A Crack Is Not a Crack: End Region Cracking in Prestressed Concrete Components

by Dr. Oguzhan Bayrak, University of Texas at Austin

This article, which is the fourth in this series, addresses the cracking that may occur in the end regions of prestressed concrete components. Within this context, we will address pretensioned and post-tensioned components separately.

Pretensioned Concrete Components

When I think about the end regions of pretensioned concrete girders, I am reminded of the historical evolution of standard girder types in the United States. Since the inception of pretensioning technology, the criticality of the complex stress state that occurs within the end regions of pretensioned concrete girders was understood. At early stages, end blocks and special reinforcement details were employed to ensure proper transfer of the pretensioning force to surrounding concrete and to interior portions of the beam, away from the end regions. When sufficient experience and knowledge were gained, the use of end blocks was eliminated as owners began taking advantage of sections such as AASHTO standard girder shapes. Meanwhile, through advances in materials science, the tensile strength of pretensioning strand was increased from 250 to 270 ksi. The stress-relieving process used in the early days of strand technology gave way to a better method (strain tempering) intended to reduce relaxation-related losses. With the advance of strain tempering, low-relaxation strands were introduced. More recently, ½-in.-diameter strands, which were once used routinely in highway construction, have been largely replaced by 0.6-in.-diameter strands. Current research continues to push those boundaries by exploring the use of 300 ksi material, as well as 0.7-in.-diameter strands.[1] With a typical 2-in. spacing of strands, these changes amount to a greater prestressing force within the bottom flanges of the girders, and where harped strands are used, in the web as well. It is worth noting that with the introduction of 0.6.-in.-diameter strands, a similar increase in the prestressing force was introduced to the typical 2-in. grid by increasing the force per strand compared with 0.5-in.-diameter strands.

Parallel to developments in prestressing reinforcement technology, concrete materials science also advanced. Chemical admixtures, supplementary cementitious materials, and microfine aggregates all helped improve particle packing and reduced water use in modern concrete mixtures. As a result, high-performance, high-strength concretes became widely available. With higher-performing materials, the quest to optimize sections for better structural performance led to the variety of different bulb tees that we see in the industry today. Such sections commonly have large bottom flanges to accommodate the large number of strands. These advances also helped concentrate additional prestressing forces in bottom flanges, allowing for longer span capabilities for a given beam section with a given depth.

An unintended consequence of increasing the pretensioning force within the end region of pretensioned girders and optimizing the concrete section is the end-region cracking that we sometimes see in modern pretensioned girders. **Figure 1** shows one such example. The *CEB-FIP Model Code 1990*[2] identifies various types of cracks that may form in the end regions of pretensioned girders (**Fig. 2**). As we can see in Fig. 2, the complex state of stress within approximately one component depth *h* from the end of a pretensioned girder causes multiple effects. Let us examine spalling stresses first. As part of a thought exercise, let's make a horizontal cut between the top flange of a pretensioned girder and the web. Let us also imagine that our beam is made from an ordinary kitchen sponge so that we can envision exaggerated displacements. When we apply a compressive force at the bottom flange (that is, squeeze the bottom flange inward), the girder will camber upward. As the girder cambers upward, the top flange will separate and move away from the web at the location where we introduced the horizontal cut. Of course, in an actual beam, there is no cut; therefore, we must restore the integrity of the beam such that deformations of the top flange can be made compatible with the rest of the beam. In an actual concrete beam (Fig. 1), cracking occurs and engages the vertical reinforcement. Such cracks are called "spalling cracks" or "compatibility cracking" in the terminology used in the Model Code 1990. The cracks near the top flange of the beam and upper portions of the web in Fig. 1 are examples of spalling cracks.

