A REDUNDANT STEEL TRUSS BRIDGE FOR THE CITY OF MINNEAPOLIS

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BIOGRAPHY

Greg Hasbrouck is a Supervising Bridge Engineer with Parsons in the Chicago office and was the Truss Bridge Design Lead for the steel truss segment of the St. Anthony Parkway Bridge project. He has over 12 years of experience working on complex bridge projects including the design of the Hastings Tied Arch Bridge in Hastings, Minnesota and the Christopher S. Bond Cable-Stayed Bridge over the Missouri River in Kansas City, Missouri. Mr. Hasbrouck received his Master's in Structural Engineering from Princeton University and his Bachelor's in Civil Engineering from Duke University and is a registered professional engineer.

Martin Furrer is a Principal Project Manager with Parsons in the Chicago office and was the Engineer of Record for the steel truss segment of the St. Anthony Parkway Bridge project. He has over 20 years of experience in the project management, design, analysis, load rating and construction of complex bridges including fifteen bridges over major waterways. Mr. Furrer was the Engineer of Record for North America's longest freestanding arch bridge in Hastings, Minnesota and the 1000 ft long Christopher S. Bond Cable-Stayed Bridge over the Missouri River in Kansas City, Missouri. Mr. Furrer received his Master's in Structural Engineering from the Swiss Federal Institute of Technology, Zurich and is a registered professional and structural engineer.

SUMMARY

The St. Anthony Parkway steel truss bridge replacement project, located in Minneapolis, Minnesota and crossing over the BNSF Northtown rail yard, incorporates unique load path redundancy measures including eliminating fracture critical steel truss members and gusset plates and using a post-tensioned concrete bottom chord. The replacement of the existing truss spans requires specialized construction techniques to minimize impacts to BNSF's second busiest rail yard in the country including employing launching beams to remove the existing trusses and install the new truss span.

Parsons' goal was to design a truss that provided complete load path redundancy and to eliminate all fracture critical members and the necessity of fracture critical member inspection. Redundancy in the main truss diagonals and top chord is achieved through open H-section members with the flanges spliced at nodal members providing two continuous load paths in each truss through the flanges. For tension members, the H-section is split into double-tees, with each tee capable of carrying the full fracture load from the loss of the other and resulting in visible deformation of the truss. Bottom chord redundancy is achieved through use of a steel U-shaped section designed to resist fabrication and launching loads. Once the truss is launched and prior to deck pour, the U-shaped section is filled with concrete and post-tensioned becoming a non-fracture critical post-tensioned concrete member.

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Introduction

The St. Anthony Parkway over Northtown Yard Bridge replacement project is located in Minneapolis, Minnesota. The existing bridge consists of five simple span Warren through trusses built in 1925 and crosses over 22 railroad tracks within the BNSF Northtown rail yard. It is owned by BNSF and maintained by the City of Minneapolis and carries both vehicular and non-vehicular pedestrian traffic. The steel truss span structures, shown in Figure 1, are both structurally and geometrically deficient containing fracture critical members, narrow travel lanes and substandard vertical clearances.

Figure 1 – Existing Bridge

In 2013 Parsons was selected, as a subconsultant to SEH, by the City of Minneapolis to provide preliminary and final design services for replacement of the existing St. Anthony Parkway truss bridge. Parsons' final design of the main span of the new three-span replacement structure consists of a 305 foot redundant steel truss structure, shown in Figure 2, incorporating unique load path redundancy measures including eliminating fracture critical steel truss members and gusset plates and using a post-tensioned concrete bottom chord. The approach spans designed by SEH consist of two-span continuous steel plate girders and all three spans incorporate a full-depth cast-in-place concrete deck with 54 foot wide travel way. The deck is formed using stay-in-place metal decking in order to improve safety and minimize construction impacts to the rail yard below.

Specialized construction methods for removal of the existing truss spans and installation of the new bridge were incorporated as part of the design. The design allows pairs of the new approach span girders to be used as launching beams for both removal and installation of the trusses to minimize in place debris removal and construction impacts to BNSF's heavily used rail yard below.

