SPLICED GIRDER BRIDGES OVER THE INTRACOASTAL WATERWAY

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ABSTRACT

The Intracoastal Waterway is a 3,000 mile long recreational and commercial waterway along the Atlantic and Gulf Coast States over which PB has designed a number of bridges. During the last decade, the precast spliced girder solution has seen a rise in popularity and has thus been used for a number of these crossings. Vertical and lateral navigational clearances for Intracoastal Waterway (ICW) crossings are such that span lengths from 200 feet to 300 feet are required.

It is the intention of this paper to highlight a few of such spliced girder bridges over the ICW in which PB has designed. Major points are as follows: construction staging, falsework scenarios for precast elements, design issues for individual precast elements, fabrication issues, and post-tensioning.

The result and conclusion of the presentation will be to educate the audience on the use of spliced girders for this application and associated particulars of their construction. The paper will also reiterate the viability of spliced girders as an increasingly popular solution.

Keywords: Spliced Girder, Post-Tensioning, Bulb Tee

INTRODUCTION

Over the past two decades the spliced girder option for bridges over the Intracoastal Waterway has gained significant popularity. The Intracoastal Waterway is a body of water, often between barrier islands and the mainland, which runs 3,000 miles along the Atlantic Coast and Gulf Coast States. This waterway often serves as a route for commercial vessels, thus requiring vertical and lateral clearances regulated by the Coast Guard.

Many aging bascule bridges built in the WWII era are often determined to be replaced by fixed bridges which provide necessary vertical clearance. The most common solution for these fixed bridges is represented by approach spans consisting of prestressed AASHTO bulb tee sections in the range of 120 feet to 150 feet, and a channel crossing requiring a span length of at least 200 feet for vessel navigation. The required channel span may reach up to 300 feet given the skew of the channel and crossing roadway. Required spans of these lengths have generated an excellent solution from the precast industry. Spliced girder bridges used for simple overpasses in the 1980's were essentially modified, using slightly adapted popular precast beam shapes to achieve these ranges. Examples of such bridges can be seen in Figure 1 and Figure 2.

Other bridge types suitable for this span range are steel plate girder, steel box girder and segmental concrete box girder. However concrete spliced girders are often preferred over the others for the following reasons:

- Concrete spliced girders provide better corrosion protection than steel girders.
- Concrete spliced girders provide economy over segmental box girders in this application due to the number of precast segments required to be competitive with other structure types.

Fig. 1 Replacement of existing bridge

Fig. 2 New parallel alignment

DEFINITION AND ASSEMBLY OF THE SPLICED GIRDER SYSTEM

NCHRP Report 517[1] defines a spliced girder as "…a precast prestressed concrete member fabricated in several relatively long pieces (i.e., girder segments) that are assembled into a single girder for the final bridge structure. Post-tensioning is generally used to reinforce the connection between girder segments."

A typical arrangement for a three-span channel crossing is presented in Figure 3. It should be noted this system could be easily adapted to more or less spans, however a three span arrangement provides good basis for description. It can be seen in Figure 3 the system is comprised of three segment types. These consist of pier segments, end segments, and a drop-in segment. The pieces are held in place by a series of falsework and beam hangers (steel strong backs) temporarily until closure pours between segments are poured and post-tensioning is applied, providing a continuous unit.

