Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web

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TRANSPORTATION RESEARCH BOARD

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COOPERATIVE RESEARCH PROGRAMS

FOREWORD

By Waseem Dekelbab
Staff Officer
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This report establishes a user's manual for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking. The report also proposes revisions to the *AASHTO LRFD Bridge Design Specifications* and provides recommendations to develop improved crack control reinforcement details for use in new girders. The material in this report will be of immediate interest to bridge engineers.

Precast/prestressed concrete girders are widely used in the United States for bridge construction. Longitudinal web cracks have been observed during prestress transfer, particularly at the ends of girders. With the use of higher strength concrete, deeper girders, and significantly higher prestress forces, these cracks are becoming more prevalent and, in some cases, larger. Reactions to these cracks have ranged from doing nothing to rejecting girders. Other reactions include debonding strands at the ends, reducing permissible prestress force, reducing allowable compression stress at the time of transfer, injecting sealants into cracks, and coating the ends of girders with sealants. Clearly, there is no consensus on the causes of longitudinal cracking and what level of longitudinal cracking is unacceptable.

A thorough understanding of whether longitudinal web cracks are of structural significance is needed. If these cracks are not structurally significant, an understanding of whether they reduce durability is required. Although published guidance exists regarding acceptance and repair criteria, these documents need validation.

The research was performed under NCHRP Project 18-14 by the University of Nebraska – Lincoln with the assistance of the George Washington University, Washington, DC. The project established procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking. A user's manual for the application of these procedures was prepared. The report also provides recommendations for improved crack control reinforcement details for use in new girders, and proposes revisions to Article 5.10.10 of the *AASHTO LRFD Bridge Design Specifications* as warranted.

Appendices A through G from the research agency's final report are not published herein but are available on the TRB website. These appendixes are titled as follows.

- Appendix A—Literature Review
- Appendix B—National Survey
- Appendix C—Structural Investigation & Full-Scale Girder Testing
- Appendix D—Sealant Specifications
- Appendix E—ASTM Specifications
- Appendix F—Field Inspection of Bridges
- Appendix G—Design Examples of End Zone Reinforcement

AUTHOR ACKNOWLEDGMENTS

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S U M M A R Y

Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web

Precast/prestressed concrete bridge girders are widely used in the United States. Longitudinal web cracks, often called *end zone cracks,* at the ends of pretensioned concrete girders are commonly observed at the time of strand detensioning, an event generally referred to as *prestress transfer.* During the last two decades, especially with the use of relatively high concrete strength, deep girders, and high levels of prestress, these cracks have become more prevalent. Longitudinal cracks will always develop in prestressed girders if the vertical bursting stresses generated by prestress transfer are greater than the tensile capacity of the concrete. Conventional reinforcement is generally placed to keep the cracks within acceptable width.

In practice, there is no consistent understanding of the impact of end zone cracking on the strength and durability of the girders. Thus, the decisions made by bridge owners vary from doing nothing to total rejection of the girders. Other reactions include debonding of strands at the girder ends, limiting prestress levels, reducing allowable compression stress at the time of prestress transfer, injecting grout into the cracks, and coating the girders' ends with sealants. There is no consensus among owners on the level of tolerance to these longitudinal cracks.

Concerns regarding end zone cracks are based on the possibility of having reduced structural capacity and future durability issues from strand and bar corrosion. End zone cracks that run parallel with and intersect the prestressing strands, reflecting strand locations, could cause debonding. This would result in an increase in the transfer and development lengths, which may consequently reduce the shear and flexural capacity of the girder. Wide reflective cracks along the strands that are exposed to chloride solutions may cause strand deterioration. Therefore, a thorough understanding was needed to determine whether longitudinal web cracks are of structural significance. If these cracks are not structurally significant, an understanding of whether they reduce durability was required.

Published guidelines regarding acceptance and repair criteria of prestressed concrete girders consider many types of cracking that may be reported but do not adequately address the uniqueness of end zone cracking. Also, most of these guidelines are greatly influenced by the criteria developed for flexural cracking in beams, which is fundamentally different in cause and effects from end zone cracking. For example, flexural cracks in beams tend to grow in width and depth with the application of superimposed loads. On the contrary, end zone cracks tend to become narrower with the application of superimposed loads and the development of long term prestress losses.

The primary objective of the work conducted in this research project was to establish a user's manual for the acceptance, repair, or rejection of precast/prestressed concrete girders

with longitudinal web cracking. To achieve this objective, guidelines were required to be established for various cracking categories as follow:

- Cracks that are not required to be repaired,
- Cracks that are required to be repaired, including the methods and materials of repair, and
- Cracks that cause structural capacity to be compromised and thus cause the girders to be rejected.

Additional objectives were to propose revisions to the AASHTO LRFD Bridge Design Specifications as warranted, and to develop improved crack control reinforcement details for use in new girders.

The work conducted by the research team to achieve these objectives consisted of the following:

1. **Structural investigation and full-scale girder testing** to study the effect of end zone cracking on shear and flexural capacities, and to investigate the performance of different amounts and details of end zone reinforcement.
2. **Epoxy injection testing** to investigate the ability of epoxy injection to restore the tensile capacity of cracked concrete across the entire girder web width.
3. **Durability testing** to investigate what repair method and material should be used, if repair is required, and to investigate if the end zone surface should be sealed with a surface sealant—regardless of whether cracks are required to be filled with a patching material.
4. **Field inspection of bridges** to check if the in-service condition of end zone cracking changes with time. The field inspection also was used to investigate if unrepaired end zone cracking leads to corrosion of the reinforcement and/or delamination of the concrete.

Based on the listed tasks and available previous work, the research resulted in establishing the following proposed cracking limits:

- Cracks narrower than 0.012 in. may be left unrepaired.
- Cracks ranging from 0.012 to 0.025 in. should be repaired by filling the cracks with approved specialty cementitious materials, and the end 4 ft of the girder side faces should be coated with an approved sealant. Recommendations are given about several products currently available for this repair and about repair procedure.
- Cracks ranging from 0.025 to 0.050 in. should be filled with either epoxy injection or cementitious patching material, depending on crack width, and then the surface should be coated with a sealant.
- For girders exhibiting cracks wider than 0.05 in., the research team recommends that the girder be rejected. For such girders, the research team believes that the cause of cracking may be beyond just the expected bursting force effects. If the owner wishes to reconsider these girders, it is recommended that a thorough structural analysis for the cause and effect of the cracking be conducted and appropriate measures taken.

Based on full-scale experimental observations and previous research, it was found that end zone cracks can be effectively controlled by concentrating the reinforcement as near the girder ends as allowed by requirements for concrete cover to reinforcement and minimum reinforcement spacing. Further, reinforcement should be gradually reduced within a distance approximately equal to half of the member depth in order to prevent cracks from reopening beyond the zone of concentrated reinforcement. It is thus recommended that the required reinforcement amount in the current AASHTO LRFD Bridge Design Specifications be retained, but the distribution of the reinforcement changed. The reinforcement is still

determined for a bursting force of 4% of the prestressing force and a stress limit of 20 ksi. But, at least 50% of that steel should be placed in the end h/8 of the member (where h is total girder depth). The full amount of bursting steel should be placed in the end h/2 of the member (not h/4) as currently specified. Anchorage of the steel into the top and bottom flanges is most critical for the bars in the h/8 zone as the steel stress rapidly diminishes beyond that zone. Even in the h/8 zone, it is not necessary to develop bars for yield strength as this reinforcement is only for crack control and would experience its highest possible stress in the early stages of girder production and handling. End zone reinforcement should be anchored into the top and bottom flanges to develop at 30 ksi. Recommended reinforcement details are given in this report.

CHAPTER 1

Background

1.1 Problem Statement

Precast/prestressed concrete girders are widely used in the United States for bridge construction. Longitudinal web cracks have been observed during prestress transfer, particularly at the ends of girders. With the use of higher strength concrete, deeper girders, and significantly higher prestress forces, these cracks are becoming more prevalent and, in some cases, larger.

Reactions to these cracks have ranged from doing nothing to rejecting girders. Other reactions include debonding strands at the ends, reducing permissible prestress force, reducing allowable compression stress at the time of transfer, injecting sealants into cracks, and coating the ends of girders with sealants. Clearly, there is no consensus on the causes of the longitudinal cracking and what level of longitudinal cracking is unacceptable.

Concerns regarding end zone cracks are based on the possibility of having reduced structural capacity and future durability issues from strand corrosion. End cracks that run parallel with and intersect with the prestressing strands, reflecting strand locations, could cause debonding. This would result in lengthening of the transfer and development lengths, which may consequently reduce the shear and flexural capacity. Open cracks that travel along the strands and are exposed to chloride solutions may cause strand deterioration. Therefore, a thorough understanding was needed to determine whether longitudinal web cracks are of structural significance. If these cracks are not structurally significant, an understanding of whether they reduce durability was required. Although published guidance exists regarding acceptance and repair criteria, these documents needed validation.

The reader should be aware that the expressions *longitudinal web cracking* and *end zone cracking* are synonymous, and they are used interchangeably in this report.

1.2 Control of Cracking in Concrete Structures

Cracking of concrete structures has been the focus of researchers for decades. A 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.

Some information on cracking due to other effects—such as shrinkage, temperature, and alkali silica reaction—also was found in the literature. However, only a small amount of information on the effects of web cracking due to prestress release in member ends was found. Web end cracking is most severe when the product 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 partially enclosed in the diaphragm concrete. Thus, it appears to be logical to have less restrictive cracking limitations on web end cracking than on conventionally reinforced concrete sections subject to flexure.

1.2.1 Evolution of Permissible Crack Widths

The evolution of, and recommendations for, permissible crack widths developed between 1935 and 1970 can be traced from several references (*1–6*). A summary of the recommendations from these publications is compiled in Table 1.1. It should be noted that the majority of these recommendations were based on flexural cracking in beams. The statistical representation of these recommendations showed that the flexural crack width in beams, at 40 ksi tensile stress in reinforcement

Table 1.1. Permissible crack widths developed between 1935 and 1970.*

Source	Maximum Crack Width (in.)	Conclusion or Exposure Level	Notes
N.J. Rengers, 1935, as contained in Reference 1	≤ 0.012 0.012–0.04 0.040–0.080	- Tolerable crack width - Excessive crack width - Some corrosion danger	Only one specimen tested.
Abeles, 1937, as contained in Reference 1	0.012–0.016	- Present no danger of rusting	Provided there are no special chemical influences.
Tremper, 1947, as contained in Reference 1	0.005–0.050	- Cracks of fairly large widths in a sound concrete will not promote serious corrosion of the reinforcement	Sixty-four concrete blocks were exposed to a marine environment for 10 years.
Brocard, 1957, as contained in Reference 1	0.004 0.024	- Corrosion was not appreciably accelerated. - Corrosion rate increased by a factor of 5 to 10.	The reinforcement consisted of thin walled steel tubes embedded in concrete prisms.
Engel and Leeuwen, 1957, as contained in Reference 1	0.008 0.012	- Unprotected structures (external) - Protected structures (internal)	Recommendations are made from investigations of structures existing for more than 15 years.
Brice, 1957, as contained in References 2 and 7	0.004 0.008 0.012	- Severe exposure - Aggressive exposure - Normal exposure	Flexural cracking in beams.
Rusch, 1957, as contained in References 2 and 7	0.008 0.012	- Aggressive (salt water) - Normal exposure	Flexural cracking in beams.
Etsen, 1957, as contained in References 2 and 7	0.002–0.006 0.006–0.010 0.010–0.014	- Severe to aggressive - Normal exposure (outside) - Normal exposure (inside)	Flexural cracking in beams.
Voellmy, 1958, as contained in Reference 1	0.008 0.008–0.020 > 0.020	- No corrosion occurred - Slight corrosion at isolated regions - More localized corrosion	Cracked beams were exposed to the atmosphere for 10 years. The locations varied from rural to industrial areas.
Bertero, 1958, as contained in Reference 1	0.001–0.006 0.010–0.014	- Exposure to seawater, smoke, etc. - Indoor exposure	These allowable crack widths apply to structural elements under permanent loading with 1-inch cover.
Haas, 1959, as contained in Reference 1	0.008 0.012 > 0.012	- Exposed structures (external environment) - Protected structures (internal environment) - Permissible in the absence of heating, humidity, and other aggressive conditions	These values are applicable only in cases where the reinforcement is adequately covered and where the loads are permanent.
Shalon and Raphael, 1959, as contained in Reference 1	< 0.008 0.008	- Structures exposed to saline air - Exposed structures	
Hendrickson, as contained in Reference 1	0.010	- Acceptable limit for reinforced concrete pipes	Crack widths smaller than 0.01 in. may often close by means of autogenous healing and therefore present little danger of severe corrosive attack.
ACI 1963 Building Code, Section 1508, as contained in Reference 1	0.010 0.015	- Exterior members - Interior members	Determined by tests on actual full-scale flexural members
CEB, 1964, as contained in References 5 and 7	0.004 0.008 0.012	- Interior or exterior, aggressive and watertight - Aggressive - Normal	Flexural cracking in beams.
U.S. Bureau of Public Roads (Maximum crack width at steel level under service load), 1966, as contained in References 6 and 7	DL Causes Compression and LL Causes Tension		Flexural cracking in beams.
	0.008 0.008 0.010 0.012	- Seawater and seawater spray, alternate wetting and drying - Deicing chemicals, humidity - Salt, air water and soil - Air or protective membrane	
	DL and LL Cause Tension		
	0.006 0.006 0.008 0.010	- Seawater and seawater spray, alternate wetting and drying - Deicing chemicals, humidity - Salt, air water and soil - Air or protective membrane	

*Permissible crack widths provided in this table are taken from References 1 through 6.

Table 1.2. Tolerable crack widths in reinforced concrete structures (7).

Exposure Condition	Tolerable Crack Width, in. (mm)
1. Water-retaining structures (excluding non-pressure pipes)	0.004 (0.10)
2. Seawater and seawater spray, wetting and drying	0.006 (0.15)
3. Deicing chemicals	0.007 (0.18)
4. Humidity, moist air, soil	0.012 (0.30)
5. Dry air or protective membrane	0.016 (0.41)

bars, ranged from 0.0025 to 0.016 in, with the majority of the results ranging from 0.005 to 0.010 in.

In the early 1970s, Committee 224 of the American Concrete Institute published the first edition of the ACI 224 report, *Control of Cracking in Concrete Structures,* which give principal causes of cracking in reinforced/prestressed concrete and recommended crack control criteria and procedures (7). Since then, the report has undergone several revisions. 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 report gives the following guidelines, shown in Table 1.2, for tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. The report recommends 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. Although the report does not give any guidelines on tolerable crack size specifically for end zone cracking in pretensioned members, it can be interpreted from the report that the limits presented in Table 1.2 are applicable to all types of cracks regardless of 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 1975, CEB Eurocode No.2 (8) developed limits for cracks developed in beams under flexure and concrete members under direct tension, see Table 1.3.

The limits given in Table 1.3 were developed based on environmental criteria. A summary of the CEB procedure to check bar spacing to control the crack width can be found in Reference 9. In this paper, the author recommended that the maximum crack width be limited to 0.008 in. to avoid any concerns by casual observers and the public.

In 1983, the PCI Committee on Quality Control Performance Criteria developed a report on *Fabrication and Shipment Cracks in Prestressed Hollow-Core Slabs and Double Tees* (10). The report provides a collection of various cracks that may occur in hollow-core slabs and double tees during casting, stripping, or shipping. The objectives of the report are to help precast producers and design engineers identify possible sources of cracking and make decisions on the acceptability of the product. The report recognizes end-of-beam cracking as follows:

- For hollow-core slabs the report provides two types of web cracking that may occur at prestress release due to the bursting forces. The first type is above the strands and the second type is at or near the strands, as shown in Figure 1.1(a) and 1.1(b), respectively. The report states that the crack width of the first type can range from a hairline up to 0.25 in. (6.3 mm). However, it does not provide a crack width for the second type. The report states that these cracks can

Table 1.3. Tolerable crack widths in reinforced concrete structures (8).

Exposure	Maximum Crack Width at Extreme Tensile Fiber of the Concrete Section (in.)	90th Percentile of the Maximum Crack Width (in.)	Appearance
Severe: • Corrosive gasses or soils • Corrosive industrial or maritime environment	0.0012	0.004	Difficult to see with the naked eye
Moderate: • Running water • Inclement weather without aggressive gasses	0.0160	0.008	
Mild: • Conditions where high humidity is reached for a short period in any one year	0.0200	0.012	Easily visible

(a) Above the Strands   (b) At or Near the Strands
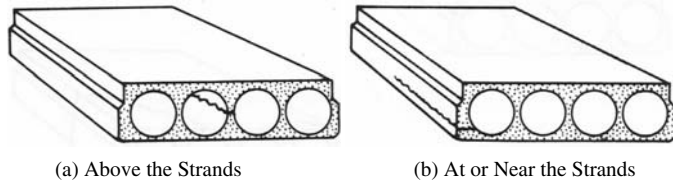
Figure 1.1. End-of-member cracks for hollow-core slabs.

Figure 1.2. End-of-member cracks for double tees.

reduce shear capacity, but it does not give any criteria on when to reject the product. The report gives some repair procedures that range from epoxy injection for small cracks to solid grouting of the voids.

- For double tees the report recognizes horizontal end cracking in the stem during prestress release, as shown in Figure 1.2. It states that the crack length can extend horizontally for a distance of from several inches to a few feet. However, the report does not give any guidelines regarding the crack width or when to reject the product. The report states that if the crack plane coincides with a strand, it may affect the bond between the strand and concrete and increase transfer and development length.

In 2006, the Precast/Prestressed Concrete Institute (PCI) published the *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (*11*). 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 1.4.
- 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.

More information on the permissible crack width is provided in Appendix A, Literature Review, of this report.

1.2.2 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 the following 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

Table 1.4. End-of-beam cracks that should be injected (*11*).

Exposure Condition	Crack Width, in. (mm)
1. Concrete exposed to humidity	> 0.012 (0.30)
2. Concrete subject to deicing chemicals	> 0.007 (0.18)
3. Concrete exposed to seawater and seawater spray, wetting and drying cycles	> 0.006 (0.15)

increase or decrease the likelihood of this longitudinal end zone cracking in pretensioned girders. Within the literature search and the survey responses, the following multiple sources were suggested:

- **Method of detensioning:** As previously explained, 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 departments of transportation (DOTs) mandate its use because it requires the precast plants to restructure the existing prestressing beds.
- **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 may produce 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.
- **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 not to 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 (*12–13*).

- **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 1978 (*12*), 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.
- **Friction with the bottom form of the prestressing bed (*11, 14*):** In cases where the bottom form of the prestressing bed is not properly oiled or has indentions, horizontal cracks are developed in the bottom flange. When cutting the strands the beam may be moved horizontally along the bed floor, causing friction on the surface between the concrete and steel.
- **Heat concentration during flame cutting (*11, 15*):** It was reported that concentrated heating of a strand leads to high sudden shock of the released prestress force. It is always recommended to heat the strand over a long distance to allow slow elongation (annealing), and that flame cutting of strands be done by trained and experienced workers.
- **Lifting the precast member from the bed (*16*):** 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 ft 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.
- **Hoyer Effect (*17*):** Upon release of the prestress, the diameter of the strand expands and pushes against the surrounding concrete. This action, which is known as the Hoyer Effect, improves the bond between the strand and the concrete and helps in transferring the prestress force to concrete. However, it creates radian tensile stresses in the concrete volume, which leads to a radial crack that extends from the strand to the nearest concrete surface at the end sur-

face of the member. This type of cracking generally would be controlled with bottom flange confinement of the concrete around the strands.

- **Use of large strands:** With the increasing use of concrete with high strength, a number of state highway agencies have begun using 0.6-in. diameter strands at the standard 2-in. spacing in place of the conventional 0.5-in. diameter strands. Also, a demonstration project by the Nebraska Department of Roads is under way to implement the use of 0.7-in. strands on the Pacific Street overpass over Interstate I-680 in Omaha. A full-scale specimen was fabricated for the University of Nebraska research team. The specimen, an NU 900 (36-in. deep) I-girder was prestressed with twenty-four 0.7-in. strands at 2.2 in. horizontally and 2.25 in. vertically. This prestress was the same amount required for a two-span bridge, 100-ft span, 10-ft, 10-in. spacing. Previous research at the university established that cracks are more extensive with the larger 0.6-in. and 0.7-in. strands than with the 0.5-in. strands.
- **Inadequate design of end zone reinforcement:** Increased vertical reinforcement concentrated at the ends of the girder has been shown to reduce the lengths and widths of end zone cracks. Therefore, insufficient amounts of end reinforcement or misplacement of the bars too far away from the edges may increase the amount of cracking experienced. Also, the lack of confinement stirrups around the prestressing strands may increase cracking. It should be noted that the end zone reinforcement is not presented to eliminate end zone cracking but to control it.
- **Concrete type:** Lightweight concrete has a reduced tensile strength capacity and modulus of elasticity, and is therefore less able to withstand the extreme prestressing forces. This leads to longer, wider, and a larger quantity of cracking along the ends.
- **Low concrete release strength:** The concrete must be allowed to set and cure long enough to reach certain strength before release. This strength value, known as the minimum release strength, assures that the concrete is strong enough to handle the prestressing forces. If the concrete does not reach this strength, it may be too weak to resist the prestressing forces, leading to cracking.
- **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 most commonly cited cause was strand distribution (72%), and the second most commonly cited cause was detensioning (50%). More discussion on sources of end zone reinforcement is given in Appendix A, Literature Review, of this report.

1.2.3 Design of End Zone Reinforcement

Design of end zone reinforcement details is typically done by (1) estimating the bursting force (vertical tensile force developed at ends of pretensioned precast concrete girders during prestress release) as a percentage of the total prestressing force just before release, (2) setting a limit on the stress of the required end zone reinforcement that allows the designer to control the size of the cracks and keeps them within acceptable limits, and (3) providing a scheme on how to distribute the end zone reinforcement.

A literature search has shown that most of the design methods require that the end zone reinforcement be designed to resist about 4% of the total prestressing force at transfer, and that the reinforcement must be designed for a service stress not exceeding 20 ksi. However, there is no agreement on how to distribute this reinforcement in the end zone areas. For example, Article 5.10.10.1 of the AASHTO LRFD specifications (*18*) states that it should be located within h/4 (one-fourth of the depth of the girder) from the end of the girder, while recent research conducted by the University of Nebraska (*16*), and adopted by Alberta DOT, Canada, has recommended that 50% of this reinforcement should be placed h/8 (one-eighth of the depth of the girder) from the end of the beam and the remainder should be placed between h/8 and h/2 from the end. In addition to the disagreement on the distribution of the end zone reinforcement, some highway authorities require a specific way of anchoring the end zone reinforcement. For example, the Illinois Department of Transportation (IDOT) requires that the end zone reinforcement should be made of ¾-in. diameter threaded rods that are welded to a 1-in.-thick plate embedded in the bottom flange, as shown in Figure 1.3. Also, the threaded rods are anchored at the top surface of the girder through a ¾-in.-thick plate with nuts. Another example is the bursting reinforcement detail recommended and used by Central Pre-Mix Prestress Co. of Spokane, Washington, where a single #8 bar that travels vertically through the center of the girder is bent back into the interior of the beam at both the top and bottom, as shown in Figure 1.4.

More discussion on end zone reinforcement details used by various highway authorities is given in Appendix A, Literature Review, of this report.
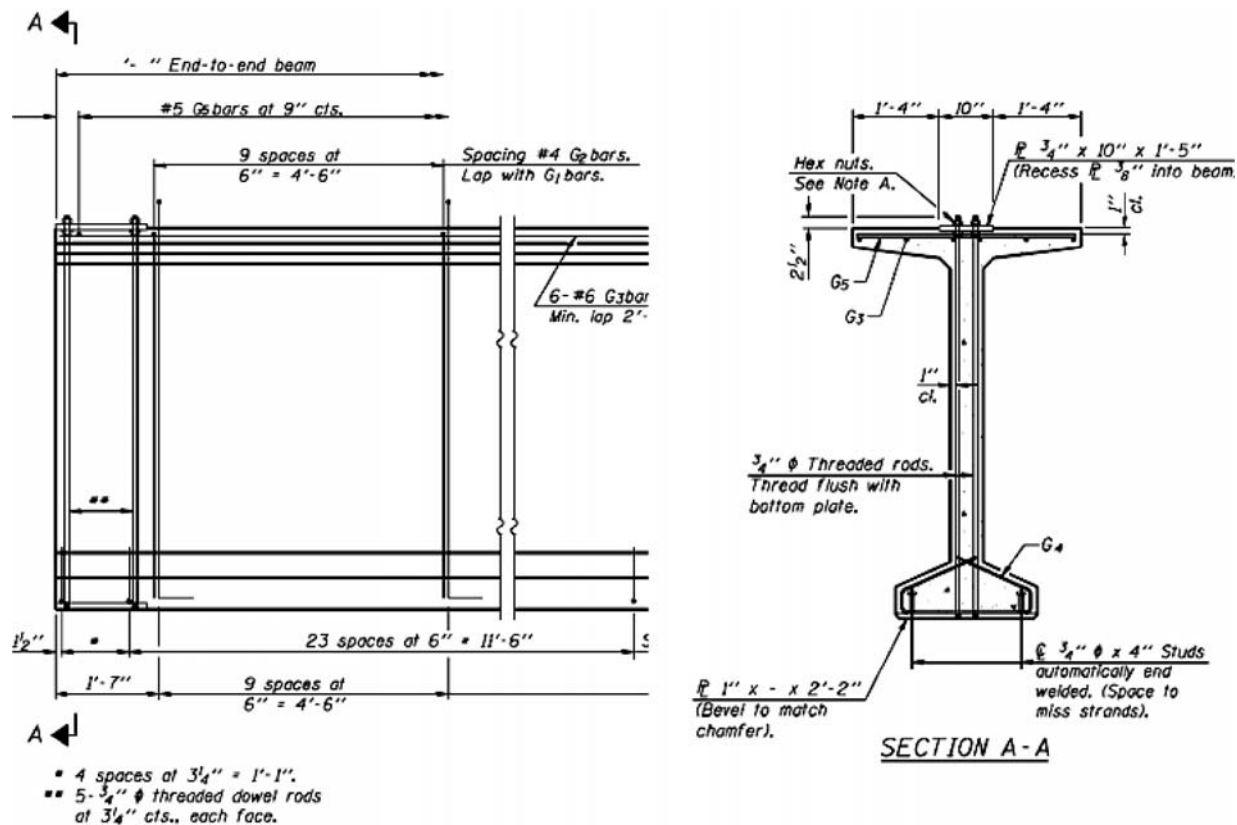
Figure 1.3. End zone reinforcement detail used by IDOT.

