crib filled with stone at the upstream (north) nose. The crib is approximately 9 feet tall from the stream bed upward and extends 15 feet upstream of the nose of the pier. There are timber forms 1 foot from the pier and up to 1 foot above the stream bottom at the downstream nose. The crib was installed in 1903 to improve the protection of the pivot pier with the upstream side completed, but the downstream side was determined not to be needed and the work left in place.

The reinforced concrete piers are in fair to good condition with minor deficiencies. There is abrasion or scaling damage typically up to 1.5 feet deep at and just below the water surface due to ice flows. Pier 14 has abrasion damage up to 4.0 feet at the downstream nose. Pier 15 south bearing pedestal is delaminated up to 6 inches deep on the south face. All 4 faces of north bearing pedestal at Pier 16 are delaminated up to 1.0 foot. For the steel elements, the steel frames at Piers 2 and 15 and the exposed portions of the steel towers from Piers 4 to 14 have minor surface corrosion throughout but no other significant damage or deterioration.

2.6 Scour Conditions

The river or lake bed is made up of rock for the foundations of the piers and abutments. No undermining was noted during the underwater inspection except that Pier 3 had past undermining due to the presence of sheet piling along the south side. Also, some footings were exposed at some of the piers. Pier 15 has the top tier of the footing exposed up to 1.0 foot wide and 3.0 feet tall, and the bottom tier is exposed up to 1.0 foot wide and 2.0 feet tall. Pier 16 has the top tier of the footing exposed up to 1.0 feet wide and 3.0 feet tall, and the bottom tier is exposed up to 1.0 foot wide and 2.0 feet tall. Pier 17 footing is exposed at the downstream nose up to 1.5 feet wide and 1.5 feet tall, and Piers 18 and 19 have the footing exposed up to 1 foot high.

The river bed profile taken during this inspection from the channel survey are compared with the original profile as shown in the 1902 drawings and to the 1977 plotted elevations as shown on following channel profile table and in Appendix A. Refer to the following table for the comparison. An elevation view of the bridge is included with the channel profile as described above. In review of the data from the table to evaluate long-term results of possible scour from 1902 to 2009, only Abutment 1 has a significant degradation of over 7 feet from the 1902 survey with more than a 3 foot change from a previous measurement. The 3 foot value of a change in measurement at a channel location from one period to the next is a rule of thumb dimension to monitor scour. A number of midspan locations show areas with degradation but not to the degree of Abutment 1 and, in addition, these areas do not affect the substructures. Other areas show aggredation of over 3 feet but are not as much of a concern. The changes from 1977 to 2009 show the channel bed to be very stable as only one location shows up to 4.12 feet of degradation but is located at midspan between Piers 3 and 4.

Significant scour does not appear to occurring at the bridge site, and only Abutment 1 appears to show a possible problem. The underwater inspection did not observe any undermining or scour at Abutment 1.

MERRIMAC RAILROAD BRIDGE No. 334 - CHANEL PROFILE -

		Profile Elevation			Profile Change (ft)		
Pier	**Sta.**	**1900**	**1977**	**2009**	**1900 - 1977**	**1977 - 2009**	**1900 - 2009**
23	2020.77	762.70	772.40	773.06	-9.70	-0.66	-10.35
*	1965.73	752.07	759.21	757.56	-7.14	1.65	-5.49
22	1910.03	752.62	753.72	753.63	-1.10	0.09	-1.01
22	1899.02	752.83	754.30	754.06	-1.47	0.24	-1.24
	1843.21	753.23	747.81	745.49	5.42	2.32	7.73
21	1787.00	752.99	752.32	751.93	0.67	0.39	1.06
21	1776.01	752.62	752.24	751.72	0.38	0.52	0.90
	1731.95	752.23	750.87	748.51	1.36	2.36	3.72
20	1694.13	753.10	751.86	750.34	1.24	1.52	2.76
20	1668.16	753.32	753.00	752.46	0.32	0.54	0.86
	1629.79	750.57	749.40	749.48	1.17	-0.08	1.09
19	1585.10	751.12	753.38	754.74	-2.26	-1.36	-3.62
19	1570.11	752.64	756.20	756.14	-3.56	0.06	-3.50
	1501.21	743.41	751.53	751.17	-8.12	0.36	-7.76
18	1432.21	751.65	755.98	755.15	-4.33	0.82	-3.51
18	1417.44	752.91	756.86	756.11	-3.96	0.76	-3.20
	1365.79	752.04	752.73	752.13	-0.69	0.60	-0.09
17	1312.31	753.64	758.05	761.93	-4.41	-3.88	-8.29
17	1301.18	752.61	757.50	762.05	-4.89	-4.55	-9.45
	1267.92	752.68	755.79	753.29	-3.11	2.50	-0.61
16	1234.82	752.56	755.83	755.04	-3.27	0.79	-2.47
16	1223.51	751.78	756.15	756.44	-4.37	-0.29	-4.66
	1175.68	752.14	754.16	752.09	-2.01	2.07	0.06
15	1127.71	753.08	756.67	751.00	-3.60	5.67	2.07
15	1116.43	751.50	757.04	753.10	-5.55	3.95	-1.60
	1082.70	753.85	755.24	752.88	-1.39	2.36	0.97
14	1052.10	757.30	756.98	755.19	0.32	1.79	2.11
14	1034.58	758.32	756.98	758.80	1.34	-1.82	-0.48
	1018.74	759.18	756.29	755.63	2.89	0.66	3.55
13	1003.03	757.69	757.03	755.09	0.66	1.94	2.60
13	985.45	757.68	757.35	756.75	0.33	0.60	0.93
	969.71	757.23	756.63	754.94	0.60	1.69	2.29
12	954.17	756.71	757.58	755.31	-0.86	2.27	1.40
12	936.22	756.75	756.99	756.90	-0.24	0.09	-0.15
	920.66	758.14	756.55	757.01	1.59	-0.46	1.13
11	904.89	757.77	757.92	758.29	-0.14	-0.38	-0.52
11	887.45	758.25	758.42	759.20	-0.17	-0.78	-0.95
	871.59	759.05	758.06	759.02	0.99	-0.95	0.04
10	855.81	757.89	758.68	759.85	-0.79	-1.16	-1.96
10	838.36	758.12	759.64	759.39	-1.52	0.25	-1.26
	822.49	758.03	758.47	758.54	-0.44	-0.07	-0.51
9	806.63	758.07	758.57	759.10	-0.49	-0.54	-1.03
9	789.21	758.10	758.97	759.89	-0.87	-0.92	-1.79
	773.34	758.03	758.35	759.45	-0.31	-1.11	-1.42
8	757.63	757.31	758.39	759.20	-1.08	-0.81	-1.89
8	739.97	757.62	758.17	759.20	-0.55	-1.03	-1.58
	724.38	757.70	757.43	756.67	0.27	0.76	1.03
7	708.94	756.79	758.33	758.67	-1.54	-0.34	-1.88
7	690.90	756.22	757.88	760.50	-1.66	-2.62	-4.28
	675.86	757.13	756.68	756.58	0.46	0.10	0.56
6	660.71	756.79	757.72	757.98	-0.93	-0.26	-1.19
6	643.18	758.79	757.86	758.33	0.93	-0.46	0.46
	627.82	759.66	757.05	757.45	2.61	-0.40	2.21
5	612.53	758.39	757.04	757.80	1.35	-0.76	0.59
5	595.07	757.52	756.92	757.45	0.59	-0.53	0.07
	579.96	757.86	756.23	755.88	1.63	0.35	1.98
4	564.85	757.58	757.36	757.10	0.22	0.26	0.48
4	547.39	758.37	758.82	757.10	-0.45	1.72	1.27
	511.18	758.74	760.95	756.83	-2.21	4.12	1.90
3	472.07	746.10	751.51	750.55	-5.41	0.96	-4.45
3	460.29	743.10	751.64	748.53	-8.54	3.11	-5.43
	414.76	744.04	749.47	749.05	-5.43	0.41	-5.01
2	368.57	748.27	754.06	754.12	-5.79	-0.05	-5.84
2	357.80	750.40	755.51	753.85	-5.11	1.65	-3.45
	330.99	757.89	762.66	761.73	-4.77	0.94	-3.84
1	304.70	781.69	774.82	773.81	6.87	1.01	7.88

* Value at midspan (Typ)

3. Structural Analysis and Load Rating

3.1 Analysis Procedure

The structural analysis of the Merrimac Railroad Bridge No. 334 was performed using a 3D computer model; the software used was GTSTRUDL™. From this model we obtained the member forces for all the different Cooper loads, including the additional loads, and using the load combinations specified in the 2009 AREMA Manual.

Model

The structure geometry for each span was extracted from the existing plans, verified using the pictures and measurements taken during the bridge inspection, and then input into the model.

To input the member properties in the model we followed the following procedure: First we obtained the member geometry from the structural plans. Then with the help of the computer program we calculated the member properties required by the GTSTRUDL™ model and required in the load rating calculations. Then these values were manually input in the model input. Members that share the same section geometry were grouped.

The member releases and support releases were also obtained from the existing plans and verified in the field.

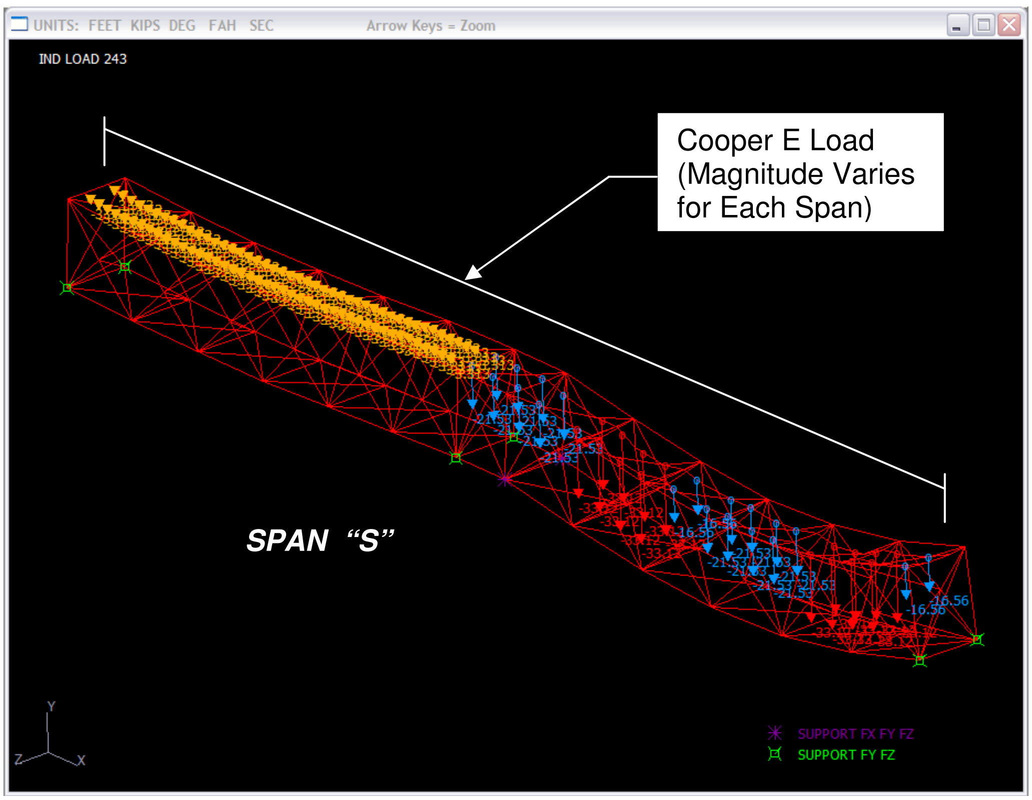


Figure 3-1 GTSTRUDL™ Model – Load Rating

Materials

The yield stress (Fy) or tensile strength (Fu) of the steel used in this structure is not specified in the existing plans; the only specification given is that "*Medium*" steel is required for the stringers and "*Soft*" steel for the rest of the members.

According to the AISC Steel Design Guide No. 15 "*AISC Rehabilitation and Retrofit Guide – A Reference for Historic Shapes and Specifications*" Table 1.1a, the steel used in structures built around 1900 was likely to be ASTM A7 with an Fy = 32 ksi for "Soft" steel and Fy=35ksi for "Medium" steel and Fu = 52 ksi for "Soft" steel and Fu=60ksi for "Medium" steel. These were the material properties used in our analysis.

Loads

The loads applied to the structural model are the ones specified in the 2009 AREMA Manual Section 7.3.3 and 1.3.

Live Load

The live load configuration adopted in the AREMA Manual is the Cooper E series loading. Figure 3-2 illustrates a unit Cooper E loading. Other Cooper loads are obtained by multiplying the axle factor and the cars distributed load by the Cooper load number; the axle spacing remains constant.

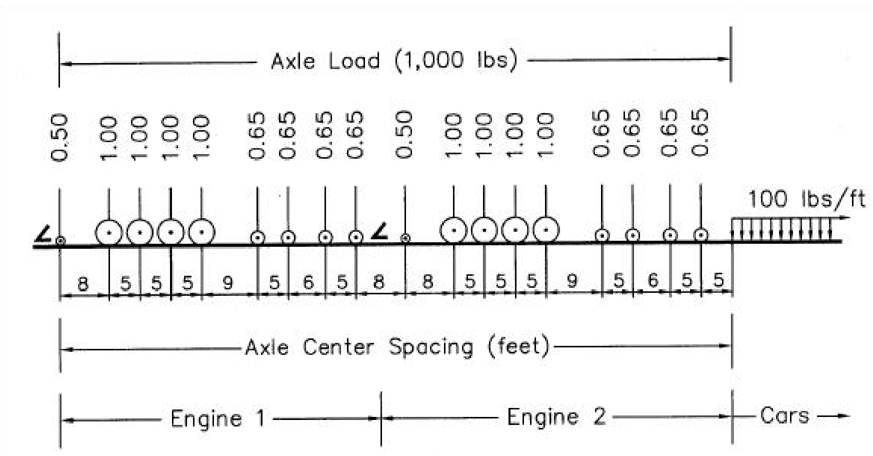


Figure 3-2 Unit Cooper Load.

In the GTSTRUDL™ model, the live load (Cooper E load) was input as a moving load that crosses the entire span; the program generates member forces for each live load position and then extracts the controlling forces for each member.

Impact Load

The impact load applied in the model is calculated using the equations given in AREMA manual Section 7.3.3.3 and Section 1.3.5. This factor is a function of the span length. The impact factors calculated for this bridge range from 0.21 to 0.36.

Wind Load

The wind load is calculated using the AREMA manual Section 7.3.3.5. This load includes wind load on the train and wind load in the structure. These loads are based on the assumption that when the wind velocity exceeds 70 mph, a train will operate at a reduced speed, if it operates at all.

Longitudinal Force

The longitudinal force in the structure is calculated as specified in the AREMA manual Section 7.3.3.8 and Section 1.3.12. This force is taken as the larger of the force due to breaking and the force due to traction; both values are a function of the span length. In most of the cases traction is what controls the longitudinal force. The values obtained for this bridge range from 1 to 2 kips per feet of rail.

See Appendix D for section property calculations, load calculations, structural model input, and analysis results for all spans.

3.2 Load Rating Procedure

The objective of the load rating in railroad bridges is to find the minimum Cooper E load that produces member stresses equal to corresponding allowable stress in any member of the span.

Load rating was performed in accordance to Section 7.3.1 of the AREMA manual. The load rating is divided in two types, Normal or Continuous rating and Maximum rating.

Normal Rating (Continuous)

Normal rating is the load level which can be carried by the structure continuously for its expected service life. The allowable stresses used for the Normal Rating are the values given in AREMA manual Section 1.4.1, these values are the same values specified for a new bridge design.

Maximum Rating

Maximum rating is the load level which the structure can support at infrequent intervals, with any applicable speed restrictions. The allowable stresses used for the maximum rating are the values given in the AREMA manual section 7.3.4.3, these values are usually higher than the values used for the normal rating, therefore the rating is usually higher. In case the Normal Rating is greater than the maximum rating, the lesser rating governs.

See Section 3.5 for Normal and Maximum Load Rating Results.

Rating Procedure

The Load Rating calculations are performed in the load rating spreadsheets (See Appendix D). The first step is to insert all the input data required in the calculations, this data includes the following:

- **Member Information:** Label, Group, Type (Frame or Truss), Length.
- **Section properties:** Area (Gross and Net), Inertia, Radius of gyration, centroid, and percentage of area loss if available.
- **Material:** Ultimate stress, yield stress, and modulus of elasticity.
- **Member Forces:** The member forces are extracted from the model output in two different ways. The first set of values consists of the member forces corresponding to the maximum axial force in the member and the second set consists of the member forces corresponding to the maxim bending moment in the member.

For truss members the load rating factor (Allowable / Actual) is calculated for the following cases: Tension in the gross area, tension in the net area and compression. For frame members the load rating is calculated for the following cases: Tension and bending, compression plus bending and shear. Shear rating is calculated only in members with substantial shear forces like stringers and floor beams.

Using the member forces obtained from the model and the section properties calculated previously, the maximum member stresses, axial (fa), bending (fb) and shear (fv) are calculated and placed in the respective column in the load rating table.

The allowable stresses are calculated using the section properties and material properties and following the equations given by the AREMA manual Section 1.4.1 for normal rating and Section 7.3.4.3 for maximum rating, for combinations of axial and bending forces, the combined stress ratios were calculated using Section 1.3.14 for normal rating and Section 7.3.4.3 paragraph C. for maximum load rating.

The entire load rating is calculated twice. In the first calculation, the longitudinal loads are not included in the maximum member forces. In the second set of calculations, the longitudinal forces are included, but the member allowable stresses are increased 25 percent, then the minimum rating between these two cases is selected as the controlling rating. This is done to comply with the requirements for lateral load application given in the AREMA manual Section 7.3.3.8 paragraph E.

After calculating the minimum rating factor for all members, a summary was created for each member group, in this summary we show the controlling member for each group with its rating factor and other relevant information.

To find the Cooper E rating for each span an iterative process was required. The first step is to run the model with an initial Cooper load, in our case we ran the first model with a Cooper E80 load, then the minimum rating factor is calculated and if the value is less than 1, the Cooper E load applied is reduced until the minimum rating factor is equal or very close to 1. This process has to be done for both Normal and Maximum rating.

Load Rating as designed and constructed

To obtain the Cooper E load for the structure as designed and constructed it is required to do the load rating calculations using the sections properties as shown in the original structural drawings but including the latest structural modifications. This is basically the load rating of the structure without including any deterioration.

Load Rating based on existing structure condition

In the current bridge inspection it was found that most of the deterioration in this bridge is generalized and only in few cases it is specific to certain members, taking this in to account and after exploring different alternatives, we decided that the best way to reflect the current structure deterioration in the load rating was to estimate a “Structure Condition Factor” this factor is estimated based on the deterioration found in the latest bridge inspection. This procedure is similar to the one used by AAHSTO (American Association of State Highway and Transportation Officials) in the load rating of highway bridges. See Section 3.3 for the condition factors given to all spans.

3.3 Structure Description

Span A

Span A is a plate girder structure 64.4 ft. long; this is the first span in the East side and is located between Piers 1 and 2. This span was originally built in 1885 and modified in 1903. During this modification the structure was replaced with a structure with the same span length and same structure type. Construction plans for the last modification are available, according to the plans, this span was designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Above average member condition”, and a condition factor of 1 was assigned.

Span B

Span B is a truss structure 20 ft. high and 12 ft. wide, it is located between Piers 2 & 3. This span was originally built in 1885 and modified in 1930. Construction plans for the last modification are available, during this modification, the clear span between supports was changed from 144.3 ft to 103.83 ft, the west support was moved two bays and one bay was removed, vertical members were added over the supports and some members in the end bay where added or strengthened. (See Figures 3-3 and 3-4).

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Below average member condition”, and a condition factor of 0.9 was assigned.

Span C

Span C is a plate girder structure originally 48.28 ft. long and later modified to 89.00 ft. This span was originally built in 1885 and modified in 1930. During the last modification, Pier 3 was moved to the east and a new superstructure was erected (See Figures 3-2 and 3-3).

Construction plans for the original structure are not available but for the last modification they are, according to the plans, this span was designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Below average member condition”, and a condition factor of 0.9 was assigned.

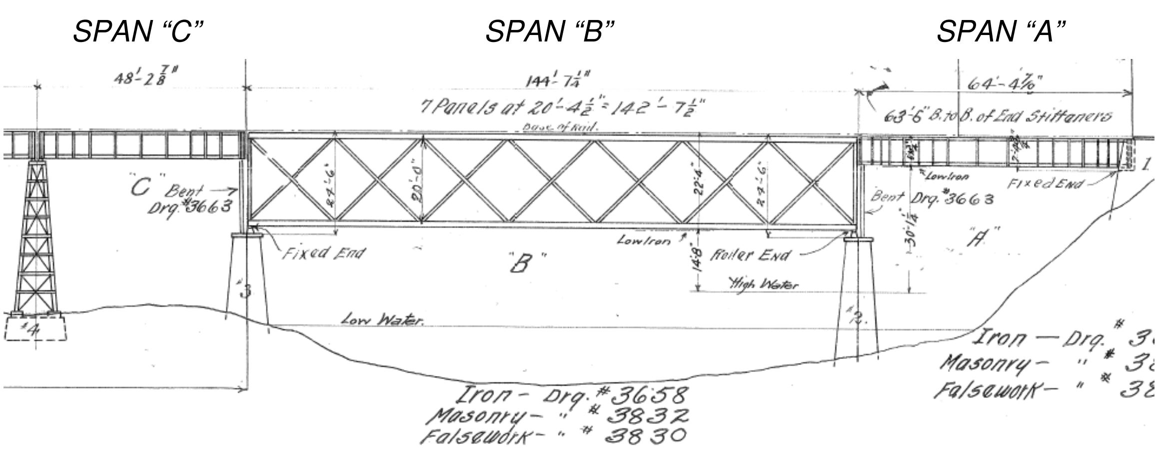


Figure 3-3 Original Configuration for Spans A, B and C.

