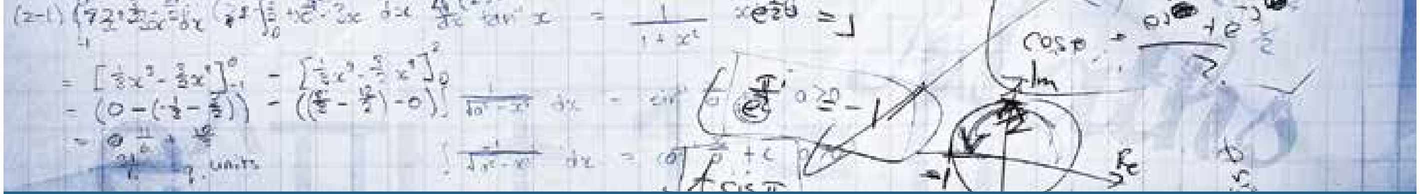

Discussion

>>

PCI Design Handbook Appendix A: Blast-resistant design of precast, prestressed concrete components

This report is very well written and I want to extend my thanks to the PCI Blast Resistance and Structural Integrity Committee. I would appreciate if you can answer a question that I have. This question is related to Eq. (A.3).

For a multi–degree of freedom system (MDOFS), the dynamic equation could be reduced to that of a single degree of freedom system (SDOFS) to determine modal-specific responses by converting the mass, stiffness, and damping matrices of the structure to values effective to a given mode of vibration. To achieve that, each of the mass, damping, and stiffness matrices are pre- and post-multiplied by transpose and column modal shape factors, respectively, for a given mode of vibration. Factor K_{LM} in Eq. (A.3) of the report appears to have been used to determine effective mass, where this factor could be determined by pre- and post-multiplying the mass matrix with the deflected shape factor of first mode of vibration. However, the term stiffness $R(y(t))$ in that equation is not multiplied by any of the deflected shape factors (to determine the stiffness factor). Can you explain why this has not been done? Can the equation produce an incorrect result if the stiffness is not multiplied by the deflected shape factors?

Mehedi Rashid
Senior Structural Engineer, Moffatt & Nichol
Norfolk, Va.

Reference

1. Naito, Clay, Chuck Oswald, and PCI Blast Resistance and Structural Integrity Committee. 2014. “PCI Design Handbook: Appendix A: Blast-Resistant Design of Precast, Prestressed Concrete Components.” *PCI Journal* 59 (1): 137–159.

Authors’ response

The committee appreciates the question. The K_{LM} factor is appropriately applied in accordance with dynamic concepts discussed in *Introduction to Structural Dynamics* by J. M. Biggs.[1] Please refer to the textbook for further information on the approach. Regarding the modes for consideration, in most applications only the first mode is considered. This again is discussed in detail in Biggs, which is included as reference 11 in appendix A.

PCI Blast Resistance and Structural Integrity Committee

Reference

1. Biggs, J. M. 1964. *Introduction to Structural Dynamics*. New York, NY: McGraw-Hill Book Co.

Evaluation of common design policies for precast, prestressed concrete I-girder bridges

The following comments relate to "Evaluation of Common Design Policies for Precast, Prestressed Concrete I Girder Bridges," by Richard Brice, Stephen J. Seguirant, and Bijan Khaleghi, which appeared in the Fall 2013 issue of *PCI Journal*.[1]

The authors present an excellent study of interest to bridge designers and owners. The paper details the benefits and disadvantages of commonly adopted design policies despite their being more stringent than the *AASHTO LRFD Bridge Design Specifications*.[2] Policies to combine design with gross or transformed section properties, reduced allowable tensile stress, and simple-span moments for superimposed dead and live loads are included, and their sensitivity in the precast, prestressed concrete bridge girder design is analyzed.

I would like to thank the authors, who should be complimented for producing a paper with valuable information, and *PCI Journal* for the opportunity of offering the following comments and questions, mainly about some of the assumptions and simplifications used in the analysis and prestress losses.

In particular, all of the strands have been considered to be at the centroid of the effective prestress force for computing transformed section properties, and the centroid of effective prestress force is located at 5 in. (130 mm) above the bottom of the girder. However, some analyses have been based on various prestress values by varying the number of strands (Fig. 3 and 5) about the influence of adopted policies on span capability. As it is possible to design different strand distributions for the same concrete cross section, did the authors use any algorithm to consider strand distributions with the same centroid in all cases? Transformed section properties can vary along the girder length according to changes in strand number and/or position, mainly by harping and/or debonding the prestressing strands in girder end zones. Did the authors use harped strands and/or debonded strand lengths in their analyses? In addition, the debonded length of strands increases transfer length, defined as the distance over which the strand should be bonded to the concrete to develop the effective stress in the prestressing steel,[3] and the effective stress is completely transferred to the concrete when concrete stresses are assumed to have a linear distribution, which occurs beyond the dispersion length.[4] Transfer length depends on several factors,[5] such as strand stress and diameter, release method, and concrete properties,[6,7] while transfer length provisions differ according to distinct codes and researchers.[8,9] Furthermore, the dispersion length depends on girder depth, the position of the strands, and debonding, among other factors, and it is longer than transfer length. Once again, the assumptions related to the centroid of the effective prestress force strongly influence the calculations. Finally, transformed section properties vary over time because of the concrete modulus of elasticity. It is noticed that an early concrete modulus of elasticity at prestress transfer can be obtained based on prestress loss due to elastic concrete shortening and transformed cross-section properties.[10,11]

On the other hand, the *AASHTO LRFD Bridge Design Specifications*[2] provide refined and approximate methods for estimating prestress losses. The differences between the two methods can be considerable, as stated for the cases of prestress losses due to concrete creep and shrinkage.[12] To clarify this point, can the authors explain how the approximate method was calibrated and correlated with the refined method?

