The Ironto Wayside Footbridge:
Historical Survey and Structural Analysis

Scope of study

The Ironto Wayside Footbridge is the oldest standing metal bridge in Virginia. The bowstring arch-truss is a unique design that combines a tied arch with diagonal truss elements integral with the top and bottom arch chords as shown in Figure 1.

Figure 1 Ironto Wayside Footbridge

This study consists of a historical survey of the bridge through a literature review and a structural analysis of the current condition of the bridge using modern analysis methods. The results of the analysis are verified through a load test.

Literature Review: History of the footbridge

In order to construct a complete history of the Ironto Wayside footbridge, several sources were used. Most of these were made available at the Virginia Transportation Research Council (VTRC) in Charlottesville, Virginia. The bridge now known as the Ironto Wayside footbridge was constructed in Bedford County in 1878. It was originally located in the south side of the county and was moved to Route 637 to cross a small stream called Roaring Run just a few years later (Newlon, 1973).

The footbridge is a bow-string arch truss, a system patented by Squire Whipple in 1841 (Miller and Clark, 2007). In 1866, Zenas King of the King Iron and Bridge Company in Cleveland, Ohio submitted a patent for the design used in this particular span called the "Tubular Arch Bridge." The company was commissioned to construct six similar bridges in Bedford County to replace bridges that had been destroyed in a flood (Diebler, 1976).

The bridge served almost one hundred years over Roaring Run. In 1972, it was replaced by a steel pipe culvert. The VTRC took notice of span because if its historical significance and began plans to move it to the Wayside, a state operated rest stop off I-81 N at mile 129, in 1976 (Miller and Clark, 2007). VTRC was careful to consider the historical integrity in addition to the structural integrity of the bridge during the planning process for the relocation. Much work was

put into determining the original paint scheme and recreating it once the bridge was in place. The new site was selected to maintain the function of the bridge by spanning a topological depression. Also, to ensure that no pieces were lost, the reassembling was done directly after it was dismantled (Diebler, 1976).

Figure 2 The bridge in Bedford County, VA with flooring system removed

Modeling the Bridge in 2D

After gathering historical data on the bridge, a 2D computer model was created using SAP2000 (2008) so that the structure could be analyzed. All dimensions for the bridge were taken from the survey of the bridge before it was moved to the rest stop. The complete survey can be found in Attachment 1.

The arch is modeled as eight linear segments continuous for bending, shear and axial force. It is held in compression by a tension tie rod that has a rectangular cross-section. The ends of the tied arch system are pin-roller supported. As seen on page 2 of the attachment, there is a unique bracing system perpendicular to the truss that restrains lateral movement of the arch with a force-couple about the *x*-axis. For 2D purposes, it was not necessary to include this system because no forces would be applied or induced in the out-of-plane direction.

The floor stringer is elevated 12” above the tie rod and is pin-roller supported. It is connected to the arch by three evenly-spaced cables with a diameter of 0.798”. The bridge also has cross bracing members (Figure 1). It is assumed that the main purpose of the cross braces is to prevent longitudinal motion of the bridge deck as it hangs from the vertical cables. All the loads applied to the model were strictly vertical, so the cross braces were not considered in the structural analysis. The horizontal tie rod is supported at each of the vertical cables, so each vertical pair of

tie and stringer nodes are constrained in the *z* direction (i.e., the stinger and tie nodes move together in *z* while maintaining 12” spacing).

Table 1 Section Properties

Section	Label	Cross-sectional Description
Arch	A	5 x 5 x ¼ square tube
Floor Stringer	B	7” I x 3⅝” x ¼”
Tie Rod	C	½” x 3” rectangle
Cables	D	Diameter = 0.798”

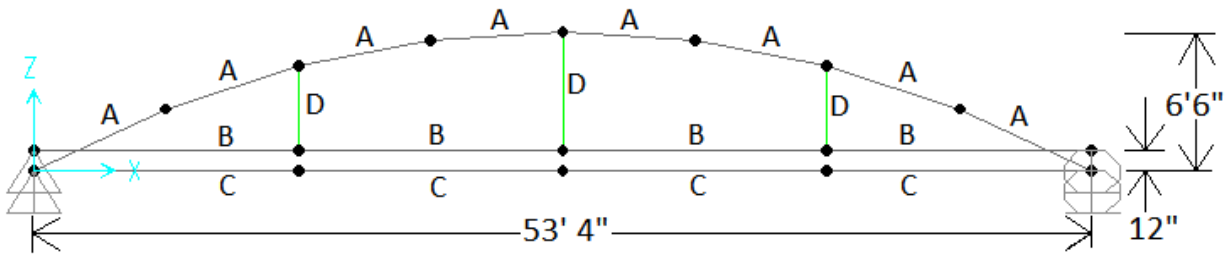


Figure 3 Single truss model created in SAP2000

All members of the bridge are wrought iron. The material profile employed in SAP2000 is listed in Table 2.

Table 2 Material properties of Wrought Iron

Weight per unit volume	485 pcf
Modulus of Elasticity, E	28,000 ksi
Poisson’s Ratio, U	0.3
Coefficient of Thermal Expansion, A	6.4×10^{-6}
Minimum Yield, F_y	23 ksi
Minimum Tensile Stress, F_u	28 ksi

<u>Analysis</u>

Several loads were selected for the analysis. First was the dead load based on the self-weight of the members. For the live loads applied, values based on historical documents were used. A uniformly distributed load of 75 psf for ordinary country bridges shorter than 60 ft was taken from the *Practical Treatise on the Construction of Iron Highway Bridges* (Boller, 1893) Also, point loads associated with a 6 ft wide by 8 ft long wagon weighing 5 tons were applied (Kulicki and Stuffle, 2006).