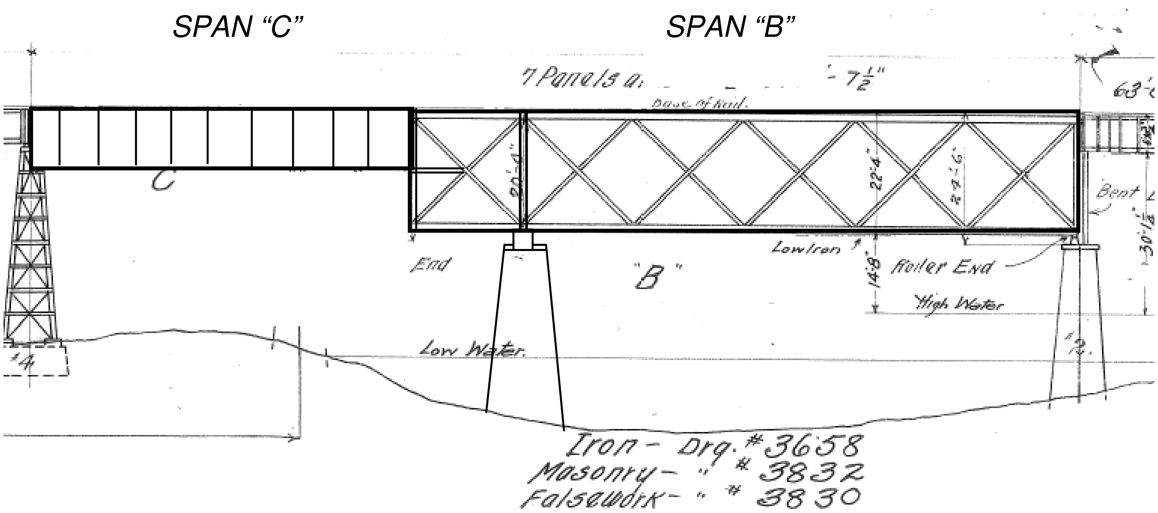


Figure 3-4 Current Configuration for Spans B and C.

Spans D to M

Spans D to M is the only section of the bridge that never has been modified, these spans were built in 1895, the span lengths range from 47.84 ft. to 49.15 ft. Construction plans are not available for this section of the bridge, therefore it was required to recreate the drawings using field measurements.

Based on the rating results obtained and since the rest of the bridge was designed for an E40 load we can assume that these spans were also designed for an E40 Cooper load.

After analyzing the inspection results for the entire structure and comparing these spans with the rest of the structure, we classified these spans as "Average member condition," and a condition factor of 0.95 was assigned.

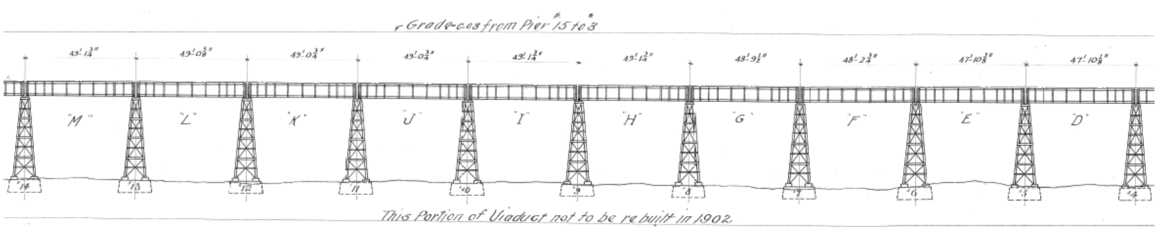


Figure 3-5 Spans D to M Original and Current Configuration.

Span N

Span N is a plate girder structure originally 49.8 ft. long and later modified to 78.92 ft. This span was originally built in 1885 and modified in 1930. During the last modification, Pier 15 was moved to the west and a new superstructure was erected (See Figures 3-5 and 3-6).

Construction plans for the original structure are not available but for the last modification they are, these plans have all the structural information but they don't specify the structure design load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as "Average member condition," and a condition factor of 0.95 was assigned.

Span O

Span O is a truss structure 15.5 ft. high and 12 ft. wide, it is located between Piers 15 and 16. This span was originally built in 1885 and modified in 1930. Construction plans for the latest modification are available. During this modification the east and west piers were moved about 30 ft. to the west keeping the same span length and re-using the same superstructure. (See Figures 3-6 and 3-7).

According to the plans, the original structure and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as "Below average member condition", and a condition factor of 0.9 was assigned.

Span P

Span P is a truss structure 15.5 ft. high and 12 ft. wide, it is located between Piers 16 and 17. This span was originally built in 1885 and modified in 1930. Construction plans for the latest modification are available. During this modification the east pier was moved about 30 ft. to the west reducing the span length from 107.19 ft. to 77.42 ft. The same superstructure was used but two bays from one of the ends were removed. (See Figures 3-6 and 3-7).

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as "Average member condition," and a condition factor of 0.95 was assigned.

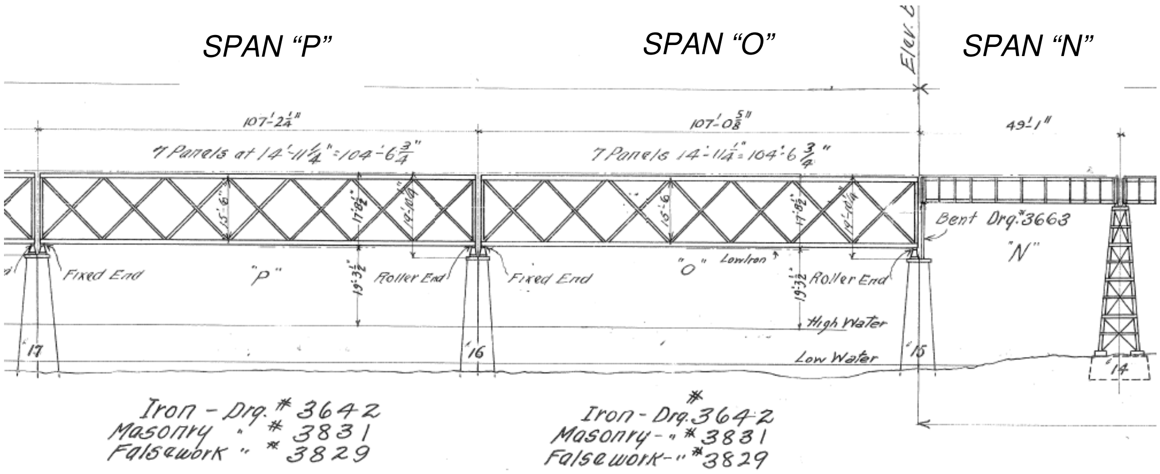


Figure 3-6 Original Configuration for Spans N, O and P.

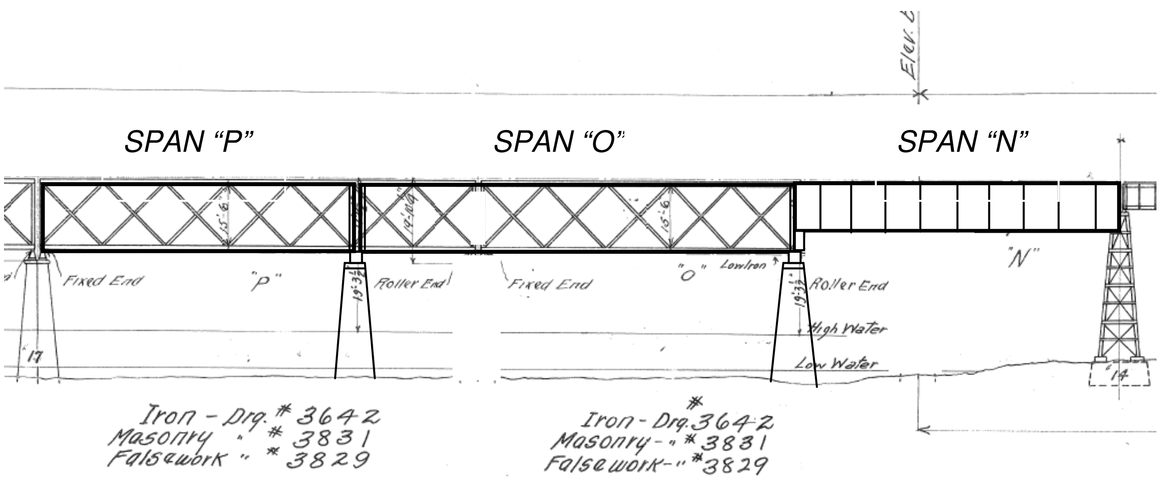


Figure 3-7 Current Configuration for Spans N, O and P.

Span Q

Span Q is a truss structure 15.5 ft. high, 12 ft. wide and 118.32 ft. long, it is located between Piers 17 and 18. This span was originally built in 1885 and modified in 1903. Construction plans for the latest modification are available. During this modification the previous superstructure was replaced by this truss keeping the same span length. (See Figure 3.-8).

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Average member condition,, and a condition factor of 0.95 was assigned.

Span R

Span R is a truss structure 22 ft. high, 12 ft. wide and 152.52 ft. long, this is the longest span in the bridge, it is located between Piers 18 and 19. This span was originally built in 1885 and modified in 1903. Construction plans for the latest modification are available. During this modification the previous superstructure was replaced by this truss keeping the same span length. (See Figure 3-8).

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Below Average member condition,” and a condition factor of 0.9 was assigned.

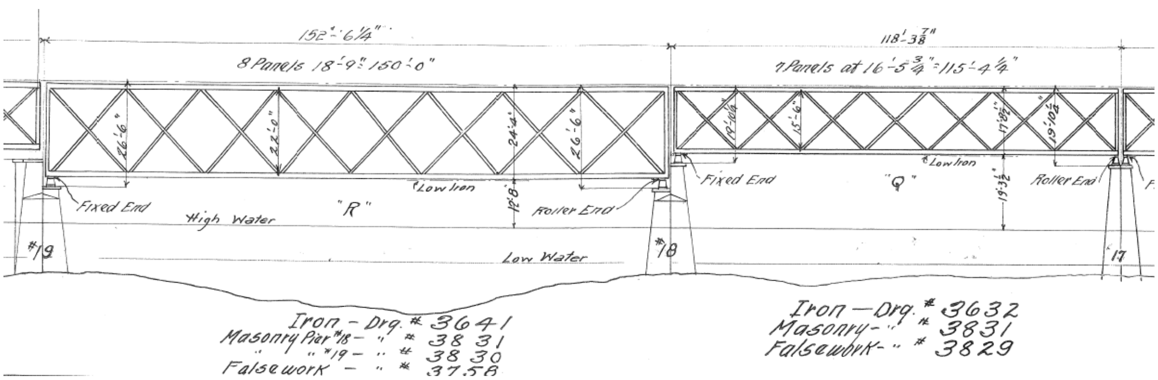


Figure 3-8 Spans Q and R Configuration.

Span S

Span S is a Swing Truss structure 15.5 ft. high, 12 ft. wide, the east span is 104.64 ft., the west span is 99.69 ft. and the total length is 204.32 ft. This structure is located between Piers 19 and 21 on the pivot pier, Pier 20. This span was originally built in 1885 and modified in 1903. Construction plans for the latest modification are available. During this modification the previous superstructure was replaced by this truss keeping the same span length. (See Figure 3-9).

The swing mechanism is currently disabled but we don’t know exactly for how long it’s been like that, the load rating calculations were done assuming that the swing span is always closed.

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load.

After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, this span was classified as “Average member condition,” and a condition factor of 0.95 was assigned.

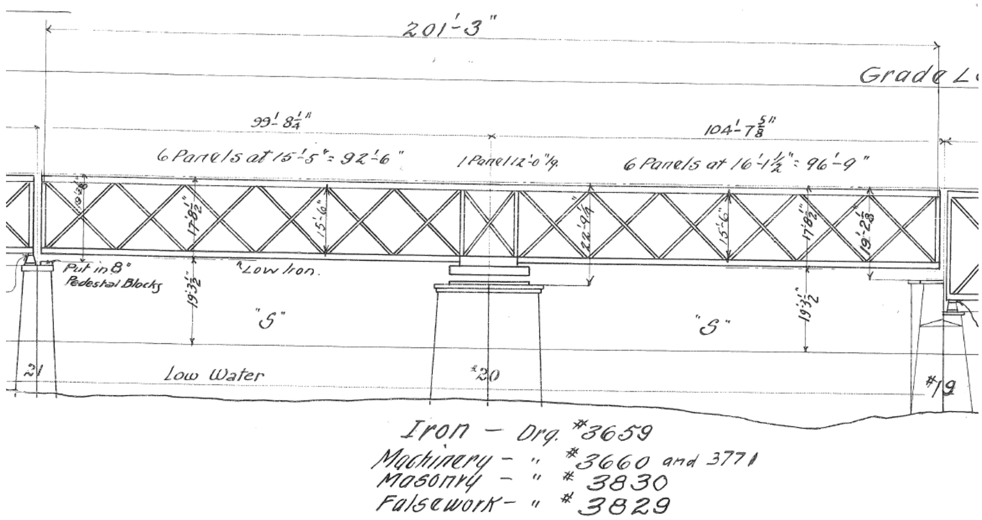


Figure 3-9 Span S Configuration.

Spans T and U

Spans T and U are truss structures 15.5 ft. high, 12 ft. wide. Span T is 122.84 ft. long and Span U is 122.19 ft. long, these spans are located between Piers 21 and 23. These spans were originally built in 1885 and modified in 1903. Construction plans for the latest modification are available. During this modification the previous superstructure was replaced by this truss keeping the same span lengths. (See Figure 3-10).

According to the plans, the original design and the latest modification were designed for a Cooper E40 Load. After analyzing the inspection results for the entire structure and comparing this span with the rest of the structure, span T was classified as "Average member condition", and a condition factor of 0.95 was assigned and span U was classified as "Below average member condition," and a condition factor of 0.9 was assigned.

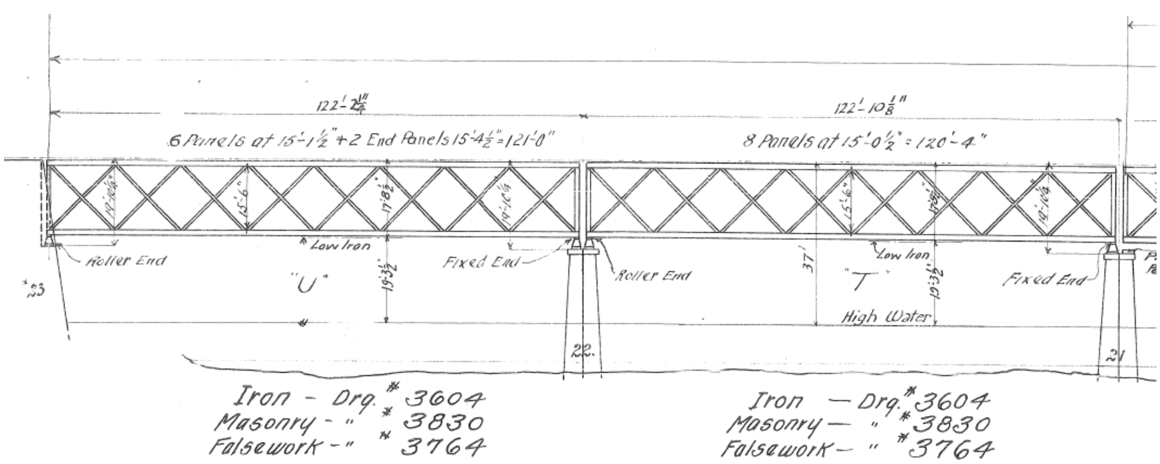


Figure 3-10 Spans T and U Configuration.

3.4 Structure Description Summary

Table 3-1 shows a summary of the basic structure information for each span of the Merrimac Railroad Bridge, we also included the current structure condition. The structure condition is based on the findings of the Bridge Inspection performed by Ayres Associates on May 2009. See Appendix B for more details.

Table 3-1 Structure Description Summary.

SPAN	ORIGINAL LENGTH (ft)	MODIFIED LENGTH (ft)	STRUCTURE TYPE	YEAR BUILT	LAST MODIFICATION	PLANS AVAILABLE	STRUCTURE CONDITION	CONDITION FACTOR
A	64.41	-	Plate Girder	1895	1903 (R)	Y	Above average	1.00
B	144.60	103.83	Truss	1895	1930 (M)	Y	Below average	0.90
C	48.24	89.01	Plate Girder / Truss	1895	1930 (R)	Y	Below average	0.90
D	47.84	-	Plate Girder	1895	-	N	Average	0.95
E	47.86	-	Plate Girder	1895	-	N	Average	0.95
F	48.23	-	Plate Girder	1895	-	N	Average	0.95
G	48.79	-	Plate Girder	1895	-	N	Average	0.95
H	49.15	-	Plate Girder	1895	-	N	Average	0.95
I	49.15	-	Plate Girder	1895	-	N	Average	0.95
J	49.06	-	Plate Girder	1895	-	N	Average	0.95
K	49.06	-	Plate Girder	1895	-	N	Average	0.95
L	49.05	-	Plate Girder	1895	-	N	Average	0.95
M	49.15	-	Plate Girder	1895	-	N	Average	0.95
N	49.08	78.92	Plate Girder	1895	1930 (R)	Y	Average	0.95
O	107.05	-	Truss	1895	1930 (M)	Y	Below average	0.90
P	107.19	77.42	Truss	1895	1930 (M)	Y	Average	0.95
Q	118.32	-	Truss	1895	1903 (R)	Y	Average	0.95
R	152.52	-	Truss	1895	1903 (R)	Y	Below average	0.90
S1	104.64	-	Truss	1895	1903 (R)	Y	Average	0.95
S2	99.69	-	Truss	1895	1903 (R)	Y	Average	0.95
T	122.84	-	Truss	1895	1903 (R)	Y	Average	0.95
U	122.19	-	Truss	1895	1903 (R)	Y	Below average	0.90

NOTES: (R): The superstructure was replaced by a new structure in this year. (See Section 3.3 for more details)
(M): The structure was only modified in this year. (See Section 3.3 for more details)

3.5 Load Rating Results

The load rating results for the "As designed and constructed" and "Current Condition" are given in Table 3-2 and Figure 3-11.

Table 3-2 Bridge Rating Summary.

Span	AS DESIGNED AND CONSTRUCTED		CURRENT CONDITION	
	Normal Load Rating	Maximum Load Rating	Normal Load Rating	Maximum Load Rating
A	E45	E79	E45	E79
B & C	E47	E78	E42	E70
D to M	E46	E68	E44	E65
N	E92	E159	E87	E151
O	E40	E65	E36	E59
P	E56	E88	E53	E84
Q	E40	E66	E38	E63
R	E41	E69	E37	E62
S	E53	E82	E50	E78
T	E40	E53	E38	E50
U	E40	E53	E36	E48
Minimum	**E40**	**E53**	**E36**	**E48**

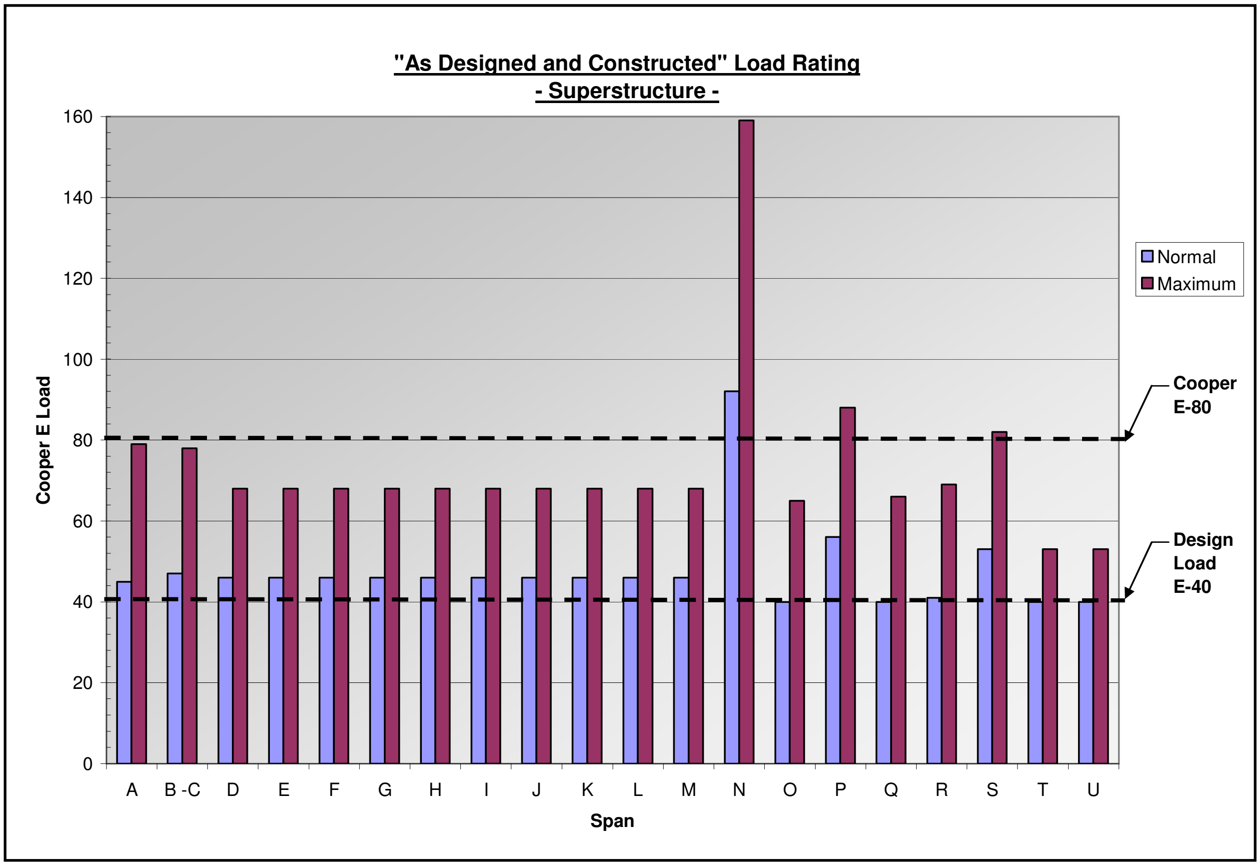


Figure 3-11 Load Rating Results – "As Designed and Constructed".