José R. Martí-Vargas

Professor, Institute of Concrete Science and Technology, Universitat Politècnica de València

Valencia, Spain

References

1. Brice, Richard, Stephen J. Seguirant, and Bijan Khaleghi. 2013. “Evaluation of Common Design Policies for Precast, Prestressed Concrete I-Girder Bridges.” *PCI Journal* 58 (3): 68–80.
2. AASHTO (American Association of State Highway and Transportation Officials). 2012. *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington, DC: AASHTO.
3. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*. Farmington Hills, MI: ACI.
4. CEN (Comité Européen de Normalisation). 2004. *Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings.* European standard EN 1992-1-1:2004:E. Brussels, Belgium: CEN.
5. Martí-Vargas, J. R., F. J. Ferri, and V. Yepes. 2013. “Prediction of the Transfer Length of Prestressing Strands with Neural Networks.” *Computers and Concrete* 12 (2): 187–209.
6. Martí-Vargas, J. R., W. M. Hale, E. García-Taengua, and P. Serna. 2014. “Slip Distribution Model along the Anchorage Length of Prestressing Strands.” *Engineering Structures* 59: 674–685.
7. Martí-Vargas, J. R., E. García-Taengua, and P. Serna, P. 2013. “Influence of Concrete Composition on Anchorage Bond Behavior of Prestressing Reinforcement.” *Construction and Building Materials* 48: 1156–1164.
8. Martí-Vargas, J. R., P. Serna, J. Navarro-Gregori, and L. Pallarés. 2012. “Bond of 13 mm Prestressing Steel Strands in Pretensioned Concrete Members.” *Engineering Structures* 41: 403–412.
9. Martí-Vargas, J. R., and W. M. Hale. 2013. “Predicting Strand Transfer Length in Pretensioned Concrete: Eurocode versus North American Practice.” *ASCE Journal of Bridge Engineering* 18 (12): 1270–1280.
10. Martí-Vargas, J. R., L. A. Caro, and P. Serna-Ros. 2014. “Size Effect on Strand Bond and Concrete Strains at Prestress Transfer.” *ACI Structural Journal* 111 (2): 419–429.
11. Martí-Vargas, J. R., E. García-Taengua, L. A. Caro, and P. Serna. 2014. “Measuring Specific Parameters in Pretensioned Concrete Members Using a Single Testing Technique.” *Measurement* 49: 421–432.
12. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2013. “Prestress Losses Evaluation in Prestressed Concrete Prismatic Specimens.” *Engineering Structures* 48: 704–715.

Authors’ response

The authors would like to thank José Martí-Vargas for his questions, comments, and discussion. Our paper reported the results of a parametric study performed to examine the influence that common design policies have on precast, prestressed concrete I girder bridges.[1] First, *baseline* designs were established in accordance with the minimum requirements of the *AASHTO LRFD Bridge Design Specifications*.[2] Then more conservative policies were examined individually, with all other parameters held constant to compare the relative sensitivity of the more conservative assumptions with the baseline designs.

One parameter that was held constant throughout the study was the location of the effective prestress force transferred to the concrete section. Martí-Vargas asks whether an algorithm was used to consider strand distributions with the same centroid in all cases. In typical Washington State Department of Transportation (WSDOT) girders, the location of the effective prestress force and distribution of strands varies with the number of strands. However, the authors thought that incorporating such an algorithm into the study would skew the comparative results for the specific parameter under evaluation. Thus, no distribution algorithm was used in the study and all strands were assumed to be lumped at 5 in. (130 mm) above the bottom of the girder. This would have only a small effect on the calculation of transformed section properties because the actual distribution of strands is still entirely within the confines of the bottom flange at the sections considered in the study. Transformed section properties were calculated on the basis of the specified concrete strength in service, as early-age strengths were not relevant to the study.

Martí-Vargas asks whether harped strands and/or debonded strand lengths were used in the analyses. This study did not investigate sections where harped or debonded strands might alter the magnitude or location of the effective prestress force. The designs were assumed to be governed by the service III limit state, which is the case for the majority of prestressed I girder bridges. The critical design section was assumed to be at or near midspan for the bridge configurations studied. Strands are typically harped or debonded near the ends of girders to control concrete stresses at the time of prestress transfer. This was not a governing case in this study.

Last, Martí-Vargas asks for an explanation of how the approximate method for estimating prestress losses was calibrated and correlated with the refined method. The approximate method for estimating prestress losses was mathematically derived from the refined method as detailed in the National Cooperative Highway Research Project (NCHRP) report 496.[3] Typical values for geometric properties of girders, ratio of creep occurring before and after deck placement, and proportions of dead, superimposed dead, and live loads for slab-on-girder bridges were substituted into the equations for the refined method and simplified to yield the equations for the approximate method. Table 22 of NCHRP report 496 shows measured versus estimated total prestress losses for previously reported experiments. The table includes estimated total prestress losses computed with the approximate and refined methods. The average ratio of the estimated loss to the loss measured during experimental investigations is reported to be 1.08 and 1.00 for the approximate and refined methods, respectively. Given the high degree of correlation between estimated and measured losses for both the approximate and refined methods, the authors concluded that the approximate method was sufficiently accurate for this study.

Richard Brice
Bridge software engineer, Washington State Department of Transportation
Olympia, Wash.

Stephen J. Seguirant
Vice president and director of engineering, Concrete Technology Corp.
Tacoma, Wash.

