BRIDGE CONSTRUCTION PRACTICES USING INCREMENTAL LAUNCHING

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SUMMARY

Bridge construction over deep valleys, water crossings with steep slopes, or environmentally protected regions can offer many challenges. The incremental launching method (ILM) for bridge construction may offer advantages over conventional construction, including creating minimal disturbance to surroundings, providing a more concentrated work area for superstructure assembly, and possibly increased worker safety given the improved erection environment. The ILM involves assembly of the bridge superstructure on one side of an obstacle to be crossed, and then movement (or launching) of the superstructure longitudinally into its final position. Despite potential advantages for certain situations, the use of the ILM for bridge construction has been very limited in the United States. The objective of the work summarized in this report was to provide bridge owners, designers, and contractors with information about the ILM, including applications, limitations and benefits.

To clarify the ILM procedure and the current state of practice, a comprehensive literature search and survey were conducted. Recommendations pertaining to best practices for planning, design, and construction activities, as well as applications and limitations for the ILM are also provided. Case studies are presented, which provide specific ILM bridge project information. The use of the ILM for bridge construction will never be the most efficient way to construct every single bridge. However, it is thought that a wider understanding of the applicability and potential benefits would allow potential owners, designers, and contractors to make well-informed decisions as to its use for their upcoming projects.

INTRODUCTION

Bridges have been constructed using the incremental launching method (ILM) for many years. In this method of construction, the bridge superstructure is assembled on one side of the obstacle to be crossed and then pushed longitudinally (or “launched”) into its final position. The launching is typically performed in a series of increments so that additional sections can be added to the rear of the superstructure unit prior to subsequent launches. The launching method has also been applied to tied-arch or truss spans, although these are fully assembled prior to launching.

The incremental launching method will never become the most economical procedure for constructing all bridges. The ILM requires a considerable amount of analysis and design expertise and specialized construction equipment. However, the ILM may often be the most reasonable way to construct a bridge over an inaccessible or environmentally protected obstacle.

When used for the appropriate project, the ILM offers a number of 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
- Increased worker safety since all erection work is performed at a lower elevation

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

It is estimated that over 1,000 bridges worldwide have been constructed using the incremental launching method. Swanson (1979) states that the first incrementally launched highway bridge in the United States was constructed near Covington, Indiana in 1977. One of the earliest published reports in North America, however, describes the construction of a railroad truss span for the Canadian Pacific Railway in 1907. Despite the advantages listed, the incremental launching method of construction has seen very limited application in the United States. The reason for this disparity is unclear and it is one of the goals of the proposed work to ascertain the reasons for and attempt to eliminate this potential “knowledge gap” for bridge owners, designers and contractors. Specifically, the project objective is to provide bridge owners, designers, and contractors with information and understanding about the ILM, including applications and benefits.

REVIEW OF CURRENT PRACTICE

In order to better understand the current state of practice within the United States and the world, the research team conducted a comprehensive literature search

In addition, a survey was conducted of all members of the AASHTO Subcommittee on Bridges and Structures (which included all state bridge engineers) to attempt to understand how much they understand about the incremental launching method and where the current study may be most useful to them in considering future projects.

Literature Review

The following information is provided as an overview of the technical literature available on the topic of incremental launching; the coverage is broad and includes historical background, studies (primarily analytical) that focus on detailed technical issues related to the launch process, structural monitoring of the launch process, and brief incremental launch project descriptions that provide overview information.

Background

It is estimated that over 1,000 bridges worldwide have been constructed by the incremental launching method (Gohler 2000), the vast majority of which have been post-tensioned concrete box girder bridges. Their main application has been in Europe, but the method has now spread around the world and the technology has been applied to steel I-girder and box girder bridges as well.

In the early 1960s, the “modern” approach to launching concrete bridges was developed. The first concrete bridge constructed by launching was built over the River Caroni in Venezuela and was completed in 1963 (Podolny 1982; Baur 1977). The bridge was a post-tensioned concrete box girder bridge with a main span of 315 ft. The construction of this bridge was considered so successful that the launching method was utilized to construct a nearly identical bridge a few years later.

The first steel bridge to be launched in the United States is believed to be a Kansas City Southern Railroad box girder bridge near Redland, OK in 1970 (Durkee 1972). The nine-span continuous bridge is 2,110 ft. long with a main span of 330 ft. This bridge was launched in two trains, one from each side of the river. Closure of the bridge was accomplished at mid-span of the main span.

This method of construction can be applied to bridges made of either steel or concrete materials. The vast majority of concrete bridges built by the ILM were cast in stationary forms behind an abutment. Each new segment is cast directly against the preceding one; then, once proper curing has taken place, the entire structure is launched to create sufficient room for casting the subsequent segment. A steel bridge constructed by ILM is completely assembled (typically one span or more at a time), including steel cross frames and bracing, prior to launching operations.

During the launching operation, the bridge superstructure is supported by a series of rollers or sliding bearings. These rollers are removed following the launching and the bridge is lowered to rest on permanent bearings identical to those used for a conventionally constructed bridge. The thrust required to launch the bridge forward can be provided by a variety of jacking systems, including hydraulic pistons or hollow-core strand jacks more commonly used for post-tensioning.

In order to reduce the cantilever moments and the amount of deflection that occurs during launching operations, one of two systems (and sometimes both) may typically be employed. The contractor can construct a tapered launching nose on the leading end of the girders. The launching nose reduces the dead load of the cantilever span and utilizes its tapered profile to assist in "lifting" the mass of the girders as they are launched forward onto the landing pier. In other cases, the contractor may elect to use a kingpost system utilizing temporary stays to reduce the deflection of the leading end of the girders during launching.

It is more economical, and thus more common, to perform all launching operations from one end of the bridge. This permits the contractor to utilize only one set of jacking equipment and supporting rollers or sliding bearings. There have been examples, however, where the contractor has elected to launch the bridge superstructure from both ends of the bridge and join the two cantilevers somewhere near the center of the bridge.

The launching of bridges made of concrete requires a somewhat different set of solutions than those required for steel bridges. The design of the post-tensioning system must consider not only the in-place dead load stresses, but also the considerable stress reversals that occur during launching. Although the steel superstructure is considerably lighter than concrete, there are a number of issues related to large contact stresses applied to the girder bottom flange as well as the torsional stiffness of an open section, such as an I-girder, that must be addressed by the designer.

Historical Studies

Perhaps some of the best known examples of bridges constructed by incremental launching are the Bailey Bridges, which were used by Allied military forces during World War II. The Bailey bridge system consists of three main components (truss panels, transoms or floorbeams, and stringers). Each unit, when assembled, creates a single, 10-foot-long section of bridge with a 12-foot-wide roadway. After each such unit is complete, it is typically launched forward over rollers on the abutment and another section is built behind it. The two are then connected with pins pounded into holes in the corners of the panels. Additional load capacity can be developed by adding truss panels outboard of the first, stacked vertically, and sometimes both. The components are light enough to be assembled by infantry troops and launched by pushing with a truck or tracked vehicle (McLaughlin 2005). The success of this system is proven by the fact that, more than 60 years later, a number of temporary bridging systems currently in use around the world continue to borrow heavily from the Bailey Bridge concept.

The use of incremental launching is not limited to highway structures. In fact, the use of innovative construction methods to reduce the amount of “down time” for installation has been common in the railway industry almost since its inception.

In October 1907, the Canadian Pacific Railway launched a 415-foot span through-truss bridge over the French River near Sudbury, Ontario (Monsarrat 1908). Due to the deep water at the site, the entire truss span was erected on the north approach embankment and launched into its final position using “two specially constructed steel pulley blocks having fourteen sheaves each, through which was reaved a 5/8 inch diameter steel wire cable and powered by a 32 horsepower Beatty hoisting engine capable of pulling 8,000 pounds on a single line.” It should be noted that, although the equipment employed for bridge construction has been considerably improved in the past century, the basic launching technique has not really changed significantly.

The first major steel-deck railway bridge in America was constructed by incremental launching and opened to traffic in June 1971. The bridge consists of a continuous box-girder structure with nine spans ranging from 175 to 330 feet in length. The bridge was launched from both directions and joined at the center of the 330-foot span. The bridge plans and specifications called for the box-girder sections to be erected using incremental launching. However, during bid preparation, limited time was invested in the consideration of specific details of the launching procedure, which proved to be costly later in the project. Sliding-type supports were found to have limitations and on future operations the contractor would give serious consideration to the use of articulated roller-type supports (Durkee 1972).

A temporary roadbed and railway track were installed behind each abutment to accommodate “erection dollies,” or trucks on which the girder sections were erected and launched. During early launches, lateral deflection of the girder due to the sun’s heat caused considerable problems. A lateral misalignment of up to 6 inches was easily eliminated by pulling laterally on the leading end of the launching nose. Vertical girder deflections during launching closely matched predicted values.

Detailed erection calculations included both review of maximum cantilever conditions as well as the continuous beam condition behind the cantilever portion. In addition, a detailed study of web buckling behavior was conducted (Durkee 1972).

Analytical and Finite Element Modeling Studies

An important issue pertaining to launched steel girders is the load carrying capacity due to concentrated forces. The load on a launched girder is unique because in addition to a bending moment, a traveling concentrated load exists, which is applied by the temporary roller bearing. The concentrated load, also called a patch load, is transferred from the bottom flange of the girder into the web. The support reaction “moves” along the girder each time the launched segment passes over a pier bearing. It is important that patch loading does not introduce residual deformation or damage to the web plate. The effects of patch loading must be understood in order to know what web thicknesses are required. Even small increases in web thickness can add great weight and extra costs.

In order to better understand the patch loading phenomenon, finite element models were carried out on three types of girders: normal, slender and stocky (Granath 2000(B)). The girders were modeled with varying bending moments along with traveling concentrated loads to determine the ultimate load-carrying capacity for each girder type. The results showed that, when no bending moment is applied at the girder ends, the girders could be damaged by a traveling patch load at a level of 59 to 68 percent of the ultimate load-carrying capacity. No damage, however, occurred to the girders at levels of 42 to 49 percent of the ultimate load-carrying capacity with a similar load configuration. The load-carrying capacity was reduced by irreversible deformations caused by traveling patch loads. To avoid damage and reduced capacity, the study suggests an establishment of serviceability limit state criteria in terms of the attained stress and/or strain levels in the web plate. The analyses also showed that the girders experienced damage in the form of accumulating plastic deformation at higher load levels. The authors recommended that finite element analysis be used to determine the stress distribution at launching bearings for each individual launch and that no yielding should be allowed in the web plate, since this may accumulate into residual deformations that could be potentially harmful.

Granath (2000(A)) addresses the issue of establishing a service load level criteria for web plates by developing an easy, closed form design method for evaluating steel girders subject to patch loading. The method is based on the premise that no yielding is allowed in the web plate. The formulas presented in the study were developed by means of finite element methods and regression analyses.

Granath (1998) also evaluated the distribution of support reactions against a steel girder on a launching shoe. Reported in this study are the results from laboratory experiments, finite element analysis, and analytical calculations. These three evaluation methods focused on the distribution of the reaction force when a steel girder is launched on a launch shoe with a slide bearing or when a girder is placed on a tilted launch shoe with a polythene slide top plate. The design calculations for the pertinent load were performed with equations valid for a uniform distribution of bearing stresses. The results of this study indicate that the support reaction was not uniformly distributed, but the distribution of pressures can be described with an analytical model and finite element models.

Rosignoli (2002) presented a very detailed discussion of local launch stresses and instabilities in steel girder bridges. The author discussed the factors that contribute to a complex state of stress in the bottom flange of launched steel girder bridges. These factors include the following the movement of a precambered steel girder over launch bearings, thermal gradients in the structural steel, torsion and distortion resulting from misaligned launch bearings, local web compressive stresses generated by the dispersal of support reactions into girder webs, launch friction, and the gradient of the launch plane.

Rosignoli states that a non-stiffened web panel subjected to a concentrated support reaction applied through the bottom flange is affected by three collapse modes that depend on load intensity and on the slenderness of the web panel. These modes are local web yielding directly above the load, local buckling in the lower part of the web for a vertical depth of about 50 times the plate thickness, and general web buckling of the web panel. The author suggests a number of equations for checking the adequacy of the girder sections subjected to launch bearing loads.

Rosignoli also suggests the design support reaction be increased by at least 30% above the maximum theoretical support reaction to account for the expected misalignment of launching bearings and geometric irregularities in the bottom flange due to fabrication and assembly tolerances.

Bridge Design Studies

An excessive amount of calculations can be compiled during the design of a launched bridge due to the infinite number of support scenarios during the launching sequence (Rosignoli 1999(B)). In practice however a finite number of calculations are completed using closely spaced support configurations to acquire, with adequate reliability, an envelope of forces. The transfer-matrix method is currently an established algorithm used for determining the bending moment and shear forces in a launched bridge. The transfer-matrix limits the risk of mistakes and can easily be implemented in a computer program, however, as the launch progresses, the number of redundant conditions increases and thusly increases the time and decreases the simplicity needed to obtain results. The development of the reduced-transfer-matrix method has allowed for an exact, simple, and economical way to solve continuous beams involved in launched bridges. The reduced-transfer-matrix uses repetitive manipulations of square matrices, which have very few varying terms. The algorithm takes advantage of the repetitiveness of launch bridge segments by multiplying small matrices of constant dimension. The reduced-transfer-matrix can be done quickly with only a small computer.

The incremental launching of prestressed concrete bridges produces temporary stresses in the deck above fixed bearing locations (Rosignoli 1999(A)). Additionally, as the bridge is launched, the deck is needed to resist the same transitory stresses. The cross-sectional moment of inertia and web thickness size are affected by these temporary construction stresses and result in increased cost of materials. Comparing the cross-sectional dimensions of bridges built by incremental launching with other construction methods can allow statistical justification for launching prestress concrete bridges and help with presizing new structures. The decks of launched bridges have greater depths and uniform thickness over the length of the bridges relative to conventionally constructed bridges. The structural efficiency is improved by deeper deck depths; however, it also requires larger quantities of structural materials. The higher cost associated with increase in material quantities is balanced however, by lower technological cost, versatility of launching bridges in a wide range of spans and dimensions, and the increased quality of construction in a controlled environment. Since the material cost is directly related to the dead load of the precast deck, the cost can be reduced by using external prestressing, light weight concrete, high-performance concrete, or a prestressed composite section.

Rosignoli (1998(C)) also notes that during launching there is a signification difference in stresses between that of the support nearest the cantilever end and the rear supports. Generally, in order to obtain a cross section at the front stress zone to meet the capacity of the cantilever the entire structure would be burdened. Conversely, if the rear stress zone cross section was optimized the front section would have inadequate capacity. Due to the cyclical nature of the optimization it becomes economical to introduce devices to reduce the stress caused by the cantilever. To reduce the front zone stresses, combinations of the three different design solutions have been implemented in the past: adding temporary supports or decreasing the clear distance of the existing support, reduce the weight of the cantilever, and/or support the cantilever end. The implementation of a launching nose reduces the weight of the cantilever and provides temporary support prior to the cantilever reaching its maximum allowable stress. The launching nose acts as an extension of the deck and has become a standard design element in the last 30 years. The launching nose has proven to be safe, fast, and economical. The nose-deck system is controlled by three parameters: the ratio of nose length to cantilever span length, the ratio of nose weight to unit weight of front zone deck, and the ratio of flexural stiffness of nose to the stiffness of the front zone deck. Rosignoli presents a theoretical model for the optimizing the three parameters when designing the launch nose. The launching nose, in nearly all cases, will produce savings in structural material when optimized properly.

Construction Process Studies

Computer simulation has been performed to replicate the incremental bridge launching process (Marzouk 2007). Computer simulation is a useful to tool gain a better understanding of scheduling and the ILM process. Analyses can give a contractor a better understanding of time delays from limited resources, equipment breakdowns, and working environment issues. Incrementally launching bridges are becoming more common because they use significantly less temporary falsework than cast-in-place methods. Casting of incrementally launched concrete bridges involves three phases: 1) bottom flange and web fabrication, 2) top flange fabrication, and 3) the prestressing process. Two different launching procedures have been modeled with the computer simulation: single form launching and multiple form launching.

Single form incremental launching is a method in which the fabrication of the segments to be launched is done at one station and then launched. Multiple form launching takes advantage of two separate form stations. In the multiple form launching method, more than one segment can be fabricated at the same time. This speeds up the fabrication process.

A specific bridge project in Cairo, Egypt was used as a case study to model an incremental launching process. The bridge was built as a single form project in the field, but the computer simulation was done both with single form and multiple forms. The computer predicted the process would take about 397.1 days to complete fabrication using the single form method. This is slightly less than the actual time it took to complete the bridge in the field. The simulation was done again as a multiple forms project. The fabrication process was reduced to 374.43 days. The benefit of computer simulation is the factors could be changed to simulate different variations in the field. It was found that doubling the rebar crew gave a 37.39 day reduction in fabrication. Furthermore, if the entire crew is doubled the fabrication process can be completed in 330.14 days.

General Studies

A significant number of steel bridges have been constructed in Europe using the launching method. Svensson (2001) points out that the cantilever moments during launching can be six times larger than the final support moments on a continuous structure and that the maximum cantilever reaction can be greater than twice the regular support reaction after construction. In order to prevent local web crippling under these high loadings, it is necessary to use either heavy duty rollers or, as is in the practice in Germany, a sliding bearing which utilizes a Teflon-coated neoprene pad beneath the steel girder. Svensson presents a series of examples of steel bridges that have been successfully launched including I-girder systems, box girder systems, and, in a few cases, steel arch spans all using various types of support systems.

Incremental launching of bridges is a method that has been used by Russian contractors for over twenty years (Zhuravov 1996). ILM is a preferred method for projects with limited construction space. Bridges can be entirely launched from one abutment or they can be launched simultaneously from both abutments and locked at the midspan.

The most common launching method in Russia involves using jacks to push the bridge horizontally across piers with special sliding devices on supports to lessen friction forces. The pushing device is made up of hydraulic jacks and clamps. The superstructure is first clamped with a set of steel plates. The jacks then launch the bridge with the help of the clamps. When the cycle is complete, the clamps are released and re-attached to their initial position. One cycle can push the bridge up to 1.5 m and takes about 10–20 minutes. Sliding devices reduce the friction forces on the steel bridge as it crosses the supports. Sliding devices are made up of common elastomeric bearings, a stainless steel sheet covering, and sliding panels of plywood sheathed with PTFE antifriction material. The plywood sliding panels are placed between the steel girder of the bridge and the stainless steel sheet. The bearings are used to distribute stresses evenly across the girder.

Launching processes are computer simulated to anticipate the behaviors of the launch. The launch is then monitored to ensure stresses and deformations to not exceed set limitations. Bridges can be launched from one abutment or from both abutments. When launching takes place at both abutments, the two bridge segments are locked using a full-penetration weld at the closure joint. After the launch is complete, the superstructure is raised on jacks, permanent bearings replace the sliding devices, and the superstructure is lowered to its final position.

Textbooks

Three books have been published in the past which present a very comprehensive investigation of the design and construction of bridges constructed by incremental launching. These references are highly recommended for owners, designers and contractors desiring a thorough knowledge of the ILM. The books also make reference to several bridges that have previously been built by use of the ILM. The detail presented for these bridges, however, is insufficient to provide summary information for this report. A list of the bridges mentioned is provided in Appendix A for future consideration by bridge owners. The three books are listed with a brief bulleted summary of their content.

<u>Incrementally Launched Bridges: Design and Construction</u> (Gohler 2000)

- Overview of ILM
- Historical development
- Evaluation of ILM used for various crossings
- Design criteria and considerations for design

- Construction considerations

Bridge Launching (Rosiginoli 2002)
- Overview of ILM for prestressed concrete bridges
- Evolution of ILM for concrete bridges
- History of analytical knowledge
- Obstacles encountered prior to and during launch
- Details and components for effectively launching bridges
- Design and construction philosophies for prestress, composite, and prestress composite bridges

Launched Bridges: Prestressed Concrete Bridges Built on the Ground and Launched into their Final Position (Rosignoli 1998(A))
- Detailed analytical and conceptual information pertaining to the following:
    - Design
    - Organization
    - Economics of construction techniques
    - Construction methods
    - Launching techniques
    - Additional effects (i.e. thermal, time, etc.)
- Alternate launch methods (i.e., rotation, side translation, etc.)
- Trends and ongoing research

Brief Project Summaries

An article by Bergeron (2002) describes the launching of the four-lane Clifford Hollow Bridge in Moorefield, West Virginia. This 1522-ft. long, continuous I-girder bridge consists of six spans with two 210-ft. end spans and four 275-ft. interior spans requiring nearly 5.2 million pounds of steel. The original design, which was to erect the superstructure with conventional sequential construction of the girders with the use of cranes from below, was redesigned to use the incremental launching method due to constraints imposed by high piers and restricted access to the jobsite. After learning of the success of previous incremental launching projects, the bridge was redesigned to be launched. A series of 400-ft. long sections of the steel superstructure, consisting of steel plate girders, bolts and cross-bracing were preassembled and launched utilizing four hydraulic jacks that pushed each segment across the land-based track rollers on the higher abutment until the leading cantilever nose reached the temporary rollers on the piers. A cable-stay to the end of the bridge section was provided by a kingpost frame in the assembly area. Emergency brakes that are comprised of chains at the abutment were used for moving the section back up the track if needed. The launching required some modification to the girder designs to accommodate the launching stresses. The modification of the original design increased the weight of the girders but the longitudinal and transverse stiffeners were eliminated, offsetting the additional steel cost by reducing fabrication labor. The launching process turned out to be beneficial, especially from an environmental standpoint, as disturbance to the surrounding landscape and trees was reduced from that expected if conventional erection methods had been used.

An example of successful incremental launching over areas with high site restrictions can be found on three prestressd concrete bridges in the center of Milan, Italy (Rosignoli 1998(B)). This incremental launching construction involved in these bridges represents one of the most complex applications performed in Italy. The original interchange consisted of a single bridge, the Palizzi Overpass, spanning a six-lane railway that was replaced with two road bridges and a tramway bridge by this project. Severe site restrictions were placed on the project due to high traffic (including train) volume, limited site access, low vertical clearance of the bridges, the close proximity of electrical wires, and settlement problems. The first bridge launched was a three-span continuous beam, spanning a distance of 93.5 m. The construction process was restricted to a period of two hours each night to keep the train interruption to a minimum. After the first bridge was completed, the other two bridges were allowed to be launched without any time restraints and traffic was rerouted to the new bridge for demolition of the existing bridge. In spite of numerous site restrictions, the project was completed on schedule and without significantly disrupting either rail or road traffic.

The construction monitoring of the Paraná River Bridge is noted elsewhere in this report (Malite 2000). There are a number of significant features associated with the launching of the 2600-m-long bridge. First, the intermediate piers of the Paraná River Bridge were stabilized by two sets of steel cables anchored to the end piers and the central pier, which were designed to resist the horizontal launching forces. Second, the total length of the Paraná River Bridge girder was split into four segments with two segments launched from each side of the river. Third, the Paraná River Bridge consisted of a box shaped truss. Therefore, when the structure crossed each roller, intermediate forces were induced directly to the lower chord of the truss. Finally, the measured strain differed significantly from theoretical design values. In some cases, the theoretical model underestimated the bottom chord strain by a factor of two. The model was thought to be inadequately modeling the roller system and the non-uniform variation of temperature. However, the experimental and theoretical values were found to be in closer agreement for the upper chord members.

The Reggiolo Overpass is a monolithic, fully prestressed concrete bridge that spans 26 m over the Verona-Mantua Railway (Rosignoli 2001). Despite the short span length, several constraints, including the owner’s preferences for minimal maintenance, settlement problems at the site, complex geometry, and railway traffic below the bridge, made the design and construction of the bridge difficult and unique. These demanding constraints were resolved by casting the entire prestressed concrete superstructure on one side of the railway and then monolithically launching it to its final position. Another interesting aspect of the project was the prestressed concrete launching nose used to compensate for negative moments during launch. It was found that using detachable concrete launch noses could be less expensive than steel girder launching noses. This project proved the use of monolithic launching to be a safe, reliable way to construct a bridge in a short time over a railway or traffic.

The Petra Tou Romiou Viaduct is an eight-span, continuous, curved, mono-cellular, concrete box girder bridge in southern Cyprus (Llombart 2000). The deep valley below the bridge made incremental launching a favorable method of construction. The structure is also governed the high seismic activity in the area. The 422.6 m long bridge is made of post-tensioned concrete. The bridge was launched by means of hydraulic jacks with temporary launching pins in the girders to facilitate the launch. Neoprene-Teflon pads covered by a stainless steel sheet were used to reduce friction over the pier bearings. The project was scheduled to launch one 18.45 m deck segment per week. A combination of short and tall piers was used to support the superstructure. Piers range in height from 16 m to 60 m. The abutments and shorter piers are connected to the superstructure by dampers. These dampers compensate for horizontal loads caused by seismic activity and wind. The taller piers are connected to the superstructure by fixed bearings.

The Easton Bridge is a three-span, steel girder bridge that crosses a deep ravine in the Cascade Mountains of Washington (ENR 1998). The bridge spans a distance of 255 feet and serves as a recreational trail. Boss Construction Co. was hired to erect this bridge in place of a trestle that had been swept away by floods. The bridge was supposed to be erected from either side of the ravine but limited crane access on one side prompted the contractor to use a launching method. The bridge was mounted on two dollies at the western side of the ravine and pushed over rollers on the first pier to the second pier 195 feet away. A crane then dragged the bridge the last 60 feet to the eastern abutment. Push-pull jacks were used to counter deflection of the bridge as it was launched across the ravine. The jacks, located on the piers, raise the bridge high enough to pass over the piers. The construction, originally planned for one day, had to be delayed over a weekend. The bridge was pushed halfway across and stabilized on Friday. The following Monday, construction was finished.

Bridge launching was a successful method for a bridge replacement in the Paddington Station area of London (ENR 2005). Westminster City decided to relieve congestion near the Station by removing an older bridge and constructing a new wider bridge over the continuously running train tracks. A bridge launching method was chosen because it would necessitate the least amount of rail line closures. The new steel girder bridge with composite concrete deck spans a distance of 180 m. Construction began by raising the old bridge on four temporary jacking towers. The bridge was then launched underneath the old bridge. After completion, the old bridge was lowered and removed. Varying girder depths along the length of the bridge were an issue as the bridge was launched across the piers. Jack levels were adjusted continuously during the launch to compensate for these irregularities.

The first incremental launching method in Netherlands was used in construction of the Ravensbosch Viaduct, as shown in Fig. 1**,** that forms part of the motorway connecting Maastricht and Heerien in Southern Netherlands (VSL 1977). The bridge crosses the valley of Strabekervloedgraaf near Valkenburg at a height of about 25 m. Its superstructure is comprised of two parallel box girders with a 37.77-m wide deck slab on top. With a total length of 420 m forming 8 spans, it is uniformly curved with a radius of 2,000 m.

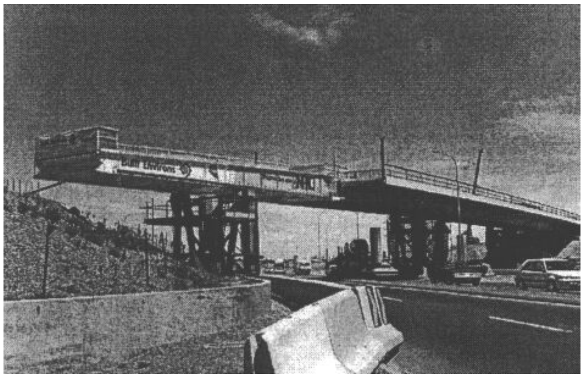
During the launching operation, specially designed bearings consisting of a block of concrete covered with a stressed sheet of chrome steel were installed on all permanent and temporary piers. Steel/neoprene/Teflon plates were placed between the leading box girder and these bearings to keep the friction to a minimum. The friction recorded at each launching operation was approximately five percent, which was close to the assumption made during the design.

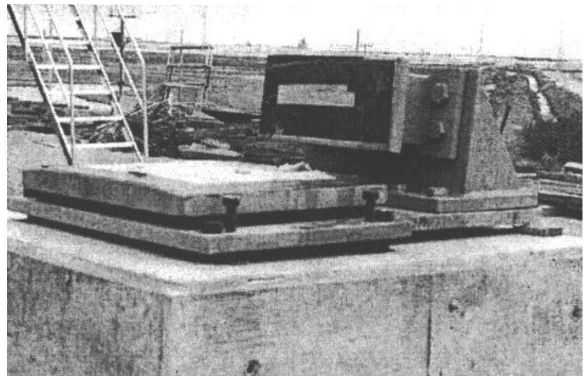
Two hydraulic jacks, fixed to steel girders that were placed in front of the eastern abutment, were used with each stroke generating 200 mm of launching. On average, it took six hours to complete the launching of each 19-m segment girder. Lateral guides were provided on both sides at every permanent pier and on the inner side at the temporary piers to ensure correct alignment of the structure during launching.

Figure 1. Launching nose resting on a temporary pier

The bridge over Port Wakefield Road consist of twin, four-span, single-cell prestressed concrete box girder bridges that carry eight lanes of traffic and an estimated traffic flow of 59,000 vehicles per day (Alistair 2000). Due to the high traffic volumes in the vicinity and safety concern, the bridge was erected by incremental launching and it was one of the first incrementally launched bridges built in South Australia. The bridge launch is shown in Fig. 2a. The 112-m long bridge consists of four spans of 23, 33, 33 and 23 m, respectively. The span lengths were governed by the three piers located in the medians of the roadway below. The superstructure consists of twin single-cell prestressed box girders. Box girders were used because they provide a high torsional stiffness. The box girders contain two stages of longitudinal prestress. The first stage is intended for launching stresses and the second stage provides strength for service loads.

The original construction plan required temporary piers to be built between the 33 m spans to account for the large span length. It was, however, later proposed to eliminate the temporary piers by using a launching nose. The permanent pier bearings could not be used during the launch because of punching failure to the box girder. Temporary columns were set up, one on each side of a pier, and temporary launch bearings were installed. These temporary bearings, seen in Fig. 2b, were installed under the box girder webs, and were later replaced with permanent bearings once the launching was completed. Elastomeric launch pads were used at the launching bearings to keep the friction to a minimum and lateral guides were provided at the piers and abutments to control the bridge along the correct alignment.





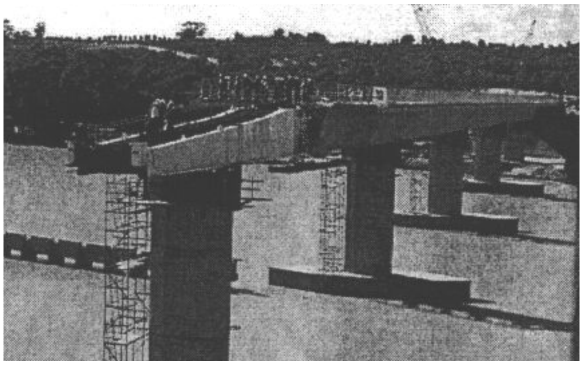
(a) Nose resting on a temporary pier **(b) Launching bearing**

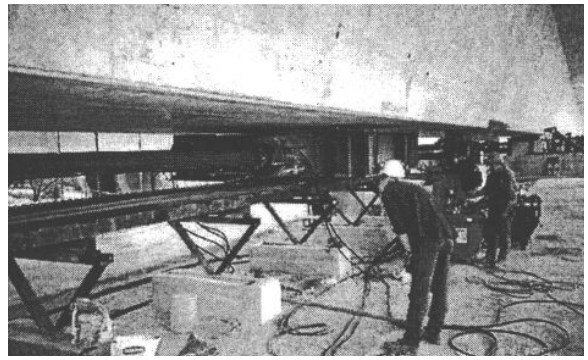
Figure 2. Port Wakefield Road Bridge launching - Australia

Experiences from the launching of the Bridge over Port Wakefield Road assisted in the construction of the Blanchetown Bridge crossing the Murray River in Blanchetown, Australia (Alistair 2000). The bridge is a 407-m long single cell prestressed concrete box girder that consists of 7 spans with seven 50-m interior spans and two 25-m and 30-m end spans. The bridge was built to replace an existing bridge that was structurally deficient. The deck carries two lanes of traffic with an additional path for bicycles and pedestrians. The bridge was erected by incremental launching due to the long span length and cost saving associated with the construction. The bridge launch is shown in Fig. 3a.

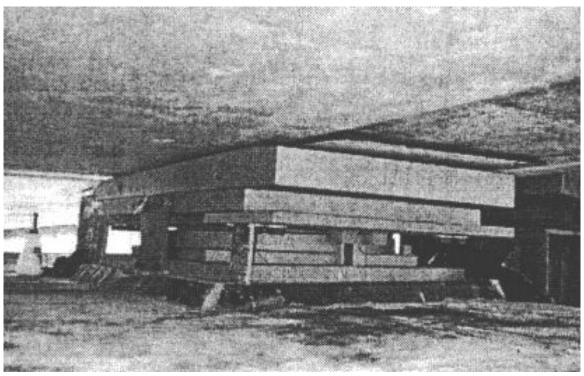
The superstructure was launched in segments of 25 m from pier to pier using a custom-made jacking frame. Two jacks were used to launch the bridge away from the frame, as seen in Fig. 3b. These segments were cast in two concrete pours and launched on weekly cycles. Permanent bearings, with a steel plate and elastomeric bearings placed between the bottom flange and the permanent bearings, were used for launching, shown in Fig. 3c. The use the permanent bearings for launching eliminated the use of temporary bearings normally used in incremental launching operation. 'Spray-on' silicone grease was used to keep the friction between the superstructure and the bearings to a minimum.

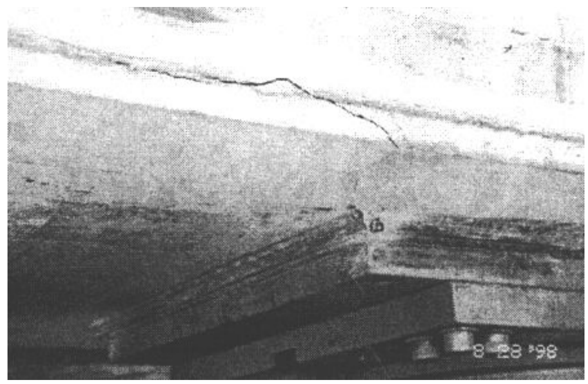
Following construction, three types of cracks were found in the box girder: punching shear cracks, flexural cracks and cracks adjacent to the cast-in bearing plates. The punching shear cracks were caused by the concentrated force from the permanent bearings on the box girder. The flexural cracks caused by eccentricity of the bearing reactions on the box girder were found on the top flange of the box girder. Cracks also formed near the cast-in bearing plates, seen in Fig. 3d, because the contractor welded the launch blocks to the cast-in plates instead of bolting them together. The thermal expansion and contraction of the metal caused the cracks to form. Lessons were learned from the project that the launch bearings should be positioned as close to the box girder webs as possible since eccentricities between the bearings and web cause flexural cracking during the launch. This may be difficult to accomplish when using permanent bearings. The experience also suggested that the launch bearing contact area of the girder be stiffened if permanent bearings are used for launching. In addition, it is critical to meet the construction tolerances on the box girder profile since small irregularities can cause uneven stress distribution.





