Appendix A–Literature Review

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1 Introduction

This appendix contains a review of the literature and other background information germane to the experimental and analytical research presented in subsequent appendices. Table 1 lists the sections and topics contained in this appendix and those appendices for which the topics specifically apply. To assist the reader, additional background information is also briefly discussed in the individual appendices.

Table 1–Literature review topics and relevant appendixes

Section	Topic	Relevant Appendices
A.2	Glossary	General to entire report
A.3	Confinement Reinforcement	Appendix B – Small Beam Test Program Appendix D – FIB-54 Test Program Appendix F – Finite Element Analyses Appendix G – End Region Design Models
A.4	Web Splitting	Appendix E – FIB-63 Test Program
A.5	Flange Effect on Shear Strength	Appendix C – SR-72 Test Program
A.6	Curb Contribution to Girder Behavior	Appendix C – SR-72 Test Program
A.7	Truss and Arch Action	Appendix C – SR-72 Test Program

2 Glossary

This section discusses some of the terms, phrases, and concepts germane to the study of pretensioned concrete I-girders.

Confinement reinforcement. Confinement reinforcement is mild steel reinforcement placed around prestressing strands in the bottom flange of precast pretensioned concrete girders (Figure 1).

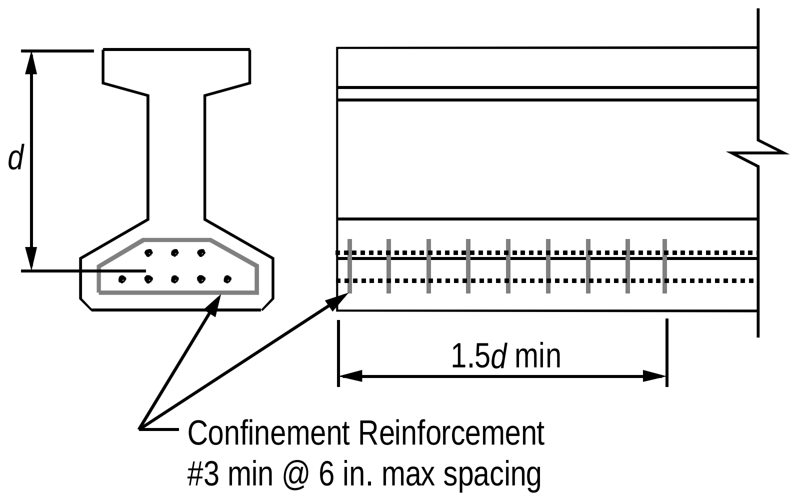


Figure 1–Confinement reinforcement

End region. The end region is loosely defined as the portion of a girder located within one and a half member depths from the girder end (Figure 2). The end region serves two critical functions: 1) Force transfer between the prestressing strands and concrete, and 2) Delivery of shear forces to the support. Mild reinforcement, including confinement reinforcement, is placed in the end region to aid in these functions.

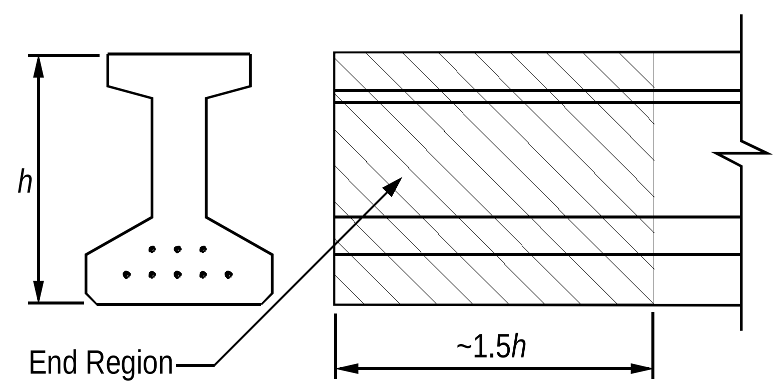


Figure 2–End region

Prestress transfer. In precast pretensioned concrete construction, concrete is cast around steel strands that have been preloaded in tension. After concrete is sufficiently cured, tension force in the strands is released, thereby transferring force into the surrounding concrete. This event is referred to as prestress transfer. Flame cutting of the strands is a common method of releasing the prestress force. The effects of prestress transfer are important considerations in the design of end region reinforcement.

Strut-and-tie behavior. After cracking, the behavior of concrete members can be modeled by a truss analogy. In this analogy, concrete struts carry compressive forces, and steel ties carry tensile forces (Figure 3). Behavior modeled by the truss analogy is referred to as strut-and-tie behavior. Strut-and-tie modeling is one approach for designing of end region reinforcement.

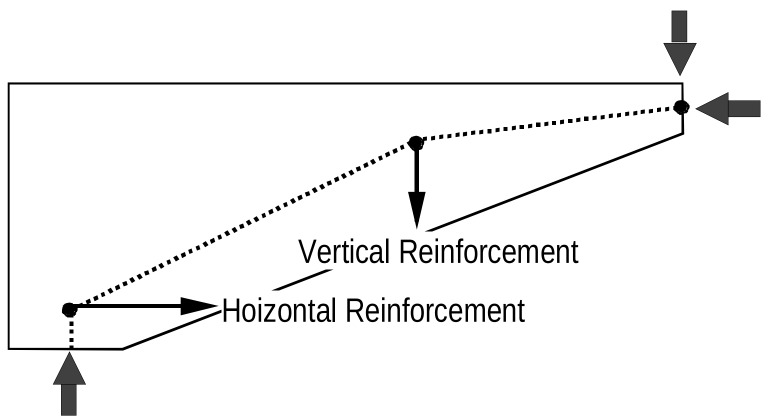


Figure 3–Strut-and-tie behavior

Shear span. Shear span is the horizontal distance from the support to the point of load application (Figure 4). Shear span-to-depth ratio (or a/d ratio) is often used to describe the loading condition of concrete members. Shear behavior is, among other factors, a function of the shear span-to-depth ratio. Strut-and-tie modeling is generally appropriate for members loaded with relatively small shear span-to-depth ratios.

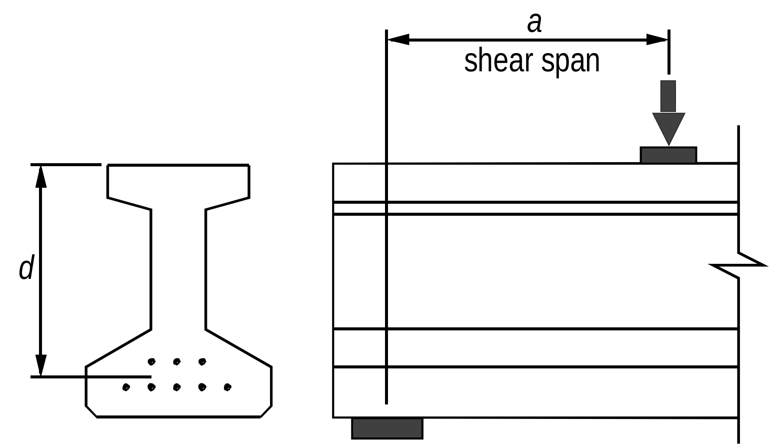
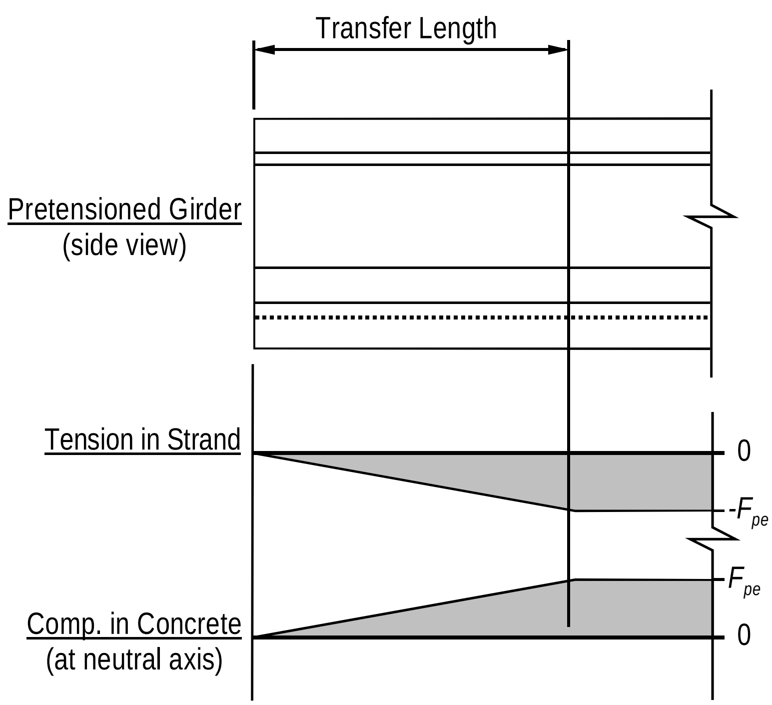


Figure 4–Shear span

Transfer length. Transfer length is the length over which prestressing force is transferred from prestressing strands into the surrounding concrete (Figure 5). Transfer length occurs within the end region. By design the prestress force in a strand is always less than ultimate capacity. Hence transfer length is not equivalent to the length required for full strand development.