Bijan Khaleghi
State bridge design engineer, Washington State Department of Transportation
Olympia, Wash.

References

1. Brice, Richard, Stephen J. Seguirant, and Bijan Khaleghi. 2013. "Evaluation of Common Design Policies for Precast, Prestressed Concrete I-Girder Bridges." *PCI Journal* 58 (3): 68–80.
2. AASHTO (American Association of State Highway and Transportation Officials). 2012. *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington, DC: AASHTO.
3. Tadros, M. K., N. Al-Omaishi, S. J. Seguirant, and J. G. Gallt. 2003. "Prestress Losses in Pretensioned High Strength Concrete Bridge Girders." NCHRP (National Cooperative Highway Research Project) report 496. Washington, DC: TRB (Transportation Research Board).

Reliability-based sensitivity analysis for prestressed concrete girder bridges

The following comments relate to "Reliability-Based Sensitivity Analysis for Prestressed Concrete Girder Bridges" by Anna M. Rakoczy and Andrzej S. Nowak.[1]

The authors have made a remarkable contribution and they should be congratulated for producing an excellent paper, which presents the results of a reliability analysis for prestressed concrete girders using the most recent live load and resistance models. Important parameters affecting reliability indices have been identified by sensitivity analysis, including functions for various parameters, such as loads, material strengths, and section geometry characteristics.

The discusser would like to thank the authors for the detailed information contained in the paper and *PCI Journal* for the opportunity to offer the following comments and questions with the intention of pointing out some aspects included in the paper.

The discusser has not found a detailed definition of effective depth d in the paper. According to ACI 318-11,[2] effective depth is the distance measured from extreme compression fiber to the centroid of longitudinal tension reinforcement. As distances d_p, d_s, and d'_s are included in the notation section in a detailed manner, can the authors clarify this fact or any redundancy to offer a better understanding of the definition of effective depth d? The results of the sensitivity analysis indicate that the reliability index strongly depends on effective depth d and prestressing steel $f_{ps}A_{ps}$, as observed in Fig. 6, 7, and 8. In these figures, the ratio of the girder reliability index is based on changes (in percentage from the nominal value) in the analyzed parameters. Prestressing steel is involved in important topics such as transfer length (at release[3] and with time[4]), development length,[5] dispersion length,[6] camber,[7] and prestress losses.[8] Because changes in the area of prestressing steel and/or the strand positions can affect these topics, can the authors report some details on how the changes of prestressing steel from the nominal value were made and their implications on the aforementioned topics?

Besides, although both material properties and bridge loads have changed, as stated by the authors, ACI 318 code provisions on transfer length first appeared in 1963 and remain to date despite a considerable number of proposed modifications[9] and analyzed factors.[10]

José R. Martí-Vargas
Professor, Institute of Concrete Science and Technology, Universitat Politècnica de València
Valencia, Spain

References

1. Rakoczy, Anna M., and Andrzej S. Nowak. 2013. "Reliability-Based Sensitivity Analysis for Prestressed Concrete Girder Bridges." *PCI Journal* 58 (4): 81–92.
2. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*. Farmington Hills, MI: ACI.
3. Martí-Vargas, J. R., W. M. Hale, E. García-Taengua, and P. Serna. 2014. "Slip Distribution Model along the Anchorage Length of Prestressing Strands." *Engineering Structures* 59: 674–685.
4. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2012. "Time-Dependent Evolution of Strand Transfer Length in Pretensioned Prestressed Concrete Members." *Mechanics of Time-Dependent Materials* 17 (4): 501–527.
5. Martí-Vargas, J. R., E. García-Taengua, and P. Serna. 2013. "Influence of Concrete Composition on Anchorage Bond Behavior of Prestressing Reinforcement." *Construction and Building Materials* 48: 1156–1164.
6. CEN (Comité Européen de Normalisation). 2004. *Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings.* European standard EN 1992-1-1:2004:E. Brussels, Belgium: CEN.
7. Storm T. K., S. H. Rizkalla, and P. Z. Zia. 2013. "Effects of Production Practices on Camber of Prestressed Concrete Bridge Girders." *PCI Journal* 58 (1): 96–111.
8. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2013. "Prestress Losses Evaluation in Prestressed Concrete Prismatic Specimens." *Engineering Structures* 48: 704–715.
9. Martí-Vargas, J. R., and W. M. Hale. 2012. "Predicting Strand Transfer Length in Pretensioned Concrete: Eurocode versus North American Practice." *ASCE Journal of Bridge Engineering* 18 (12): 1270–1280.
10. Martí-Vargas, J. R., F. J. Ferri, and V. Yepes. 2013. "Prediction of the Transfer Length of Prestressing Strands with Neural Networks." *Computers and Concrete* 12 (2): 187–209.

Authors' response

Thank you for the discussion calling for further clarification of the discrepancy between the effective depth d and the distances d_p, d_s, and d'_s. The effective depth d represents all three distances d_p, d_s, and d'_s. In calculations, all three variables are functions of d. Therefore, the changes in effective depth d are reflected in changes in d_p, d_s, and d'_s.

The objective of this paper was to present the results of the reliability analysis for prestressed concrete girders using the most recent live load and resistance (flexure) models.[1] An important part of the research was derivation of sensitivity functions for various parameters that affect performance of prestressed concrete girders. The contribution of several resistance parameters, such as reinforcement area and yield stress, was considered. Other factors, such as transfer length, development length, dispersion length, camber, and prestress losses, were not specifically considered. However, they are accounted for in the change of parameters: $A_{ps}f_{ps}$, bf'_c, and d. Reliability indices were calculated for the nominal values of these parameters and then for the overall resistance reduced by multiples of 10%.

