

5-2008

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Recommended Citation

Hasenkamp, Christie J.; Badie, Sameh S.; Tuan, Christopher Y.; and Tadros, Maher K., "Sources of End Zone Cracking of Pretensioned Concrete Girders" (2008). *Civil Engineering Faculty Proceedings & Presentations*. 5.

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SOURCES OF END ZONE CRACKING OF PRETENSIONED CONCRETE GIRDERS

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ABSTRACT

Recent developments of high performance concrete, increasing amounts of prestressing, and increasing use of deep girders have resulted in increasing popularity of precast pretensioned concrete girders in bridge construction. These developments have increasingly contributed to end zone cracking. This paper summarizes the interim results of an ongoing research sponsored by the National Cooperative Highway Research Program (NCHRP) Project 18-14. The objectives of the research are: (1) to establish procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking, and (2) to prepare a user's manual for the application of these procedures. The results from a national survey of fabricators and users of pretensioned concrete girders and an extensive literature review are presented in this paper.

Keywords: Cracks, End zone, Prestressed, Concrete, Pretensioned, Bridges

INTRODUCTION

Longitudinal web cracks have been observed during prestress transfer, in the ends of precast pretensioned concrete I-girders. The cracks are most visible at the time of lifting girder from the prestressing bed, shortly after prestress release. End zone cracks have also been observed in other girder shapes, such as box girders, voided slabs and tee beams. With the increasing use of high strength concrete, deep girders, thin webs and high prestress forces, these cracks are becoming more prevalent and, in some cases, larger. The current AASHTO LRFD provisions for end zone reinforcement¹ were developed for lower concrete strength and prestress levels than current practice. There is no consensus on predictive methods of longitudinal cracking, level of longitudinal cracking tolerance, and acceptable repair procedures. Although some publications, such as the PCI Repair Manual², ACI Committee 224 report³ and the report by the PCI Committee on Quality Control Performance Criteria⁴ provide guidance on acceptance and repair criteria, these documents need to be validated and combined to establish a unified national approach. The current practice among designers is to provide semi-empirically determined special reinforcement, as close to the member ends as possible, in order to control cracking, and to use crack fillers to fill and seal the cracks that are arbitrarily determined to be too wide.

The University of Nebraska-Lincoln and the George Washington University have been commissioned by the National Cooperative Highway Research Program (NCHRP) to investigate this topic in Project 18-14. The objectives of this project are: (1) to establish procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking, and (2) to prepare a user's manual for the application of these procedures.

This paper gives a summary of the results of a national survey of fabricators and users of pretensioned concrete girders regarding their experience with end zone cracking. It also gives a summary of a literature review.

NATIONAL SURVEY

The national survey was sent to all the state DOTs, selected bridge consultants, bridge girder producers, selected Canadian transportation agencies, members of the PCI Committee on Bridges, and PCI Bridge Producers Committee. The questionnaire included surveys on reinforcement details, strand release process, criteria for repair and rejection of cracked members, and repair methods. Results from the questionnaire have been most helpful in seeing how organizations in the U.S. and other countries have been dealing with this issue.

Forty-four responses were received. Thirty-two responses were from State DOTs, ten responses were from precast concrete producers, one response has been from a consultant, and one response was from a researcher.

Most responses indicated experience in the design, fabrication, or construction of thousands or more linear feet of precast/prestressed concrete girders annually. As anticipated, most state

DOTs deal with I-girders, bulb tees, and box girders. Some also stated that they deal with voided slabs, double tees, and among others, inverted tees. Thirty-six respondents, or 82% of those who replied, said that they experienced longitudinal or diagonal cracks in the webs of the end zones of their girders while only eight said they did not encounter the problem. I-girders and bulb tees seem to be experiencing longitudinal cracking the most. About half stated that only 1-10% of their girders experienced cracking, while the other half stated that cracking occurred in 80-100% of their girders.

