WALK THIS WAY: PEDESTRIAN BRIDGES - UNIQUE DESIGN AND ANALYSIS

Irvin J. Lopez, PE

Ken Price, PE, SE, PEng

Rachel Cullen, CPEng, MIEAust

Joshua Sletten, PE, SE, PMP

Joseph Smith, PE, SE

BIOGRAPHY

Irvin J. Lopez is a Senior Bridge Engineer with the Movable Bridge Group at WSP USA. He is an alumnus of North Carolina State University (Master) and University of Puerto Rico, Mayaguez Campus (BSCE). Irvin was the lead designer for the deck and steel superstructure of the bridge.

Ken Price is the Manager of the Complex Bridge group at WSP USA. Ken was the Engineer-of-Record for this bridge and has functioned at various levels in the US bridge engineering community for many years.

Rachel Cullen is a Senior Bridge Engineer at WSP USA. She received her Bachelor of Engineering degree at the University of Queensland in Brisbane, Australia. Rachel was the Design Lead for the UVU Pedestrian Bridge and is currently taking on the role of Project Engineering throughout construction.

Joshua Sletten is the Bridge Group Lead in the Salt Lake City office at WSP USA. He is an alumnus of Purdue University (Master) and South Dakota School of Mines & Technology (BSCE). Joshua served as the Project Manager for this bridge.

Joseph Smith is a Supervising Bridge Engineer at WSP USA. He is an alumnus of University of California at San Diego (MS). Joseph was the lead engineer for the redundancy analysis.

SUMMARY

Pedestrian structures function at the small end of the bridge scale for size and cost, but they often make a large-scale contribution to visual interest at the user level and to technical complexity at the engineering level. Experience in the USA has shown this to be true and the Utah Valley University (UVU) pedestrian bridge is no exception.

At the user level and the engineering level, this 971 feet long pedestrian bridge in Orem, Utah, provides students, faculty and the general public a safe crossing to the main campus over Interstate I-15, a transit line, private property, and several tracks of a US Class 1 Freight Line. The bridge is on a horizontally curved alignment and is comprised of four spans that are carried by a two-girder system. The bridge has an uneven span arrangement due to site constraints, uses of 3 fixed piers due to large seismic loads, deck heating system, roof structure, high value to aesthetics, and a redundant two-girder system as determined by a fracture and redundancy analysis. This project is also being delivered as a CM/GC project.

This presentation focuses on the unique design features and practical lessons learned for refined analysis of a two-girder system. It will also present some of the benefits of the CM/CG process and its impact during the design phase.

WALK THIS WAY: PEDESTRIAN BRIDGES - UNIQUE DESIGN AND ANALYSIS

Introduction

Utah Valley University (UVU) has grown significantly since its inception and is now the largest four-year university in Utah with over 40,000 students.

Currently, movement between the UVU campus to the east and the Intermodal Transit Center to the west requires pedestrians, cyclists and shuttle traffic to travel through the busy University Parkway Interchange in Orem, Utah. UVU plans to expand their campus to the west, further increasing the need for mobility between these two locations. The proposed UVU Pedestrian Bridge will provide students and transit users with safe, direct access over Interstate-15 (I-15) and the railroad between the Transit Center and the UVU campus.

This project was a collaborative effort between the Utah Department of Transportation (UDOT), UVU, and the Utah Transit Authority (UTA) to fund and build a signature pedestrian bridge.

Due to right-of-way limitations in the area, the bridge needed to clear span both I-15, and the seven-track railroad corridor including both UPRR and UTA tracks. The challenging location and long spans required significant construction staging and erection planning, while minimizing disruptions to I-15 and railroad traffic during construction.

Concept Development and Acceptance

As UVU requested an aesthetically distinct design, WSP, in conjunction with Architect, Method Studio, developed multiple alternatives and worked with all project stakeholders to select a structure type.

Based on stakeholder input, an Inverted Fink Truss on an 'S' curve alignment was the initial preferred bridge type. As design progressed for this unique structure, it became clear that an integrated construction manager approach, known as a Construction Manager / General Contractor (CM/GC) contract could benefit this project. A CM/GC contract involves the Contractor providing constructability and pricing feedback during design which helps to reduce construction risk and cost.

