ABSTRACT

During 2005 and 2006, Hardesty & Hanover performed bridge condition inspection and load analysis of four, prominent railroad bridges. The first bridge is Amtrak’s Little Hell Gate Bridge in New York City. The Little Hell Gate Bridge, completed in 1917, is a three-track, 1,156-foot long, unique, underdeck inverted bow string steel truss structure with pins and eyebars, soaring high above the urban landscape of New York, designed by the renowned Gustav Lindenthal and Othmar Ammann. The second and third bridges are the Tunkhannock and Martin’s Creek Viaducts near Scranton, Pennsylvania, inspected for Canadian Pacific Railway. The viaducts were constructed circa 1915, and are massive, world-record, reinforced concrete structures of 2,375 feet and 1,600 feet length respectively, that carry a single-track railroad as high as 240 feet above rural, forested mountain valleys of northeastern Pennsylvania. The fourth bridge is Conn Bridge on Amtrak’s electrified Northeast Corridor, spanning the Connecticut River near its mouth, at Old Saybrook, CT. Conn Bridge is a two-track, 1,584-foot long, through-truss bridge with rolling lift drawspan, completed in 1907. For bridge inspection, this presentation provides information on unique access requirements, specialty testing, operating railroad accommodation, issues of age and features of special interest for these unusual structures. For analysis, discussion of the application of special historical research, analysis for current design loadings such as railroad longitudinal forces, and modern traffic loadings vs. original design assumptions is provided.

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Inspection, Rating, Access, Bridge, Railroad

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INTRODUCTION

The period between approximately the mid-1890s through World War I in 1917 was a “golden age” of railroad bridge engineering in North America.

Hardesty & Hanover LLP (H&H) had the privilege between mid-2005 and mid-2006 to inspect four, prominent railroad structures that date to the first 20 years of the 20th Century. The bridges are as follows, presented in the order that these were inspected by H&H:

- Little Hell Gate Bridge in New York NY, owned by Amtrak
- Tunkhannock Viaduct in Nicholson PA, owned by Canadian Pacific Railway
- Martin’s Creek Viaduct near Kingsley PA, owned by Canadian Pacific Railway
- Connecticut River Bridge in Old Saybrook CT, owned by Amtrak

In addition to inspection, three of four of the structures were completely analyzed for load rating capacity (seismic excluded). Specialized ultrasonic pin testing and concrete testing were performed.

This paper presents some of the unique challenges that these bridges provided, and solutions offered, in the areas of

- Field inspection access
- Specialty Testing
- Load rating analysis

Little Hell Gate Bridge

The Little Hell Gate Bridge was designed by the famous bridge engineer, Othmar Ammann, as part of the New York Extension of the Pennsylvania Railroad. While the Little Hell Gate Bridge is a significant structure in its own right, it is overshadowed by its nearby sibling structure, the famous Hell Gate Bridge through-arch bridge, designed by Ammann’s boss on the
project, the renowned bridge engineer, Gustav Lindenthall. The Little Hell Gate bridge construction was completed in 1917, and has carried passenger and freight trains since.

The Little Hell Gate Bridge (See Figure 1) is a four span, inverted bowstring truss bridge over the former Little Hell Gate between Randall’s and Ward’s Islands. The structure was originally designed to carry four tracks, but the southernmost set has been taken out of service and removed. The bridge carries both eastbound and westbound trains between Boston and Penn Station. In its history, the Little Hell Gate Bridge has seen little maintenance or rehabilitation. Since its construction, the bridge has been painted twice (in 1939 and 1995) and a rehabilitation contract was performed in 1992 for minor steel repairs.

Approximately 20 years after the bridge was constructed, the Little Hell Gate waterway below was filled in with construction excavations and debris during the 1930s construction of the Triborough Bridges. Randall’s and Ward’s Islands are now a single land mass bordered by the East River to the west, the Bronx Kill to the east and the Hell Gate to the south. The area directly to the east (south) of the bridge currently houses a sewage treatment plant and the New York City Fire Academy. Directly to the west (north) of the Little Hell Gate Bridge are the approach spans of the Triborough Bridge, local access roadways, waste areas and parkland.

