Design and Construction Considerations for Continuous Post-Tensioned Bulb-Tee Girder Bridges

Hugh D. Ronald, P.E.
Senior Structural Engineer
JE Sverdrup
Jacksonville, Florida

This article reflects the author's experience with post-tensioned spliced girder bridges over the past ten years, during which he has practiced both as a bridge designer and construction engineer.

Post-tensioned bulb-tee girders provide an extremely cost-effective structural system for bridge construction. Use of high strength concrete and further refinement of the structural system promises to yield longer spans, while providing even greater economy and durability. However, the analysis, design and construction of a prestressed, post-tensioned bulb-tee girder bridge requires careful consideration of a multitude of factors. This article attempts to outline the essential design and construction considerations for a composite prestressed and post-tensioned system in accordance with the AASHTO LRFD Bridge Design Specifications. The structural system reviewed consists of precast, pretensioned bulb-tee girders, field spliced and post-tensioned in two stages for a continuous composite structural system spanning three to five spans. This type of structural system has been used extensively and economically for bridges requiring moderately long spans ranging up to 320 ft (97.5 m).

Precast, prestressed bulb-tee girders post-tensioned for continuity have proven to be one of the most cost-effective structural systems for bridge spans of 200 to 320 ft (61 to 97.5 m). In the past ten years, this structural system has been used extensively, particularly for intracoastal crossings in the state of Florida (see Fig. 1).

This article addresses key aspects of the design and construction of these structures. Rather than expound on the specifics of a particular project, details of the structural sys-

Fig. 1. Moore Haven Bridge over Okeechobee Waterway in Central Florida. At 320 ft (97.5 m), this bridge has currently the longest span using staged post-tensioning. Sverdrup Civil provided construction inspection of this bridge, and the channel unit was designed by Janssen and Spaans.

tem are discussed. The intent is to relate lessons learned, while providing guidelines for design and detailing, and suggestions for further refinement of the structural system. Background information on the material presented herein is provided in References 1 through 5.

The analysis and design of a post-tensioned, bulb-tee girder bridge are very similar to the analysis and design of a segmental bridge. A load-history analysis of the structural system is required to properly accumulate actions and reactions at each stage of erection. In addition, the accuracy of the result requires that the program used for analysis address the time-dependent effects of creep, shrinkage and relaxation, as well as slip, wobble and curvature losses in the post-tensioning tendons.

To bridge spans of 200 to 320 ft (61 to 97.5 m), precast, prestressed bulb-tee girders are field-spliced and post-tensioned, usually in two stages. First-stage post-tensioning allows girders to be made continuous prior to the addition of a composite deck, while second-stage post-tensioning provides residual compression in the deck for serviceability and deflection control.

Depending on the section properties of the precast, pretensioned girder, girder spacing and span length, three or four tendons may be used. In general, two tendons are tensioned when the section is non-composite, with the balance of the tendons tensioned after placement of the deck.

Because this construction technique falls somewhere between conventional precast composite construction and segmental construction, code provisions of the American Association of State Highway and Transportation Officials (AASHTO), American Concrete Institute (ACI) and American Segmental Bridge Institute (ASBI) do not adequately address many aspects of the analysis or design. Nor do most of the programs available for segmental construction provide comprehensive algorithms for live load envelopes, ultimate strength checks, and other limit state checks of composite post-tensioned systems.

The Florida Department of Transportation (FDOT) has used precast prestressed, post-tensioned girders since the mid-seventies. The first composite post-tensioned bridge in Florida was the Chipola Nursery Road Bridge over I-10. Simple span prestressed AASHTO Type IV girders were post-tensioned for continuity, for a composite system with spans of 127 ft (39 m). Post-tensioning simple span AASHTO girders provided the opportunity and experience to bridge longer spans and use fewer girder lines.

The first post-tensioned spliced girder system of the type described in this article was constructed in 1988. Using two-stage post-tensioning, the FDOT designed a three-span channel unit to bridge the navigation channel in Choctawhatchee Bay. The channel unit had spans of 160, 200 and 160 ft (49, 60 and 49 m), respectively. Since that time, the system has been used repeatedly and refined extensively.

Post-tensioning precast girders allows the use of long simple spans or greater beam spacing. The cost of post-tensioning steel is offset by the reduced number of girders or fewer

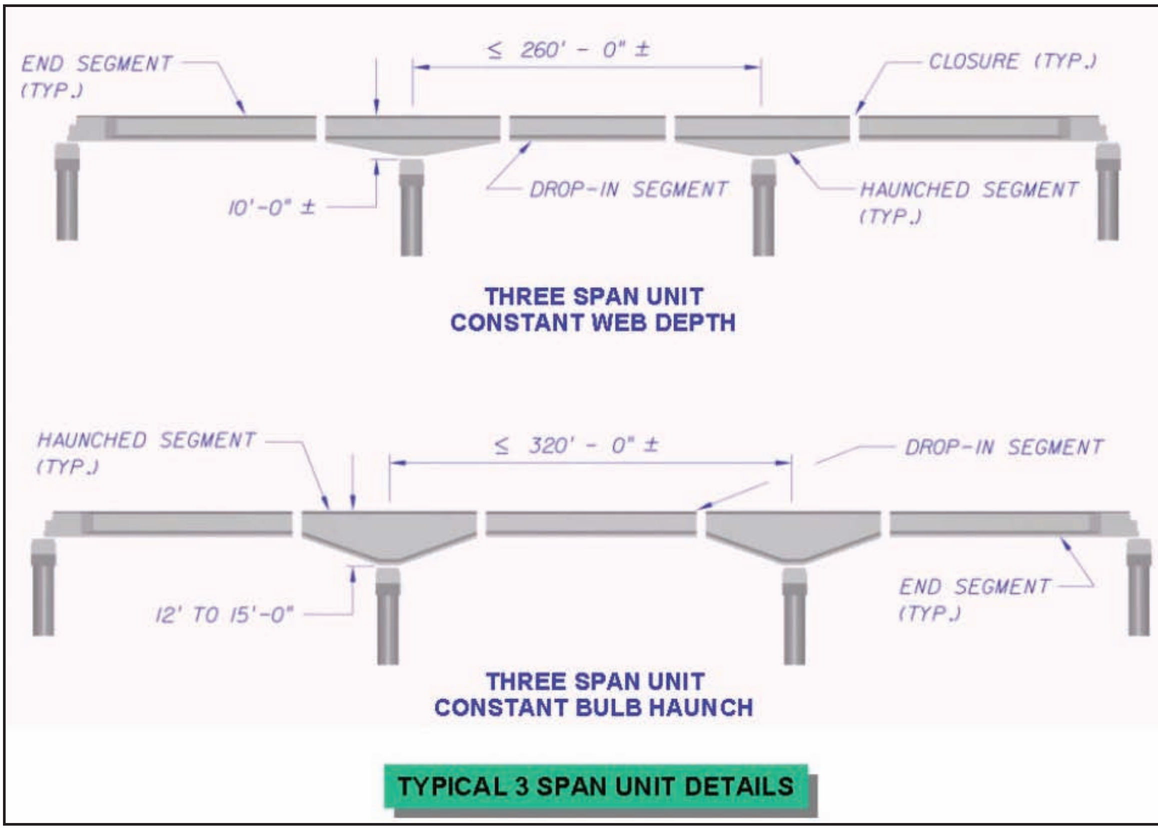

Fig. 2. Typical three-span unit details for constant web depth (top) and constant bulb haunch (bottom).

substructure units. But the economy of the system is greatest when span lengths are pushed beyond the practical limits of standard precast shapes, and advantage is taken of structural continuity.

The practical limits of current and future "standard" shapes are site access restrictions, handling loads, and transportation requirements. It is impractical or uneconomical to transport a 200 ft (61 m) girder on most roads. Similarly, it is impractical to handle and erect massive structural elements using anything but highly specialized lifting equipment. Hence, there is a compelling need to develop a segmental approach to precast composite girder design.

CONTINUOUS POST-TENSIONED STRUCTURAL SYSTEMS

Two very similar structural systems are discussed. The first is a three-span unit, typical of the structural systems used for intracoastal crossings in Florida. Several variations of the three-span unit have been designed and constructed in Florida. One of the most recently designed three-span systems is the channel unit of the Wonderwood Connector, in Jacksonville, Florida. The second system discussed is a five-span unit, proposed for the St. George Island design-build project.

The three-span units that have typically been constructed in Florida range from 650 to 750 ft (198 to 229 m) in length, with center spans of 240 to 320 ft (73.2 to 97.5 m). Each bridge girder line comprises five segments; two haunched pier segments, two end segments, and a center drop-in segment. The five-span unit for the St. George Island Bridge is 1180 ft (360 m) long, and each girder line comprises four haunched segments, three drop-in segments, and two end segments. Typical three-span unit details are provided in Fig. 2.

A brief description of the Wonderwood and St. George Island Bridges serves to highlight the project constraints leading to the choice of the post-tensioned bulb-tee girder system, and some background information on each bridge. Both projects are constructed using the 78 in. (1.98 m) deep Florida Bulb Tee, or FBT-78, as the primary prestressed member.

The Wonderwood Connector crosses the Atlantic Intracoastal Waterway just south of the St. Johns River in northeast Jacksonville. The proposed bridge is approximately 3500 ft (1070 m) in length, with approach spans of 142 ft (43.3 m), and a three-span main channel unit with spans of 195, 250, and 195 ft (59.4, 76.2, and 59.4 m), respectively. Approach units utilize simple span FBT-78s, and the channel unit utilizes modified FBT-78s, all at a girder spacing of 11 ft 6 in. (3.50 m).

There are eight girder lines, providing an overall structure width of approximately 90 ft (27.4 m). The haunched segments of the channel unit are 110 ft (33.5 m) long and 12 ft (3.65 m) deep. The main channel unit superstructure is post-tensioned in two stages using four tendons. Two tendons are post-tensioned in the non-composite FBT-78, and two are tensioned after the deck is cast. The AASHTO LRFD Code was used for design and SI units were specified for detailing.

The Intracoastal Crossing is located in an extremely aggressive marine en-

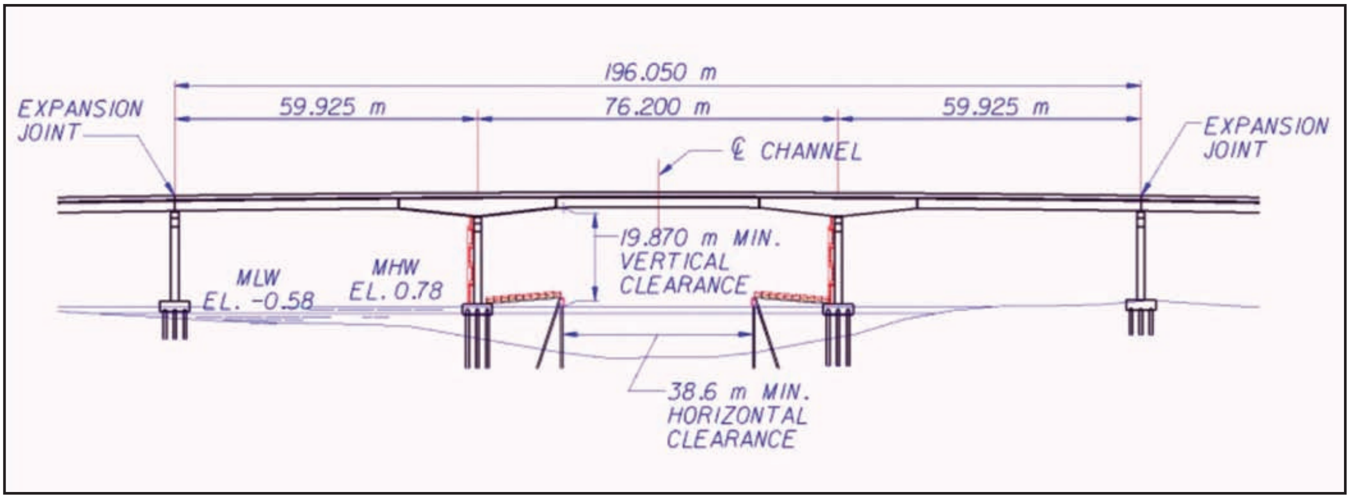

Fig. 3. Three-span channel unit of Wonderwood Bridge.

vironment, and will serve as a primary hurricane evacuation route for the Mayport Naval Station and nearby coastal residents. The Atlantic Intracoastal Waterway experiences a considerable amount of commercial barge traffic year round. In addition, the site has strong tidal currents, and the bridge is in an area susceptible to hurricane tidal surge.

The Coast Guard requires navigational clearances of 125 ft (38.1 m) horizontally and 65 ft (19.8 m) vertically. Vessel impact criteria were determined from the AASHTO Guide Specification for Vessel Collision Design. In addition, the channel piers have been designed for 40 ft (12.2 m) of scour. The maximum ship impact resistance of the piers is 1400 kips (6223 kN), based on an analysis using Method II of the Vessel Collision Guide Specifications.

The channel unit incorporates several innovations previously untried in Florida. No intermediate diaphragms are used, closure pours are 1 ft 6 in. (450 mm) wide, the web is 9 in. (225 mm) thick, and the haunched segment is 12 ft (3.66 m) deep with a constant bottom flange (bulb) thickness. These design elements will be discussed in the text to follow. Details of the main channel unit are provided in Figs. 3 and 4.

The St. George Island Bridge Replacement project is a design-build program in the Panhandle of Florida. The existing bridge is being replaced after only 35 years, based on rapid deterioration of the piling, functional obsolescence (narrow lanes and shoulders), and concern that the ship impact resistance of the piers is inadequate.

The new bridge will be 21,700 ft (6600 m) in length, spanning St. George Sound and Apalachicola Bay. At 1180 ft (360 m) in length, the five-span, post-tensioned, spliced, bulb-tee channel unit is the longest of its kind in the United States.

