ALBERTA TRANSPORTATION EXPERIENCE WITH DEBONDED PRE-TENSIONING STRANDS IN PRECAST NU GIRDERS

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ABSTRACT

Precast concrete NU girders have been used in Alberta Transportation bridges since 2001 with spans ranging from 30 - 65m. These bridges consist of pre-tensioned NU girders which are typically post-tensioned in the field for longitudinal continuity. It is standard practice in Alberta to utilize deflected strands as well as debonded strands to minimize stresses at the ends of these girders during fabrication. The Canadian Highway Bridge Design Code (CHBDC S6-06) limits the number of debonded strands to a maximum of 25% of the total number of strands in the girder. Design engineers have found the code clause to be somewhat unclear and have often questioned whether the limits are applicable to girders which are pre-tensioned and post-tensioned.

In Alberta, it has been typical practice for designers to limit the percentage of debonded strands to between 25% and 40% of the total number of pre-tensioning strands in the girder and to stagger the strand debonding locations. On a recent project, NU girders were fabricated with over 50% debonded strands, all of which were debonded concurrently. This paper presents the engineering rationale that was developed to support the acceptance of these girders.

INTRODUCTION

The practice of debonding pre-tensioning strands at the ends of precast prestressed concrete girders has been in use for quite some time. Strand debonding is typically accomplished by sheathing the prestress strands in plastic tubing for a given length from the ends of a girder. The plastic tubing prevents the strand from bonding to the concrete (Figure 1). The purpose of debonding the strands is to reduce the pre-tensioning stresses in the girder ends, particularly just after strand release when the pre-tensioning forces are the highest and the girder concrete strengths are the lowest.

The Canadian Highway Bridge Design Code CAN/CSA-S6-06 (CHBDC) places limits on the use of debonded strands in prestressed girders. The CHBDC states, "Where bonding of the strand does not extend to the ends of the component and tension occurs at the serviceability limit state within the development length, l_d, a development length of $2l_d$, shall be used. The number of strands where the bonding does not extend to the ends of the member shall not exceed 25% of the total number of strands." The 2010 AASHTO LRFD Bridge Design Specifications (AASHTO) contain similar limits as well as additional requirements that limit the pattern and location of debonded strands. In practice, the 25% limit is often exceeded without any noticeable negative effect on the girder behaviour. In the commentary section, AASHTO acknowledges that the 25% limit may be exceeded based on local past experience and testing, but advises that when debonding exceeds 25%, "the shear resistance in the region should be thoroughly investigated".

ALBERTA TRANSPORTATION EXPERIENCE

For Alberta Transportation (AT) projects, the NU precast concrete girder shape is typically used for longer span bridges. Typical spans for these girders range between 30 - 65m, with girder depths ranging from 1.2 - 2.8m deep. These girders are typically pre-tensioned and then post-tensioned for continuity in the field.

It has been AT's approach to provide only minimal guidance and standards for NU girders and to allow the industry (independent consultants and girder fabricators) to develop local practices for designs and details. As a result of this "hands-off" approach, there has been significant variation in consultants' approach to utilizing debonded strands and their interpretation of the code limits. Some consultants have not used debonded strands at all, preferring to use deflected strands, while other consultants have regularly exceeded the debonded strand code limits. Over the years, it has been common for girder designs to specify 30 - 40% debonded strands. Typically, designers have staggered the locations where strands are debonded. Although this staggered debonding requirement is not mentioned in the CHBDC, it is consistent with AASHTO's specifications.

For a recent AT project, NU girders were designed and fabricated with debonded strand quantities that significantly exceeded the code design limits. This project is a two span bridge (48.5m - 48m) with ten lines of 2.4m deep NU girders. The girders contained 50-15.2 dia. pre-tensioning strands, with 26 of those debonded (52% debonded). All of the debonded pre-tensioning strands were debonded concurrently at 4.5m from each end of the girder. The girders also contained two post-tensioning tendons with 9-15.2 dia. strands each and one post-tensioning tendon with 12-15.2 dia. strands.

AT required that a specialist in prestressed concrete investigate the basis for the debonded strand limits in the CHBDC and propose a rational approach for the design of precast prestressed concrete girders when more than 25% of the pre-tensioned strands are debonded. Once AT was satisfied with the proposed rational design approach, the project designers checked the girders to confirm their capacity and integrity. This paper summarizes the Technical Memo that was submitted presenting the basis for the code limits and the rational design approach.