(a) Bridge launching **(b) Jacking operation**





(c**) Launch bearing** **(d) Crack near the cast-in bearing plate**

Figure 3. Blanchetown Bridge launching - Australia

Structural Monitoring during Construction

The use of structural monitoring during construction of an incrementally launched bridge has received considerable attention from both owners and university researchers. Structural performance information through monitoring can supplement visual observations and may provide critical alerts during the launch stages at structure locations during the launch process. It can also provide validation of the design and construction process, which is useful for implementation of subsequent ILM projects. Several representative examples of ILM projects where monitoring was implemented are provided below.

The Paraná River Bridge, a 2600-m-long steel structure with 26 spans of 100 m, is the largest bridge combining highway and railway systems in Brazil, and was erected by incremental launching (Malite 2000). Four bridge segments, two 600 m long and two 700 m long, were assembled on the riverbank, then pushed longitudinally into their definitive positions. Several parameters were monitored during launching, including the stresses at critical points of the steel structure, displacements at the tops of the piers and at the end of the cantilever, forces required for launching, and ambient temperature.

Monitoring of a steel plate girder superstructure launch was performed on the Iowa River Bridge crossing (Wipf 2004). This report documents the launching procedure and monitoring and evaluation of various bridge components during numerous launches of the Iowa River Bridge. The bridge components were instrumented and monitored to assess the launch procedure and the subsequent structural impact on the superstructure and substructure. The overall objective of the project was to validate the assumptions made by the bridge designers, HNTB Corporation, and the contractor's erection engineer, Ashton Engineering. These launch assumptions included such things as:

- The force applied to piers during launch events
- The frictional resistance of roller system during launch events
- Behavior of piers caused by large horizontal forces applied to capbeam during launch events
- Girder flexural behavior during launch events, including contact stress and bending
- Load transfer mechanism between girders
- Horizontal force necessary to launch various construction stages

Generally the measured structural response of the superstructure and substructure elements was relatively consistent with design and construction expectations. Some selected results include 1) the measured contact stresses in the flange/web region during girder passage over a launch roller were relatively high, 2) pier column stresses during launching were relatively moderate and consistent with expected structural performance, and 3) measured launch forces were consistent with forces expected using the hydraulic pressure gages.

Survey of State DOT Bridge Engineers

Survey Process

An electronic survey was developed with the original intent to contact the chief bridge engineers of the 50 state DOTs. In order to ensure a wider representation of the bridge community, the survey was in fact directed to each member of the AASHTO Subcommittee on Bridges and Structures. The subcommittee comprises 116 individuals from a wide variety of owner agencies, including the following:

- 50 state Departments of Transportation (1 to 3 committee members per state)
- Federal Highway Administration
- American Association of State Highway and Transportation Officials
- Canadian provincial transportation agencies (5 members)
- Other bridge owner agencies (e.g. turnpike authorities, US Army Corps of Engineers, etc.)

The survey was developed, distributed and collected through an online survey service entitled SurveyMonkey.com, allows respondents to access an online version of the survey, respond to each question, and submit their answers via an easy-to-use form. In addition, the survey data can be continually analyzed by the research team to monitor trends. Each response can be traced back to the email address of a respondent. Follow-up reminder notes were sent to each survey recipient who did not respond initially.

Survey Results

Overall, a total of 40 survey responses were recorded by the online system, for a response rate of 34 percent, which compares reasonably with past surveys performed by the research team. A presentation of data and discussion of responses from *selected* survey topic questions are briefly summarized in this section. The complete survey is provided in Appendix B of this draft report. In the data tables associated with some questions presented in this section, those responses that received more than 50% of the total are highlighted in red for those questions where a respondent was asked to evaluate the significance or usefulness of various alternatives.

<u>Selected Survey Topic 1: Familiarity with ILM</u>

This question focused on the personal familiarity of the engineer with the incremental launching method for bridge construction. In addition to asking that question, the engineer was asked how they first learned about incremental launching.

The response to the first question resulted in 55% stating they were personally NOT familiar with incremental launching. Only 40 replies were received for this question, and it is very possible that a much higher percentage of surveyed engineers are similarly NOT familiar with launching as a construction practice.

Regarding the question about how the respondents found out about incremental launching, the majority (33%) indicated that conference presentation was how they were exposed to the topic, with 11% indicating a technical journal article was the source of their information. Trade publications, books and "other" comprised the other responses. It was interesting that 0% responded to the medium of documentary video/DVD as a source of information. This response is noteworthy because the Iowa Department of Transportation created a documentary video following the completion of the US 20 Iowa River Bridge in 2003. The project involved the incremental launching of a steel plate girder bridge superstructure. The video was mailed to all state DOTs and FHWA division offices. However, upon discussion with some respondents, unclear wording of the question was found to be a possible reason why they did not indicate the video as a source of information. The video includes information associated with construction and monitoring of the launched bridge.

Based on the response to the questions above, it appears that technical information regarding incremental launching has not spread widely. One possible reason for the lack of technical literature on this topic is that the designers and contractors are not very interested in sharing information, although it is fair to state that publication by these two groups is usually not a high priority.

Selected Survey Topic 2: Level of familiarity

As a follow-up to the questions noted above, the survey participants were asked how they would rate their familiarity with the ILM construction method. The question asked about the level of familiarity on a [4] point scale from [1] extremely knowledgeable (personally involved) to [4] completely unfamiliar. The majority of the respondents (73%) indicated a rating of [3] where they had read an article or had attended a presentation, supporting the observation for the above question that there are few engineers with working knowledge of incremental launching.

Selected Survey Topic 3: Advantages of ILM

This question attempted to determine the perception of the respondents regarding advantages of the ILM when compared to conventional construction methods. The question was somewhat leading in that the types of advantages were provided, although the respondents had opportunities to provide their own replies.

Table 1 summarizes the response to the question. As noted, the two primary advantages (based on a cumulative response of “very significant” and “significant” responses, appear to the 1) minimal disturbance to surroundings (95%) and 2) reduced access required beneath the bridge (77%). It is also noted that smaller equipment required for construction, increased worker safety, and increased construction speed are also perceived to be significant benefits of incremental construction.

Table 1. Perceived advantages of ILM compared to conventional construction

	Very Significant	Significant	Somewhat Significant	Not Very Significant
Minimal disturbance to surroundings	39%	**56%**	0%	6%
Reduced access required beneath bridge	33%	44%	22%	0%
Smaller, more concentrated work area	6%	29%	41%	24%
Increased worker safety due to ground-level assembly	6%	44%	33%	17%
Increased speed of construction	11%	44%	39%	6%
Smaller equipment required for construction	0%	**56%**	22%	22%

Selected Survey Topic 4: Disadvantages of ILM

In contrast to the question noted above, this question tried to determine the perception of the engineer regarding potential disadvantages of the ILM as compared to more conventional construction. As the previous question did, this question provided leading response topics.

Table 2 summarizes the response to the question. As shown in the tabular responses, the biggest concerns the respondents had with incremental launching compared to conventional construction methods are 1) perceived risk to owner and contractor, 2) increased costs and 3) contractor unfamiliarity with the incremental launch method.

Table 2. Perceived disadvantages of ILM compared to conventional construction

	Very Significant	Significant	Somewhat Significant	Not Very Significant
Perceived risk to owner and contractor	12%	**65%**	18%	6%
Increased costs	29%	**59%**	12%	0%
Increased time for construction	6%	41%	18%	35%
Requires specialized hardware (rollers, jacks, etc.)	12%	44%	31%	12%
Contractor unfamiliarity with method	41%	**53%**	6%	0%
Increased horizontal forces on substructure	6%	24%	41%	29%
Access requirements behind abutment(s)	6%	47%	35%	12%

Some pertinent comments received with this question, apparently related to why an owner did not choose incrementally launching include 1) the launching of haunched girders is very difficult, 2) there is an increasing problem with clear spanning rivers without enough back span for this method to work (e.g. when you have a 350 ft. main span with no or minimal approach). This problem arises when permitting for river access or construction is denied for various reasons. Launching has only been possible with adequate pier placement, 3) some grand failures have been recorded over the years associated with launched bridges and 4) potential structure redesign.

<u>Selected Survey Topic 5: ILM projects completed, under construction or planned</u>

The intent of these questions was intended to determine the level of experience and activity by the respondents in implementing ILM. Specifically how many bridges has an agency either completed or currently have under construction using ILM, and further, is the agency CURRENTLY CONSIDERING an incrementally launched bridge project for future construction.

The reply from the respondents was very brief regarding any past, current or planned ILM activity. Respondents identified two launched bridges for which the research team was not previously familiar. Additional follow-up with this respondent was made to obtain additional information, but unsuccessfully thus far. The owner did offer to share copies of the project plans and construction details and photos, and the research team will pursue this information for inclusion in the final report. The two bridges are 1) the Queets River Bridge (WA), steel I-girder, completed 1991, and 2) the Yakima River Bridge (WA), steel I-girder, completed 1999. Additionally, no agencies reported any potential future incremental launch construction.

Selected Survey Topic 6: Useful tools to assist design of ILM projects

Recognizing that perhaps one reason the ILM has not caught on routinely in the United States, this question was intended to determine what design tools would be most helpful to engineers. The objective of this question was to also provide the research team input regarding the format and content of this report in order to be most helpful to the enginerering community.

Table 3 summarizes the response to the question. Based upon the responses shown in the table, it seems reasonable to state that the most useful tools for preliminary and/or final design would be 1) series of illustrative case studies; 2) detailed list of recommendations; 3) collection of proven launch details; and 4) description of launching limitations. It is interesting that even though a database of case studies has existed online for sometime (see Appendix A for further information on the database), apparently this has not had a major impact on more use of incremental launching. This could possibly be due to the engineering community not being aware of the database information. The authors of this report have not been able to find a significant amount of information related to the other desirable pieces of information.

It is also noted from the table that 1) preliminary design assistance; 2) final design consultation; and 3) independent review for constructability are also desirable tools for engineers, and the lack of these perhaps have had a negative impact on incremental launch use.

Table 3. Types of useful tools for design of ILM projects

	Very Useful	Useful	Somewhat Useful	Not Useful
Description of launching limitations	48%	35%	17%	0%
Series of illustrative case studies	26%	**65%**	9%	0%
Detailed list of recommendations	**57%**	35%	9%	0%
Collection of proven details (jacks, rollers, etc.)	**52%**	35%	13%	0%
Preliminary design assistance	13%	**57%**	26%	4%
Final design consultation	13%	**52%**	30%	4%
Independent review for constructability	13%	**61%**	26%	0%
Detailed list of published technical papers and reports	22%	43%	35%	0%

Selected Survey Topic 7: Potential interest and intent in implementing ILM

These questions attempted to determine the level of interest that the bridge community has in implementing ILM. Specifically, one question asked if the agency would BE WILLING TO CONSIDER an incrementally launched bridge project for future construction. Additionally, they were asked how likely they would estimate a launched bridge would be utilized by their agency in the future.

These related questions, associated with potential interest and intent regarding implementing an ILM, interestingly yielded somewhat contradictory responses. To the question of would the agency be willing to consider a project, 83% of the replies were yes. In contrast, the estimate of how likely your agency would implement a project, approximately 44% stated that would not be likely. Approximately 11% stated it would be very likely. This would seem to suggest that agency would very much like to consider incremental launching as a construction process, but either have insufficient confidence to do so, or simply have few projects that would require launching.

Selected Survey Topic 8: Useful tools in consideration of ILM for future projects

A previous question had asked what tools would be most useful for design of ILM projects. This question asked what types of tools would be most useful to the engineer in even considering incremental launching for future construction projects, recognizing the level of perceived disinterest to date by the engineering community. The survey question contained specific concepts for tools that the research team thought could address the lack of interest and/or activity to date. Another objective of the question, similar to some of the other questions asked in the survey, was to help the research team determine the format of the product of this research.

Table 4 summarizes the response to the question. Based on the results of this survey, it is clear that bridge owners are frequently lacking in a general understanding of the incremental launching method and its potential benefits in the appropriate location. Filling this knowledge gap was one of the overall goals of the current research project. Perhaps the most useful information that can be extracted from the survey results is the types of tools that bridge owners feel would be most valuable to them in the planning and design of future launched bridges. This conclusion is supported by the results from the previous two questions.

Table 4. Types of useful tools to promote consideration of ILM projects

	Very Useful	Useful	Somewhat Useful	Not Useful
Flowchart for planning activities	4%	39%	43%	13%
Summary of launching applications	4%	**70%**	26%	0%
Description of launching limitations	35%	48%	17%	0%
Series of illustrative case studies	22%	43%	35%	0%
Detailed list of recommendations	39%	43%	17%	0%
Preliminary design assistance	13%	35%	43%	9%

MANUAL OF BEST PRACTICE

Innovative construction methods have been used since a tree was felled across a stream allowing the first bridge engineer to cross without getting their feet wet. Bridge owners in today's political and economic climate must often consider whether a potential innovative bridge construction method may be suitable for use at a particular site.

Frequently, there are two types of bridge projects – those where an innovative construction method is recognized early in planning stages as the only feasible way to complete the crossing and those where a resourceful contractor proposes a clever value engineering solution to a challenging problem.

Based on the personal experiences of the research team and a review of both successful and unsuccessful bridge launching projects described in the literature review and provided as case studies elsewhere in this report, we can offer a number of issues to be considered for future projects.

The manual of best practice highlights a few recommended planning, design, and construction activities that should be considered when developing a project for construction by incremental launching. Many of these activities would be useful in cases where other forms of innovative bridge construction would be appropriate as well.

Preliminary Design and Planning Considerations

Recognize Critical Restrictions

Early recognition of project site challenges such as environmental issues or sites which offer only limited access for construction make it easier to consider the value of alternative construction methods as early in the preliminary design phase as possible.

Establish Advisory Panel Early in Process

The value of an advisory panel for any specialized project that has not been attempted by a particular owner cannot be overemphasized. Owners, designers and contractors are available who are willing to share their experiences (positive and negative) and assist the owner by providing examples of previous projects. One source of this information is the project case study summaries provided herein.

Two primary cases exist in which the ILM may be useful as a potential construction method for a particular project: either the proposed bridge must cross an obstacle (such as a sensitive waterway, deep valley, or railyard), which makes site access problematic; or there is a need to accelerate construction using a limited footprint behind one or both abutments. In either case, the detailed design of the launching system to be used is typically performed by the selected contractor's engineer along with the specialty equipment providers who sell or lease the high-capacity hydraulic jacks and rollers which are available.

The establishment of a contractor advisory panel should be considered well in advance of the project letting date and preferably early in the design phase of the project. The members of this panel should include experienced bridge contractors of moderate to large size from the surrounding area and, depending on the size and location of the project, this could certainly include surrounding states as well. In order to encourage participation and valuable contributions, it should be made clear that that the members of this advisory panel should not be excluded from bidding on the project. In addition, the panel representatives should be invited to participate through the local chapter of the Associated General Contractors in order to eliminate the appearance of bias toward a particular contractor.

The research team recommends that the panel be convened at least twice during the design of the project – once at the beginning of final bridge design and again as the design is approximately 90 percent complete. At the initial meeting, the panel members should be given an opportunity to visit the proposed project site. The visit should be in conjunction with the design team as well as representatives from the owner's construction staff and they should be provided with at least some details of the proposed bridge alignment, preliminary plans, and an explanation for why the particular project might be considered for launching. The panel should be asked to provide recommendations regarding necessary clearances, crane swing radii, working areas that might be necessary for material storage and laydown, which might be helpful in property acquisition, etc. A second meeting near the end of final bridge design should be used to review the launching details for the bridge as well as to ensure that all questions and concerns are thoroughly and completely addressed.

Input received from this type of advisory panel would potentially be useful to owners, designers and contractors alike. The owner may feel confident that more reasonable bids may be anticipated from a well-informed contracting community. In fact, the need for launching may perhaps be eliminated by an innovative contractor who is able to devise a system to construct the same bridge by alternative means. The design team (either agency designers or consultants) will gain valuable input at critical stages of the process which can be used to adjust both the preliminary layout as well as the final design details that may result in a better overall product at a more reasonable bid price. The contractors, in turn, will be better able to plan their work and begin early conceptual engineering of their own which will help reduce the need for rapid engineering on their part during the bidding process and the consequent bids which must be magnified to address the additional risk they feel due to the uncertain nature of a complex project.

An alternative approach which could offer similar advantages would be to require the designer to have an experienced ILM contractor included in the design team. This contractor would, of course, be precluded from bidding on the ultimate construction project. In addition, a number of national engineering consultants exist who specialize in providing advice to bridge owners and designers on constructability issues. Alternatively, the owner and designer could seek the advisory services of an experienced ILM contractor, perhaps from outside the region, with the understanding that they will not submit a bid (or team with a bidder) on the particular bridge project under consideration.

Engage Specialty Equipment Manufacturers

Manufacturers of specialty bearings, rollers and jacking equipment should be contacted to obtain examples of innovative solutions which have been used for similar projects. The use of incremental launching in particular is one method which has seen relatively widespread use in Europe and around the world which has spurred the development of specialized equipment.

Final Design Phase Considerations

Substructure Effects Caused by Launching Forces

The forces applied to a substructure element due to launching a bridge include three vector components which include the following:

- Vertical loads due to representing the dead load support reaction at the pier
- Longitudinal loads generated by the friction and other resistance forces in the bearings as well as the local grade of the launch surface
- Transverse horizontal component generated by the lateral guide system

Rosignoli (2002) presents a detailed presentation of the substructure forces which should be carefully considered during final bridge design. On some steel girder bridge projects, the horizontal component of the substructure forces must also include the resultant force generated as the tapered transition ramp (launching nose) encounters a pier roller bearing. Researchers at Iowa State University (Wipf 2004) attempted to document the impact of these forces during launching of the US 20 Bridge in 2002. It should be noted that for particularly short piers, the impact of these forces could be significant.

Lateral Guidance and Steering Control during Launching

An adequate lateral guidance system must be provided for the superstructure during launching operations. It is well-known that steel girder bridges are subject to sun-induced curvature prior to placement of the concrete deck. Essentially, the girder face exposed to the sun warms considerably quicker than the face which is shaded. This phenomenon is not typically problematic on a conventional bridge construction project and is commonly ignored. However, when this curvature occurs during a launching event, there can be significant problems in maintaining the alignment of the girders and providing a means to keep them tracking along the desired path.

A guidance system is recommended which provides lateral resistance of at least 10 percent of the vertical reaction at a given pier during the entire launching process. This lateral resistance also contributes to resist the lateral forces due to wind forces which are applied to the length of the girders and any fabrication and assembly tolerances that may exist.

Wind Forces during Launching

The design and contractor team are highly recommended to consider the effects of wind on a potential launched bridge project. The effect of both static and dynamic wind forces during the construction of the bridge using incremental launching must be considered, particularly in the case of a lighter-weight steel superstructure. An analysis of the static wind forces applied to the superstructure at maximum cantilever is not sufficient to include the possible effects of buffeting caused by a blunt body. In some cases for longer spans, the use of wind fairings to help improve the aerodynamic performance of the cantilever span has been used with reasonable success.

In order to eliminate potential problems with wind effects during a launching operation, a clause is suggested to be included in the project special provisions which prohibit launching of the bridge when forecast conditions indicate a likelihood of wind speeds on a given day in excess of a particular threshold, perhaps 20 to 30 miles per hour. The recent availability of internet-based weather documentation and prediction forecast sites make it routine practice to verify the predicted wind speeds for 12 hour periods in advance of a critical event.

Reversible Launching System

In order to reduce the chance that a bridge is left in a vulnerable position with a long cantilever for an extended period of time, the utilization of a launching system that is reversible is recommended – in other words make it possible to retract the cantilever span back to a suitably stable position in the event of a mechanical problem. It would also be wise to ensure that each launch event be suspended at a stable position with only a minimum cantilever extended.

Lateral Bracing System for Steel Girder Spans

The modern concept for incremental launching was developed in the 1960s, primarily for use on concrete box girder superstructures. These girders are inherently very stiff and provide considerable resistance against torsional buckling during the launching phase. However, this same resistance is not pertinent for a typical steel I-girder bridge. The advantage of a steel superstructure is a significant savings in dead load resulting in potentially smaller rollers and bearings, as well as reduced jacking force needed to launch the superstructure. This makes these an attractive alternative for moderate spans.

A system of upper-and-lower lateral bracing is highly recommended to be included in the design of steel girder superstructures in order to provide the necessary torsional stiffness during launching operations. This bracing should be designed as a primary member for calculated loads during the cantilever stage. In particular, the bracing is of critical importance in the leading span which undergoes reverse bending during the cantilever stage of construction. The bracing is likely not needed in the final condition and could be removed following completion of the bridge deck. However, due to the cost and difficulty of this operation, it may be more economical to simply leave the bracing in place for the final condition.

Temporary Supports and Auxiliary Piers

The need for temporary piers constructed at midspan of the permanent crossing can rarely be justified except in the case of extremely long spans. The Millau Viaduct, which was recently completed in southern France, utilized temporary piers to reduce the cantilever length but the cost of these towers was significant. The design team was able to justify the cost due the extreme wind forces which have been recorded in the Tarn Valley. The desire of the design team was to reduce the free cantilever length as much as possible. A review of the literature fails to show the use of these temporary supports on spans less than 450 ft unless it is necessary to launch the span along a horizontal curve. The cost of these temporary towers can quickly exceed the cost of a longer launching nose or temporary kingpost system.

Steel Girder Flange Contact Stresses and Girder Web during Launching

There has been considerable research into the subject of contact stresses on the bottom flange of heavily loaded steel girder bridges which is presented elsewhere in this report. It should be noted that large contact stresses must be considered during design and appropriate consideration must be given to both localized effects on the bottom flange as well as web buckling and crippling concerns.

When launching a bridge superstructure over a series of roller supports which are fixed in position, essentially any point along the length of each girder line serves as a support point at some point during the launching operation for the non-composite steel dead load. It is critical that the girder web be stiffened appropriately to resist this loading without the risk of local web buckling due to the combined flexure/shear acting at this point.

Required Jacking Forces to Overcome Friction and Longitudinal Grade

The use of a low friction roller system is recommended for use on all future launched girder bridge projects. These rollers are typically assumed to provide a frictional resistance of 5 percent when rolling across a surface covered with steel plating sufficient to resist deformations due to the heavily concentrated load. Laboratory testing has shown this friction coefficient may be as low as 1 to 2 percent under static conditions.

It is certainly possible for a bridge to be launched along a longitudinal grade of up to several percent in either positive or negative grade. Certainly the idea of launching the bridge along a positive grade (uphill) offers some advantages in that there is no concern of allowing the bridge to roll unencumbered in the event of a mechanical failure during a launch event. Conversely, the additional force required to overcome not only the inherent friction in the roller system along with the energy to raise the mass of the bridge superstructure during the launching must be designed into the jacking system and may require larger equipment. The decision as to which end of the bridge will best accommodate the jacking system is a function of the local access and restrictions and should not be seen as being controlled by the mechanics of the launching system.

Analysis of Erection Stages

Much has been written about the challenges of analyzing a bridge for incremental launching. Essentially, an envelope of flexural moment and shear forces must be calculated over an infinite number of support conditions as the superstructure is launched. These calculations are compounded in the case of a bridge constructed with post-tensioned concrete as the additional effects of creep and shrinkage must be included along with thermal gradient concerns.

Design of Specialized Bridge Components

Due to the significant number of these projects which have been completed in Europe, there has been an opportunity to develop standard bridge launching equipment which is commonly specified. Particular components to be selected or designed include:

- Design or selection of bearings/rollers. Past projects have typically used proprietary rollers but a few projects were constructed using rollers which were custom-made for the specific application;
- Design of launching nose;
- Design of lateral guides; and
- Design of kingpost and cable-stay system (if required). The need for additional girder stiffeners at the location beneath the kingpost must be considered.

Recommended Construction Phase Considerations

Review of Contractor's Engineering Submittals

Innovative bridge construction projects, such as incremental launching, place an additional burden on the contractor and their construction/erection engineer to thoroughly calculate loads and stresses placed on the structure through the chosen construction method. In addition, details of connections or stiffeners added to the permanent structure, falsework required to construct the bridge or any other substantial modifications to the contract plans must be detailed for review. These calculations and details are submitted to the owner and the engineer of record for review and approval prior to the start of construction.

Often times, a contractor and their engineer will develop an erection procedure which differs significantly from that shown in the contract plans and specifications. In this case, the contractor should be requested to submit a complete set of structural analysis calculations. The review of these calculations will often necessitate the engineer of record to perform an independent modeling of the contractor's launching stages and construction loadings. The time required for this independent modeling is greatly reduced by the ability to reuse the original design model with only slight modifications.

The complete and timely review of these contractor submittals is critical and cannot be overemphasized. It is recommended that open communication between all parties is maintained in order to facilitate the review and reduce or eliminate the need for resubmittals.

Structural Monitoring during Construction

Some concerns naturally exist when implementing new technology (e.g. incremental bridge launching). Because launching is "very serious business" and can often be relatively new to contractors as well as the owner/designer, there should be some steps considered to minimize problems. It has been shown that structural performance instrumentation and monitoring of existing bridges provides supplemental information to the design and evaluation process. Similarly, instrumentation and monitoring of bridges during construction phases can provide valuable validation of the design/construction process and timely feedback during the actual construction process. This is particularly true for incremental construction of bridges, especially given the use of relatively unfamiliar construction techniques and equipment. The discussion above regarding the various incremental launch issues provides excellent information about where structural performance monitoring may be useful. By using strain, displacement and tilt sensors, some of the critical bridge superstructure and substructure elements, as well as launch equipment and launch components, can be monitoring during the launch process.

At a minimum, it is recommended that the contractor consider positioning experienced personnel at each supporting pier location to monitor the relative position and performance of the superstructure throughout the launching operations. These personnel should be equipped with radio communications to be able to immediately suspend launching operations in the event that a problem is observed.

The following are some general considerations if incremental launching projects are undertaken:

- For monitoring of future launched bridges, contract language should be included to provide reasonable access and assistance to the monitoring staff. Coordination among the contractor, the monitoring consultant, and the structural designer is essential to the success of the project.
- A comprehensive monitoring program, which alerts the contractor/designer/owner of potential problems, should be implemented to insure that allowable stresses are not exceeded. The designer should develop a design model showing the expected stresses and the anticipated load distribution during the launch. These values for allowable stresses/forces covering all anticipated modes should be developed in advance.
- A pre-launch and post-launch survey of the structure should be performed.
- Use a set of mirrors or some other system to monitor the plumbness of the piers during and after launching operations.
- Crossframe members of the superstructure are particularly vulnerable to unusual launch forces and potential monitoring should be considered if the crossframe members, girders and connections have not been designed to support the weight of one girder supported only by the crossframe connections to the adjacent girder.
- Designers should develop a launching system that is reversible. In other words, there

should be a method of retracting the cantilevered girders in the event of an unexpected problem. Monitoring of the cantilevered portion of the superstructure could provide useful information regarding potential problems.

- It may be advisable to monitor the structural response of the piers to the touchdown forces during the launch and during the passage of the superstructure over the pier.
- A number of other behaviors that would be useful to monitor would be 1) girder flexural behavior during launch events, including contact stress and bending; 2) load transfer mechanism between girders; and 3) horizontal force necessary to launch various construction stages.

Applicability and Limitations of Incremental Launching

The use of the incremental launching method for bridge construction will never be the most efficient way to construct every single bridge. However, in the right location, the ability to erect the bridge superstructure without the need to intrude into either congested, restricted or environmentally sensitive areas beneath the bridge offers tremendous benefits to the owner, contractor and other stakeholders including:

- Minimal disturbance to surrounding area;
- Smaller, but more concentrated area required for erection; and
- Increased worker safety since all erection work is performed at a lower elevation

During the launching of a bridge, the superstructure acts as a continuous beam supported on roller or sliding bearings and is transversely restrained by lateral guides that prevent drifting movement. Any constraint eccentricity (vertical misplacement of launching bearings or transverse misalignment of lateral guides) will cause unintended secondary stresses and may cause launching problems such as excessive wear of bearing devices (Rosignoli, 2002).

The case studies presented in this report highlight the fact that incremental launching is applicable to a wide variety of challenging bridge sites. The recent FHWA scanning tour of Europe and Japan has identified a number of bridge launching projects for which launching was considered the most efficient solution to a difficult bridge construction problem. Although virtually all bridge projects can offer their share of challenges, the K.S. Tubun flyover bridge is exceptional in the number of degree of difficult circumstances. This bridge, located in Jakarta, was designed to cross a navigable drainage canal along with the city's largest railway junction all while passing with less than 2 feet below high voltage power lines. In addition, the contract documents stated that there could be no disturbance to the railway traffic at any time during construction. In order to eliminate the need for a temporary pier located in the rail yard, the bridge was designed with a particularly long launching nose – approximately 70% the length of the permanent span. The K.S. Tubun flyover bridge launching nose can be seen in Fig. 4.

Figure 4. K.S. Tubun flyover bridge launching nose

Essentially, the incremental launching method is worthy of consideration for project sites which face challenges such as:

- Steep slopes or deep valleys which make delivery of materials difficult,
- Deep water crossings.
- Environmental restrictions which prevent or severely limit access.
- Access to area beneath bridge limited by heavily traveled roadways or railways.

Ideally, a bridge intended for incremental launching would be designed along a tangent alignment in both horizontal and vertical planes to simplify fabrication and construction. However, the bridge site which fits these ideal conditions is extremely scarce especially when combined with the close proximity of the potential site restrictions listed. Although somewhat more challenging, it is possible to construct a bridge by incremental launching while maintaining a curved alignment in either or both planes. In order to eliminate the relocation and adjustment of lateral bearings it is necessary, however, that these surfaces remain perfectly aligned with the superstructure during launching operations, which can only be guaranteed in the case of a common geometry. Rosignoli (1998(A)) states that a bridge constructed by launching must be designed with one of the following alignments:

- Tangent in plan and tangent or circular in profile.
- Circular in plan and horizontal in profile (no launch gradient).
- Circular in plan and included with respect to the horizontal plane.
- Curvilinear both in plan and in profile.

The geometry of curved structures and the desire for uniform distribution of launch stresses strongly favor the use of constant depth superstructures such as a parallel flange I-girder. It is possible to utilize a variable depth steel superstructure by using temporary steel plate or trussed extensions of the bottom flange. A variable depth superstructure is greatly complicated by the higher dead load present during launching operations.

Case Studies

The incremental launching method has been used for bridge construction at a wide range of sites and for a broad variety of purposes. As stated previously, it has been estimated that nearly 1000 bridges have been constructed using this method worldwide. Unfortunately, the competitive nature of bridge design and construction has considerably limited the amount of detailed information which has been published.

Assembled herein is a brief summary of eleven bridges constructed by incremental launching which is intended to provide contact information associated with the projects, as well as highlight the potential benefits of the method for bridge owners and contractors faced with a bridge site which might be unsuitable for more conventional "stick-built" construction methods. Appendix A includes a table summarizing the information for the bridges described in case studies within this report, as well as, information pertaining to an online launched bridge database containing additional case studies.

The project summaries have been selected to highlight both steel and concrete superstructures and are not intended to suggest that one material may be more suitable for this type of construction than another. It is important to note that, as opposed to conventionally constructed bridges, bridges built by the incremental launching must be designed with the final manner of construction firmly in mind from the earliest stages of the project.

Perhaps the most critical consideration in the selection of a bridge construction method is the cost involved when compared to more conventional construction. Although the contractor is responsible for employing an experienced, licensed professional to provide erection engineering and selecting the specialized construction equipment to be used on a particular project, these costs are passed on to the owner either in the form of a particular bid item such as "Bridge Launching (Lump Sum)" or as subsidiary to other bid items on the project.

As is often the case in the highly-competitive construction industry, the cost of these specialized bridge construction bid items are not widely published and are not available without considerable research into each specific project. Therefore, the projects presented in the following case studies do not present this information. As evidence of this variability, the U.S. 20 Iowa River Bridge project was completed at a cost of approximately $150 per square foot while the Stoney Trail Bridge was constructed for nearly $450 per square foot.

Due to the large number of widely-ranging variables involved in each particular project including bridge material, site location, local economic factors, fabrication processes, environmental constraints, inflation, currency exchange rates, etc. it is not possible to present a confident estimate of general launched bridge construction costs.

It is recommended that bridge owners anticipate some reasonable cost premium for any innovative bridge construction method when compared to conventional "stick-built" construction methods. For budgeting and planning purposes, it is recommended that a cost premium of 10–

15% above the conventional superstructure construction cost be considered as a reasonable estimate.

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
U.S. 20 Iowa River Bridge Hardin County, IA Construction Completed 2002	Superstructure 5 – 302' steel I-girder spans with lateral bracing	Substructure CIP concrete piers with driven steel piles and drilled shaft foundations	Total Cost = $21M (USD)

PROJECT CONTACT INFORMATION

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Iowa Department of Transportation Mr. Ahmad Abu-Hawash, P.E. Iowa Department of Transportation 800 Lincoln Way Ames, IA 50010 ahmad.abu-hawash@dot.iowa.gov (515) 239-1393	HNTB Corporation Mr. Michael LaViolette, P.E. 715 Kirk Drive Kansas City, MO 64105 mlaviolette@hntb.com (816) 472-1201	Jensen Construction Mr. Dan Timmons 5550 NE 22nd Street P.O. Box 3345 Des Moines, IA 50316 dtimmons@rasmussengroup.com (515) 266-5173

PRIMARY REASONS FOR LAUNCHING

The bridge was constructed using the launching method due to a number of very stringent environmental restrictions near the project. These environmental issues included endangered mussel species residing in the Iowa, endangered plant species near the site and Native American artifacts near the site. In addition, a bald eagle roosting area was identified near the site. An extensive environmental monitoring program was established and maintained during construction.

BRIDGE STRUCTURAL DESCRIPTION

The bridge consists of two parallel deck superstructures, each with five equal spans of 92 m (302'). A 19 m (62') prestressed concrete jump span is provided on each end of the steel unit. The I-girders were fabricated from ASTM A709 Grade 345 weathering steel; they are 3450 mm (11') deep and spaced at 3600 mm (12') centers. The web-depth choice was based not on strength requirements, but rather to reduce dead-load deflection during the cantilever-launching phase to a reasonable level. Since any point along the girder length could become a bearing location during launching operations, the constant 22 mm (7/8") web thickness was designed to serve as an unstiffened element for steel dead load.

