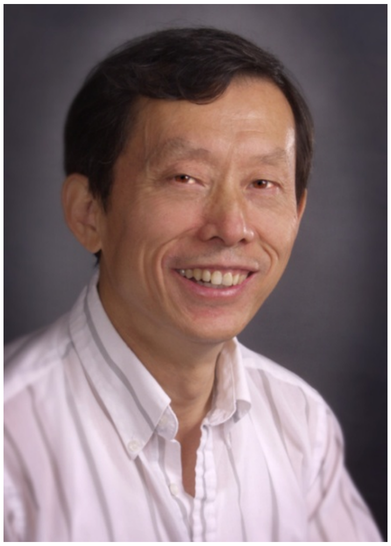
DESIGN AND CONSTRUCTION OF THE SR522/US2 INTERCHANGE FLYOVER RAMP



Hongzhi Zhang, Ph.D. PE

BIOGRAPHY

Hongzhi Zhang received a Bachelor's Degree of Science in Material Science in late 1960's and a Master's Degree in Civil Engineering in 1980's, while in People's Republic of China. In the early 1990's, Hongzhi was awarded with his Ph. D. in Structural Dynamics and Earthquake Engineering from Old Dominion University, here in the United States.

Hongzhi has been working for the Washington State Department of Transportation in the Bridge and Structures Office as a senior design engineer and seismic specialist for more than 20 years. Beyond his extensive research work, Hongzhi has vast experience in bridge design and seismic retrofit on a variety of structure types including suspension, cable-stayed, moveable and fixed span concrete/steel bridges. His published research papers on Pipeline and Highway Bridges have been presented at Regional, National and International Conferences. He also co-authored, with Chinese Professor, Dr. Zhong-wei Wu, in 1990, a technical book (in Chinese) titled "Expansion Concrete".

Hongzhi also has many years of teaching experience in graduate/undergraduate facilities both in P. R. China and in the U.S.

SUMMARY

Located in Monroe, Washington, this structure, at a total length of 712 feet, is a four span, curved bridge, designed with a two-steel plate girder system. The main span, crossing over the BNSF Railroad Line, is 230 feet in length with a curve of 300 feet in radius.

The two girder system is allowed on ramp structures per WSDOT's design policy. It is fracture critical but has the least construction cost within bridge life time.

Because of the sharp curve of the bridge, the bracing frames were designed as the primary members with a very close spacing on the curve portion. "Two Way Slab" was applied on the design of the bridge deck to ease the design of the drilled shaft in the high seismic zone.

The differential vertical deflections between the exterior and interior girders and the associated torsional rotation and warping posed many challenges on girder erection and deck pours during construction.

The bridge was opened on December 1st, 2011.

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Introduction

A four span steel plate girder bridge was designed overcrossing the BNSF Railroad in the vicinity of Monroe, Washington (Figure 1). The bridge is 27 feet wide with a total length of 718 feet (163'-230'-188'-137'). The main span crossing over the BNSF Railroad is 230 feet (span length/superstructure depth=27). Two thirds of the bridge in length was curved with a 300 feet in radius.

