# Innovative Steel Bridge Erection Techniques

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This paper presents the innovative erection techniques that were developed and successfully executed to erect three recent steel bridges in Canada. The three bridges presented are:

- Progressive Cantilever Erection of the 199A St. Viaduct, Golden Ears Project, BC, Canada
- Incremental Launching of the Athabasca River Bridge, Ft. McMurray, AB, Canada
- The Transverse Launching of the Mount Hunter Creek Bridge, Golden, BC, Canada

## PROGRESSIVE CANTILEVER ERECTION OF THE 199A CURVED STEEL BOX GIRDER RAMP

The 199A viaduct is part of the $800 million Golden Ears Bridge project in Langley, British Columbia, Canada. The viaduct is an off ramp to the mainline south approach viaduct. While segmental concrete structures are commonly designed and built, an unusual steel segmental bridge, constructed using the progressive cantilever method, was selected for the steel portion of this viaduct.

The steel portion of the 199A viaduct is a continuous, four-span bridge with 65m-82m-66m-55m long spans which carry two lanes of highway traffic. The horizontal alignment includes spiral portions with a minimum radius of 130m in the main span. The 82m-long main span crosses existing railway lines and the newly constructed south approach viaduct, precluding freedom of crane access in the main span or the possibility of placing temporary piers. This had a big impact on the lifting capabilities of the cranes since they could only be placed outside the railway clearance envelope or on the newly constructed south approach viaduct.

Figure 1. The Erected 199A steel box girder viaduct crossing the South Approach viaduct

## First Design and Redesign

Initially the viaduct was designed as a composite three steel plate girder system with splices at the typical stress reversal locations. Lifting of two adjacent connected plate girder segments as a pair was beyond crane capacities, while the instability of the curved elements ruled out a single girder lift. Moreover, the plate girder solution required heavy cross-frames and bottom lateral bracing due to its tight curvature. The weight of the structural steel for the plate girder option was more than 900t.

After a brainstorming session, the design-build team selected a composite single steel box girder superstructure due to its effectiveness in carrying torsion and the fact that this design made it possible to split the bridge into several segments. To save structural steel the box was haunched at the two supports of the main span. An optimised box girder design had the potential to save up to 20% of structural steel compared to the original plate girder design, and the savings in structural steel would offset the additional cost of redesign and manufacturing.

The design team completed the redesign within an aggressive 12-week schedule; the plate sizes and quantities were delivered in six weeks. To minimise the steel tonnage, structurally-efficient details were designed. The bottom flange structurally interacts with longitudinal and transverse stiffeners. The transverse and longitudinal stiffeners act as a structural part of the bottom flange and significantly reduce the required flange thickness. This principle is widely used for orthotropic decks although in other regions with smaller fatigue demands cost savings are even higher. (1) European and Asian designers and manufacturers take great advantage of this principle and have developed methods to compensate for more detailing and welding requirements. For example, simple plate stiffeners following the curved geometry with a polygonal line and bend only at transverse stiffener locations eliminate the need of a second flange to stabilize stiffeners in transverse direction. This reduces welding and the related distortion significantly.

## Fabrication, Shipping and Erection

The fabrication of the bridge was outsourced by the contractor, Bilfinger Berger, to China. Due to shipping and lifting capacities of the cranes on site, the bridge was split into segments which were limited to 10m length and 35t weight. The fabrication details designed for structural efficiency were more labour intensive compared to the standard practice in North America, however they did not make a significant difference to the fabrication costs in China as they would have in North America due to low labour costs and available know how. A complete trial assembly of the bridge was conducted in China before shipping to Canada. Ten metre long bridge segments were shipped in containers along with the corresponding splice plates.

The main span was constructed with a progressive cantilever method as illustrated in Figure 2, using cranes positioned on the south approach viaduct for the south cantilever and outside the railway envelope for the north cantilever. The approach span segments were spliced on the ground and lifted into place using temporary support piers. The achieved geometrical accuracy was impressive and highlighted the importance of the trial assembly approach before shipping to site.