1.3 Methods and Materials Used for Repair

The following three methods of repair were found in the literature review:

1. Epoxy injection,
2. Batching and sealing the cracks, and
3. Sealing the cracks.

Elevation   End View
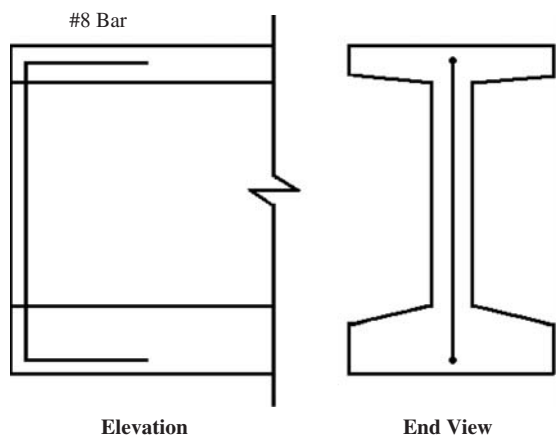
Figure 1.4. End zone detail used by Central Pre-Mix Prestress Co., Spokane, Washington.

Use of any of these methods depends on the crack width and criteria used by the highway authorities.

1.3.1 Epoxy Injection Procedure by PCI Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products

The PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (*11*) provides detailed information on the process and steps used in the epoxy injection of cracks in precast, prestressed concrete bridge products. The responses to the national survey that was conducted in this research project have shown that this manual is currently used by many state DOTs and precast producers for repair of end zone cracking. Chapter 4 of the PCI publication states that the epoxy injection procedure applies to cracks that are wider than 0.006 in. or to cracks that are noticeable after soaking with water. The publication provides the epoxy injection procedure for two cases (1) cracks accessible and visible from both sides, and (2) blind cracks that are not visible or accessible from both sides. Chapter 5 of the same publication provides information on how to prepare these cracks. It also states that 75 psi to 200 psi injection pressure is commonly used for crack widths in the range of 0.006 to 0.007 in.

1.3.2 Batching Materials and Sealants

Batching materials typically are used to fill the cracks but not to restore the concrete tensile strength at the cracks; sealants are used to create impermeable concrete surfaces that prevent moisture from getting into the girders and causing corrosion of the reinforcement. A wide range of commercial batching materials and sealants are available in the market. Precast producers make their choice based on experience and performance of the selected material. Appendix A of this report provides some examples of batching materials and sealants used by precast producers in Nebraska and Washington State.

1.4 Objective and Scope of the Research

The NCHRP objective of this project is stated as follows:

To establish procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking. A user's manual for the application of these procedures will be prepared based on the research findings.

The research team also attempted to include as many of the following related objectives as practicable, without interfering with the accomplishments stated in the primary objective:

1. Develop limits for cracking that can not be tolerated, resulting in product rejection.
2. Develop repair methods for cracking that is tolerable but must be repaired before the girder is used.
3. Develop guidelines for cracks that are not required to be repaired. These guidelines must be based on both initial and long-term performance and on the potential for reinforcement corrosion or further crack propagation. This objective must focus on how the girders are used and in what environmental conditions.
4. Develop a user's manual for acceptance criteria and repair materials and methods at the precast plant. Repair materials and methods must be validated before they are included in the manual.
5. Develop improved crack control reinforcement details for use in new girders.
6. Propose revisions to the AASHTO LRFD Bridge Design Specifications as warranted.

To accomplish these objectives, the following tasks were performed:

Task 1. The research team reviewed and interpreted U.S. and international practices, performance data, research findings, and other information related to the types, causes, mitigation, evaluation, and acceptance or repair of longitu-

dinal web cracking in precast/prestressed concrete girders. In addition to the literature review, a national survey was developed to collect information on the experience regarding longitudinal end zone cracking. The national survey was sent to all state DOTs, other owner agencies, selected bridge consultants, and precast concrete producers. It was also sent to about 150 PCI bridge product producers, the PCI Committee on Bridges, the PCI Bridge Producers Committee, and selected Canadian agencies. The questionnaire included questions on reinforcement details, strand release process, criteria for repair and rejection of cracked members, and repair methods.

Task 2. Using information assembled in Task 1, the factors that alone or in combination may initiate or propagate longitudinal web cracking were identified and documented.

Task 3. Using the findings from Tasks 1 and 2, the research team proposed evaluation criteria for assessing the strength and durability consequences of longitudinal web cracking. The evaluation included consideration of when repairs are required and what repair methods are appropriate.

Task 4. The research team prepared a future work plan to develop and validate the evaluation criteria and repair methods proposed in Task 3.

Task 5. Tasks 1 through 5 were conducted within four months of the contract start and an interim report, documenting the results of Tasks 1 through 4, was developed and submitted to NCHRP for review. The report included the future work plan envisioned by the research team to reach the project objectives (Tasks 6 through 8). Two months were given to the members of the project panel to review the interim report. Then, the research team had a meeting with the panel to discuss their comments on the interim report and future work plan. Based on this meeting, a modified version of the future work plan was developed by the research team and submitted to NCHRP for final approval.

Task 6. Based on the results collected from Task 6, the research team prepared a manual for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking. The manual went through several revisions based on the comments received from the project panel.

Task 7. The research team developed a final report describing the entire research effort.

1.5 Applicability of Results to Highway Practice

The project was structured to provide a manual of procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking. The manual, which is provided in Chapter 3 of this report, can

be used by highway authorities to develop their own criteria for acceptance or rejection. The information provided in Appendices A and B also can provide the decision maker with a clear view of possible sources of longitudinal web cracking, available reinforcement details used to control longitudinal web cracking, and acceptance/rejection criteria used by other highway authorities.

1.6 Organization of the Report

This report consists of four chapters and seven appendices, as follows:

- Chapter 1 provides the problem statement, current knowledge of the problem, research objectives, and scope of the research, and applicability of results to highway engineering practice.
- Chapter 2 summarizes the approach developed by the research team to reach the objectives of the project.
- Chapter 3 provides information on the project activities and findings, which include (1) a summary of results of the national survey, (2) full-scale testing of eight girders, (3) a durability test, which included three phases, (4) a manual of procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with longitudinal web cracking, and (5) recommended end zone reinforcement detail.
- Chapter 4 summarizes the significant conclusions of this project and suggestions for future research.

The following appendices are not published herein. To find Appendices A through G for this report, go to www.trb.org and search for "NCHRP Report 654".

- Appendix A provides a summary of the information collected from the literature review and national survey.
- Appendix B provides the national survey and its responses.
- Appendix C provides information on the full-scale girder test regarding fabrication, testing, and analysis of test results.
- Appendix D provides information on the sealants that were used in the durability testing.
- Appendix E provides the ASTM Specifications used in the durability testing.
- Appendix F provides information on the field inspection of bridges in Nebraska and Virginia.
- Appendix G provides design examples of end zone reinforcement using the LRFD Specifications and the proposed details.

CHAPTER 2

Research Approach

The research team developed and executed the following approach to achieve the project objectives listed in Chapter 1:

- **Literature search:** The research team searched the literature looking for performance criteria and data of prestressed girders where end zone cracking was reported. The search included national and international resources such as journal papers and reports published in the past 60 years. Also, the search tried to collect information on any acceptance or rejection measures developed for prestressed girders with end zone reinforcement. The research team found that most of the available measures are related to flexural cracks in conventionally reinforced beams. Very few publications that deal with end zone cracking of prestressed bridge girders were available. The literature search showed that there is no unified approach or set of criteria that is available and widely accepted by highway authorities in the United States. The majority of publications on end zone cracking agreed that crack width is the best measure that can be used to develop practical acceptance/rejection criteria.
- **National survey:** After searching the literature, and due to the lack of information on polices used by precast producers and highway authorities regarding acceptance/refusal of girders with end zone cracking, the research team developed a national survey to collect information on the experience regarding longitudinal end zone cracking. The national survey was sent to all of the state DOTs, other owner agencies, selected bridge consultants, and precast concrete producers. It was also sent to about 150 PCI bridge product producers, the PCI Committee on Bridges, the PCI Bridge Producers Committee, and selected Canadian agencies. The questionnaire included questions on reinforcement details, strand release process, criteria for repair and rejection of cracked members, and repair methods. The national survey and its results are provided in Appendix B. Also, a summary of the survey results is presented in Chapter 3.

- **Required tasks:** Based on the information collected from the literature review and analysis of the national survey results, the research team identified the following set of issues that, if addressed, would help establish procedures for the acceptance, repair, or rejection of precast/prestressed concrete girders with end zone cracking. These issues are
  1. Effect of end zone cracking on structural capacity, durability and aesthetics of prestressed concrete girders.
  2. Improvement of the current design of end zone reinforcement to reflect recent usage of high-strength concrete and high levels of prestress.
  3. Methods and material of end zone crack repair, if required.
- **Work plan:** To investigate these issues, the research team developed the following work plan. The plan has four Task 6 subtasks of the project, as shown in Table 2.1. A set of questions were developed to be answered by each subtask. Details of work in each subtask including results and conclusions are presented in Chapter 3.
- **Project deliverables:** After the subtasks listed in Table 2.1 were conducted and the results were analyzed, the research team developed
  1. A user's manual for acceptance criteria and repair materials and methods.
  2. Improved crack control reinforcement details for use in new girders.
  3. Proposed revisions to the AASHTO LRFD Bridge Design Specifications.

Table 2.1. Questions and corresponding work-plan subtasks developed to reach project objectives.

Questions	Work-Plan Subtasks
1. Does end zone cracking negatively affect the flexural and shear capacities of prestressed girders? 2. Do variations of the end zone reinforcement details have significant effect on the number, width, and pattern of end zone cracks?	1. Structural Investigation and Full-Scale Girder Testing Analysis of previous work, identification of influencing parameters, and determination of potential structural effect of end zone cracking led to the design and testing of eight full-scale girders. Two girders were fabricated in each of four states: FL, TN, VA and WA. Shear capacity, flexural capacity, and variations of end zone reinforcement details were included. These details included LRFD recommendations, proposed detail, and, if available, local practices.
3. If epoxy injection is used to repair end zone cracking, can repair restore the tensile capacity of the cracked concrete? 4. Is epoxy injection capable of completely filling the crack through the width of the web?	2. Epoxy Injection Testing Two 12-ft long specimens were fabricated by Concrete Industries (CI), Lincoln, NE, as part of an NU 1350 (53-in. deep) bridge girder production. The first specimen was fabricated with only shear reinforcement and no additional end zone reinforcement. The second specimen was fabricated with a different method of end zone reinforcement at each end (LRFD method and the proposed method). The specimens were repaired using epoxy injection by CI staff using the procedure from PCI's *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products (11)*. Sections of the web at the ends were saw cut, visually examined, and structurally tested.
5. If repair is required, what repair method and material should be used? 6. Should the end zone surface be sealed with a surface sealant regardless of whether cracks are required to be filled with a patching material?	3. Durability Testing This testing was conducted in three stages. In Stage I, 60 4×8-in. concrete cylinders were fabricated and sealed using five commercial sealants. ASTM D6489-99 Specification, *Water Absorption Test of Hardened Concrete Coated with Water Repellant*. Sealant effectiveness was evaluated. The best performing sealants were selected for further testing. In Stage II, 49 prisms were fabricated with preformed cracks ranging in width from 0.007 to 0.054 in. The results of Stage II were verified through a series of tests on 69 prisms in Stage III.
7. Does the width of end zone cracking change with time? 8. If end zone cracking is detected at the precast plant and no repair was conducted, do these cracks lead to corrosion of the strands and bars, or delamination of the concrete?	4. Field Inspection of Bridges The research team developed criteria for bridges to be inspected. Two states (Nebraska and Virginia) were targeted. Several bridges were inspected in each state. The inspection process included • Collection of reports of inspection conducted at the plant. Examination of the report and identification of repair method and material. • Collection of inspection reports of the bridges in service. • Visits by the research team of the bridges under study. The inspection included observation of crack growth since production and of signs of reinforcement corrosion and concrete delamination.

CHAPTER 3

Research Findings

This chapter presents the results and findings of the work plan developed by the research team and reported in Chapter 2. In order to keep the size of this report within acceptable limits, detailed discussions on the material covered in this chapter are provided in Appendices B through G, which are not provided herein (to find Appendices A through G for this report, go to www.trb.org and search for "NCHRP Report 654"). The contents of Chapter 3 and the corresponding appendices are as follow.

Subtasks and Deliverables	Section	Appendix
• National Survey	3.1	B
• Structural Investigation and Full-Scale Girder Testing	3.2	C
• Epoxy Injection Testing	3.3	—
• Durability Testing	3.4	D and E
• Field Inspection of Bridges	3.5	F
• Manual of Acceptance, Repair, or Rejection	3.6	—
• Improved Crack Control Reinforcement Details for Use in New Girders	3.7	G
• Proposed Revisions to the AASHTO LRFD Bridge Design Specifications	3.8	—

3.1 National Survey

The research team developed a questionnaire to survey experiences regarding longitudinal end zone cracking. It was sent to all the state DOTs, other owner agencies, selected bridge consultants, and precast concrete producers. It was also sent to about 150 PCI bridge product producers, the PCI Committee on Bridges, the PCI Bridge Producers Committee, and selected Canadian agencies. The questionnaire included surveys on reinforcement details, strand release process, criteria for repair and rejection of cracked members, and repair methods. A copy of the survey is presented in Appendix B. Results from the questionnaire have been most helpful in seeing how organizations around the country and beyond have been dealing with this issue.

The research team received 44 responses, which have been compiled and summarized in Appendix B. There were 32 responses from state DOTs, 10 responses from precast concrete producers, 1 response from a consultant, and 1 response from a researcher.

Most responses indicated experience in the design, fabrication, or construction of thousands of 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, but only eight said they did not encounter the problem. I-girders and bulb tees seem to be experiencing the longitudinal cracking the most. About half of the responses stated that only 1% to 10% of their girders experienced cracking, while the other half stated that cracking occurred in 80% to 100% of their girders.

Of those who experienced longitudinal web cracking, 56% 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 width determines the need for, and level of, repair. The literature review shows that cracks that are 0.01 in. wide or smaller can be sealed just by using a brush-on sealant, but cracks that are in the range of 0.01 to 0.025 in. must be repaired by epoxy injection. Most of these ranges were set for durability aspects, to protect the reinforcement from corrosion and the crack width from growing during freeze and thaw cycles.

Most inspectors stated that routine inspection is used to determine the extent of cracking. However, 17 of the 36 who experienced end zone cracking used crack comparators, shown in Figure 3.1, and 5 used magnifying scopes.
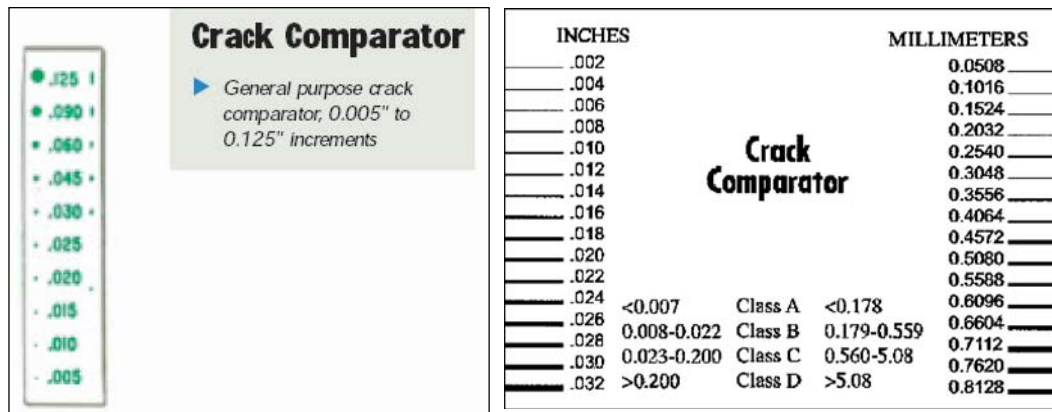
Figure 3.1. Crack comparator.

When asked about established criteria for deciding when to repair cracks, 16 of the 36 who responded said they had no established criteria. The rest of the respondents repaired cracks based on the size of the crack. Many used the PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (11) 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. Large cracks are injected and small cracks are just coated. Almost all respondents use a form of epoxy to seal or inject the cracks.

Of the 36 who experienced end zone cracking, 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 of the 36, 89%, do not believe it is even necessary to restore the tensile strength of the girder.

In regard to rejecting a girder due to end zone cracking, most respondents 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 placement 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 36 who experienced end zone cracking, 31 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.

Of those who responded, 72% 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. Other theories given 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.

3.2 Structural Investigation and Full-Scale Girder Testing

3.2.1 Introduction

The objectives of the full-scale girder testing were to investigate (1) if end zone cracking negatively affects the flexural and shear capacities of prestressed girders, and (2) if variations of the end zone reinforcement details have significant effect on the number, width, and pattern of end zone cracks. The test plan had eight full-scale girders fabricated in four states with different end zone reinforcement details. This was done through direct contact between the research team and precast concrete girder producers in four states (Tennessee, Florida, Virginia, and Washington). Each precast producer agreed to fabricate two specimens as part of an actual bridge girder project. This was done through the following steps:

1. Each precast concrete girder company picked an actual bridge girder project where the precast girders would be manufactured in its yard. The criteria for a good candidate project were: (a) the girders should be packed with a large number of strands in order to produce end zone cracking, and (b) the type of girder should be different from those picked by other precast producers in order to have four different types of girders tested in the project.
2. The precast concrete girder companies provided the research team with details of reinforcement of the actual girders and their time plan to fabricate these girders.

3. For each bridge project, the research team designed two 42-ft-long specimens using the same number of strands used in the actual bridge. At least one of the four ends of the specimens had to have the same end zone reinforcement details used on the actual bridge.

4. The precast concrete girder companies reviewed the details of the specimens and tried to find the right time to cast them next to some of the girders of the actual bridge project. Therefore, the specimens were fabricated using the same material in the production of the girder of the actual bridge and received the same level of treatment regarding curing and strand release technique.

5. The precast concrete girder companies in Washington, Virginia, and Florida allowed the research team to be present at time of prestress release and to record any end zone cracking that might appear. Most of the specimens

were shipped to the structures laboratory in Omaha, Nebraska, within a month from their production date. Concrete cylinders made during production and coupons of rebars were also sent to the structures laboratory with the specimens.

3.2.2 Description of the Test Specimens and Test Setup

Table 3.1 summarizes the details of the eight specimens. The details in this table include the specimen type, type of end zone reinforcement (EZR) details, material properties, number of prestress strands, and type of failure. Specimens are listed in the order in which they were fabricated and tested.

The "proposed" detail was developed by the research team based on the research that was conducted at the University

Table 3.1. Design criteria of the full-scale specimens.

State / Girder Type	Girder #1		Girder #2	
	Left End — EZR Type — Repair	Right End — EZR Type — Repair	Left End — EZR Type — Repair	Right End — EZR Type — Repair
Tennessee Type III AASHTO Beams	TN1L LRFD 2007 EZR No repair	TN1R Proposed EZR No repair	TN2L TN DOT EZR No repair	TN2R Proposed EZR* No repair
	Construction Products, Inc., Jackson, Tennessee Designed to fail in <u>flexure</u> $f_{ci}' = 6{,}000$ psi, $f_c' = 7{,}000$ psi Bottom: 30 straight 0.5 in., 270 ksi, low relaxation strands stressed to 33.8 kips Top: 2 straight 0.5 in., 270 ksi, low relaxation strands stressed to 5 kips 7.5 in. thick CIP slab was added in the lab, $f_c' = 9{,}000$ psi			
Washington State WF58G (Wide Flange Super Girder)	WA1L Proposed EZR* No repair	WA1R LRFD 2007 EZR No repair	WA2L NO EZR No repair	WA2R NO EZR Epoxy Injection
	Concrete Technology Corporation (CTC), Tacoma, Washington Designed to fail in <u>shear</u> $f_{ci}' = 6{,}000$ psi, $f_c' = 8{,}000$ psi Bottom: 38 straight 0.6 in., 270 ksi, low relaxation strands jacked to 43.9 kips Top: 20 straight 0.6 in., 270 ksi, low relaxation strands jacked to 43.9 kips + 4 "temporary" post-tension 0.6 in. diameter strands			
Virginia PCEF45 (VA new Bulb-Tee)	VA1L No EZR No repair	VA1R No EZR No repair	VA2L LRFD 2007 EZR No repair	VA2R Proposed EZR* No repair
	Bayshore Concrete Products, Cape Charles, Virginia Designed to fail in <u>flexure</u> $f_{ci}' = 6{,}000$ psi, $f_c' = 8{,}500$ psi Bottom: 38 straight 0.6 in., 270 ksi, low relaxation strands jacked to 43.9 kips Top: 14 straight 0.6 in., 270 ksi, low relaxation strands jacked to 43.9 kips 4-in. thick, 47-in. wide deck slab was cast monolithically with the top flange			
Florida 60-in. deep inverted T beams	FL1L FL DOT EZR No repair	FL1R Mod. FL DOT EZR No repair	FL2L LRFD 2007 EZR No repair	FL2R Proposed EZR* No repair
	Standard Concrete Products, Tampa, Florida Designed to fail in <u>flexure</u> $f_{ci}' = 6{,}000$ psi, $f_c' = 8{,}500$ psi Bottom: 36 straight 0.6 in., 270 ksi, low relaxation strands jacked to 43.9 kips 10-in. thick, 24-in. wide CIP deck was added in the lab, $f_c' = 10{,}000$ psi			

* Proposed EZR is the end zone reinforcement recommended by Tuan et al. (*16*) and discussed in Section 1.2.3 of this report.

of Nebraska (*16*). As explained in Chapter 2, the LRFD Specifications (*18*) and University of Nebraska proposed detail (*16*), recommend that the end zone reinforcement should be designed to resist 4% of the total prestressing force at transfer, and that the reinforcement should be designed for a service stress not exceeding 20 ksi. However, the LRFD Specifications states that this reinforcement should be distributed within h/4 (one-fourth of the depth of the girder) from the end of the girder, while the University of Nebraska proposed detail recommends that 50% of this reinforcement should be placed within h/8 (one-eighth of the depth of the girder) from the end of the beam and the remainder should be placed between h/8 and h/2 from the end.

