ANCHORAGE ZONE CRACKING EVALUATION IN DEBONDED DEEP BULB-TEE BRIDGE GIRDERS

Pinar Okumus, PhD, Dept. of Civil, Structural and Env. Eng., University at Buffalo, NY
Emre Kizilarslan, Dept. of Civil and Env. Eng.,University of Wisconsin, Madison, WI
Michael Oliva, PhD, Dept. of Civil and Env. Eng.University of Wisconsin, Madison, WI
Cancan Yang, Dept. of Civil, Structural and Env. Eng., University at Buffalo, NY

ABSTRACT

Anchorage zone cracks in pretensioned bridge girders are a concern for durability. Previous research used analytical methods to show that these cracks can be controlled by de-bonding selected strands near the girder end for bulb-tee girders. The current work, presented in this paper, tests the effectiveness of this method with actual 72 inch deep bulb-tee girders. Full scale bridge girders were designed with and without de-bonded strands. They were instrumented in precast plants and strains were measured during de-tensioning to evaluate the effects of debonding on cracking. Strains were detected in the anchorage zone using an array of surface strain gages on reinforcing bars, and vibrating wire gages in concrete. Transfer length of strands was also measured. Comparison of strain results from girders with and without de-bonded strands shows that even limited debonding at the AASHTO 25% limit can reduce anchorage zone cracking and tension strains by half. The most efficient reinforcing bars in restraining cracking were identified. Stresses on reinforcing bars for girders with and without debonding were compared to AASHTO requirement for end zone reinforcing design.

Keywords: Anchorage zone, Cracks, Prestressed concrete, Field instrumentation, Debonded Strands, Splitting reinforcement

INTRODUCTION

Prestressed concrete bulb tee girder anchorage zones crack after prestress release due to the transfer of prestress to a slender concrete body. The amount of prestress transferred to concrete directly impacts anchorage zone stresses and cracking. Girder end cracks are characterized as inclined cracks (0.004 in.-0.010 in. wide), horizontal web cracks (0.004 in.-0.010 in. wide), and Y cracks (0.02 in.-0.06 in. wide) as shown on a 72 in. deep girder in Fig. 1. Cracks cause durability concerns, especially for Y type cracks that are in close proximity to strands. Due to their size, Y cracks may need to be repaired by cementitious material or by epoxy filling[1] and in some states could lead to the rejection of the girder.

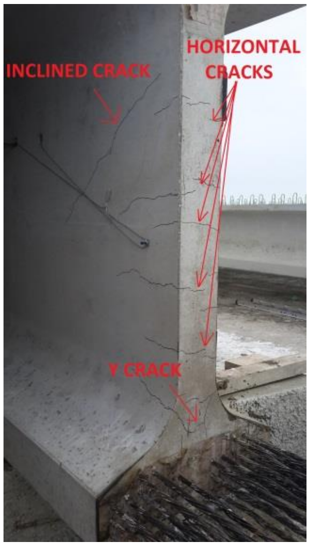


Fig. 1 Types of prestressed concrete girder end cracks

A previous study[2,3] evaluated anchorage zone crack control methods including addition of reinforcement, changing strand cutting order, lowering or spreading harped strands, debonding all strands for a short distance from the girder end, and debonding some strands over the anchorage zone. The evaluation indicated that debonding was the most effective method to restrain all types of cracks and the only effective method to restrain Y type cracks. Number and patterns of debonded strands that could minimize end cracks were recommended using finite element analysis[4]. Strains that cause Y cracks were minimized when the most exterior strands across the girder width were debonded, and no two adjacent columns of strands were debonded. Configurations that do not follow this debonding pattern could increase strains in the Y crack region.

Others investigated methods to improve anchorage zones through decreasing stresses by using vertical post-tensioning[5], changing strand cutting order[6] or by restraining cracking through reinforcement[1,7-11]. Work presented in this paper focuses on debonding selected strands over the anchorage zone and evaluates the decrease in concrete and reinforcing bar strains due to debonding. Although some states and precast concrete manufacturers use debonding to control stresses, the amount of debonding is determined to limit tension at the extreme fiber at the plant. The impact of debonding on anchorage zone cracks is only known through observations. This study measured concrete and reinforcing bar strains on bulb tee girders to quantify the impact of debonding on anchorage zones.

Two 72 in. deep bulb-tee girders were monitored at a precast concrete plant for strains before and right after prestress release. The two girders were identical except for the number of strands bonded at the girder end. This allowed a direct comparison and evaluation of impact of debonding on concrete and reinforcing bar stresses.