Note: F_{pe} = Effective Prestress Force

Figure 5–Transfer length

Development length. Development length is the length of concrete embedment required to fully anchor prestressing strands. Strands with full development length can carry their ultimate tensile capacity (Figure 6). Development length is greater than transfer length.

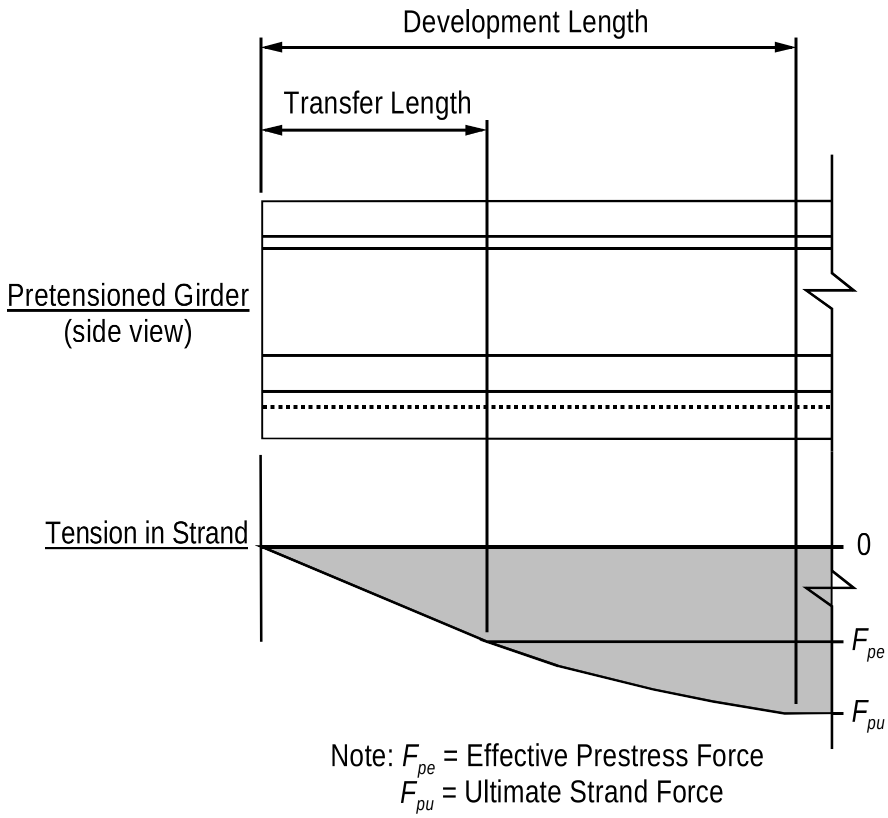


Figure 6–Development length

Strand debonding (shielding). Stress in pretensioned concrete girders can be controlled by selectively preventing force transfer between strands and concrete. To prevent force transfer, strands are debonded (also called shielding), at the ends of the beam. Debonding is accomplished by placing a sleeve over the strand to prevent bond with the concrete. Debonding moves the transfer length of shielded strands away from the girder end (Figure 7).

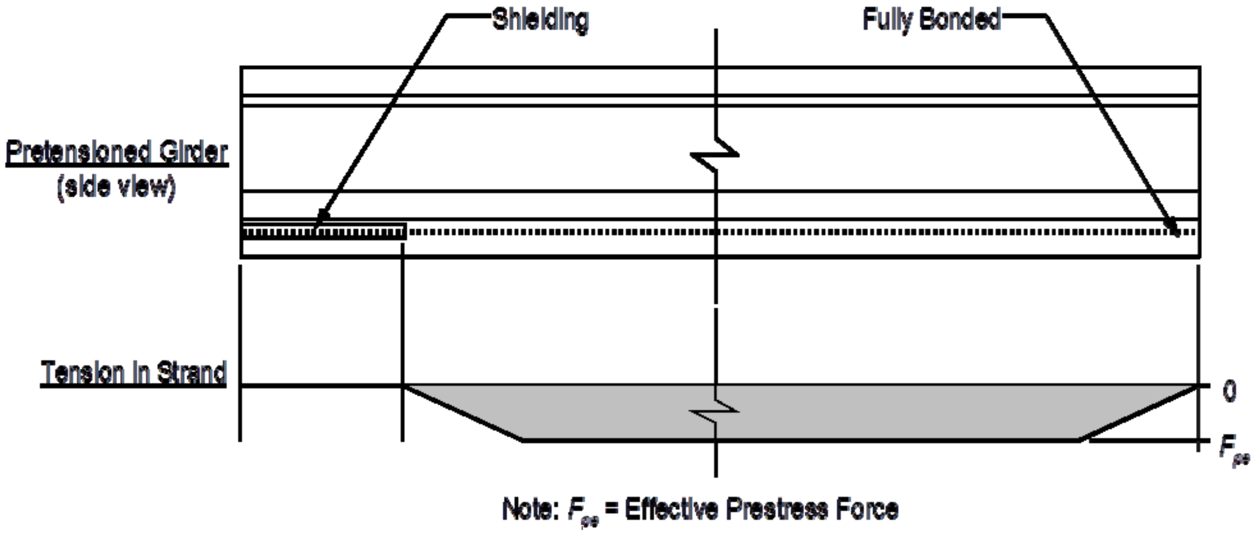


Figure 7–Strand debonding (shielding)

Strand Slip. Strand slip is movement of prestressing strands relative to the surrounding concrete due to applied loads. Strand slip generally occurs after the formation of cracks within the strand development length. These cracks reduce the available bond length for developing the strands, thus leading to strand slip once load on the strands exceeds capacity along the reduced embedment length. Strands can still partially develop even after the onset of slip.

3 Confinement Reinforcement

This chapter summarizes the work of other researchers regarding confinement reinforcement and other relevant topics. It is organized topically, beginning with a discussion of end region failure modes. Code requirements regarding confinement reinforcement and end region detailing are also summarized.

3.1 End Region Failure Modes

The end region of pretensioned girders can fail in modes other than basic "flexural failure" or "shear failure." Two such failure modes are bond-shear failure and lateral-splitting failure. Both of these modes have been observed experimentally, and have relevance in the study of confinement reinforcement.

3.1.1 Bond-Shear Failure

After cracks form in the end region, load is carried by a strut-and-tie mechanism (Figure 8). If cracks form near the girder end then strand anchorage is interrupted, and the strands may slip (Figure 9). Bond-shear failure occurs when strands can no longer support load, or when the compression zone crushes due to the rotation allowed in-part by strand slip. This type of failure can be very sudden and has also been called "bond-tension failure" or "bond failure." Bond-shear failure has been observed during load tests of girders without confinement reinforcement (Maruyama and Rizkalla 1988, Kaufman and Ramirez 1988, Englekirk and Beres 1994, Llanos et al. 2009), as well as during tests of girders with confinement reinforcement (Deatherage et al. 1994, Barnes et al. 1999, Kuchma et al. 2008).

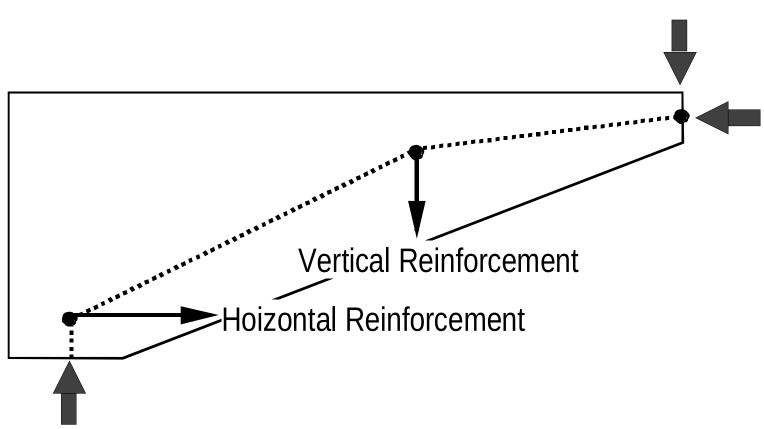


Figure 8–End region strut-and-tie behavior

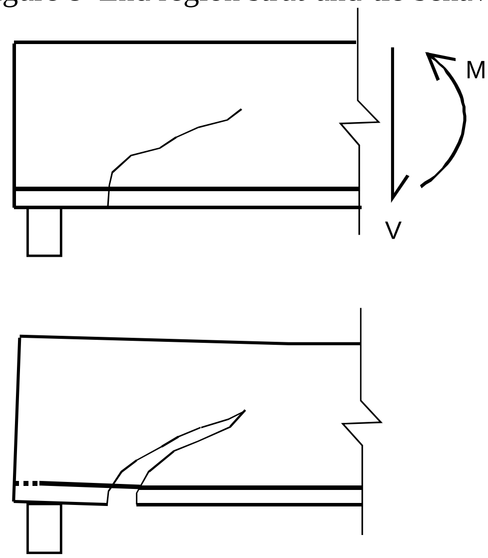


Figure 9–Bond-shear failure.

