# DESIGN OF THE LONG-SPAN SPLICED CONCRETE GIRDER CHANNEL SPAN UNIT AT FANTASY HARBOUR

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## ABSTRACT

This paper describes the design of a long-span bridge crossing the Intracoastal Waterway in Myrtle Beach, SC. The three span channel unit has a record-setting center span of 330 feet. Girders are 6.5 feet deep with 15 feet deep haunched sections over the pier and a girder spacing of 8'-5". The overall nominal length of the channel unit is 870 feet with post-tensioning tendons running the full length of the unit. Design features of the project are presented with emphasis on practical issues encountered during design and detailing of the structure.

**KEYWORDS:** Prestressed Concrete, Spliced Girders, Post-Tensioning, Design Issues, Seismic, Long Spans

## BRIDGE OVERVIEW

## ENTIRE PROJECT

The South Carolina Department of Transportation (SCDOT) retained Ralph Whitehead Associates, Inc. (RWA) to design the multi-lane extension of Harrelson Boulevard across the Intracoastal Waterway to George Bishop Parkway in Myrtle Beach, South Carolina. The project consists of one mile of roadway approach, one 1800' long main bridge, one single span bridge and rail modification of an existing bridge on Harrelson Boulevard over US 17. The main bridge on the project is also referred to as the Fantasy Harbour Bridge. The design of this bridge is the topic of this paper.

The Fantasy Harbour Bridge represents a typical high-level Intracoastal Waterway crossing. It will span the navigable channel of the Intracoastal Waterway with about a 15° skew to the centerline of channel and will provide 65' vertical clearance above the mean high water elevation, and a minimum 110' horizontal clearance for Intracoastal Waterway traffic (measured perpendicular to the waterway). The roadway alignment is tangent across the bridge, but has a significant vertical curve on the bridge to provide the required vertical clearance. The waterway has an average width of approximately 260 feet at the bridge site along centerline of the proposed bridge. The channel maximum water depth is approximately 20 feet. The proposed main bridge is located between recreational and commercial business areas. The Myrtle Beach International Airport is located approximately 3000' from the project site and the bridge site is located in the flight path for one of the runways. As a result, limitations have been placed on construction equipment and scheduling to prevent interference with air traffic.

The main bridge consists of 5 continuous units as shown in Figure 1. Unit 1 is defined as Approach Region 1, in which 72" bulb tee girders are utilized and made continuous for live load. Units 3, 4 and 5 are defined as Approach Region 2 with the same superstructure type as Approach Region 1. Unit 2 is defined as the main channel unit which spans the Intracoastal Waterway.

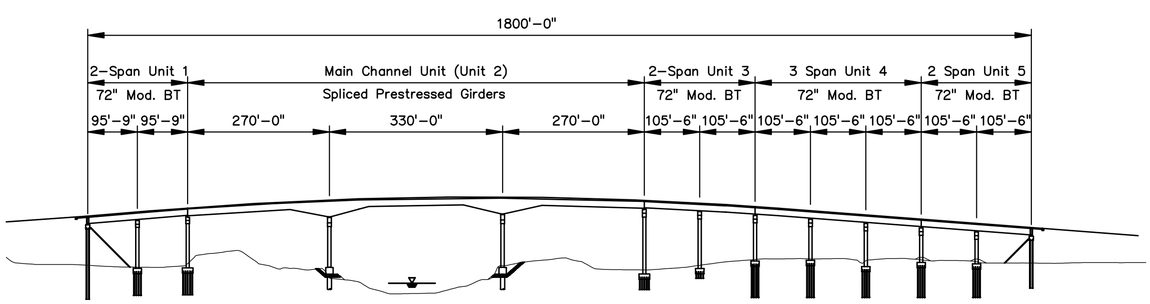


Figure 1 Elevation of Project

The South Carolina Department of Transportation has always given consideration to providing aesthetically pleasing structures that will have minimal long-term maintenance costs. The site and structure layout for the Fantasy Harbour project provided opportunities to accomplish both of these objectives. Several span arrangements were investigated using prestressed concrete and structural steel for the main channel unit. The span layout selected has a main span of three hundred thirty feet (330'), which is the minimum span required to keep the substructure units out of the Intracoastal Waterway. This layout eliminated a fender system (a costly maintenance item) and reduced the number of deck joints, which accomplished the goal of minimizing long-term maintenance costs.

The span layout for the main channel unit is 270'-330'-270'. The 330' center span not only provides the minimum 65' vertical clearance and 110' horizontal clearance without a fender system, but also places the pier footings where they can be constructed with minimum impact on the waterway.

During preliminary design, it was determined that precast post-tensioned concrete girders were a viable solution even for a span of this length. Because of the construction limitations resulting from the close proximity of the airport, SCDOT required that an alternate design for the main channel unit be prepared using a structural steel plate girder. The same span layout was used for the concrete and steel alternates. Construction cost estimates indicate that the concrete alternate is significantly more economical.

The clearances required over the roadway and waterway underneath the bridge exceed the requirements imposed by SCDOT for the spliced concrete girder alternative. This is a result of location geometry and the uncertainty of using either a 95" structural steel plate girder or 78" bulb tee girder in the main span during preliminary design.

The project is located near one of the most seismically active regions on the east coast. The Fantasy Harbour Bridge is classified as a "Critical Bridge" and the seismic performance category is "D". To properly consider the behavior of the weak soil (soil profile class "E"), a site-specific study was required by SCDOT. The site-specific acceleration coefficient at the 1 second period is 0.43g. Two levels of analysis (functional evaluation and safety evaluation) were performed for the seismic design. For the safety evaluation, an elastic multi-mode spectra analysis and a push-over analysis were performed. To ensure that the components of the structure had the desired plastic deformation capacity, ductility demands of the components were limited to 2.0.

The bridge was designed using the *AASHTO LRFD Bridge Design Specification* 2$^{nd}$ Edition[1] with interims through 2003, and the 2001 *SCDOT Seismic Design Specifications for Highway Bridges*[2].