Even for shorter bridges with relatively simple geometry the trial assembly approach offers great advantages. On-site remedial work, such as reaming of bolt holes, ordering new splice plates, exchange of bracing components, can be very costly and time consuming. If components are forced into position to overcome geometry problems – in particular cross frames at bearing locations – overstressed components, i.e. bearings, may be the consequence. For the 199A viaduct, a segmental erected curved steel box girder, the requirements were pushed even further to avoid erection problems and overstressed components. Prefabricated pieces of each segment (flanges and webs) were welded together using the fully completed neighbour segment as a template to achieve a perfect match of segments similar to the match casting process developed for segmental concrete box girders. (2)

As anticipated, the final tonnage achieved a 20% saving in structural steel. As well as having shipping and handling advantages, the lighter superstructure reduced demands on the substructure and foundations under seismic loads. The bridge was carefully analysed for the erection stages, particularly to address concerns over the stability of the structure during the slab pour, but no changes were necessary.

Figure 2. Progressive cantilever erection of the main-span over the railway lines and South Approach viaduct

A typical segment splice has about 500 bolts. All of them needed to be fully torqued to achieve critical slip requirements as illustrated in Figure 3. Although the local engineering community was cynical about this erection method, doubting a good fit of segments and predicting a time consuming erection process, the erection of the bridge progressed smoothly without any delays. On the contrary, the main span was completed in only two weeks, one week ahead of schedule. The box girder the top lateral bracings is extraordinarily stiff for torsion during erection and the twist during critical cantilevering stages could be ignored – a fact that was confirmed at the design stage by a simplified single spine FEM model and a detailed 3-D analysis. This torsional stiffness of box girders with a top lateral bracing system acting as a 'quasi' top plate allows equal camber values for both webs at cross frame locations and simplifies segment fabrication significant. The predicted camber for bending deflections was accurate and the fabrication and construction remained within the respective tolerances. Jacks at the piers next to the main span piers were used to fine-tune the vertical alignment of the cantilever ends. (3)

Figure 3. Bolting of a typical segment splice

### Concluding Remarks

The tightly curved main span alignment of the 199A Viaduct crosses active railway lines and the South Approach Viaduct. The site constraints in the main-span precluded erection of a plate girder system through conventional method using cranes. A feasible erection methodology was developed using a progressive cantilever method that required the use of a torsionally stiff box girder system. The box girder fabricated in China was split into segments that size of which were determined by shipping and lifting constraints. The reduction in steel due to the structural efficiency of the box system on a curved alignment compared to a plate girder system paid for the additional fabrication costs. Tight fabrication specifications were followed including a full trial assembly to minimise potential misalignment onsite. The main-span was erected and closed one week ahead of schedule.

## TRANSVERSE LAUNCH FOR STEEL GIRDERS AT MOUNT HUNTER CREEK BRIDGE

Mount Hunter Creek Bridge, constructed in 2009, is a new four lane structure built to replace an aging non-composite two lane bridge on the Trans-Canada Highway along the section referred to as the Kicking Horse Canyon Route situated in the Rocky Mountains just east of Golden, British Columbia. The new Mount Hunter Creek Bridge consists of six steel plate girders and a composite cast in place concrete deck. Each girder is 3.5 meters deep and spans 80 meters. Mount Hunter creek runs approximately 25 meters below the bridge soffit. Because of the mountainous site geography and single span design, installation of the plate girders using either conventional crane erection or longitudinal launching was impractical.

After assessing the steel erection possibilities it was decided that the best alternative was to use a staging area to pre-assemble the girders in pairs then use dollies to transport the girders onto the existing bridge. Using the existing bridge to temporarily stage pre-assembled girders for the new bridge, two 300T cranes performed a tandem pick to lift steel plate girders onto the new bridge abutments. A transverse launch was then employed to move the girders to their permanent bearing locations. Each of these critical stages for this engineered steel erection is summarized under the following sections: girder pre-assembly and transportation, tandem lifts, and transverse launch.