The bridge is approximately 47 ft (14.3 m) wide, and the high-level structure uses five girder lines 9 ft 6 in. (2.90 m) on center. Haunched segments are constant bulb-tee members 12 ft (3.66 m) deep, and 115 ft (35 m) in length. Closure pours are 1 ft $8^1/_2$ in. (502 mm) wide. Again, no intermediate diaphragms are used at the clo-

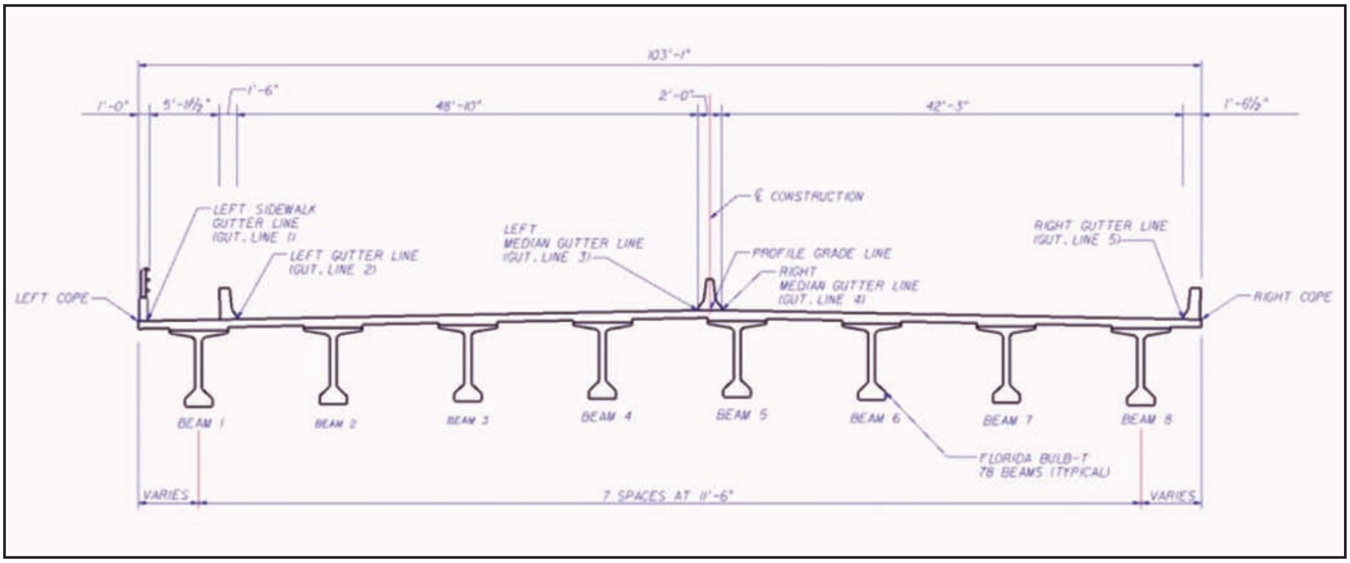

Fig. 4. Typical cross section of Wonderwood Bridge.

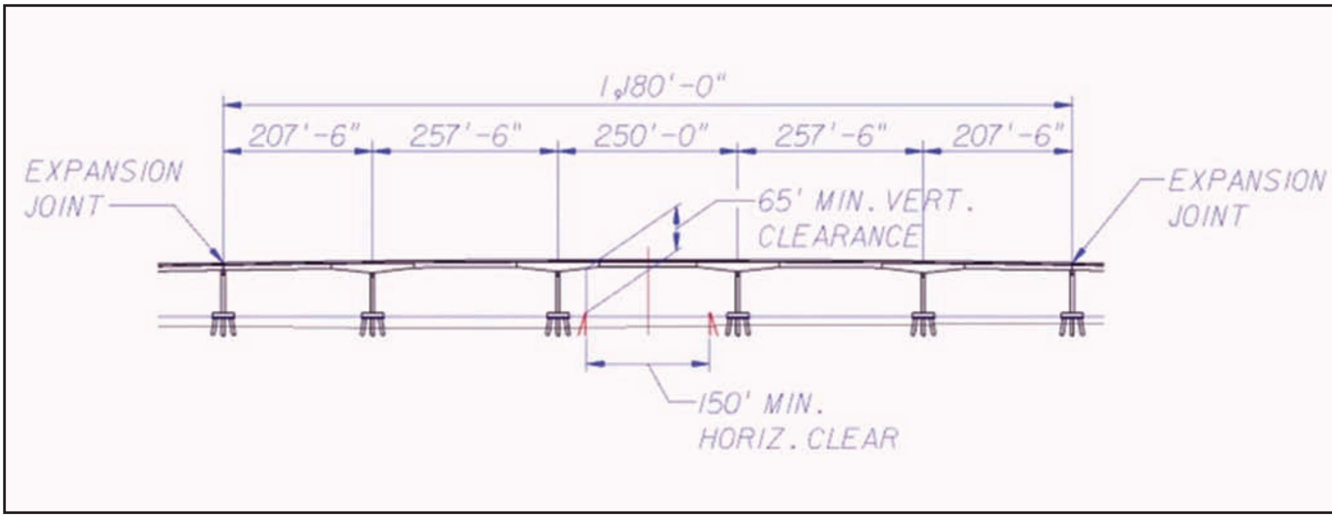

Fig. 5. Five-span channel unit of St. George Island Bridge.

sure pours. The post-tensioning system utilizes four 15-strand tendons, stressed in two stages. Design is in accordance with the AASHTO LRFD Code, and U.S. Customary units are used on the project. Details of this system are shown in Figs. 5 and 6.

The bridge spans the Gulf Intracoastal Waterway on a reach of the waterway that has a significant amount of vessel traffic. Large ocean-going barges laden with petroleum are common on this stretch of the Gulf Intracoastal Waterway. Approximately 4200 ft (1280 m) of bridge substructure is considered to be highly susceptible to ship impact, and the ship impact resistance of the piers varies from 3250 kips (14460 kN) adjacent to the navigation channel to 1075 kips (4782 kN), approximately 2100 ft (640 m) left and right of the channel centerline.

A horizontal navigation clearance of 150 ft (45.7 m) and vertical clearance of 65 ft (19.8 m) is provided over the Gulf Intracoastal Waterway. The bridge is located in an area very susceptible to hurricanes, and though tidal currents are minimal, wind induced currents in the shallow bay can produce significant scour.

Ship impact criteria dictated the configuration of the high-level structure. Short spans require a greater number of relatively massive piers, though superstructure costs are very reasonable. The longer the span, the higher the superstructure cost. Consequently, to provide the most competitive structure for the design-build competition, a process of optimization was completed.

Different span configurations and pier sizes were evaluated to provide the best possible balance of superstructure and substructure cost. The end result was a five-span channel unit with spans of 205, 260, 250, 260 and 205 ft (62.5, 79.2, 76.2, 79.2 and 62.5 m), respectively. Moment distribution was used to balance positive and nega-

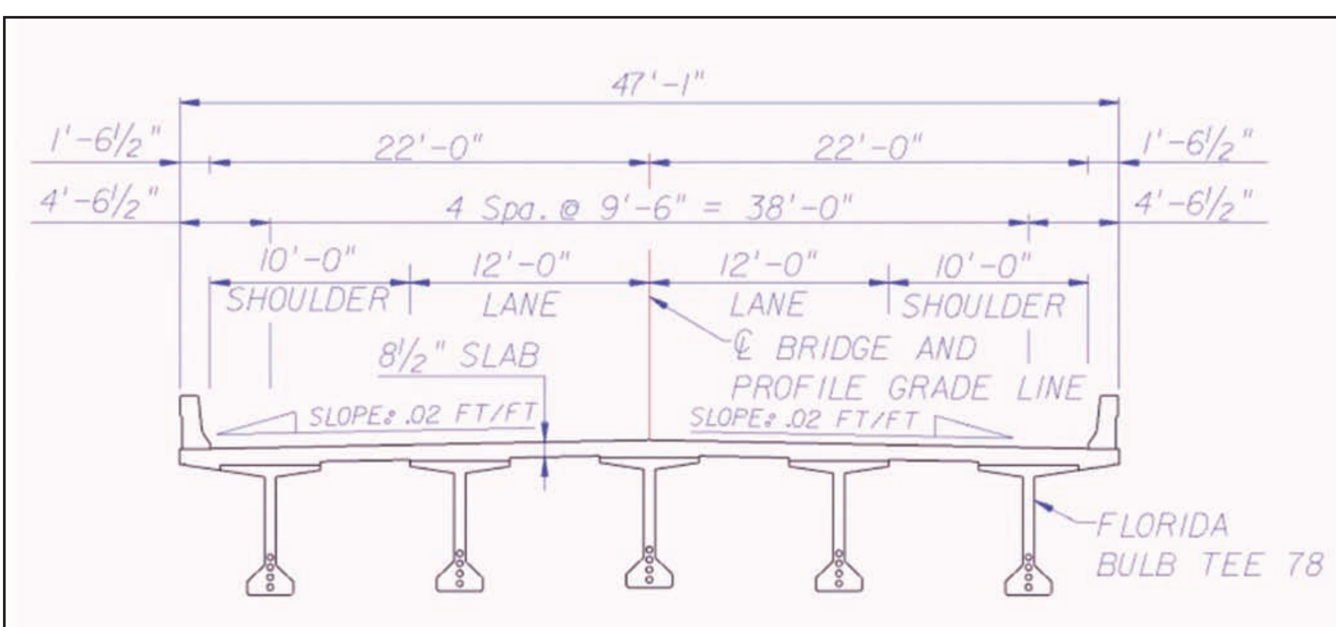

Fig. 6. Typical cross section of St. George Island Bridge.

tive moments during preliminary design, resulting in a channel span of 250 ft (76.2 m) and flanking spans of 260 ft (79.2 m).

GENERAL DESIGN CONSIDERATIONS

At the outset of any project in which a three-span continuous unit is considered, it is recommended that a preliminary frame analysis of the continuous girder be completed. The moment diagram for a uniformly distributed load (or more accurately the moment diagram for a distributed load that reflects the member self-weight distribution) should be generated.

Inspection of the moment diagram will provide some insight into the location of inflection points, and the relative magnitude of positive and negative moments. Though the location of closure pours need not fall directly over inflection points, closure pours should be located in close proximity to inflection points.

Several different haunch depths should be considered to assess the sensitivity of the structural system to the stiffness of the haunches. The deeper the haunch, the greater the negative moment drawn to interior piers. In addition, by varying the pier spacing and relative span lengths, peak moments may be reduced to some degree. However, excessive reliance on initial runs that do not reflect incremental construction of the structural assembly is unwarranted.

Three separate analyses of the structure using a general-purpose structural analysis program are needed for more accurate information. The first analysis considers the non-composite section hinged at closure pours. The next analysis considers the non-composite girder continuous from one end to the other with equivalent externally applied loads to represent the first-stage post-tensioning. The third run considers the composite girder with second-stage post-tensioning and live load envelopes. The results of these analyses are superimposed to approximate the ultimate structural behavior, but even then it should be understood that important aspects of the structure behavior of the bridge are ignored.

Several initial computer analyses may be used to get a better "feel" for the relationship of different geometric configurations, and an estimate of the total prestress required. Beyond that, the use of a computer program that allows incremental construction, and accumulation of prestress losses, creep and shrinkage, is believed to be necessary. Time-dependent effects can be significant factors affecting both the stress and deflection of the structural system.

The computer program used should also be capable of generating stresses due to temperature changes and thermal gradients. The AASHTO LRFD Code requires that the design consider nonlinear thermal gradients.

Material Properties for Design

The strength and time-dependent properties of the concrete used for precast, prestressed members, for the composite deck, and for the prestressing strand and tendons are essential parameters in the analysis of composite spliced girders. The design parameters used for the St. George Island and Wonderwood projects take advantage of the latest revisions to the FDOT Structures Design Guidelines and Standard Specifications. Concrete strength and prestressing strand properties are listed below.

Note that creep and shrinkage are discussed in the section that follows Concrete and Prestressing Steel.

Concrete

Precast Girders and Closure Pours
- Specified compressive strength: f'_c = 8500 psi (58.6 MPa)

Deck and Diaphragms
- Water-cementitious material ratio (maximum): 0.40
- Specified compressive strength: f'_c = 6500 psi (45 MPa)

Prestressing Steel

Strands – ASTM A416, Grade 270 low relaxation

Strand diameter = 0.6 in. (15 mm) (pretensioning and post-tensioning steel)

The use of 8500 psi (58.6 MPa) high strength concrete for girder design is a recent development at the FDOT. So too is the use of 0.6 in. (15 mm) diameter strand for pretensioning precast girders. Both developments have substantially increased the capacity and span range of precast girder bridges. However, a cautionary note is warranted, because very long slender bulb-tee girders exhibit a tendency to deflect and twist during handling and erection.

As the trend towards long slender members evolves with the use of high strength concrete, the lateral stability of members will become a more important issue. The FBT-78 used extensively in Florida has a 5 ft (1.52 m) wide top flange, and is considerably more stable than many other bulb-tee shapes, making it a good candidate for a pilot project using high strength concrete.

The deck concrete used for the Wonderwood and St. George post-tensioned channel units has a compressive strength of 6500 psi (45 MPa) and maximum water-cementitious material ratio of 0.40. The design strength is significantly higher than specified for conventional simple span construction, in part to boost the effectiveness of the composite member, as well as increase the allowable tensile stress in the deck.

Because shrinkage is particularly detrimental to the performance of the composite system, a maximum water-cementitious material ratio is specified. Use of proprietary shrinkage-reducing additives for the deck appears to offer additional benefits.

Creep and Shrinkage Parameters for Analysis

Creep and shrinkage are concrete properties that have a marked effect on the stress and deflection in continuous prestressed members and an even more significant influence on composite prestressed members. The ultimate creep and shrinkage coefficients used in analysis can determine the number of tendons employed to satisfy stress limit states.

An excessively conservative value of creep or shrinkage can make satisfaction of allowable stress design nearly impossible (without reducing span length or using deeper members). On the other hand, unconservative values of creep or shrinkage will yield unconservative estimates of tendon requirements, and provide stress resultants that will not be obtained in the field.