Bursting cracks occur as the concentrated force transferred into the concrete section starts to spread in the transverse direction. Let us recall that the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*[3] specifies that transfer length is 60

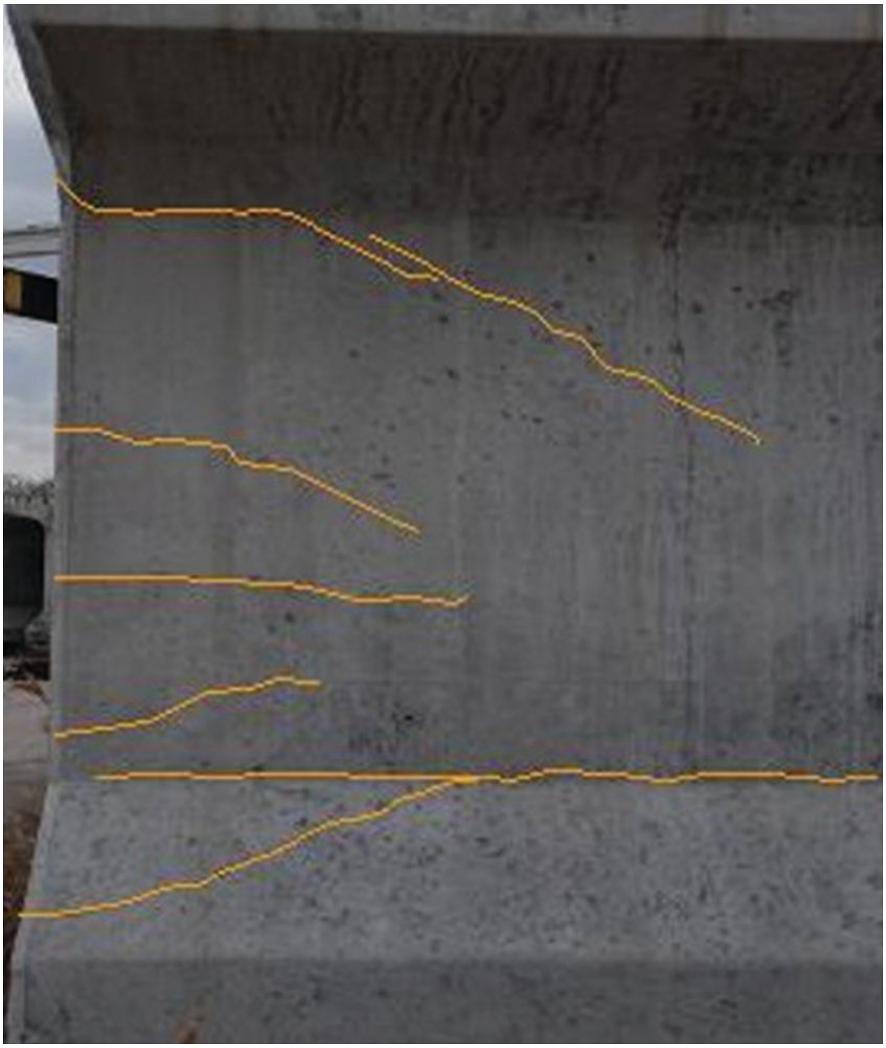


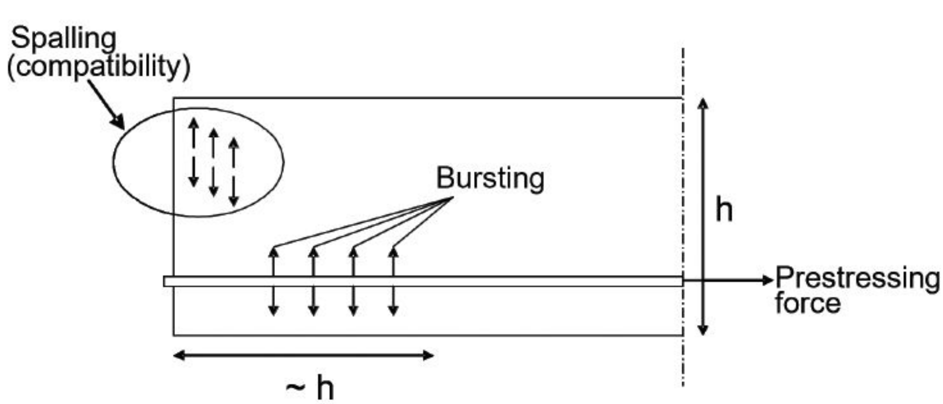
Figure 1. End-region cracking (enhanced) in a pretensioned concrete I-girder. Photo: Matt O'Callaghan and FSEL Researchers.[4]