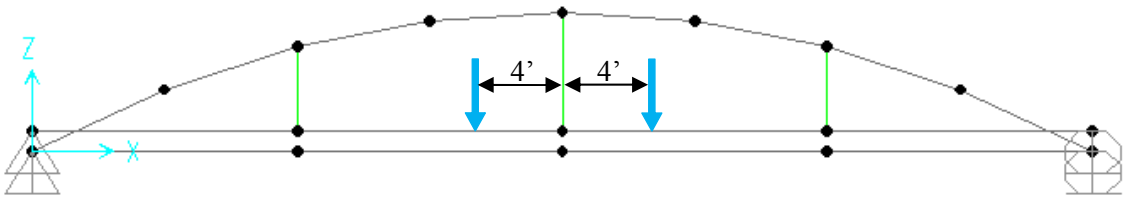


Figure 4 Location of wagon wheel loads (blue arrows)

An analysis for each load was run separately through SAP2000 and the axial forces and moments in the arch and tie rod were recorded. The maximum deflection of the midspan of the arch and the base of the floor stringers were also noted.

The axial and bending stresses were calculated using Microsoft Excel (Microsoft, 2007)and the following equations:

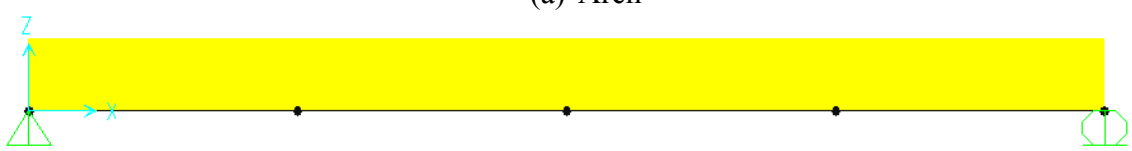
$$\sigma_{axial} = \frac{P}{A},\ \sigma_{bending} = \frac{Mc}{I} \tag{1}$$

where P is the axial force (+ is tension and – is compression), M is the bending moment in the member, A is the cross-section area and I the moment of inertia (about the y-axis), and c is the distance from the member centroid to the extreme fiber (assuming a symmetric section).

Results

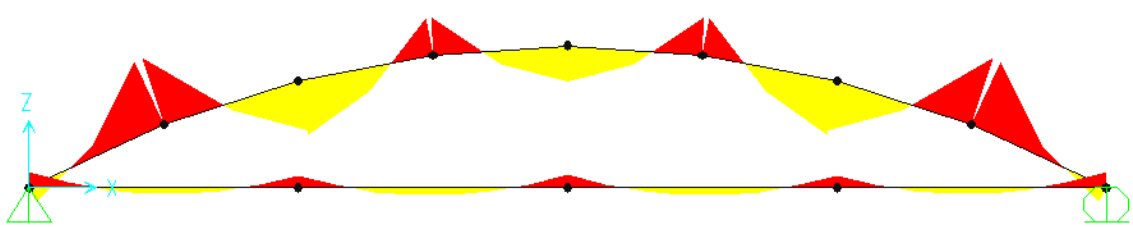
The axial force and moment diagrams are shown for each load case in Figure 5 to Figure 7. Red areas indicate compression and yellow areas indicate tension.

(a) Arch

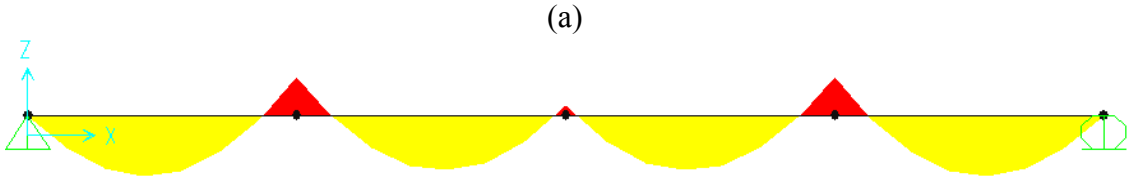


(b) Tie rod

Figure 5 Axial force in arch and tie rod for all load patterns



(a)



(b)

Figure 6 Moment diagrams of tied arch and floor stringer under uniform or dead loads.

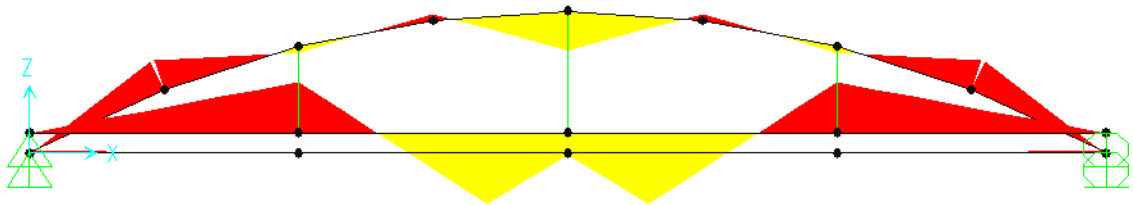


Figure 7 Moment diagram under wagon loads.

The stress results are summarized in Table 3 to Table 5. The maximum stresses in the arch were found at the node between the two segments at the ends of the arch. The maximum stresses in the tie rod were at the ends for the dead and uniform load and in the middle of the span for the wagon loads.

Table 3 Maximum stresses in arch

Load	σ_{axial} (ksi)	$\sigma_{bending}$ (ksi)	σ_{top} (ksi)	σ_{bottom} (ksi)	Location
Dead	-0.5	0.7	0.2	-1.2	Node between two segments at the end of the arch
Uniform	-5.7	13.4	7.7	-19.1	
Wagon (5 ton)	-1.8	5.8	4.0	-7.6	
Wagon (3.5 ton)	-1.3	4.1	2.8	-5.3	

Table 4 Maximum stresses in tie rod

Load	σ_{axial} (ksi)	$\sigma_{bending}$ (ksi)	σ_{top} (ksi)	σ_{bottom} (ksi)	Location
Dead	0.6	1.4	2.0	-0.7	end
Uniform	7.6	6.4	14.0	1.1	end
Wagon (5 ton)	2.4	2.8	5.2	-0.5	center
Wagon (3.5 ton)	1.7	2.0	3.7	-0.3	center

Table 5 Deflections at midspan in inches

Load	**Arch**	**Tie rod**
Dead	-0.071	-0.070
Uniform	-0.948	-0.920
Wagon (5 ton)	-0.573	-0.561
Wagon (3.5 ton)	-0.401	-0.393

The deflected shape of the truss under uniform loading (Figure 8) is the same for the dead load. Similarly, the deflected shape of the truss with the 3.5 ton wagon load is the same as the 5 ton wagon load (Figure 9). The figures below show the deflected shape scaled up 50 times.

