STRUCTURAL EVALUATION REPORT FOR LOWER DECK BIKE PATH

CUYAHOGA COUNTY, OHIO

THE DETROIT-SUPERIOR BRIDGE

BR#: CUY-6-1456
SFN: 1800930

CLEVELAND URBAN DESIGN COLLABORATIVE

Report Submitted: April 4, 2014

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CUYAHOGA COUNTY, OHIO

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EXECUTIVE SUMMARY

GENERAL

The Detroit-Superior Bridge (CUY-6-1456), located in downtown Cleveland, Ohio, carries US Route 6 over the Cuyahoga River, numerous local streets including the Center Street Swing Bridge, surface parking lots, and RTA railroad tracks. The bridge is approximately 2,880 feet long, consisting of a steel main span and numerous concrete approach spans and tunnels (see Photo 1). The bridge was originally designed as a double-deck structure, supporting vehicular and pedestrian traffic on the top roadway surface and street railway traffic on the lower deck. Use of the lower deck for streetcars has since been abandoned, and the space is occasionally opened to the public for various events by the City of Cleveland.

Photo 1 – South elevation of the Detroit-Superior Bridge.

Constructed by the King Bridge Company, the structure was the largest steel and reinforced concrete bridge in the world at the time of completion in 1918. Since then, the bridge has received numerous modifications and rehabilitations, including major work in 1967 and 1994. In addition, a large pedestrian sidewalk was added on the north side of the bridge in 2003. Significant modifications during the major rehabilitations include strengthening or replacing deteriorated steel and concrete, structural steel painting in Span 4, updating safety features, and improving the drainage system.

TranSystems has been contracted by the Cleveland Urban Design Collaborative to provide professional engineering services and a structural evaluation report for a proposed shared use pedestrian and bike pathway on the lower deck of the Detroit-Superior Bridge. This investigation is part of the Detroit-Superior Bridge Project Connectivity plan, an initiative that aims to re-open the former streetcar level of the structure as a public thoroughfare for cyclists and pedestrians, as well as a venue for various events, programming and recreation.

DESCRIPTION OF ALTERNATIVES

The project team performed a structural evaluation of two different alternatives for the proposed lower decks, as described below:

- Alternative #1: This option includes replacement of the steel open grid deck with a reinforced concrete deck within the same footprint (19'-0" wide) with minor structural upgrades (see Figure 1 and Appendix A). Weights for concrete railing and vandal protection fences are included in the analysis. The deck configuration constitutes a path that is confined to the area between interior concrete column lines in the concrete approach spans and between truss lines in the steel main span. In this alternative, the fiberglass reinforced grating will remain on the lower floorbeam cantilevers and will not be included in the bike path area.

Figure 1 – Conceptual rendering of Alternative #1.

- Alternative #2: This option includes evaluation of a shared bike and pedestrian path that includes the full width of the lower deck (80'-0" wide) with minor structural upgrades (see Figure 2 and Appendix B). The open grid steel deck and both fiberglass reinforced pedestrian walkways will be removed in the steel arch main span. A reinforced concrete deck will be designed to cover the entire footprint of the lower deck in the main span. In addition, a reinforced concrete deck will be evaluated for the concrete arch spans between the northern exterior and northern interior column lines, as this section of deck was removed in 1994.

Figure 2 – Conceptual rendering of Alternative #2.

AS-INSPECTED CONDITIONS

The Detroit-Superior Bridge is in Poor Condition [4-NBIS] overall, due to deterioration noted throughout the structure. The Main Truss Span lower chord members and gusset plates exhibit advanced section loss. This loss is typically concentrated to the areas below the upper level deck, occurring at the last five panel points at each end of the span. In addition, isolated framing elements exhibit advanced section loss due to water infiltration at locations were the trusses or eyebar hangers pass through the deck, and at stringer saddle bearings (see Photo 2). Framing elements at the west and east ends of the Main Truss Span also exhibit heavy section loss due to water infiltration at the joints.

Photo 2 – Typical lower deck stringer with 100% section loss in web and bottom flange at saddle bearing.

Photo 3 – Typical spall with exposed reinforcement in concrete column.

The reinforced concrete approach spans exhibit numerous deficiencies throughout. Concrete floorbeams, columns, and jack arches commonly exhibit minor to moderate spalls with exposed reinforcing (see Photo 3). The concrete arch ribs commonly exhibit areas of spalled or delaminated concrete, as well as longitudinal and map cracking.

2014 REHABILITATION

The Detroit-Superior Bridge (CUY-6-1456) is scheduled to begin a major structural rehabilitation project during the upcoming 2014 and 2015 construction seasons. The design plans for this project (PID 77040) were completed by TranSystems for the Ohio Department of Transportation in 2013, with work scheduled to begin in April 2014.

The scope of work for this project as described in the plans includes patching of the upper deck wearing surface and concrete substructure and superstructure, concrete corbel replacement (see Figure 3), steel arch span zone painting, steel floorbeam web retrofits, drainage and sinkhole repairs, and inspection access improvements. Many of the spalled and delaminated concrete areas as described in this report will be repaired as part of this rehabilitation contract. The concrete repairs have been prioritized in order to ensure that members exhibiting major deficiencies or public safety risks are addressed. Only minor steel repairs and zone painting are being performed in Span 4 under this project.

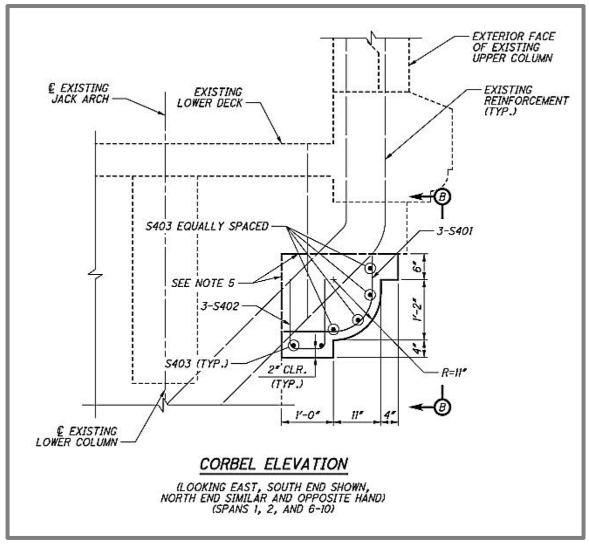


Figure 3 – Typical concrete corbel replacement detail from the 2014 Rehabilitation Plans.

LOAD RATING ANALYSIS

The proposed lower decks in each alternative were analyzed with a pedestrian load of 75 psf over the area between the bridge railings, which is considered to act simultaneously with the vehicular and lane loadings in effect on the upper deck. The bridge superstructure was analyzed for HS20-44 truck and lane loading for inventory and operating levels, and for the operating levels of the Ohio Legal Loads 2F1, 3F1, 4F1 and 5C1. In addition, the effect of two truck trains, one composed of a series of HS20-44 trucks and one of Ohio 5C1 trucks, were considered at the operating level for the steel aches, which qualify as "long span." All applicable loadings are to be considered in combination in accordance to provisions in the ODOT Bridge Design Manual (BDM).

Structural components were analyzed and load rating capacities were calculated using a combination of hand calculations, spreadsheets, and various finite element software. Capacities and dead loads were calculated by hand and by using Microsoft Excel workbooks. Maximum live load effects were found utilizing STAAD.Pro V8i or hand calculations. Impact and multiple presence factors were applied to the live loads in accordance with AASHTO 3.8.2 and 3.12.1. The load rating formulas were applied inside of Excel workbooks. All capacities and loads were generated based upon Load Factor Rating.

The as-inspected rating factors for each section of the bridge for Alternative #1 are shown in Table 1.

ALTERNATIVE #1: AS-INSPECTED SUMMARY CONTROLLING RATING FACTORS											
Location		HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck	5C1 Truck	Pedestrian Inv	Pedestrian Oper
Deck - All Sections		1.20	2.00	3.20	2.81	3.10	3.15				
Concrete Spans	Arch Ribs	2.15	3.58	6.35	4.33	3.95	4.54				
	Columns	2.21	3.69	5.01	3.62	3.49	3.26				
	Upper Floorbeams	0.79	1.31	1.72	1.51	1.34	1.53				
	Lower Floorbeams									5.16	8.62
	Jack Arches	1.12	1.86	2.98	2.63	3.03	2.63				
Steel Main Span	Truss Members	0.58	0.73	5.48	3.61	3.12	2.18	0.77	0.96		
	Pins and Hangers	0.96	1.31	2.50	1.69	1.51	1.73				
	Upper Floorbeams	0.93	1.44	2.79	1.83	1.62	1.91				
	Upper Stringers	0.88	1.47	2.15	1.56	1.46	1.66				
	Lower Floorbeams									5.01	8.36
	Lower Stringers									2.84	4.74
	Gusset Plates	1.58	1.97	5.30	3.50	3.02	2.33				

Table 1 – Controlling as-inspected rating factors for CUY-6-1456 under Alternative #1 (numbers below 1.0 are red).

The load rating analysis for Alternative #1 is controlled by the main truss members in Span 4, with a governing HS20 Inventory rating factor of 0.58 and Operating rating factor of 0.73. Nine (9) lower chord members exhibit operating rating factors below 1.10 for this alternative, and these members would require structural rehabilitation in order to accommodate the additional loads proposed by Alternative #1. All other members throughout the Detroit-Superior Bridge have operating rating factors above 1.10 for this option.

The as-inspected rating factors for each section of the structure for Alternative #2 are shown in Table 2.

ALTERNATIVE #2: AS-INSPECTED SUMMARY CONTROLLING RATING FACTORS											
Location		HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck	5C1 Truck	Pedestrian Inv	Pedestrian Oper
Deck - All Sections		1.20	2.00	3.20	2.81	3.10	3.15				
Concrete Spans	Arch Ribs	2.12	3.54	6.19	4.24	3.95	4.51				
	Columns	2.21	3.69	5.01	3.62	3.49	3.26				
	Upper Floorbeams	0.79	1.31	1.72	1.51	1.34	1.53				
	Lower Floorbeams									5.16	8.62
	Jack Arches	1.12	1.86	2.98	2.63	3.03	2.63				
Steel Main Span	Truss Members	0	0	0	0	0	0	0	0		
	Pins and Hangers	0.65	0.82	1.56	1.05	0.94	1.08				
	Upper Floorbeams	0.93	1.44	2.79	1.83	1.62	1.91				
	Upper Stringers	0.88	1.47	2.15	1.56	1.46	1.66				
	Lower Floorbeams									3.79	6.33
	Lower Stringers									2.95	4.92
	Gusset Plates	0	0	0	0	0	0				

Table 2 – Controlling as-inspected rating factors for CUY-6-1456 under Alternative #2 (numbers below 1.0 are red).

The load rating analysis for Alternative #2 is controlled by the main truss members and gusset plates in Span 4. Specifically, thirty-one (31) lower chord truss members, twenty-four (24) upper pins and eight (8) gusset plate locations exhibit operating rating factors below 1.10, indicating that structural rehabilitation would be required on these members in order to accommodate the loads proposed in this option. Furthermore, the majority of these members have operating rating factors below 1.10 in the as-built condition, with no measureable section loss noted on many of these members. Accordingly, the rehabilitation efforts on these members would be significant, as the members would need to be strengthened beyond their original capacity.

Seventeen (17) of the deficient lower chord members and two (2) gusset plates do not have sufficient capacity to support the dead and pedestrian loads proposed by Alternative #2. This results in members that do not possess any residual capacity with which to support the live loads. Accordingly, these members were reported to have rating factors equal to zero (0). Substantial rehabilitation efforts would be needed on these members in order to ensure the structural integrity of the bridge under Alternative #2.

COST ANALYSIS

Based on the results of the load rating analysis, the project team was able to develop conceptual cost estimates for each alternative based on the proposed lower deck modifications and necessary levels of structural rehabilitation in order to accommodate such replacements. The preliminary evaluation and analysis of each alternative was performed to determine base costs for a planning-level cost estimate, as well as the advantages and disadvantages provided by each alternative. Note that these conceptual estimates only cover the costs associated with the structural rehabilitation of the existing bridge and installation of the reinforced concrete lower deck. Additional costs such as lighting, earthwork, overlooks, aesthetic improvements, benches, signage, safety features and other amenities were not included.

Under Alternative #1, the existing lower deck of the Detroit-Superior Bridge would upgraded to provide a more suitable riding surface and safer conditions for pedestrians and bicyclists on the structure. This lower deck modification would occur within the same footprint as the existing 19'-0" wide lower deck. According to the load rating analysis, select lower chord members would be structurally deficient in their as-inspected condition as a result of the additional loads being applied to the structure. Structural rehabilitation on these members would be generally minor and would occur below the main deck. No maintenance of traffic will be required for this alternative.

Under Alternative #2, a full width reinforced concrete deck would be provided on the lower deck of the Detroit-Superior Bridge. This option would include removal of the steel open grid deck and both pedestrian walkways in the steel main span, as well as an additional reinforced concrete deck in the north bay of the concrete approach spans. According to the load rating analysis, numerous components in the steel main span would be structurally deficient under the loads proposed in this alternative. Structural rehabilitation efforts would include 31 lower chord members, 24 upper pins and 8 gusset plate locations. Maintenance of traffic will be required for this alternative, as much of the work would occur over the main deck.

The alternatives and their associated construction cost estimates are as follows:

Alternative Description	2014 Cost	2019 Cost*
Alternative 1 - Partial Width Deck	$1,696,000	$1,967,000
Alternative 2 - Full Width Deck	$6,148,000	$7,128,000

*Cost based on 3% annual inflation

INTRODUCTION

TranSystems has been contracted by the Cleveland Urban Design Collaborative to provide professional engineering services and a structural evaluation report for a proposed shared use pedestrian and bike pathway on the lower deck of the Detroit-Superior Bridge (see Location Map). This investigation is part of the Detroit-Superior Bridge Project Connectivity plan, an initiative that aims to re-open the former streetcar level of the structure as a public thoroughfare for cyclists and pedestrians, as well as a venue for various events, programming and recreation. Use of the lower deck for streetcars was abandoned in 1955, but this level of the structure has occasionally served as a multi-functional space for the City of Cleveland in recent years.

Location Map

The scope of services for this structural investigation project includes load rating analysis of the CUY-6-1456 (Detroit-Superior) Bridge for two proposed design alternatives, as well as recommendations and a conceptual cost analysis comparing the two options. Each alternative will include removal of the plywood-covered, existing 5" steel open grid deck and replacement with a 6" thick reinforced concrete deck. Standard bridge parapets and 6'-0" vandal protection fences will be included for each alternative. This design configuration is being investigated in order to provide a worst case loading condition for analysis, as well as a basic price point. Lighter deck designs could also be feasible for this application but would come at a higher cost.

- Alternative #1: This option includes replacement of the steel open grid deck with a reinforced concrete deck within the same footprint (19'-0" wide) (see Appendix A). Weights for concrete railing and vandal protection fences are included in the analysis. The deck configuration constitutes a path that is confined to the area between interior concrete column lines in the concrete approach spans and between truss lines in the steel main span. In this alternative, the fiberglass reinforced grating will remain on the lower floorbeam cantilevers and will not be included in the bike path area.

- Alternative #2: This option includes evaluation of a shared bike and pedestrian path that includes the full width of the lower deck (80'-0" wide) (see Appendix B). The open grid steel deck and both fiberglass reinforced pedestrian walkways will be removed in the steel arch main span. A reinforced concrete deck will be designed to cover the entire footprint of the lower deck in the main span. In addition, a reinforced concrete deck will be evaluated for the concrete arch spans between the northern exterior and northern interior column lines, as this section of deck was removed in 1994.

The bridge superstructure components were rated utilizing the following specifications and documents:

- ODOT Bridge Design Manual, 2004 Edition
- AASHTO Standard Specifications for Highway Bridges, 17th Edition, 2002
- FHWA Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates In Truss Bridges, February 2009
- AASHTO Manual for Bridge Evaluation, 2nd Edition, 2011
- AASHTO Guide Specifications for Fatigue Evaluation of Existing Steel Bridges, 1990
- Transportation Research Board Record 1814, Paper No. 02-2889 "Numerical Load Rating of Reinforced Concrete Compression Members"

The updated load rating analysis is based on the *"2012 Load Rating Report of Detroit-Superior Bridge"* performed by TranSystems and submitted to the Ohio Department of Transportation (ODOT) on January 25, 2013. The technical assumptions for this analysis are consistent with the methods from this previous load rating, with loads updated in accordance with the proposed lower deck modification alternatives.

The proposed lower decks in each alternative were analyzed with a pedestrian load of 75 psf over the area between the bridge railings, which is considered to act simultaneously with the vehicular and lane loadings in effect on the upper deck. The bridge superstructure was analyzed for HS20-44 truck and lane loading for inventory and operating levels, and for the operating levels of the Ohio Legal Loads 2F1, 3F1, 4F1 and 5C1. In addition, the effect of two truck trains, one composed of a series of HS20-44 trucks and one of Ohio 5C1 trucks, were considered at the operating level for the steel aches, which qualify as "long span." All applicable loadings are to be considered in combination in accordance to provisions in the ODOT Bridge Design Manual (BDM).