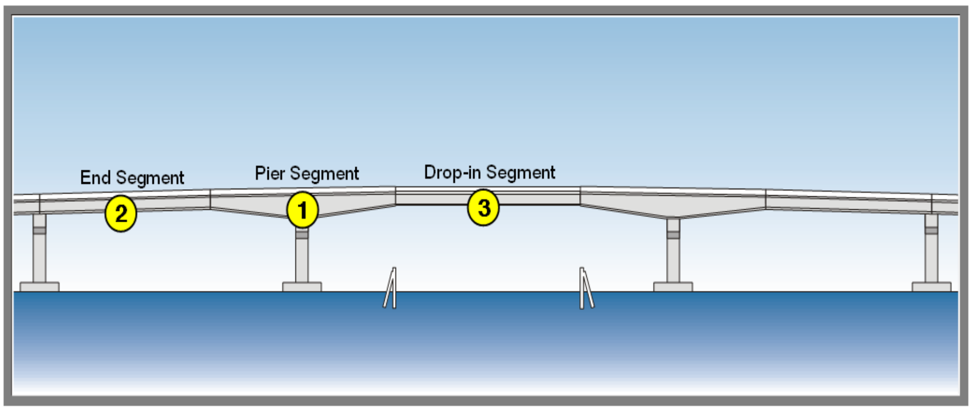
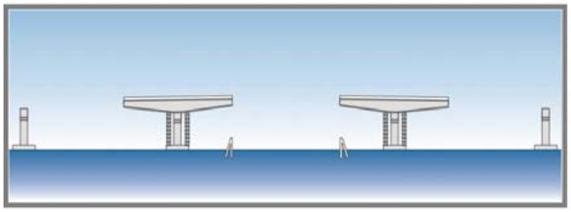
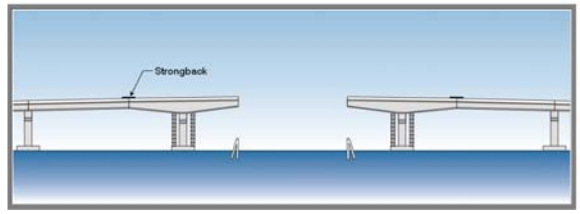


Fig. 3 Typical three span arrangement

Erection of the girder unit assembly is divided into three stages. These can be seen in Figure 4. Stage 1 consists of erecting the pier segments, commonly referred to as "haunched segments" due to often varying in depth. This piece is then stabilized by connection to falsework towers. Although other methods of support are possible such as cables or threaded post-tensioned bars, falsework towers are most common. Stage 2 involves erection of end segments, which are normally hung from the end of the pier segment using beam hangers or "strongbacks". Stage 3 consists of erecting a drop-in segment which connects to the two pier segments, again using a beam hanger system. Placement of this segment will normally balance the reactions at falsework towers, but will induce significant negative bending in the pier segment. It should be noted that falsework towers have only been installed in the flanking spans, or on top of channel pier pile caps, as to keep the navigation span clear for passage of vessels.



Stage 1: Place Pier Segments



Stage 2: Place End Segments

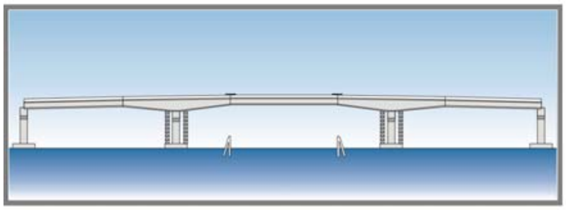


Fig. 4 Construction Staging

Stage 3: Place Drop-In Segment

POST-TENSIONING

As stated earlier in the spliced girder definition, the unit is comprised of a marriage between pre-tensioning contained in each segment and post-tensioning at splice locations. In most all cases, post-tensioning tendons will run from the beginning of the unit to the end without interruption. The result is a system where resistance of the precast section is provided by a combination of pre-tensioning and post-tensioning, and resistance at splice locations is provided only by post-tensioning.

After erection of the segments, beam end locations are left for “splicing” of post-tensioning ducts as shown in Figure 5. Once the ducts are joined, a cast-in-place diaphragm (or continuation of beam cross-section) is provided at the splice point and post-tensioning is applied to make the system continuous.

