## Analytical investigation and monitoring of end-zone reinforcement of the Alaskan Way viaduct super 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.
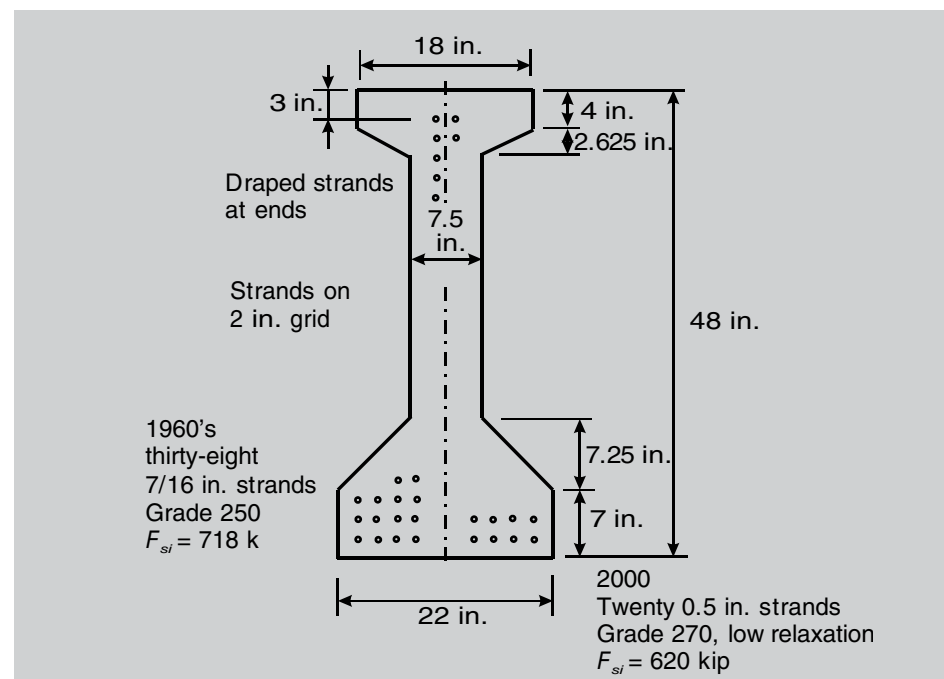
Figure 1. Illinois 48 in. pretensioned girder. Note: F_{si} = design pretentsion force before transfer. Grade 250 = 1720 MPa; Grade 270 = 1860 MPa; 1 in. = 25.4 mm.

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 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 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.

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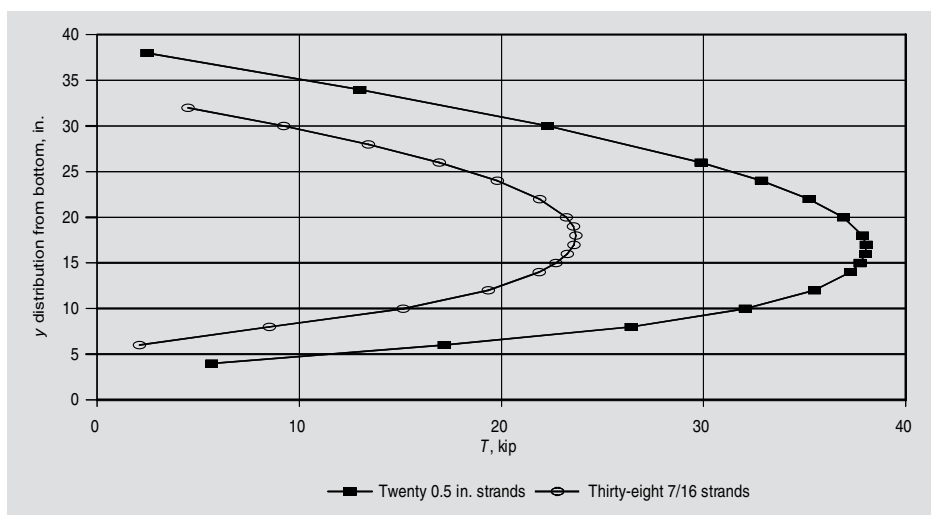
Figure 2. Results of Gergely analysis of girders.