BASIS FOR DEBONDED STRAND LIMITS IN CODES

The CHBDC contains two main design criteria related to the use of debonded strands:

1. The percentage of debonded strands shall not exceed 25%.
2. The development length of debonded pre-tensioning strands shall be ℓ_d, but shall be doubled to $2\ell_d$ if the girder experiences tension within the development length at the

serviceability limit state. The value for ℓ_d is given in Clause 8.15.4 and is the same for debonded strands as it is for fully bonded strands.

25% Debonded Strand Limit

Shahawy, M., Robinson, B., and Batchelor, B. de V, 1993 conducted a research program involving 33 AASHTO Type II girders. Of the 33 girders tested, 8 contained debonded strands; 2 with 25%, 2 with 27.3%, 2 with 45.5% and 2 with 50% of the strands debonded towards the ends of the girders. Shahawy et al observed that the specimens with 25% and 27.3% debonded strand behaved very similar to those specimen without any debonded strands, while the specimens with 45.5% and 50% debonded strand exhibited shear failures with decreased ductility.

It would appear that the CHBDC adopted the 25% debonded strand limit as a convenient practice that would allow the existing code design provisions for flexure and shear to be easily applied without significant additional and intensive calculations.

Development Length for Debonded Strands

Kaar and Magura, 1965 recognized that standard development lengths may not be adequate for debonded strands located in concrete regions subjected to cracking. They tested five 34 foot long Type III AASHO T-beams. Three were flexural tests subjected to up to 5 million service load cycles prior to testing to ultimate load. Two were shear tests. Within each group there was a control specimen with no debonded strands. They used staggered debonding but had much longer debonded lengths than typically used in current practice. For the specimen with 50% of the strands debonded, the debonded length was up to 31.6% of the span. This specimen only had the regular development length. For the two specimens with 33% of the strands debonded, the debonded length was up to 17.7% of the span. These two specimens had double the regular development length as required for the test loading.

Kaar and Magura found that there was little change in strand stress with cyclic loading, and no detrimental effects of debonding with up to 5 million load cycles. Bond slip generally occurred after cracking and primarily when static loads were increased. They found no detrimental effects on shear due to debonding. Their most important finding was that members with debonded strands and twice the normal development length closely matched the behaviour of members without debonded strands.

Barnes, Burns and Kreger 1999 carried out research involving girders with significantly higher percentages of debonded strands. They present a comprehensive study of 72 tests of AASHTO Type 1 girders with debonded strands percentages varying up to 75%. This research yielded two important observations:

- Even with 75% debonded strands, girder behaviour was acceptable as long as adequate anchorage was provided to resist the longitudinal flexural forces at all locations along the girder.

- If cracking occurs in the girder within 20 strand diameters (20 d_b) of the strand transfer length, general bond slip occurred. The bond slip resulted in changes in the steel stress along the bonded anchorage length and an increase in the development length for the strands.

Furthermore, *Barnes et al.* present two approaches for addressing the design of girders with debonded strands. They referred to these two alternate approaches as Method A and Method B. Figure 2 illustrates these two approaches.

Method A was developed based primarily on the research completed by *Barnes et al.* at the University of Texas at Austin. This approach assumes that strand slip occurs within the strand transfer length. It involves checking to ensure that cracks do not occur within 20 d_b of the strand transfer length. If calculations show that there are no such cracks, normal transfer lengths ℓ_t and development lengths ℓ_d apply. Furthermore, the rules in the CHBDC (and AASHTO) that limit the percentage of debonded strands are unnecessary. Unfortunately, Method A is calculation intensive because it requires the designer to check extreme concrete fibre stresses as well as principle tension stresses in the web. Unless it is clear what part of the web controls, the principle tension stresses need to be computed at the girder centroid, at the junction of the girder web and top flange, and at the junction of the girder web and bottom flange. These additional checks would need to be done using ultimate limit states (ULS) load combinations for all construction and in-service stages of the girder life. The main benefit of Method A is that a shorter development length can be utilized because the calculations have shown that cracking will not occur within the strand transfer lengths.