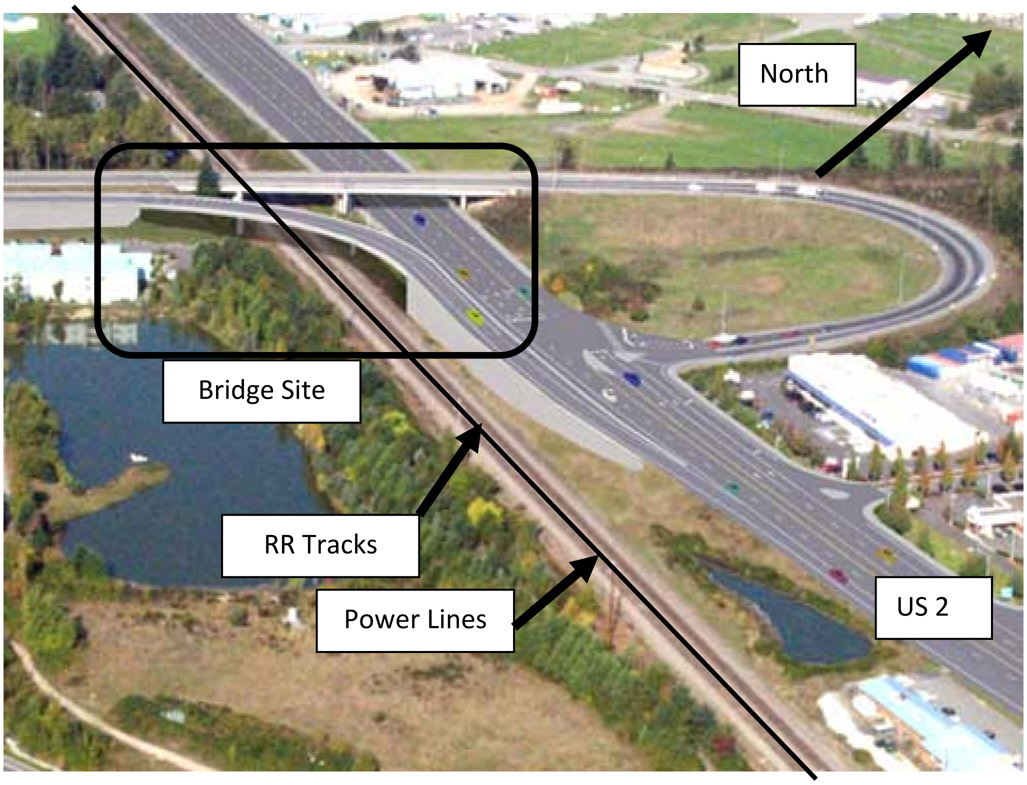
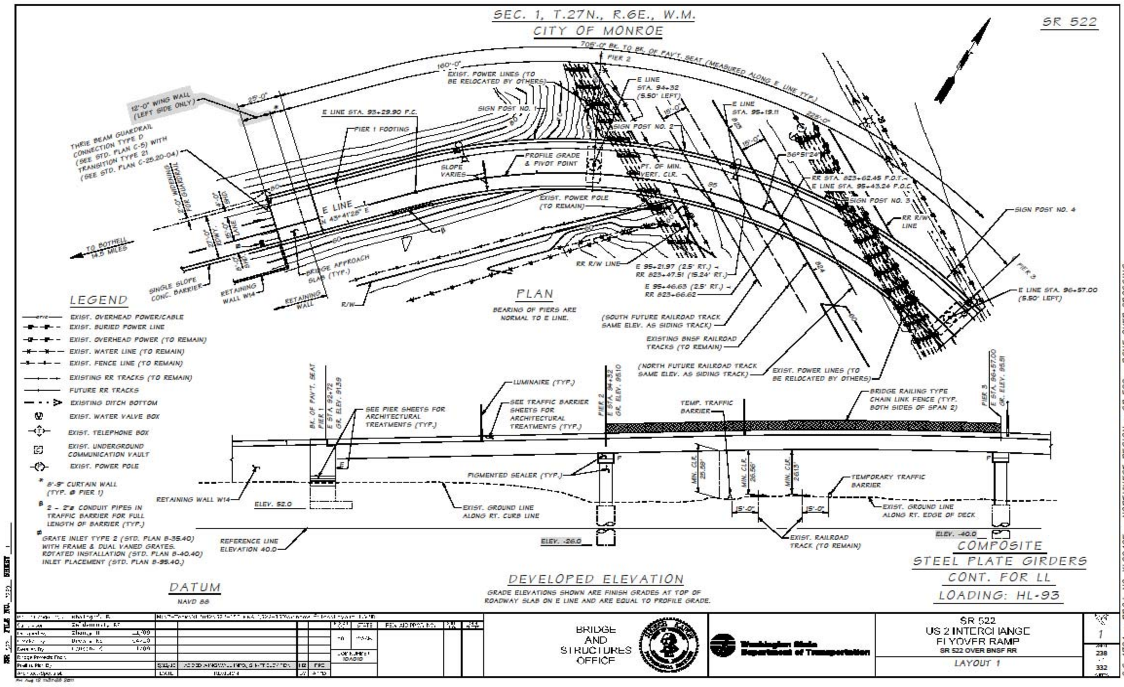


