LESSONS LEARNED IN THE LAUNCHING OF STEEL GIRDER BRIDGES

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SUMMARY

Launching of steel girder bridges is a construction method that is implemented when the terrain conditions allow the assembling of the structure behind one of the abutments. This methodology may be safer, faster, and more cost effective than other traditional erection procedures since most of the operations are conducted on firm soil.

This paper presents the details of the launching operations of two bridges recently constructed by incremental launching. The first case study, the Los-Pajaros Bridge, is a three-span continuous I-girder structure with a total length of 639’-6” that was originally designed as three independent simple spans and was converted to continuous spans in a value engineering improvement to facilitate launching. The second case study is the Villorita Bridge, a two-span continuous tub-girder structure with span lengths of 171’-4” and 304’-3”. The paper focuses on three aspects of bridges erected with launching methods: the implementation of the incremental launching method and its benefits, the design calculations and the limit states that apply to girders in cantilever, and the expected versus real structural behavior in terms of girder stresses and cantilever displacements.
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ABSTRACT

Launching of steel girder bridges is a construction method that is implemented when the terrain conditions allow the assembling of the structure behind one of the abutments. This methodology may be safer, faster, and more cost effective than other traditional erection procedures since most of the operations are conducted on firm soil. This paper presents the details of the launching operations of two bridges recently constructed by incremental launching. The first case study, the Los-Pajaros Bridge, is a three-span continuous I-girder structure with a total length of 639’-6” that was originally designed as three independent simple spans and was converted to continuous spans in a value engineering improvement to facilitate launching. The second case study is the Villorita Bridge, a two-span continuous tub-girder structure with span lengths of 171’-4” and 304’-3”. The paper focuses on three aspects of bridges erected with launching methods: the implementation of the incremental launching method and its benefits, the design calculations and the limit states that apply to girders in cantilever, and the expected versus real structural behavior in terms of girder stresses and cantilever displacements.

INTRODUCTION

The incremental launching method (ILM) is a technique in which a bridge structure is assembled behind one of the end supports and then it is moved longitudinally to cross over an obstacle, passing over the intermediate piers until it reaches the far abutment. When proper considerations are taken, the steel erection with ILM can be safer, faster, and more cost-effective than other methods. The assembly of the structure on a firm surface is an advantage that reduces the possibilities of accidents and normally, requires less crane capacities than other erection schemes. Similarly, the construction time is usually reduced since the launching operation is almost continuous, from the moment that the structure starts moving longitudinally, until it reaches the end support.

This paper presents the lessons learned during the construction of two steel girder bridges erected using this method: the Los-Pajaros Bridge and the Villorita bridge, both located in Quito, Ecuador. These bridges were constructed as part of the Simon Bolivar Ave. extension, which improves the connection between the north part of the city and the neighboring towns of Pusuqui and San Antonio. Figure 1 shows an aerial view of the Simon Bolivar Ave. extension and bridge locations.

Case Study 1: Los-Pajaros Bridge

The Los-Pajaros Bridge is a 195.0m (640ft) long structure with three equal spans that accommodates five lanes: four for motorized traffic and one for bicycles. Figure 2 shows an elevation view, the bridge cross-section, and the geometry of a typical girder. To facilitate the construction, the bridge is divided in two structures of four and five girders named Structures 1 and 2, respectively. The composite superstructure consists of I-girders connected with inverted “V” type cross-frames, topped with a 20cm (8in.) thick reinforced concrete deck. The superstructure is made of ASTM A588 Gr.50 steel and is supported by typical reinforced concrete abutments at the ends and two equal intermediate piers with cell-type sections of an approximate 35m (115ft) height. The exterior supports are abutments designed to resist the vertical, transverse, and longitudinal forces that are transferred from the composite superstructure. During the design phase, two options were considered to erect the superstructure. In the first one, provisional erection towers would be placed at approximately the third points of each span. Then, cranes located at the road level and on the sloped terrain would manipulate the girder segments to support them on the provisional towers and the substructure. The last step would be to make the field connections and remove the towers.
This method, however, has two major disadvantages: Firstly, large crane capacities are needed to manipulate heavy girder segments at more than 30m (98ft) of height. Secondly, this method requires the construction of provisional structures that would not be used after the bridge is erected.

The other option was to use the ILM by assembling the entire steel structures behind Abutment “B”, where there is enough space to maneuver and conduct all girder assembly operations. This method proved to be safer and more cost-effective than the previous. Some of the benefits of the ILM are that all operations are conducted on firm soil, there is no need to locate any construction items in the sloped terrain, and it is significantly faster than the other method. For these reasons, the ILM was selected for the construction of this bridge, and specific design features were implemented in the steel structure geometry to facilitate the launching process. LaViolette et al (2007) present a thorough discussion of aspects that need to be considered for successfully implementing ILM in steel girder bridges. These recommendations were considered in the design of the Los-Pajaros Bridge.