The structure has undergone a number of rehabilitations and modifications since its initial erection from 1914 to 1918, including the addition of a large pedestrian sidewalk on the north side of the bridge. The As-Built analysis utilizes the members currently in place without section loss from the following list of available drawings:

- 1912 Original Design Drawings
- 1913-1916 Original Shop Drawings
- 1965-1967 Rehabilitation Plans/Shop Drawings
- 1994/2000 Rehabilitation/As-Built Plans
- 2003 Sidewalk Addition Plans

The as-inspected analysis applies the section losses noted during the following TranSystems bridge inspections:

- 2011 Routine Inspection
- 2012 In-Depth Inspection

Structural components were analyzed and load rating capacities were calculated using a combination of hand calculations, spreadsheets, and finite element software. Capacities and dead loads were calculated by hand and using Excel workbooks. Maximum live load effects (moments and shears) were found utilizing STAAD.Pro V8i in conjunction with hand calculations. The load rating formulas were applied inside of Excel workbooks. The steel truss loads were calculated through a combination of hand calculations and Excel workbooks, and these loads were applied to the three-dimensional models in STAAD. The load effect outputs were inserted into customized Excel workbooks which calculated the member capacities and load rating factors.

In addition, the gusset plate load rating was generated by inputting truss forces into a modified version of the 2009 ODOT Rating Excel workbook provided by the Office of Structural Engineering. This workbook is based upon the methods described in FHWA "Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges" (FHWA-IF-09-014).

BRIDGE DESCRIPTION

The CUY-6-1456 (Detroit-Superior) Bridge carries four lanes of US Route 6 approximately 2,880 feet over numerous local streets including the Center Street Swing Bridge, surface parking lots, RTA railroad tracks, and the Cuyahoga River. The bridge was designed as a double-deck structure, carrying vehicular and pedestrian traffic on the upper deck and street railway traffic on the lower deck. Use of the lower deck for streetcars was abandoned in 1955, but now serves as a multi-functional space for the City of Cleveland.

The Detroit-Superior Bridge has received several rehabilitations and modifications from original design during its service life, including major work in 1967 and 1994, as well as the addition of an expanded north sidewalk in 2003. Key rehabilitation efforts include, but are not limited to, updating safety features, improving the drainage system, and strengthening or replacing deteriorated steel and concrete sections.

The Detroit-Superior Bridge consists of three distinct sections, including:

Section I	–	West Approach (Reinforced Concrete Spans) (See Figure 4)
Section II	–	Trussed, Three-Hinged Through-Arch Main Span (See Figures 4 and 5)
Section III	–	East Approach (Reinforced Concrete Spans) (See Figure 6)

Section I – West Approach (West Tunnels, West Approach Spans, and Spans 1A through 3)
The West Approach section consists of double-deck reinforced concrete open-spandrel arches, two cellular spans, and two tunnel sections. Typical approach sections consist of concrete arch ribs supporting open spandrel columns with jack arches and floorbeams at the both deck levels. The tunnel sections below Detroit Avenue and West 25th Street utilize similar column/jack arch constriction. The cellular construction spans are located in Spans 1A and 1B and consist of reinforced concrete walls supporting the upper deck columns above.

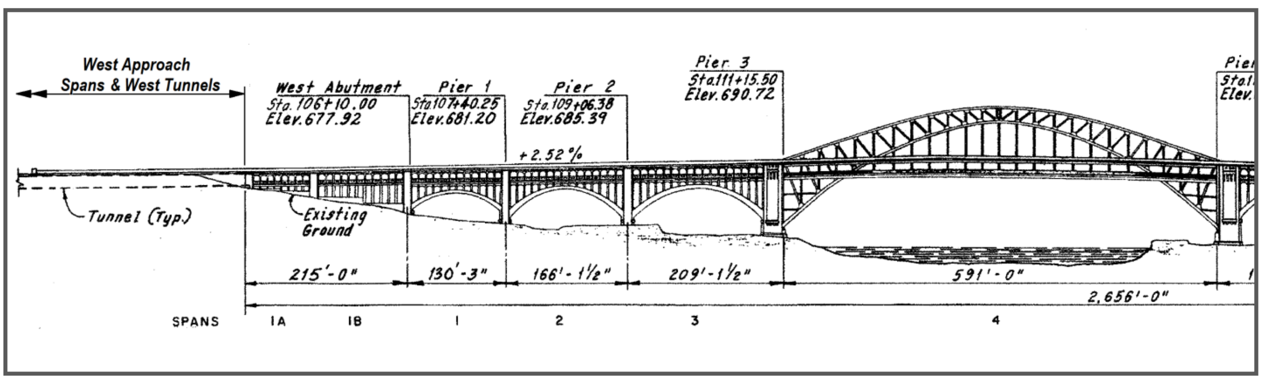


Figure 4 – Partial south elevation of CUY-6-1456 (Span 1A through Span 4 shown).

Section II – Main Truss Span (Span 4)
Span 4 is a steel, 591 foot long three-hinged, trussed arch (Pratt design). The lower chord is pin connected to hangers (eyebars) from panel points 4 to 4' where the decks are below the arch. Members from Panels 0 to 3 and 3' to 0' are framed directly into the arch lower chord. Both deck levels consist of a stringer-floorbeam system with cantilevered brackets. The upper deck in Span 4 was replaced during the 1994 Rehabilitation and consists of an 8" slab. A wide sidewalk was added on the north side of the upper deck in 2003.

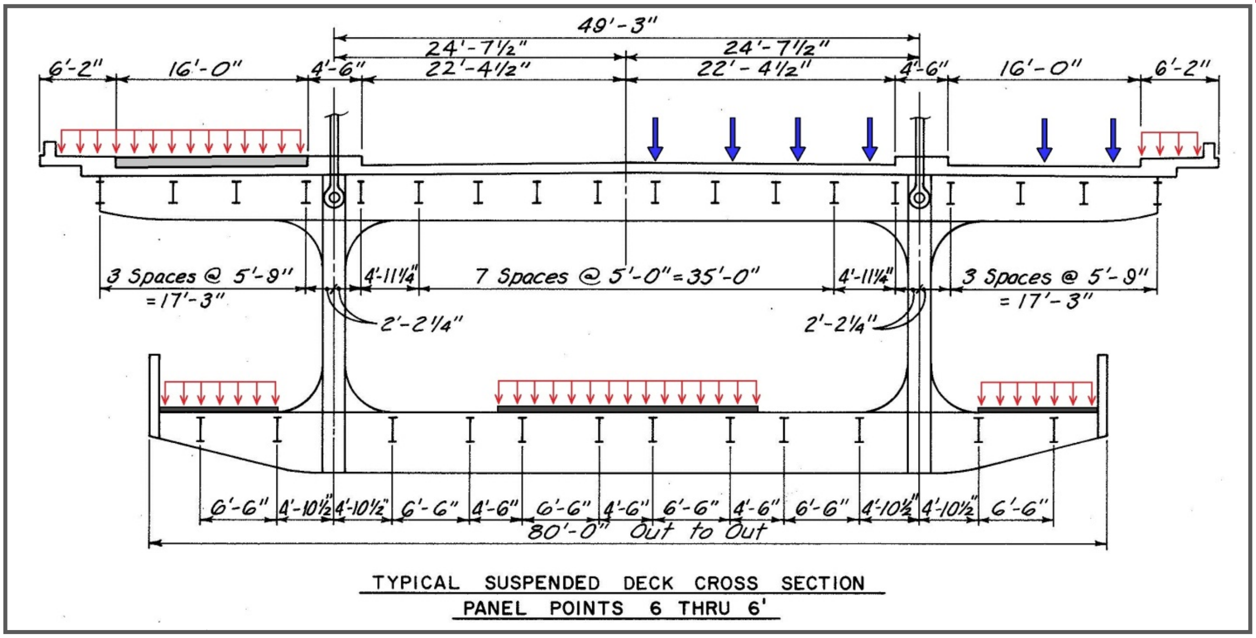


Figure 5 – Typical cross section in steel main span, looking east. Pedestrian loading is shown in red, and truck wheel locations are in blue (governing South Truss truck positions shown, North Truss similar but not shown for clarity).

Section III – East Approach (Spans 5 through 13)

The East Approach spans consist of double-deck reinforced concrete open-spandrel arches with jack arch and floorbeam framing similar to Spans 1 through 3 of the West Approach. Span 12 is unique in that the lower deck is supported by reinforced concrete hangers, thus making the span a through-arch system. The East Approach is comprised of an open, framed system consisting of jack arches and floorbeams.

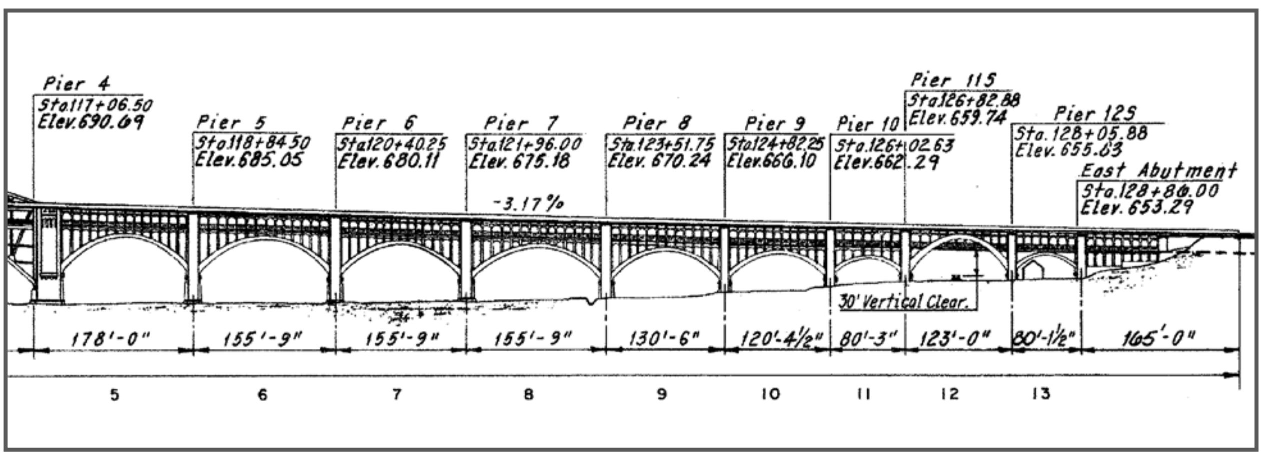


Figure 6 – Partial south elevation of CUY-6-1456 (Span 5 through Span 13 shown).

GENERAL LOADING AND RATING ANALYSIS ASSUMPTIONS

All capacities and loads were generated based upon Load Factor Rating (LFR) methodology. The primary load carrying members were analyzed with the AASHTO HS20-44 truck and lane loads and ODOT 2F1, 3F1, 4F1 and 5C1 trucks (See Figure 7). In addition, in the main truss span which is longer than 200 feet, two truck trains, one consisting of HS20-44 trucks and one consisting of 5C1 trucks, were utilized. The train length was varied to maximize load effects on individual members. Multiple loaded lanes were applied along with impact factors in accordance with AASHTO. Vehicular loading was applied to the upper deck only, while pedestrian loading was applied on all existing and proposed lower deck access walkways and upper deck sidewalks.

	Load Designation	Load Configuration	Gross Weight
AASHTO	HS20-44	8 k, 32 k, 32 k; 14', varies 14' to 30'	36 Tons
OHIO LEGAL LOADS	2F1	10 k, 20 k; 10'	15 Tons
	3F1	12 k, 17 k, 17 k; 10', 4'	23 Tons
	4F1	12 k, 14 k, 14 k, 14 k; 10', 4', 4'	27 Tons
	5C1	12 k, 17 k, 17 k, 17 k, 17 k; 12', 4', 31', 4'	40 Tons

Figure 7 – AASHTO Truck Load and Ohio Legal Loads

The material properties used in the original construction and major rehabilitations are shown in Figure 8. Note that the yield stress of nickel steel used during original construction was not specified directly, but was calculated based upon the allowable stresses provided in the original shop drawings.

Material Properties	Original Plans	1965 Rehabilitation	1994 Rehabilitation	2003 Sidewalk Addition	Weight
Structural Steel - F_y (ksi)	30.0 (carbon)	36.0 (A36)	36.0 (A36)	50.0 (A572)	490 pcf
	42.0 (nickel)	50.0 (A441)	50.0 (A572)		490 pcf
Reinforcing Steel - F_y (ksi)	32.0	40.0	60.0	60.0	490 pcf
Lightweight Concrete - f'_c (ksi)	---	---	4.5	4.5	113-117 pcf
Normal Weight Concrete - f'_c (ksi)	2.0	4.0	4.5	4.5	150 pcf

Figure 8 – Material properties.

GENERAL DESCRIPTION OF LOAD RATING ANALYSIS

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load ratings require engineering evaluation in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. A rating factor of less than 1.0 indicates that the structure does not have sufficient capacity to carry the specified loading. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection.

The Inventory Rating (Inv) generally corresponds to the customary design level of stresses, but reflects the existing bridge and material conditions with regard to deterioration and loss of section. Load ratings based on the Inventory level allow comparisons with the capacity for new structures and, therefore, result in a live load which can safely utilize an existing structure for an indefinite period of time.

Load ratings based on the Operating Rating (Oper) level generally describe the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at the Operating level may shorten the life of the bridge.

The Load Factor Rating method was used to rate all primary members of the bridge. The LFR method is based on analyzing a structure subject to multiples of the actual loads (factored loads). Different factors are applied to each type of load, which reflect the uncertainty inherent in the load calculations. The rating is determined such that the effect of the factored loads does not exceed the strength of the member (See Figure 9).

$$Rating\ Factor = \frac{Capacity - A_1 \times (Dead\ Load)}{A_2 \times (Live\ Load + Impact)}$$

$$Rating\ Factor = \frac{Capacity - A_1 \times (Dead\ Load + Pedestrian)}{A_2 \times (Live\ Load + Impact)}$$

Figure 9 – Primary load rating equations with and without pedestrian load applied.

For both Inventory and Operating ratings, a factor of 1.3 is applied to all dead loads. The Inventory rating includes multiple loaded lanes and utilizes a factor of 2.17 on all live loadings, while this factor is reduced to 1.625 if live load vehicles and pedestrian loads are considered simultaneously. For Operating ratings, this factor is decreased to 1.3 independent of whether pedestrian load is applied in tandem with the live loading. Some members on the structure are subject to pedestrian loads only, and a modified rating factor equation was developed in order to account for the lack of vehicular loading in these locations (See Figure 10).

$$Rating\ Factor = \frac{Capacity - A_1 \times (Dead\ Load)}{A_2 \times (Pedestrian\ Load)}$$

Figure 10 – Modified load rating equation for pedestrian load only.

STRUCTURE RATINGS

DECK – ALL SECTIONS

The top concrete deck was fully replaced during both major rehabilitations, including widening the steel main span deck in 1965 to allow for traffic on the north and south cantilevers. The deck was once again replaced in 1994, and an additional large pedestrian sidewalk was added on the north side of the structure in 2003. The deck in the steel main span consists of lightweight concrete with an 8" thick slab. The concrete approach spans have normal weight concrete decks that are 8 3/4" thick.

The concrete deck was analyzed as continuous in each of the sections. A concrete 1" thick wearing surface was considered for dead load purposes but neglected in the deck capacity. The lightweight concrete deck in the steel main span was considered using AASHTO Case A, as longitudinal stringers support a transverse deck with primary reinforcement perpendicular to traffic. The live loads in this section were determined by hand with the equations provided in AASHTO. The remaining concrete spans were analyzed in accordance with AASHTO Case B, with longitudinal main reinforcing spanning between transverse floorbeams. The live loads for these sections were based upon a series of simple, continuous span, two-dimensional STAAD models. The deck rates above 1.0 for inventory and operating load cases in each span (see Table 3).

AS-INSPECTED DECK CONTROLLING RATING FACTORS							
Location		HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
West Approach	Spans 1A & 1B	1.47	2.45	3.78	3.02	3.20	3.41
	Spans 1 - 3	1.37	2.28	3.60	2.81	3.10	3.15
Main Span	Span 4	1.20	2.00	3.20	3.76	4.57	3.76
East Approach	Spans 5 - 13	1.37	2.28	3.60	2.81	3.10	3.15

Table 3 – Controlling as-inspected concrete deck rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

Photo 4 – Typical view of deck underside looking south at joint location between Spans 7 and 8.

Overall the condition rating of the concrete deck is controlled by West Approach station and tunnel sections of the structure, which were not included in the scope of this load rating. The deck components in the remaining spans are without major deficiencies (see Photo 4), with minor cracking in the deck floor in isolated locations. As a result, the as-inspected load ratings are governed by the as-built condition.

The deck load rating is not affected by the proposed lower deck modifications because the upper deck sections are only subject to applied vehicular loads and the upper deck sidewalks. As a result, the load ratings for each alternative are equal to those presented in the 2012 load rating performed by TranSystems.

CONCRETE APPROACH SPANS

The concrete approach spans are comprised of three distinct sections. West Approach Spans 1A and 1B are cellular units with columns and walls supporting transverse floorbeams and longitudinal jack arches. West Approach Spans 1 through 3 consist of four concrete arch ribs supporting open spandrel columns with jack arches and transverse floorbeams at both deck levels. The East Approach Spans 5 through 13 are similar double-deck reinforced concrete open spandrel arches with jack arch and floorbeam framing members. However, Span 12 is unique in that the lower deck is supported by reinforced concrete hangers, resulting in a through-arch system.