## MAIN CHANNEL UNIT

The three span main channel unit for the concrete alternate consists of prestressed concrete girders fabricated in five sections that are spliced with post-tensioning to create the continuous unit shown in Figure 2.

The site conditions and geometry governed the span layout of the channel unit for all alternatives considered. Typically, the ratio of the lengths of exterior to interior spans in a continuous bridge of this type is lower than the 0.82 ratio used for this project. Refer to NCHRP Report 517[3] for examples and a discussion of span arrangements that have been used for spliced girder bridges. When completed, the main span will be one of the longest concrete spliced girder spans in North America.

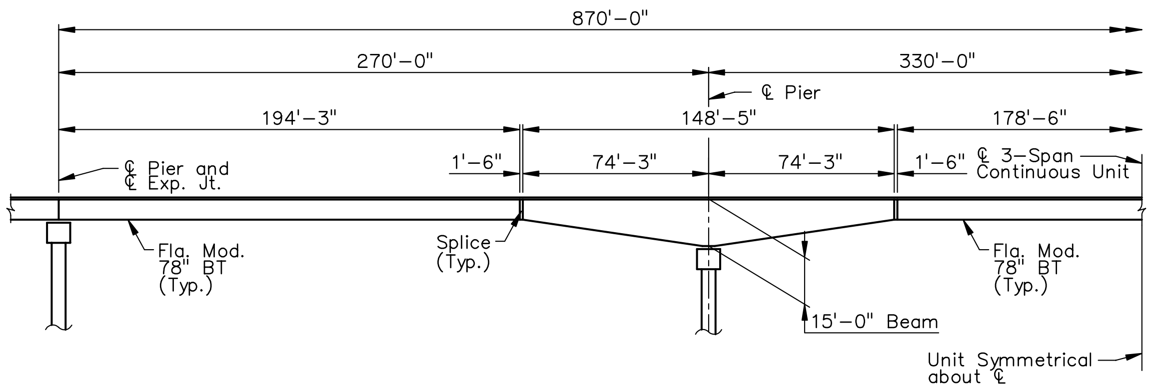


Figure 2 Half Elevation of Bridge Girder Layout

The girders are made continuous by casting a splice between the ends of the girder segments and post-tensioning for the full length of the unit. Post-tensioning is performed in two stages: Stage 1 Tendons are stressed prior to casting of the deck slab and Stage 2 Tendons are stressed after casting the deck slab. Therefore, the girders are continuous for the deck slab dead load and for all loads applied to the composite section.

The end and drop-in segments are modified 78” Florida Bulb Tee Girders. The typical 78" Florida Bulb Tee Girder was modified by spreading the side forms apart by 2 inches resulting in a web width of 9 inches as necessary for post-tensioning duct cover requirements discussed later in this paper. The variable depth pier segments are based on the same cross-section with the depth varying linearly from 78" at the ends to a maximum depth of 15' for three feet on either side of the interior pier. The depth of the bottom flange of the pier segment varies linearly from 8" to 15" to address strength issues. See strength design section for more details. The weights and lengths of the segments are summarized in Table 1.

As shown in Table 1, the weights of the segments are substantial. This directly affects the construction equipment and the sequence used for girder segment erection. Due to these weights, pot bearings are used and the segments are expected to be transported using barges.

Table 1 Girder Segment Lengths and Weights

| Segment | Length (ft) | Weight (tons) |
|---|---|---|
| End | 193'-9" | 138 |
| Pier | 148'-6" | 143 |
| Drop-in | 178'-6" | 117 |

The bridge cross-section uses eleven girder lines. The girders are spaced at 8'-5" on center with 3'-2½" overhangs each side as shown in Figure 3.