In order to make the I-girder superstructure act as much like a torsionally rigid box girder as possible during launching, a stiff system of diaphragms and lateral bracing was used. A diaphragm spacing of 7,000 mm (23') was used for spans two through five, but was reduced to 3,500 mm (11'6") in the leading span that would be cantilevered during launching.

BRIDGE LAUNCHING SYSTEM

The bridge superstructure was completely erected on steel falsework and custom-made 18" diameter rollers behind the east abutment. A 146' long, tapered steel launching nose was erected at the leading end of the girders and used to reduce the cantilever deflection during each launching operation. After each span was launched forward, additional steel girder sections, including diaphragms and bracing, were pushed forward to land on the subsequent pier. The process was completed five times for each steel superstructure. After the complete launching of the eastbound girders, the falsework was removed and reinstalled to perform an identical launching of the westbound superstructure.

STRUCTURAL MONITORING SYSTEM

A structural monitoring program was developed by the ISU Bridge Engineering Center to evaluate critical aspects of the incremental launch procedure and the corresponding effect on the superstructure and substructure so that design assumptions could be verified. For the substructure, monitoring included measuring strain in the pier columns, rotation of the pier cap, and general displacement of the substructure system. Superstructure monitoring included measuring longitudinal strains at selected cross-sections in the steel girders, longitudinal strains in select cross-frame members, and contact strains in the girder bottom flange and web. In addition, the force required to launch the bridge was monitored.

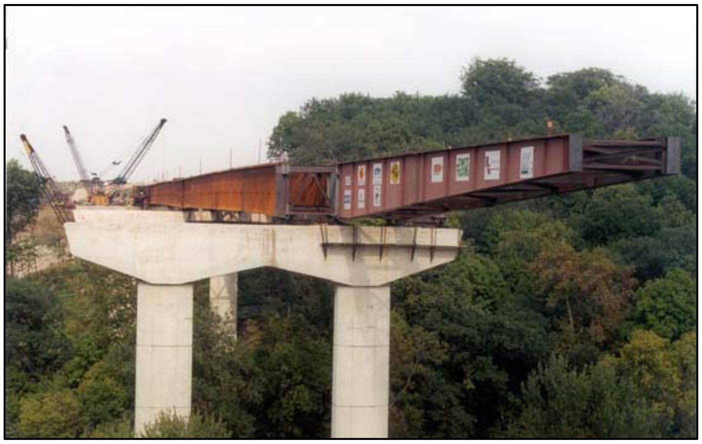
HNTB performed the preliminary and final design for the bridge and provided full-time onsite resident engineering expertise during construction. Jensen Construction served as general contractor on the project.

REFERENCE

LaViolette, M., "Pushing", Structural Engineer, May 2003.
LaViolette, M., McDonald, D., "Landmark Launch", Modern Steel Construction, February 2004.
Rogowski, D., "Green Giant", Bridge Builder, January-March 2003.

Vertical support roller and guide roller during launching

Launching system including transverse jacking beam



Tapered launching nose on leading end of girders

Aerial view showing project worksite and girder bracing

Girder erection performed in launching pit

Project completed – opened 2003

<table>
<tr><td rowspan="2">TITLE AND LOCATION (City and State)

The Stoney Trail Bridge
Calgary, Alberta, Canada
Construction Completed 1997</td><td colspan="2">BRIDGE TYPE AND MATERIAL</td><td>CONSTRUCTION COST</td></tr>
<tr><td><u>Superstructure</u>
5 – span double-celled concrete box girder</td><td><u>Substructure</u>
CIP concrete abutments and piers</td><td>$48M (Canada)</td></tr>
</table>

<table>
<tr><td colspan="3">PROJECT PARTICIPANTS</td></tr>
<tr><td>PROJECT OWNER

City of Calgary, Canada
800 Macleod Trail SE
P.O. Box 2100, Stn. M.
Calgary, AB Canada T2P 2M5
Mail Code #230
Phone: (403) 268-CITY (2489)
Fax: (403) 538-6111</td><td>PROJECT DESIGN FIRM

J.R. Spronken & Associates Ltd.
550 6 Avenue SW,
Calgary, AB T2P0S2
Tel. 403-265-1123</td><td>PROJECT CONTRACTOR

Walter & SCI Construction (Canada) Ltd.
Yarmouth, NS
Phone: (902)742-2665
(N/A)</td></tr>
</table>

BACKGROUND

The Stoney Trail Bridge is a horizontally curved, segmentally constructed bridge and was the second incrementally launched reinforced concrete bridge to be built in North America (first in Canada). Each of the concrete segments was built on one bank and then jacked horizontally into its final position atop 30 m high ‘Y’ shaped concrete piers. This structure is the featured element of a $48M (Canada) project forming the first leg of a long awaited northwest perimeter transportation corridor for the city of Calgary. The incremental launching technique was particularly well suited for this project because of the $1.5M (Canada) cost savings that this method offered, and also because of the sensitive nature of the surrounding environment: the south bank contains one of the few stands of Douglas Fir trees in this area.

BRIDGE DESCRIPTION

The bridge is a 476 m (1562 ft) x 21 m (68 ft), 5-span structure with a main span of 102 m (335 ft), 40 m (131 ft) above the Bow river valley. The superstructure consists of a 4.5 m (15 ft) deep girder elements. It consists of cast-in-place concrete abutments, piers and superstructures. The superstructure section is a post-tensioned, double-celled monolithic concrete box structure and was cast in two stages: soffit and webs cast together, followed by the deck in two segmental casting beds. The box girder and deck was assembled in segments (total of 19 segments) on the north bank, post-tensioned with steel reinforcing cables, and then pushed from the north abutment to the south. Each completed segment (1200 tons) is 25.5 m (84 ft) long with the exception of end segments which are 22 m (72 ft) long.

BRIDGE CONSTRUCTION AND LAUNCHING

The bridge construction involved curved, post-tensioned segmental concrete placed with hydraulic jacks for both vertical lifting and horizontal sliding. The bridge superstructure was pushed and pulled with hydraulic jacks over a system of temporary sliding bearings and lateral guides that were mounted on permanent and temporary piers. During the launching, external post-tensioning was performed inside each cell to provide the structural support.

The bridge was launched on a 3 percent uphill grade, which required both friction and gravity forces be overcome. The entire casting yard was curved and superelevated to match the superstructure grade. Although casting yard production permitted a launching sequence to be undertaken every 7 days at the peak of operations, the overall fabrication of the 19 segments and 20 launching sequences took longer (approximately 40 weeks to complete) than anticipated primarily due to a slow learning process and weather constraints.

A 180 tons steel launching nose was used to guide the concrete segments onto the piers. The 32 m (105 ft) nose served as a relatively lighter cantilever section to reach the next pier. This launching nose reduced bending stress in the precast bridge sections and ensures clearance with the next pier.

Five intermediate temporary steel piers allowed the bridge to be launched between permanent piers with a 51 m (167 ft) cantilever span. Sliding bearings that consisted of groups of steel laminated elastomeric pads supported by concrete pedestals were placed on top of each of the temporary and permanent piers. In addition, thin low friction Teflon pads were installed on the top of each pier to reduce the friction between the pier and the bridge sections during launching operations. The Teflon pad moved with the superstructure over the bearing, requiring a crew to be stationed at bearing locations to pick up the slider pad, which would otherwise drop off the bearing, for reuse at the interface of the moving superstructure and the bearing.

The lift/launch/drop mechanism required 2 sets of 3 hydraulic jacks, one set placed at the abutment and one on the first temporary pier. The front hydraulic jacks were used to compensate for vertical deflection as the nose landed at the oncoming sliding bearing. Each jack pushed 1300 tons and extended about 250 mm (10 in.). This lift/push/drop sequence moved the superstructure in 250 mm (10 in.) increments. Approximately 5 to 8 hours were taken to launch a segment length of 25.5 m (84 ft).

NOTE

This bridge was constructed through a popular recreation area, Bowness Park. One of the main challenges of the project was to comply with environmental requirements for the sensitive areas of Douglas Fir trees and to ensure minimal disruption of Bowness Park. By utilizing the incremental launch technique, it was possible to concentrate the majority of the construction activities away from these areas. The bridge was assembled on the north bank and 'launched' over the environmentally sensitive areas. The use of a temporary bridge over the Bow River allowed construction access to the south bank, with no access through Bowness Park. The precise/prestressed construction methodology reduced the amount of equipment and limited work crew contact with environmentally sensitive sites. Special attention was given to minimize the amount of runoff discharge directly into the river.

REFERENCES

Skeet, J., Lester, W., McClary, C., "Incremental Launch: The Stoney Trail Bridge:, Concrete International, Vol. 20, Issue 2, February 1998.

McGarth, R., "Concrete Thinking in Engineered Structures", Cement Association of Canada, 2002.

PROJECT PHOTOS

Launching nose supported by temporary steel pier

Completed Bridge

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
Brides Glen Bridge Dublin, Ireland Construction Completed 2003	Superstructure 3 – span post-tensioned concrete box girder	Substructure CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
N/A	Roughan and O'Donovan (ROD) Tony Gee and Partners (TGP)	Main contr.: ASCON Sub-contr.: VSL Systems (UK) Ltd. and TGP

BACKGROUND

In late 2003, VSL Systems (UK) Ltd. completed the construction of a pair of incrementally launched box girder bridges, which were one of a series of bridges on the South Eastern Motorway project at Brides Glen in Dublin, Ireland. Tony Gee and Partners (TGP), commissioned by VSL, worked closely with the contractor in this environmentally sensitive area to develop the overall launch methodology and a detailed deck design for the twin structures. The overall design was engineered to minimize material quantities and to optimize the construction advantages by incrementally launching the structures from one side to the other.

BRIDGE DESCRIPTION

Each bridge is a 160 m (525 ft) long and 20 m (65.6 ft) wide, three span post-tensioned concrete box girder structure. Each deck is divided into 10 segments varying between 15.3 m (50.2 ft) and 16.3 m (53.3 ft) in length. The segments were cast in two stages within a specially constructed casting cell behind one of the abutments. When the final segment was launched, the total weight of each bridge exceeded 6,000 tons.

BRIDGE CONSTRUCTION AND LAUNCHING

Given the unique nature of the bridge-launching operation, VSL and TGP was sub-contracted by the main contractor, ASCON, for the detailed bridge deck design and construction that was appropriate to the specific casting and launching method. While the outside shape of the deck and span layout from the original design was maintained, effort was given to simplify the internal arrangement of the deck and reduce the overall material quantities. First, the total number of segments was reduced from 13 to 10 per deck to shorten the overall casting program, which also suited the internal variations in web thickness. The section of the deck was also optimized with reduced web and slab thicknesses. While the original design proposed the use of both internal and external tendons, only internal prestressing tendons were used. This resulted in eliminating the heavy mid-span deviators.

Permanent incremental launch pot bearings that were specially designed for both the temporary and permanent loading conditions were used in preference to temporary bearings at the piers and abutments. These special pot bearings consisted of a profiled top plate and stainless steel sliding surface for launching and avoided the need to substitute permanent bearings for the temporary ones after completion of the launch.

The bottom slab and webs including the diaphragms were initially cast, followed by the top slab cast in a second stage. When sufficient strength of the concrete was attained, each segment was post-tensioned and launched out of the casting yard using hydraulic jacks. TGP undertook the design of the 28 m (92 ft) steel launching nose that was connected to the leading segment.

The construction and launching of each segment was designed to be performed in a 7-day cycle. However, the actual construction and launching of the two decks took longer (11 months) than originally anticipated, partially due to difficulties in achieving sufficient concrete strength (50 MPa or 7.3 ksi). From the early trials, the contractor believed that concrete strengths of 25 MPa (or 3.6 ksi) could be attained within 36 hours with the use of super-plasticized concrete. However, it took up to 4 days for the concrete to achieve adequate strength for stressing and launching. As a result, over 10 days were taken for the average cycle time for a segment to be completed.

NOTE

With the launch bearings placed beneath the webs during launching operations, the adoption of permanent bearings creates offset inside the web, which generates punching shear and localized bending in the bottom flange and box webs. Recognizing the lessons learned that the incorrect placement of the temporary bearings played an integral role in the collapse of the Injaka Bridge in South Africa in 1998, finite element analyses of the deck webs and bottom flanges were undertaken to investigate the behavior above the bearings during the launch. These analyses took into account the maximum possible inward eccentricity of the slipper pads during the launch and the effect of the un-grouted tendon ducts on the flow of stresses in this area was considered. To this end, it was decided to add additional slab and web reinforcement to resist the local bending and shear stresses co-existing with the global forces during the launch.

In order to validate assumptions made in the design, the project team monitored the loads and deflections in the deck during the launching operations. This monitoring was conducted by measuring the reactions of the hydraulic jack at the temporary supports and by surveying the deck levels at predetermined intervals during each launch phase. The comparison of the monitoring results with theoretical values resulted in a good general agreement. Where necessary, some adjustments of the reactions and levels were made within a prescribed range that reflected the design limits for the launch position.

Following summarize some additional issues regarding incrementally launching prestressed concrete bridges:
Details must account for both incremental launching and in-service stages to produce an efficient prestressing scheme.
The designer must analyze complex stress distributions around incremental launch bearings to produce a safe design.
Realistically achievable construction tolerances must be considered and incorporated into the design assumptions.
The construction cycle is governed by the required early strength gain of concrete.

REFERENCES

Hewson, N. and Hodgkinson, A., “Incremental Launch of Brides Glen Bridge, Ireland”, Concrete, Vol. 38, Issue 7, 2004

PROJECT PHOTOS

Launching nose rested on a CIP concrete pier

Casting of the first segment

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
Vaux Viaduct Vaud, Switzerland Construction Completed 1999	Superstructure 14 (13) – span steel-concrete composite girder	Substructure CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Etat de Vaud	Giacomini & Joliet Realini & Bader SA	Steel construction: Zwahlen & Mayr SA Prestressing: VSL International Pot bearings and expansion joints: Mageba SA

BACKGROUND

The Vaux Viaduct, located beside the Lake of Neuchatel between Lausanne and Bern, Switzerland, is one the major bridges on the A1 highway. Due to environmental concerns in and around the region, the bridge was constructed by launching two large spans (130 m {426.5 ft} each). At the time this was one of the largest launched curved spans in the world that did not use any intermediate supports.

BRIDGE DESCRIPTION

The Vaux Viaduct is a steel-concrete composite bridge with a total length of a 945 m (3100 ft) and a width of 13.46 m (44 ft). It consists of two independent structures, one for each driving direction. The north bridge is comprised of 14 spans while the south has 13 spans. The heights of the central piers are nearly 100 m (328 ft) above the Vaux Valley. Each bridge crosses the valley with two 130 m (426.5 ft) spans; the remainder of the bridge consists of shorter spans that are 56 m (184 ft) to 62 m (203 ft) in length. The horizontal geometry consists of two circular curves, each with a radius of 1000 m (3281 ft).

BRIDGE CONSTRUCTION AND LAUNCHING

The steel superstructure is made of weathering steel and was prefabricated and transported to the construction site in segments; the largest segment had a lengtm (105 ft) and a weight of 58 tonnes (64 tons). The transverse steel section for the long spans is a closed box girder with a depth linearly varying from approximately 6 m (20 ft) over the highest piers to 3.86 m (12.7 ft) at the end of the 130 m (426.5 ft) spans. A traditional twin plate girder section was used for the shorter spans.

Traditional construction equipment and methods, such as cranes and simple launching, were used to place the steel superstructure for the short spans and for the parts of the bridge with short piers. The longer spans (box girders) were launched from east to west along the curved geometry of the highway creating a maximum cantilever of 130 m (426.5 ft), which was one of the longest in the world. A 35 m (115 ft) long launching nose was used to reduce the bending moments in the cantilever. This was followed by a two-beam girder of about 45 m (147.6 ft) in length with no provisional staying attached to the girder.

Segments of 32 m (105 ft) girders were assembled and launched every two weeks (on average) with the total length of the launched girder and the maximum weight of a launch reaching approximately 400 m (1312 ft) and 16000 kN (3597 kips), respectively. Generally, one day was required to carry out the launching operations for a single stage with a launching speed of about 10 m/hr (33 ft/hr). Hydraulic jacks, placed on the top of the piers with a temporary anchorage system, were used to lift the girder nose. The maximum deflection that occurred as the launching nose reached the piers was 4.5 m (14.8 ft).

NOTE

Uncertainties and the relatively complex geometry of the bridge - including horizontal curvature and variable depth box girders - caused significant challenges that needed to be addressed during construction. The erection procedures and the launching operations required careful planning based on detailed calculations to evaluate the uncertainties regarding the support reactions and the patch-loading resistance.

Considering the uncertainties with respect to the patch loading resistance and due to the sensitivity of the support reactions caused by the large torsional stiffness of the box girder, the indirect loads and the construction tolerances, it was decided to monitor the bridge in real-time in order to better control the reaction distribution and to make corrections if necessary. The continuously measured reactions were compared with the predicted values and the level of the supports was adjusted whenever the reactions diverged more than 15 percent from the predicted values. Two adjustments were typically needed for the launching of a 30 m (98.4 ft) section. These adjustments, which did not cause significant delays, were made by either placing thin plates under the sliding shoes on the fixed supports on several piers or varying the levels of the supports behind the abutment.

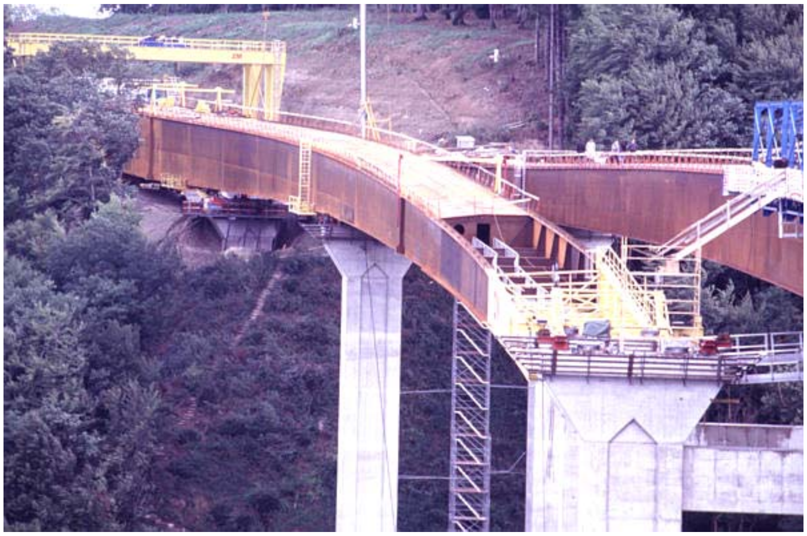
With the real-time monitoring of the bridge, the project team was able to correct the support reactions, keep the applied patch-loads within the accepted limits, and properly adjust the vertical support positions during launching, allowing the successful completion of the complex erection process.

REFERENCE

Lebet, J., "Composite Construction in Steel and Concrete V", Proceedings of the 5th International Conference, July 2006.

PROJECT PHOTOS

Launching of the steel structure, 130 m cantilever (Courtesy of Jean Jecker)



TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
Serio River Bridge Bergamo, Italy	<u>Superstructure</u> 19 – span double-cell precast box girder	<u>Substructure</u> CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
N/A	N/A	N/A

BACKGROUND

The Serio River Bridge, located near Bergamo in northern part of Italy, crosses a distance of approximately 800 m (2600 ft) over the Serio River and was constructed in an area with sloping topography and a wide turbulent riverbed. The design and construction of this bridge was governed by these restraints, ruling out the use of erection on false work. Instead, incremental launching was chosen for the construction and medium spans and circular piers were adopted to minimize scour and erosion from the rushing currents. This project was one of the most significant applications of the incremental launching techniques utilized in Italy, and one of the few such constructions that has a double-celled cross section.

BRIDGE DESCRIPTION

The bridge consists of 38 segments of a slender precast box girder structure with seventeen 42.6 m (140 ft) interior spans and two 36.4 m (120 ft) end spans. The deck is 2.3 m (7.5 ft) deep in depth with a length-to-height ratio of 18.5. The bridge is supported by cast-in-place piles that are 1.2 m (3.9 ft) in diameter. These piles are drilled 15 m (49.2 ft) into the ground. The piers and pier caps are cast in steel formwork. The superstructure is fixed to the central pier to reduce movements at bearings and at expansions joints.

The girder consists of a central web and lateral webs. The central web was designed to resist the majority of the shear force while the lateral webs channel eccentric loads towards the central web. This design allowed the girder to resist torsion and distortion during both launch and service conditions. A board-marked finish was used to give texture to the bridge.

BRIDGE CONSTRUCTION AND LAUNCHING

The bridge launching construction method was chosen for this bridge because of the previously mentioned turbulent river conditions. The bridge was friction-launched across the river valley in 38 segments. These 38 segments box girders were match-cast in a yard just beyond the east abutment. Each segment was cast on-site and transported to the staging area by a gantry crane. The steel cage for the first casting phase (bottom slab, webs, cantilevers, and side curbs) was assembled and inserted into the formwork by a lattice hangar. Later, the internal steel cage was placed into the form. The gantry crane was covered to shield it during bad weather. This allowed the project teams to cast a deck segment per week (3 m {10 ft} of complete superstructure per calendar day, which required less than 5 hours of labor per square yard of deck surface including mobilization and demobilization).

The superstructure was launched by means of a friction launcher over bearings spaced 1.1 m (3.6 ft) apart in the transverse direction (i.e., the deck was moved over launching bearings on each pier cap). During each launch, the superstructure was guided by pivots acting in an axial offset. This approach allowed the rounding of the deck corners, otherwise used as transverse constraint. The parameters of the superstructure and launching system were monitored to assure they stayed within the specified limits.

NOTE

The Serio River Bridge project had to overcome two problems that were caused by launching bearings and work stoppage. The bearings used to support the bridge during launch were vital to the success of the launch. Due to geometric irregularities in formwork, positional tolerances, and human involved errors, small misplacements between the two bearings were created, which in turn generated secondary stresses (torsion and distortion) on the superstructure. These were minimized by imposing strict tolerances in formwork and bearing positioning. In addition, plasticization of launching bearings was used to limit stresses in the deck.

When the 8th deck segment was completed, there was a contract dispute causing a work stoppage for 21 months. This stoppage produced large creep deformations in the superstructure. Although these deformations did not cause an impediment to the completion of the project, a few alterations to the launching device (e.g., launching nose, launching bearings and external tendons) had to be made to account for effect of creep deformation; the bottom flanges of the launching nose were shimmed to account for the deformation in the front deck zone; the launching bearings were realigned by inserting shimming steel plates and all neo-flon plates were replaced; finally, a pair of temporary prestressing tendons was anchored to the deck to account for the moment capacity being exceeded. However, no adjustments were made on the structure itself. Even with these unusual problems that the project team had to overcome, the incremental launching method proved to be a viable method of construction.

REFERENCE

Rosignoli, M., "Creep Effects During Launch of the Serio River Bridge", Concrete International, Vol. 22, Issue 3, March 2000

PROJECT PHOTOS

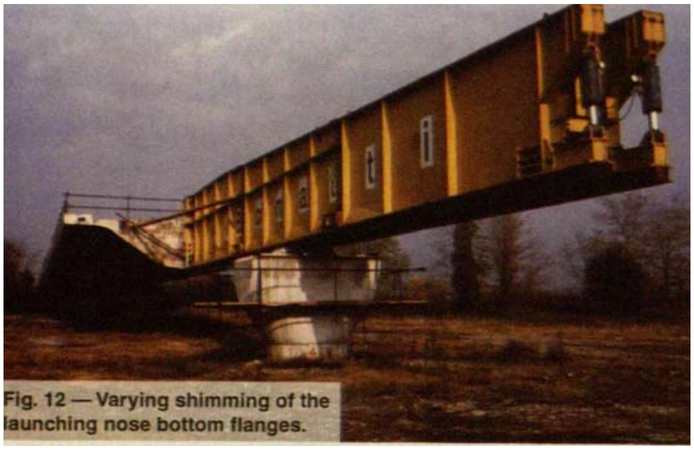
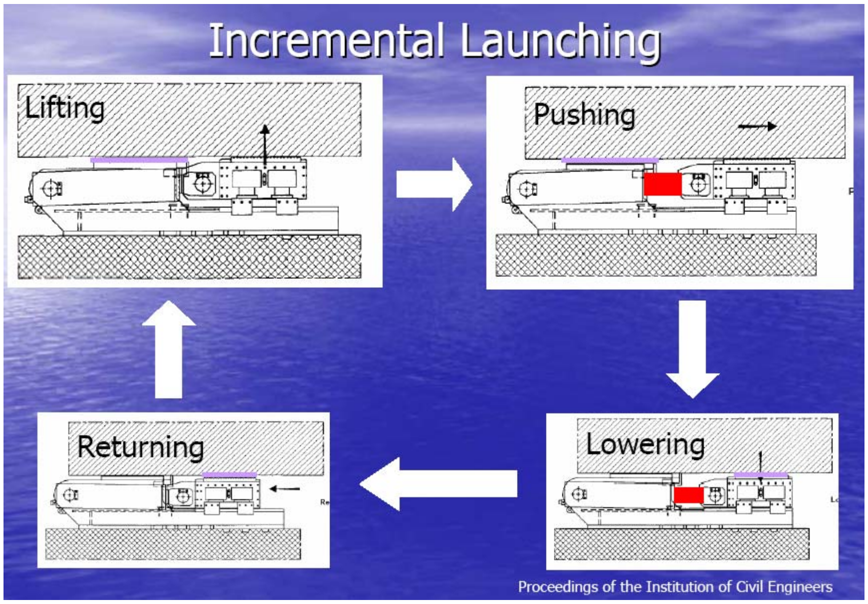


Fig. 12 — Varying shimming of the launching nose bottom flanges.



TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
Woronora River Bridge New South Wales, Australia Construction Completed 2001	Superstructure 10 – span single-cell prestressed concrete box girder	Substructure CIP concrete abutments and piers	$44.8 million (Australia)

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Roads and Traffic Authority of New South Wales	Structural design: RTA & Taylor & Herbert Consultants Pty. Ltd. Field service and design: PERI Australia	Contractor: Barclay Mowlem Pty. Ltd. Launching: Leonhardt Andra & Partner

BACKGROUND

The 521 m (1709 ft) long Woronora River Bridge connects the suburbs of Sutherland and Menai with the southern part of Sydney, Australia. At the time of its construction, it was the largest incrementally launched bridge in Australia. A downhill grade of 4.7 percent also makes this bridge one of the steepest incrementally launched bridges in the world. The horizontal alignment consists of a 450 m (1476 ft) radius curve which extends the entire length of the bridge.

BRIDGE DESCRIPTION

The bridge a 521 m (1709 ft) x 19.6 m (64.3 ft), 4 m (13 ft) deep single cell prestressed concrete box structure consisting of 10 spans with varying span lengths: 36 m (118 ft), 47 m (154 ft), 6 x 58.7 m (193 ft), 49 m (161 ft) and 36 m (118 ft). It provides 4 traffic lanes with an additional lane provided at each end of the bridge for left turning traffic. In addition to traffic lanes, a 3.5 m (11.5 ft) wide pedestrian lane was provided as a suspended structure beneath the northern superstructure cantilever. The superstructure is supported by 9 hollow piers that are up to 36 m tall. These piers are supported on piled foundations or spread footings and reduce in size towards the top with a side elevation taper of 1:100.

BRIDGE CONSTRUCTION AND LAUNCHING

Two 30 m long section installations were used for the superstructure construction. The base slab and webs of the first section were constructed first, followed by the concreting of the roadway slab in the next cycle. The sections were launched over the piers using hydraulic jacks on a weekly cycle. Due to site constraints, construction involved incremental launching on a downhill slope of 4.7 percent that caused the biggest obstacle for the contractor. Because of this large downhill grade and to ensure maximum control of the launch, the superstructure was launched using a cable braking/launching system, which resembles a heavy lift system; this system used modified prestressing jacks and cables that are comprised of prestressing strands. In this use the cables and jacks were rotated to be parallel to the soffit of the box girder. Three fixed prestressed cables were installed between the casting bed and the abutment and heavy jacks were installed at each cable. The two outside jacks provided the braking force while the central jack provided the launching force.

Typically, Australian bridge launches use temporary bearings during launching operation that are later replaced with permanent pot bearings once the last segment is launched. The replacement of bearings was avoided in this project by using permanent laminated elastomeric bearings that are capable to deform transversely and longitudinally in all directions. Piers and launching bearings were continuously monitored during the launching operation to prevent them from being overloaded.

NOTE

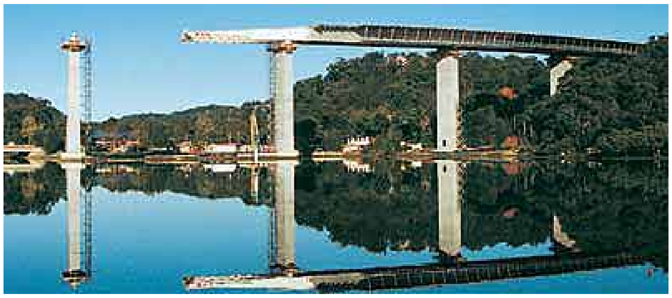
The construction process was completed without any major impediments except the steep downhill. The spans were cast in two separate segments due to their large size. Two separate casting beds were used to speed up the casting process. In the first casting bed, the bottom flange and webs were constructed and precast ribs installed. In the second casting bed, the top flange was constructed and the prestressing operations were carried out.

The Woronora River Bridge is one of the largest incrementally launched bridges in Australia. By using the incremental launch method, the need for scaffolding was eliminated. However, the challenge of downhill launching required exact design engineering and absolute precision during the construction and launching operations. The safety outcome on the project was thought to be good given the risks that needed to be managed on an incrementally launched system of such complexity, size and nature. After launching the 521-m (1709-ft) long superstructure, which was carried out 36 m (118 ft) above water, the $44.8 M (Australia) project reached its destination within accuracy of 2 mm (0.08 in.).

REFERENCE

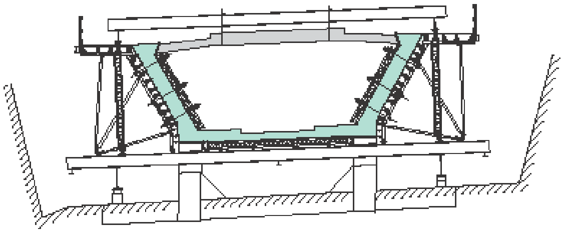
Bennett, M. and Taylor, A., “Woronoroa River Bridge, Sydney”, Structural Engineering International, Vol. 12, No. 1, February 2002.

PROJECT PHOTOS




Three sets of VARIO fitted with 18 mm Fin-Ply was enough to construct the piers in Class 2 fair-face concrete to Australian standards.

PERI supplied an ideal formwork concept. It led to a fast and economical completion of the bridge pillars.



After erecting the internal web formwork the precast reinforced concrete beams were concreted with the trough. Then it was pushed hydraulically by one section length.

Placing the precast reinforced concrete beams and assembling the internal diaphragm formwork in casting bed 1.

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
Bandera Bridge Slovenia Construction Completed 1995	<u>Superstructure</u> 9 & 8 – span prestressed concrete box girders	<u>Substructure</u> CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Republic of Slovenia	Viktor Markelj, Ponting Inc., Maribor	SGP Primorje, Ajdovscina

BACKGROUND

The Bandera Bridge, located on the Ljubljana-Trieste Highway, is the first externally prestressed concrete bridge that was erected by incremental launching in Slovenia. The bridge has a horizontal curve of approximately 1500 m (4921 ft) in radius and a longitudinal inclination of 5 percent.

BRIDGE DESCRIPTION

The Bandera Bridge consists of two separate viaducts, both with varying span lengths: one with 9 spans (2 x 24 m {78.7 ft}, 6 x 33.6 m {110.2 ft} and 22.8 m {74.8 ft}) and the other with 8 spans (2 x 24 m {78.7 ft} and 6 x 33.6 m {110.2 ft}). Each viaduct is an externally prestressed hollow concrete box girder bridge and is 13.72 m (45 ft) wide.

BRIDGE CONSTRUCTION AND LAUNCHING

The construction of each individual 16.80 m (55.1 ft) long box girder segment took approximately one week. The box girders were prestressed with straight bonded strands in the upper and lower slabs. External polygonal strands run the inside the box. The box girder was designed to be a trapezoidal form with two cantilevers that are 3.23 m (10.6 ft) long extending from each side of the girder. The thickness of the webs and of the upper slab of the box girder was designed to be constant for simplified, efficient fabrication and launching.

The bridge was launched, using a cable braking system, over the piers toward the lower abutment. The highest and lowest friction coefficients of 0.080 and 0.015, respectively, were used in dimensioning the pushing and braking devices. During launching of the girder, the displacements of the column heads were monitored for correct alignment.

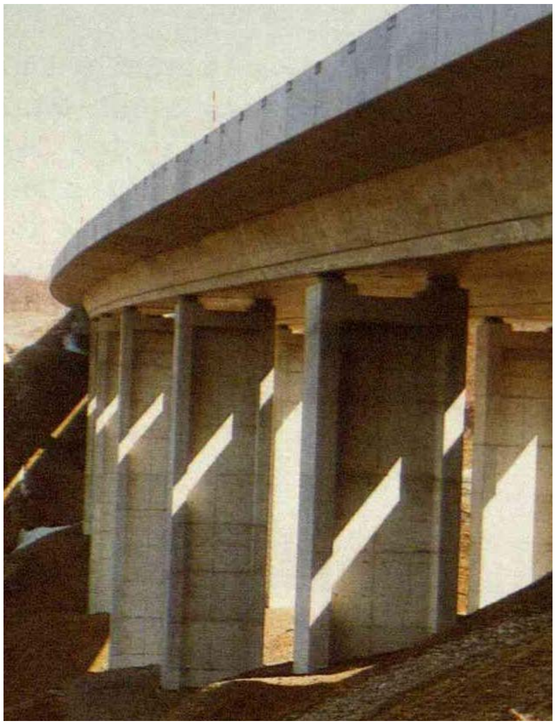
NOTE

The Bandera Bridge is a good example of a cost-effective bridge that was erected by incremental launching in Slovenia. The bridge was fabricated and launched in a very short period of time without any significant issues. The use of bonded straight internal tendons and additional external unbounded tendons in the box girder allowed for simplified execution and rationalization of the structure. Static and dynamic tests performed on the bridge verified that the bridge behavior was as expected.

REFERENCE

Saje, F. and Markelj, V., “Bandera Bridge, Slovenia”, Structural Engineering International, Vol. 7, No. 1, February 1997.