Anna M. Rakoczy
Senior engineer II, Transportation Technology Center Inc.
Pueblo, Colo.

Andrzej S. Nowak
Professor and chair of Civil Engineering, Auburn University
Auburn, Ala.

References

1. Rakoczy, Anna M., and Andrzej S. Nowak. 2013. "Reliability-Based Sensitivity Analysis for Prestressed Concrete Girder Bridges." *PCI Journal* 58 (4): 81–92.

Long-term lateral deflection of precast, prestressed concrete spandrel beams

The following comments relate to "Long-Term Lateral Deflection of Precast, Prestressed Concrete Spandrel Beams" by Bulent Mercan, Arturo E. Schultz, Henryk K. Stolarski, and Rafael A. Magaña.[1]

Given that the use of slender precast concrete spandrels that span longer distances results in greater cost effectiveness of precast concrete parking structures, the authors present a detailed paper that provides realistic estimates of long-term deflections and offers sensitivity analyses on the effects of the main parameters affecting these deflections.

The authors should be congratulated for this paper. The discusser agrees that the scientific-technical literature available on this topic is limited, making the paper that much more valuable. In addition, the discusser would like to thank *PCI Journal* for the opportunity of offering the following comments, mainly about the assumed transfer length and restrained slips in the spandrel beam modeling so that the authors can consider them in further analyses and research.

In the section "Description of Prototype Spandrel Beams," the authors have detailed the use of prestressing steel Grade 270 (1900 MPa), of 0.5 in. (13 mm) in diameter, stressed to 75% of ultimate. Afterwards, in the section "Modeling of Spandrel Beams," the discusser has not found the strand diameter, and among other data, it appears that transfer length is assumed to be 20 in. (500 mm). Transfer length according to ACI 318-11[2] ranges from 27 to 33.75 in. (690 to 860 mm) for 0.5 in. (13 mm) diameter strands, depending on the prestress losses accounted.[3,4] A value of 20 in. (500 mm) would mean a reduced diameter, a reduced initial strand stress, a lower bound value,[5] a result from an optimized prediction[6] or another equation,[7] and a particular specimen size,[8] among other possibilities. However, a justification for the assumed transfer length of 20 in. (500 mm) has not been included in the paper.

Finally, the authors have stated that the relative displacements between concrete and steel were fully restrained in the modeling of spandrel beams. Was this restriction applied along the entire spandrel beam? Slips are involved in bond phenomena concerning the transfer and development lengths of prestressing steel strands.[9–11]

José R. Martí-Vargas
Professor, Institute of Concrete Science and Technology, Universitat Politècnica de València
Valencia, Spain

References

1. Mercan, Bulent, Arturo E. Schultz, Henry K. Stolarski, and Rafael A. Magaña. 2013. "Long-Term Lateral Deflection of Precast, Prestressed Concrete Spandrel Beams." *PCI Journal* 58 (4): 93–115.
2. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11).* Farmington Hills, MI: ACI.
3. Martí-Vargas, J. R., P. Serna, J. Navarro-Gregori, and L. Pallarés. 2012. "Bond of 13 mm Prestressing Steel Strands in Pretensioned Concrete Members." *Engineering Structures* 41: 403–412.
4. Caro, L. A., J. R. Martí-Vargas, and P. Serna. 2013. "Prestress Losses Evaluation in Prestressed Concrete Prismatic Specimens." *Engineering Structures* 48: 704–715.
5. Martí-Vargas, J. R., and W. M. Hale. 2013. "Predicting Strand Transfer Length in Pretensioned Concrete: Eurocode versus North American Practice." *Journal of Bridge Engineering* 18 (12): 1270–1280.
6. Martí-Vargas, J. R., F. J. Ferri, and V. Yepes. 2013. "Prediction of the Transfer Length of Prestressing Strands with Neural Networks." *Computers and Concrete* 12 (2): 187–209.
7. Martí-Vargas, J. R., P. Serna, J. Navarro-Gregori, and J. L. Bonet. 2012. "Effects of Concrete Composition on Transmission Length of Prestressing Strands." *Construction and Building Materials* 27: 350–356.
8. Martí-Vargas, J. R., L. A. Caro, and P. Serna-Ros. 2014. "Size Effect on Strand Bond and Concrete Strains at Prestress Transfer." *ACI Structural Journal* 111 (2): 419–429.
9. Martí-Vargas, J. R., P. Serna, and W. M. Hale. 2013. "Strand Bond Performance in Prestressed Concrete Accounting for Bond Slip." *Engineering Structures* 51: 236–244.
10. Martí-Vargas, J. R., W. M. Hale, E. García-Taengua, and P. Serna. 2014. "Slip Distribution Model along the Anchorage Length of Prestressing Strands." *Engineering Structures* 59: 674–685.
11. Martí-Vargas, J. R., E. García-Taengua, L. A. Caro, and P. Serna, P. 2014. "Measuring Specific Parameters in Pretensioned Concrete Members using a Single Testing Technique." *Measurement* 49: 421–432.

Authors' response

The authors would like to thank José Martí-Vargas for his insightful questions and informative comments.