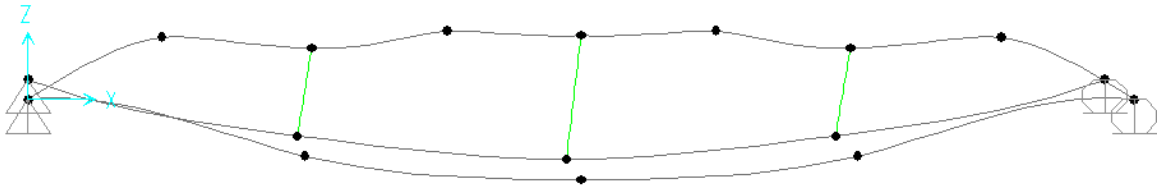


Figure 8 Deflected shape of truss under uniform load

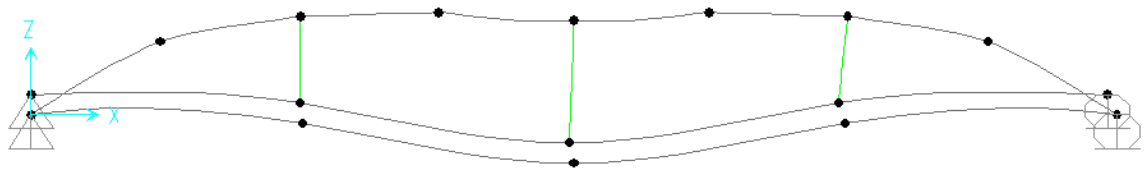


Figure 9 Deflected shape of truss under wagon load

Discussion

The primary structural system for this bridge is a tied arch, where the applied loads are carried through the arch in compression to the supports. The tie rod is designed to withstand the high tension forces created as it prevents the arch from "flattening out".

The bending stresses are higher than the axial stresses in the arch and tie rod. This is because the force on the floor is transferred vertically through the cables to the arch as a point load. (Arches are not as efficient at carrying point loads as they are at carrying uniform loads.) The tie rod has a low moment of inertia, so it does not have much resistance to the moments that are transferred to it from the interaction with the floor system. Therefore the bending stress is almost double the axial stress. The axial forces in the floor stringers are equal to zero in the model because the roller end of the member is free to move in towards the structure until it reaches equilibrium.

The uniform load creates the highest stresses and highest deflections. As noted in Table 1, the minimum yield stress for wrought iron is 23 ksi. The stress induced on the bottom of the arch by the uniform load is 19.1 ksi in compression which is 84% of the maximum yield stress. Generally, stresses are limited in a modern bridge to 60% of the yield stress. In its current location, the bridge will most likely never see such a high uniform load because pedestrian traffic over it is neither constant nor high enough and there is no way for water to pool on the wooden decking.

The deflected shape is very different between the uniform loads and the wagon loads. The uniform load and dead load result a generally parabolic deflected shape. The wagon loads cause a sharp deflection down towards the midpoint of the span. This is because the point loads from the wheels of the wagon are four feet on either side of the middle vertical cable, so the deflection is highest where the load is concentrated. Also, it should be noted that the spacing between the floor stringers and the tie rod is maintained through each deflection because of the constraint included in the model.

Extension of the model from 2D to 3D

The model was extended from two to three dimensions so that the behavior of the bridge as an entire unit could be studied. Particularly important to the behavior in three dimensions are the triangular bracing systems at each hanger that keep the arch from buckling in the out-of-plane direction. Also, the diagonal braces were added to the 3D model to evaluate their influence on bridge deflection. The model, created in SAP2000, is shown in Figure 10 below. The dimensions of each member can be found in Attachment 1.

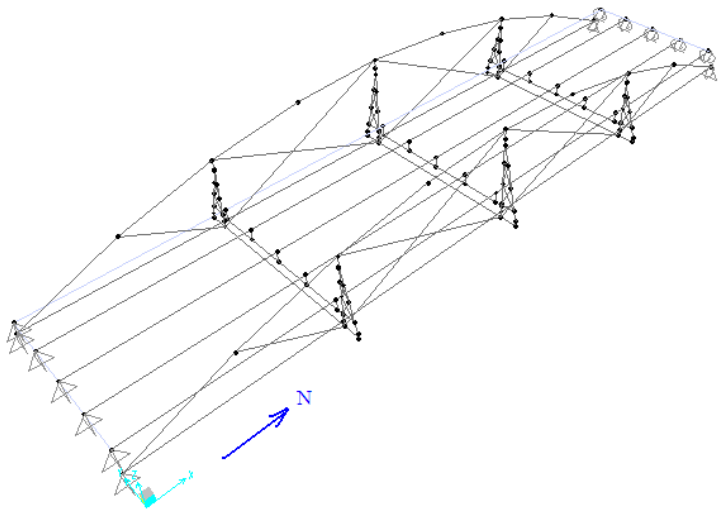


Figure 10 Three-dimensional model of the bridge in SAP2000

Each arch, tie, and floor stringer are pin-roller supported. The tie is supported by the three vertical hangers and the floor stringers rest on the floor beams which connect to the vertical hangers. The cross braces are modeled as cylindrical frame elements with no self weight and moment releases at the connections to the arch and vertical hangers.