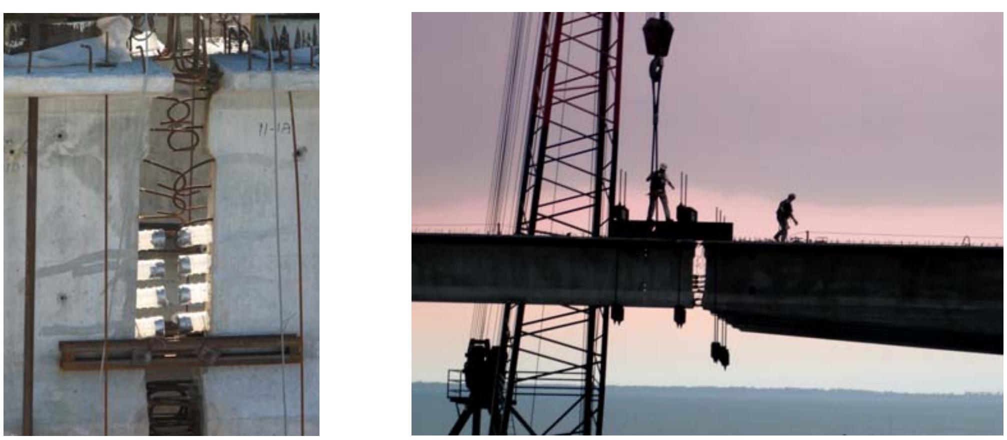


Fig. 5 Beam Splices

Traditionally post-tensioning for Intracoastal Waterway bridges has been applied in two stages. The first stage provides continuity of the girder system so a cast-in-place deck can be applied, followed by a second stage of post-tensioning after the deck has cured to place precompression into the deck. Most girders have 3 to 5 tendons, ranging in size from nine to fifteen 0.6" diameter strands in each tendon. Normally 50% to 75% of the tendons are stressed on the non-composite system, with the remaining stressed on the composite system after the deck has cured.

It should be noted that second stage post-tensioning makes the total system, beam and deck, serviceable with regard to longitudinal stresses. Although the second stage of post-tensioning has not been used on all spliced girder bridges, some benefits are listed below:

- Generally speaking it is more efficient to post-tension a composite system.
- Deflections of the system can be estimated using an elastic analysis (disregarding deck cracking).
- Extended service life of deck system.

Conversely, drawbacks to using second stage post-tensioning are as follows:

- Subcontractor performing tensioning and grouting operations must mobilize equipment and personnel a second time.
- Chance of more tedious methods required to replace bridge deck in the future.
- Perception of providing more post-tensioning than required to make bridge deck serviceable as well (i.e. precast girders continuous for live load do not place precompression into the bridge deck).

BEAM CROSS-SECTION SELECTION AND MODIFICATIONS

Due to required span ranges, the most common sections used for spliced girders have been bulb-tees. Some states have adapted cross-sections capable of extending beyond the standard AASHTO bulb-tee span ranges, mainly by modifying depth of the section and increasing top flange dimensions, as well as bottom flange dimensions for accommodation of more prestressing strands. These modified cross-sections have played a major role in the ability to stretch spliced girder spans into the 300 foot range.

In addition to bulb-tee girders, spliced post-tensioned tub sections have also been used for a span range of 200 feet (Reference No. 5). An advantage of the tub section is the ability to adapt to curvature of roadway geometry and provide a larger torsional capacity for curved bridges.

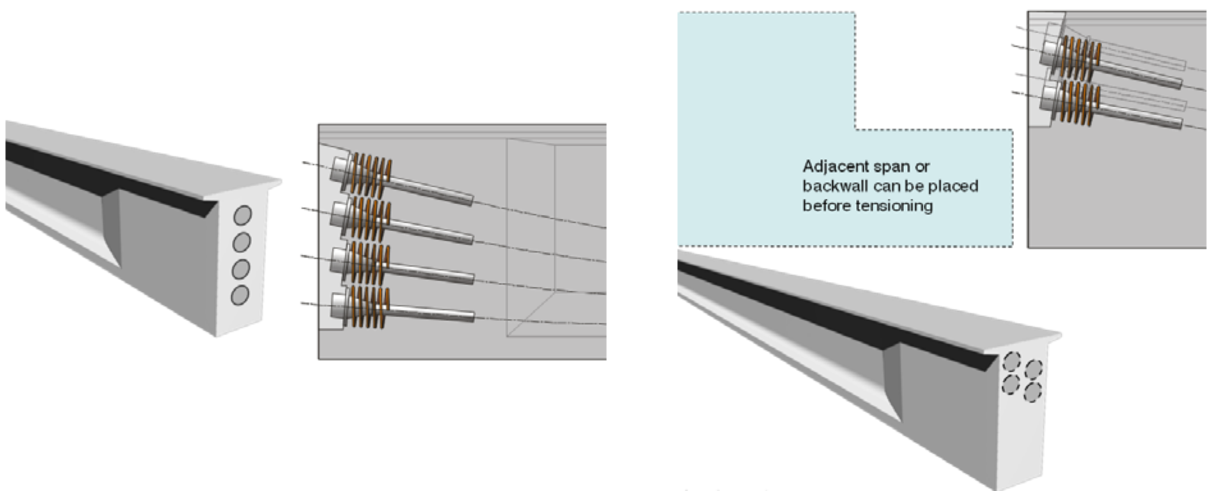
Most standard beam shapes will need minor modifications to be suitable for spliced girder construction. The following section will describe such modifications along with specific issues related to each segment type.