3.2.2.1 Tennessee Specimens

Construction Products, Inc. of Jackson, Tennessee, fabricated two 42-ft-long Type III AASHTO I-girders for the project. Each specimen had thirty 0.5-in. diameter, 270 ksi, low relaxation prestressing strands, stressed to 33.8 kips per strand. They also contained two partially stressed 0.5-in. diameter strands in the top flange, stressed to 5 kips per strand. The specimens were designed to fail in flexure. Of the four ends, two had the end zone reinforcement designed to the proposed design; one was designed using LRFD specifications, and one contained the same end zone reinforcement existing on the typical Tennessee production girders. Figures 3.2 through 3.5 show the details of the Tennessee specimens.
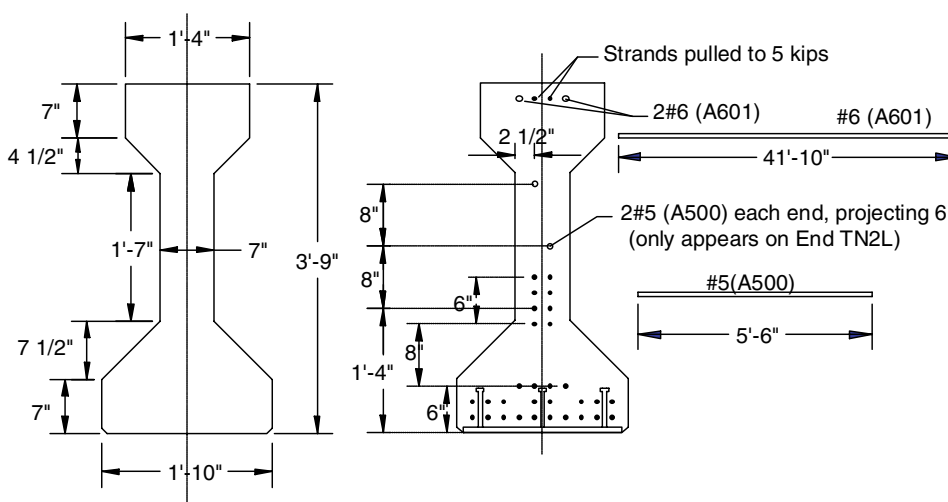
(a) Longitudinal Reinforcement
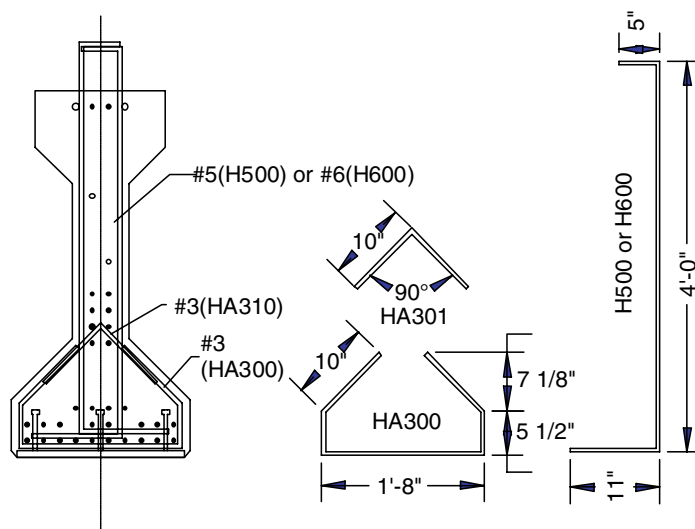
(b) Shear and Confinement Reinforcement

Figure 3.2. Cross section details of Tennessee specimens.
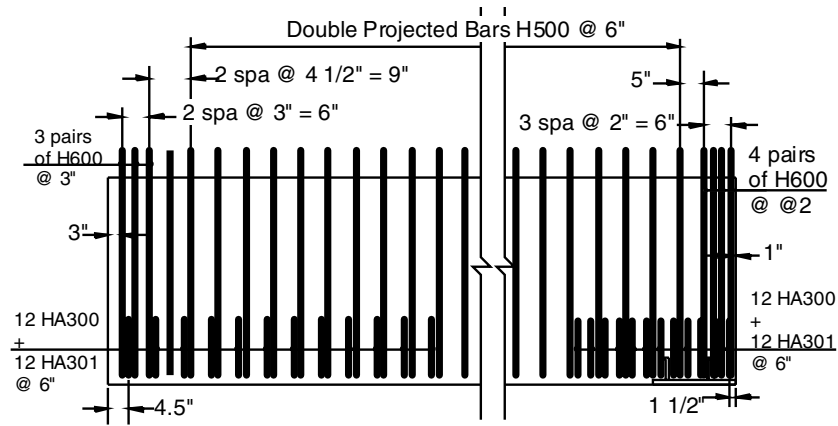
Figure 3.3. End zone reinforcement details of TN1L (LRFD) and TN1R (proposed).

3.2.2.2 Washington State Specimens

Concrete Technology Corporation (CTC) of Tacoma, Washington, produced two 42-ft long, 58-in. deep Washington Super Girders (Wide Flange Girders). Each specimen contained 38 straight 0.6-in. diameter, 270 ksi, low relaxation prestressing strands in the bottom portion of the girder, jacked to 43.9 kips per strand. At the top of the web, each specimen contains 20 additional straight 0.6-in diameter prestressing strands and 4 "temporary" post-tension 0.6-in diameter strands. The four "temporary" strands were included in an attempt to amplify the end zone cracking as much as possible.

The production girders that the Washington specimens were modeled after contained 20 draped prestressing strands. However, none of the girder specimens manufactured for the structural testing contained draped strands, so the top strands in the Washington specimens remained straight. Having the prestressing strands remain straight at the top of the girder

was more critical than having the strands draped. The force at the top of the girder from the prestressed strands and the post-tensioned strands created additional stresses in the girder web, amplifying the end zone cracks. The top and bottom strands apply opposing flexural moments creating vertical tensile forces in the web.

The specimens were designed to fail in shear. The first girder had one end designed using the AASHTO LRFD specifications and the other using the proposed improved reinforcement design procedure. The other girder did not contain any additional end zone reinforcement other than the typical shear reinforcement. This was done to create the maximum amount of end zone cracking possible for the girder. The precast producer stated that if they had a production girder that showed the extent of end cracking experienced by the test specimens, it would not be accepted. One of the ends that did not contain additional end reinforcement received an epoxy injection repair at the precast yard using the typical epoxy repair

Figure 3.4. End zone reinforcement details of TN2L (TN) and TN2R (proposed).

Figure 3.5. Girder TN1 decking cross section.

(a) Longitudinal Reinforcement

(b) Shear and Confinement Reinforcement

Figure 3.6. Details of Washington specimens.

procedure outlined in *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (*11*). The matching end was not repaired in any way. Figures 3.6 through 3.8 show the details of the Washington State specimens.

3.2.2.3 Virginia Specimens

Bayshore Concrete Products of Cape Charles, Virginia, donated two 42-ft long, 45-in. high bulb-T girders with thirty-eight 0.6-in. diameter, 270 ksi, low relaxation prestressing strands in the bottom flange and fourteen 0.6-in. diameter, 270 ksi, low relaxation prestressing strands in the top flange of the girders, each tensioned to 44 kips. The straight prestressing strands in the top of the girder were designed to create additional vertical tensile stresses in the girder web, amplifying end zone cracks. These girders were designed to fail in flexure. Much like the Washington girders, two of the girder ends did not contain any additional end zone reinforcement other than the typical shear reinforcement. The remaining ends were designed using the AASHTO LRFD Specification and the proposed improved details. In the end that was designed using the proposed details, a #8 C-shaped bar was placed at 1 in. clear cover, in the same cross-sectional plane as one of the pairs of #5 bars in order to get a larger amount of steel closer to the girder end. None of the four ends were repaired in any way. Figures 3.9 through 3.11 show the details of the Virginia specimens.

3.2.2.4 Florida Specimens

Standard Concrete Products of Tampa, Florida, produced two 42-ft long, 60-in. deep inverted-T girders for the project.

They had thirty-six 0.6-in. diameter, 270 ksi, low relaxation prestressing strands in the bottom flange, tensioned to 43.94 kips each, and six #6 bars of mild steel reinforcement along the top of the web. These six bars were placed to resist the top tensile forces produced by the prestressing strands. On specimen FL1, one end contained the exact same configuration for end reinforcement as the Florida production girders, while the other end was designed to resemble the Florida end reinforcement design. On specimen FL2, one end was designed using AASHTO LRFD specifications and the other was designed using the proposed detail. After receiving the
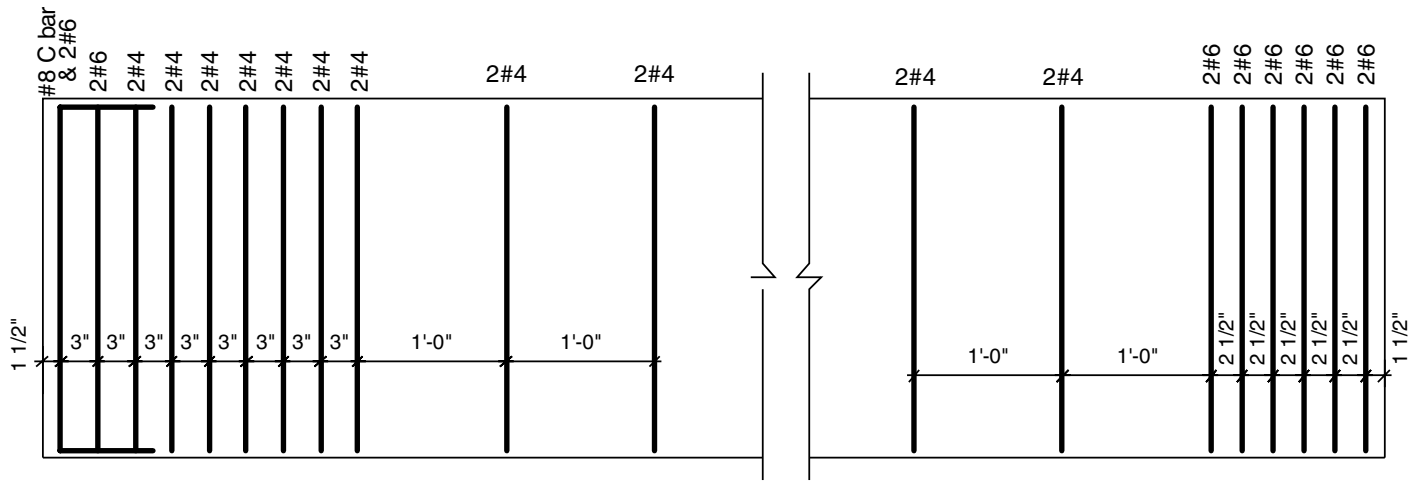
Figure 3.7. End zone reinforcement details of WA1L (proposed) and WA1R (LRFD).

specimens in the structures laboratory, a 10×24-in., 12,500 psi concrete deck was formed and made composite with the girder. Figures 3.12 through 3.14 show the details of the Florida specimens.

3.2.3 Test Setup

In order to simulate the decking system that would be placed on the girders in the actual bridge, a deck was cast in place on top of the Tennessee and Florida specimens in the structural laboratory, while a deck was cast monolithically with the top flange during fabrication of the Virginia and Washington specimens in the precast yard. The existing vertical reinforcement was extended in deck to act as horizontal shear reinforcement to create a composite system. The deck weight helped to increase the amount of stress in the bottom strands of prestressing steel at flexural

Figure 3.8. End zone reinforcement details of WA2L (no EZR) and WA2R (no EZR).

failure. Examples of the CIP deck and the deck cast during fabrication of the specimens are shown in Figures 3.5 and 3.9, respectively.

To test the first end of a specimen, the specimen was supported at 6 in. from both ends, leaving an unsupported length of 41 ft. A point load was applied at 12 ft from the end being tested and 30 ft from the other end, as shown in Figure 3.15. Once the test on this end was complete, the support on this end was moved 12 ft inside the specimen and the load setup was placed 12 ft from the second support. This setup helped to test both ends of every specimen while avoiding any effect from the tested end on the performance of the second end of the specimen.

Since the dead loads, applied after a girder is installed on a bridge, help in closing the end zone cracks, a clamping force mechanism was provided in the test setup at 30 in. away from the end of the girder in order to simulate this load. The clamping force was provided by using a hydraulic jack attached to a self-equilibrium frame built around the specimen as shown in Figures 3.16 and 3.17. This clamping force was calculated as the balance between the reaction developed by the actual bridge girder (due to the slab, barrier, wearing surface, and utilities weight) and the reaction generated by the 42-ft long specimen. The clamping mechanism was placed only at the end being tested.

The load was applied at a rate of about 5 kips per second in stages of 100 kips. After each additional 100 kips, the loading was paused so that the girder could be checked and marked for cracks. Once the estimated failure load was reached, the loading was stopped and the girder was checked for signs of failure and the cracks were marked. Then the loading was resumed until failure was reached. In some cases, as will be discussed in the following sections, failure could not be
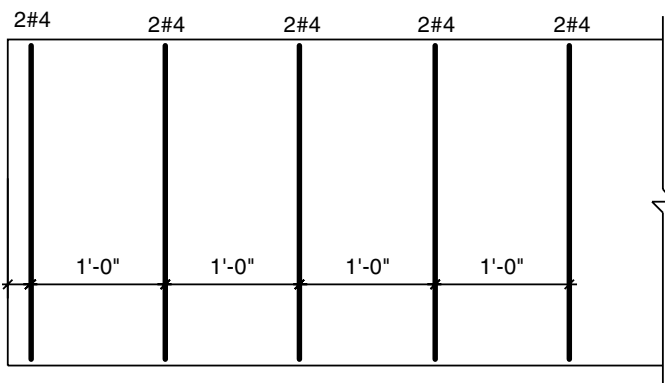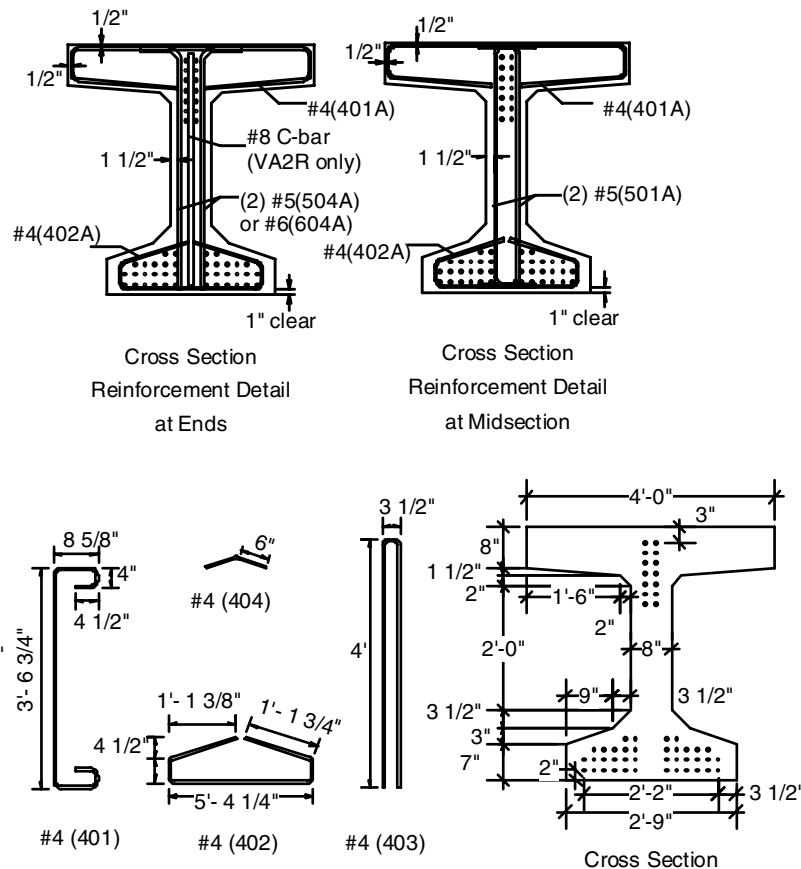
Figure 3.9. Cross section details of the Virginia specimens.

reached as the failure load was beyond the capacity of the loading frame. The failure load was calculated using the measured material properties of the concrete cylinders made during fabrication of the specimens and the coupons taken of the reinforcing bars.

Figure 3.10. End zone reinforcement details of VA1L (no EZR) and VA1R (no EZR).

3.2.4 Test Results

A summary of test results is given in Table 3.2. The table gives the failure mode and failure moment including those calculated based on the specified and measured material properties and those obtained from the test. A summary discussion related to each set of specimens is given in the following sections. More details about all fabrication and testing of all specimens are given in Appendix C.

3.2.4.1 Tennessee Specimens

Upon inspection after the release of the strands, neither girder appeared to have experienced any visible end zone cracking. The research team believes that the lack of end zone cracking is due to the limited amount of prestressing force, the presence of end zone reinforcement, and the size and shape of the girder. The girders contained thirty 0.5-in. diameter strands. This amount was the largest available to the producer at the time the specimens were made. The relatively small girder size and amount of prestressing, compared to the depths, spans, and levels of prestress in other states, has been a challenge to the research team. This is because the research

Figure 3.11. End zone reinforcement details of VA2L (LRFD) and VA2R (proposed).
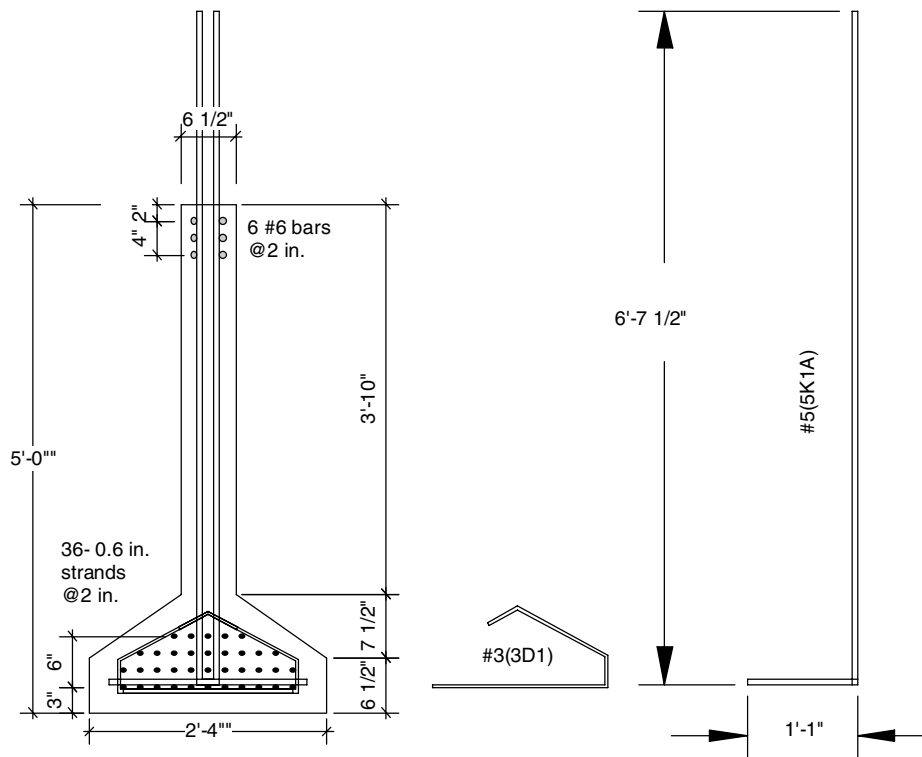
Figure 3.12. Cross section details of Florida specimens.
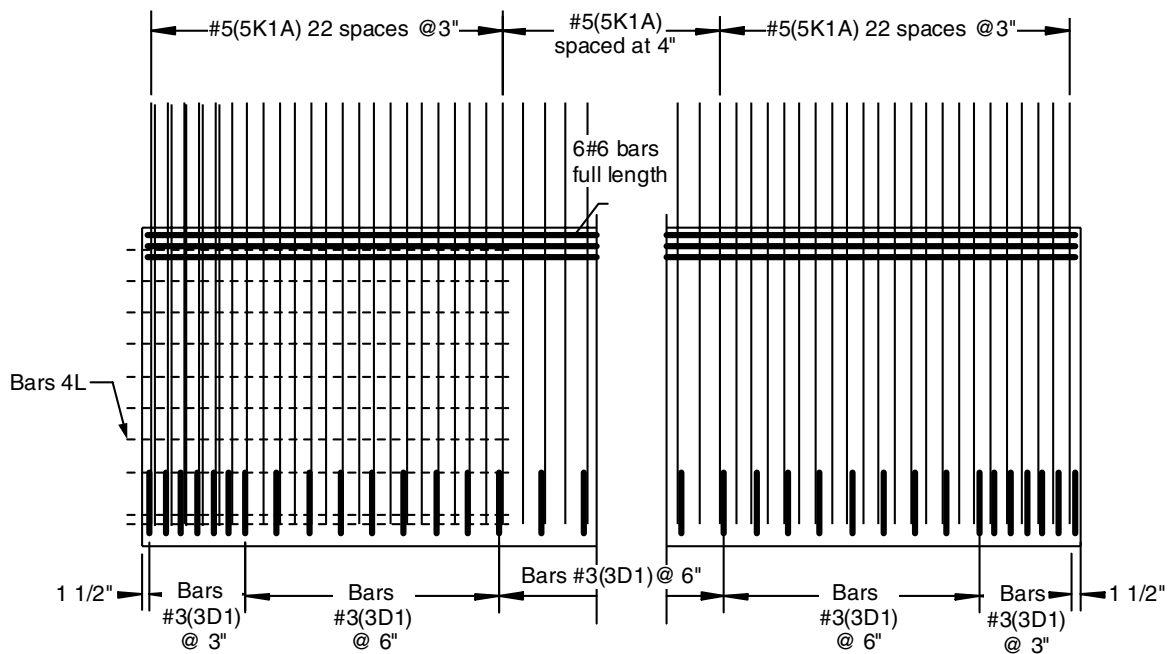
Figure 3.13. End zone reinforcement details of FL1L (FL) and FL1R (modified FL).

Figure 3.14. End zone reinforcement details of FL2L (LRFD) and FL2R (proposed).

Figure 3.15. Support and loading arrangement of the full-scale specimens.

Figure 3.16. End clamping detail.

Figure 3.17. Location of the point load and clamping mechanism.

Table 3.2. Summary of the test results of the full-scale specimens.

