DESIGN FOR LAUNCHING AND OWNING A CURVED 2-GIRDER BRIDGE

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BIOGRAPHY

Patrick Gallagher graduated from Washington State University in 1999 having earned a Bachelor of Science Degree in Civil Engineering. Following graduation, Pat worked the next four years at several engineering firms and in various engineering capacities prior to coming to work for Washington State in 2004. Pat is currently employed as a Bridge Engineer in the Bridge and Structures Design Office, for the Washington State Department of Transportation (WSDOT).

Pat's experience in bridge design covers steel and pre-stressed concrete girder bridges, both single and multiple span bridges, bridges in high seismic regions, retro-fit structures including repairs, bridge inspections and is a steel sign bridge specialist.

In addition to his work with the WSDOT, Pat also operates his own engineering company. Pat recently was a guest speaker at the *Highways for Life Conference* held in Orlando, Florida, 2010.

Known for his enthusiasm and his thinking "outside the box" attitude, Pat routinely champions aspects of bridge design that interests him most. These include: Accelerated Bridge Construction, Isolated Abutments, and Fracture Critical Steel Structures.

SUMMARY

Fracture critical (FC) bridges bring with them a history of fear and criticism from those in the design and maintenance community. This paper discusses where two fracture critical bridges were designed for constructability, allowing the contractor to launch the girders. We'll discover significant cost savings and a logical setting for an FC bridge, whether or not the girders are launched. The discussion will also illustrate how the risk associated with FC was managed in design and followed through to maintenance. Furthermore, we'll address the structural design, life cycle cost of the bridges, and how one job site might lend itself better to an FC bridge than another.

As this paper will demonstrate, there is a place for FC bridges in a bridge engineer's array of choices and suggests that bridge designers ought to look more closely at this underutilized method of achieving a span. Perhaps a fracture critical bridge really is the best choice for the right project.

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Introduction

Vancouver, Washington is a fast growing community in Southwest Washington State bordering Portland, Oregon with the Columbia River between the two cities. In the Bridge Engineering world, this area draws a lot of attention with the new Columbia River Crossing being planned along Interstate 5. However, along the north edge of the city of Vancouver sits a little known highway, State Route 500, located between Interstates 5 and 205. Those who do not live in the area would hardly notice this short stretch of road and have little reason to travel along it. But to those who live in Vancouver, it's a significant roadway. While in the shadows of a larger project gaining national attention, there's interesting things happening on SR 500, specifically with two bridges near the new Saint Johns Boulevard intersection currently being rebuilt.

Historically this highway was a rural two lane road. In the 1980s it was widened from two lanes to four. With recent growth to the area, the Washington State Department of Transportation (WSDOT) has constructed or planned a series of four or five projects converting this old rural highway to a freeway style road with free flowing interchanges. The intersection of Saint Johns Boulevard and SR 500 is the next intersection on the list to be improved and it's currently under construction.

Overall, the project consists of three bridges and one culvert improvement, see Figure 1. The on and off ramps east of Saint Johns Boulevard will be the focus of this paper. Above the site of the steel bridges are high voltage power lines that must remain in service during construction. These power lines are very close to the SE Bridge and 57' above the end of the NE Bridge. They require buffers around them to ensure heavy equipment will not ground the wires, endangering the equipment operator. The ramp bridges are parallel to an existing bridge that crosses Burnt Bridge Creek and the Burnt Bridge Creek Pedestrian Trail.