Kraemer North America LLC (KNA), hired by UDOT as the General Contractor during design, collaborated with the team to re-evaluate the solution. Ultimately, these workshops led to changing the proposed bridge type to a curved, four-span, twin steel plate girder alternative, a bridge type more familiar to the local transport infrastructure industry in Utah. The bridge has four-spans, a total length of 971 feet and a constant plan radius of 2,400 feet along the bridge centerline. The longest span measures 305 feet and crosses over I-15. A rendering of the bridge is shown in Figure 1. The cross-section consists of a 16.83 feet wide 8-inch thick concrete deck carried by two welded-plate girders spaced at 10 feet as shown in Figure 2. The girders have a varying web depth of 5 feet to 9 feet. Floorbeams and cross-frames are spaced at 15 feet, and there are upper and lower lateral braces on Spans 1 thru 3. The floorbeams coincide with outrigger brackets on the outside of the girders which support the roof columns. The deck is continuous throughout with the only expansion joints located at the ends of the bridge.

Figure 1: Rendering of the UVU Pedestrian Bridge.

Though more conventional than an Inverted Fink Truss, this bridge still presented many challenges and required innovative solutions to meet our Client and Stakeholders' expectations.

The bridge required an aesthetic design that was compatible with the aesthetics of the University. To meet this requirement, WSP and Method Studios incorporated numerous design features, including a variable height roof to replicate the "mountain peaks" which is congruent with the architecture of the existing University buildings. The roof uses transverse steel joists with longitudinal steel members to create an efficient structural system that is simple to fabricate and erect.

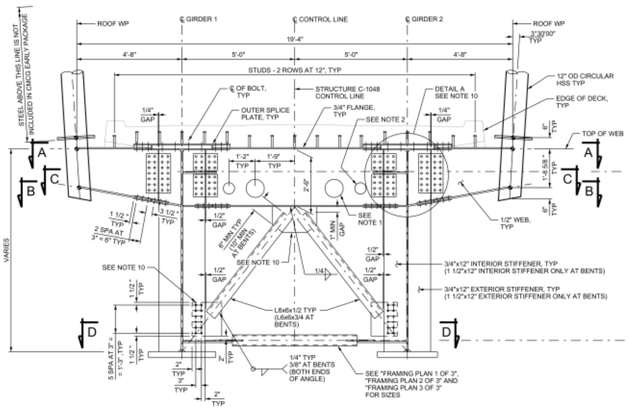


Figure 2: Typical cross-section of bridge

Additionally, the perforated steel side enclosure, heated deck system, oversized freight elevators and aesthetic treatment of to the landing towers added to the aesthetic nature of the bridge.

CM/GC Process

Throughout the design process, collaboration workshops were held with the Contractor, Fabricator and Erector which helped to adopt design details that would help reduce the construction time, cost and risk.

The CM/GC process helped identify that an early release steel package was required to meet the construction schedule deadlines with the long lead times on steel. The early steel package plans were released for construction seven months before the full package was released. The early steel package included the steel girders, cross bracing, lateral bracing and all connection details. Figure 3 presents an infographic presenting some of the benefits of the CM/GC process for this project.

The site has many critical infrastructure elements in close proximity to each other including a sewer line, commuter and freight rail lines, and mainline I-15 supported on MSE walls. Throughout the CM/GC process, WSP worked with KNA to ensure the design mitigated potential risks related to these.

During the design process, there were several value engineering proposals taken on by WSP and KNA. The initial design included drilled shafts, however due to the increased testing requirements of shafts in Utah, the foundations were re-designed using driven piles, which also significantly reduced the cost. In addition to this, a number or architectural items were adapted to meet budget requirements while maintaining the aesthetics. WSP and KNA studied multiple material types and products to find efficient solutions for the 'timber lookalike' roof soffit, the perforated steel side enclosure panels, the architectural lighting and the landscaping at each end of the bridge.