Regarding the first item raised by the commenter, the authors used ½ in. (13 mm) diameter, Grade 270 strand. A description of the strand is given in the section of "Description of the Prototype Spandrel Beams," so it is not mentioned again in the section "Modeling of Spandrel Beams."[1]

Regarding the transfer length l_t, the commenter is correct in stating that the selected value of 20 in. (500 mm) is somewhat low relative to the ACI 318-11 recommendations ($l_t = f_{se}d_b/3000$) depending on the magnitude of prestress losses.[2] However, the transfer region in prestressed concrete members introduces significant uncertainty relative to its length and the parameters describing the transfer of stresses through bond. Given the large computational undertaking described in Mercan et al.,[3] simplifying assumptions were necessary to accommodate the large size of the finite element models of the spandrels and the length of the time-dependent deformation analysis. For the same reasons, a relatively coarse model was selected for the transfer region, namely a series of steel elements with decreasing cross-sectional area over the transfer length. Analyses of the beams described in the section titled "Experimental Verifications," but not included in the paper due to length limits, suggested that long-term deflections were not highly sensitive to small variations in transfer length that validated our approach. Furthermore, as the commenter notes, underestimating the transfer length can result in overestimation of member deflections. Thus, any possible overestimation was considered to be consistent with the present study because upper-bound estimates of the long-term increments in vertical and lateral deflections and twist angle due to gravity loads were sought.

Regarding the third item, the commenter is correct in noting that reinforcing bar and prestressing strand were embedded in the concrete assuming perfect bond between reinforcement and concrete. However, the bond versus slip interaction between cracked concrete and reinforcement was included indirectly by modifying the post-peak behavior of concrete invoking the so-called tension stiffening effect for concrete in tension. More details about the authors' treatment of the *tension stiffening* effect can be found elsewhere.[3]

Bulent Mercan
Engineer, 2H Offshore Inc.
Houston, Tex.

Arturo E. Schultz
Professor and director of graduate studies, Department of Civil Engineering,
University of Minnesota, Twin Cities
Minneapolis, Minn.

Henryk K. Stolarski
Professor, Department of Civil Engineering, University of Minnesota, Twin Cities
Minneapolis, Minn.

Rafael A. Magaña
President, Precast Engineering Systems Inc.
Tampa, Fla.

References

1. Mercan, Bulent, Arturo E. Schultz, Henry K. Stolarski, and Rafael A. Magaña. 2013. "Long-Term Lateral Deflection of Precast, Prestressed Concrete Spandrel Beams." *PCI Journal* 58 (4): 93–115.
2. ACI (American Concrete Institute) Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary (ACI 318R-11)*. Farmington Hills, MI: ACI.
3. Mercan, B., A. E. Schultz, and H. K. Stolarski. 2010. "Finite Element Modeling of Prestressed Concrete Spandrel Beams." *Engineering Structures* 32 (9): 2804–2813.

Comparison of details for controlling end-region cracks in pretensioned concrete I-girders

"Comparison of Details for Controlling End-Region Cracks in Pretensioned Concrete I-Girders" by B. E. Ross, M. D. Willis, H. R. Hamilton, and G. R. Consolazio and "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders" by A. Arab, S. S. Badie, M. T. Manzari, B. Khaleghi, S. J. Seguirant, and D. Chapman are collectively an important addition to the knowledge of end-zone cracking.[1,2] The photos of the crack patterns are not greatly different from those shown in Gamble[3] for a girder produced in 1966, though the recent photos show more cracks because the members are larger and much more heavily stressed. At that time, the extent of end-zone cracking did not seem to be widely appreciated or understood, even though this was slightly after publication of the important papers by Gergely et al. that are referenced in the two *PCI Journal* papers.[4,5]

As we were trying to understand the extent of the problem, two or three graduate students and I examined a group of 50 similar I-girders on a very cold morning in February 1967. We could reach 96 girder ends, but the other 4 were buried in snow drifts. Of the 96 ends, 94 had anchorage zone cracks, so one must conclude that all girders of the type must crack. **Figure 1** shows the cross section of the Illinois standard 48 in. (1220 mm) I girder with the strand arrangement shown in the left half of the drawing. There were thirty-eight 7⁄16 in. (11 mm) strands with 10 draped strands, and the girders were 75 ft 1 in. (22.89 m) long.

The anchorage zone reinforcement consisted of six no. 5 (15M) bars located near each end of the beam. The area was probably selected using the Gergely analysis method.[4,5]

There were from 1 to 4 cracks in the end zones, with an average of 1.5 cracks per end. Four cracks were found in 2 ends, 3 cracks were found in 8 ends, 2 cracks were found in 25 ends, and the remaining 59 ends each had 1 crack. In nearly all cases, the