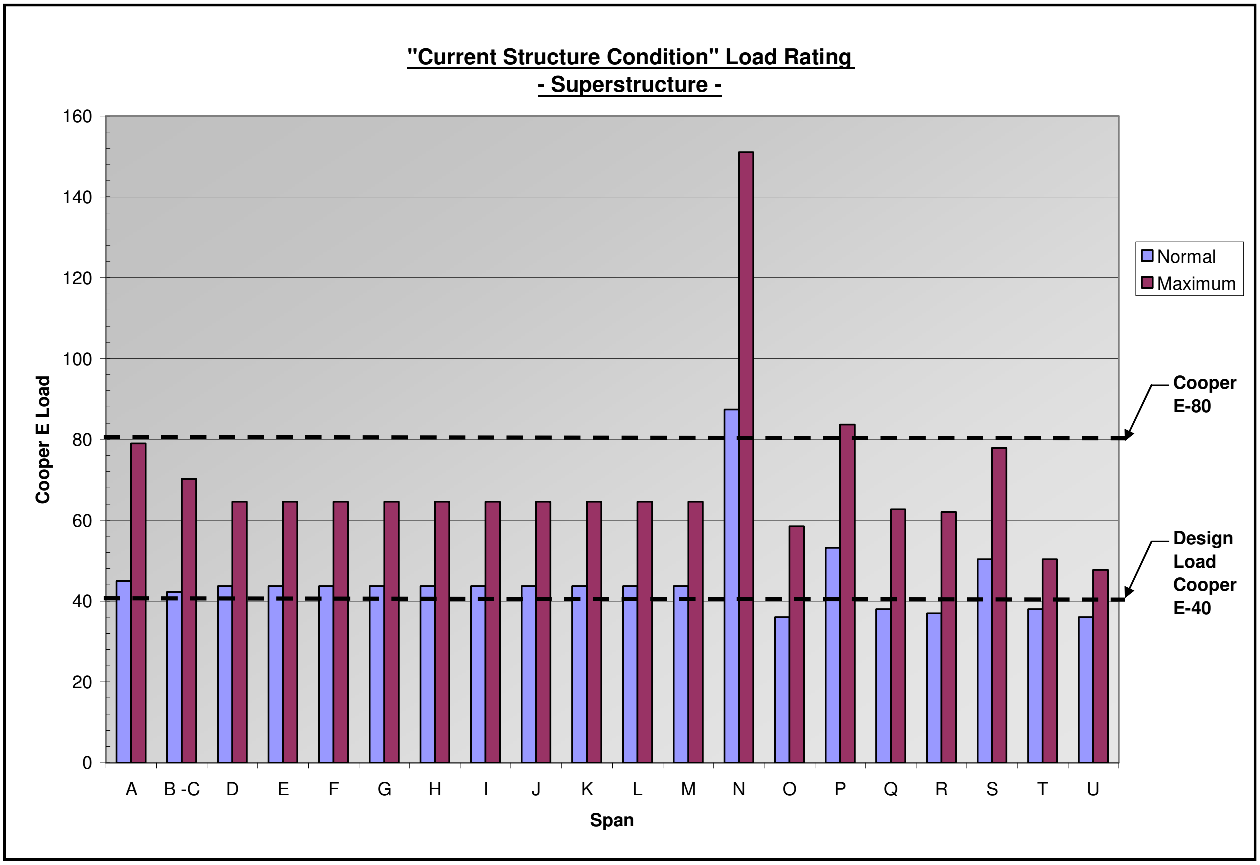


Figure 3-12 Load Rating Results – "Current Structure Condition"

Looking at the load rating results for the "As Designed and Constructed" conditions we can see that the normal load rating for most of the spans is between Cooper E40 to Cooper E47. This is consequent with the bridge design load (Cooper E40) shown in the structural drawings.

Only three spans give Normal ratings higher than that range; Span N is the span with the highest Normal Rating in the entire bridge, according to our calculations it has a Normal rating of E92, the reason for this could be that since this span was replaced almost 30 years after the rest of the structure (1930) it probably was designed for a higher Cooper load than the rest of the bridge.

Span P also gives a relative high Normal Rating value (Cooper E56), this is because in 1930 the structure length was reduced from 107.19 ft. to 77.42 ft. therefore reducing the member stresses and increasing the Load Rating. Another span with a relative high Normal Rating is Span S (Cooper E53) this is because this is a swing structure that works as a continuous span over the center pier, and also this span has heavier members than other spans with similar span length.

The Maximum load rating results are not that relevant because according to its definition (See Section 3.2), this load should be applied to the structure only at infrequent intervals and with applicable speed restrictions.

4. Cooper E80 Structural Evaluation

One of the objectives of this comprehensive study is to determine if this bridge is capable of carrying a continuous Cooper E80 Load and if is not capable; determine the structural upgrades required to carry such load.

The Cooper E80 load (See Figure 4-1), is the recommended design load for new railroad structures in the current AREMA Manual (2009).

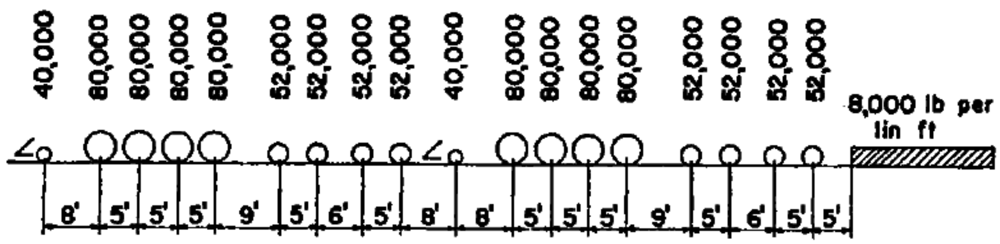


Figure 4-1 Cooper E80 Load (lb)

From the previous load rating results (Section 3.5) we can see that only Span N is capable of carrying a continuous Cooper E80 load, therefore we focus this structural evaluation in to identifying the members in each span that have to be upgraded to carry such load.

4.1 Analysis Procedure

The analysis procedure used for this structural evaluation was similar to the one used for the Normal Load Rating (Section 3.2), the only difference is that instead of changing the live load until the ratio of allowable stress over actual stress is equal to one, the Cooper E80 is the only live load applied and the ratio of allowable stress over actual stress is calculated for every member. When the stress ratio is less than 1 it means that the member is overstressed and has to be upgraded.

The allowable stresses used to calculate the stress ratio are the values given in AREMA section 1.4.1, these values are the same values used for a new design.

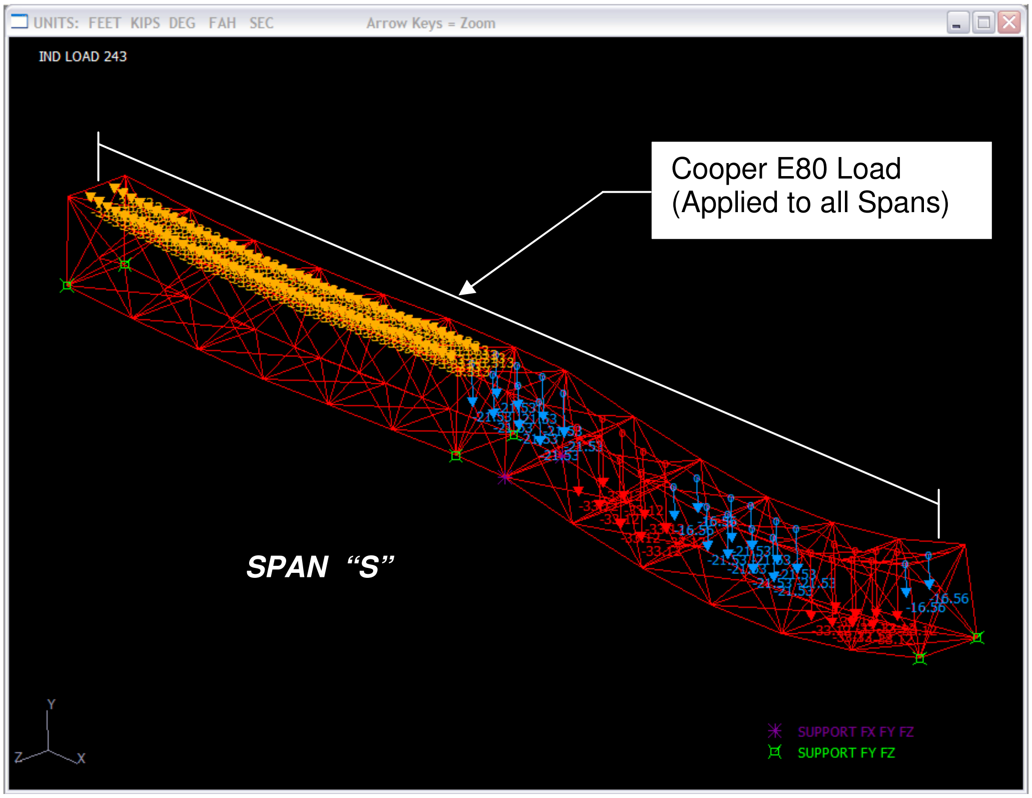


Figure 4-2 GTSTRUDL™ Model – Cooper E80 Evaluation -

4.2 Structural Evaluation Results

This structural evaluation shows that only Span N is capable of carrying a continuous Cooper E80 load without any modification to its superstructure. The rest of the spans have to be upgraded to carry such load.

Based on the overstress ratios previously calculated, we divided all the member of the structure in to 4 groups: Members with no overstress, members with overstress ratio between 1% to 20%, members with overstress ratio between 21% to 40% and members with overstress ratio of 40% or higher.

Then we calculated the total weight of the structure and the total weight of the members in each group after that, we calculated the percentage by weight of members in each group. These percentages provide a good overall view of the structure stresses under a Cooper E80 load.

See Figures 4-3 to 4-5 and Table 4-1 for a summary of the Cooper E80 structural evaluation results. Detailed results and calculations are provided in Appendix E.

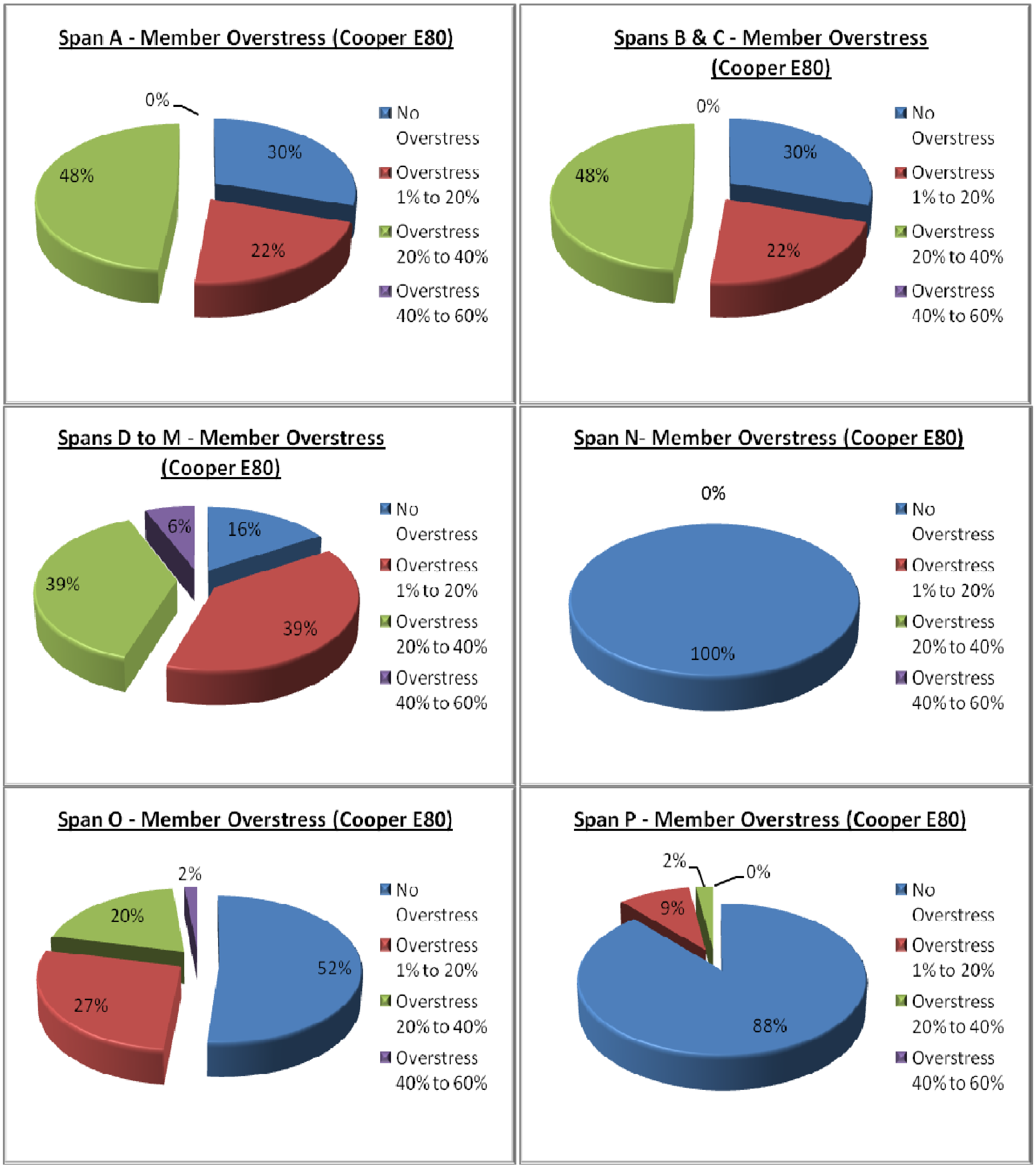


Figure 4-3 Spans A to P – Member Overstress (Cooper E80)

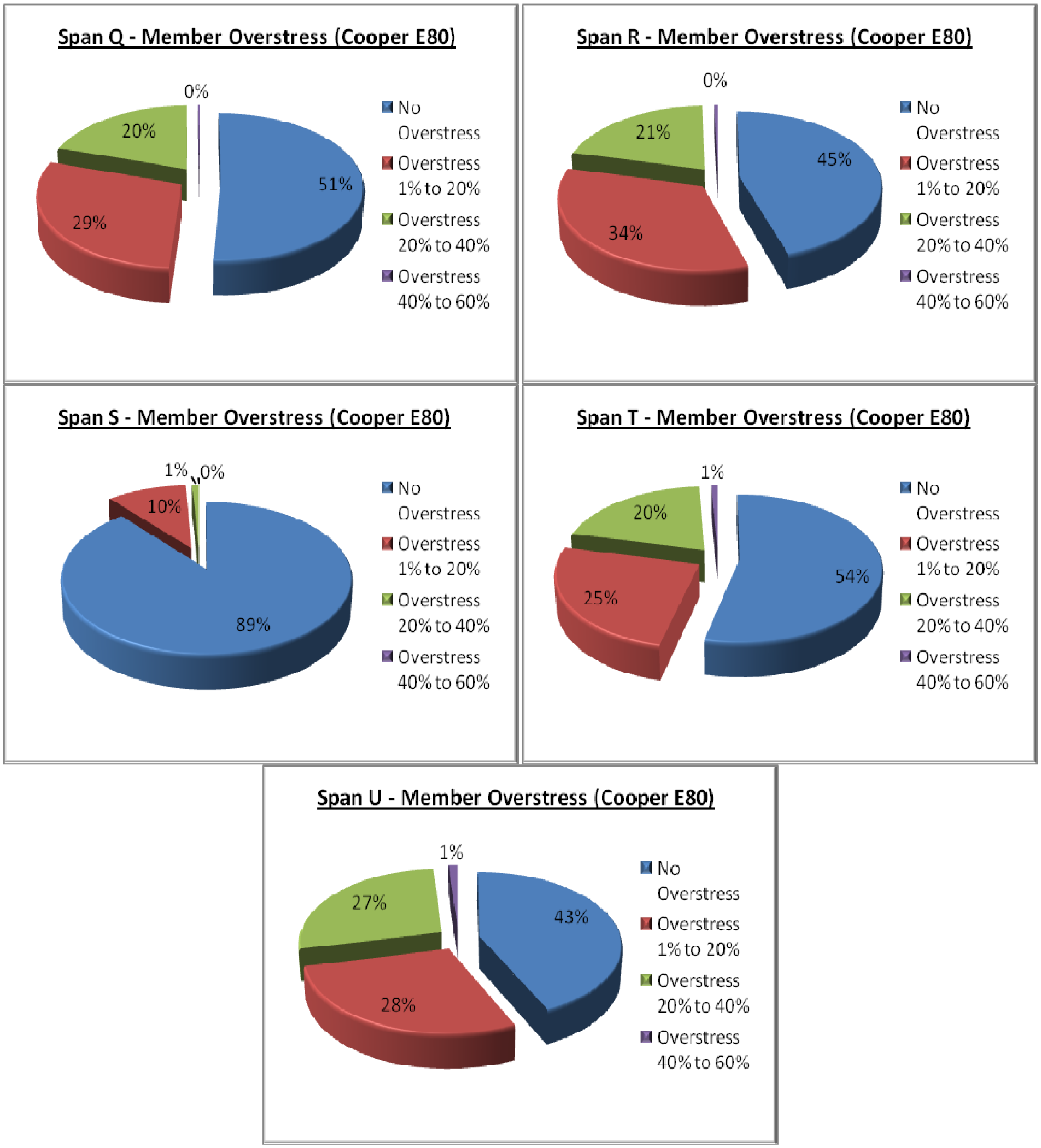


Figure 4-4 Spans Q to U – Member Overstress (Cooper E80)

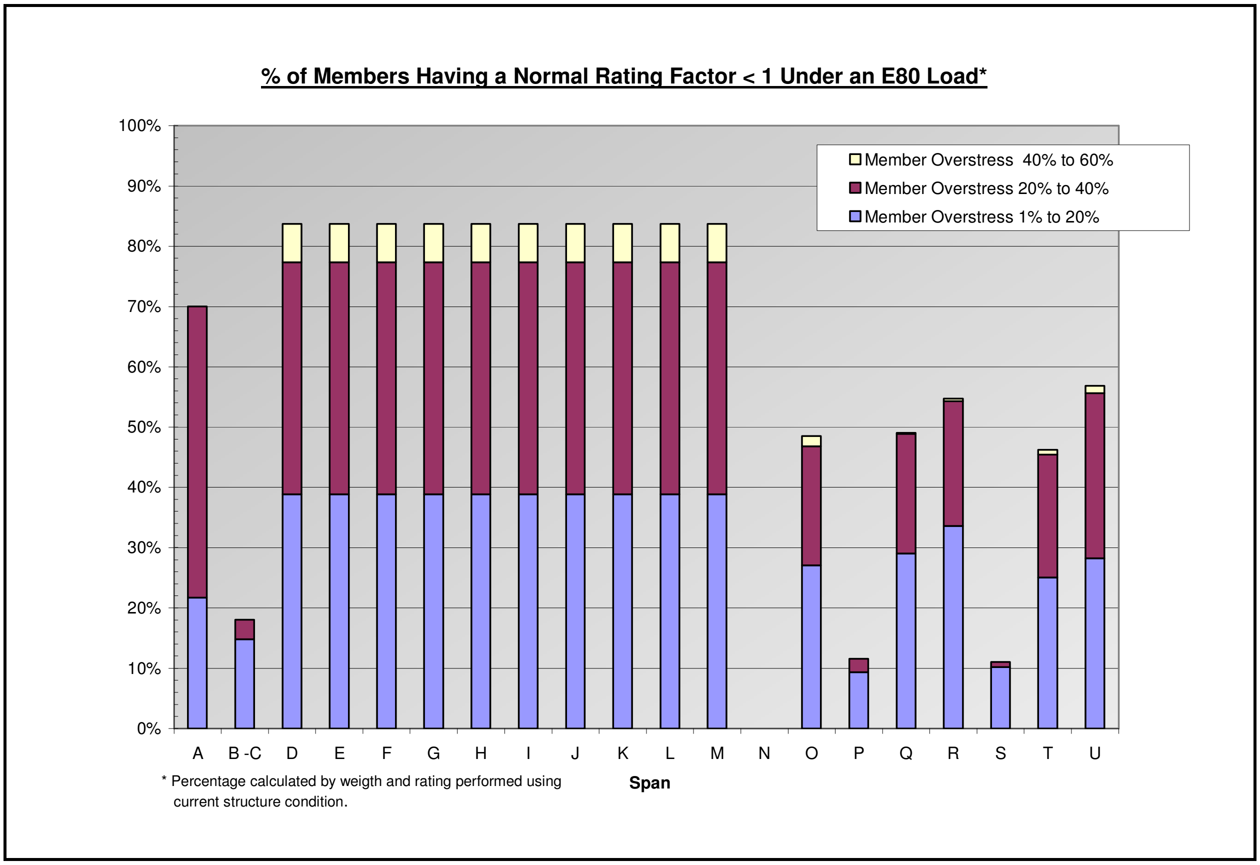


Figure 4-5 Percentage of Failing Members by Weight (E80 Load)

Table 4-1 Cooper E80 Structural Evaluation Results

Span	Min. Normal Rating Factor	Percentage of Failing Members by Weight			
		Total	Member Overstress 1% to 20%	Member Overstress 20% to 40%	Member Overstress 40% to 60%
A	0.64	70%	22%	48%	0%
B -C	0.61	18%	15%	3%	0%
D	0.56	84%	39%	39%	6%
E	0.56	84%	39%	39%	6%
F	0.56	84%	39%	39%	6%
G	0.56	84%	39%	39%	6%
H	0.56	84%	39%	39%	6%
I	0.56	84%	39%	39%	6%
J	0.56	84%	39%	39%	6%
K	0.56	84%	39%	39%	6%
L	0.56	84%	39%	39%	6%
M	0.56	84%	39%	39%	6%
N	1.08	0%	0%	0%	0%
O	0.52	48%	27%	20%	2%
P	0.71	12%	9%	2%	0%
Q	0.58	49%	29%	20%	0%
R	0.55	55%	34%	21%	0%
S	0.68	11%	10%	1%	0%
T	0.57	46%	25%	20%	1%
U	0.54	57%	28%	27%	1%

4.3 Fatigue Evaluation

The deck truss and deck girder structure is considered a fracture critical structure with the 2 truss lines or girder lines for each of the spans. A fracture critical structure is where, if a main member fails, the entire structure could collapse due to a lack of redundancy. Fatigue prone details and connections will limit the tensile stresses at locations throughout the girders or trusses with typical examples shown in AREMA Table 15-1-9. Bridges today are designed with these details allowed for due to the history of particular details that have proven to be susceptible such as a pin and eyebar detail. In addition, the primary connections to be evaluated would typically involve welded connections due to induced stress from the welding operations but riveted and bolted connections also need to be evaluated as well. The structural makeup of the bridge includes primarily riveted connections having Category C or D details depending on the condition and range of cycles applied to the bridge throughout its life. Connections and details require close inspection for fracture critical bridges and the stresses and load ratings can be significantly affected by the deterioration of the connections or a reduction in remaining life is reasonable due to the condition.