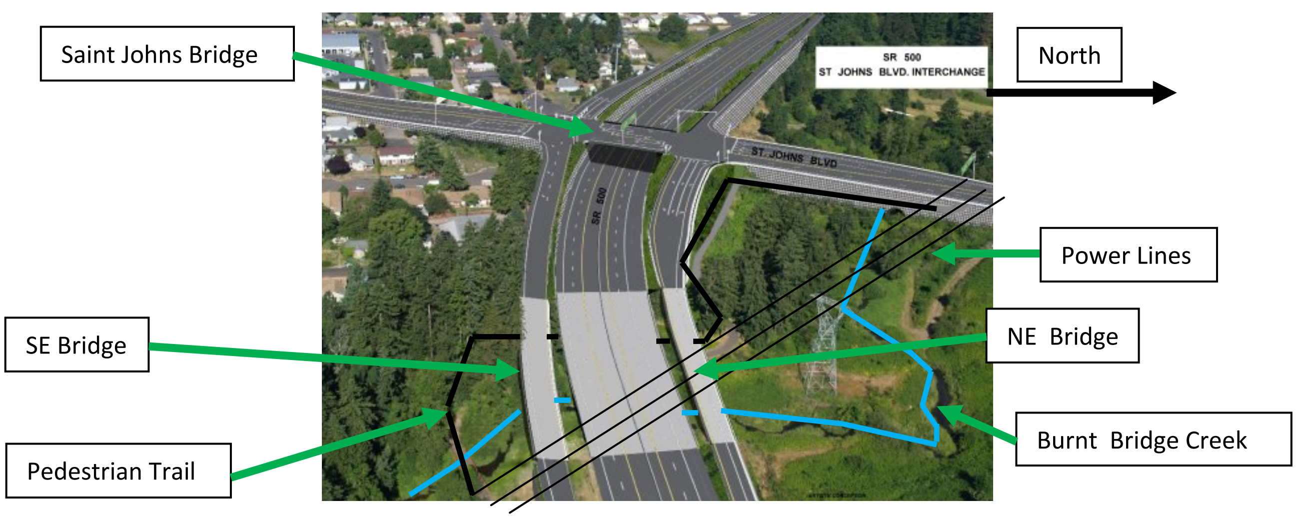


Figure 1 ~ Project Description

To fully understand the details of this project, a few structural details need to be understood. The on ramp, SE Line Bridge, spans 208'. The off ramp, NE Line Bridge, spans 248'. Both bridges consist of two steel plate girders, simply supported, each constructed with three steel segments. The SE Bridge has a constant web depth of 10'; the NE Bridge web is 10'-9" deep. The cross frames are steel K-Frames with moment connections at the ends of all the members, see Figure 4 further ahead in this paper. The deck spans between the two girders, with overhangs on either side of the steel superstructure. These girders rest on disc bearings sitting on L-Type abutments supported on spread footings. Further ahead in this paper, Figure 7 shows a WSDOT L-Type abutment.

Selection Process for a Two Girder Superstructure

With the resistance of the bridge maintenance industry, somewhat recent history of bridge failures, and the cautious nature of a structural engineer, choosing a Fracture Critical (FC) superstructure had to be evaluated for more than just ease of construction. Providing a launching option was what brought the project to consider an FC superstructure, but not the only factor. The designers anticipated a launching sequence that minimized the number of times an operation was performed. A lighter superstructure as a whole and fewer girders were considered easier to launch than heavy ones. They expected the girders to be launched in pairs. So a two girder system was considered in order to minimize weight.

Choosing an FC bridge ultimately came down to managing risks. Realize that AASHTO (1) and the Bridge Welding Code (2) fully support an FC design. AASHTO's load rating code (3) also considers FC for load rating. So it's not too much of a stretch to consider an FC Bridge at any site. History, experience, thorough analysis, and careful thought and attention to detail ought to be considered carefully. But an FC Bridge can be utilized and be completely up to code! Maintenance and inspection is discussed further in this paper.

An FC bridge is not completely void of redundancy. There is some internal redundancy in the connections and cross frames. There are 11 cross frames in the SE Bridge, each connection has numerous bolts, and many plates connecting the larger parts. While not enough to declassify this bridge as FC, it does ensure some mechanism is available to prevent total collapse as observed in other bridges throughout the country that exhibited fracture without total collapse.