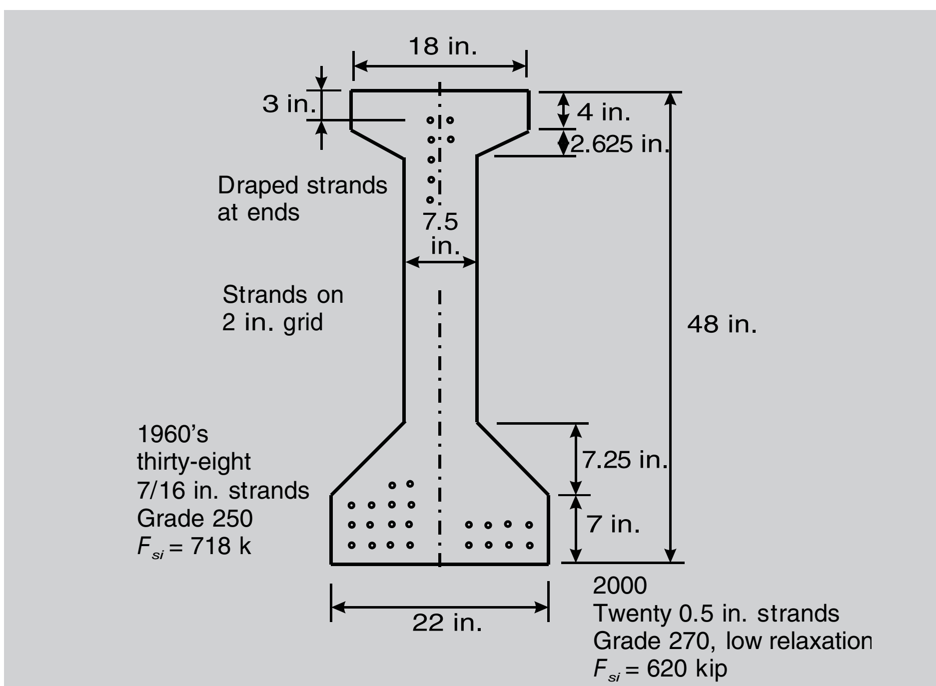

Figure 1. Illinois 48 in. pretensioned girder. Note: F_{si} = design pretentsion force before transfer. Grade 250 = 1720 MPa; Grade 270 = 1860 MPa; 1 in. = 25.4 mm.

single crack was found a few inches, usually no more than 6 in. (150 mm), above the bottom flange-web junction. In the multiple-crack cases, the lowest crack was usually close to the flange-web junction.

Because of the site conditions, only a few crack width measurements were made. The largest single crack found was 0.016 in. (0.41 mm), measured on the side of a beam at the end. About 10 cracks were 0.010 in. (0.25 mm) or greater in width. The estimate of the average width was about 0.006 in. (0.15 mm).

The beams ranged from 10 days to 3 months old, and no correlation between age and state of cracking could be found. One of the producer's engineers said that nearly all such members cracked but that sometimes it took 2 weeks for the cracking to occur.

Both *PCI Journal* papers reference the AASHTO LRFD anchorage zone reinforcement requirement of 4% of the prestressing force,[6] to be resisted by deformed bars at 20 ksi (140 MPa). I find the retention of this value, which is carried over from earlier AASHTO specifications, to be puzzling because it recognizes neither the cross section shape nor the strand arrangement. The provision is also slightly ambiguous in that it does not state whether the prestressing force before or after transfer is to be considered. I have used the pretransfer value because it is slightly more conservative.

The cross section shown in Fig. 1 can be used to illustrate the problems with the 4% rule. The left side of the drawing shows the 1960s steel arrangement used for a beam with a span of about 72 ft (22 m) and a beam spacing of 8 ft (2.4 m). The right side shows the equivalent steel arrangement from the year 2000. The larger, stronger strands lead to a significantly greater eccentricity and, thus, to a smaller initial pretensioning force. The low-relaxation material also contributes to the reduction. The forces noted as F_{si} are the design pretensioning force, before transfer. On the basis of the 4% rule, the new design requires less end-zone reinforcement.

However, a Gergely-type analysis gives a quite different outcome. **Figure 2** shows the results of Gergely analyses for the two different cases. The case with twenty 0.5 in. (13 mm) strands has a tension force that is much larger than the earlier design and much larger than the 4% rule requires. Four percent of 620 kip (2800 kN) is 24.8 kip (110 kN), while the Gergely analysis gives a force of 38 kip (125 kN). The 4% rule is safe for the earlier case because 4% of 718 kip (3190 kN) is 28.7 kip (128 kN), while the Gergely analysis gives a maximum force of 23.7 kip (105 kN). A Gergely-type analysis is not too difficult, but it does require many details of the cross sections, including the area, moment of inertia, centroid, and the detailed variation in width of

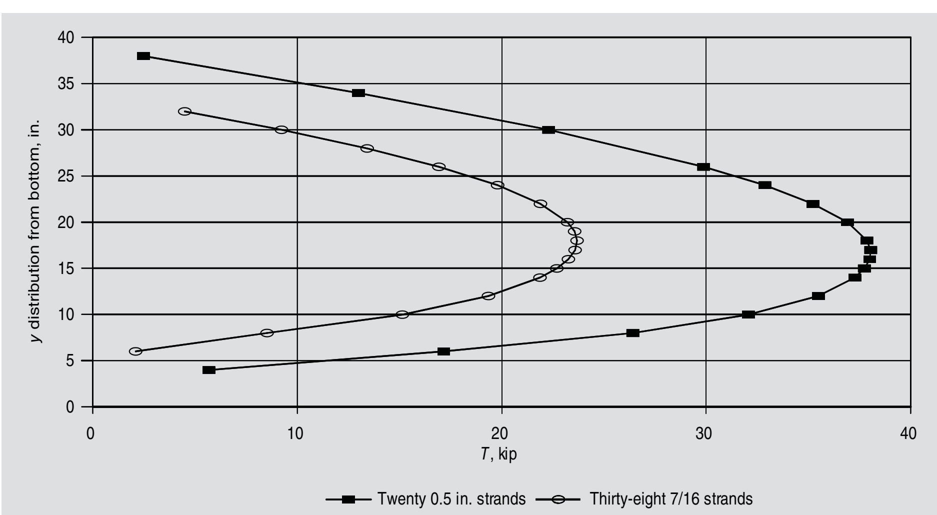

Figure 2. Results of Gergely analysis of girders.

the section in the lower part of the member. The FIB-63 member is a bit messy in this respect, while the Alaskan Way super girders are fairly straightforward. While the Gergely analysis is mentioned in both papers, it is not clear whether it was actually used. The results might have been instructive.

William L. Gamble
Professor Emeritus of Civil and Environmental Engineering, University of Illinois Urbana-Champaign, Ill.