In the absence of project-specific testing for determination of creep and shrinkage coefficients, values of ultimate creep and shrinkage should be used that have been obtained through previous projects or mix design testing. The FDOT has provided conservative values of creep and shrinkage to be used with ACI Code provisions for time-dependent analysis of post-tensioned structural systems.

Based on research conducted at the University of Florida, an ultimate creep coefficient of 2.0 and ultimate shrinkage coefficient of 0.0004 have been specified for typical mix designs utilized by the FDOT for precast, prestressed girders. No distinction between the coefficients of deck and precast concrete mixes is made. Further research is probably warranted for composite systems, to better estimate creep and shrinkage in deck concrete originating at ready-mix batch plants.

It is useful to run analyses of the structural behavior of the post-tensioned system under investigation using several different values of creep and shrinkage, alternately varying creep and shrinkage. This analysis serves to underscore the effect each variable has on the system, and the sensitivity of the structure to variations in creep and shrinkage. Several general observations regarding creep and shrinkage are warranted.

Examples of problems that have manifested themselves in the design of composite spliced bulb-tee girder bridges follow.

• First is the differential shrinkage between a precast girder, perhaps 120 to 180 days old, and a deck pour made after first stage post-tensioning. It is particularly detrimental over haunched girders where the deck will undergo tension due to restraint of shrinkage, so that compression derived from second-stage post-tensioning of the deck is sometimes negated. Consequently, allowable tension stresses may be exceeded in the deck with the application of live load and introduction of negative bending stresses.

• Second is the development of a tensile force in the deck over the drop-in and end segments which tends to compress the top flange of the precast girder (through an equal and opposite reaction), and consequently reduce the effectiveness of the prestress in the bottom of the girder.

Note that the second effect is often realized in simple span structures, where the effect of deck shrinkage on mature girders tends to bow the beam down.* These phenomena point to the necessity of carefully selecting a shrinkage coefficient that is representative of the concrete being used.

Shrinkage in composite systems can create sizable forces between the deck and girders. On the Wonderwood project, discussed earlier, analysis indicates that a 50 psi (0.345 MPa) deck tension due to differential strain between the deck and girder elements cast at different times occurs. Note that the stress pertains to the final design of the Wonderwood Channel unit, where every reasonable attempt was made to mitigate differential shrinkage stresses.

Unfortunately, even sophisticated computer programs may overestimate the shrinkage force that develops between members. Programs do not provide a stress reduction to reflect development of shrinkage cracks. Instead, differential strain between two members due to differing rates of shrinkage is calculated as stress, using Code specified elastic moduli.

In addition, the tensile stress that develops due to differential shrinkage leads to a force unaffected by shear lag. In more tangible terms, the differential strain in a deck element is not reduced by shear lag between the deck and girder. Nor is there a lessening of tension due to development of shrinkage cracks.

The stress that develops due to differential shrinkage may approach the allowable tensile stress specified by the AASHTO and ACI Codes, and current software does not address cracking and subsequent stress relief. Shrinkage stresses in excess of 100 psi (0.69 MPa) are sometimes computed using programs performing time-dependent analysis.

There are effective countermeasures that can be taken to alleviate differential shrinkage:

• First, low water-cementitious materials ratios in the deck concrete, or shrinkage-reducing admixtures may be used. Use of low shrinkage deck concrete will reduce tensile stresses significantly. The author has performed analyses using shrinkage coefficients ranging from 0.0004 to 0.0003 for the deck, and a constant of 0.0004 for the girders. These analyses show dramatic improvement in the stress distribution due to shrinkage.

• Second, realization that differential shrinkage is greatest when the age difference between the deck and girder is pronounced offers a viable solution. Haunched members, if cast late in the production schedule, will still be relatively young when the deck is placed. This will reduce the magnitude of differential shrinkage. The issue of the casting schedule as it affects time-dependent analysis will be further discussed in the text to follow.

Creep can have similar effects on the performance of a composite system. The effect of creep is less marked, since it is primarily a function of the applied load. A load must be present to induce creep. Because creep will be greatest in young concrete, it may be advisable to delay post-tensioning to reduce creep losses. On the other hand, creep will tend to shed load from highly stressed members to less highly stressed members.

One notable effect of creep on both three- and five-span units is the change in stress that occurs between the beginning of service life and the end of service life. It has generally been found that the deck compression over haunches relaxes with time, and that end and drop-in girder bottom flange compressive stress also relaxes. The effect is essentially a reduction in the system stiffness over time. Consequently, it is good practice to review service load stresses at both the begin-

* A similar phenomenon may be responsible for the difference between measured and predicted camber in Florida Bulb Tee (FBT) girders. Shrinkage in the exposed top flange of the girder can result in web compression. The resultant compression in the top of the web may counteract the effective prestress in the member.

ning and end of the projected service life of this structure.

Effect of Casting Schedule on Analysis and Design

It is apparent from the discussion of creep and shrinkage parameters above that the schedule used to construct a composite segmental structure can significantly affect the calculated stresses in deck and girder members. In addition, it is unlikely that the design engineer will have control over the casting schedule used by the contractor building the bridge once it is designed. Consequently, it is good practice to produce at least two different casting schedules for incremental construction of the structure.

The first should reflect an "optimized" schedule, with precast, prestressed members cast and deck pours made for least detriment to the composite system stress distribution. The second should be a casting schedule that departs from the "optimum" schedule, so as to adversely affect member stresses. Both should be viable achievable schedules.

On the one hand, the "optimum" schedule will minimize differential shrinkage between members (all members cast and poured in the same time frame, i.e., a compressed schedule). At the same time, and to the extent possible and consistent with the first condition, the "optimum" schedule will allow girders, deck and closure pours to cure sufficiently to reduce immediate and pronounced creep (all precast and deck pours made four or more weeks before post-tensioning) before stressing tendons.

On the other hand, the adverse schedule will have deck pours made well after all precast members are cured (aggravating differential shrinkage), and post-tensioning will be carried out as soon as the concrete strength permits (aggravating creep). These are, in the author's opinion, the opposite ends of the spectrum as related to the casting schedule.

Temperature and Thermal Gradient Effects

The effect of a thermal gradient on the ultimate strength of a continuous concrete structure is not believed to be significant. At ultimate strength, the continuous member will most likely undergo considerable stress redistribution due to cracking and subsequent redistribution of loads, including thermal and hyperstatic actions.

This condition is reflected in the latest edition of the AASHTO LRFD Code, in which the load factor applied to the strength limit states for a thermal gradient is zero, without project specific information to the contrary. It is noteworthy that the AASHTO LRFD Code does not consider stresses due to thermal gradients.

The AASHTO LRFD Code does provide a nonlinear gradient for review of service limit states, and specifies both positive and negative temperature gradients in Section 3.12.3. The thermal gradient produces two sets of stresses that must be superimposed: Self-equilibrating stresses:

$$\sigma_{se}(y) = \sigma(y)_{restrained} + [-(P/A) - (M\, y_{bar}/I)]$$

and stresses due to an equivalent linear thermal gradient:

$$\sigma(y)_{equiv\ linear} = P_c/A + (M_c y/I)$$

where P_c and M_c are continuity moments obtained from an analysis of the equivalent linear thermal gradient on the continuous structure.

Only a few of the computer programs currently available will handle a nonlinear gradient (most will handle a linear gradient), requiring manual superposition of the self-equilibrating stresses to obtain final results.

Stress resultants from nonlinear thermal gradients can be significant. Positive gradients tend to compress the deck over haunches, while increasing tension in the bottom of girders near the center of the spans. Negative gradients reduce compression in the deck over haunched segments and add compression at the base of girders near the center of the spans.

Load factors in the AASHTO LRFD Code do not give much weight to thermal gradient stresses, since live load is given a load factor of 0.5 when combined with the thermal gradient. It is the author's hope that further discussion of thermal gradient effects be included in future revisions of the applicable codes. This appears to be an area where further research is warranted.

Allowable Stress and Ultimate Strength Design Considerations

The design of precast, prestressed members, as well as precast post-tensioned members, must satisfy both allowable stress design and ultimate strength. The LRFD Code recognizes this by providing both strength and service load limit states. The code provisions for allowable stress for prestressed and post-tensioned members is intended to limit or prevent cracking, so as to protect strand from corrosion. Applied to the composite post-tensioned structure, allowable stress design leaves some room for interpretation.

As a practical matter, the extreme fibers of the deck and girders are checked to verify conformance with allowable stress design. However, the deck is neither prestressed nor post-tensioned in the sense that the section contains no prestressed strand. The deck uses mild steel reinforcement.

The deck of a composite steel structure does not have to satisfy allowable stress design. Instead, the deck is reinforced in negative moment regions to reduce the width of potential hairline cracks, but is not considered a part of the nominal moment-resisting system.

From a strength perspective, there does not appear to be any compelling reason to treat the deck in the post-tensioned bulb-tee composite deck any differently. The deck contains no prestressing steel. Nonetheless, current practice likes to provide sufficient second-stage post-tensioning to precompress the deck against anticipated live load bending moments and to satisfy maximum allowable tensile stresses (less than the modulus of rupture).

It is arguable that the practice of limiting stress in deck elements is done to ensure that the analysis properly describes the stiffness of the composite system. Should the deck crack, the stress distribution is improperly described by linear elastic behavior in the deck. Deflections are also improperly described once cracks develop.

Stress in the underlying precast member increases substantially once cracks develop. These phenomena are not easily modeled using standard methods of analysis.

If not for the need to ensure an accurate model for analysis, the second-stage post-tensioning required in the bulb-tee systems might be substantially reduced. The amount of prestress in the bulb-tee system used to satisfy allowable stress design generally provides nearly twice the steel needed by a review of strength limit states. This apparent discrepancy is not discussed or considered by the code at this time. It may be that future code revisions (and computer models) will address this issue in light of the increasing popularity of the spliced post-tensioned bulb-tee design.

DETAILING AND ERECTION CONSIDERATIONS

This section discusses the various factors that affect the detailing and erections of the girders and post-tensioning practices.

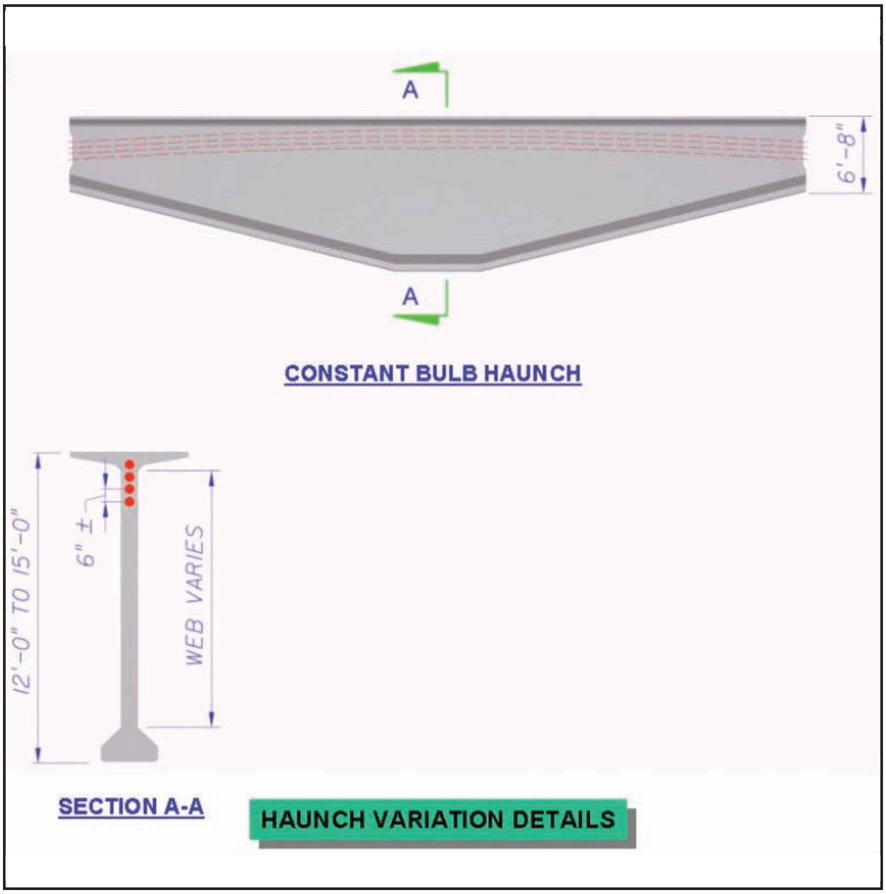

Fig. 7a. Bulb-tee girder with constant bulb depth.