The type of train traffic is not specifically known so that the fatigue life cannot be accurately predicted, however, the present owner, WSOR, has a fairly good knowledge that, for some of the spans, the normal or continuous rating has been exceeded since the 1950's with locomotives and since the 1970's with locomotives and rail cars. With the reduction in traffic in the last decade or so, the cycles do not appear to have approached the design cycles as noted in the AREMA manual for design. Even if the connection members may be approaching their fatigue life according to specified limits, the bridge was originally designed for the Cooper E40 loading and does not have the normal rating capacity to accept much higher loadings that could cause a reduction of fatigue life. In addition, the construction of the trusses and girders are made up of riveted connections which typically have good fatigue characteristics as long as overloading is not prevalent.

The conditions of the girders and trusses do not indicate that connections and details have been overstressed due to fatigue as cracking would occur in the base metal of the main member or connection piece or rivets would start working loose. In addition, overloading could cause the members to distort out of plane. Since the inspection noted that connections are tight and no cracking was located in the members of the girders or trusses and that no distortion was occurring other than from pack rust, fatigue was not considered to be a significant condition warranting further investigation. For any fracture critical bridge, routine inspections are necessary to monitor the condition of the details that are susceptible to fatigue.

If the traffic of the line is to be significantly increased and heavier loadings applied to the structure, further analysis may be warranted to assess the fatigue life of the structure but at this time, no reduction in service life is expected from the recommended value based on the type of traffic currently being used by the structure and the assumption that additional traffic that is overloaded would not be allowed to use the structure.

5. Improvement Alternatives

The following sections describe the proposed alternatives that will be evaluated to upgrade or maintain the existing bridge superstructures and substructures to the design or specified Cooper E loadings. Estimated costs are provided in Section 8 to further assist the owner in prioritizing their funding for repair or replacement. As noted in the previous sections, the existing structure is in good condition considering its age but requires minor repairs and routine maintenance.

5.1 Deck

The timber ties of the deck were noted to be in good condition but are not an element to be evaluated for improvement. The life span of the ties can be as long as 30 years but will tend to deteriorate rapidly toward the end of the life period. Since the ties are about 20 years old at this time, consideration to replace is suggested with the repair costs.

5.2 Superstructure

One of the objectives for this evaluation is to review the spans that do not meet the Normal rating condition of Cooper E40 loading using the current condition of the structural elements. Spans O, Q, R, T and U are the only spans that have analysis results below the Cooper E40 rating loading with values of E36, E38, E37, E38 and E36, respectively. All are truss spans. Also, each of the spans has a condition rating factor of 0.90, 0.95, 0.90, 0.95 and 0.90, respectively to represent the current condition. The above spans require some member improvements to bring the spans back up to the E 40 level.

The specific members for each of the spans to be evaluated are the following:

> Span O, the floor beams in tension and bending.
>
> Span Q, the top lateral bracing attached to the floor beam in compression and bending.
>
> Span R, the floor beams in compression and bending.
>
> Spans T and U, the floor beams and top lateral bracing attached to the floor beam both in compression and bending.

As shown above, in general the floor beams and top lateral bracing are the primary members that need to be improved. The floor beams are riveted built-up plate girders and the lateral bracing are angles that are riveted to the gusset plates of the floor beams. For the lateral bracing, since the rating factors are just short of the limit of 1.0, they can easily be removed and replaced with a slightly larger section. The floor beams are also just short of the 1.0 rating limit but are more difficult to upgrade as the improvements would need to be made to the flanges to improve the bending. Cover plates can be added to the outsides of the flanges but the cross frames below would have to be modified as well.

Since the rating factors include a general condition rating reduction and all Cooper loadings are equal to or above E 40 before the section loss was applied, it would be more prudent to monitor the elements at this time during routine inspections to verify if any deficiencies have increased.

If the floor beams or top lateral bracing starts to show section loss due to corrosion or any local conditions, then repairs would be recommended.

5.2.1 Critical Structural Element Upgrades

Since the deck girder and deck truss superstructures are in relatively good condition, there were no elements that require critical upgrades. The inspection revealed areas that were noted throughout the deck trusses that need routine repairs. The structure appears to not have specific spans that develop deficiencies more rapidly than others. Refer to Section 5.1.3 for the recommended maintenance repairs.

5.2.2 Cooper E80 Upgrade Improvements

Another primary objective of this study is to evaluate the bridge components to see if they can be upgraded to be capable of allowing Cooper E80 loadings. To improve the structure to the higher loading, the following alternatives will be evaluated if they are reasonable:

1. Attach additional plate material to the lower capacity members.

   In reference to the results of Section 4.0, the estimated percentage of members needing improvement to upgrade to the Cooper E80 loading is from between 46% to 84% showing that the number of members needing improvement is not a reasonable alternative solution. Only 5 of the 22 spans, Spans B, C, N, P and S, have a percentage that is low enough that would appear to be reasonable to include member repairs for upgrade. An alternative solution to improve 50% or more members for a span is not a feasible solution such that this alternative will not be further evaluated.

2. Replace lower capacity members.

   Similarly as noted in the Alternative 1 discussion above, too many members would need to be replaced to improve to the Cooper E80 loading. As such, an alternative solution to replace 50% or more members for a span is also not a feasible solution so that this alternative will not be further evaluated as well.

3. Add a deck girder or deck truss line.

   Adding a deck girder line or deck truss line is a possible solution since the existing deck girder and deck truss lines can stay in place; however, the floor beams, diaphragms, cross frames and all other internal members between the primary truss or girder lines will need modification. The main disadvantage to this alternative is the age differential between the girder or truss lines of 100 years or more even though the existing members are in good condition. Some of the substructures would also need major rehabilitation or even replacement such as the concrete encased steel tower piers. A geotechnical analysis would be necessary to verify the foundation capacities since the increase in load for each pier will be about 60% of the original loading at each location.

4. Replace the structure.

   The most complete procedure to ensure that the bridge is upgraded to meet the latest Cooper E80 loading is to replace the entire structure. This procedure eliminates the piecemeal process and provides superstructures and substructures specifically designed to the latest loading capacity. If the same spans are to be used some of the existing substructures may be reused; however, the more prudent method would be to develop a new alignment while keeping the existing line in service. The disadvantage is the extra right-of-way cost that would be encountered.

5.2.3 Maintenance Repairs

As noted in the inspection, the deck girder span members were noted to have minor surface corrosion that is correctable by a thorough paint system as discussed below. Otherwise, the small areas of section loss noted in the webs of the girders adjacent to the floor beams and in the webs of the floor beams adjacent to the stringer connections are not severe enough to need repairs other than a paint system and monitoring.

For the deck truss spans, the members were also noted to be in good condition with only minor surface corrosion correctable by a thorough paint system. The inside gusset plates with section loss at the lower chords do not need repairs at this time since the entire connection is reduced by only 12.5% and should be painted and monitored at this time. However, the areas with pack rust along the bottom chords will require repair since this has caused minor distortion of some of the members. These areas will require some component removal and replacement after the corrosion has been removed or ground out.

The bearing for Span U at Pier 22 with a cracked plate is recommended to be repaired by welding on the fractured section and grinding down the weld.

In addition, the steel elements appear to have been painted in the past and plans originally called for a one coat system. A paint system is typically on an owner’s routine maintenance program where a total structure sand blast procedure is performed at 10 to 15 year intervals and interim painting is performed as needed or at 5 year intervals.

5.3 Substructure

The substructures normally do not control a load capacity of the bridge unless there are severe deficient areas that reduce the support capability. The inspection revealed that the upper areas of the piers near the bearing areas have a loss of section or stone masonry due to abrasion or ice damage. In addition, scour does not appear to be occurring to potentially increase the risk of the substructures although some footings are visible. To verify the subsurface foundations, borings would be necessary.

5.3.1 Critical Structural Element Upgrades

The deficient conditions of the piers do not cause a reduction in supporting capacity such that upgrades are necessary. Repairs will fall under routine maintenance repairs in the following section.

5.3.2 Cooper E80 Upgrade Improvements

Although geotechnical borings and analysis will be required to verify the foundation capacities, further analysis would be necessary to verify that the stone masonry abutments and piers can support the additional loads for the Cooper E80 loading as well. The concrete encased steel tower piers will also require analysis to verify the existing supporting capacity. In addition, some destructive testing may be needed to check the encased steel sections, although the inspection does not appear to show any specific signs of corrosion damage at the water line.

5.3.3 Maintenance Repairs

In general the abutments and piers are in good condition as noted in the inspection, however, some areas are in need of repair. Pier 2, and Piers 18 to 20 have missing stone masonry blocks that need to be replaced. The stone masonry abutments and piers also have a loss of mortar jointing that requires repair. The reinforced concrete encased steel tower piers have abrasion damage near the water line in need that will require repair. Also, the pedestals for Piers 15 and 16 have delaminations to be repaired.

5.4 Other Cooper Loading Levels

Referring back to Table 3-2 for the Bridge Rating Summary, the results provide an indication of different levels of Cooper ratings that can be further evaluated. For example, to upgrade to a Cooper E50 rating for Normal Load Rating at the current condition, some improvements would still need to be provided for all but 3 of the 22 spans whereas, if the Maximum Load Rating is used, only 1 span needs to be upgraded for the E 50 level but only 2 spans need to upgraded to the E 60 level. This analysis assumes that the Normal Rating is the method to be used for normal operations, however, the owner can then allow for an occasional overload if the need arises as long as upgrades are provided to get to the higher Cooper E ratings.

6. Welded Rail Evaluation

Replacing the existing jointed sections of rail and replacing with continuous welded rail provides the advantage removing or reducing the number of joints on the bridge therefore providing a smoother ride.

This evaluation explores the feasibility from the structural point of view of replacing the existing rail with continuous welded rail. The proposed continuous welded rail has to be installed in accordance to the AREMA manual.

The following are the requirements given in the AREMA manual for the anchorage of continuous welded rail to open deck steel bridges.

> **"8.3.3.1** (Longitudinal Anchorage of Rail on Bridge Approaches)
>
> *On roadbed approaches to bridge of length over 50 feet, rail shall be box anchored longitudinal at each tie a distance of 200 feet unless otherwise specified by the engineer."*
>
> **"8.3.3.5(b)** (Longitudinal Anchorage of Continuous Welded Rail without Expansion Joints on Open Deck Bridges)
>
> *b. On bridges with total length of 300 feet or greater, rail anchors shall be applied as follows:*
>
> *(1) For individual spans of 100 feet or less, rail anchors, if used, shall be applied throughout the span, at all ties anchored to bridge spans.*
> *(2) For individual spans exceeding 100 feet, rail anchors shall be applied only in the first 100 feet from the fixed end, at all times anchored to the bridge spans.*
>
> *c. Bolted joints connecting strings of continuous welded rail shall not be located on bridges nor on roadbed approaches within 200 feet of the ends of the bridges."*
>
> ***"8.3.4.4** Number and Positioning of Rail Expansion Joints on Bridges with Continuous Welded Rail.*
>
> *e. The spacing and design of joints shall be such that the maximum length L, in feet of rail, causing movement through each joint shall be as follows, except that L shall not exceed 1,500 feet."*

The AREMA manual Section 5.2.1 covers the present practice for laying and maintenance of continuous welded rail (CWR). This section states:

> *"CWR (Continuous Welded Rail) should not be laid across long open bridge decks without special consideration."; "Rail anchors should not be used on a open deck bridge without special precaution"; and "If structural stresses are significant on bridge, CWR can be laid stress free by using sliding rail joints."*

To find if the additional structural stresses applied to the structure by the continuous welded rail are significant, the structure was modeled and load rated with the continuous welded rail and then these results compared with the load ratings obtained for the structure as designed and constructed (See Chapter 3)

The most significant impact of a continuous welded rail in a structure like this is the fact that the continuous rail restricts the movement of the top of the structure changing the behavior of the truss from simple supported span to a continuous span.

The structural evaluation for Span R is presented in this chapter; the results obtained in the other spans are similar to the results obtained in Span R.

Figure 6-1 shows the continuous welded rail configuration for span R, this configuration follows the requirements of the AREMA manual Section 8.3.3.5 (b). The continuous welded rail is replicated in the structural model by adding a member at the end of the stringers, and by assigning a fixed support at the end of these members, See Figure 6-2.

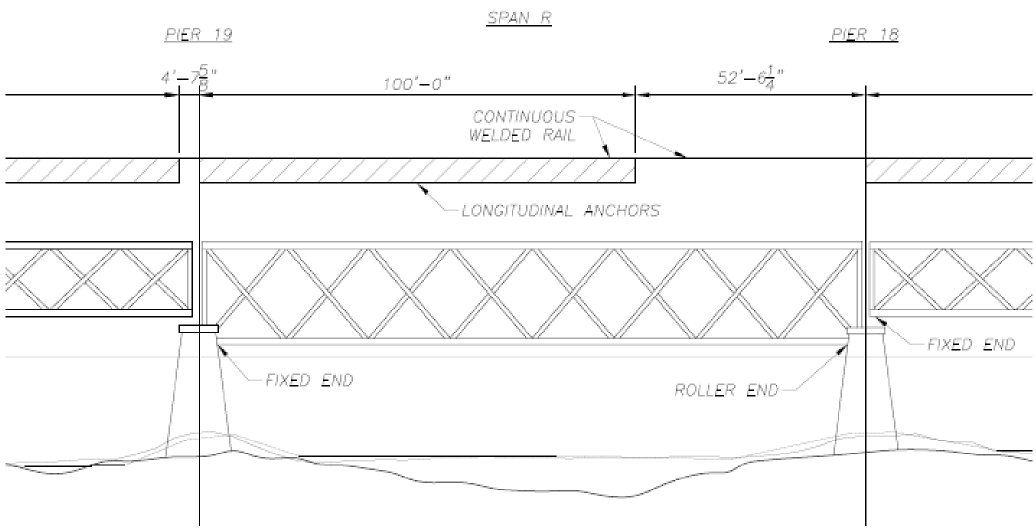


Figure 6-1 Continuous Welded Rail Configuration for Span R

The comparison of structural stresses between the span with and without continuous welded rail is going to be based on the Normal Load Rating results. The normal load rating for this span as designed and constructed was a Cooper load E41 therefore the live load used in this model is a Cooper E41 load, the other loads used in this model are the same loads described in Chapter 3. Figure 6-3 shows the deformed structure under the controlling load.

The load rating procedure was the same procedure described in Section 3.2, the only difference is that in this case we wanted to know what rating factors will produce an E41 load in this span with a continuous welded rail, and compare those factors with the values obtained for the same live load without the continuous welded rail.

A summary of the load rating results for each group of members with and without the continuous welded rail is given in Tables 6-1 and 6-2.

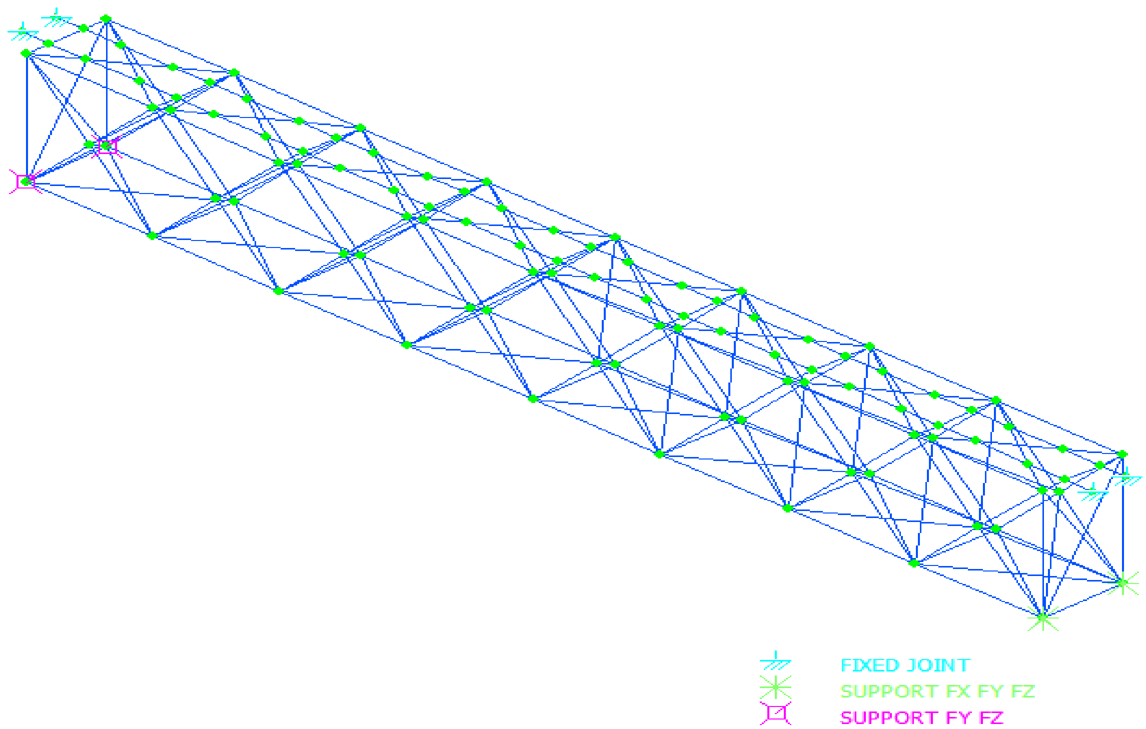


Figure 6-2 Span R Structural Model with Continuous Welded Rail

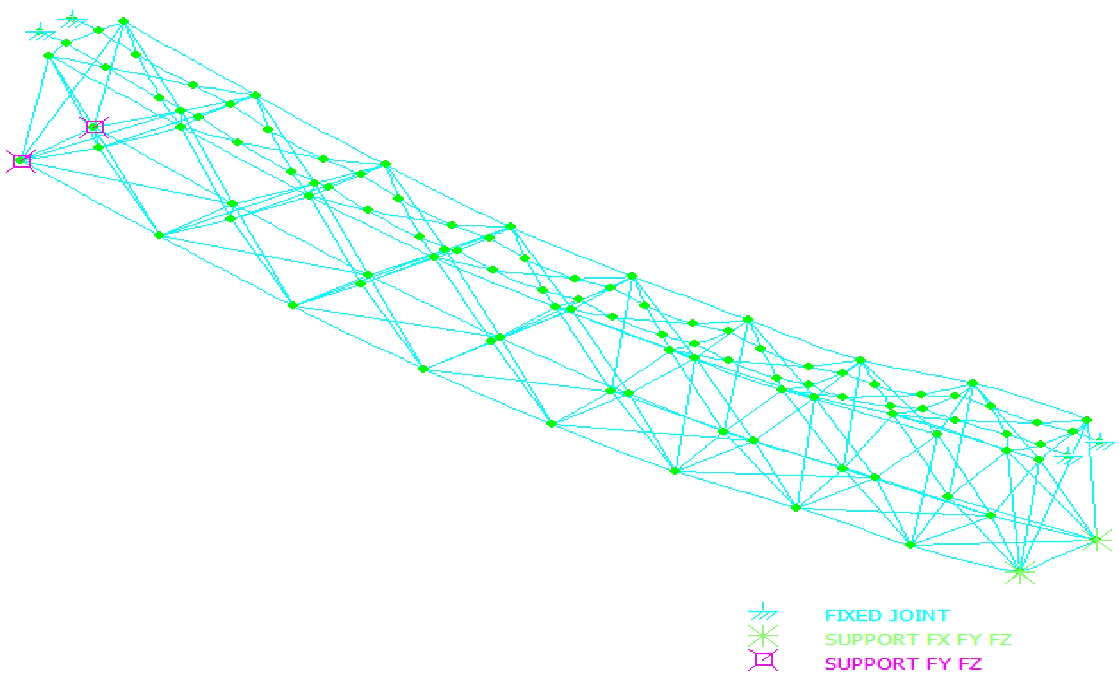


Figure 6-3 Span R Deformed Structure with Continuous Welded Rail

Group	RATING SUMMARY (NORMAL) - E41 - (CONTINUOUS WELDED RAIL)										
	Controlling Member	Controlling Load	FX	FY	FZ	MX	MY	MZ	Rating	Controlling Provision	Longitudinal Load Controls?
BC1	19	FX	125.85	-0.33	0.14	0.00	-2.03	6.13	1.58	Tension & Bending (Tension Fiber)	No
BC2	20	FX	313.93	0.23	0.19	0.00	-3.89	5.84	1.57	Tension & Bending (Tension Fiber)	No
BC3	21	FX	422.53	-0.28	0.26	0.00	-4.38	14.60	1.48	Tension & Bending (Tension Fiber)	No
BC4	22	MZ	439.73	1.25	-0.23	0.00	-4.37	22.34	1.58	Tension & Bending (Tension Fiber)	No
BL12	188	FX	-29.49	--	--	--	--	--	1.06	Pure Compression	No
BS1	36	FX	-13.35	--	--	--	--	--	3.97	Pure Compression	No
CF	191	FX	-40.59	--	--	--	--	--	3.77	Pure Compression	No
DC1	83	FX	-285.14	--	--	--	--	--	1.26	Pure Compression	No
DC2	85	FX	-185.45	--	--	--	--	--	1.39	Pure Compression	No
DC3	87	FX	-154.43	--	--	--	--	--	1.43	Pure Compression	No
DT1	115	FX	189.29	--	--	--	--	--	1.47	Pure Tension (Gross Area)	No
DT2	117	FX	164.62	--	--	--	--	--	1.30	Pure Tension (Gross Area)	No
EP	73	FX	-86.55	--	--	--	--	--	4.43	Pure Compression	No
ES	43	MZ	0.00	0.00	-0.21	-0.01	-1.25	0.00	7.04	Compression & Bending	No
FB	68	FX	87.16	33.91	-20.97	-0.15	-73.38	-118.67	0.50	Tension & Bending (Tension Fiber)	No
S	280	MZ	252.70	25.47	35.47	0.05	-101.21	199.26	1.07	Tension & Bending (Tension Fiber)	No
SB	155	FX	-8.16	--	--	--	--	--	4.28	Pure Compression	No
TC1	17	MZ	-73.04	4.86	-0.15	0.01	-0.55	50.45	3.07	Compression & Bending	No
TC2	15	MZ	-225.94	2.50	-0.27	0.01	-3.92	37.55	2.33	Compression & Bending	No
TL1	281	FX	125.82	0.00	0.00	0.00	0.00	0.00	0.63	Tension & Bending (Tension Fiber)	No
TL2	271	FX	-83.75	0.00	0.00	0.00	0.00	0.00	0.82	Compression & Bending	No
TL3	259	FX	-48.14	0.00	0.00	0.00	0.00	0.00	1.09	Compression & Bending	No
									0.50		

Table 6-1 Normal Load Rating for Span R with Continuous Welded Rail.