Figure 2 – Proposed Bridge

Location

The St. Anthony Parkway Bridge is located in northeast Minneapolis's industrial area and spans the BNSF Northtown rail yard, see location map in Figure 3. The BNSF's Northtown rail yard is one of the most heavily used freight and commuter rail yards in the Midwest. The Minneapolis/St. Paul metro area's Northstar Commuter Rail Line travels through the rail yard with current service of 10 trains per day. In addition, St. Anthony Parkway is on the "Grand Rounds Scenic Byway System" (the Grand Rounds) in Minneapolis which has been recognized by the FHWA as a premier National Urban Scenic Byway for its cultural, historic, natural, recreational, and scenic qualities. The Grand Rounds is managed by the Minneapolis Park and Recreation Board and is an important element in their regional park system, servicing a large percentage of pedestrian and bicycle traffic in northeast Minneapolis.

Figure 3 – Project Location

Preliminary Design

As part of the Grand Rounds, the City of Minneapolis required the replacement bridge structure to provide a wider cross section, shown in Figure 4, to accommodate two 14 foot travel lanes with a 10 foot sidewalk on one side and a 14 foot trail for non-motorized bicycle and pedestrian access on the other. BNSF required a reduction to a maximum of two piers in the rail yard to provide operations flexibility for future expansion and meet current code requirements for pier offsets in the rail yard for worker safety.

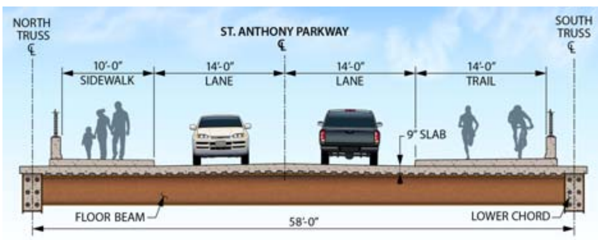

Figure 4 – Cross Section

The industrial feel of the area around the project site, historic nature of the existing structure and its designation on the Grand Rounds required public involvement in determining the future bridge at the site. Community input resulted in a request for a steel truss replacement structure, visually similar to the existing truss spans, to be studied for feasibility instead of a more modern arch structure. Parsons developed preliminary steel truss concepts to meet project design and geometric constraints while in parallel developing an integrated approach for a redundant design of a steel truss.

Parsons provided two concept truss types that fully met the redundancy and aesthetic requirements of the City of Minneapolis and Minnesota Department of Transportation. The first being a Pratt truss, Figure 5, and the second being a Warren truss, Figure 6. Both truss types use an H section for the top chord and compression diagonals or verticals as well as the same arrangement for the bottom chord. The top chord of both trusses is sloped to provide adequate drainage for the H sections and prevent collection of rainwater/debris. The primary difference between the two systems is in the arrangement and constructability of the diagonals.

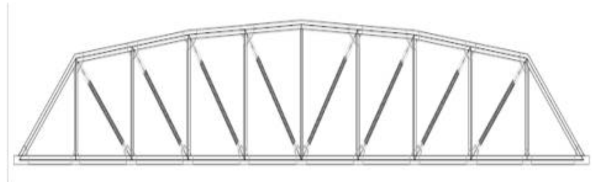

Figure 5 – Pratt Truss

The Pratt truss system utilizes vertical compression members and diagonal tension members. The advantage of this system is that all the diagonals are in tension, allowing the use of a multiple hanger cable arrangement to create redundancy, and all the vertical elements are in compression, which are shorter for a smaller buckling length. Cable hangers would be easier to replace than a solid member in the event of a failure, however, due to the forces restrained by the diagonal members, large diameter cables would be required. Cables of this size and weight would likely not allow for in-house maintenance/ replacement and such work would require a specialized contractor.

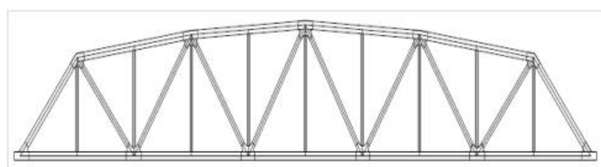

Figure 6 – Warren Truss

The Warren truss system utilizes diagonal compression and tension elements. The use of diagonal compression members requires longer and thus heavier compression members. However, this configuration creates the most uniform load path through the connections, simplifying detailing. The diagonal tension members use a split member to provide tension redundancy. Since there would be no cables to maintain, maintenance requirements would be less than that of the Pratt truss. Additionally, since there are fewer primary members with smaller secondary vertical members, the Warren truss provides a less 'busy,' more aesthetically pleasing system.