*Tunkhannock and Martin’s Creek Viaducts*

The Tunkhannock Viaduct (Bridge 653.22 Freight Main Line – See Figure 2) is a twelve span reinforced concrete arch bridge that, at the time of its completion in 1915, was the largest concrete bridge in the world. The structure was designed by Abraham Burton Cohen and constructed by the Delaware, Lackawanna and Western Railroad as part of the ‘Hallstead Cutoff’. 
Tunkhannock (also known as the Nicholson Viaduct) was constructed in order to eliminate the difficult grade changes that the railroad encountered along its original path on what is now Pennsylvania Route 11. The construction of the bridge also shortened the travel distance of trains between New Milford and Clarks Summit by three and a half miles. It was originally designed to carry two tracks, but today carries only one along its eastern side that services both northbound and southbound freight trains from Binghamton NY (North) to Scranton PA (South). The bridge stretches 2,375 feet across the rural valley over which it is sited, and soars 240 feet above the east branch of Tunkhannock Creek at its highest point. The Tunkhannock Viaduct is a designated historic, civil engineering landmark of the American Society of Civil Engineers.

The structure has ten visible spans, each measuring 180 feet in length, and is flanked by two abutment spans that were buried during construction. Each of the piers is founded on bedrock, which at the time of construction was up to 138 feet below grade. Nearly half of the structure’s mass is buried underground.

As previously noted, a single track operates on the bridge at present and carries only freight trains (See Figure 3). There are reportedly an average of ten freight trains per day carried on the line at present. The closed western track line area now serves as an access road along the full length of the bridge. The track is supported on timber ties that rest in a ballast.
and gravel bed. The ballast is supported by a reinforced concrete bridge deck, which was constructed with an applied waterproofing system. Three-foot thick reinforced concrete parapets extend the length of the bridge along each fascia. At the visible spans, the concrete deck was poured atop reinforced concrete spandrel arches, which are then supported by reinforced concrete spandrel walls. In each span, there are twelve spandrel walls, ten of which are supported on the arch ribs themselves. The two end walls comprise a portion of the tower at each support pier. The spandrel walls apportion the span into eleven equal-length panels. The main structural elements of each span are the east and west reinforced concrete arch ribs (the spandrel walls separate in an archway at varied elevations above the ribs) that spring from each pier 125 feet below the top of deck.

The Martin’s Creek Viaduct (Bridge 644.25 Freight Main Line – See Figure 4) is a ten span reinforced concrete arch bridge, built in conjunction with the Tunkhannock Viaduct (nine miles distant) as part of the “Hallstead Cutoff” project. The Martin’s Creek Viaduct is 1600 feet long with 150-foot high arches. The structure was completed in 1914 and was originally constructed with three tracks. Today it carries a single center track, which services both northbound and southbound trains on the rail line.

The geometric makeup of the bridge varies for the first three spans. Spans 1 and 2 are short single arch spans that directly support the railway deck. Spans 3 through 10 are similar to the visible spans of the Tunkhannock Viaduct. Each span consists of two arch ribs that support a series of spandrel walls, which in turn support the spandrel arches and deck above. Reinforced
concrete parapets extend the length of the bridge along each fascia. Span 3 is shorter than Spans 4 through 10; the span supports seven spandrel walls, five of which rest directly on the arch ribs. Spans 4 through 10 each support eleven spandrel walls, nine of which rest directly on the arch ribs. The two end walls comprise a portion of the tower at each support pier. The ten piers supporting the structure are buried deep below grade. The north abutment span similarly consists of two reinforced concrete arch ribs supporting four spandrel walls, which in turn support the deck above. Concrete sidewalls block in the span along the east and west fasciae; a significant portion of the north abutment span is buried under fill material.