56% of those who experienced longitudinal web cracking do not have any official criteria for classifying it. The others use a combination of crack width and crack length. The most prevalent answer in the surveys for acceptance/rejection was criteria based on crack width in the range of 0.006 to 0.025 in. The size of the crack width determines the need for and level of repair. Review of the survey results shows that cracks that are 0.01 in. wide or smaller are sealed by brushing a sealant on the cracks, while cracks that are in the range of 0.01 to 0.025 in. are repaired by epoxy injection. Most of these ranges were set for durability concerns such as to protect the reinforcement from corrosion, and to prevent crack width from growing during freeze and thaw cycles.

Most inspectors used naked eye to determine the extent of cracking. However, seventeen out of the thirty-five who responded also used crack comparators, and five used magnifying scopes.

When asked about established criteria for deciding when to repair cracks, sixteen of the thirty-five who responded said they had no established criteria. The rest repaired cracks based on the crack width. Many used the PCI Repair Manual (PCI MNL-37-06, 2006) as a guideline for repair procedure. Repair is done by either painting a substance over the cracks or by injecting a substance into the crack itself. Larger cracks are injected while smaller ones are just coated. Almost all respondents use a form of epoxy to seal or inject the cracks.

Of the thirty-six who responded, 58% believed that their repair methods do not restore the tensile capacity of the member and 20% believe it only partially restores the tensile capacity. Thirty-two out of those same thirty-six, 89%, do not even believe it is necessary to restore the tensile strength of the girder.

With regards to rejecting a girder due to end zone cracking, most responses said they deal with the beams on a case-by-case basis. Rejection would be based on the width and length of the crack along with its location on the beam, the number of cracks and their proximity to one another. Most stated that rejection is rare or they have never seen a beam rejected for these reasons. The literature review showed that it is a common belief among design engineers, precast producers, and contractors that repaired girders can be used as long as the end zone cracks are sealed and the cracked part of the girder is embedded in the diaphragm. Some DOT agencies such as Washington State DOT believe that these cracks will close up to some extent due to the weight of the girder, deck slab, and barriers. This is because usually the direction of the end zone cracks is normal to the direction of shear cracks, which means

that the end zone cracks will be subject to diagonal compressive stresses that help to close them up.

Of the thirty-five that responded, thirty-one used flame cutting of individual strands as their only method or one of their methods for strand release. Eight used a hydraulic release (jack down) of all strands in one step or of individual strands. Most respondents used a mix of 0.6 in. and 0.5 in. diameter strands in their girders. There was an equal distribution of those that used only 0.6 in. strand diameters and those that used only 0.5 in. strand diameters, so there seems to be no bias towards a preferred strand diameter.

72% of those who responded believe strand distribution contributes to end zone cracking, and 50% believe it is due to detensioning. A few others think that strand size, lifting method, insert locations, and concrete strength also contribute to end zone cracking. Other theories cited were the uneven support of the beam after detensioning, eccentricity of prestressing strand groups, changes in temperature, restraint of forms during curing, form geometry, limitations of debonding, and the presence of draped strands.

LITERATURE REVIEW

CONTROL OF CRACKING IN CONCRETE STRUCTURES

Cracking of concrete structures has been the focus of researchers for decades. Typically, concrete cracks when tensile stresses become higher than the tensile capacity of the concrete. Cracks that are visible to pedestrians can be objectionable from an appearance point of view. Certain types of cracks may present a durability issue if they contribute to corrosion of the reinforcement. Furthermore, cracked horizontal surfaces that are subjected to wetting and drying may deteriorate over time, especially if freeze-thaw cycles occur.

Most of the literature has focused on flexural cracking in reinforced concrete members. Information was also found on cracking due to effects such as shrinkage, temperature, and alkali silica reaction. Information on the effects of web cracking due to prestress release in member ends is almost non-existent.