The Warren truss main span with approach spans shown in Figure 7 was selected for consistency with the existing structure and for its use of simpler connections and low maintenance detailing. The unpainted weathering steel provides a low maintenance and industrial brown aesthetic.

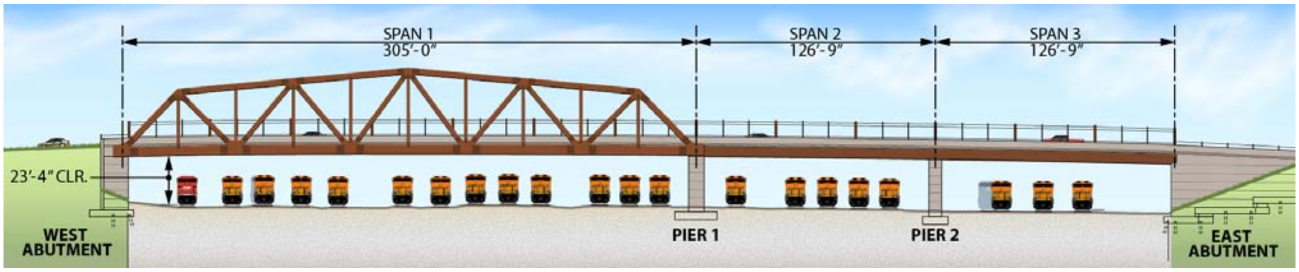

Figure 7 – Proposed Bridge Layout

Truss Redundancy

Typical truss design results in non-redundant and fracture critical tension member elements. Given Minnesota's current practice of not allowing fracture critical members on new bridges, developing load path redundancy concepts to eliminate such members and connections was of primary importance. In addition to this design goal, minimizing long term maintenance and inspection requirements by eliminating fracture critical inspections could be achieved. Finally in developing the redundant truss concepts, the aesthetic nature of the truss must be preserved to achieve the project goals for replacing the existing bridge with a similar truss structure.

A fracture critical member (FCM) is defined as a steel tension member or element whose failure is expected to result in the partial or full collapse of the bridge. In a non-redundant bridge, system identification and management of FCMs are an essential part of the design, fabrication, inspection and maintenance of the structure. Providing redundancy means the bridge has the ability to safely carry some level of live load in a damaged condition without collapse. This can be achieved through load path redundancy, structural redundancy and/or internal member redundancy.

To achieve load path redundancy for tension members, multiple main supporting members must be provided between points of support. The typical definition of this is more than two parallel girders or trusses. However providing more than two parallel trusses was not an option in this case; therefore the concepts developed provide an additional load path within the truss element for failure of any tension member.

Structural redundancy can be provided by developing three-dimensional system load redistribution. Should one element fail, collapse would be prevented by the redistribution of load to adjacent spans or members. FHWA has noted in a recent memo (1) that structural redundancy can be used to limit in-service fracture critical inspection requirements as long as the members are fabricated to FCM standards. FHWA has classified these members as "System Redundant Members" (SRMs), as such they are designated SRM on the plans and fabricated to fracture critical standards. Changes in the condition of the bridge elements that affect the analysis to classify them as SRMs could result in them becoming FCMs and requiring fracture critical inspection in the future.

In tied arch design, internal redundancy of the tie girder tension member is often achieved by bolting together the built-up section to limit propagation of fracture through the cross section and designing for the failure of any of the plates. This strategy provides additional safety if an element of the built up section were to fracture, however because it may not be visually evident that a plate has fractured it is likely to go undetected without a hands-on fracture critical inspection. Therefore the member is still considered a FCM per FHWA.

Of these redundancy approaches, only load path redundancy qualifies a tension member to be identified as a non fracture critical member for the purpose of design and fabrication to a higher quality standard.

Development of the Truss

The primary steel member sections evaluated for the truss include closed box and open H or I sections as shown in Figure 8.