SEGMENT CONSIDERATIONS

END SEGMENT

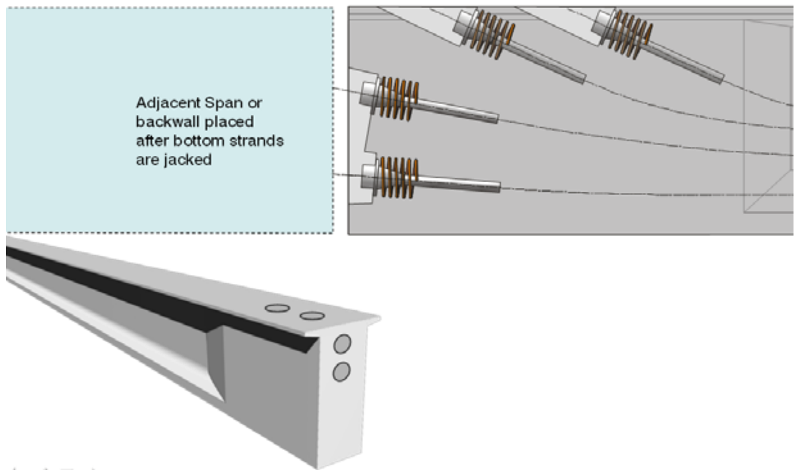
Most bulb-tee shapes have standard web widths of 6 or 7 inches. Due to the inclusion of post-tensioning ducts, webs may need to be widened to a minimum of 8 inches depending on the duct size. Adequate room should be provided for placement of the ducts, shear reinforcing, and to ensure proper placement and consolidation of concrete below the ducts. This is of increasing importance lately as some agencies are transitioning to the exclusive use of plastic ducts for grouting purposes. The new plastic ducts have a slightly larger diameter than their metal predecessor. The AASHTO LRFD Bridge Design Specifications[2] also have ratios which need to be satisfied with regard to duct diameter versus web width. Although not required by code, the authors recommend conducting a principle tension stress check in the web of the girder to assure cracking will not occur under service loading.

Due to the need to house tendon anchorages, a modified end section will be required at the end segments where tendons are normally anchored as depicted in Figure 6a. Another issue to keep in mind is the erection schedule of adjacent span girders with respect to the spliced girder unit. If all tendons are placed on the vertical face of the beam end, access for post-tensioning operations could be compromised if these adjacent span girders are present. Multiple options are available to overcome this. A few solutions are presented in Figure 6b and 6c.



(a) (b)

Fig. 6 Anchorage block and adjacent span conflict resolution



(c)

Fig. 6 (continued) Anchorage block and adjacent span conflict resolution

It should be noted that anchoring tendons at the top of the girder as shown in Figure 6c will significantly increase the anchorage block length and thus the weight of this segment. In one case, the authors used a system in which four tendons were anchored at the top of the segment. This resulted in an anchorage block length of approximately 20 feet.

To resist bursting forces induced by large concentrated anchor loads, strut and tie models can be developed to estimate the required amount of reinforcing in this area. Proper attention should be given to the order in which tendons are stressed, as different areas of the anchorage block will undergo splitting forces at each stage, as shown in Figure 7.