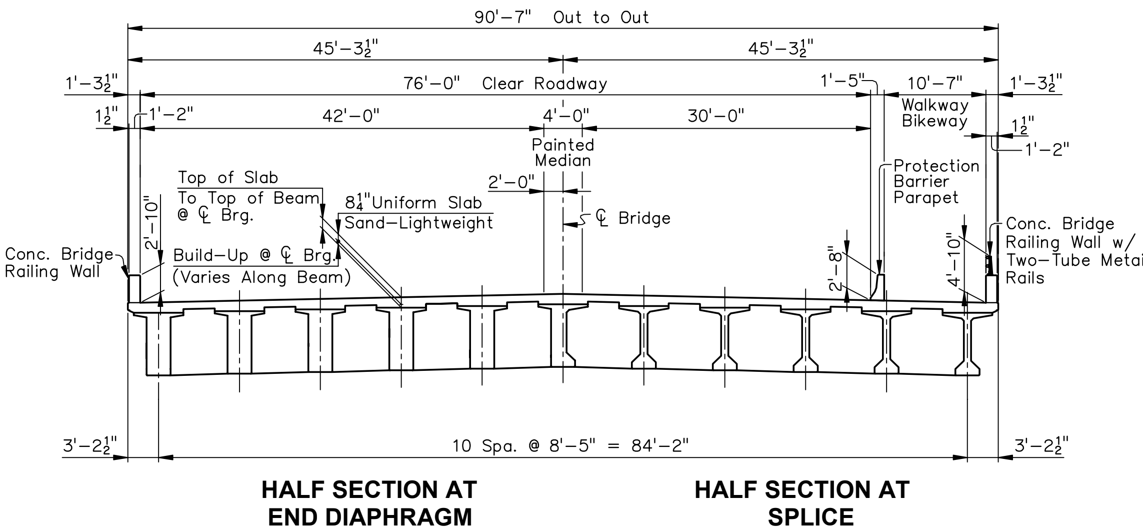


Figure 3 Typical Section

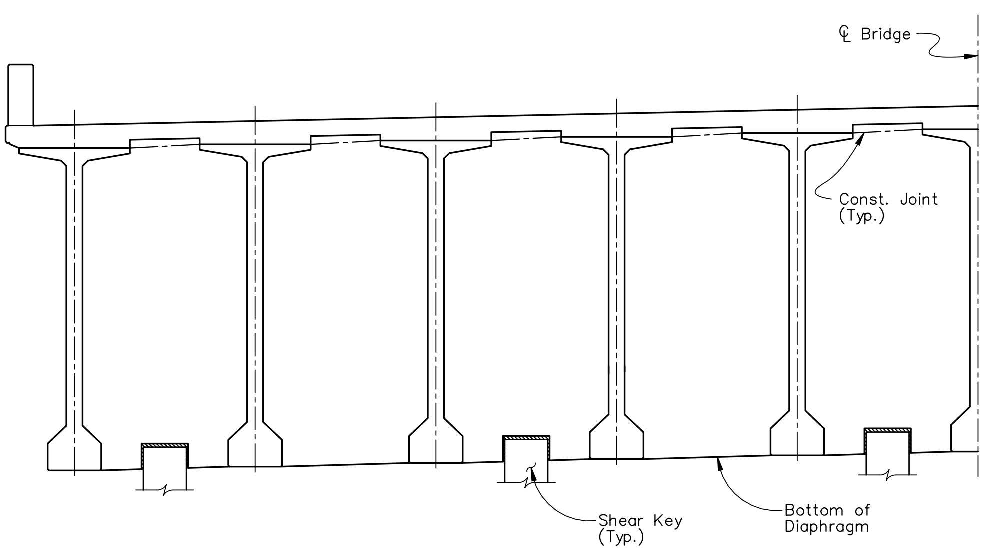


Figure 4 Typical Section at Interior Pier

As shown in Figures 3 and 4, the roadway section is asymmetrical with three lanes on one side and two lanes of traffic and a walkway/bikeway on the other side. This arrangement accounts for the possibility of a future sidewalk constructed over the current walkway/bikeway, and a turning lane located on the other side of the bridge. This required that several different live and dead load conditions be considered during design.

The *LRFD Specifications* do not provide allowable stress limits for intermediate stages during the construction of a spliced girder bridge. The allowable tensile stress limits specified in LRFD Table 5.9.4.2.2-1 were used for intermediate stages. The allowable compression stress limit was taken as 0.60 $f'_c$, which is normally used for the stresses resulting from the total effects of prestress, dead loads, and live loads. These limits were used since the loads applied during the construction stage can be considered temporary. For all stages after the bridge was completed and opened to vehicular traffic, the typical allowable stress limits and load combinations were used.

## ESTABLISHING A COMPUTER MODEL

Developing a computer model for a typical prestressed concrete girder bridge is normally straight forward with the bridge girders being analyzed at only two key times: release and after all losses have occurred. This has been done for years with no indications of problems in completed bridge structures. However, a bridge constructed using staged prestressing and splicing requires a more detailed evaluation with stresses, shears, ultimate capacity, and deflections examined at each stage of construction. Furthermore, time-dependent effects (creep and shrinkage) may be significant for this type of structure. As a result of these issues, typical design software was inadequate to assist with the design so the use of an advanced software package was necessary. Several software packages are currently available for this type of design including, but not limited to, Consplice, BD2, and LARSA 4D. BD2 was selected for this project. The software computed the effect of time-dependent effects, temperature gradient effects and also prestress losses.

The number of analysis stages was determined by considering the stages of construction and software requirements necessary to obtain the information required to design the bridge. For the main channel unit, 13 stages were used as shown in Table 2. The first stage was at release of pretensioning in the girder segments. The next three stages model the girder erection sequence discussed later in this article. The casting of the splices between girder segments, post-tensioning Stage 1, deck/slab casting, and post-tensioning Stage 2 are in the following stages. The remaining stages occur during the actual service life of the structure.

As shown in Table 2, two construction schedules were modeled and analyzed for the main channel unit to address the effects of creep, differential shrinkage, and prestress losses. One schedule was developed reflecting an aggressive "Fast Paced" construction sequence for which the bridge would be completed within 180 days and the other schedule was developed to reflect a "Slow Paced" construction sequence for which the bridge would be completed in 550 days. The time frames were determined based on experience with other projects, estimates of construction time required for each stage, anticipated availability of materials, and an effort to maximize or minimize the effects of creep, differential shrinkage and prestress losses. The two schedules were intended to bracket the actual construction schedule that may be used.

Table 2 Stages in Analysis

| Stage | | Time of Occurrence | |
|---|---|---|---|
| | | "Fast Paced" | "Slow Paced" |
| 1. | Release of Pretensioned Strands** | 2 | 2 |
| 2. | Erect Pier Segments | 60 | 365 |
| 3. | Erect End Segments | 60 | 365 |
| 4. | Erect Drop-in Segments** | 60 | 365 |
| 5. | Cast Splices | 70 | 375 |
| 6. | PT Stage 1 | 85 | 400 |
| 7. | Cast Diaphragms | 100 | 420 |
| 8. | Cast Deck | 115 | 450 |
| 9. | PT Stage 2 | 135 | 480 |
| 10. | Cast Barrier/Parapet | 150 | 490 |
| 11. | Bridge Open to Traffic** | 180 | 550 |
| 12. | Apply Future Wearing Surface | 4,000 | 4,000 |
| 13. | End of Service Life of Bridge** | 27,560 | 27,560 |

Note: Time is measured in **Days** from casting the girders.
** - Stages which controlled the design.

Since the effects of the dead loads and the initial effects of the prestress, without time-dependent losses, do not vary between the two schedules, the difference in stress in the girder and deck slab between the two schedules for the end of service life stage is the result of creep, differential shrinkage and prestress losses.

The "Fast Paced" schedule indicated higher stresses in some areas of the girders at the time that the bridge is first opened to traffic and at the end of the service life than the "Slow Paced" model. Similarly, the "Slow Paced" model exhibited higher stresses at other areas of the girder when compared to the "Fast Paced" model.

An item to note is that these time frames will not work for all situations. The use of different span lengths, girder cross-sections and spacing, materials, and prestress layouts and amount, will affect the creep, differential shrinkage and prestress losses differently. In some cases, the creep and differential shrinkage may be negligible depending on the shrinkage coefficients for a particular deck and girder or they may be major and can induce significant stresses into the girder and/or deck slab.