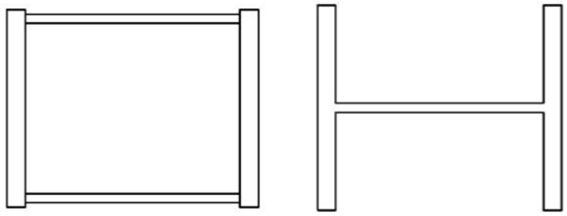

Figure 8 – Closed Section vs. Open Section

Both sections are capable of being incorporated into a redundant design; however, the closed section poses certain challenges. Due to the size of the truss structure and the magnitude of the forces carried by the individual members, the size of the closed section required would be in the order of 2'-0" to 2'-6" square. This is relatively small for a closed box section and would not facilitate inspection from within the member. Additionally, since internal access would be limited, fabrication of a box section this size would be difficult. On the other hand, an open section would provide complete access for fabrication and inspection and would not require sealing. For these reasons an open cross section was selected for the primary truss elements.

The open section can be oriented with either the web in the vertical plane (I section) or with the flanges in the vertical direction (H section) as shown in Figure 9.

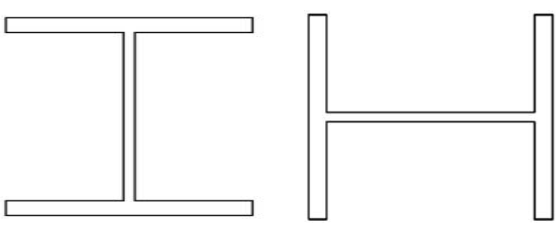

Figure 9 – I Section vs. H Section

The vertical web I section allows for an integral truss connection which eliminates traditional gusset plates as shown in Figure 10. Additionally, the I section does not create areas that trap water and debris as is possible with the H section. However, this configuration creates cause for concern for the following reasons:

- Discontinuous Load Path – As load transitions from the flanges through the web connection and back to the flanges, a discontinuous load path is created. This creates high stress regions at the interfaces between the web and flanges.
- Stress Concentration – As load is transferred from the compression diagonals to the tension diagonals through the integral connection, a high stress zone is formed in the web of the connection.
- Non-redundant Load Path – As load travels through the diagonals to the top and bottom chords it must travel through a single web plate. Failure of this plate would cause failure of the system and without load path redundancy, FCM inspections and maintenance would be required.

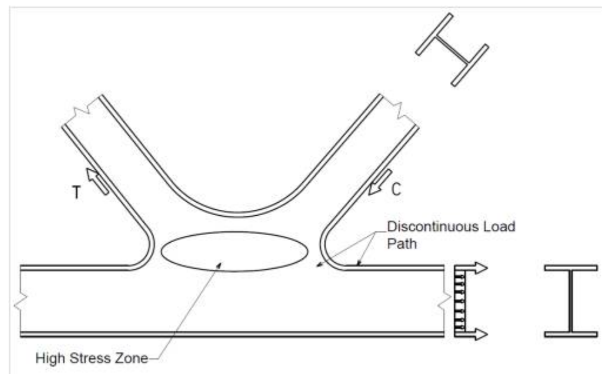

Figure 10 – I Section Integral Truss Connection

The shortcomings of the I section can be mitigated through the use of a vertical flange H section. With the H section, redundancy is provided by allowing the load to continuously travel through the flanges of one section to the flanges of another section through the use of redundant nodal connection plate members. This provides for simplified load path continuity without the flange to web to flange transitions required with the I section. One drawback of the H orientation is that it creates the possibility of trapping debris and water on horizontal sections; however this can be mitigated by sloping the top chord. With redundancy as a primary concern, the vertical flange H section configuration was selected for the primary truss elements.

To provide continuity of load path and simplify connection detailing, the top chord and diagonals are designed to be H sections of the same overall width. This allows load to flow continuously from chord and diagonal flanges to the nodal connection plate member flanges through splice plates, all along the same plane. This dual plate system creates load path redundancy as each connection plate is sized to accommodate the fracture load. The top chord connection concept with vertical flange H sections is shown in Figure 11.