The lower deck of the concrete approach spans consists of four column lines with three bays. The lower deck in the northernmost bay was removed during the 1994 Rehabilitation, although floorbeams are still present in this area. The existing riding surface in the concrete approach spans is generally adequate for bicycle and pedestrian traffic. As a result, no structural modifications would be necessary in the concrete approach spans for Alternative #1. However, a deck would need to be added to the northernmost bay in order to achieve Alternative #2. Therefore, no additional load rating analysis was necessary for Alternative #1 because the ratings are equal to those from the 2012 load rating. Select members were reanalyzed for Alternative #2 due to the addition of concrete deck in the northernmost bay and the subsequent pedestrian load in this area.

Arch Ribs

West Approach Spans 1 through 3 and East Approach Spans 5 through 13 consist of four concrete arch ribs supporting two concrete decks (see Photo 5). The top deck is the main traffic deck, carrying four vehicular lanes, as well as pedestrian loading on the north and south sides of the bridge. Originally intended for street railway traffic, the lower concrete deck has been closed to the public and is only periodically used as a multi-functional setting for the City of Cleveland.

Photo 5 – North elevation of concrete arch ribs in Span 3.

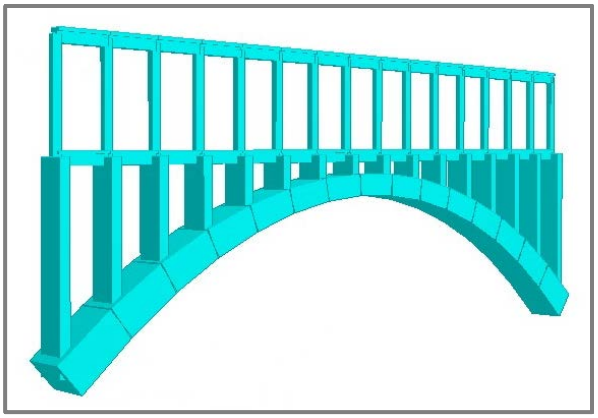


Figure 11 – Rendered view of Span 2 interior concrete arch rib STAAD model.

Many of the concrete approach items have been replaced or rehabilitated during the numerous modifications done to the bridge since its initial erection, including full deck replacement in 1994. Numerous floorbeams, columns, and jack arches were either retrofitted or replaced at this time. However, the concrete arch ribs have not experienced any significant upgrades over the lifetime of the bridge. Accordingly, the rehabilitations to the structure were considered for dead and live load purposes only, while the original plans were used for capacity calculations.

The structural analysis for the concrete arch ribs was done with two-dimensional STAAD models. The nodal geometry for the concrete arch ribs, columns, and both decks was developed in Microstation based on information available in the original plans. Straight beam elements were used in the STAAD models with tapered cross sections where necessary to accurately represent the stiffness and dead load of the elements. Each concrete arch span was modeled independently with fixed supports at both ends such that the load effects of one arch do not influence the loadings on adjacent arches (see Figure 11).

Nodal dead loads were calculated by hand and with Excel spreadsheets, considering the original construction, applicable rehabilitations and the proposed lower deck modifications. Single live load vehicles and lane loadings were analyzed on the top deck of the models, and appropriate live load distribution factors were calculated for each arch line in order to determine the unique load effects on each arch. Distribution factors were also calculated for pedestrian loads on top deck sidewalks and throughout the lower decks where applicable.

The load effects and section properties were utilized in an iterative axial-moment interaction spreadsheet in Excel, with a series of macros developed in order to streamline the iterative process. Final load effects were calculated by multiplying the STAAD results by the appropriate distribution factors and impact, and pedestrian loading was applied in conjunction with live loads where appropriate. The capacities for each arch segment were calculated based on the load effects and resultant load eccentricity, and the rating equations were iterated until final rating factors were obtained.

All arch lines rate well above 1.0 in the current configuration, an expected result given that the lower deck was originally intended to take significantly higher loadings in tandem with upper deck loads. For Alternative #1, the south interior arch governs the rating in each span because the top deck live loads and lower deck pedestrian loads are maximized over this arch. The controlling load effect was typically the maximum negative moment at a support with its concurrent axial force. See Table 4 for the governing Alternative #1 ratings in each span, which match those from the 2012 load rating.

ALTERNATIVE #1: AS-INSPECTED CONCRETE ARCH CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1	3.30	5.51	11.30	7.45	6.55	7.70
Span 2	2.95	4.93	10.85	7.20	6.25	7.70
Span 3	3.36	5.62	12.58	8.27	7.13	7.67
Span 5	3.49	5.83	12.47	8.27	7.18	8.69
Span 6	3.57	5.97	12.77	8.46	7.35	8.86
Span 7	3.69	6.17	13.20	8.75	7.60	9.19
Span 8	3.57	5.96	12.67	8.40	7.30	8.85
Span 9	3.77	6.29	12.74	8.50	7.46	8.89
Span 10	3.90	6.51	12.70	8.53	7.55	8.95
Span 11	2.77	4.62	8.07	5.53	5.17	5.89
Span 12	2.59	4.32	8.84	5.87	5.15	6.00
Span 13	2.15	3.58	6.35	4.33	3.95	4.54

Table 4 – Controlling as-inspected concrete arch rating factors for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

The concrete arch ribs were reanalyzed for additional deck dead load and pedestrian load under Alternative #2 in order to investigate the feasibility of a full width lower deck. While this proposed modification lowered the ratings of Arches A and B on the north side of the bridge, the load ratings were still governed by Arch C in all but two of the spans due to the combination of live loads and pedestrians loads being maximized over this arch. However, in Spans 11 and 12 at the east end of the bridge, the upper deck sidewalk begins to taper down, allowing for additional live load over Arch B. As a result, the governing load ratings for Spans 11 and 12 are reduced by Alternative #2, while the ratings in the other spans are identical to those in Alternative #1. See Table 5 below for the governing Alternative #2 ratings in each span (ratings that differ from Alternative #1 shown bold).

ALTERNATIVE #2: AS-INSPECTED CONCRETE ARCH CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1	3.30	5.51	11.30	7.45	6.55	7.70
Span 2	2.95	4.93	10.85	7.20	6.25	7.70
Span 3	3.36	5.62	12.58	8.27	7.13	7.67
Span 5	3.49	5.83	12.47	8.27	7.18	8.69
Span 6	3.57	5.97	12.77	8.46	7.35	8.86
Span 7	3.69	6.17	13.20	8.75	7.60	9.19
Span 8	3.57	5.96	12.67	8.40	7.30	8.85
Span 9	3.77	6.29	12.74	8.50	7.46	8.89
Span 10	3.87	6.46	12.59	8.45	7.48	8.87
Span 11	2.12	3.54	6.19	4.24	3.96	4.51
Span 12	2.59	4.32	8.84	5.87	5.15	6.00
Span 13	2.15	3.58	6.35	4.33	3.95	4.54

Table 5 – Controlling as-inspected concrete arch rating factors for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

The reinforced concrete arch ribs were generally in fair condition due to spalled or delaminated concrete with isolated map and longitudinal cracking (see Photo 6). Although deterioration is present in isolated locations, the reinforcing steel typically has negligible section loss. In addition, the spalled locations are minor relative to the size of the sections and do not occur at a controlling location on the arch ribs. As a result, the as-inspected rating factors are equal to the as-built rating factors.

Photo 6 – Map cracking throughout underside of Arch A in Span 10, looking west toward Pier 9.

Concrete Columns

Concrete columns are present in all of the concrete approach spans, and there are two distinct types. The lower columns consist of the open spandrel arch columns which support the lower deck in Spans 1 through 3 and Spans 5 through 13. The upper columns extend between the lower deck and upper deck, supporting the upper deck floorbeams and jack arches in the framing system. These columns are present in all of the concrete approach spans, including the arch spans and the cellular construction in Spans 1A and 1B (see Figure 12).

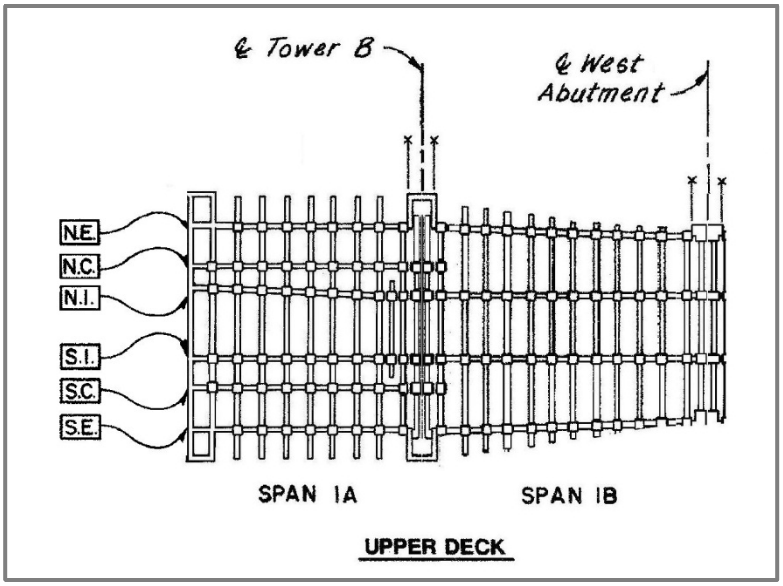


Figure 12 – Plan view of framing system and column locations in Spans 1A and 1B.

The columns were rated using an approach similar to that of the concrete arches, utilizing the same axial-moment interaction Excel spreadsheet with a modification to include slenderness effects. All columns in the arch spans were analyzed in the same STAAD models as the concrete arches, while two additional models were created in order to evaluate Spans 1A and 1B. The lower open spandrel columns and upper concrete columns were rated separately due to their differences in section properties.

Similar to the concrete arch ribs, the controlling rating factors for the lower spandrel columns in Alternative #1 occur in south interior column line where the live load and pedestrian load effects are maximized. All of the lower columns rate well above 1.0 for all load cases and match the values from the 2012 load rating (see Table 6). The concrete columns and hangers in Span 12 could not be rated due to the lack of information in the available existing plans.

ALTERNATIVE #1: AS-INSPECTED LOWER CONCRETE COLUMNS CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1	3.07	5.13	7.50	5.03	4.96	6.03
Span 2	2.39	3.99	5.01	3.62	3.49	3.26
Span 3	2.70	4.51	7.24	5.26	4.91	3.85
Span 5	2.44	4.08	7.56	5.35	5.00	4.91
Span 6	2.40	4.00	7.29	5.25	4.92	4.72
Span 7	3.61	6.04	8.89	6.48	6.16	5.50
Span 8	2.21	3.69	6.53	4.68	4.41	4.10
Span 9	2.38	3.97	7.11	4.95	4.73	5.96
Span 10	2.82	4.71	8.55	5.99	5.11	6.94
Span 11	4.07	6.80	9.83	7.30	6.06	7.35
Span 13	8.70	14.22	25.83	17.92	15.99	18.45

Table 6 – Controlling as-inspected lower concrete column rating factors for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

The lower spandrel columns were reanalyzed for the additional deck dead load and pedestrian load for Alternative #2 in order to evaluate the feasibility of a full width deck. While the majority of the controlling ratings are governed by columns over Arch C which are unaffected by the lower deck modification, isolated controlling values have decreased and are governed by columns over Arch B. Specifically, the governing load ratings decreased in Spans 10 and 11 for all load cases, with load ratings in Spans 6, 7, 9 and 13 decreasing for only select load cases. Nonetheless, all of the load ratings are above 1.0 for all load cases. See Table 7 below for the governing Alternative #2 ratings in each span (ratings that differ from Alternative #1 shown bold).

ALTERNATIVE #2: AS-INSPECTED LOWER CONCRETE COLUMNS CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1	3.07	5.13	7.50	5.03	4.96	6.03
Span 2	2.39	3.99	5.01	3.62	3.49	3.26
Span 3	2.70	4.51	7.24	5.26	4.91	3.85
Span 5	2.44	4.08	7.56	5.35	5.00	4.91
Span 6	2.40	4.00	7.29	5.25	4.83	4.72
Span 7	3.45	5.76	8.89	6.48	6.16	5.50
Span 8	2.21	3.69	6.53	4.68	4.41	4.10
Span 9	2.38	3.97	7.11	4.95	4.49	4.65
Span 10	2.79	4.67	8.48	5.51	4.75	5.80
Span 11	3.12	5.21	7.53	5.60	4.64	5.63
Span 13	8.70	14.22	25.83	17.92	15.99	16.81

Table 7 – Controlling as-inspected lower concrete column rating factors for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

Overall the condition rating of the concrete columns is controlled by deficiencies in the West Approach station and tunnel sections of the bridge, which will not be affected by any of the lower deck modifications and are not included in the scope of this load rating (see Photo 7). In Spans 1 through 13, the lower spandrel columns exhibit isolated spalls with exposed reinforcing and delaminations (see Photo 8). Despite the deterioration to these elements, the exposed reinforcing typically exhibits negligible loss, and the spalls do not occur at the critical locations on the columns. Thus, the governing as-inspected rating factors are identical to those in the as-built condition.

Photo 7 – Easternmost column in West Side Station jack arch Line C with heavy spalling and exposed rebar on all faces.

Photo 8 – Northeast face of Span 2, arch line D, Column 13 with 5' high by 1' wide spall with exposed reinforcing.

The load ratings for the upper concrete columns are not affected by the proposed lower deck modifications because these columns are only subject to the upper deck loadings which remain unchanged. As a result, the load ratings for each alternative are equal to those presented in the 2012 load rating performed by TranSystems. Similar to the lower spandrel columns, the controlling load ratings for the upper concrete columns are above 1.0 for all load cases (see Table 8).

AS-INSPECTED UPPER CONCRETE COLUMNS CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1A	3.29	5.49	8.11	5.50	5.57	6.09
Span 1B	2.49	4.13	7.94	5.12	4.55	5.32
Span 1	2.77	4.62	8.07	5.76	4.95	5.64
Span 2	2.87	4.79	7.75	5.19	5.01	4.77
Span 3	2.98	4.97	8.03	5.46	5.18	4.77
Span 5	3.03	5.06	8.41	5.69	6.87	5.75
Span 6	2.92	4.87	8.19	5.64	5.36	5.71
Span 7	2.95	4.92	8.21	6.33	4.98	5.17
Span 8	2.82	4.71	7.94	5.46	5.19	4.73
Span 9	2.93	4.89	8.37	6.63	5.79	5.90
Span 10	3.09	5.16	8.71	6.73	6.16	6.08
Span 11	3.62	6.04	10.31	7.76	6.29	7.22
Span 13	2.92	4.85	7.74	5.82	5.06	5.43

Table 8 – Controlling as-inspected upper concrete column rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

In Spans 1A through 13, the upper concrete columns supporting the floorbeams and jack arches exhibit isolated spalls with exposed reinforcing and delaminations. These upper columns also commonly have large spalls to the decorative capitals (see Photo 9), and many of these locations have been repaired or patched. Despite this deterioration, the spalls typically do not occur at critical locations on the columns and exposed reinforcing bars typically exhibit negligible section loss. As a result, the governing as-inspected rating factors are identical to those in the as-built condition.

Photo 9 – Span 7, jack arch line B, Column 7 with west face of decorative capital fully spalled.

Concrete Floorbeams

All of the concrete approach spans in the structure between Spans 1A and 13 consist of transverse floorbeams supporting a longitudinal deck. These concrete floorbeams are reinforced to be continuous, spanning between the various column lines. Many of these concrete floorbeams have been replaced or strengthened during the major rehabilitations of the structure. The end floorbeams in each span were typically replaced during the 1965 Rehabilitation, and isolated interior floorbeams were partially or fully replaced at this time. Numerous floorbeams were also strengthened with the addition of concrete top or bottom flange retrofits with supplemental reinforcement (see Figure 13). The 1994 Rehabilitation also resulted in the replacement of all end floorbeams, as well as partial repairs or strengthening of the remaining floorbeams. During both rehabilitations, much of the top reinforcing was retained and used in the new floorbeams or deck (see Figure 14).

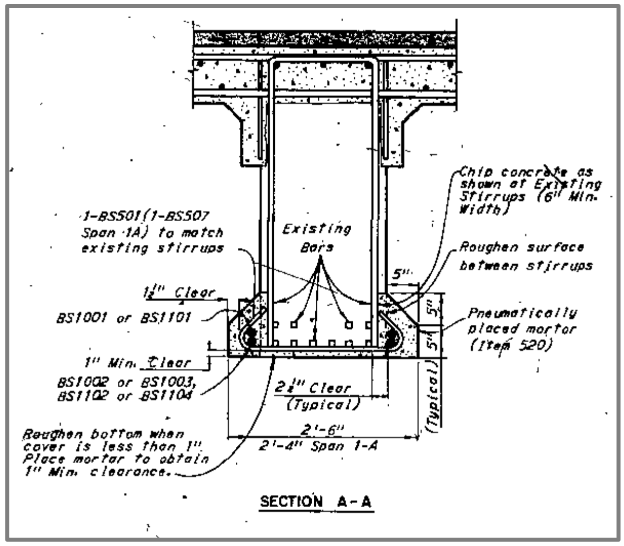


Figure 13 – Typical cross section showing floorbeam bottom flange strengthening detail during 1965 Rehabilitation.