Connecticut River Bridge

The 100-year-old Connecticut River Bridge is part of the rail infrastructure that has served New England for many years (See Figure 5). CONN bridge is one of nine movable railroad bridges along the Connecticut coast main passenger line between Stamford in the west and Mystic in the east. The existing bridge, constructed circa 1907, is a two-track, open deck, electrified railroad bridge, 1564 feet long, consisting of seven thru-truss spans, two deck girder spans, and one 158-ft Scherzer-type Rolling Lift span. The bridge carries approximately 35 passenger and six freight trains per weekday. In recent years, the draw span of the bridge has experienced approximately 3700 yearly openings to accommodate navigation on the Connecticut River. The bridge spans the Connecticut River near its mouth at Navigation MP 3.4 in the towns of Old Saybrook and Old Lyme, CT. The approach span structures were designed by the New York, New Haven and Hartford Railroad (N.Y.N.H. & H.R.R.); the bascule span was designed for the N.Y.N.H. & H.R.R. by the Scherzer Rolling Lift Bridge Company in

Figure 5. Amtrak Connecticut River Bridge
1905. The structure was constructed by American Bridge Company beginning in 1906. The structure is used primarily for passenger trains for the AMTRAK Northeast Corridor with one or more daily round trips for the Providence and Wooster (P&W) freight train and the occasional maintenance rail cars.

**INSPECTION ACCESS**

The inspection of these four, prominent railroad bridges required addressing many of the typical challenges of major bridge inspection, but also provided some unique challenges. In addition to a brief discussion of some of the standard railroad bridge inspection issues, some of the unique inspection access challenges are also presented here, with an account of how these challenges were met in the field.

*Major Railroad Bridge Inspection Typical Issues*

Some typical issues that must be addressed for performing inspection of existing railroad bridges include those that were addressed as part of the work on these projects.

- Safety against falls, as detailed further below.
- Proper inspection scope: What level of detail is time and cost effective?
- Proper testing scope: how much and what type of specialty testing is appropriate?
- Site Specific Work Plans – plan in detail field activities for efficiency and safety.

*Field Safety*

Federal Railroad Administration 49 CFR 214.103 Subpart B, “Bridge Worker Safety Standards” regulates minimum safety standards on bridges where fall hazards may exist. Hardesty & Hanover has also developed some simple and practical safety rules for performance of bridge condition inspections follows:
“While performing condition inspection of railroad bridges at 12 feet or more above the ground, and for bridges above water more than four feet deep, employees shall comply with the following:

1. Employees shall not perform inspections alone, but shall work in groups of two or more.

2. The employee shall not climb steel unaided by equipment, except where specifically and formally trained in safe climbing techniques.

3. When walking on the bridge, employees shall perform inspection work, to the greatest extent possible, from maintenance access walks, from behind safety railings, and from areas between the outside rails of the track, in which areas federal regulations do not require special fall protection.

4. Where access to heights is required, the employee shall use approved access equipment that has been properly placed and secured for inspection use. Examples of equipment include bucket trucks, underbridge inspection units, lift trucks and scaffolding.

5. Where used, ladders shall be founded on solid ground, in areas safe from vehicular impact, to access relatively low heights only. One person shall remain at the base of the ladder and hold the ladder stiles while the inspector uses the ladder.

6. If any employee does not understand the site safety provisions for inspection, or feels that conditions are unsafe, the employee shall notify the field supervisor of their concerns, and is not required to perform field inspection work until that employee understands the safety provisions and/or has been provided reasonable demonstration that safe conditions are provided.”
Another area of special field safety that sometimes needs to be addressed is the area of confined spaces per OSHA regulations.

Bridge Inspection Unique Project Access Issues

Several atypical issues arose which needed to be addressed as part of the inspection work for these projects. Some of these problems encountered, and solutions offered, are briefly presented here.

Martin’s Creek Viaduct Access Road Deterioration

At Martin’s Creek Viaduct, during inspection the field crew found that the railroad right-of-way access road to the site had been severely rutted during the winter and early spring due to recent, heavy rains, such that the inspection equipment could not gain access to the bridge. The remote and mountainous terrain prohibited alternate access means. CP Railway crews were not available to respond to the situation, because of emergent conditions on other railway bridges within the territory, also caused by the recent heavy rains and severe flooding in the Susquehanna River valley. This situation threatened costly delays to the project.