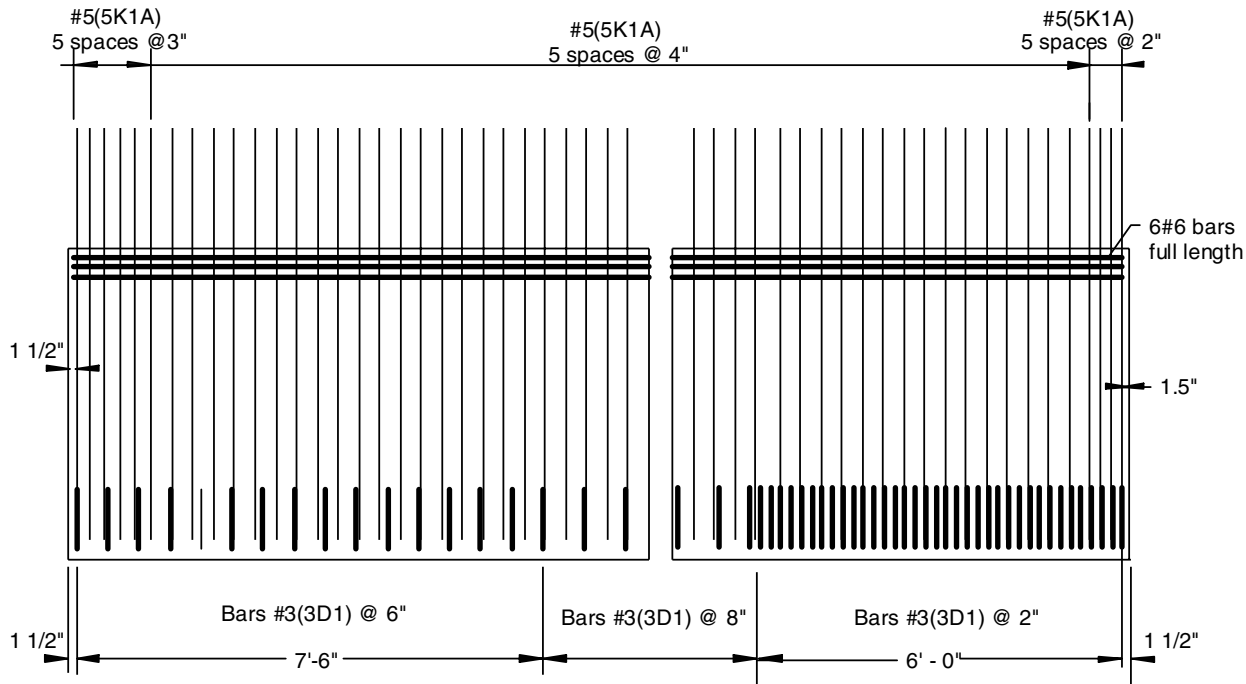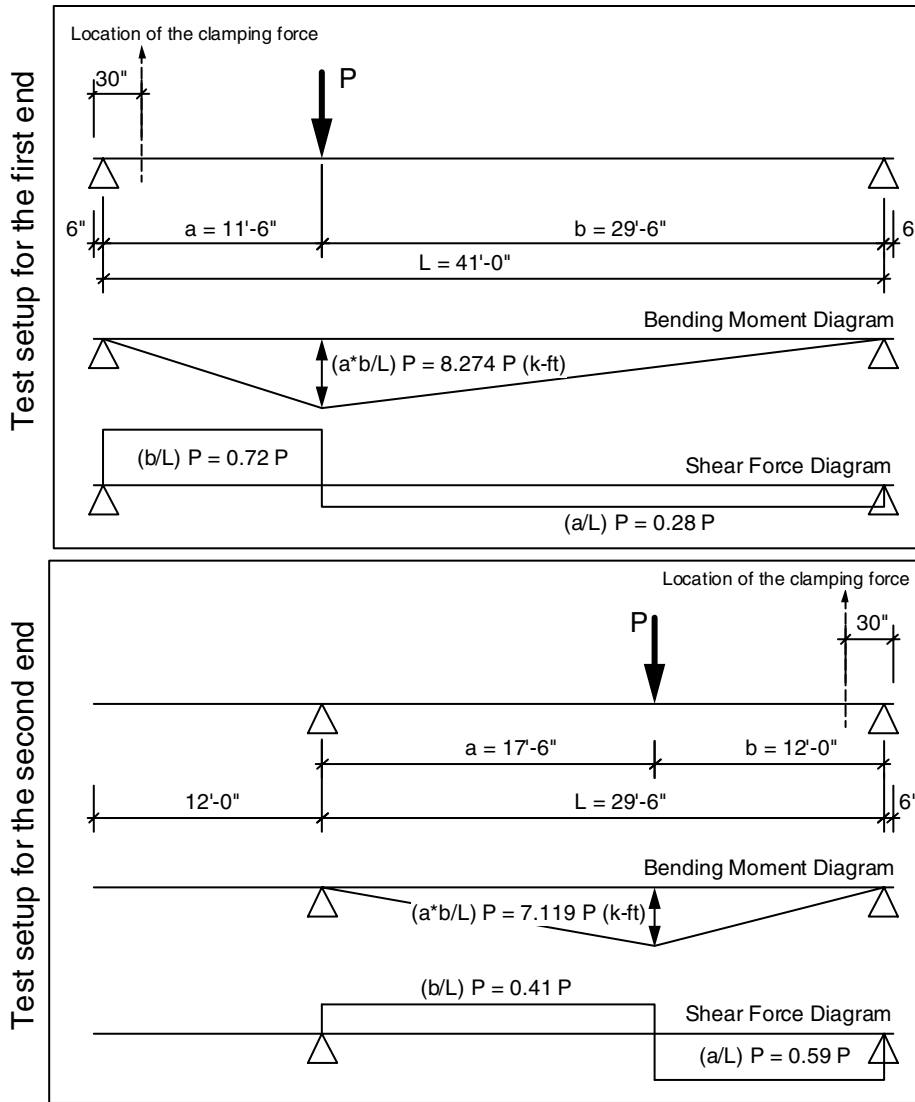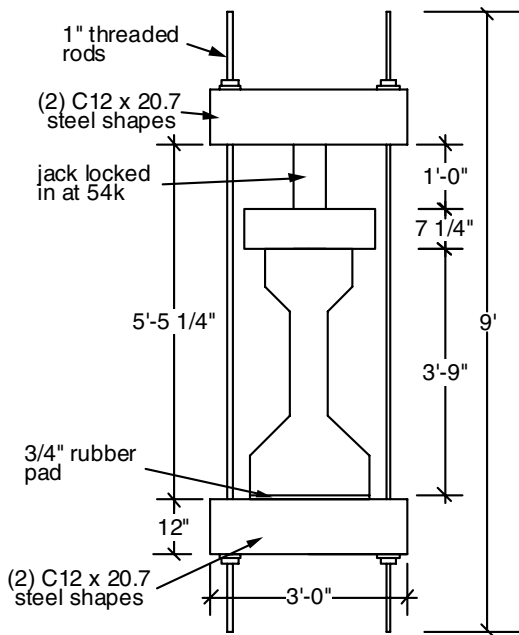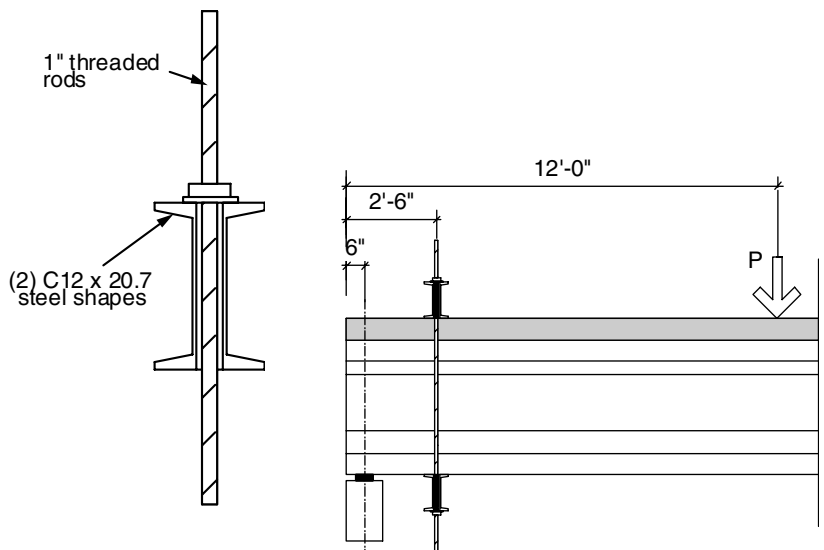| State | End | Design | Mode of Failure | Strength Calculated Using: | | | % Difference | |
				Specified Values	Measured Values	Test Data	Specified and Test	Measured and Test
Tennessee	TN1L	LRFD	Flexure	4,204 k-ft	4,299 k-ft	4,539 k-ft	8.0	5.6
	TN1R	Proposed*	Flexure	4,204 k-ft	4,299 k-ft	4,494 k-ft	6.9	4.5
	TN2L	TN DOT	Flexure	4,204 k-ft	4,299 k-ft	4,649 k-ft	10.6	8.1
	TN2R	Proposed*	Flexure	4,204 k-ft	4,299 k-ft	**	--	--
Washington State	WA1L***	Proposed*	Shear	311.3 k	319.8 k	508.5 k	63.3	59.0
	WA1R	LRFD	Shear	311.3 k	319.8 k	***	--	--
	WA2L	Non	Shear	311.3 k	319.8 k	434.2 k	39.5	35.8
	WA2R	Non	Shear	311.3 k	319.8 k	457.5 k	47.0	43.1
Virginia	VA1L	Non	Flexure	7,471 k-ft	7,809 k-ft	7,852 k-ft	5.1	0.6
	VA1R	Non	Bearing	7,471 k-ft	7,809 k-ft	7,593 k-ft	1.6	2.8
	VA2L	LRFD	Flexure	7,471 k-ft	7,809 k-ft	8,215 k-ft	10.0	5.2
	VA2R	Proposed*	Flexure	7,471 k-ft	7,809 k-ft	8,492 k-ft	13.7	8.7
Florida	FL1L	FL DOT	Flexure	10,039 k-ft	10,317 k-ft	9,890 k-ft	1.5	4.1
	FL1R	Mod. FL DOT	Flexure	10,039 k-ft	10,317 k-ft	**	--	--
	FL2L	LRFD	Flexure	10,039 k-ft	10,317 k-ft	**	--	--
	FL2R	Proposed*	Flexure	10,039 k-ft	10,317 k-ft	**	--	--

* Proposed EZR detail is the end zone reinforcement recommended by Tuan et al. (*16*) and discussed
in Section 1.2.3 of this report.
** The girder exceeded the setup capacity.
*** Girder end was epoxy repaired.

team wanted to have specimens with end zone cracking to see their effect on the girder capacity. To achieve this goal, deep girders with a large number of strands should be used. However, these large girders would be a challenge to load to failure in the structures laboratory because they require a large amount of applied force that might be beyond the capacity of the testing facility.

Reviewing the test results revealed that classical flexural failure occurred in all specimens at a load higher than the estimated load. The flexural failure was associated with loss of bond between the strands and the concrete at the girder end, as shown in Figure 3.18.

Figure 3.19 shows the load-deflection relationship of end TN1L, which clearly shows the elastoplastic behavior of the concrete section.

3.2.4.2 Washington State Specimens

As expected, upon release of the prestress force, the ends without end zone reinforcement (WA2L and WA2R) expressed much more end zone cracking than the ends that contained additional reinforcement (WA1L and WA1R). The comparison is shown in Figure 3.20. Both ends that contained additional end reinforcement experienced similar widths and patterns of end zone cracking.

In both of the reinforced ends, there was a delay in the location of the end zone cracks where they did not start until a few inches into the girder. The responsible factor for this may be the concentrated amount of reinforcing steel located near the end preventing cracks from starting at the very edge, but then

allowing them to form once the presence of reinforcing steel decreases. This phenomenon is shown in Figure 3.21. The figure also shows the end reinforcement for WA1L containing a C-shaped #8 bar 1.5 in. from the girder end. This bar was placed in conjunction with the pairs of #6 bars to locate a greater amount of steel close to the girder end.

All four ends failed in shear and reached much higher capacities than the estimated requirements. Figure 3.22 shows how the shear cracks run in the opposite direction of the end zone cracks. The load on the girder produces a force that works in a direction to close the end zone cracks. This demonstrates that even with excessive amounts of end zone cracking, the structural capacity of the girders was not reduced below acceptable limits. Both the repaired and unrepaired girders reached capacities greater than the theoretical calculated capacities. It was also noted that the end that was repaired by epoxy injection did not perform noticeably better than the unrepaired end, having similar percent differences between the theoretical and actual results, as shown in Table 3.2. This gives evidence that epoxy injection does not necessarily improve the structural capacity of girders with end zone cracks.

An example of the load-deflection relationship is presented in Figure 3.23, which shows how the test data far exceeds the estimated capacities. The curve is for End WA1R that was designed using LRFD specifications. In this case, the load required to fail the girder in flexure was greater than the 800 kips capacity of the two hydraulic jacks in the test setup. The curve shows where loading was halted, but it can be
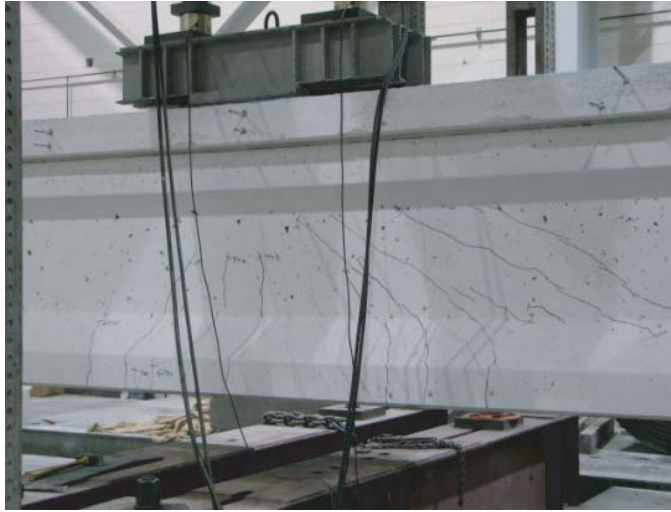
(a) Classical Flexural Cracking

(b) Strand Rotation

(c) Strand Slippage

Figure 3.18. Typical flexural failure and loss of bond in the Tennessee specimens.
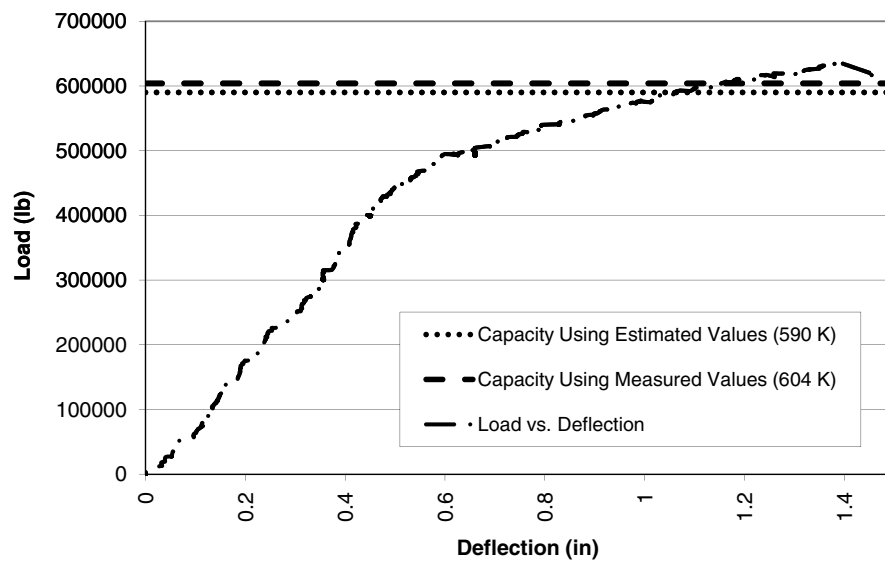
Figure 3.19. Load-deflection curve of TN1L.
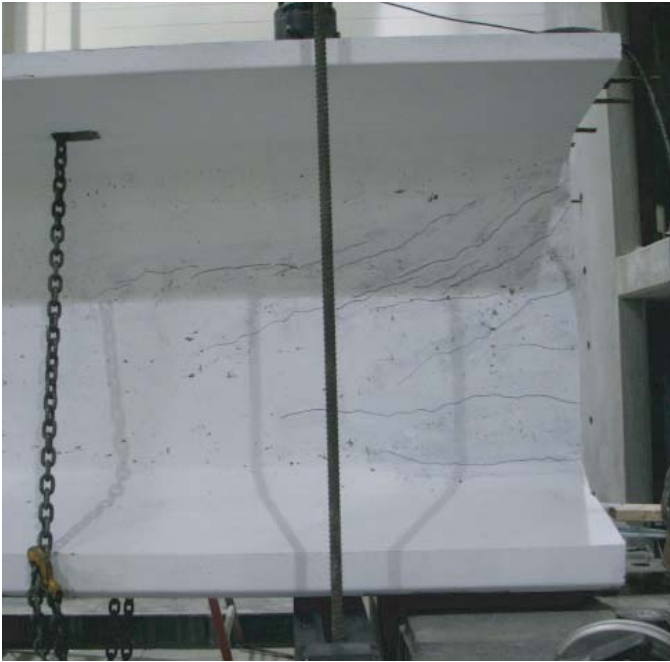
(a) WA2L (no EZR)

(b) WA1R (LRFD)

Figure 3.20. End zone cracking of WA2L and WA1R.

Figure 3.21. Crack pattern of WA1L and WA2R.

Figure 3.22. End WA1L after shear failure.

presumed that the girder would continue to take load until flexural failure.

3.2.4.3 Virginia Specimens

Upon release of the prestress force, all of the Virginia specimens experienced cracking in the range of 0.004 to 0.010 in. in width and extending no more than 3 ft from the end. The cracks on the ends without end zone reinforcement were wider and more prevalent. The end designed using the AASHTO LRFD specifications experienced the least amount of cracking. See Figure 3.24.

End VA1R was the first Virginia end tested and it failed prematurely due to inadequate bearing area, as shown in
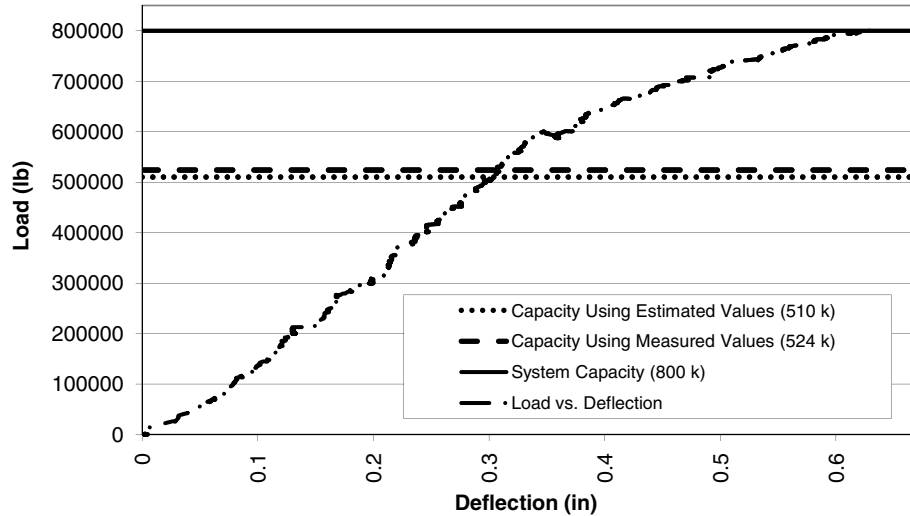
Figure 3.23. Load versus deflection curve for End WA1R.

Figure 3.25. This occurrence prompted the team to devise a pivoting support with a larger bearing area, shown in Figure 3.26. The three remaining ends failed in shear as designed, where all ends held loads higher than their design capacities. This is further proof that end zone cracks, even in cases where the cracks are wider and longer than any typically reported, do not reduce the structural capacity of girders below the design limits.

3.2.4.4 Florida Specimens

Upon release of the prestress force, all end zone cracks were around 0.004 to 0.006 in. in width and did not extend farther than 3 ft into the specimens. The two ends designed using LRFD specifications and the proposed method had cracking patterns of similar severity. However, the end designed using the LRFD experienced slightly more cracking than the proposed method. The two ends designed from Florida details were similarly cracked as well. The end that had some of the end reinforcement removed experienced slightly more cracking than the end with more reinforcement, however, the improvement is not significant enough to justify using that amount of extra steel for reinforcement. Comparisons are shown in Figures 3.27 and 3.28.

The girders were originally designed to fail in flexure. However, a mix-up at the precasting plant caused one or both of

(a) VA1R (No EZR)

(b) VA2R (Proposed)

Figure 3.24. End zone cracking of VA1R and VA2R.

Figure 3.25. End VA1R after bearing failure.

the girders to contain only half the amount of shear reinforcement requested, leading to the premature failure of the ends in shear. One girder was observed to contain half of the specified shear reinforcement when it burst in shear failure. However, the girder that was tested first did not fail, keeping the reinforcement hidden, so it is only assumed that it also contained half the specified shear reinforcement. In most cases, the design capacity of the specimens was more than the test setup could apply. The team decided to continue with testing and use the first half of the load versus deflection curves to determine when the girders would fail.

Figure 3.29 shows the load-deflection curve for End FL2R. The loading of this end had to be stopped before failure because the three hydraulic jacks had reached capacity at 1,200 kips. However, it can be inferred that if more load had been applied, the test values would have risen above the experimental values.

Figure 3.26. Support with roller.

| (a) FL2R (Proposed EZR) | (b) FL2L (LRFD EZR) |

Figure 3.27. Comparison of ends FL2R and FL2L.

The corresponding image, Figure 3.30, shows the shear cracks experienced by End FL2R just before the loading was stopped. The figure illustrates how the shear cracks form in the opposite direction as the end zone cracks.

End FL1L unexpectedly failed in shear due to the lack of adequate shear reinforcement. Images of the shear failure are shown in Figure 3.31. One can see how the prestressing force pulled the bottom flange in toward the center of the girder once there was no web to resist it. This is the same force that pulls on the web of precast girders causing end zone cracking. Once the reinforcement had been exposed, the team was able to calculate the theoretical shear design capacity for the girder to be around 700 kips. The experimental failure point was still greater than the calculated shear capacity value.

3.2.5 Full-Scale Testing Conclusions

The full-scale tests on eight full-scale girders has indicated that end zone cracking due to prestress bursting forces does

| (a) FL1L (FL Typical EZR) | (b) FL1R (FL Modified EZR) |

Figure 3.28. Comparison of ends FL1L and FL1R.
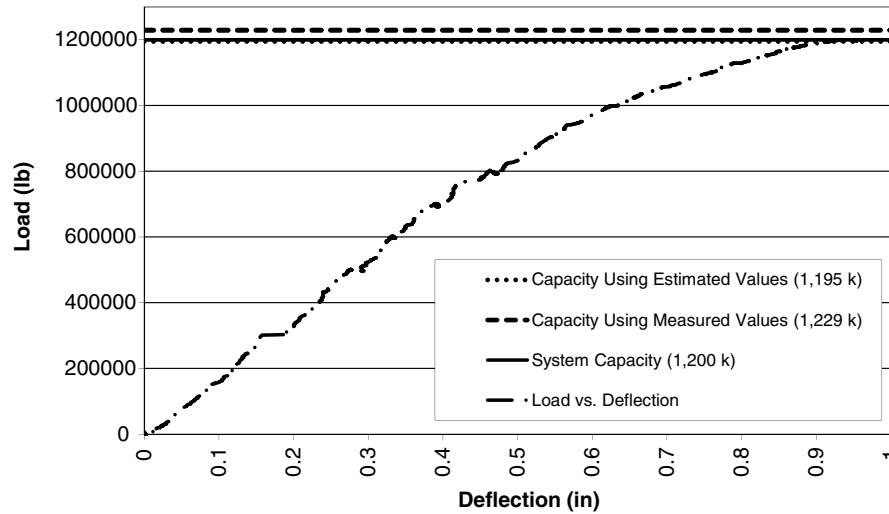
Figure 3.29. Load versus deflection curve for End FL2R.

not cause a reduction in the structural capacity of prestressed concrete girders. The orientation of the diagonal cracks is nearly perpendicular to the forces caused by diagonal tension (i.e., shear). When external loads are applied, they induce compressive stresses across the bursting force cracks, and therefore the types of cracks are not cumulative to each other. Even when end zone cracks were induced in the testing that were significantly larger than cracks commonly observed in practice, there still was not a measurable reduction in structural capacity. All specimens had capacities at or higher than the expected theoretical capacity.

When repairing was performed with epoxy injection in an attempt to restore concrete tensile capacity across the cracks, there was no significant change in capacity between repaired
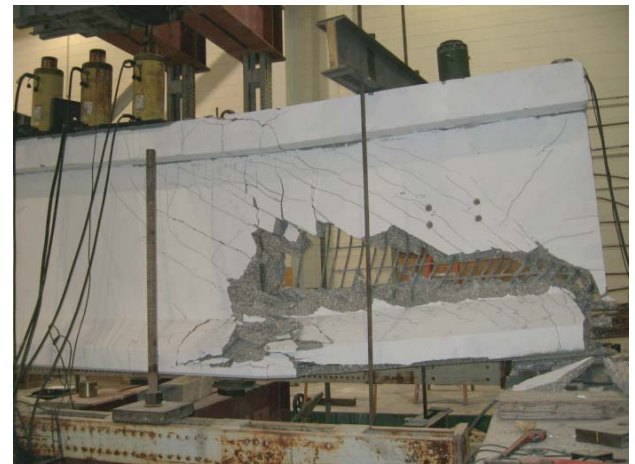
(a) General View
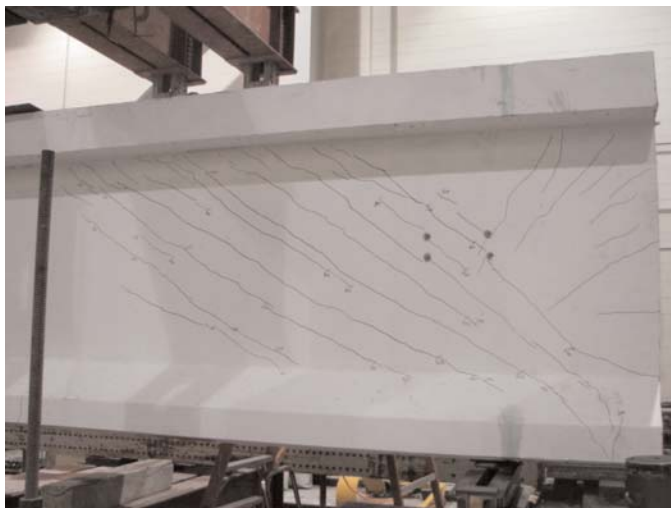
Figure 3.30. Shear cracks on End FL2R after loading was stopped.

(b) Close View Showing Relative Movement of the Bottom Flange

Figure 3.31. End FL1L after failure, east side.

and unrepaired specimens. Using epoxy injection to restore tensile capacity of concrete cracked under the effect of pre-stressing bursting forces is unwarranted and misleading. Even if the injected cracks are assured to be completely filled with epoxy and the interface surface between epoxy and concrete have adequate adhesion, the tensile capacity restoration would only be assured at the injected cracks. In the meantime, there would be numerous cracks, some of which are too narrow to effectively inject or are even invis-ible. At these locations, the tensile capacity of the concrete perpendicular to the crack lines would be lost even if wider cracks are satisfactorily injected with epoxy. Thus, the value of epoxy is to act as a sealant preventing penetration of water and salts into the concrete member. For this purpose, epoxy sealing may not be the most economical or efficient method, unless the cracks are very wide.

The full-scale testing also validated the statement that prop-erly designed and detailed end zone reinforcement is impor-tant in controlling end zone cracking. The AASHTO LRFD method produced acceptable results. The proposed method resulted in further improvements in crack control. The exper-iments demonstrated that reinforcement should not just be placed at the very end of the girder. The reinforcement should gradually diminish over a distance equal to h/2 of the girder depth. If reinforcement is placed only at the very end, there may be instances where wider cracks appear beyond the concen-trated reinforcement. This was confirmed in the Washington State experiments, where relatively large prestressing was applied.

3.3 Epoxy Injection Testing

3.3.1 Introduction

The epoxy injection test was developed and conducted at an early stage of the project to investigate (1) if epoxy injec-tion repair of end zone cracking is able to restore the tensile capacity of the cracked concrete, (2) if epoxy injection is capa-ble of completely filling the crack through the width of the web, and (3) if variations of the end zone reinforcement details have a significant effect on the number, width, and pattern of end zone cracks.

To shed light on these issues, the research team had two 12-ft long specimens fabricated by Concrete Industries Inc., Lincoln, Nebraska, as part of an NU 1350 (53 in. deep) bridge girder production. Details of these specimens and discussion on the experimental activities conducted on them are given in the following sections.