3.1.2 Lateral-Splitting Failure

As with bond-shear failure, lateral-splitting failure occurs after cracks form in the end region and the girder begins strut-and-tie behavior. In this failure mode, splitting cracks (Figure 10) form due to transverse stresses above the support (Llanos et. al. 2009, Csagoly 1991). Splitting cracks can lead to strand slip relative to the surrounding concrete. In girders without confinement reinforcement the formation of splitting cracks and the associated strand slip can lead to sudden girder failure. Development of transverse forces and their relationship to confinement reinforcement are primary considerations of the research presented in this document.

Figure 10 shows a splitting crack forming at the centerline of the cross-section. Splitting cracks in the outer portion of the bottom flange have also been observed experimentally (Llanos et. al. 2009). Splitting cracks in the flange can also lead to sudden girder failure.

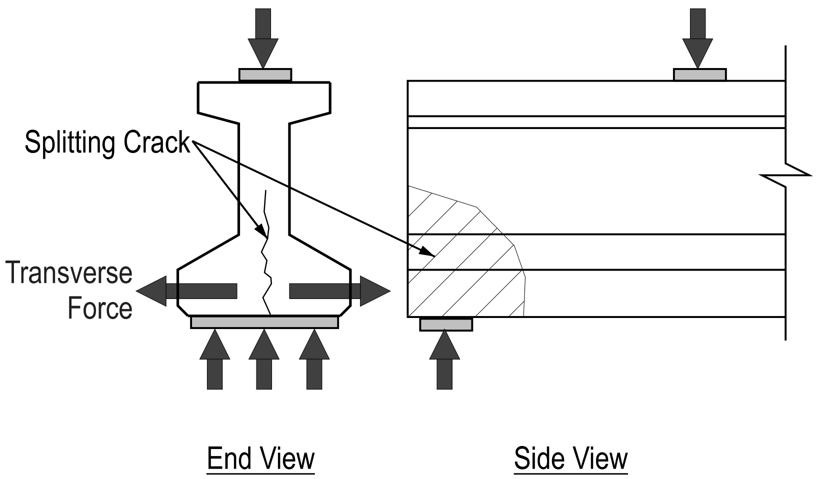


Figure 10–Splitting crack

3.2 Code Requirements

Confinement reinforcement is required by AASHTO LRFD article 5.10.10.2 (AASHTO 2009). Figure 1 graphically presents the confinement reinforcement requirements. The requirements are prescriptive, meaning that confinement reinforcement is not designed, but is rather specified according to the strict “recipe.” The function of confinement reinforcement is not discussed in the code or in the associated commentary.

Strand shielding is addressed by AASHTO LRFD article 5.11.4.3. This article limits shielding to no more than 25% of strands in a girder. Limits are also placed on the percentage of shielded strands in a given row (40%) and the quantity strands that can have shielding terminate at the same section (greater of 40% or four strands). Shielding is required to be symmetric about the cross-section centerline.

Article 5.8.3.5 addresses the amount of longitudinal steel required at any section, including sections near the supports. Requirements at the support are based on a strut-and-tie model similar to that shown in Figure 3. Sufficient steel must be provided to support the horizontal tie force above the bearing. To prevent bond-shear failures the article states that, “Any lack of full development shall be accounted for.”

3.3 *Confinement Reinforcement during Prestress Transfer*

In addition to design for ultimate loads, pretensioned girders must also be designed and detailed for serviceability criteria arising from fabrication, shipping, deck placement, and service. Loads from prestress transfer in particular can have negative consequences on performance and durability of girders. Llanos et al. (2009) observed splitting cracks at prestress transfer (Figure 11) in the bottom flange of test girders. Splitting cracks formed due to a problematic strand-shielding pattern wherein fully bonded strands were placed in the outer portion of the flange and shielded strands were placed below the web. Splitting cracks were allowed to propagate because confinement reinforcement was not present. Other researchers (Russell and Burns 1996) recommended the use of confinement reinforcement to prevent splitting at prestress transfer.

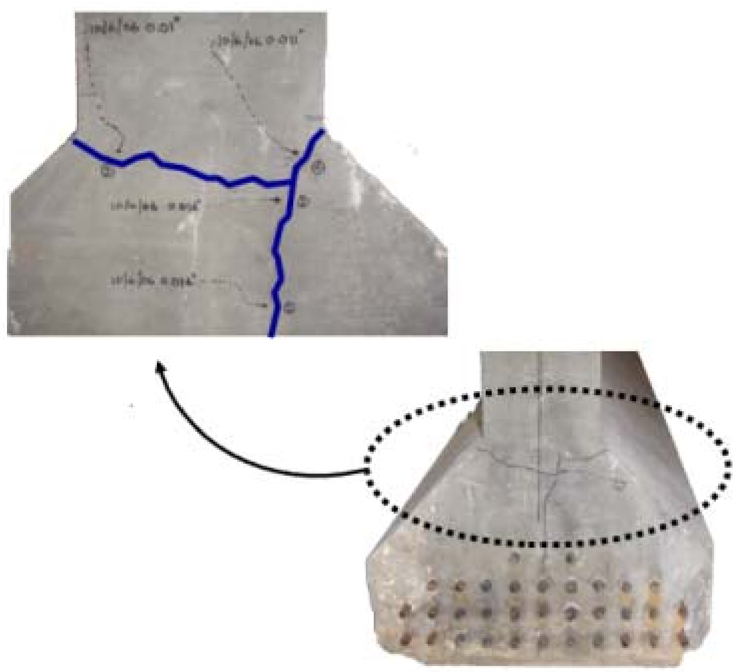


Figure 11–Splitting crack at prestress transfer

Russell and Burns (1996) investigated transfer length for 0.5 and 0.6 in. diameter prestressing strands in girders with and without confinement reinforcement. The authors' state:

Overall, confining reinforcement had little or no effect in improving the transfer lengths. In fact, the measured transfer lengths for strands confined by mild steel reinforcement were marginally longer than strands where confinement was not provided.

Other authors have reported similar findings for 0.7 in. diameter strands (Patzlaff et al. 2010, Akhnoukh 2010). None of the authors investigating transfer length in confined girders reported splitting cracks during prestress transfer. Each concluded that the negligible impact on transfer length was due to the inactivity of confinement reinforcement in the absence of cracking.

Work regarding confinement reinforcement in the anchorage zone of post-tensioned concrete members has been conducted by Breen et al. (1991) and Roberts (1990). These works, while useful for post-tensioned members, do not apply directly to the design of pretensioned members. In post-tensioned concrete, prestress force is transferred to the concrete in a relatively small local zone. In pretensioned concrete, however, the prestress force is transferred to the concrete over the relatively large strand transfer length. The local zone in post-tensioned members is analogous to an axially loaded column, and confinement reinforcement for the local zone can be designed using an approach similar to columns. As demonstrated in subsequent chapters of this document, confinement in the bottom flange of pretensioned I-girders is used to control cracks and to carry transverse tension forces. These functions are distinct from confinement reinforcement in columns and post-tensioned local zones, which provide confinement to axially loaded concrete.

3.4 Confinement Reinforcement during Loading

Confinement reinforcement has been shown to affect girder performance under applied loads. This section summarizes previous research investigating the effects of confinement reinforcement on girder strength and behavior during loading.

3.4.1 <u>Shear Capacity</u>

Csagoly (1991) tested 16 pretensioned girders, some with confinement reinforcement and some without. Confinement reinforcement improved shear capacity by an average of 13% relative to girders without confinement. Shahawy et al. (1993) also tested girders with and without confinement reinforcement; results indicated that confinement reinforcement improved shear capacity by 10% to 17%.

3.4.2 Transverse Reinforcement

In some cases, equilibrium of strut-and-tie systems requires the formation of tension ties in bottom flange in the transverse direction. If confinement reinforcement is not provided to act as the transverse tie, concrete must carry the tension force. This condition is undesirable, and has been found culpable in splitting cracks by Csagoly (1991) and Llanos et al. (2009).

According to Csagoly, splitting cracks result from "the spreading of the reaction force above the bearing." This concept is shown graphically in Figure 12. A transverse force *T* is formed to maintain static equilibrium as the reaction force is spread from the web to the bearing pad. Splitting cracks form when the transverse force exceeds the concrete tensile capacity.