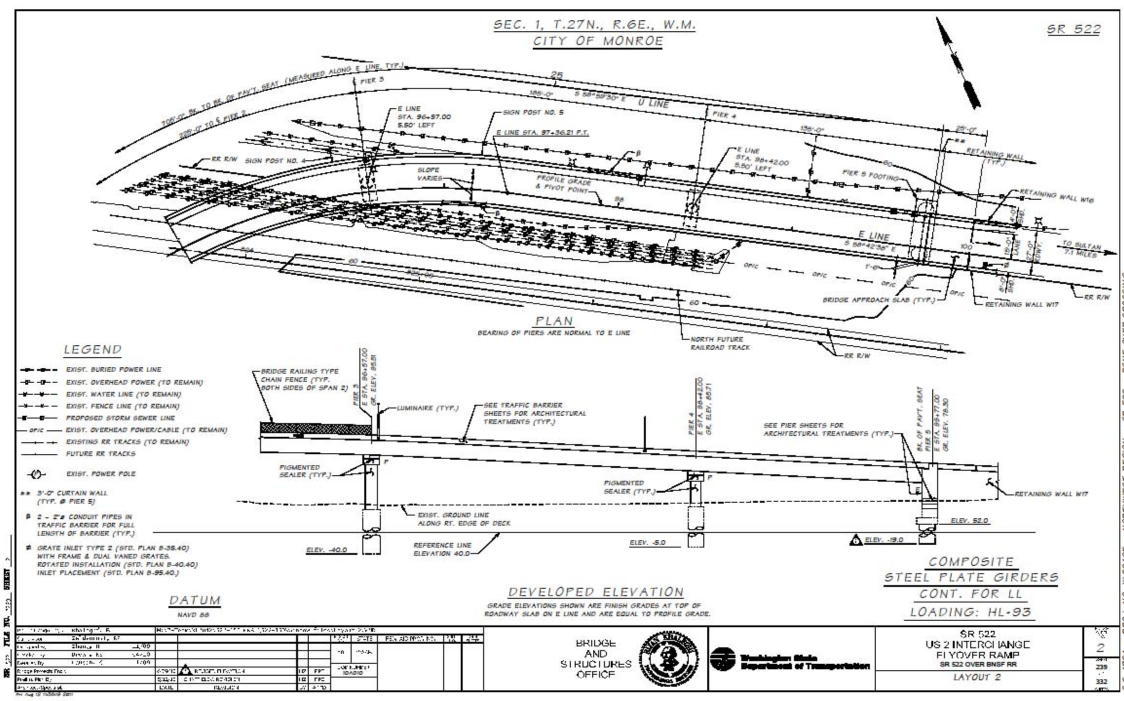
Figure 1 Bridge Vicinity

Because of the sharp curve of the bridge, the lateral instability caused by torsional moment was the primary concern. A steel box girder was the first consideration because it is the most efficient structural type for torsional moment resistance. However, shipping a maximum length of 120 foot curved box girder portion between two field splices points would need to occupy a minimum of 28 feet width on the highways from the manufacturer's shop in Oregon to the bridge site. It may be possible to do so, on the major highway like I-5, but not on the smaller highways or local roads. If increasing more field splices in the design to reduce the shipping problem, the construction efforts, time and overall cost will increase.

A concrete box girder was also ruled out because the required vertical clearance crossing over the BNSF railroad would not provide adequate space for falsework while allowing the railroad track to remain in operation.

A steel plate girder was then studied and selected. Several alternatives of steel plate girder systems are available: (a) a three same size girders system or; (b) a three girders system with two large size exterior girders and a smaller size interior girder and (c) a two girder system. By comparing the three systems, the two girder system would have the least construction cost in the bridge life time.





Figures 2 & 3 Bridge Layout Sheets

Structural Design Considerations

The current WSDOT's practice allows two plate girder's system on a ramp structure[1] and therefore, a two steel plate girder bridge with a 16 feet girder spacing was selected as the most cost effective solution after the comparison (Figures 2 & 3). Since a two plate girder system is classified as fracture critical due to the lack of structural redundancy, certain welding procedures and bridge inspection rules had to be applied.

The bridge is in high seismic zone. The surrounding soil of the drilled shaft foundations has the potentials of liquefaction and lateral spreading. If the conventional design method for the deck (One Way Slab) was used, the bridge deck would require a minimum thickness of 10 ½ inches. In order to reduce the weight of the superstructure, the design utilized a “Two Way” slab to reduce the slab thickness to 8 ½ inches. Shear studs would be installed on the top the members of the cross frames and the deck longitudinal rebar would increase significantly, but the thin deck made the design of the single column/single drilled shaft system possible.