The type of steel bridges used on the Saint Johns project was important. It's hard to quantify due credit to recent code changes when considering an FC bridge. When one hears FC, they associate this term with older bridges, designed to older codes, designed for lighter trucks, and different industry standards. Modern steel bridges give special attention to tension regions, welding practices, material handling, etc. that was not as dominant in the days when much of the interstate system was constructed.

Modern WSDOT steel bridges are designed to a Fatigue Category C', improving the performance of the welding details over older designs. The structural details used for the SE and NE Bridges eliminate many of the connection types that cause fatigue cracking in older bridges, such as bottom laterals and their connecting gussets. This eliminates tri-axial stresses on those welds. The stress range for the tension members in the SE and NE Bridges turned out to be below the Constant Amplitude Fatigue Threshold (CAFT) for infinite life as listed in AASHTO Table 6.6.1.2.5-3 (1). In addition, the structural details of these bridges do not allow for distortion-induced fatigue. The structural details WSDOT uses today do not have a history of fatigue problems observed in older WSDOT bridges. Comparing a modern steel bridge to older bridges in the context of FC is not a reasonable comparison.

These girders have a 27' wide deck on top constructed with two layers of epoxy coated reinforcing. The steel girders are not fully exposed to the weather and are not as vulnerable as other FC superstructures of getting corroded by road debris, rain, or snow. In addition, Washington State uses a less corrosive product than salt to remove ice from the roads, providing further protection of the steel.

The project site itself also helped ensure other risks would be minimized. There is no road below the bridge which might introduce a high load hit to the girders. The stream 30' below the SE and NE Bridges bridge has very little flow, even in the event of a flood; ensuring flood debris will not damage the bridge. Since the bridges are ramps, near a modern undercrossing, in a residential area off of the interstate, it's unlikely an overloaded vehicle would travel on them. The bridges have wide shoulders allowing easy inspection. If, in the future, another lane is added to the bridge as noted in AASHTO Article 3.6.1.1.1 (1), traffic could flow on the bridge with one of those lanes closed for the duration of an inspection. The only traveled way beneath the bridges is a pedestrian path, which poses no structural threat to the bridges, and actually improves access to the underside of the bridges for inspection. Since the bridges are simply supported and on non-liquefiable soils, the seismic risk is also low.

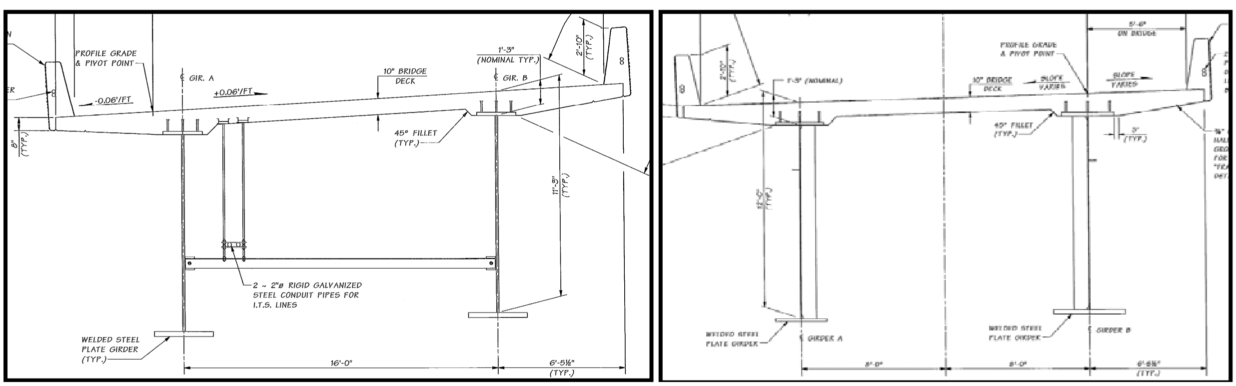
Outside of launching, selecting FC was not much of a concern for construction. Had the superstructure had three or four girders, except for fabricating the steel, the construction technique would have been evaluated by construction inspectors much the same is it was for the FC design selected.