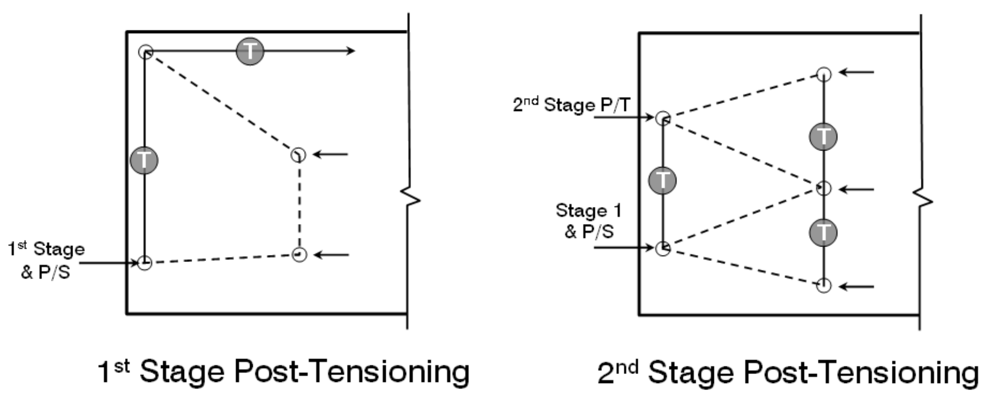


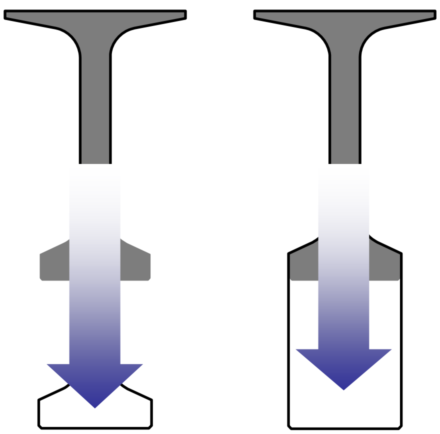
Fig. 7 Anchorage block bursting during tendon stressing

Due to smaller positive moments in the side spans, one may find only small amounts of pre-tensioned strands are required in conjunction with the provided post-tensioning. If this is the case, one should check if adequate pre-tensioning is provided to resist bending moments during handling and erection.

PIER SEGMENT

Pier segments are often the most intricate members of the unit. Depending on span length, the designer is faced with a decision to keep a constant depth segment over the piers, or use a variable depth section. When using custom bulb tee shapes with depths over 8'-0", one will normally use a constant depth segment for most applications. However when using AASHTO bulb tee sections, span lengths equal to or larger than 225 feet normally require a variable depth section.

Deepening of the segment can be done in two ways, as depicted in Figures 8a and 8b. Generally speaking maximum depth of the haunched segment alone should be around L/22 of the span length. For example, a 250 foot span would produce a depth of approximately 11'-0", not including slab thickness. If one was using a standard shape of 6'-6", the depth of additional beam would be 4'-6". Conversely, if a 300 foot span is required, a pier segment depth of 13'-6" is required, thus adding a haunch depth of 7'-0". This additional 7'-0" could pose a significant weight addition if accomplished using the method in Figure 8b. If the method used in Figure 8a was used, a significant weight savings could be seen, although at the cost of a custom side form. Weights should be given proper attention during design so as to not exceed available crane capacities at the precast plant or construction site.



(a) (b)

Fig. 8 Precast pier segment haunch

Prior to post-tensioning, the precast segments rely on pre-tensioning to resist forces during handling, transportation, erection, and staging. As can be seen in Figure 9, pier segments can go through a reversal in moments during the erection process. During handling and transportation, it is possible for the girder to be subjected to positive bending. When drop-in segments are placed during staging, the girder will have a large negative moment. Satisfaction of this reversal in moments is accomplished by fine tuning the resulting eccentricity of the pre-tensioning force. It has been standard practice to place large amounts of prestressing strands in the top flange of pier segments to resist these forces.