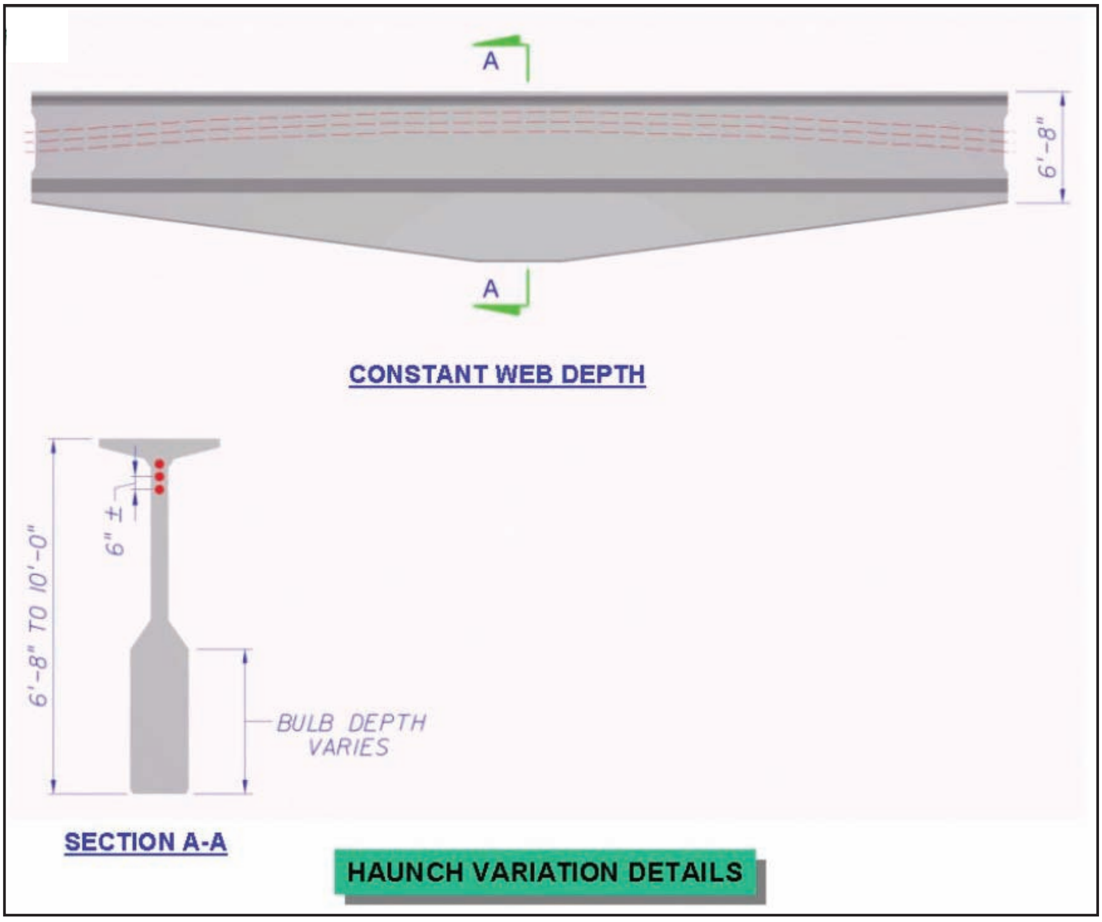

Fig. 7b. Bulb-tee girder with constant web depth.

Girder Segment Geometry

All Florida Bulb Tee (FBT) segments have length limitations imposed by the amount of prestress that can be fitted within the girder cross section. In the case of the end and drop-in segments, prestress is required in the bottom bulb, since segments will be simple span members subject to combined self-weight and prestress. Stresses must be maintained within allowable limits throughout the length of the girder (particularly at the girder centerline and ends).

In addition, Florida (and other states) has provisions for minimum camber in prestressed concrete members under girder self-weight. This provision ensures that there is evidence of adequate prestress in a member when force is transferred from the casting bed. Erection equipment load limitations or transportation and access requirements may impose a further length restriction, but these parameters rarely control the length of end and drop-in segments.

Haunched segments require the preponderence of the prestress force in the top flange of the member, since these segments cantilever off of piers and must be capable of handling the weight of end and drop-in segments supported from the ends of the segment prior to post-tensioning. Haunched segment length restrictions are subject to several variables:

• First is the limitation imposed by the size of the top flange. This limits the amount of prestress that can be employed to resist cantilevered bending. To compensate for a possible lack of adequate cross-sectional area in the top flange, the flange thickness can be increased. An increase in top flange depth of only 2 in. (50 mm) can provide sufficient additional cross section to double the member strands of the top flange. Increasing top flange thickness is not a significant forming problem.

• The second limitation, related to the amount of prestress in the member, is the bending stress resulting from prestressing the top flange. Until the segment is supported from the pier, and the weight of the drop-in and end segment(s) is applied at the ends of the haunches, there is a considerable positive moment due to prestress in the center of the haunched segment.

The positive moment can precipitate cracking of the bottom flange and reinforcement must be added at the base of the member to resist the net tensile force. Tensile stresses must be limited to satisfy the provisions of AASHTO 5.9.4.1.2. The moment can, and should be, offset by adding supplemental prestress as low as possible in the cross section to lower the tensile stresses in the bottom of the haunch.

• Third is the transportation and handling limitation imposed due to practical weight restrictions of the member. As service load negative moment over the pier increases, the haunch depth must be increased for additional bending resistance. The weight of the segment increases disproportionately, a factor further described below.

For center spans of 240 ft (73.2 m), the haunched segments have generally been 10 ft (3.05 m) deep, whereas for center spans of 320 ft (97.5 m), the haunched segments may be as deep as 15 ft (4.57 m). The majority of haunched segments use a standard bulb-tee section with a variable depth bottom flange. This is practical if the haunch depth is 10 ft (3.05 m), but becomes more and more unwieldy as the haunch depth increases.

For deeper haunches required for longer spans, it is better to form the haunch using a constant bulb thickness to reduce the weight of the segment. The difference in weight between segments with constant web depth as opposed to constant bulb or flange thickness is substantial, and must be considered when evaluating casting yard limitations and erection picks. Fig. 7 provides details of the haunch segment variations discussed above.

End segments are formed using the standard bulb-tee section, but have in common (on every project) an end block to accommodate tendon anchorage. Tendon anchorage details may vary substantially between projects. Where all tendons are anchored on the vertical face of the end segment, post-tensioning must be done prior to placement of the abutting girder lines in the bridge.

If all tendon block-outs are in the top of the end segment, tendons may be stressed at any time. More commonly, the first-stage tendon anchorages are located in the ends of the segment, and the second-stage tendon anchorages are located on the top of the segment. This allows second-stage post-tensioning after deck pours are made on all girders.

A general rule of thumb for determining the length of the end block is to set it equal to 1.5 times the depth of the girder. This will ensure that stress concentrations at anchorages have smoothed out across the entire cross section within the reinforced end block. The variations to end block design are shown in Fig. 8. More attention will be given to end block design in a later section.

Drop-in segments are the simplest members in terms of detail and design. They are essentially the standard precast section with minimal modification.

Modifications to Accommodate Post-Tensioning Ducts

Proper fit of an appropriately sized tendon duct within the web of a bulb-tee girder must satisfy concrete cover requirements over shear stirrups and generally requires adjustment of the web thickness. Early post-tensioned bulb-tee girders used a web thickness of 7 in. (178 mm). To accommodate a tendon with upwards of twelve 0.6 in. (15 mm) diameter strand, the tendon duct in the girder was specified to be elliptical (oval) rather than circular, to provide greater side cover. Experience in Florida with oval ducts proved to be unsatisfactory. It was not always possible to thread twelve (or more) strands through the tendons, due to the limited cross section of the duct.

In some cases, the oval duct was found to have been indented or otherwise mis-shapen when girders were cast, probably due to hydrostatic pressure or impact by hand-held vibrators. At the Edison Bridge in Ft. Myers, Florida, an oval duct was used and web splitting was observed. Some bridge engineers theorized that the contact stresses between the sharply rounded oval duct and web promoted the splitting.

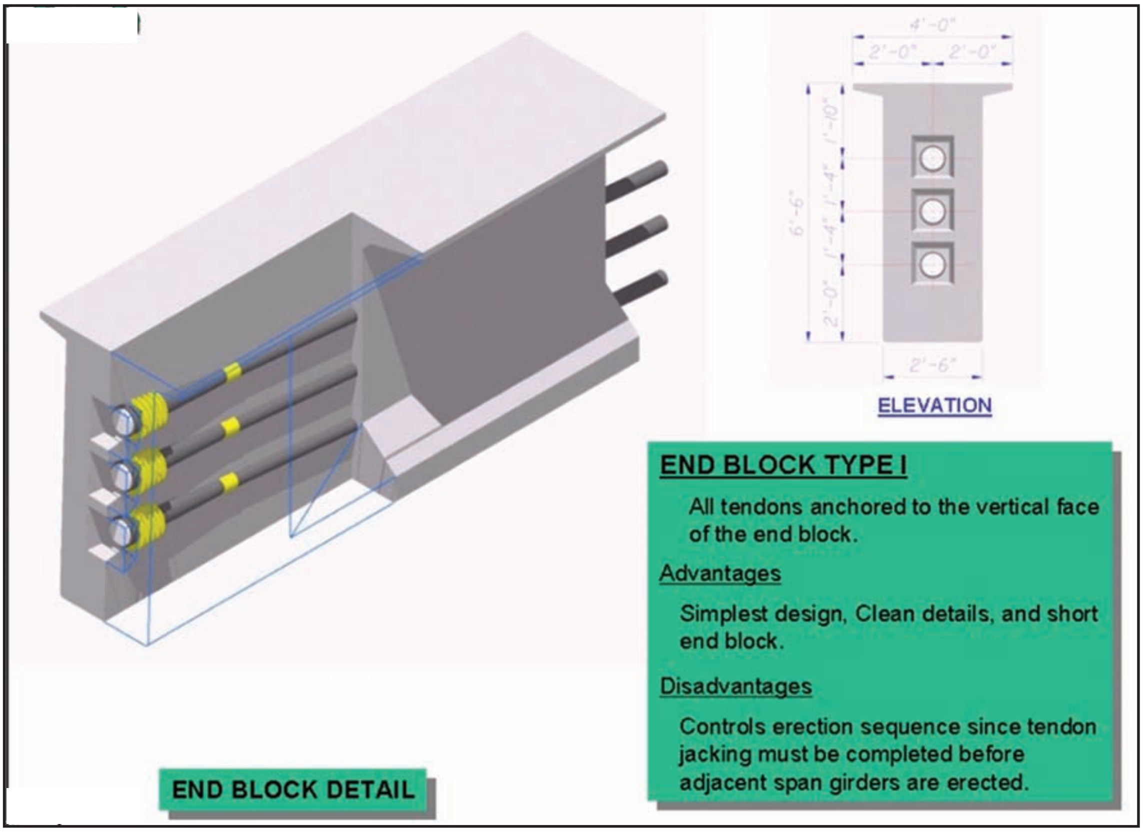

Fig. 8a. End block detail (Type I).

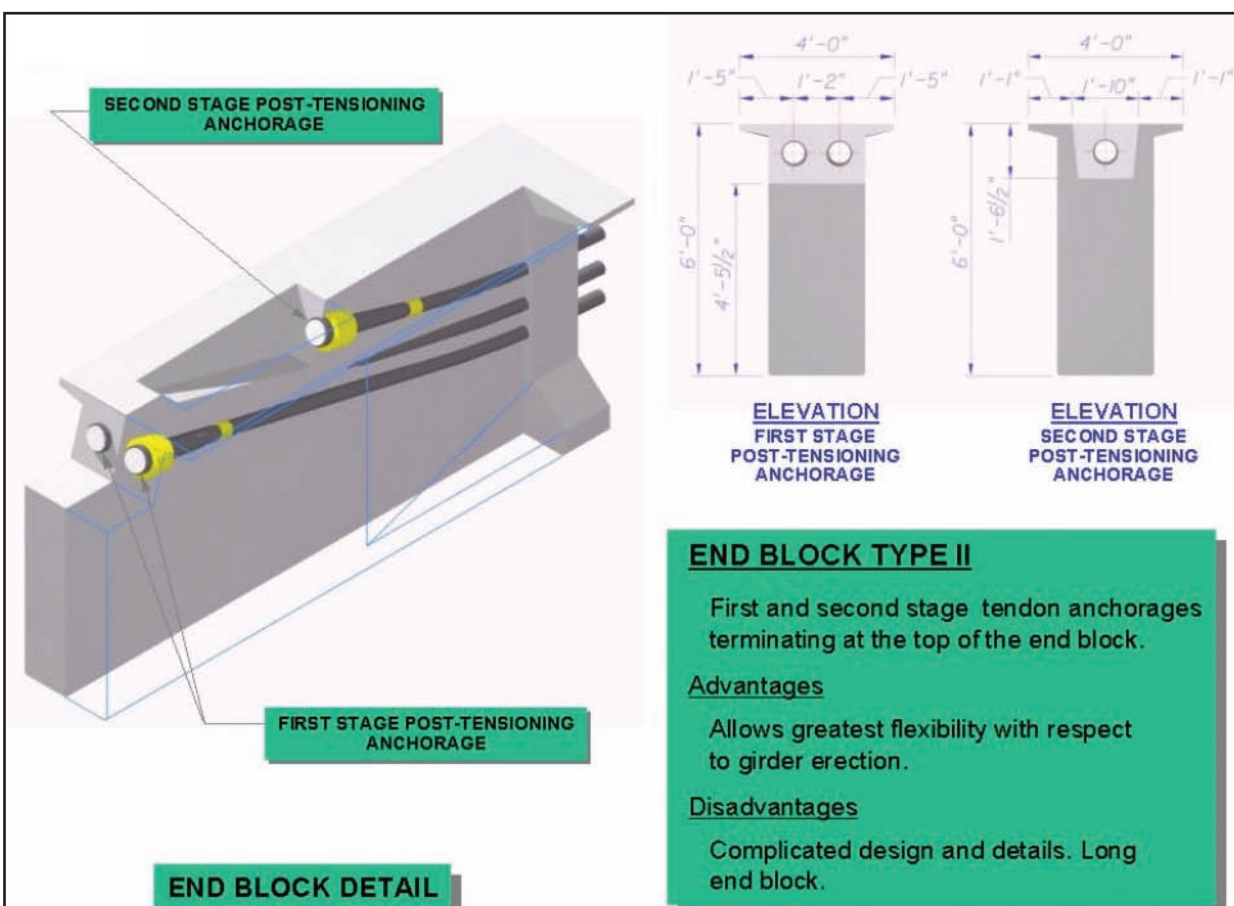

Fig. 8b. End block detail (Type II).