Traditionally, the main concern of reinforced/prestressed concrete superstructures design is the strength, but for steel superstructures the key is overall structural stability. The design of the steel structures with the newly developed High Strength Steel (High Performance Steel) would have smaller cross section area and less thickness of each structural member and therefore, would have more potentials of structural instability. There is also a significant difference between a straight steel structure and a curved one, especially for the subject bridge with a very sharp curved geometry. On a straight steel structure, the design may focus on the stability of each individual girders and the cross frame (bracing) may be designed as the secondary members. But for a curved bridge, the lateral stability on both the individual member level as well as the global system level becomes the primary concern and the lateral bracing will become a part of the primary load path, not secondary. In the design of the SR522 RR Flyover, a cross frame spacing of approximately 15 feet was designed on the curved portion and 25 feet for the straight portion. Reinforced concrete end diaphragms were installed at both abutments to increase the fixity and resist uplift on the outside girder, further stabilizing the two girder system against the torsional moment.

A 3-D Finite Element Analysis with GTSTRUDL was performed and then compared with the results from MDX to insure the correctness and accuracy of the analysis. The analysis showed that the design of all the girders and the bracing frames met the AASHTO code requirements(2).

According to the structural analysis, the interior girder carries higher shear force and the exterior girder carries higher flexural loads under both the dead load of the steel only and the dead load of the composite cross section. The redundancy may have different meanings for sharply curved bridges and even a three girder’s system would not satisfy the requirement of redundancy.

The design used the same thickness webs on both the interior girder and the exterior girder, but the width and thickness of the top and bottom flanges were designed based on the flexural demands. Vertical stiffeners were used to increase the girder’s shear capacity. No horizontal stiffener was designed for this structure because WSDOT office practice discourages its use.

Due to the eccentricity of the load, the maximum vertical deflection at the mid-span of the exterior girder under dead load was almost doubled the deflection of the interior girder (Figures 4 & 5). The rotations that were introduced by the differential vertical deflections between these two girders would make the girder erection more challenging.

AASHTO(1) allowed the maximum live load deflection of a bridge to be L/800 (L is the span length of the bridge). Since the exterior girder would have twice vertical deflection than the interior girder, a 90 inches deep girder web had to be used for this project.

Pinned disc bearings were designed on the intermediate piers to carry the vertical loads and to resist the lateral seismic loads. One-directional movable disc bearings were utilized for the end abutments to allow free longitudinal movement. Reinforced concrete shear blocks were installed on both sides of the concrete end diaphragms at the abutments with special buffers on both surfaces of the shear blocks and the diaphragms to protect pounding damages during a strong earthquake.