As depicted in Figure 2, Method B requires a longer development length which accounts for the effects of cracking in the strand transfer length and assumes a different distribution of steel stress along the development length. This approach is required if calculations show that cracking occurs within 20 d_b of the debonded strand transfer length, or if the crack check calculations are not carried out. The main benefit of Method B is that the calculation work is easier.

The CHBDC uses an approach similar to Method B, incorporating a longer development length while not requiring intensive principle tension crack check calculations. Based primarily on the work by *Kaar and Magura*, the CHBDC incorporated the recommendation to double the development length for debonded strands. However, the CHBDC also adopted an exception to this “double development length rule” based on research by *Rabbat, B.G., Kaar, P.H., Russell, H.G., and Bruce, R.N. Jr. 1979*, who “demonstrated that if zero tension were allowed in the concrete at service load, then debonded strands required only one times the development length”. This SLS tension check required by the CHBDC is a simple “P/A + MY/I” calculation and not the intensive principle tension calculations required with Method A. This simple tension check is used to ensure that repetitive service loads will not deteriorate the bond of the pre-tensioning strands.

RATIONAL DESIGN APPROACH

Various studies have investigated the behaviour of girders with higher percentages of debonded strands. *Barnes, Burns and Kreger 1999* present the most comprehensive study and it is the basis for much of the Rational Design Approach presented in this section. What follows is a discussion of the various criteria identified as critical to the behaviour of girders with higher percentages of debonded strands as well as a rational approach to handling these issues that is consistent with current design practices.

Interaction of Flexure and Shear

The force developed at each location along the strand must be adequate to satisfy flexure and shear. The CHBDC uses a version of the modified compression field theory which automatically accounts for the interaction of flexure and shear and resulting tension force requirements in the reinforcement.

Barns et al. 1999 had no shear failures in their 72 tests to failure. There were a small number of bond failures which were precipitated by slippage of the debonded strands when cracks occurred in or near the transfer length. The result was that not all of the strands were fully yielded, and the moment resistance was reduced by up to 7% in the worst case. However, all of the failures were ductile and exhibited significant cracking and deflection prior to failure.

It is worth noting that some of the flexural failures were also accompanied by some strand slippage when the cracks were in or near the transfer zone (say within 20 d_b). This type of slippage will produce large cracks and effectively large longitudinal strains ε_x. In the CHBDC, these larger longitudinal strains can lead to a reduction in V_c and should be accounted for by doubling the computed ε_x.

Considerations When Curtailing Flexural Reinforcement

Barns et al. 1999 recommended that rules for curtailment of non-prestressed reinforcement in flexural tension zones be applied to debonded strands. This recommendation was based on judgement rather than test data. Without experimental evidence the degree of conservatism is not known. Many of the rules for curtailment of reinforcement are no longer explicitly stated in the CHBDC because they are implicitly embedded in the modified compression field code formulation.

Fatigue

The cyclic fatigue tests done by *Kaar and Magura 1965* included two specimens with debonded strands and one specimen without debonded strands. They found that the stress change resulting from 5 million cycles was small for all strands in all of the tested beams. The stress change of 40MPa (5.8 ksi) to 50MPa (7.3 ksi) was considered negligible compared to the effective prestress of 965MPa (140 ksi).

Fatigue tests done by *Russell and Burns 1994* included four tests (DB850-F1A/B, and DB850-F2A/B) with staggered debonding totaling 50% of the strands, and two tests (DB850-F3 and BS850-F4) with concurrent debonding of 50% of the strands. The specimens were initially precracked and subjected to at least one million cycles of load. Periodically, large static loads were applied to simulate large permit loads that were 30% to 60% greater than the normal service load. “Strand slips showed a tendency to stabilize under repeated loading; additional bond slips occurred largely through the application of large static overloads and not as a result of repeated applications of service loads.” The two specimens with concurrent debonding of 50% of the strands were tested with embedment lengths of 1.25 and 1.5 times the normal development lengths. (i.e. they had less than double the normal development lengths.) They concluded that “… fatigue tests demonstrated that the anchorage of debonded strands can be ensured by designing the debonded lengths so that cracking is not likely to intersect the transfer zone of the debonded strands.”

Barns et al 1999, noted that under serviceability limit state (SLS) load combinations the concrete should be uncracked to avoid any possibility of fatigue issues, or deterioration of bond due to cyclic loading.

Checking for zero tension in the extreme concrete fibre under SLS combinations is a conservative approach, and consistent with existing code requirements.