times the strand diameter. For 0.6-in.-diameter strands, this distance is 36 in. At a section approximately 2.5 to 4 ft from the end of a beam, we would see the highest transverse forces that can be attributed to bursting effects. These cracks typically form within the bottom flange and in close proximity to the bottom flange–to–web interface. Both the vertical reinforcement that crosses the bottom flange–to–web interface and the confinement reinforcement we provide within the bottom flange of pretensioned girders help us address the demand imposed by large prestressing forces in modern bulb tees.

The confinement reinforcement provided within the bottom flange also helps in tackling the Hoyer effect–related transverse stresses within the very end of a beam, over the transfer length. The Hoyer effect can be visualized as the tendency of the wires that form a seven-wire strand to expand back to their original (unstressed state) diameter and their original orientation as the strands are wedged in the surrounding concrete at prestress transfer. This effect is the primary mechanism by which the force transfer occurs, in addition to the mechanical interlock that stems from the helical nature of the circumferential wires that surround the king wire in a seven-wire strand.

Of all the cracks shown in Fig. 1, the horizontal crack at the bottom flange–to–web interface has been a primary cause for concern for my research team at the University of Texas at Austin. More specifically, we have focused our attention on nontraditional shear failure modes in a number of studies, starting with an experimental study conducted at the Phil M. Ferguson Structural Engineering Laboratory, *Tensile Stresses in the End Regions of Pretensioned I-Beams at Release*.[4] We have researched the additive nature of stresses imposed by reinforcing bars that cross the end-region cracks and those imposed by the shear stresses caused by the dead and live loads as they are transferred to the supporting bents. With all strength and serviceability considerations, we concluded that bundling some bars with traditional shear reinforcement in the end regions of the bulb tees would be a good detailing practice to (a) materialize the shear strength calculated by the shear design provisions based on modified compression field theory in the AASHTO LRFD specifications, and (b) keep the end-region crack widths to a minimum to ensure long-term durability. Today, several states employ similar details where several sets of stirrups are bundled with additional or special reinforcement in the end regions, with the ultimate goal of reducing transverse stresses in those bars and limiting the widths of the cracks that form in the end regions at prestress transfer.

Figure 2. Schematic of spalling and bursting stresses in the end region of a pretensioned concrete component.[2,4]



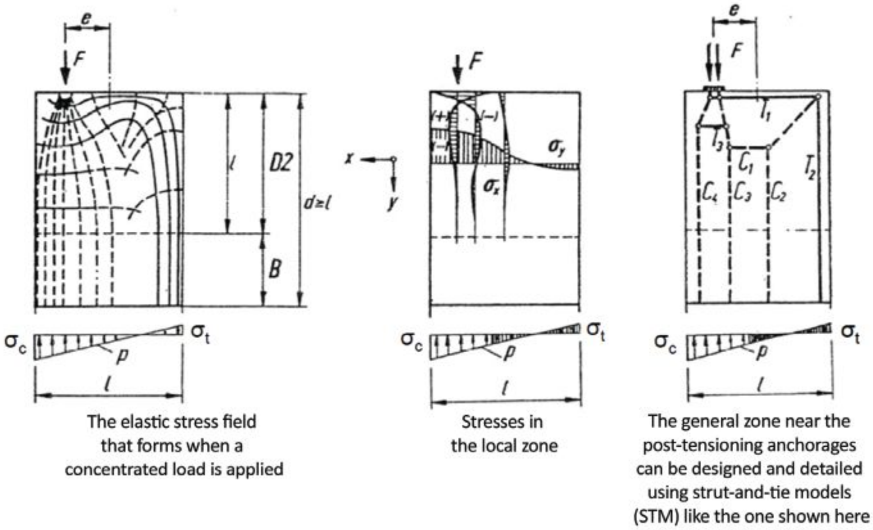


Figure 3. The stress immediately adjacent to a concentrated load on a member are complex. Source: Figure C5.8.2.7[1-3] from the *CEB-FIP Model Code 1990*.[2,5]

What Do the AASHTO LRFD Specifications Say?