3.3.2 Description of the Test Specimens

Figure 3.32 shows the cross section of the test specimen. An NU 1350 section was used in making the specimens. Specified release strength was 6,500 psi and final strength was 8,000 psi. The bottom flange was reinforced with thirty-two 0.6-in., 270 ksi, low relaxation straight strands in two rows, and the web top was reinforced with twelve 0.6-in., 270 ksi, low relax-ation straight strands. The strand stress just before release was 202.5 ksi. Four additional 0.5-in., 270 ksi strands stressed at 13.2 ksi were provided in the top flange. Vertical shear web
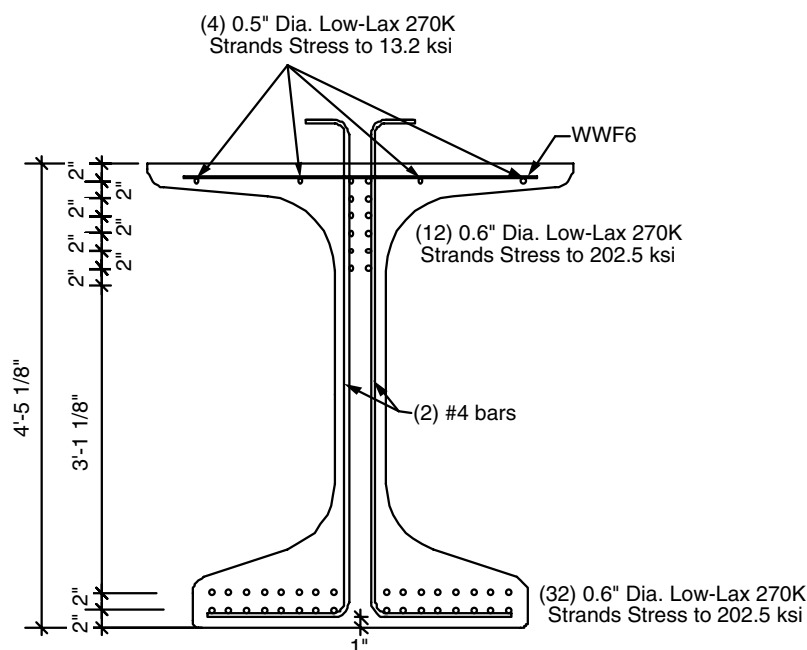
Figure 3.32. Cross section of test specimen (NU 1350).
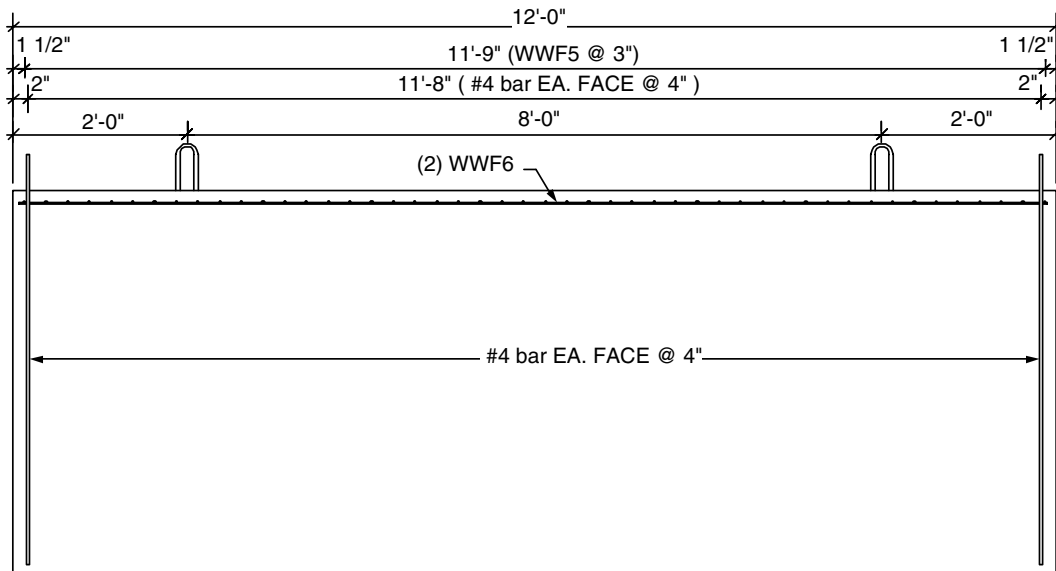
Figure 3.33. Specimen #1: S1L (no EZR), S1R (no EZR).

reinforcement consisted of pairs of #4 at 4 in. for the full 12-ft length of each specimen. No confinement reinforcement was provided in the bottom flange.

No special end zone reinforcement was provided in either end of the first specimen, as shown in Figure 3.33. Both ends of the second specimen were provided with special end zone reinforcement, where the left end was designed using the LRFD Specifications (*18*) and the right end was designed using the proposed end zone reinforcement detail that is given in Section 3.7 of this report, as shown in Figure 3.34. The proposed detail was developed at the University of Nebraska (*16*).

The test specimens were not provided with sole plates at the ends, or with transverse confinement reinforcement in the bottom flange. The research team believes in the importance of these two elements. However, they were intentionally omitted to demonstrate their value. The production girders, made in the same production run as the test specimens, had these elements, thus offering an opportunity for comparison.

Both specimens, as well as the production girders, experienced end zone cracking. Figures 3.35 through 3.37 show the end zone cracks of the test girders. As expected, both ends of the first specimen, S1L and S2R, which had no special end zone
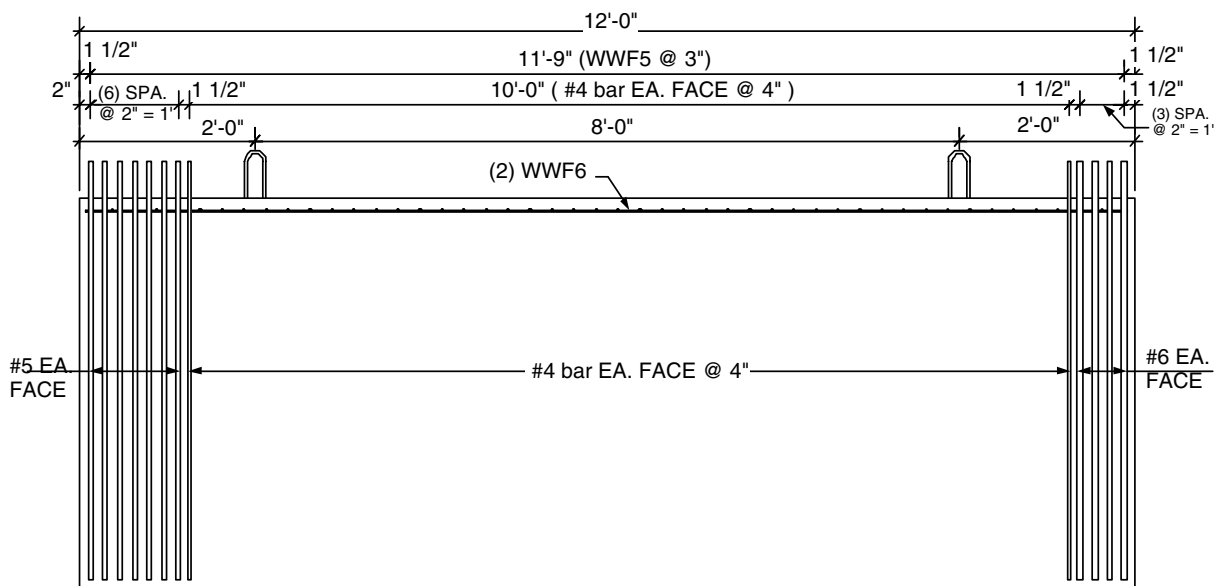
Figure 3.34. Specimen #2: S2L (LRFD), S2R (proposed).

34

Figure 3.35. Specimen #1 with no special end zone reinforcement (cracks traced for clarity).

Figure 3.36. Specimen #2: S2L (LRFD).

reinforcement, experienced a greater amount of cracking than those of the second specimen, which was provided with special end zone reinforcement. One end of Specimen #1 has cracks that were longer and wider than the other end. For Specimen #2, the end designed according to the AASHTO LRFD specifications experienced more severe cracking than the end designed using the proposed detail. The lack of bottom plate and bottom flange reinforcement contributed to increased cracking near the bottom flange. At one end, splitting cracks occurred at a corner strand. The precast producer used epoxy injection to repair one end of Specimen #1 (the girder without bursting end reinforcement) and the end designed according to the LRFD specifications of Specimen #2. The epoxy injection repair was conducted according to the procedure given in the *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products (11)*. Then,
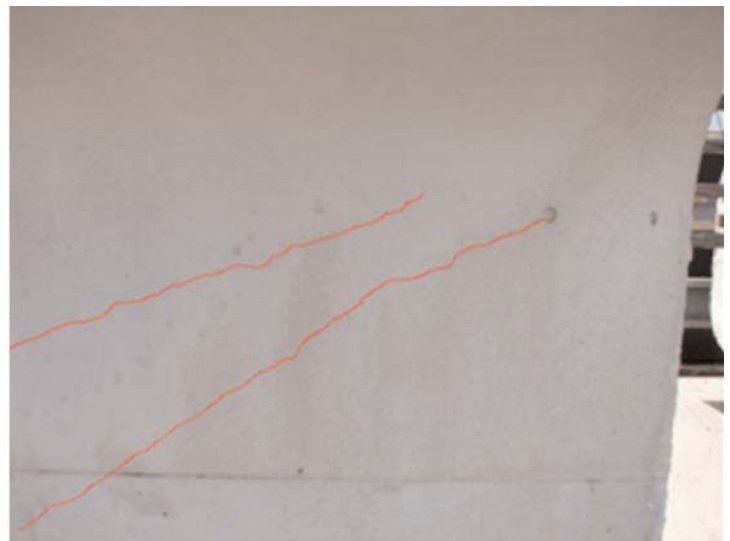
Figure 3.37. Specimen #2: S2R (proposed).

Figure 3.38. Bottom flange during cutting.

Figure 3.40. End zone cracking extending the full length of the web.

the specimens were shipped to the structures laboratory in Omaha, Nebraska, for testing.

3.3.3 Preparation of the Test Specimens

The team's objective was to find the most suitable method of testing the cracked and repaired ends for tensile capacity, and to compare them with the capacity of the uncracked zone in the mid-length of the specimen. This was done by cutting out sections of the web, turning them over on their sides, and loading each section as if it were a beam.

The top and bottom flanges were cut away, leaving only the webs of each girder. The thickness of the web required two cuts of the saw, one on each side of the web, as shown in Figure 3.38. The bottom flange contained a large prestressing force in the 32 strands. This force had been resisted by

the full section, before the bottom flange was separated from the rest of the section. When the bottom flange was cut away from the web, the web was no longer able to oppose these forces, and the bottom flange cracked. Figure 3.38 shows the bottom flange while being cut from the web. Although there was a great deal of cracking, the section remained intact, as shown in Figure 3.39.

When the beam was cut, the full extent of the interior cracking became visible, as shown in Figure 3.40. Upon inspection, it was clear that the epoxy did not totally fill the cracks as anticipated. From the cut section, the epoxy could only be seen entering approximately 0.2 in. into the crack, as shown in Figures 3.41 and 3.42. Also, visual inspection revealed a lack of adhesion between the concrete and the epoxy.

Figure 3.39. Bottom flange completely cut from the specimen.

length of penetration of epoxy seal

Figure 3.41. End zone cracking extends vertically and horizontally.

Figure 3.42. Very limited penetration of the epoxy repair.

The web sections were cut into 16-in. strips, as shown in Figure 3.43. One strip was extracted from each of the four ends of the two girders. Each specimen was turned on its side and subjected to a bending test, as shown in Figure 3.43. The structural testing was performed to find the cracking moment and tensile capacity of the specimens. The supports were set 18 in. apart. A two-point loading system was used, with the two points 6 in. apart. Table 3.3 provides the description and properties of each specimen.

3.3.4 Test Results

3.3.4.1 Specimen S1L (No End Zone Reinforcement, Repaired)

Figure 3.43. The 16-in.-wide strips and test setup.

The last recorded load point was 47.8 kips, but the specimen actually reached a load of 56 kips before failure. While testing, a crack began to form at the bottom center of the

Table 3.3. Properties of test specimens.

Girder Specimen	Specimen #1 (Without Special End Reinforcement)			Specimen #2 (With Special End Reinforcement)	
Test Specimen	S1L	S1R	S1MS	S2L	S2R
Location of the Specimen	Left End	Right End	Midspan	Left	Right
Repaired (R) or Unrepaired (UR)	R	UR	UR No cracks	R	UR
End Reinforcement Design	No special end reinforcement			LRFD	Proposed
Reinforcement on Each Face	4 #4 @ 1.25 in.			7 #5 @ 1.31 in.	4 #6 @1.31 in. + 3 #4 @ 1.25 in.
Dimensions	b (width) = 16 in., h (depth) = 6 in., l (length) = 24 in.				
Concrete Strength	8,000 psi				

beam due to the bending stress, as shown in Figure 3.44(a). The lack of a bearing surface contributed to premature failure, as shown in Figure 3.44(b). Figure 3.44(c) shows cracks that formed on the bottom on the specimen during testing. Before testing, this specimen had a crack along the length of a vertical bar. It appeared to have formed during prestress release. This preexisting crack was not visible before the saw cutting and was not grouted with epoxy. The final failure cracking was a continuation of this preexisting crack. The failure load resulting in this test may be considered to be unreliable due to the setup issues (bearing and data acquisition). An important lesson learned, besides refining the setup for future testing, was that the epoxy repair used may not be effective in restoring the tensile capacity, and may not totally fill and seal all cracks.

3.3.4.2 Specimen S1R (No End Zone Reinforcement, Unrepaired)

The specimen failed at a load of 109 kips with a maximum deflection of 0.236 in. This maximum load was much greater than that of the repaired Specimen S1L. The unrepaired section had larger bearings, each with a 4 × 12-in. contact area. This specimen also did not contain a crack along its rebar as observed in Specimen S1L. Figure 3.45 shows the specimen setup before and after failure.

3.3.4.3 Specimen S1MS (No End Zone Reinforcement, Midspan Strip)

This specimen had no cracks, and therefore no repair was required. The specimen failed at a load of 103 kips and reached a maximum deflection of 0.251 in. Figure 3.46 shows both ends of the specimen after failure. The cracks formed vertically along the rebar.

3.3.4.4 Specimen S2L (LRFD End Zone Reinforcement, Repaired)

Similar to Specimen S1L, this specimen contained a crack along the rebar before testing, as shown in Figure 3.47(a). The specimen failed at 103.4 kips and reached a maximum deflection of 0.260 in. Figures 3.47(b) and (c) show both ends of the specimen after failure. This specimen also split along the direction of the rebars at failure. Figure 3.47(d) shows how the splitting occurred through the epoxy injection. Instead of the concrete failing, the epoxy-crack separated. This shows that the epoxy repair of these specimens did not appear to be effective. The rest of the cracking occurred in the same planes as the pre-existing crack along the rebar, as shown in Figure 3.47(e).

(a) Flexural Cracking

(b) Bearing Failure at the Support

(c) Bottom Surface at Failure

Figure 3.44. Test specimen S1L after failure.

(a) Setup with Wider Bearing Plates

(b) At Failure

Figure 3.45. Specimen S1R before and after testing.

3.3.4.5 Specimen S2R (Proposed End Zone Reinforcement, Unrepaired)

The specimen contained four pairs of #6 bars and three pairs of #4 bars, as shown in Figure 3.48(a). The specimen failed at a load of 154.2 kips and reached a maximum deflection of 0.246 in. Figures 3.48(b) and 3.48(c) show the specimen after failure. Once again, the cracks formed in the same direction as the rebar.

3.3.5 Discussion and Conclusions

Calculations were performed to estimate the cracking and failure moments of the five specimens. Table 3.4 gives the calculated cracking and failure load, and the test results of the five specimens. Cracking load is measured as the load at the intersection between the steep and flat lines on the load-deflection diagram. None of the specimens exhibited a discernible "kink" in the load-displacement curve, implying that there was practically no cracking load capacity. Another less accurate method of measuring cracking is by visual inspection as the load is gradually applied. The computer-aided data acquisition system is more accurate, because micro cracks are impossible to detect visually. These observations led the team to conclude that (1) all specimens became cracked transverse to the prestressing direction at the time of prestress release, and (2) epoxy injection for these specimens was ineffective in restoring them to a pre-cracked condition.

The epoxy injection testing demonstrated the following:

1. Cutting coupons from the web end of a pretensioned I-beam was not an effective method of testing for structural tensile capacity;

Figure 3.46. Specimen S1MS after failure.

(a) Pre-Existing Crack across Bottom Layer of Reinforcement

(b) View at Bottom Flange Junction

(c) View at Top Flange Junction

(d) Splitting Occurred Right through Epoxy Injection

(e) Splitting Occurred Right through Pre-Existing Crack at Lower Layer of Reinforcement

Figure 3.47. Specimen S2L (LRFD end zone reinforcement, repaired specimen).

2. Prestressing release causes end zone cracking some of which cannot be epoxy injected or even seen with the naked eyes;

3. The epoxy injection used on the specimens, even though it was applied by experienced professionals in a precast concrete plant, was not a reliable method of totally filling the injected cracks across the entire web width;

4. The tested specimens had no concrete tensile capacity, indicating that epoxy injection does not restore concrete tensile capacity of repaired end zones even if the injection totally repairs the individual cracks being injected;

5. The AASHTO LRFD method was effective in controlling end zone cracks;

6. The proposed reinforcement was more effective than the AASHTO LRFD method; and

7. The bottom flange confinement reinforcement and the base plate should be treated as an integral part in crack control of the end zone (they are highly recommended in all stemmed prestressed concrete girders).

(a) Before Testing

(b) At Failure

(c) At Failure

Figure 3.48. Specimen S2R (UNL end zone reinforcement unrepaired specimen).

3.4 Durability Testing

3.4.1 Introduction

To prevent chloride penetration into end zone cracks, especially for coastal regions and areas where deicing chemicals are used in winter, waterproofing sealants can be used. However, since the cracks tend to open and close with the loading of the girder, the sealant must be able to withstand this movement while spanning the gap created by the crack. The objectives of the durability testing were to investigate the following issues:

1. If repair is required, what sealant material should be used?
2. Is it required that end zone cracks be filled with a patching material before a surface sealant is applied?

3.4.2 Durability Test, Stage I

The research team devised an experiment to gauge the effectiveness of commonly used sealants in order to select characteristics that can be recommended to the public for use in crack repair. This test also observed and recorded the properties of each sealant as well as the method and ease of application.

3.4.2.1 Stage I Test Procedure

The test slightly modified the test from ASTM D6489-99, *Standard Test Method for Determining the Water Absorption of Hardened Concrete Treated with a Water Repellant Coating.* (ASTM Standard D6489-99 is provided in Appendix E.)

Concrete cylinders were used as test specimens in the procedure. Sixty specimens were produced at Concrete Industries' precast plant in Lincoln, Nebraska, from a self-consolidating concrete production line. After the cylinders were received at the structures laboratory, they were washed and cleared of debris and then heated in a draft oven for 24 hours to remove any moisture. They were then coated with the sealants, which were mixed, prepared, and applied according to the manufacturers' specifications, taking care not to leave any uncovered spots. A control group was designated, which received no sealant coating. All of the specimens were then immersed completely in water and left to soak, as shown in Figure 3.49. At 24 h, and again at 96 h the specimens were towel dried and weighed. By taking the weight of the specimens before and after submersion the percent absorption was calculated and averaged for each sealant type.

Five different sealants were tested for effectiveness in protecting against water and chloride penetration as follow:

1. Pipewipe®,
2. DuralPrep® A.C.,
3. Transpo Sealate® T-70,
4. Xypex® Concentrate, and
5. DegaDeck® Crack Sealer Plus.

Table 3.4. Test results of the five specimens.

Girder Specimen	Specimen #1 (Without Special End Reinforcement)			Specimen #2 (With Special End Reinforcement)	
Test Specimen	S1L	S1R	S1MS	S2L	S2R
Location of the Specimen	Left End	Right End	Midspan	Left	Right
Nominal Cracking Load (kip)	23.1			25.5	25.0
Nominal Ultimate Load (kip)	77.3			175.6	153.2
Test Results					
Cracking Load (kip)	---	---	---	---	---
Failure Load (kip)	56	109	103	103	154
Midspan Deflection (in.)	---	0.236	0.251	0.260	0.246

Appendix D provides technical information on the sealants. Figure 3.50 shows the cylinders after being sealed with the five sealants.

- Pipewipe® is a cement-based product designed to produce a "sack rubbed" finish to the concrete. It fills in any voids or cracks to create a smooth, uniform coating. Its primary purpose is to be a cosmetic repair product. Pipewipe® contains a polymer-bonding agent along with very fine aggregates. It expands and contracts at the same rate as normal concrete, therefore preventing cracking at extreme temperature changes. As a liquid, the product is thick and easy to apply by hand. However when dry, the coating could easily be rubbed away and was also water absorbent. Due to its cement base, Pipewipe® is not intended to be a waterproofing sealant. Therefore, the research team decided not to consider Pipewipe® for further testing.
- DuralPrep® A.C. is a water-based epoxy modified Portland cement bonding agent and anti-corrosion coating. It contains a migratory corrosion inhibitor and claims that it can be used as a coating for steel reinforcement. The mixture requires the blending of three separate chemicals. The mixture is non-volatile and does not give off any harmful fumes. The product is very viscous, making it difficult to apply a thin coating. Once dry, the product is rough and cement-like in texture, giving the impression of being porous.
- Transpo Sealate® T-70 is a high molecular weight methacrylate resin system. It is designed to fill and seal concrete cracks. It has a very low viscosity that is designated to penetrate deep into cracks of small widths. Due to this low viscosity, the product works best on horizontal surfaces where it cannot flow away. It is a three-part substance that, when its chemical components are combined, may produce skin irritation and will give off harsh, volatile fumes. However, the seal that is produced appears glassy and has water-resistant characteristics. The team believes that the product itself is water resistant, but the method of application and orientation of the cylinders prevented a solid coating around the specimens. Due to the ultra low viscosity of the product, it easily flowed off the surface before setting up. This shows that the product is satisfactory for the water and chloride prevention requirement, but may not be effective for vertical application on the webs of precast girders.
- Xypex® Concentrate is designed to waterproof and repair concrete. When exposed to water, this product causes a catalytic reaction that produces a non-soluble crystalline formation within the pores and capillary tracts of concrete and cement-based materials. It is mixed with water and can be made into different consistencies to match the application method. The results for this sealant may be confusing due to initial saturated surface and the moist curing procedure. It may be that the Xypex® layer absorbs a limited amount of water, yet would not allow any to leak through. The Xypex®-coated specimens absorbed more water than the uncoated control specimens.
- DegaDeck® is a reactive methacrylate resin designed to penetrate and seal cracks in concrete. It produces a hard, clear, matte, water repellant surface. The product has a low enough viscosity to flow easily into small cracks. DegaDeck® is

Figure 3.49. Specimens submerged in water.

| (a) Pipewipe® | (b) DuralPrep® A.C. | (c) Transpo Sealate® T-70 | (d) Xypex® Concentrate | (e) DegaDeck® Crack Sealer Plus |
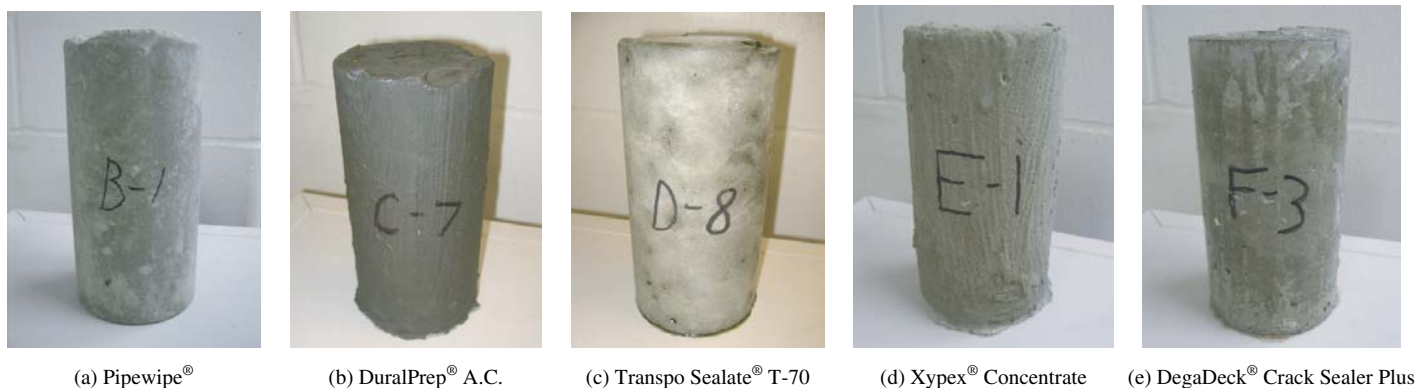
Figure 3.50. Specimens coated in sealants.

two-part substance comprised of liquid and powder hardener. It is recommended for horizontal surfaces only. However, the team found that the product performed well on vertical surfaces, such as girder webs, and did not flow off the exterior. DegaDeck® Crack Sealer Plus was the best performing sealant of the five tested. It also had a very easy workability and mixing procedure. However, when mixed, the chemical is highly volatile and produces harsh, potentially dangerous fumes. It is also a skin and eye irritant. The fumes, as well as the liquid, are flammable.