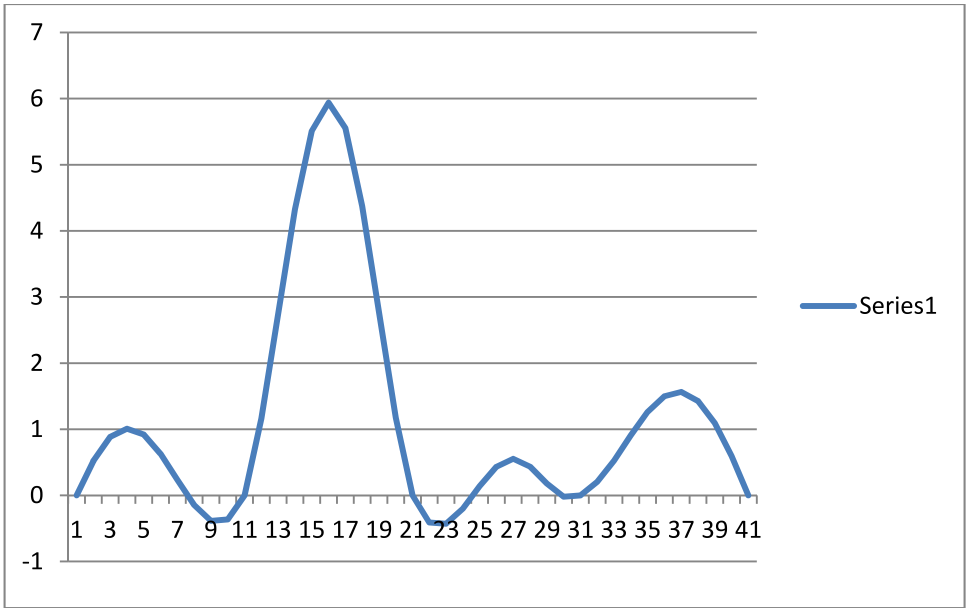


Figure 4 Interior Girder Dead Load Deflections

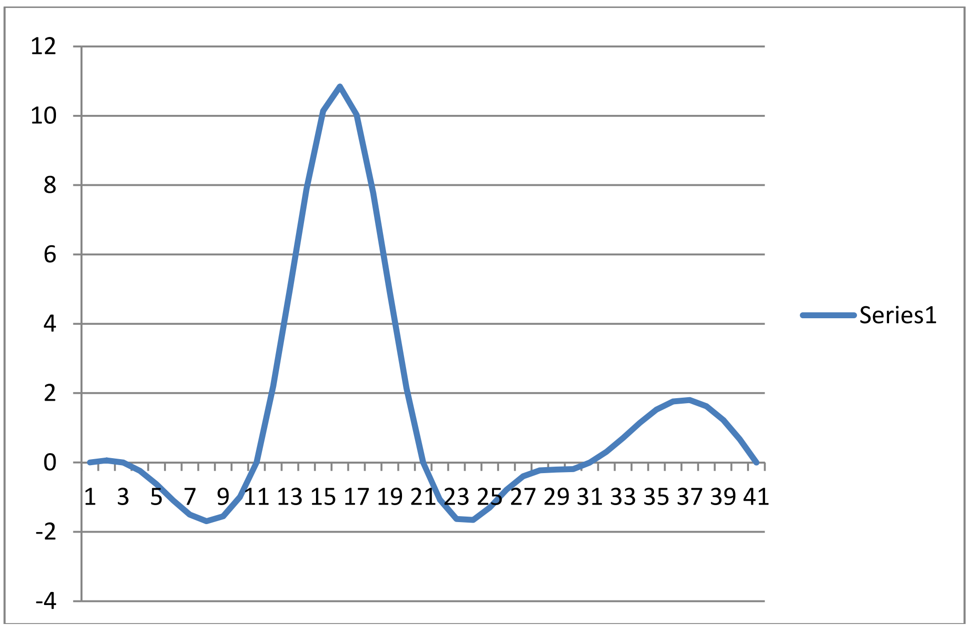


Figure 5 Exterior Girder Dead Load Deflections

Note: The locations of the five piers are at 1, 11, 21, 31, 41 on the horizontal axis in Figures 4&5

Constructability

At the bridge site, an existing bridge was on the left side of the beginning of the new bridge and an apartment complex on the rights. Two very tall timber utility poles were very close to the edge of the bridge deck (Figure 6). The BNSF Railroad Company had restrictions keeping any temporary supports and construction equipments 15 feet away from the two railroad tracks. This made it very difficult to position the cranes for erecting the girders.

Figure 6 Bridge Construction Site

Under the circumstances, a "Suggested Girder Erection Plan" was provided on the design plan. This included the foot print of the temporary shoring towers, the locations to set up cranes and the calculated girder weight between every two field splice points (Figure 7). A special Notice was also included in the "Suggested Girder Erection Plan" warning the contractor to consider the torsional rotation, warping and the differential vertical deflections between the two girders. To ease the girder erections, the design plan suggested that the two girders crossing the railroad be assembled together prior to the installation on the temporary shoring towers. According to the analysis, "One piece at a time" installation method would also work, but it would be very difficult to design the anchor system to keep the segment stable on the temporary shoring towers. The contractor decided to place the girder segments "one piece at a time".