Previous research has not indicated correlation between the flexural crack width and reinforcement corrosion. Many researchers believe that cracking transverse to the reinforcement has little impact on corrosion. When the ACI 318 building code⁵ introduced serviceability requirements into the code for conventionally reinforced flexural design, the committee purposely modified the Gergely-Lutz crack width equation to emphasize reinforcement detailing rather than crack width. The equation calculated a fictitious "z" factor that was limited to different levels for interior and exterior exposure. The intent was to disguise calculated crack widths to avoid possible unnecessary litigation. The "z" factor was 145 kip/in. and 175 kip/in. for exterior and interior exposures, representing surface crack widths of 0.013 and 0.016 in., respectively. However, these values of anticipated crack width were intentionally omitted from the Code to avoid being taken as exact deterministic values.

Starting from 1999, the ACI building code replaced the “z” factor with minimum reinforcement spacing requirements. The commentary states that “the current provisions for spacing are intended to limit surface cracks to a width that is generally acceptable in practice but may vary widely in a given structure”. The current (2005) provisions⁵ do not have distinction between interior and exterior exposures because “research shows that corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels”.

AASHTO LRFD Specifications¹ have generally followed the ACI building code in adopting serviceably provisions for reinforced concrete, although the current AASHTO spacing provisions are in a slightly different form than ACI. They are slightly more conservative and allow for two classes of exposure.

Web end cracking is most severe when the girder is lifted off the bed. The cracks tend to get smaller and sometimes totally disappear as the vertical gravity loads are introduced by superimposed loads and support reaction. When these cracks are diagonal, they are “normal” to that of the compression struts created by the shearing effects and, thus, are not additive to the principal tensile stresses due to shear. When diaphragms are used, the most severe cracks, at the member ends, are enclosed in the diaphragm concrete. Thus, it appears to be more logical to have less restrictive cracking limitations on web end cracking than on conventionally reinforced concrete sections subject to flexure.

CRACK CONTROL

Review of the literature has shown that crack width has been the most common measure used to quantify acceptable levels of cracks in reinforced concrete structures. The majority of the cracking studies were conducted to investigate flexural cracking in reinforced concrete beams. Flexural cracks are formed on the tension side of a beam, typically at right angles to the reinforcing bars. They largely depend on the concrete cover, level of stress in the steel reinforcement, and distribution of the reinforcement. The majority of the studies concentrated on providing information on sources of cracking, factors affecting crack width, and formulas used to estimate crack width.

In his paper, Nawy⁶ presented the state of knowledge on cracking of concrete structures. The paper focused on flexural cracking behavior in beams and acceptable formulas that could be used to estimate the crack width. Also, the paper gave a discussion and tabulation of the permissible crack width in concrete structures under various exposure conditions. Nawy summarized the flexural crack width values that were collected from his research and others, such as Kaar & Mattock⁷, and Hognestad⁸. The statistical representation of these results showed that the flexural crack width in beams, at 40 ksi tensile stress in reinforcement bars, were in the range from 0.0025 to 0.016 in, with the majority of the results were in the range from 0.005 to 0.010 in. Nawy also gave a summary of the permissible crack widths in concrete structures, under various exposure conditions, which were available at that time. Justification for these limits was more based on “experience” than proven detrimental effects. A reproduction of these limits is given in [Table 1](#).

Table 1 Permissible Crack Widths in Reinforced Concrete Structures⁸

| Source | Exposure condition | Max. Crack Width, inch | |
|---|--|---|-----------------------|
| Brice ⁹ | Severe | 0.004 | |
| | Aggressive | 0.008 | |
| | Normal | 0.012 | |
| Rusch ⁹ | Aggressive (salt water) | 0.008 | |
| | Normal | 0.012 | |
| Etsen ⁹ | Severe to Aggressive | 0.002-0.006 | |
| | Normal (outside) | 0.006-0.010 | |
| | Normal (inside) | 0.010-0.014 | |
| ACI 318-63 | Exterior | 0.010 | |
| | Interior | 0.015 | |
| CEB ¹⁰ | Interior or exterior, aggressive and watertight | 0.004 | |
| | Aggressive | 0.008 | |
| | Normal | 0.012 | |
| US Bureau of Public Roads (Maximum crack width at steel level under service load) ¹¹ | Air or protective membrane | DL causes Compression & LL causes Tension | DL & LL cause Tension |
| | Salt, air water & soil | 0.012 | 0.010 |
| | Deicing chemicals, humidity | 0.010 | 0.008 |
| | Sea water & seawater spray, alternate wetting & drying | 0.008 | 0.006 |
| | | 0.008 | 0.006 |