3.4.2.2 Stage I Test Results

Tables 3.5 and 3.6 present a summary of the percent absorption of each specimen at 24 h and 96 h, respectively. Also, these tables provide the average, standard deviation, and variance for the five sealants.

The five sealants were rated from the analysis of the absorption results at 24 h and 96 h and ease of application, and from the best to the worst were (1) DegaDeck® Crack Sealer Plus,

(2) DuralPrep® A.C., (3) Pipewipe®, (4) Transpo Sealate® T-70, and (5) Xypex® Concentrate.

The top performing sealants retained for Stage II of the durability tests were DegaDeck® Crack Sealer Plus, DuralPrep® A.C., and Transpo Sealate® T-70.

3.4.3 Durability Test, Stage II

For the second stage of the durability test, the team observed how assorted sealers perform in preventing water from penetrating into concrete specimens exhibiting various sizes of cracks. The procedure was modified from two ASTM Standards, G109-99a *Standard Test Method for Determining the Effects of Chemical Admixtures on the Corrosion of Embedded Steel Reinforcement in Concrete Exposed to Chloride Environments,* and D6489-99 *Standard Test Method for Determining the Water Absorption of Hardened Concrete Treated with a Water Repellent Coating.* (ASTM Standard D6489-99 is provided in Appendix E.)

Table 3.5. Summary of percent absorption of all sealants at 24 hours.

Specimen	Sealant					
	Control Specimens	Pipewipe®	DuralPrep® A.C.	Transpo Sealate® T-70	Xypex® Concentrate	DegaDeck® Plus
1	2.65%	1.58%	0.38%	2.52%	2.99%	0.62%
2	2.49%	1.31%	0.37%	2.45%	3.16%	0.52%
3	2.43%	1.71%	0.45%	2.59%	3.12%	0.44%
4	2.65%	1.41%	0.58%	2.23%	3.15%	0.29%
5	3.02%	1.38%	0.37%	2.13%	3.21%	0.27%
6	3.01%	1.39%	0.38%	2.29%	3.15%	0.68%
7	2.74%	1.50%	0.51%	2.59%	3.24%	0.23%
8	2.80%	1.55%	0.48%	2.53%	3.14%	0.16%
9	2.69%	1.46%	0.68%	2.05%	2.55%	0.18%
10	2.74%	1.54%	0.62%	2.71%	3.00%	0.15%
Average	2.72%	1.48%	0.48%	2.41%	3.07%	0.35%
Stand. Dev	0.190	0.118	0.112	0.219	0.199	0.195
Variance	0.036	0.014	0.012	0.048	0.040	0.038
Rating (1 = best, 5 = worst)		3	2	4	5	1

Table 3.6. Summary of percent absorption of all sealants at 96 hours.

Specimen	Sealant					
	Control Specimens	Pipewipe®	DuralPrep® A.C.	Transpo Sealate® T-70	Xypex® Concentrate	DegaDeck® Plus
1	2.76%	1.67%	0.66%	2.92%	3.23%	0.82%
2	2.60%	1.39%	0.63%	2.81%	3.40%	0.67%
3	2.54%	1.80%	0.65%	2.98%	3.35%	0.58%
4	2.77%	1.48%	0.83%	2.60%	3.40%	0.47%
5	3.14%	1.45%	0.63%	2.55%	3.46%	0.42%
6	3.15%	1.47%	0.60%	2.70%	3.44%	1.05%
7	2.86%	1.59%	0.74%	2.99%	3.52%	0.34%
8	2.92%	1.61%	0.81%	2.89%	3.36%	0.87%
9	2.81%	1.55%	1.04%	2.36%	2.74%	0.27%
10	2.87%	1.63%	1.11%	3.07%	3.27%	0.26%
Average	2.84%	1.56%	0.77%	2.79%	3.32%	0.58%
Stand. Dev.	0.199	0.122	0.177	0.226	0.219	0.273
Variance	0.040	0.015	0.031	0.051	0.048	0.074
Rating (1 = best, 5 = worst)		3	2	4	5	1

The sealants chosen for this experimentation are the three best-performing sealants from the first durability test (Dega-Deck® Crack Sealer Plus, DuralPrep® A.C., and Transpo Sealate® T-70) along with SilACT®, which was recommended by Central Pre-Mix Prestress Co. of Washington State. The specimens were made from the same concrete mix design, with a concrete strength of about 5,000 psi. Although this concrete mix is relatively more porous than the concrete normally used in precast girders, it was used to amplify the amount of water absorbed if the sealers failed.

3.4.3.1 Stage II Test Procedure

The concrete specimens were made in the structures laboratory of the University of Nebraska, in the form of $3 \times 3 \times 12$-in. rectangular prisms. Artificial cracks were formed with metal and plastic shims, penetrating down 2.25 in. from the top surface of the specimens and measuring 9 in. in length, as shown in Figure 3.51. These shims were placed in the concrete while it was still wet and removed when it began to set. The artificial cracks were produced in a variety of widths, ranging from 0.007 to 0.054 in.

After all specimens were fabricated, they were placed in a draft oven for 24 h to remove any moisture. When cooled, their weight was recorded as W_A, and then the sealants were used to cover the four sides and bottom face of each specimen, leaving only the top surface containing the crack uncoated. These sides were covered to prevent moisture from either entering or escaping the areas not being tested.

There were two sets of specimens for each sealant, with each set containing prisms with cracks of each available size. The first set was sealed only with the specified sealant. The second set had a Hilti® Brand hydraulic cementitious material, REM 800, rubbed into the cracks by hand, and then sealed with the same sealant as the first set. An additional set was made as the control, where the specimens were not repaired with any sealant at all and did not contain any artificial cracks. Table 3.7 shows the test plan.

The specimens were placed on their sides and the selected sealants and REM 800 were applied to their specific sets. This orientation mimics the orientation of the cracks on the webs of production girders. Care was exercised not to leave any concrete surface uncovered or to allow any air bubbles to form. The REM 800 was rubbed into the cracks by hand but the sealants were applied with a roller.

Once all of the specimens dried, they were turned upright and a 3-in.-tall rectangular plastic dam was built on the top

Figure 3.51. Specimens with metal shims.

**Table 3.7. Plan for the durability tests, Stage II.**

Crack Width (in.)	Control Batch	DegaDeck®		Transpo Sealate®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
Number of Specimens, Stage II (Total = 46 Specimens)									
0.000	1								
0.007	1	1	1	1	1	1	1	1	1
0.012	1	1	1	1	1	1	1	1	1
0.016	1	1	1	1	1	1	1	1	1
0.033	1	1	1	1	1	1	1	1	1
0.054	1	1	1	1	1	1	1	1	1
Total	6	5	5	5	5	5	5	5	5

surface of each specimen around the artificial crack so that water could pond on the repaired surface. Waterproof caulking material was used to secure the plastic walls in place, as shown in Figure 3.52. With the dam in place, the specimens were weighed, recording the data as W_1.

The specimens were all placed face up in an area where they would not be disturbed. Each dam was then filled to the top with water. The specimens were given the opportunity to absorb water for 24 h. Every effort was made to ensure that the dam remained filled with water at all times. At 24 h, the water in each dam was emptied. Then, the specimens were towel dried. The weight of each sample was measured and recorded as W_2. The percent of water absorption by each sample can be found using the following equation:

$$Percent\ Absorption = \frac{100 \cdot (W_2 - W_1)}{W_A} \qquad \text{(Equation 1)}$$

Where W_A is the weight of the concrete specimen after drying, but before exposure to the sealant and before dam placement, W_2 is the weight of the sealed specimen after soaking, and W_1 is the weight of the sealed specimen before soaking.
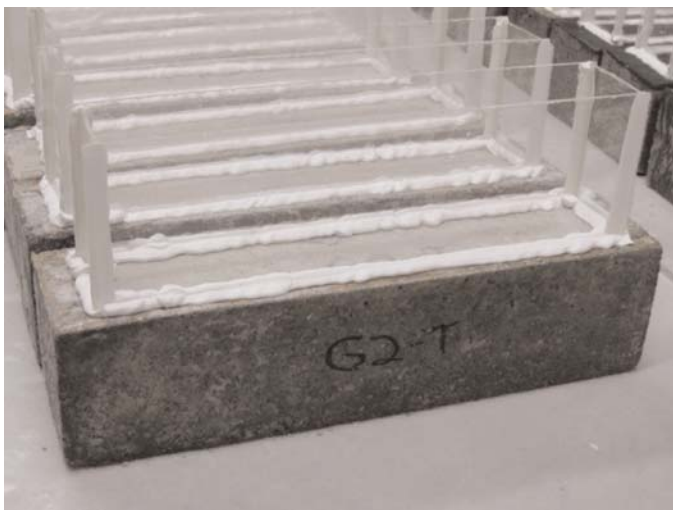
Figure 3.52. Specimens with water dams.

3.4.3.2 Stage II Test Results

The test results show that packing larger cracks with a thick, cementitious material (REM 800) allowed the cracks to be closed, while repair with a sealant alone failed in most cases with large cracks. Typically, the specimens with REM 800 were able to keep the water out better than the specimens without REM 800. The material packed into the crack created a bridge, over which the less viscous water-resistant sealants were allowed to lay, forming an unbroken seal across the entire surface. Without REM 800, the sealants with a water-like consistency (DegaDeck®, Transpo Sealate®, and SilACT®) were not able to adequately fill the large-sized cracks when applied on a vertical surface. Table 3.8 gives a summary of the 24-h percent absorption of the specimens.

This experiment was designed to exaggerate actual bridge conditions to which end zone cracks would be exposed. In service, the crack surface would not be continuously under water, as the specimens were, but the exposure to wet environmental conditions would extend for a much longer period of time.

DegaDeck® Crack Sealer Plus was effective when coupled with REM 800, but without the hydraulic cementitious material enough water penetrated into the crack for it to be considered ineffective. About half of these DegaDeck® specimens remained relatively water resistant while the remaining seals failed. The sealant was not thick enough to be able to bridge the gap created by the crack on its own, as shown in Figure 3.53.

Transpo Sealate® was not considered effective with or without REM 800. Except for a few outliers, the specimens containing REM 800 collectively had a much lower percent absorption of water than the specimens without REM 800. The ineffectiveness of Transpo Sealate® may be attributed to the thin, water-like consistency of the product. When applied to the vertical surface, most of this sealant flowed off of the sample. Therefore, the layer that remained was not thick enough to prevent water infiltration. The product is recommended for horizontal application and the experiment confirms that this is where it would be most useful. Figure 3.54 shows the specimens sealed with Transpo Sealate®.

Table 3.8. Summary of 24-hour percent absorption for the durability test, Stage II.

Crack Width (in.)	Control	DegaDeck®		Transpo Sealate®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
0.000	4.28								
0.007	1.81	0.22	4.10	2.06	4.63	0.42	0.66	0.09	2.17
0.012	1.49	0.11	1.44	2.96	0.87	0.51	2.78	0.13	0.46
0.016	2.59	0.36	0.69	2.82	1.03	1.07	3.25	0.19	1.45
0.033	4.19	0.35	0.37	2.29	4.04	0.33	1.46	0.17	1.94
0.054	1.69	0.08	3.81	1.34	4.03	0.72	0.54	0.15	2.43

(a) With REM 800

(a) With REM 800

(b) Without REM 800

Figure 3.53. Specimens coated with DegaDeck®.

(b) Without REM 800

Figure 3.54. Specimens coated with Transpo Sealate®.

DuralPrep® A.C. was moderately effective with an undercoating of REM 800, but was not effective without it. The sealant was mixed by combining a powder with two liquid chemicals. This created a thick, slurry-like liquid that was able to bridge the space created by the cracks, even without REM 800 and with the largest 0.54-in. crack. DuralPrep® A.C. was the only sealant of the four tested that did not gap when applied over the crack, especially when voids appeared at the crack location. However, the product performed well when the specimen was batched with REM 800. Figure 3.55 shows the specimens sealed with DuralPrep® A.C.

SilACT® was effective at preventing water penetration with REM 800, but was ineffective without the cementitious packing material. The manufacturer states that SilACT® chemically bonds with the substrate and creates a water-resistant layer just below the concrete surface that repels water but allows gasses to flow through. Therefore, SilACT® has a different method of water resistance than the other sealants tested. There is no

hard, water-resistant outer shell that covers the specimen, as is the case with the other sealants. Instead, the water-resistant layer is actually within the concrete. Without the hard, outer layer, there is nothing to bridge the crack gap. The strength of SilACT® comes from being able to be soaked into the concrete. This is why it performed well after soaking into the REM 800 layer. Without the patching material, the crack was left open and SilACT® was not able to soak all of the way into the crack when the opening was located on a vertical surface. If the surface had been horizontal and the product had been allowed to soak all the way into the crack, the results would have been more effective, but this would not be representative of the actual end zone crack position. Figure 3.56 shows the specimens sealed with SilACT®.

In comparison to what the study team expected, the data had quite a few inconsistent results. It seemed that whether the sealant was effective or not depended largely on how well the application was executed. Specimens (such as the 0.016-in.

(a) With REM 800

(a) With REM 800

(b) Without REM 800

Figure 3.55. Specimens coated with DuralPrep®.

(b) Without REM 800

Figure 3.56. Specimens coated with SilACT®.

Table 3.9. Plan for the durability tests, Stage III.

Crack Width (in.)	Control Batch	DegaDeck®		Transpo Sealate®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
Number of Specimens, Stage III (Total = 69 Specimens)									
0.000	3								
0.007	3	3	3	-	-	3	3	3	3
0.012	-	-	-	-	-	-	-	-	-
0.016	3	3	3	-	-	3	3	3	3
0.033	3	3	-	-	-	3	-	3	-
0.054	3	3	-	-	-	3	-	3	-
Total	15	12	6	-	-	12	6	12	6

and 0.033-in. cracks treated only with DegaDeck®) that were unexpectedly water resistant in relation to the other specimens with that sealant may have been unintentionally administered extra sealant. The sample may also have been inadvertently tipped during application, allowing the sealant to pool in the crack before drying. There was only one sample of each specific combination of sealant and crack size created, so an average of multiple specimens could not be found.

These inconsistent results led the team to repeat sections of the Stage II test before conclusions could be drawn. (See the following section for information on the Stage III test.)

From Stage II of the durability tests, the four sealants were rated from the analysis of the absorption results at 24 h and the ease of application. Ratings are as follows, from the best to the worst:

1. SilACT®,
2. DegaDeck® Crack Sealer Plus,
3. DuralPrep® A.C., and
4. Transpo Sealate® T-70.

3.4.4 Durability Test, Stage III

For Stage III of the Durability Test, the research team repeated the procedure used in Stage II for DegaDeck®, Dural-Prep® A.C., and SilACT® for the crack widths 0.007-in., 0.016-in., 0.033-in., and 0.054-in. Transpo Sealate® T-70 was removed from the testing because it did not perform well with a vertical application, as shown in Stage II.

The procedure used for Stage III of the durability test is identical to the procedure for Stage II. However, each case containing the same crack width and sealant combination had three separate specimens. Three batches of concrete were required to manufacture the 69 specimens for Stage III. Table 3.9 shows the Stage III durability test plan. Figure 3.57 shows hand application of REM 800 on Stage III specimens.

3.4.4.1 Stage III Test Results

Water was allowed to soak in the dams on the specimens for 48 h. Readings were taken at both 24 h and 48 h, and the percent absorption for each specimen was determined. The percent absorption results for Stage III of testing are given in Tables 3.10 and 3.11.

The final results were fairly similar for each group of identical specimens. This shows that the results gathered are consistent with one another and are repeatable. For clarification purposes, the results from the three identical specimens were averaged together and are shown in Tables 3.12 and 3.13.

Tables 3.12 and 3.13 show that DuralPrep® A.C. was the best performing sealant. It was the thickest sealant of those that were tested. It performed well both with and without the REM 800 cementitious packing material, showing almost no measurable absorption of water in either case. Even with the largest cracks, this thick sealant was able to fill the gap created by the crack without leaving voids into which the water could seep. The reason that DuralPrep® A.C. did not perform as well in Stage II of testing was that small openings appeared at the crack location, allowing water to seep into the hole in the sealant. The product itself is waterproof, however, the person applying the sealant must take care not to leave any open voids in the layer of sealant. DuralPrep® A.C. was the only sealant that gave acceptable results without REM 800.

The second-best-performing sealant was DegaDeck® Crack Sealer Plus. With the REM 800, the specimens showed

Figure 3.57. Hand application of REM 800.

Table 3.10. Summary of 24-hour percent absorption for the durability test, Stage III.

Crack Width (in.)	Control	DegaDeck®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
0.000	2.164 2.383 2.293						
0.007	4.056	0.003	0.493	0.000	0.000	0.069	3.182
	3.871	0.000	2.850	0.000	0.000	0.325	2.712
	4.116	0.000	3.610	0.000	0.000	0.071	1.355
0.016	4.624	0.000	3.854	0.000	0.000	0.092	3.549
	4.737	0.003	4.063	0.000	0.000	0.091	2.447
	4.630	0.000	0.170	0.000	0.000	0.058	0.999
0.033	4.475	0.000		0.000		0.115	
	4.650	0.000		0.000		0.111	
	4.657	0.000		0.000		0.064	
0.054	5.091	0.000		0.000		0.299	
	4.951	0.000		0.000		0.149	
	5.120	0.000		0.005		0.127	

Table 3.11. Summary of 48-hour percent absorption for the durability test, Stage III.

Crack Width (in.)	Control	DegaDeck®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
0.000	2.724 2.995 2.940						
0.007	4.358	0.000	0.762	0.000	0.000	0.085	3.861
	4.360	0.000	3.644	0.000	0.000	0.411	3.524
	4.406	0.000	3.942	0.000	0.000	0.089	3.000
0.016	4.817	0.000	4.224	0.000	0.000	0.126	4.081
	4.919	0.000	4.285	0.000	0.000	0.110	3.177
	4.781	0.000	0.212	0.000	0.000	0.082	1.479
0.033	4.690	0.000		0.000		0.164	
	4.842	0.000		0.000		0.164	
	4.784	0.000		0.000		0.084	
0.054	5.182	0.000		0.000		0.401	
	5.069	0.000		0.006		0.199	
	5.225	0.000		0.000		0.172	

Table 3.12. Summary of 24-hour percent absorption for the durability test, Stage III.

Crack Width (in.)	Control	DegaDeck®		DuralPrep® A.C.		SilACT®	
		With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
0.000	2.280						
0.007	4.014	0.001	2.318	0.000	0.000	0.155	2.416
0.016	4.663	0.001	2.696	0.000	0.000	0.081	2.332
0.033	4.594	0.000		0.000		0.097	
0.054	5.054	0.000		0.002		0.192	

Table 3.13. Summary of 48-hour percent absorption for the durability test, Stage III.

Crack width (in.)	Control	DegaDeck®		DuralPrep® A.C.		SilACT®	
0.000	2.886	With REM 800	Without REM 800	With REM 800	Without REM 800	With REM 800	Without REM 800
0.007	4.375	0.000	2.783	0.000	0.000	0.195	3.462
0.016	4.839	0.000	2.907	0.000	0.000	0.106	2.912
0.033	4.772	0.000	✕	0.000	✕	0.137	✕
0.054	5.158	0.000	✕	0.002	✕	0.257	✕

almost no measurable absorption of water. However, without the REM 800, the cracks were not able to be bridged nor sealed, and water was allowed to seep into the open cracks. DegaDeck® is too thin to bridge even the smallest cracks tested. For the specimens without REM 800, the specimens with a crack size 0.016 in. absorbed more water than the specimens with a crack size 0.007 in. However, neither absorbed as much water as the control group that did not have a sealant. DegaDeck® Crack Sealer Plus has performed well in all phases of testing and has been near the top in each experiment.

SilACT® did not perform as well as the other two sealants tested in this experiment. It performed relatively well with REM 800, but did not perform well without REM 800. With REM 800, only a small amount of water was allowed to seep into the concrete. However, in all cases water was continuing to seep into the concrete from Day 1 to Day 2, and would continue to do so as time went on. The sealant was thin enough that, without REM 800, the product was not able to bridge the void created by the crack. This left a large opening that water was able to pass through and absorb into the concrete. The water that absorbed into the SilACT® specimens was still less than the water absorbed by the control group that did not have a sealant.

The control group absorbed the highest volume of water, as expected. The results in Tables 3.10 through 3.13 also show a relationship between the amount of water absorbed and the crack width. Typically, wider cracks absorbed more water, and narrower cracks absorbed less water. This is not apparent in all cases, but most of the specimen results follow this statement.

These results are different than those acquired in Stage II of testing. This may be due to procedural error and the fact that there were multiple specimens from which to take an average. By Stage III, the team had become more familiar with the testing procedure and would have been more careful with the sealant application.

The team was able to propose that when using thin sealants, packing cracks with a thick cementitious material allows the cracks to be closed when the sealant alone is not adequate. In order to make this a universal statement and to avoid confusion on limits on sealant viscosity, a packing material is recommended with the use of all sealants. Typically, the specimens

treated with REM 800 were able to prevent water absorption better than the specimens that were not treated with REM 800. The material packed into the crack created a bridge over which the less viscous water resistant sealants were allowed to lay, forming an unbroken seal across the entire surface.

This experiment was designed to exaggerate actual bridge conditions to which end zone cracks would be exposed. In service, the crack surface would not be continuously under water, as the specimens were, but the exposure to wet environmental conditions would extend for a much longer period of time.

3.4.5 Chemical Composition of the Sealers

In order to help design engineers specify the appropriate sealer for a project, Table 3.14 lists the sealers used in this study and their chemical composition.

3.5 Field Inspections of Bridges

3.5.1 Introduction

The objectives of the field inspection of highway bridges were to determine the following:

1. Does the width of end zone cracking change with time?
2. If end zone cracking was detected at the precast plant and no repair was conducted, do these cracks lead to corrosion of the strands and bars, or delamination of the concrete?

To investigate these issues, the research team selected two pilot states, Nebraska and Virginia. Two bridges were selected from Nebraska and three bridges were selected from Virginia for field inspection. The inspection process included

- Collection of reports for inspections conducted at the plant, examination of the reports, and identification of repair method and material;
- Collection of inspection reports for the bridges in service; and
- Research team visits of the bridges under study for inspections that included observation of crack growth since

Table 3.14. Sealers used in this study and their chemical composition.

Product	Chemical Compound
Pipewipe®	Cementitious product: silicon dioxide + Portland cement
DuralPrep® A.C.	Water-based, epoxy-modified Portland cement bonding agent and anti-corrosion coating • Part A: water + bisphenol A polyglycidyl ether resin + phenol + glycidyl ether + octylphenoxypolyethoxy ethanol + benzyl alcohol • Part B: water + polyamine polymer+ polyamine + tetraethylene pentamine • Part C: Portland cement + Portland cement + amorphous silica + vinyl acetate copolymer
Transpo Sealate® T-70	A specially formulated, high molecular weight methacrylate resin system
Xypex® Concentrate	A crystalline waterproofing system
DegaDeck® Crack Sealer Plus	A reactive methacrylate resin
SilACT®	A clear penetrating silane with ethylsilicate treatment specially formulated to treat limestone; the treatment causes concrete, masonry and many natural stones to become repellent to water, chloride, and other waterborne contaminants and weathering elements
REM 800	Fast-setting concrete patching material; self-bonding patching compound with special cement and additives

production and examination for signs of reinforcement corrosion and concrete delamination.

The complete inspection reports for all bridges inspected in this project are presented in Appendix F.

3.5.2 Nebraska Department of Roads (NDOR)

With help from NDOR, the research team selected two bridges for inspection. The first bridge was located on Highway 6, near the 168th Street exit, over a branch of Papillion Creek, in Omaha, Nebraska. The second bridge is located on I-80 over the Platte River in Cass County, Nebraska.

3.5.2.1 Papillion Creek Bridge

This bridge is located on Highway 6, near the 168th Street exit, over a branch of Papillion Creek, in Omaha, Nebraska. The bridge consists of three spans (95 ft, 122 ft, and 95 ft, respectively), with a bridge deck 117 ft wide on the east end and 124 ft wide on the west end, as shown in Figure 3.58.

The bridge was constructed in 2002 to 2003 and girder ends were consistently encased from the top flange to the top of the bottom flange. The team members were able to get close access to all of the girder ends in order to look for end zone cracks. No visible cracking was noted at the ends of any of the girders, as shown in Figure 3.59. The concrete encasing the ends of each girder extended about a foot from the end. It is possible, although unlikely, that very small end zone cracks may have existed within a foot from the end of the girder, but these would have been covered by the end block. The end block would prevent any water or chlorides from penetrating these possible cracks; therefore, they would not be a threat to the girder. The research team was not able to get the records of end zone cracking from the precast producer.