GIRDER PROPERTIES

The two girders instrumented were 72 in. deep bulb-tee girders designed to be used for a fourteen span bridge on Wisconsin Highway 96, in Wrightstown, Wisconsin. Each span had seven identical girders. The two girders instrumented over the end zones were 154.75 ft. long. The design called for forty-eight, 0.6 in. diameter, 270 ksi ultimate strength, low-relaxation prestressing strands. Eight of these strands were draped in the original design.

One of the two girders instrumented was re-designed with de-bonded strands. Compliant with AASHTO LRFD Bridge Design Specifications (BDS)[12] 5.11.4.3, the number of debonded strands was limited to 25% of the total number of strands. This led the girder to have a combination of 12 debonded strands and 6 draped strands. Debonding was terminated for 4 strands at a time at 4.5 ft., 9.0 ft. and 13.5 ft. from the girder end. Debonding was achieved using two layers of plastic sheathing as shown in Fig. 2. Plastic sheathing is expected to transfer some prestress compared to rigid sheathing[13], but this prestress is negligible compared to the prestress transferred by bonded strands.

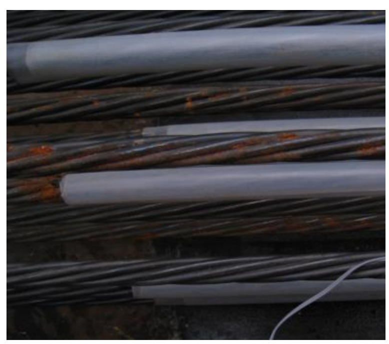
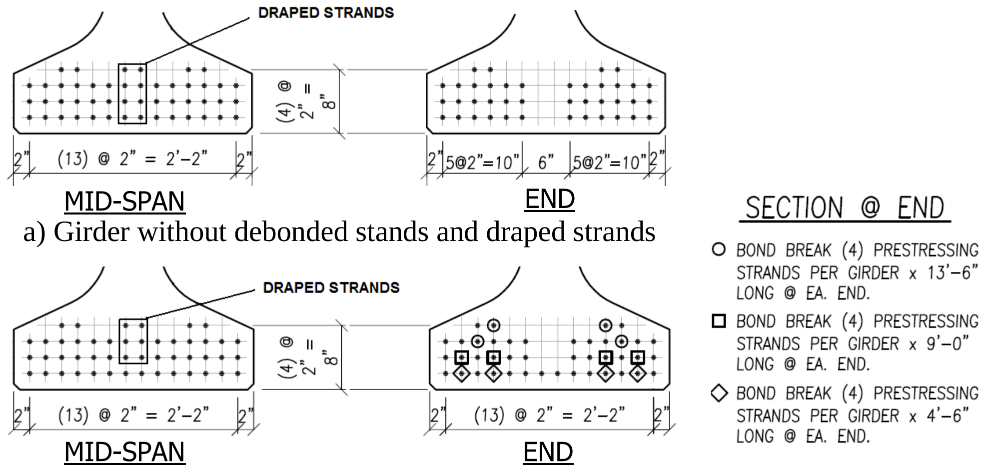


Fig. 2 Plastic sheathing used for debonding strands

The pattern of debonded strands was selected to minimize the strains in concrete in the bottom flange-web junction area where Y type cracks form. This configuration requires the most exterior strands to be debonded[4]. In order to make the girder design compliant with AASHTO LRFD BDS 5.10.14.1, however, the most exterior strand of each row of strands was left bonded to concrete. Strand pattern of the girder with debonded strands is shown in Fig. 3.



a) Girder without debonded stands and draped strands

b) Girder with debonded strands and draped strands

Fig. 3 Strand patterns for girder with (a) and without (b) debonded strands

Splitting and confinement reinforcement of the two girders were the same, and met the requirements of AASHTO LRFD BDS 5.10.10.1 and AASHTO LRFD BDS 5.10.10.2, respectively. The anchorage zone reinforcement area of the girder with debonded strands was kept the same as the girder without debonded strands, to understand the impact of strand debonding alone on girder end zone stresses. Splitting reinforcement within a distance h/4, where h is the girder depth, from the girder end was designed to resist 4.6% of prestress at transfer. If debonding is considered when calculating the prestress at transfer, this ratio was 6.1%. Girder end zone reinforcement standard details are available elsewhere[14]. Both girders had 2109 kips of total prestress applied through two hydraulic jacks: one for draped and one for straight strands. The girders were detensioned gradually using hydraulic jacks, releasing 88 kips of prestress force at a time. After each release step, two strands were cut at the dead end using a torch.

INSTRUMENTATION

LOCATIONS OF GAGES

Strains during detensioning were measured on 1) strands, 2) anchorage zone reinforcing bars, and 3) directly in concrete. Strains on strands were used to obtain the transfer length, shape of the strand-concrete bond function, and prestress at transfer. Strains in reinforcement bars and concrete were used to evaluate anchorage zone stresses with and without debonding.