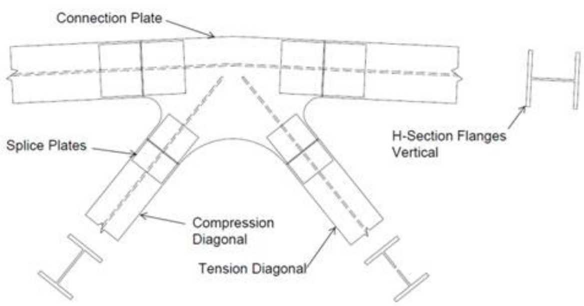

Figure 11 – H Section Top Chord Connection

Tension Diagonal Redundancy

The tension diagonals (and verticals) are H sections split into two T shaped members that are individually connected to the nodal connection plate members with a bolted splice at the flange, creating two independent tension members for each diagonal. Each tension member in the diagonal is sized to accommodate the fracture load individually, thus full load path redundancy is achieved through the tension members.

Since the tension members are connected only to the flanges of the H shaped nodal member which are in the same plane, failure of any tension member will create an eccentric load on the nodal member, which will result in visibly detectable member deformation. Due to the fact that the tension diagonals and the node connections will both provide load path redundancy and that member failure will be easily identifiable, these members are load path redundant. As a result they will not be subject to the more stringent FCM inspection and maintenance requirements. However, to achieve higher material and fabrication standards, the tension members have been classified as "SRM" on the plans. A rendering of the top chord nodal connection including vertical members and upper bracing is shown in Figure 12.

Bottom Chord Redundancy

The primary challenge of a truss or truss-like structure is providing tension member redundancy to the bottom chord. Approaches to achieving this include:

- Utilizing more than two truss lines/panels allowing for complete redistribution of the remaining truss lines/panels in the event of a failure.
- Doubling of the chord section of a single truss panel to allow a complete second load path within the same panel in the event of failure.
- Providing a secondary backup system such as external post-tensioning tendons or deck/stringer system to accommodate the load in the event of a main chord failure.
- Utilizing a composite post-tensioned concrete bottom chord in compression to eliminate the fracture critical tension member designation.

Figure 12 - Rendering of Top Chord Connection

After consideration of the alternatives with regard to redundancy, constructability, cost, durability, maintenance and inspection, a post-tensioned concrete bottom chord was chosen for the St. Anthony Parkway Truss. Post-tensioned concrete members are precompressed and highly internally redundant with the tension force taken by multiple tendons that each consist of multiple prestressed strands, made up of seven individual wires wrapped together to form the strand. The tendons are encased in concrete and grouted after stressing to provide continuity along the tendon. Additional redundancy of the bottom chord is achieved by designing the chord to withstand the loss of one post-tensioning tendon. Parsons' recent design of the new, highly redundant Hastings Tied Arch Bridge over the Mississippi River for the Minnesota Department of Transportation also incorporates a similar post-tensioned tie girder (2).

Construction of the post-tensioned bottom chord is accomplished in two phases. First, a "temporary" bottom chord consisting of a built-up steel U-shaped section is constructed for erection as seen in Figure 13 along with bottom chord nodal connection member.

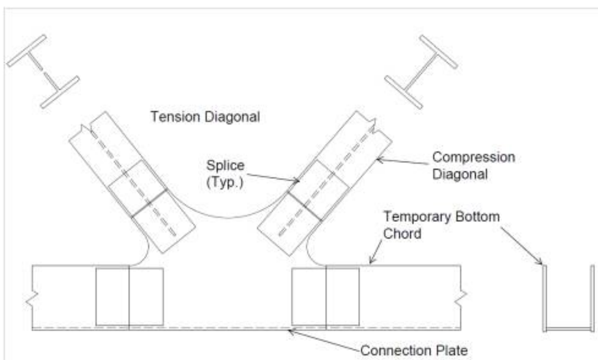

Figure 13 – Bottom Chord Connection

This steel section serves as the bottom chord during erection only. Second, once the truss has been launched into place and prior to construction of the deck, post-tensioning tendons are installed on the interior of the U-shaped section which serves as a form for construction of the permanent post-tensioned concrete bottom chord as shown in the cutaway rendering in Figure 14.