Group	RATING SUMMARY (NORMAL) - E41 -										
	Controlling Member	Controlling Load	FX	FY	FZ	MX	MY	MZ	Rating	Controlling Provision	Longitudinal Load Controls?
BC1	34	FX	223.59	0.39	0.44	0.00	1.41	0.02	1.19	Tension & Bending (Tension Fiber)	Yes
BC2	20	FX	355.42	0.45	0.18	0.00	-4.32	4.73	1.40	Tension & Bending (Tension Fiber)	No
BC3	24	MZ	474.02	1.14	-0.23	0.00	-5.46	26.42	1.27	Tension & Bending (Tension Fiber)	No
BC4	23	FX	557.81	-0.98	0.19	0.00	-5.46	25.53	1.26	Tension & Bending (Tension Fiber)	No
BL12	187	FX	38.90	--	--	--	--	--	1.62	Pure Tension (Gross Area)	Yes
BS1	41	FX	-20.58	--	--	--	--	--	3.21	Pure Compression	Yes
CF	189	FX	-74.45	--	--	--	--	--	2.05	Pure Compression	No
DC1	83	FX	-249.29	--	--	--	--	--	1.44	Pure Compression	No
DC2	85	FX	-165.44	--	--	--	--	--	1.56	Pure Compression	No
DC3	87	FX	-124.92	--	--	--	--	--	1.77	Pure Compression	No
DT1	115	FX	171.52	--	--	--	--	--	1.63	Pure Tension (Gross Area)	No
DT2	117	FX	133.52	--	--	--	--	--	1.61	Pure Tension (Gross Area)	No
EP	73	FX	-137.91	--	--	--	--	--	2.78	Pure Compression	No
ES	43	MZ	0.00	0.00	-0.23	-0.02	-1.37	0.00	6.45	Compression & Bending	No
FB	65	MZ	-3.67	-89.48	-6.77	-0.01	-23.69	313.18	1.00	Compression & Bending	No
S	242	MZ	-210.45	-1.13	-1.17	0.00	-16.83	235.32	1.46	Compression & Bending	No
SB	155	FX	-10.86	--	--	--	--	--	3.21	Pure Compression	No
TC1	17	MZ	-164.39	2.53	-0.18	0.01	-0.47	36.77	2.35	Compression & Bending	No
TC2	15	MZ	-296.70	-1.22	0.42	0.00	-2.48	36.87	1.96	Compression & Bending	No
TL1	197	FX	-67.66	0.00	0.00	0.00	0.00	0.00	1.23	Compression & Bending	No
TL2	271	FX	-61.58	0.00	0.00	0.00	0.00	0.00	1.12	Compression & Bending	No
TL3	259	FX	-38.79	0.00	0.00	0.00	0.00	0.00	1.36	Compression & Bending	No
									1.00		

Table 6-2 Normal Load Rating for Span R as Designed and Constructed.

After comparing the load rating factors given in the rating summaries for each case, we found significant additional stresses in some of the members when the continuous welded rail is added. The members that take these additional stresses are the following: Stringers (S), Floor

Beams (FB) and Top Transverse Bracing (TL1, TL2 & TL3). The increase in the stresses leads to several members with rating factors less than 1. In the rest of the members the stresses are reduced or remain unchanged. These results were anticipated due to the continuous span behavior induced by the continuous welded rail.

Looking at a more detailed set of results for members FB, S, TL1, TL2 and TL3 we found the following:

Member Group	Members with Incresed Stresses	Stress Increase Range	Minimum Rating
FB	60%	2% to 66%	0.50
S	25%	4% to 51%	1.07
TL1	92%	3% to 64%	0.63
TL2	50%	18% to 66%	0.82
TL3	50%	13% to 60%	1.09

Table 6-3 Span R Member Stress Increases with Continuous Welded Rail.

After conducting a detailed analysis of the structural effects of adding a continuous welded rail to this bridge we can say that this modification in the rail will add significant structural stresses to the bridge structure. When this happens, the AREMA manual Section 5.2.1 recommends laying the continuous welded rail stress-free by using sliding rail joints. See figure 6.4.

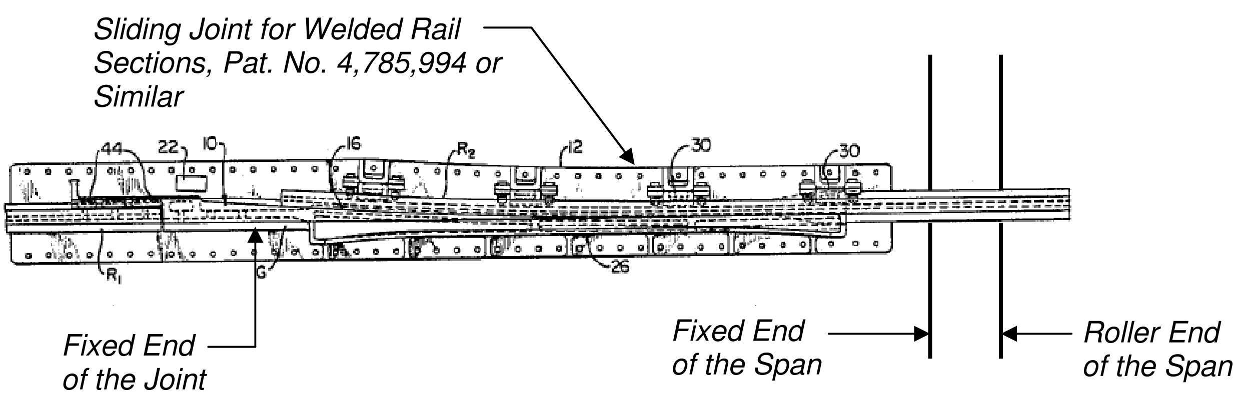


Figure 6-4 Sliding Joint for Welded Rail Sections.

In conclusion, this bridge doesn't require any upgrades to accommodate a continuous welded rail, as long as the continuous welded rail is installed stress-free by using a sliding rail joint on each span

7. Prairie du Sac Dam Evaluation

In 1907, the Prairie du Sac Dam was constructed approximately 7 miles downstream from the bridge causing the backwater to rise at the bridge. Refer to Figure 7-1. As a result, the steel tower piers from Pier 4 to Pier 14 were encased in concrete to protect from constant water submersion. The change in high water elevation rose from approximately Elevation 763 ft. to an estimated maximum Elevation 775 ft. when the water is at top of the closed tainter gates for the dam. However, for Piers 4 to 14, the original construction in 1913 left top of the concrete protection below the high water line and would allow the steel tower elements to become submerged. So, the protection for each of the piers was then raised further to get the tops out of the water to Elevation 778.0 in 1915. No other protective work appears to have been provided to the bridge due to the construction of the dam.

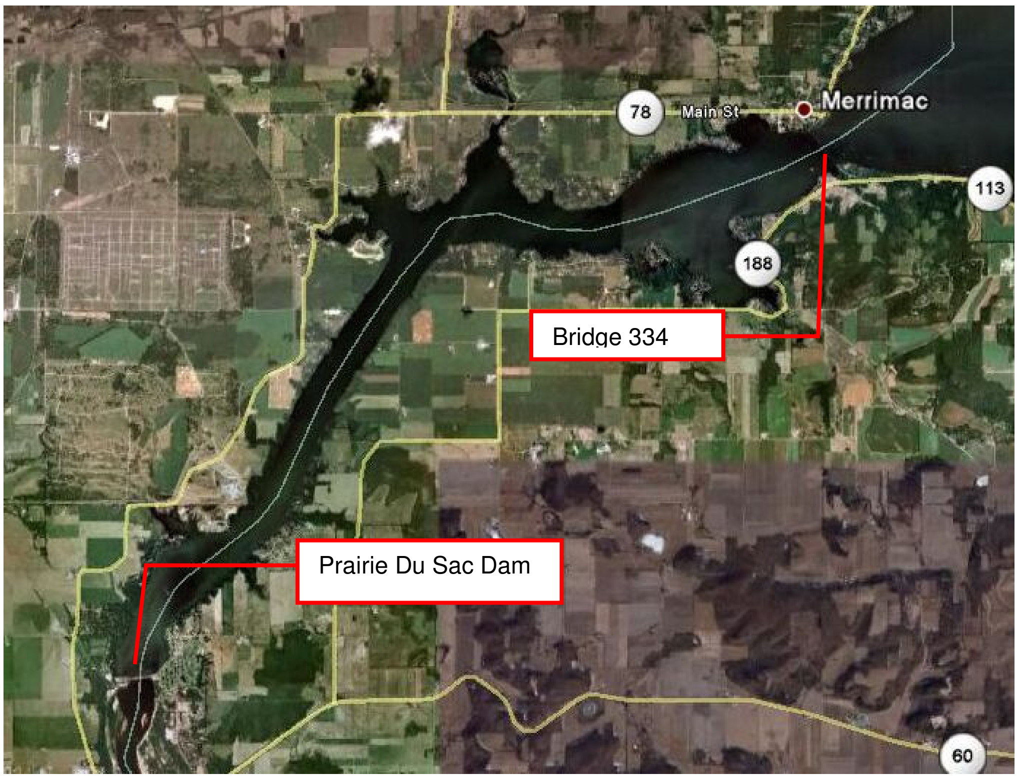


Figure 7-1 Location of Dam.

With the change in water elevations, the steel bearings and some of the steel truss members are closer to the water surface and thus more prone to corrosion and accelerated deterioration. In addition, the potential for increased damage from ice buildup is another condition that could result. Chapter 8 will discuss in more detail the vulnerability to ice damage from flow or buildup.

The following list shows the approximate clearances between the new high water line and the top of caps at the bearings or the bottom of the truss members:

Abutment / Span	Clearance *	Remarks
Abutment 1	N/A	
Piers 2 to 3	1.0 ft.	Top of Cap
Piers 4 to 14	3.0 ft.	Top of Conc. Encasement
Piers 15 to 17	5.7 ft.	
Piers 18 to 19	Possible Submergence	Top of Span R Cap
Pivot Pier 20	5.7 ft.	Top of Cap
Piers 21 to 22	5.5 ft.	Top of Cap
Abutment 23	5.5 ft.	Top of Cap
Span A	N/A	Girder Span
Span B	3.2 ft.	Truss Span
Span C to N	N/A	Girder Spans
Span O to Q	7.8 ft.	Truss Spans
Span R	1.2 ft.	Truss Span
Span S	5.7 ft.	Movable Truss Span
Spans T to U	5.7 ft.	Truss Spans

* N/A if not significant to corrosion or submersion

Table 7-1 High Water Line Clearances with Bridge Components

As can be seen from the above table, the locations at greatest risk due to corrosion are the bearings at Piers 2, 3, 18 and 19, the steel tower column elements at Piers 4 to 14, and the bottom chords and laterals for the steel trusses in Spans B and R. The other areas listed may see some water spray from time to time but are not expected to be regularly saturated.

For the bearings at Piers 2, 3, 18 and 19, the inspection did not reveal any differing conditions from other bearings indicating that, from a maintenance perspective, no more significant corrosion has occurred that would require additional improvements. The bearings typically have soil and debris build-up that has caused the minor corrosion. In addition, since the adjustments have been made over 90 years ago with no adverse effects, no additional improvements to the bearings are recommended with respect to the adjusted water levels.

Figure 7-2 Pier 2 Bearings.

Figure 7-3 Pier 19 Bearings.

For the tower piers at Piers 4 to 14, no accelerated corrosion appears in the steel columns particularly at the top of the concrete cap surface. In addition, the joints between the added top concrete sections in 1915 to the first added section 1913 do not have spalling that is exposing the steel members. The spalling that is occurring on the noses of the concrete caps appears to be due to ice build-up or flow and does not affect the steel columns at this time. However, further spalling may eventually expose the columns underwater and provide the potential for increased corrosion. Damage to the concrete caps due to ice will be discussed further in Chapter 8. Although the conditions of the column members encapsulated in the concrete are unknown, an indication of corrosion problems would be apparent at the steel members exposed just at the top of the caps. Since this does not appear to be the case and, the conditions of the columns are relatively good for the length of period of exposure, no additional improvements are recommended.

Figure 7-4 Pier 14 Columns

Figure 7-5 Span B Truss.

The trusses in Spans B and R have the most vulnerability in the lower chords due to the change in water elevation for the lake. In addition, all the truss spans are safety hazards to recreational boats due to the lack of clearance. Typically, the design of bridges over waterways should have 2 ft. clearance over the design high water and, for small recreational boating, a minimum of 6 ft. of clearance is recommended above the high water elevation if the channel is not designated as a navigational waterway. Span R could also be vulnerable to ice damage since the clearance is only 1 ft. above the high water line but will be discussed more in Chapter 8. However, review of the steel components in regard to possible increased corrosion for the sections closer to the water, the inspection noted that Span R had pack rust between the bottom chords and the vertical or diagonals, but was also noted similarly in Span U. Other deteriorated conditions were similar in the trusses throughout. All of the bottom chords typically had similar deficiency conditions where the inside vertical gusset plates connecting the lower chord to the diagonals and verticals have moderate pitting along the top of the bottom chord and the lower lateral transverse brace angles have developed pack rust between the angles. The pack rust between the angles and the section loss for the gusset plates will be recommended to be monitored at this time as noted in Chapter 5 whereas repairs are recommended for the pack rust between the lower chord and the diagonals and verticals.

In summary, the steel members have not been significantly affected by the change in water elevation in regard to corrosion. Affects due to ice will be discussed in more detail in the following chapter.

8. Ice Damage Vulnerability Evaluation

Bridge structures over waterways for Wisconsin will typically need to be designed to resist the forces from ice formation due to expanding sheet development and to the impact forces due to ice flow or the change in the hydraulic forces in the case of ice jams. If not accounted for in design, the damage that ice formation or ice flow can cause could lead to serious repair issues or possibly to a total loss of the structure if precautions are not taken. With the change in the water surface elevation due to the building of the Prairie du Sac dam, the structure's vulnerability to ice damage was increased particularly to the superstructure elements. Piers 4 to 14 were initially designed not in the water and were steel tower type piers whereas the piers originally designed for the water were designed with substantial mass to counteract the water flows and possibly ice forces although the design calculations were not available for review. With the age of the structure and affected elements nearing 100 years old, there is a long period that can be referenced for historical conditions.

For superstructure spans, a minimum vertical freeboard clearance over the water surface is required to pass the design storm water, ice or debris under. Design guidance from AREMA, AASHTO and WisDOT all provide clearance recommendations above the high water line surface. Chapter 1, Part 3 Natural Waterways gives guidance for hydraulic design to establish the waterway opening and scour analysis to aid in abutment or pier protection and countermeasure design. However, freeboard clearances to the low beam or chords elements were not specifically provided other than to provide sufficient clearance above the design storm. However, WisDOT provides the more definitive guidance where the low beams or chords of a bridge span should be 2 ft. above the 100-yr design storm and, if no other information is available for ice thickness, the design clearance should be at least 12" to allow ice to pass through. These limitations are for bridges designs today but earlier limitations are not known. Only Span R is at risk due to the clearance being at 1 ft. or equal to the recommended WisDOT clearance. From the inspection though, no distortion of the lower chords were observed to indicate that damage due to the ice impact was or is occurring to any significant degree. Without further analytical study to verify ice thickness and forces, no modifications of the truss in Span R is recommended. However, WSOR should continue to monitor the bridge during floods especially during spring thaws to verify that no damage has taken place. If ice formation events are beginning to occur with some frequency, then further study may be warranted.

Pressures from ice flow forces are provided in the AREMA manual under Chapter, Section 2.2.3 and vary from 100 psi to 400 psi depending upon the temperature and pier ice splitting characteristics to be applied laterally to the piers. The lower pressures indicate that the ice is in melting state and the piers have good splitting capability. WisDOT recommends a pressure of 250 psi if no other information is available. In addition, for an expanding ice sheet, WisDOT also recommends a lateral expansion force of 8 ksf, similar to AASHTO, to be applied laterally to the pier as well. But, due to the unknown capacity of the masonry materials and unknown substructure foundation capacity, and with the knowledge that the velocity of water flow is partially controlled by the dam, a structural analysis would require more in depth geotechnical study and investigation to provide the required parameters to be valid even though a lower pressure could be assumed.

The inspection of the bridge with emphasis on ice effects and a survey can provide the necessary information to base a sound conclusion on whether movement has occured. Since movement of the bridge and its piers are the major sign of any detrimental effects that ice formations can cause by sliding or tilting the piers and thus the entire structure out of alignment, these effects would be readily seen on the track rails due to being out of alignment or the ties showing tie plates pulling out or lateral movement of the ties on top of the stringers. This does not appear to have been taken place as these conditions were not noted. In addition, the underwater inspection may show that the piers have slid along the channel bottoms and this also does not appear to have occurred.

The primary issues to review and provide repair recommendations include the abrasion of concrete piers to ice flow and the loss of masonry blocks from freeze – thaw cycles affecting the masonry piers.

Piers 4 to 14 have abrasion loss typically up to 1.5 ft. on both the upstream and downstream noses of the piers with some piers having as much as 4 ft. of abrasion. These areas that have the abrasion are the second concrete pours to increase the height of the piers to protect from the new dam. The remaining portions of piers with concrete sections at the water line also have up to 1 ft. of abrasion. The piers are typically sharp nosed piers that are the better type to reduce ice impacts and scour. The abrasion is occurring just below the top of the caps creating ledges that can cause the ice pieces to flip and stack up on to the piers. Repairs to reestablish the pier form will reduce this effect. Coatings around the concrete ice zone may help with increasing the durability against ice damage for this condition, but since the damage has occurred in a period of 100 years, repairs without coatings would still be expected to outlive the useful life of the structure and are not recommended.

The masonry piers and abutments are also at risk for abrasion but also have the added condition of losing stones and grout due to the freeze-thaw cycles. The masonry stones do not have any scouring though. Piers 2, 18 to 22 have masonry stone or stones missing. Not only do the voids increase the likelihood of flipping ice pieces, but some of the voids are under bearing areas that can also reduce the support capacity of the upper portion and be more at risk for the freeze-thaw affects. With the replacement of the missing stones and repair of the loss of grout, the piers will be upgraded to their design level and reduce the effects of ice damage.

Repair procedures and costs will be discussed in more detail in Chapter 10.

9. Scour Evaluation

The field evaluation portion of a Level 1 scour evaluation form was filled out during the dive inspection on April 30, 2009.

The scour evaluation is done using guidelines as presented in FHWA HEC-18, Evaluating Scour at Bridges and HEC-20, Stream Stability at Highway Structures

The bridge is located at sections 2 and 11, T10N, R7E. It is found on the Lodi and Durwards Glen USGS 7 ½' quad maps. The north end of this bridge is in the village of Merrimac.

This bridge is over the Wisconsin River at Lake Wisconsin. The stage elevations for the site are controlled by the Prairie du Sac Dam.

There is a detailed Flood Insurance Study (FIS) for this site. The Department of Natural Resources (DNR) web site has the FIS HEC-RAS hydraulic model available for download. The model was obtained to check the hydraulics for this site. The flows used in the FIS are based on a detailed Wisconsin River model analysis done by the USGS that accounts for the affects of all of the dams and reservoirs in the basin. The 500-year flood analysis shows velocities through the bridge being less than 3 feet-per-second (fps).

The dive inspection found that some piers appear to have riprap protection; some appear to have exposed gravel and cobble stream bed while others have areas of silt and sand at the base of the piers. The silt and sand deposits are typical for much of the river bed near the piers.

During a 500-year flood event, some of the fine silt & sand deposits is anticipated to being removed near the piers. However, with velocities below 3-fps, there will be minimal scour at this site.

The one area that might be of concern is the piers where riprap or cobble was not found. Placement of small riprap around the upstream 1/3 of the pier 1 to 2 feet thick extending 3-feet from the pier would provide protection against any anticipated scour at and near these piers.

With placing of riprap at the piers with silt and sand, a scour code of 5 is recommended for this bridge. The descriptions associated with a code 5 are:

- The bridge foundations are determined to be stable for the assessed scour condition.
- The scour is determined to be within the limits of the footings.
- The foundations are determined to resist scour within the service life of the bridge.

10. Engineering Cost Analysis

Preliminary repair, rehabilitation and replacement costs are developed to provide assistance to WSOR in determining their funding priorities for Bridge No. 334. The costs developed are to maintain the bridge at a Cooper E40 rating with repairs, or to rehabilitate or replace the structure entirely to upgrade to a Cooper E80 rating. However, due to several unknown factors of the existing substructure elements and foundations, the rehabilitation alternative is to be viewed with caution until extensive geotechnical testing is performed to verify the foundation capacities.

Improvements or enhancements of the existing structure to increase the capacity to a Cooper E80 rating are not feasible solutions since too many members would need to be modified in the majority of the spans and the majority of the members are riveted built-up sections. As noted previously 17 of the 22 spans would need to have 50% or more of the members adjusted or modified to meet the E 80 loading. A new structure or a major rehabilitation such as installing an additional girder or truss line is the recommended method to upgrade to the E 80 rating.

The following sections describe in more detail the alternatives evaluated.

10.1 Bridge Repairs

For the current condition of the structure, the remaining life expectancy of the structure is controlled by the substructure elements near the water line surface. The ice damage and potential damaging effects will encroach on the support and bearing locations of the superstructure. If no substructure repairs are implemented, the remaining useful life is estimated to be approximately 15 years. The superstructure elements have weathered very good and would not be a concern for at least 25 years with no repairs made other than routine cleaning.

To maintain the structure at the Cooper E40 rating and to provide some longevity the life of the structure as noted above, a number of repairs are recommended.

1. Repair the pack rust along the bottom chords.
2. Repair and replace the missing stone masonry block.
3. Repair the loss of mortar in the stone masonry.
4. Repair the areas of abrasion on the upstream and downstream pier noses.
5. Repair the delaminated bearing areas.
6. Add rip rap to piers

The estimated cost for providing the maintenance repairs is approximately **$4.2 million** where $1.5 million is for replacing the timber ties for the deck. Although the original plans called for a paint system, the bridge has not been painted for some time and does not show any adverse effects other than some corrosion that is more caused by debris build-up. For the work on the piers, much would need to be done underwater but estimates include dewatering systems to be able to do the work in the dry for the abrasion repair and masonry replacement and point repairs. Performing this work and providing for some spot paint repairs is expected to allow the structure to have a life expectancy to 25 years as stated above. Refer to Appendix F for a summary of the costs and calculations.