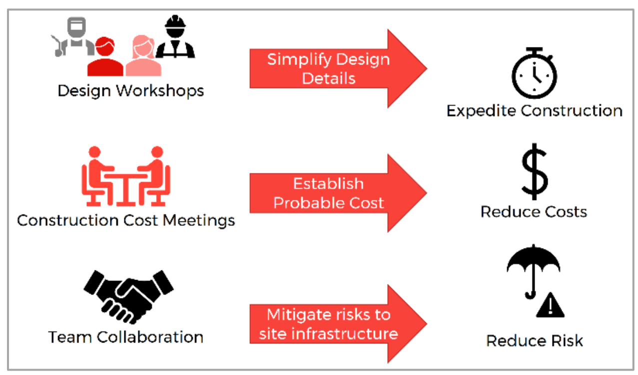


Figure 3: Benefits of CM/GC Process

Structural Analysis and Design

Horizontally curved bridges are considered complex structures in part due to their inherent three-dimensional response to gravity loads. Their tendency to "roll" under vertical loading requires

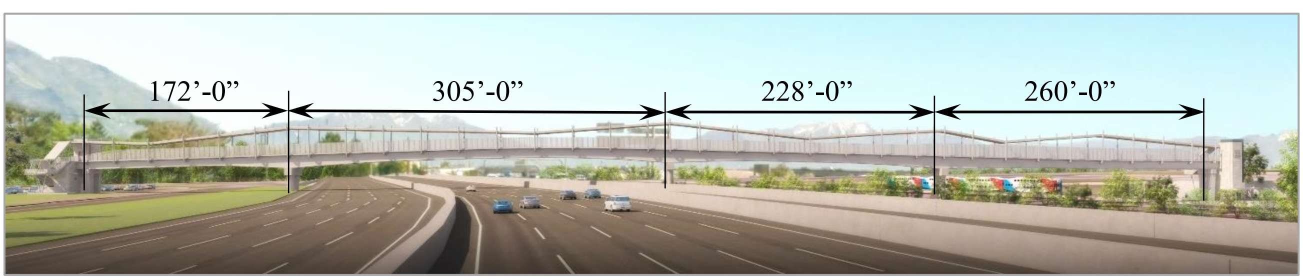


Figure 4: Rendering of the UVU Pedestrian Bridge North Elevation

special consideration during the analysis and design to ensure lateral stresses are properly addressed. During the preliminary analysis phase, it was found that the lateral distribution of loads varied significantly between girders, and thus using the same steel cross-section for both girders would not be economical. The curve radius, span length, and support conditions were such that even after decreasing the spacing of cross-frames the lateral distribution of loads was still greatly uneven. The AASHTO LRFD Bridge Design Specifications[1] (AASHTO, 2017) recommends that refined analysis methods be used for determining force effects on structures curved in plan where stiffness between girders varies as was the case for this bridge. A rendering of the north elevation of the bridge showing the span arrangement is shown in Figure 4.

CSIBridge®[2] (Computers and Structures, Inc., 2018) was the analysis and design software used. Figure 5 shows a rendering of the analytical model highlighting some of the analyses and design exercises carried out. All main load carrying structural elements were discretely modeled, including the roof structure. The structural model was comprised of a total of 3,756 joints, 1,765 frames, 2,528 plates, 30 springs. The deck and girder web were modeled as plate elements (also referred to as shells in CSIBridge), while the girder flanges, stiffeners, connection plates, cross-frames, floorbeams, lateral bracing, roof beams and columns, pier caps and columns, and piles, were modeled as beam elements (also referred to as frames in CSIBridge). Joint constraints were used to model the composite behavior between the deck and girders. Springs were used to model the bearing at each pier. Such springs would connect the bottom flange of the girders to the top of the pier cap beams. Another set of rigid springs would then transfer the loads from the bearings to the centroid of the pier cap beams. Although not critical for the analysis and design of the superstructure, the substructure was also discretely modeled. This allowed an integrative design process between the superstructure and substructure designers. For instance, the seismic demands were such that three interior piers needed to be fixed to be able to withstand the seismic demands. This change was easily updated in the model, with minimum rework from either designer.

The roof structure was included in the analysis model to investigate global behavior and interaction between the two structures with a significant difference in stiffness. A separate analytical model was developed to analyze and design the roof structure itself.