This arrangement is beneficial for several reasons. First, it allows the truss to be launched without the weight of the concrete bottom chord and eliminates the necessity of concrete formwork over the active railroad lines below. Second, it utilizes a redundant post-tensioned concrete element as the primary structural tension member which is precompressed and not a fracture critical member.

Diagonals are connected to the bottom chord in much the same way as to the top chord. A connection plate nodal member provides a continuous load path between diagonals through splice plates. Connection plates are in plane with the lower chord vertical plates and forces into the concrete are developed through shear connectors within the U-shaped nodal member. The connection is open above the deck and the deck sloped to drain water away from the connection.

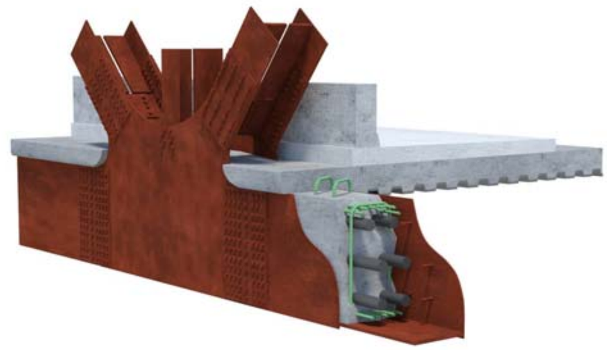

Figure 14 – Bottom Chord Rendering with Deck

This connection configuration simplifies construction and provides load path redundancy through the dual plate nodal connection member as in the top chord connection. Since the bottom chord is not a FCM and the tension diagonals and connection plates are also not FCMs due to load path redundancy, stringent FCM inspection requirements will not apply to the bottom chord or bottom chord connections.

The concrete deck extends over the bottom chord and is raised around the truss connections to channel water away from the connections protect the bottom chord. The truss connection penetrations through the deck and adjacent to the travel way are painted to protect against excessive corrosion due to salt spray minimizing future deterioration and maintenance concerns.

Fracture Design Criteria

The following fracture design criteria were used to evaluate the loss of any single tension element or connection in the truss. Loss of the entire tension member and any resulting eccentric loading on the remaining members was considered in the analysis. The remaining members of the truss were checked against progressive collapse under the following extreme event fracture load combination: 1.1 DC + 1.35 DW + 0.75 (LL+IM) + 1.1 FDF, where LL+IM is the full live load in striped lanes plus pedestrian loading and FDF is the fracture dynamic force.

The extreme event fracture load combination provides a reduced loading in an extreme event situation at which the structure can safely support the full dead load and a portion of live load and is based on the Post-Tensioning Institute (PTI) stay cable loss combination (3). The FDF for steel tension elements are determined based on the PTI stay cable loss recommendations using twice the force in the member acting in the opposite direction. The loss of a single post-tensioning tendon is considered for the post-tensioned bottom chord with no FDF considered for the grouted tendon.

The remainder of the structure is designed to meet the following performance criteria under the extreme event fracture load combination. The demand to capacity (D/C) ratio for all members is designed to be less than or equal to 1.0 at the extreme event limit state, where $\varphi = 1.0$. The steel tension elements may exhibit local yielding where plastic section analysis design is permitted in accordance with AASHTO LRFD (4). Post-tensioned concrete elements may be in tension and were checked for combined axial and moment to satisfy Equation 1 below and shear and torsion in accordance with AASHTO LRFD 5.8.

Equation 1: $$\frac{N_u}{\phi N_n} + \frac{M_u}{\phi M_n} \leq 1$$

To safely support the structure under a reduced loading, it must be evident that a fracture has occurred so the bridge can adequately be repaired in a timely manner to return the structure to normal operating conditions. As such, the redundancy approach required the structure (and any portion of the structure) to not collapse under extreme event fracture loads but exhibit visible deformation under service loads.

Floor Beam Redundancy

The floor beams span between the bottom chords of the through truss and are attached with bolted connections to the temporary steel bottom chord that is made integral with the permanent post-tensioned bottom chord through shear connectors. The floor beams are spaced at approximately 10 feet and directly support the concrete deck that spans longitudinally between the floor beams in the primary direction. The deck is made integral over top of the concrete bottom chord and thus supported on all four sides of each bay.