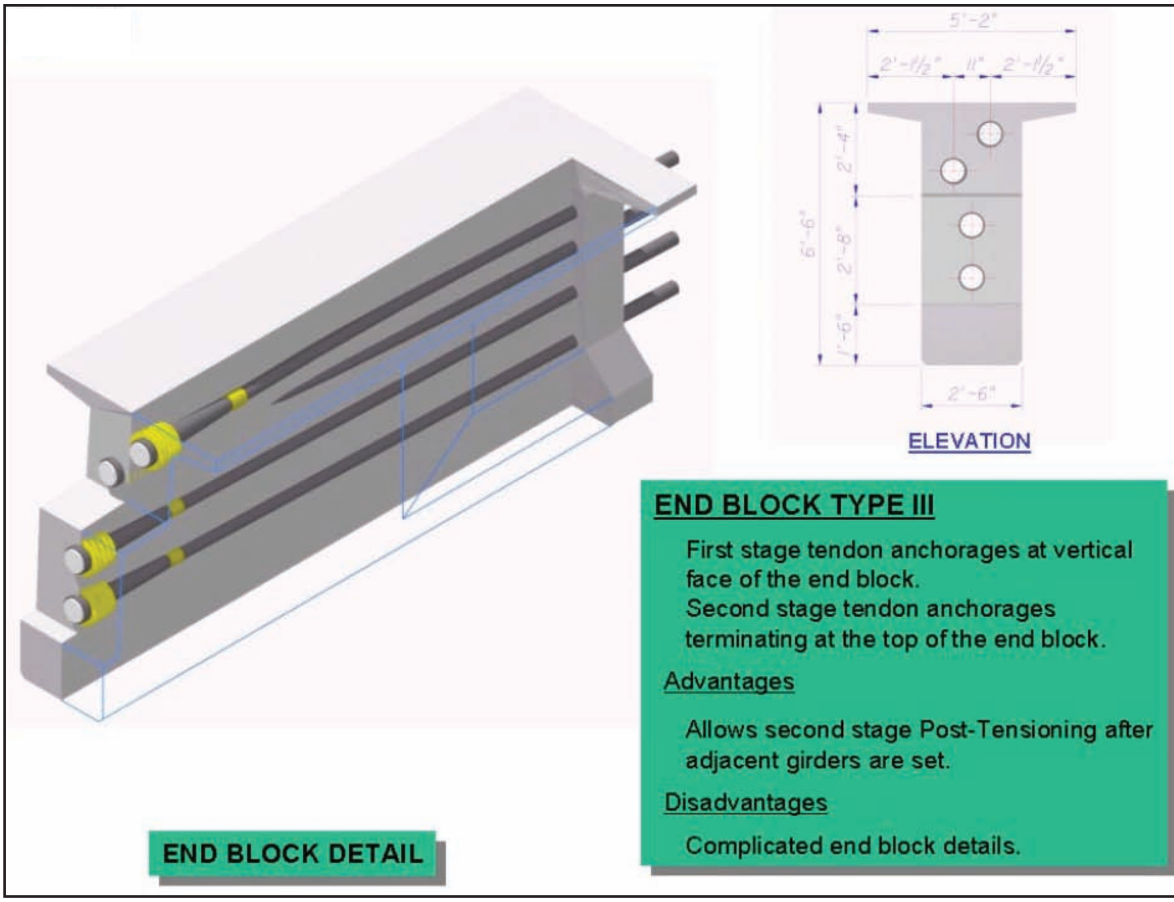

Fig. 8c. End block detail (Type III).

Regardless, the use of oval ducts for post-tensioned spliced bulb-tee girders has been terminated in Florida. Circular duct is now used exclusively and either corrugated metal or polyethylene duct is permitted. Typically, the side forms of bulb-tee girders are spread apart to permit use of an 8 in. (203 mm) web with circular ducts. This provides more tolerance for the placement of ducts while maintaining required cover. It has proven to be a satisfactory adaptation.

Nonetheless, the author has, for the last several projects, specified use of a 9 in. (229 mm) web. This is due to a continuing concern over the ability to thread up to 15 or 16 strands over long distances through narrow undulating tendons, and a desire to provide greater web thickness for both strength and serviceability. The subcontractors who install strand and post-tension the spliced girders generally agree that an "oversized" duct is preferable when post-tensioning a length of tendon in excess of 600 or 700 ft (183 or 213 m).

There also appear to be strong arguments favoring larger ducts to better accommodate tendon grout after strand is installed. Tendon grouting will be further discussed in the text to follow. Note that whether the girder web thickness is 8 or 9 in. (203 or 229 mm), AASHTO 5.4.6.2 specifies that the duct shall not exceed 0.4 times the gross concrete thickness at the duct. This is tied to shear provisions referenced later in the AASHTO Code.

This provision is often ignored (or violated) in the design of post-tensioned bulb-tee girders. Because tendons are ultimately grouted, it may be argued that post-tensioning ducts are not discontinuities during the service life of the bridge. However, the voids do exist when all dead load shear forces are introduced. The Standard and Modified FBT-78 are presented in Fig. 9. Note the different strand template in the bottom flange of the girder to accommodate tendon ducts for post-tensioning.

Closure Pour Details

Another design consideration typical of all segments in a spliced bulb-tee girder bridge is the detailing and dimensioning of closure pours. Closure pours are typically located near points of dead load contraflexure to minimize flexural stress across the joint. Most spliced girder projects have adapted use of a 1 ft (305 mm) gap, or closure pour, between segments. This joint must be large enough to allow tendon duct splicing, but should be short enough to minimize the length of the closure joint since there will be minimal longitudinal reinforcement and no pretensioning across the joint.

It is difficult to effectively splice tendon ducts in a 1 ft (305 mm) gap; similarly, it is difficult to splice longitudinal reinforcing bars across a 1 ft (305 mm) joint, no matter what size of bar is proposed. Several details have emerged to compensate for these difficulties.

Shear keys have been employed in the webs of girders. These indentations provide some mechanical interlock between the precast segments and closure pour concrete. Just as impor-

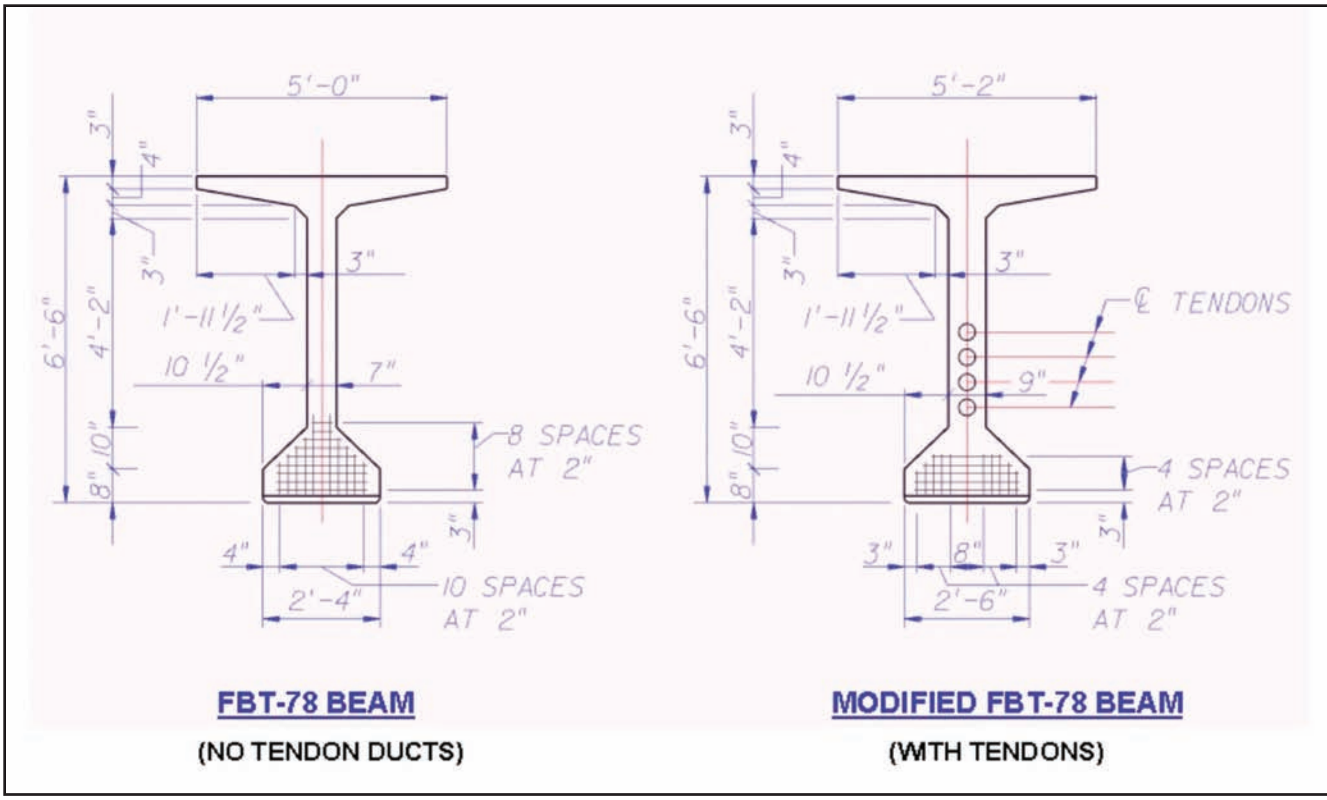

Fig. 9. Florida bulb-tee girder variations.

tantly, they provide more space to affect the tendon splices, and smooth out any discontinuities in the duct profiles between adjacent segments should they occur.

Splice details proposed for closure pours include use of overlapping U-bars. This detail was prevalent in earlier spliced girder projects. It has not always been an effective detail. On the most recent projects completed by the author, the closure pour gap has been increased from 1 ft (305 mm) to as much as 1 ft 9 in. (533 mm). This allows greater flexibility in the detailing of tendon splices and permits longitudinal bars in adjacent precast segments to be spliced using code-specified lap details.

It is important to note that the integrity of longitudinal bar splices are critical, since it is recognized that the shear transfer mechanism in concrete girders depends in part on the contribution of longitudinal reinforcement.

The tendon profile across closure pours is also an important consideration. The offset of the tendons with respect to the neutral axis of the section and the effect that secondary (hyperstatic) moment has on the distribution of flexural stress in the [closure pour] cross section must be considered. The combined effects of the primary prestress, secondary moments, and dead load stress distribution across closure pours should not be ignored in design.

Consideration of these actions may dictate adjustment of the tendon profile. Also, the tendons provide the bulk of the tensile force contributing to the internal moment in the cross section at ultimate strength. In addition, ultimate live load moments at closure pours may subject the section to stress reversals depending on closure pour location. This stress reversal indicates that strength must be provided to resist both positive and negative moments at closure pours.

Intermediate Diaphragms

Intermediate diaphragms have typically been used at closure pour locations. They are used to provide lateral stability. Diaphragms are generally cast at the same time closure pours are made. Diaphragm reinforcement between girders adds an additional level of congestion within the closure pours, where longitudinal reinforcement between segments, tendon duct splices, and shear stirrups already compete for available space.

Diaphragms are necessary for spliced bulb-tee systems that have to be kinked at closure pours to accommodate horizontal curves in the roadway alignment. On straight alignments, they will not normally distribute forces between girders, except for transient effects of live load and wind. They add a focused mass within the superstructure that may increase the inertial response to a seismic acceleration.

On both the Wonderwood and St. George Island projects, they have been omitted to simplify detailing, and reduce unnecessary weight. Temporary cross bracing between girders and erection shoring towers provide lateral stability until the deck is cast, whereby lateral stability is provided through diaphragm action of the deck.

Tendons and Tendon Profiles

Parabolic tendon profiles have been used on both the Wonderwood and St. George Island projects. The tendon profiles are set to counteract the dead

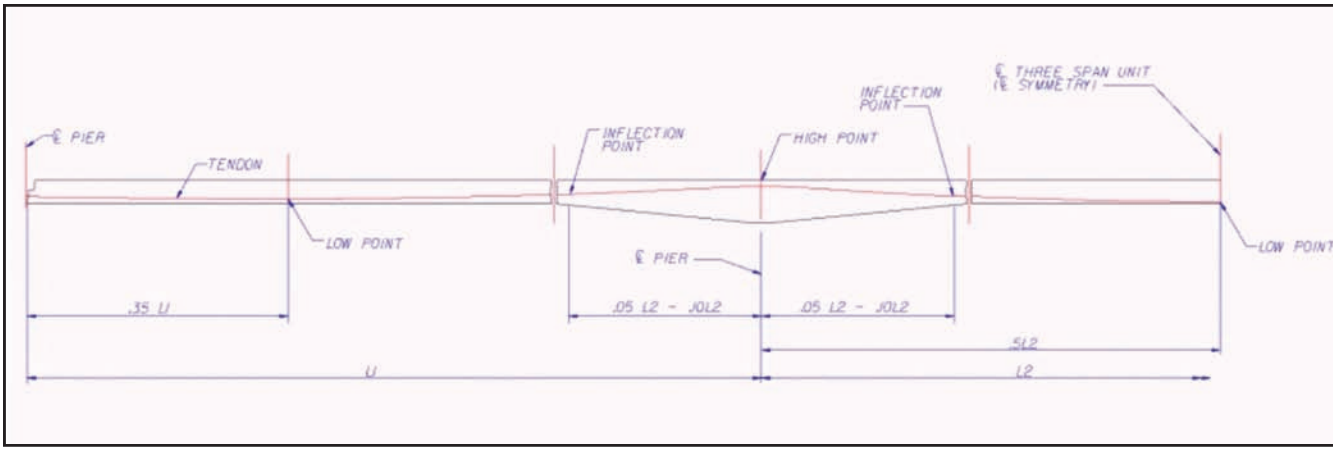

Fig. 10. Parabolic tendon profile.

load moments in the continuous unit. The method is essentially a load balancing procedure. It reduces the magnitude of secondary moments by providing tendon profiles that are as close to concordant profiles as possible.

A good rule of thumb often applied for preliminary design is to set the low point in the exterior spans at a distance of 0.35 times the span length (from exterior supports) and locate high points directly over piers. Inflection points in the tendon profiles should be located 0.05 to 0.10 times the span length away from interior piers. Tendon high points invariably occur over interior supports.

There have been a number of projects in which the profile of the top tendon in the haunched segments rises above the top flange of the girder. This has been done to ensure compression of the deck over interior supports. This presents a handling problem, since the duct will be exposed until the deck is cast. However, unless the duct is completely exposed, it will most likely be embedded in the build-up between deck and girder.