## PRETENSIONED STRAND LAYOUT

To keep concrete stresses in the spliced girders within the stress limits for both tension and compression, a large quantity of pretensioned strands were required in the design.

The large number of pretensioned strands resulted in high concrete stresses in both the end segments and the drop-in segment. Since post-tensioning ducts were located within the web, draping of pretensioned strands was not possible, so debonding was used to control stresses in these segments.

With the large number of pretensioned strands that was required in the end segment, controlling the stresses at the ends and middle of the segment at both release and service proved to be difficult. The design required 8 debonded strands in the top flange at the center of the segment and 12 debonded strands in the bottom flange at the ends of the segment. SCDOT requirements limit the number of debonded strands in a row to 40% of the number of strands in that row and does not allow any debonding in the bottom row. Available locations for top strands in the end segment was limited by the 24" wide recess for access to the post-tensioning anchorages and by allowing a location for tie rods to secure strong-backs to the girder for erection. Therefore, the maximum number of top strands that could be placed in the top flange was 10, which was used in the design (see **Figure 5**).

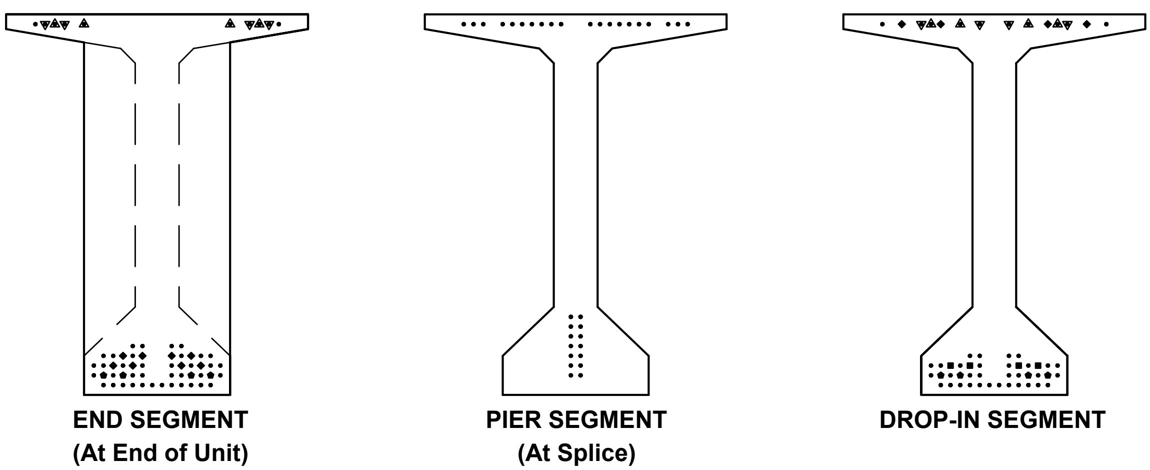


Figure 5 Pretensioned Strand Layout and Debonding

The drop-in segment also required debonded strands in the top and bottom flange as shown in Figure 5. The pier segment used only straight strands in the top flange and a group of straight strands in the web.

The debonded top strands in the end and drop-in segments will not be detensioned until the girders have been erected. This will improve the stability of the long beams during shipment and will reduce initial camber and camber growth. Slots are detailed in the top flange at midspan to provide access to the debonded strands for detensioning. The plans indicate when and how the debonded strands in the top flange will be detensioned and warn the contractor that the strands should not be simply cut, but rather must be detensioned according to the procedure given. After detensioning, the access holes will be filled with epoxy non-shrink grout meeting girder strength (moisture non-sensitive).

Pretensioned strands were also added to provide additional camber so that the anticipated beam profile would better match the grade line of the bridge, thus reducing the anticipated build-up over the beams.

## POST-TENSIONING DUCT

Selection of the size and type of post-tensioning duct presented a major design challenge for the bridge. The duct size is directly related between the amount of prestress required, the size of the girder cross-section, the post-tensioning anchorages from different companies, and the material used for the duct itself. This leads to an iterative process that can be very time consuming.

Early in the design process, it was decided that the standard girder cross-section should be modified so that the web would be 9" wide instead of 7". This was a result of previous experience and consideration of LRFD Art. 5.4.6.2, which requires that the duct size not exceed 40% of the web width. Using this requirement, the maximum duct size for 7" and 9" webs is 2.8" and 3.6", respectively. This limit is frequently ignored and larger ducts have been used successfully on other bridge projects. The Florida Department of Transportation allows 50% of the web width for instance. However, for this project, the LRFD requirement was used.

A second requirement in LRFD Art. 5.4.6.2 requires that the area of the duct be at least 2 to 2.5 times the area of the prestressing strands contained within the duct. Using this requirement, the number of prestressing strands in each tendon was limited to 11 and 18-0.6" diameter strands for a 7" and 9" web, respectively, assuming a 2.5 multiplier. After consulting post-tensioning suppliers, it was determined that their preference was to use a multiplier of no less than 2.5. Therefore, a tendon using 18-0.6" diameter strands could have been used given the duct size and LRFD requirements.

After further study of combinations of anchorages and spacing recommendations provided by post-tensioning suppliers, it was determined that an anchorage with more than 15-0.6" diameter strands would not fit within the cross-section. However, a 15 strand anchorage was only available from one supplier, and if a 19 strand anchorage were provided by the other supplier, it would not fit in the cross section. To resolve this issue, additional discussions were made with post-tensioning anchorage suppliers. See additional discussion later in this section.

The standard duct size used with the post-tensioning anchorages exceeded the maximum diameter size as imposed by the *LRFD Specifications*. After additional conversation with the post-tensioning suppliers, a slightly undersized duct was selected for the girders. This duct, due to the reduced size, had to be metal instead of polyethylene which is required by several DOTs. After discussions with representatives from several DOTs, it was decided that a metal duct could be used, but special consideration for corrosion prevention had to be made. The metal ducts also allowed the duct layout used to have a curvature less than the 30', a minimum allowed for polyethylene ducts (LRFD Art. 5.4.6.1). On a side note, some suppliers only carry a few sizes of polyethylene ducts which they use for the anchorages they provide. This may result in the use of ducts that are larger than necessary for some anchorages.