With the relatively close floor beam spacing and integral deck over the floor beams and the bottom chord, the floor beams are shown to be structurally redundant by removing a floor beam and analyzing the deck spanning between two floor beams and supported on each end by the bottom chords. A yield line analysis shows sufficient capacity without adding additional rebar. The removal or fracture of a floor beam will result in noticeable deflection in the deck and the floor beams are classified as SRMs on the plans for better material and fabrication while not being considered fracture critical.

Redundancy Approach Summary

- All steel truss tension members and connections achieve redundancy through a direct alternate load path capable of safely supporting a reduced loading in an extreme event situation providing load path redundancy.
- Redundancy is provided to the main bottom chord tension member by a highly internally redundant post-tensioned concrete member that takes all of the tension force and is not fracture critical.
- The floor beams are structurally redundant members after considering the behavior of the deck when a floor beam is removed.
- Visible deformation will occur in the event of a fracture.

Construction Challenges

The construction approach is centered on minimizing interruptions to rail traffic in the busy yard below. The configuration and weight of the existing trusses foster the removal via a modified launching technique. This allows for much of the work to be safely performed adjacent to the live tracks. The preferred concept is to minimize the modifications required to the existing trusses and substructure and leaving the existing substructure in place is a key element in the installation of the new truss. Installation of the new truss is accomplished by a similar launching technique with the demo of the substructure to follow the installation.

Favorable consideration was given to the modified launching approach as this allows for better containment of the environmental contaminates during demolition if performed on grade away from the active rail yard. It also provides for a reduction of equipment required in the rail yard below the structure.

Coordination with the railroad to allow continual traffic through the yard is accomplished by switching the through traffic to either end span, allowing as large of a construction area as possible. This method promotes optimization of the construction work area, but also has the challenge of working within allowable track time given by the railroad.

Launching Concept

The project requirements were such that a conceptual removal and erection method was required to be included in the bid documents. By utilizing a modified launching technique, Parsons recognized the benefit of preserving the existing support conditions of the existing trusses. This provided for a reduced cost through minimizing structural modifications and minimized risk considering that the existing truss structure contained asbestos and lead paint.

During the design, consideration was given to utilizing the permanent girders in the approach spans for the temporary removal and erection loading. This was accomplished by pairing two girders and forming a box section. The box section more efficiently resists the lateral loading that can develop as launching progresses and the lateral travel restraints are engaged. The span lengths were of sufficient length to allow the approach span girders to span the existing piers and provide temporary support during both the removal of existing trusses and erection of the new truss. A marginal increase in the flange plate sizes was required to accommodate the launching scheme. The option to use four of the approach girders for launching was provided to contractors in the bid documents, as the use of permanent materials for temporary works is typically not permitted.

The existing piers are positioned along the skew and are two square column piers which rest on spread footings and are connected by a web wall. The existing trusses are supported directly by the columns. Due to the skew, the square columns leave a large corner portion exposed. This spawned the idea of using this exposed corner to support a launching beam. The launching beam would be positioned on the existing piers, the existing trusses raised up and slid along the launching beams for removal. The same configuration would then be used to launch the new truss.

It was determined from field measurements, however, that the exposed corners of the columns were not large enough or in satisfactory condition to support the launching beams without a retrofit. Two retrofit options were considered. The first was to modify the existing column cap and increase its size to support the launching beam. This would require exposing the existing reinforcement and casting a new cap with the existing trusses in place. The second option considered was to mount a pipe pile directly to the outside of the column and support the pile on the existing spread footing. As this would not require demolition of a portion of the existing columns and does not rely on the questionable condition of the existing column, the second option was selected as the most prudent recommended launching beam support method.

The conceptual removal and erection plan utilized the existing piers to support the launching beams. The steel column is affixed to the exterior of each existing pier allowing for the temporary column to be braced to the existing pier wall and the axial load resisted by the top of the existing footings. This method accommodates both the existing structure width of 37 feet and the new truss width of 58 feet. Adjusting between the structures widths was accomplished by shifting the support location from the exterior for the existing trusses to the interior for the new truss through the open deck.