Locations of strain gages on reinforcing bars and concrete were selected to coincide with the regions where inclined cracks, horizontal web cracks, and Y cracks were expected. These locations were determined using finite element analyses of a similar girder, and through observations of cracking in similar girders. Strain gages on the strand were placed across the expected transfer length, at 8 in., 20in., and 36 in. from the girder end. Location of the instrumented strands, strain gages on reinforcing bars, and in concrete are shown in Fig. 4.

The locations of all strain gages were identical for the girder without debonded strands and the girder with debonded strands.

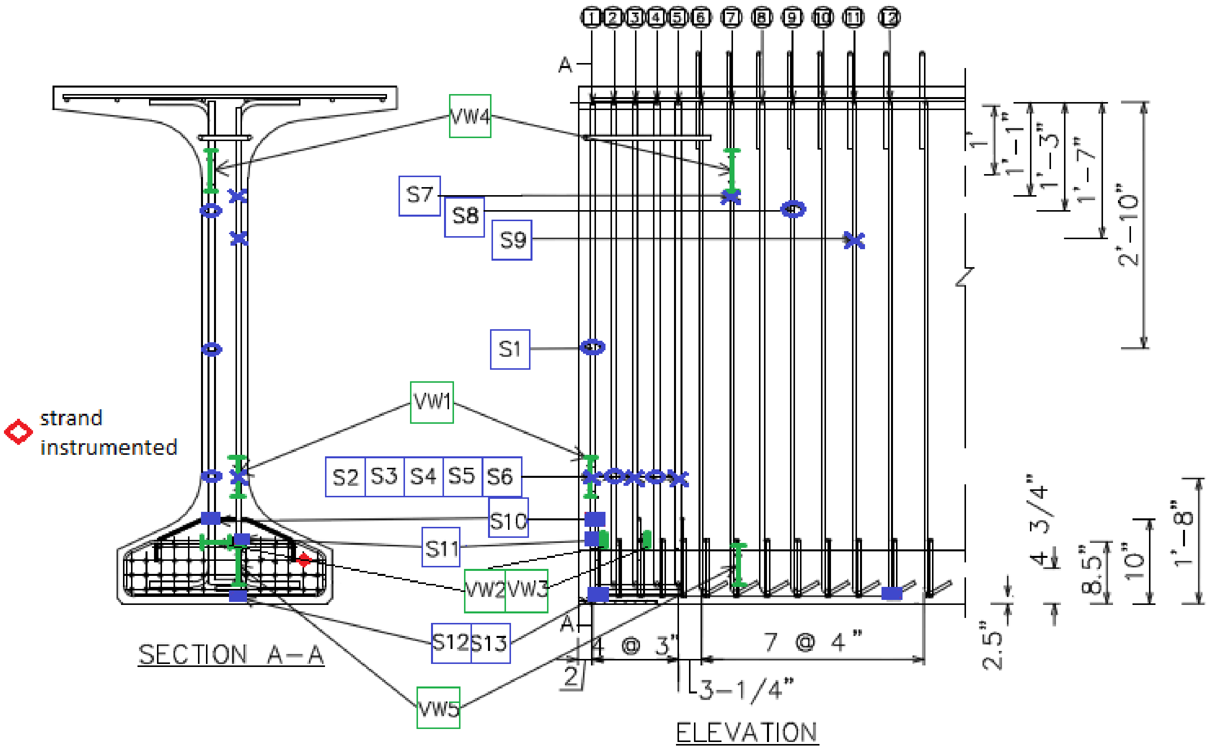


Fig. 4 Locations of gages on strands, reinforcing bars and in concrete

TYPES OF GAGES

Electric resistance surface gages of 120 Ω and 350 Ω were used on strands and reinforcing bars. These gages were small in size (0.040 in. or 1 mm), in order not to disturb the bond between concrete and strand, and the bond between concrete and reinforcing bars. Surface of the reinforcing bar was sanded to create a flat surface. The gages were glued, water proofed using a wax coating, and protected against mechanical impact. Strain gages on strands were placed on a single wire of a seven-wire strand. A strain gage glued on the rebar surface, and the strain gage after all protective coats are placed are shown in Fig. 5.

Fig. 5 Strain gage on re-bar before (left) and after (right) protective layers

Vibrating wire strain gages of 2 in. and 4 in. lengths were embedded in the concrete to obtain strains directly in concrete. These gages had an accuracy of ±15 με at full strain range.

DATA ACQUISITION

Surface strain gages attached to strands and reinforcing bars were connected to a 16 channel data acquisition system. These strains were recorded continuously at a rate of 1 reading per 2.5 seconds during prestress release. Readings from the vibrating wire gages embedded in concrete were obtained at discrete intervals using a hand-held device.