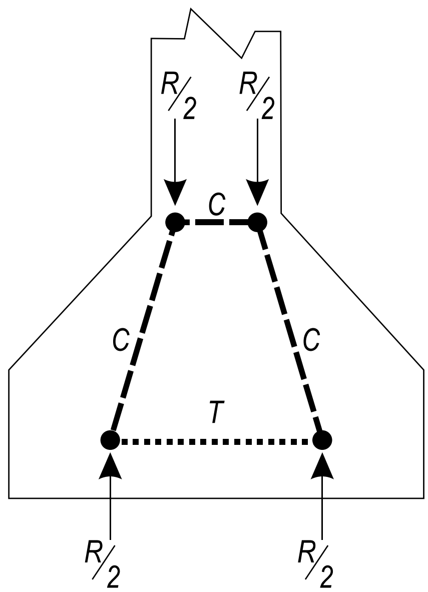


Figure 12–Transverse tie above bearing (after Csagoly 1991)

Llanos et al. (2009) also observed splitting failures in load tests of full-scale girders, and likewise attributed these failures to transverse forces above the support. Mechanics leading to these transverse forces were different from the Csagoly model, and are described by Figure 13. Figure 13a is a model of a girder with bonded strands below the web. Inclined compressive forces travel through the web, arriving at a node above the support. A tie resists the horizontal component of the inclined force. Because strands are fully bonded, they can act as the tie, and equilibrium is maintained. In contrast to Figure 13a, Figure 13b models the condition from the Llanos et al. test girders in which fully bonded strands were located at the edges of the flange. Because only the strands in the outside portion of the flange were bonded near the support, they were the only strands able to act as ties (Figure 13b). This resulted in a disruption of the node at the support point. Because of the offset between the strut in the web and the two ties (fully bonded strands) in the bottom flange, secondary struts formed to transfer the load laterally to the nodes at the ties. Additional secondary struts were essential between the support and the nodes at the ties to complete the load path to the support. Both pairs of secondary struts induced horizontal components that acted transverse to the beam.

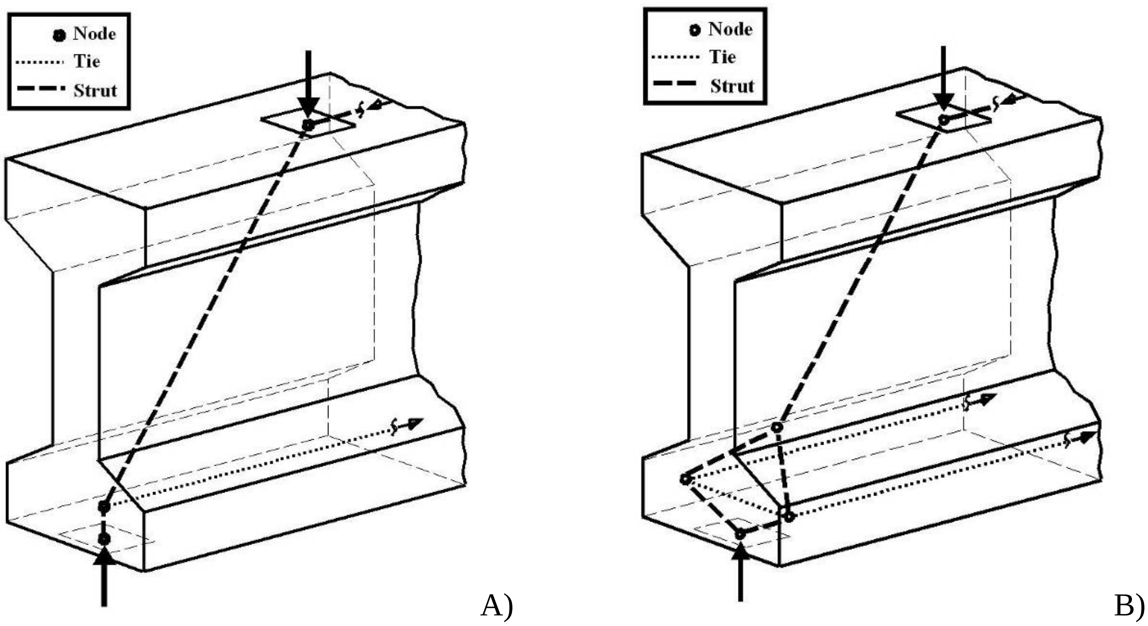


Figure 13–End region strut and tie models A) with fully bonded strands below web and B) debonded strands concentrated below web.

Test beams had no reinforcement (such as confinement reinforcement) to support the transverse force, and edges of the bottom flange peeled or split away at failure (Figure 14). The authors speculate that "[confinement] reinforcement might have held the bulb together after cracking."

A)

B)

Figure 14–Flange cracking in girder with debonded strands concentrated below web A) side view and B) end view.

3.4.3 <u>Vertical Reinforcement</u>

Another possible function of confinement reinforcement is that of vertical, or "shear" reinforcement. Csagoly (1991) proposed this function and included it in a strut-and-tie model of the end region. In the model, inclined cracks crossing confinement reinforcement mobilize the confinement steel, thereby generating a vertical force that contributes to the end region shear capacity (Figure 15).

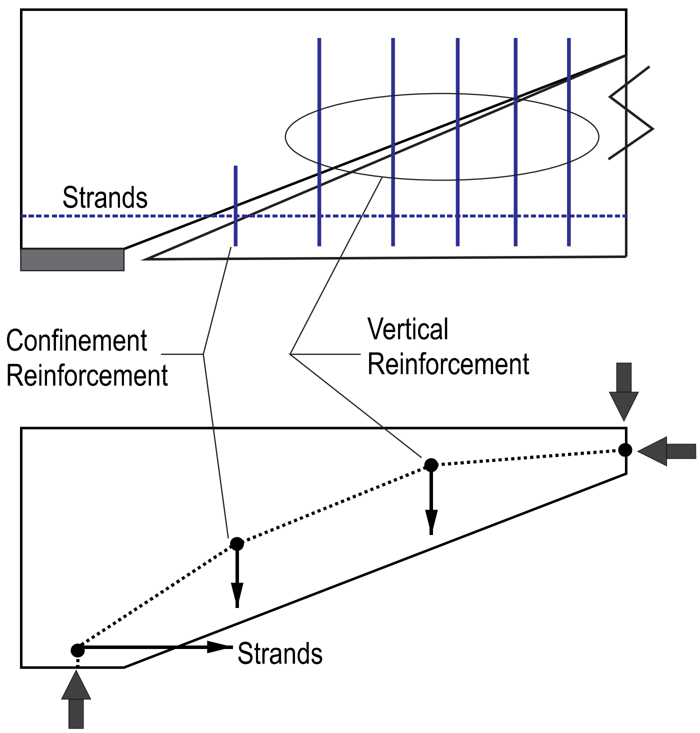


Figure 15–Confinement reinforcement as shear reinforcement

3.4.4 Ductility

Morcous et al. (2010) noted an improvement in ductility in girders with confinement reinforcement. Work presented in Appendix B and Appendix D confirm Morcous's finding and provide information on the mechanisms by which confinement reinforcement leads to improved ductility.

3.4.5 Development Length

By restraining cracks, confinement reinforcement may reduce strand development length. To test this possibility, Patzlaff et al. (2010) load tested (6) pretensioned concrete T-beams with varying configurations of confinement reinforcement. In all cases, the girders failed in flexure, and at loads sufficient to fully develop the strands. Because the strands reached full development in all tests, no conclusions were made regarding the effect of confinement reinforcement on development length.

Akhnoukh (2010) investigated the effects of confinement reinforcement on development length by conducting pullout tests of 0.7 in. diameter strands embedded in 4 ft, 5 ft, and 6 ft long concrete prisms. Specimens also varied in the quantity and spacing of confinement reinforcement. Pullout tests were terminated when the strand ruptured or when the strand slipped relative to the concrete. The 4 ft specimens with (5) #3 confinement hoops always flailed by strand rupture, whereas the majority of the 4 ft specimens with (3) #3 confinement hoops failed by strand slip. The author concluded that confinement reinforcement decreased the development length.

3.4.6 Shear Friction

Akhnoukh (2010) proposed a shear friction method for designing confinement reinforcement. Currently this is the only design method in the literature. The Akhnoukh method is based on an assumed crack running through a row of strands, which engages the confinement reinforcement thereby inducing normal and friction forces on the crack plane (Figure 16). Equilibrium is applied in the longitudinal direction to equate the friction force with the force in the strands, resulting in Equation 1. Akhnoukh applied this model to design confinement reinforcement for the strand pullout tests discussed previously. It was concluded that the shear-friction concept can be used to quantify the effect of confinement reinforcement on strand development.