Build-up over haunched segments is typically $4^1/_2$ to $5^1/_2$ in. (114 to 140 mm), based on camber requirements. Extension of the tendon duct above the top flange of the precast section is undesirable, because of potential damage to the duct during handling. Yet, because strands will not be placed in the tendon until the deck is cast, it is sometimes done. If the time-dependent analysis indicates that tension is a problem in the deck over supports, raising the tendon profile is only one of several options available.

As discussed earlier, it may be more appropriate to investigate the tension resulting from shrinkage and attempt to alleviate differential shrinkage before raising the tendons or adding strand. A typical tendon profile for a three-span unit is shown in Fig. 10. Also shown is a cross section of the haunched segment over an interior pier with the top tendon extending above the top flange of the member.

The "optimum" tendon profile and number of strands requires an iterative analysis. Some adjustment of the profile will probably be desired after closure pour stresses are reviewed and maximum live load envelopes are superimposed on the structure's dead load moment envelopes.

Tendon profiles should be smooth and parabolic, without sharp curvature, to facilitate tendon placement in girder formwork and provide efficient load balancing. Nonetheless, departure from smooth profiles may be necessary in the end blocks to accommodate tendon anchorage. If so, deviation stresses and local effects may require design of supplemental reinforcement.

Departure from parabolic tendons that oppose the moment diagram in the members will almost certainly increase secondary moments (alternatively called hyperstatic, parasitic or continuity moments). This can be turned to an advantage, though the secondary moments can also be a significant detriment to the design. Careful consideration of the tendon profile is, therefore, necessary.

Normally, tendons in three- to five-span continuous units will be stressed from both ends, to compensate for friction losses.

There are practical limitations on the length of continuous post-tensioned structures. If the post-tensioning strands run the full length of the continuous unit, the effective prestress at the center of the unit will be considerably less than at the ends due to friction losses. Satisfaction of allowable stresses may be difficult, requiring deeper sections or a greater number of strands.

An additional concern is related to the constructibility of a long continuous reach of strand. Tendons can be pushed through ducts provided the length of the duct is not too great and the duct is sufficiently large with no obstructions or discontinuities. This is generally done for tendon lengths of 650 to 750 ft (198 to 229 m). The logistics of the operation are manageable as well. Strand is shot through the duct at relatively high speed, as it is unwound from a spool at one end. One-by-one strand is sent through the tendon duct.

For longer spans, e.g., the St. George Island Bridge, it is questionable whether strand can be successfully pushed through the tendon ducts. The St. George Island Main Channel Unit will be 1180 ft (360 m) in length. The length of the tendon makes pushing strand questionable. Instead, for girder lengths in excess of 800 ft (244 m), strand should be pulled.

Pulling strand is logistically more complicated than pushing strand. It requires a laydown area behind the ten-

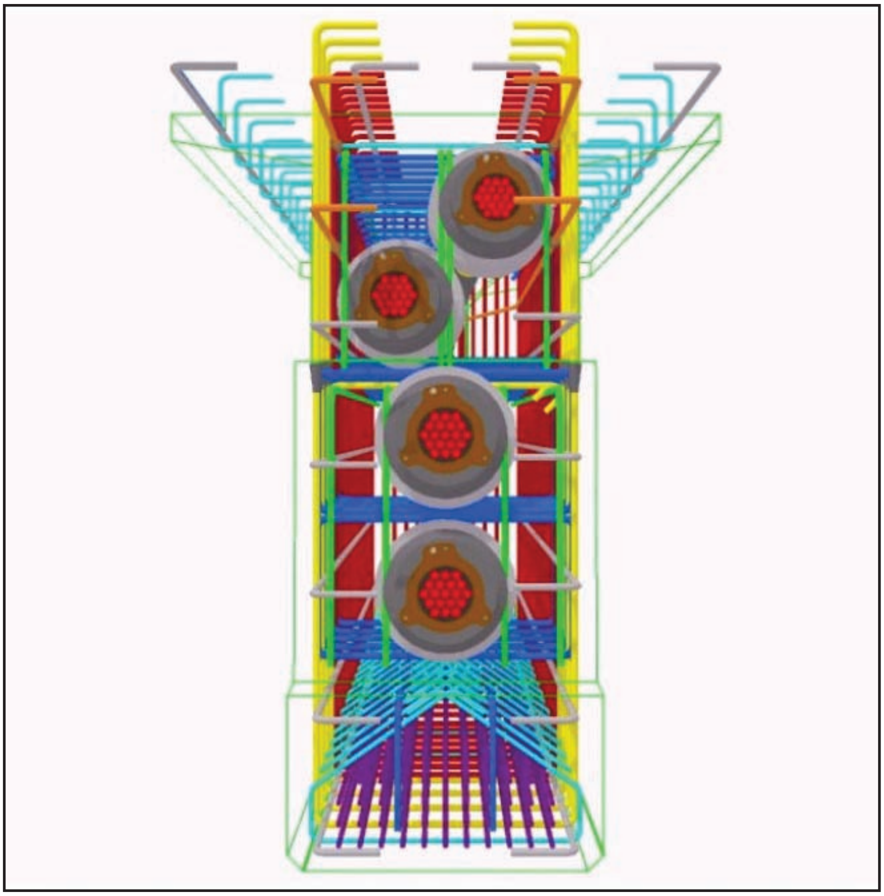

Fig. 11a. End view of end block anchor zone reinforcement detail.

don duct so that strand can be bundled. When tendons are pulled, all strand is pulled together. The laydown area will necessarily be 1180 ft (360 m) long on St. George. The laydown area is the deck behind the channel unit, on the approach spans. It must be protected from spalling due to pulling of the strand across the deck, and the strand must be similarly protected. Consequently, it is a more costly operation than pushing strand.

At some point, the length of the structure will make a long continuous run of post-tensioning strand impractical. To overcome the need to push or pull strand long distances for continuous tendons, and to compensate for the loss of prestress due to friction losses, it appears feasible to propose overlapping tendons, with tendon anchorages at blisters located left and right of closure pours.

Tendons for the negative moment regions over supports can be run through the haunched segments near the top flange of the girder, and anchored in end blocks cast in the adjacent drop-in segments.

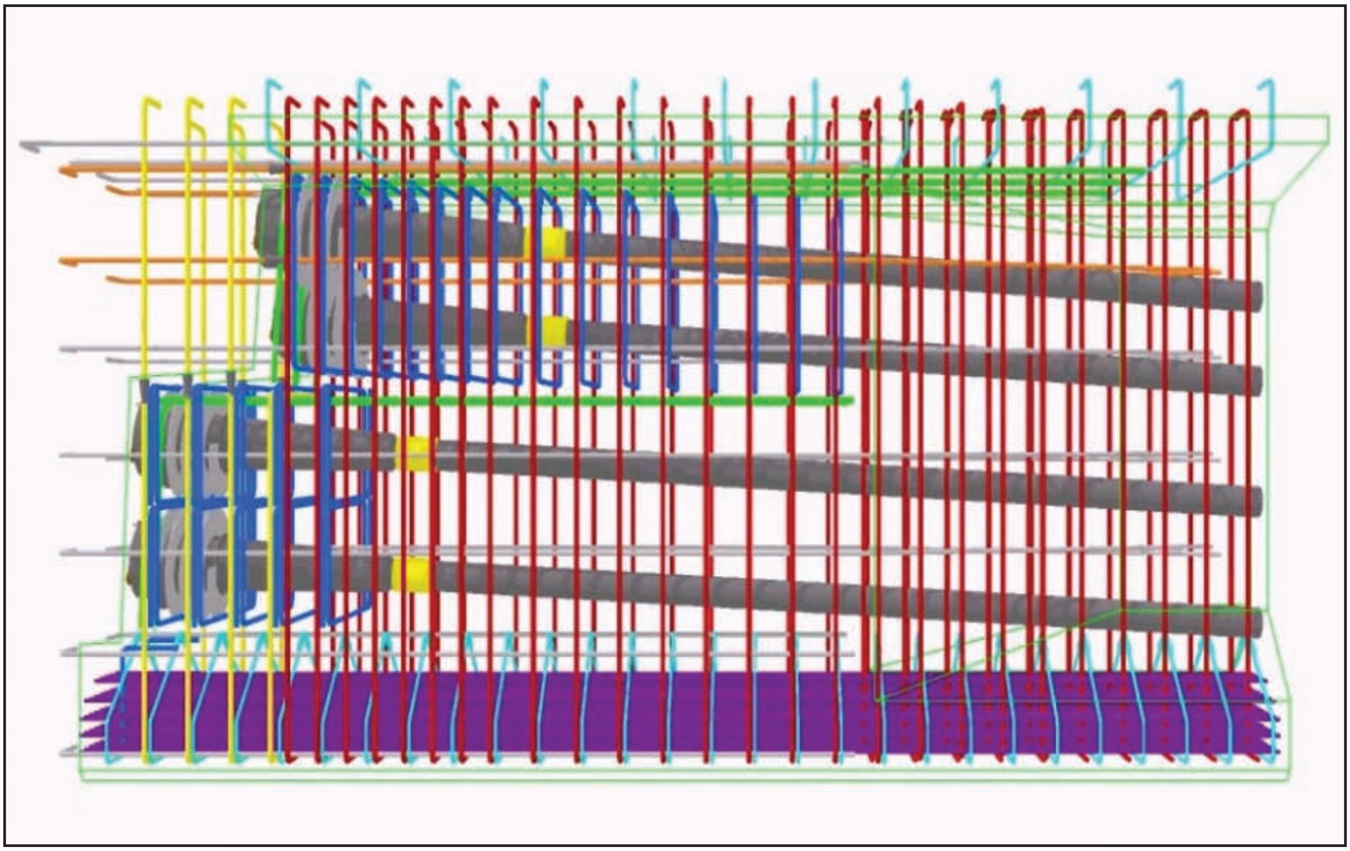

Fig. 11b. Side view of end block anchor zone reinforcement detail.

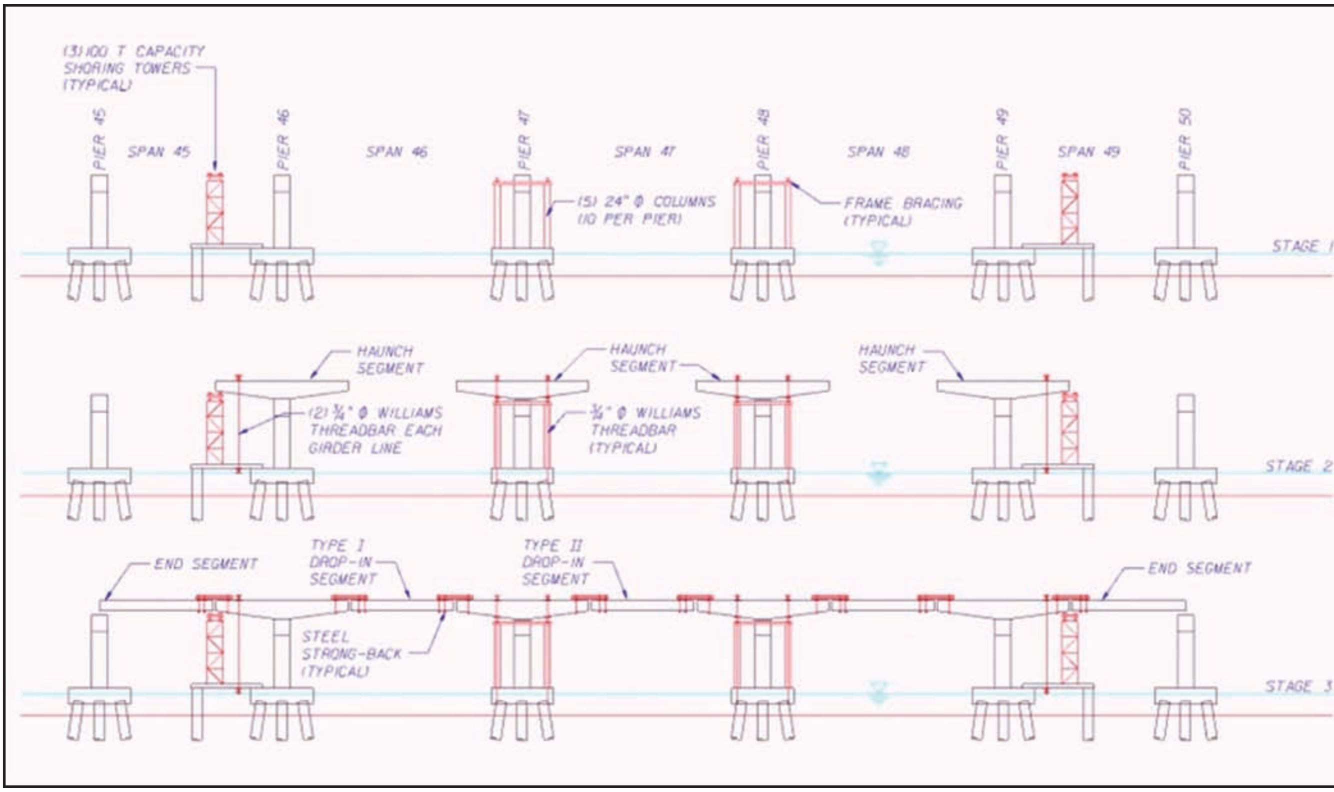

Fig. 12a. Erection sequence schematic (Stages 1 to 3).