**Case Study 2: Villorita Bridge**

The Villorita Bridge is a steel tub-girder structure with two continuous spans of 52.25m (171.4ft) and 92.75m (304.2m) of length. Similarly, to Case Study 1, this bridge has a total width of 23.60m (77.4ft) with four traffic lanes and one lane for bicycles. Figure 3 shows the elevation view and a typical cross-section of the bridge.

The structural system selected for the construction of this bridge is different to the steel I-girder configuration used in Los-Pajaros Bridge. The main reason for this change is the torsional stiffness of the tub-girders, which increases the lateral-torsional buckling strength of the system during launching, as it is discussed in further detail in later sections.

**IMPLEMENTATION OF ILM IN THE CONSTRUCTION OF THE CASE STUDIES**

As discussed in this section, ILM was the most effective construction method for both of the case studies; however, the launching method will likely never become the most economical procedure for constructing a particular bridge. Launching a bridge requires a considerable amount of analysis of the unusual loading and support conditions, design expertise to accommodate these conditions and specialized construction equipment including rollers, jacks and launching nose. However, launching may often be the most reasonable way...
to construct a bridge over an inaccessible or environmentally protected obstacle.

Figure 2. General dimensions and superstructure details of the Los-Pajaros Bridge
When used for the appropriate project, the ILM offers several significant advantages to both the owner and the contractor, including the following:

- Minimal disturbance to surroundings including environmentally sensitive areas.
- Smaller, but more concentrated area required for superstructure assembly.
- Smaller cranes required due to reduced boom reach to erect steel.
- Increased worker safety since all erection work is performed much closer to the ground.

The ILM can be used to construct a bridge over a wide range of challenging sites which feature limited or restricted access, including those with the following characteristics:

- Deep valleys
- Deep water crossings
- Steep slopes or poor soil conditions making equipment access difficult
- Environmentally protected species or cultural resources beneath the bridge

In the case of the Los-Pajaros Bridge, the use of the ILM was an obvious decision. The area behind Abutment “B” was free of obstacles, and there was no infrastructure that may interfere with launching operations. As depicted in Figure 4, the entire 195m-long structure was assembled behind the abutment at an elevation that was only 400mm (15.5in.) above the final position that is when the girders are supported on the bearing pads.

In this case, the structure was launched, and displaced vertically until the girders rested in the
bearings with minor effort since the structure was lowered with jacks of a short stroke.

![Image of Structure Assembly](image-url)

**Figure 4. Structure assembly of the Los-Pajaros Bridge**

In the Villorita Bridge, the surrounding conditions complicated the launching process of the structure. As shown in Figure 5a, the structure had to be assembled at an elevation approximately 4m (13ft) above the bearing level. The structure could not be assembled at a lower elevation due to the presence of a sewer system and ducts for electricity under the structure. For that reason, temporary steel frames had to be erected at four locations to conduct the launching process (Figure 5b). In addition, the girders were launched in pairs, given the site constraints. As shown in the figure, at Support B’, there was not enough space to construct the foundation for the four girders; therefore, the first two girders were launched, lowered and mounted on skids, moved transversely, and lowered again to their final position on the bearings.

Lowering the first pair of girders supposed a major operation during the construction process. Once the longitudinal movement of the girders was finished, the structure had to be jacked up to remove the temporary frames and lowered 4m (13ft) until it rested in the sliding skids. For this operation, the entire structure was jacked at three points, Abutment “A”, Pier “B”, and Pier “C”. Strict control of the forces in the strand jacks was taken as the girders moved down. Also, anemometers were installed to measure wind speeds. According to the structural analyses, operations had to be halted at wind speeds above 5m/s, that is the speed at which the pendulum effect exerts significant lateral forces in the support structures.

Due to the different maneuvers involved in the construction of the Villorita Bridge (i.e., launching, lowering to skids, sliding, and lowering to bearings), in an early design stage, it was decided to construct this structure with steel tub-girders. The torsional stiffness of the quasi-closed sections increased the overall rigidity of the girder system, which helped control the geometry of the structure during the entire process.

In general, the ILM was used for the construction of these bridges because of its advantages over other traditional methods. By comparing the construction of the Los-Pajaros Bridge and the Villorita Bridge, it is clear that the second was considerably more difficult than the first, due to the site constraints, being the lowering operation, the most challenging step in the construction sequence. Whenever it is possible, as in the case of the Los-Pajaros Bridge, it is recommended to assemble the bridge structure at an elevation that is close to the final bottom flange elevation, so the equipment needed to bring down the girders to the bearing pads is limited to short-stroke jacks. This practice reduces the time, the effort, and the risk level implicit in the structure’s lowering since this operation may take as many resources as the launching itself.