### Girder Pre-Assembly and Transportation

Steel plate girders for the new Mount Hunter Creek Bridge were fabricated in Armstrong, British Columbia, and transported 300 kilometres to site using piloted steering dollies pulled by a semi-tractor. Each of the girders was fabricated using three segments and two field splices; consequently, a complete pre-assembled pair of girders was composed of six girder segments.

Splicing the girders on grade presented several benefits including: increased safety for iron-workers; significant schedule advantages as girders were assembled outside of the project critical path; and open access for inspections improving quality assurance. During the day preceding each night closure a tandem pick, as illustrated in Figure 4, was employed to lift a pair of girders and place them on trailer beds supported by six sets of independent dollies and the fifth wheel of a semi-tractor. In this manner the girders were ready to be transported immediately following the road closure.

Road closures over three consecutive nights were permitted allowing transportation of the girders from their staging area into their critical lift position enabling the tandem lifts used to place the girders onto the launch rails at each abutment. The closure on the first night was of longest duration extending for a period of eight hours, followed by closures of six and five hours on the second and third night respectively. These improvements on the second and third night were possible due to the heavy lift crew becoming familiar with the ‘hands-on’ details associated with the engineered lift sequence.

Figure 4. Two 300T cranes lifting a pair of girders onto dollies and a semi-tractor

On the night of the first critical lift, a semi-tractor backed the pair of girders down the center of the highway from the staging area to the site of the new bridge. Despite appearances, the semi-tractor was not used to steer the girders; the leading set of dollies was equipped with a steering mechanism that was used to guide the girders into their critical lift position.

**Tandem Lifts**

With each pair of girders weighing approximately 220 tons, choosing a precise location to position each of the cranes was a critical step during the development of the lift design. A 300T all-terrain hydraulic and a 300T conventional crawler crane was utilized at the east and west abutments respectively. After consideration of several alternatives, a precise position for each of the cranes which provided a picking radius that served the lift demands without exceeding chart capacity was determined.

Temporary works constructed prior to the critical lift included support towers for the outriggers of the 300T all-terrain crane and a work bridge for one of the tracks of the conventional crawler crane. Design and construction of these temporary works was included in the steel erection sub-contract. Pre-assembly of all three pairs of girders and construction of the temporary works was completed outside of the project critical path in the weeks prior to the road closures.

Immediately following closure of the Highway on the night of the first critical lift, both cranes were mobilized and a pair of girders was transported from their staging area and into position for the first lift. In total, these tasks took approximately two and a half hours. Rigging the all-terrain crane was the critical path at this first stage of the night closure.

With both cranes mobilized and the girders in position, the lifting radii at each of the pick and drop points was checked. This was done by moving the main line of the cranes to each of these points so that the operator could read the radii from the crane consul. Each radius was then checked manually using a chain. Two photographs from the first critical lift are presented in Figure 5.

Figure 5. First critical lift for girder installation viewed from the west approach (left) and east approach (right)

The conventional crawler crane picked the girders 6,450 mm from the end of the girders. This pick point was analyzed by the erection engineer and girder reinforcement was not required. The first lift was used to lower the end of the girders directly onto sliders which were positioned in the launch rail at the east abutment while the west end of the girders was lowered onto a temporary bent located between the conventional crawler crane and the west abutment. This temporary bent is visible near the bottom left corner of the right photograph in Figure 5.

For the second lift the pick point for the conventional crane was moved to reduce the lifting radius and enable the capacity required to lower the girders onto the sliders positioned in the launch rails at the west abutment. After the second lift the cranes were de-mobilized and the Highway was re-opened. The lateral launch of the first pair of girders was performed late that afternoon. Closure of the Highway was not required for the lateral launch.