The Hardesty & Hanover inspection team leader at the site was experienced with operation of small construction equipment. Without delay, the inspector rented a small bulldozer for the day, performed the necessary grading work of the access road (all work was outside the limits of the FRA fouling envelope for the active tracks). After the minor but necessary roadway repair work was done, the inspection team accessed the site, and the inspection team leader informed the home office of the completed work.
Hands-on inspection of the Connecticut River Bridge included working around 25,000 Volt railroad electrification wires and feed lines. Safety rules require that inspectors maintain 15 feet clear distance to live wires. In 2000, electrification was added to the bridge. The high-voltage, negative feed lines of the system were mounted on the outboard faces of the through trusses, just below the level of the upper chord. This combined with the catenary system mounted immediately above each of the two tracks, prevented inspection access to the entire top half of the trusses while the electrification system was live, and the hands-on inspection work could only be performed while at least one track was out of service, and the electrification system for that track de-energized and grounded.

Night work would be a possibility for work requiring taking a track out of service with the electrification system de-energized; however, inspecting a structure at night, even with expensive floodlights, is not usually effective.

The field team was able to arrange with Amtrak to piggy-back the inspection with planned, 55-hour weekend single track outages being made for other construction activity. This weekend work permitted hands-on inspection of the upper portions of the truss work during daylight hours.
Connecticut River Bridge Security Notification

For bridges in navigable waterways, security clearance of employees is required, which means that the US Coast Guard needs to be notified, and the social security numbers of each employee proposed to work at the bridge needs to be provided. The use of Social Security Numbers presents some privacy concerns. It is hoped that a simple system such as the rail security registration systems that some railroads are adopting for contract employees working on their right-of-way may be adopted for more universal use, to include work at bridges over navigable waterways.

In order to inspect the underside of the bridge and to provide as little interference with rail operations as possible, the inspection work was performed using a manlift on a barge. The barge was propelled by a push-tug. A safety and inspection powerboat was also provided.

Work in the navigable channel of the Connecticut River was coordinated with river traffic. During the summer months, the Connecticut River has constant recreational boating.

Tunkhannock and Martin’s Creek Extreme Height

The viaducts have a height of up to 240 feet above the valley floor. The inspection plan required coordinating a variety of access equipment to effectively access portions of the structure for hands-on inspection.

- As a safety measure, the local fire and rescue departments were notified prior to the inspection of the work being done.
performed, so that they might be prepared in the unlikely event of the need for emergency rescue.

- To access the lowest portions of the structures, a 135-foot manlift was used where ground access permitted.

- To access the upper portions of the structure, an Underbridge Inspection Unit (UBIU) with a 60-foot reach was used (See Figure 7). Rail-mounted units are available; however, where an access road is available, using road-based vehicles is vastly preferable, to avoid scheduling and other access issues with track time. Track time availability was very limited for our use at Tunkhannock and Martin’s Creek Viaducts.

- To access the mid-height portions of the structure, a spider basket attached to a truck-mounted crane was used (See Figure 8).

- Safe climbing using harnesses was also employed, along the top and center of the arches where the edge falling hazard was more than one body-length distant (See Figure 9).

The detailed, site-specific work inspection plan was particularly important for efficiently coordinating the use of crews and equipment at these two structures. The comprehensive inspection of these massive, concrete structures was accomplished in just a few weeks time. Due to the difficulty in accessing some areas from the top of deck or from ground level (a result of structure size, site topography and obstructions), it was
determined to be economically unfeasible to inspect every surface of the structure “hands-on”. The best use of inspection resources within the allotted schedule was to perform a complete and thorough visual inspection of the structures using reasonable access methods, and then to focus on close-up inspection of a representative span at each structure, and to extrapolate this inspection data to other spans. By following this methodology, the inspection teams focused more on the behavior of the structure and on overall rehabilitation needs. The similarities in deterioration conditions and locations throughout the structure were noted.

*Movable Bridge Facilities*

The Connecticut River Bridge includes a 100-year-old, rolling lift bascule draw span. Inspection of the electrical and mechanical facilities of these bridges requires trained individuals who are knowledgeable in and experienced with:

- movable bridge technology and practices,
- railroad operations environment,
- interaction of structural-mechanical-electrical systems of the movable bridge,
- signaling interface with the electrical operating system, and
- electrification interface with the bridge drawspan operations.