Another issue considered when selecting the duct size was the layout of shear stirrups. The combination of duct and stirrups must provide the required concrete cover over the stirrups. The design uses a galvanized corrugated metal duct with an outside diameter of 3.90" (99mm) and an inside diameter of 3.54" (90mm).

## POST-TENSIONING TENDON LAYOUT

The post-tensioning tendon layout required consideration of several code and supplier requirements or recommendations including duct spacing, minimum duct curvature, and anchorage zone features. LRFD Article 5.10.3.3.2 requires the clear spacing between ducts to be the greater of 1.5" or 1.33 times the size of the maximum coarse aggregate. When the outer diameter of the ducts used is incorporated with the clear spacing, a minimum duct-to-duct spacing of 5.5" was determined. This spacing was used in the anchorage zone to handle the varying duct curvatures. For the remainder of the duct layout, a spacing between duct centers of 5.5" to 6" was used.

The location of the inflection points of the post-tensioning duct was governed by the amount of stress in the spliced girder at the locations of the ends of the girder segments. Depending on the location of the inflection points, the stresses in the girder exceeded the maximum allowable compressive stress at the time the bridge was first open to vehicular traffic and remained so until the expected end of service life at 75 years. Shifting the inflection point slightly changed the post-tensioning eccentricity and affected the post-tensioning friction losses as a result of curvature change. This resulted in an unsymmetrical duct layout in the pier segments. It was original planned for the duct layout to be symmetrical in the pier segment so that placing the girder segment backwards would require no corrective measures.

As shown in Figure 6, ducts are placed as low as possible considering pretensioned strand layout and the required concrete cover at the low points for both the end and drop-in segments. This duct location was dictated by deflection consideration rather than service or strength limit states. The high points of the tendon layout occur at the center of the pier segments as shown in Figure 6. The concrete cover over the top duct at these locations is less than would normally be required if the surface would be exposed to weather, because a concrete deck will be cast on the girder. To simplify constructability of the spliced girders, the duct layout at splice locations is nearly linear, which simplifies splicing of the ducts and reduces the possibility of kinks in the duct.

As specified in LRFD Art. 5.9.1.6, the offset between the center of the duct and the center of the tendon was considered. A maximum offset of ¾ in. was used, as provided in LRFD Figure C5.9.1.6-1, to account for the difference between the center of the duct and tendon at the extreme points along the duct layout. A linear variation of the maximum offset was used at locations between the extremes.

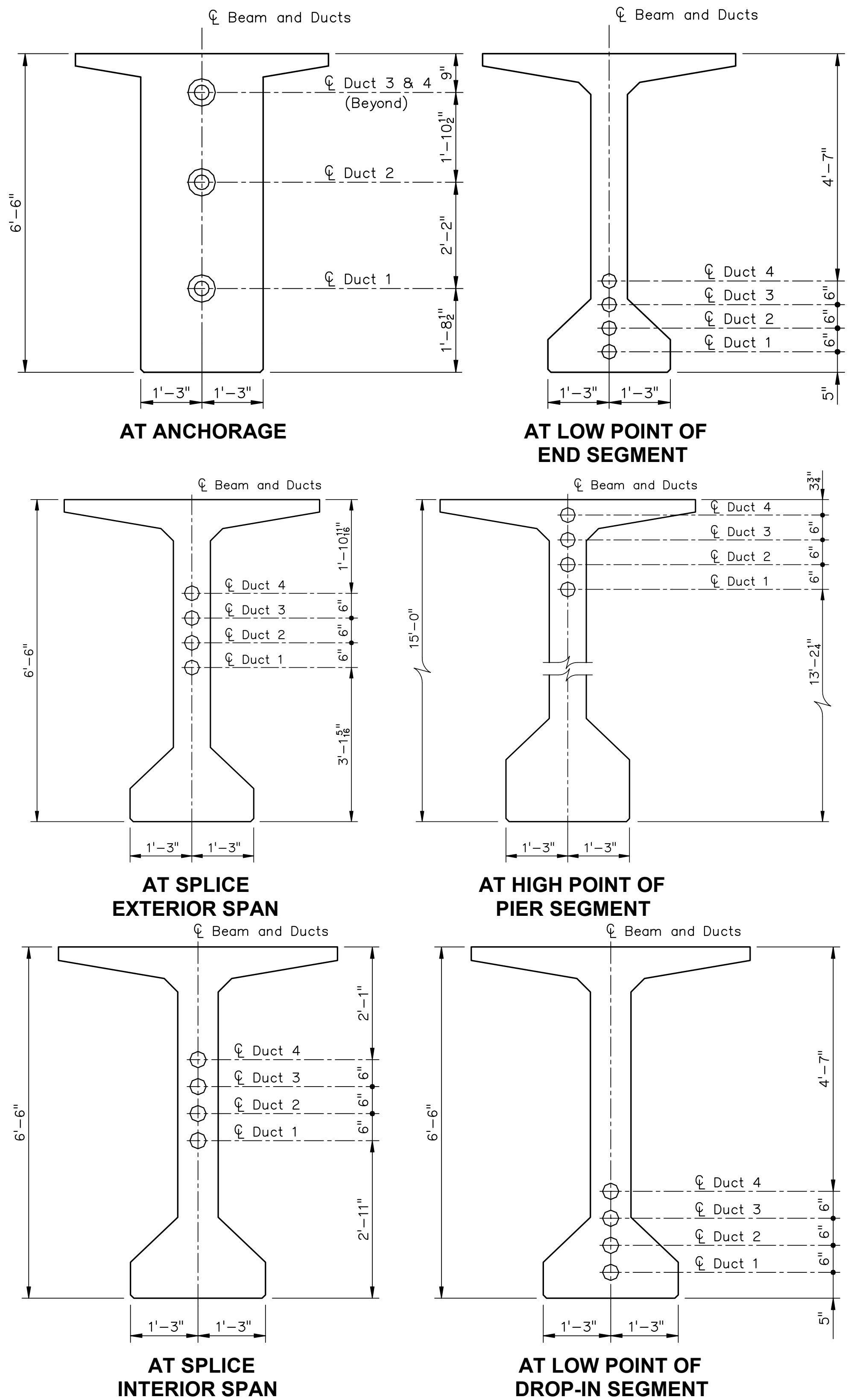


Figure 6 Duct Locations at Selected Sections Along the Bridge

The duct layout required compound curves for the top three ducts at anchorage locations. A minimum radius of 20' was used for the top two ducts before the four ducts became parallel throughout the remainder of the length of the spliced girder.