Another unique aspect incorporated into the structural analysis model was the vertical profile curve which varied from 1.35% to 3.61%. This allowed the design team to obtain a more accurate approximation of the analysis results. It is worth mentioning that bending stresses increased up to 2% after adding the vertical profile curve in the analysis model.

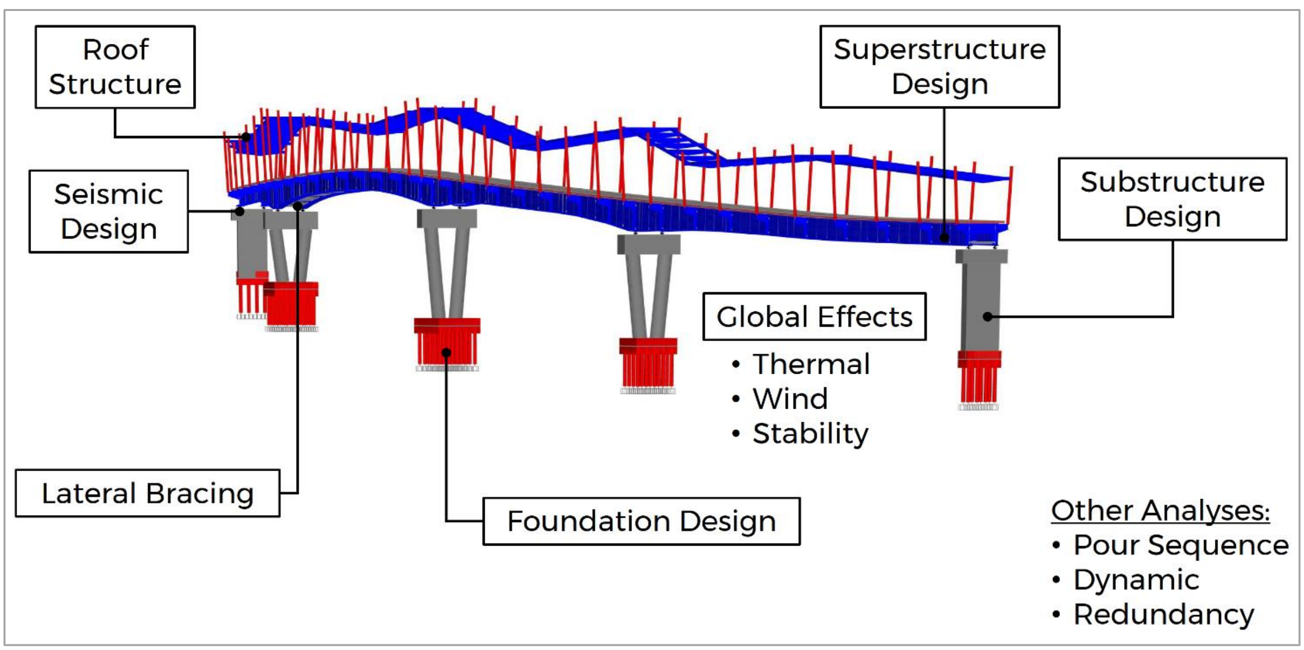


Figure 5: 3D Finite Element Analytical Model of the UVU Pedestrian Bridge

Global Effects

The AASHTO LRFD Bridge Design Specifications[1] (AASHTO, 2017) provide a simplified procedure under Article 6.10.3.4.2 to evaluate the global lateral-torsional resistance of a span during deck placement. The formula provided for estimating the elastic global lateral-torsional buckling resistance of the span acting as a system is applicable for spans of straight I-girder bridge units with three or fewer girders. This resistance was then compared to the largest total factored girder moment during deck placement (e.g. with unhardened concrete deck) to determine whether excessive amplification of lateral and vertical displacements would be expected. For horizontally curved girder bridges this approach can only provide a general indication of the susceptibility to second-order amplification effects under noncomposite loadings. For this reason, using CSIBridge, a linear buckling analysis was carried out to evaluate the noncomposite loading condition in which all the wet concrete load was applied in a single pour. This was considered conservative since a pouring sequence was specified in the plans. The results of this analysis showed that buckling under the noncomposite loads was not expected.