<u>Intermediate Anchorage Considerations</u>

The concentrated introduction of force from a large number of pre-tensioned strands beginning their bonding at the same location along a member is somewhat similar to an intermediate post-tensioning anchorage. The two primary issues are the development of tension ahead of the "anchorage" (ahead of the anchorage refers to the girder area on the support side of the debonding point) and the development of bursting forces at the "anchorage".

If one considers the beam as an elastic isotropic continuum, half of the concentrated force is resisted by compression behind the anchorage. The other half of the concentrated force is resisted by tension ahead of the anchorage. The usual practice with intermediate post-tensioning anchors is to design the bearing plate and concrete compression zone behind it to resist 100% of the anchored force, and to ensure that there is enough longitudinal reinforcement (or prestress) ahead of the anchor to "hold back" a service load equal to 25% of the anchored force (see *Rogowsky and Marti, 1991* for further details). If one does the calculations with ultimate factored resistances, the hold-back force should be 50% of the anchored force as per CHBDC Clause 8.16.5. Note that this 50% force is not a real external tension force. It is merely a convenient method of dealing with potential strain compatibility cracks which might form ahead of the intermediate anchorage. The "50% rule" from the CHBDC is intended for post-tensioning anchors which truly have a high local concentrated force at the anchorage bearing plate. It is undoubtedly a conservative approach for debonded pre-tensioning strands where the force is introduced gradually along the transfer length. The worst case for cracking is immediately after transfer when the concrete has minimum strength and the pre-tensioning strands have maximum force.

It would be prudent and conservative to check the bottom fibre stresses at the beginning of the strand bonding for SLS1 with a 25% hold back force to ensure that there is zero tension under service loads. This would consist of a simple "P/A + My/I" type calculation. The code check for a 50% hold back force should be done with ultimate factored resistances.

Design codes provide rules for bursting reinforcement at the end of a girder, but are silent on bursting reinforcement when a large number of debonded strands transfer their force to the concrete away from the girder end. Again, this is similar to an intermediate post-tensioning anchorage. In the case of post-tensioning, Menn's pragmatic approach is to increase the factored shear force by 20% in the zone in question (*Rogowsky and Marti 1991*). One could investigate this region using strut-and-tie models or finite element analyses. Alternatively, one could apply the same code provisions to debonded strands that are used for bursting reinforcement at girder ends (CHBDC cl. 8.16.3.2). Because of its simplicity, the later approach is recommended.

It should be noted that none of the references consider the bursting stresses when computing principle tension stresses in the web. It is not that bursting stresses do not exist, but that the bursting stresses are not included in the calculation of web cracking shear V_{cw}. This is the standard engineering assumption at supports and it is reasonable to apply this assumption at the debonding point. If one were to include bursting stress effects in the calculation of the principle tension stresses in the web, one would need to formulate a different criteria for limiting the principle tension stress. It is proposed that the conventional calculations and conventional stress limits be used which explicitly ignore bursting stresses. The references did not note any cracking concerns due to bursting stresses.

<u>Staggered Debonding vs Concurrent Debonding</u>

The CHBDC does not specifically address the issue of concurrent versus staggered debonding, although AASHTO does specify that "not more than 40 percent of the debonded strands, or four strands, whichever is greater, shall be debonded at any section".

Russell and Burns 1993, recommend that staggered debonding be used when multiple strands require long debonding lengths. The basis for this requirement seems to stem from *Russell, Burns and ZumBrunnen 1994*, who report on two specimens (DB850-5 and DB850-6) that had 50% of the strands debonded concurrently at 78" from the girder ends and a companion specimen (DB850-4) that had 25% of the strands debonded at 39" and 25% of the strands debonded at 78". They argue that concurrent debonding increases the possibility that cracking will occur within the development length of the debonded strands, while staggered debonding avoids this. This argument is presented conceptually in Figure 3. They argued that cracking would be expected at Point B for the beam with concurrent debonding while not for the beam with staggered debonding. It must be pointed out that the researchers contrived the specimens to make concurrent debonding more critical than staggered debonding when they established the debonding length. This was done in order to test the validity of their "bond failure prediction model". The debonding length which determines the location of Point B was selected to produce cracking as shown. Reducing the length of the concurrent debonding would move the step in the cracking moment (Point B) to the left. With an appropriate length of concurrent debonding, one could produce a test specimen that is less critical than staggered debonding.