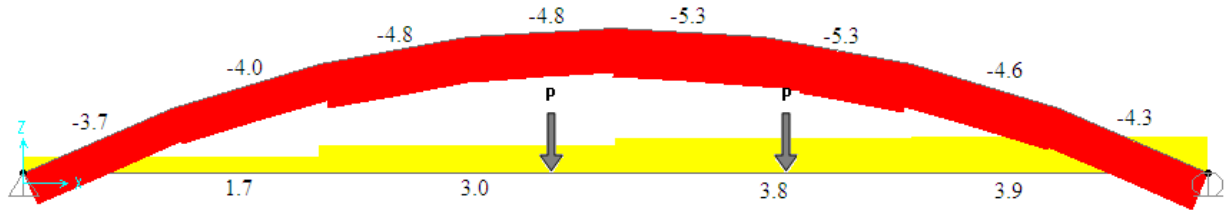
Analysis

A load representing a GMC pick-up truck loaded to a gross weight of 3.06 tons was applied to the model. The truck has a wheelbase of 133 inches and axle widths of 66 inches and the weight is dispersed evenly with 1530 lbs on each axle. The maximum deflection of the midpoint of the span was found to be 0.1028 inches downward. The resulting stresses are summarized in Table 1 below. Positive numbers indicate tensile and negative numbers indicate compressive forces.

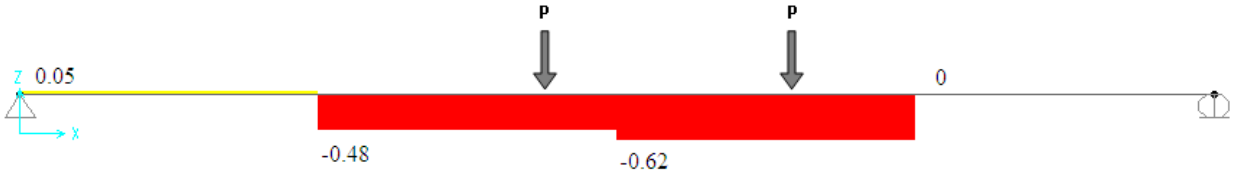
Table 6 Summary of maximum stresses in the arch, tie rod, and stringers under load from 3.06 ton truck

	σ_{axial} [ksi]	$\sigma_{bending}$ [ksi]	σ_{top} [ksi]	σ_{bottom} [ksi]	Location
Tie	1.20	1.28	2.48	-0.08	End
Arch	-0.91	2.89	1.98	-3.80	Center
Stringer	0.238	4.261	4.500	-4.023	Under Load

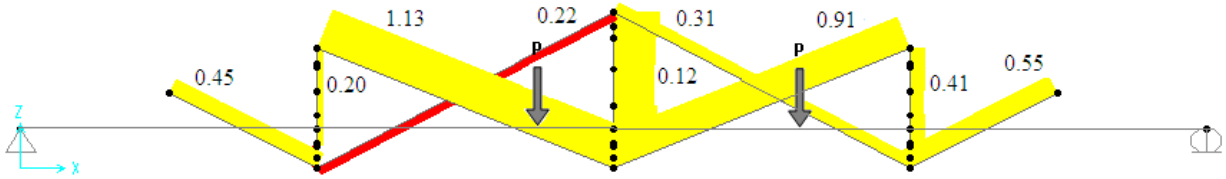
The axial force and moment diagrams are shown for each load case in Figure 11 and Figure 12. Red areas indicate compression and yellow areas indicate tension.



(a) Arch and tie rod

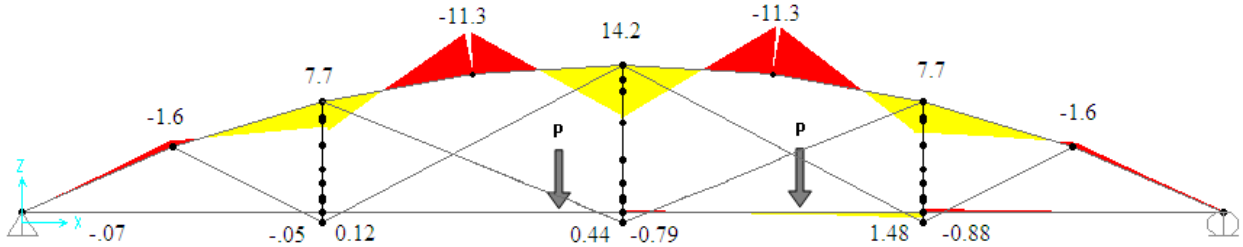


(b) Stringer supporting load

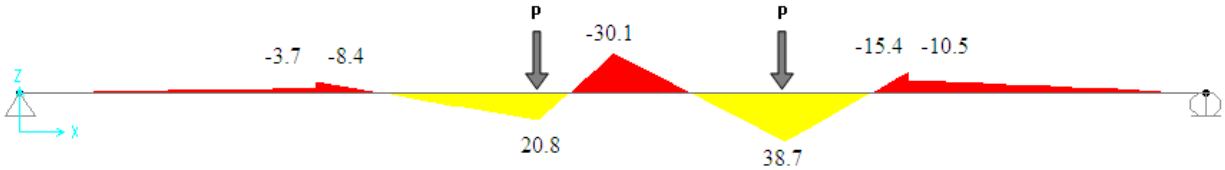


(c) Cross-braces

Figure 11 (a-c) Axial force diagram of truss members. Axial forces shown are in kips.



(a) Arch and tie rod



(b) stringer

Figure 12 Moment diagram of truss members. Moments shown are in kip-inches.

Load test

In order to verify the accuracy of the computer model, a load test was planned and carried out at the beginning of December 2009. The deflection at the center of each truss was recorded as a three ton truck was driven across the bridge, pausing every five feet to measure the change in deflection.

Plan

The Virginia Department of Transportation (VDOT) Salem District Structure and Bridge Division was contacted to perform a non-destructive load test on the footbridge. The two main concerns with the test were yielding of the materials and strength of the connections. The model confirms that the stresses induced by the three ton truck are well below the yield stress of wrought iron.