Evaluating Girder Launching

At WSDOT, the bridge engineers do not design for specific construction methods except where absolutely required. A careful WSDOT bridge engineer will consider a wide array of options a contractor might choose and design for the envelope of all of them, or design for a well documented plan suggested in the plans.

For the SE Bridge, the designer accounted for a schematic of shoring towers. The designer demonstrated that shoring towers could be placed at enough locations and still be in compliance with environmental and access restrictions such that launching could be achieved without overstressing the girders.

For the NE Bridge, the designer took the approach used on the SE Bridge one step further. In an effort to make the girders lighter, each girder was designed with a different configuration of flanges, and a longitudinal stiffener to minimize weight, See Figure 2. The NE Bridge designer created a “Suggested Construction Sequence” where the truck that delivered the middle segment would have one of the end pieces bolted to it, overhanging the back of the truck, and it would be used to “launch” that segment under the power lines into place, See Figure 3. The steps following would be a more normal, pick and place method for the rest of the girder segments.



SE Bridge NE Bridge

Figure 2 ~ SE and NE Cross Sections

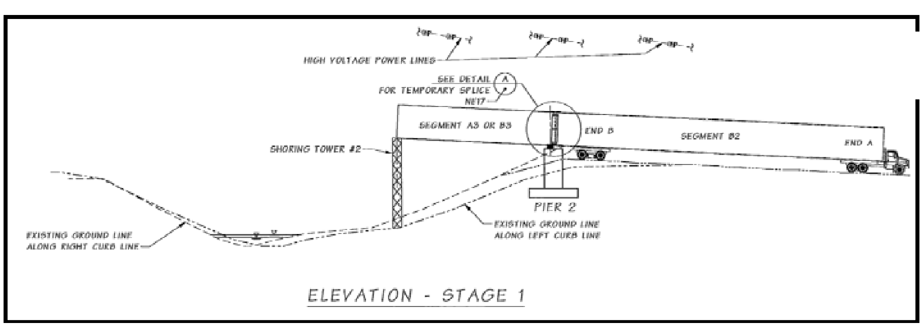


Figure 3 ~ NE Bridge Girder Placement

Both designers added mechanical rebar couplers to the abutments such that the abutment construction could be staged, and they confirmed there was enough room to facilitate the construction activities. The designers were also careful to locate girder field splices in locations that would work best with what they thought was the most likely place to put shoring towers.

Structural Analysis Method and Design Considerations

Precision was the driving force behind the structural analysis. Aspects of the design carefully reviewed specifically for FC concerns were fatigue, bearing and support conditions, and ensuring the structural design and details reflects what will really happen in the field assuming a variety of problems that might occur during the life of the bridge. Construction was restricted in the specifications to prevent oversized holes, tack welding for erection purposes, and a rigorous quality control program was implemented for the steel fabrication.

According to AASHTO Article 4.6.1.2.4a (1), both bridges were required to be analyzed as curved bridges, introducing torsion into the analysis. The superstructure's cross section does not provide a continuous path for torsion stresses to flow within the beams alone. So the cross frames had to be designed to withstand higher moment than the designer initially expected within the cross frames to provide an adequate torsion resisting load path. With the curve, and the simple span configuration of the girders, the torsion in the girders tended the girders to twist off their foundations on the inside of the curve; particularly when a design level truck and lane was applied to the outside of the curve. Uplift at the bearings on the inside of the curve was evaluated. Had this been a multiple span bridge, the adjoining spans would have provided some resistance to the twisting forces.

Since the cross frames are part of the torsional resistance system, which support FC girders, it was decided that the cross frames were also fracture critical, along with the stiffeners and web. The only portion of the bridge not fracture critical was determined to be the top flange of the girders.