One challenge during the removal of the existing trusses comes from the end bays of each truss. The floor beams of the existing trusses are normal to traffic with the stringer lengths varying with the skew of the bridge. In the end bays of each truss, one end of the stringers is supported by a floor beam and the other by the pier wall. To move the existing truss, the load has to be removed from the stringer ends supported by the pier wall. The conceptual method to accomplish this is to utilize a transverse beam run along the skew of the bridge adjacent to the joint near the end of each truss. The deck dead load is carried by the transverse beams through anchors into the deck and stringer ends.

An alternate concept of removal of the end bay of the truss deck was considered, but deemed less desirable as it required in-situ demo and left poor access on the bridge deck. Conceptual launching sketches are shown for the removal of the existing trusses and installation of the new truss in Figures 15 and 16 respectively.

Conclusions

The unique constraint of the highly active BNSF Northtown rail yard, one of the most heavily used rail yards in the Midwest including the Northstar Commuter Rail Line's 10 trains per day, greatly affected the design and construction approach for the St. Anthony Parkway Bridge replacement. In order to minimize both construction and long term maintenance impacts to the rail yard below and provide increased worker safety, the final structure is designed to accommodate unique erection methods, provide low maintenance details and eliminate FCMs and their associated inspection requirements.

St. Anthony Parkway Bridge replacement project demonstrates several unique approaches to providing for redundancy in steel truss member design and provides load path redundancy within the truss to eliminate fracture critical members. The project also demonstrates specialized bridge construction concepts utilizing launching beams to both remove existing truss spans and move new truss spans into place, thereby minimizing disruption of service to one of the nation's critical railways.

Acknowledgements

Owner: City of Minneapolis, Minneapolis, MN

Prime Consultant: Short Elliott Hendrickson Inc., Minneapolis, MN

Architect: Touchstone Architecture, Tallahassee, FL

Reviewing Agency: Minnesota Department of Transportation, St. Paul, MN

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(2) Greg Hasbrouck and Martin Furrer, *An Innovative Tied Arch Solution to Spanning the Mississippi.* 2012 World Steel Bridge Symposium, NSBA/AISC, 2012.

(3) *Recommendations for Stay Cable Design, Testing, and Installation*, 6th Edition, 2012. Post-Tensioning Institute, Farmington Hills, MI.

(4) *AASHTO LRFD Bridge Design Specifications*, 6th Edition, 2012. American Association of State Highway and Transportation Officials, Washington, DC.

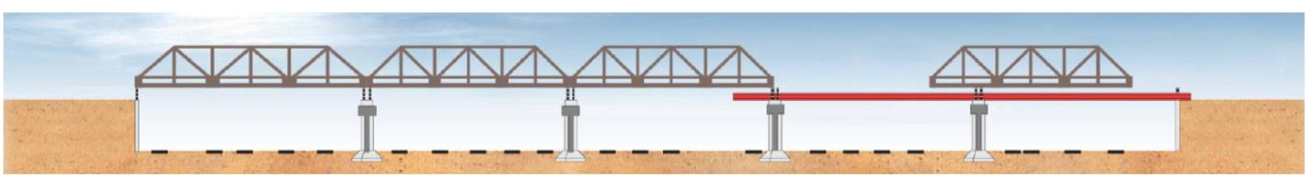

Figure 15 – Existing Truss Removal

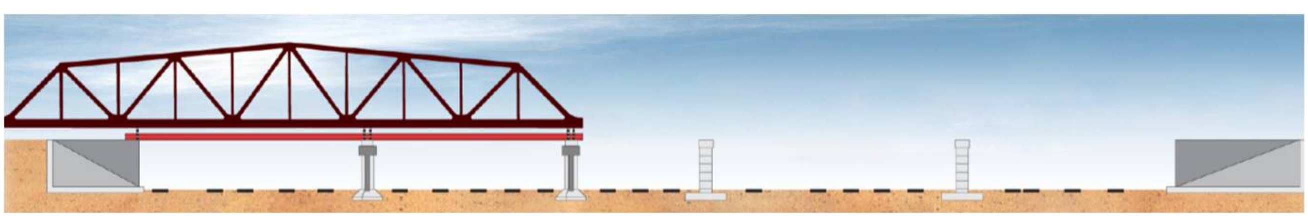

Figure 16 – Installation of New Truss