First edition of the ACI 224 report³, “Control of Cracking in Concrete Structures,” was published by ACI Committee 224 on Cracking in early 1970s. Since then, the report has undergone several revisions. The objectives of the report are to give principal causes of cracking in reinforced/prestressed concrete and recommended crack control criteria and procedures. The report discusses many possible sources of cracking, such as shrinkage cracking, flexural cracking, tension cracking, and end-zone cracking on prestressed concrete members. The ACI report gives the following guidelines, shown in Table 2, for tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions.

Table 2 Tolerable Crack Widths in Reinforced Concrete Structures³

| Exposure Condition | Tolerable Crack Width, inch |
|--|-----------------------------|
| 1. Dry air or protective membrane | 0.016 |
| 2. Humidity, moist air, soil | 0.012 |
| 3. Deicing chemicals | 0.007 |
| 4. Seawater and seawater spray, wetting & drying | 0.006 |
| 5. Water-retaining structures (excluding non-pressure pipes) | 0.004 |

Although the ACI 224 report recommends this table as a practical guide, it states that these values of crack width are not always a reliable indication of steel corrosion and deterioration

of concrete to be expected. The report states that engineering judgment should be exercised and other factors, such as concrete cover, should be taken into consideration to revise these values.

The ACI 224 report recognizes the fact that bursting cracks can develop at ends of prestressed concrete members. The report does not give any guidelines on tolerable crack size for this specific type of cracks. However, it can be interpreted from the report that the limits presented in [Table 2](#) are applicable to all types of cracks regardless their source. The report states the importance of proper design of the bursting reinforcement, and that the first row of the bursting reinforcement should be placed as close as possible to the member end and the rest should be distributed over a certain distance.

In 2006, the Precast/Prestressed Concrete Institute (PCI) published the “*Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products*”². The objective of the report is to achieve a greater degree of uniformity among owners, engineers, and precast producers with respect to the evaluation and repair of precast, prestressed concrete bridge beams. The report recognizes end-of-beam cracking in “Troubleshooting, Item #4.” A summary of the report findings and recommendations are as follows:

- For cracks that intercept or are collinear with strands but without evidence of strand slippage (significant retraction of strand into the beam end), the report recommends injecting the cracks with epoxy.
- The report uses the crack width values developed in ACI 224R-01 as guidelines whether or not to inject cracks. These values are shown in [Table 3](#).
- For cracks that intercept or are collinear with strands with evidence of strand slippage (significant retraction of strand into the beam end), the report recommends injecting the cracks with epoxy and re-computation of stresses after shifting the transfer and development length of affected strands.
- The report recognizes the fact that this type of cracking does not grow once the beam is installed on a bridge. On the contrary, the cracks will close to some extent due to applied dead and live loads, as end reactions provide a clamping force.
- The PCI report does not give any guidelines on when to reject a beam with end cracks.

[Table 3 End-of-Beam Cracks that should be Injected](#)²

| | Exposure Condition | Crack Width, inch |
|----|--|-------------------|
| 1. | Concrete exposed to Humidity | > 0.012 |
| 2. | Concrete subject to Deicing chemicals | > 0.007 |
| 3. | Concrete exposed to seawater and seawater spray, wetting & drying cycles | > 0.006 |

For the environmental criteria, [Table 4](#) gives the maximum crack widths that were recommended by the CEB and Eurocode No. 2¹². These values are valid for a concrete cover of 1.18 in. and for bar diameter not greater than 1.0 in. These cracks width limits were driven based on investigating cracks developed in beams under flexure and concrete members under direct tension and the effect of bar diameter and spacing on the crack width. A summary of the CEB procedure to check bar spacing to control the crack width can be found by

Leonhardt¹³. In this paper, Leonhardt recommended to limit the maximum crack width to 0.008 in. to avoid any concerns by casual observers and the public.