Hal Coleman, District Safety Inspection Engineer at VDOT Salem, looked over the bridge to provide a professional opinion of the bridge's readiness for a test. He found that the primary truss members, connections, and floor beams are in good condition for a structure the age of the bridge. This is likely due to the fact that it does not receive the deicing salts that vehicular structures do. There were several stringers that had section loss of up to 75% on the top flange near the midspan, but after a rough analysis deemed them suitable to supporting up to 8 tons. His recommendation was the three ton load would stay well within the range of the structure and avoid causing damage to the wooden flooring system.

The equipment for the test can be described in two parts. First, a weight was hung from the lateral strut directly below the center of each arch. Second, a dial gage, mounted on a magnetic stand and securely attached to a concrete block, was placed on the concrete streambed below the weights. One dial gage was analog, accurate to 0.001" and the second dial gage was digital, accurate to 0.0001". Steel angles were used to prevent the weights from swaying during the test.

Figure 13 Test set up with dial gage, weight, and steel guides

Figure 14 Load test in progress. The truck is centered transversely across the width of the bridge and two dial gages with stands are below the midpoint of each truss.

Measurements were taken to determine the centerline of the bridge and a tape measure was put out on the bridge to guide the truck as it drove across. The truck was then driven from the northern to southern abutment, stopping every five feet, plus once where the center of gravity of the truck was over the midpoint of the span. At each stop, the deflection of each arch was recorded. The test was repeated to verify the behavior of the bridge.

Results

The deflection of both trusses was measured as the truck was driven across the bridge, pausing every five feet. The results are summarized with the predicted results from SAP2000 in Table 2. Positive values indicate downward deflection. The maximum deflections for each trial are highlighted.

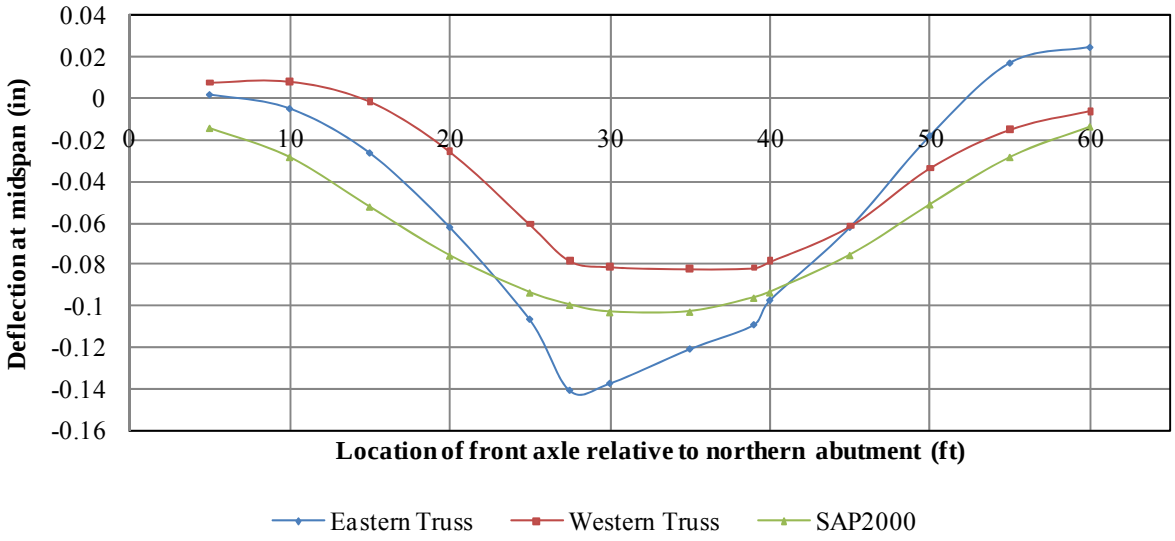


Figure 15 Deflection at midspan of bridge

Table 7 Deflection at bridge midspan

Location of front axle	Trial 1		Trial 2		Average		SAP Results
	Western	Eastern	Western	Eastern	Western	Eastern	
[ft]	[in]	[in]	[in]	[in]	[in]	[in]	[in]
5	0.0010	-0.005	-0.0048	-0.010	-0.002	-0.008	0.0146
10	0.0080	-0.006	0.0017	-0.010	0.005	-0.008	0.0285
15	0.0293	0.006	0.0230	-0.003	0.026	0.002	0.0523
20	0.0629	0.025	0.0611	0.026	0.062	0.026	0.0757
25	0.1069	0.054	0.1057	0.067	0.106	0.061	0.0934
27.5	0.1436	0.077	0.1375	0.079	0.141	0.078	0.0993
30	0.1410	0.080	0.1334	0.082	0.137	0.081	0.1028
35	0.1233	0.078	0.1180	0.086	0.121	0.082	0.1026
39	0.1071	0.078	0.1108	0.085	0.109	0.082	0.0959
40	0.0842	0.073	0.1102	0.084	0.097	0.079	0.0933
45	0.0198	0.039	0.1042	0.084	0.062	0.062	0.0755
50	0.0108	0.018	0.0250	0.049	0.018	0.034	0.0512
55	-0.0219	0.004	-0.0124	0.026	-0.017	0.015	0.0285
60	-0.0204	-0.002	-0.0291	0.014	-0.025	0.006	0.0139

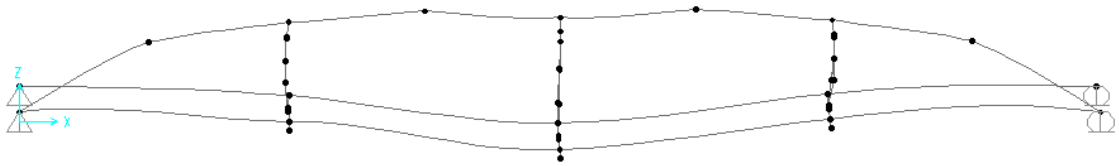
The eastern truss deflected 0.06 inches less than the western truss. Also, both trusses showed an upward deflection at the midpoint when the truck was at one end of the bridge.