**AASHTO DESIGN CHECKS DURING CONSTRUCTION**

The AASHTO Bridge Design Specifications (AASHTO 2017) requires the designer to conduct five different strength checks during construction for steel I-girders subject to flexure and shear. These are:

- **Compression flange yielding**
  
  \[ f_{hu} + f_r < \phi R_f F_{cy} \]

- **Compression flange stability**
  
  \[ f_{hu} + f_r / \phi < \phi R_f F_{cw} \]
- Web bend-buckling  
  \[ f_{bu} \leq \phi_f F_{crw} \]

- Tension flange yielding  
  \[ f_{tu} + f_t < \phi_y R_y F_{yt} \]

- Web shear strength  
  \[ V_u \leq \phi_v V_{cr} \]

Also, to limit the levels of flange lateral bending, the specifications require the \( f_t \) stresses not to exceed \( 0.6 F_{yt} \).

In the context of the launching process, as the cantilever length increases, the girder panel at the cantilever support is subject to a combination of all the limit states described above, i.e., bending, shear, and stresses due to point loads. As a result, this panel experiences the largest stresses in the structure. AASHTO (2017) does not require checking of the interaction between bending, shear, and point load (known as M-V-P interaction) at the regions near to the support; instead, strength checks are conducted individually, by computing and comparing the required strength versus the design strength of each of the seven limit states described previously. For the specific case of bridges erected with ILM, in addition to the required AASHTO checks, the authors recommend conducting further specialized analyses to verify the integrity of the girders at the cantilever supports.

For the Los-Pajaros Bridge, the structural behavior during launching was studied using finite element analysis (FEA). The characteristics of the FEA are described in Ponton et al (2016). The launching process is simulated by increasing the cantilever 5m per step. In the finite element model (FEM), the bridge supports are moved accordingly, so the structural responses are captured at each increment. Figure 6 depicts the deformed shape of the structure when the cantilever is equal to 55m, and it is subject to its self-weight. In the figure, the stress contour shows that the largest Von Mises stresses are in the region near the cantilever support.

For the calculation of the strength checks described above, AASHTO (2017) requires consideration of two types of loading: the structure’s self-weight, DC, and the wind load acting on the superstructure, WS. The design wind velocity used for the calculation of WS is set to 5m/s, assuming that launching will not take place at wind velocities higher than this value. These two types of loads are used to calculate the required strength for two load combinations: Strength IV, 1.5DC, and the specific combination applied to construction, 1.25DC+1.25WS. These are the load combinations that need to be considered according to AASHTO (2017) for construction of steel girder bridges.
Figure 7 shows the plots corresponding to the strength checks for four of the seven limit states mentioned previously. The plots show the design strength calculations and the required strength for the construction load combination and Strength IV for girder G1 (i.e., an exterior girder). The design strength calculations are based on the equations provided in the AASHTO Specifications. The stress values $f_{bw}$ and $f_{f}$ used to compute the required strength in the load combinations are obtained from elastic geometric nonlinear analyses conducted with the characteristics discussed previously.

In the plots, each pair of points represents the required and design strength for a given cantilever length at the cantilever support. For example, in Figure 7a, the design strength of the girder at the cantilever support, for the compression flange stability limit state, when the cantilever is equal to 55m (180ft), is 324.68MPa (47.1ksi). Similarly, the required strength of girder G1 for the same cantilever length, at the cantilever support, when considering the construction load combination is equal to 134.42MPa (19.5ksi).

These plots show that for the Los-Pajaros Bridge, the AASHTO strength checks are fulfilled at the girder panels near the cantilever support. In all the checks included in the plots, the girder section with the largest strength demands is at this location. As the cantilever increases from 35m (114.8ft) to 65m (213.2ft), the different limit state checks are satisfied, showing that the structure complies with the AASHTO (2017) requirements for construction. The results of this study show that in terms of flexural and shear strength, this structure may be erected using ILM.
The strength calculations show that the demands are relatively low versus the structure’s capacity. However, the AASHTO design checks should be complemented with a more detailed study of the structure, where the M-V-P interaction is captured to verify its integrity during launching. Analyzing the structure with a 3D FEA, as the one shown for the Los-Pajaros bridge, is essential to properly capture the bridge behavior in the construction process.