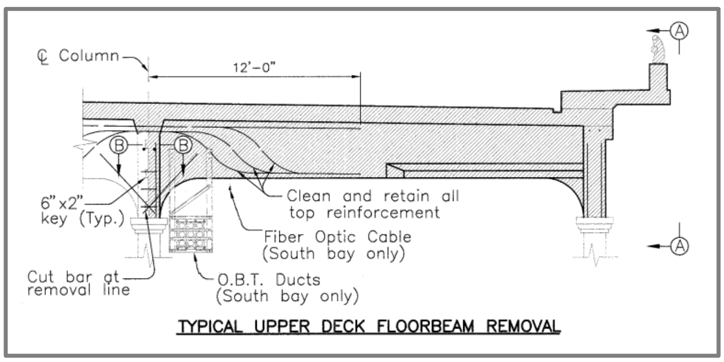


Figure 14 – Typical removal detail for concrete upper floorbeam during 1994 Rehabilitation. Note retained reinforcement adjacent to column.

Concrete floorbeam load effects were calculated using a combination of hand calculations and two-dimensional STAAD models. Transverse floorbeams were modeled as multi-span continuous with pinned supports at column locations, and longitudinal load effects for each truck type were calculated by hand in order to determine distribution factors on both the end and interior floorbeams. All critical unique floorbeams were rated for each span, including separate load ratings for original and replaced floorbeams due to their different capacities.

The upper deck floorbeam load ratings are not influenced by the proposed lower deck modifications because these floorbeams are only carrying upper deck vehicular and pedestrian loads which would remain unchanged. As a result, the load ratings for each alternative are equal to those presented in the 2012 load rating performed by TranSystems. The controlling floorbeams in each span rate below 1.0 for HS20 Inventory and above 1.0 for all subsequent load cases (see Table 9).

AS-INSPECTED UPPER CONCRETE FLOORBEAMS CONTROLLING RATING FACTORS							
Location		HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
Span 1A	Moment	0.86	1.43	2.14	1.62	1.60	1.68
Span 1B	Moment	0.79	1.32	2.11	1.54	1.35	1.54
Span 1	Moment	0.88	1.47	2.35	1.72	1.51	1.72
Spans 2, 6, 7, 8	Shear	0.86	1.44	2.30	1.68	1.47	1.68
Span 3	Shear	0.82	1.37	2.20	1.60	1.41	1.60
Span 5	Shear	0.84	1.40	2.25	1.64	1.44	1.64
Spans 9 and 10	Moment	0.88	1.47	1.72	1.51	1.51	1.72
Span 11	Moment	0.79	1.31	2.10	1.53	1.34	1.53
Span 12	Shear	0.79	1.33	2.12	1.55	1.36	1.55
Span 13	Shear	0.80	1.33	2.13	1.56	1.37	1.56

Table 9 – Controlling as-inspected upper concrete floorbeam rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

The reinforced concrete floorbeams in the East and West Approach Spans exhibit minor deficiencies throughout. Isolated minor to moderate spalls with exposed reinforcement are commonly present on the members which have not been repaired during the last major rehabilitation. The exposed reinforcement typically exhibits minimal section loss if any, and spalled concrete typically occurs on exterior faces of floorbeams with insufficient concrete cover or around tension reinforcement in the bottoms of the floorbeams (see Photo 10). As a result, the upper concrete floorbeam rating factors are governed by the as-built condition.

Photo 10 – West face of Floorbeam 9 in Span 3 between Columns B and C with spall and exposed reinforcement in beam underside near midspan.

Photo 11 – Span 3 Lower Floorbeam 6 between Arch C and Arch D with large spall to floorbeam underside with exposed reinforcement exhibiting active corrosion.

Similar to the upper deck floorbeams, a number of the lower floorbeams have been replaced during the various rehabilitations of the bridge. Many of the lower concrete floorbeams exhibit moderate spalling with exposed reinforcement and isolated delaminated areas (see Photo 11).

In the current configuration of the concrete arch spans, the lower deck is only present between Arches B and D, meaning there are no applied dead or live loads on floorbeams in the northernmost bay. While this configuration would be maintained for Alternative #1, a reinforced concrete deck would need to be added to the northernmost bay for Alternative #2. Despite this additional dead load and subsequent pedestrian load, the controlling rating factors for each alternative are equal because the controlling load case includes pedestrian load applied in only two of the three bays, a condition which is covered by the current configuration.

Because the lower deck is no longer used for vehicular traffic but would potentially be used for pedestrians or bicycle traffic, the lower deck concrete floorbeams were rated for pedestrian loading only. The lower floorbeams in the concrete approach spans rated well above 1.0 for all load cases (see Table 10). As-inspected losses were not applied to the lower floorbeams, as the reinforcing in these members typically exhibits minimal loss with rather high rating factors due to the applied loading. Lower floorbeams in Spans 1A, 1B and 12 were not rated due to a lack of information available in the as-built plans.

AS-INSPECTED LOWER CONCRETE FLOORBEAMS RATING FACTORS			
Location		Pedestrian Loading	
		Inventory	Operating
Span 1	Shear	6.29	10.49
Span 2	Moment	7.71	12.86
Span 3	Moment	6.26	10.44
Span 5	Moment	7.37	12.31
Spans 6, 7, 8	Moment	7.83	13.07
Spans 9 and 10	Moment	7.98	13.32
Span 11	Moment	5.16	8.62
Span 13	Moment	6.38	10.65

Table 10 – Controlling as-inspected lower concrete floorbeam rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

Jack Arches

Concrete jack arches span longitudinally between the concrete columns along each column line below both decks (see Photo 12). Because the primary deck reinforcement is also longitudinal, the transverse floorbeams were rated using the assumption that they support all vehicular loading transferred from the deck. As a result, the jack arches are not truly load-carrying members. However, a conservative calculation was made in order to rate the jack arches based on a single wheel line passing directly over the jack arch. This configuration would transfer load directly into the jack arch rather than through the deck and into the floorbeams. The jack arches are non-composite sections following the 1994 deck replacement, and top rebar are not continuous over the columns with a roughened construction joint at each end. Accordingly, the jack arches were treated as simply supported beams.

Photo 12 – Typical jack arch north elevation (Span 9 jack arch line C between Floorbeams 12 and 13).

The jack arch load ratings are not influenced by the proposed lower deck modifications because the jack arches were only rated for a vehicular wheel load from the upper deck. The lower deck jack arches are not subject to significant applied loads because the transverse floorbeams were assumed to transmit lower deck loads directly into the lower spandrel columns and arch ribs. As a result, the load ratings for each alternative are equal to those presented in the 2012 load rating performed by TranSystems. The controlling jack arches in each span rate above 1.0 for all load cases (see Table 11).

AS-INSPECTED JACK ARCH CONTROLLING RATING FACTORS						
Rating Type	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
Moment	1.12	1.86	2.98	3.24	3.65	3.24
Shear	1.29	2.15	3.44	2.63	3.03	2.63

Table 11 – Controlling as-inspected concrete jack arch rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

Overall the condition coding of the concrete jack arches is controlled by elements in the west approach station and tunnel sections of the bridge, which were not included in the scope of this load rating. Isolated jack arches in the remaining concrete spans exhibit spalls with exposed reinforcing and delaminated areas. In locations with exposed reinforcing, the steel typically exhibits negligible or no section loss (see Photo 13). Therefore, the capacities are not affected, and the as-inspected rating factors are equal to the as-built. Isolated jack arches throughout the structure have either been patched or replaced during the various rehabilitations of the structure (see Figure 15).

Photo 13 – Jack arch exhibiting an underside spall with exposed reinforcement in Span 1 Column Line B. Note that no section loss has occurred on exposed rebar.

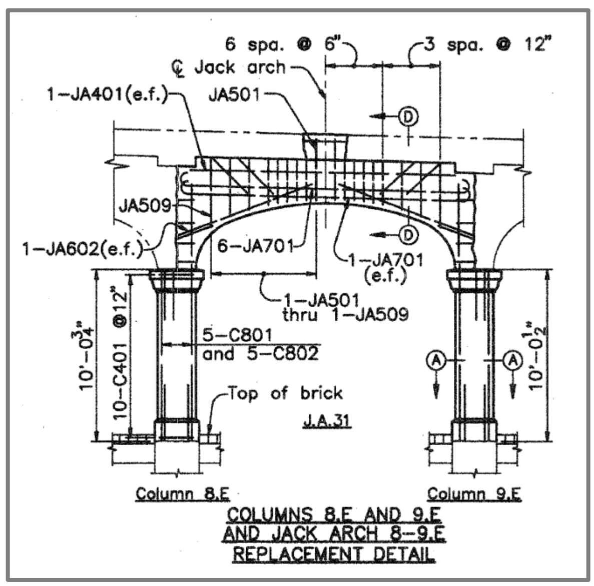


Figure 15 – Typical concrete jack arch replacement detail from 1994 Rehabilitation, showing overall geometry and steel reinforcement.

STEEL TRUSS SPAN

The Main Truss (Span 4) consists of a 591 foot long three-hinged, trussed arch with a modified Pratt design. The deck at the first four panel points at each end are framed directly into the arch vertical members, while the remaining panel points are suspended from hangers (eyebars) which are pin connected to the lower chords above the decks (see Photo 14). Both deck levels utilize a stringer-floorbeam framing system with cantilevered brackets which are continuous with the interior floorbeams. The existing lower deck in Span 4 consists of a plywood-covered, existing 5" steel open grid deck between the truss lines. Access walkways are present outside the truss lines on the lower deck, consisting of fiberglass reinforced grating that is supported by the lower floorbeam cantilevers.

Photo 14 – South elevation of west half of Span 4. Note that Panel Points 0 to 3 are framed directly into the truss, while Panel Points farther east are suspended.

The main span is on a straight alignment with a vertical curve such that the deck elevation is highest at midspan. Panel points are numbered west to east, and the bridge is symmetric in this direction. Because of the symmetry, most existing plans label the Panel Points from 0 (west end) to 12 (at midspan) and then utilize a prime notation, numbering down to 0' (east end). However, for simplicity during load rating and inspection, the nomenclature was revised so the panel points are numbered west to east from 0 to 24.

The deck and framing elements underwent major rehabilitations in 1965 and 1994, as the upper concrete deck was fully replaced along with numerous modifications to the floorbeams and stringers in both levels. In addition, the traffic pattern was revised in 2003 with the addition of a large pedestrian sidewalk on the north side of the bridge (see Figure 16). Originally carrying three lanes of traffic in both directions, the bridge now carries two westbound and one eastbound traffic lanes between the arch lines, as well as one additional eastbound lane outside the South Arch.

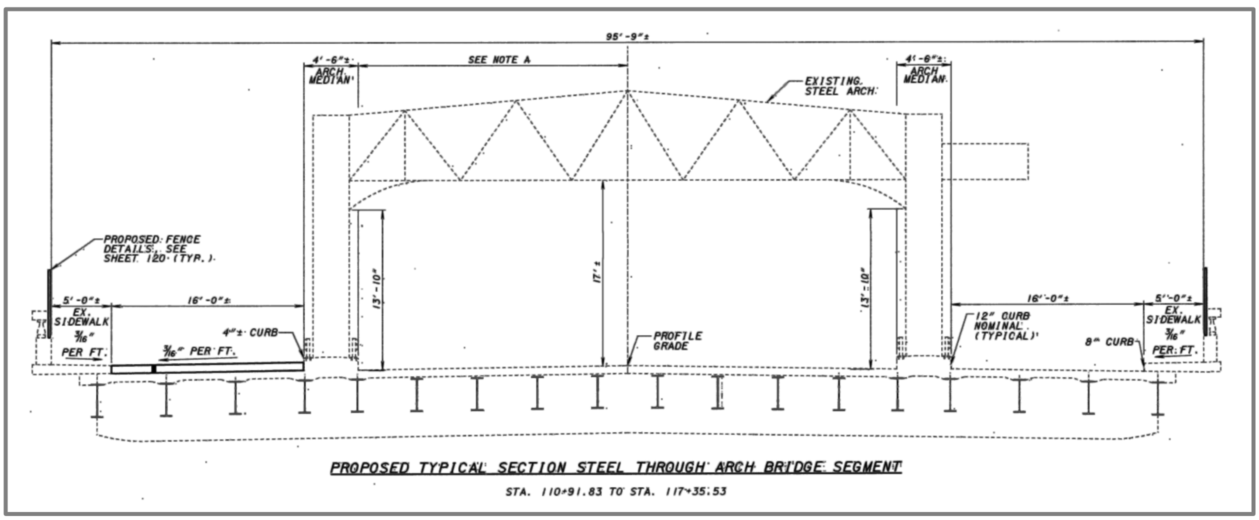


Figure 16 – Roadway cross section in Span 4 showing added pedestrian sidewalk (from 2003 Plans).

The Main Truss was analyzed with a series of three-dimensional models in STAAD.Pro (see Figure 17). The vertical curvature of the roadway was taken into account with the truss geometry. Pinned supports were used at the ends of each arch to match existing conditions, and member releases were utilized to mimic the truss member connections, hangers, secondary members, and framing system. The eyebar hangers could not be modeled as "Tension Only" members in STAAD without causing instabilities in the model; thus, the hangers were modeled as purely axial members and were found to never experience compression under the applied loading. In addition, because of the hinge at L12, upper chord members L11-L12 and L12-L13, as well as vertical member L12-U12, are zero force members with axial force released in the models. Truss forces were input in Excel spreadsheets and modified with impact and multiple presence factors in order to obtain final load effects with which to rate both the truss members and gusset plates.

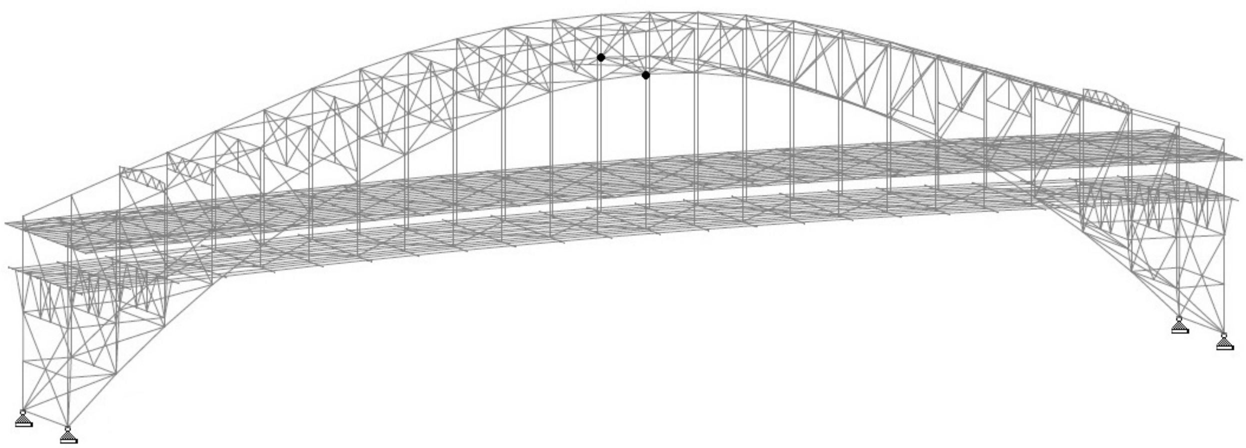


Figure 17 – South Elevation of Main Arch Span STAAD model.

Framing members were analyzed using a series of hand calculations, two-dimensional STAAD models, and Excel spreadsheets. The calculations for these members were used in order to develop the applied dead loads used in the Main Truss dead load three-dimensional model.

Dead Loads

The dead load STAAD model consists of the arch primary and secondary members, floorbeams and stringers. Each type of member was input into STAAD using its base section properties. In order to account for the additional weight of connection, tie, and fill plates, as well as lacing and internal diaphragms, a calculated average increase "bump-up factor" was applied uniquely to each member type. The weight of gusset plates, splice plates, and larger connection plates were applied at specific individual panel points or nodes. Sign supports attached to the structure were also applied directly at appropriate joint locations. Two utility items running the length of the bridge were also included in the dead load calculation: utility ducts and fiber optic cable mounted to upper deck stringers adjacent to the South Arch, and electric lines mounted to the south access walkway on the lower deck level. These items were applied as distributed loads directly to the stringers carrying the utilities.

Deck and parapet loads were distributed evenly between the roadway stringers. This includes the subsequent dead load due to the new sidewalk added to the north side of the structure in 2003. All existing deck concrete in the main span, including this pedestrian sidewalk, is lightweight concrete with a unit weight of 115 lb/ft^3. The applied dead load on the lower stringers is based on the tributary area of any attached deck elements. Lighting fixtures and small architectural items on the walkways were considered negligible and not specifically calculated in the dead loading.

The concrete deck and parapet loads for the proposed lower deck modifications were distributed directly to the lower deck stringers. In order to utilize a worst case loading condition for the analysis of Alternative #1 and Alternative #2, normal weight concrete with a unit weight of 150 lb/ft^3 was assumed. In reality, lightweight concrete would be a realistic option for the proposed lower deck modifications in order to reduce the impact of additional dead load on the structure; however, this would come with an increased construction cost.

The vast majority of the dead load is taken as compression in the truss lower chords. See Chart 1 for a comparison of lower chord dead loads for Alternative #1. Note that the loads are roughly symmetric about the midspan of the bridge; however, the South Truss takes marginally more dead load. This difference is due primarily to fact that the utilities were applied directly to the appropriate stringers on the south side of structure, while all other superimposed dead loads are taken equally be the stringers. In addition, the sign structures are applied directly to nodes, introducing a lack of symmetry. If these loads are removed from the STAAD model, the North Truss and South Truss have equal levels of dead load in all members.