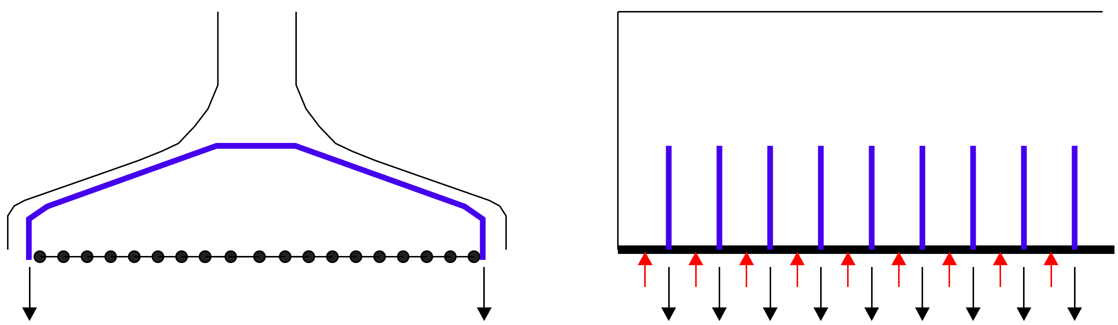


Figure 16–Shear-friction model (Based on Akhnoukh 2010).

$$A_{ts} = \frac{A_{ps} \cdot f_{ps}}{\mu \cdot f_{tsy}} \qquad \text{Equation 1}$$

Where:

A_{ts} = Area of transverse (confinement) reinforcement crossing crack (in^2)
A_{ps} = Total area of prestressing strand (in^2)
f_{tsy} = Yield strength of transverse (confinement) reinforcement (ksi)
f_{ps} = Stress in prestressing strands at ultimate capacity (ksi)
μ = Coefficient of friction

3.5 Hoyer Effect

The diameter of prestressing strands decreases due to the Poisson effect as strands are pretensioned (Figure 17). Upon release, strand tension at the end of the member is relieved, and the strand diameter increases. The surrounding concrete resists the increase in diameter, which causes tensile stress in the concrete and develops mechanical bond between the strand and concrete. Radial strand expansion and concrete tensile stress are greatest at the member end. Radial strand expansion and the associate concrete tensile stresses are zero at the end of the transfer length. This condition is referred to as the Hoyer Effect and is the primary contributor to strand-concrete bond capacity. This effect is named for Ewald Hoyer, the German Engineer who first discussed radial expansion of prestressing strands (Hoyer 1939).

Oh et al. (2006) derived a model for transfer length that accounts for the Hoyer effect. The Oh model is rigorous, utilizing equilibrium, material constitutive properties, and strain compatibility. The model can be used for calculating stresses at the strand-concrete interface and for calculating tensile concrete tensile stresses. Building on the equilibrium-compatibility portion of the model, Oh then considers the effects of concrete cracking adjacent to the strands within the transfer length. The full model (including crack effects) was compared to experimental tests of transfer length and found to have good correlation.

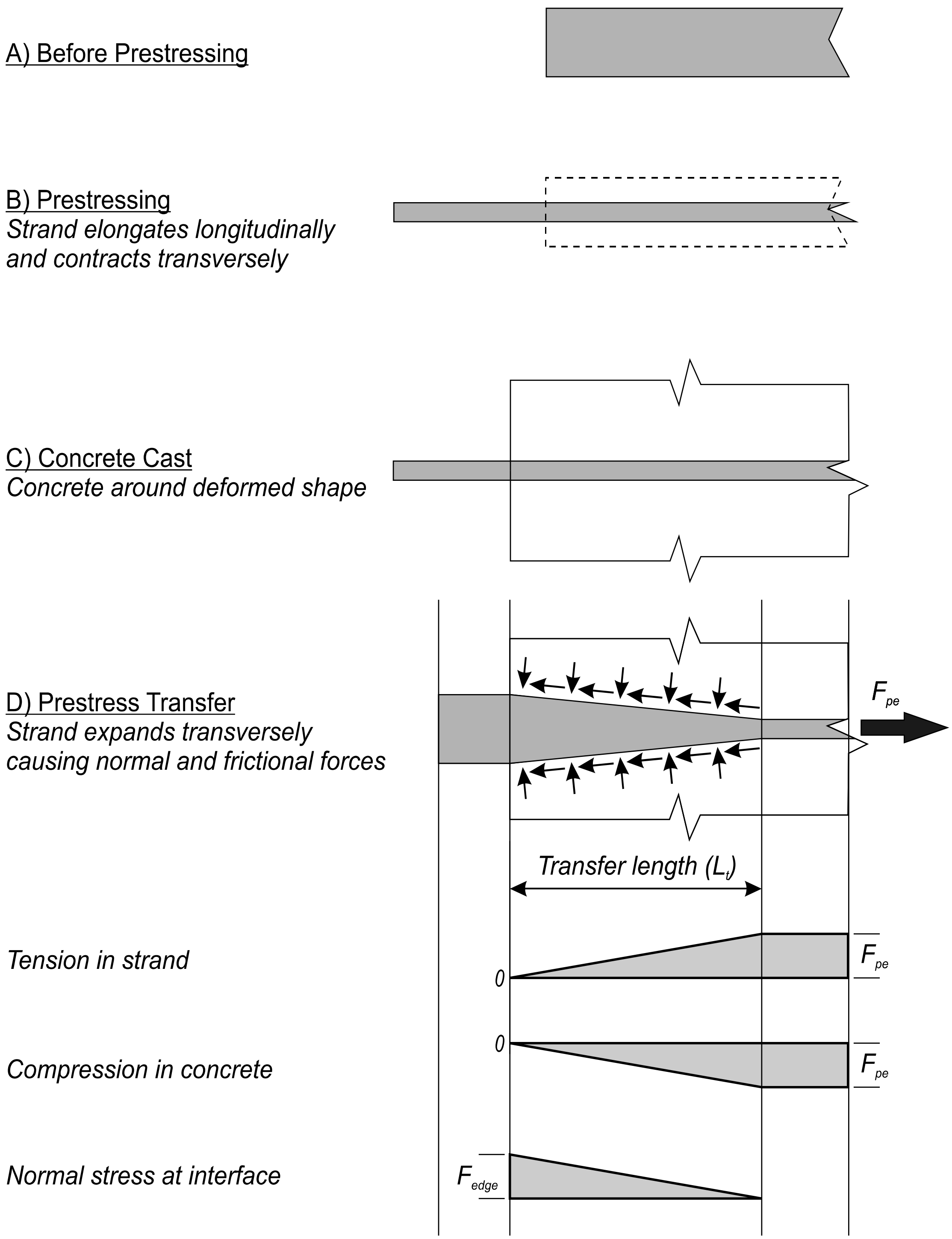


Figure 17–Hoyer effect in A) Strand before stressing, B) strand after prestressing, C) concrete cast around strand, and D) stresses and forces after transfer

3.6 Summary

The following conclusions are made based on the aggregate findings of the relevant literature. Results from two or more authors support each conclusion.

- The presence of confinement reinforcement does not prevent strand slip
- Confinement reinforcement is inactive until engaged by cracks in adjacent concrete
- Confinement reinforcement has negligible effect on transfer length in uncracked concrete
- Confinement reinforcement improves shear capacity and ductility of girders
- Transverse tensile forces forming above the bearing can lead to splitting cracks in concrete girders
- Transverse tensile forces form due to applied loads and the Hoyer effect

Based on limited treatment or complete absence in the literature, the following topics are deserving of additional attention:

- Function of confinement reinforcement at prestress transfer and ultimate strength
- Effect of confinement reinforcement on development length
- Optimal quantity and placement of confinement reinforcement
- Relationship between confinement reinforcement and splitting failure
- Effect of prestress force on bottom flange transverse tensile stress
- Effect of confinement reinforcement on strand-concrete bond capacity
- Interaction of confinement reinforcement with other end region variables
- Rational confinement reinforcement design methods

4 Web Splitting

Web splitting cracks are the horizontal or diagonal cracks that form in the end region of pretensioned concrete I-girders during or following the prestress transfer (Figure 18). Elsewhere these cracks are referred to as "bursting", "spalling", or "splitting" cracks. Reinforcement for controlling these cracks is referred to as "bursting" resistance in the 2007 AASHTO LRFD code and "splitting" resistance in the 2010 code. For the purposes of this report these cracks will be referred to as web splitting cracks.

Figure 18–Web splitting cracks (enhanced in blue)

Beginning the 1950s, many researchers have attempted to model stresses in the concrete that lead to web splitting cracks. Other researchers have focused on controlling web splitting cracks through strength evaluation of the vertical reinforcement in the end region. This review of literature will discuss both of the above approaches as well as outlining current code requirements and crack treatment protocols.