PROJECT PHOTOS



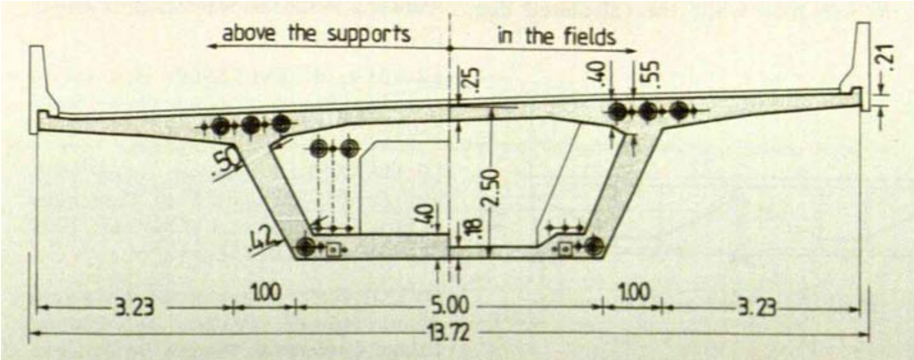


Fig. 2: Box girder cross section

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
San Cristobal Bridge Chiapas, Mexico Construction Completed February 2006	Superstructure 3 spans - curved steel composite and orthotropic box girders	Substructure CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Mexican Secretary of Communication and Transportation	T.Y. Lin International	Ingenieros Civiles Asociados – ICA T.Y. Lin International

BACKGROUND

The San Cristobal Bridge is a 3-span, continuous curved steel composite and orthotropic box girder bridge that crosses a deep canyon. This bridge provides an important link between the cities of Tuxtla-Gutierrez and San Cristobal. The steep topography across the deep canyon made cast-in-place construction questionable and the designer decided that the incremental launching of the superstructure from both sides of the canyon would be an economical solution. Initial construction of the bridge began in early 2003. Shortly after all segments had been launched, the structure on the Tuxtla-Gutierrez side collapsed while the San Cristobal side of the bridge remained erect. After the collapse, T.Y. Lin International was hired by the new contractor (Ingenieros Cliviles Asociados) to investigate the cause of the collapse, to redesign the structure, and to complete the erection.

BRIDGE DESCRIPTION

The 323-m (1060-ft) long bridge consists of thee spans with a 180-m (591-ft) interior span and two 71.5-m (234.6-ft) end spans. The superstructure is supported on two intermediate piers and two end abutments. The structural system of the deck is comprised of an unconventional mix of orthotropic steel deck segments with a composite post-tensioned box girder. The central portion of the main span is comprised of lighter orthotropic box girder segments while the rest of the main span and the end spans consist of heavier composite (concrete-steel) box girder segments. These variations were intended to prevent the overturning and uplift at the abutments, and to provide stability during launching.

CAUSE OF COLLAPSE AND CORRECTIONS MADE

The site investigation led to a conclusion that the primary cause of the collapse was due to the failure of the shear connectors, which were inadequately designed and poorly welded to the top flange. This resulted in the loss of composite action of the girder cross section over the pier on the Tuxtla-Gutierrez side.

Before the re-construction was begun, damage assessment was performed. Significant delamination of the concrete slab was found on the Tuxtla-Gutierrez side while noticeable cracks were found on both sides. In addition, one of the piers had substantial damage from the collapse. Several alterations and modifications were made to ensure the safe erection of the structure including the addition of shear studs, increasing deck post-tensioning during launching, increasing the concrete slab strength, and the addition of plate stiffeners to the bottom flange and webs.

BRIDGE CONSTRUCTION AND LAUNCHING

The bridge was launched from both abutments, half of the bridge from the Tuxtla-Gutierrez side and the other half from the San Cristobal side; the segments were subsequently connected at midspan. The launching sequence used during the second launch was similar to the original launching sequence. The San Cristobal side was partially pulled back to make necessary alterations on the box girders, after which the bridge was launched back to its final position. The project encountered minor impediments on the San Cristobal side when the cantilever was launched back out to the piers. Minor cracks in the concrete slab caused significant deflection in the cantilever. This problem was corrected by raising the abutment supports and inducing rigid body rotations of the deck to match the elevations at both ends.

The new segments on the Textla-Gutierrez side were fabricated and assembled directly behind the abutment. Limited space on the launching platform forced the contractor to assemble and launch simultaneously.

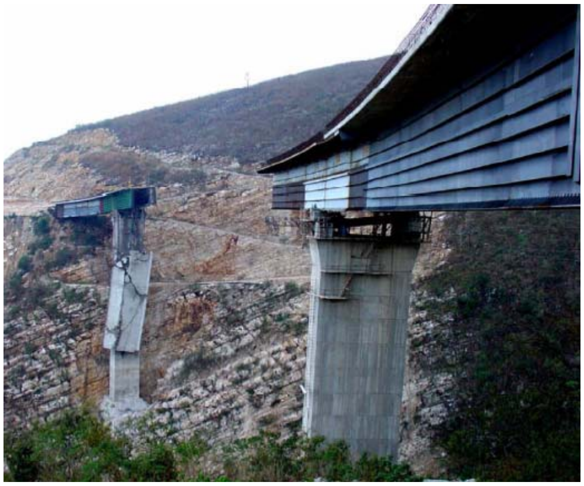
The launching of the composite segments with the concrete deck already cast in place could cause large negative moments to the composite segments. Extra longitudinal post-tensioning was, therefore, provided to overcome these negative moments introduced by this unusual combination, and to prevent tension and cracking of the concrete slab.

NOTE

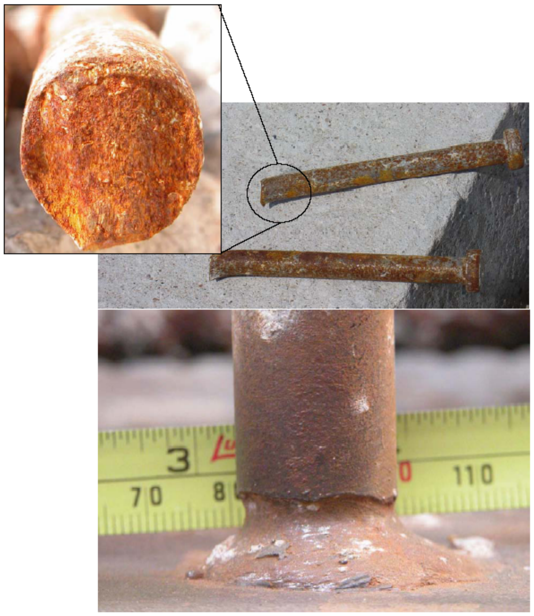
The San Cristobal Bridge was constructed using incremental launching methods mainly due to the steep topography at the site. The launching of a composite section with post-tensioned slab, however, appeared to be not practical method because of the complexities and uncertainties involved in the actual stress distribution and effective width of the slab. Even with a careful analysis and control of the loads, the bridge experienced some cracking in the slab and deflections that were larger than anticipated.

REFERENCE
Nader, M., Manzanarez, R., Lopez-Jara, J., De La Mora, C., "Launching of the San Cristobal Bridge", Proceedings from the Transportation Research Board, 2007

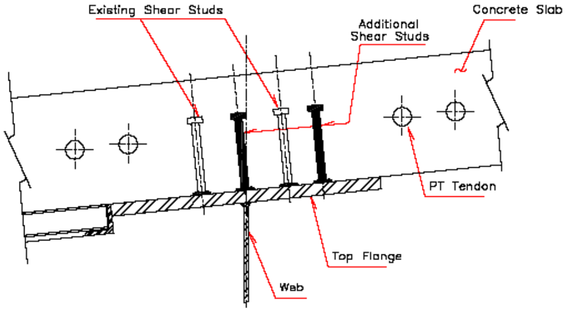
PROJECT PHOTOS



Collapsed structure on the Tuxtla-Gutierrez side



Failed shear studs and poor welding of shear studs on top flange



Existing and additional shear studs

Completed structure of the new San Cristobal Bridge

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
	Superstructure	Substructure	
Ile Falcon Bridge Valais, Switzerland Construction Completed 1998 & 1999	15 – span Prestressed concrete box girder	CIP concrete abutments and piers	Constr. Cost = $20M (USD)

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
N/A	SD Ingenierie Deneriaz & Pralong Sion Bureau d'ingenieurs SA Andenmatten SA	Ambrosetti & Zschokke Freyssinet SA

BACKGROUND

The Ile Falcon highway bridge is two prestressed concrete box girders that cross the Rhone River in the mountainous Valais region of Switzerland. The project involved the construction of two parallel 720 m (2400 ft) long curved bridges that are similar in design and construction. The construction of each bridge was completed in 1998 (north bridge) and 1999 (south bridge). Only the construction of the north bridge is summarized herein.

BRIDGE DESCRIPTION

The bridge consists of 15 spans and features varying girder depths, span lengths, and top slab widths. The girder is 2.15 m (7.1 ft) deep at the abutments and has the maximum depth of 3.7 m (12 ft). The span lengths varies from 27.4 m (90 ft) at the bridge ends to 73 m (240 ft) in the central portion that crosses the river. The superstructure is supported by 5-m (16-ft) diameter circular piers that were designed to also provide lateral stability during both launching and service. Fixed bearings were used at the middle piers to stabilize the bridge in the longitudinal direction.

BRIDGE CONSTRUCTION AND LAUNCHING

Bridge launching operations were carried out in 41 weekly stages with a standard launching stage of 18.25 m (60 ft) long. A U-shaped channel for the next stage was case each week along with the top slab of the previous stage and the parapets of the before-last stage. Due to the varying web depth, the channel and the top slab were cast with offsets.

The launching operation involved downhill launching due to the road transitions adjacent to a tunnel at the lower western abutment; this was also done to reduce the launching forces. Because the girder was launched from the higher east abutment to the lower west abutment, the project team developed a system that was capable of both pushing the bridge to initiate the sliding and holding it back once the initial friction resistance at the launching saddles is overcome (i.e., a braking mechanism). The holding mechanism was required due to the kinematic friction coefficient being significantly less than the downhill slope. In the last launching stages, engineers artificially increased the friction at the launching saddles by using timber plates at some launching saddles.

During the launching operations, the project team used a lateral guiding system that provided guiding forces perpendicular to the girder axis (i.e., deviating forces for the axial launching force in the girder) to keep the curved girder on the correct alignment. This lateral guiding system was installed on the permanent piers at approximately 250 m (820 ft) intervals.

NOTE

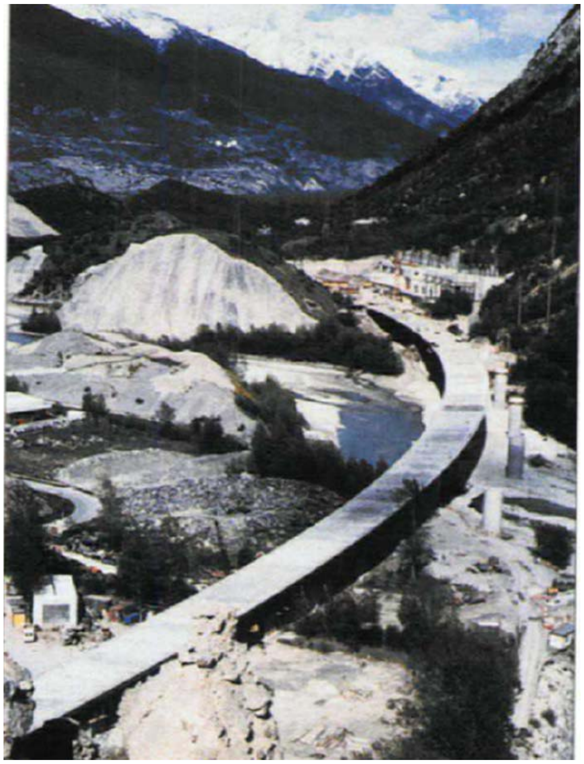
One of the main challenges of the project was to comply with unusually complex geometry of the superstructure. This geometrical complexity required significant attention during both the design and construction stages to define and implement the correct geometry. Equally critical was the rigorous topographical control of the casting bed, particularly because the launching had a large number of casting/launching stages. The Ile Falcon bridge project demonstrated that incremental launching can be a viable construction method for a curved bridge if proper attention and rigorous quality control are involved. Overall, the project proceeded smoothly and proved to be economical when compared to alternative construction methods. The bridge construction was completed on time and within the anticipated budget.

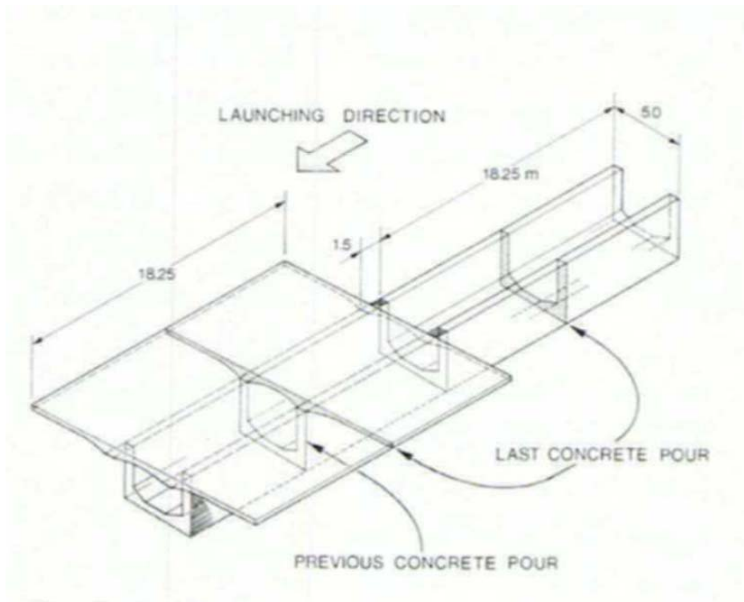
REFERENCE

Favre, R., Badoux, M., Burdet, O., Laurencet, P., "Incremental Launching fo the Ile Falcon Bridge", Concrete International, Vol. 21, Issue 2, February 1999.

Favre, R., Badoux, M., Burdet, O., Laurencet, P., "Design of a Curved Incrementally Launched Bridge", Structural Engineering International, Vol. 9, Issue 2, May 1999.

PROJECT PHOTOS


Bridge under Construction



Bridge casting sequence


Permanent and temporary piers of the north bridge

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
	Superstructure	Substructure	
Panval Nadhi Viaduct India Construction Completed 1995	11 – span prestressed concrete box girder	CIP concrete abutments and piers	N/A

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Konkan Railway Corporation Ltd.	Shrish Patel & Assoc. Ltd.	Larson & Toubro Ltd. – ECC Group Wayss & Freytag AG, Germany

BACKGROUND

The 760 km long Konkan Railway in western India required the construction of 143 major bridges, 1670 minor bridges and 75 tunnels. One of these bridges is the Panval Nadhi Viaduct. With columns up to 65 m (213 ft) in height, this bridge is the tallest bridge on the Konkan Railway and is an essential link in the Konkan Railway. Due to the deep valley (20 to 60 m {66 to 197 ft}) that had to be crossed, commonly used cast-in-place erection methods were ruled out as a viable construction techniquen and an incremental launch method was chosen for the construction.

BRIDGE DESCRIPTION

The bridge consists of 11 spans with two 30-m (98.4-ft) exterior spans and nine 40-m (131.2-ft) interior spans. The superstructure is a continuous prestressed concrete box girder that supports the track and a cable duct, with a footpath on one side of the track. The continuous deck is supported on low-friction polytetraflouride (PTFE) bearings at the abutments and at the piers. All of the substructure elements are founded on solid rock. Each pier is 3.8 m (12.5 ft) wide at the cap level and has a hollow, tapered octagonal cross section with a constant wall thickness of 325 mm (12.8 in.). The superstructure is anchored at one abutment with expansion joints at the other. The piers and abutments were designed for primarily for transverse wind and earthquake-induced loads.

BRIDGE CONSTRUCTION AND LAUNCHING

A casting yard was located 80 m (262.5 ft) behind the abutment that was fitted with the expansion joint to assure alignment of the pre-cast girder. In order to control the exact alignment of the bottom of the box girder, 50-tonne (55-ton) capacity hydraulic jacks were used to support the girder at 5-m intervals. A 30 m (98.4 ft) long steel launching nose, connected to the lead segment, was used to reduce cantilever bending stresses during launching. Temporary sliding bearings, consisting of 30 mm (1.2 in.) thick machined steel plates covered with 4 mm (0.16 in.) thick stainless steel plates, were installed at the grade level of each pier and each abutment to facilitate launching.

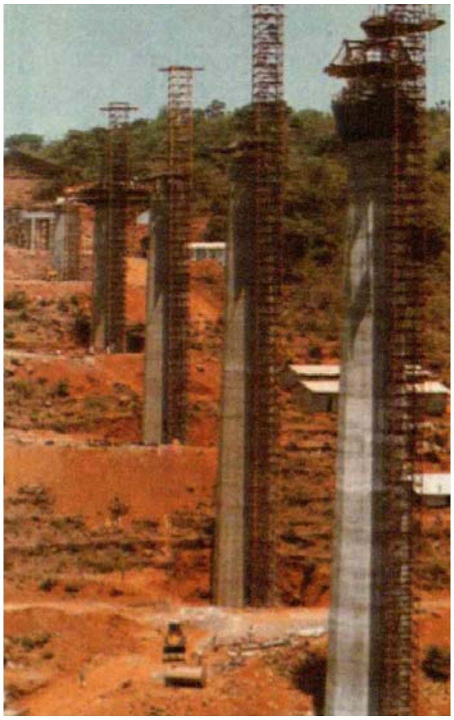
The bridge was launched across the valley from pier to pier using two prestressing jacks that were placed at the free abutment; the jacks reacted against a temporary A-frame anchored at the top of the abutment. When the jacks were activated, a set of prestressing strands, 15.2 mm (0.6 in.) in diameter, were locked at the rear end of the box girder by spreader beam. Once the launching force exceeded the static frictional force between stainless steel and PTFE layer, the box girder started sliding. Each jack had a strok of approximately 180 mm (7 in.).

NOTE

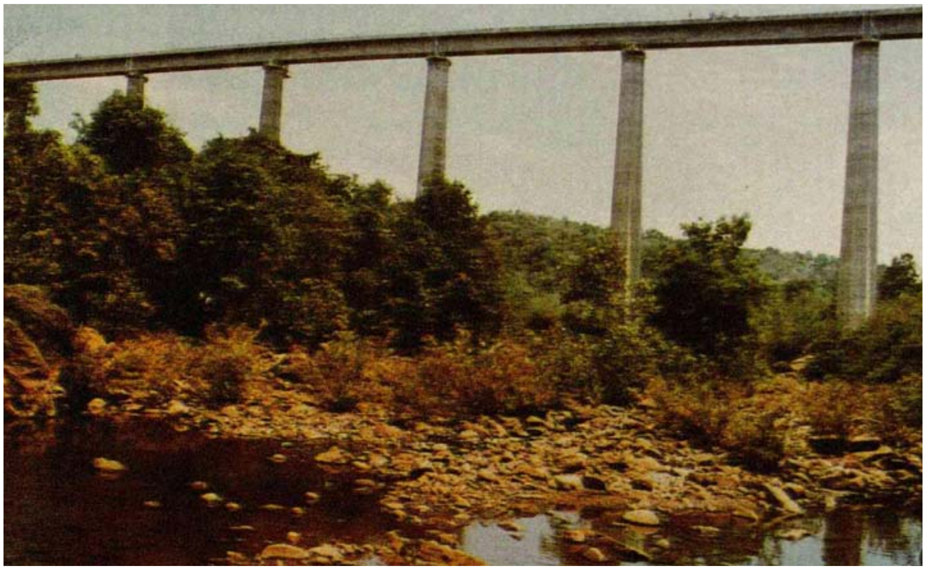
A high degree of accuracy was required to ensure that the box girder was aligned correctly because any unanticipated misalignment could cause secondary and unaccounted for stresses in the launched box girder. This accuracy was controlled by utilizing lateral guide bearings fixed every 10 m (32.8 ft) to temporary columns, abutments, and piers. The contract called for a maximum accepted disparity of 20 mm (0.8 in.) slope-out in the overall height of a pier. For the tallest pier, the tolerance was 0.00033 mm/m ($4.0x10^{-6}$ in./ft). This accuracy was achieved by close monitoring during launching operations. Overall, the construction proceeded smoothly and the entire bridge was fabricated and erected in time with no significant issues.

REFERENCE

Ramakrishna, A. and Sankaralingam, C., “Panval Nadhi Viaduct, India”, Structural Engineering International, Vol. 7, No. 3, August 1997.



Pier slipforming



Completed bridge

TITLE AND LOCATION *(City and State)*	BRIDGE TYPE AND MATERIAL		CONSTRUCTION COST
The Wabash River Bridge Covington, Indiana Construction Completed 1977	Superstructure 6 – span double-celled concrete box girder	Substructure CIP concrete abutments and piers	Superstructure = $1.67M (USD)

PROJECT PARTICIPANTS

PROJECT OWNER	PROJECT DESIGN FIRM	PROJECT CONTRACTOR
Indiana DOT Anne Rearick, P.E. 100 N. Senate Ave, Room N642 Indianapolis, IN 46204	VSL Corporation, Los Gatos, Calif.	Roger Construction Co. Weddle Brothers Construction Co. Launching: VSL Corporation

BACKGROUND

The Wabash River Bridge, a replacement for a structural steel bridge build in 1915, is a part of the US Route 136, located in the vicinity of Covington, Indiana, approximately 128.7 km (80 miles) west of Indianapolis, Indiana. It is thought to be the first incrementally launched concrete box girder bridge to be built in the US. Its construction involved the use of a launching nose, temporary bridge supports, and hydraulic launching jacks to advance the girders from the fabrication area across the river.

BRIDGE DESCRIPTION

The Wabash River Bridge is a 285 m (935 ft) by 14.17 m (46.5 ft) double-celled concrete box girder structure with two 28.5 m (93.5 ft) end spans and four 57 m (187 ft) interior spans. The box girders are 2.4 m (8 ft) in depth and are supported by 6.1 m (20 ft) wide, 2 m (6.5 ft) thick, and approximately 12.2 m (40 ft) high solid wall piers that are founded on solid rock. The structure is straight and level both in elevation and in plan. The piers are skewed 10 degree while the abutments are orthogonal (not skewed).

BRIDGE CONSTRUCTION AND LAUNCHING

The superstructure was constructed in 20 segments with each 14.25 m (46.8 ft) segment constructed in two stages; specifically, the bottom slab was constructed first followed by the webs and deck slab. Post-tensioning of the various elements was carried out once the concrete strength reached 24.1 MPa (3.5 ksi). Four temporary piers were used in the interior spans during launching, dividing the structure into 10 equal spans of 28.5 m (93.5 ft). On average, it took 2.5 hours to launch one 14.25 m (46.8 ft) segment.

Two hydraulic jacks (one for horizontal sliding and the other for vertical lifting with the capacity of 2670 kN [300 ton] and 3560 kN [400 ton], respectively), designed and manufactured by VSL, were used for launching. A horizontal jack was connected to the casting bed and the abutment on one end and to the vertical jack at the other end.

Two cross-braced structural steel plate girders attached to the cantilever end of the first segment were used as a launching nose. Due to the axis of the piers not being perpendicular to the center line of the bridge, these plate girders were fabricated in two different lengths (16.5 m [54 ft]and 17.7 m [58 ft]) so that they would reach the pier at the same time.

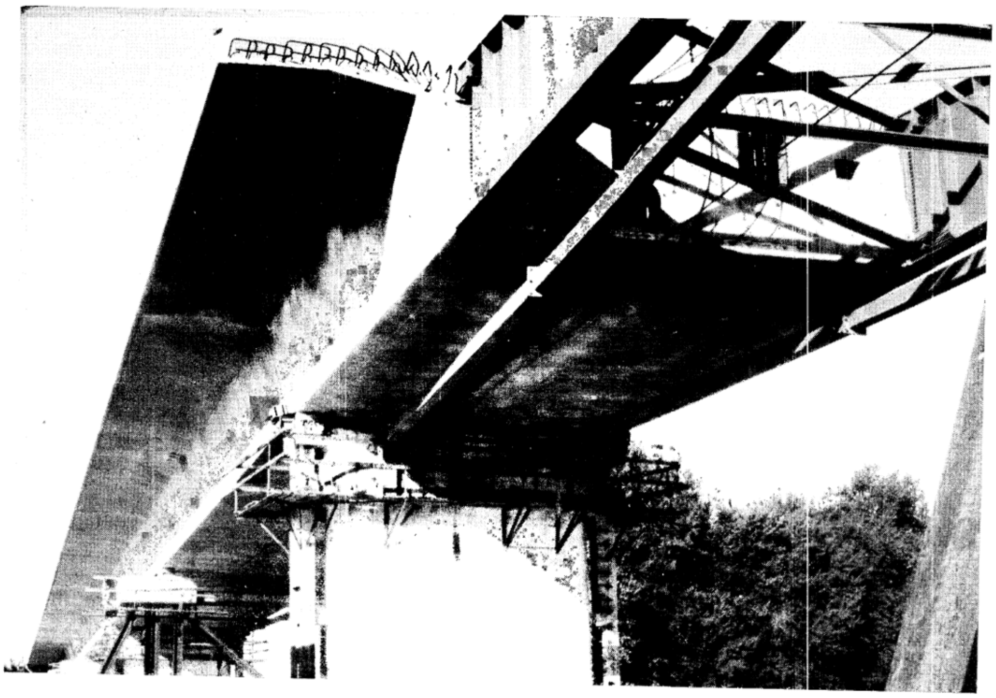
On each pier, permanent bridge bearings were placed 1 in. below the final elevation such that the superstructure would pass over them. These bearings were raised and welded onto steel plates after the bridge was set into its final position.

NOTE

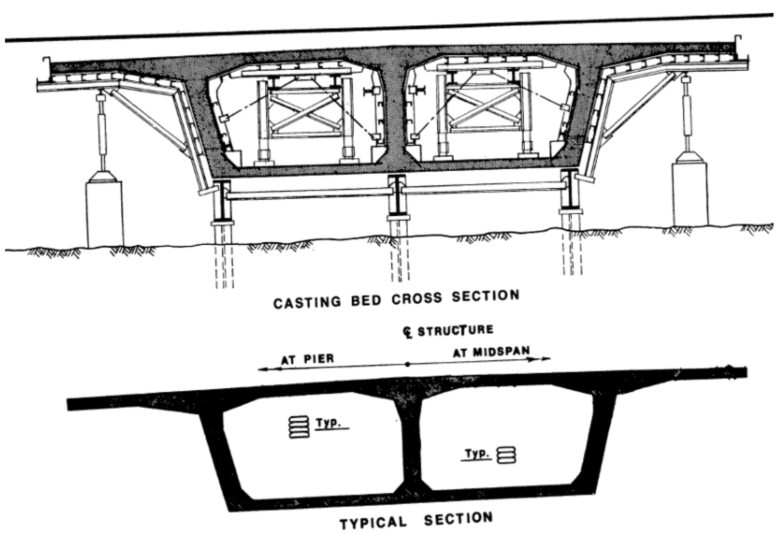
Four different designs were initially considered: a precast cantilever method (original design), an incrementally launching method, segmental construction on falssework, and the commonly used cast-in-place method. Some unpredictable characteristics of the Wabash River ruled had some influence in selection of the erection method. Although the bridge was to be constructed 11 m (36 ft) above the mean water level, the Wabash River can rise considerably, as much as 6.1 m (20 ft) in 24 hours and can happen almost any time of the year. Thus, the conventional construction methods were thought to cause some danger/risk and were therefore ruled out. In addition, among the four alternatives, the incremental launching method, proposed by VSL, turned out to be the most economical with the cost saving of approximately $200,000 over the precast cantilever method.

REFERENCE

Swanson, David T, “Launching a Concrete Bridge Saves $200,000”, Concrete International, Vol. 1, Issue 4, April 1979.



Steel plate girder launching nose attached to first bridge segment



Bridge casting bed cross section

STRATEGIC PLAN FOR INCREASING USE OF INCREMENTAL LAUNCHING METHOD

The implementation of research results, no matter how comprehensive and practical, is perhaps the most difficult part of any research project. In order to increase the application of an innovative construction process such as the ILM, the entire bridge community must be engaged for a variety of reasons. In order to construct a bridge over a challenging obstacle, the bridge owner must be committed to the additional risk and expense that a new or untested process entails. Bridge designers must begin to consider the construction method early in the design process and understand the additional analysis that will be required. Contractors, and their erection engineers, must be willing to work cooperatively with the design team to solve the problems that almost inevitably arise with a new process and design a launching system which is well-suited to existing or readily available specialty equipment such as jacks and rollers.

A recommended strategic plan to promote the wider use of the ILM consists of a number of approaches that, in concert, would be expected to increase the exposure of this bridge construction technology to a wider audience. It is recognized that incremental launching is not the ideal construction method for every bridge project. However, it is thought that a wider understanding of the applicability and potential benefits would allow potential owners, designers and contractors to make a well-informed decision as to its use for their upcoming projects. The elements of the recommended strategic plan are as follows:

- Organize an expert group of owners, designers and contractors with personal experience with bridge launching who would be willing to advise owners regarding the value and applicability of the launching method to their particular project. This program might be patterned after the ongoing FHWA Accelerated Construction Technology Transfer (ACTT) program which arranges a group of qualified experts in a wide-range of disciplines to present a multi-day workshop for a particular project. The workshop is used to promote brainstorming and develop critical recommendations for accelerated construction projects around the county.
- Establish a series of cooperative agreements with bridge-related technical organizations associations such as NSBA, PCI, ASBI and other similar groups to provide information and encourage the writing of technical papers and presentations at future national/regional meetings.
- Encourage the publication of practical case study-type articles in trade publications such as Civil Engineering, Engineering News Record and Concrete International. One potential location for wide-spread exposure to the industry is the new PCI Aspire magazine which is distributed free of charge to the target audience of bridge owners, designers and contractors. It should be noted that an article has just been published in the October 2007 issue of Concrete International entitled “Launch and Shift of the Tiziano Bridge”. This article provides a detailed case study of a twin-concrete box girder bridge in which the first girder was launched and then slid transversely to permit the subsequent launch of a second parallel girder using the same equipment.
- In the past few months, a number of presentations have been made at regional and national conferences to address the growing interest in using the incremental launching method. Each of these presentations was well-attended and generated much

interest among state DOT engineers. These presentations include the following:

  - A series of “brown bag” presentations was delivered to the Utah and Oregon DOT bridge engineering staff
  - An eight hour seminar devoted exclusively to bridge construction by incremental launching was presented at an ASCE conference in Sacramento in September 2007.
  - A presentation entitled “Incremental Launching of Bridges in Europe” was delivered at the Western Bridge Engineers Seminar in Boise, ID in September, 2007
  - The design and construction of the innovative curved steel girder Kicking Horse Canyon Bridge was presented at the World Steel Bridge Symposium in December 2007

- Secure the assistance of specialty equipment manufacturers such as Hilman, VSL, Freyssinet, Enerpac and others to provide additional examples, details and technical assistance to support the use of incremental launching for appropriate project locations.
- Promote cross-collaboration between concurrent and closely related research projects. The research team for the current study has recently been contracted through the Strategic Highway Research Program to serve as co-investigators on project R04 Innovative Bridge Designs for Rapid Renewal. During this study, additional investigation of accelerated bridge construction techniques will be performed with the intent of developing design specifications for rapidly constructed bridges.
- Assist interested state DOT bridge owners in applying for funding for innovative bridge construction methods through the FHWA Innovative Bridge Research and Deployment program. This program has been established with the expressed intent of directing discretionary funding to projects which will yield tangible transportation and safety benefits.

It is anticipated that a combination of these efforts, as well as the publication of a technical paper based on the results of the current study will be effective in generating interest within the US bridge community to consider the incremental launching method for appropriate project sites.

REFERENCES

Baur, Willi, "Bridge Erection by Launching is Fast, Safe and Efficient", Civil Engineering – ASCE, Vol. 47, No. 3, March 1977.

Bergeron, J. and Laurie Sawicki, "The incremental adaptation of the Clifford Hollow Bridge", Structural Engineer, May 2002.

Bennett, M. and Taylor, A., "Woronoroa River Bridge, Sydney", Structural Engineering International, Vol. 12, No. 1, February 2002.

Durkee, Jackson L., "Railway Box-Girder Bridge Erected by Launching", Journal of the Structural Division, ASCE, Vol. 98, No. ST7, Proc. Paper 9028, July, 1972, pp. 1443-1463.

Durkee, Jackson, "Steel Bridge Construction", pgs 45-58, Bridge Engineering Handbook, CRC Press, 2000.

Engineering News Record, "Span Launched over Deep Gap", Engineering News Record, Vol. 241, No. 22, December 14, 1998.

Favre, R., Badoux, M., Burdet, O., Laurencet, P., "Incremental Launching fo the Ile Falcon Bridge", Concrete International, Vol. 21, Issue 2, February 1999.

Favre, R., Badoux, M., Burdet, O., Laurencet, P., "Design of a Curved Incrementally Launched Bridge", Structural Engineering International, Vol. 9, Issue 2, May 1999.

Gohler, Bernhard and Pearson, P., Incrementally Launched Bridges Design and Construction, Ernst and Sohn, Berlin, 2000.

Granath, P., "Distribution of support reaction against a steel girder on a launching shoe." Journal of Constructional Steel Research, Vol. 47, No. 3, pp. 245-270, 1998.

Granath, P., "Serviceability limit state of I-shaped steel girders subjected to patch loading", Journal of Constructional Steel Research, Vol. 54, No. 3/2000(A), pp. 387-408.

Granath, P., Anders Thorsson, and Bo Edlund, "I-shaped steel girder subjected to bending moment and travelling path loading", Journal of Constructional Steel Research, Vol. 54, No. 3, June 2000(B), pp. 409-421.

Hewson, N. and Hodgkinson, A., "Incremental Launch of Brides Glen Bridge, Ireland", Concrete, Vol. 38, Issue 7, 2004

LaViolette, M., "Pushing", Structural Engineer, May 2003.

LaViolette, M., McDonald, D., “Landmark Launch”, Modern Steel Construction, February 2004.
Lebet, J., “Composite Construction in Steel and Concrete V”, Proceedings of the 5th International Conference, July 2006.

Llombart, J. A.and Jordi Revoltos, “Petra Tou Romiou Viaduct, Cyprus”, Journal of Structural Engineering International, Vol. 10, No. 4, pp. 233-234, November 2000.

Malite, M., Takeya, T., Goncalves, R. and Jairo de Sales, J., “Monitoring of the Parana River Bridge During Construction”, Structural Engineering International, March 2000, pg. 193-196.

Marzouk, Mohamed, Hisham Zein El-Dein and moheeb El-Said, “Application of computer simulation to construction of incrementally launching bridges”, Journal of Civil Engineering and Management, Vol. 13, No. 1, 2007, pp. 27-36.

McGarth, R., “Concrete Thinking in Engineered Structures Stoney Trail Bow river Bridge”, Cement Association of Canada, 2002.