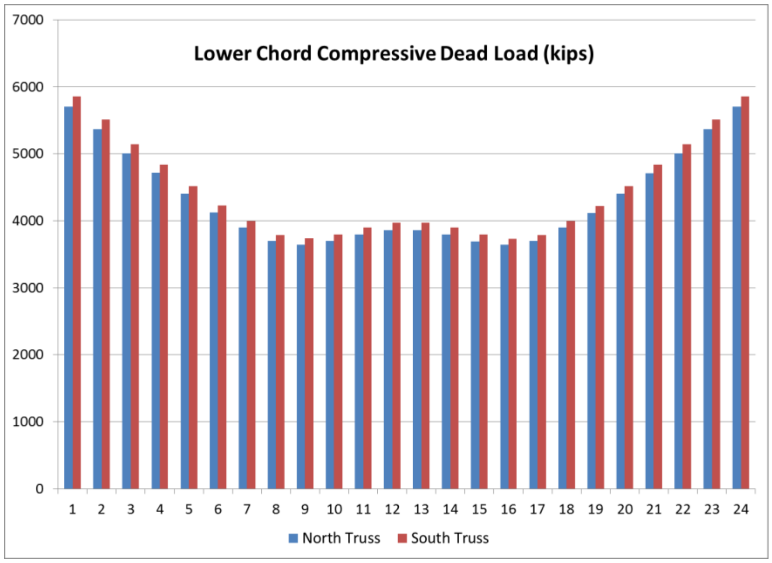


Chart 1 – Comparison in main span lower chord compressive dead loads between North and South Trusses for Alternative #1.

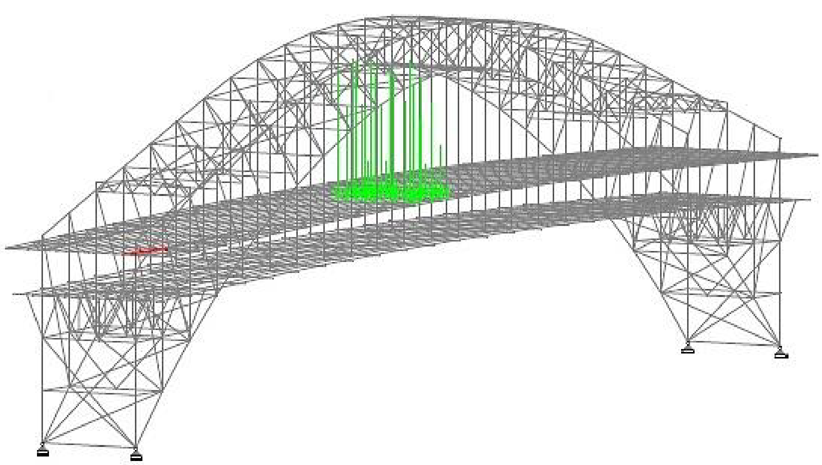


Figure 18 – Typical live load generation showing three HS20 trucks loading the South Truss (red line indicates rightmost initial truck position, green arrows show location and magnitude of wheel loads).

Live Loads

To develop appropriate live load distribution into the truss panel points, individual truck generations were run directly on the upper deck framing elements in STAAD. Each truss line is governed three loaded lanes with a multiple presence factor of 0.9. The South Truss live load consists of one vehicle on the south cantilever and an additional two vehicles on the south side of the interior portion of the deck (see Figure 18), while the North Truss is controlled by three vehicles positioned adjacent to the North Truss between the truss lines.

Lane loading was applied to maximize the loading in each truss, utilizing the same lane configurations as the truck loading. Because the Main Arch spans over 200 feet, the structure qualifies as "long span" and is subject to HS20 and 5C1 truck trains. These loadings were applied in the rightmost lanes in each direction in conjunction with the standard vehicles in the rest of the lanes, utilizing only whole trucks.

In accordance with ODOT BDM Section 914, a pedestrian load of 75 lb/ft^2 was applied on both upper deck sidewalks and all lower deck access walkways. The pedestrian loading for Alternative #1 is identical to that applied in the 2012 load rating, as this configuration would require an upgrade to a concrete deck but would occupy the same space as the existing lower deck. However, Alternative #2 represents a much larger applied loading to the structure because pedestrian load must now be applied to the entire width of the lower deck. These loads were considered to act simultaneously with the live load and were subtracted from the capacity of the member per the appropriate load rating equations. Pedestrian load was used only in locations that would maximize the load effects in each truss line.

Analysis Results

Truss Members

Force effects were taken from the STAAD output files and sorted into the truss rating sheets. Impact and multiple presence factors were applied in the forces spreadsheet. Capacities were calculated within the rating sheet based on section and material properties, considering as-built information from existing plans and any applicable field measurements. The primary truss members were rated for each alternative in the as-inspected condition, as the proposed deck modifications would occur with the current section losses present on the existing members.

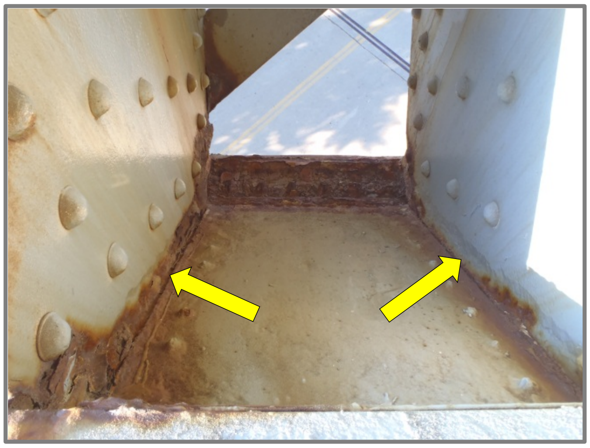


Photo 15 – South Truss L23-L24 with pitting on interior web plates up to 3/8" along the internal diaphragm at L24.

In the as-inspected condition, 17 out of 194 truss members exhibited more than 5% section loss, three locations exhibited between 10% and 20% section loss, and one vertical member (L3-U3 South) exhibited over 20% section loss. Significant deterioration was noted primarily in lower chord and web members below the upper deck level, as well as gusset plates. The controlling lower chord losses typically occur on the interior faces web plates along internal diaphragms or gusset plate edges (see Photo 15). Many of the internal diaphragms and stay plates at these locations exhibit isolated areas of 100% section loss. Upper chord members and lower chord members above the main deck typically exhibit minimal or no section loss (see Photo 16).

Photo 16 – General view of South Truss above the main deck, looking northwest.

In the as-inspected condition for Alternative #1, isolated lower chord members have inventory and operating rating factors below 1.0 for the HS20 and Truck Train load cases, while rating factors for the remaining load cases are above 1.0 (see Table 12). The added dead load from the proposed concrete deck and parapets results in a total of nine (9) lower chord members exhibit operating rating factors below 1.10. Six of these members are present on the South Truss, while the remaining three members are on the North Truss. Note that all of these members are located below the lower deck where the most widespread section loss is typically present. All of these members rate above 1.10 in the as-built condition, suggesting that rehabilitation of these truss members would be feasible in order to restore sufficient capacity of the truss members for the proposed additional loads. The pedestrian load configuration is the same as the existing condition, as Alternative #1 does not modify the limits of pedestrian traffic.

ALTERNATIVE #1: AS-INSPECTED MAIN TRUSS CONTROLLING RATING FACTORS											
Location		Member	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck Train	5C1 Truck Train	Section Loss
North Truss	Lower Chord	L0-L1	0.58	0.73	5.61	3.67	3.13	2.18	0.78	0.97	4.15%
		L1-L2	0.85	1.07	7.42	4.85	4.14	2.90	1.13	1.39	3.04%
		L23-L24	0.75	0.94	7.25	4.74	4.05	2.82	1.01	1.25	2.60%
	Upper Chord	U19-U20	5.89	8.93	20.58	13.53	11.63	8.83	10.14	10.94	---
		U17-U18	6.05	8.92	20.58	13.55	11.63	8.67	9.29	10.13	---
		U6-U7	6.06	8.93	20.59	13.57	11.64	8.68	9.27	10.12	---
	Vertical	L21-U21	3.67	4.72	10.92	7.20	6.21	4.70	4.46	5.04	5.80%
	Diagonal	L19-U20	5.34	8.06	18.43	12.17	10.53	8.29	8.77	9.76	6.03%
South Truss	Lower Chord	L0-L1	0.68	0.85	6.80	4.45	3.79	2.61	0.89	1.11	7.84%
		L1-L2	0.69	0.87	6.31	4.12	3.52	2.45	0.90	1.11	8.86%
		L2-L3	0.80	1.00	6.56	4.29	3.66	2.56	1.00	1.23	9.84%
		L21-L22	0.86	1.08	7.03	4.60	3.93	2.74	1.07	1.32	9.14%
		L22-L23	0.78	0.97	7.07	4.62	3.94	2.74	1.00	1.25	7.86%
		L23-L24	0.59	0.73	5.88	3.84	3.28	2.26	0.77	0.96	8.95%
	Upper Chord	U14-U15	4.30	6.84	15.63	10.34	8.95	6.84	7.42	8.46	---
		U19-U20	4.32	6.88	15.84	10.41	8.95	6.83	7.52	8.25	---
		U6-U7	4.55	7.15	16.44	10.84	9.30	7.00	6.96	7.79	---
	Vertical	L3-U3	1.73	2.37	5.48	3.61	3.12	2.37	2.18	2.49	23.07%
	Diagonal	U3-L4	3.88	6.18	14.10	9.34	8.09	6.34	6.69	7.42	8.92%
		L18-U19	4.14	6.62	15.03	9.96	8.65	6.88	6.67	7.75	---

Table 12 – Controlling as-inspected truss member rating factors for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

In the as-inspected condition for Alternative #2, numerous lower chord members exhibit rating factors below 1.0 (see Table 13). Several of these members do not have sufficient capacity to support the additional dead load and pedestrian load proposed by the full width reinforced concrete deck in Alternative #2. These members are shown to have a rating factor of zero (0), as these elements do not have the capacity to support any applied live loads.

ALTERNATIVE #2: AS-INSPECTED MAIN TRUSS CONTROLLING RATING FACTORS											
Location		Member	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck Train	5C1 Truck Train	Section Loss
North Truss	Lower Chord	L0-L1	0	0	0	0	0	0	0	0	4.15%
		L1-L2	0	0	0	0	0	0	0	0	3.04%
		L2-L3	0	0	0	0	0	0	0	0	2.80%
		L3-L4	0	0	0	0	0	0	0	0	1.29%
		L4-L5	0.48	0.59	2.96	1.94	1.66	1.16	0.57	0.69	---
		L5-L6	0.28	0.35	1.60	1.05	0.90	0.63	0.34	0.40	4.95%
		L10-L11	0.51	0.64	3.01	1.96	1.68	1.17	0.65	0.76	---
		L11-L12	0	0	0	0	0	0	0	0	---
		L12-L13	0	0	0	0	0	0	0	0	---
		L13-L14	0.52	0.65	3.02	1.97	1.68	1.17	0.65	0.76	---
		L18-L19	0.78	0.98	4.46	2.92	2.50	1.76	0.95	1.13	0.77%
		L19-L20	0.48	0.60	2.98	1.95	1.67	1.17	0.58	0.69	---
		L20-L21	0	0	0	0	0	0	0	0	0.58%
		L21-L22	0	0	0	0	0	0	0	0	0.93%
		L22-L23	0	0	0	0	0	0	0	0	0.74%
		L23-L24	0	0	0	0	0	0	0	0	2.60%
	Upper Chord	U19-U20	5.69	8.36	19.28	12.68	10.89	8.27	9.50	10.25	---
		U17-U18	5.79	8.20	18.90	12.45	10.69	7.97	8.54	9.31	---
		U6-U7	5.80	8.21	18.92	12.46	10.70	7.98	8.52	9.30	---
	Vertical	L21-U21	2.93	3.66	8.46	5.58	4.81	3.64	3.46	3.90	5.80%
	Diagonal	L19-U20	5.13	7.50	17.15	11.33	9.80	7.71	8.16	9.08	6.03%
South Truss	Lower Chord	L0-L1	0	0	0	0	0	0	0	0	7.84%
		L1-L2	0	0	0	0	0	0	0	0	8.86%
		L2-L3	0	0	0	0	0	0	0	0	9.84%
		L3-L4	0	0	0	0	0	0	0	0	6.85%
		L4-L5	0.67	0.84	4.46	2.92	2.50	1.74	0.80	0.98	1.85%
		L5-L6	0.87	1.09	5.30	3.47	2.96	2.07	1.05	1.26	2.86%
		L10-L11	0.85	1.06	5.13	3.35	2.86	1.98	1.05	1.24	---
		L11-L12	0.27	0.34	1.90	1.24	1.06	0.74	0.34	0.41	---
		L12-L13	0.27	0.34	1.91	1.25	1.06	0.74	0.34	0.41	---
		L13-L14	0.85	1.06	5.14	3.36	2.87	1.98	1.05	1.24	---
		L19-L20	0.75	0.94	4.98	3.26	2.79	1.95	0.89	1.09	0.97%
		L20-L21	0.25	0.31	1.80	1.18	1.01	0.70	0.30	0.36	2.82%
		L21-L22	0	0	0	0	0	0	0	0	9.14%
		L22-L23	0	0	0	0	0	0	0	0	7.86%
		L23-L24	0	0	0	0	0	0	0	0	8.95%
	Upper Chord	U14-U15	4.13	6.36	14.54	9.62	8.33	6.36	6.90	7.87	---
		U19-U20	4.17	6.43	14.80	9.73	8.37	6.39	7.03	7.71	---
		U6-U7	4.35	6.55	15.07	9.94	8.53	6.41	6.38	7.14	---
	Vertical	L3-U3	1.25	1.57	3.62	2.38	2.06	1.57	1.44	1.64	23.07%
	Diagonal	U3-L4	3.74	5.77	13.16	8.71	7.55	5.91	6.24	6.92	8.92%
		L18-U19	4.01	6.24	14.15	9.38	8.14	6.48	6.28	7.29	---

Table 13 – Controlling as-inspected truss member rating factors for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

Overall the added dead load and subsequent pedestrian load in Alternative #2 result in a total of thirty-one (31) truss members with operating rating factors below 1.10, consisting of sixteen North Truss members and fifteen South Truss members. Note that all of these members are lower chords, as these members carry the majority of the applied loads as thrust into the truss bearings. Furthermore, all but one of these members exhibit operating rating factors below 1.10 in the as-built condition, and many of the truss members exhibiting low rating factors do so without any measureable section loss. This suggests that any structural rehabilitation in order to accommodate Alternative #2 would be large in scale, as the lower chord would need to be strengthened well beyond its original capacity.

Of the thirty-one (31) truss members with operating rating factors less than 1.10, seventeen (17) lower chord members do not have sufficient capacity to support the dead and pedestrian loads in Alternative #2 without any additional live loads present. In accordance with the rating factor equations in Figure 9, these members were reported to have rating factors equal to zero. This further suggests that substantial rehabilitations would be necessary on the lower chord truss members in order to make Alternative #2 feasible.

Both alternatives represent a substantial reduction in the truss load ratings due to the additional dead loads being applied to a long span bridge that carries the majority of its applied load as compressive force in the lower chords. See Table 14 below for the controlling as-inspected load ratings in the structure's existing configuration.

EXISTING CONDITION: AS-INSPECTED MAIN TRUSS CONTROLLING RATING FACTORS											
Location		Member	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck Train	5C1 Truck Train	Section Loss
North Truss	Lower Chord	L0-L1	1.17	1.47	11.33	7.41	6.32	4.40	1.58	1.96	4.15%
	Upper Chord	U19-U20	6.00	9.11	21.00	13.80	11.86	9.01	10.34	11.16	---
		U17-U18	6.19	9.16	21.11	13.91	11.93	8.90	9.54	10.40	---
		U6-U7	6.20	9.16	21.13	13.92	11.94	8.91	9.51	10.38	---
	Vertical	L21-U21	3.88	5.07	11.72	7.73	6.66	5.04	4.79	5.41	5.80%
	Diagonal	L19-U20	5.44	8.24	18.84	12.44	10.77	8.47	8.97	9.97	6.03%
South Truss	Lower Chord	L23-L24	1.01	1.26	10.10	6.60	5.63	3.88	1.32	1.64	8.95%
		L2-L3	1.22	1.52	9.93	6.49	5.55	3.88	1.51	1.87	9.84%
	Upper Chord	U14-U15	4.38	6.99	15.96	10.56	9.14	6.98	7.58	8.64	---
		U19-U20	4.40	7.02	16.15	10.61	9.13	6.97	7.66	8.41	---
		U6-U7	4.65	7.32	16.85	11.11	9.53	7.17	7.14	7.98	---
	Vertical	L3-U3	1.88	2.62	6.05	3.99	3.45	2.62	2.41	2.75	23.07%
	Diagonal	U3-L4	3.96	6.31	14.39	9.53	8.25	6.46	6.83	7.57	8.92%
		L18-U19	4.21	6.74	15.29	10.14	8.80	7.00	6.78	7.88	---

Table 14 – Controlling as-inspected truss member rating factors for existing structure (numbers below 1.0 are red, controlling values are shaded in yellow).