4.1 Code Requirements

AASHTO provisions for web splitting reinforcement were first introduced in 1961 and have undergone little revision since that time (Tadros 2010). Provisions were influenced by the work from Marshall and Mattock (1962).

The 2010 AASHTO LRFD Bridge Design Specifications require sufficient vertical reinforcement to resist a force equal to at least 4% of the total prestressing force while limiting the allowable stress in the reinforcement to 20 ksi. The reinforcement must be within *h/4* (where *h* is the height of the beam) of the end of the beam and the end bar should be as close to the end of the beam as practicable. The required end region vertical reinforcement can be taken as:

$$A_s = 0.04\frac{F_{pi}}{f_s} \qquad \text{Equation 2}$$

where:

A_s = total area of vertical reinforcement located within the distance h/4 from the end of the beam (in^2)

F_{pi} = prestress force at transfer

f_s = stress in steel not exceeding 20 ksi

4.2 Modeling of Web Splitting Cracks

As noted by Dunkman (2009) models addressing web splitting cracks have typically been based one of two approaches. The first approach focuses on calculating concrete stresses that lead to web splitting. The other approach focuses on calculating the required strength of the end region, particularly vertical reinforcement, to prohibit excessive crack sizes. Both modeling approaches are discussed in this section.

4.2.1 Stress Modeling

Numerous methods have been used to calculate stresses that cause web splitting cracks. Analytical methods employing elastic, inelastic, and plastic assumptions have all been used with various degrees of success.

Currently finite element analysis (FEA) is the favored analysis approach due to method's ability to model stresses in members with complicated loadings and geometry. FEA has been used by Breen et al. (1994), and Kannel et al. (1997). Other analysis approaches include infinite series (Iyengar 1962) and finite difference approaches (Gergely et al. 1963). Guyon (1955) developed a method of analysis using a symmetric prism method, which is limited to the region directly adjacent to the prestressed force and subsequently only useful in estimating bursting forces and not splitting forces. This approach has been influential in the development of European codes (Dunkman, 2009).

Inelastic analysis was employed by Gergely et al. (1963) to produce the Gergely-Sozen model. Their approach assumed an initially cracked cross section and was based on equilibrium of the end region. The Gergely-Sozen model gives a conceptual framework for describing the formation of tensile forces in the web (Figure 19).

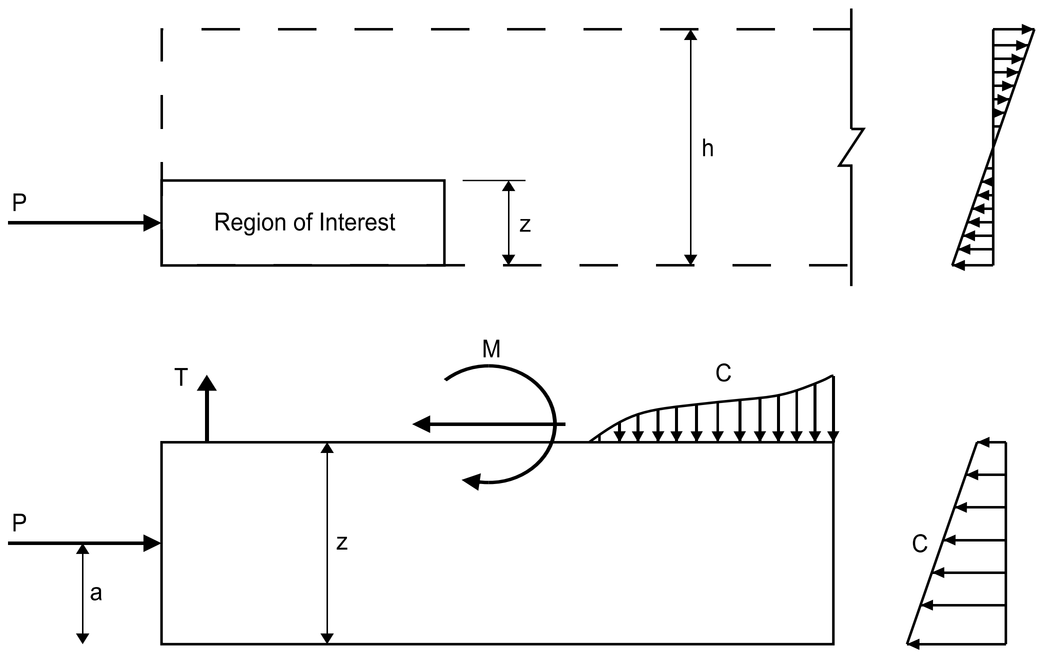


Figure 19–Free-body diagram based on Gergely-Sozen model (after Gergely et al. 1963)

where:

P = prestress force

z = height of section analyzed

h = member depth

a = height of prestressing force

T = web splitting tensile force

M = resulting moment and shear forces

C = resulting compression fields

4.2.2 Strength Modeling

Marshall and Mattock (1962) and Gergely et al. (1963) are frequently credited as landmark studies that recognized the need to develop a pragmatic approach for designing transverse reinforcing steel in the end region. The Gergely-Sozen model discussed above provides one method for estimating the force to be resisted by the transverse steel. Marshall and Mattock developed Equation 4 for calculating the required area of steel in the end region. This equation was incorporated into AASHTO design standards with slight modifications. To be more conservative the 1962 AASHTO code implicitly changed the ratio of h/l_t to 2. More recently, an experimental investigation by Tuan et al. (2004) has confirmed Marshall and Mattock's equation and suggested that 50% of the area of steel be concentrated to within *h/8* from the end of the member and the balance between *h/8* and *h/2*.

$$A_r = 0.021\frac{P_i}{f_s}\frac{h}{l_t} \qquad \text{Equation 3}$$

where:

A_r = required transverse reinforcement at the member end

P_i = prestress force at transfer

f_s = maximum allowable stress in reinforcement (20 ksi)

h = member depth

l_t = strand transfer length

Strut-and-tie modeling (STM) has also been used to design reinforcement for controlling web splitting cracks. STM has been influential in the development of European codes and has been investigated by numerous researchers (Schlaich et al. 1987, Castrodale et al. 2002, Davis et al. 2005, and Crispino 2007).

4.3 Treatment of Web Splitting Cracks

Treatment protocols for web splitting cracks are typically based on the beam exposure conditions and width and location of cracks. Historical recommendations for the allowable crack widths and treatment procedures are provided in Tadros et al. (2010). Table 2 and Table 3 list treatment procedures from Tadros et al. (2010) and FDOT Specification 450 (2012). Although not listed in this document, recommendations for treating web splitting cracks have also been made by PCI (1999) and CEB (1993).

Table 2–Crack treatment procedures as recommended by Tadros et al. (2010)

Crack Width	Recommended Action
< 0.012 in.	Left unrepaired
0.012 - 0.025 in.	Repaired by filling the cracks with approved specialty cementitious materials, and the end 4 ft of the girder side faces coated with an approved sealant
0.025 - 0.050 in.	Filled with either epoxy injection or cementitious patching material (depending on crack width) and the surface coated with a sealant
> 0.05 in.	Girder rejected

Table 3–Crack treatment procedures as recommended by FDOT Specification 450 (2012)

Crack Classification	Environment	Action
Cosmetic cracks (< 0.006 in.)	Slight - Moderate	Do not treat
	Extreme	Apply penetrant sealer
Minor cracks (0.006 - 0.012 in.)	Slight	Do not treat
	Moderate	Beam Elevation > 12 ft - Do not treat Beam Elevation < 12 ft - Apply penetrant sealer
	Extreme	Inject epoxy
Major crack (> 0.012 in.)	All	Engineering evaluation required

5 Flange Effect on Shear Strength

Contributions of compressive flanges to shear capacity of concrete beams are typically not considered in design codes. This practice is conservative as multiple researchers have shown that flanges, such as those on T-beams, do indeed contribute to shear capacity. This section summarizes the available research regarding shear contribution of flanges.

Leonhardt and Walther (1961) tested a series of RC beams in which the shear reinforcement, flange width and thickness were held constant while the web thickness varied (Figure 20). Under both uniform and concentrated loads, the shear capacity of the beams increased as the web thickness increased. The increase was less pronounced when the web thickness was greater than about half of the flange width. The increase was also less pronounced for beams tested with concentrated loads.