**Transverse Launch**

The system employed for the lateral launch was simple and efficient. H-pile was fastened, with its flanges in the vertical position, along the full width of both abutments. Solid wood blocking filled the void between the underside of the H-pile web and the concrete abutment. To support one pair of girders, four steel sliders were fabricated to fit within the H-pile. To reduce friction during the launch, Teflon was fastened to the underside of the sliders with countersunk screws. Figure 6 shows the preparation, and execution, of the lateral launch for the first pair of girders.

Figure 6. Installation of rigging (left) and lateral launch of girders (right) for the new Mount Hunter Creek Bridge

A block, visible in both of the photos, was temporarily anchored into one end of the abutment in line with the center of the launch rail. To reduce demand on the cranes used for the launch, a second block was fastened to the leading slider. At each abutment, 65T rough terrain cranes were used to power this transverse launch. This system successfully launched all three pairs of girders to their permanent bearing locations.

**Concluding Remarks**

The transverse launch employed for Mount Hunter Creek accelerated the erection of structural steel, improved quality assurance, and increased safety for iron workers and inspectors. By pre-assembling the structural steel, all six girders were installed in a 72 hour period with less than 19 hours of combined closures to the Trans-Canada Highway. The success of this project can be attributed to excellent team work between all parties involved while utilizing an experienced steel erection sub-contractor in combination with an erection engineering consultant familiar with bridge construction practices and methods of incremental launching.

## INCREMENTAL LAUNCH FOR THE NEW ATHABASCA RIVER BRIDGE

To serve the on-going expansions of Oil Sand developments in Northern Alberta a bridge has recently been built across the Athabasca River in Fort McMurray. This new bridge, shown in construction below, runs parallel to the existing two bridges and will double the capacity of Highway 63 at its crossing of the Athabasca River.

Figure 7. Incremental Launching of Steel Plate Girders for the Athabasca River Bridge

The superstructure for the new Athabasca River Bridge consists of ten steel plate girders and a cast-in-place composite concrete deck. To facilitate the extra-ordinary loads that are common in Northern Alberta this new bridge was designed using a CL800 design truck load with special provisions for an oversize vessel weighing ( this information is on CH2M Hills design drawings… I think it was around 10,000 kN). To support these demands, over 6,000 tonnes of structural steel was required.

The steel plate girders were installed using a combination of incremental launching and conventional crane erection. Of the total bridge span, 394 meters was launched and a flared 78 meter long end section was crane erected. During the launch, the girders cantilevered a maximum clear span of 76 meters without the use of temporary supports. An inclined launching nose attached to the leading segment made touch down at the piers.

## Erection Design

Erection engineering included a detailed launching manual and the design of all erection equipment including the launching nose, girder supports and guides, a pushing assembly, and the launching pad. In addition, all components of the permanent structure in particular girders, piers and abutments needed to be reviewed for construction demands. Figure 8 presents a schematic view illustrating the situation in plan and elevation.