*SPECIAL TESTING*

There are a few basic questions to ask when considering the need for specialized materials or other testing:
• What is the value to the railroad of testing for the time and money expended?
• How will the anticipated results be used?
• How much testing is needed?
• Is the inspection phase the right time to take tests, or is it better done as part of
detailed design development, or even during construction?

For three of the four bridges on this project, specialty testing was undertaken. For the
Tunkhannock and Martin’s Creek Viaducts, limited concrete coring and laboratory examination
were undertaken, partly to confirm suppositions made from visual observations and partly to
confirm results from previous testing performed on the viaducts. At Little Hell Gate Bridge, all
78 truss joint pins were ultrasonically tested for hidden flaws. At Connecticut River Bridge,
specialty testing was not performed as part of the 2006 inspection. Extensive pin testing had been
performed at the Connecticut River Bridge for the previous, in-depth inspection, at which time no
immediately worrisome conditions had been found. Rehabilitation work on the mechanical and
electrical systems of the bridge had also just recently been completed, so that new, specialized
movable bridge testing was not warranted.

Challenge for Ultrasonic Pin Testing at Little Hell Gate Bridge

The main challenge for testing of the pins at the
Little Hell Gate Bridge was not testing the pins. It was
accessing the face of the pins (See Figure 10).

For nearly 90 years, the pins at Little Hell Gate
Bridge had been retained and protected by cast iron caps.
Temporarily removing these caps for access to the faces
proved much more difficult than anticipated. It was
initially planned that the access equipment supplier would remove the caps as part of their scope. The supplier could not remove the rust-frozen bolts that retained the caps.

The solution was to hire a small, ironwork contractor, who employed a specialized, hydraulic torque wrench to loosen the through bolts, remove the caps, and then reinstall the caps and bolts. After breaking the first bolt head while trying to figure out how to do the work, the contractor was able subsequently to successfully remove and reinstall the remainder of the caps (See Figure 11). Through very tight scheduling and coordination of the access equipment usage, the ironwork contractor and the testing technician, budget and schedule were maintained.

Figure 11: Pin Cap Removed

Concrete Testing at Tunkhannock and Martin’s Creek Viaducts

Limited concrete investigations were performed for the viaducts, the overall purpose being to sample the concrete to check compressive strength against rating analysis assumptions, and to verify some of the mechanisms causing deterioration for estimating the repair quantities and costs. After initial examination of core samples indicated evidences of Alkali-Silica Reactivity (ASR), petrographic analysis was performed on two of the core samples to check the extent and the activity of the ASR. It was found that the concrete, while having suffered the effects of freeze-thaw and ASR, was generally in a stable state. It was recommended that a rehabilitation that addresses the visually observed areas of surface deterioration would likely suffice to repair the structure for continuous, acceptable service.
LOAD RATINGS AND HISTORICAL RESEARCH

Included in the scope of work for three of the four bridges, was a comprehensive live load capacity rating of the structures. For the fourth structure, the Connecticut River Bridge, rating effort was limited to checking as-inspected rating capacities for any areas where member deterioration had advanced since the previous comprehensive inspection of nearly ten years earlier.

For these major bridges designed during the early years of the 20th century, understanding the load rating analyses of the bridge members today, required research into the original assumptions of the designers. Because of the prominence of the bridges, published information from transaction papers was available to unlock a few mysteries during initial load rating analyses of the structures.

Major Railroad Bridge Load Rating Typical Issues

Some typical issues that must be addressed for performing load rating analysis of existing railroad bridges include these that were addressed as part of the work on these projects.

1. Fatigue analysis of steel tension members per AREMA 15-7.3.4.2.

2. Longitudinal forces per current design codes versus allowances in older codes and actual forces anticipated per local operations

3. Acceptance criteria and current operating equipment, for both normal and maximum live load rating conditions

Load Rating Unique Projects Issues

Several atypical issues arose which needed to be addressed as part of the load rating analysis for these projects. Some of these problems encountered, and solutions offered, and here briefly presented.
Little Hell Gate Bridge Variable Allowable Stresses

An unusual result occurred from our initial load and capacity analysis of the Little Hell Gate Bridge, and detailed research provided an interesting explanation.