Horizontally curved steel bridges tend to have large web rotations, also referred to as girder layover, at midspan during steel erection and deck placement. During the design and analysis of the bridge, the web lateral deflections were investigated to ensure they were within acceptable limits. The AASHTO/NSBA Guide Specification S10.1-2014[3] (AASHTO/NSBA, 2014) suggests that end web rotation be kept below 0.6 degrees (1/8 inch of layover per foot of web depth) to prevent fabrication and field fit-up issues. Furthermore, the *Skewed and Curved Steel I-Girder Bridge Fit*[4] publication (NSBA, 2016) suggests that web rotations of 2 degrees ($^{7}/_{16}$ inch of layover per foot of web depth) or less do not present strength or stress concerns. During the analysis, web rotations of 2.1 degrees were noted at midspan of Span 1 under steel and wet concrete dead load.

At this stage, no lateral bracing had been incorporated in the design. Through various iterations a lateral bracing layout was defined which reduced the girder layover to 1.8 degrees. Accounting for the pouring sequence in the analysis also helped in reducing the girder layover. Figure 6 shows the final lateral bracing layout in Span 1.

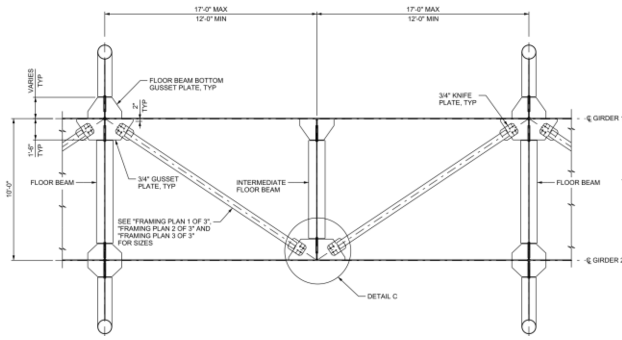


Figure 1: Typical Bay of Lateral Bracing

Having a roof structure covering the deck means a temperature gradient between the concrete deck and girders is unlikely to result in significant stresses. However, since the roof and bridge superstructure have significantly different stiffness and support conditions, several temperature gradient cases were studied. Four main temperature cases were investigated:

1) Roof is 50°F warmer than Superstructure
2) Deck is 50°F warmer than roof and girders
3) Superstructure is 50°F warmer than roof

These analyses were conducted to ensure proper dimensioning of the roof expansion joints at bridge ends, to investigate torsional moments in the outrigger brackets supporting the roof columns and their connections, and the roof beam-to-roof column connections. Case 2 and 3 were included to represent the scenario in which the deck heating system warms the deck and superstructure during a cold winter day. Case 1 represents a typical sunny day where the roof is exposed to sunlight. Figure 7 shows the deflected shape at Pier 1 under Case 1.

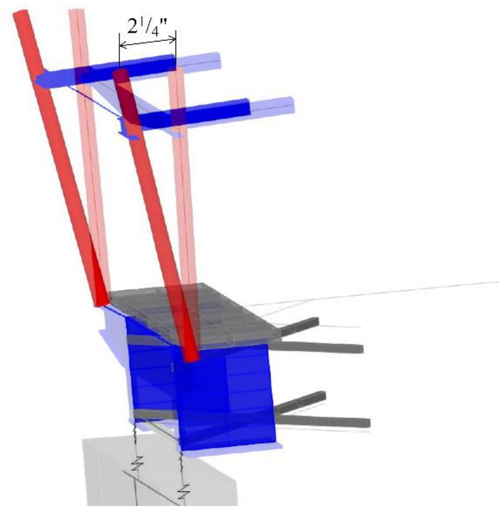


Figure 2: Roof expansion at Pier 1 under Case 1

Another interesting finding–often ignored in most bridges having only one fix pier–was the development of large horizontal bearing reactions from vertical loads. This is a result of having the interior piers fixed paired with seated bearing connections (e.g. an eccentric connection with respect to the girder center of gravity). As the bridge deflects and rotates under vertical loads, since the bottom flange is restricted from movement, a horizontal reaction will develop and thus needs to be accounted for when having more than one fixed bearing. In lieu of accounting for these forces in the design of the bearings and substructure, designers could consider specifying the use of temporary expansion bearings or hydraulic jacks in front of the final bearing location that could accommodate these rotations and movements during steel erection and concrete deck. Once concrete deck hardens, girders can be lowered and attached to the fixed bearing.