INSTRUMENTATION RESULTS

CONCRETE-STRAND BOND STRESSES

Gages on strands collected data on distribution of bond stress between concrete and strands, and transfer length. One strand was instrumented on each girder. The change in strain on strands during de-tensioning is given in Fig. 6 (left). Strain gages were located at 8 in., 20 in., and 36 in. from the girder end. The strain at 0 in. from the girder end, shown in Fig. 6 (left), was calculated assuming the end of the strand was free to slip and the strain change was due to detensioning stress. Strands were stressed to 75% of the strand ultimate strength.

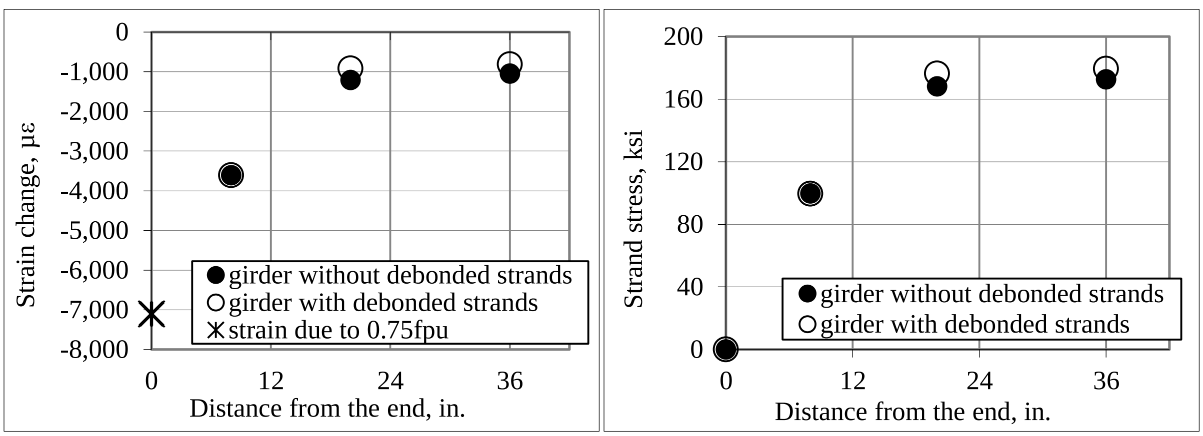


Fig. 6 Strain change in strands (left) and stress in strands (right)

Fig. 6 (right) shows the stress in strands calculated from the change in strains shown in Fig. 6 (left). Strands on the girder with and without debonded strands had practically identical values of strains. Increase in stresses in both strands is nonlinear, indicating a non-uniform bond stress distribution.

The average of stresses on the two strands converges to 176 ksi. Initial prestress after elastic shortening is calculated as 184 ksi. This indicates that strand stresses could increase by 4.5% beyond 36 in. from the girder end. This expected increase in stress is insignificant, as stresses only increased by 3.7 ksi between 20 in. and 36 in. from the girder end. Bond stresses after 36 in. are anticipated to be very small.

STRAINS IN REINFORCING BARS

Reinforcing bars were instrumented with strain gages at the same locations for the girder with and without debonded strands. Instrumentation was placed over the horizontal web cracking region (gages S1 through S6), inclined cracking region (gages S7 through S9) and Y cracking region (S10 through S13). The locations of gages S1 through S13 are shown in Fig. 4. Strains were measured during de-tensioning continuously for both girders.

Fig. 7 to Fig. 9 show the increase in strains in reinforcing bars during detensioning for the locations of reinforcement in the horizontal web cracking region, inclined cracking region and Y cracking region, respectively. Each figure has reinforcement bar strains for the girder without debonding (left) and the girder with debonding (right). Strain of 690 με corresponding to the 20 ksi limit on splitting zone reinforcement of AASHTO LRFD BDS 5.10.10.1 for crack control is also shown on the figures on Fig. 7 and Fig. 9. Gage S9 malfunctioned for the girder without debonding and is omitted from Fig. 8.