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Authors' response

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 Bridge Design Specifications*[1] that are currently used in the United States, and for his constructive comments on the research conducted on the end-zone reinforcement

of the Alaskan Way Viaduct project and reported in "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders."[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 Arab et al.[2] were reported in all WF100G girders used in the monitored span of the Alaskan Way Viaduct project. The authors believe that, at 100 in. (2500 mm) deep and 205 ft (62 m) long, with eighty 0.6 in. (15 mm) diameter strands jacked to an initial stress of 202.5 ksi (1400 MPa), 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 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. While 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 (16M) 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 project, the authors used the Gergely-Sozen procedure[7] to determine the elevation 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.

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Analytical investigation and monitoring of end-zone reinforcement of the Alaskan Way viaduct super girders

The following comments relate to "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders," by Amir Arab, Sameh S. Badie, Majid T. Manzari, Bijan Khaleghi, Stephen J. Seguirant, and David Chapman, which appeared in the Spring 2014 issue of *PCI Journal*.[1]

Based on experimental research and nonlinear finite element analyses, the authors have reported interesting findings on the behavior of end-zone cracking of deep pretensioned concrete I-girders immediately after release. The authors should be complimented for providing a detailed paper that compares the results obtained from several methods (traditional elastic analyses, strut-and-tie method, and a finite element method) and, based on shear-friction analogy, proposes an alternative method for estimating the tensile stresses in the end-zone reinforcement at the web and bottom-flange interface of I-girders. The discusser would like to thank the authors for their contribution and *PCI Journal* for the opportunity to offer the following comments, mainly about the pretensioning transfer path.

The authors state that pretensioning transfer occurs along a parabolic path instead of linearly, as prescribed by the AASHTO LRFD specifications.[2] As observed in Fig. 7, the percentage of prestress transferred along the transfer length L_t is as follows (distances from the member end): 45% at $L_t/4$, 80% at $L_t/2$, 95% at $L_t(3/4)$, and 100% at L_t. These percentages coincide with the upper values of the ranges of prestress transferred established by Rôs[3] on the basis of theoretical and experimental results: 35% to 45% at $L_t/4$, 75% to 80% at $L_t/2$, 90% to 95% at $L_t(3/4)$, and 100% at L_t. However, other authors have stated that an inelastic response occurs for more than 90% of transfer length,[4,5] and the hypothesis of uniform bond stress distribution along the transfer length (pretensioning transfer along a linear path) has been assumed by several codes[6]—ACI 318-11, Eurocode 2, Model Code 2010—and authors.[7–10]

This certainly seems to be a controversial topic. Based on the transfer length determination from the longitudinal concrete strains profile on the member surface,[11,12] both parabolic[13] and linear[14] paths have been observed. Initial linear paths have also been observed, which became parabolic with time.[15,16] Regarding the transfer length estimation from the free end slip, two hypotheses were considered by Guyon:[17] uniform bond stress distribution or linear variation in strand stress ($\alpha = 2$ in Guyon's expression) and linear descending bond stress distribution or parabolic variation in strand stress ($\alpha = 3$ in Guyon's expression). Different α values, ranging from 2 to 3, have been reported in the literature.[18] Specifically, $\alpha = 2.44$ has been obtained for 13 mm (0.5 in.) seven-wire prestressing steel strands,[19] which means that pretensioning transfer occurs along an intermediate path that is between linear and parabolic. Regarding the transfer length determination from strand slip sequences,[20] a linear tendency was observed. Furthermore, for the transfer length determination from prestressing strand force,[21,22] the test results showed variation in transferred prestressing force, which was practically linear, and no elastic zone in the transfer length has been observed,[5,23] even with a varying specimen size.[24]

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Authors' response

The authors would like to thank José Martí-Vargas for sharing his experience about the distribution of the prestress force transferred along the transfer length L_t. The authors agree with Martí-Vargas regarding the variety of the assumptions and test results corresponding to the stress path along the transfer length. This is because the transfer length is a multiparameter phenomenon affected by many factors, such as strand surface conditions, interlocking between the strands and concrete, adhesion at the interface between strands and concrete, and confinement.