Discussion

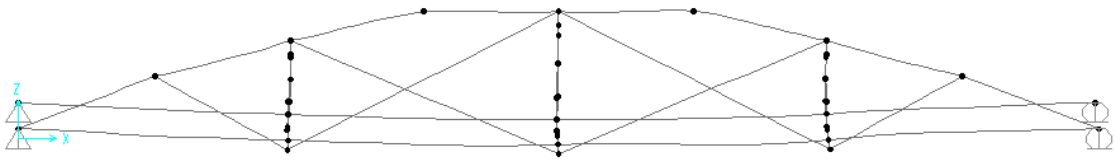
The average maximum measured deflection of the eastern truss was 0.08 inches. The predicted deflection was 0.11 inches, a difference of 20%; however, the western truss deflected 0.14 inches on average which is 40% more than predicted. It was observed that the cross-bracing of the eastern truss under load and the western cross-braces were not consistently taught. The eastern diagonals were loose, even when the bridge was carrying the load of the truck. To more effectively represent the as-built structural behavior, the bracing was removed from the eastern truss in the SAP model and the truck load was applied again.

With the uneven support, the western truss (without cross-bracing) deflected 0.25 inches downward and the eastern truss (with cross-bracing) deflected 0.10 inches. The variance between each truss better represents the actual behavior of the bridge but still overestimates the deflection. The smaller deflection of the actual bridge may be due to the wooden deck and hand railing (Figure 1) that is not accounted for in the model. The deck is bolted to the floor stringers and adds stiffness to the bridge which reduces the overall deflection.

The deflected shapes of each truss can be seen in Figure 16. The cross-braces restrain the arch from bulging out so far near the endpoints. They also maintain a straighter deck and tie by allowing the deck and arch to act as a unit instead of deflecting separately. The cross-braces tie the arch and deck together creating one deep beam instead of two shallow beams. The moment of inertia is proportional to the height of the beam cubed, so the deeper beam has a significantly larger moment of inertia and therefore smaller deflection.



(a) western truss, maximum deflection = 0.2464



(a) eastern truss, maximum deflection = 0.1015

Figure 16 Deformed shape of each truss with adjustments for tightness of bracing

Out-of-plane bracing system

Arches have a tendency to buckle out of plane because they carry loads in compression. Typically transverse bracing members are placed perpendicular between arches to prevent this lateral buckling mode, but not on this bridge. The arches of Ironto Wayside footbridge are supported from buckling in the out-of-plane direction by triangular bracing members as seen in Figure 17 below. Details of each member can be found on page 2 of the attachment.

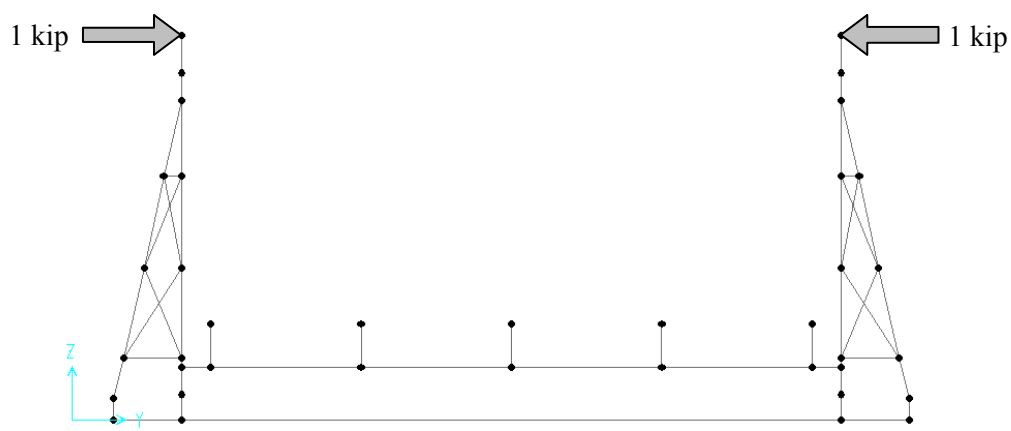


Figure 17 Out-of-plane bracing system

To determine the lateral stiffness contribution of the bracing system to the out-of-plane support to the arch, the arch-hanger system was analyzed as a system of two springs in parallel:

$$K_{total} = K_{arch} + K_{hanger} \tag{2}$$

where K_{arch} is the lateral stiffness contribution of the arch and K_{hanger} is the lateral stiffness contribution of the hanger.

K_{total} is calculated by applying a horizontal point load of one kilopound to the center of each arch (Figure 17). It was found that each arch moved 0.5122 inches towards the center of the span with the bracing system resulting in K_{total} = 1 kip/0.5122 in. = 1.95 kip/in. With the hangars removed (Figure 18), the lateral displacement of the arches was 3.085 in. resulting in K_{arch} = 0.324 kip/in.

Solving Eq. (2) for K_{hanger} and substituting K_{total} and K_{arch} obtained from the SAP model, K_{hanger} = 1.63 kip/in which is 83% of K_{total}. Therefore, the hanger system provides a significant portion of the total truss out-of-plane stiffness. Another option to support the trusses would be a lateral member connecting the arches. However, with the low profile of this bridge, connecting the trusses across the deck in any manner would have made the bridge impassible.