McLaughlin, Mike., "The practical and portable British Bailey Bridge helped Allied troops remain on the march." *Military Heritage Presents: WWII History*, pp. 10-15, 76, 2005.

Monsarrat, C.N., “Erection of French River Bridge – Canadian Pacific Railway”, The Canadian Engineer, June 5, 1908, pg. 400-404.

Nader, M., Manzanarez, R., Lopez-Jara, J., De La Mora, C., “Launching of the San Cristobal Bridge”, Proceedings from the Transportation Research Board, 2007

Nahawi, K., and Banchik, C., “North Halawa Valley Viaduct Design and Construction”, Journal of Concrete International, Vol. 16, No. 2, pp. 39-43, February 1994.

Paul, Alistair, “Large and Small Incrementally Launched Structures”, Transportation Research Record 1696, Paper No. 5B0060, Transportation Research Board, Washington D.C., 2000.

Podolny, Walter and Jean M. Muller, Construction and Design of Prestressed Concrete Segmental Bridges, John Wiley and Sons, 1982.

Ramakrishna, A. and Sankaralingam, C., “Panval Nadhi Viaduct, India”, Structural Engineering International, Vol. 7, No. 3, August 1997.

Reina, P., “London Crews Launch Spans over Tracks with Little Room”, Engineering News Record, Vol. 255, No. 2, July 18, 2005.

Rogowski, D., “Green Giant”, Bridge Builder, January-March 2003.

Rosignoli, M., “Creep Effects During Launch of the Serio River Bridge”, Concrete International, Vol. 22, Issue 3, March 2000

Rosignoli, Marco, Launched Bridges: Prestressed Concrete Bridges Built on the Ground and Launched into Their Final Position, ASCE Press, 1998(A).

Rosignoli, M., “Site Restrictions Challenge Bridge Design”, Concrete International, Vol. 20, No. 8, August 1998(B).

Rosignoli, M. “Presizing of Prestressed Concrete Launched Bridges”. ACI Structural Journal, Title No. 96-S77, Vol. 96, No. 5, Sept-Oct 1999(A), pp. 705-711.

Rosignoli, M. “Nose-Deck Interaction in Launched Prestressed Concrete Bridges”. Journal of Bridge Engineering, February 1998(C). pp. 21-27.

Rosignoli, M. “Reduced-Transfer-Matrix Method of Analysis of Launched Bridges”. ACI Structural Journal, Vol. 96, No. 4. July-August 1999(B). pp. 603-608.

Rosignoli, M.,“Monolithic Launch of the Reggiolo Overpass”, Concrete International, Vol. 29, No. 2, February 2001.

Rosignoli, Marco. Bridge Launching. Thomas Telford Ltd, Parma, Italy, 2002.

Rosignoli, M. and Rosignoli, C., “Launch and Shift of the Tiziano Bridge”, Concrete International, American Concrete Institute, October 2007, pp. 44-49.

Saje, F. and Markelj, V., “Bandera Bridge, Slovenia”, Structural Engineering International, Vol. 7, No. 1, February 1997.

Skeet, J., Lester, W., McClary, C., “Incremental Launch: The Stoney Trail Bridge:, Concrete International, Vol. 20, Issue 2, February 1998.

Svensson, H.S., “Incremental Launching of Steel Bridges”, paper presented at World Steel Bridge Symposium, Chicago, IL, 2001.

Swanson, David T, “Launching a Concrete Bridge Saves $200,000”, Concrete International, Vol. 1, Issue 4, April 1979.

VSL Technical Report, ‘The Incremental Launching Method in Prestressed Concrete Bridge Construction”, April 1977, VSL International Ltd., Berne, Switzerland.

Wipf, T.J., B.M. Phares, R.E. Abendroth, B. Chang and S. Abraham, “Monitoring of the Launched Girder Bridge over the Iowa River on US 20”, Final Report CTRE Project 01-108, Center for Transportation Research and Education, Iowa State University, Ames, IA 50010, March 2004.

Zhuravov, L.N., O.I. Chemerinsky and V.A. Seliverstov, “Launching Steel Bridges in Russia”, Journal of International Association for Bridge and Structural Engineering (IABSE), Vol. 6, No. 3, 1996, pp. 183-186.

APPENDIX A

Database of Incrementally Launched Bridges

Table A.1 provides an information summary of the bridges that have been previously described within this report in both the literature review and the case study sections. Following Table A.1 is information regarding an online database for launched bridges from around the world that will provide additional information.

Table A.1. Launched bridge information

Name	Location	Year Built	Featured Crossed	Superstructure Type	Function / Usage	Contractor	Designer	Owner
U.S. 20 Iowa River Bridge	Steamboat Rock, Hardin County, Iowa U.S.A.	2002	Iowa River Valley	Steel I-girder	Road bridge	Jensen Construction	HNTB Corporation	Iowa Department of Transportation
Stoney Trail Bridge	Calgary, Alberta, Canada	1997	Bow River	Double-celled concrete box girder	Road bridge	Walter & SCI Construction (Canada) Ltd.	J.R. Spronken & Associates Ltd.	City of Calgary
Brides Glen Bridge	Dublin, Ireland	2003	Brides Glen Valley	2-Post-tensioned concrete box girders	Road bridge	Main contr: ASCON; Sub-contr: VSL Systems (U.K.) Ltd. and Tony Gee and Partners	Roughan and O'Donovan; Tony Gee and Partners	N/A
Vaux Viaduct	A1 Highway; Vaud, Switzerland	1999	Vaux Valley	2-Steel-concrete composite girder bridges	Road bridge	Steel: Zwahlen & Mayr SA; Prestressing: VSL International; Pot bearings and expansion joints: Mageba SA	Giacomini & Joliet; Realin & Bader SA	Etat de Vaud
Serio River Bridge	Bergamo, Italy	N/A	Serio River	Double-cell precast box girder	N/A	N/A	N/A	N/A
Woronora River Bridge	New South Wales, Australia	2001	Woronora River	Single-cell prestressed concrete box girder	Road bridge	Contr: Barclay Mowlem Pty. Ltd; Launching: Leonhardt Andra & Partner	Structural: RTA & Taylor & Herbert Consultants Pty. Ltd.; Field: PERI Australia	Roads and Traffic Authority of New South Wales
Bandera Bridge	Ljubljana-Trieste Highway, Slovenia	1995	Natural valley	2-Externally prestressed concrete box girder2	Road bridge	SGP Primorje; Ajdovscina	Viktor Markelj, Ponting Inc., Maribor	Republic of Slovenia

N/A= Information not available

Table A.1 (continued). Launched bridge information

Name	Location	Year Built	Featured Crossed	Superstructure Type	Function / Usage	Contractor	Designer	Owner
San Cristobal Bridge	Chiapas, Mexico	2006	Chentic Creek Canyon	Curved steel composite and orthotropic box girder	Road bridge	Final Contr: Ingenieros Civiles Asociados	Final Designer: T.Y. Lin International	Mexican Secretary of Communication and Transportation
Ile Falcon Bridge	Valais, Switzerland	1998 & 1999	Rhone River	2-Curved prestressed concrete box girders	Road bridge	Ambrosetti & Zschokke; Freyssinet SA	SD Ingenierie Deneriaz & Pralong Sion; Bureau d'ingenieurs SA; Andenmatten SA	N/A
Panval Nadhi Viaduct	Konkan Railway, western India	1995	Panval Nadia Valley	Prestressed concrete box girder	Railway bridge	Larson & Toubro Ltd. ECC Group; Wayss & Freytag AG, Germany	Shrish Patel & Assoc. Ltd.	Konkan Railway Corporation Ltd.
Wabash River Bridge	Covington, Indiana, U.S.A.	1977	Wabash River	Double-cell prestressed concrete box girder	Road bridge	Roger Construction Co.; Weddle Brothers Construction Co.	VSL Corporation, Los Gatos, Calif.	Indiana Department of Transportation
Clifford Hollow Bridge	Moorefield, West Virginia	N/A	N/A	Steel I-girder bridge	Road bridge	Dick Corporation	Parsons; HDR Engineering	West Virginia Department of Transportation
Palizzi Overpass	Milan, Italy	N/A	Six-lane railway	Prestressed concrete box girder	Road & tramway bridge	Bonatti SpA.	Marco Rosignoli	Milan Underground Railway Authority
Parana River Bridge	Brazil	N/A	Parana River	Two welded truss beams w/box cross sections	Road & railway bridge	N/A	N/A	N/A
Reggiolo Overpass	Reggiolo, Italy	2003	Verona-Mantua railway	Multi-cellular prestressed concrete plate girder	Road bridge	N/A	N/A	N/A

N/A= Information not available

Table A.1 (continued). Launched bridge information

Name	Location	Year Built	Featured Crossed	Superstructure Type	Function/ Usage	Contractor	Designer	Owner
Petra Tou Romiou Viaduct	Limassol-Paphos Highway, Cyprus	2001	Natural valley	2-Post-tensioned mono-cellular concrete box girders	Road bridge	China Wanbao Eng. Corp. Beijing; MeKano4, Barcelona	EIPSA, Madrid	Republic of Cyprus, Public Works Department
Easton Bridge	Cascade Mountains, Washington, U.S.A.	N/A	Yakima River & Hall Creek	Steel I-girder	Pedestrian & biking bridge	Main contr: Boss Construction Co.; Sub-contr: Engineered Transport and Lifting Co.	N/A	N/A
Paddington Bridge	London, England (U.K.)	N/A	Railway & subway	Steel girder w/composite deck	Road bridge	Hochtief Construction Ltd. (U.K.)	Cass Hayward Ltd., Chepstow	Westminster City
Ravensbosch Viaduct	Maastricht and Heerlen Motorway, Netherlands	N/A	Valley of Strabekerv-loedgraaf	2-Single-cell post-tensioned concrete box girders	Road bridge	Internationale Gewapend Betonbouw; Societe Belge des Betons	Bouvy, van der Vlugt, van der Niet, Scheveningen	Provinciale Waterstaat Limburg Maastricht
Port Wakefeild Road	South Australia, Australia	N/A	Major highway	2-Single-cell prestressed concrete box girders	Road bridge	N/A	N/A	N/A
Blanchetown Bridge	Blanchetown, South Australia, Australia	N/A	Murray River	Single-cell post-tensioned concrete box girder	Road bridge	N/A	N/A	N/A

N/A= Information not available

During the completion of this work a comprehensive database related to bridge construction was identified. This database contains a specific subcategory of bridge construction related to launching bridges. The public database is located at http://en.structurae.de/structures/mtype/index.cfm?ID=3001. Several screen captures from the database are shown in Figures A.1 and A.2.

To view project information, the database is setup to allow a user to browse by 1) name; 2) structural type; 3) function; 4) construction method; 5) geographic location; and 6) year of completion.

The projects summarized in this report that were not previously contained in the Structurae database have been submitted to the webmaster for their entry into the database. A bridge owner/designer/contractor can contribute to the Structurae database by following the instructions on the website and filling out electronic submission forms or by sending pertinent data via email. The website does not accept anonymous submissions.

Presented below is a list of bridges that were not described within this report due to insufficient information and were not found within the existing database. The bridges were, however, briefly mentioned by several references (Rosignoli, 1998(A), Rosignoli 2002, and Gohler, 2000). The bridges are as follows:

- Ager Bridge, Austria
- Amiens Viaduct, France
- Boivre Viaduct, France
- Boivre Bridge, Poitou-Charente, France
- Bubiyan Bridge, Kuwait
- Canyon Creek, Idaho, USA (2006)
- Charix Viaduct, Rhone-Alpes, France
- Charolles Bridge, Charolles, France
- Dal Bridge, Avesta, Sweden
- Hamburg Bridge, Utrecht, Netherlands
- Juneau River, Juneau Alaska, USA (1999)
- Kicking Horse Canyon Bridge, Canada
- Kufstein Bridge, Germany
- Lawyers Creek, Idaho, USA
- Neckarburg Bridge, Baden-Württemberg, Germany
- Queets River Bridge, Washington State, USA (1991)
- Rio Caroni Bridge, Venezuela
- Sathorn Viaduct, Bangkok, Thailand
- Schnaittach Bridge, Germany
- Schrotetal Bridge, Germany
- Skye Bridge, Scotland
- Val Restel Bridge, Italy
- Veitschochheim Bridge, Bavaria, Germany
- Wandre Bridge, Belgium

- Yakima River Bridge, Washington State, USA (1999)
- Zilwaukee River, Michigan, USA(1984)

Lastly, a brief list of noteworthy bridges from other sources is presented, but again, insufficient information was found for report summaries. The bridges are as follows:

- Chiapas I Bridge, Chiapas, Mexico
- Damsumlo Bridge over Skeena River, Hazelton, British Columbia, Canada
- North Halawa Valley Bridge, Oahu Island, Hawaii
- Tai Po Bypass, Hong Kong

Name	Year	Location	Status
A43 Overpass	1993	Grenay (38)	in use
A5 Ceiriog Viaduct		Chirk	in use
Abend Viaduct	2006	Arenshausen (TH)	in use
Abéou Aqueduct	1968	Saint-Paul-lès-Durance (13)	in use
Aich Valley Bridge	1983	Aichtal (BW)	in use
Aiguilly Bridge	1982	Aiguilly (42)	in use
Albetal Viaduct		Siegen (NRW)	
Alconétar Viaduct	2006	Cáceres	in use
Åmål Motorway Bridge	2009	Åmål	under construction
Amolanas Bridge	2000	Chile	in use
Antonius Bridge	1982	Meschede (NRW)	in use
Aranda Viaduct	2000	Ricla	in use
Arnoya Viaduct	1998	Allariz	in use
Arsac Bridge		Andrézieux-Bouthéon (42)	in use
Atenquique, Puente (I)	1989	Mexico	in use
Atenquique, Puente (II)	1989	Mexico	in use
Avesta, Dal Bridge at	1972	Avesta	in use
Ayalon Bridge		Tel Aviv	
Bajer Bridge		Croatia	
Bardonnex Viaduct		Saint-Julien-en-Genevois (74)	in use
Beesedau, Saale River Bridge at	2000	Beesedau (ST)	in use
Berbke Viaduct		Arnsberg (NRW)	in use
Bergères Viaduct		Bourg-Lastic (19)	in use
Béziers, Pont de la rocade nord de		Béziers (34)	
Bhaira Bridge	1997	Bhaira	in use
Boyeros Viaduct	1990	Córdoba	
Bul-Jeong Bridge	2004	Mungyeong	under construction
Buxach Viaduct	1987	Buxheim (BY)	in use
Caguanas River Bridge	1991	Puerto Rico	in use
Calapa Bridge		Calapa	
Caracas Viaduct		La Guaira	under construction
Caroni River Bridge	1963	Puerto Ordaz	
Carrich Viaduct	1995	Kyle of Lochalsh	
Chalifert Viaduct	1993	Chalifert (77)	in use
Churreteles Viaduct	1990	Córdoba	
Cieszyn-Cesky Tesin Bridge	1992	Cieszyn (SL)	
Cissé Viaduct		Indre-et-Loire (37)	in use
Colo River Bridge		New South Wales	in use
Concejo Viaduct	1990	Córdoba	
Cortaceros Viaduct	1990	Córdoba	
Crould Viaduct		Goussainville (95)	in use

Figure A.1. Partial list of database projects


Structurae [en]: Data Submission: Submission of a new structure - Mozilla Firefox

File Edit View History Bookmarks Tools Help

http://en.structurae.de/submit/structures.cfm

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Data Submission / Submission of a new structure

Browse by:

- Forms for:
- A Structure
- A large-scale project
- A person
- A firm
- A product
- A publication
- A journal
- An external link
- Photos

Google Search

Structurae web

- Data submission
- About Structurae
- Advertising on Structurae
- Imprint
- Contact Structurae
- Acknowledgements
- Photos used on Structurae

Wallpaper

Click to download!

Submission of a new structure

Please use this form only to submit data for a structure which has not yet been added to the Structurae database. **This is not a search form nor a form to send message to the editors of Structurae**

Submission criteria:

1. Every structure such as a bridge, tower, dam, skyscraper, off-shore platform, stadium or tunnel
2. Every contemporary or historical structure that has an interest for professionals in the field of civil or structural engineering or construction
3. Every structure must have been built (even if demolished or destroyed since), be currently under construction or its construction reasonably certain in the near future.

The fields that definitely have to filled out are marked with a red asterisk *

Your Name: *

Your e-mail: *

Please add the data you would like to submit in the following fields:

Name of structure: *

Other names:

Other names under which the structure is or has been known, for example historical names or in the local language, etc.

Structural Type: *

Please describe in as much detail as possible the type of structure. For example: gravity-anchored suspension bridge with diagonal hangers and a deck truss, etc.

Function / usage: *

What function does this structure have? Is this a road bridge, a hydroelectric dam, a transmission tower, etc.?

Figure A.2. Example of electronic form for submitting project information

APPENDIX B

Survey of State DOT Bridge Engineers

Appendix B lists each question sent to the bridge engineering community. The response results received as well as the number of responses are also shown.

Are you personally familiar with the incremental launching method for bridge construction?	Response Percent	Response Count
Yes	**55.0%**	22
No	45.0%	18
answered question		**40**
skipped question		**0**

Do you know of anyone in your agency who is personally familiar with the incremental launching method for bridge construction?	Response Percent	Response Count
Yes	3.7%	1
No	**85.2%**	23
If yes, please provide contact information (Name, email address, phone, etc.)	11.1%	3
answered question		**27**
skipped question		**13**

How did you first learn about incremental launching?	Response Percent	Response Count
Technical journal article (example ASCE Journal of Bridge Engineering)	11.1%	2
Trade publication article (example Modern Steel Construction)	16.7%	3
Book	5.6%	1
Conference presentation	**33.3%**	6
Documentary video/DVD	0.0%	0
Other (please specify)	**33.3%**	6
answered question		**18**
skipped question		**22**

How would you rate your familiarity with this construction method?		Response Percent	Response Count
1 = Extremely knowlegeable (personally involved in one or more projects)		0.0%	0
2 =		22.2%	4
3 = Read article(s) or attended presentation(s)		**72.2%**	13
4 =		5.6%	1
5 = Completely unfamiliar		0.0%	0
		answered question	**18**
		skipped question	**22**

How would you rate the significance of perceived advantages of the incremental launching method as compared to more conventional construction?	Very significant	Significant	Somewhat significant	Not very significant	Response Count
Minimal disturbance to surroundings	38.5% (10)	**50.0% (13)**	7.7% (2)	3.8% (1)	26
Reduced access required beneath bridge	34.6% (9)	**46.2% (12)**	19.2% (5)	0.0% (0)	26
Smaller, more concentrated work area	4.0% (1)	32.0% (8)	**44.0% (11)**	20.0% (5)	25
Increased worker safety due to ground-level assembly	11.5% (3)	**50.0% (13)**	26.9% (7)	11.5% (3)	26
Increased speed of construction	11.5% (3)	**50.0% (13)**	34.6% (9)	3.8% (1)	26
Smaller equipment required for construction	0.0% (0)	**57.7% (15)**	23.1% (6)	19.2% (5)	26
				answered question	**26**
				skipped question	**14**

Please list other perceived advantages of the incremental launching method not provided above.		
		Response Count
		2
	answered question	2
	skipped question	38

How would you rate the significance of potential disadvantages of the incremental launching method as compared to more conventional construction?					
	Very significant	Significant	Somewhat significant	Not very significant	Response Count
Perceived risk to owner and contractor	16.0% (4)	**52.0% (13)**	28.0% (7)	4.0% (1)	25
Increased costs	28.0% (7)	**48.0% (12)**	24.0% (6)	0.0% (0)	25
Increased time for construction	8.0% (2)	**32.0% (8)**	28.0% (7)	**32.0% (8)**	25
Requires specialized hardware (rollers, jacks, etc.)	8.3% (2)	**54.2% (13)**	29.2% (7)	8.3% (2)	24
Contractor unfamiliarity with method	**44.0% (11)**	**44.0% (11)**	12.0% (3)	0.0% (0)	25
Increased horizontal forces on substructure	4.0% (1)	**40.0% (10)**	36.0% (9)	20.0% (5)	25
Access requirements behind abutment(s)	8.0% (2)	**40.0% (10)**	**40.0% (10)**	12.0% (3)	25
				answered question	25
				skipped question	15

Please list other potential disadvantages of the incremental launching method not provided above.		
		Response Count
		4
	answered question	4
	skipped question	36

How many incrementally launched bridges has your agency completed or have under construction?

	Response Percent	Response Count
None	81.3%	26
1	12.5%	4
2	6.3%	2
3 or more	0.0%	0
	answered question	**32**
	skipped question	**8**

Please complete the information below for your launched bridge project #1.

	Response Percent	Response Count
Location	**100.0%**	6
Feature crossed (river, railroad, highway, etc.)	**100.0%**	6
Completion date (year)	**100.0%**	6
Material (steel, concrete)	**100.0%**	6
Structure type (I-girder, box girder)	**100.0%**	6
Designer	**100.0%**	6
Contractor	**100.0%**	6
Bridge construction or launching engineer	83.3%	5
Specialty manufacturer (bearings, jacks)	66.7%	4
	answered question	**6**
	skipped question	**34**

Please complete the information below for your launched bridge project #2 (if applicable).

	Response Percent	Response Count
Location	**100.0%**	2
Feature crossed (river, railroad, highway, etc.)	**100.0%**	2
Completion date (year)	**100.0%**	2
Material (steel, concrete)	**100.0%**	2
Structure type (I-girder, box girder)	**100.0%**	2
Designer	**100.0%**	2
Contractor	**100.0%**	2
Bridge construction or launching engineer	50.0%	1
Specialty manufacturer (bearings, jacks)	50.0%	1
	answered question	**2**
	skipped question	**38**

Would you be willing to share copies of project plans and specifications, construction details, photos, etc.?

	Response Percent	Response Count
Yes	16.7%	1
No	16.7%	1
If yes, please provide contact person to acquire this information (Name, date, author, link to online document, etc.)	**66.7%**	**4**
	answered question	**6**
	skipped question	**34**

Were these projects documented with technical reports, journal articles, conference presentations, etc.?		
	Response Percent	Response Count
Yes	16.7%	1
No	**66.7%**	**4**
If yes, please provide details (title, publication, date, author, link to online document, etc.)	16.7%	1
	answered question	**6**
	skipped question	**34**

If 3 or more projects have been completed, would you be willing to email the requested information to the research team?		
	Response Percent	Response Count
Yes	0.0%	0
No	0.0%	0
Would it be acceptable for the research team to contact you? Please provide contact information.	0.0%	0
	answered question	**0**
	skipped question	**40**

Is your agency CURRENTLY CONSIDERING an incrementally launched bridge project for future construction?		
	Response Percent	Response Count
Yes, project is in final design	0.0%	0
Yes, project is in preliminary design	0.0%	0
Yes, project is in early concept stage	3.1%	1
No	**96.9%**	**31**
	answered question	**32**
	skipped question	**8**

Please complete the information below for your proposed launched bridge project			
		Response Percent	Response Count
Location		100.0%	1
Feature crossed (river, railroad, highway, etc.)		100.0%	1
Structure type (I-girder, box girder)		100.0%	1
Material (steel or concrete)		100.0%	1
Designer (if known)		100.0%	1
Contractor (if known)		100.0%	1
Bridge construction or launching engineer (if known)		100.0%	1
Specialty manufacturer (bearings, jacks) (if known)		100.0%	1
		answered question	1
		skipped question	39

What types of tools would be most useful to you in preliminary/final design of an incremental launching project?					
	Very Useful	Useful	Somewhat Useful	Not useful	Response Count
Description of launching limitations	**45.2% (14)**	35.5% (11)	19.4% (6)	0.0% (0)	31
Series of illustrative case studies	22.6% (7)	**67.7% (21)**	9.7% (3)	0.0% (0)	31
Detailed list of recommendations	**64.5% (20)**	25.8% (8)	9.7% (3)	0.0% (0)	31
Collection of proven details (jacks, rollers, etc.)	**54.8% (17)**	32.3% (10)	12.9% (4)	0.0% (0)	31

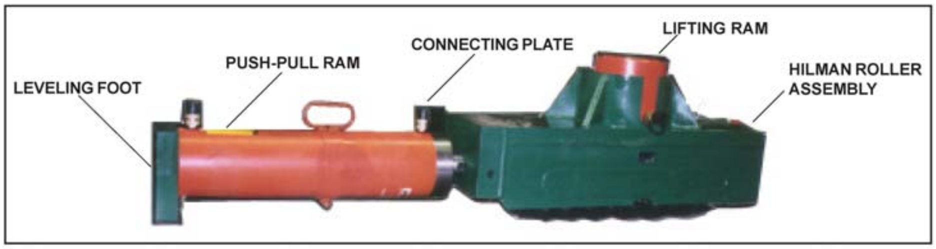
APPENDIX C

Details of Incremental Launching Systems

The selected sheets include examples provided by Hilman from previous projects along with a sheet on their jack/roller bridge launching unit. This unit combines both vertical lift capabilities (e.g. for jacking up the girders to insert permanent bearings) along with horizontal thrust to provide launching force component. This information is provided at the risk of appearing to endorse a commercial product, which is not the case. The fact is that they are THE heavy moving specialists for this kind of application and have many rollers that have been widely used and proven over time.

Hilman Data Sheet

INCREMENTAL BRIDGE LAUNCH SYSTEM



Combining high capacity Hilman Rollers with single and double-acting hydraulic rams creates an efficient, time-saving lift and roll system for launching and positioning bridge segments.

The Hilman Incremental System consists of two centralized hydraulic pumps and hoses connected to push/pull rams. These rams are mounted to an equal number of Hilman Rollers via an adapter which is threaded into a connecting push plate. Single-acting hydraulic lift rams are mounted on top of the Hilman Rollers; they are also connected to a hydraulic pump. The Bridge Launch System is set under the bridge or on a launch platform. "Idler" Rollers are placed at appropriate contact points to carry the load as it is moved. The bridge rests on piers or some falsework when the system is not activated.

The rollers used can have a variety of capabilities and load carrying capacities. Some require a hydraulic ram atop the roller as pictured. This allows the ram to be raised, the section blocked, and the roller to be moved or repositioned. The rollers are often used in the inverted position. Many bridges have a slight radius or "bank" to them. This translates into a minimal but tolerable side drag on the rollers.

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08/15/02
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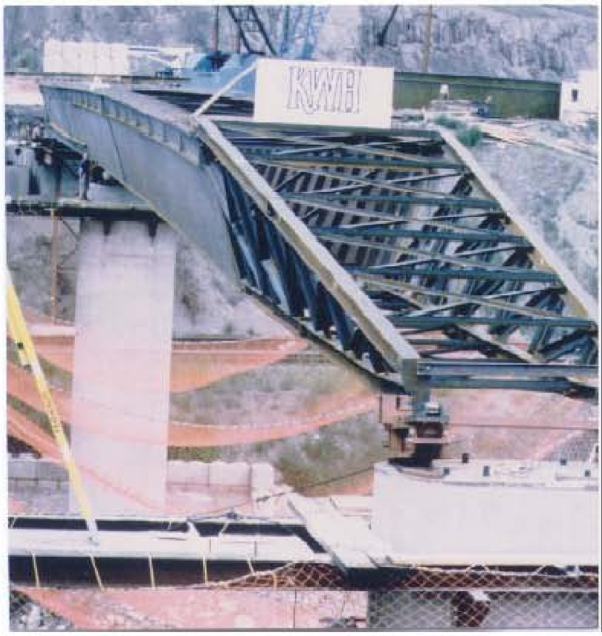
BRIDGE LAUNCH

Ashcroft Bridge, Ashcroft, BC, CANADA
1990



Hilman Rollers were used when this new five span bridge at Ashcroft, British Columbia replaced a sixty year old steel truss structure. Erecting and installing the new 783 foot (238m) long bridge over the Thompson River posed a problem: the river flow becomes a dangerous torrent during run-off. Because of this the contractor chose to launch the steel girders from both sides, closing in the middle.

To accomplish the unusual launch, the bridge was launched over rocker beams on which 75-OT Hilman Rollers were mounted upside-down. The system was designed to minimize critical web buckling stresses and simplify passage of differing plate thicknesses.

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STEEL BRIDGE INCREMENTAL LAUNCH

West Duffin Creek Bridge, Ontario, CANADA
1993

Early in 1993 Dominion Bridge Ltd. completed the movement of five bridge spans with box girders. The spans ranged in length from 160 to 236 feet long, for a total of 1115 feet. Combined weight of all the spans was 2400 tons.

Due to the curved design of the spans, three 150 ton capacity Hilman Rollers, mounted upside-down, were used to launch them—two Rollers on the South web and one Roller on the North web. With this method, the contractor was able to launch a pair of girders at one time, and complete this job in good time.

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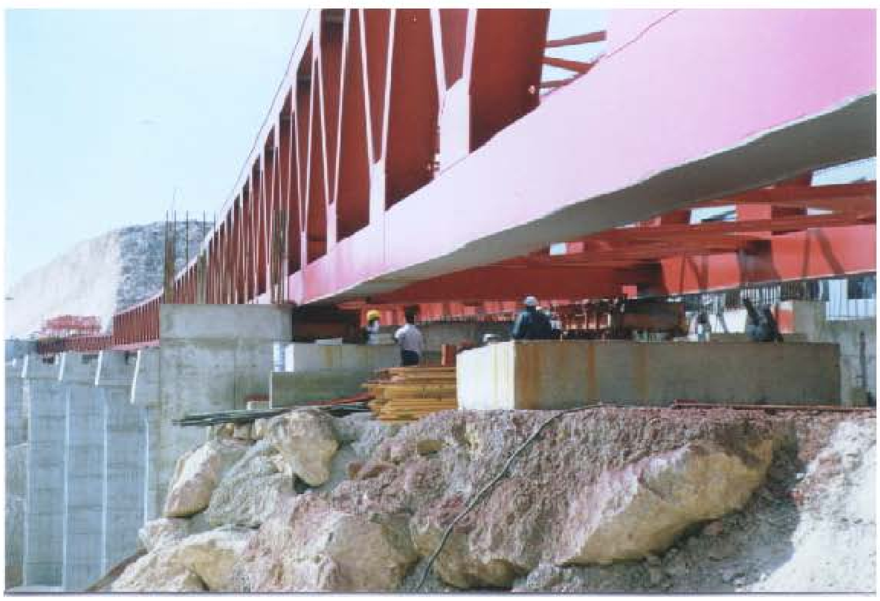
STEEL
BRIDGE
LAUNCH

Almeria, SPAIN
1995

Pictured here is the December 1995 launch of a steel bridge span at Almeria, Spain. 150 ton capacity Hilman Rollers are installed upside-down on 20 concrete piers spaced every 15 meters (approx. 45 ft.).
The bridge span is slid over the top of the Rollers into place.



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DAMSUMLO BRIDGE LAUNCH

Skeena River, North of Hazelton, B.C., CANADA
1995

At its highest point, the 600 foot long Damsumlo Bridge sits 140 feet above the beautiful Skeena River. Due to its limited access, The British Columbia contractor constructed the bridge from one side using a launching method in which Hilman Rollers were an integral part. A series of 50-OT Hilman Rollers were used to launch and guide the spans 245 feet to the touchdown point on the other side. Some Rollers were mounted upside down as load bearing Rollers; others were mounted sideways for horizontal alignment. The pleased contractor reports that the Hilman Rollers made it possible to move over one million pounds of superstructure with a bulldozer the size of a D8 over an uphill grade of 4%.

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TEMPORARY BRIDGE LAUNCH

Sagticos Parkway on Long Island, New York USA
1989 & 1991

Strings of inverted Hilman Rollers with pivoting bases were used to launch a temporary bridge used during the construction of the Sagticos Parkway Overpass. The entire launching process of the temporary bridge was completed in only one hour and twenty minutes. The Hilman Rollers system was again used in 1991 when the temporary bridge was removed in less than twenty minutes.

Close up of one of the Hilman Rollers used in this bridge launching application. The OT style Rollers have LRP style rocker tops. When mounted with the LRP top down, allows the temporary bridge to be rolled over the top of the Rollers. The pivoting ability of the LRP top helps to keep the bridge level as it is installed and removed.

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TWIN GIRDER INSTALLATION

Gitwinksihlkw Bridge Site, British Columbia, CANADA
1995

Four 150 ton capacity Hilman rollers, used in the inverted position, were used on the main launch pier to launch twin girders the main span distance of approximately 335 feet (102 meters). A 131 foot (40 meters) launching nose and 49 foot (15 meters) falsebent were also used in the span placement. The use of Hilman rollers provided a practical and economical method for launching the large span.