References

1. Ross, B. E., M. D Willis, H. R. Hamilton, and G. R. Consolazio. 2014. "Comparison of Details for Controlling End-Region Cracks in Pretensioned Concrete I-Girders." *PCI Journal* 59 (2): 96–108.
2. Arab, A., S. S. Badie, M. T. Manzari, B. Khaleghi, S. J. Seguirant, and D. Chapman. 2014. "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders." *PCI Journal* 59 (2): 109–127.
3. Gamble, W. L. 1970. "Field Investigation of a Continuous Composite Prestressed I-Beam Highway Bridge Located in Jefferson County, Illinois." Civil Engineering Studies, structural research series no. 360, Department of Civil Engineering, University of Illinois at Champaign-Urbana.
4. Gergely, P., M. A. Sozen, and C. P. Siess. 1963. *The Effect of Reinforcement on Anchorage Zone Cracks in Prestressed Concrete Members.* University of Illinois structural research series no. 271. Champaign, IL: University of Illinois.
5. Gergely, P., and M. A. Sozen. 1967. "Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams." *PCI Journal* 12 (2): 63–75.
6. AASHTO (American Association of State Highway and Transportation Officials). 2012. AASHTO LRFD Bridge Design Specifications. 6th ed. Washington, DC: AASHTO.

Authors' response

The authors appreciate William Gamble's thoughtful comments and wish to thank him for writing a substantive response to our paper, "Comparison of Details for Controlling End-Region Cracks in Pretensioned Concrete I Girders."[1] His story about examining girders in the snow has historical interest and is also instructive regarding the ongoing problem of end-region cracking.

Gamble discusses the 4% rule from the *AASHTO LRFD Bridge Design Specifications*.[2] This approach for designing splitting resistance in pretensioned anchorage zones (in other words, vertical end-region reinforcement) originates from a model created by Marshall and Mattock[3] in the early 1960s. In presenting their model, Marshall and Mattock use the prestressing force at transfer to calculate the requisite vertical reinforcement, an approach consistent with Gamble's calculations. The Marshall-Mattock model was based on experimental data, and as Gamble's analysis has demonstrated, it may not be suitable for modern girder design and materials.

Regarding Gamble's question as to the application of the Gergely-Sozen[4] model to the FIB test specimens, we did not conduct a Gergely-Sozen analysis of our specimens; however, a finite element model was developed in recent work by Willis.[5] The

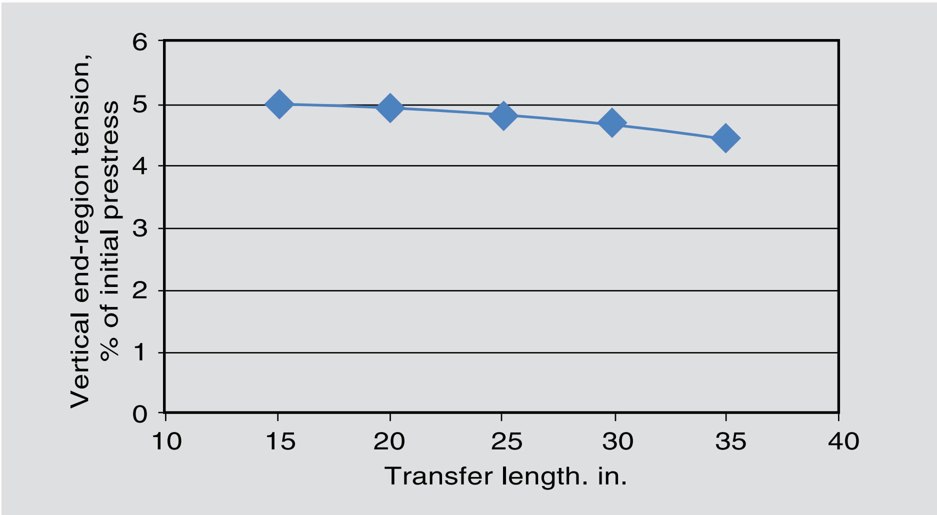

Figure 3. Vertical end-region tension as a function of transfer length. Source: data from Willis (2014).

finite element model was validated using strain data measured during prestress transfer from the FIB specimens. Results of the finite element model (**Fig. 3**) show that the vertical tension force in the end region of the FIB specimens was equal to approximately 5% of the initial prestress force. Moreover, the model shows that vertical tension force is a function of the transfer length. Interestingly, the original model by Marshall and Mattock also recognized that vertical tension was a function of transfer length.

Considering the analyses presented by Gamble and Willis, the authors conclude that the current AASHTO LRFD specifications provisions for splitting resistance in pretension anchorage zones may be unconservative for some (if not many) modern girders. The authors agree with Gamble's statements regarding the merit of considering cross-section shape and strand configuration when designing splitting reinforcement. The Gergely-Sozen model and finite element analysis offer two means of approaching this problem.

Brandon E. Ross
Assistant professor, Glenn Department of Civil Engineering, Clemson University
Clemson, S.C.

Michael D. Willis
Graduate student, Glenn Department of Civil Engineering, Clemson University
Clemson, S.C.

H. R. Hamilton
Professor, Department of Civil and Coastal Engineering, University of Florida
Gainesville, Fla.

Gary R. Consolazio
Associate professor, Department of Civil and Coastal Engineering, University of Florida
Gainesville, Fla.