For “Normal Rating” per AREMA Chapter 15 Section 7.3, all structural members were found to have capacity in excess of Cooper E-60, except for a few members that were found to have a calculated rating capacity of slightly less than the basic acceptance criteria of Cooper E-60, between E-57 and E-59.

An investigation was made as to why these members would rate less than what the bridge was originally designed for, namely Cooper E-60 live load. This was especially puzzling, since the original design drawings specifically stated that two full tracks of load are applied per truss, as compared to a computed maximum 1.6875 tracks tributary to the trusses for current conditions and design standards. Also, the original design would have likely included higher, steam impact. So why would the bridge rate less than E-60, especially with lighter, current design loadings?

In researching the possible reason for these lower than anticipated initial ratings, the answer was found in an ASCE transaction paper that had been written by designer Othmar Ammann1. Ammann stated that unit stresses used for steel materials were taken as “from five eighths to three fourths of the minimum elastic limit”; i.e., 0.625 Fy to 0.75 Fy. Normal rating per AREMA Chapter 15 is based upon 0.55 Fy, and this is sufficient to explain the less-than E-60 capacity of the low-rating members. Increasing the allowable stress for normal rating to an amount very little above 0.55 Fy will increase the rating capacity from E-57 to above E-60.

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1 Ammann, Othmar, ASCE Transactions Vol. LXXII, Paper No. 1417 presented Nov. 21, 1917
Little Hell Gate Bridge Eyebar Allowable Stresses

For pin-connected members, allowable tension stress is $K = 0.45 \ F_y$, per AREMA Section 15-1.4 2004 update, Table 15-1-11. In Table 3-11 of the 1994 update, no special provision for pin-connected truss members was indicated. The 2004 commentary for this section references a single, 1939 ASCE transaction paper as the source for the more conservative allowable stress applied to pin-connected members. We obtained a copy of the paper, and found that forged eye bars are specifically excluded from the investigation, as stated in the first paragraph of the paper: “The present investigation is concerned with pin-connected plates which do not have reduced width in the body as is the case of the forged eye-bar…” Therefore, basic allowable stress for the pin-connected, eye-bars at Little Hell Gate were assumed the same as for other tension members, or $0.55F_y$.

Tunkhannock and Martin’s Creek Viaducts Original Design Loading Assumptions

The Tunkhannock and Martin’s Creek Viaducts are composed of Portland Cement concrete, with steel reinforcement. However, the design for the structures is not based upon modern reinforced concrete principles, but rather on unreinforced masonry design practices of the 19th century, using mass concrete in place of stone masonry. Additionally, loading conditions used for the original design differ from current AREMA recommendations.

For Live Loads and Impact Loads for Tunkhannock Viaduct, the 1911 stress sheets show a non-Cooper series of locomotive wheel concentrations (See Figure 12).

The wheel concentrations were positioned on the spans to produce the maximum stresses in the arch rib for two live load cases, namely, 1) Live load plus impact applied to the full span length, and 2) Live load plus impact applied to one half the span length.
The original stress sheets apply the live load to the arch rib as concentrated loads at the spandrel columns. Since the concentrated loads are proportional to the spacing of the spandrel columns rather than the magnitude and location of the wheel concentrations, it is apparent that an equivalent uniform live load was used as part of the original design. It appears that the original stress sheets calculated the concentrated loads at the spandrel columns as follows:

1. Locomotive axle loads were positioned on the span for maximum stress at the arch rib.
2. Once positioned, the concentrated loads were summed and divided by the loaded length to generate a uniform load.
3. A distribution factor was applied to the live load.
4. The live loads were then divided by the width of the arch rib to generate an equivalent uniform load per unit width of arch rib.

Figure 12: Present Day Cooper E80 Live Load arrangement compared to 1911 Delaware and Lackawanna Railroad Live Load
5. The concentrated loads at the spandrel columns were then determined by calculating the reactions of the equivalent uniform live load, considering the tributary area to be one half of adjacent concrete deck span lengths. From review of the concentrated live loads tabulated in the stress sheets and working backwards using the above procedure uniform track loads were derived.