The controlling as-inspected members for HS20 Inventory for the North and South Trusses are lower chord members adjacent to the truss bearings. The North Truss is governed by L0-L1, while the South Truss is governed by L23-L24. Vertical member L3-U3 also governs some of the Ohio legal loads for the South Truss. After section losses are applied as reductions to capacity, the truss members rate above 1.0 for all load cases with the governing rating factor for HS20 Inventory at 1.01.

Pins and Hangers

The decks in the suspended portion of the structure are pin-connected to the lower chord with a series of eyebar hangers. All of the pins and hangers were intended to be replaced in the 1994 Rehabilitation; however, according to the as-built plans, work at each end panel point (4 and 4') was not performed with fabricated material turned over to the Cuyahoga County Engineer.

The eyebar hangers were rated for axial tension, considering both gross section yielding in the shank and new section fracture in the heads. The pins were rated for bearing and for combined moment and shear. In order to evaluate the pins, the moments and shears were input into an Excel spreadsheet which utilized a macro in order to solve an iterative formula. The standard AASHTO condition equation (see Figure 19) was modified in order to yield a rating factor (β), and an example inventory rating equation would be set up as shown in Figure 20.

$$\frac{6M}{D^3F_y}+\left(\frac{2.2V}{D^2F_Y}\right)^3\leq 0.95$$

Figure 19 – Standard AASHTO moment-shear condition equation.

$$\frac{6(2.17\beta M_{LL}+1.3M_{DL})}{D^3F_y}+\left(\frac{2.2(2.17\beta V_{LL}+1.3V_{DL})}{D^2F_Y}\right)^3\leq 0.95$$

Figure 20 – Modified AASHTO combined moment-shear condition equation to allow for calculation of pin rating factor (β).

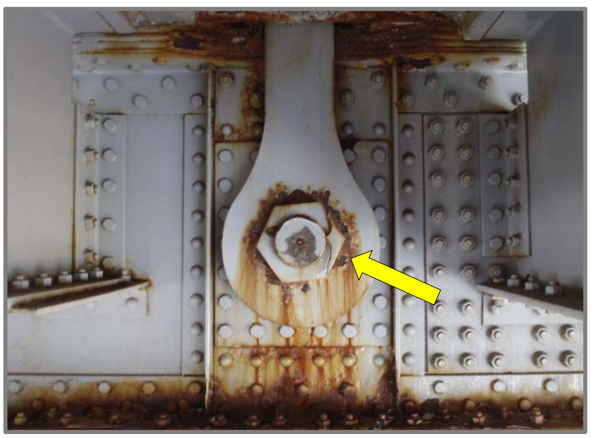


Photo 17 – East face of eyebar at connection to north lower pin at Panel Point 17.

The pins and hangers were rated with consideration of section losses noted during the inspection. Active corrosion was noted at the majority of lower pin locations and on isolated eyebar heads and shanks (see Photos 17 and 18), although section loss at most of these locations was negligible. In order to account for the corrosion at these locations, a conservative level of section loss (1/16") was applied to both faces of all eyebars and around the circumference of all lower pins, unless more significant section loss was specifically noted.

The pins and hangers are generally controlled by elements in the South Truss due to the increased live load from the south exterior traffic lane over the floorbeam cantilevers. Despite having no section loss, the upper pins control the load rating and are governed by combined shear and bending. Note that the hangers at the first two panel points at each end are supporting the lower deck only, and thus, were not rated for vehicular loading.

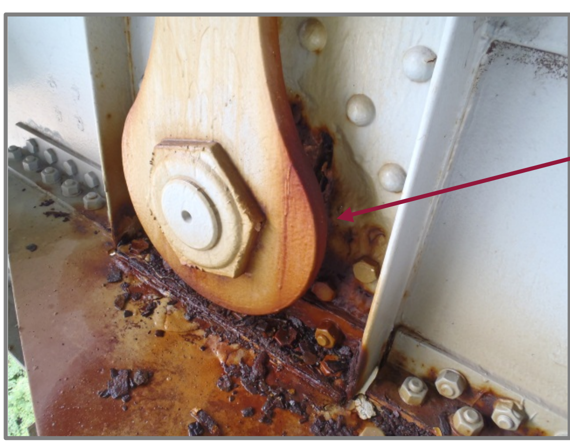


Photo 18 – East lower eyebar connection at L4 North. Note active corrosion in area and up to 3/8" pitting on pin plate.

In the as-inspected condition for Alternative #1, all of the operating rating factors remain above 1.10, with a governing HS20 Inventory rating factor of 0.96 for the South Truss upper pins (see Table 15). The remaining elements on the South Truss and all elements on the North Truss exhibit rating factors above 1.10. Because each panel point is subject to similar loadings in the suspended portion of the span, all of the upper pins have very similar rating factors, with a similar phenomenon also true for the lower pins and the eyebar hangers. The lowest operating rating factor remains above 1.10; as a result, Alternative #1 would be feasible with no structural rehabilitation necessary on the pins and hangers.

ALTERNATIVE #1: AS-INSPECTED PIN & HANGER CONTROLLING RATING FACTORS								
Location		Panel Points	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
North Truss	Upper Pin	9 / 9'	1.24	1.55	2.96	2.00	1.79	2.05
	Lower Pin	8 / 8'	2.07	2.66	5.04	3.41	3.07	3.50
	Hanger	8 / 8'	2.15	2.79	5.30	3.59	3.23	3.68
South Truss	Upper Pin	9 / 9'	0.96	1.31	2.50	1.69	1.51	1.73
	Lower Pin	8 / 8'	1.43	2.09	3.98	2.69	2.42	2.76
	Hanger	8 / 8'	1.77	2.66	5.06	3.42	3.08	3.50

Table 15 – Controlling as-inspected pin and hanger rating factors for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

In the as-inspected condition for Alternative #2, the primary upper chord pins (all except U12) have HS20 Inventory and Operating rating factors below 1.10, with isolated legal loads also exhibiting rating factors below 1.10 in the operating level (see Table 16). The governing component is the upper pin at U9 on the South Truss, which has an HS20 Inventory rating factor of 0.65 and Operating rating factor of 0.82. The lower pins and eyebar hangers on each truss have operating rating factors above 1.10. All common elements on each truss line have similar rating factors due to the consistent loading applied at each panel point. Because the upper pins have operating rating factors below 1.10 with no section loss noted, they would require strengthening through a structural rehabilitation in order to accommodate the added loads from Alternative #2. These rehabilitation efforts would be difficult to perform and would come at a substantial cost.

ALTERNATIVE #2: AS-INSPECTED PIN & HANGER CONTROLLING RATING FACTORS								
Location		Panel Points	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
North Truss	Upper Pin	9 / 9'	0.71	0.88	1.68	1.14	1.02	1.17
	Lower Pin	8 / 8'	1.59	1.99	3.78	3.69	2.30	2.62
	Hanger	8 / 8'	1.70	2.13	4.04	2.73	2.46	2.80
South Truss	Upper Pin	9 / 9'	0.65	0.82	1.56	1.05	0.94	1.08
	Lower Pin	8 / 8'	1.26	1.60	3.05	2.06	1.85	2.11
	Hanger	8 / 8'	1.60	2.17	4.13	2.79	2.51	2.86

Table 16 – Controlling as-inspected pin and hanger rating factors for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

Floorbeams and Stringers

The main span steel framing has undergone a series of modifications since original construction. The entire upper framing system was either modified or replaced during the 1965 Rehabilitation, including isolated locations where the original stringer was retained and a new, smaller rolled section was bolted to the top flange (see Photo 19). The second major rehabilitation occurred in 1994, which included full removal and replacement of the top concrete deck. In addition, numerous upper framing elements were modified at this time, including full replacement of Floorbeams 5 and 19, as well as several stringers. The lower framing system also received modifications during the 1994 Rehabilitation, including four floorbeams and all of the stringers in 6 of the 24 bays.

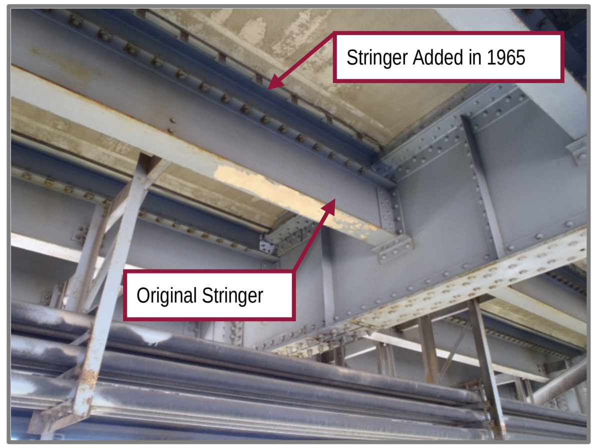


Photo 19 – Typical stringer configuration showing original stringer and additional stringer bolted to top flange.

All framing elements were rating using a combination of hand calculations and simple STAAD models, with analysis results and load rating equations typically processed in Excel spreadsheets. The load ratings for the upper floorbeams are not affected by the proposed lower deck modifications because these members are subject to the upper deck loadings only. As a result, the load ratings for each alternative are equal to those presented in the 2012 load rating performed by TranSystems. The upper deck floorbeams from Panel Point 6 to Panel Point 18 rate below 1.0 for HS20 Inventory. This governing HS20 Inventory rating factor is 0.93, which applies to Floorbeams 6 through 18 in the as-built condition. The rating factors for all floorbeams are above 1.10 for all other load cases (see Table 17).

AS-INSPECTED UPPER STEEL FLOORBEAMS CONTROLLING RATING FACTORS						
Location	HS20 Inventory	HS20 Operating	2F1 Operating	3F1 Operating	4F1 Operating	5C1 Operating
FB 0	1.46	2.32	4.14	2.95	2.80	3.04
FB 24	1.74	2.79	4.98	3.55	3.36	3.65
FB 1, FB 23	1.05	1.64	3.20	2.11	1.86	2.19
FB 2, FB 22	1.03	1.60	3.13	2.06	1.81	2.14
FB 3, FB 21	1.13	1.77	3.44	2.26	1.99	2.35
FB 4, FB 20	1.47	2.34	4.56	3.00	2.64	3.12
FB 5, FB 19	2.22	3.59	7.01	4.61	4.06	4.79
FB 6 to FB 18	0.93	1.44	2.79	1.83	1.62	1.91

Table 17 – Controlling as-inspected upper steel floorbeam rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

Many of the upper steel floorbeams exhibit heavy section losses, including isolated locations of 100% section loss to the web plates along upper framing lateral bracing connection plates at hanger locations. However, since the floorbeams are typically governed by positive bending at midspan, only the end floorbeams at Panel Points 0 and 24 are governed by as-inspected conditions. These floorbeams exhibit heavy section loss to the webs and 100% section loss to multiple transverse web stiffeners (see Photo 20). In addition, the bottom cover plates and flange angles exhibit isolated pitting which has been cleaned and painted. Despite these levels of deterioration, the as-inspected end floorbeams rate above 1.10 for all load cases, and the remaining floorbeams are controlled by as-built conditions with rating factors also above 1.10.

Photo 20 – West face of Floorbeam 0 with heavy losses to the web plate, web stiffeners and bottom cover plates.

The upper steel stringers consist of four different sections: The fascia stringers are all W36x135 wide flange sections, and the interior stringers consist of W30x99 sections or original W20x59 stringers with smaller wide flange sections bolted on to the top flanges during the 1965 Rehabilitation. All of the upper steel stringers rate above 1.0 in the as-built configuration (see Table 18). Similar to the upper floorbeams, the load ratings of these elements are not influenced by the proposed lower deck modifications.

AS-BUILT UPPER STEEL STRINGERS CONTROLLING RATING FACTORS								
Panel Number	Stringer Label	Section	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
1 to 4, 7 to 18	N	W20x59 + W10x33	1.42	2.37	3.46	2.51	2.34	2.68
21 to 24	N	W20x59 + W12x40	1.62	2.71	3.96	2.87	2.68	3.06
All	U	W36x135	5.90	10.03	14.67	10.62	9.92	11.34
5, 6, 19, 20	N	W30x99	3.28	5.48	8.02	5.81	5.42	6.20

Table 18 – Controlling as-built upper stringer rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

The stringers in the main span are in poor condition overall due to isolated heavy section losses in the stringer webs and bottom flanges. These section losses typically occur in stringers adjacent to the truss lines, as these stringers experience accelerated corrosion from rainwater passing through the deck at truss or eyebar locations (see Photo 21).

Photo 21 – Stringer P between Floorbeams 20 and 21 with 100% section loss in web and bottom flange.

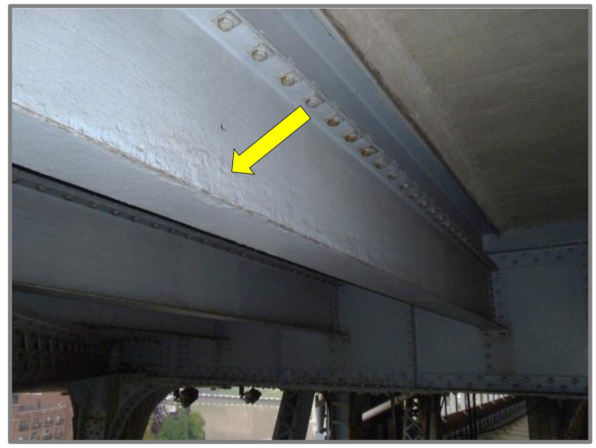


Photo 22 – Controlling Stringer H between Floorbeams 21 and 22. Note losses to web and bottom flange of original stringer.

The controlling as-inspected stringer rating factors for each type of cross section are shown in Table 19. Only one stringer rates below 1.10 for HS20 Inventory, while rating factors are above 1.10 for all other stringers and load cases. The controlling as-inspected stringer is Stringer H between Floorbeams 21 and 22, which consists of an original W20x59 stringer with an additional W12x40 bolted to the top flange (see Photo 22). Note that as-inspected rating factors were not calculated for fascia stringers, as these members typically had minimal section loss and high as-built rating factors.

AS-INSPECTED UPPER STEEL STRINGERS CONTROLLING RATING FACTORS								
Panel Number	Stringer Label	Section	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper
2	F	W20x59 + W10x33	1.19	1.98	2.90	2.10	1.96	2.24
22	H	W20x59 + W12x40	0.88	1.47	2.15	1.56	1.46	1.66
21	P	W30x99	1.80	3.01	4.40	3.19	2.98	3.40

Table 19 – Controlling as-inspected upper stringer rating factors (numbers below 1.0 are red, controlling values are shaded in yellow).

The lower deck is no longer used for regular vehicular traffic and has been converted into a multifunctional space for the City of Cleveland. Accordingly, rather than rate the lower framing members for live load vehicles, rating factors were developed for pedestrian loading only. A modified rating factor equation was used, which treats the pedestrian load as a dead loading rather than a live load. Floorbeams were modeled as continuous through the truss lines, and the last three floorbeams at each end were modeled like a truss due to presence of truss sway bracing members that frame into the floorbeams at intermediate support points (see Photo 23).

Photo 23 – West elevation of Lower Floorbeam 24. Note truss sway bracing which frames into the floorbeam between web plates.

As-built rating factors for pedestrian loading were calculated for all unique lower floorbeams during the 2012 load rating performed by TranSystems. As-inspected losses were applied to each floorbeam as noted during the field inspection. According the results of the previous load rating, Floorbeams 4 and 20 govern the as-built rating, and Floorbeam 20 controls the as-inspected rating due to the advanced section losses adjacent to the truss and eyebar locations. Because all of the floorbeams were being rated for pedestrian load only, the Inventory and Operating rating factors are all well above 1.0 despite the isolated heavy deterioration documented on the members (see Table 20). Note that the as-built and as-inspected rating factors for Floorbeams 5 and 19 are equal, as there were no significant section losses noted on these members.

EXISTING CONDITION: PEDESTRIAN LOADING LOWER STEEL FLOORBEAMS CONTROLLING RATING FACTORS					
AS-BUILT			AS-INSPECTED		
Location	Inventory	Operating	Location	Inventory	Operating
FB 0, FB 24	11.69	19.51	FB 24	11.08	18.49
FB 1, FB 23	12.65	21.11	FB 1	12.19	20.35
FB 2, FB 22	12.66	21.12	FB 22	11.99	20.02
FB 3, FB 21	8.47	14.14	FB 3	7.29	12.16
FB 4, FB 20	8.32	13.89	FB 20	5.01	8.36
FB 5, FB 19	9.89	16.51	FB 5, FB 19	9.89	16.51
FB 6 to FB 18	8.32	13.89	FB 13	5.47	9.13

Table 20 – Controlling as-built and as-inspected lower steel floorbeam rating factors under pedestrian loading (numbers below 1.0 are red, controlling values are shaded in yellow).

Because Floorbeam 20 was the governing element and exhibited large rating factors during the 2012 load rating analysis, only this floorbeam was analyzed for the proposed lower deck modifications. Even with the significant additional dead and pedestrian loads added to the structure in both alternatives, the lower floorbeams have large rating factors well above 1.0 (see Table 21). These floorbeams were originally designed for the capacity to carry streetcars on the lower deck, resulting in members that are now overdesigned even for the very substantial proposed loads in each alternative. Thus, the lower floorbeams have sufficient capacity for each alternative without any structural rehabilitation necessary.