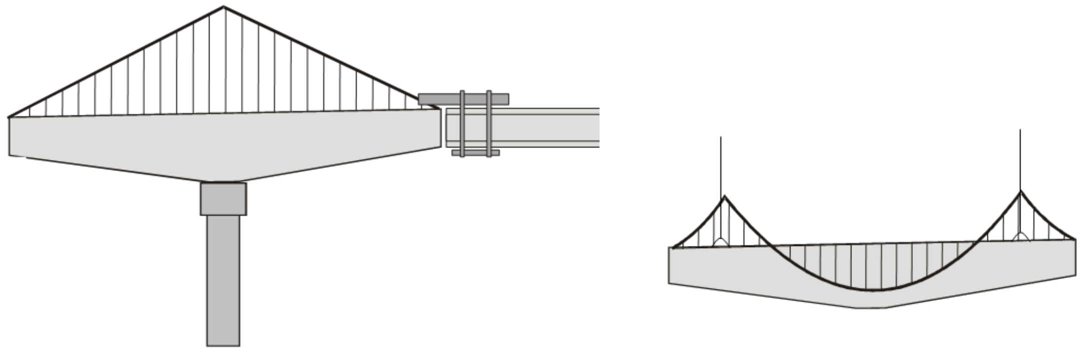


Fig. 9 Reversal of pier segment moments during construction

The need for pre-tensioned strands in the top flange of haunched segments often presents a complication for precasters. This is due to the resulting prestressing force being located at an eccentricity above which has been designed for the pre-tensioned bulkheads to resist. To overcome this, many precasters have constructed compression frames, where one pier segment is cast at a time, rather than the long-bed method. An example of this is shown in Figure 10. Another solution is to install ducts in the top flange and apply post-tensioning after the girder is cast. A drawback to this method is the need for an anchorage block to accommodate post-tensioning anchorages.

Fig. 10 Compression frame for haunched girder pre-tensioning

Cross-section dimension changes may also be required in cases where girders possess smaller top flanges. One may find the top flange incapable of accommodating the required pre-tensioned strands to support negative moments caused by the drop-in segment, thus requiring the top flange to be re-sized. In some instances the authors have added one or two inches to the thickness of the top flange to place the required amount of pre-tensioning.

Proper detailing of the pier segment ends will need to be considered to resist bursting as the pre-tensioned strands develop their forces. Splitting forces can be induced in elevation view, as shown in Figure 11, as well as in the transverse direction due to prestressing in the top flange. These phenomenons can be adequately resisted by placing proper amounts of mild reinforcing at these locations.

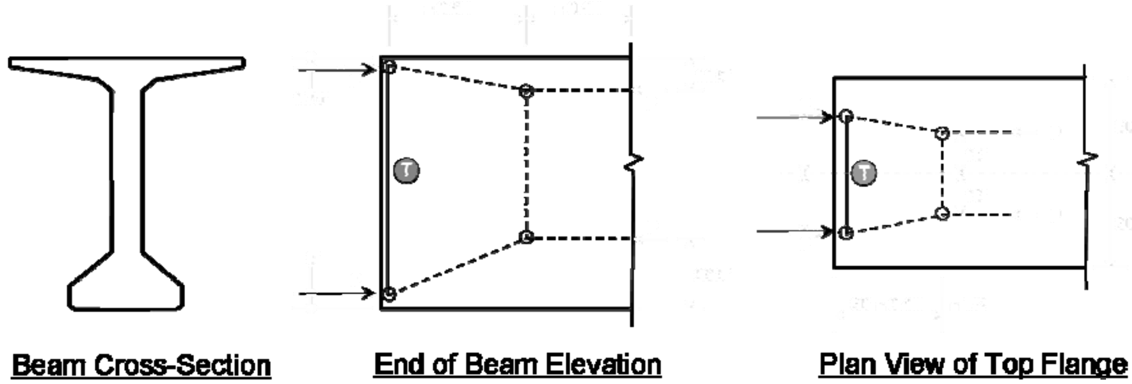


Fig. 11 Vertical and lateral bursting forces due to prestressing

DROP-IN SEGMENT

This segment may require the least amount of modification compared to others. Issues to keep in mind are to provide enough pre-tensioned strands to satisfy bending moments under transportation and erection, and satisfy anchorage zone bursting forces at the segment ends due to pre-tensioning.