The repairs to the existing structure will not be able to upgrade the rating of the bridge but will help maintain the Cooper E40 rating.

10.2 Bridge Rehabilitation

One method to upgrade the existing bridge while using the existing truss or girder lines would be by adding a line of new trusses and girders between the existing. Estimated costs were developed installing a deck truss or girder line between the existing lines but will require major replacement of members for the interior frames and lateral bracing. If the line is to be kept in service, then shoring would need to be provided but was not included in the costs provided. Since the existing outside truss and girder lines are to remain and many of the piers are likely to remain, the service life is estimated to be about 30 years as long as some the repairs as noted above are provided. But since the components of the existing bridge are already over 100 years old, the additional girder would only provide a nominal increase in useful life from the repair recommendation. The cost for a major rehabilitation is about $19.8 million but the repairs would need to be added for a total of **$24.0 million**.

Note that although rehabilitating the structure appears to be a good solution, the unknown foundation factors cause reservations regarding this alternative and is only provided in the event that WSOR has been able to determine the foundation capacities and are agreeable to reuse 100 year components for much longer than originally designed for.

10.3 Bridge Replacement

A new structure would provide a life expectancy of 75 to 100 years depending upon the maintenance programs. The primary advantage for a new structure is that it will be designed for the Cooper E80 loading or higher but the disadvantage will be the cost. Different alternatives were analyzed with one alternative reusing the same configuration as the existing bridge which is unlikely. Other alternatives reviewed include spanning the lake with 16 spans with 4 – 110' deck steel plate girders per span supported by 4 – 4' dia. drilled shafts; or 12 spans with 4 – 145' deck steel plate girders per span supported by 4 – 5' dia. drilled shafts; or 8 220' steel deck trusses supported by 4 – 6' dia. drilled shafts.

The alignment of a replacement structure would be best suited just to the south of the existing bridge to reduce the impact to the adjacent the Town of Merrimac on the west bank and Okee on the east bank. Right-of-way would be needed and approaches would be required to tie the line back into the exiting.

Discussion with the Wisconsin Department of Natural Resources indicated that for a new alignment just downstream of the current alignment, no issues would preclude replacement from an environmental standpoint based on a cursory review of the concept of moving the bridge minimally (40') downstream. However, the new design would need to address the following:

- Low chord for the spans should accommodate the taller type watercraft on the lake. This includes pontoons and larger fishing/water skiing boats.

- Verify that no "special concern" or endangered species which would preclude us from relocating the alignment just downstream.

- Address archaeological sites located in or near the park in Okee.

In addition to right-of-way concerns, the approaches to the bridge would need to designed to minimize impacts to the "Ice Age" trail on the Merrimac (Sauk County) side and minimize any park land takings on the Okee (Columbia Co) side.

Preliminary engineering costs for the total replacement alternatives are:

1. Replace steel spans in kind = **$ 48.3 million**
2. 16 spans of 110 ft. Deck girders only = **$ 35.0 million**
3. 12 spans of 145 ft. Deck girders only = **$ 39.5 million**
4. 8 spans of 220 ft. Truss spans only = **$ 65.3 million**

Refer to Appendix F for the cost calculations.

The following table is provided as a guide for improving or replacing the existing bridge:

Table 9-1 Repair or Replacement Cost Summary.

Repair Method	Estimated Cost	Estimate Remaining Service Life
Routine Repairs	$4,200,000	25 years
3rd Line of Girders, Trusses	$24,000,000	30 years
New Structure	$35,000,000 to $65,300,000	75 years

11. Final Recommendations

In the previous sections, an evaluation was provided based on the field inspections and available plans to determine the necessary repairs required to maintain the structure at its assumed Cooper E40 rating level and to determine if possible improvements or upgrades can be implemented to increase the Cooper E rating level to an E80. From the structural analysis, the current condition of the bridge was determined to be as low as an E36 rating. However, the rating was based on a general condition state rating reduction. Overall, the condition of the bridge can be considered sufficient with minor repairs made and routine maintenance provided where the original design level can be used and the bridge can be considered at a Cooper E40 rating.

Improvements or upgrades to specific structural members to improve the rating to a Cooper E80 level would be too numerous and are considered not feasible as more than 50% of the members would require some modifications or enhancements. This generally includes the main chords and girders for the associated spans of riveted built-up construction.

The primary upgrade alternatives evaluated include constructing an additional truss or girder line as a major rehabilitation method or for a total replacement to upgrade to the Cooper E80 loading level. The additional truss and girder line may not be a feasible alternative due to the unknown foundation factors. This alternative is not recommended since it would be unlikely that the foundations can safely handle the additional loads. For the major upgrade alternative, additional geotechnical analysis would be necessary to ensure that the additional loads can be accommodated by the foundations, and additional analysis is needed to verify that the substructures have the capacity as well. The advantage of this method is that the existing alignment and existing trusses and girders can be reused although there would be a large age difference in the materials in regard to allowable stresses. The major rehabilitation option would be asking that the existing components end up having a service life at around 140 years.

To upgrade to normal Cooper E80 loading, replacement of the existing bridge is the recommended alternative that will also provide for the expected service life for a new structure. The different replacement alternatives analyzed were based on conservative estimates, but a replacement structure would need to be evaluated in more detail as to the final configuration. To meet the proper clearance requirements and loading requirement, replacing the structure would be necessary. The preferred location is just south of the existing structure and would allow the line to remain open until the traffic can be switched. Preliminary review of environmental impacts does not appear to show that there would be issues to preclude the new alignment. However, additional right-of-way would be necessary for the new alignment.

Continuous welded rail was structurally analyzed for the existing bridge and found to cause an increase in stresses, thus reducing the rating capacity in some of the truss members to an unacceptable level. However, additional expansion joints can be installed and would be recommended throughout the existing bridge while the rail is secured using sliding joints.

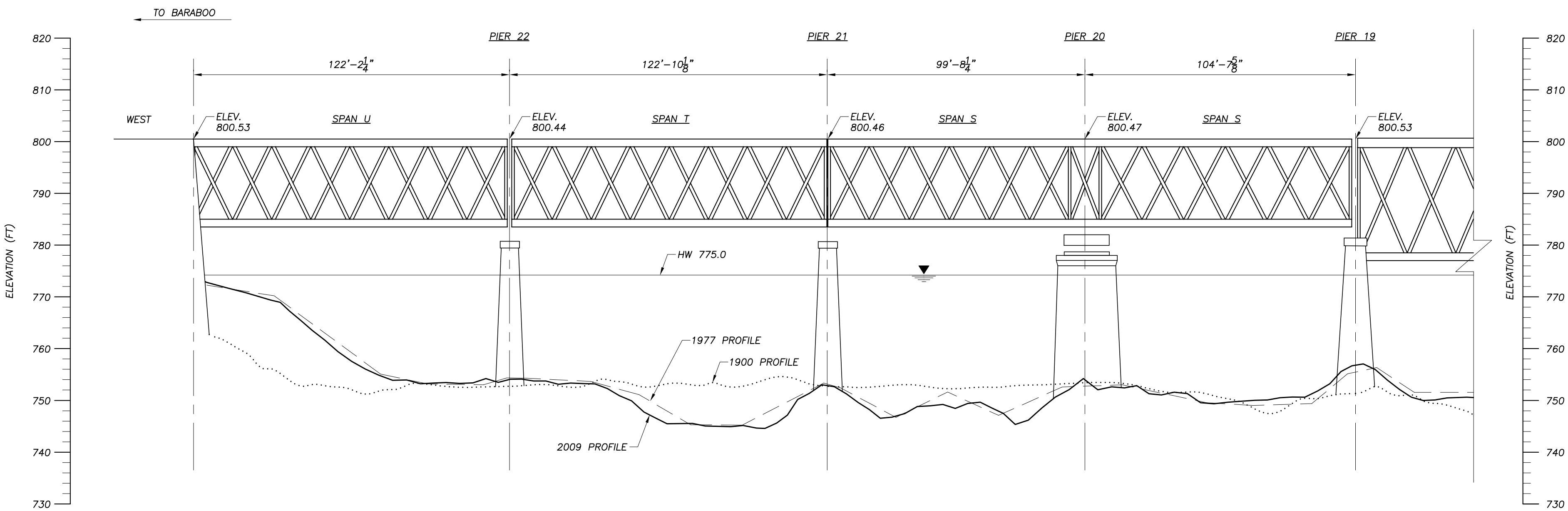
The effects of the rise in backwater elevations from the Prairie du Sac Dam were evaluated. The corrosion rate of the steel components that are now closer to the water surface does not appear to have increased because of the installation of the dam. The structure has similar corrosion areas throughout, indicating that very little effect has occurred. The rise in backwater elevations has increased the potential for ice damage and also creates more safety hazards for boaters due to the low chords. Spans B and R are the spans with very low clearance. A replacement structure would eliminate the above hazard.

The existing bridge was also evaluated for vulnerability to ice pressures and ice flow, particularly the spans and bearings closer to the water surface. In review of the supporting data and referring to the inspection results, it was found that the concrete piers are being abraded by the flow of ice or the impact of ice sheets, and the masonry piers appear to be losing some stones due to freeze-thaw cycles. In both cases, repairs to the piers are recommended and will be sufficient to counteract the effects of ice.

If the priorities of WSOR are to maintain current rail traffic while allowing additional traffic not to overload the structure, implementing the repairs are recommended to maintain the structure at its current Cooper E40 normal operating level as long as locomotive and car equipment are available that meet the lower ratings. If the priorities are to increase the traffic level and use of the line where standard locomotives and cars are expected, then a new structure is recommended to upgrade to the Cooper E80 rating levels.

Appendix A

Plan and Elevation View



ELEVATION

NOTE:

INFORMATION FOR ELEVATIONS OF POINTS AT BASE OF RAIL OVER PIERS TAKEN FROM DRAWING No. 11499 "GENERAL ELEVATION OF BRIDGE" (08–07–1913)

LEGEND

- ········ 1900 PROFILE
- — — 1977 PROFILE
- ——— 2009 PROFILE

REVISIONS	BY	DATE

SURVEY BOOK No.:		
SURVEY DIVISION	BY	DATE
SURVEYED		
TECHNICIAN		
CHECKED		
DESIGN DIVISION		
DESIGNED		
DRAWN	IAG	11/09
CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

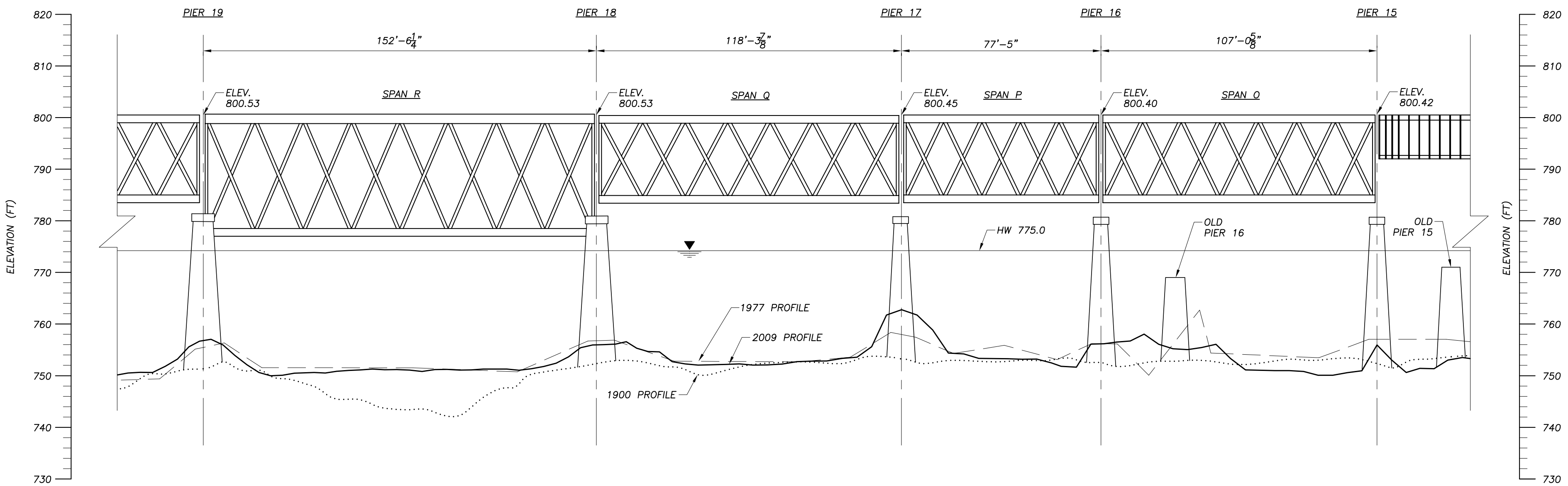
CHANNEL PROFILE
(1 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES

8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION NO. 4356

DATE: 11/2009
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 1 of 4



ELEVATION

LEGEND

1900 PROFILE
1977 PROFILE
2009 PROFILE

REVISIONS	BY	DATE	SURVEY BOOK No.:		
			SURVEY DIVISION	BY	DATE
			SURVEYED		
			TECHNICIAN		
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			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL PROFILE
(2 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES
8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION
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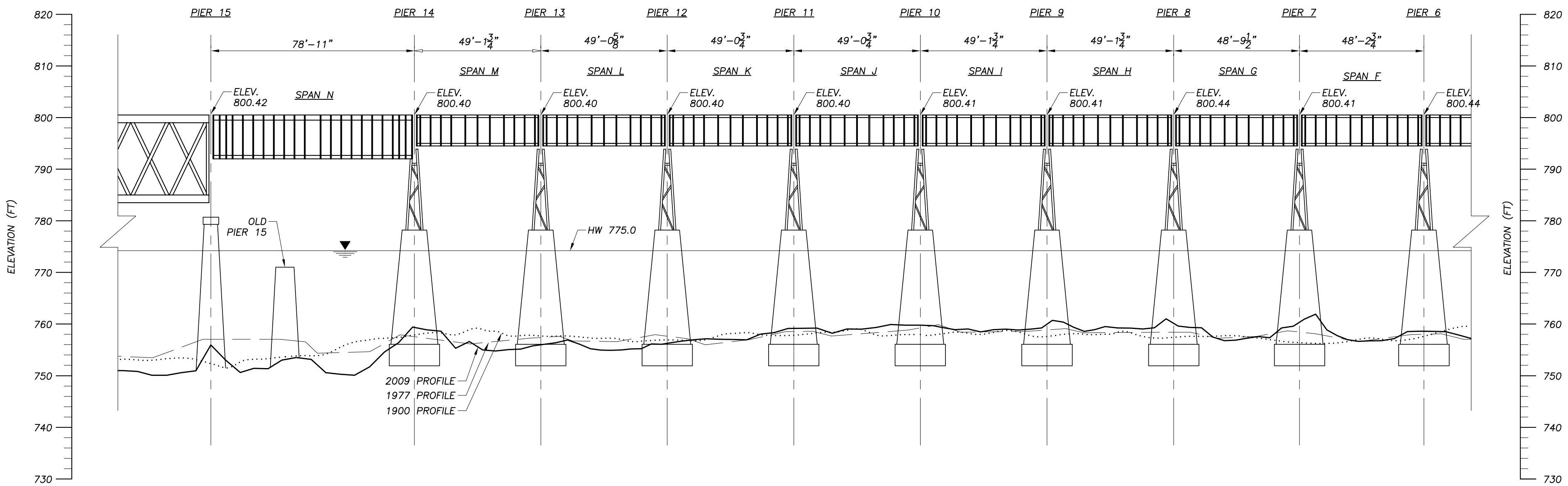
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 2 of 4



ELEVATION

LEGEND

1900 PROFILE
1977 PROFILE
2009 PROFILE

REVISIONS	BY	DATE	SURVEY BOOK No.:		
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			SURVEYED		
			TECHNICIAN		
			CHECKED		
			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL PROFILE
(3 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES

8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION
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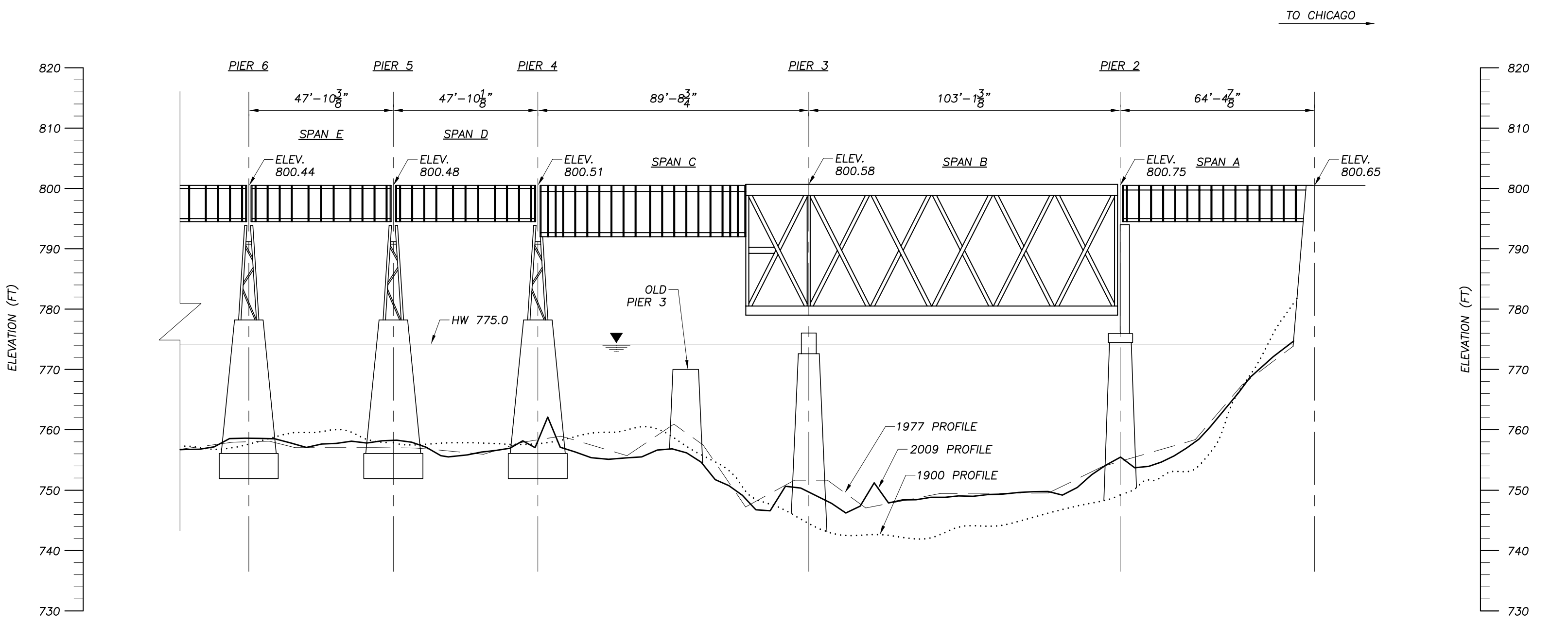
DATE: 11/2009
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 3 of 4



ELEVATION

LEGEND

- 1900 PROFILE (dotted line)
- 1977 PROFILE (dashed line)
- 2009 PROFILE (solid line)

REVISIONS	BY	DATE	SURVEY BOOK No.:	BY	DATE
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			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

MERRIMAC RAILROAD BRIDGE No. 334 COMPREHENSIVE STUDY

CHANNEL PROFILE (4 OF 4)

WISCONSIN & SOUTHERN RAILROAD CO.

AYRES ASSOCIATES
8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION NO. 4356

DATE: 11/2009
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 4 of 4

Appendix B

Inspection Report

BRIDGE INSPECTION REPORT

Inventory Data

Feature On: Wisconsin & Southern Railroad		Maintainer: WI & Southern RR	Structure Number: 334
Feature Under: Wisconsin River		Sect/Twn/Rng:	
Location: 0.2 mi SE of JCT STH 78		County: Sauk / Columbia	Municipality: Merrimac
Inv Rating:	Deck Width: 14.0 ft	Deck Width: 14.0 ft	Existing Posting:
Oper Rating:	Total Length: 1729.8 ft	Deck Area: 24217.2 sf	ADT On: Yr: ADT Under: Yr:

Inspection Type (* = Additional Applicable Form(s) Required)

	Routine Visual	Fracture Critical*	In-Depth*	UW-Dive*	UW-Surv.*	UW-Probe/ Visual*	Movable*
Last Insp.							
Frequency							
Recom. Freq.	24 Months	24 Months		60 Months			

	Initial*	Damage	Interim	Load Posted	SI & A Field Review*
Last Insp.					
Frequency	N/A				
Recom. Freq.	N/A				Item No. Needing Change

Load Rating Information

Overburden	File Meas. (in):	File Insp. Date:	Insp. Meas. (in):	Type:
Section Loss	File Meas. (%):	File Insp. Date:	Insp. Meas. (%):	Describe:
Should structure be re-rated for load carrying capacity? (Y/N)			Reason:	Date last rated:

Expansion Joints

Location	Type	Temp. File Insp. Date	File Insp. (in.)	New Insp. (in.)