PEDESTRIAN LOADING FLOORBEAM 20 CONTROLLING RATING FACTORS					
ALTERNATIVE #1			ALTERNATIVE #2		
Rating	Inventory	Operating	Rating	Inventory	Operating
As-Built	7.58	12.65	As-Built	5.12	8.55
As-Inspected	5.01	8.36	As-Inspected	3.79	6.33

Table 21 – Controlling as-built and as-inspected lower steel floorbeam rating factors for the proposed lower deck modifications (numbers below 1.0 are red, controlling values are shaded in yellow).

The lower deck stringers were also rated for only pedestrian loading. Lower deck stringers were labeled south to north from Stringer 1 to Stringer 12. Because pedestrian load was only applied to stringers supporting a pedestrian walkway, Stringers 3, 4, 9 and 10 were not rated for Alternative #1, as these members do not carry any applied loads. All stringers were considered for Alternative #2 because of the inclusion of a full width concrete deck.

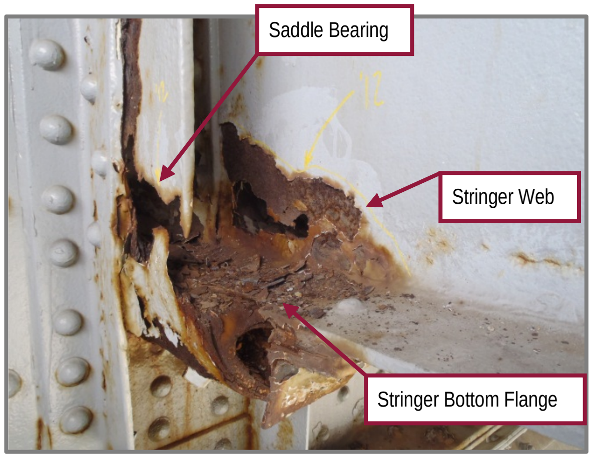


Photo 24 – Lower deck Stringer 10 at west face Floorbeam 21 saddle bearing. Note 100% section loss on the stringer web and bottom flange, as well as the saddle bearing.

The lower deck stringers comprise of two unique sections, original Bethlehem 24"x84# I-sections and new W24x68 rolled sections from the 1994 Rehabilitation. The most significant section losses occurred at the stringer ends at saddle bearing locations, where 100% section loss was commonly noted in stringer webs and bottom flanges (see Photo 24).

Despite these section losses, all of the lower deck stringers for Alternative #1 rate well above 1.0 for pedestrian loading in the Inventory and Operating conditions for Alternative #1 (see Tables 22 and 23). Note that the as-built load rating is governed by the exterior stringers, which are not affected by the proposed lower deck modification. However, the as-inspected ratings are affected by the additional dead loads over Stringer 8, which carries the existing steel open grid deck.

ALTERNATIVE #1: AS-BUILT LOWER STEEL STRINGERS RATING FACTORS			
Stringer Label	Section	Pedestrian Loading	
		Inventory	Operating
12	W24x84	6.64	11.09
12	W24x68	6.13	10.23

Table 22 – Controlling as-built lower stringer rating factors under pedestrian loading for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

ALTERNATIVE #1: AS-INSPECTED LOWER STEEL STRINGERS RATING FACTORS				
Panel Number	Stringer Label	Section	Pedestrian Loading	
			Inventory	Operating
7	8	W24x84	5.38	8.98
12	8	W24x84	6.09	10.16
22	8	W24x84	2.84	4.74

Table 23 – Controlling as-inspected lower stringer rating factors under pedestrian loading for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

Similarly, the as-built and as-inspected lower stringer ratings for Alternative #2 are well above 1.0 for under the proposed pedestrian loading (see Tables 24 and 25). Note that the as-inspected ratings for Alternative #2 are higher than those in Alternative #1. This is because the proposed bridge parapets are distributed over more stringers in the full width alternative.

ALTERNATIVE #2: AS-BUILT LOWER STEEL STRINGERS RATING FACTORS			
Stringer Label	Section	Pedestrian Loading	
		Inventory	Operating
2	W24x84	4.42	7.37
12	W24x68	4.83	8.07

Table 24 – Controlling as-built lower stringer rating factors under pedestrian loading for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

ALTERNATIVE #2: AS-INSPECTED LOWER STEEL STRINGERS RATING FACTORS				
Panel Number	Stringer Label	Section	Pedestrian Loading	
			Inventory	Operating
7	8	W24x84	5.38	8.97
12	8	W24x84	6.05	10.10
22	8	W24x84	2.95	4.92
21	2	W24x84	4.83	8.07

Table 25 – Controlling as-inspected lower stringer rating factors under pedestrian loading for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

GUSSET PLATES

The gusset plates were analyzed and load rated to account for their as-built and as-inspected conditions, utilizing the following specifications:

- ODOT Bridge Design Manual, 2004 Edition
- AASHTO Standard Specifications for Highway Bridges, 17th Edition 2002
- FHWA Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges, 2009

The gusset plates were analyzed for the HS20-44 truck and lane loads for inventory and operating levels, and for operating levels of the Ohio legal loads 2F1, 3F1, 4F1, and 5C1. ODOT's truck train configuration for spans over 200 feet long was considered. Pedestrian live load combinations were considered in the same manner as in the Main Truss analysis. The as-inspected analysis utilized findings noted during the 2012 In-Depth Bridge Inspection.

Panel points are numbered west to east from 0 to 24 and are symmetric about Panel Point 12 (at midspan). Gusset plate nomenclature consists of upper or lower (1 – upper, 2 – lower) and panel point number (0 through 24).

The Load Factor Rating was performed using a modified version of the 2009 Rating Excel spreadsheet provided by the ODOT Office of Structural Engineering that is based upon the FHWA Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges (FHWA-IF-09-014). Modifications to the Rating Excel spreadsheet are as follows:

1. Allows for vertical members to be non-perpendicular to the chord.

2. As-inspected losses columns were added to allow for individual losses for Tensile, Whitmore and Shear regions to be applied to each gusset.

3. Allows for three (3) connector yield strengths to be identified in truss member connections.

4. Allows for two (2) connector diameters to be identified in truss member connections.

5. Allows for connector shear check to be ignored where truss member bears directly on pin.

6. Allows for force reduction factors to be applied where splice plates are present.

7. Allows for buckling checks to be ignored where members are in tension only and where vertical members extend through chord connections.

Photo 25 – Typical lower chord gusset plate connection (L2 North shown).

Rating Assumptions

Chord members typically connect to the gusset plate outboard face while diagonal and vertical members connect to the inboard face (see Photo 25). Chord members terminating at gusset plates are also connected by additional web and flange splice plates.

Due to the non-standard member layouts, several Whitmore width and block shear capacity measurements intersect adjacent truss members. Additionally, global horizontal shear checks intersect through vertical truss members, making this an improbable failure mode.

Vertical member connections typically extend through the chords, eliminating tension planes from block shear capacity measurements. Additionally, these vertical members were not analyzed for local compression buckling.

The chords bear directly on pins at L0, L24, U0, U24 and L12 (midspan). At these locations, the chord connectors act in bearing and were not analyzed for shear.

No gusset plate is present at U12, which connects zero force members with a sliding plate connection (see Photo 26).

See Figure 21 for an overview of how the failure plane measurements were used in order to determine capacities utilized for each gusset plate.

Photo 26 – U12 Panel Point on the North Truss with zero force member restraint, looking north.

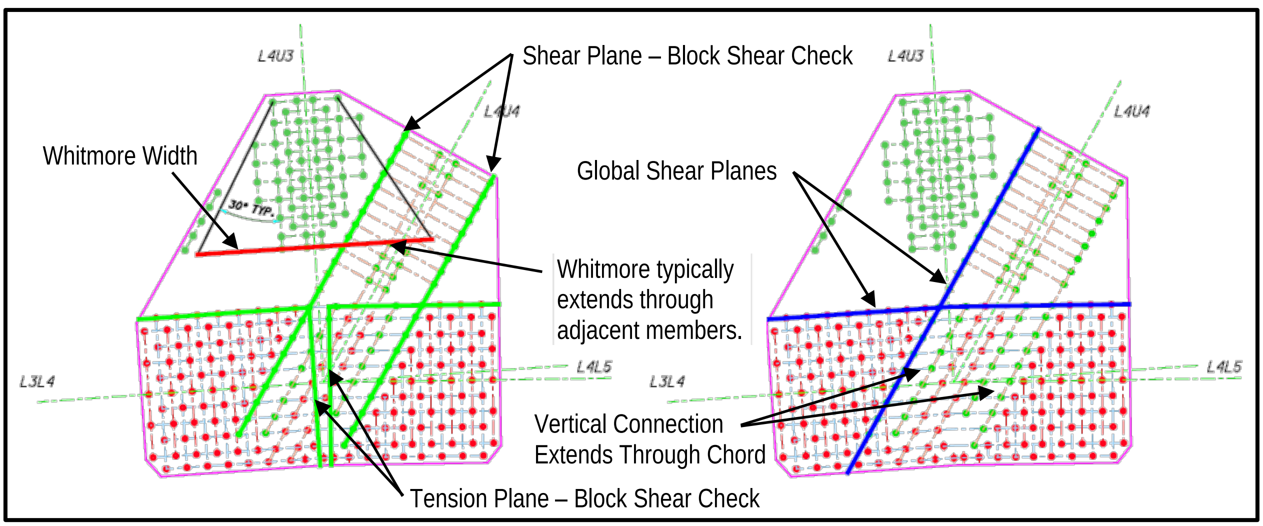


Figure 21 – Typical capacity measurements along lower gusset plates (south elevation of L4 shown).

As-inspected conditions were accounted for by applying percentage reductions to the appropriate capacity measurements illustrated in Figure 18. Gusset plate section loss was primarily noted along the interface of the bottom chord (see Photo 27), and less commonly along strut connections and vertical truss members. Where no significant section loss was noted during the 2012 in-depth inspection, a 5% reduction to the capacity was utilized along all failure planes to account for any gusset plates which may have cleaned and painted surface corrosion.

Photo 27 – Typical gusset plate section loss along bottom chord (L2 of the South Truss shown).

Below is a list of assumptions made for the rating of the gusset plates:

1. Gusset plates consist of Nickel Steel, with F_y=42 ksi and F_u=60 ksi
2. The connectors (rivets) consist of Carbon Steel, with F_y=30 ksi
3. The connector hole diameter was assumed to be 1/8" wider than the connector for net section calculations (AASHTO 10.16.14.6). Per the shop drawings, rivets are 7/8" diameter throughout except for the majority of lower chord rivets being 1" diameter. All truss joint field connections were reamed 1/4" while trusses were assembled at the shop.
4. Per the gusset plate shop drawings, chord members were milled to bear throughout.
5. Lateral constraints to gusset plates were ignored.
6. The design K value used in the analysis is 1.20.
7. Where splice plates are present, a calculated portion of the chord axial force (based on the capacity of each splice plate and splice plate connection) is transferred to the gusset plates in lieu of the full force.
8. Connector shear checks are ignored where truss members bear directly on pins.
9. Forces used in the rating are the envelope live load and dead load forces taken from the truss member rating.
10. Buckling checks were ignored where vertical member connections extend through chords. Buckling checks were also ignored for non-reversal tension members.
11. Ignored fill plates from capacity calculations.
12. 5% section loss was assigned to gusset plates with no significant section loss noted during the 2012 inspection.
13. Forces used in the rating are the maximum live and dead load forces taken from the Main Truss Analysis.

Analysis Results

From the previous assumptions, the following as-built rating factors were developed for the controlling upper and lower chord panel points for Alternative #1 (see Table 26). All of the as-built rating factors are governed by rivet capacity, with the North Truss controlling for HS20 loadings and the South Truss controlling for the remaining legal loads. The lowest HS20 rating factor for a gusset plate failure plane analysis was 4.25, indicating that the limiting factor for the gusset plates is the rivet capacity by a relatively wide margin. Analyses with pedestrian load considered controlled the load rating for each truss line. Because the load ratings for Alternative #1 are all above 1.10, no structural rehabilitation would be necessary on the gusset plates for this alternative.

ALTERNATIVE #1: AS-BUILT / AS-INSPECTED GUSSET PLATE CONTROLLING RATING FACTORS										
Truss	Chord	Panel Point	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck Train	5C1 Truck Train
North	Upper	U4 & U20	2.09	2.62	6.02	3.98	3.43	2.63	2.38	2.70
	Lower	L1 & L23	1.58	1.97	15.25	9.97	8.51	5.92	2.13	2.64
		L4 & L20	2.99	3.74	8.62	5.69	4.91	3.76	3.41	3.86
South	Upper	U4 & U20	1.84	2.31	5.30	3.50	3.02	2.33	2.01	2.31
	Lower	L1 & L23	1.69	2.11	13.68	9.02	7.78	5.87	2.21	2.76
		L4 & L20	2.50	3.13	7.18	4.74	4.10	3.15	2.73	3.14

Table 26 – Controlling as-built and as-inspected gusset plate rating factors for Alternative #1 (numbers below 1.0 are red, controlling values are shaded in yellow).

The same gusset plate locations that govern for Alternative #1 also control the load ratings for Alternative #2 (see Table 27). However, due to the increased dead load and pedestrian load from the full width deck in the second alternative, eight (8) gusset plate locations exhibit operating rating factors below 1.0. Because these gusset plates are controlled by the capacity of the rivets, this analysis applies to the as-built condition. Gusset plates L1 and L23 on the North Truss do not have the capacity to support the applied dead and pedestrian loads from this alternative; accordingly, the rating factor for these gusset plates is reported as zero (0). The deficient gusset plates would need to be rehabilitated in order to make this alternative feasible, which would include replacing the existing rivets on these plates with high strength bolts, and possibly also adding splice plates with additional fasteners to these members.

ALTERNATIVE #2: AS-BUILT / AS-INSPECTED GUSSET PLATE CONTROLLING RATING FACTORS										
Truss	Chord	Panel Point	HS20 Inv	HS20 Oper	2F1 Oper	3F1 Oper	4F1 Oper	5C1 Oper	HS20 Truck Train	5C1 Truck Train
North	Upper	U4 & U20	1.19	1.49	3.43	2.26	1.95	1.50	1.36	1.54
	Lower	L1 & L23	0	0	0	0	0	0	0	0
		L2 & L22	0.16	0.20	1.40	0.91	0.78	0.55	0.21	0.26
South	Upper	U4 & U20	1.16	1.45	3.32	2.19	1.89	1.46	1.26	1.45
	Lower	L1 & L23	0.36	0.45	3.57	2.34	1.99	1.37	0.47	0.58
		L2 & L22	0.63	0.78	5.69	3.72	3.17	2.21	0.81	1.00

Table 27 – Controlling as-built and as-inspected gusset plate rating factors for Alternative #2 (numbers below 1.0 are red, controlling values are shaded in yellow).

COST ANALYSIS

Based on the results of the load rating analysis, the project team was able to develop conceptual cost estimates for each alternative based on the proposed lower deck modifications and necessary levels of structural rehabilitation in order to accommodate such replacements. The preliminary evaluation and analysis of each alternative was performed to determine base costs for a planning-level cost estimate, as well as the advantages and disadvantages provided by each alternative. Note that these conceptual estimates only cover the costs associated with the structural rehabilitation of the existing bridge and installation of the reinforced concrete lower deck. Additional costs such as lighting, earthwork, overlooks, aesthetic improvements benches, signage, safety features and other amenities were not included.

ALTERNATIVE #1

This alternative includes replacement of the steel open grid deck with a reinforced concrete deck within the same footprint (19'-0" wide) (see Figure 22). The deck configuration constitutes a path that is confined to the area between the interior column lines in the concrete approach spans and between the truss lines in the steel main span. The fiberglass reinforced grating will remain on the lower floorbeam cantilevers and will not be included in the bike path area for this alternative. In addition, see Appendix A for a 3D rendering of this proposed option.

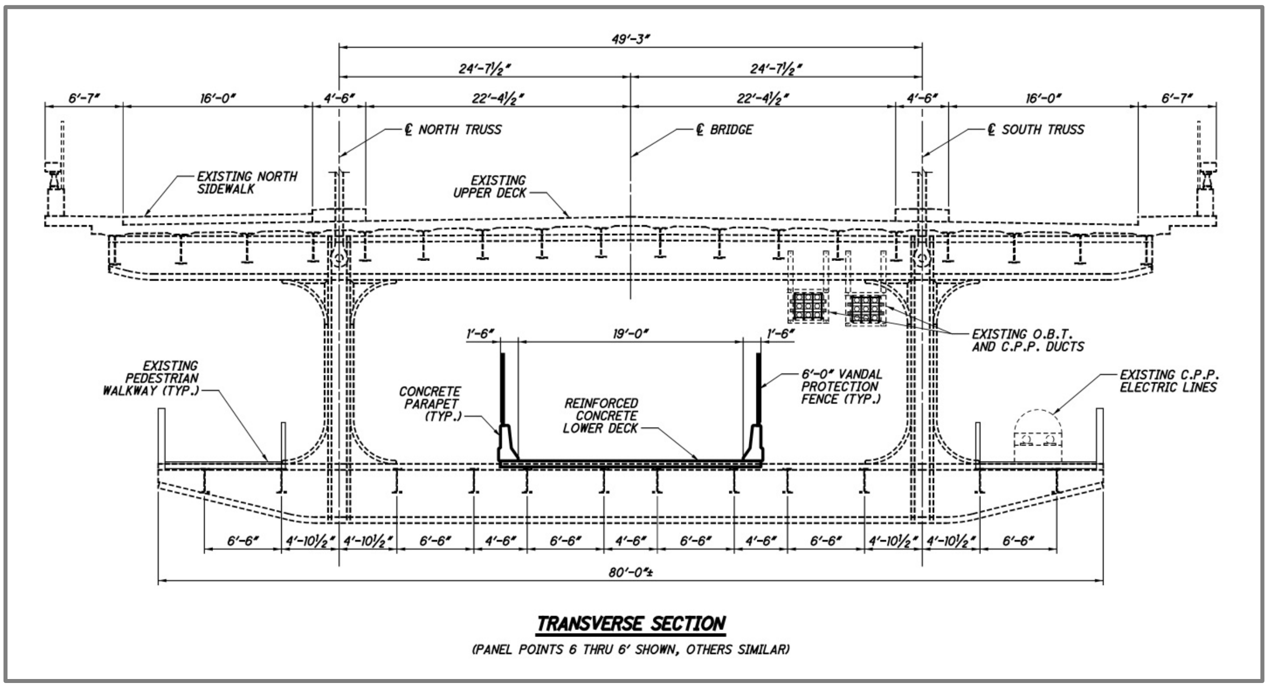


Figure 22 – Alternative #1 proposed transverse section.