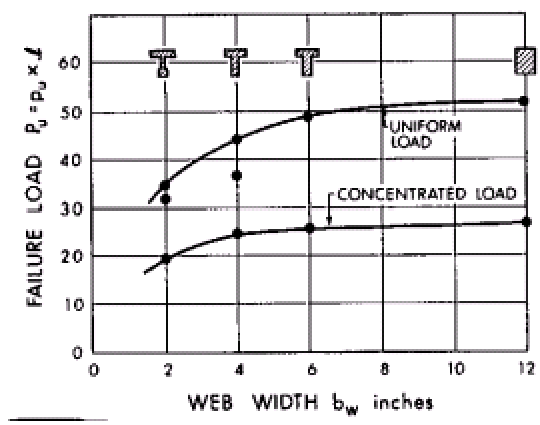


Figure 20–Effect of web width (Leonhardt and Walther 1961)

Placas and Regan (1971) conducted shear tests on beams with constant web width and varying flange sizes. Figure 21 presents the experimental relationship between shear capacity and flange width. For constant web thickness, the presence of a flange increases the shear capacity. The increase in shear capacity, however, is essentially independent of the flange width for the range tested. The shear capacity also increased as flange thickness increased.

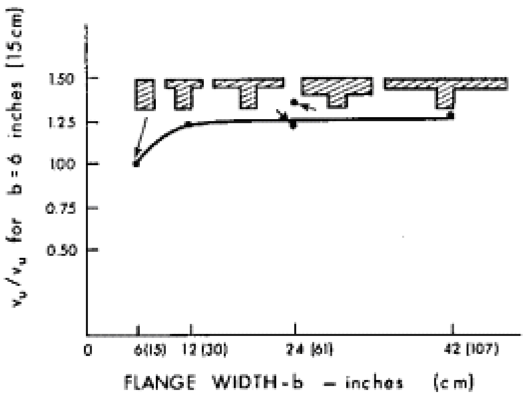


Figure 21–Effect of flange thickness (Placas and Regan 1971)

Placas and Regan (1971) present Equation 4 for calculating the area of the flange that is effective in resisting "shearing failure" of the compression zone. Shearing failure is distinguished from shear-compression failure, which the authors indicate is not a critical mode in T-beams. Because "shearing failure" is only a function of the compression zone, the web is not considered to contribute shear strength for this failure mode.

$$A_f = t(b_w + x) \qquad \text{Equation 4}$$

Where:

A_f = Area of the flange effective in resisting shearing failure of the compression zone

t = Thickness of compressive flange

b_w = Web width

x = 6" (Based on curve fit with available data)

ASCE-ACI committee 426 (1973) presented a document summarizing research on shear capacity of reinforced concrete beams. Research by Leonhardt and Walther (1961) and Placas and Regan (1971) presented in the previous paragraphs was among the work summarized by committee 426. The committee reported that flexural-compression stresses in the flange are distributed over a greater width than are the shear stresses and that only the portion of the flange "immediately adjacent" to the web can contribute to V_{cz} (shear contribution of the compression zone). Accordingly, Equation 5 is presented to address shear contribution from portions of the flange immediately adjacent to the web. This equation assumes that the shear carried by the concrete in the flange is only a function of the flange thickness. The committee suggests that “For design purposes, however, it seems reasonable to ignore the strengthening effect of the flange [beyond the portion immediately adjacent to the web].”

$$V_c = v_c\left(b_w d + 2h_f^2\right) \qquad \text{Equation 5}$$

Where:

V_c = Concrete contribution to shear capacity

v_c = Shear stress in the concrete

b_w = Web width

d = Distance from extreme tension fiber to centroid of tensile reinforcement

h_f = Depth of compression flange

Giaccio et al. (2002) conducted a series of tests on 15 RC T-beams to evaluate the flange contribution to shear strength. The test beams where loaded in three-point bending. Variables included the flange width and thickness. The authors report that portions of the flange beyond those adjacent to the web will contribute to the shear capacity if a flange thickness is large enough relative to the beam depth. The flange beyond the web intersection contributed to the shear capacity if the flange thickness was at least 0.25*d* (where d is the distance from the extreme compression fiber to the centroid of the tensile reinforcement). For flanges of sufficient thickness, the width of the flange contributing to the shear capacity was $4b_w$ (where b_w is the web thickness). For beams with wide flanges (b_f>$4b_w$), the contribution of the flange was governed by its punching strength. The ratio of b_f>$4b_w$ corresponded to a change in failure mode from a beam shear mechanism to a punching shear mechanism.

Kostovos et al. (1987) conducted a series of four tests on RC beams to evaluate the effect of the flange on the shear capacity. Test beams were loaded uniformly, or with two symmetric point loads. Tied-arch behavior was observed after cracking. Failure of the T-beam shown in Figure 22, was attributed to instability of the concrete arch after the onset of tied-arch behavior. Comparing their test results with data from the literature, the authors found that for a given a/d ratio, the normalized shear carrying capacity of the tested tee beams were as much as 200%

larger than rectangular beams. No attempt was made, however, to quantity the effects of the flange on the shear capacity.

Figure 22–T-beam failure after tied-arch behavior

Chong and Arthur (1987) tested twenty-one RC beams under uniform load. Flange thickness and width were varied. The tee beams behaved as tied-arches, and failed due to instability of the arch. The authors report that increased flange thickness increased the load at the first shear crack, but did not affect the ultimate shear force supported by the beams.

Ibell et al (1999) conducted tests on individual rectangular and T-beams, as well as beam-slab structures (Figure 23). Individual beams were loaded in three-point bending, and failed in shear or flexure. The beam-slab structures where loaded by multiple point loads, and failed in punching-shear, tearing-shear, or flexure. The tearing shear failure was characterized by cracks in the slab running perpendicular to and between the beams. The tears occurred because the slab could not support the load induced by differential movement between adjacent loaded and unloaded beams. Using ACI and BD codes of practice, the authors calculated the predicted capacity of both the beam-slab structures and the individual beams. Comparing the calculated capacities with the experimental results, it is noted that: "…while these codes-of-practice predict relatively accurate two-dimensional beam capacities, they substantially underpredict the enhancement in shear strength that the redundancy of beam-and-slab bridge provides. This underprediction, due to membrane enhancement from the surrounding concrete slab, is in the region of 25 to 35 percent herein, depending on which code-of-practice is used as the basis for comparison." Interior beams particularly benefited from membrane effects and load distribution to adjacent girders. By monitoring the reactions in the beam-slab structures during testing, it was observed that the distribution of load was greater than predicted by a grillage analysis, and that the relative load distribution between beams did not change during the course of the test.

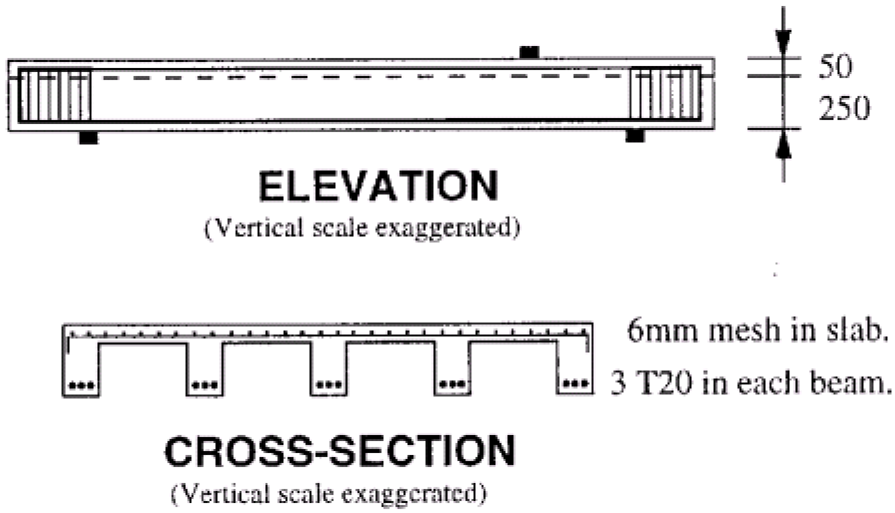


Figure 23–Beam-slab test specimen (Ibell et al. 1999)

Ruddle et al (2003) investigated the effects of arching action on the strength of rectangular and T-beams. The authors report both analytical and experimental work. An analytical model for shear capacity of T-beams was presented, which considers the contribution of the flange away from the web up to $3b_w$ (where b_w is the thickness of the web). The authors report that "the ultimate flexural and shear strength of longitudinally restrained beams are enhanced by the development of arching action."

Zararis et al. (2006) developed a shear model for concrete T-beams and compared the model with experimental data from the literature. The Zararis model considers shear to be carried by the shaded portion of T-beams shown in Figure 24. Comparisons with the available literature show that the proposed model results in more accurate, but less conservative calculated capacities as compared to the ACI code.

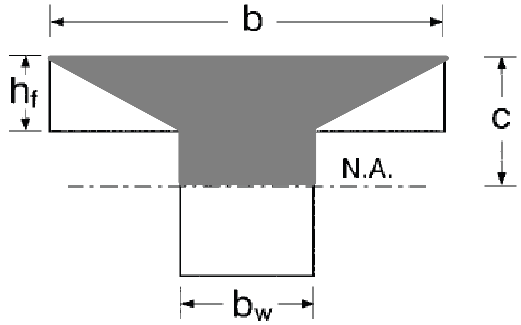


Figure 24–Shear contribution of flange (Zararis 2006)

Although there are differences in approach, the general trend in the literature is that flanges increase the shear capacity of T-beams beyond that of similar rectangular beams. The area of the flange contributing to shear strength is limited to those portions immediately adjacent to the web. Numerous authors suggest that the portion of the flange effective in supporting shear is a function of the geometric properties of the flange and/or web.