FALSEWORK SYSTEM

Common falsework support systems can be seen in Figure 12. If wide pile caps are used for channel piers, falsework can often be founded on the permanent pile cap. This provides the benefit of not requiring a pile foundation for the falsework and also increases the amount of salvaged material after construction. One drawback of the pile cap founded system is the channel falsework tower receiving tension when the end segment is placed, thus requiring a tension tie for stability.

Fig. 12 Common temporary falsework scenarios

Figure 13 details an example of beam hangers, or “strongbacks”, as commonly referred to. Responsibility of designing the falsework system and strongbacks normally falls upon a construction engineer hired by the contractor, or by their in-house staff. However thought should be given to how these elements will be connected to the girders. The designer may expect small detailing changes during the shop-drawing process to accommodate these systems. For example, the strongback beam could conflict with horizontal shear stirrups at the top of beam surface, or pre-tensioned strands in the top flange may need to be shifted to accommodate a hole for threaded bars to clamp the strongback together.

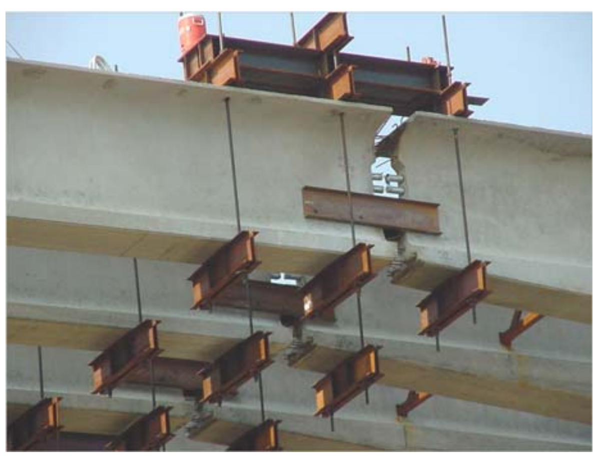


Fig. 13 Common beam hanger or “strongback” concept

DESIGN CONSIDERATIONS

Of particular importance in the design of spliced girders is the construction staging and resulting analysis. Due to various stages during construction, resulting forces and stresses will not be identical to those as if the unit were one continuous system from the beginning of construction.

Although one could perform the analysis of these structures without the use of specially adapted software, the authors recommend using a program which can account for the following:

- Time dependent analysis
- Incremental summation of construction staging
- Accommodation to changes in static system
- Ability to place and remove temporary supports
- Post-tensioning effects and secondary forces
- Preferably the ability to pour the bridge deck in stages
- Non-linear temperature gradients

Even a decade ago, there were few commercially available software programs capable of such an analysis. However today there are multiple programs on the market with such capabilities.

The AASHTO LRFD Bridge Design Specifications[2] have made significant progress in the latest few editions through inclusion of Articles addressing spliced girder construction. These specifically address which elements of the girder unit are designed using a segmental approach, and those designed using a pre-tensioned approach. Essentially spliced areas in which a joint is present are designed using segmental construction guidelines, and those areas without joints (i.e. girder segments) are designed following classical pre-tensioned guidelines.

The importance should be noted of creating an assumed construction sequence drawing for inclusion in the bridge plans. This will instruct the contractor how to build the bridge and which assumptions were made in design. If the contractor proposes deviations to this sequence, alternate methods can be prepared for approval.

CONSLUSIONS

Although spliced girder construction has seen significant progress with regard to Intracoastal Waterway crossings, many designers and owners remain unfamiliar with this method of construction. Spliced girders can serve as an attractive solution for bridges requiring span lengths greater than achieved by full-span precast methods, or where construction staging inherent with spliced girders provides an advantage over other construction types.

It is the intent of this paper to familiarize interested readers with common techniques and challenges encountered in the authors' spliced girder construction experience. While this paper does not attempt to detail every topic associated with this construction type, hopefully it can provide insight and encourage others to consider it in future designs.

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