The argument presented by *Russell et al* does not conclusively eliminate the use of concurrent debonding. More specifically, it seems to suggest that concurrent debonding could be used effectively as long as proper attention was given to detailing the debonding location. However, when one couples this observation with the effects of introducing large concentrated forces (as discussed previously), there may well be merit in staggering the debonding so as to gradually introduce the loads into the girder.

<u>Load Combinations</u>

All relevant serviceability and ultimate limit state load combinations from CHBDC Table 3.1 must be checked at the various interim and final stages of the girder life. Specifically, load combinations SLS1, ULS1 and ULS2 should be checked. For ULS1, the effects of bearing friction should be included by adding "1.00K" to the load combination, where "K" only includes bearing friction as determined by Clause 11.6.3.7 of the CHBDC. The addition of bearing friction to ULS1 is done out of caution.

For all other load combinations the "K" loads should include all applicable effects at the various stages of construction including bearing friction, positive thermal gradient effects, deck shrinkage effects and the effects of creep redistribution of the girder dead load and pretensioning moments after the girders are made continuous.

When checking stresses in the section for cracking or zero tension, the “P” loads should include the effects of primary and secondary moments due to the prestressing.

DESIGN RECOMMENDATIONS AND ADDITIONAL DESIGN CHECKS

Based on the discussion presented in the previous section, the following design recommendations and additional design checks are proposed for prestressed precast concrete girders containing greater than 25% debonded strands. This summary is by no means recommended for future designs and is presented here primarily for the purpose of further discussion and consideration in future code provision development.

When girders are designed with greater than 25% debonded strands, the following design approach is recommended:

- Avoid cracking in or near the transfer length of the debonded strands;
- Avoid cracking in the zone ahead (on the support side of the debonding location) of the debonded strands;
- Ensure sufficient longitudinal reinforcement is provided for shear capacity;
- Accommodate additional bursting forces associated with the introduction of large prestress forces at the debonding location.

Specifically, the following additional design checks should be carried out.

1. Check for cracking in or near the transfer length of debonded strands:
    a. At SLS, limit the concrete extreme fibre stress to zero tension within 20 d_b of the strand transfer length (ℓ_t).
    b. At ULS, limit the principle concrete tensile stress to be no more than $0.4\sqrt{f'_c}$ at the extreme fibre and no more than $0.33\sqrt{f'_c}$ in the web (checking at the girder centroid, at the web to top flange junction, and at the web to bottom flange junction) within 20 d_b of the strand transfer length.
    c. Indirect control of shear cracks can be done by using the traditional limits for curtailing reinforcement in a flexural tension zone:
        i. V_f should not exceed (2/3) of V_r
        ii. Use additional A_v equivalent to $0.4b_ws/f_y$

2. The existing CHBDC shear capacity provisions already ensure that sufficient longitudinal reinforcement is provided under normal conditions. However, if it is found that the cracking check in 1(b) above is not satisfied, some additional bond slip will occur. This can be accounted for by increasing ε_x by a factor of two when computing the shear resistance anchorage at any location within 20 d_b of the transfer length.

3. Check “hold-back” force effects at the start of the bonding of the strands.
    a. Check SLS1 to see that there is zero tension at the beginning of the bonded section with a “hold-back” force equal to 25% of the force being introduced at the beginning of the bond.

b. A ULS check to ensure that there is sufficient bonded reinforcement, or pre-compression to resist 50% of the force in the debonded strands.

c. If either of these checks can not be met, additional bonded reinforcement can be provided. This could be accomplished by providing additional non-prestressed reinforcement or by moving the strand debonding location.

4. To accommodate the additional bursting forces at debonding locations, one can conservatively add transverse (shear reinforcement) to resist 8% of the force in the strands being anchored at the debonding point. This reinforcement should be placed over a distance of H/4 starting from the beginning of bond. The 0.08 F_{pu} is a factored force.

All of the above checks must be done at the various interim and final stages of the girder life. In addition, all relevant serviceability and ultimate limit state load combinations from the CHBDC must be checked at each interim stage.