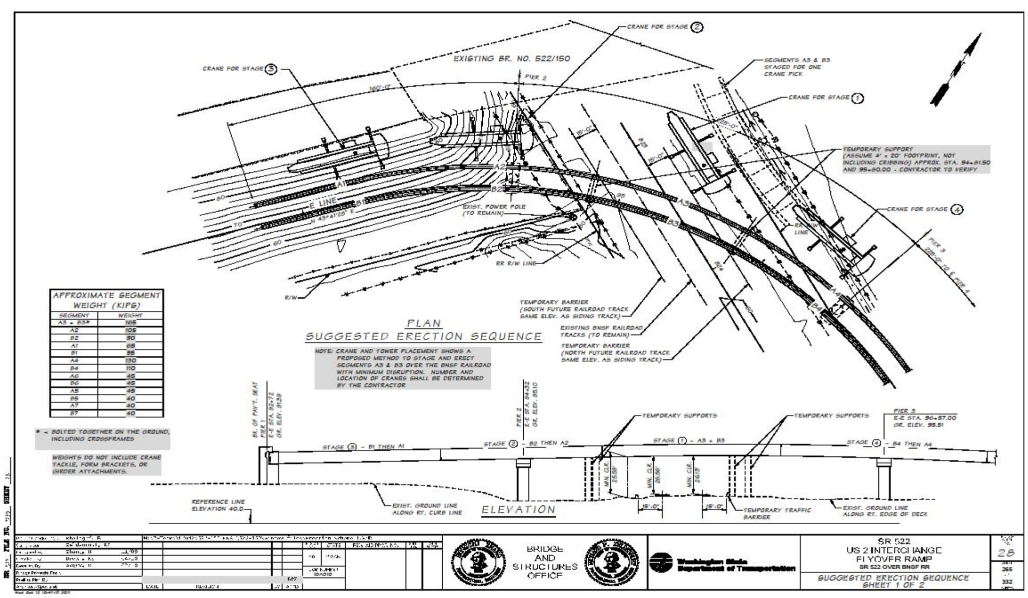


Figure 7 Suggested Girder Erection Plan

The construction of the SR522 RR Flyover also experienced many challenges.

First, there were two 120 KV power lines one on each sides of the BNSF Railroad. They would post a threat to the shaft installations and girder erections. Through negotiations, one owner agreed to temporarily shut down their power line during construction but the other had to be paid for rerouting.

Second, the BNSF Railroad required a construction window and would not allow any temporary supports being installed with 15 feet from their tracks. The designed field splice locations had to be adjusted in order to match the assumed locations of the temporary supports.

Since the contractor decided to pick and place one segment at a time, another significant challenge was how to stabilize just one girder segment at a time on the temporary support during girder erection for such a sharp curved bridge. Recall the design plan suggested to assemble the two adjacent girder segments together on the ground first and then put them on the temporary support tower. It would be much easier to design anchors to stabilizing the assembled two girders on the support. But erecting two girders in one pick would require higher capacity crane. After the contractor's erection plan was reviewed by our office, we recommended the contractor to use two cranes to stabilize one curved girder segment on the temporary support tower or pier. One higher capacity crane would lift the whole piece during the erection, while the other crane would hold the middle point of the segment as long as needed for connecting the adjacent girder segment to the first one with all the cross frames (see Figure 8).

Figure 8 Two Cranes Held the Curve Girder during Erection

Under the dead load of the bridge deck, the curved bridge will not only experience differential vertical deflections between the interior girder and exterior girder, but will also experience the rotation caused by the torsional moment. The rotation will change the super elevation of the bridge deck. Calculations of girder camber under steel load and the composite sections load were calculated and listed in a table in the contract plan. This table is based on the final configuration of the bridge. From the information in that table, the contractor had to anticipate the rotation as they placed the deck concrete.

At the beginning of the deck pour, the contractor did not understand that the deck's formwork could not be set with the final super elevation of the bridge deck because the deck pour would introduce a significant rotation to the bridge deck. Per WSDOR practice, certain calculations were done to represent the bridge deck super elevations before and after casting the deck, but they were usually not included in the Contract Plans. Based on the calculations, the revised formwork before deck pour was quite different from the final super elevation but would match the designed super elevation accurately after the deck pouring (see Figure 9).

Figure 9 Deck Formwork

The two steel plate girder design of the SR522/US2 Interchange Flyover Ramp provided an efficient, cost effective, and safe solution for the very challenge project. Currently, the bridge construction of the project is completed and will be opened to the public on December 1st, 2011.

Acknowledgements:

Many thanks are due to Scarsella Bros., Inc/SB for the completion of this complex construction of the SR522 RR Flyover Project and to Fought, Inc., the fabricator of the curved girders.

Special thank is due to Mr. Nathan Brown, WSDOT Bridge Steel Specialist, for his checking of the design. Supports from Mr. Rich Zeldenrust, our Design Unit Supervisor, are also greatly appreciated.

References:

(1). WSDOT Bridge Design Manual (LRFD), 2011
(2). AASHTO LRFD Bridge Design Specifications. 5th Edition, 2010