Seismic Design

Typically, for continuous bridges in non-seismic regions only one interior pier is fixed to resist all the horizontal forces from the superstructure (e.g. Braking Force and Wind), however, this was not the case for the UVU Bridge. The bridge is located on a Site Class D with a peak ground spectral acceleration of 0.352g. As mentioned previously, CSIBridge was used for both the superstructure and substructure design, including the seismic design. The use of a single analytical program minimized the seismic modeling time, post-processing of results, and updating structural model. The results from the seismic analysis confirmed the need to fix the three interior piers to withstand the seismic demands. Noting that proper modeling of the support conditions is critical for the superstructure and substructure design, the structural model support conditions were adjusted accordingly to fix all bearings in Piers 2, 3, and 4.

Wind and Vibration Study

AASHTO requires that bridges deemed wind-sensitive be investigated for wind-induced motions such as: buffeting, vortex excitation, galloping, flutter, and static divergence. During the preliminary design of the bridge a modal analysis was conducted to determine the fundamental periods of the bridge. The computer results showed several modes with fundamental periods larger than 1 second, and therefore, a sectional model test in a wind tunnel was conducted to investigate further and determine appropriate design wind pressures and verify instabilities from wind-induced motions. A 1:30 scale model was developed by RWDI Consulting Engineers and Scientists as shown in Figure 8. Some of the tests included looking at normal weather conditions under a smooth flow with the fences either uncovered or fully covered. The results showed that the cross-section was aerodynamically stable. It was only at significantly higher wind speeds than those specified for this project site, that some galloping instability was revealed, and thus the bridge was deemed stable from wind-induced motions.

Figure 3: Photo of the wind sectional model test

Due to the accelerated schedule of the project, the designers used AASHTO and ASCE 7 wind loads for the superstructure and roof, respectively, for the design. This was a risk taken by the design team to meet the early steel package submittal deadline. Upon completion of the wind load tests, the wind load design pressures used for design were found to be conservative when compared to those proposed by RWDI.

Redundancy and Fracture Analysis

Up until recently, structural engineers would have to refer to NCHRP Report 406 "Redundancy in Highway Bridge Superstructures"[5] (Ghosn and Moses, 1998) for guidance on redundancy of bridges. This report is one of the first attempts at quantifying redundancy in highway bridges by analyzing redundancy of a bridge system rather than determining redundancy based only on number of members. In 2018, AASHTO released two new guide specifications[6,7] (AASHTO, 2018) that provide guidance on analysis and identification of fracture critical members and system redundant members for both existing and new bridges, and internal redundancy of mechanically-fastened built-up steel members. All three publications deal directly with highway bridges and although they provide a solid baseline for what a redundancy analysis should consist of, they don't directly address pedestrian bridges. For instance, the fatigue load for highway bridges is a truck load whereas for pedestrian bridges is wind.

Experience has shown that pedestrian bridges are seldom governed by fatigue loading. In fact, the current pedestrian design live load of 90psf.[8] (AASHTO, 2009) is such that for large spans the design could exceed the HL-93 design live load of highway bridges as was the case for this bridge. Since fatigue load induced cracks are unlikely on pedestrian bridges, other potential fracture conditions were considered. Two factors that must be considered are constructability risks and damage in-service (e.g. from vehicle impact).