Based on the structural evaluation of Alternative #1, minor structural rehabilitation would be necessary. Nine (9) lower chord members in the main steel span would require strengthening due to the added dead load from the reinforced concrete deck. The remaining steel members are sufficient in their current condition for this proposed alternative, and the concrete arch spans would be unaffected by Alternative #1 because pedestrian and bicycle traffic could travel on the existing center bay without major modifications.

Advantages

- Necessary structural rehabilitation would be rather minor, consisting of lower chord strengthening on nine (9) members.
- No significant work would be necessary in the concrete approach spans.
- The work would be performed below the upper deck, minimizing the impact on existing vehicular and pedestrian traffic.
- The lower deck in the steel main span would remain open between the center concrete deck and the existing pedestrian walkways, providing a better view for pedestrians and bicyclists.
- The existing Cleveland Public Power electric lines on the south walkway would not be disturbed.
- Construction cost would be significantly lower than Alternative #2.

Disadvantages

- Alternative #1 would provide significantly less area for pedestrians and bicycle traffic than Alternative #2.
- Limited capacity for public events due to space constraints in the main span.

Cost Analysis

The total estimated construction cost of the partial width deck described in Alternative #1 would be approximately $1,967,000. This would include the proposed deck modifications in the main steel span and the necessary structural rehabilitations on the lower chord members. A cost breakdown is presented below in Table 28.

ALTERNATIVE #1: 19'-0" LOWER DECK						
Estimated Construction Cost						
ITEM	EXT	QTY	UNIT	DESCRIPTION	2014 UNIT COST	2014 COST
202	11002	1	LUMP	PORTIONS OF STRUCTURE REMOVED, OVER 20 FOOT SPAN	$75,000	$75,000
509	10000	19050	POUND	EPOXY COATED REINFORCING STEEL	$2	$38,100
511	34444	240	CU YD	CLASS QC2 CONCRETE, BRIDGE DECK	$800	$192,000
513	95030	9	EACH	STRUCTURAL STEEL, MISC.: LOWER CHORD STRENGTHENING	$12,500	$112,500
517	73200	3368	FT	RAILING (DEFLECTOR PARAPET TYPE)	$175	$589,400
607	39900	3368	FT	VANDAL PROTECTION FENCE, 6' STRAIGHT, COATED FABRIC	$85	$286,280
624	10000	1	LUMP	MOBILIZATION	$120,000	$120,000
					SUBTOTAL	$1,413,280
					20% CONTINGENCY	$282,656
					2014 TOTAL	$1,696,000
				FUTURE WORTH, AT AN INFLATION RATE OF 3%	2019 TOTAL	$1,967,000
NOTE: THESE QUANTITIES ARE NOT FINAL						

Table 28 – Alternative #1 estimated construction cost.

ALTERNATIVE #2

This option includes the implementation of a full width (80'-0" wide) reinforced concrete deck for pedestrian and bicycle traffic on the lower deck of the structure (see Figure 23). The open grid steel deck and both fiberglass reinforced pedestrian walkways will be removed in the steel arch main span. In addition, a reinforced concrete deck would be included for the concrete arch spans between the northern exterior and northern interior column lines, as this section of deck was removed in 1994. In addition, see Appendix B for a 3D rendering of this proposed option.

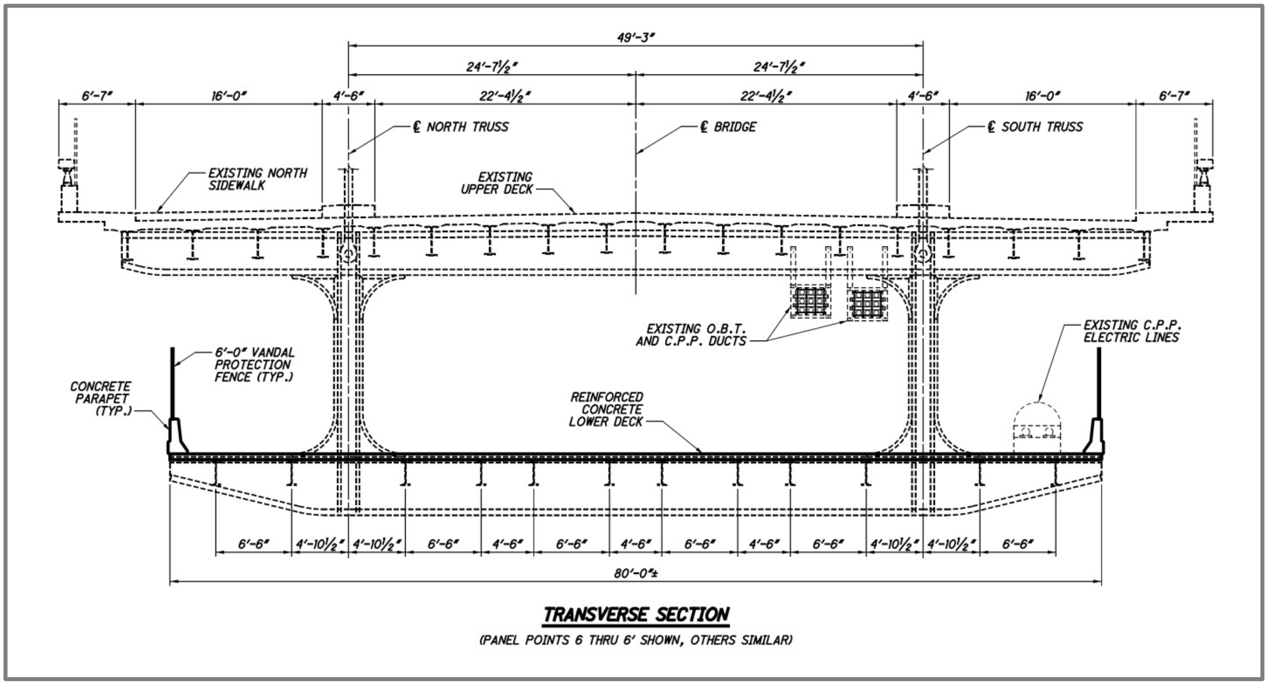


Figure 23 – Alternative #2 proposed transverse section.

Based on the structural evaluation of Alternative #2, substantial rehabilitation efforts would be necessary in order to make this option feasible. Due to the significant dead and pedestrian loads that would be added to the bridge within this alternative, structural rehabilitations would need to be performed on several of the lower chord members, upper pins and gusset plates in the steel main span. No structural rehabilitation would be needed in the concrete arch spans beyond the physical installation of the reinforced concrete deck between Arches A and B.

Advantages

- Increased area for bicyclists and pedestrians throughout the lower deck of the structure.
- Added capacity for public events with access to all areas on the lower deck of the structure.
- Increased potential for benches and greenery on the lower deck.
- Great view of Cleveland from the north and south sides of the bridge.

Disadvantages

- Substantial structural rehabilitations necessary to make this option feasible, including rehabilitation of 31 lower chord members, 24 upper pin locations, and 8 gusset plates in the steel main span.
- Additional construction efforts necessary in the concrete approach spans.
- Isolated lower chord and upper pin members would require construction work above the main deck, resulting in large maintenance of traffic efforts.
- The current view of the Cuyahoga River through the lower framing would now be lost.
- The existing Cleveland Public Power electric lines would need to be either relocated or protected from the public.
- Significant utility coordination and relocation would be necessary.
- Construction costs would be significantly higher than Alternative #1.

Cost Analysis

The total estimated construction cost of the partial width deck described in Alternative #1 would be approximately $7,128,000. This would include the proposed deck modifications in the main steel span and concrete approach spans, as well as the necessary structural rehabilitations on the lower chord members, upper pins and gusset plates. A cost breakdown is presented below in Table 29.

ALTERNATIVE #2: FULL WIDTH LOWER DECK						
Estimated Construction Cost						
ITEM	EXT	QTY	UNIT	DESCRIPTION	2014 UNIT COST	2014 COST
202	11002	1	LUMP	PORTIONS OF STRUCTURE REMOVED, OVER 20 FOOT SPAN	$150,000	$150,000
509	10000	142090	POUND	EPOXY COATED REINFORCING STEEL	$2	$284,180
511	34444	1790	CU YD	CLASS QC2 CONCRETE, BRIDGE DECK	$650	$1,163,500
513	95030	31	EACH	STRUCTURAL STEEL, MISC.: LOWER CHORD STRENGTHENING	$32,000	$992,000
513	95030	24	EACH	STRUCTURAL STEEL, MISC.: UPPER PIN STRENGTHENING	$50,000	$1,200,000
513	95030	8	EACH	STRUCTURAL STEEL, MISC.: GUSSET PLATE STRENGTHENING	$80,000	$640,000
517	73200	1182	FT	RAILING (DEFLECTOR PARAPET TYPE)	$175	$206,850
607	39900	3368	FT	VANDAL PROTECTION FENCE, 6' STRAIGHT, COATED FABRIC	$85	$286,280
614	11000	1	LUMP	MAINTAINING TRAFFIC	$80,000	$80,000
624	10000	1	LUMP	MOBILIZATION	$200,000	$120,000
					SUBTOTAL	$5,122,810
					20% CONTINGENCY	$1,024,562
					2014 TOTAL	$6,148,000
				FUTURE WORTH, AT AN INFLATION RATE OF 3%	2019 TOTAL	$7,128,000
NOTE: THESE QUANTITIES ARE NOT FINAL						

Table 29 – Alternative #2 estimated construction cost.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the results of the Structural Evaluation Report for the Detroit-Superior Bridge, the project team has researched and evaluated the feasibility of two alternatives for the addition of a shared use pedestrian and bike path on the structure. This work included the development of two alternatives, load rating calculations on the affected structural members for each alternative case, and a construction cost estimate for each alternative, as well as conceptual sketches and renderings for each alternative.

Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load ratings require engineering evaluation in determining a rating value that is applicable to maintaining the safe use of a bridge and arriving at posting and permit decisions. A rating factor of less than 1.00 indicates that the structure does not have sufficient capacity to carry the specified loading. TranSystems performed a structural analysis and load rating on the as-built and as-inspected structure for each alternative, considering the additional dead and pedestrian loads applied to the structure as a result of each alternative.

Under Alternative #1, the existing lower deck of the Detroit-Superior Bridge would upgraded to provide a more suitable riding surface and safer conditions for pedestrians and bicyclists on the structure. This lower deck modification would occur within the same footprint as the existing 19'-0" wide lower deck. According to the load rating analysis, select lower chord members would be structurally deficient in their as-inspected condition as a result of the additional loads being applied to the structure. Structural rehabilitation on these members would be generally minor and would occur below the main deck. No maintenance of traffic will be required for this alternative.

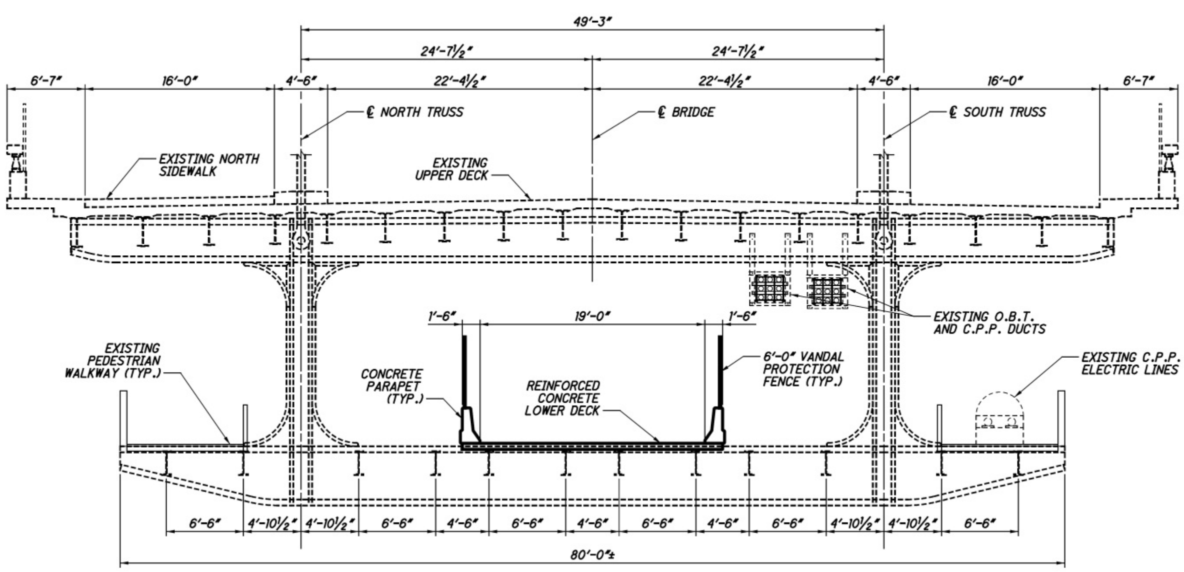
Under Alternative #2, a full width reinforced concrete deck would be provided on the lower deck of the Detroit-Superior Bridge. This option would include removal of the steel open grid deck and both pedestrian walkways in the steel main span, as well as an additional reinforced concrete deck in the north bay of the concrete approach spans. According to the load rating analysis, numerous components in the steel main span would be structurally deficient under the loads proposed in this alternative. Structural rehabilitation efforts would include 31 lower chord members, 24 upper pins and 8 gusset plate locations. Maintenance of traffic will be required for this alternative, as much of the work would occur over the main deck.

The alternatives and their associated construction cost estimates are as follows:

Alternative Description	2014 Cost	2019 Cost*
Alternative 1 - Partial Width Deck	$1,696,000	$1,967,000
Alternative 2 - Full Width Deck	$6,148,000	$7,128,000

*Cost based on 3% annual inflation

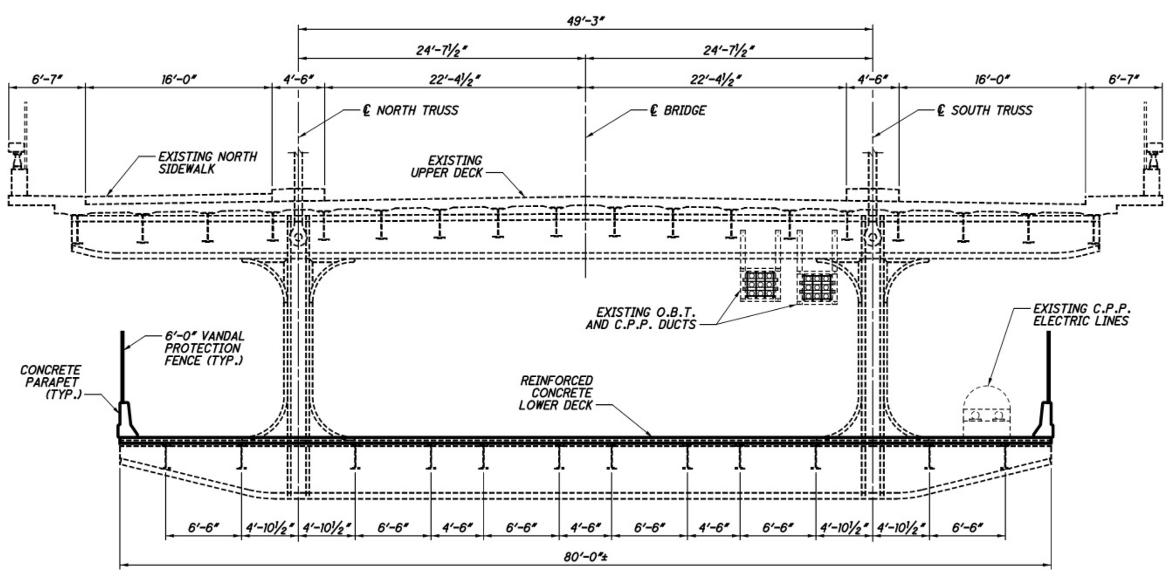
Appendix A
Conceptual Rendering of Alternative #1



TRANSVERSE SECTION

(PANEL POINTS 6 THRU 6' SHOWN, OTHERS SIMILAR)

Appendix B
Conceptual Rendering of Alternative #2



TRANSVERSE SECTION

(PANEL POINTS 6 THRU 6' SHOWN, OTHERS SIMILAR)