AASHTO Article 6.13.1 (1) states that the cross frames need only be designed for the force effects determined in the structural model. This is what is typical at WSDOT. However, the designers considered the cross frames as part of the "primary load carrying system," so they felt it was worthwhile to design the cross frames for 75% of the member's capacity as suggested elsewhere in that AASHTO Article. The cross frames provide torsional stability of the entire system, and if they were to fail, the bridge could collapse or at least show some large deformations. What made the 75% rule interesting is that loads did not drive the cross frame design. The designers had to iterate between member sizes and connection designs to gain 75% of that member's capacity. By following the 75% rule, the cross frames had much more capacity than otherwise needed to support the design load. See Figure 4.

Pier Cross Frame | Intermediate Cross Frame

Figure 4 ~ Cross Frame Designs

WSDOT places a lot of value and emphasis on the quality control of their designs. The engineers designing, checking, and reviewing the plans had recent steel bridge design experience. A total of nine bridge engineers reviewed the designs at varying depths, including complete analysis and re-analysis of various parts of the bridge by six of them. Three engineers focused deeper on construction and overall project quality. All nine engineers focused on these issues at some depth. Typically two or three engineers completely analyze and re-analyze a WSDOT bridge design. Some reviewers were very experienced in steel bridge design, including WSDOT's steel bridge specialist, managers who have designed many steel bridges themselves, and WSDOT's bridge construction office. The SE and NE designers made efforts to observe one another's work and verify the designs were consistent. While not foolproof, the quality assurance of the design was representative of the risk a design error might produce.

Load Rating

Being an FC Bridge, and a bridge on a curve, two aspects of the load rating were rather unusual. First, while rating the bottom flange field splices, the designer found the bolts and plates didn't check out to be 1.0 or greater. For a new design, this should theoretically be 1.0 according to load rating theory. However, the design process increases the loads by 5% for non redundant structures. Unless the engineer elects to use the AASHTO LRFD factors, the load rating process reduces the capacity of non redundant members by 15%. If the bridge is designed perfectly, the minimum rating factor should be 0.81 (1/1.05*0.85 = 0.81). To ensure this new design gets a good load rating, the designer added bolts to this connection to get a rating factor of 1.0 as a minimum for all HL-93 trucks (3). While the designer could have used 1.0 instead of 0.85, at the time of the load rating, there was some dispute as to what to do with redundancy and some concern with FC. Using 0.85 and adding a few bolts to the field splice was simple and effective for ensuring a safe load rating. Second, when a truck travels on the outside edge of the curve, the bearings on the inside of the curve have a smaller vertical reaction. While there is no rating factor one could assign to this, the designer did check this reaction for all of the trucks and concluded that the superstructure will not uplift off of its bearings.

Maintenance

Most of the difficulty in deciding to design a fracture critical bridge comes from calming fears, both in design and in maintenance. As noted above, the structural design well represented the FC concerns. When considering maintenance concerns, the designers of these bridges considered strongly the capacity of WSDOT's Bridge Inspection Program and considered maintenance costs for the life of the structure, assuming the future will be like the past.

WSDOT's Bridge Inspection Program far exceeds the minimum requirements that have been established by the FHWA. The program is executed by licensed Professional and Structural Engineers with bridge design experience. Inspection teams consist of at least one PE, often two. Bridge repairs are designed by an SE and the program is led by an SE, both experienced in bridge design. The inspection program has a number of specialists, some focusing on issues specific to steel.