With these in mind, a commons sense approach was developed. First, a damage criterion was defined which represented complete failure of one girder at a critical location (e.g. bottom flange at mid-span) with concurrent live load overload. This scenario was considered particularly due to the presence of a bolted splice connection near mid-span of Span 3 over I-15. Although enough vertical clearance was provided, the risk of there being a collision due to an oversized load or a fracture resulting from other sources is still possible. Following the damage criteria, a collapse limit state was established which meant the bridge would not collapse under the damage criteria condition. Additionally, a functional limit state was defined to consider the bridge in service after the failure event. The following load combinations were used to represent these limit states:

1) Maximum Expected Loading: (LL = 90psf)
   1.00DC + 1.00LL + 1.00FDF

2) Minimum Expected Loading: (No live load)
   1.00DC + 1.00 FF
   where,

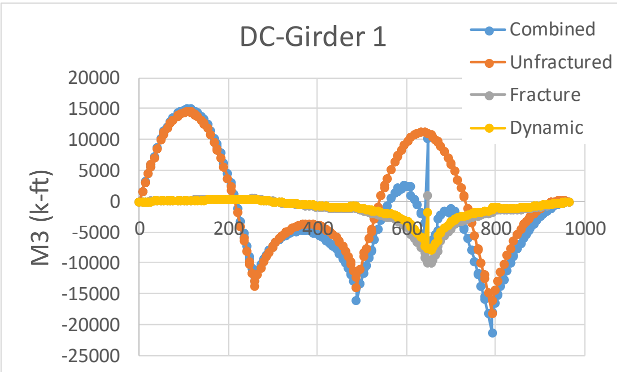
3) Overload Scenario (LL = 135 psf)
   1.00DC + 1.00LL + 1.00FDF

where, FDF $= (1 + DA_R)(DC + LL)$
FF $= (1 + DA_R)(DC)$
DA_R = Dynamic amplification factor

$DA_R = 0.85$

The dynamic amplification factor used is the maximum theoretical magnitude for a single degree of freedom[9] (Chopra, 1995). This value is conservative given the many degrees of freedom of the structure and the inherent damping of the structure.

The redundancy analysis was also conducted in CSIBridge®. First, the fracture was introduced in Girder 1 (which is the girder on the outside of the curve) at mid-span of Span 3. This was done by eliminating the girder frame elements and the deck shells elements at mid-span of Span 3. Following the fracture, the analyses were carried out for dead load and live loads which were then increased by the dynamic factor. Figure 9 below is a plot of the major axis bending moment of both girders under dead load case. The results from the unfractured, fractured, and dynamic factor are shown and superimposed (shown as the Combined curve). It is evident that load is shifting from Girder 1 to Girder 2. In some sense, Girder 2 is now acting as interior support for Girder 1 at the fracture location. This behavior is a result of the load path that is achieved through the composite action between girders and deck, closely spaced cross-frames (spaced at 15 feet), and the lateral bracing members.



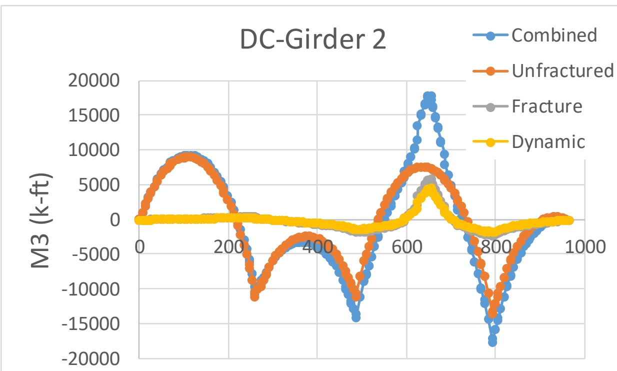


Figure 4: Dead Load Moment Diagrams for Girders Before and After Fracture.

This exercise involved an iterative process. After the fracture was introduced the nominal capacity of the primary members' (e.g. girders, floorbeams, cross-frames, lateral bracing, and deck) was compared to the analysis results. If capacity was exceeded, the member would be removed from the model and the structure reanalyzed. Multiple lateral brace members were identified as having reached their capacity and were removed from the analytical model. As for the floorbeam, cross-frames, and girders, none of them reached their capacities. In fact, it was found that the modeled bridge could withstand an overloading case of 135 psf., which is nearing the limit of number of persons that can physically be placed on the bridge. Through this exercise the bridge was found to be redundant system.

In terms of the functional limit state, or serviceability, the maximum total deflection under the overload case was 15 inches. Comparing this result with the recommended deflection limit of 72 inches (L/50 for Redundancy II Load Case), as specified on the AASHTO Guide Specifications[6] (AASHTO, 2018), it is clear the functional limit state is also met.