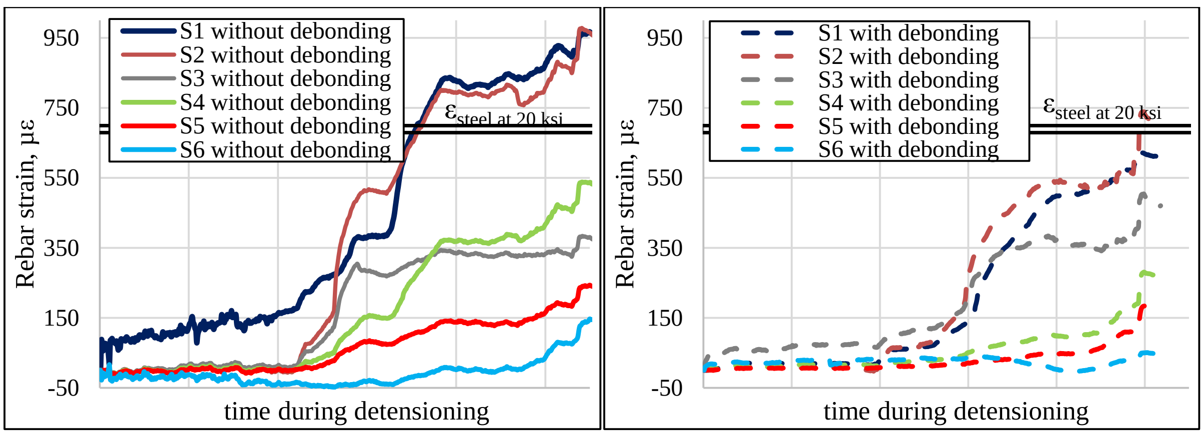


Fig. 7 Strain increase in the web crack region without (left) and with (right) debonding

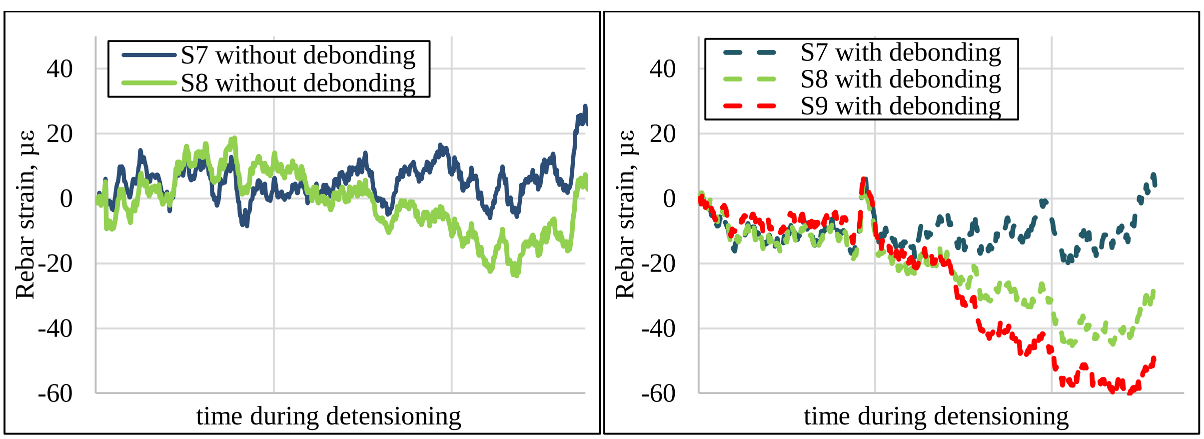


Fig. 8 Strain increase in the inclined crack region without (left) and with (right) debonding

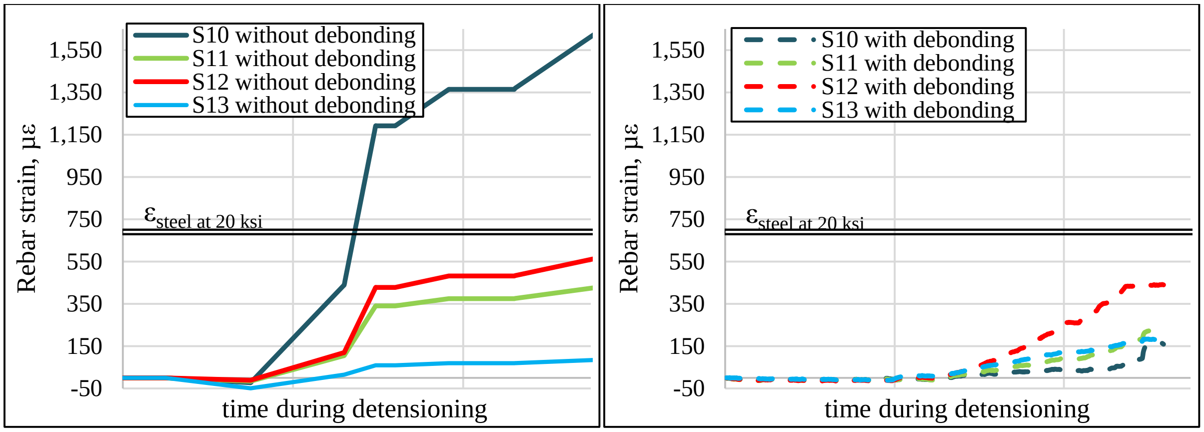


Fig. 9 Strain increase in the Y crack region without (left) and with (right) debonding

Fig. 7 to Fig. 9 show that reinforcing bars have a similar strain trend with and without debonding during de-tensioning. All strains except at locations S3 (web cracking region) and S13 (Y crack region) decreased or stayed the same due to debonding. The increase in strains at S3 and S13 was negligible. Fig. 8 shows that strains on reinforcing bars in the inclined cracking region were small, less than 60 με. This was due to mis-predicting the location of the inclined crack while determining the locations of gages. Gages were located above the inclined crack, and failed to capture strains created due to the crack.