As stated in the paper,[1] the authors strongly recommend using nonlinear finite element models in investigating stresses and strains within the end-zone region. This is due to the fact that tensile stresses higher than the tensile capacity of concrete are generated in this region at prestress release. Nonlinear finite element models are capable of detecting where the end-zone cracks will be developed and accurately determining the stresses in the end-zone reinforcement after the concrete cracks and redistribution of the stresses takes place. Because building these accurately is elaborate and time consuming, especially when draped strands are used, the authors recommend limiting their use only for analysis of the stresses and strains in the end-zone regions.

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

The following comments relate to "Comparison of Details for Controlling End-Region Cracks in Precast, Pretensioned Concrete I-Girders" by Brandon E. Ross, Michael D. Willis, H. R. Hamilton, and Gary R. Consolazio, which appeared in the Spring 2014 issue of *PCI Journal*.[1]

The paper presents an interesting comparison of four detailing schemes for controlling end-zone web-splitting cracks at prestress transfer of precast, pretensioned concrete I-girders. The authors should be congratulated for producing a detailed paper, which is acknowledged by the discusser. The discusser would also like to thank *PCI Journal* for the opportunity of offering the following comments, mainly about specimen design, analysis of the results, and potential ways of cracking.

Certainly this paper focuses on an unsolved topic that requires more research. As a matter of fact, the same issue of *PCI Journal* includes another research work[2] that focuses on end-zone reinforcement and longitudinal web cracks. In this case, a new detailing scheme based on the distribution of the required end-zone reinforcement is suggested: 50% of the required end-zone reinforcement should be placed within $h/8$ and the remaining 50% between $h/8$ and $h/2$; however, according to the AASHTO LRFD specifications,[3] the end-zone reinforcement should be distributed within a distance of $h/4$ from the girder end, and no end-zone reinforcement is required beyond $h/4$.

The authors state that web-splitting cracks were first observed in specimens CT, LB, and SL during prestress transfer and in specimen PT the day after transfer; however in Fig. 6 (an excellent figure with valuable information on cracking evolution over time) one can observe that specimen PT also exhibits cracks during prestress transfer. Perhaps the authors considered these cracks located on the bottom flange instead of in the web. If so, then it seems that there are no cracks in the web except for the inclined cracks that appeared 30 days after transfer. It is also observed from Fig. 6 that the inclined cracks from the top appeared in specimens CT, PT, and LB on day 30 post-transfer and also in specimen SL before load testing 112 days after transfer. Therefore, it seems that inclined cracks are not due to a specific effect of posttensioned threaded rods. Nevertheless, in this case only one wide inclined crack resulted.

The discusser appreciates the authors' efforts in innovating detailing schemes and testing full-scale girders and encourages them to continue their research on this topic. Highly relevant conclusions have been drawn from only two girders (four end zones). However, in addition to the different detailing schemes, some parameters, such as distance of placement of vertical bars and excess of limits from AASHTO LRFD specifications,[3] changed for the different end zones. In the discusser's opinion, this fact does not allow an analysis of effects in an isolated manner.

Finally, the possible distinction of the three potential ways of cracking[4] (bursting, spalling, and splitting) is suggested for further studies as well as the measuring transfer and development lengths. Regarding the ACI 318-11[5] requirements, transfer length is about 36 in. (910 mm) and development length is about 48 in. (1220 mm) for the strands with a diameter of 0.6 in. (15 mm). Besides, the transfer and development lengths in each girder can differ as they are related to concrete properties.[6,7] In particular, this is the main difference between European and North American practices.[8] Moreover, transfer length is related to strand-free end slip,[9–11] and experimental studies on bond behavior after strand slippage at loading have been conducted.[12,13]

Furthermore, the use of vertical posttensioned threaded rods may modify girder deformability in addition to the transformed cross-section properties.[14]

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Authors' response

The authors are pleased to see that their work, "Comparison of Details for Controlling End-Region Cracks in Pretensioned Concrete I-Girders,"[1] has generated national and international interest, and thank José Martí-Vargas for his insightful comments. His comments provide a welcome opportunity to clarify and expand our discussion of the research results and to advance the body of knowledge on this highly relevant topic.