## POST-TENSIONING ANCHORAGE ZONE

Designing the anchorage zone proved to be a tedious task due to the cross-section restrictions, available anchorages, and considerations for adjacent spans. Since this project is a traditional design-bid-build project and the contractor will be selected through a competitive bid process after completion of the design plans, a generic anchorage was used in the design. This anchorage was taken as the larger of two possible anchorages provided by two different post-tensioning suppliers. As mentioned previously, this anchorage, 19-strand tendon anchorage, would have been too large for the girder cross-section to accommodate, but after discussions with the supplier, the anchorage spacing was reduced to an acceptable size. In addition to this consideration, the anchorages and reinforcement had to fit within the large number of pretensioned strands required in the end segments of the spliced girder.

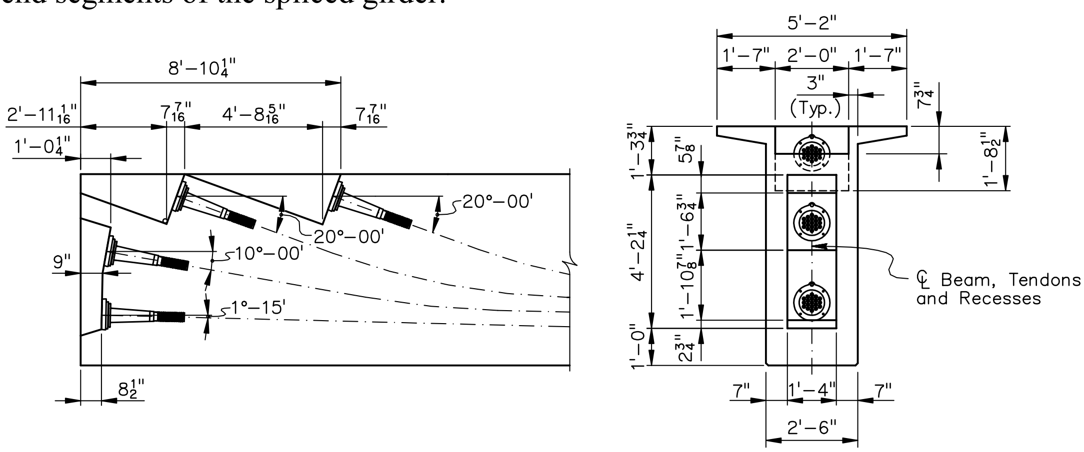


Figure 7 End Block Anchorage and Recess Details

Since the Fantasy Harbour Bridge had approach spans on either side of the spliced girder unit, the possibility of adjacent span girder erection before the second stage post-tensioning was considered in the design. This resulted in the two Stage 1 post-tensioning anchorages being located in the end face of the girder and the two Stage 2 anchorages being recessed in the top of the girder, as shown in Figure 7. The recesses were detailed to accommodate the stressing equipment. The recesses for the two top anchorages were 24" wide with sufficient bottom and top clearances.

The final layout provided the required concrete cover, duct spacing, anchorage spacing, clearance to pretensioned strands, and consideration for the adjacent spans.

Special requirements and materials were employed to address corrosion protection for the post-tensioning anchorages. See End Block Design/Detailing for further discussion.

## GIRDER ERECTION SEQUENCE

As mentioned during the bridge overview, there are several factors which directly affected the design of the bridge as well as the future construction. The presence of the Myrtle Beach Airport restricted the height of cranes that the contractor may use. See the discussion on the special provisions for more details. The presence of the airport, Intracoastal Waterway, and the existing roadway affected the construction methods and sequences used, as well as the weights and lengths of the individual girder segments.