Fig. 7 and Fig. 9 show that although the splitting reinforcement was designed to meet AASHTO LRFD BDS, strains in reinforcing bars exceeded the AASHTO limit. For both regions, strains decreased to or below the limit for the girder with debonded strands. Fig. 7 also shows that the splitting reinforcement stresses are the maximum for the bar closest to the girder end, and rapidly decrease for bars away from the girder end. The second splitting reinforcement bar from the girder end had stresses smaller than the AASHTO LRFD BDS limit even for the girder without debonding.

The highest strain in the Y cracking region was on S10 on the single bar at the very end of the girder without debonding. This shows that this single reinforcement is effective in restraining the Y crack. For the girder with debonding, Y crack strain decreased considerably, by 70%. It is evident that debonding can have a significant impact on controlling the Y-crack – particularly if more debonding was attempted. Strains on confinement reinforcement (S13) at a length equal to a transfer length away from the girder end were small for both girders.

Decrease in the maximum reinforcing bar strain for horizontal web cracking and Y regions due to debonding were 350 με and 1430 με, respectively. For concrete initial modulus of elasticity of 4,800 ksi, and assuming that the reinforcing bar strains are identical to the surrounding concrete strains, the theoretical decrease in concrete stress due to debonding is calculated as 1.68 ksi and 6.86 ksi. For all analysis methods that assume concrete elasticity, these stresses are multiple times larger than the tensile strength of concrete, 0.63 ksi. For inelastic analyses, these stresses would not occur in concrete due to cracking but they are calculated as a theoretical scale for concrete strain reductions.

STRAINS OR CRACK OPENINGS IN CONCRETE

Vibrating wire gages were embedded in concrete in the inclined cracking region (VW4), horizontal web crack region (VW1), and around the Y crack region (VW2, VW3, VW5). Locations of these gages are shown in Fig. 4. Fig. 10 shows the strains measured and strain at which concrete cracks, 132 με. Gage VW5 of the girder with debonding malfunctioned and is omitted from the results.

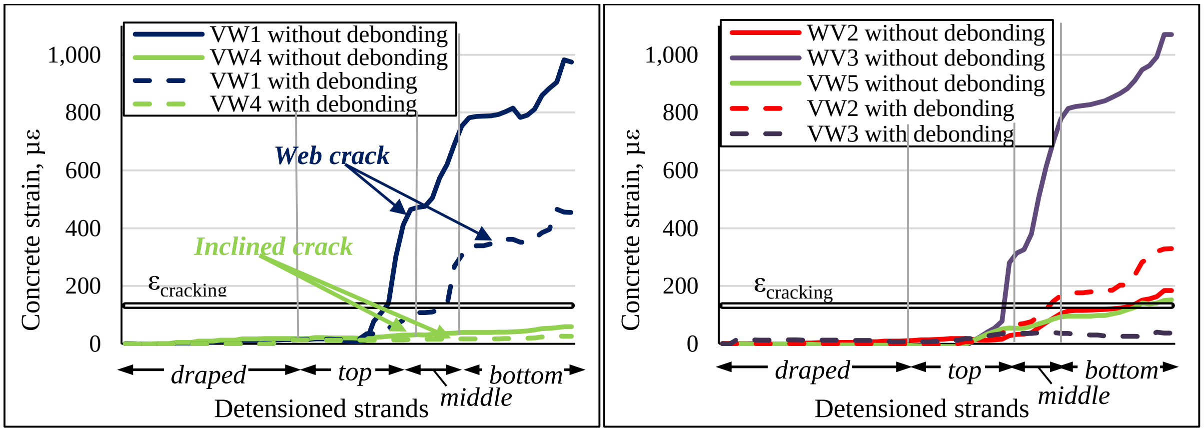


Fig. 10 Concrete strains in inclined and web crack regions (left) and in Y crack region (right)

As shown in Fig. 10, the measured change in vertical strain in the concrete of the lower web region (VW1) for the girder with debonding was 54% less than the girder without debonding. In both beams, however, the web cracking strain was still above the level at which a horizontal web crack is expected. The results from the VW4 gages suggest that concrete strains in the inclined crack region are small due to gage locations not aligning with the inclined crack location.