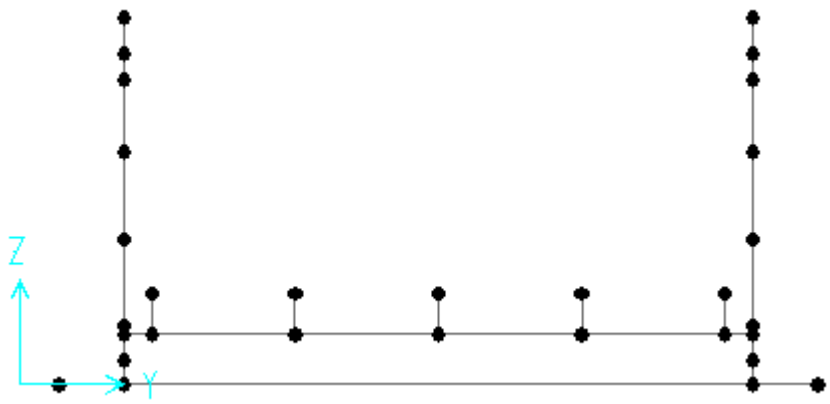


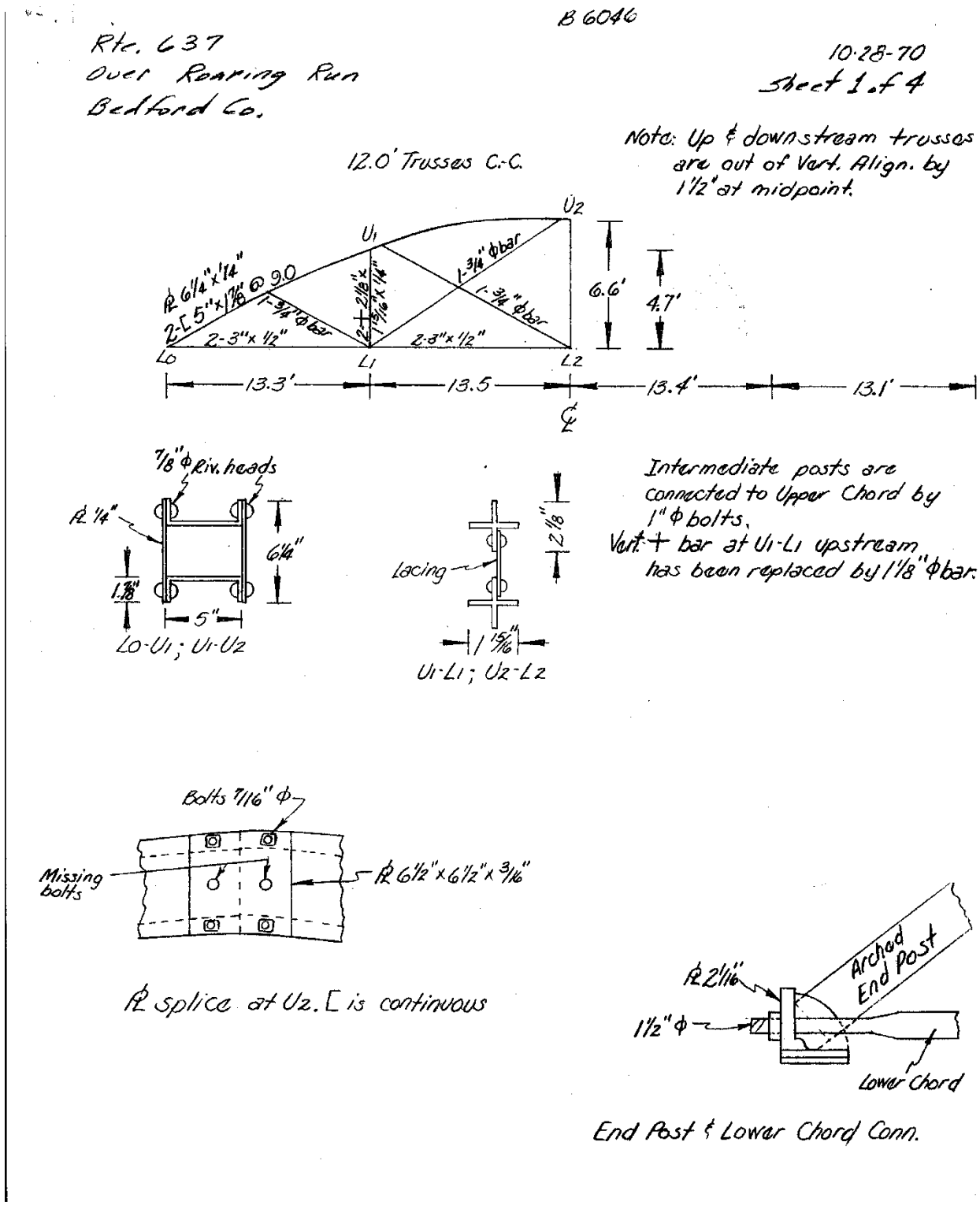
Figure 18 Cross-section of bridge without lateral bracing

Conclusion

The Ironto Wayside footbridge, the last remaining bowstring arch-truss in the state is a significant landmark in Virginia. It is useful to study historic landmarks as they can guide us in the design of future structures. The model created in this study can be used as a stepping stone for models of other bridges or a more detailed representation of the footbridge. Perhaps the analysis will aid in the future condition assessment of the bridge as it continues to be preserved as a historic landmark.