Article 5.9.4.4.1 of AASHTO LRFD specifications is devoted to splitting resistance in pretensioned concrete components. That section states the following:

> *The factored splitting resistance of pretensioned anchorage zones provided by reinforcement in the ends of pretensioned beams shall be taken as:*
>
> $$P_r = f_s A_s \qquad (5.9.4.4.1\text{-}1)$$
>
> *where:*
>
> f_s = *stress in steel not to exceed 20.0 ksi*
>
> A_s = *total area of reinforcement located within the distance* h*/4 from the end of the beam (in.*^{2}*)*
>
> h = *overall dimension of precast member in the direction in which splitting resistance is being evaluated (in.)*
>
> *For pretensioned I-girders or bulb tees,* A_s *shall be taken as the total area of the vertical reinforcement located within a distance of* h*/4 from the end of the member, where* h *is the overall height of the member (in.).*
>
> *For pretensioned solid or voided slabs,* A_s *shall be taken as the total area of the horizontal reinforcement located within a distance of* h*/4 from the end of the member, where* h *is the overall width of the member (in).*
>
> *For pretensioned box or tub girders,* A_s *shall be taken as the total area of vertical reinforcement or horizontal reinforcement located within a distance* h*/4 from the end of the member, where* h *is the lesser of the overall width or height of the member (in.).*
>
> *For pretensioned members with multiple stems,* A_s *shall be taken as the total area of vertical reinforcement, divided evenly among the webs, and located within a distance* h*/4 from the end of each web.*
>
> *The resistance shall not be less than four percent of the total prestressing force at transfer.*
>
> *The reinforcement shall be as close to the end of the beam as practicable.*
>
> *Reinforcement used to satisfy this requirement can also be used to satisfy other design requirements.*

In accordance with Article 5.9.4.4.1, we must pay attention to providing special end-region reinforcement through the web, as close to the beam end as possible. We must appreciate the fact that the 20-ksi limit on the reinforcement stress is placed as a serviceability limit to control the widths of the cracks that may form within the end regions. In a beam that does not have sufficient reinforcement to meet the required quantity of reinforcement specified in Article 5.9.4.4.1, we can expect to see end-region cracks that are wider than those implied with a 20-ksi stress limit. Put simply, if we provide one-half of the reinforcement required by Eq. (5.9.4.4.1-1), we can expect to see end-region cracks that are approximately twice as wide. While this ratio will not directly translate to structural capacity lost due to wider cracks, it may have adverse implications for the durability of the pretensioned girders, depending on the environmental exposure conditions.

In addition to the previously discussed requirements, Article 5.9.4.4.2 of the AASHTO LRFD specifications covers confinement reinforcement design considerations that reflect decades' worth of experience gained through construction and structural testing of pretensioned girders. The confinement reinforcement requirements of AASHTO LRFD specifications are as follows:

> *For the distance of 1.5*d *from the end of the beams other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands.*
>
> *For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.*

Some states and designers find it practical to pair the confinement reinforcement with the shear reinforcement, ultimately resulting in a greater quantity of confinement reinforcement. Such decisions made by local jurisdictions reflect local experiences that complement AASHTO LRFD specifications and further enhance the performance of pretensioned girders.