150 OT Hilman Roller used in the inverted position to skid bridge span.

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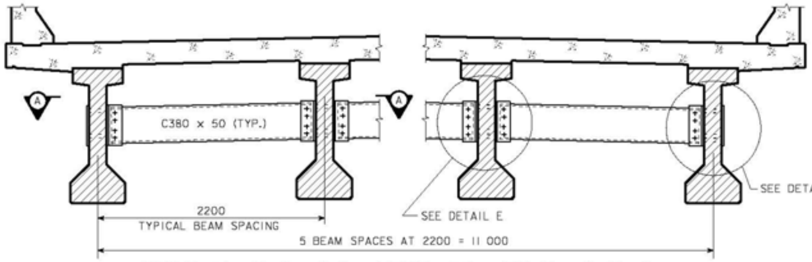
APPENDIX D

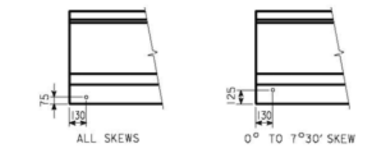
Example Details for Launched Bridge Projects

This appendix includes example details from the U.S. 20 Iowa River Bridge which was successfully completed using the ILM. Included are a variety of contract plan drawings which illustrate the following:

- Bridge erection sequence
- Launching nose and kingpost details
- Roller and sliding bearing details
- Miscellaneous details such as tapered ramp plates for bolted splices on steel girders

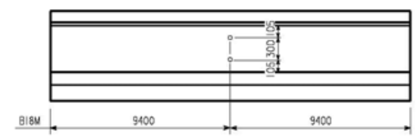


SECTION SHOWING INTERMEDIATE DIAPHRAGM



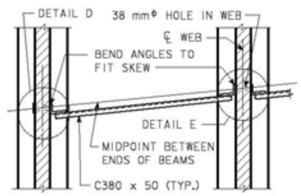
INTEGRAL ABUTMENT EXPANSION PIER

BEAM COIL TIE LOCATIONS



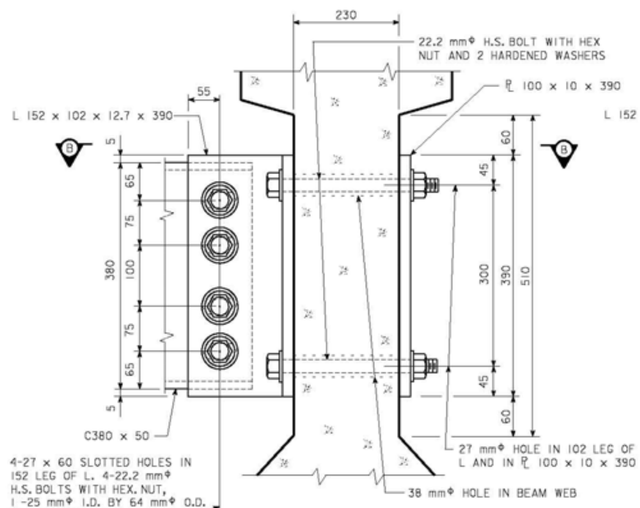
INTERMEDIATE DIAPHRAGM BOLT LOCATION

(0° TO 7°30' SKEW)

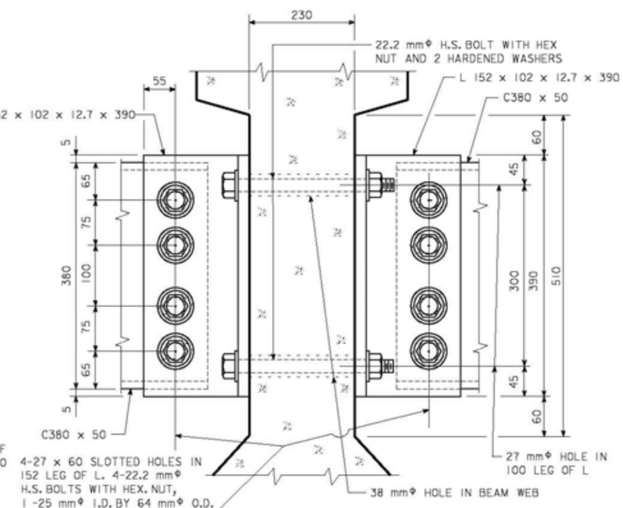


SECTION A-A

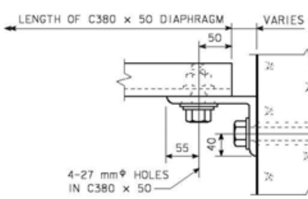
FOR BRIDGES SKEWED LESS THAN OR EQUAL TO 7°30'



DETAIL D



DETAIL E



SECTION B-B

INTERMEDIATE DIAPHRAGM STRUCTURAL STEEL

ONE CONNECTION DETAIL "E"

WEB THICKNESS	LENGTH OF H.S. BOLTS	MASS PER DETAIL "E"	NUMBER OF DETAIL "E"	MASS
2 - 22.2 mmΦ x LENGTH H.S. BOLTS WITH NUTS & WASHERS				
230	305	2.4	16	38.4
2 - L 152 x 102 x 12.7 x 390 = 18.8 kg			16	300.8

ONE CONNECTION DETAIL "D"

WEB THICKNESS	LENGTH OF H.S. BOLTS	MASS PER DETAIL "D"	NUMBER OF DETAIL "D"	MASS
2 - 22.2 mmΦ x LENGTH H.S. BOLTS WITH NUTS & WASHERS				
230	305	2.4	8	19.2
1 - BACKING ℘ 100 x 10 x 390 = 3.1 kg			8	24.8
1 - L 152 x 102 x 12.7 x 390 = 9.4 kg			8	75.2

ONE C380 x 50 DIAPHRAGM

BEAM SPACING / WEB THICKNESS	1960 * LENGTH	1960 UNIT MASS (kg)	2000 * LENGTH	2000 UNIT MASS (kg)	2030 * LENGTH	2030 UNIT MASS (kg)	2150 * LENGTH	2150 UNIT MASS (kg)	2200 * LENGTH	2200 UNIT MASS (kg)
230									1885	94.3

* THE LENGTH OF THE C380x50 SHOWN IN THE TABLE IS BASED ON A VARIABLE CLEARANCE OF 40 mm TO 55 mm BETWEEN THE FACE OF BEAM WEB AND END OF C380 x 50.

DIAPHRAGM MASS

UNIT MASS	NUMBER OF DIAPHRAGMS	
94.3 kg	20	1886
kg		

DIAPHRAGM CONNECTION BOLTS

8 - 22.2 mmΦ x 70 H.S. BOLTS WITH NUTS AND WASHERS, PER UNIT DIAPHRAGM	NUMBER OF DIAPHRAGMS	
4.7 kg	20	94

INTERMEDIATE DIAPHRAGM STRUCTURAL STEEL - TOTAL (kg)	2438

NOTES:

STRUCTURAL STEEL FOR DIAPHRAGMS AND CONNECTION ANGLES SHALL CONFORM TO ASTM A709, GRADE 250.

ALL DIAPHRAGM MATERIALS, INCLUDING BOLTS, NUTS AND WASHERS SHALL BE GALVANIZED.

SHOP DRAWINGS OF THE STEEL DIAPHRAGMS SHOWING LAYOUT AND DETAILS OF THE DIAPHRAGMS SHALL BE SUBMITTED FOR APPROVAL.

ALL COSTS FOR FURNISHING AND INSTALLING STEEL INTERMEDIATE DIAPHRAGMS SHALL BE INCLUDED IN THE PRICE BID FOR STRUCTURAL STEEL.

THE 38 mmΦ HOLES FOR THE 22.2 mmΦ H.S. BOLTS SHALL BE CAST INTO THE WEB. DRILLING IS NOT ALLOWED.

THE 22.2 mmΦ H.S. BOLTS THROUGH THE WEB SHALL HAVE A THREAD LENGTH OF 75 mm MIN. AND 100 mm MAX.

ALL BOLTS ARE TO BE TIGHTENED PRIOR TO PLACING BRIDGE FLOOR CONCRETE WITH THE FOLLOWING EXCEPTION : BOLTS IN DIAPHRAGMS LOCATED UNDER LONGITUDINAL BRIDGE FLOOR CONSTRUCTION JOINTS SHALL NOT BE TIGHTENED UNTIL STAGE TWO OF THE BRIDGE FLOOR HAS BEEN PLACED.

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

SUPERSTRUCTURE DETAILS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

DESIGN SHEET NO. A26A OF ____ FILE NO. 29212 DESIGN NO. 199

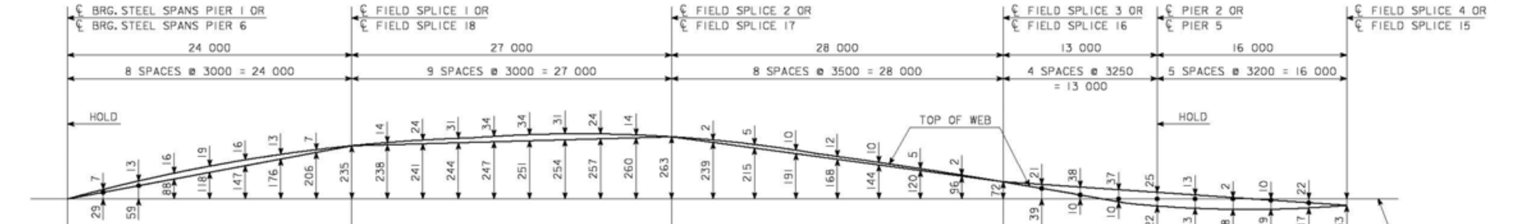
DESIGNED BY PJH CHECKED BY EDY

DETAILED BY MBG CADD FILE

STEEL INTERMEDIATE DIAPHRAGMS FOR PC BEAM BRIDGES | MODIFIED STANDARD SHEET M1036 | HARDIN COUNTY | PROJECT NUMBER

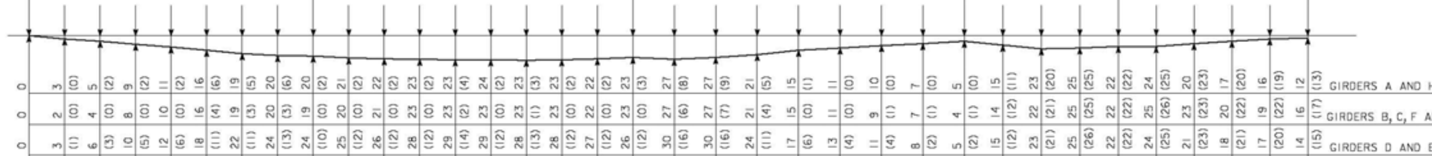
STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

D-3



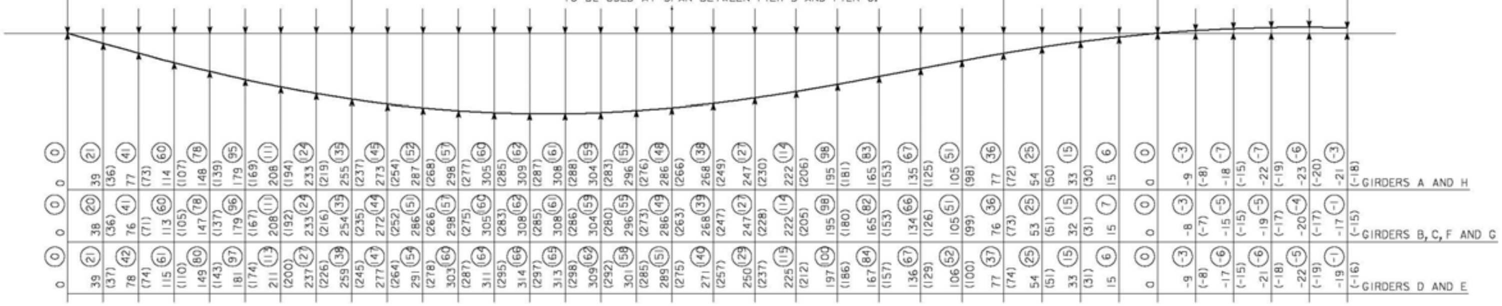
GIRDERS AS FABRICATED HORIZONTALLY

NOTE: DOES NOT INCLUDE DEFLECTION DUE TO WEIGHT OF STEEL OR CONCRETE.



HAUNCH THICKENING DIAGRAM

NOTE: FOR ESTIMATING PURPOSES ONLY. (XX) INDICATES VALUES TO BE USED AT SPAN BETWEEN PIER 5 AND PIER 6.



ANTICIPATED DEAD LOAD DEFLECTION DIAGRAM

NOTE: ENCIRCLED FIGURES INDICATE ANTICIPATED DEFLECTION DUE TO WEIGHT OF CONCRETE (SLAB, WEARING COARSE, BARRIERS) ONLY. (XX) INDICATES VALUES TO BE USED AT SPAN BETWEEN PIER 5 AND 6.

MOMENT AND REACTION TABLE

GIRDERS	ITEM	UNIFORM LOAD (kN/m)	DISTRIBUTION FACTOR (LANES OF MS18)		POSITIVE MOMENT (kN-m)					NEGATIVE MOMENT (kN-m)				REACTION (kN)					
			MOMENT	REACTION	SPAN 2	SPAN 3	SPAN 4	SPAN 5	SPAN 6	PIER 2	PIER 3	PIER 4	PIER 5	PIER 1	PIER 2	PIER 3	PIER 4	PIER 5	PIER 6
A & H	DEAD LOAD "A"	19.94	——	——	18 569	5370	9149	5370	18 564	-28 066	-20 407	-20 409	-28 062	1037	3084	2546	2546	3084	1037
	DEAD LOAD "B"	5.11	——	——	3320	1305	1847	1305	3320	-4667	-3556	-3556	-4666	185	533	458	458	533	185
	LIVE LOAD+IMPACT	——	0.849	0.849	9314	6894	7783	6894	9313	-10 019	-9120	-9120	-10 018	504	1095	1045	1045	1095	504
	TOTAL	——	——	——	31 203	13 569	18 779	13 569	31 197	-42 752	-33 083	-33 085	-42 746	1726	4712	4049	4049	4712	1726
B, C, F & G	DEAD LOAD "A"	25.27	——	——	22 025	6369	10 852	6370	22 020	-33 290	-24 206	-24 208	-33 286	1230	3658	3020	3020	3658	1230
	DEAD LOAD "B"	5.11	——	——	3320	1305	1847	1305	3320	-4667	-3556	-3556	-4666	185	533	458	458	533	185
	LIVE LOAD+IMPACT	——	1.074	1.153	11 812	8650	9849	8650	11 811	-12 644	-11 645	-11 645	-12 643	653	1399	1340	1340	1399	653
	TOTAL	——	——	——	37 157	16 324	22 548	16 325	37 151	-50 601	-39 407	-39 409	-50 595	2068	5590	4818	4818	5590	2068
D & E	DEAD LOAD "A"	20.62	——	——	18 538	5469	9324	5470	18 534	-28 558	-20 432	-20 433	-28 555	1041	3126	2564	2564	3126	1041
	DEAD LOAD "B"	5.11	——	——	3310	1306	1873	1306	3309	-4693	-3531	-3531	-4693	185	533	457	457	533	185
	LIVE LOAD+IMPACT	——	0.849	0.849	9314	6894	7783	6894	9313	-10 019	-9120	-9120	-10 018	504	1095	1045	1045	1095	504
	TOTAL	——	——	——	31 162	13 669	18 980	13 670	31 156	-43 270	-33 083	-33 084	-43 266	1730	4754	4066	4066	4754	1730

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

SUPERSTRUCTURE DETAILS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

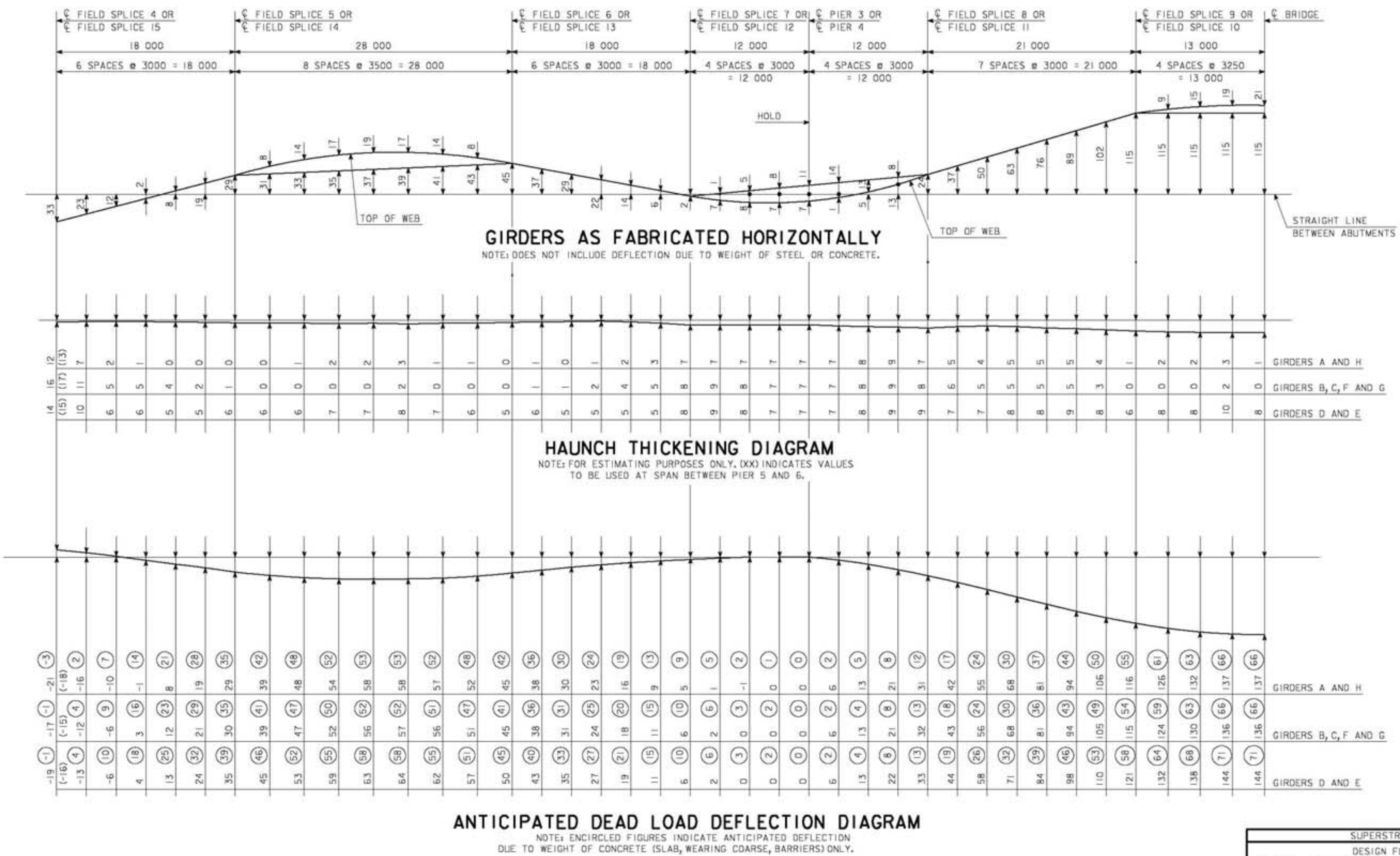
DESIGN SHEET NO. /A30/ OF ____ FILE NO. 29212 DESIGN NO. 199

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

DESIGNED BY CMS CHECKED BY EDY
DETAILED BY CAT CADD FILE

HARDIN COUNTY PROJECT NUMBER

PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$



SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

SUPERSTRUCTURE DETAILS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

DESIGN SHEET NO. A300 OF ___ FILE NO. 29212 DESIGN NO. 199

DESIGNED BY CMS CHECKED BY EDY
DETAILED BY CAT CADD FILE

HARDIN COUNTY

PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

PLOTTED: $$USERNAME$$
PLOTTED: $$DATE AND TIME$$ $$DGNSPEC$$

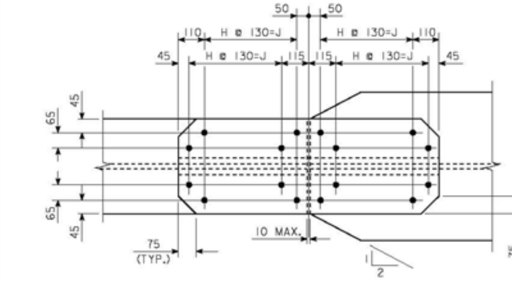
D-5

FIELD SPLICE DATA - GIRDERS A, B, C, F, G AND H

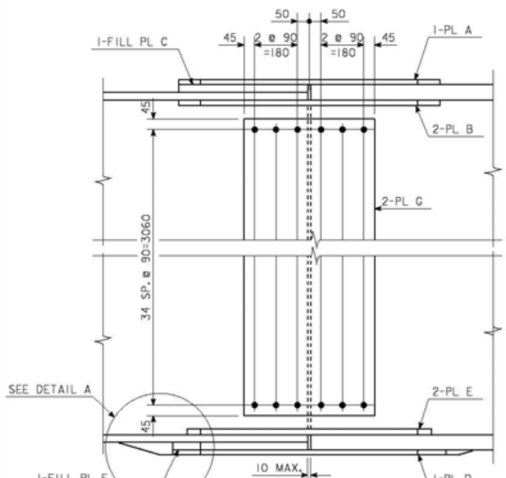
LOCATION	PL A	PL B	FILL PL C	PL D	PL E	FILL PL F	PL G	H	J	K	L
FIELD SPLICE 1 OR 18	PL 25x400x1880	PL 32x165x1880	PL 10x400x935	PL 32x500x2460	PL 40x200x2290	PL 14x500x1225	PL 16x730x3190	6	780	10	900
FIELD SPLICE 2 OR 17	PL 22x400x1620	PL 28x165x1620	PL 14x400x805	PL 32x500x2460	PL 40x200x2290	PL 14x500x1225	PL 16x730x3190	5	650	10	900
FIELD SPLICE 3 OR 16	PL 22x400x1620	PL 28x165x1620	PL 5x400x805	PL 20x500x1170	PL 25x200x1000	PL 12x500x580	PL 14x730x3190	5	650	4	360
FIELD SPLICE 4 OR 15	PL 20x400x840	PL 25x165x840	PL 22x400x415	PL 20x500x1170	PL 25x200x1000	PL 12x500x580	PL 16x730x3190	2	260	4	360
FIELD SPLICE 5 OR 14	PL 20x400x840	PL 25x165x840	———	PL 20x500x990	PL 25x200x820	PL 3x500x490	PL 12x550x3150	2	260	3	270
FIELD SPLICE 6 OR 13	PL 20x400x840	PL 25x165x840	———	PL 20x500x990	PL 25x200x820	PL 3x500x490	PL 12x550x3150	2	260	3	270
FIELD SPLICE 7 OR 12	PL 20x400x840	PL 25x165x840	PL 40x400x415	PL 20x500x1170	PL 25x200x1000	PL 32x500x580	PL 16x730x3190	2	260	4	360
FIELD SPLICE 8 OR 11	PL 20x375x840	PL 25x155x840	PL 40x375x415	PL 20x500x1170	PL 25x200x1000	PL 32x500x580	PL 14x730x3190	2	260	4	360
FIELD SPLICE 9 OR 10	PL 20x375x840	PL 25x155x840	PL 3x375x415	PL 20x500x1170	PL 25x200x1000	PL 6x500x580	PL 14x730x3190	2	260	4	360

FIELD SPLICE DATA - GIRDERS D AND E

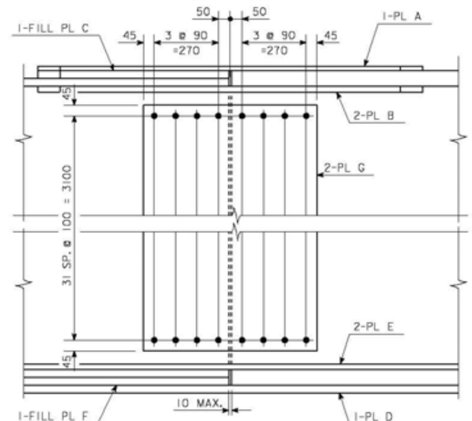
LOCATION	PL A	PL B	FILL PL C	PL D	PL E	FILL PL F	PL G	H	J	K	L
FIELD SPLICE 1 OR 18	PL 20x400x1620	PL 25x165x1620	PL 6x400x805	PL 28x500x2100	PL 35x200x1930	PL 10x500x1045	PL 14x730x3190	5	650	8	720
FIELD SPLICE 2 OR 17	PL 20x400x1100	PL 25x165x1100	PL 10x400x545	PL 28x500x2100	PL 35x200x1930	PL 10x500x1045	PL 14x730x3190	3	390	8	720
FIELD SPLICE 3 OR 16	PL 20x400x1100	PL 25x165x1100	PL 10x400x545	PL 20x500x1170	PL 25x200x1000	PL 12x500x580	PL 14x730x3190	3	390	4	360
FIELD SPLICE 4 OR 15	PL 20x375x840	PL 25x155x840	PL 22x375x415	PL 20x500x990	PL 25x200x820	PL 16x500x490	PL 14x730x3190	2	260	3	270
FIELD SPLICE 5 OR 14	PL 20x375x840	PL 25x155x840	———	PL 20x500x990	PL 25x200x820	———	PL 12x550x3150	2	260	3	270
FIELD SPLICE 6 OR 13	PL 20x375x840	PL 25x155x840	———	PL 20x500x990	PL 25x200x820	———	PL 12x550x3150	2	260	3	270
FIELD SPLICE 7 OR 12	PL 20x375x840	PL 25x155x840	PL 35x375x415	PL 20x500x990	PL 25x200x820	PL 25x500x490	PL 14x730x3190	2	260	3	270
FIELD SPLICE 8 OR 11	PL 20x375x840	PL 25x155x840	PL 35x375x415	PL 20x500x990	PL 25x200x820	PL 25x500x490	PL 14x730x3190	2	260	3	270
FIELD SPLICE 9 OR 10	PL 20x375x840	PL 25x155x840	———	PL 20x500x990	PL 25x200x820	———	PL 14x730x3190	2	260	3	270



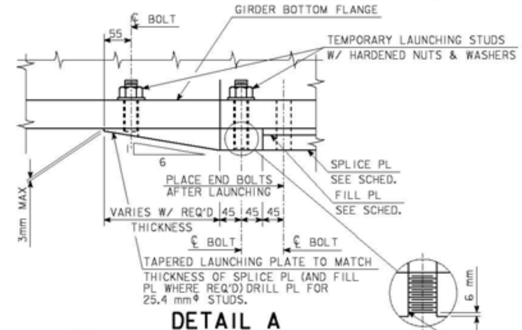
PLAN-TOP FLANGE
FIELD SPLICES 1 THRU 18



ELEVATION
FIELD SPLICES 5, 6, 13 AND 14



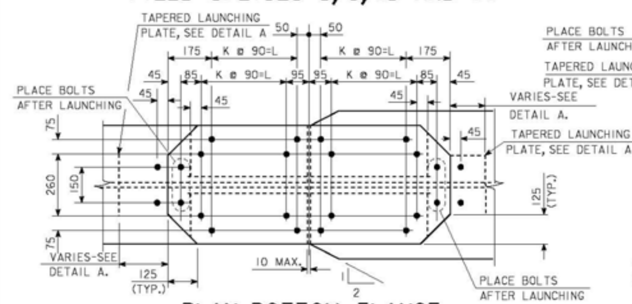
ELEVATION
FIELD SPLICES 1 THRU 4, 7 THRU 12
AND 15 THRU 18



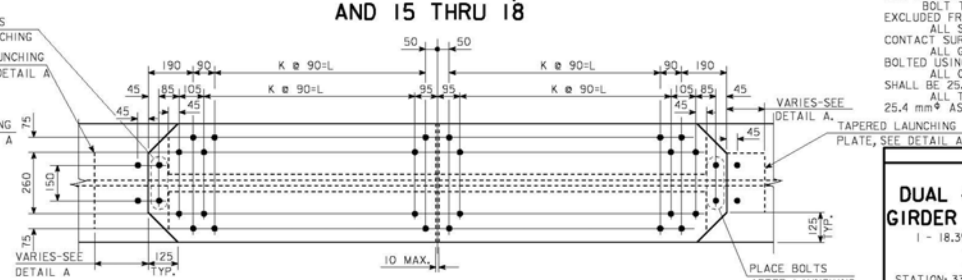
DETAIL A

NOTE:
TAPERED LAUNCHING PLATE SHALL BE INSTALLED PRIOR TO LAUNCHING OF GIRDERS AND SHALL BE REMOVED UPON COMPLETION OF LAUNCHING OPERATION. OPEN BOLT HOLES IN BOTTOM FLANGES SHALL BE FILLED WITH NEW 25.4 mmΦ ASTM A325, TYPE 3 BOLTS.

NOTES:
ALL SPLICE PLATES SHALL BE ASTM A709, GR. 345W.
FOR NOTES PERTAINING TO FILL PLATE THICKNESS, SEE DESIGN SHEET /A36/.
BOLT THREADS AND THREAD RUN-OUT SHALL BE EXCLUDED FROM THE SHEAR PLANE.
ALL SPLICE PLATES SHALL HAVE A CLASS B CONTACT SURFACE PREPARATION.
ALL GIRDER FIELD SPLICE CONNECTIONS SHALL BE BOLTED USING "HIGH STRENGTH BOLTS".
ALL OPEN HOLES SHALL BE 27 mmΦ AND ALL BOLTS SHALL BE 25.4 mmΦ ASTM A325M TYPE 3.
ALL TEMPORARY LAUNCHING PLATE BOLTS SHALL BE 25.4 mmΦ ASTM A325M, TYPE 3.



PLAN-BOTTOM FLANGE
FIELD SPLICES 3 THRU 16



PLAN-BOTTOM FLANGE
FIELD SPLICES 1, 2, 17 AND 18

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

FIELD SPLICE DETAILS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

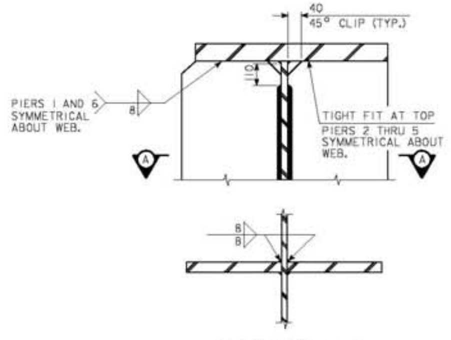
DESIGN SHEET NO. /A31/ OF ____ FILE NO. 29212 DESIGN NO. 199

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

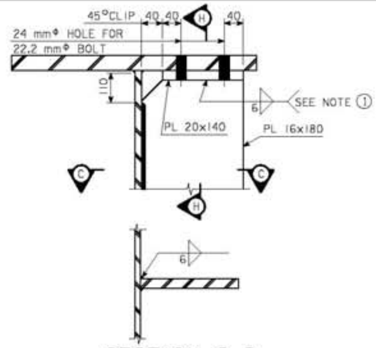
DESIGNED BY RGB CHECKED BY EDY
DETAILED BY CAT CADD FILE

HARDIN COUNTY PROJECT NUMBER

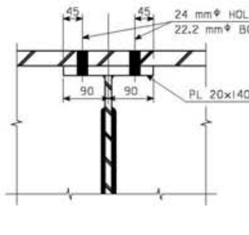
PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$



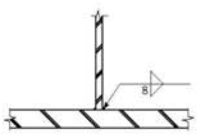
SECTION A-A



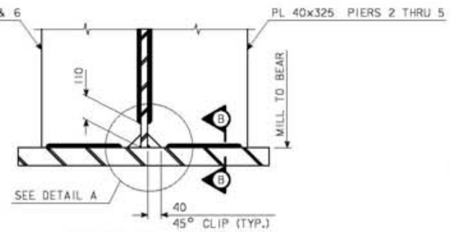
SECTION C-C



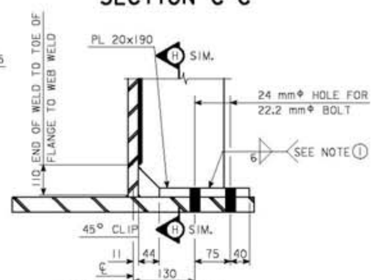
SECTION H-H



SECTION B-B



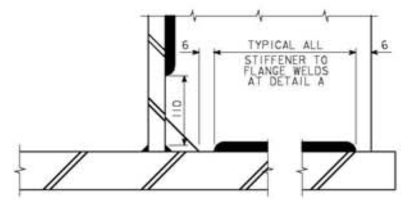
PIER BEARING STIFFENER



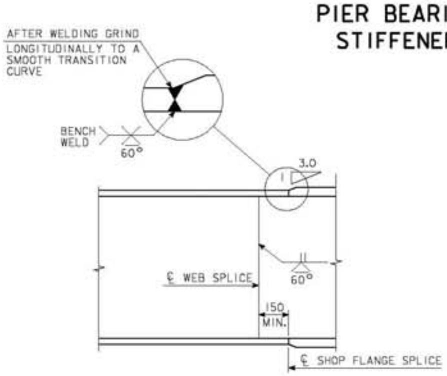
INTERMEDIATE DIAPHRAGM STIFFENER

NOTE 1

WELD AFTER PLATE IS BOLTED TO FLANGE. (SEE DETAIL A FOR WELD LOCATION).



DETAIL A

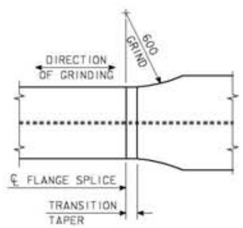


ELEVATION SHOP FLANGE & WEB SPLICE

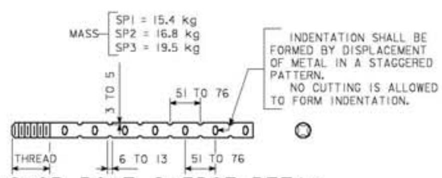
NOTE:

WELDED SHOP WEB AND FLANGE SPLICES MAY BE PERMITTED WHEN DETAILED ON THE SHOP DRAWINGS AND APPROVED BY THE ENGINEER. NO ADDITIONAL PAYMENT WILL BE MADE FOR OPTIONAL WEB AND FLANGE SPLICES ADDED FOR THE CONTRACTOR'S CONVIENIENCE.

ALL FLANGE BUTT WELDED JOINTS SUBJECT TO TENSION WITH REVERSAL OF STRESS ("TENSION ZONE") ARE TO BE RADIOGRAPHED FULL WIDTH. ALL FLANGE BUTT WELDED JOINTS SUBJECT TO COMPRESSION ONLY ARE TO BE RADIOGRAPHED FOR A MINIMUM OF 50% OF THE WIDTH. FOR LOCATIONS OF TENSION ZONES, SEE ELEVATIONS ON DESIGN SHEETS /A27/ THRU /A29/.



PLAN SHOP FLANGE SPLICE



ANCHOR BOLT SWEDGE DETAIL

NOTE:

FOR POT BEARING DEVICE DETAILS, SEE DESIGN SHEET /A33/.

NOTES:

THIS SHEET IS PRIMARILY FOR THE USE OF FABRICATOR'S, WORKMEN AND IOWA DEPARTMENT OF TRANSPORTATION INSPECTORS IN INTERPRETING PLAN DETAILS. IT COVERS THE LOCATIONS OF WELD TERMINI THAT ARE NOT SPECIFIED BY TYPICAL WELD SYMBOLS.