PREDICTED VERSUS REAL BEHAVIOR DURING LAUNCHING

Two structural responses that are commonly monitored during the launching process are the vertical deflections and the major-axis bending stresses in the girder flanges. The first response is measured with standard surveying equipment, while the stresses are measured with strain gauges. Since the deformations in the girders are elastic, the measured strains are multiplied by the steel elastic modulus to transform them into stresses. Sanchez et al (2018) presents the details of the instrumentation used in the launching of the Los-Pajaros Bridge to measure strain and stresses.

Figure 8 presents the plots with the response predictions using FEA and the measurement taken during the launching process. The launching length, which ranges between 0m (0ft) and 195m (639.6ft), is shown in the horizontal axis. The vertical axis in Figure 8a shows the vertical displacements at the girder tip. As depicted in this figure, during the first stages of launching, the measured girder deflections are larger than those predicted by the FEA. For example, when the launching length is equal to 40m (131.2ft), the measured and predicted deflections are 577mm (22.7in.) and 434mm (17.1in.), respectively. The differences between the expected and real responses are due to the continuity effects given by the launching skids, located behind the abutment. The jacks in the skids can be moved up or down to control and distribute the forces in these supports, as the structure is moved forward. When the pistons in the skids that are closer to the cantilever support are moved up, the launching nose and the girders tend to deflect more. In general, steel girders are flexible, and changes in support conditions can affect the vertical displacement at the girder tip. As launching progresses, and the continuity effects due to the skids are less influential, the expected and measured responses converge. Figure 8a shows that between x=130m (426.4ft) and x=170m (557.6ft) both responses are essentially the same. In this part of the launching, the structure moves from Pier 1 towards Abutment “A” (see Figure 2) that is the final part of the launching process. The reason why both responses are similar at this stage is because the analytical prediction properly represents the support conditions in the real structure.
At the launching length of 170m (557.6ft), for example, the structure is supported at Abutment “B”, Pier 2, Pier 1, and at a few skid pair. At this point, however, the effect of the skids in the cantilever displacement is negligible.

For a bridge erected with the ILM, predictions of the girder cantilever deflections may be off significantly due to the structure’s flexibility and the effect of support conditions. To compensate for the girder deflections and to reduce the cantilever length, a steel truss is usually connected at the free end, as shown in Figure 9. This truss is known as the “launching nose,” and it has a tapered shape.

Figure 9. Launching Nose
At the connection with the girders, the nose and girder heights are the same. At the free end, the truss height is shorter, so it accommodates the cantilever deflections and facilitates the landing on the rollers. The theoretical height difference between both ends would be equal to the girder deflection at the maximum cantilever length (i.e., the longest span minus the nose length). However, as previously mentioned, due to continuity effects, the girders tend to deflect more than the analytical predictions. Hence, it is recommended to design the taper of the launching nose considering at least a 30% deflection increase, so there is certainty that the nose will land on the rollers without difficulties.

The other response of interest that is measured during the construction process is the flange stresses due to major-axis bending. Figure 8b and 8c show the stress measurements and predictions at the top and bottom flanges, respectively, at a girder cross-section located at 40m (131.2ft) from the cantilever free end. As shown in the figure, the analytical model is an accurate representation of the real behavior. This result is expected since the cantilever is an isostatic structure, so the support reactions may be calculated with accuracy even with hand calculations. Contrary to the girder deflections, the stress responses are not sensitive to the support conditions.

These figures show that during the entire launching process, the girders are subject to relatively low stress levels. Both tension and compression stresses are bound between 200MPa (28.9ksi) and 170MPa (24.6ksi), which represent less than 60% of the yielding point of the Gr.50 steel used in the construction of this bridge. This observation is in agreement with the calculations of the AASHTO strength checks that show that during the entire launching process, the required strengths are well below the design strengths for the different limit states.

CONCLUSIONS

This paper discusses the implementation of the ILM for the construction of the Los-Pajaros Bridge and the Villorita Bridge. The launching operations were successful, and there were not significant complications during this process. To implement the ILM, it is necessary to conduct specialized studies that confirm that the structure is stable when the girders are in cantilever. Strength checks according to the AASHTO Specifications must be considered to verify that as a result of the high demands experienced during launching, the bridge will not be subject to stress levels that may compromise its performance once it is in operation. Additionally, it is suggested to conduct further studies to verify the strength of the girder at the cantilever support due to the M-V-P interaction since this check is not explicitly required by the AASHTO Specifications.

These bridges are the first to be erected in Ecuador using this methodology. The coordination between all the parts involved in the project was a key factor to successfully complete the erection of the structures. The ILM is not necessarily the most economical procedure to construct a bridge; however, given that almost all operations are conducted on firm soil, it is a methodology that reduces the risk of accidents, facilitates the inspection of the structural components, and considerably reduces the construction times.

REFERENCES


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