The failure modes observed in T-beams can vary from those observed in isolated rectangular beams; punching-shear being one observed failure mode, and arching instability of the compressive zone due to being another. Accordingly, flange contribution to shear capacity can be limited by the flange punching strength and/or compression zone arching capacity.

6 Curb Contribution to Girder Behavior

Concrete curbs, barriers, walls, and other appurtenances are often attached to concrete bridge decks, and are especially common above exterior girders. Although their contribution to the strength and stiffness of the girders are not considered in codes of practice, their effects are still present and can be significant. These effects are greatest when the elements and bridge behave compositely; however detailing of the elements can affect a system's ability to sustain composite action. In additional to affecting the strength and stiffness of individual girders, they can also influence load distribution between girders. Two studies of are presented in this section that demonstrate the effects of curbs, etc. on bridge capacity and behavior.

Harries (2009) tested concrete box girders that were salvaged from the partially-collapsed Lake View Drive Bridge in Pennsylvania. The collapse occurred in one of the exterior girders. Tests were conducted on the other exterior girder as well as the interior girder adjacent to the collapse (Figure 25). The exterior text girder supported a curb and a barrier wall. Test data confirmed a degree of composite action between the curb and the exterior girder; however the composite action deteriorated with increasing load. Because of the composite action, the curb and barrier wall "…clearly increase[d] the capacity of the box girder section, although this increase [was] tempered by an increase in the degree of asymmetry of the section; thus the increased capacity is not as significant as the increase in moment of inertia would suggest." Commenting on the approach designers might take when considering curbs and barrier walls, the author states that "It is not recommended that their contribution to girder capacity be relied upon under ultimate load conditions. Under service load conditions, however, it is likely that they may be considered to be composite with the girder."

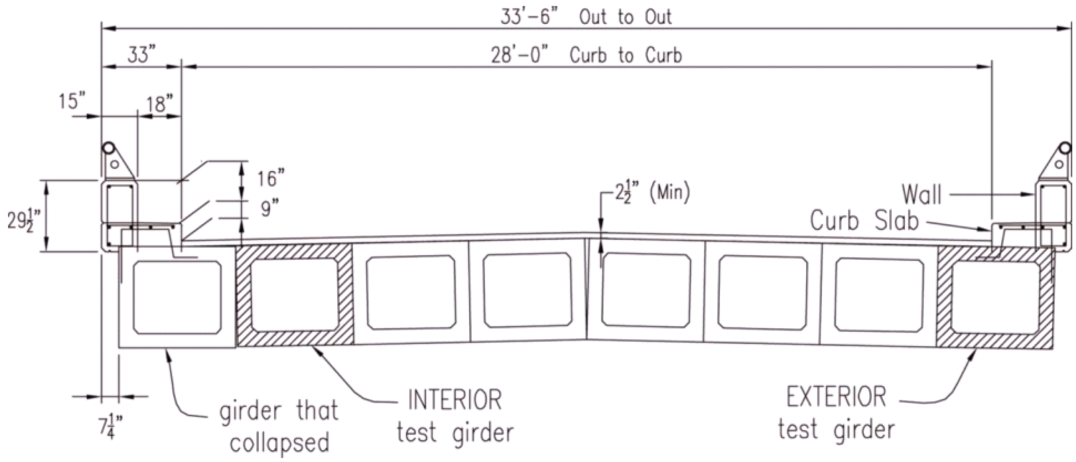


Figure 25–Cross section of bridge (Harries 2009)

Oh et al (2002) conducted an on-site load test of an existing bridge. A cross-section of the bridge is shown in Figure 26. Point loads were applied above the right-most girders near mid-span. The medium curb strip acted compositely with the deck and girders until it separated from the deck when the load reached about 50% of the eventual maximum load. Failure was initiated by compression crushing of the curb near the load points. The test was terminated after crushing occurred in the slab. The authors note that crushing of curbs could be used as a warning indicator preceding failure. Regarding the strengthening of the bridge by the median strip and curb, the authors state that a "…quantitative assessment of [their] contribution greatly depends on the integrity of those elements with the main slab system."

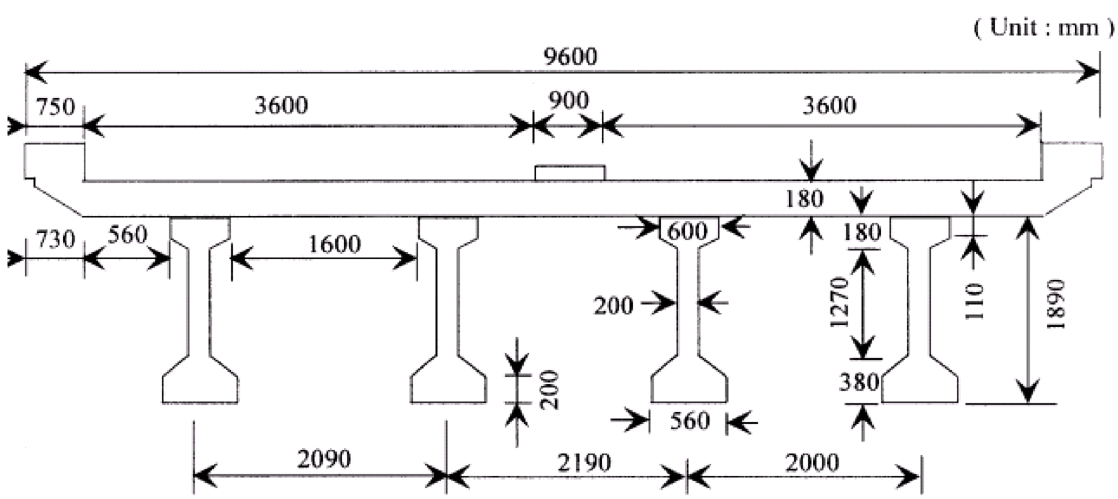


Figure 26–Cross section of test bridge (Oh et al. 2002)

7 Truss and Arch Action

Prior to cracking, RC beams transfer loads according to elastic beam theory. After cracking, loads in RC beams are transferred in “arch action” or “truss action.” Both types of behavior are shown in Figure 27, and are discussed further in the proceeding paragraphs.

Ritter (1899) proposed a truss analogy for modeling the transfer of forces in reinforced concrete beams. In the truss analogy, the concrete at the top of the beam is considered as a compression chord, the steel reinforcement in the bottom of the beam as a tension chord, concrete in the web as diagonal compression struts, and vertical reinforcement as tension members. This type of load transfer will be referred to as “truss action” within this report. In truss action, equilibrium requires that the vertical force component in the compression struts be equal and opposite of the vertical force in the transverse steel ties. Truss action will only occur in beams with transverse reinforcement. Additionally, the formation of cracks at the location of the vertical reinforcement is required to mobilize the reinforcement tie force.

In beams without transverse reinforcement, forces are transferred to the supports by arching of the concrete. To balance the horizontal force component at the supports, a tension tie is formed by the steel reinforcement in the bottom of the beam. This mechanism of load transfer will be referred to as “arch action” within this report.

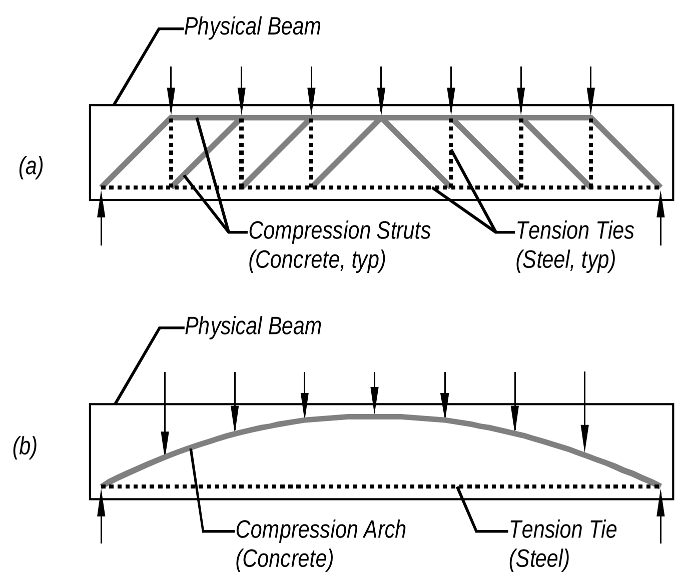


Figure 27–Beam behavior (a) truss action and (b) arch action.