In looking at the strain of VW2 and VW3, indicators of bottom flange Y cracking, there is little difference between the two beams at the VW2 (first stirrup) location. At the VW3 (third stirrup) location, however, the bonded beam had strains nearly 21 times as high as the debonded beam had. This difference suggests that the Y crack may be induced by strains developing inside the end of the beam where more prestress is transferred to concrete.

GIRDER INSPECTION RESULTS

After detensioning, the girders were lifted and taken to the storage yard where the girders were inspected for cracking. Number of cracks, crack widths and crack lengths were documented. It should be noted that the crack length and sizes are expected to increase as the girders are lifted. Due to the limited access of the researchers to the casting bed, girders were not inspected for cracks right after de-tensioning.

Fig. 11 shows the crack patterns observed in the girder with debonding and the girder without debonding. Although strains in concrete and reinforcing bars decreased significantly due to debonding, the number of cracks did not change considerably with 25% debonding. The cracks formed, however, were shorter in length and smaller in width.

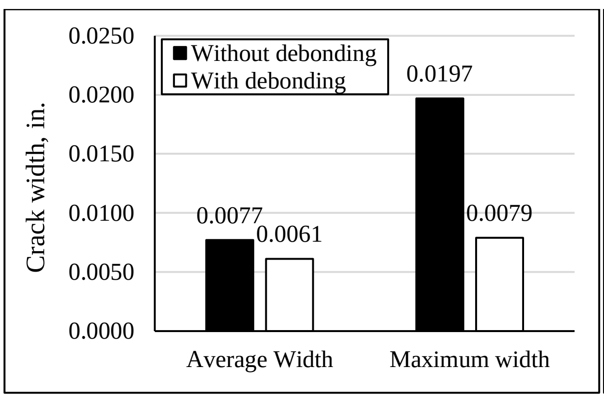


without debonding

without debonding with debonding with debonding

Fig. 11 Crack patterns of girders with and without debonding

Fig. 12, on the left, shows the average and maximum crack widths measured on the girders with and without debonding. Fig. 12, on the right, shows the total length of cracks on both beams, measured only at one end of each girder.



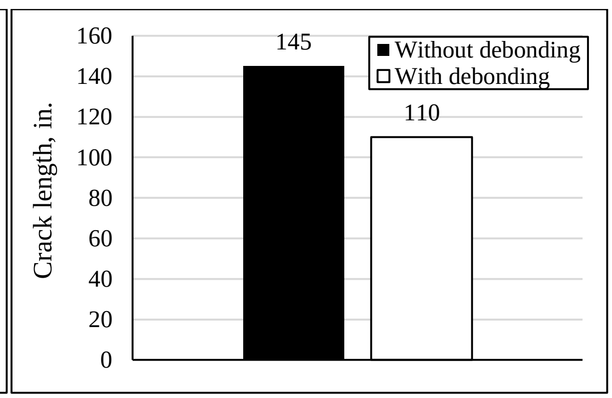


Fig. 12 Crack width (left) and crack length (right) of girders with and without debonding

The average width of cracks did not change with debonding, but the girder with debonded strands had less than half the maximum crack width of the girder without debonded strands. The maximum width was observed at the undesirable Y crack. Debonding did not eliminate Y cracking, but reduced its width considerably. The total length of cracking was reduced by 25%, by debonding 25% of the total number of strands.

CONCLUSIONS

Debonding 25% of the strands over the anchorage zone decreased strains in reinforcing bars, and concrete significantly by decreasing prestress transferred at girder end, and by decreasing the number of draped strands needed. Reinforcement bar strains were compared to the limit on strains (or stress) on reinforcing bars per AASHTO LRFD BDS 5.10.10.1 to restrain cracking. Concrete strains were compared to the strain at which concrete cracks.

Even though the splitting reinforcement of the girders was designed to have stresses below 20 ksi per AASHTO LRFD BDS, stresses on reinforcing bars on the girder without debonding exceeded this limit. The stresses as high as 28 ksi in splitting reinforcement and 45 ksi for confinement reinforcement were observed on the girder without debonding. 25% debonding brought reinforcing bar stresses below the AASHTO limit for crack control for all cracks. This 20 ksi stress limit, however, exists to control crack width and cannot prevent cracking. Reinforcing bar stresses rapidly decreased away from the girder end. The second bar from the girder end had stresses lower than 20 ksi for both girders.

Concrete strains in the web cracking (splitting) region show that 25% debonding decreased tensile strains in concrete by 50%, but not below the cracking limit and did not prevent cracking. The decrease in concrete strains in the splitting region is mainly due to the reduction of prestress transfer to concrete, and partly due to the small reduction in number of draped strands in the girder with debonded strands.