Post-tensioned Concrete Components

Elements of the previous discussion also apply to post-tensioned concrete components, but there are distinctive aspects of behavior in post-tensioned components that warrant additional discussion. While post-tensioning technology provides opportunities that cannot be materialized by pretensioned components, the technology also provides challenges that are rooted in specific aspects of behavior that require separate treatment. With this in mind, let us examine a portion of a 1987 paper by Schlaich et al.[5] **Figure 3** illustrates the elastic stress field that forms when a concentrated load is applied (F in Fig. 3). The dashed lines illustrate the compression trajectories, and the solid lines signify the tension fields that develop under the load application. The stress trajectories in the left side of the figure indicate the complex nature of the stresses immediately adjacent to the bearing device. This region, also known as the local zone, is best studied through testing conducted by, or on behalf of, the post-tensioning system suppliers. Design provisions in the AASHTO LRFD specifications do not provide explicit guidance for detailing the local zone. That said, this region is typically reinforced with spiral reinforcement. Confinement offered by the spiral reinforcement facilitates the transfer of high levels of compressive stresses into the post-tensioned concrete component. Engaging transversely positioned confining reinforcement by applying longitudinal compressive stresses in this area may result in cracking within the immediate vicinity of the anchorage device. The general zone near the post-tensioning anchorages can be designed and detailed using strut-and-tie models (STM) like the one shown on the right side of Fig. 3. As can be seen in that STM, the post-tensioning force spreads, moving away from the anchorage point, creating a transverse tension field (T_3 in Fig. 3). This bursting effect is similar to the one discussed earlier for the pretensioned sections. Edge tension force T_1 and longitudinal tension force T_2 are additional tensile forces that must be resisted by the reinforcement provided within these regions. To resist T_1, T_2, and T_3 shown in Fig. 3, typical unfactored post-tensioning forces may cause stress or strain levels in ordinary reinforcement that will be in the linear elastic range of the material response but large enough to exceed typical concrete cracking strains. In other words, if the tie reinforcement is to meaningfully engage and reinforce the general zone, we expect to see some cracking. While these cracks are expected due to bursting, spalling (edge tension), or longitudinal tension effects, their widths and propensity can be controlled by using a sufficient quantity of well-detailed reinforcement in compliance with the AASHTO LRFD specifications.

The actual structural details used and the geometry of concrete within and ahead of an anchorage zone play important roles in determining the bursting as well as edge tension effects, especially when there are discontinuities that disturb the flow of forces. In the AASHTO LRFD specifications, Commentary C5.8.4.5.1 includes a discussion that clearly highlights this fact, as follows:

> *Discontinuities, such as web openings, disturb the flow of forces and may cause higher compressive stresses, bursting forces, or edge tension forces in the anchorage zone. Figure C5.8.4.5.1-1* [reproduced herein as **Fig. 4**] *compares the bursting forces for a member with a continuous rectangular cross-section and for a member with a noncontinuous rectangular cross-section. The approximate equations may be applied to standard I-girders with end blocks if the longitudinal extension of the end block is at least one girder height and if the transition from the end block to the I-section is gradual.*

Figure 4 shows that the presence of an opening can increase the bursting effects by as much as a factor of 2 or more compared with a section without an opening. While this comparison may be viewed as an extreme case, we must recall the fact that the actual geometry of a concrete component must be properly considered in our STM selection to appropriately detail the general zones.

Conclusion

In a Professor's Perspective published in the Spring 2015 issue of *ASPIRE*®, I focused on Fritz Leonhardt's classic textbook, *Prestressed Concrete Design and Construction*.[6] Within the context of that article, I addressed the "ten commandments for the prestressed concrete engineer" by borrowing terminology from Professor Leonhardt. I would like to finish this article with his fifth recommendation: "Provide ordinary reinforcing bars transverse to the direction of prestressing force within the transfer length." After all these years, much research, and many success stories, those first principles in concrete construction remain as important as ever.

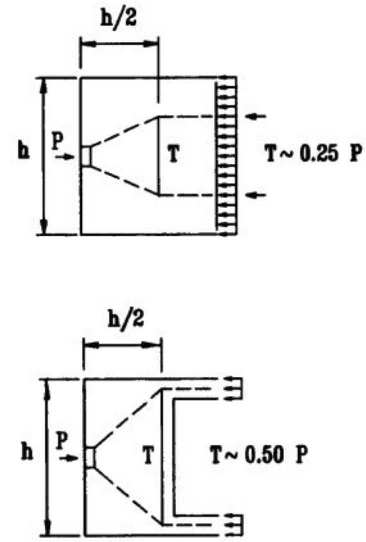


Figure 4. Effect of an opening in the anchorage zone. Source: Figure C5.8.4.5.1-1 from the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[3]

References

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