Signing Condition

Type of Marker	File	Y N N/A	Comments
Bridge Markers			
Narrow Bridge			
One Lane Road			
Vertical Clearance			
Weight Limit			
Other(Addl. Sign)			

Clearances (Cardinal = N or E)	File Meas. (ft.)	File Date	New Meas. (ft.)
Min. Vertical Clearance Under (Cardinal)			
Min. Vertical Clearance Under (Non-Cardinal)			
Min. Vertical Clearance On			

Structure Type

Material	Configuration	# of Spans	Overall Length (ft)
Steel	Deck Girder	12	700.5
Steel	Deck Truss	8	940.3
Steel	Deck Truss & Deck Girder	1	89.0

Construction/Rehabilitation History

Year	Work Performed	Plan	Shop
1895	Erected New Spans C - N	1387	
1903	Major Reconstruction and Rehabilitation	3639	
1909	Erected Steel Towers for Piers 4-14	8174	
1911	Encased Steel Towers with conc. -18.5’	10390	
1915	Encased Steel Towers with conc. -7.5’	12080	
1923	Reconstruction of Pier #17	14740	
1930	Built New Piers 3, 15, & 16 and	17926	
	Rehabilitated Adjacent Spans		
1943	Remod. Span S Bearings and	24864	
	Anchorage against uplift		

Inspection Information

Special Requirements	Y/N	Comments
Traffic Control	Y	Schedule Inspection with Road Master
Access Equipment	Y	Dive Boat and Climbing Gear
Other		

Inspector Information

Team Leader Name and No. Printed: Brian K. Schroeder (9540)	Team Member(s) Name(s) Printed: Matthew Rynish (9580)	
Team Leader Signature:	Insp. Date: 05/01/2009	Inspection Agency: Ayres Associates

NBI Ratings

NBI	File	New	NBI	File	New
Deck		6	Culvert		N
Superstructure		6	Channel		8
Substructure		5	Waterway		8

Maintenance Recommendations

Item	Cost	Comments

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	

Abutment: 1 – East – Masonry (Remodeled in 1903) | Insp. Date: 04/29/2009

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	Element Inspection (X) Check Elements Inspected					Quantity in Condition States				
X	217	2	Masonry Abutment	LF	20		20			
	Comments:									
X	402	2	Masonry Wingwalls	EA	2		2			
	Comments:									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Length: 64’-8”	Insp. Date: 05/01/2009

Span: A – Deck Girder (Erected in 1903)

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	Element Inspection (X) Check Elements Inspected					Quantity in Condition States				
X	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
X	106	2	Unpainted Steel Open Girder	LF	127		127			
	Comments: Girder length = 63’-6”.									
X	112	2	Unpainted Steel Stringer	LF	259		259			
	Comments:									
X	151	2	Unpainted Steel Floor Beam	LF	48		48			
	Comments:									
X	171	2	Unpainted Steel Diaphragm	EA	24		24			
	Comments:									
X	174	2	Unpainted Steel Lat. Bracing	EA	6		6			
	Comments: Members within a plane of a bay are considered 1 brace.									
X	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
X	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Pier: 2 – Masonry (Remodeled in 1903)	Insp. Date: 04/27/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	1		1			
	Comments: Members within a plane of a bay are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	2		2			
	Comments:									
	211	3	Masonry Pier Wall	LF	27		27	2		
	Comments: 1 displaced stone under south bearing at the waterline on the west face. 8 inch max penetration of grout loss extending to 3 feet below the water surface.									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Span: B – Deck Truss (Erected in 1903 and Remodeled in 1929)	Length: 103'-10" / Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	415		415			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	204		180	24		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 101'-10 ½". 2 flame cut holes in ext. top flange of north bottom chord in bay 1. Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	204		204			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 101'-10 ½". 2 flame cut holes in exterior bottom flange of north top chord is bay 5.									
	151	2	Unpainted Steel Floor Beam	LF	72		72			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	40		40			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	16		10	5		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
		2	Concrete Counterweight	EA	1		1			
	Comments: Follow condition states for "Reinforced Concrete Elements." Located at Pier 2.									
	Comments:									

General Inspection/Maintenance Notes

Includes all truss elements for Bays 1-5.

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 3 – Reinf. Concrete (New in 1929)

Insp. Date: 04/27/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	210	3	Reinf. Concrete Pier Wall	LF	33		33			
	Comments: Sheet piling exposed at DS (south) end of pier. Scaling up to 3 inches deep at the water surface ending 1 foot below the water surface. Scaling has exposed steel at the DS (south) end of the pier above the water line.									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: C – Deck Girder (Erected) and Deck Truss (Remodeled) – (1930)

Length: 89'-0" Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	139		121	32		
	Comments: Girder length = 69'-4 ½". Pitting up to 1/8 inch on vertical leg of interior bottom flange at horizontal gusset plates.									
		3	Unpainted Truss – Bot. Chord	LF	41		33	8		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 20'-4 ½". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	41		41			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 20'-4 ½".									
	151	2	Unpainted Steel Floor Beam	LF	48		48			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	27		27			
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		22			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

The Lateral Bracing and Floor Beam between Bays 5 and 6 of the Deck Truss are included is Span B. There are 2 Floor Beams in the deck truss and 2 Floor Beams in the deck girder.

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 4 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Insp. Date: 04/27/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.5 feet at the US and DS noses.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: D – Deck Girder (Erected in 1895)

Length: 47'- 9 1/2"
Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	94		84	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	96		96			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace. Lower brace at Floor Beam 4 is bent 6 inches out of plane at 4 feet from South Girder.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
	Insp. Date: 04/27/2009

Pier: 5 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.5 feet at the US and DS noses.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Length: 47'- 9 1/2"	Insp. Date: 05/01/2009

Span: E – Deck Girder (Erected in 1895)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	94		84	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	96		96			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Pier: 6 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)	Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.5 feet at the US and DS noses.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Span: F – Deck Girder (Erected in 1895)	Length: 48'-1 1/2" / Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	94		84	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	96		96			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Floor Beam 2 has 1/8 inch diam. hole and section loss up to 50% up to 2 inches tall and 3 inches wide at stringer shelf plate. Floor Beams 1 and 3 – 5 have section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	

Pier: 7 – Concrete Encased Steel Tower (New - 1909, Encased – 1915) Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected

						Quantity in Condition States				
Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		36	4		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.5 feet at the US nose and 2.5 feet at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	

Span: G – Deck Girder (Erected in 1895) Length: 48'-9 1/2" Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

						Quantity in Condition States				
Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR

Structure Number: 334

Pier: 8 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 2.0 feet at the US nose and 1.0 foot at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR

Structure Number: 334

Span: H – Deck Girder (Erected in 1895)

Length: 49'-1 1/2"

Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 9 – Concrete Encased Steel Tower (New - 1909, Encased – 1915) | Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.5 feet at the US and DS noses.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: I – Deck Girder (Erected in 1895) | Length: 49'-1 1/2" | Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	

Pier: 10 – Concrete Encased Steel Tower (New - 1909, Encased – 1915) | Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.0 foot at the US nose and 1.5 feet at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	

Span: J – Deck Girder (Erected in 1895) | Length: 49’-1 1/2” | Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/4 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 11 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.0 foot at the US nose and 2.0 feet at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: K – Deck Girder (Erected in 1895)

Length: 49'-1 1/2"
Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 12 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Insp. Date: 04/30/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
						Quantity in Condition States				
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		37	3		
	Comments: Typical scaling for Piers 4 -14 with a max penetration up to 1.5 feet at the water surface at the US and DS noses.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: L – Deck Girder (Erected in 1895)

Length: 49’-1 1/2” Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
						Quantity in Condition States				
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 13 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		36	4		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 1.0 foot at the US nose and 3.0 feet at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: M – Deck Girder (Erected in 1895)

Length: 49'-1 1/2" Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	96		86	10		
	Comments: Pitting up to 1/8 inch deep by 1 inch tall by 1 inch wide on interior web at horizontal gusset plate at bottom flange of floor beams.									
	112	2	Unpainted Steel Stringer	LF	98		98			
	Comments:									
	151	2	Unpainted Steel Floor Beam	LF	60		50	10		
	Comments: Section loss up to 1/8 inch deep by 1 inch tall by 2 inches wide on both sides of web at stringer shelf plate.									
	174	2	Unpainted Steel Lat. Bracing	EA	9		9			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
	Insp. Date: 04/29/2009

Pier: 14 – Concrete Encased Steel Tower (New - 1909, Encased – 1915)

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	4		4			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	4		4			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	40		35	5		
	Comments: Typical scaling for Piers 4 -14 with a max penetration at the water surface up to 3.0 feet at the US nose and 4.0 feet at the DS nose.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Length: 78’-11”	Insp. Date: 05/01/2009

Span: N – Deck Girder (Erected in 1930)

Element Inspection (X) Check Elements Inspected

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	106	2	Unpainted Steel Open Girder	LF	158		122	36		
	Comments: Girder length = 78’-10”. Pitting up to 1/8 inch on vertical leg of interior bottom flange at horizontal gusset plates.									
	151	2	Unpainted Steel Floor Beam	LF	12		12			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	16		16			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 15 – Reinf. Concrete (New in 1930)

Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	174	2	Unpainted Steel Lat. Bracing	EA	1		1			
	Comments: Members within a plane are considered 1 brace.									
	201	3	Unpainted Steel Columns	EA	2		2			
	Comments:									
	210	3	Reinf. Concrete Pier Wall	LF	34		32	2		
	Comments: South face of south bearing pedestal is delaminated up to 6 inches deep. Scaling up to 1.0 foot at the water surface at the DS nose. The top tier of the footing is exposed up to 1.0 foot wide and 3.0 feet tall and the bottom tier is exposed up to 1.0 foot wide and 2.0 feet tall.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: O – Deck Truss (Erected in 1903, Moved in 1930)

Span Length: 107'-1"
Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	428		428			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	209		187	32		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss Length = 104'-6 ¾". Flame cut holes in interior top flange of south bottom chord, 3 in bay 6 and 1 in bay 4. Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	209		209			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss Length = 104'-6 ¾".									
	151	2	Unpainted Steel Floor Beam	LF	96		96			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	56		56			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	23		16	7		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Pier: 16 – Reinf. Concrete (New in 1930)	Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	210	3	Reinf. Concrete Pier Wall	LF	34		26	8		
	Comments: Scaling up to 1.0 foot at the water surface at the DS nose. All 4 faces of north bearing pedestal are delaminated up to 1.0 foot. The top tier of the footing is exposed up to 1.0 feet wide and 3.0 feet tall and the bottom tier is exposed up to 1.0 foot wide and 2.0 feet tall.									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Span: P – Deck Truss (Erected in 1903, Remodeled in 1930)	Length: 77'-5" Insp. Date: 05/01/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	310		300			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	149		125	24		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss Length = 74'-8 ¼". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	149		149			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss Length = 74'-8 ¼".									
	151	2	Unpainted Steel Floor Beam	LF	72		72			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	40		40			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	16		11	5		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Pier: 17 – Reinf. Concrete (New in 1923)	Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	210	3	Reinf. Concrete Pier Wall	LF	30		30			
	Comments: Scaling up to 1 inch at the water surface at the US and DS noses. The footing is exposed at the DS (South) nose up to 1.5 feet wide and 1.5 feet tall.									
	Comments:									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Span: Q – Deck Truss (Erected in 1903)	Length: 118'-4" / Insp. Date: 04/28/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	473		473			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	231		199	32		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 115'-4 ¼". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	231		231			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 115'-4 ¼".									
	151	2	Unpainted Steel Floor Beam	LF	96		96			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	56		56			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	23		16	7		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR

Structure Number: 334

Pier: 18 – Masonry (Remodeled in 1903) | Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	211	3	Masonry Pier Wall	LF	36		32	4		
	Comments: 1 row of blocks missing just below the water surface at the US and DS noses. Section loss of block up to 1.0 foot deep at 3 feet below the water surface. The footing is exposes up to 1.0 foot wide and 1.0 foot tall. Mortar loss up to 1 foot deep from water surface to 6 feet below.									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR

Structure Number: 334

Span: R – Deck Truss (Erected in 1903) | Length: 152'-6" | Insp. Date: 04/28/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	610		610			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	300		264	36		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 150'-0". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord. Pack rust up to 3/8 inch between diagonals and vertical gusset plates at bottom chords. 1 flame cut hole in exterior top flange of north bottom chord and 1 in exterior top flange of south bottom chord in bay 7.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	300		300			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 150'-0".									
	151	2	Unpainted Steel Floor Beam	LF	108		108			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	64		64			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	26		18	8		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Pier: 19 – Masonry (Remodeled in 1903) | Insp. Date: 04/29/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	211	3	Masonry Pier Wall	LF	36		33	3		
	Comments: Section loss up to 6 inches deep 3 feet tall at the water surface at the DS nose. 1 row of blocks missing at the water surface at the US nose. The footing is exposes up to 1.0 foot wide and 1.0 foot tall. Mortar loss up to 1.0 foot deep from water surface to 6 feet below.									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR
Structure Number: 334

Span: S – Movable Deck Truss spanning Piers 19 - 21 (Erected in 1903) | Length: 204'-3 1/2" | Insp. Date: 04/28/2009

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	817		817			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	403		345	58		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 201'-3". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord. North bottom chord has 2 linear feet of pack rust up to ¼ inch between the south top flange and south web plate in the 4th bay from the east end.									
		3	Unpainted Deck Truss - Excl Bottom Chord	LF	403		403			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 201'-3".									
	151	2	Unpainted Steel Floor Beam	LF	168		168			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	102		102			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	40		40			
	Comments: Members within a plane of a bay are considered 1 brace.									
	311	2	Movable Bearing	EA	4		4			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	1		1			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

General Inspection/Maintenance Notes

The mechanism to move Span S has been removed.

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
	Insp. Date: 04/29/2009

Pier: 20 – Masonry (Remodeled in 1903)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	211	3	Masonry Pier Wall	LF	25		21	4		
	Comments: 1 block missing and other blocks with section loss up to 8 inches deep at the water surface at the DS nose. 1 block with section loss up to 6 inches deep at the water surface on the west face. Section loss up to 6 inches deep by 1 foot long by height of 1 block at the water surface at the US nose. There is a timber crib filled with stone from the stream bottom to 9 feet above the stream bottom at the US nose. There are timber forms 1 foot from the pier and up to 1 foot above the stream bottom at the DS nose. Mortar loss up to 1.0 foot deep from water surface to 6 feet below.									
	Comments:									
	Comments:									
	Comments:									
	Comments:									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
	Insp. Date: 04/28/2009

Pier: 21 – Masonry (Remodeled in 1903)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	211	3	Masonry Pier Wall	LF	27		25	2		
	Comments: The DS nose has section loss up to 6 inches deep at the water surface and up to full thickness of stone at the 1st row below the water surface. The US nose has section loss up to 8 inches deep at the water surface. Mortar loss up to 1.0 foot deep from water surface to 6 feet below. There is a concrete shelf along the west face and US nose up to 3 feet tall and 2.5 feet wide.									
	Comments:									
	Comments:									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
Length: 122'-10"	Insp. Date: 04/28/2009

Span: T – Deck Truss (Erected in 1903)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	491		40			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	241		205	36		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 120'-4". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord caused by soil and debris.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	241		241			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 120'-4".									
	151	2	Unpainted Steel Floor Beam	LF	108		108			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	64		64			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	26		16	8		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR	
Structure Number: 334	
	Insp. Date: 04/28/2009

Pier: 22 – Masonry (Remodeled in 1903)

Element Inspection (X) Check Elements Inspected — Quantity in Condition States

Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	211	3	Masonry Pier Wall	LF	27		27			
	Comments: Mortar loss up to 1.0 foot deep from water surface to 7 feet below.									
	Comments:									
	Comments:									
	Comments:									

General Inspection/Maintenance Notes

Field Notes:

Owner: Wisconsin & Southern RR										
Structure Number: 334										
Span: U – Deck Truss (Erected in 1903)						Length: 123'-8 1/2"		Insp. Date: 04/28/2009		
Element Inspection (X) Check Elements Inspected						Quantity in Condition States				
Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	31	3	Timber Deck - Bare	EA	1		1			
	Comments:									
	112	2	Unpainted Steel Stringer	LF	491		491			
	Comments:									
		3	Unpainted Truss – Bot. Chord	LF	242		206	36		
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 121'-0". Typical pitting up to 25% of inside vertical gusset plate thickness at top of bottom chord caused by soil and debris. Bottom batten plates at L7 have a hole up to 4 in by 2 in with surrounding pitting. Pack rust up to 5/8 inch between diagonals and vertical gusset plates at bottom chord.									
		3	Unpainted Deck Truss - Excl. Bottom Chord	LF	242		242			
	Comments: Follow condition states for "Unpainted Steel Elements." Truss length = 121'-0".									
	151	2	Unpainted Steel Floor Beam	LF	108		108			
	Comments:									
	171	2	Unpainted Steel Diaphragm	EA	64		64			
	Comments:									
	174	2	Unpainted Steel Lat. Bracing	EA	26		17	9		
	Comments: Members within a plane of a bay are considered 1 brace. Pack rust up to 1 inch at between transverse lower lateral bracing angles that span between lower panel points. L3 to U4 diagonal bracing, bottom horizontal has pitting with holes up to ½ inch diameter - minor.									
	311	2	Movable Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	312	2	Fixed Bearing	EA	2		2			
	Comments: Minor soil buildup with minor corrosion.									
	Comments:									

Field Notes:

Owner: Wisconsin & Southern RR										
Structure Number: 334										
Abutment: 23 – Masonry (Remodeled in 1903)								Insp. Date: 04/29/2009		
Element Inspection (X) Check Elements Inspected						Quantity in Condition States				
Ck	Elem.	Env.	Description	Unit	Total QTY	1	2	3	4	5
	217	2	Masonry Abutment	LF	20		20			
	Comments: Mortar loss up to 1.0 foot deep from water surface to the stream bottom.									
	400	2	Concrete Wingwalls	EA	2		1	1		
	Comments: South Wingwall has max penetration of 2.0 feet of mortar loss. North Wingwall has mortar loss up to 1.0 foot deep.									
	Comments:									

General Inspection/Maintenance Notes

West side of Abutment 1 (East)

East Side of Pier 2

East Side of Pier 3

East Side of Pier 4

East Side of Pier 5

East Side of Pier 6

East Side of Pier 7

East Side of Pier 8

East Side of Pier 9

East Side of Pier 10

East Side of Pier 11

East Side of Pier 12

East Side of Pier 13

East Side of Pier 14

East Side of Pier 15

East Side of Pier 16

East Side of Pier 17

East Side of Pier 18

East Side of Pier 19

East Side of Pier 20

East Side of Pier 21

East Side of Pier 22

East Side of Abutment 23 (West)

Pier 2 at south (DS) bearing has a missing block on the west face at the waterline

Pier 3 south (DS) bearing pedestal has spalling with exposed steel at the SE corner

Abutment 23 has up to 50% section loss of angles on horizontal gusset plate above SW Bearing

Typical bearing at Abutment 23 (West)

Span U batten plate at L7 on bottom of North Lower Chord has a hole

Typical pack rust up to 5/8 inch between diagonal and vertical gusset plate at lower chord

Span U NE bearing at Pier 22 has a cracked bottom plate at NE bolt.

Typical pitting up to 25 percent of thickness of vertical gusset plate at lower chord connections where rock and soil are present

Typical pitting up to 25 percent of thickness of vertical gusset plate at lower chord connections where rock and soil are present

Span S North Lower Chord in 4th bay west of Pier 20 has pack rust up to ¼ inch between the south top flange and south web plate

Typical pitting up to ¼ inch deep and 1 inch tall by 3 inches wide in FB web at stringer shelf plate in Spans D through M

Typical Interior of Deck Girder

Typical Diagonal Lateral Bracing

Typical Transverse and Diagonal Lateral Bracing

Typical Exterior of Stringer

Typical Deck Underside

Typical Stringer to Floor Beam and Floor Beam to Deck Girder Connections

Typical Interior of Bearings

Typical Interior of Bearing

Typical Floor Beam and Cross Frame at Pier

Typical Cross Frame and Top of Steel Tower at Pier

Typical Top Flange of Deck Girder

Typical Top of Deck

Typical Concrete Encased Steel Tower Pier

06.24.2009

Typical Deck Girder Span

Typical Bottom Side of Floor Beam to Deck Girder Connection

Typical Top Side of Floor Beam to Deck Girder Connection

Typical Stringer to Floor Beam Connection

Typical Stringer to Floor Beam Connection

Typical Exterior of Bearings

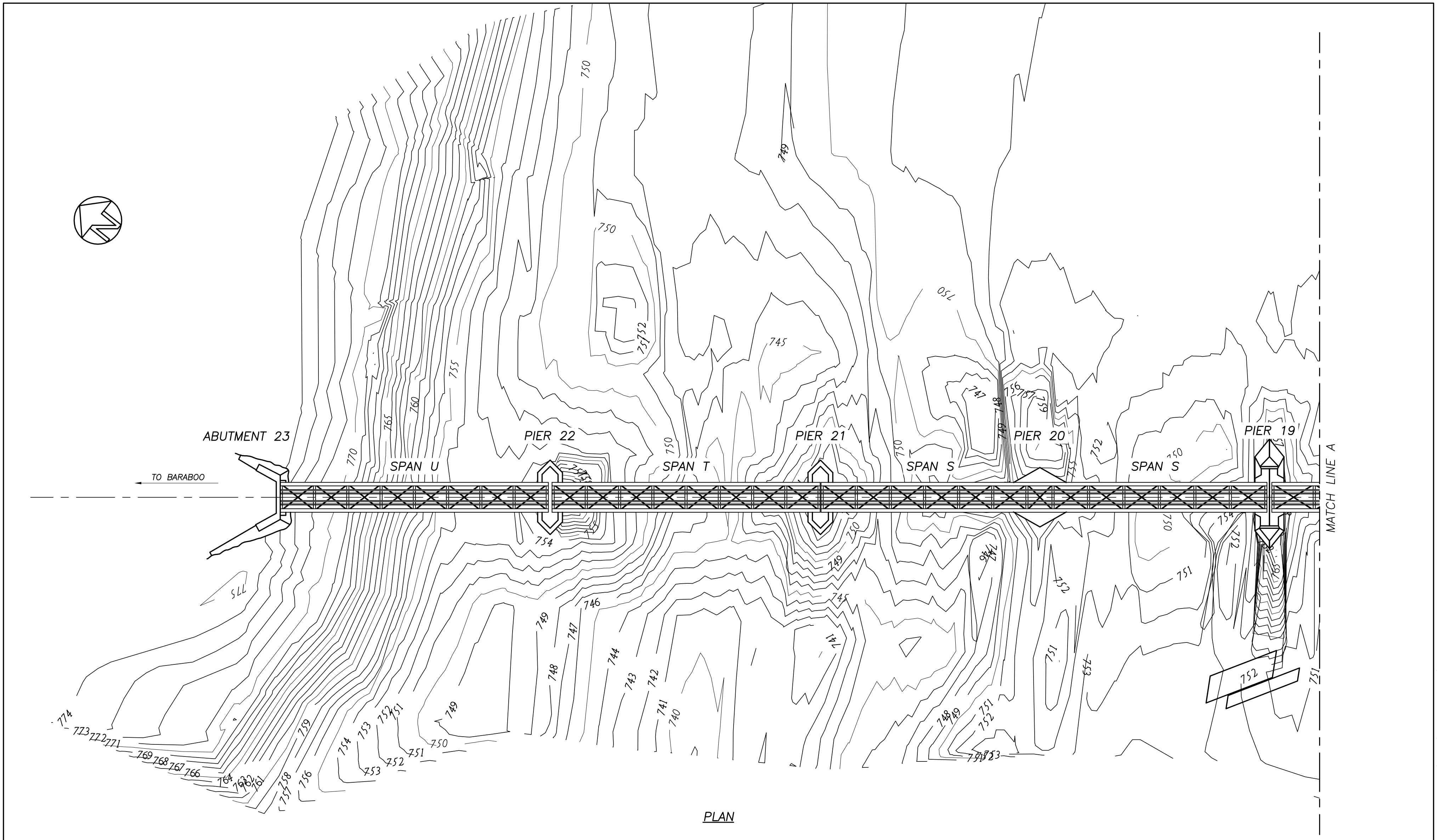
Typical Exterior of Bearing

Typical Underside of Bearing Pedestal at Top of Steel Tower

Typical Downstream (South) Profile of Deck Girder Span

Appendix C

Survey



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			SURVEYED		
			TECHNICIAN		
			CHECKED		
			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