Similar to live load, a separate impact force was calculated for each spandrel wall location and applied to the main arch rib as a concentrated load. The 1911 stress sheets show that impact was calculated per the Impact Formula: \( I = \frac{LL^2}{LL + DL} \), where \( I \) = Impact Load, \( LL \) = Live Load and \( DL \) = Dead Load. The magnitude of the impact force is a function of the ratio of live load to dead load. Therefore, the impact at each spandrel walls varies, impact being greatest at the crown and lowest near the skewbacks. As a percentage of live load, the 1911 design impacts are roughly equivalent to 2006 AREMA diesel impact per AREMA 8.2.2.3.

1911 Temperature Loads for Tunkhannock Viaduct: Temperature loads are calculated on the 1911 stress sheets based on a temperature rise of 30° F and a temperature fall of 30° F.

1911 Load Combinations were:

- Dead Load + Temperature
- Live Load + Impact on half the arch span + Dead Load + Temperature
- Live Load + Impact on full arch span + Dead Load + Temperature.

No other loads or combinations were used in the original design.

_Tunkhannock Viaduct 1911 Construction Method_

The main arch ribs were constructed by the alternate block or voussoir method. With this method, the arch rib is constructed in transverse blocks of such size that each
block can be completed at one pouring, or within about a day’s work. The blocks are poured in such an order as to give a uniform deflection of the centering, and also prevent the crown of the arch from rising as the lower arch loads are placed. This method has the advantage of reducing shrinkage stresses in the arch ring to a minimum (See Figure 13).

Steel centering was employed in the construction of the Tunkhannock viaduct. From 1918 “Concrete Engineers Handbook” by Hool and Johnson, the use of three hinged steel centering had the following advantages:

- Crown deflection using steel centers is usually much less than that obtained by employing timber falsework.
- It is possible to compute the deflection of each point of a steel center with some degree of accuracy while, in the case of a wooden center, the probable settlement at each bent is much less definitive.
- The advantage of allowing the deflection to be quite accurately computed makes it possible to give the centers a preliminary camber so that when the concrete is in place and the centering withdrawn, the arch ring will assume its true position.
The unique feature in the steel centering used in constructing the Tunkhannock Viaduct was an adjustable panel at the crown of the steel arch trusses.

The above information on construction method is important to the proposed load rating analysis methods for the following reasons:

- The alternate block method of construction tells us that concrete shrinkage stresses during construction were kept to a minimum.
- The arch ribs are not monolithic but are composed of a series of concrete blocks with a series of construction joints along the entire rib length. It is assumed that if a particular section of the rib is subjected to high bending moments, these construction joints will crack, effectively forming a hinge to redistribute the forces.
- Since steel centering trusses were used it is assumed that deflection of the centering was accounted for during construction and arches were constructed in their “true” position.

**Tunkhannock and Martin’s Creek Viaducts Modeling Assumptions**

Analyzing the structures using finite element computer programs provided many opportunities for deep thought, review of assumptions, re-analysis and further, iterative computational processes, until rational analysis results were obtained.

Some of the modeling assumptions that were found to have significant effects on distribution of loadings and structural stresses included the following:

- How is the arch spring restrained? Is the main arch best modeled as a two pin, three pin or fixed structure? The answer to this question
required research into the original construction method employed for the arch, as explained above, and as further explained below.

- Do the spandrel walls sit atop the arch simply as dead load? For Tunkhannock Viaducts dead loads, the 1911 stress sheets apply the design dead load from the ballasted deck and spandrel walls to the arch ribs as concentrated loads at the base of the spandrel walls. The self weight of the arch rib is applied as a series of concentrated loads along the length of the arch rib. If the spandrel walls interact with the arch, is their connection pinned or fixed? The arch ribs were modeled as a series of continuous segments or members with fixed end supports at the skewbacks. During the initial analysis of the 180-foot spans, the “dead load plus temperature fall” load combination generated relatively high moments at the arch crown. It should be noted that the original 1911 stress sheets showed tensile forces in the arch rib at these locations under the “dead load plus temperature fall” load combination. It was assumed that under the “temperature fall” load combinations the concrete construction joints at the arch crown crack and the steel reinforcement yields, thus creating a hinge and redistributing the moment throughout the rib. This is consistent with the 1911 Construction Method used, as discussed above. A separate computer model was therefore created for the “temperature fall” load combinations with a hinged crown at the arch to model the behavior of the arch and verify that the rib has sufficient capacity to accommodate the load redistribution. The deck and spandrels were included as part of
the computer model for the arch rib load rating. Deck expansion joints were incorporated into the model at locations shown on the original plans. Spandrel columns were assumed pinned at their bases since there is a minimal steel reinforcement and a construction joint at the spandrel wall to arch rib connection.