CONCLUSION

The use of debonded strands in precast girders has several benefits, including reduction of pre-tensioning stresses at girder ends, reduction of girder end zone web cracking and increased fabrication economy and safety realized by eliminating the use of deflected strands. Various jurisdictions successfully utilize significant amounts of debonded strands with specific girder types. Further testing and code development may well allow designers in all jurisdictions to regularly use greater amounts of debonded strand in girder designs. However, until this work is completed, it is maintained that excessive amounts of debonded strands should be avoided. This is primarily due to the large number of questions that still exist and the calculation intensive work that is required to validate the design. Furthermore, excessive debonding can often be avoided simply through the use of temporary or permanent pre-tensioning strands in the top girder flange and thorough detailing of girder end zone reinforcing.

It is the authors' opinion that the Rational Design Approach presented above provides a safe and acceptably conservative approach for assessing girder capacity when more than 25% of the pre-tensioning strands are debonded. However, due to the significant additional design work required and the lack of local experience using this approach, AT is not comfortable giving a blanket endorsement to this Rational Design Approach for future projects. For the foreseeable future, AT will continue to require consultants to explicitly meet the debonded pre-tensioning strand provisions contained in CHBDC clause 8.15.4.

Future Development

It is recommended that researchers and code committees review and consider the following concerns for future development:

- It is unclear as to whether the existing 25% limit should apply only to the number of pre-tensioned strands or to the combined number of pre-tensioned and post-tensioned strands in a girder?

- The current provisions specify that tension should be checked within the development length (ℓ_d) rather than within 20 d_b of the transfer length (ℓ_t) as stated in the literature.

- The current provisions specify that tension should be checked at SLS. Based on the literature research it is understood that this is intended as a simple "P/A + My/I" check and not a principle tension check. It would be worthwhile to expand the explanation of this check in the commentary.

- It appears that the "double the development length ($2\ell_d$)" provision in CHBDC results in a significantly longer development length than obtained using the appropriate provisions in AASHTO.

- The additional debonded strand requirements in AASHTO (debonded strand pattern and location) that are not included in the CHBDC.

- Is it reasonable to add code provisions for girder designs with greater than 25% debonded strands. This may require more extensive calculations but could allow for more economical designs.

FIGURES

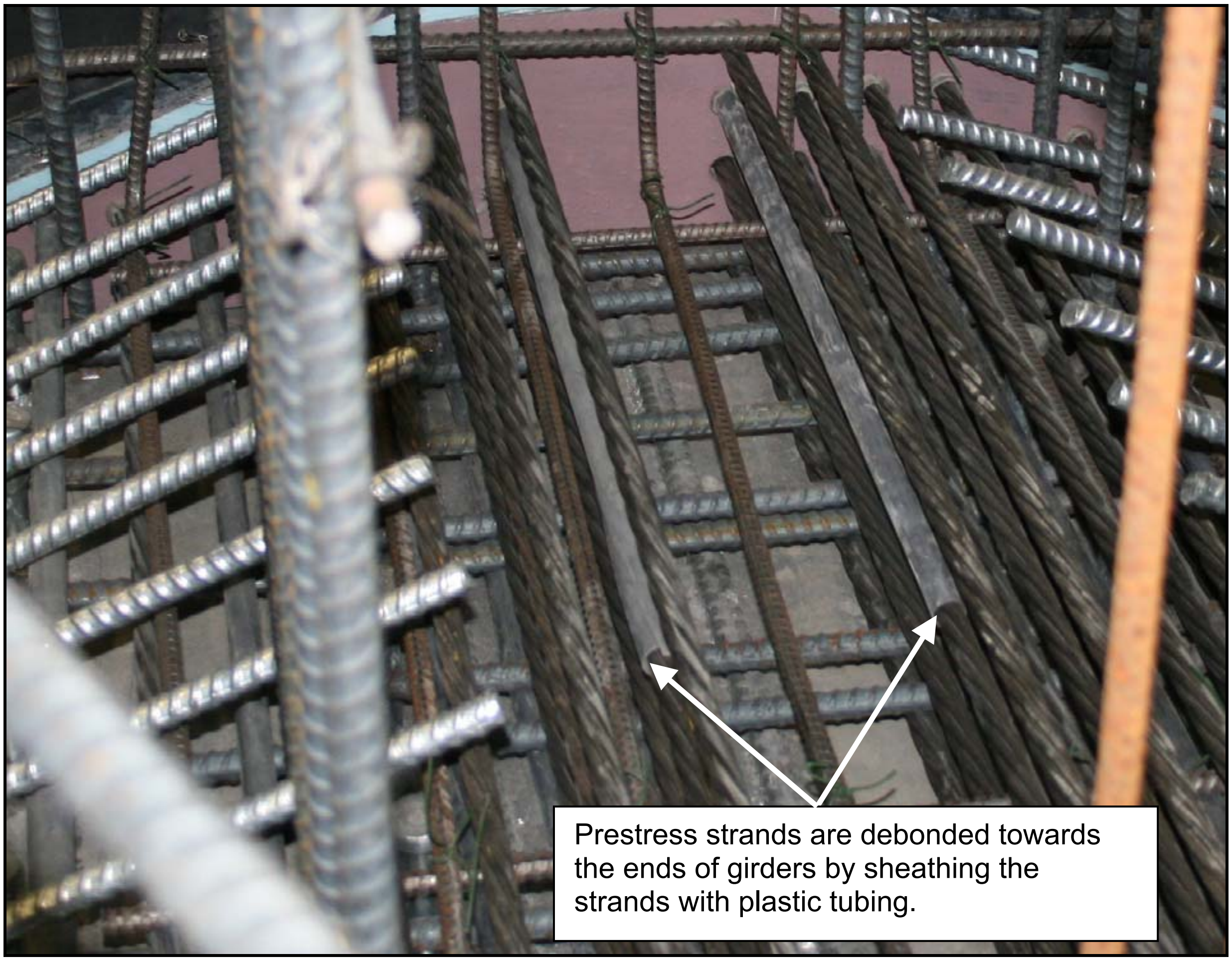
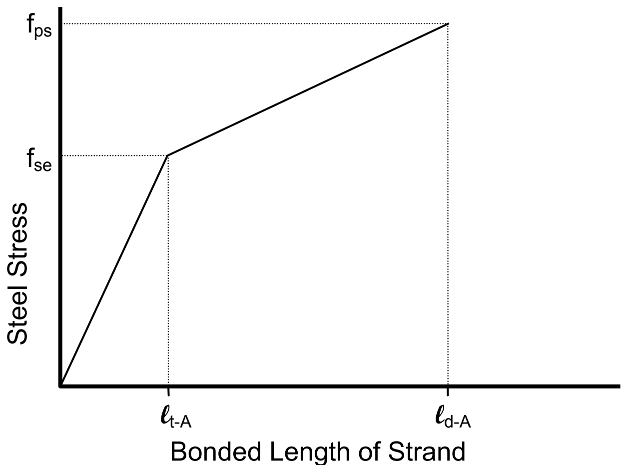
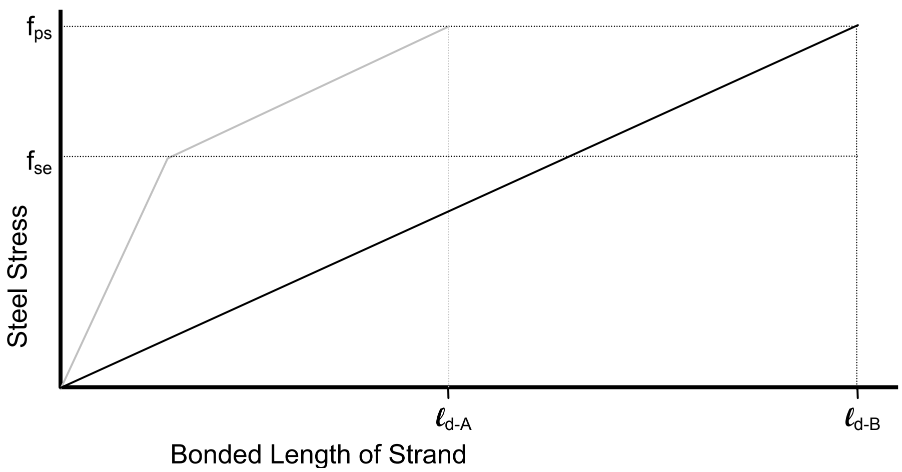


Figure 1. Debonded pre-tensioning strands



Method A: If no cracking occurs within 20 d_b of ℓ_t, the transfer length is not degraded and a shorter development length may be used.



Method B: If cracking occurs within 20d_b of ℓ_t, or if a cracking check is not done, a degraded transfer length is assumed and a longer development length shall be used.

Figure 2. Comparison of development lengths (modified from Figures 2.17 and 2.18 in Barnes, Burns and Kreger, 1999)

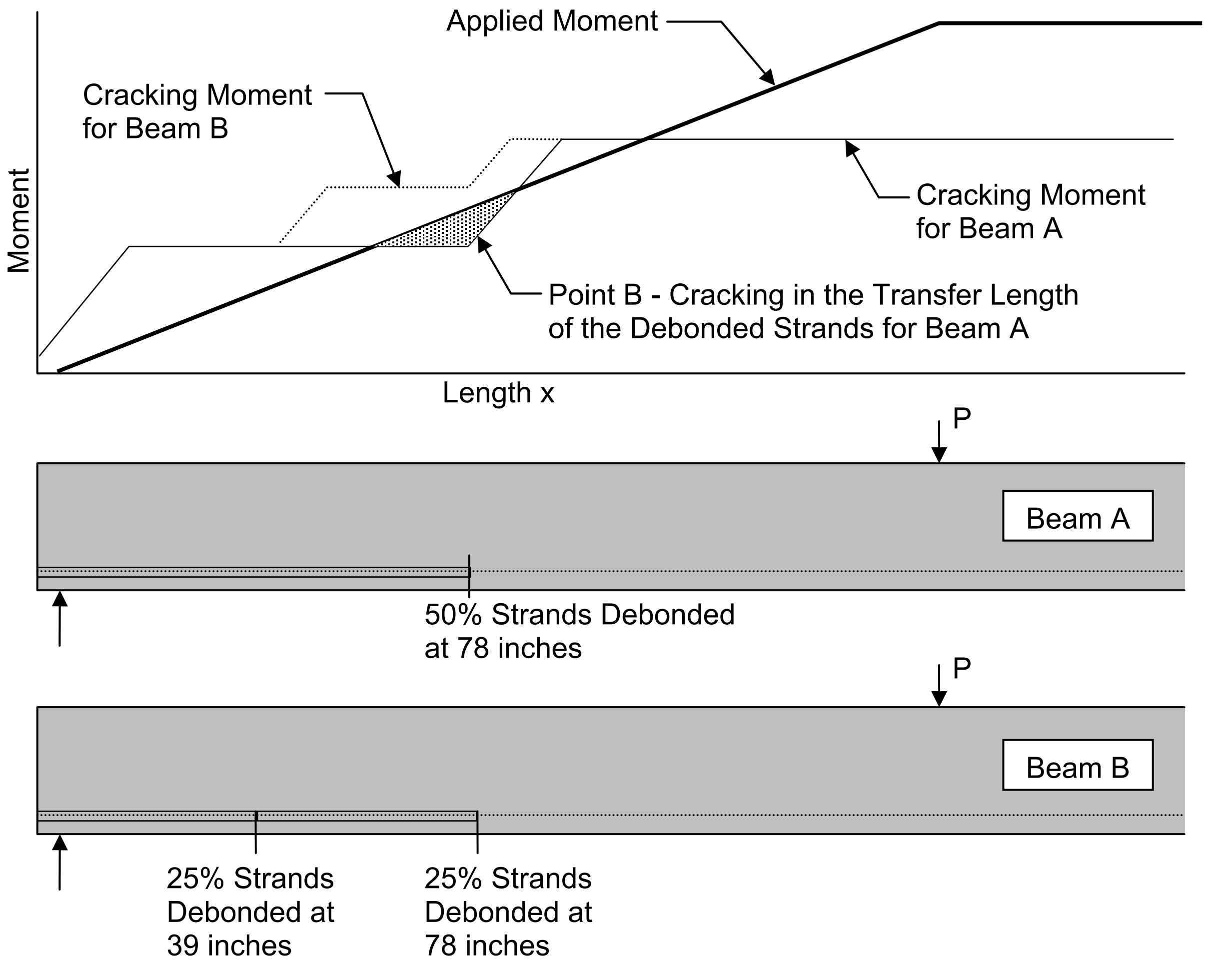


Figure 3. Conceptual Comparison of a Beam with Concurrent Debonding to a Beam with Staggered Debonding (modified from Figs. 2 and 3 in Russel, Burns and ZumBrunnen, 1994)

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