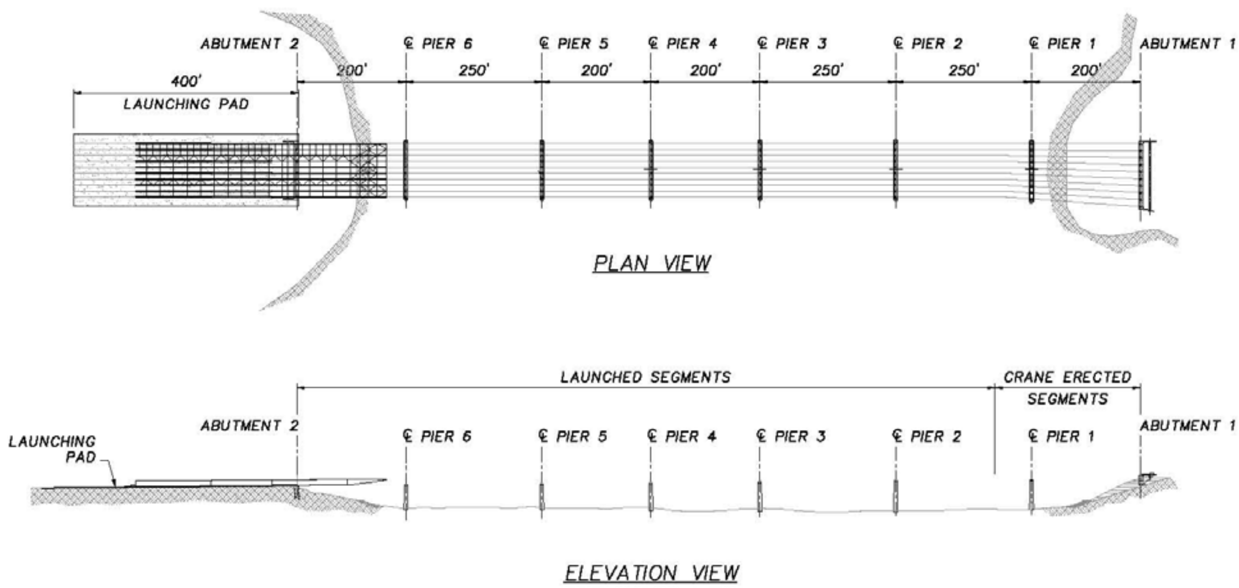


Figure 8. Schematic view of erection concept

Launching Nose - A launching nose with an inclined bottom flange was attached to each of the leading girder segments. Cross and plan braces tied each launching nose segment together creating a single structural system. As the girders approached each pier the bottom flanges of the launching nose touched down on Hilman rollers then acted as a guide to correct vertical alignment of the girders.

One of the challenging aspects of this launch was proposing a simple and efficient way for the Hilman rollers to transition from the inclined bottom flange of the launching nose to the horizontal underside of the girders. To enable this transition a rotating flange plate slightly longer than a Hilman roller was attached to each launching nose with a steel pin. This plate, referred to as a rocker, was able to rotate so that it could match the inclination of the launching nose flange as well as the orientation of plate girder flange.

The mechanism is described as follows: As the rocker assembly approached the Hilman roller “the path of minimum resistance” guided the launching nose up along the inclined Hilman roller as shown in practice in Figure 9 (left) and in theory in Figure 10 (Step 1). As soon as the distance between the two pivot points of the rocker (top) and the Hilman roller (bottom) was minimized, Step 2 of Figure 10, a rotation was triggered just by pushing the girders. During the rotation process (Step 2 to Step 3 of Figure 10) the Hilman roller did not move relative to rocker due to “the principle of minimum deformation energy”. In other words, the rotation was controlled solely by the pushing force since the launching nose was slightly lifted up during the rotation process. In Step 4 of Figure 10 the Hilman roller moved again and cleared the rocker. Site observations confirmed the stable and smooth rotation and verified this simple energy principle.

Figure 9. Launch nose (left) and girders (right) supported by a Hilman Roller during the launch