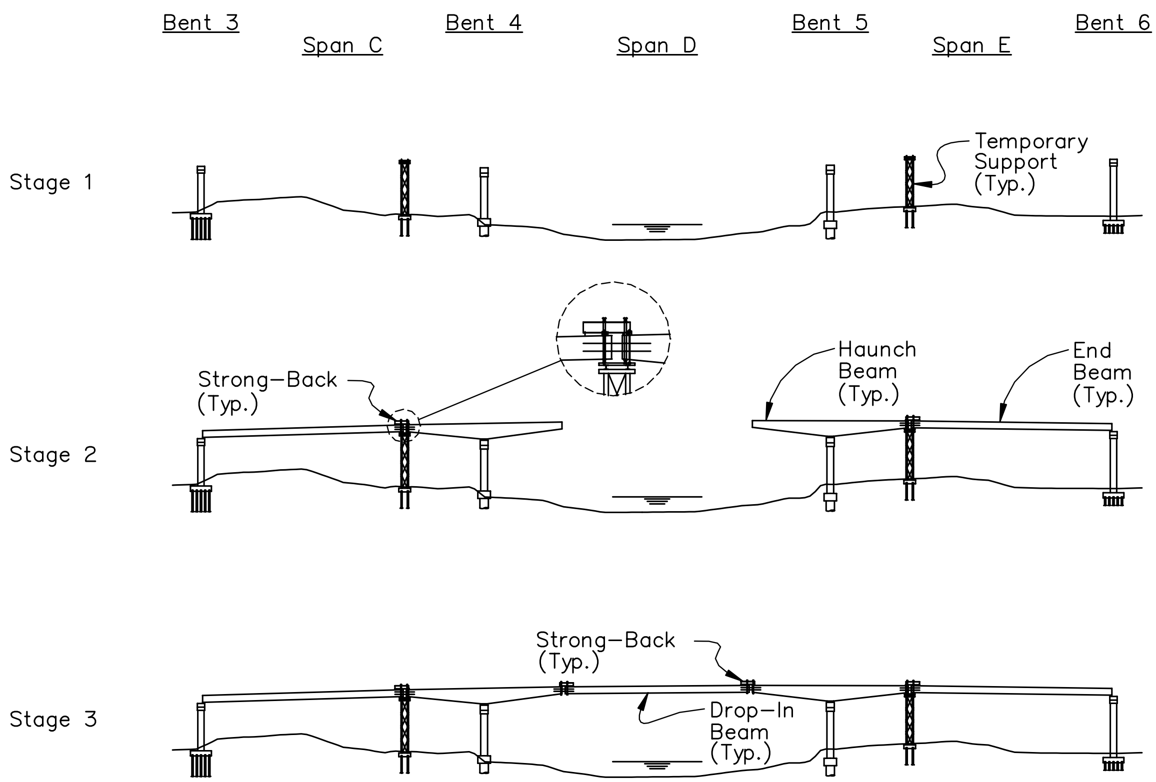


Figure 8 Girder Segment Erection Sequence

The girder erection sequence, as provided in the plans, is shown in Figure 8. The lack of maneuvering room for the cranes resulted in erection of the end segments first upon permanent bents 3 and 6 and a temporary support erected at the location of the splice between the end segments and the pier segments. The pier segments will then be erected on the temporary support and permanent bents 4 and 5. The end segment and pier segments will be connected using a strong-back system. The strong-backs are used to offset the loads on the temporary support resulting from the erection of the drop-in segments on the cantilever ends of the pier segments. Without the connection, portions of the temporary support would experience an uplift force while the other portion would have a downward force from the reaction of the end segments. The weight of the end segments is sufficient to overcome the uplift force resulting from the placement of the drop-in segment. Thus, this simplifies the design of the temporary support.

## DEFLECTIONS, CAMBERS, AND ROADWAY PROFILE

This bridge is on a vertical curve with the high point located near the connection between the pier segment and drop-in segment on one side of the bridge. This vertical curve along with the long spans can result in large amounts of build-up. To further reduce the anticipated build-up, the elevations of the pier segment at the temporary supports were set so that the final position of the spliced girders would better conform to the vertical curve of the roadway. As mentioned previously, pretensioned strands were also added in the drop-in segment to help reduce the build-up. Even with these special efforts to control the amount of build-up, a maximum build-up of approximately 6.5" is estimated to occur in the exterior spans.

The estimated build-up varies from 1.5" to 6.5" in the exterior spans and 1.5" to 4.5" in the main span. This large variation in build-up required that the stirrups be detailed for two different heights. Without this height adjustment, the stirrups would have interfered with deck reinforcement or have been too short to project into the deck.

## DESIGN FOR STRENGTH LIMIT STATE

As a result of the large quantity of prestressed reinforcement provided in the spliced girder, the equations given in the *LRFD Specifications* do not adequately assess the flexural capacity of the girder at the Strength Limit State. The equation provided in the *LRFD Specifications* indicates that the section is over-reinforced, so the over-reinforced capacity equation (Art. C5.7.3.3.1) should be used. However, this equation does not properly account for the compression in the top flange of the girder when used for a girder with a composite deck for which the compression zone extends into the girder.

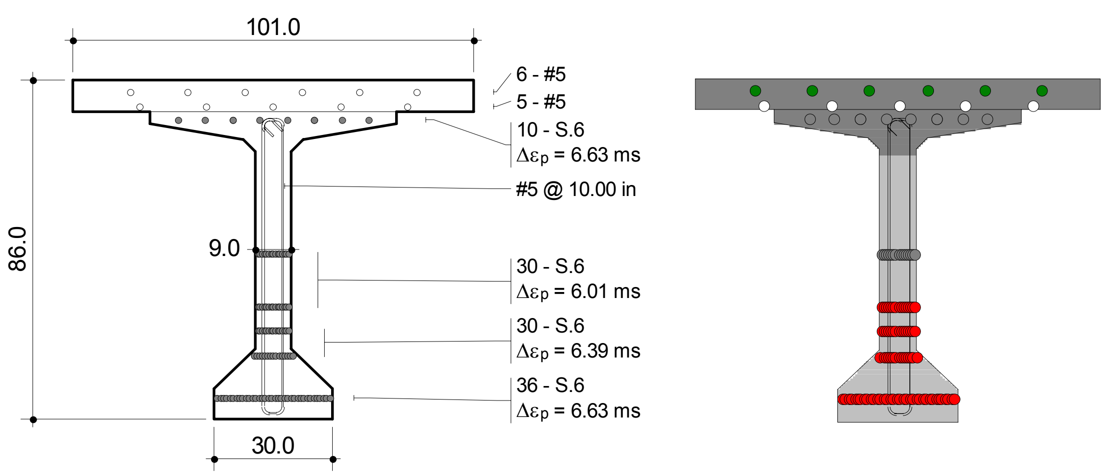


Figure 9 Example of Moment Curvature Analysis

Therefore, a more refined method of analysis was used to determine the flexural capacity of the girder. The moment curvature analysis software package, Response 2000, was used to determine the capacity of the section at the Strength Limit State. This analysis considered all aspects of the girder and deck dimensions as well as the different concrete strength in the deck and girder.

To increase the negative moment capacity of the pier segments, the depth of the bottom flange was increased from 8" to 15" at the interior bents. The bottom flange is reinforced with 8 # 29 bars, which further increases the strength of the pier segment.

Using the refined method, the flexural capacity exceeded the factored moment at all locations on the bridge except for the splice locations. The flexural capacity at the splices was increased by the addition of mild reinforcement projecting from the ends of the segments.