The bottom line for maintenance is the frequency of inspections. Since these bridges are fracture critical, they will require more FC inspections, which require close inspection from an inspector walking the bottom flange, tied off to a safety rail, or in an Under Bridge Inspection Truck (UBIT). With the residential areas adjacent to the bridge, safety rails were not installed in an effort to prevent tempted residents from walking the flange unprotected from a fall. The traffic use of the bridge allows for easy and undisruptive use of the smallest UBITs WSDOT owns. There is no pedestrian rail or throw fences to maneuver a UBIT bucket over. The utilities under the bridge were located to minimize the disruption the utility supports would have on UBIT access. This bridge should be very easy inspected with a UBIT. Furthermore, the two FC bridges are close together, so scheduling a UBIT inspection for one bridge would work conveniently with the other, overlapping cost and effort.

Costs Associated with Fracture Critical

A cost comparison is tabulated below for an FC bridge and a non-FC bridge with the same span capacity, See Table 1. Also below is some explanation describing the values listed in this table. This table provides a reasonable and conservative cost comparison that demonstrates that FC bridges can be less expensive than non-FC bridges.

Only items significantly differing between a two and three girder design were used for the comparison, and only for the SE Bridge. Costs would be nearly the same for the NE Bridge. As such, the costs should be at least doubled for the sake of the entire Saint Johns Project. Realize that the SE and NE Bridges would have been steel regardless of the FC issue. Despite improved tension design and fabrication of modern steel bridges, the bridge inspection rates were assumed to increase at the same rate as older steel bridges.

Notes Regarding Table 1:

The cost of producing a fracture critical girder is about 10% higher than non-fracture critical according to the manufacturer of the Saint Johns Bridge girders. This additional 10% is due to the extra quality control efforts required to protect and test the material in the shop.

Inspection Items not Considered:

1) An FC Bridge requires fewer bearings, although they are somewhat larger.
2) The initial painting of the FC Bridge costs less than non-FC.

For the estimation of painting costs, realize that the estimating tools used by the designers were used. Actual projects costs are not fully understood at the current state of construction, and being a new bridge, actual maintenance costs will not be realized for decades to come. Based upon past experiences, steel plate girder bridges in Western Washington are expected to be repainted 35 years after initial construction, and each successive 20 years throughout the life of the bridge. The inspection cost addresses the frequency of inspections increasing gradually over the life of the bridge. Table 2 lists how the inspection costs were broken down.

TABLE 1 ~ COST OF THE SE BRIDGE DUE TO AN FC DESIGN:

		NON-FRACTURE CRITICAL			FRACTURE CRITICAL			Difference
ITEM	Unit of Measure	Unit Cost	Quantity	Cost	Unit Cost	Quantity	Cost	
Steel	lbs	$1.50	484843	$727,264.50	$1.65	370707	$611,666.55	$115,597.95
Deck Concrete	cubic yards	$750.00	177	$132,750.00	$750.00	208	$156,000.00	-$23,250.00
Maintenance Inspections	NA	See Table 2	See Table 2	$100,884.00	See Table 2	See Table 2	$180,000.00	-$79,116.00
Repainting Costs	square foot*	$30.00	56727	$1,701,810.00	$30.00	44562	$1,336,860.00	$364,950.00
TOTAL COSTS								$378,181.95

* Square footage based upon three paintings.

TABLE 2 ~ BREAKDOWN OF SE BRIDGE INSPECTION COSTS:

		NON-FRACTURE CRITICAL			FRACTURE CRITICAL		
Inspection Type	Inspection Cost ($ each)	Inspection Quantity (each) First 48 Years:	Next 26 Years:	Inspection Cost ($ / 76 years)	Inspection Quantity (each) First 48 Years:	Next 26 Years:	Inspection Cost ($ / 76 years)
UBIT*:	$3,600.00	12	14	$93,600.00	24	26	$180,000.00
Routine:	$607.00	12	0	$7,284.00	0	0	$0.00
TOTAL COSTS				$100,884.00			$180,000.00

* 1 hour of "Routine" time included.

A careful study of the tables above will reveal that for the items pertaining to construction, the cost of this FC Bridge is less, and also for maintenance. Overall costs of the steel and deck, and also overall costs of inspection and repainting are lower for the FC Bridge than for the non-FC Bridge. This makes the bridge less expensive to construct, and less expensive to maintain.