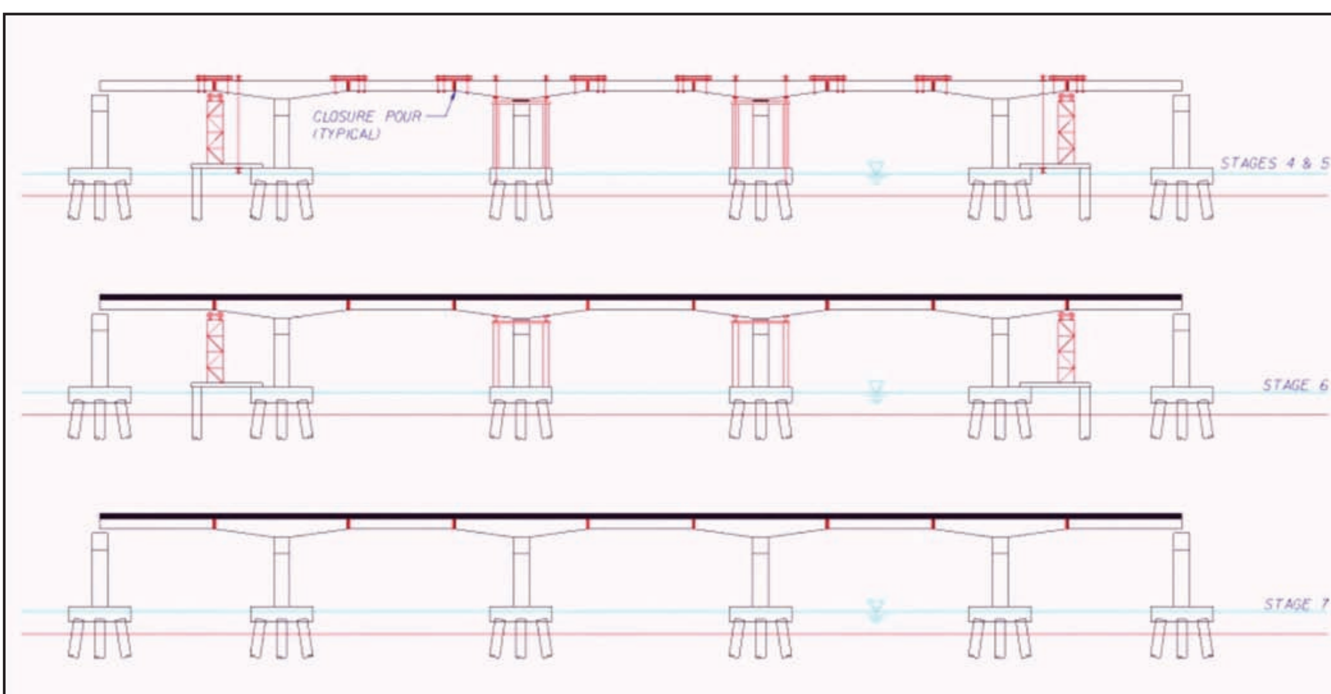

Fig. 12b. Erection sequence schematic (Stages 4 to 7).

Conversely, tendons for positive moment in the drop-in and end segments can be run through the bottom of drop-ins and anchored to end blocks cast in the adjacent haunched segments. In this manner, a continuous structure of indefinite length may be constructed in the future. A more economical variation of this theme would have a tendon continuous over two or three spans, then anchored in the adjacent drop-in unit. These variations of the spliced bulb-tee design appear to be untried. The method would most likely produce significant secondary moments. A future project may warrant giving this approach closer scrutiny.

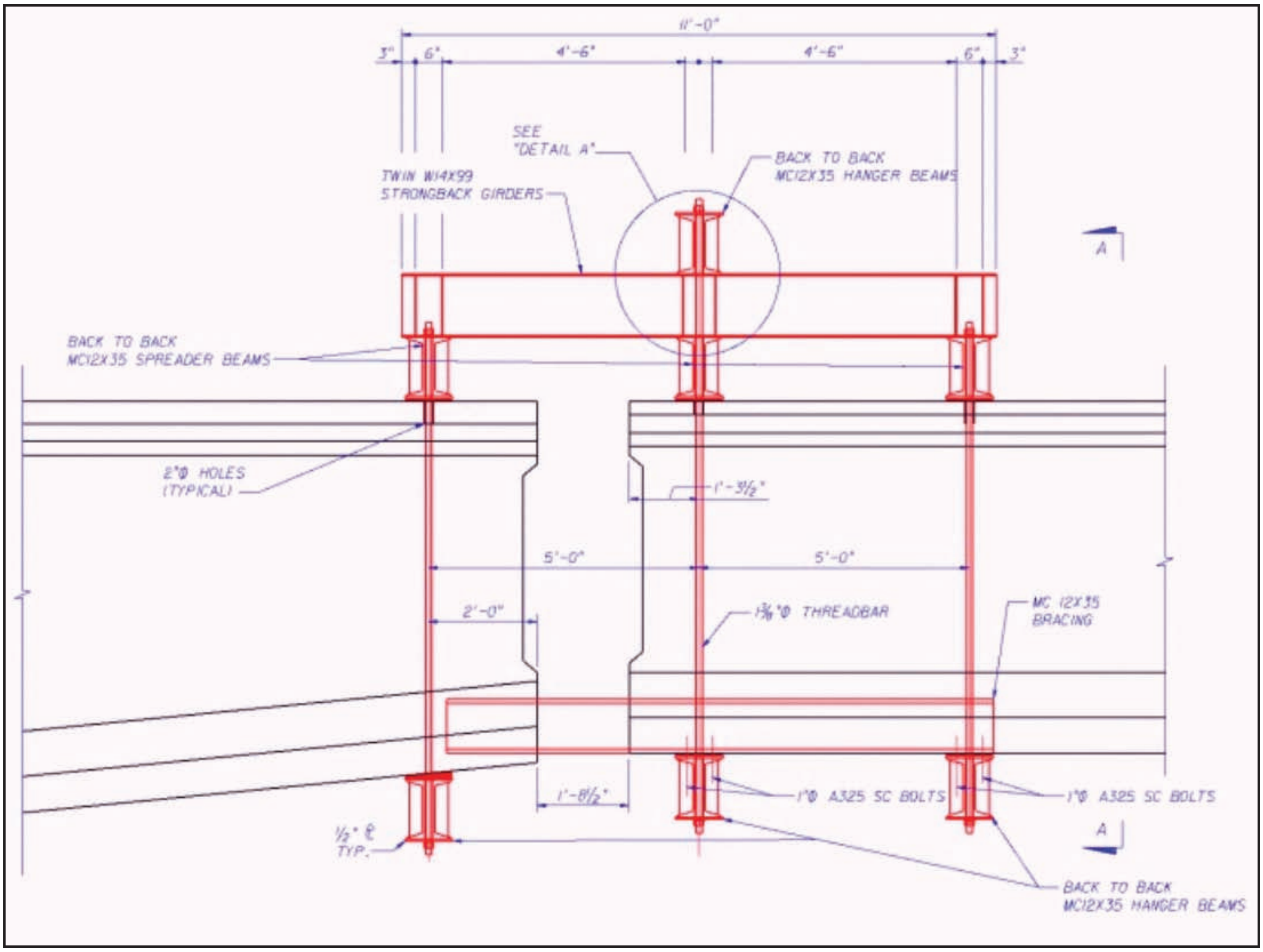

Fig. 13a. Elevation at closure showing temporary shoring strongback details. (For section, see Fig. 13b.)

End Block Design and Details

The design of end segment anchor zones can have a significant impact on the erection sequence used to construct the bridge. As indicated earlier, there are three generic end block details that have been used for the post-tensioned spliced bulb-tee system.

There are essential design aspects common to all end block regions. These include both local and general anchorage zones, with corresponding stress concentrations that must be considered. And in each case the design of the anchor zone must consider the effect that staged post-tensioning has on stress patterns.

AASHTO has incorporated extensive provisions for the design of anchor zones into the LRFD Code, and the provisions of Section 5.10.9 are applicable to anchor zone design. The equations of Section 5.10.9.6 are, in general, most appropriate to end block design for post-tensioned bulb-tee girders similar to the FBT-78. These are the equations the author has employed to size anchor zone bursting reinforcement and check compressive stresses.

Note that because tendons are stressed sequentially (i.e., Tendon 1, then Tendon 2, etc.), reinforcement must be appropriate for each stage of loading. For example, though the net jacking force to be considered when only one or two tendons are stressed is much lower than the total jacking force, the eccentricity of the tendons will lead to different bursting stresses, and equally important, to different centroids of the net bursting force.

The bursting stresses for one or two tendons at a large eccentricity from the centroid of the bulb tee may be higher than the bursting stresses when the last tendon is stressed. Since all tendons act together, they may not be effectively smoothed out over such a short distance. Edge tension forces must be examined sequentially as well.

In addition to compressive, bursting and edge tension stresses, there are shear forces and radial forces on deviated tendons. These must be calculated independently and superimposed. Radial forces on deviated tendons can be assessed using the method of equivalent external loads. The curvature of the tendon leads to radial forces on the concrete that can be determined from the prestressing profile.

For example, a circular profile leads to a uniform distributed load of P/R along the tendon, where P is the tendon prestress force, and R is the radius of curvature. Similar equations apply to parabolic and "kinked" tendons.

It should be cautioned that all of

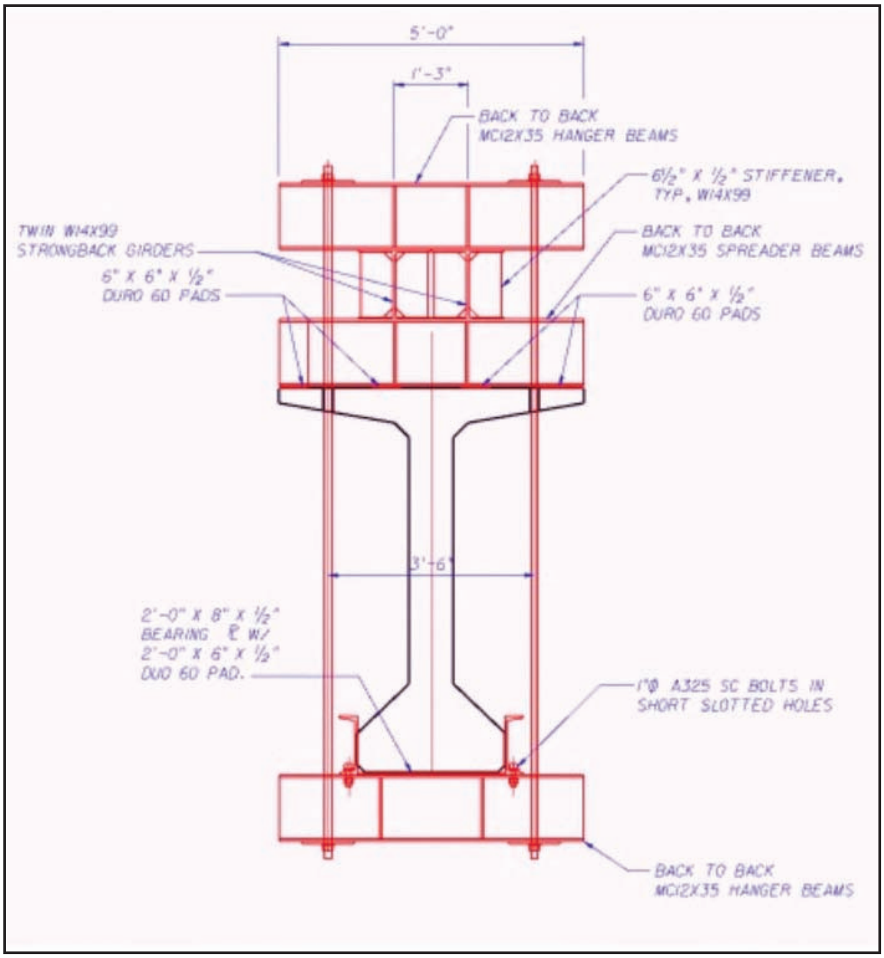

Fig. 13b. Cross section at closure pour. (For elevation, see Fig. 13a.)

these methods are approximate and that actual stress distributions within anchor zones are complex and nonlinear. Superposition of each action is a practical simplification. It is recommended that the engineer designing end block regions back up his or her design with a suitable strut-and-tie model, and in the case of very complicated or unusual end blocks, perform a finite element analysis to supplement any approximate equations used.

One further consideration should be noted for end block regions where all tendons terminate at the top of the girder. There will be stresses in the concrete due to the abrupt change in compression at a tendon anchor. The concrete immediately behind the tendon anchorage may not be compressed, while the concrete in front of the anchorage experiences a considerable compressive stress. The stress and strain gradients between these two regions may result in tensile cracking.

This type of anchorage merits special consideration and is a good candidate for supplemental strut-and-tie modeling or finite element analyses. The equations of AASHTO Section 9.10.9.6 should not be literally applied when the tendon anchorage terminates on the top of the girders. Fig. 11 provides three-dimensional isometrics of end block reinforcement details. These isometric views are sometimes required to verify that potential reinforcing bar conflicts have been adequately considered.

Tendon Grouting Considerations

Tendon grouting has recently received considerable attention in Florida. Grouting is the operation used to bond tendons to the precast members and provide corrosion protection to the prestressed strands. Typically, a neat cement grout has been used, sometimes with an aluminum powder additive to provide some expansive properties. (Note that aluminum powder is no longer an accepted additive in Florida.)

The grout consistency must be highly fluid to allow it to be pumped under pressure through long tendon ducts. Grouting ports and vents are fitted to tendons at or near high and low points along the tendon profile. Grout has traditionally been injected at tendon anchorages, proceeding along the length of the tendon. Grout vents are used to expel contaminants out of the tendons prior to grouting and to discharge air pockets and excess grout as the tendon fills.

On long tendons, grout may be injected at multiple high points along the length of the girder. Ports are closed after a sufficient discharge of grout establishes that air pockets have been eliminated. Unfortunately, there is widespread evidence that neat cement grout and the traditional grouting procedures used have been ineffective in properly sealing tendons. Bleed water has been shown to collect at high points, demonstrating segregation of the grout constituents.

In addition, despite venting grout at high and low points along the tendons, air pockets remain trapped in the viscous grout and are not expelled at ports as the tendon is filled. They eventually migrate to high points as the grout sets, leaving pockets of trapped air and bleed water that have sometimes resulted in corrosion of the strand.