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

TYPICAL SUPERSTRUCTURE DETAILS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

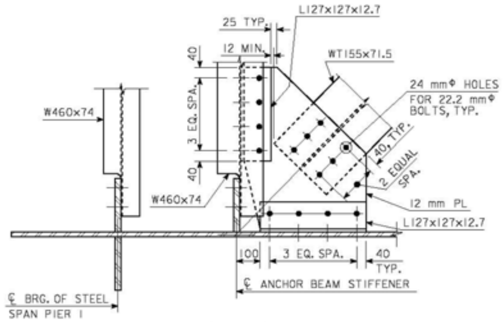
DESIGN SHEET NO. /A32/ OF ____ FILE NO. 29212 DESIGN NO. 199

PLOTTED: $$USERNAME$$
PLOTTED: $$DATE AND TIME$$ $$DGNSPEC$$

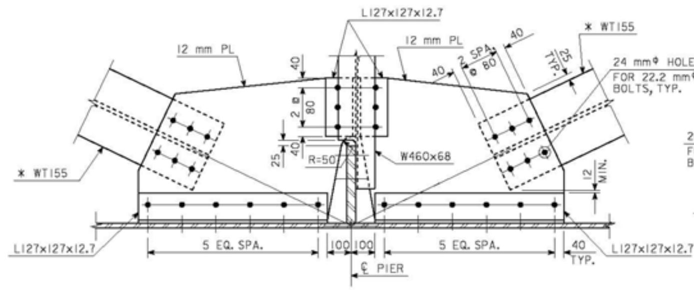
DESIGNED BY RGB	CHECKED BY EDY	HARDIN COUNTY	PROJECT NUMBER
DETAILED BY MBG	CADD FILE		

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

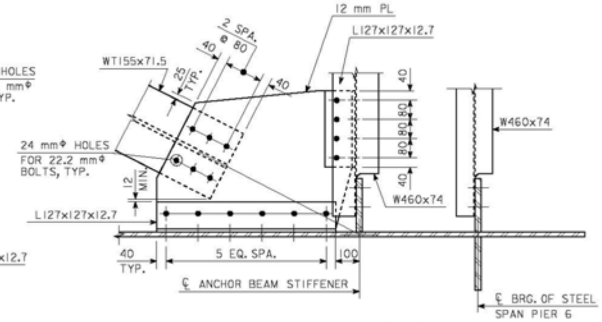
D-7



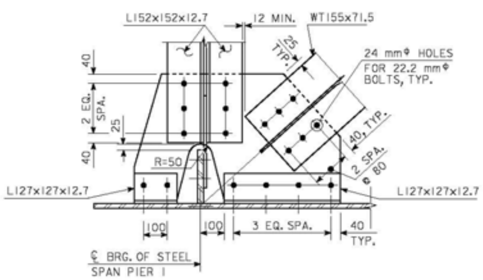
PIER 1
ANCHOR BEAM DIAPHRAGM - TOP



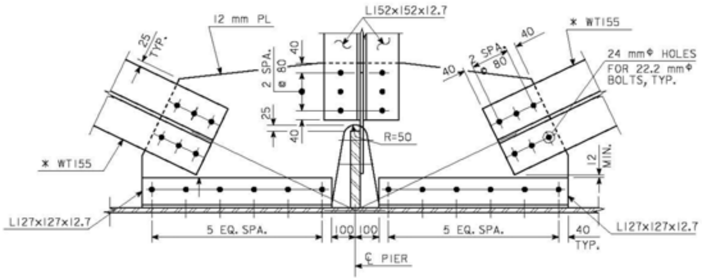
PIERS 2 THRU 5
PIER DIAPHRAGM - TOP



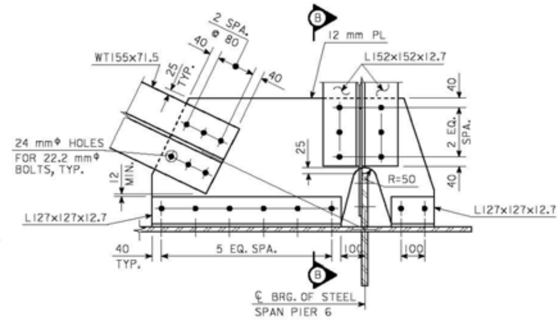
PIER 6
ANCHOR BEAM DIAPHRAGM - TOP



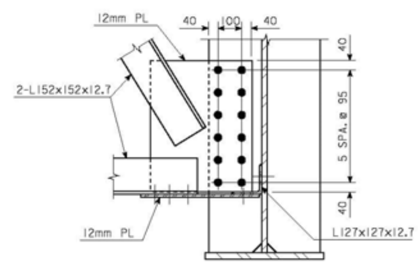
PIER 1
PIER DIAPHRAGM - BOTTOM



PIERS 2 THRU 5
PIER DIAPHRAGM - BOTTOM



PIER 6
PIER DIAPHRAGM - BOTTOM



SECTION B-B

NOTES:
FOR STEEL SUPERSTRUCTURE NOTES, SEE DESIGN SHEET /A36/.
FOR PIER DIAPHRAGM DETAILS, SEE DESIGN SHEETS /A36/ AND /A37/.
* DIAGONAL MEMBER VARIES, FOR SIZES AND LOCATIONS, SEE DESIGN SHEETS /A27/ THRU /A29/.

SUPERSTRUCTURE CONTRACT
DESIGN FOR 0 DEGREE SKEW
DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS
1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN
LATERAL BRACING DETAILS
STATION: 338+20.657
HARDIN COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION
DESIGN SHEET NO. A32a OF ____ FILE NO. 29212 DESIGN NO. 199

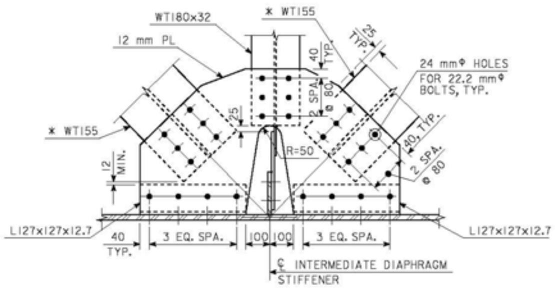
DESIGNED BY JPB	CHECKED BY DMR
DETAILED BY CAT	CADD FILE

HARDIN COUNTY

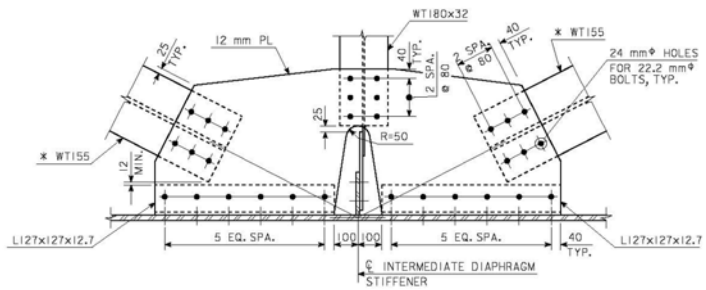
PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

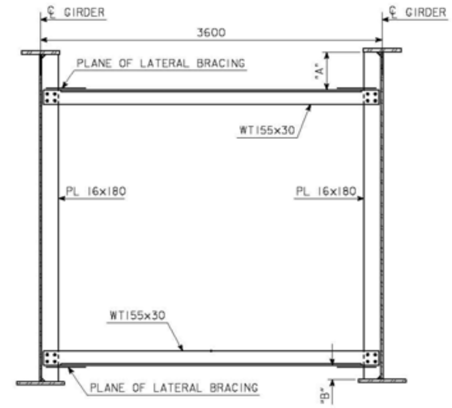
PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$



INTERMEDIATE DIAPHRAGM - TOP
BETWEEN PIERS 1 AND 2

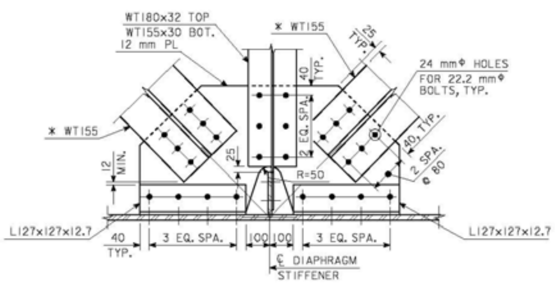


INTERMEDIATE DIAPHRAGM - TOP
BETWEEN PIERS 2 AND 6



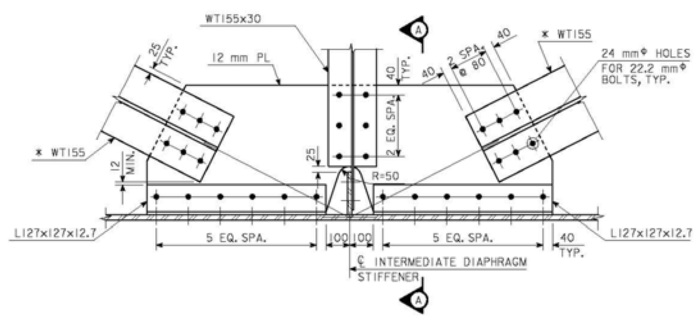
LATERAL BRACE DIAPHRAGM
BETWEEN PIERS 1 AND 2

DIMENSIONS		
LOCATION	"A"	"B"
GIRDER A & H	304	246
GIRDER B & G	376	174
GIRDER C & F	400	150
GIRDER D & E	328	222

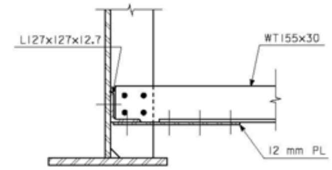


LATERAL BRACE DIAPHRAGM - TOP AND BOTTOM
INTERMEDIATE DIAPHRAGM - BOTTOM
BETWEEN PIERS 1 AND 2

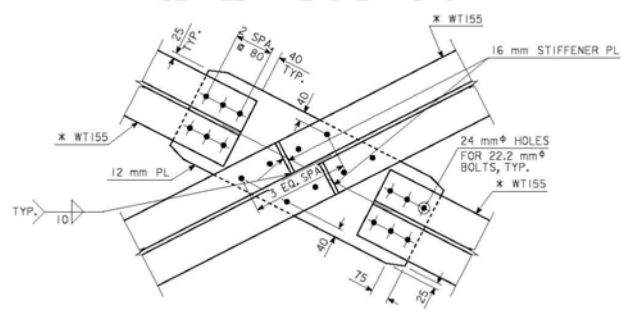
NOTE:
DETAIL SHOWN AT BOTTOM OF LATERAL BRACE AND INTERMEDIATE DIAPHRAGMS. MIRROR DETAIL ABOUT ℄ OF DIAPHRAGM FOR SIMILAR CONDITION @ TOP.



INTERMEDIATE DIAPHRAGM - BOTTOM
BETWEEN PIERS 2 AND 6



SECTION A-A



TYPICAL INTERSECTION OF LATERAL BRACING

NOTES:
* DIAGONAL MEMBER VARIES, FOR SIZES AND LOCATIONS, SEE DESIGN SHEETS /A27/, /A28/ AND /A29/.
FOR INTERMEDIATE DIAPHRAGM DETAILS, SEE DESIGN SHEET /A36/.
FOR LOCATION OF LATERAL BRACE DIAPHRAGMS, SEE DESIGN SHEETS /A27/, /A28/ AND /A29/.

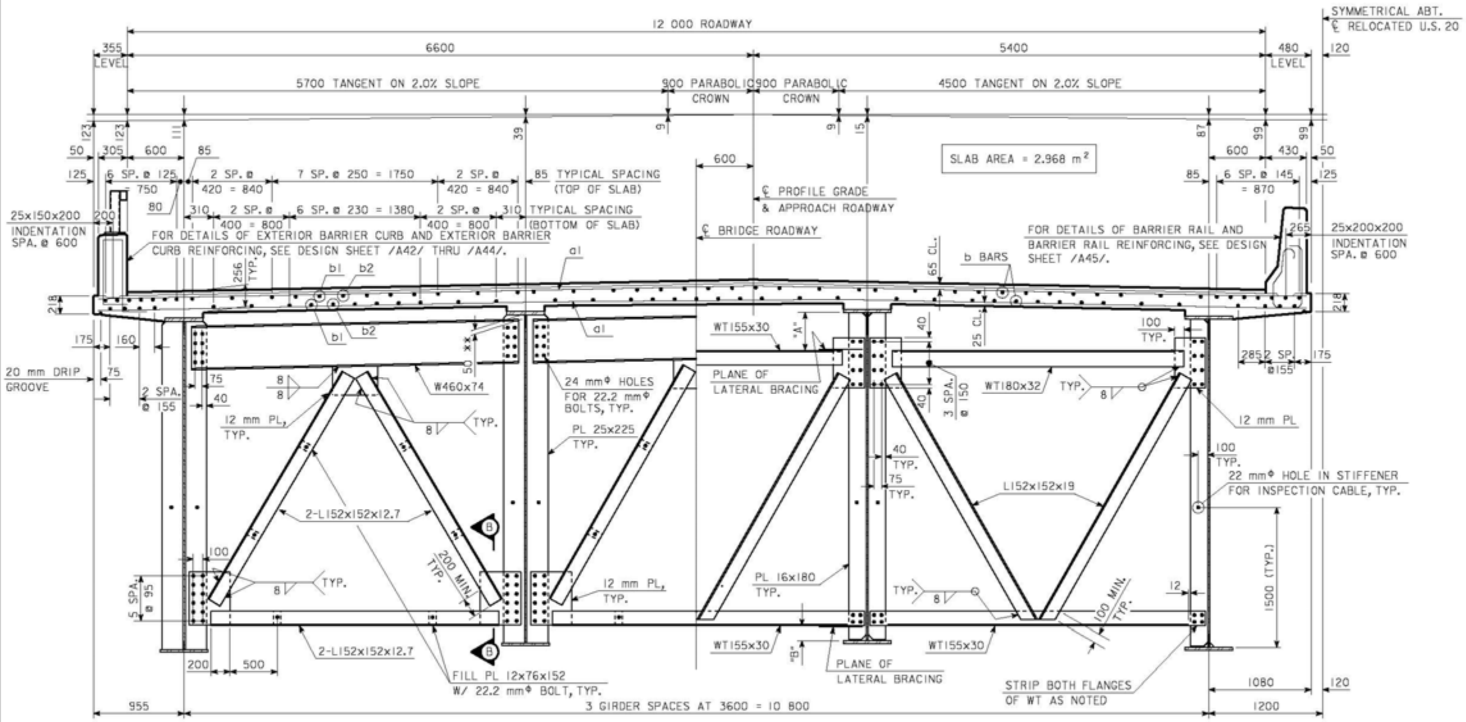
SUPERSTRUCTURE CONTRACT
DESIGN FOR 0 DEGREE SKEW
DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS
1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN
LATERAL BRACING DETAILS
STATION: 338+20.657
HARDIN COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION
DESIGN SHEET NO. /A32B/ OF ____ FILE NO. 29212 DESIGN NO. 199

DESIGNED BY JPB CHECKED BY DMR
DETAILED BY CAT CADD FILE

HARDIN COUNTY | PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$



HALF SECTION AT PIER 1 AND 6 — HALF SECTION NEAR INTERMEDIATE DIAPHRAGM

NOTE:
TOP AND BOTTOM LATERAL BRACING NOT SHOWN FOR CLARITY.
** TOP OF WEB TO TOP OF W460x74.

STEEL SUPERSTRUCTURE NOTES:

FORMS FOR THE SLAB AND BARRIER RAIL ARE TO BE SUPPORTED BY THE GIRDERS.

CLEAR DISTANCE FROM FACE OF CONCRETE TO NEAR REINFORCING BAR SHALL BE 50 mm UNLESS OTHERWISE NOTED OR SHOWN.

TOP TRANSVERSE REINFORCING STEEL IS TO BE PARALLEL TO AND 65 mm CLEAR BELOW TOP OF WEARING SURFACE. BOTTOM TRANSVERSE REINFORCING STEEL IS TO BE PARALLEL TO AND 25 mm CLEAR ABOVE BOTTOM OF SLAB. TOP AND BOTTOM REINFORCING STEEL IS TO BE SUPPORTED BY INDIVIDUAL EPOXY COATED METAL BAR CHAIRS SPACED AT NOT MORE THAN 900 mm CENTERS LONGITUDINALLY AND TRANSVERSELY, OR BY CONTINUOUS ROWS OF EPOXY COATED METAL BAR HIGH CHAIRS OR SLAB BOLSTERS SPACED 1200 mm APART.

ALL REINFORCING BARS ARE TO BE EPOXY COATED.

TRANSVERSE SLAB REINFORCING MAY BE SPLICED WITH ONE LAP LOCATED AS FOLLOWS:

TOP BARS - LAP MIDWAY BETWEEN GIRDERS (MIN. LAP = #15-770 mm/#20-960 mm)

NO LAPS WILL BE ALLOWED IN THE EXTERIOR BAY.

BOTTOM BARS - LAP OVER GIRDERS (MIN. LAP = #15-770 mm/#20-960 mm)

PAYMENT FOR REINFORCING BARS SHALL BE BASED ON NO SPLICES, AND NO ALLOWANCE SHALL BE MADE FOR THE ADDITIONAL LENGTH OF BAR REQUIRED FOR THE USE OF SPLICES.

ALL FIELD CONNECTIONS EXCEPT FOR GIRDER SPLICES ARE TO BE BOLTED USING "HIGH STRENGTH BOLTS", ASTM A325M, TYPE III. UNLESS OTHERWISE NOTED, ALL OPEN HOLES ARE TO BE 24 mm⌀ AND ALL BOLTS ARE TO BE 22.2 mm⌀.

BOTTOM FLANGES ARE TO BE PERPENDICULAR TO WEBS AT THE REACTION POINTS.

FILL PL THICKNESSES SHOWN ON PLANS ARE BASED ON NOMINAL GIRDER DIMENSIONS. THESE THICKNESSES ARE TO BE VERIFIED OR ADJUSTED DURING FABRICATION TO SECURE A CLOSE FIT. EACH FILL PLATE SHALL FIT TO THE NEAREST 2 mm IN THICKNESS AND SINGLE PLATES ARE REQUIRED AT EACH FILL LOCATION. GIRDERS ARE TO BE TRULY SQUARE AT SPLICE POINTS WITH FLANGES PERPENDICULAR TO WEBS.

MAGNETIC PARTICLE INSPECTION OF WELDS, IN ACCORDANCE WITH THE STANDARD SPECIFICATIONS, WILL BE REQUIRED FOR THE WEB TO FLANGE WELDS AND THE BEARING STIFFENER WELDS OF THE GIRDERS.

SHEAR STUD CONNECTORS ARE TO BE WELDED IN THE SHOP OR IN THE FIELD AT THE LOCATIONS SHOWN ON THE DESIGN PLANS OR THE APPROVED SHOP DRAWINGS.

THE DESIGN DRAWINGS INDICATE AWS PREQUALIFIED WELDED JOINTS, AND UNLESS OTHERWISE NOTED THE DESIGN JOINT DETAILS ARE FOR MANUAL SHIELDED METAL-ARC WELDING. ALTERNATE JOINT DETAILS MAY BE SUBMITTED FOR APPROVAL.

FOR TYPICAL DETAILS AT GIRDERS, SEE DESIGN SHEET /A27/ AND /A32/.

FOR LATERAL BRACING DETAILS, SEE DESIGN SHEET /A32a/ AND /A32b/.

FOR INSPECTION CABLE DETAILS, SEE DESIGN SHEET /A32C/.

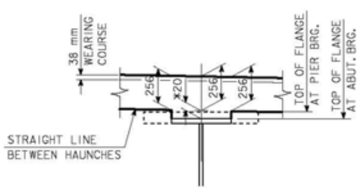
FOR EXPANSION JOINT DETAILS, SEE DESIGN SHEET /A46/ THRU /A46c/.

FOR SECTION B-B, SEE DESIGN SHEET /A37/.

ALL STRUCTURAL STEEL SHALL BE ASTM A709, GRADE 345W UNLESS NOTED OTHERWISE IN THE PLANS OR SPECIFICATIONS.

DIMENSIONS

LOCATION	"A"	"B"
GIRDER A & H	304	246
GIRDER B & G	376	174
GIRDER C & F	400	150
GIRDER D & E	328	222



TYP. SLAB & HAUNCH DETAIL

* THE HAUNCH DIMENSION SHOWN IS THE NOMINAL HAUNCH DIMENSION NEAR PIER 1 AND 6 BEARINGS. FOR THE SLAB THICKNESS OVER THE GIRDER AT ANY LOCATION, THE NOMINAL HAUNCH DIMENSION IS TO BE DECREASED BY THE ADDITIONAL FLANGE THICKNESS AT THAT POINT AND INCREASED OR DECREASED BY THE AMOUNT INDICATED ON THE "HAUNCH THICKENING DIAGRAM" SHOWN ON DESIGN SHEET /A30/ OR /A30a/ AND MAY BE ADDITIONALLY INCREASED OR DECREASED TO COMPENSATE FOR CONSTRUCTION INACCURACIES. THE MAXIMUM HAUNCH ALLOWED IS 75 mm AND THE MINIMUM HAUNCH ALLOWED IS 0.

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

CROSS-SECTION AT STEEL SPAN

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

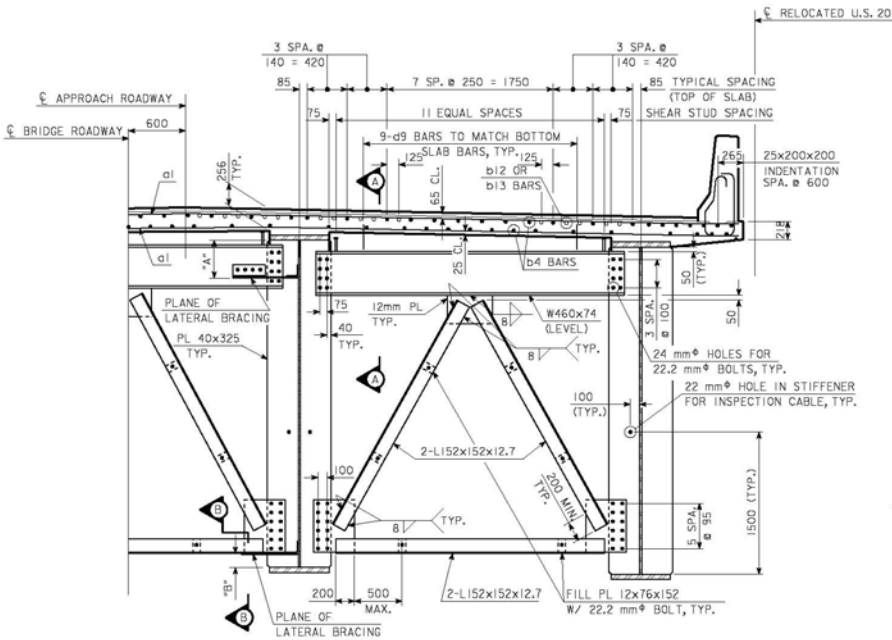
DESIGN SHEET NO. /A36/ OF ____ FILE NO. 29212 DESIGN NO. 199

DESIGNED BY RGB CHECKED BY EDY/DMR
DETAILED BY CAT CADD FILE

HARDIN COUNTY — PROJECT NUMBER

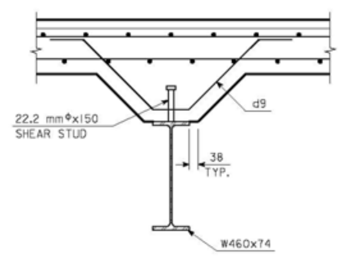
STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$

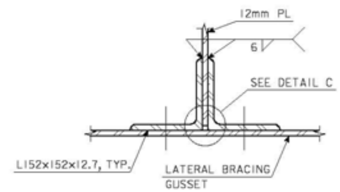


HALF SECTION AT PIERS 2, 3, 4 AND 5

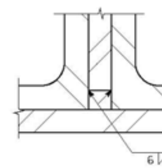
NOTE:
HALF SECTION IS SHOWN NEAR PIER 2. HALF SECTION NEAR PIERS 3, 4, AND 5 IS SIMILAR.
TOP AND BOTTOM LATERAL BRACING NOT SHOWN FOR CLARITY.



SECTION A-A



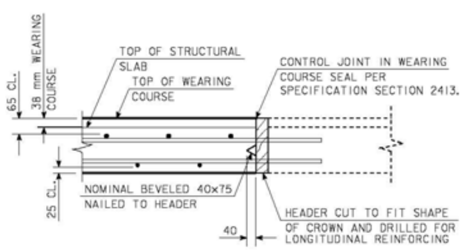
SECTION B-B



DETAIL C

DIMENSIONS

LOCATION	"A"	"B"
GIRDER A & H	——	246
GIRDER B & G	376	174
GIRDER C & F	400	150
GIRDER D & E	——	222



TRANSVERSE SLAB CONSTRUCTION JOINT

SUPERSTR. ESTIMATED QUANTITIES

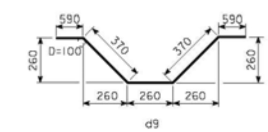
ITEM	UNIT	WESTBOUND QUANTITY	EASTBOUND QUANTITY
STRUCTURAL CONCRETE	M3	1366.2	1366.2
REINFORCING STEEL-EPOXY COATED	kg	300,412	300,412
STRUCTURAL STEEL	kg	2,105,035	2,105,035

NOTE:
FOR STEEL SUPERSTRUCTURE NOTES, SEE DESIGN SHEET /A36/.

SPANS 2, 3, 4, 5 AND 6 EPOXY REINF. BAR LIST-ONE SUPER.

MARK	SIZE	LOCATION	SHAPE	NO.	LENGTH	MASS
a1	20	SLAB TRANS. TOP & BOTTOM	——	5104	12 735	153,074
b1	20	SLAB LONG. TOP & BOTTOM	——	92	8060	1,746
b2	20	SLAB LONG. TOP & BOTTOM	——	90	7100	1,505
b3	20	SLAB LONG. TOP & BOTTOM	——	728	17 000	29,145
b4	20	SLAB LONG. TOP & BOTTOM	——	182	15 280	6,549
b5	20	SLAB LONG. TOP & BOTTOM	——	484	15 800	18,009
b6	20	SLAB LONG. TOP & BOTTOM	——	424	11 310	11,293
b7	20	SLAB LONG. TOP & BOTTOM	——	546	14 210	18,272
b8	20	SLAB LONG. TOP & BOTTOM	——	182	17 680	7,578
b9	20	SLAB LONG. TOP & BOTTOM	——	182	12 760	5,469
b10	20	SLAB LONG. TOP & BOTTOM	——	228	17 860	9,590
b11	20	SLAB LONG. TOP & BOTTOM	——	90	10 360	2,196
b12	20	SLAB LONG. TOP AT PIERS	——	30	14 320	1,012
b13	20	SLAB LONG. TOP AT PIERS	——	30	13 360	944
b14	20	SLAB LONG. TOP AT PIERS	——	30	13 230	935
b15	20	SLAB LONG. TOP AT PIERS	——	30	14 190	1,003
b16	20	SLAB LONG. TOP AT PIERS	——	30	16 720	1,181
b17	20	SLAB LONG. TOP AT PIERS	——	30	15 760	1,113
b18	20	SLAB LONG. TOP AT PIERS	——	30	14 680	1,037
b19	20	SLAB LONG. TOP AT PIERS	——	30	15 640	1,105
b20	20	SLAB LONG. AT DRAINS	——	90	5000	1,060
b21	20	SLAB LONG. AT DRAINS	——	60	2750	389
b22	20	SLAB LONG. AT DRAINS	——	135	1410	448
b23	20	SLAB LONG. AT DRAINS	——	60	1750	247
d9	15	SLAB PIER HAUNCH		108	2070	351
		1 BARRIER CURB - SEE BARRIER CURB SHEET				9717
		1 BARRIER RAIL - SEE BARRIER RAIL SHEET				15,444
		REINFORCING STEEL - EPOXY COATED TOTAL (kg)				300,412

BENT BAR DETAILS



CONCRETE PLACEMENT QUANTITIES

LOCATION	WESTBOUND m^3	EASTBOUND m^3
SECTION 2	111.9	111.9
SECTION 3	208.7	208.7
SECTION 5	111.9	111.9
SECTION 6	208.7	208.7
SECTION 7	154.1	154.1
SECTION 8	154.1	154.1
SECTION 9	134.8	134.8
SECTION 10	134.8	134.8
SECTION 11	147.2	147.2

SUPERSTRUCTURE CONTRACT
DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

CROSS-SECTION @ STEEL SPANS

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION
DESIGN SHEET NO. /A37/ OF ____ FILE NO. 29212 DESIGN NO. 199

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COUNTY | PROJECT NUMBER

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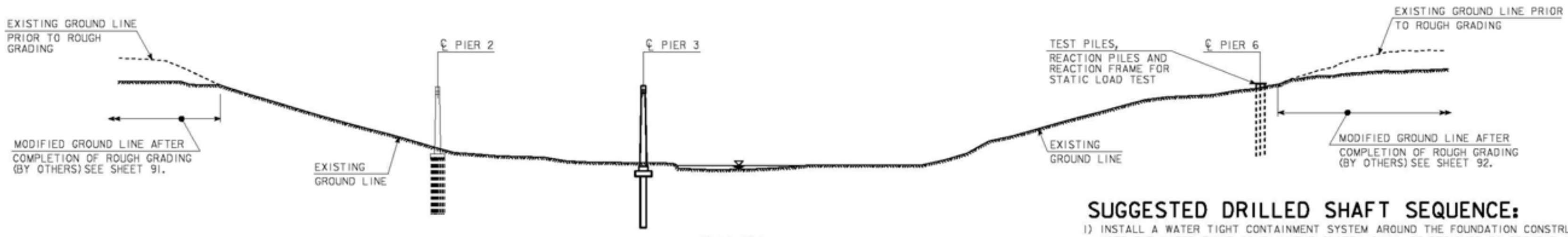
IOWA	7			



STAGE - 1

1) PREPARE SITE AS REQUIRED FOR CONSTRUCTION OF PIERS 2 AND 3 AND WITHIN THE LIMITS OF THE ENVIRONMENTAL CONTROL PLAN ON DESIGN SHEET /A55/ AND IN THE SPECIAL PROVISIONS.

   NOTE:
   THE CONTRACTOR SHALL IMMEDIATELY IMPLEMENT TEMPORARY DRAINAGE AND EROSION CONTROL MEASURES AS NOTED ON DESIGN SHEET /A55/ AND IN THE SPECIAL PROVISIONS.

2) CONSTRUCT PIERS 2 AND 3.

   NOTE:
   PROVIDE TEMPORARY WATER-TIGHT PROTECTION FOR ANCHOR BOLT WELLS.

3) UPON COMPLETION OF THE CONSTRUCTION OF PIERS 2 AND 3, THE CONTRACTOR SHALL RESTORE THE SITE SURROUNDING THE PIERS TO ORIGINAL GRADES AND INSTALL PERMANENT EROSION CONTROL MEASURES EXCEPT AS NOTED BELOW:

   a) THE IMPROVEMENTS MADE TO THE ACCESS ROAD AND THE ZONE TYPE C IMPROVEMENTS AROUND & BETWEEN PIERS 2 AND 3 AS SHOWN ON DESIGN SHEET /A55/ SHALL REMAIN IN PLACE TO ALLOW THE SUPERSTRUCTURE CONTRACTOR ACCESS TO THE SITE.

4) PERFORM STATIC LOAD TEST @ PIER 6, AS NOTED IN THE SPECIAL PROVISIONS.

GENERAL ERECTION SEQUENCE NOTES:

THE FOLLOWING ASSUMPTIONS WERE USED TO DEVELOP THE ERECTION SEQUENCE.

1) ALL EXISTING GROUND LINES MODIFIED BY THE CONTRACTOR TO FACILITATE CONSTRUCTION ACTIVITIES SHALL BE RESTORED TO EXISTING GRADES UNLESS APPROVED OTHERWISE BY THE ENGINEER.

2) THE CONTRACTOR MUST CONSTRUCT THE BRIDGE WITHIN THE GUIDELINES INDICATED ON THE ENVIRONMENTAL CONTROL PLANS AND IN THE SPECIAL PROVISIONS. DURING CONSTRUCTION, THE CONTRACTOR MAY PROPOSE ADJUSTMENTS TO FACILITATE CONSTRUCTION ACTIVITIES. THESE ADJUSTMENTS SHALL BE PER A CASE BY CASE BASIS AND SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW.

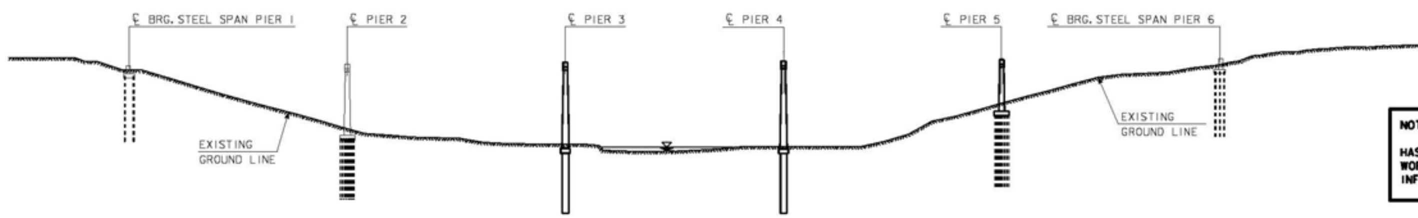
3) ALLOWABLE CONSTRUCTION ACTIVITIES SHALL BE RESTRICTED DURING CERTAIN TIMES OF THE YEAR AS NOTED IN SPECIAL PROVISIONS.

> THE SUGGESTED ERECTION SEQUENCE SHOWN REPRESENTS THE SEQUENCE OF CONSTRUCTION ASSUMED IN THE DESIGN OF THE GIRDER SYSTEM. THE CONTRACTOR MAY PROPOSE AN ALTERNATIVE SEQUENCE WITHIN THE "DETAILED ERECTION SEQUENCE".
>
> WHETHER THE CONTRACTOR PROPOSES AN ALTERNATIVE SEQUENCE OF CONSTRUCTION OR ONE SIMILAR TO THAT SHOWN ON THIS DRAWING, THE CONTRACTOR SHALL PREPARE A "DETAILED ERECTION SEQUENCE", INCLUDING COMPUTATIONS AND DETAILED DRAWINGS, TO BE SUBMITTED TO THE ENGINEER FOR REVIEW AS PART OF THE SHOP DRAWINGS.
>
> THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ERECTION SEQUENCE SHOWN IN THE AGREED UPON "DETAILED ERECTION SEQUENCE".

SUGGESTED DRILLED SHAFT SEQUENCE:

1) INSTALL A WATER TIGHT CONTAINMENT SYSTEM AROUND THE FOUNDATION CONSTRUCTION AREA.
   NOTE: THE WATER TIGHT CONTAINMENT SYSTEM SHALL BE DESIGNED TO PREVENT WATER FLOW TO THE RIVER.
2) DRILL SHAFT TO ELEV. 284.00 USING SLURRY AND TEMPORARY CASING.
3) INSTALL PERMANENT CASING TO BOTTOM OF DRILLED HOLE AND ADVANCE CASING DOWN TO THE TOP OF ROCK BY INCREMENTALLY SCREWING OR DRILLING CASING AND REMOVING OVERBURDEN FROM INSIDE THE CASING.
4) SCREW PERMANENT CASING INTO ROCK A MINIMUM DEPTH AS INDICATED IN THE CONTRACT DOCUMENTS OR AS APPROVED BY THE ENGINEER.
5) REPLACE SLURRY INSIDE CASING WITH WATER.
6) DRILL ROCK SOCKET TO PLAN DEPTH OR AS APPROVED BY ENGINEER.
7) CLEAN SHAFT AND ROCK SOCKET BY AIRLIFT AND CAMERA INSPECT ROCK SOCKET.
8) INSTALL REINFORCING CAGE.
9) PLACE CONCRETE IN SHAFT BY UNDERWATER METHODS.
10) GROUT ANNULUS AROUND PERMANENT CASING AND REMOVE TEMPORARY CASING.
11) PERFORM CROSSHOLE SONIC LOGGING.
12) INSTALL TEMPORARY SHEET PILING FOR COFFERDAM.
13) EXCAVATE AS REQUIRED WITHIN COFFERDAM AND INSTALL SEAL.
14) CLEAN TOP OF DRILLED SHAFT AS REQUIRED TO ACHIEVE SOUND CONCRETE.
15) FORM AND CONSTRUCT DRILLED SHAFT CAP.
16) BACKFILL AS REQUIRED.
17) REMOVE COFFERDAMS.