Selected Construction Method

The bridge contractor and their erection subcontractor decided not to pursue launching. They chose to pick the pieces from a delivery truck and place them on shoring towers, see Figure 5. For the area closest to the power lines, they chose to use a high capacity extension crane with flat picks and a short boom length. They decided not to place the crane on the adjacent bridge because they did not want to pay for the cost of the engineering to determine where to place the crane.

According to the contractor, launching is slow and difficult. This contractor would rather go to great lengths to avoid launching. The reasons they didn't select launching is as follows:

1) Launching the girders would require the retaining walls, abutments, and fill to be staged. Equipment, materials, and personnel would have to be mobilized twice for each bridge.
2) The suggested construction sequence the NE Bridge designer chose would require a very smooth road to precisely place the girder segment. Since they are scheduled to place the girders in November, getting a smooth road in a wet climate on a construction site would be expensive and difficult.
3) Jacks would be required to precisely place the girder segment into its intended position.
4) For overall stability, the contractor felt that 8', not 10', was a good maximum limit for a launchable girder depth.
5) The end segments are under the power lines, and they were shorter and lighter than the segments near midspan. With only two to place, and their relative light weight, picking these pieces with a flat boomed crane was rather easy. Launching was simply too complicated when compared to the expense of a larger crane and flat boom.

Figure 5 ~ Girder Erection

Construction and Fabrication Inspection

Outside of activities within the steel girder manufacturing facility, the construction inspection is pretty routine, despite the FC items. Had the girders been launched, then there would have been more effort required to ensure the girders ended up at their final location. Since that option was not selected, any additional effort is rather theoretical.

One construction aspect that was considered in the design largely due to FC was the rotation of the girder webs as the dead load of the bridge is added. The girder webs are constructed in a no load condition, with the webs plumb. With the curve, the girders rotate as they're released from the shoring towers and then the deck and barriers placed. The rotation is evident in the different cambers in each girder. The designers determined that the webs translate laterally about 1" when the barriers are placed. The designers determined this rotation would not make the steel unstable or overstressed. Careful attention was given to the bracing scheme to ensure that this rotation was minimized.

The girders were fabricated about five miles from the job site, See Figure 6. Shipping was not a concern for this project. In addition to this, WSDOT specifications require steel girders be fit up in the shop in a no load condition, in the configuration the pieces will be assembled in the field. Shipping and fit-up should not be a concern for this project.