Gages on reinforcing bars in the Y cracking region showed that the single bar at the very end of the girder crossing the Y crack had very high strains, and therefore was effective in restraining the Y crack. The strains on this rebar with debonding were reduced to 10% of the stress, but a narrower Y crack still occurred in the girder with debonding. Concrete strain gages in the Y cracking region indicated that the Y crack might start 8 in. into the girder and propagate towards the girder end. A higher amount of debonding is necessary to completely prevent Y cracking. The change in the number of draped strands does not impact Y cracking strains as these cracks are caused by the eccentric distribution of strands in the direction of the width of the girder, and the vertical reaction force of the girder self-weight.

Debonding 25% of the strands reduced the maximum crack width of the Y cracks by more than half, and decreased the total length of cracks by 25%. Use of debonding has the potential to eliminate cracks, particularly if more than 25% of the strands can be debonded.

ACKNOWLEDGMENTS

This research was conducted with support from Wisconsin Highway Research Program (WHRP). The authors would like to acknowledge WHRP for financial support and the members of the Project Oversight Committee for technical guidance. The conclusions in this paper are those of the authors and do not necessarily reflect the views of WHRP. The authors would also like to thank Spancrete for accommodating the research team at their plant.

REFERENCES

1. Tadros, M. K., Badie, S. S., and Tuan, C. Y., “Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web,” NCHRP Report 654, Transportation Research Board, Washington, DC, 2010, pp. 76.
2. Okumus, P., and Oliva, M. G., "Evaluation of Crack Control Methods for End Zone Cracking in Prestressed Concrete Bridge Girders," *PCI Journal*, V. 58, No. 2, Spring 2013, pp. 91-105.
3. Okumus, P., Oliva, M. G., and Becker, S., "Nonlinear Finite Element Modeling of Cracking at Ends of Pretensioned Bridge Girders," *Engineering Structures*, V. 40, July 2012, pp. 267-275.
4. Okumus, P., and Oliva, M. G., "Strand Debonding for Pretensioned Bridge Girders to Control End Cracks," *ACI Structural Journal*, V. 111, No. 1, Jan.-Feb. 2014, pp. 201-210.
5. Ross, B. E., Willis, M. D., Hamilton, H. R., and Consolazio, a. G. R., "Comparison of Details for Controlling End-Region Cracks in Precast, Pretensioned Concrete I-girders," *PCI Journal*, V. 59, No. 2, Spring 2014, pp. 96-108.
6. Kannel, J., French, C., and Stolarski, H., "Release Methodology of Strands to Reduce End Cracking in Pretensioned Concrete Girders," *PCI Journal*, V. 42, No. 1, Jan.-Feb. 1997, pp. 42-54.
7. Arab, A., Badie, S. S., Manzari, M. T., Khaleghi, B., Seguirant, S. J., and Chapman, D., "Analytical Investigation and Monitoring of End-Zone Reinforcement of the Alaskan Way Viaduct Super Girders," *PCI Journal*, V. 59, No. 2, Spring 2014, pp. 109-128.
8. Tuan, C. Y., Yehia, S. A., Jongpitaksseel, N., and Tadros, M. K., "End Zone Reinforcement for Pretensioned Concrete Girders," *PCI Journal*, V. 49, No. 3, May-June 2004, pp. 68-82.
9. Dunkman, D. A., Hovell, C. G., Moore, A. M., Avendaño, A., Bayrak, O., and Jirsa, J. O., “Bursting and Spalling in Pretensioned Concrete Beams,” *The third FIB International Congress and Exhibition, PCI Annual Convention and Bridge Conference*, Washington, DC, Vol. 2, May 29-June 2, 2010, pp. 1633-1644
10. Marshall, W. T., and Mattock, A. H., "Control of Horizontal Cracking in the Ends of Pretensioned Prestressed Concrete Girders," *PCI Journal*, V. 7, No. 5, Oct. 1962, pp. 56-74.
11. Gergely, P., and Sozen, M. A., "Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams," *PCI Journal*, V. 12, No. 2, April 1967, pp. 63-75.
12. American Association of State Highway and Transportation Officials (AASHTO), AASHTO LRFD Bridge Design Specifications, U.S. Units, 7th Edition, with 2015 Interim Revisions, Washington, DC, 2014, pp. 1960
13. Rigoberto Burgueño, Y. S., and Melissa Donoso, "Stress Transfer Characteristics of Sheathed Strand in Prestressed Concrete Beams: Experimental Study," *PCI Journal*, V. 60, No. 2, March-April 2015, pp. 104-120.
14. WisDOT Bureau of Structures, “Standard Detail Drawings,” Retrieved from http://on.dot.wi.gov/dtid_bos/extranet/structures/LRFD/standards.htm , Updated February 2015, Retrieved May 2015