- **What concrete strength should be assumed?** This was a particularly interesting problem. Concrete modulus of elasticity was shown as two million pounds per square inch (psi) on 1911 stress sheets. From 1918 “Concrete Engineers Handbook” by Hool and Johnson, a modulus of elasticity for concrete equal to 1/15 that of steel may be assumed when the strength of the concrete is taken as greater than 800 psi and less than 2,200 psi. Based on the above information, the load rating analysis assumed a design concrete strength $f_{c'} = 2,000$ psi. Concrete core samples, however, all tested well in excess of 3000 psi capacity. Still, the original design assumptions regarding elastic modulus were found to be most appropriate for obtaining rational and balanced results.

Load ratings were computed in accordance with AREMA (2006) Chapter 8, Concrete Structures and Foundations, Section 19, Rating of Existing Concrete Structures. A two-dimensional (2D) linear element analysis was performed using SAP 2000 Integrated Software for Structural Analysis and Design. Similar to the 1911 stress sheets, the arches were analyzed as independent spans. For the load rating evaluation of the arch rib and spandrel walls, ultimate strength axial force versus bending moment (P-M) interaction diagrams were constructed and the main arch ribs and spandrel walls were
rated by the load factor method. The deck sections were analyzed independently by hand calculation using the working stress method.

The following load rating analysis conclusions were reached:

- For analysis Load Cases 1, 2 and 3 (dead load, live load and temperature combinations), all member capacities meet basic acceptance criteria for Normal and Maximum service levels.

- For analysis Load Cases 4, 5 and 6, which include current-day design longitudinal forces, member capacities for the 180-foot arch rib and for spandrel walls do not meet basic acceptance criteria; however, this does not appear significant to the continued serviceability of the structure under current operating conditions. Longitudinal loads were not included in the original design. Field inspection evidences showed no indication of distress from longitudinal forces. Site conditions indicate that actual longitudinal forces and/or their application to the structural elements are likely to be much less than current-day design longitudinal force analysis might indicate.

CONCLUSIONS

Four significant railroad bridge structures were inspected and rated for capacity. Besides the typical issues of large bridge inspection, a number of unique issues arose in the execution of these projects.

For inspection access, some of the unique occurrences included these:

- At Martin’s Creek Viaduct, the inspection team leader graded an access roadway.
• At Tunkhannock and Martin’s Creek Viaducts, the extreme height of the structures required that three, different pieces of access equipment be used, in addition to safe climbing.

• At Connecticut River Bridge, the railroad electrification facilities required special scheduling of the inspection

• At Connecticut River Bridge as at all bridges over navigable waterways, present day security concerns require that inspectors provide personal identification data.

For special testing, some of the unique occurrences included these:

• At Little Hell Gate, removing eyebar pin retainer caps that had been in place for 90 years required special effort and tools.

• At the Tunkhannock and Martin’s Creek Viaducts, just enough concrete testing was performed to determine that the structure concrete, though deteriorated, was stable and that repairs could address visible signs of surface deterioration.

For load rating analysis, some of the unique occurrences included these:

• For Little Hell Gate, historical research into papers written at the time of the initial construction revealed the designer’s assumptions of variable safety factor multipliers to yield strength for various steel components of the structure.

• For Little Hell Gate Bridge, research into the background documents supporting current AREMA design code suggestions provided information on the proper stress capacity allowances to apply.

• For Tunkhannock and Martin’s Creek Viaducts, extensive research into original design reports and papers indicated assumptions for assumed loadings, concrete properties and construction methods that had significant impacts on the distribution of loadings and design stresses within the structure.
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