To remedy the tendon-grouting problem, Florida has elected to require a premixed grout in lieu of site-mixed, neat cement grout. It is a proprietary product with numerous additives and superplasticizers. This grout, mixed according to the manufacturer's directions, has a very low water-cementitious materials ratio, yet remains highly fluid with moderate viscosity. Because it is pre-formulated, there is less likelihood that the grout will be of the wrong consistency.

The product is Master Builders GS-1205 Cable Grout. At the time of this writing, it is the only product of its kind and it comes in 95 lb (43.1 kg) bags rather than bulk quantities. Several competitors are producing their own tendon grouts and the material will soon be available in bulk quanti-

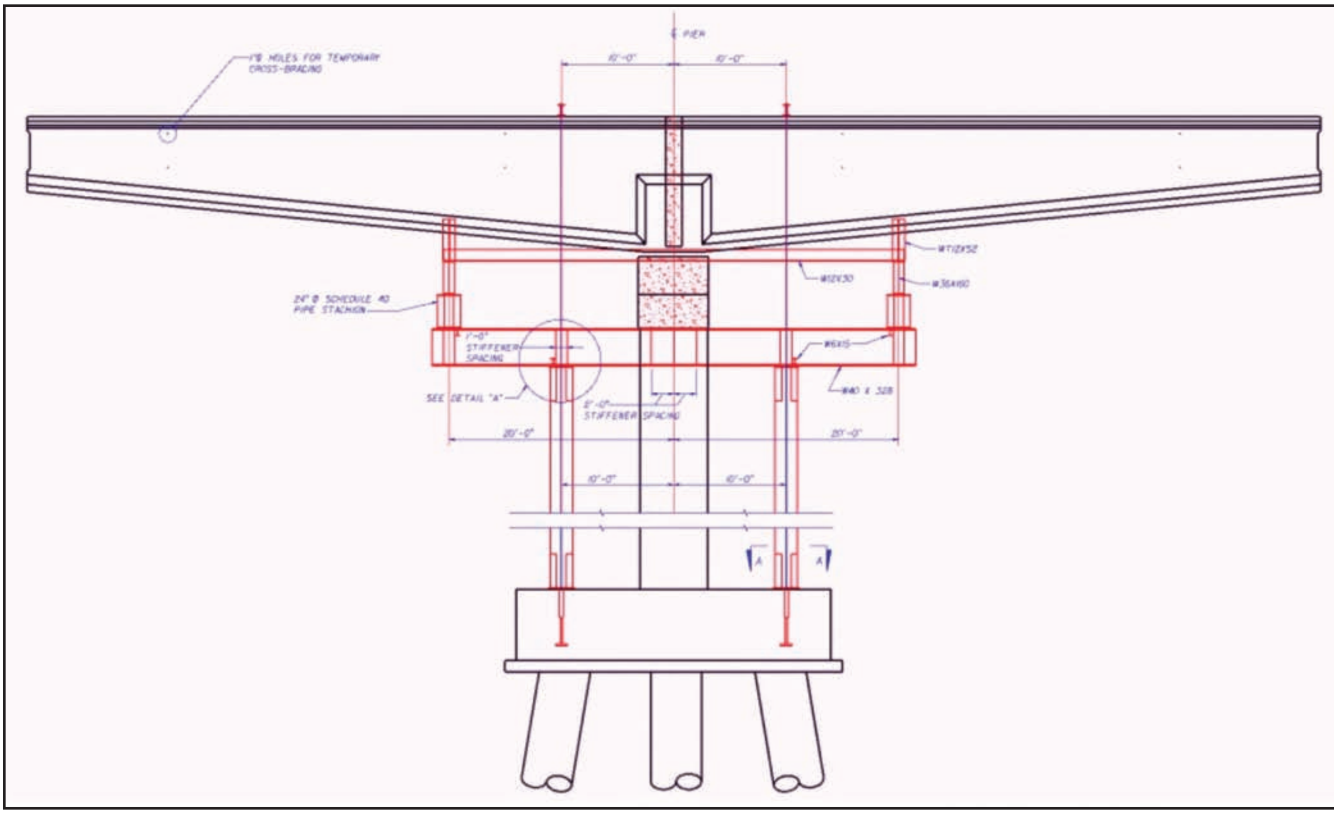

Fig. 14. Elevation of temporary shoring tower.

ties from several manufacturers. In the interim, it will be a fairly expensive product.

In addition to the use of a premixed specially formulated cable grout, Florida has drafted stringent criteria for tendon grouting operations. It will no longer be sufficient to grout from high points only. Grout must be injected from both low and high points and in both directions – left to right then right to left.

This procedure is intended to ensure that tendon ducts are completely filled and all air pockets are expelled during the grouting process. It is evident that tendon grouting will no longer be taken lightly and that quality control will be much tighter. Costs will reflect the stringent material and execution requirements, as well as higher quality control standards.

Bearing Design Considerations

Pot bearings and reinforced neoprene bearings have been used for continuous precast, post-tensioned structures. On both the Wonderwood and St. George Island Bridges, reinforced neoprene pads were chosen based on the simplicity of the design, reliability, and relatively low cost. The design of standard reinforced neoprene pads and reinforced pads with composite slide bearing assemblies is covered in detail in Section 14 of the LRFD Code. These provisions will not be reiterated here. But there is one design issue that merits discussion, since it is a function of the structural system used, and not the pad design.

Reinforced neoprene pads used on long continuous post-tensioned structures will be subjected to considerable translation due to elastic shortening during the construction of the unit, thermal movement, and later due to time-dependent creep and shrinkage of the girders. All of these movements are additive. It may not be practical to provide a pad that is sized to accommodate the entire projected pad displacement.

At the ends of the unit, slide bearings must be used because of the magnitude of elastic and long-term shortening. But use of slide bearings for the interior piers of the unit will not provide the stability required of the system to resist nominal wind and/or minor seismic events. It is best to provide reinforced pads with shear strength sufficient to absorb these loads. However, in so doing, the pad thickness may become inordinately large, based on anticipated longitudinal movement.

On the St. George Island Bridge, the interior haunched segments bear on neoprene pads. After first-stage post-tensioning, the bearing pads will have accumulated $2^1/_2$ to 3 in. (64 to 76 mm) of longitudinal deformation. Additional longitudinal displacement due to elastic shortening, thermal contraction, creep, and shrinkage will lead to excessive deformation of the pad. Therefore, stress relief of the pads is specified.

Haunched segments will be lifted off their pads to relieve the longitudinal deformation. Subsequent creep and shrinkage deformation will then be well within the pad design parameters. Lifting the haunched segments is accommodated using the temporary shoring jacks already in place to facilitate erection of the segments.

Temporary shoring, though generally not designed by the engineer of record, has to be considered by the engineer. The sequence of erection plans

Fig. 15. Column pier with cantilevered haunched pier beam segments (Fuller Warren Bridge, Florida).

must accompany the design documents, since the structural system is incrementally constructed, and the erection sequence and shoring details dictate the stresses that are ultimately locked into the structure. With adequate forethought, shoring towers can be designed to provide platforms for installation and dismantlement of diaphragm formwork and cross bracing, or stress relief in bearings.

Sequence of Erection Drawings

The stress in a segmental structure must be calculated for each stage of erection and accumulated for final determination of the service life stresses. The sequence of construction is, therefore, an integral part of the design and construction, and must be included in the contract documents. For a post-tensioned bulb-tee system, the sequence of erection must include pier construction, temporary shoring tower erection and placement of haunched, end, and drop-in segments.

A suggested sequence of erection for a five-span structure is shown in Fig. 12. This erection sequence was provided in contract documents for the St. George Island Project. The temporary shoring details have a significant effect on the construction stresses in the structure. Strongback details, tower details, segment weights and construction loads must be developed in order to tabulate erection loads for the erection sequence plans.

Temporary Shoring

The primary components of the temporary shoring for a spliced bulb-tee system are the strongbacks that suspend end and drop-in segments off of haunched segments, and towers used to support and stabilize the haunched segments. Strongbacks are fixed to end and drop-in segments, then eased onto haunched segments and secured to the haunched segments with high-strength threadbar. Along with strongbacks, web clamps are secured across closure pours to prevent girder roll.

All girder segments must be detailed to accommodate strongbacks used during erection of the girders. Strongback tie-downs have typically been designed to extend from holes cast into the top flange of bulb tees. In addition, the engineer should consider strongback details to accommodate shear stirrups that protrude through the top flanges of girders.

Several incidents have occurred because the design and installation of strongbacks were not carefully considered. On at least one project, the strongback bearing pressure on the bulb-tee flange caused the top flanges to crack. Bearing between strongbacks and girders should be concentrated on the area over girder webs.

On another project, web clamps had not been fastened tightly to the haunched segments and the drop-in segments slowly but progressively twisted until the strongbacks failed and the girders fell. Should there be any sweep in the girders, torsion may cause the girders to roll. The design of strongbacks cannot be neglected. Typical strongback details are shown in Fig. 13.

There are several variations to the design of shoring towers. For a three-span unit, two towers are generally constructed, one under each closure pour in the exterior spans. After haunched segments are set on CIP piers, the outboard end of the segments are tied down to the shoring towers. End segments are then erected and eased onto haunched segments using strongback attachments, or set directly onto the towers adjacent to the haunches.

There is an advantage to setting end segments directly onto the haunches. They provide resistance to uplift which otherwise must be provided by tie-downs. Uplift occurs when the drop-in segments are suspended from the opposite end of the haunched segments. The loading of each stage is considered in the design of shoring towers and must be noted on the erection sequence drawings.

If the CIP piers supporting the haunched segments are founded on pile caps of sufficient size, it is feasible to support temporary towers on the pile caps. There is a definite advantage to using the pile cap for the base of the temporary towers. Towers located under closure pours generally require temporary pile foundations.

The towers and foundations cannot be removed until construction of the superstructure is complete, and once the superstructure is in place over the temporary piling, it is difficult to salvage the shoring. On two recent bulb-tee projects, the pile caps have been used to support shoring towers. The St. George Island Bridge is one of these projects. The typical shoring tower design for St. George Island is shown in Fig. 14.

Various sequences of the erection of a bridge superstructure are shown in Figs. 15 through 20.

Temporary shoring is critical for the safe erection of girders and can be a costly bid item. Therefore, it warrants careful consideration and forethought.

CONCLUDING REMARKS

Continuous composite post-tensioned bulb-tee girder systems have proven to provide a very adaptable structural system for intermediate and long-span bridge construction. Future improvements to the system promise even greater adaptability, and the possibility that longer spans may be bridged. The discussion of design and construction considerations in this paper provides some background on the problems that have been encountered and resolved, and highlights the issues that the bridge engineer must address during design development.

Though the analysis and design of incrementally constructed bulb-tee girders is considerably more involved than for typical prestressed girder systems, construction is a great deal simpler than for incrementally constructed span-by-span segmental and balanced cantilever systems. Formwork, shoring, and post-tensioning requirements make the erection of the bulb-tee system a much easier and more

Fig. 16. Drop-in segment fabrication at the Moore Haven Bridge, Okeechobee Waterway, Florida.

Fig. 17. Drop-in girder with strongbacks attached, Moore Haven Bridge, Okeechobee Waterway, Florida.

Fig. 18. Drop-in segment erection, Moore Haven Bridge, Okeechobee Waterway, Florida.

cost-effective structural system than span-by-span and balanced cantilever construction.

The AASHTO Code does not refer to this type of construction directly, but adherence to the allowable stress provisions for prestressed concrete members is the current standard for design. The author hopes that future code revisions will address this type of system directly. Guidelines for construction loads, creep and shrinkage parameters (in the absence of laboratory testing), and minimum reinforcement requirements or allowable stresses in the deck should be discussed. The construction schedule assumed in design should also be required on the Sequence of Erection Drawings.

A final note regarding post-tensioned girder systems: there are a number of commercially available programs for analysis of incrementally constructed structures. These include ADAPT, BD-II, TANGO, and others. Each has advantages and disadvantages. It is recommended that the engineer become very familiar with the program used for time-dependent

Fig. 19. Strongback installation at closure pour, Moore Haven Bridge, Okeechobee Waterway, Florida.

Fig. 20. Moore Haven Bridge nearing completion, Okeechobee Waterway, Florida.

analysis of composite structures. Simple structural systems should be analyzed first and compared with known results. It is difficult to gauge the accuracy and reliability of the results of a complex analysis if the first problem solved is the design of a two-staged post-tensioned composite system.

ACKNOWLEDGMENT

The author wishes to express his appreciation to Henry Bollmann of the Florida Department of Transportation for his review of this paper. Mr. Bollmann provided valuable guidance and perspective on the content and clarity of the work. However, the author accepts responsibility for any inaccuracies that may be found in the article.

REFERENCES

1. Janssen, Hubert H., and Spaans, Leo, "Bids Reveal Economy of Precast Post-Tensioned Girders on Bridge Project," PCI JOURNAL, V. 37, No. 1, January-February 1992, pp. 86-87.
2. Ficenec, Joseph A., Kneip, Steven D., and Tadros, Maher K., "A New Continuous Precast Concrete Bridge System," IBC Paper Number IBC-93-26, International Building Code, Falls Church, VA, June 1993.
3. Alami, Bijan O., "Design Fundamentals of Segmentally Constructed Bridges with Particular Reference to AASHTO LRFD Method," American Association of State Highway Officials, Washington, DC, June 1997.
4. Abdel-Karim, Ahmad M., and Tadros, Maher K., "State of the Art of Precast/Prestressed Concrete Spliced I-Girder Bridges," for the PCI Committee on Bridges, Precast/Prestressed Concrete Institute, Chicago, IL, Second Printing, 1995, 134 pp.
5. Caroland, William B., Depp, David, Janssen, Hubert H., and Spaans, Leo, "Spliced Segmental Prestressed Concrete I-Beams for Shelby Creek Bridge," PCI JOURNAL, V. 37, No. 5, September-October 1992, pp. 22-33.