Fabrication

Aesthetics played a significant role on this project, even when sizing the girder bottom flanges. The UDOT Bridge Manual[10] has policies in place that require I-girder bridges to use same width bottom flanges throughout the length of the bridge. For this bridge, the governing bottom flange moment was in Span 1 resulting in a bottom flange of 36 inches wide by 3 inches thick. This mean carrying a 36-inch wide flange throughout the whole bridge. Although compromising on weight efficiency may not seem so elegant to a structural engineer, for this complex project it meant simpler and faster fabrication, and better aesthetic value. Additionally, having considerable reserved capacity certainly helped in achieving system redundancy.

Given the tight schedule, the design team worked together with the Contractor, Erector, and Fabricator to determine the final bolted field splice locations. All flange transitions were done at the bolted field splice locations, which helped expedite fabrication. There was a total of eight girder field sections with the longest section measuring 183 feet and weighing approximately 70 tons. This long section was in Span 1 over the Union Pacific Railroad and the Utah Transit Authority tracks. Due to site constraints and the limited locations to place temporary supports for erecting the steel girders, the Contractor requested a bolted field splice near midspan of Span 3 over the I-15 Interstate.

Originally, a Steel Dead Load Fit (SDLF) was specified as recommended in Table 4 of the *Skewed and Curved I-Girder Steel Bridge Fit*[4] publication (NSBA, 2016) for horizontally curved I-girder bridges with maximum ratio of length over radius (L/R_{max}) less than 0.2. The L/R_{max} for this bridge was about 0.13 taken at Span 3. After final design, the Contractor and Fabricator requested to change the bridge fit to a Total Dead Load Fit (TDLF). Although this requires special detailing by the fabricator, obtaining plumb girders after total dead load was deemed more critical by the Contractor to facilitate the field connections between the roof columns base plates and the outrigger brackets.

Conclusions and Lessons Learned

Complex bridges benefit greatly from alternative delivery methods. In this case, the CM/GC process worked well for this project. The numerous workshops and on-going dialogue between the design team and contractor were key in delivering a project on time and on budget.

Through engineering and proper use of advanced analysis tools, it was proven that continuous two-girder systems can sometimes be deemed redundant. In this case a simple and conservative approach was utilized to prove redundancy.

Steel erection risks and fabrication efficiency should be considered during the design phase as they could have significant cost impacts.

Acknowledgements

The authors would like to thank the following contributors involved in the management, design and construction of the UVU Pedestrian Bridge.

- Utah Department of Transportation (UDOT)
- Utah Valley University (UVU)
- Utah Transit Authority (UTA)
- Kraemer North America LLC
- Method Studio, Architect

References

1. AASHTO (2017). *AASHTO LRFD Bridge Design Specifications*, 8th Edition. American Association of State Highway and Transportation Officials, Washington, DC.
2. Computers and Structures, Inc. (2018), CSIBridge Version 21.0.0 (Computer software)
3. AASHTO/NSBA (2014). *S10.1-2014 Steel Bridge Erection Guide Specification*. AASHTO/NSBA Steel Bridge Collaboration
4. NSBA (2016). *Skewed and Curved Steel I-Girder Bridge Fit*. NSBA Technical Subcommittee
5. Ghosn, M. and Moses, F. (1998). *Redundancy in Highway Bridge Superstructure, NCHRP Report 406*. Transportation Research Board, National Research Council, Washington, D.C.
6. AASHTO (2018). *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*, 1st Edition. American Association of State Highway and Transportation Officials, Washington, DC.
7. AASHTO (2018). *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members*, 1st Edition. American Association of State Highway and Transportation Officials, Washington, DC.
8. AASHTO (2009). *LRFD Guide Specifications for the Design of Pedestrian Bridges*, American Association of State Highway and Transportation Officials, Washington, DC.
9. Chopra, A (1995). *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, 4th Edition. Prentice Hall, Upper Saddle River, NJ.
10. UDOT (2017). *Structures Design and Detailing Manual*, September 2017, Utah Department of Transportation, Salt Lake City, UT