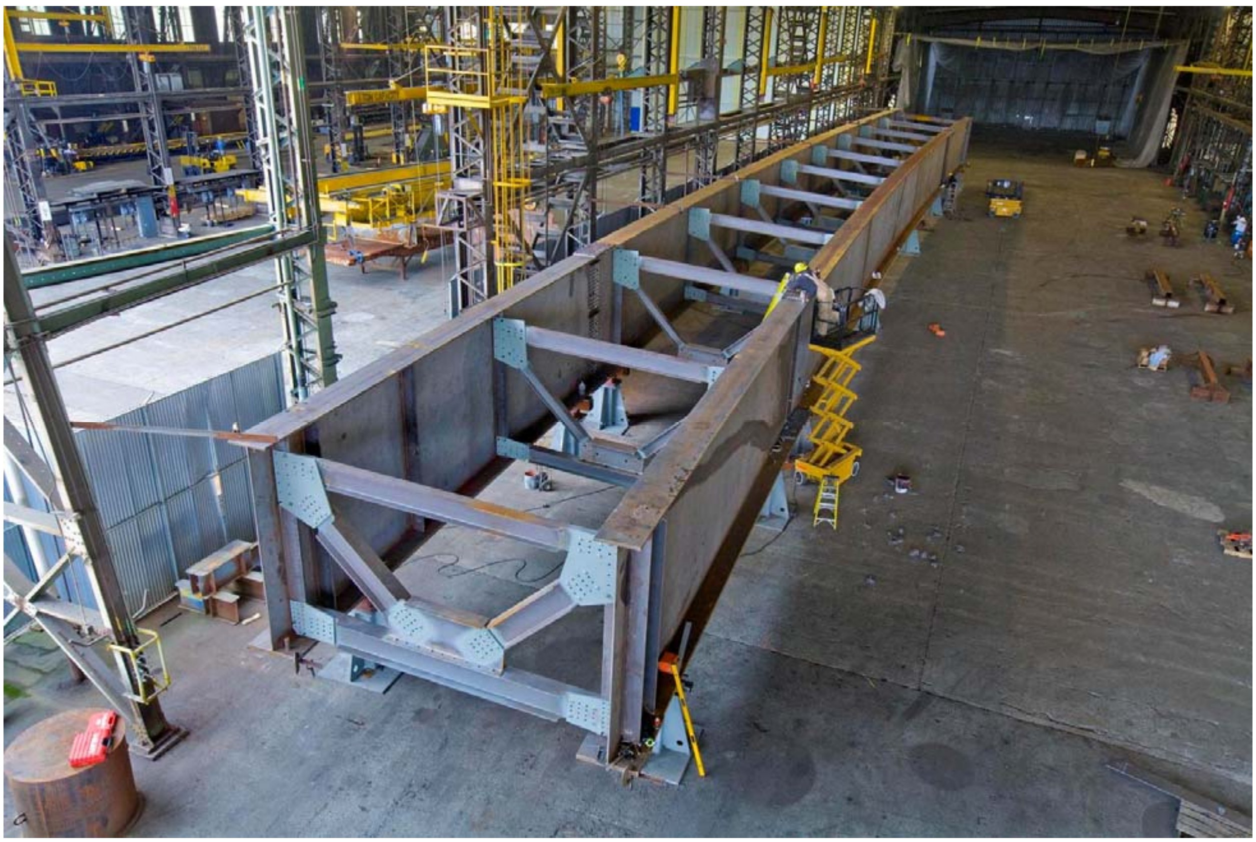


Figure 6 ~ Steel Fabrication

Conclusion

While girder launching was not selected by the contractor for this project, the fruit of the design effort made quite a statement for a modern fracture critical bridge. Launching proved to be too complicated, too extreme, and just not the best choice for this contractor at this site. But the advantages of an FC bridge might not have been realized at Saint Johns had it not been for the design effort allowing girder launching.

The Saint Johns project demonstrates that there is room in the arsenal of bridge spanning techniques for a fracture critical bridge. This engineer believes that FC Bridges are underrated and too often overlooked. FC Bridges are supported by the code and can be designed for infinite fatigue life. While structural redundancy is the easiest method of ensuring a safer structure, proper thought, attention to detail, structural design, and ensuring quality materials can also provide a safe structure while providing the potential cost savings of an FC bridge. Broad brushing all FC Bridges as “bad” is perhaps overkill and provides unneeded expense in some steel bridges.

A designer ought to more strongly consider a fracture critical bridge as an option for the right job site. With easy access under the bridge, wide shoulders on top of the road, or brave inspectors willing to take advantage of safety rails on steel girders, a fracture critical steel bridge can be a good investment. It takes careful analysis for the structural design and the life cycle cost of the bridge, a careful evaluation of the use of the bridge, and it needs to consider the load rating of the bridge, especially at the connections. But, as with any project worth taking on, good engineering will produce an excellent end product.

Figure 7 ~ SE Bridge Project as of February 2012

(1) AASHTO LRFD Bridge Specifications, Fifth Edition, 2010
(2) AASHTO/AWS D1.5M/D1.5:2010 Bridge Welding Code
(3) AASHTO Manuel for Bridge Evaluation, LRFD First Edition, 2008 and interims through 2010