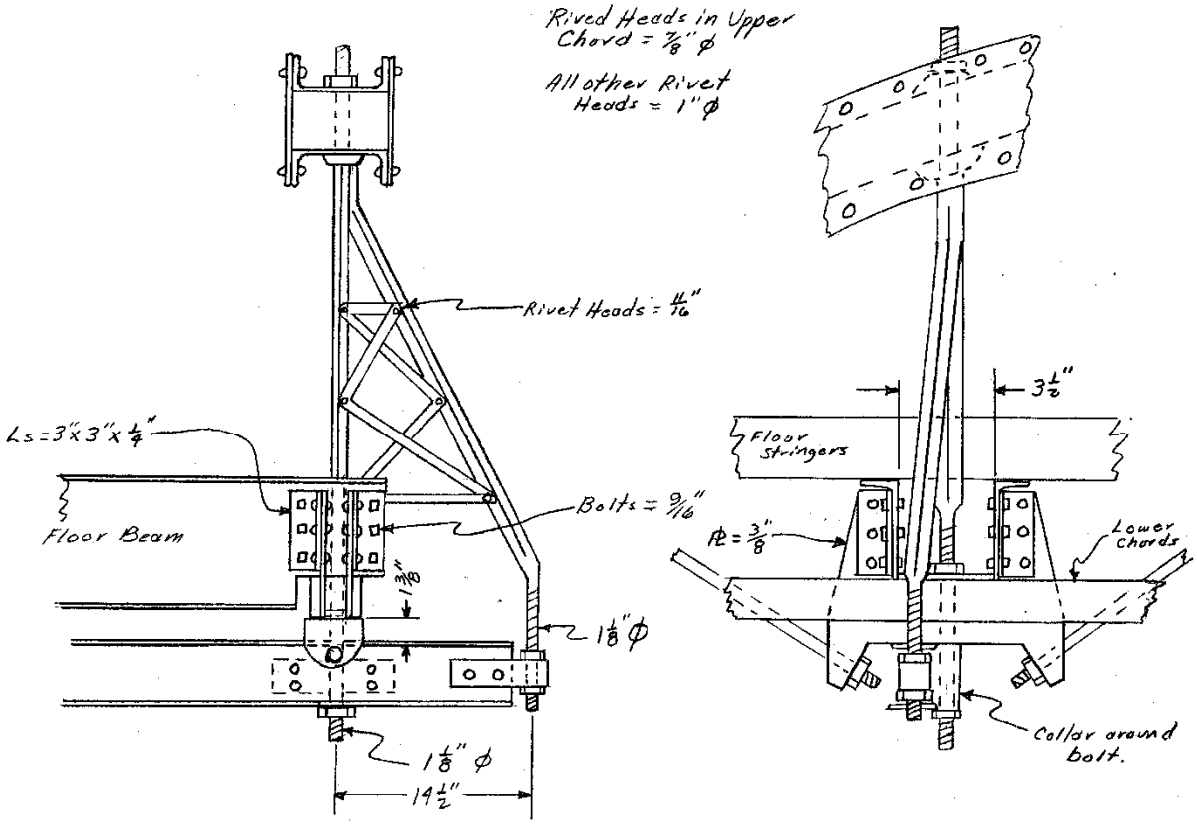
Attachment 1

Survey of Ironto Wayside Bridge (Deibler, 1975)



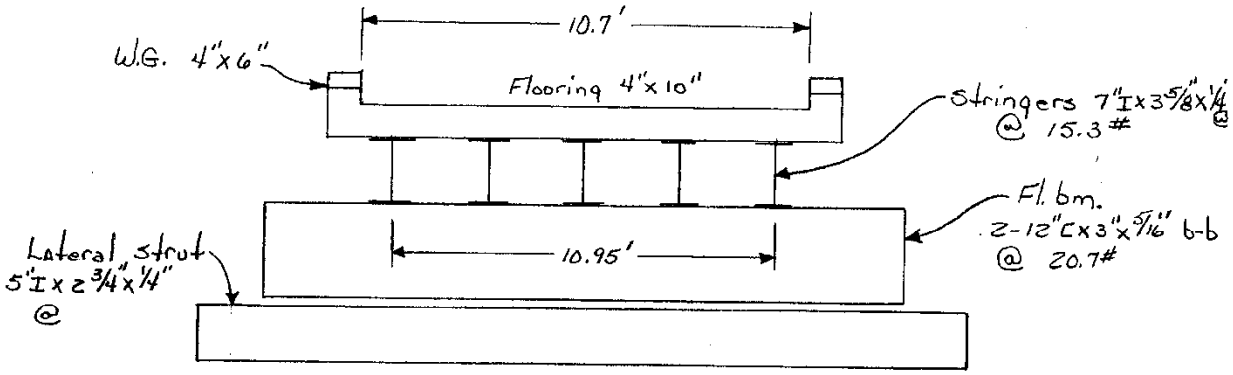
Rte. 637
Over Roaring Run B6046 Bedford Sheet 2 of 4

Floor Beam & Upper Chord Connections



Rte. 637
Over Roaring Run B6046 Sheet 3 of 4
Bedford Co.

X- Section of Truss Deck



Typical stringer spacing 32 3/4"
Stringers rest at Abt. A & B on Γ 3 1/2" x 2 1/2" x 1/4"
Stringers are Continous from L_0 at Abt. A to L_2
& from L_2 to L_4 at Abt. B.
Fl. bm & Lateral strut are connected by threaded
ends of intermediate post as per sketch.
End reaction from Fl. bm transferred to strut
at bottom chord and intermediate post.

Rte. 637 B1046 10-28-70
Over Roaring Run Bedford Co. Sheet 4 of 4

STREAM PROFILE - LOOKING DOWNSTREAM
1" = 10' VER. & HOR.

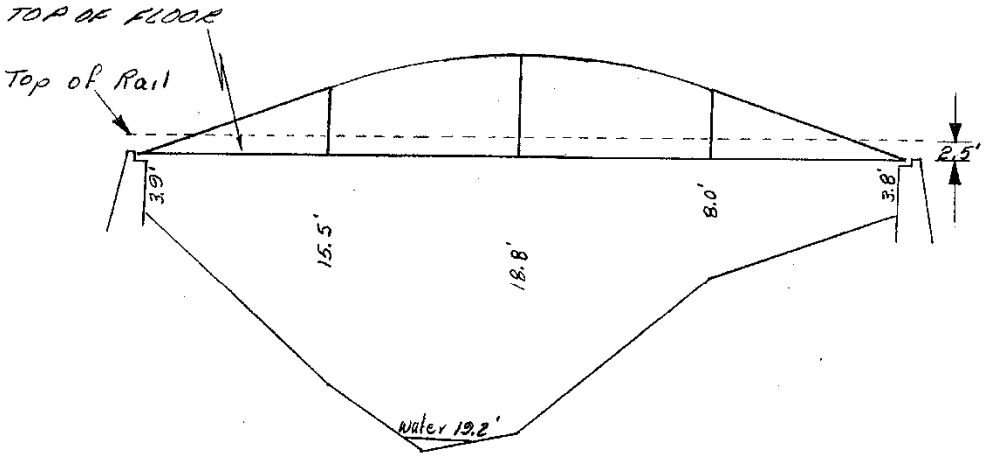


Photo Credits

Figure 1. Elaine Huffman (2009)

Figure 2. Wallace McKeel (1977)

Figure 13. Jim Stroup (2009)

Figure 14. Jim Stroup (2009)

References

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