D-11



NOTE: THE ERECTION SEQUENCE SHOWN ON THIS SHEET HAS BEEN INCLUDED IN THE SUBSTRUCTURE CONTRACT. WORK NOTED ON THIS SHEET HAS BEEN PROVIDED FOR INFORMATION ONLY AND IS N.I.C.

STAGE - 2

1) PREPARE SITE AS REQUIRED FOR CONSTRUCTION OF PIERS 4, 5 AND 6 AND WITHIN THE LIMITS OF THE ENVIRONMENTAL CONTROL PLAN ON DESIGN SHEET /A56/ AND IN THE SPECIAL PROVISIONS.

   NOTE:
   THE CONTRACTOR SHALL IMMEDIATELY IMPLEMENT TEMPORARY DRAINAGE AND EROSION CONTROL MEASURES AS NOTED ON DESIGN SHEET /A56/ AND IN THE SPECIAL PROVISIONS.

2) CONSTRUCT PIERS 4 AND 5.

   NOTE:
   PROVIDE TEMPORARY WATER-TIGHT PROTECTION FOR ANCHOR BOLT WELLS.

3) CONSTRUCT PIER 6.

   NOTE:
   PROVIDE TEMPORARY WATER-TIGHT PROTECTION FOR ANCHOR BOLT WELLS.

4) CONSTRUCT PIER 1.

   NOTE:
   PROVIDE TEMPORARY WATER-TIGHT PROTECTION FOR ANCHOR BOLT WELLS.

5) UPON COMPLETION OF THE CONSTRUCTION OF PIERS 4, 5 AND 6, THE CONTRACTOR SHALL RESTORE THE SITE SURROUNDING THE PIERS TO ORIGINAL GRADES AND INSTALL PERMANENT EROSION CONTROL MEASURES EXCEPT AS NOTED BELOW:

   a) THE NEW CAUSEWAY, THE IMPROVEMENTS MADE TO THE EXISTING DRAW AND ZONE TYPE C IMPROVEMENTS AROUND PIERS 4 AND 5 AS SHOWN ON DESIGN SHEET /A56/ SHALL REMAIN IN PLACE TO ALLOW THE SUPERSTRUCTURE CONTRACTOR ACCESS TO THE SITE.

SUPERSTRUCTURE CONTRACT

DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

SUGGESTED SUBSTR. EREC. SEQ.

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

DESIGN SHEET NO. ____ OF ____ FILE NO. 29212 DESIGN NO. 199

PLOTTED: $$USERNAME$$ $$DATE AND TIME$$ $$DGNSPEC$$

DESIGNED BY DMR CHECKED BY DMR
DETAILED BY MBG CADD FILE

HARDIN COUNTY | PROJECT NUMBER

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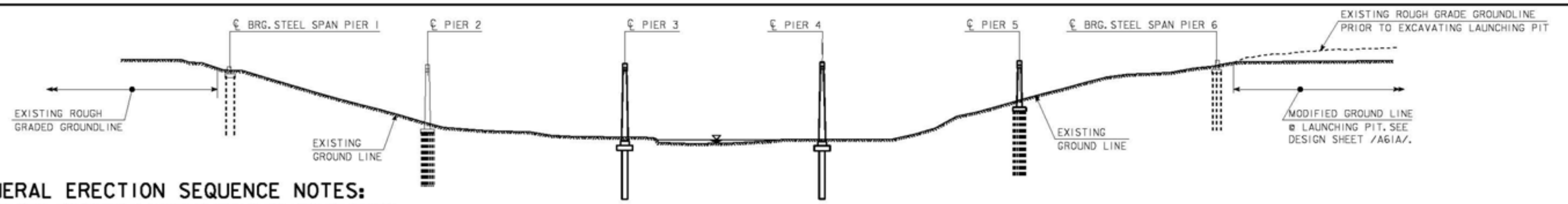


GENERAL ERECTION SEQUENCE NOTES:

THE FOLLOWING ASSUMPTIONS WERE USED TO DEVELOP THE ERECTION SEQUENCE.

1) ALL EXISTING GROUND LINES MODIFIED BY THE CONTRACTOR TO FACILITATE CONSTRUCTION ACTIVITIES SHALL BE RESTORED TO EXISTING GRADES UNLESS APPROVED OTHERWISE BY THE ENGINEER.

2) THE CONTRACTOR MUST CONSTRUCT THE BRIDGE WITHIN THE GUIDELINES INDICATED ON THE ENVIRONMENTAL CONTROL PLANS AND IN THE SPECIAL PROVISIONS. DURING CONSTRUCTION, THE CONTRACTOR MAY PROPOSE ADJUSTMENTS TO FACILITATE CONSTRUCTION ACTIVITIES. THESE ADJUSTMENTS SHALL BE PER A CASE BY CASE BASIS AND SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW.

3) ALLOWABLE CONSTRUCTION ACTIVITIES SHALL BE RESTRICTED DURING CERTAIN TIMES OF THE YEAR, AS NOTED IN THE SPECIAL PROVISIONS.

4) THE SUGGESTED ERECTION SEQUENCE ASSUMES THAT THE LAUNCHING OF THE BRIDGE SUPERSTRUCTURES SHALL OCCUR FROM THE EAST (BEHIND PIER 6) WORKING TOWARDS THE WEST (PIER 1). AT THE CONTRACTOR'S OPTION, THE BRIDGE MAY BE LAUNCHED FROM THE WEST (BEHIND PIER 1) TOWARDS THE EAST (PIER 6) HOWEVER IT SHOULD BE NOTED THAT THE MOST SEVERE WINTER CONSTRUCTION RESTRICTIONS REVOLVE AROUND THE BALD EAGLE'S ROOSTING AREA IN THE DRAW AS SHOWN ON DESIGN SHEET /A55/ AND NOTED IN THE SPECIAL PROVISIONS.

5) THE SUGGESTED ERECTION SEQUENCE ASSUMES THAT THE BRIDGE SUPERSTRUCTURES WILL BE ERECTED SIMULTANEOUSLY BUT LAUNCHED INDEPENDENT OF EACH OTHER (I.E. THE 4-GIRDER SYSTEM OF THE EAST BOUND LANES SHALL BE LAUNCHED ONE SPAN AT A TIME SEPARATELY FROM THE 4-GIRDER SYSTEM OF THE WEST BOUND LANES). THE CONTRACTOR, AT HIS OPTION, MAY REVIEW THE CONSTRUCTION SCHEDULE, RE-USE OF ROLLERS, AND AVAILABLE WORKING ROOM FOR STEEL ERECTION IN THE LAUNCHING PIT TO DETERMINE WHETHER THE LAUNCHING OF ONE 4-GIRDER SYSTEM COULD BE COMPLETED PRIOR TO LAUNCHING THE OTHER.

6) THE SUGGESTED ERECTION SEQUENCE ASSUMES THAT THE PERMANENT POT BEARINGS ARE POSITIONED ATOP THE PIERS PRIOR TO THE LAUNCHING OF THE GIRDERS. THE CONTRACTOR MAY PROPOSE ALTERNATE SEQUENCES OF INSTALLATION PROVIDED THE PROPOSED SEQUENCE CLEARLY INDICATES THE METHOD AND DETAILS FOR TRANSFERING LOADS FROM THE TEMPORARY BEARINGS TO THE PERMANENT POT BEARINGS.

7) A REACTION FOUNDATION BLOCK HAS BEEN SHOWN TO FACILITATE THE "PUSHING" OF THE GIRDERS THE LAUNCHING OPERATIONS. AN ANTICIPATED TOTAL MAXIMUM HORIZONTAL REACTION PER 4-GIRDER SYSTEM OF 3650 kN (410 TONS) HAS BEEN ASSUMED FOR DESIGN.

NOTE:
AT THE CONTRACTOR'S OPTION THE GIRDERS MAY BE "PULLED" AGAINST PIER 6. PROVIDED THE CONTRACTOR CHECKS THE COMPONENTS OF PIER 6 FOR OVER-STRESS DUE TO THIS OPTION AND SUBMITS COMPUTATIONS TO THE ENGINEER FOR REVIEW.

8) A LAUNCHING SKID WAS UTILIZED TO LIGHTEN THE LEAD CANTILEVER SPAN AS WELL AS TO GUIDE THE DEFLECTED STRUCTURE OVER THE TOPS OF PIERS. THE CONTRACTOR MAY PROPOSE ALTERNATIVE METHODS OF CONTROLLING THE DEFLECTION OF THE LEAD CANTILEVER.

9) ALL CONCRETE FORMS AND REBAR ARE ASSUMED TO BE INSTALLED ON GIRDERS PRIOR TO LAUNCHING EXCEPT FOR THE FIRST 44 METERS OF SPAN 1. NO ADDITIONAL DEAD LOAD EXCEPT FOR THE GIRDERS AND THE ASSOCIATED BRACING ARE ASSUMED FOR THE FIRST 44 METERS OF SPAN 1.

10) THE ANTICIPATED MAXIMUM VERTICAL GIRDER REACTION DURING LAUNCHING OF 2450 kN (275 TONS) HAS BEEN ASSUMED FOR DESIGN OF THE PIER STRUCTURES AND LAUNCHING FRAMES.

STAGE - 1

1) BEGIN EXCAVATION OF LAUNCHING PIT.

NOTE:
THE CONTRACTOR SHALL IMMEDIATELY IMPLEMENT TEMPORARY DRAINAGE AND EROSION CONTROL MEASURES WITHIN THE LAUNCHING PIT AS NOTED ON DESIGN SHEET /A61A/ AND IN THE SPECIAL PROVISIONS. SEE DESIGN SHEET /A48c/ FOR PHASE 2 "TIE-IN" TO PERMANENT DRAINAGE SYSTEM.

THE CONTRACTOR SHALL COORDINATE THE STORAGE OF THE EXCAVATED MATERIAL RIGHT OF STATION 352+00.000, WITH THE ENGINEER.

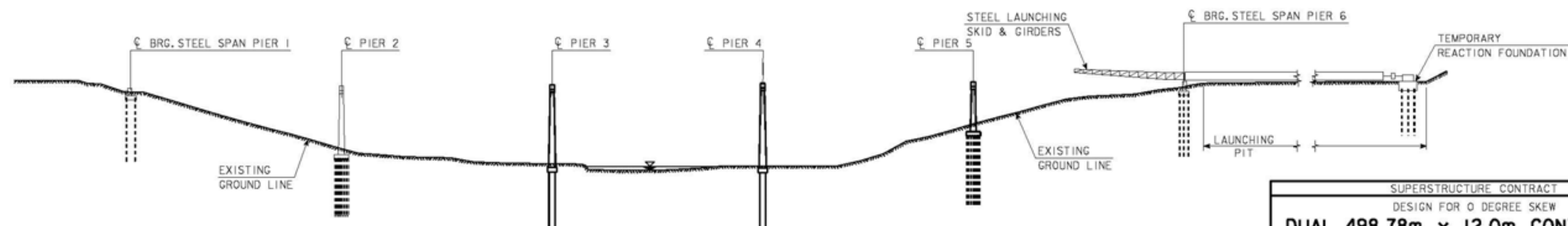
2) INSTALL AND PROTECT PERMANENT POT BEARINGS, TEMPORARY ERECTION FRAMES AND ROLLER BEARINGS ON PIERS 1 THRU 6.

NOTE:
DO NOT GROUT IN ANCHOR BOLTS FOR PERMANENT POT BEARINGS.
MAINTAIN WATER TIGHT PROTECTION OF ANCHOR BOLT WELLS.
POT BEARINGS ARE NOT TO BE USED DURING LAUNCHING OPERATIONS EXCEPT AS NOTED IN THESE PLANS.

THE SUGGESTED ERECTION SEQUENCE SHOWN REPRESENTS THE SEQUENCE OF CONSTRUCTION ASSUMED IN THE DESIGN OF THE GIRDER SYSTEM. THE CONTRACTOR MAY PROPOSE AN ALTERNATIVE SEQUENCE WITHIN THE "DETAILED ERECTION SEQUENCE".

WHETHER THE CONTRACTOR PROPOSES AN ALTERNATIVE SEQUENCE OF CONSTRUCTION OR ONE SIMILAR TO THAT SHOWN ON THIS DRAWING, THE CONTRACTOR SHALL PREPARE A "DETAILED ERECTION SEQUENCE", INCLUDING COMPUTATIONS AND DETAILED DRAWINGS, TO BE SUBMITTED TO THE ENGINEER FOR REVIEW AS PART OF THE SHOP DRAWINGS.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ERECTION SEQUENCE SHOWN IN THE AGREED UPON "DETAILED ERECTION SEQUENCE".



STAGE - 2

1) CONSTRUCT TEMPORARY REACTION FOUNDATION AT END OF LAUNCHING PIT.

2) BEGIN ERECTING GIRDERS IN LAUNCHING PIT.

3) ERECT STEEL LAUNCHING SKID.

SUPERSTRUCTURE CONTRACT
DESIGN FOR 0 DEGREE SKEW

DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS

1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN

SUGGESTED ERECTION SEQUENCE

STATION: 338+20.657

HARDIN COUNTY

IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION

DESIGN SHEET NO. /A50/ OF ____ FILE NO. 29212 DESIGN NO. 199

DESIGNED BY DMR CHECKED BY DMR
DETAILED BY MBG CADD FILE

HARDIN COUNTY PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

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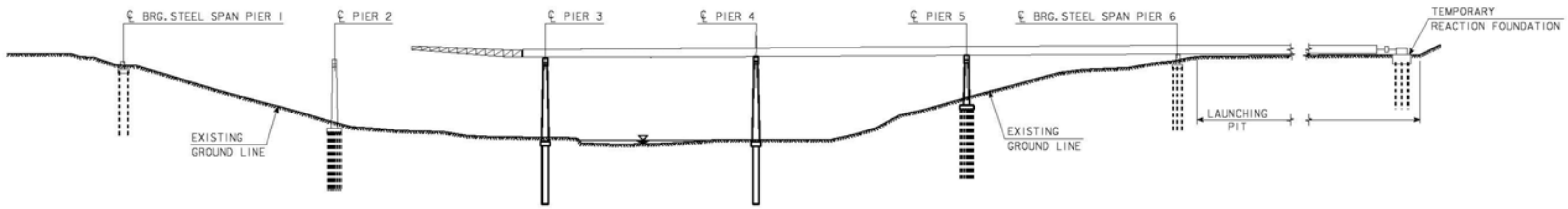
D-13



STAGE - 3

1) BEGIN SPAN BY SPAN LAUNCHING OF GIRDERS BY STAGGERING LAUNCH CYCLES OF THE EAST AND WEST BOUND 4-GIRDER SYSTEMS.

NOTE:
THE GIRDERS ARE TO BE LAUNCHED IN ONE SPAN INCREMENTS TO MINIMIZE THE EXPOSURE TIME OF THE FREE CANTILEVER.

UPON COMPLETION OF EACH ONE SPAN LAUNCH INCREMENT, AND PRIOR TO LAUNCHING THE SUBSEQUENT 4-GIRDER SYSTEM, THE GIRDERS OF THE PREVIOUS LAUNCH SHALL BE LOWERED ONTO (BUT NOT FASTENED TO) THE PERMANENT POT BEARINGS TRANSFERRING THE VERTICAL REACTION AT EACH GIRDER LINE TO THE CENTERLINE OF BEARING AT EACH PIER. LOWER THE ROLLERS AS REQUIRED TO "UNLOAD" THE VERTICAL REACTION BUT MAINTAIN ENGAGEMENT OF THE GUIDES AT THE BOTTOM FLANGES OF THE EXTERIOR GIRDERS.

THE SUGGESTED ERECTION SEQUENCE SHOWN REPRESENTS THE SEQUENCE OF CONSTRUCTION ASSUMED IN THE DESIGN OF THE GIRDER SYSTEM. THE CONTRACTOR MAY PROPOSE AN ALTERNATIVE SEQUENCE WITHIN THE "DETAILED ERECTION SEQUENCE".

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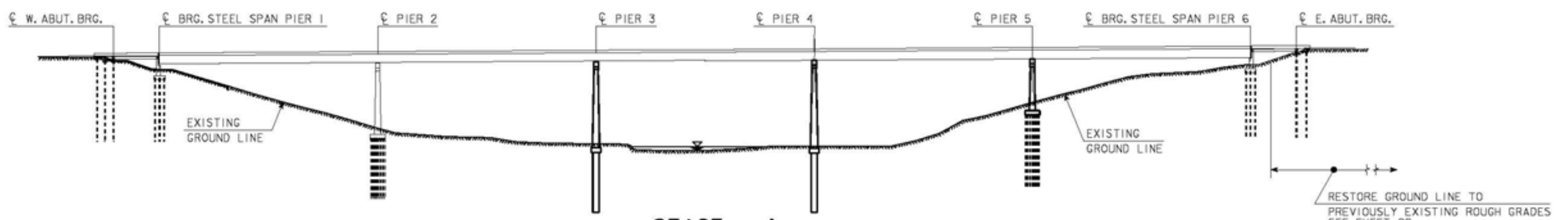

STAGE - 4

1a) ONCE THE GIRDERS HAVE REACHED THE FINAL IN-PLACE POSITION, TRANSFER LOADS FROM TEMPORARY ROLLER BEARINGS TO THE PERMANENT POT BEARINGS. GROUT IN ANCHOR BOLTS.

1b) INCREMENTALLY DIS-ASSEMBLE LAUNCHING NOSES AS GIRDERS ARE LAUNCHED OVER THE FINAL SPAN.

2) REMOVE CAUSEWAY (ZONE TYPE F ON DESIGN SHEET /A56/) CONSTRUCTED TO ACCESS PIER 4.

3) BACKFILL LAUNCHING PIT AND RESTORE GROUNDLINE TO PREVIOUSLY EXISTING ROUGH GRADES.

4) CONSTRUCT THE ABUTMENTS.

5) INSTALL FINAL CONNECTIONS TO THE PERMANENT DRAINAGE SYSTEM. SEE SHEET 89 AND DESIGN SHEETS /A48c/ AND /A48d/.

6) INSTALL MACADAM STONE.

7) ERECT THE PRECAST JUMPSPAN BEAMS.

8) CONSTRUCT THE SLAB, WEARING COURSE AND MEDIAN BARRIERS.

SUPERSTRUCTURE CONTRACT

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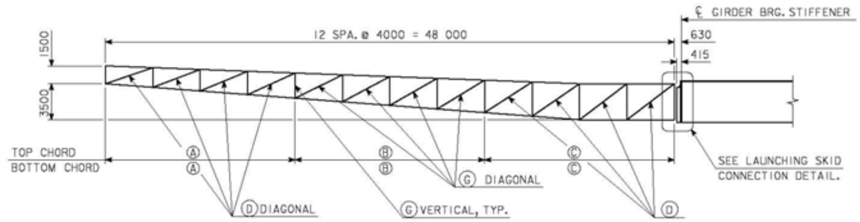
HARDIN COUNTY

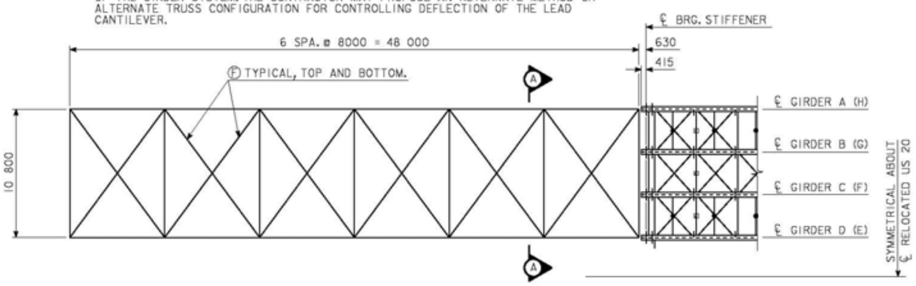
PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

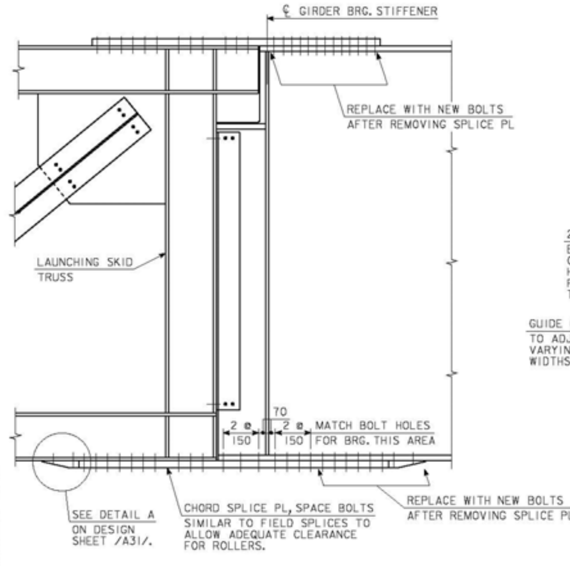


LAUNCHING SKID ELEVATION

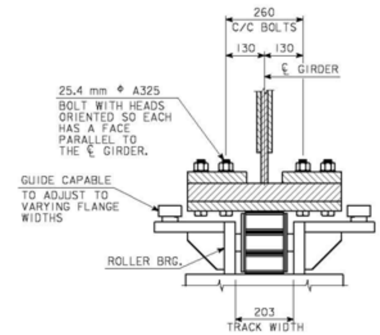
NOTE:
THE LAUNCHING SKID SHOWN REPRESENTS THAT WHICH WAS USED IN THE DESIGN OF THE GIRDER SYSTEM. THE CONTRACTOR MAY PROPOSE AN ALTERNATE METHOD OR ALTERNATE TRUSS CONFIGURATION FOR CONTROLLING DEFLECTION OF THE LEAD CANTILEVER.



PLAN VIEW OF LAUNCHING

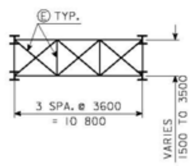


LAUNCHING SKID CONNECTION DETAIL



PROPOSED LAUNCHING DETAIL AT FIELD SPLICES

NOTE:
ORIENTATION OF BOLT HEADS OF BOLTS FARTHEST FROM ℄ OF WEB IS NOT CRITICAL.

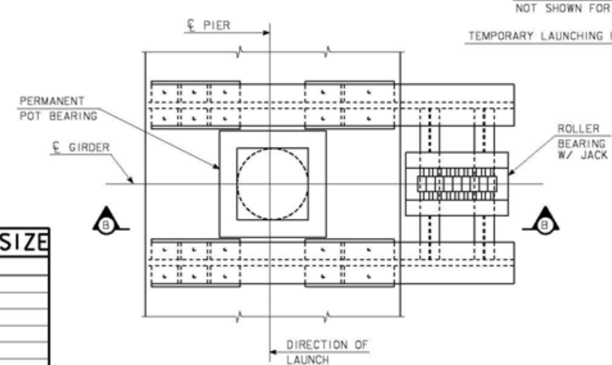


SECTION A-A

LAUNCHING SKID MEMBER SIZE

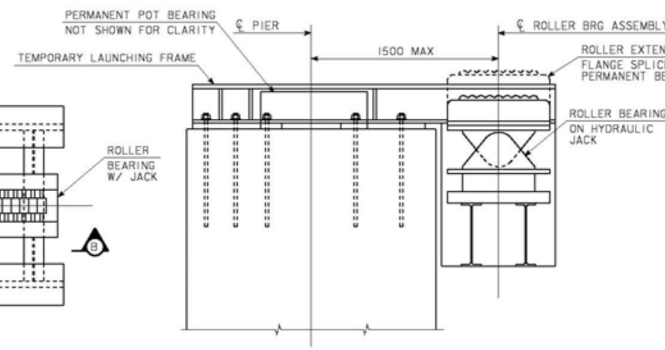
MARK	SHAPE
A	W360x179
B	W360x196
C	W360x237
D	2-WT180x39.5
E	W360x64
F	W360x134
G	2-WT180x32

NOTE:
LAUNCHING SKID MEMBER SIZES ARE PROVIDED FOR BIDDING ONLY. THE CONTRACTOR SHALL SUBMIT A DETAILED SET OF DESIGN CALCULATIONS FOR THE TRUSS CONFIGURATION DEVELOPED TO SATISFY CONTRACTOR'S MEANS AND METHODS. THE DESIGN CALCULATIONS SHALL BE SUBMITTED TO THE ENGINEER FOR REVIEW.



PLAN VIEW-LAUNCHING FRAME

NOTE:
THE LAUNCHING FRAME SHOWN REPRESENTS THAT WHICH WAS ASSUMED IN THE DESIGN OF THE PIERS WHEN CONSIDERING THE AFFECTS OF THE LAUNCHING. THE CONTRACTOR MAY PROPOSE ALTERNATE SYSTEMS, CONFIGURATIONS, OR METHODS FOR THE LAUNCHING EQUIPMENT.



SECTION B-B

NOTES:
ROLLERS SHOWN IN RETRACTED POSITION.
ROLLERS UTILIZED AT THE EXTERIOR GIRDERS OF EACH 4-GIRDER SYSTEM SHALL CONTAIN GUIDES THAT WILL ALLOW LONGITUDINAL (ALONG THE LENGTH OF THE BRIDGE) MOVEMENTS BUT RESTRICT TRANSVERSE MOVEMENT (PERPENDICULAR TO DECK).

THIS DRAWING IS NOT TO BE CONSIDERED AS PART OF THE CONTRACT DOCUMENTS. THE SUGGESTED LAUNCHING DETAILS ARE FOR INFORMATION ONLY. THE CONTRACTOR MAY PROPOSE ALTERNATIVE DETAILS.

WHETHER THE CONTRACTOR PROPOSES AN ALTERNATIVE DETAILS OR ONE SIMILAR TO THAT SHOWN ON THIS DRAWING, THE CONTRACTOR SHALL PREPARE A DETAILED ERECTION SEQUENCE (DES), INCLUDING CALCULATIONS AND DETAILED DRAWINGS, TO BE SUBMITTED TO THE ENGINEER FOR REVIEW AS PART OF THE SHOP DRAWINGS.

THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE ERECTION SEQUENCE IN ITS ENTIRETY. THE CONTRACTOR'S BID SHALL BE BASED SOLELY UPON THE DETAILED ERECTION SEQUENCE PROPOSED BY THE CONTRACTOR.

SUPERSTRUCTURE CONTRACT
DESIGN FOR 0 DEGREE SKEW
DUAL 498.78m x 12.0m CONT. WELDED GIRDER BRIDGE w/ PRECAST JUMPSPANS
1 - 18.395 m SPAN; 5 - 92.000 m SPANS; 1 - 18.395 m SPAN
SUGGESTED LAUNCHING DETAILS
STATION: 338+20.657
HARDIN COUNTY
IOWA DEPARTMENT OF TRANSPORTATION - PROJECT DEVELOPMENT DIVISION
DESIGN SHEET NO. /A51/ OF ____ FILE NO. 29212 DESIGN NO. 199

PLOTTED: $$USERNAME$$ $$DGNSPEC$$
PLOTTED: $$DATE AND TIME$$

DESIGNED BY DMR CHECKED BY DMR
DETAILED BY MBG CADD FILE

HARDIN COUNTY PROJECT NUMBER

STATE	FHWA REGION	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
IOWA	7			

APPENDIX E

Example Specifications for Steel Erection by Launching

The following example Special Provision is from the U.S. 20 Iowa River Bridge incremental launching project in Iowa. The specification is for erection of the steel superstructure by launching. This specification is provided as an example which may be useful to owners considering a special erection process and addresses the types of information the contractor may be required to submit in support of his erection engineering proposal.

SPECIAL PROVISION

For
STRUCTURAL STEEL ERECTION

THE STANDARD SPECIFICATIONS, SERIES OF 1995, ARE AMENDED BY THE FOLLOWING ADDITIONS AND MODIFICATIONS. THIS IS A SPECIAL PROVISION AND IT SHALL PREVAIL OVER PROVISIONS OF THE STANDARD SPECIFICATIONS IF THERE IS CONFLICT.

Steel.01 DESCRIPTION

This work shall consist of, but is not limited to, the fabrication, transporting, shop and field erection, the subsequent launch operation of the structural steel superstructure for the spans from Pier 1 to Pier 6 and the excavation and restoration of launching pit in accordance with the contract documents, applicable portions of **Standard Specifications, Section 2408, Special Provision for Weathering Steel,** and as specified herein.

A suggested erection sequence representing the sequence of construction assumed in the design of the girder system has been included as part of the contract documents. Utilization of this suggested erection sequence as presented is not mandatory.

All work shall be done in accordance with the approved Detailed Erection Sequence (**Article Steel.02**), as shown in the contract documents, the **Standard Specifications,** its supplements, as specified herein and as directed by the Engineer. All design computations, plans, methods and procedures prepared for submittal shall be prepared and sealed by a structural engineer licensed in the State of Iowa.

The final agreed upon Detailed Erection Sequence shall prevail over the Standard or Supplemental Specifications where there is conflict. Submittal of erection methods other than launching shall demonstrate complete adherence to environmental regulations as specified in the **Special Provision for ENVIRONMENTAL PROTECTION** and will require complete agreement by the Engineer.

Erection assumptions have been provided with the suggested erection sequence to permit analysis of the structure for the effect thereof during launching operations. If the Contractor elects to use the suggested erection sequence provided, he shall ascertain for himself the practicality thereof and shall assure complete responsibility for the Detailed Erection Sequence.

Steel.02 DETAILED ERECTION SEQUENCE

Prior to the pre-construction conference, the Contractor shall prepare and submit a Detailed Erection Sequence (DES) for the structural steel work along with or within the project Detailed

Construction Plan (DCP) for review as specified in the **Special Provision for ENVIRONMENT PROTECTION**. The DES may be similar to the suggested erection sequence or may be an alternate sequence. All DES AND DCP documents shall be prepared and sealed by a structural engineer licensed in the State of Iowa prior to submittal. The DES and DCP shall be submitted in advance of the date schedule for the pre-construction conference in order to afford the Engineer 30 calendar days of review time. Work shall not be started prior to the receipt by the Contractor of the agreed upon DES and DCP documents. The review of the DES and DCP documents by the Engineer shall not relive the Contractor of the full responsibility for the safety and adequacy of the work.

The proposed DES shall include, but not be limited to:

1. Detailed design of all launching/erection equipment, falsework, temporary bracing and other items as required for launching/erection, fabrication and installation procedures for all launching/erection equipment, materials, excavation and subsequent recompaction of the launching pit and all other associated mobilization.

2. Methods and procedures for superstructure steel placement including:

    a. Sequence and manner of steel assembly for launching.
    b. Sequence and methods of making diaphragm bolted connections during and after launching operations and after deck is poured.
    c. Sequence for deck form and deck reinforcement installation.
    d. Sequence and manner for launching girder systems.
    e. Details indicating provisions for stability of the girder systems during the various stages of the launch.
    f. Sequence for installing temporary launching equipment and permanent bearings, removing temporary launching equipment and transferring loads to permanent bearings.

3. Factors of safety for all applicable equipment and procedures to be used as agreed upon by the Engineer. These factors of safety shall be specified by the structural engineer that will submit the DES and DCP.

4. All computations consisting of, but not limited to:

    a. The expected bearing, shear, compression and tensile stresses as may be produced within the pre-assembled structural steel girder system, launching skid or temporary members due to launching operations.
    b. Minimum and maximum vertical and horizontal reactions at all support locations that will occur during launching.
    c. Verification that the permanent substructure units are not overstressed during launching (temporary bracing of the permanent substructure units shall be allowed if required per computations).
    d. Displacements at nose of launching skid during launching.

5. Methods and procedures for excavating launching pit, hauling excavated material and restoring launching pit upon completion.

The agreed upon DES shall ensure that the erection sequence is coordinated with the steel lay-up to ensure proper hole placement and camber.

Steel.03 CONSTRUCTION REQUIREMENTS

The Contractor is completely responsible for protection of the structural integrity of the superstructure steel from fabrication to final approved placement with concrete deck, wearing course, and cast in place barrier railing and curb. Any damage sustained by structural steel, reinforcement, or concrete deck forms shall be repaired or replaced by the Contractor to the satisfaction of the Engineer at no additional cost to the Iowa DOT. All launching equipment shall be furnished by the Contractor**.**

Any damaged shop painted areas of the superstructure steel after final placement of steel and deck concrete shall be touched up to the satisfaction of the Engineer at the Contractor's expense.

At the Contractor's option, the superstructure steel may be installed without deck forms and deck reinforcement in place prior to launching.

Changes in the approved DES will not be allowed unless approved in writing by the Engineer.

Upon completion of construction operations and Engineer approval of final superstructure placement, all equipment shall be removed and all existing ground lines and site conditions modified by the Contractor to facilitate construction activities shall be restored to undamaged existing condition unless approved otherwise by the Engineer.

Construction activities shall be governed by the guidelines indicated in the Environmental Control Plan sheets included in the contract documents and as specified in the **"Special Provision for ENVIRONMENT PROTECTION"**. The Contractor shall have the option to propose adjustment to these guidelines to facilitate construction activities. These adjustments shall be submitted to the Engineer and reviewed on a case by case basis.

Excavation, hauling, storage, stripping and salvaging of soil at borrow pit, and subsequent restoration of launching pit shall be per the requirements of **Division 21** of the **Standard Specifications.**

Steel.04 MEASUREMENT AND PAYMENT

All costs incurred in complying with this special provision except as specified below herein. are included into the contract unit price, per Kg, for "Structural Steel".

All costs of transporting the welded steel girder segments from the fabrication shop to the site including, but not limited to, shipping and temporary bracing required to stabilize the girders during shipment shall be included in the contract unit price, per lump, sum for "Delivery - Steel Girder Segments".

All costs of deck reinforcement shall be included in the contract unit price, per Kg, for "Reinforcement, Epoxy Coated", and furnished and installed in accordance with **Standard Specifications, Section 2404**.

The cost of deck form placement shall be included with the contract unit price per cubic meter for "Structural Concrete, (Bridge)" and constructed in accordance with the **Standard Specifications, Section 2403**.

All costs of furnishing and submitting the DES and DCP documents shall be incidental to the over-all cost of the project.

All costs of furnishing, fabrication, installation and subsequent removal of the superstructure launching equipment and any other contractor furnished equipment, all costs incurred in launching the girder systems out onto the permanent substructure units except as specified below shall be included in the contract unit price, per lump sum, for “Launching - Steel Superstructure”.

All costs of excavating, hauling, storage, stripping and salvaging of top soil at borrow pit, and subsequent restoration of launching pit upon completion of the launching operation shall be included in the contract unit price, per lump sum, for "Excavation and Restoration of Launching Pit".

Repair of undesirable site conditions created by reaction foundation removal and any other physical or environmental damage sustained by the site due to equipment operation or removal, as determined by the Engineer, or that is subject to the contract environmental regulations shall be repaired to the satisfaction of the Engineer at Contractor expense.