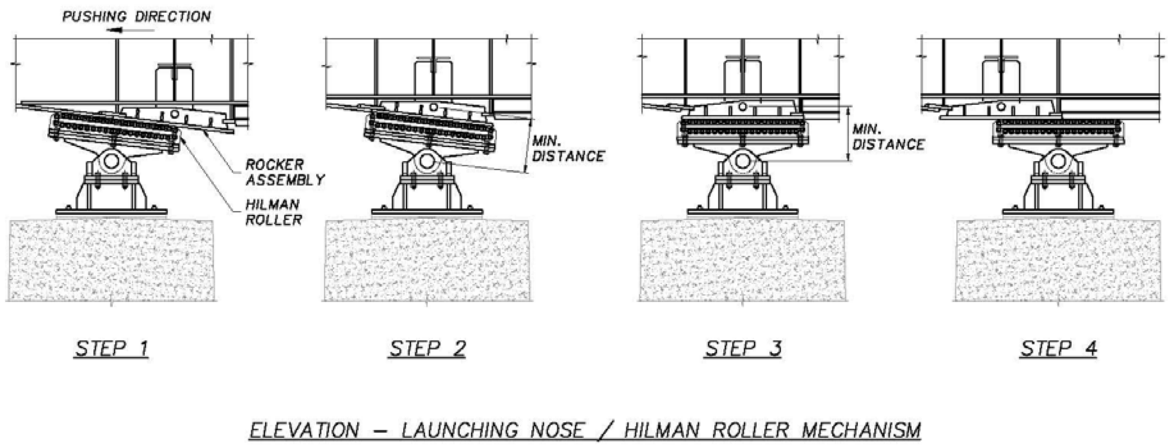


Figure 10. Rocker assembly working mechanism

Girder Support - All ten girders were supported during the launch using 150 ton and 250 ton Hilman rollers as in Figure X. Standard 150-XNTL Hilman rollers were positioned at abutment 2 and at pier 4 while 250-XOTL-08332 Hilman rollers served at piers 2, 3, 5 and 6. Lateral guides were positioned at the abutment 2 and piers 2 to 6. Several of these lateral guides housed jacks that were used to maintain transverse alignment of the girders.

Pushing Assembly - At the rear of each girder a steel sled beam was used for vertical support and to facilitate longitudinal movement associated with the incremental launch. Each girder sled had 25 mm thick Teflon runners fastened with counter sunk screws to the underside of its bottom flange. The sled beams were supported and guided by a rail system using conventional H-piles.

Strand jacks, fastened to the rear of the sled beams, provided the pushing force required for the launch. The dead ends of the strands were anchored into the abutment while the live ends were kept neatly protected in the troughs provided by the H-piles during all stages of the launch. Each girder sled was tied together with a transverse push beam. This push beam transferred the longitudinal pushing force from the sled beams to the girders.

Figure 11. Sled beams, push beam, and strand jacks (left) and sea container housing launch equipment (right)

Launch Pad - The launch pad consisted of small spread footings supporting the continuous H-pile rails. All H-piles were orientated in line with each girder axis and positioned with their flanges vertical. Solid wood blocking beneath the web of the H-pile, in combination with the intermediate spread footings distributed gravity loads to the sub-grade. The launch pad was approximately 120 meters long. Dirt access roads along either side of the launch pad provided access for cranes and service trucks.

Permanent Structure - A detailed finite element model, developed by the erection engineer, was used to assess girder, pier and abutment demands and deflections at several key stages throughout the launch. The results from this model, in combination with a detailed structural analysis, confirmed that steel erection did not require reinforcing of the girders, piers, and abutment.

Erection Sequence - Sequencing the initial launching stages is crucial for every launch design. The erection engineer has to verify that sufficient counterweight in the launching pad is provided to avoid an overturning before the first pier is reached. Typically a safety factor of 1.5 against overturning is considered. For the incremental launch of the Athabasca River Bridge, this safety margin was maintained until the touch down point was approximately 1.2 meters short of the first pier. At this point the launching nose did overshoot the pier by more than 9 meters and overturning stability was not a concern. Assuming the worst scenario, an overturning rotation at this stage would have resulted in a relative soft landing of the launching nose directly on the Hilman rollers of the first pier. If the safety factor criterion of 1.5 would have been strictly enforced the launching pad would have been required to be around 18 meters longer to accommodate another girder segment. This would have added significantly to the cost of the temporary launch pad and its equipment.

To facilitate the splice between the launched and crane erected girders, the launched girders were lowered 64 mm at pier 3 and 125 mm at pier 4 while the girders remained on the Hilman rollers at pier 2. This proved to be a successful method to align the launched girders with the crane erected segments making an anticipated temporary jacking pier unnecessary. [4] [5] [6]

**Concluding Remarks**

The steel erection of the new Athabasca Bridge proved that the old idea of launching the entire bridge superstructure is still a contemporary and cost effective construction method. The incremental launching technique contributes significantly to accelerated and safe bridge construction but requires a good understanding of construction and engineering tasks in general and in detail. Good teamwork between the parties involved is essential for success.