References

1. Ross, B. E., M. D. Willis, H. R. Hamilton, and G. R. Consolazio. 2014. "Comparison of Details for Controlling End-Region Cracks in Pretensioned Concrete I-Girders." *PCI Journal* 59 (2): 96–108.
2. AASHTO (American Association of State Highway and Transportation Officials). 2012. AASHTO LRFD Bridge Design Specifications. 6th ed. Washington, DC: AASHTO.
3. Marshall, W., and A. Mattock. Control of Horizontal Cracking in the Ends of Pretensioned Prestressed Concrete Girders. *PCI Journal*. 7, 56–75 (1962).
4. Gergely, P., and M. A. Sozen. "Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams." *PCI Journal*. 12, 63–75 (1967).
5. Willis, M. 2014. "Post-Tensioning to Prevent End-Region Cracks in Pretensioned Concrete Girders." MS thesis. Clemson University, Clemson, S.C.

The authors would like to thank William Gamble for sharing his experience and opinions about end-zone reinforcement and the provisions of the AASHTO LRFD specifications[1] that are currently used in the United States. Also, the authors would like to thank Gamble for his constructive comments on the research conducted on the end-zone reinforcement of the Alaskan Way Viaduct project.[2]

The authors agree with Gamble that end-zone cracks recorded during release of the strands of heavily prestressed precast concrete beams have become more prevalent than before. This is a direct result of using deeper girders with higher prestress than previously encountered while maintaining a cross section with relatively thin webs and flanges.[3] The end-zone cracks shown in Fig. 4 of "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders" were reported in all WF100G wide-flange girders used in the monitored span of the Alaskan Way Viaduct project. At 100 in. (2540 mm) deep and 205 ft (62.5 m) long, with eighty 0.6 in. (15 mm) diameter strands jacked to an initial stress of 202.5 ksi (1396 MPa), the authors believe that these girders are the largest fully pretensioned girders manufactured in North America to date.

The authors also agree with Gamble that the current provisions for the design of end-zone reinforcement in the AASHTO LRFD specifications[1] do not take into consideration either the shape of the beam or the strand arrangement in the cross section, nor do they clearly specify whether the prestressing force before or after transfer should be used. Although the end-zone reinforcement in the Alaskan Way Viaduct girders was sized based on 4% of the total prestress force prior to transfer, this was not the sole basis for the design. During development of the Washington State Department of Transportation (WSDOT) wide-flange girder sections in the late 1990s, finite element and confirming hand calculations were performed on several worst-case scenarios to properly size the end zone reinforcement.[4,5] The results indicated only slightly less reinforcement than 4% of the prestress force prior to release is required. On the premise that cracking is best controlled by well-distributed smaller bars, no. 5 (15M) stirrups were selected at the minimum spacing permitted until the required area was achieved. This has become the basis of WSDOT's end-zone reinforcement design[6] and was used for the analytical purposes of this research.

Finally, during the instrumentation of the WF100G girders for the Alaskan Way Viaduct, the authors used the Gergely-Sozen procedure[7] to determine the height where the splitting cracks were anticipated to initiate. The authors believe that although this procedure is simple and can be implemented using hand calculations, it does not accurately reflect the postcracking behavior of reinforced concrete sections. In addition, the amount of the end-zone reinforcement determined by this method is significantly influenced by certain assumptions, such as the distance between the tension and compression resultants corresponding to the coupling action imposed by the moment within the end zone.

Amir A. Arab
DC/VA area manager of bridge & tunnel division, Parsons
Washington, D.C.

Sameh S. Badie
Associate professor, The George Washington University
Washington, D.C.

Majid T. Manzari
Professor & chair of civil & environmental engineering, The George Washington University
Washington, D.C.

Bijan Khaleghi
State bridge design engineer, Washington State Department of Transportation
Olympia, Wash.

Stephen J. Seguirant
Vice president and director of engineering, Concrete Technology Corp.
Tacoma, Wash.

David Chapman
Chief engineer, Concrete Technology Corp.
Tacoma, Wash.

References

1. AASHTO (American Association of State Highway and Transportation Officials). 2012. *AASHTO LRFD Bridge Design Specifications*. 6th ed. Washington, DC: AASHTO.
2. Arab, A., S. S. Badie, M. T. Manzari, B. Khaleghi, S. J. Seguirant, and D. Chapman. 2014. “Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders.” *PCI Journal* 59 (2): 109–127.
3. Tadros, M. K., S. S. Badie, and C. Y. Tuan. 2010. “Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web.” NCHRP report 654: National Cooperative Highway Research Program. Washington, DC: Transportation Research Board.
4. Seguirant, S. J. 1998. “New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders.” *PCI Journal* 43 (4): 92–119.
5. Weigel, J., S. J. Seguirant, R. Brice, and B. Khaleghi. 2003. “High Performance Precast Prestressed Concrete Girders in Washington State.” *PCI Journal* 48 (2): 28–52.
6. Washington State Department of Transportation. *Bridge Design Manual LRFD*. Washington State Department of Transportation. http://www.wsdot.wa.gov/publications/manuals/m23-50.htm.
7. Gergely, P., and M. A. Sozen. 1967. “Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams.” *PCI Journal* 12 (2): 63–75.

Comments?

The editors welcome discussion of the technical content of *PCI Journal* papers. Comments must be confined to the scope of the paper to which they respond and should make a reasonable and substantial contribution to the discussion of the topic. Discussion not meeting this requirement will be returned or referred to the authors for private reply.

Discussion should include the writer's name, title, company, city, and email address or phone number and may be sent to the respective authors for closure. All discussion becomes the property of *PCI Journal* and may be edited for space and style. Discussion is generally limited to 1800 words with each table or illustration counting as 300 words. Follow the style of the original paper, and use references wherever possible without repeating available information.

The opinions expressed are those of the writers and do not necessarily reflect those of PCI or its committees or councils.

All discussion of papers in this issue must be received by February 1, 2015. Please address reader discussion to *PCI Journal* at journal@pci.org.