## END BLOCK DESIGN AND DETAILING

Several strut and tie models were developed to assist in the design of the general zone in the end blocks. Due to the staged prestress, it was determined that developing separate models for the pretensioned strands, Stage 1 post-tensioning and Stage 2 post-tensioning were necessary. The results were superimposed to determine the amount and location of the necessary reinforcement. Secondary models were used to determine the reinforcement needed in the transverse direction at each stage of prestress. The reinforcement required from the strut and tie models was substantial. The end block reinforcement was carefully detailed to satisfy the requirements of the strut and tie models, to maintain required cover, and to provide a reinforcement layout that was as simple and constructible as possible. Providing reinforcement around the anchorage recesses areas proved to be a challenge.

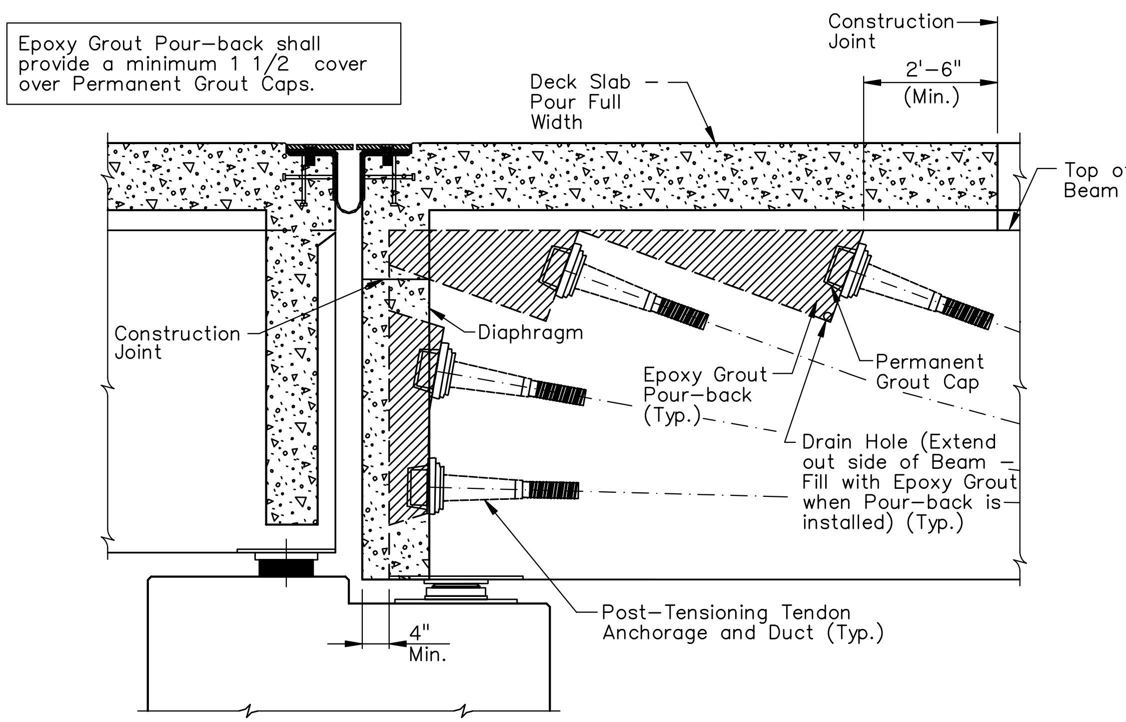


Figure 10 Details at Post-Tensioning Anchorages

The anchorage recess areas shown in Figure 10 are filled with epoxy grout shortly after grouting of the tendons is complete. The lower tendons, with recesses in the ends of the girder segments, are totally encased by the lower portion of the end diaphragm to permit erection of the girders in the approach spans. After the Stage 2 post-tensioning has been completed and the ducts are grouted, the two top recesses are filled. The final portion of the deck slab is then cast, encasing these anchorages. This system provides multiple levels of corrosion protection for the tendon anchorages.

## SPECIAL PROVISIONS

Since the SCDOT Standard Specifications do not include requirements for some of the issues related to the construction of this post-tensioned spliced girder bridge, project special provisions were developed. The special provisions developed were based on information collected from several DOTs, including California, Florida, Oregon, Texas, Virginia and Washington, as well as the Post-Tensioning Institute (PTI) and the American Segmental Bridge Institute (ASBI). The post-tensioning special provisions included certification requirements, material specification, procedures and other items.

Certification of post-tensioning and grouting personnel through programs offered by the PTI and ASBI is required. The post-tensioning crew are required to have at least one member that holds a Level I – Grouted Tendon Technician certification (PTI) and the foreman and either the Project Engineer or Project Superintendent/Manger shall hold a Level II – Grouted Tendon Installer Certification (PTI) as well as a current Grouting Technician certification (ASBI). These certifications are required to insure proper installation and protection of the post-tensioning tendons, which should reduce the possibility of corrosion of the tendons and insure a long service life for the bridge.

Other special provisions provided requirements specifying restrictions on the construction of the bridge related to the Intracoastal Waterway and airport. These restrictions, which are based on SCDOT regulations, FAA guidelines, and U.S. Coast Guard restrictions, range from equipment size restriction to time limits for certain construction activities. An example is that the total interruption of river traffic cannot exceed 72 hours for the entire construction period of the new bridge, which includes interruptions for erection of girders and the construction of cofferdams, footings, substructure, and superstructure. The special provisions also indicate that construction activities below a certain elevation can proceed without any restrictions, but also sets a maximum elevation, which will be subject to weather and time restrictions.

## CONCLUDING REMARKS

Designing the main channel unit of the Fantasy Harbour Bridge was challenging due to the record breaking main span along with the restrictions imposed on the project by the Intracoastal Waterway and nearby airport. The design is innovative and a step forward in the use of precast concrete to span longer spans, which were normally the realm of steel construction.

The letting for this project is expected to occur in August of 2004. Construction should begin the following year.

**ACKNOWLEDGMENTS**

RWA appreciates the guidance and direction received from the FHWA and the SCDOT to proceed with the design of the record setting precast concrete spliced girder main channel unit. RWA also appreciates that the Department has always provided a positive environment for their consultants to work with them and turn concepts into reality.

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