Strength Design of Pretensioned Flexural Concrete Members at Prestress Transfer*

by Panya Noppakunwijai, Maher K. Tadros, Zhongguo (John) Ma, and Robert F. Mast

Comments by Stephen J. Seguirant and Authors

STEPHEN J. SEGUIRANT†

The authors are to be commended for presenting a rational design approach to a critical issue that is impeding the advancement of the industry. However, as a reviewer of the original draft, I am still concerned with certain concepts presented in the published paper:

1. I am troubled by the premise that draped or harped strands are somehow less desirable than straight strands. The point that "this operation is not as safe as keeping all the strands straight" may be marginally true, but the same could be said of using straight pretensioned strands in lieu of mild steel reinforcement. Given proper plant limitations and procedures, harping is done safely on a routine basis. Of course, harped strands are not practical for all types of pretensioned flexural members. Where they are viable, however, limiting span capability because straight strands cause high concrete stresses at release near the ends is entirely avoidable, even under the current allowable stress methods. For reasons stated in the paper, choosing to debond straight strands in lieu of draping strands is much less desirable. I certainly would not want to discourage design engineers from using harped strands if they felt it resulted in a better design.

I also fail to see why the distinction was made between holding the strands down in a bridge beam versus pushing them down in a double tee. If the implication is that, in a double tee, the "vertical hole...becomes vulnerable to moisture and salt penetration which can cause premature corrosion of the strands," then I believe the premise is flawed. To my knowledge, only one occurrence of failure of harped strands in double tees due to corrosion has been documented. The alleged cause of this failure was improper patching materials, not a vulnerability created by a properly patched hole.

I only raise these issues because I feel the primary reason for seeking a change to the release strength requirements is being obscured. As described in the "Historical Background" section, the current code requirements are arbitrary at best, and are not based on science or research. And even when the best technology in pretensioned concrete design is used, these arbitrary provisions are limiting the span capabilities of pretensioned concrete flexural members.

2. Throughout the text, it is noted that the prestress force to be applied to the "column" is that just before transfer. In an externally loaded reinforced concrete column, the load does not diminish as the member shortens. In a pretensioned member, it does. The definition of P_i given on page 37 is the "prestress force immediately after prestress release." It seems that this is the appropriate value to use in the analysis.

3. The discussion of the 0.8 and 1.2 load factors, which are applied to M_g, is very confusing. The critical stresses at release do not occur immediately after release, when the member is still sitting in the form, but when the member is physically lifted from the form. Invariably, the lifting locations will be away from the ends towards midspan, sometimes substantially to enhance the lateral stability of the

* PCI JOURNAL, V. 46, No. 1, January-February 2001, pp. 34-52.

† Director of Engineering, Concrete Technology Corporation, Tacoma, Washington.

beam. It is difficult to envision any normal scenario where the lifting points would be beyond the member ends. The resulting self-weight moments would then be negative at the lifting locations (inducing tensile stresses in the top and compressive stresses in the bottom), and the positive moments in the midspan region would be reduced.

The stresses summarized in Table 1 do not reflect the stresses in a lifted beam. If the lifting locations were at the transfer length (25 in. from the end), the static stresses at the transfer section would be $f_t = -914$ psi and $f_b = 2832$ psi. At midspan, the static stresses would be $f_t = -758$ psi and $f_b = 2676$ psi. Under the current allowable stress methods, these are the stresses that should be used to determine the required concrete release strengths.

Assuming the lifting devices are at least a transfer length away from the ends, the critical section for beams with straight strands will be at the lifting locations. For beams with harped strands, the critical section will either be at the lifting location or at the harp point.

For the strength design method, load factors typically reflect uncertainty surrounding the type of load under consideration. For the self-weight of a precast beam, these considerations include tolerances on the dimensions of the cross section, tolerances on the unit weight of the concrete/steel arrangement, tolerances on the placement of the lifting devices, and potential impact during handling — essentially, variables that can make the beam heavier or lighter than anticipated. The load factors should be applied in combinations that produce the worst condition. For checking the condition at the lifting locations, a load factor of 1.2 should be applied to the calculated negative moment, since its effects are in the same direction as the effects of the prestressing force (creating tension in the top and compression in the bottom). In the positive moment region, a load factor of 0.8 should be applied to the calculated moment, since it works to counteract the effects of prestress. In Eq. (4), the numerator on the right-hand side of the equation should be either $0.8M_g$ or $1.2M_g$, depending on the section under consideration.

4. Assuming the assumptions stated in Items 2 and 3 are correct, the factored loads for the 16RB40 described in "Comparison with Working Stress Design Results" would be:

$$1.2P_i = P_u = 1.2(22)(202.5 - 20.25)(0.153) = 736 \text{ kips}$$

$$M_g = \frac{wa^2}{2} = -0.667(2.083)^2(12)/2 = -17 \text{ in.-kips}$$

$$M_u = P_u e - 1.2M_g = [736(13) - 1.2(-17)]/12 = 799 \text{ ft-kips}$$

This compares to $P_u = 818$ kips and $M_u = 870$ ft-kips as plotted at point +1 in Fig. 3 of the paper.

5. The section "Control of Top Fiber Cracking" concludes that "...there is no need to perform a service load cracked section analysis or to check for crack control for steel grades not greater than Grade 60." It does seem appropriate, however, to ensure that the flexural tension reinforcement is well distributed in the top flange in accordance with Section 10.6 of ACI 318-99.

6. Appendix C indicates that "an approximate formula was developed to determine the K value at the end section of the pretensioned member," ignoring the self-weight moment. Is it appropriate to ignore the self-weight moment if it is in fact negative and adds to the effects of prestress? What if the critical section is 15 ft (4.58 m) from the ends at the lifting locations, a configuration that can very easily occur with large bridge girders? Does the formula apply to critical sections in the midspan region? One of the advantages of the strength design method is that, for a given concrete release strength, there is an associated amount of top tension reinforcement required. What is the required top tension reinforcement associated with the concrete release strengths calculated according to the approximate formula? Should this be calculated according to current code requirements, and, if so, how does it correlate with the reinforcement required by the strength design method?

In my opinion, there seem to be enough questions associated with the approximate formula that it should be abandoned. I was hoping that the research would indicate a simple single value of K, somewhat higher than 0.6, that would be applicable to all conditions. This does not appear to be the case. If the designer does not wish to use the strength design method, then the current rules should apply.

AUTHORS' CLOSURE by PANYA NOPPAKUNWIJAI,* MAHER K. TADROS,† ZHONGGUO (JOHN) MA,‡ and ROBERT F. MAST§

The authors are grateful to Mr. Seguirant for his contributions as a member of the advisory committee of the research project culminating in the publication of this paper. His extensive effort was a valuable contribution to this important topic. The authors will respond to Mr. Seguirant's comments in the order they were presented by him.

1. The authors agree with Mr. Seguirant that, with appropriate quality control, draped strands can be used on a routine basis. Most producers of bridge products have plant capabilities to harp strands. Strand harping, however, requires more work than keeping all strands straight. In some situations, whether they are due to plant limitations or to cross section shape limitations, the option of strand harping may not be available.

The authors thank Mr. Seguirant for pointing out that proper patching materials and methods for strands harped by being pushed down from the top would assure strand protec-

* Research Assistant Professor, Civil Engineering Department, University of Nebraska-Lincoln, Omaha, Nebraska.

† Cheryl Prewett Professor, Civil Engineering Department, University of Nebraska-Lincoln, Omaha, Nebraska.

‡ Assistant Professor, Civil and Environmental Engineering Department, University of Alaska-Fairbanks, Fairbanks, Alaska.

§ Senior Principal, BERGER/ABAM Engineers Inc., Federal Way, Washington.

tion against corrosion. The authors are merely indicating that avoidance of strand harping would result in fewer production steps.

The authors do not endorse elimination of strand harping as common practice in pretensioned member production. Harped strands, as opposed to debonded strands, preserve the level of prestressing and induce a vertical prestress component that aids in shear resistance. The authors believe, however, that excessive harping or debonding may be avoided when the proposed rational design procedure is adopted.

As Mr. Seguirant indicated, the primary goal of the proposed procedure is to offer a rational approach of member design for prestress transfer effects. The past half-century has proved without a doubt that prestressed concrete is a superior material to non-prestressed concrete in most structural applications. It is time to remove some of the unnecessary conservatism created when this material was introduced.

2. The authors disagree with Mr. Seguirant. Actually, the definition of P_i on page 37 is incorrect. The word "after" should be replaced with the word "before." The force considered to be applied to a "reinforced" concrete member is the force just before release. When that force is released from the prestressing bed to the "reinforced concrete" member, which consists of a concrete component that is assumed to be perfectly bonded to a steel component (the strands), part of the force is resisted by the concrete component and the other part by the steel component. In unfactored load analysis, the component of the force resisted by the steel is called elastic shortening loss, and the component resisted by the concrete is used to check concrete stresses. Elastic shortening loss can be determined by applying P_i to the transformed section or applying P_o to the concrete component only. P_o is the prestress just after release.

When a factored load is introduced to check the capacity of concrete against crushing, that force should be related to P_i and the "transformed" section should be used in the analysis in order to maintain consistency with unfactored load elastic analysis. In this case, however, one cannot use the modulus of elasticity or modular ratio to account for the presence of steel as the cross section nears its ultimate strength. One has to use the basic strain compatibility relationship that is automatically used in strength calculations. In summary, one cannot account for elastic loss twice, once by using P_o, and a second time through strain compatibility calculations.

3. Mr. Seguirant makes an important point. Gravity load moment that is in the same direction as the moment due to prestress at the section being considered should be assigned a load factor of 1.2. This situation generally exists at lifting points. Moments that act opposite to the pretensioning force moment should be assigned a load factor of 0.8. This situation occurs at drape points and at midspan. The coefficient 0.8 in Eq. (4) of the paper applies only to the latter case. To make the equation less confusing, the expression "or $1.2M_g/\phi$, whichever controls" should be added at its end. The example represented by Table 1 is consistent with the presentation in the PCI Design Handbook for building members. It reflects the condition immediately after prestress release but not necessarily the most critical condition. Mr. Seguirant presents an interesting and important observation. If the lifting device is at a significant distance from the member end, the section at that location may be the most critical one for analysis for prestress release. The lifting point location is sometimes considered by bridge designers but almost never considered by building designers. Perhaps in the future, lifting point location should be included as an integral part of the initial structural design of all precast pretensioned members.

4. The values of P_u and M_u used in the paper are 818 kips and 870 ft-kips. The first figure is 120 percent of the prestress using a strand stress of $0.75f_{pu}$ without elastic shortening loss allowance. $1.2P_i = 1.2(22)(202.5)(0.153) = 818$ kips. The second figure is consistent with the assumptions given for working stress design (Table 1, and the PCI Handbook). Thus, $M_g = wab/2 = 0.667(2.083)(30 - 2.083)(12)/2 = 233$ in.-kips, and $M_u = P_u e - 0.8M_g = [818(13) - 0.8(233)]/12 = 870$ ft-kips. As indicated earlier, the value of M_u to be used in the analysis would have to be calculated for all critical sections under all critical loading combinations. We agree with Mr. Seguirant's calculation of M_u if one assumes that the critical section is at the lifting point and that the lifting point is 2.083 ft away from the member end. Besides consideration of the "loading" case immediately after transfer of prestress, where the product is assumed to be cambered and thus supported only at its ends, designers should be aware of all the other loading cases and check all critical sections accordingly. In slender long members where the lifting points are intentionally placed well inside the member to control instability due to lifting and handling, analysis for lifting conditions should be considered by the designer in cooperation with the producer. In this situation, the concrete strength at lifting may be different from that at transfer.

5. We agree that crack control should be checked according to Section 10.6 of ACI 318-99. It should be noted that the 60-ksi stress at ultimate was found to correspond to 22 ksi at service, i.e., due to unfactored loads. This value, in turn, corresponds to a bar spacing requirement of 20 in. when 2 in. cover is assumed. It may be convenient to the designer to indicate a maximum spacing of the bonded tension reinforcement, if required, of 20 in. Studies in Britain[15] have indicated that eccentrically loaded columns had controlled cracking on the tension side near ultimate regardless of the amount of reinforcement on that side, because crack width is controlled by crack depth.

The main factor influencing crack depth in a compression member is the applied load. On the other hand, the main factor influencing crack depth in a non-prestressed flexural member is the tension reinforcement. There are situations in current practice of prestressed concrete beam design where the depth of the tension zone is so small that code-required bonded tension reinforcement could not be placed close enough to the tension face to fall within the tension zone. If the tension zone is so shallow, the crack widths could not possibly become large, irrespective of the amount of reinforcement provided.

6. Part of the response to this item has already been covered earlier. The proposed approximate formula was developed for sections at the ends of various common precast concrete section shapes. The authors found that sections subjected to relatively large positive self-weight moments, e.g., midspan sections, had significantly different approximate K values than those subjected to prestress alone or to prestress with negative self-weight bending moments. Since it was felt that relief from the current 0.6 value was mostly needed at member ends, the authors decided to focus on relevant loading conditions. A designer who wishes to use the proposed equivalent K values would obviously have to determine the stresses at the appropriate section location using the appropriate loading combination. It is unfortunate, that the authors were not able to recommend a universally applicable K formula. It is still a significant step forward to realize that one can use a limit of $0.75f'_{ci}$ at the ends of double tee members, and $0.70f'_{ci}$ at the ends of rectangular members.

As Mr. Seguirant correctly pointed out, using the strength design method automatically satisfies the requirement on both the tension and compression sides of the cross section being considered. With working stress design, whether a constant $K = 0.6$ or the proposed K formula is used, the designer is still faced with the dilemma of resorting to the current empirical code provisions for calculation of the required bonded tension reinforcement.

PCI JOURNAL readers might be interested to know that the authors have developed an Excel spreadsheet that significantly simplifies application of the proposed strength design method. The program is simple to use. It is applicable to flanged as well as rectangular sections. The program output is the required minimum compressive strength for a designer-selected area of top bonded reinforcement. Application of the program will be presented in a future issue of the PCI JOURNAL. Electronic copies of the program itself will be made available from PCI.

As mentioned in the paper, the 1.2 load factor applied to prestress and to negative moment, the 0.8 factor applied to positive moment and the 0.7 strength reduction factor may be too conservative. Additional investigation by the authors after publication of the paper has indicated that these factors may be changed to 1.15, 0.85 and 0.75, respectively, without compromising acceptable safety margins. This issue will also be discussed in more detail in a future issue of the JOURNAL.

ADDITIONAL REFERENCE

15. Beeby, A.W., "The Prediction of Crack Widths in Hardened Concrete," *The Structural Engineer* (London), V. 57A, No. 1, January 1979, pp. 9-17.