Table 4 Maximum Crack Width¹²

| Ambient condition of exposure | Maximum crack width permitted, inch | Crack appearance |
|-------------------------------|-------------------------------------|-------------------------------------|
| Mild | 0.020 | Easily visible |
| Moderate | 0.016 | Difficult to see with the naked eye |
| Severe | 0.0012 | |

DESIGN SPECIFICATIONS AND DETAILS OF END ZONE REINFORCEMENT

AASHTO LRFD Specifications¹

Article 5.10.10.1 of the AASHTO LRFD Specifications¹ requires that the end zone reinforcement be designed to resist four percent (4%) of the total prestressing force at transfer. The reinforcement must be designed for a stress not exceeding 20 ksi, and should be located within h/4 (one-fourth of the depth of the girder) from the end of the girder.

$$P_r = f_s A_s \geq 0.04 f_{pi} A_{ps} \quad \text{Eq. 1}$$

Where:

P_r = bursting resistance of pretensioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state

f_s = stress in steel not exceeding 20 ksi

A_s = total area of vertical reinforcement located within the distance h/4 from the end of the beam

h = overall depth of precast member

f_{pi} = stress of the strand at transfer

A_{ps} = area of prestressing steel

Also, Article 5.10.10.2 of the AASHTO LRFD Specifications¹ requires that for the distance of $1.5d$ (where d is distance from top flange surface to centroid of tension reinforcement) from the end of the beams, other than box beams, reinforcement shall be placed to confine the prestressing steel in the bottom flange. The reinforcement shall not be less than No. 3 deformed bars, with spacing not exceeding 6.0 in. and shaped to enclose the strands. For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.

Proposed Details by University of Nebraska-Lincoln¹⁴

The proposed procedure states that the end zone reinforcement should be designed to resist four percent (4%) of the prestressing force at release with a uniform stress of 20 ksi. Fifty percent of this reinforcement should be placed h/8 (one-eighth of the depth of the girder) from the end of the beam. The remainder should be placed between h/8 and h/2 from the end.

According to the proposed procedure, the remainder of the end zone reinforcement that is provided between $h/8$ and $h/2$ from the end is not in addition to the vertical shear reinforcement. In this particular distance, i.e. between $h/8$ and $h/2$ from the end, the design engineer should compare the vertical shear reinforcement that is required through this distance with the end zone reinforcement and use whichever is greater.

The proposed details were developed in a research project funded by Nebraska Department of Roads (NDOR), where Tadros and his team of researchers tested a large number of NU I-Girders and Nebraska Inverted Tee Beams. The research concluded that:

- An upper bound on bursting force may be estimated as 3% of the prestressing force, see Fig. 1. However, since the research project did not utilize other types of girders that are commonly used in other states, such as PCI Bulb Tee and Double Tee girders, the final recommendation kept the 4% bursting force given by the LRFD Specifications.

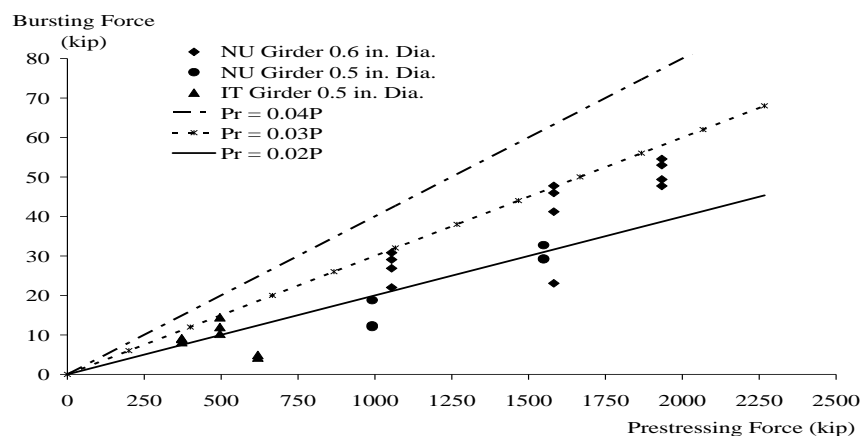


Fig. 1 Bursting Force Comparison¹⁴

- About 20% of the total stress caught by the end zone reinforcement is due to release of the harped strands. The remainder 80% of the total stress is due to release of bottom straight strands. Removal of the hold down devices of the harped strands has almost no effect.
- 60% of the bursting force develops in the end $h/4$, 85% in the end $h/2$, and 100% in the end distance h of the member, see Fig. 2.
- A steel stress limit of 20 ksi may be placed on the bursting reinforcement for crack control. However, the stress in the bursting reinforcement drops sharply with the distance from the end. At $h/8$, the center of the ($h/4$) reinforcement zone in AASHTO, the stress average is only 10.7 ksi according to the experiments, see Fig 2.
- If most of the bursting reinforcement is placed in the end $h/8$, it would have the most effective crack control with the least amount of steel.

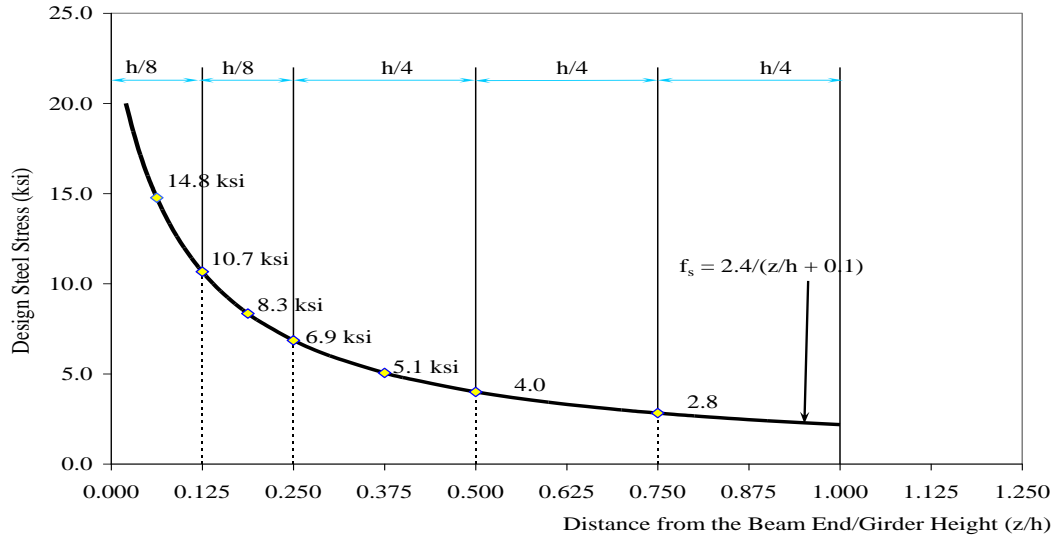


Fig. 2 Average Stress in End Zone Reinforcement vs. Distance from the Member End¹⁴

- Adequate anchorage of the bursting reinforcement should be provided, especially in the end h/8 zone where the stress is highest. To provide anchorage from the bottom side of the bars, the research team proposed welding the end zone reinforcement in this area to the base bearing plate, see Fig. 3. To provide anchorage from the topside of the bars, U-shape bars or headed bars can be used. The U-shape bars or headed bars should be completely embedded in the precast girder.

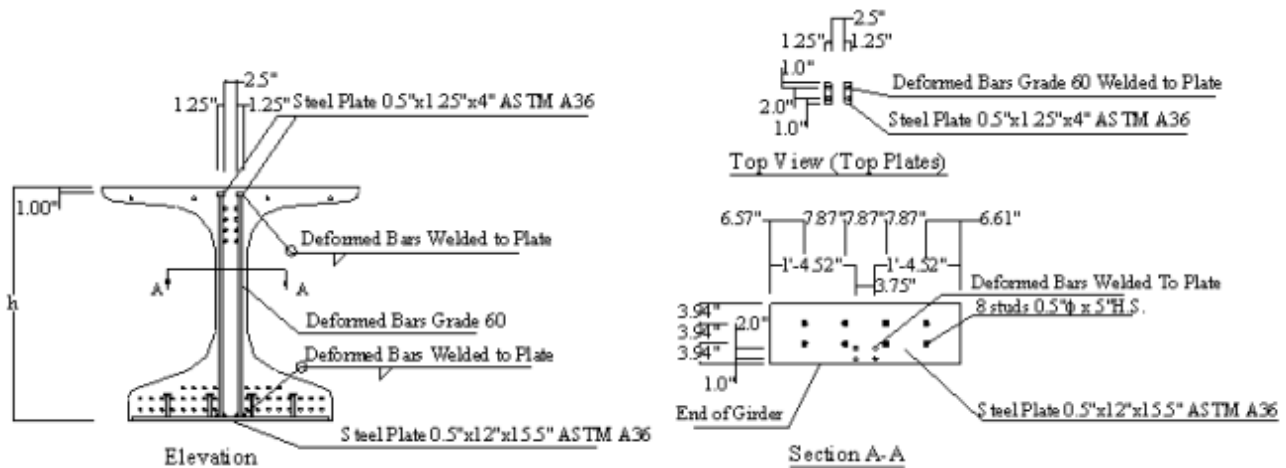


Fig. 3 Anchorage of End Zone Reinforcement for NU I-Girder¹⁴

SOURCES OF END ZONE CRACKING

Longitudinal end zone cracking occurs in pretensioned girders during release of the pretensioned strands. The draped strands are usually released first using flame cutting at the ends and then by removing the hold-down anchorage devices at the harp points. The straight strands are then released by one of two methods: (1) flame cutting, which is a practice used by a large number of precast producers, or (2) gradual release (jack down) in which the

abutment of the prestressing bed is equipped with a hydraulic system that allows it to move gradually towards the concrete member.

During release, the strands grip against the concrete, gradually transferring their force to the concrete girder through a distance known as the transfer length. The force transferred from the strands causes member shortening. The member slides on the bottom pallet, dragging the ends at the bottom. The horizontal sliding is accompanied by upward camber, and the precast member becomes supported at its ends only.

The release process is typically accompanied with formation of longitudinal cracks at the girder ends. These cracks may occur in the web or at the junction between the web and the bottom flange. There are many possible sources that may increase or decrease the likelihood of this longitudinal end zone cracking in pretensioned girders. Within the literature search and the survey responses, multiple sources were suggested:

- a) Method of detensioning: As explained before, the bottom strands can either be flame cut manually while still fully tensioned, or they can be slowly jacked down by a hydraulic release before being cut. Since flame cutting is done manually, the strands are released individually, which creates uneven forces throughout the beam and presents a more localized aggressive introduction of force to the beam. Slowly jacking down the strands prevents the sudden introduction of force that flame cutting causes and gives the concrete girder more time to accommodate the transformed compressive force. Although, hydraulic release is preferred to reduce end zone cracking, very few state DOTs mandate its use because it requires the precast plants to restructure the existing prestressing beds.
- b) Release of the top straight or draped strands before the bottom straight strands: This sequence puts the bottom flange in tension (especially with deep precast members), trying to stretch it out. Since the beam at this stage is in full contact with the bottom form of the prestressing bed, and its bottom flange is restrained by the straight strands that are not released yet, the frictional force produced at the bottom surface of the member resists this movement and produces a vertical crack at the side of the bottom flange that extends vertically towards the web/bottom flange junction. In order to treat this problem, some state DOTs require not to fully tension strands located in the top flange, reduce the height of the draped strands to the level that makes release stresses within their allowable limits, and/or uniformly distribute the draped strands across the web height rather than concentrating them close to or in the top flange.
- c) Order of release of bottom strands with the flame cutting method: Due to limited accessibility of interior strands, the edge strands on each layer are generally released before the interior strands. This order puts the tips of the bottom flange in compression and makes them act as free cantilevers, which initiates horizontal cracking at the web/bottom flange junction or sloped cracks in the web close to its junction with the bottom flange. A specific pattern must be followed in order to not increase cracking. Angular cracks can occur from the stress difference of cut and uncut prestressed strands if the cutting pattern is not idealized. Both ends of the same prestressing strand should also

be cut simultaneously to prevent uneven forces. However, researchers found that the sudden introduction of stress into the girder from flame cutting of the strands is conducive to cracking, even with a planned pattern (Mirza & Tawfik¹⁵, Kannel et al¹⁶).

- d) Length of the free strand in the prestressing bed: As the first strands are cut and the precast member is compressed causing elastic shortening, the remaining uncut strands must lengthen to accommodate the shortening of the member. The resulting tensile force in the uncut strands causes vertical cracks to form near the ends of the member, where the compression from the cut strands has not been fully imparted on the section. This source can be very detrimental in cases where more than one precast member is cast on a single prestressing bed. In a study conducted in 1987 (Mirza & Tawfik¹⁵), researchers found that this source of cracking can be eliminated by making the free strand length between the abutment and the concrete member or between adjacent members as short as needed for fabrication.
- e) Lifting the precast member from the bed (Tuan et al.¹⁴): The prestressing force causes the girder to camber so that the center of the beam is forced higher than the ends. Shortly after prestress release, the precast member is lifted from the bed and moved to the storage area. In most cases where the member is relatively long, the lifting points are generally recessed by as much as 15 to 20 feet from the member ends, at camber raised locations. The lifting point locations are subject to negative moments not only from the prestress but also from the self weight. This latter effect is often ignored by designers. It is a major contributor to the temporary crack widening that occurs at the time of lifting. At this initial lifting of the beam, the prestress force has not yet diminished and is at its highest while the concrete has not yet reached its full strength. It has been known to contribute to downward diagonal cracks in the upper part of the web.
- f) Use of 0.6-inch strands: With the increasing use of concrete with high strength, a number of state highway agencies have begun using 0.6-inch diameter strands at the standard 2-inch spacing in place of the conventional 0.5-inch diameter strands. Previous research has shown that cracks are more extensive with the 0.6-inch strands than with the 0.5-inch strands.
- g) Strand distribution: Girders with a large number of draped strands appear to have more extensive cracking than girders with fewer or no draped strands. The concentration of the prestressing force at the top of the web and the bottom flange increases the bending of the section and the vertical tensile stresses.

Other proposed variables related to end zone cracking include form geometry, beam length, the number of strands, thermal and shrinkage stresses, the number of debonded strands and the debonding lengths, residual stress from curing, restraint of forms during curing, and using forceful means to remove the side forms and bulkheads. From the survey responses, the commonly cited cause was strand distribution (72%), and the second cause was detensioning method (50%).

ACKNOWLEDGEMENTS

This work is part of the ongoing Project 18-14, funded by National Cooperative Highway Research Program (NCHRP). The authors would like to thank Mr. David Beal for his support during various phases of the project. The authors would also like to thank Mark D. Lafferty and Concrete Industries of Lincoln, Nebraska, and Steve Seguirant of Concrete Technology Corporation of Tacoma, Washington, for their contributions to the study. The authors would also like to thank many bridge engineers of state DOTs and members of the precast concrete community that offered their candid input during the national survey.

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