Martí-Vargas inquired about the discrepancy between Fig. 6, which shows the growth and formation of end-region cracks, and the statement that web-splitting cracks were observed in specimen PT the day after prestress transfer. This discrepancy is due to an unfortunate error in our manuscript. As shown in Fig. 6, cracking in the web of specimen PT was not observed until 30 days after prestress transfer. We are grateful to Martí-Vargas for bringing this discrepancy to our attention so that we might provide clarification.

With regard to the cause of inclined web cracking in specimen PT, the paper does not imply that posttensioning was the only cause. Rather, it is stated that forces induced by the posttensioning "contributed to the formation of diagonal cracking in specimen PT." Inclined cracks in the specimens without posttensioning clearly indicate that the distribution of pretensioning forces also caused inclined web cracking. It is interesting to note, however, that posttensioning appears to have had a significant effect on the width of the primary inclined crack in specimen PT. At 0.012 in. (0.3 mm), the primary inclined crack in specimen PT was the widest crack in the program. In hindsight the authors suggest that distributing the vertical posttensioning force throughout the end region would have been more effective for controlling horizontal and inclined web cracking. Creating a method for designing the magnitude and distribution of end-region posttensioning is recommended as a topic for future research.

The authors concur with Martí-Vargas that end-region cracking is an unsolved problem requiring more research. In addition to comprehensive experimental research that treats variables in isolation, validated analytical models also constitute a powerful tool for studying the problem of end-region cracking. Recent papers by Okumus and Oliva[2] and Arab et al.[3] are recommended.

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Modeling the Resistance of Precast, Prestressed Concrete Hollow-Core Slabs Exposed to Fire

The following comments relate to "Modeling the Resistance of Precast, Prestressed Concrete Hollow-Core Slabs Exposed to Fire," by V. K. R. Kodur and A. M. Shakya, which appeared in the Summer 2014 issue of *PCI Journal*.

This paper is a valuable addition to the literature on fire resistance, but it contains an error that has been repeated by multiple authors. It is concerned with the limiting temperatures on the side of the member away from the fire. The problem is in the second paragraph of page 84. The stated 139 °C is the limiting temperature *rise*, not the limiting temperature, for the average of several measuring points. So the maximum temperature is 159 °C, assuming 20 °C is the starting point. In English units, the limiting rise is 250 °F, or a final temperature of about 320 °F.

The limiting temperature rise for a single point is 325 °F (181 °C).

This has no consequences for the current study because the surface temperatures remained far below these limits but could have consequences for thinner slabs.

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References

1. Kodur, V. K. R., and A. M. Shakya. 2014 "Modeling the Resistance of Precast, Prestressed Concrete Hollow-Core Slabs Exposed to Fire." *PCI Journal* 59 (3): 78–94.

Authors' response

The authors would like to express gratitude to William Gamble for his interest and valuable feedback on "Modeling the Resistance of Precast, Prestressed Concrete Hollow-Core Slabs Exposed to Fire."[1] The discrepancy in the paper regarding the limiting temperature on the unexposed surface is due to inconsistency in the conversion of temperature on Celsius and Fahrenheit scales ($\Delta°C \leftrightarrow \Delta°F$). However, the authors are aware of the fact that the critical temperature rise for the unexposed side of the slabs is 250 °F (139 °C) for the average temperature and 325 °F (181 °C) for tem-

perature at a single point, as per ASTM-E119.[2] Thus, temperature limits of 159 °C for average temperature and 201 °C for single-point temperature were applied in evaluating failure (fire resistance) of the slabs. It should be noted that this error does not change or affect the results presented in the paper.

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1. Kodur, V. K. R., and A. M. Shakya. 2014 "Modeling the Resistance of Precast, Prestressed Concrete Hollow-Core Slabs Exposed to Fire." *PCI Journal* 59 (3): 78–94.
2. ASTM International. 2012. *Standard Test Methods for Fire Tests of Building Construction and Materials*. Test method E119-12. West Conshohocken, PA: ASTM International.

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