3.5.2.2 Platte River Bridge

Members of the research team visited a bridge on Interstate I-80 over the Platte River in Cass County, Nebraska. The bridge was in the process of being replaced with new spans of precast prestressed concrete girders. It consists of 10 spans total; two 156-ft spans and eight 166.5-ft spans. The bridge deck is 206 ft wide and the girders are prestressed with fifty-eight 0.6-in.-diameter strands. See Figure 3.60.

The research team compiled and reviewed the girder production records and post-pour product inspection reports from the precast producer, Coreslab Structures, Inc., of Omaha, Nebraska. The team was able to inspect both interior and exterior girders on the Platte River Bridge. A self-propelled scissor boom lift was used to get right up next to the girder ends on the eastbound section. The team also walked along the eastern side of the river and was able to inspect each of the girder ends resting on that bank on both the westbound and eastbound sections.

All of the girders that were inspected are NU2000s (79-in.-deep section), and they all experienced end zone cracking. The crack patterns, as well as the crack widths and lengths, were fairly consistent from one girder to another. Generally, the cracks in the end zones were reported to be 0.004-in. to 0.008-in. wide and ranged from 2 ft to 6 ft long.
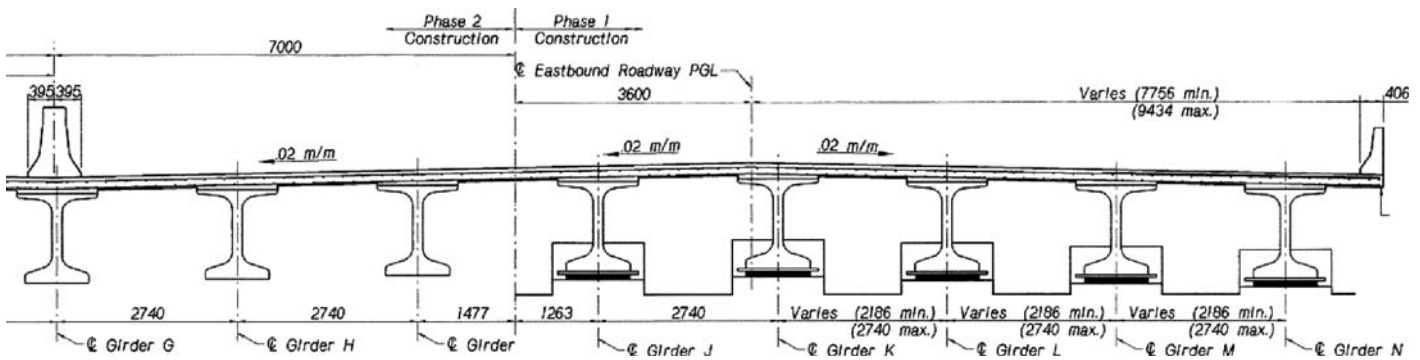
Although evidence of end zone cracking was prevalent, there were no signs of further damage to the girders (such as reinforcement corrosion or delamination). Most cracks
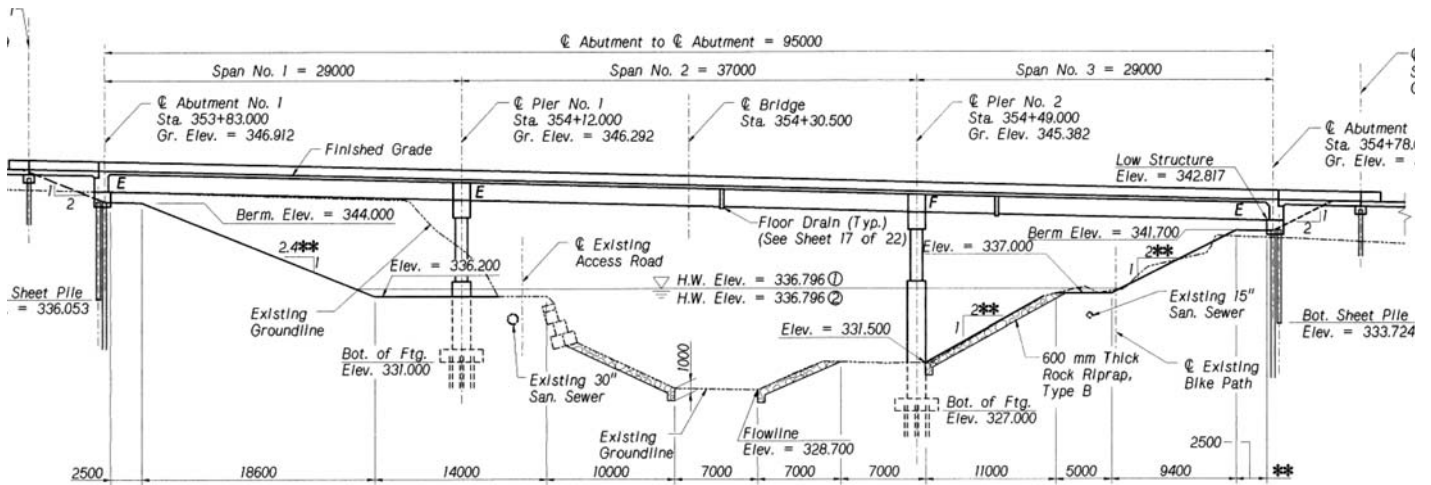
(a) Longitudinal Profile
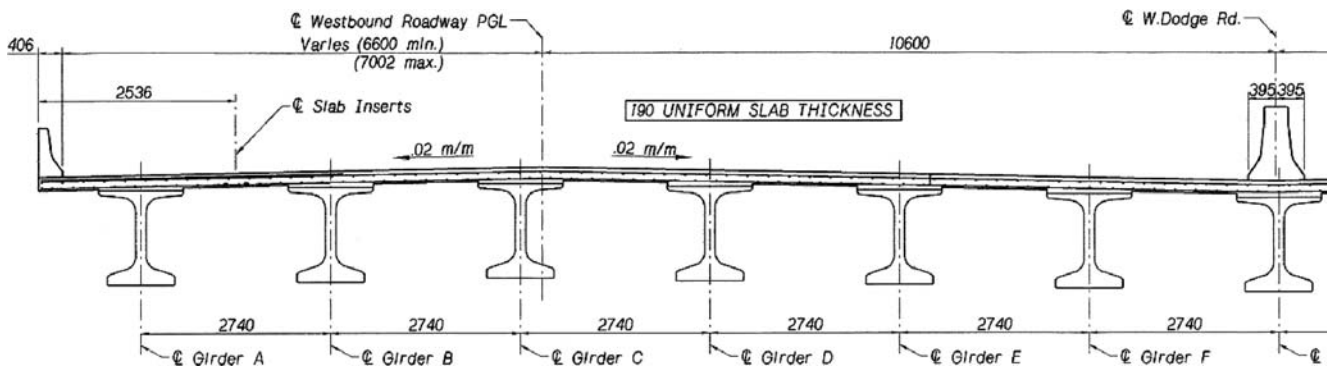
(b) Cross Section of the Westbound Side

(c) Cross Section of the Eastbound Side

Figure 3.58. Papillion Creek Bridge, Omaha, Nebraska.

had a white-colored efflorescence surrounding them. There appeared to be neither structural nor durability problems occurring. The girders have only been in place for a few years, but so far no durability issues have been observed at the girder ends. Examples of the girder ends with end zone cracks are shown in Figure 3.61 where the cracks are highlighted for clarity.

The cracks shown on the girders in Figure 3.61 are traveling in two distinct directions. Near the top portion of the web, the cracks are traveling diagonally downward. Near the bottom portion of the web, the cracks are traveling diagonally upward. The top section of cracks was caused by the prestressing force of strands in the bottom flange. Likewise, the collection of cracks on the bottom was caused by the prestressing strands

Figure 3.59. Girder showing end block.

near the top of the girder. This arrangement of prestressing strands in both the top and bottom portions of a girder creates increased stress on the girder end, amplifying the likelihood of increased end zone cracking.

The team received inspection documents from Coreslab Structures, Inc. The inspector was looking for any imperfections or damage to the girders. Each individual girder design has its own unique piece mark number. This number identifies each girder in the prestressing plant. Each number belongs to only one girder in the project, and there are no repetitions. However, this number is not carried over once the girder leaves the precast plant. Once the girder is in the field and placed on a bridge, there is no way to identify its piece mark number and there is no way to get previous information on any specific girder. Therefore, it was impossible to follow a specific girder from the precast yard to storage and then to the construction site. If a girder was repaired, one would not be able to locate this girder on the bridge to see if the original damage was causing any problems. If a girder on a bridge started to show signs of deterioration after years of use, there would be no way to look up that girder's specific history to see what kind of cracking, damage, or repair it was subjected to earlier on in its service life. For this reason, the research team recommends giving each individual girder an identification number for the entire life of the girder.

Although every single girder that was observed experienced end zone cracking, there is no record of these cracks drawn on the inspection sheets. There are records of vertical cracks and shrinkage cracks, but nothing is mentioned about end zone cracks. In their response to the research team, the precast producer made the following statement:

> . . . the inspectors do not record end zone cracks unless they exceed acceptable limits. The presence of these cracks is expected, and it would be redundant to mark down the same cracks for every girder, especially if they are inconsequential. The only way these cracks would be reported would be if the inspector felt they were severe enough to be repaired.
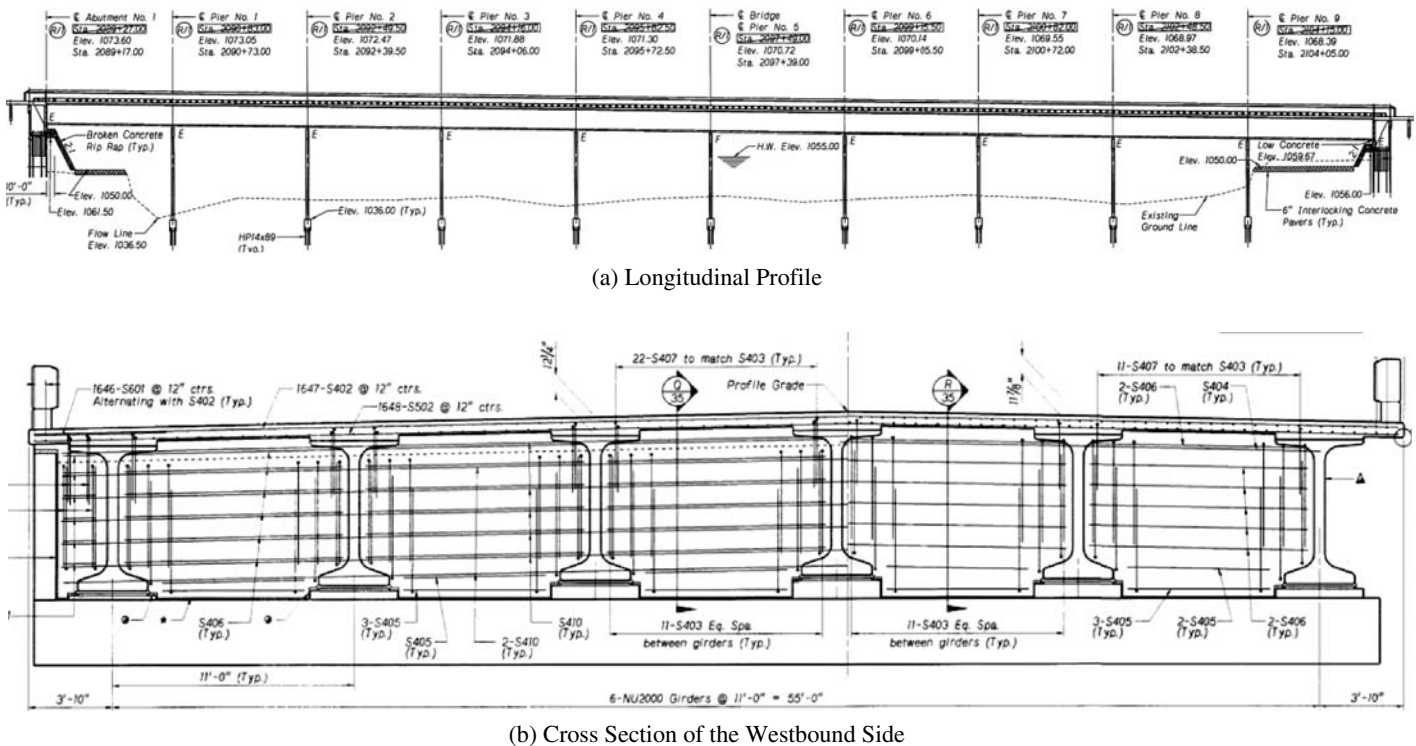

(a) Longitudinal Profile

(b) Cross Section of the Westbound Side

Figure 3.60. I-80 Platte River Bridge, Nebraska.

Figure 3.61. End zone cracks on the Platte River Bridge girders.

3.5.3 Virginia Department of Transportation (VDOT)

The research team, jointly with VDOT, selected two bridges on Route 33 and one bridge on Route 614, for inspection. The selected bridges were constructed between 2005 and 2007. Girders of these bridges experienced end zone cracking at strand release. Some of the girders were repaired in the precast yard.

The two bridges on Route 33 are located in West Point, Virginia, and they are next to each other. The first bridge is over the Mattaponi River between King William and Queen Counties, and the second bridge is over the Pamunkey River between New Kent and King William Counties. The third bridge is located on Route 614 (Hickory Fork Road) over Carter's Creek, 1.6 miles west of Route 17, in Gloucester County, Virginia.

3.5.3.1 Bridge on Route 33 over Mattaponi River, West Point, Virginia

The bridge consists of 28 spans with 7 girders at 10 ft, 6 in. and two overhangs, 3 ft, 8 in. long each, as shown in Figure 3.62. All girders were lightweight concrete of 120 pcf with a 28-day concrete strength of 8 ksi. Half-inch diameter, 270 ksi, low relaxation pretensioned strands were used in all spans. The eastbound and westbound approaches of the span were made from pretensioned VA 45-in.-deep new bulb tee girders, as

shown in Figure 3.63. The spans over the Mattaponi River were made from pretensioned/post-tensioned VA 95-in.-deep new bulb tee girders, as shown in Figure 3.64.

The concrete girders were fabricated by Standard Concrete Products (SCP), Inc. of Tampa, Florida, in October 2005. According to the shop inspection reports, end zone cracking was reported in almost of all the girders. End zone cracking in the web ranged from hairline to 0.016 in. at time of release, and some of these cracks grew up to 0.020 in. at time of shipping. For web cracks 0.009 in. and under, the precast producer sprayed on a penetrant sealer all along the crack. Sikagard 701W was used in this project. Web cracks that are 0.010 in. or greater were epoxy injected using the typical epoxy injection procedure given in the PCI *Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products* (*11*). Prime Rez 1100 High Mod LV was used in this project.

The bridge was open to traffic in September 2006. The inspection report that was generated by the district engineer at that time stated that "hairline diagonal cracks exist in the web of Spans 'j through q.'" However, the report did not provide any detailed information on crack size, length, or number.

The research team inspected the bridge on July 1, 2008, and was looking for signs of distress such as delamination (which was inspected by tapping with a hummer on the girder at the crack and its vicinity) and reinforcement corrosion. End zone cracks were visible in almost all of the girders on the bridge. The crack width ranged from 0.008 to 0.010 in. wide.

Figure 3.62. Cross section of the bridge (eastbound and westbound approaches).

END VIEW

SECTION A-A

Figure 3.63. Girder details of the eastbound and westbound approaches.

SECTION A-A
Scale: ¾" : 1'-0"

SECTION B-B
Scale: ¾" : 1'-0"

SECTION C-C
Scale: ¾" : 1'-0"

Notes:

Figure 3.64. Girder details of the spans over the Mattaponi River.

Efflorescence could be seen around some of these cracks. The inspection engineer stated that the efflorescence was reported at the time when the bridge was opened to traffic in 2006 and, based on his experience, the amount of efflorescence did not increase with time. No signs of reinforcement corrosion were reported, except in one girder in one of the spans of the eastbound approach, as shown in Figure 3.65. No delamination was reported in the girders that were inspected.

3.5.3.2 Bridge on Route 33 over Pamunkey River, West Point, Virginia

The bridge consists of 49 spans. Girders of all spans were made of 120 pcf lightweight concrete with a 28-day concrete strength of 8 ksi. Half-inch diameter, 270 ksi, low relaxation strands were used in all spans. The bridge is made of eight VA New BT girders spaced at 9 ft, 6 in. to 11 ft, 6 in., as shown in Figure 3.66. Figures 3.67 and 3.68 show the cross section of

Figure 3.65. Web end zone cracking (0.009 in.) showing some efflorescence, no delamination, and some signs of corrosion.

TRANSVERSE SECTION - UNITS J & K
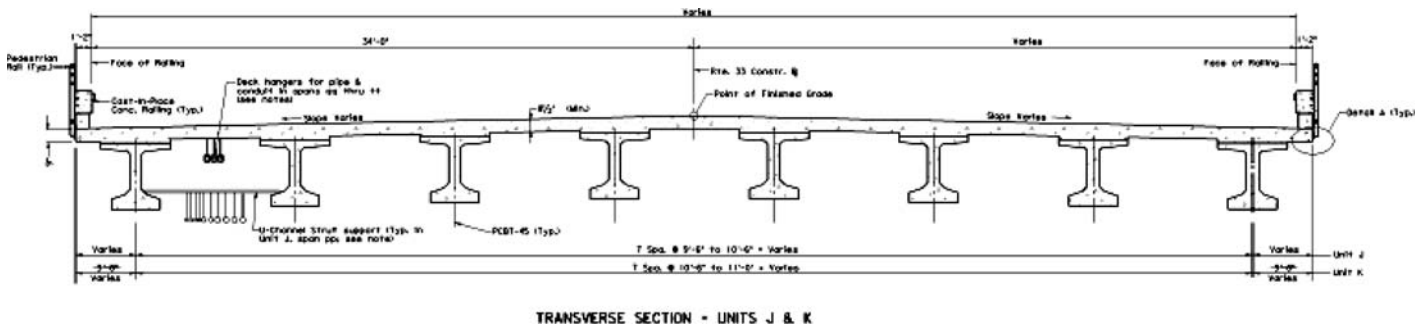
Figure 3.66. Cross section of the bridge on Route 33 over the Pamunkey River.
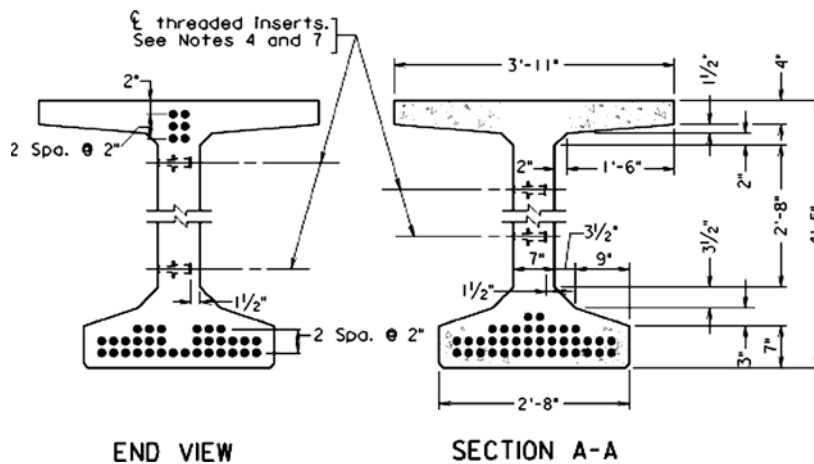

END VIEW　　　　SECTION A-A

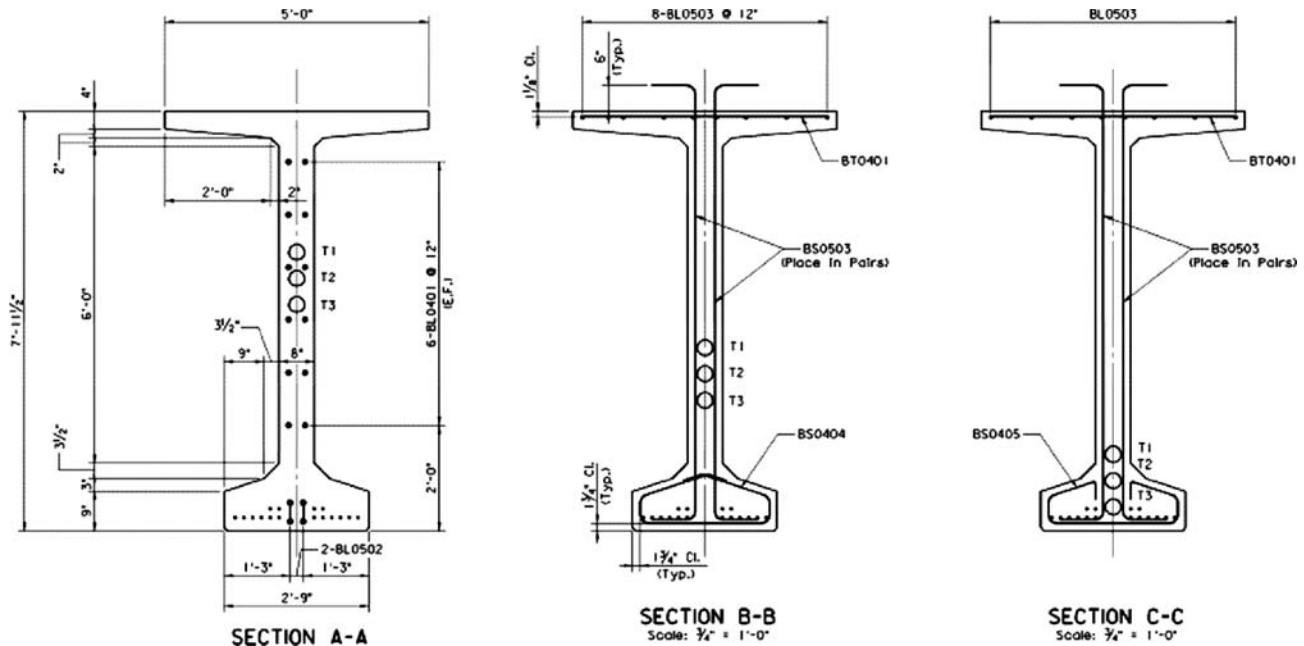**Figure 3.67. Girder details of the eastbound and westbound approaches.**

Figure 3.68. Girder details of the spans over the Pamunkey River.

the girders used on the eastbound and westbound approaches and the spans over Pamunkey River.

The bridge was opened to traffic in late 2007. The inspection report that was generated by the district engineer at that time stated that "hairline diagonal cracks exist in the web of Spans 'j through q.'" However, the report did not provide any detailed information on crack size, length, or number. The research team inspected the bridge on July 1, 2008, and was looking for signs of distress such as delamination and reinforcement corrosion. End zone cracks were visible in many exterior and interior girders on the bridge. The crack width ranged from 0.008 to 0.010 in. No efflorescence, signs of reinforcement corrosion, or delamination was reported in the inspected girders, as shown in Figure 3.69.

3.5.3.3 Bridge on Route 614 over Carter's Creek in Gloucester County, Virginia

The third Virginia bridge is located on Route 614 (Hickory Fork Road) over Carter's Creek, 1.6 miles west of Route 17, in Gloucester Co. The bridge consists of 6 spans (82.5 ft each) with one expansion joint at the center pier. The cross section of the bridge consists of 6 girders at 7 ft, 4 in., which support 8-in.-thick cast-in-place concrete slab, as shown in Figure 3.70. All of the girders are 54 in. deep AASHTO Type IV. The bridge crosses over a marsh land with a very humid environment. Also, the bridge has a very low profile that places trees in contact with the bottom flange of most of the spans, as shown in Figure 3.71. The bridge was opened to traffic in 2006.

It was evident that all of the girders have an almost identical pattern of end zone cracking. At each end of the girders, one end zone crack is formed where it extends from the top of the flange and the member end to the top surface of the bottom flange, as shown in Figure 3.72.