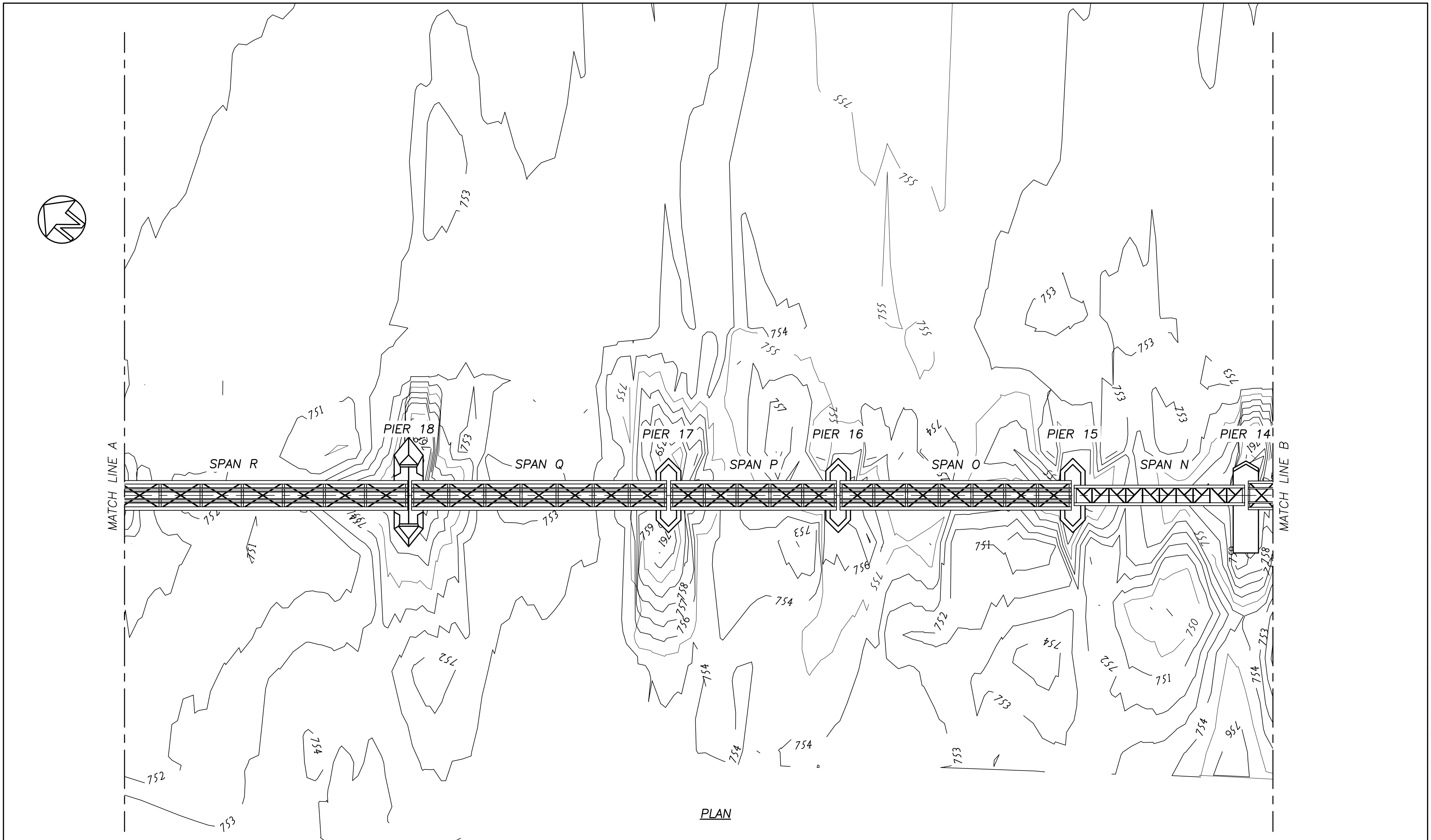
MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL BOTTOM CONTOURS (1 OF 4)

WISCONSIN & SOUTHERN RAILROAD CO.

AYRES ASSOCIATES
8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION NO. 4356

DATE: 11/2009
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 1 of 4



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			TECHNICIAN		
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			DESIGN DIVISION		
			DESIGNED		
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			CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL BOTTOM CONTOURS
(2 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES
8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION
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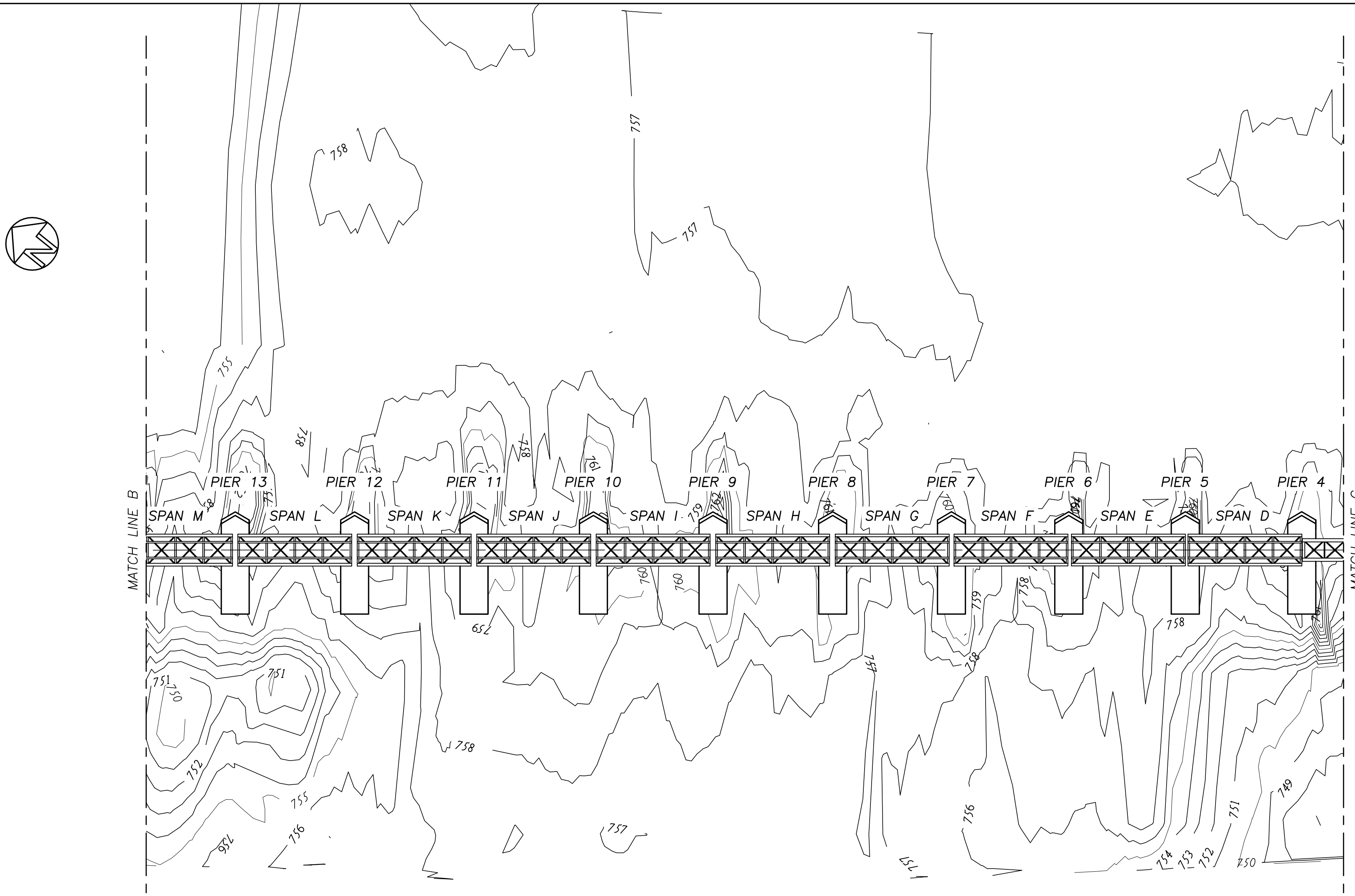

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			TECHNICIAN		
			CHECKED		
			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL BOTTOM CONTOURS
(3 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES

8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION
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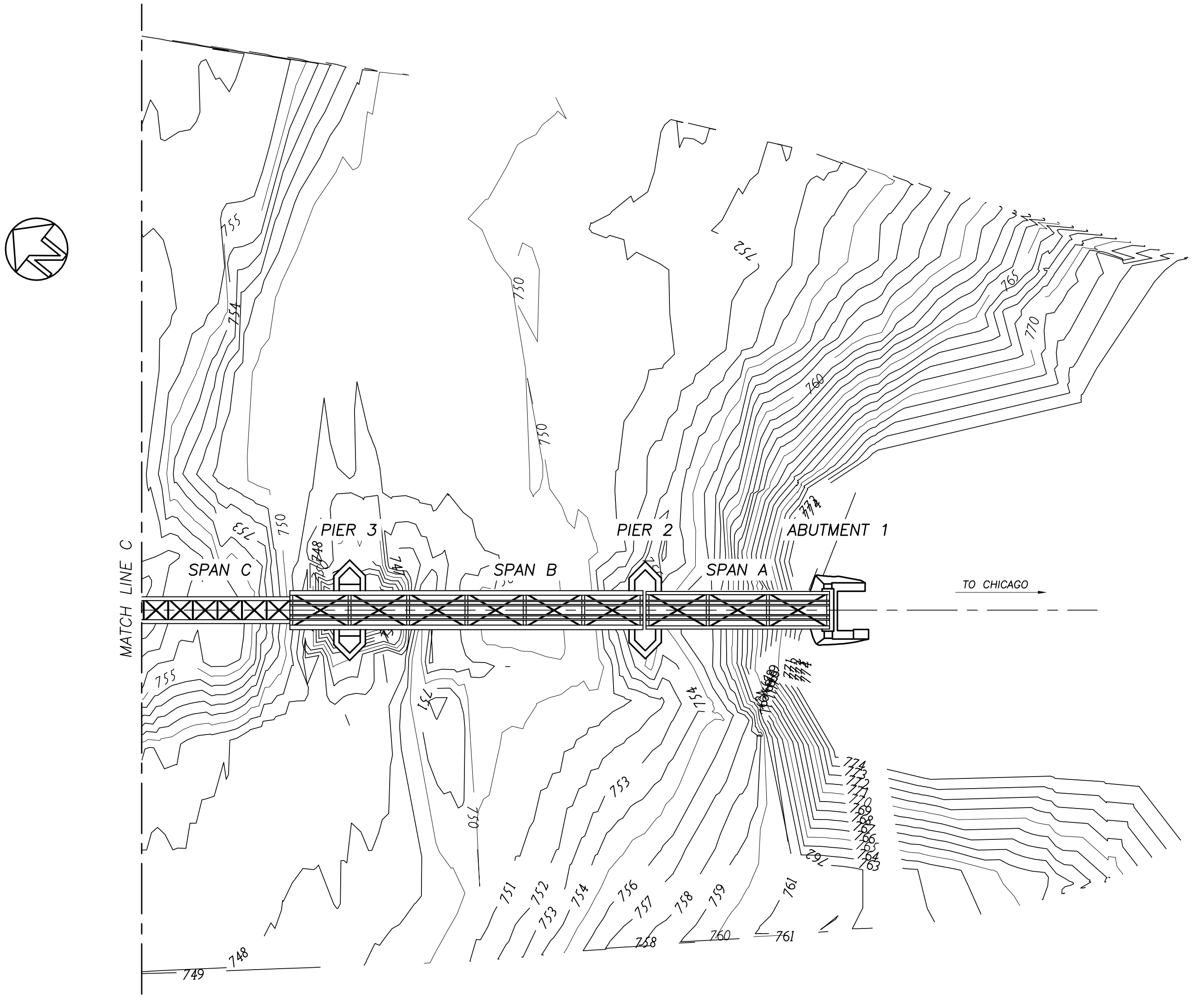
PROJECT I.D. ______
SURVEY FILE NO.:
SHEET: 3 of 4



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			DESIGN DIVISION		
			DESIGNED		
			DRAWN	IAG	11/09
			CHECKED	SML	11/09

MERRIMAC RAILROAD
BRIDGE No. 334
COMPREHENSIVE STUDY

CHANNEL BOTTOM CONTOURS
(4 OF 4)

WISCONSIN &
SOUTHERN
RAILROAD CO.

AYRES ASSOCIATES

8875 HIDDEN RIVER PARKWAY
SUITE 200
TAMPA, FLORIDA 33637
PHONE (813) 978-8688
FAX (813) 978-9369
CERTIFICATE OF AUTHORIZATION
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SURVEY FILE NO.:
SHEET: 4 of 4

Appendix D

Load Rating Calculations

(See unabridged report appendix.)

Appendix E
Cooper E80 Structural Evaluation Calculations

(See unabridged report appendix.)

Appendix F

Estimated Repair and Replacement Costs

JOB	**:**	Bridge No. 334 Rehabilitation		
DESCRIPTION	**:**	Construction Cost Estimate		
PROJECT NO.	**:**	62-0137.00		
COMPUTED BY	**:**	SML	**DATE:**	12/19/2009
CHECKED BY	**:**		**DATE:**	

Rounded Total Replacement Estimated Construction Cost - in Kind	**$48,280,000**	Deck Truss and Girders
Rounded Total Replacement Estimated Construction Cost - 110' Spans	**$34,960,000**	Deck Girders only
Rounded Total Replacement Estimated Construction Cost - 145' Spans	**$39,480,000**	Deck Girders only
Rounded Total Replacement Estimated Construction Cost - 220' Spans	**$65,270,000**	Deck Trusses only
Rounded Total Major Rehabilitation Costs	**$24,000,000**	
Rounded Total Repair Costs	**$4,200,000**	

AYRES ASSOCIATES

JOB	:	Bridge No. 334 Rehabilitation		
DESCRIPTION	:	Construction Cost Estimate		
PROJECT NO.	:	62-0137.00		
COMPUTED BY	:	SML	**DATE:**	12/19/2009
CHECKED BY	:		**DATE:**	

Complete Bridge Replacement

Superstructure Type: Replace steel spans in kind

Unit Costs

Estimated cost per linear foot, new construction of Deck Plate Girder Bridge	**$10,500 /lf**
Estimated cost per linear foot, new construction of Deck Truss Bridge	**$25,000 /lf**
Estimated cost per linear foot, 4' dia. Drilled Shafts	**$700 /lf**
Estimated cost per linear foot, 6' dia. Column with Caps	**$1,500 /lf**
Estimated cost per linear foot, demolition	**$500 /lf**

Bridge Length (Incl. Appr. Slab) (ft.)	Est. Cost per SF	Estimated Cost	
650	$10,500	$6,825,000	New Deck Girder Span Bridge Sections
1150	$22,000	$25,300,000	New Deck Truss Span Bridge Sections
3600	$700	$2,520,000	Drilled Shafts
320	$1,500	$480,000	Substructure Concrete
1800	$1,500	$2,700,000	Demolition of existing bridge
		$340,000	Approach Construction & Right-of-way

Mobilization (10%)	$3,816,500
SubTotal Estimated Construction Cost	**$41,981,500**
Contingency (15%)	$6,297,225.0
Rounded Total Estimated Construction Cost	**$48,280,000**

Notes: Estimated Construction Costs are only for the bridge.
Vertical clearance will be maintained

Complete Bridge Replacement

Superstructure Type: 16 spans of 110 ft. Deck girders only

Unit Costs

Estimated cost per linear foot, new construction of Deck Plate Girder Bridge	**$12,000 /lf**
Estimated cost per linear foot, 4' dia. Drilled Shafts	**$700 /lf**
Estimated cost per linear foot, 6' dia. Column with Caps	**$1,500 /lf**
Estimated cost per linear foot, demolition	**$500 /lf**

Bridge / Substructure Est. Lengths (ft.)	Est. Cost per SF	Estimated Cost	
1800	$12,000	$21,600,000	New Deck Girder Span Bridge Sections
3600	$700	$2,520,000	Drilled Shafts
320	$1,500	$480,000	Substructure Concrete
1800	$1,500	$2,700,000	Demolition of existing bridge
		$340,000	Approach Construction & Right-of-way

Mobilization (10%)	$2,764,000
SubTotal Estimated Construction Cost	**$30,404,000**
Contingency (15%)	$4,560,600.0
Rounded Total Estimated Construction Cost	**$34,960,000**

Notes: Estimated Construction Costs are only for the bridge.
Vertical clearance will be maintained

Complete Bridge Replacement

Superstructure Type: 12 spans of 145 ft. Deck girders only

Unit Costs

Estimated cost per linear foot, new construction of Deck Plate Girder Bridge	**$14,000 /lf**
Estimated cost per linear foot, 5' dia. Drilled Shafts	**$900 /lf**
Estimated cost per linear foot, 6' dia. Column with Caps	**$1,500 /lf**
Estimated cost per linear foot, demolition	**$500 /lf**

AYRES ASSOCIATES

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Complete Bridge Replacement

Bridge / Substructure Est. Lengths (ft.)	Est. Cost per SF	Estimated Cost	
1800	$14,000	$25,200,000	New Deck Girder Span Bridge Sections
2900	$900	$2,610,000	Drilled Shafts
240	$1,500	$360,000	Substructure Concrete
1800	$1,500	$2,700,000	Demolition of existing bridge
		$340,000	Approach Construction & Right-of-way

Mobilization (10%)	$3,121,000
SubTotal Estimated Construction Cost	**$34,331,000**
Contingency (15%)	$5,149,650.0
Rounded Total Estimated Construction Cost	**$39,480,000**

Notes: Estimated Construction Costs are only for the bridge.
Vertical clearance will be maintained

Complete Bridge Replacement

Superstructure Type: 8 spans of 220 ft. Truss spans only

Unit Costs

Estimated cost per linear foot, new construction of Deck Truss Bridge	**$25,000 /lf**
Estimated cost per linear foot, 5' dia. Drilled Shafts	**$900 /lf**
Estimated cost per linear foot, 8' dia. Column with Caps	**$2,000 /lf**
Estimated cost per linear foot, demolition	**$500 /lf**

Bridge / Substructure Est. Lengths (ft.)	Est. Cost per SF	Estimated Cost	
1800	$25,000	$45,000,000	New Deck Girder Span Bridge Sections
2600	$900	$2,340,000	Drilled Shafts
160	$2,000	$320,000	Substructure Concrete
1800	$2,000	$3,600,000	Demolition of existing bridge
		$340,000	Approach Construction & Right-of-way

Mobilization (10%)	$5,160,000
SubTotal Estimated Construction Cost	**$56,760,000**
Contingency (15%)	$8,514,000.0
Rounded Total Estimated Construction Cost	**$65,270,000**

Notes: Estimated Construction Costs are only for the bridge.
Vertical clearance will be maintained

JOB	**:**	Bridge No. 334 Rehabilitation
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COMPUTED BY	**:**	SML — **DATE:** 12/19/2009
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Major Bridge Rehabilitation

Install line of Deck Truss or Deck Girders between existing
Remove and replace all interior memebers to accommodate above

Unit Costs

Estimated cost per linear foot, new construction of Deck Plate Girder Bridge	**$1,400 /lf**
Estimated cost per linear foot, new construction of Deck Truss Bridge	**$10,000 /lf**
Estimated cost per square foot, demolition	**$25 /lf**

Bridge Length (Incl. Appr. Slab) (ft.)	Width (ft.)	Est. Cost per SF	Estimated Cost	
650	12.00	$1,400	$910,000	Deck Girder Span Line
1150	12.00	$10,000	$11,500,000	Deck Truss Span Line
1800	12.00	$150	$3,240,000	Selective Demolition of existing sections of bridge

Mobilization (10%)	$1,565,000
Major Rehabilitation Costs	**$17,215,000**
Contingency (15%)	$2,582,250.0
Repairs	$4,200,000.0
Rounded Total Major Rehabilitation Costs	**$24,000,000**

Notes: Estimated Construction Costs are only for the bridge.
Vertical clearance will be maintained

AYRES ASSOCIATES

JOB	**:**	Bridge No. 334 Rehabilitation		
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PROJECT NO.	**:**	62-0137.00		
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Bridge Repairs

	No. of Units / quantity	Est. Cost per Unit	Estimated Cost
Deck Timber Tie Replacement:			
Demolition, Tie removal	1728	$30	$51,840
Tie Installartion	1728	$850	$1,468,800
Bottom Chord Pack Rust Repair:			
Demolition, rivet removal	1000	$100	$100,000
Steel Repair (Est. at 20 repairs for a total of 5 ft ea = 100 lf)	100	$5,000	$500,000
Stone Masonry Replacement:			
Dewatering system	4	$4,000	$16,000
Stone Replacement (4 locations)	4	$15,000	$60,000
Masonry Pointing:			
Dewatering system	7	$4,000	$28,000
Mortar Repair (Avg.= 2.0 CF per location, 7)	14	$400	$5,600
Abrasion Repair:			
Dewatering system	15	$4,000	$60,000
Concrete Repair (Avg = 80 cf per ft. per location, 15)	1200	$500	$600,000
Beam Seat Spall Repair:			
Spall Repair (2 locations, 1.5 CF per location)	3	$1,000	$3,000
Bearing Repair:			
Replace Bearing (inlcudes Jacking)	1	$30,000	$30,000
Scour Repair:			
Install Rip Rap (Estimate 15 ton per pier for 15 piers)	150	$300	$45,000
Repair Costs Only			**$2,968,240**
Mobilization (10%)			$296,824
Total Repair Costs			**$3,650,935**
Contingency (15%)			
Rounded Total Repair Costs			**$4,200,000**