It was evident that efflorescence exists on about 75% of the cracks. Upon inspection, the concrete at the crack and in its vicinity looked very sound with no signs of delamination or reinforcement corrosion. The size of the crack ranges from 0.006 to about 0.009 in., with the majority of the cracks

Figure 3.69. Web end zone cracking (0.008 in.) showing no efflorescence, no delamination, no signs of corrosion.
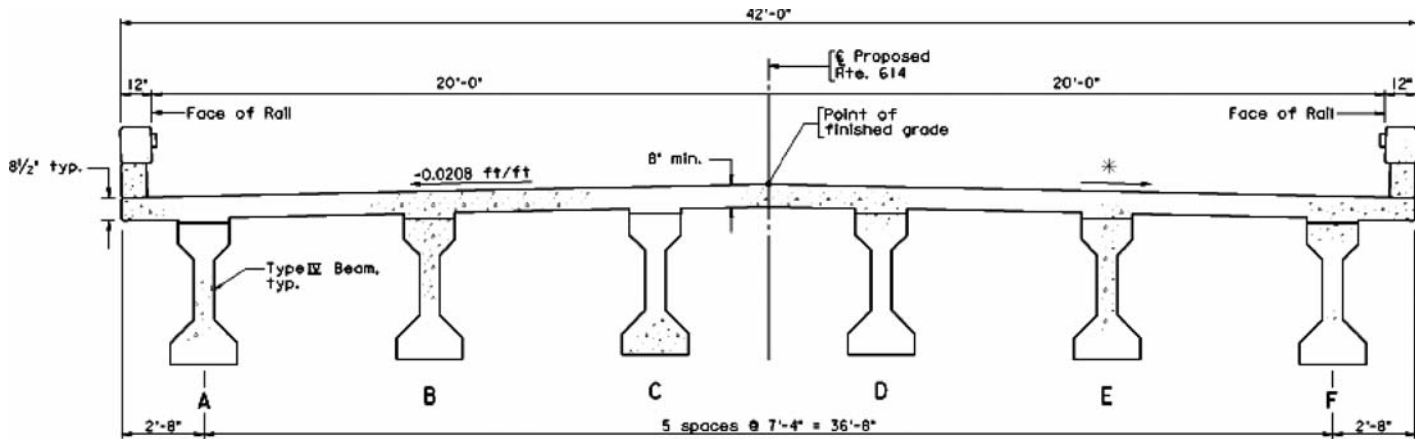
(a) Cross Section of the Bridge
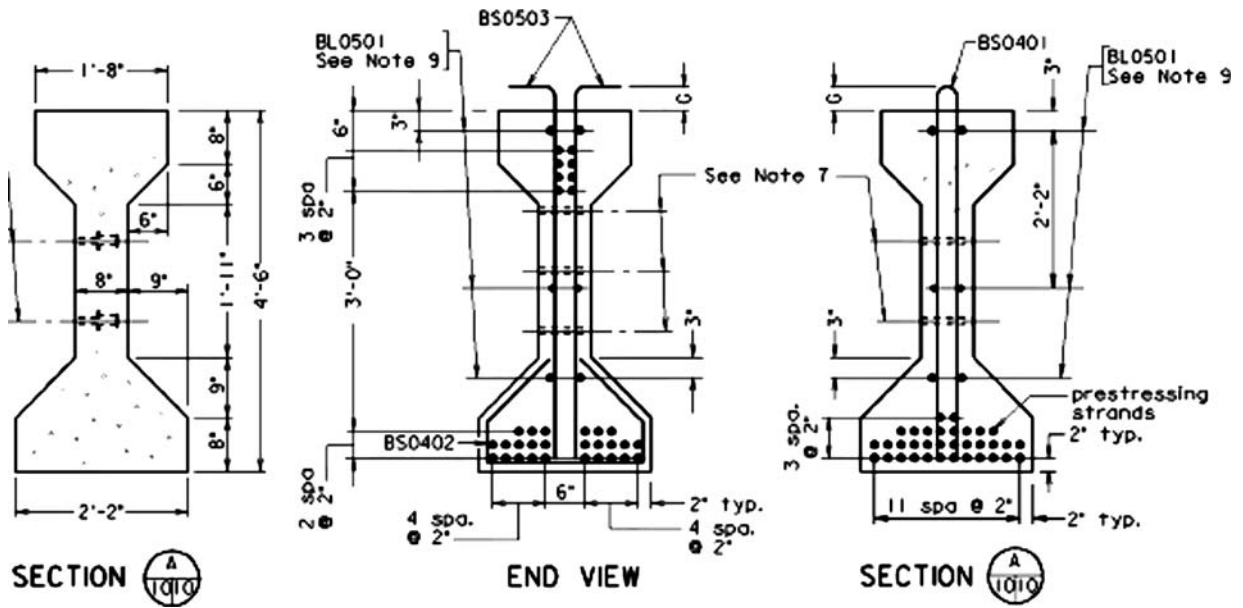
(b) Girder Details

Figure 3.70. Details of the bridge on Route 614 in Gloucester County, Virginia.

Figure 3.71. View of bridge over Carter's Creek showing its low profile.

Figure 3.72. End zone crack, Span 5, left side, exterior girder, 0.006 to 0.008 in. wide, with efflorescence, no delamination, no reinforcement corrosion.

around 0.008 in. wide. The end zone cracks extend for about 30 in. from the face of the member end. No signs of shop repair could be detected by the research team. Although no inspection reports were made available by the precast producer for review by the research team, it was clear that no repair was made to the end zone cracks.

3.5.4 Bridge Field Inspection Conclusions

Based on the field inspection conducted on five bridges in Nebraska and Virginia, the study team made the following conclusions:

- Of the five bridges inspected in Nebraska and Virginia, four bridges were built over water channels where the ambient air is humid. Field inspection of these bridges did not reveal any signs of reinforcement corrosion or concrete delamination, although end zone cracking had existed at the time of pre-stress release.
- Comparing the crack widths at the time of inspection with those documented in the inspection reports revealed no deterioration.
- There is no NDOR or VDOT policy that specifically requires field inspection reports to document end zone bursting cracks, regardless of whether they had been reported in plant inspection reports. Also, there was no consistency in girder identification between the producer's and the owner's identification systems. Thus, it was difficult for the research team to gain much value from field inspection reports. These constraints reduced the researchers' effort to recording cracks in the field without fully correlating them with cracks at the plant before the girders were shipped.
- There was no documentation relative to methods and materials used to repair end zone cracking.

3.6 Manual of Acceptance, Repair, or Rejection

Based on the information collected from field inspection in Nebraska and Virginia, and the results of the structural testing of eight full-scale girders, the research team devel-

oped Table 3.15 to provide decision criteria for acceptance and repair of web end cracking during production.

These criteria were developed based on observation of the results of structural testing of eight full-scale girders and field inspection of five bridges. The investigation shows that

- There was no deficiency of shear, bond, or flexural capacity attributed to end zone cracking whether the cracks were filled or not prior to testing.
- The epoxy repaired end of one of the Washington girders did not exhibit any improvement in load carrying capacity over the unrepaired end. Thus, if epoxy repair is desirable, it should be intended only to seal the cracks, not to restore tensile capacity of the repaired surface.
- No signs of efflorescence or corrosion due to web end cracking were reported in the inspected Virginia and Nebraska bridges.
- No cracks wider that 0.01 in. were observed, even in the cases where end zone reinforcement was extremely light (#4 at 12 in.) and the prestressing force is relatively large (sixty-two 0.6-in. diameter strands).
- Most observed crack lengths from this study and from previous reports were limited to about 36 in.

End zone cracking is quite different from flexural cracks in conventionally reinforced beams and slabs, and from tensile cracks in water storage structures. Even if one equates these cracks with flexural cracking, the 0.012-in. width is less than the 0.013 in. and 0.016 in. used in early versions of the ACI-318 Building Code for exterior and interior exposures, respectively. It corresponds to the "z" value of 130 kip/in. that was previously used in AASHTO specifications to indirectly control crack width in environments with severe exposure. Specification of crack width limits and "z" value limits for flexural design have been dropped from recent editions of the ACI-318 Building Code and from the AASHTO specifications. This was done due to evidence that flexural cracking, which is normal to the flexural reinforcement, does not correlate to reinforcement corrosion. Thus, it is quite reasonable to limit end zone cracking to 0.012 in., without need for any repair.

Table 3.15. Decision criteria for acceptance and repair of web end cracking during production.

Criterion	Crack Width (in.)	Action
1	Less than 0.012	No action
2	0.012 to 0.025	Fill the cracks and apply surface sealant to the end 4 ft as recommended in this report
3	0.025 to 0.05	Fill cracks with epoxy and apply surface sealant to the end 4 ft as recommended in this report
4	Greater than 0.05	Reject girder, unless shown by detailed analysis that structural capacity and long-term durability are sufficient

Criterion 1: Crack width less than 0.012 in.

Action recommended: No action is recommended.

With a crack width this narrow, no repair is required. However, if the owner requires repair, steps for Criterion 2 can be followed.

Criterion 2: Crack width 0.012 in. to 0.025 in.

Action recommended: Apply cementitious packing materials to cracks between 0.012 in. and 0.025 in. Apply surface sealant to the end 4 ft as recommended in this manual.

It is recommended that the crack be filled with a cementitious packing material and covered with a water-resistant surface sealant to keep water contaminated with corrosion-inducing chemicals from reaching the steel inside the girder.

The area in question should be cleaned and cleared of any debris such as dirt, dust, grease, oil, or any other foreign material. This will aid in the bonding of the material to the concrete. Cleaning products that are corrosive should not be used.

It is best that the packing material used to fill the cracks be cementitious, slightly viscous, and easily worked by hand. The material should be rubbed into the cracks either by hand or by brush until the entire outer opening is filled and a surface is created that is even with the original girder web surface. Excess material should be wiped off so the surface remains even.

The surface sealant should be water resistant and highly flowable. Its application should result in a smooth surface. The sealant should be applied with either a brush or a roller so the side faces of the girder are fully covered. The top face of the girder where it normally is connected with a cast-in-place concrete slab should not be covered with sealant. It is recommended that a minimum length of 4 ft at each end of the girder be covered.

Examples of acceptable patching and sealant materials to be used are provided in Section 3.4 of this report.

Criterion 3: Crack width 0.025 in. to 0.050 in.

Action recommended: Epoxy injection of all cracks larger than 0.025 in. Apply surface sealant to the end 4 ft as recommended in this report.

For cracks wider than 0.025 in., epoxy injection is recommended. It is important that this be performed such that the crack is completely filled and that the epoxy is effectively bonded to both surfaces of the crack. Cracks of this size in the web generally exist in the full width of the web and appear on both side faces of the member. Injection must be done in accordance with proven practices and epoxy manufacturer's specifications. Epoxy pressure should be high enough to fully penetrate the crack depth, yet the pressure should not cause a blow out of the epoxy paste material used to confine the epoxy.

Before injection, the surface and interior of the crack should be cleared of all debris such as dirt, dust, grease, oil, moisture, or any other foreign material without using corrosive chemicals. If loose particles have entered the crack, they can be blown out with filtered high-pressure air equipment, as long as they do not introduce oil into the fissure. Water, solvents, or detergents should not be used because they may compromise the ability of the epoxy to bond to the concrete.

When applying the epoxy, the crack should first be examined to determine the ideal placement for the injection ports.

Port spacing can depend on the crack width and the amount of pressure applied. Professional judgment from an experienced injector should be used. The ports should be at least 8 in. apart. However, if the crack passes through the entire web, the spacing should not exceed the thickness of the web. After the ports are installed, the exterior of the cracks are to be sealed with an epoxy paste and allowed to harden. This is to prevent the injected epoxy from leaking out of the crack. With cracks that extend on both sides of the girder, the opposite side of the injection should be sealed as well. If the cracks on each side do not connect, epoxy injection should be performed on each side individually.

After confining the cracked area is completed, the epoxy can be mixed and the injection can begin from the bottom up. Injection should be performed with an epoxy injecting machine. The lowest injection port should be filled with epoxy first until it begins to come out of the next port, which is slightly higher than the first port. The used port is to be plugged so the epoxy does not leak out. Then, the process can be repeated until epoxy begins to come out of the next port in line. This process continues until the top port is reached, and the crack is completely filled. The final port should be placed a few inches away from the termination point of the crack, but this remaining portion of the crack should still be filled with the last injection.

Criterion 4: Crack width greater than 0.050 in.

Action recommended: Reject girder, unless shown by detailed analysis that structural capacity and long-term durability are satisfactory.

Cracks exceeding a width of 0.050 in. may be symptomatic of causes beyond the normal effects of bursting forces due to prestress release. All aspects of material quality, reinforcement quality and quantity, and production practices must be examined. If a loss of structural capacity were to occur, typical methods of epoxy injection may not be sufficient to measurably return the girder back to its intended strength, especially if cracking causes excessive loss of prestress.

3.7 Improved Crack Control Reinforcement Details for Use in New Girders

Most designers follow the provisions of Article 5.10.10.1 of the AASHTO LRFD Bridge Design Specifications (*18*). However, states with recently introduced I-girder shapes that can accommodate a relatively large amount of prestressing (as many as sixty-eight 0.6-in. diameter strands) have developed supplementary requirements for end zone reinforcement.

Factored Bursting Resistance, as given in Article 5.10.10.1 of the AASHTO LRFD (*18*), indicates that the end reinforcement resistance shall not be less than 4% of the prestressing force at transfer. The end zone reinforcement is designed for 20 ksi allowable stress to control the crack size and is located within h/4 from the end of the girder, where h is total girder depth.

The following recommendations offer improvements to the AASHTO provisions, especially for cases with high

prestressing levels. Effective and simplified reinforcement detailing is proposed.

The following recommendations are based on experience in Nebraska and Washington State where very large amounts of prestressing have been provided on some projects. A research project conducted for NDOR resulted in recommendations published in a 2004 *PCI Journal* article (*16*). Results of this project have shown that the end zone reinforcement closest to the member end is the most stressed and would correspond to the widest crack, as shown in Figure 3.73. Also shown is that the stress in the vertical reinforcement drops sharply at a distance h/8 away from the girder end, with steel beyond the h/2 distance having little influence on cracking.

Since the end zone reinforcement is provided to minimize the crack width, and not for strength, there is no need to develop the full yield strength beyond the locations of the top and bottom cracks, which are assumed for design to be at the junction between the web and the flanges.

The results of this research, along with additional recommendations from the Nebraska and Washington producers involved in NCHRP Project 18-14, have been used in the full-scale testing in this project (see Section 3.2 of this report), where these recommendations have been compared with the AASHTO LRFD provisions as well as other local practices in the four states supplying eight full-scale girder specimens.

The full-scale testing confirmed that, although the AASHTO LRFD requirements provided acceptable performance in all cases, the proposed details provided better performance. More significantly, the proposed details lend themselves to optimal bar detailing with minimized end zone reinforcement congestion. The team has found it to be most effective to have a large area of vertical steel as close as possible to the end of the girder, with the steel area gradually diminishing as the distance from the end is increased. The reinforcement must be anchored well enough into the bottom and top flanges to assure no slippage at the design stress level of 30 ksi. The bottom flange must also be confined with a minimum amount of confinement steel to help resist strand slippage and boundary zone cracking.

The five proposed requirements are as follow:

(1) Provide reinforcement in the end (h/8) to resist at least 2% of the prestressing force, using an allowable stress limit of 20 ksi.
(2) Provide reinforcement in the end (h/2) to resist at least 4% of the prestressing force, using an allowable stress limit of 20 ksi. The reinforcement in the zone between the h/8 and h/2 sections must not be less than shear reinforcement requirement as stipulated in (3) below.
(3) Beyond the (h/2) zone, provide reinforcement to meet shear requirements at the nearest critical section.
(4) Determine the bar anchorage into the flanges for a maximum stress of 30 ksi.
(5) Confine the strands in the bottom flange with at least the equivalent of #3 bars at 3-in. spacing for a distance equal to at least 60 strand diameters. The #3 bars must totally enclose the bottom flange strands. Welded wire reinforce-
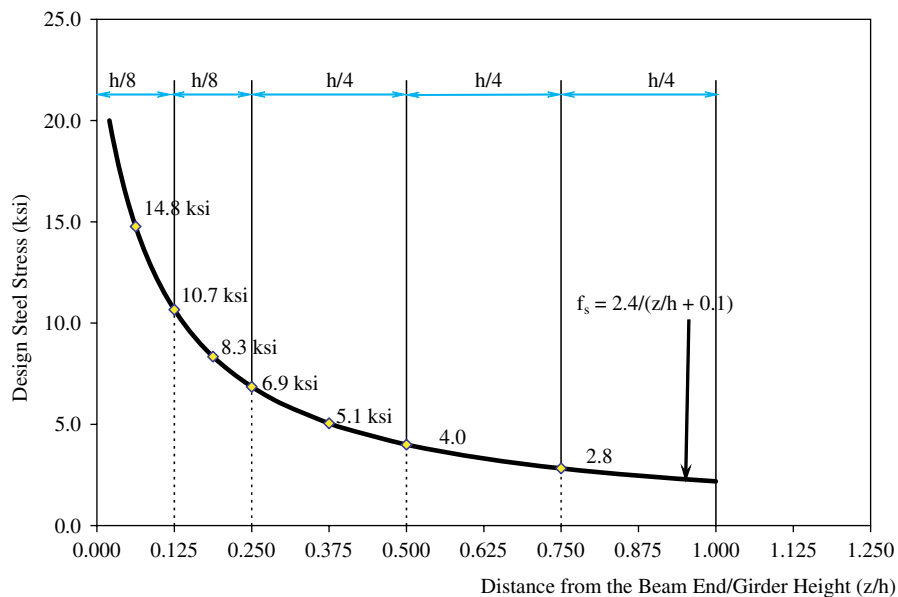
Source: Reprinted with permission from Tuan, C.Y., Yehia, S.A., Jongpitaksseel, N., and Tadros, M.K., "End Zone Reinforcement for Pretensioned Concrete Girders," *PCIJournal*, Vol. 49, No. 3, May-June (2004), Figure 15.

Figure 3.73. Average measured stress in end zone reinforcement versus distance from the member end.

ment (WWR) of the same area per unit length may be used to substitute for the #3 bars. The same amount of confinement steel must be provided at the bonded ends of all debonded strand groups.

Appendix G provides two examples for the design of end zone reinforcement using the AASHTO LRFD specifications and the proposed requirements.

3.8 Proposed Revisions to the AASHTO LRFD Bridge Design Specifications

The research team proposes the following changes to Article 5.10.10 of the AASHTO LRFD Bridge Design Specifications (*18*). Table 3.16 presents Article 5.10.10.1, with additions underlined, and deletions struck through.

Table 3.16. Proposed changes to Article 5.10.10.1 of the AASHTO LRFD specifications.

5.10.10.1 Factored Bursting Resistance	**C5.10.10.1**
The bursting resistance of pretensioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as: $$P_r = f_s A_s \quad (5.10.10.1\text{-}1)$$ where: f_s = stress in steel not exceeding 20 ksi A_s = total area of vertical reinforcement located within the distance ~~h/4~~ *h/2* from the end of the beam (in.^{2}) h = overall depth of precast member (in.) The resistance shall not be less than 4 percent of the prestressing force at transfer. ~~The end vertical reinforcement shall be as close to the end of the beam as practicable.~~ Vertical reinforcement located within the distance *h/8* from the end of the beam shall be provided to resist at least 2 percent of the prestressing force at transfer. Also, the total amount of vertical reinforcement located within the distance h/2 from the end of the beam shall be provided to resist at least 4 percent of the prestressing force. The reinforcement in the end h/2 shall be not less than that required for shear resistance. Crack control reinforcement shall be anchored beyond the anticipated extreme top and bottom cracks, an embedment adequate to develop at least a stress = 30 ksi.	This provision is roughly equivalent to the provisions of Section 9.22.1 in AASHTO Standard Specifications (1996). Results of tests conducted by the Florida Department of Transportation were taken into account. Additional research by Tuan et al. (*PCI Journal*, 2004) and Tadros et al. (NCHRP 18-14, 2009) shows that distribution of the 4% reinforcement such that at least one half of that reinforcement is concentrated in the end h/8 of the member while the balance of the 4% is distributed over a distance from h/8 to h/2 provides for arrest of the cracking at the member end and for well distributed cracks in the balance of the end zone. Since the crack control reinforcement is required to minimize the crack width, and not for strength, there is no need to develop the full yield strength beyond the locations of the top and bottom cracks, which are assumed for design to be at the junction between the web and the flanges. The bar anchorage into the flanges should be designed for a maximum stress of 30 ksi which was found (NCHRP 18-14, 2009) to be conservative.
5.10.10.2 Confinement Reinforcement	**C5.10.10.2**
For the distance of ~~1.5d~~ at least 60 strand diameters 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~~ 3.0 in. and shaped to totally enclose the strands. The same amount of confinement steel must be provided at the bonded end of all debonded strand groups. For box beams, transverse reinforcement shall be provided and anchored by extending the leg of stirrup into the web of the girder.	Welded wire reinforcement (WWR) of the same area per unit length may be used to substitute for the #3 bars.
Add the following references to the list of references of Section 5: • Tuan, C., Yehia, S., Jongpitakseel, N., and Tadros, M., "End Zone Reinforcement for Pretensioned Concrete Girders," *PCI Journal*, Vol. 49, No. 3, May-June 2004, pp. 68-82. • Tadros, M.K., Badie, S.S., and Tuan, C., "Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with Longitudinal Cracking in the Web," NCHRP 18-14, Contractor's Final Report, November 2009 (published as NCHRP Report 654).	

CHAPTER 4

Conclusions, Recommendations, and Suggested Future Research

4.1 Conclusions

Based on the work conducted in this project, the research team has developed a user's manual for acceptance criteria and repair materials and methods for prestressed concrete girders experiencing end zone cracking due to transfer of the pretensioning force. The manual consists of four criteria depending on the crack width. The first criterion is for crack widths less than 0.012 in., where no repair is recommended. The second criterion is for crack widths from 0.012 in. to 0.025 in., where it is recommended that the cracks be filled with a cementitious packing material and then covered with a water-resistant surface sealant to keep water contaminated with corrosion-inducing chemicals from penetrating the concrete and reaching the steel reinforcement. The third criterion is for crack widths from 0.025 in. to 0.050 in., where epoxy injection is recommended for cracks wider than 0.025 in. and cementitious packing material for cracks narrower than 0.025 in. The manual provides the provisions for successful epoxy injection. The fourth criterion is for crack widths greater than 0.050 in., where rejection of the girder is recommended, unless it can be shown by detailed analysis that structural capacity and long-term durability are not compromised. These criteria allow for acceptance of girders with cracks wider than those implied for flexural members in the ACI-318 Building Code and the AASHTO LRFD Bridge Design Specifications. The nature and consequences of end zone cracking are quite different from those of flexural cracking. Flexural cracking in beams has a different impact on member behavior than does end zone cracking. For example, flexural cracks in beams tend to grow in width and depth with the application of superimposed loads. They may cause a negative impact on deflection, vibration, and fatigue behavior of the member. On the contrary, the width of end zone cracks tends to decrease with the application of superimposed loads and the development of time-dependent prestress losses.

Based on the experience gained in this project, the research team was able to develop improved end zone details for use in new girders. End zone reinforcement of the developed details is determined using 4% of the prestressed force at transfer and 20 ksi allowable steel stress, which are the same criteria stated by the AASHTO LRFD specifications. However, the proposed details require that at least 50% of the end zone reinforcement be placed in the end h/8 of the member, where h is the member depth. The balance of the end zone reinforcement is recommended to be placed in the distance between h/8 and h/2 from the member end. This distribution concentrates the reinforcement where the highest bursting stresses are expected to exist. The bursting reinforcement must be embedded into the top and bottom flanges such that it can develop at least 20 ksi at the junctions of the flanges with the web. The anchorage is considerably less than that required to develop the full yield strength of the bars.

An effective detail already in use by at least one producer in the Northwest is to provide a single #8 bar in the center of the web at the end of the member. The bar is bent into a C in the longitudinal direction of the girder to allow the bar adequate anchorage. Another effective reinforcement at the end h/8 of the member is a pair of ¾-in.-diameter coil loop rods with attached nuts to the top and bottom. Please note that the end h/8 is likely to be embedded in an end diaphragm and should theoretically be exempt from minimum concrete side cover between the web face and the bar. Also, regular #4 and #5 bars that are bent into the top and bottom flanges, and not necessarily projecting above the top flange, may be calculated to be adequately anchored.

Based on the proposed end zone details, the research team proposed changes to Article 5.10.10.1 of the AASHTO LRFD specifications.

4.2 Implementation of Research Findings in Highway Communities

The research team will prepare at least one paper about the work conducted in this project and the recommendations will be presented at the 2011 TRB Annual Meeting. Another

paper will be prepared for publication in the *PCI Journal* or the *ACI Structural Journal.* Also, the research team will approach NDOR and VDOT to present the findings and recommendations of this project. This may be done through direct meetings with the bridge division engineers or through arranged seminars.

The research team will submit the proposed change to Article 5.10.10.1 of the AASHTO LRFD specifications to the T-10 AASHTO Committee for possible adoption.

4.3 Suggestions for Future Research

No further research into the acceptance, repair, and rejection of web end cracking due to prestress transfer is suggested. Further research to develop finite element modeling of the end zone of pretensioned members should be of value in optimizing the bursting reinforcement, especially as larger than 0.6-in.-diameter strands and higher than 15 ksi concrete become more common in practice.

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A P P E N D I C E S

Appendices A through G are available on the TRB website. To find Appendices A through G for this report, go to www.trb.org and search for "NCHRP Report 654". Titles of Appendices A through G are as follow:

Appendix A: Literature Review
Appendix B: National Survey
Appendix C: Structural Investigation & Full-Scale Girder Testing
Appendix D: Sealant Specifications
Appendix E: ASTM Specifications
Appendix F: Field Inspection of Bridges
Appendix G: Design Examples of End Zone Reinforcement

Abbreviations and acronyms used without definitions in TRB publications:

AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	Air Transport Association
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation