Evaluation and Repair Procedures for Precast/Prestressed Concrete Girders with End Zone Reinforcement

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ABSTRACT

Precast/prestressed concrete bridge girders are widely used in the United States. End zone cracks at the ends of pretensioned concrete girders are commonly observed at the time of strand release. End zone cracking is quite different from flexural cracks in conventionally reinforced beams and slabs, and from tensile cracks in water storage structures. 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 regarding girders with end zone cracking vary from do-nothing to total rejection of the girders. There is no consensus among owners on the level of tolerance to these longitudinal cracks.

This paper gives a summary of the research activities conducted in the NCHRP 18-14 (Report 654) in order to develop a user's manual for acceptance and repair of girders with end zone cracking. The goal of developing the manual is to provide precast concrete producers and bridge owners with unified guidelines.

Keywords: End zone cracking, end zone reinforcement, prestressed concrete girders, manual, repair, acceptance, rejection, crack width

INTRODUCTION

Precast, prestressed concrete bridge girders are widely used in the United States. End zone cracks, at the ends of pretensioned concrete girders, are commonly observed at the time of 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. Conventional reinforcement is generally placed to keep cracks widths within acceptable limits.

In practice, there is no consistent understanding of the effect of end zone cracking on the strength and durability of the girders. Concerns regarding end zone cracks include the possibility of reduced structural capacity and durability from strand- and bar corrosion. End zone cracks parallel to or intersecting 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 promote strand corrosion. Therefore, 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.

The National Cooperative Highway Research Program (NCHRP) sponsored the research project NCHRP 18-14, which addressed these issues. NCHRP Report 654[1] contains the results of this research project. To address the strength and durability concerns, the research team prepared a list of ten questions and developed a work-plan that would help to answer these questions. Table 1 shows the questions and the corresponding research tasks. The following sections provide a summary of the results of each task.

Table 1. Research-plan developed for NCHRP 18-14 Project

Questions need to be answered	Research-plan tasks
1. What are the current criteria for crack control? 2. What is the current practice used by highway authorities?	A) Literature Review & National Survey
3. Does end zone cracking negatively affect the flexural and shear capacities of prestressed girders? 4. Do variations of the end zone reinforcement details have significant effect on the number, width & pattern of end zone cracks?	B) Structural Investigation & Full-Scale Girder Testing
5. If epoxy injection is used to repair end zone cracking, can repair restore the tensile capacity of the cracked concrete? 6. Is epoxy injection able of completely filling the crack through the width of the web?	C) Epoxy Injection Testing
7. If repair is required, what repair method and material should be used? 8. Should the end zone surface be sealed with a surface sealant regardless of whether cracks are required to be filled with a patching material?	D) Durability Testing

Table 1. Research-plan developed for NCHRP 18-14 Project (cont.)

9. Does the width of end zone cracking change with time? 10. 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?	E) Field Inspection of Bridges

TASK (A): LITERATURE REVIEW AND NATIONAL SURVEY

A review of the literature[2,3,4] showed that the majority of the published guidelines regarding acceptance and repair criteria of prestressed concrete girders consider many types of cracking that may be reported and do not specifically address end zone cracking. However, all of them agree that crack width is the most convenient measure that should be used for establishing user guidelines. It was evident that 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 maximum crack width related to flexural cracks in conventionally reinforced beams varies from 0.002 in. (0.05 mm) for concrete exposed to sea water to 0.016 in. (0.41 mm) for concrete used in dry environment.

Few publications that deal with end zone cracking of prestressed concrete bridge girders were available. The majority of publications on end zone cracking agree that crack width is the best measure to develop practical acceptance and rejection criteria. In 2006, PCI published the Manual for the Evaluation and Repair of Precast, Prestressed Concrete Bridge Products.[5] The objective of the report was to achieve a greater degree of uniformity among owners, engineers, and precast concrete 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 follows:

- Cracks that intercept or are collinear with strands but without evidence of strand slippage, such as significant retraction of strand into the beam end, should be injected with epoxy.
- Cracks that intercept or are collinear with strands with evidence of strand slippage should be injected with epoxy, and a re-computation of stresses after shifting the transfer and development length of affected strands should be conducted.

The PCI report[5] uses the crack widths developed in American Concrete Institute's (ACI's) *ACI Manual of Concrete Practice, Part 2* (ACI 224R-01)[6] as guidelines for whether or not to inject cracks with epoxy. Table 2 summarizes the crack widths that should be injected with epoxy based on exposure condition. The report recognizes the fact that this type of cracking does not grow once the beam is installed in 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. However, the report does not give any guidelines on when to reject a beam with end cracks.

Table 2. End-of-beam cracks that should be injected with epoxy[6]

Exposure Condition	Crack width (in.)
Concrete exposed to Humidity	> 0.012
Concrete subject to Deicing chemicals	> 0.007
Concrete exposed to seawater and seawater spray, wetting and drying cycles	> 0.006

A survey was developed in the NCHRP 18-14[1] project to collect data on the experiences regarding longitudinal end zone cracking. The survey was sent to state highway authorities, bridge consultants, precast concrete producers, and members of the PCI Committee on Bridges and the PCI Bridge Producers Committee. One of the questions of the survey asked about established criteria for when to repair end zone cracking. Table 3 summarizes the results. The majority of state highway authorities, 36 out of 41 responses, stated that crack width is their sole criterion. Also, the majority of the respondents, who recommend using epoxy injection to repair end zone cracking, believe their repair methods do not restore the tensile capacity of the member and their repair is made only to protect the beam reinforcement against corrosion.

Table 3. Established criteria by some state DOTs for repair of end zone cracking

Crack width, in.	Action
< 0.007	Surface sealing
0.007 to 0.025	Epoxy injection
> 0.025	Reject beam

In regards to rejecting a girder due to end zone cracking, most responses stated that they deal with the beams on a case-by-case basis, considering the width, length, and number of cracks and their proximity to one another. Most stated that rejection is rare or they have not known of a beam rejected because of end zone cracking. The survey showed that it is a common belief among design engineers, precast concrete 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. Also, many respondents believe these cracks will close up to some extent after the beam is installed in a bridge due to the weight of the deck slab and barriers. This is because the direction of the end zone cracks is opposite to the direction of shear cracks, which means the end zone cracks will be subject to diagonal compressive stresses that help to close them up.

TASK (B): STRUCTURAL INVESTIGATION & FULL-SCALE GIRDER

The objective of the full-scale girder testing was to investigate whether end zone cracking negatively affects the flexural and shear capacities of prestressed concrete girders. The test plan had eight full-scale girders, 42 ft (13 m) long, fabricated in four States with different end zone reinforcement details. Four precast concrete producers from four states (Tennessee [TN], Florida [FL], Virginia [VA], and Washington [WA]) were selected. Each precast concrete producer agreed to fabricate two specimens as part of an actual bridge girder project. Both ends of each girder were tested. Table 4 summarizes the details of the eight specimens, which include the girder type, type of end zone reinforcement details, end-zone crack size, material properties, and number of prestress strands. Specimens are listed in the order they were fabricated and tested. Details of the girders and the testing plan can be found in Reference 1.

Table 4. Design criteria of the full-scale specimens

	Girder 1		Girder 2	
	Left end	Right end	Left end	Right end
Tennessee: Type III AASHTO beams All ends were designed to fail in flexure Girder concrete: $f'_{ci} = 6000$ psi, $f'_c = 7000$ psi 32- 0.5-in.-diameter, 270 ksi, low relaxation strands jacked to 33.8 kip CIP concrete slab: 7.5-in.-thick CIP concrete deck slab was added in the lab; $f'_c = 9000$ psi				
Repair type	(no repair)	(no repair)	(no repair)	(no repair)
End zone reinforcement (EZR) detail	AASHTO LRFD[7]	*Proposed EZR[8]	TNDOT EZR	*Proposed EZR[8]
End-zone crack size	≤ 0.002 in.	≤ 0.002 in.	≤ 0.002 in.	≤ 0.002 in.
Washington State: 58-in.-deep wide flange super girder WF58G All ends were designed to fail in shear Girder concrete: $f'_{ci} = 6000$ psi, $f'_c = 8000$ psi 58- 0.6-in.-diameter, 270 ksi, low relaxation strands jacked to 43.9 kip CIP concrete slab: none				
Repair type	(no repair)	(no repair)	(no repair)	(epoxy injection)
End zone reinforcement (EZR) detail	*Proposed EZR[8]	AASHTO LRFD[7]	No EZR	No EZR
End-zone crack size	0.001 - 0.005 in.	0.001 - 0.005 in.	0.003 - 0.010 in.	0.003 - 0.010 in.
Virginia: 45-in.-deep new bulb-tee PCEF45 All ends were designed to fail in flexure Girder concrete: $f'_{ci} = 6000$ psi, $f'_c = 8500$ psi 52- 0.6-in.-diameter, 270 ksi, low relaxation strands jacked to 43.9 kip Slab cast monolithically with the top flange: 4-in.-thick, 47-in.-wide; $f'_c = 8500$ psi				
Repair type	(No repair)	(No repair)	(No repair)	(No repair)
End zone reinforcement (EZR) detail	No EZR	No EZR	AASHTO LRFD[7]	*Proposed EZR[8]
End-zone crack size	0.004 - 0.010 in.	0.002 - 0.006 in.	0.004 - 0.008 in.	0.004 - 0.008 in.

Table 4. Design criteria of the full-scale specimens (cont.)

Florida: 60-in.-deep inverted tee beams All ends were designed to fail in shear Girder concrete: f'_{ci} = 6000 psi, f'_c = 8500 psi 36 straight 0.6-in.-diameter, 270 ksi, low relaxation strands jacked to 43.9 kip CIP concrete slab: 10-in.-thick, 24-in.-wide, f'_c = 10,000 psi				
Repair type	(no repair)	(no repair)	(no repair)	(no repair)
End zone reinforcement (EZR) detail	FLDOT EZR	Modified FLDOT EZR	AASHTO LRFD[7]	*Proposed EZR[8]
End-zone crack size	0.004 - 0.006 in.	0.004 - 0.006 in.	0.004 - 0.006 in.	0.004 - 0.006 in.

* Proposed EZR = EZR details developed at the University of Nebraska[8]

In addition to the end zone reinforcement (EZR) details adopted by each state, EZR details according to the AASHTO LRFD Specifications[7] and the research conducted at the University of Nebraska[8] (Proposed EZR) were used in the tested girders. The proposed EZR details are determined using four percent of the prestressed force at transfer and 20 ksi (138 MPa) allowable steel stress, which are the same criteria stated by the AASHTO LRFD Specifications[7]. However, the proposed details require that at least fifty percent of the end zone reinforcement be placed in the end $h/8$ of the member. The balance of the end zone reinforcement is recommended to be placed between $h/8$ and $h/2$ from the member end. This distribution concentrates the reinforcement where the bursting stresses are highest. The bursting reinforcement must be embedded into the top and bottom flanges such that it can develop at least 30 ksi (207 MPa) at the junctions of the flanges with the web.

Conclusions drawn from testing of the full-scale girders are as follow:

- End zone cracking has no effect on the shear and flexural capacity of the tested girders. Fourteen ends (out of the sixteen ends tested) were able to develop shear/flexure capacity higher than design capacity. Only two ends did not develop the measured capacity due to some fabrication flaws.
- End zone reinforcement appears not to have any effect on the shear or flexural capacities of a girder. Three ends (out of four) that contained zero end zone reinforcement were able to develop failure capacity higher than the design values. Only one end failed prematurely due to inadequate bearing area. The authors believe that providing adequate confinement reinforcement in the bottom flange at the end of the girder and anchoring some of the bottom flange strands in the end diaphragm provide the girder with the tension tie required to develop the design shear and flexural capacities at the girder end.
- Epoxy injection repair of end zone cracking does not enhance girder capacity.

TASK (C): EPOXY INJECTION TESTING

This task was developed to investigate: (1) if epoxy injection repair of end zone cracking is able to restore the tensile capacity of the cracked concrete, (2) if epoxy injection is able of completely filling the crack through the width of the web, and (3) if variations of the end zone reinforcement details have significant effect on the number, width & pattern of end zone cracks. Two 12-ft long specimens fabricated by Concrete Industries, Lincoln, Nebraska, as part of an NU 1350 (53 in. deep) bridge girder production. Details of these specimens are given in Table 5.

Table 5. Criteria of the specimens used for the epoxy injection testing

<table>
<tr><td rowspan="2"></td><td colspan="2">Girder 1</td><td colspan="2">Girder 2</td></tr>
<tr><td>Left end</td><td>Right end</td><td>Left end</td><td>Right end</td></tr>
<tr><td colspan="5">NU 1350 (53 in.), 12-ft long
Girder concrete: f'_{ci} = 6000 psi, f'_{c} = 8500 psi
Top flange: 32- 0.6-in.-diameter, 270 ksi, low relaxation straight strands
Bottom Flange: 32- 0.6-in.-diameter, 270 ksi, low relaxation straight strands
Top & bottom strands were jacked to 43.94 kips
No confinement reinforcement was provided in the bottom flange
Vertical shear web reinforcement consisted of pairs of #4 at 4 inch for the full 12 ft length of each specimen.</td></tr>
<tr><td>Repair type</td><td>(epoxy injection)</td><td>(no repair)</td><td>(epoxy injection)</td><td>(no repair)</td></tr>
<tr><td>End zone reinforcement (EZR) detail</td><td>No EZR</td><td>No EZR</td><td>AASHTO LRFD[7]</td><td>* Proposed EZR[8]</td></tr>
<tr><td>End-zone crack size</td><td>0.03 in.</td><td>0.03 in.</td><td>0.002 – 0.006 in.</td><td>0.002 – 0.006 in.</td></tr>
</table>

* Proposed EZR = EZR details developed at the University of Nebraska[8]

The top and bottom flanges were cut away, leaving only the webs of each girder, as shown in Figure 1. 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 beam was cut, the full extent of the interior cracking became visible, as shown in Figure 2. Upon inspection, it was clear that the epoxy injection did not totally fill the cracks as anticipated. From the cut section, it could only be seen entering approximately 0.2 inches into the crack, as shown in Figure 3. Also, visual inspection revealed a lack of adhesion between the concrete and the epoxy.

The web sections were cut into 16-inch (406 mm) strips, as shown in Figure 4(a). One strip was extracted from each of the four ends of the girder. Each specimen was turned on its side and subjected to a bending test, as shown in Figure 4(b). The structural testing was done to find the cracking-moment and tensile capacity of the specimens. The supports were set 18 inches apart. A two point loading system was used, with the two points being 6 inches apart.

Figure 1. Bottom flange completely cut from the specimen (Girder 1)

Figure 2. End zone cracking extending the full length of the web (Girder 1)

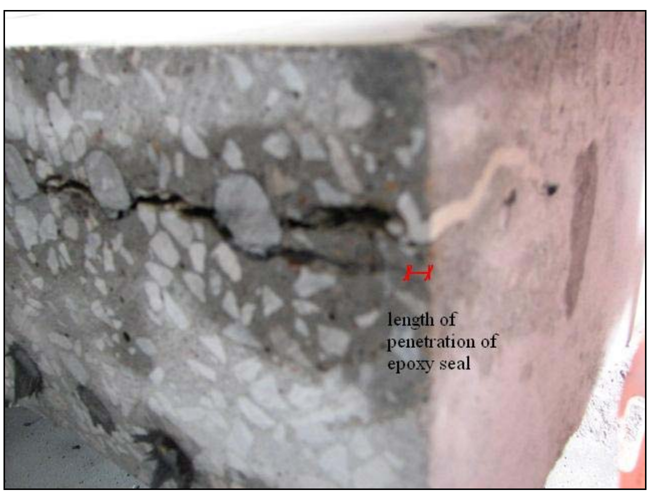
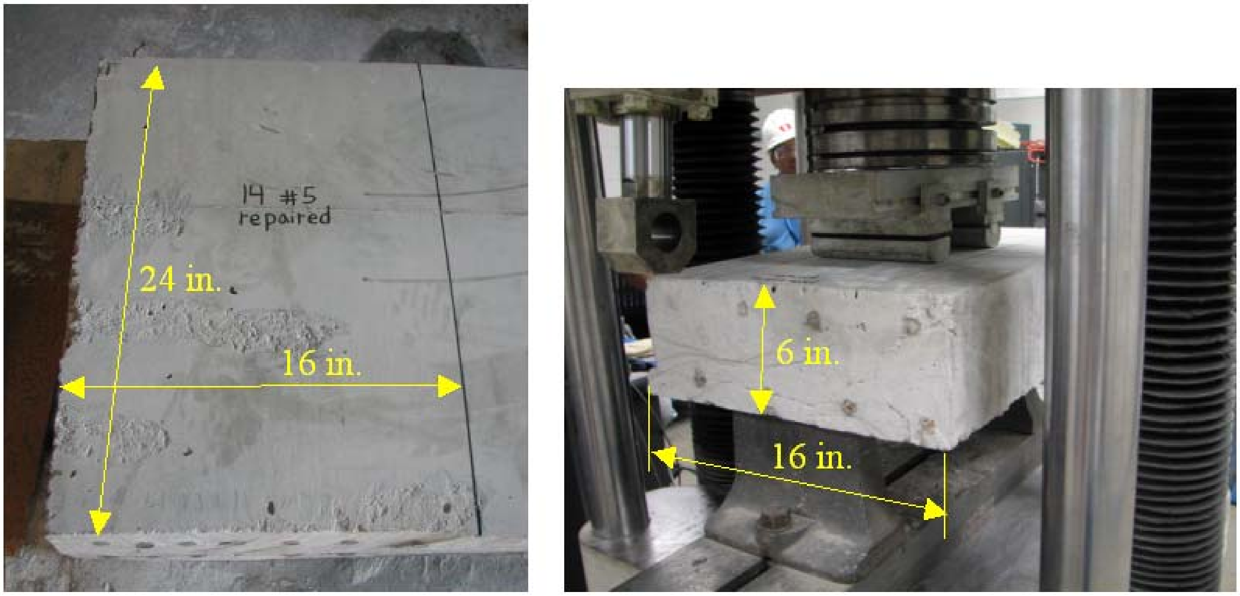


Figure 3. End zone cracking extends vertically and horizontally (Girder 1)



(a) Dimensions of the strip (b) A web strip in the testing frame

Figure 4. The 16-inch (406 mm) wide strips and test setup

Calculations were performed to estimate the cracking and failure moments of the five specimens. Table 6 gives the calculated cracking and failure load, and the test results of the five specimens.

Table 6. Test Results of the Five Specimens

Girder Specimen	Girder 1 (without special end reinforcement)			Girder 2 (with special end reinforcement)	
Location of the specimen	Left end	Right end	Midspan	Left end	Right end
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

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 as micro cracks are impossible to detect visually. These observations led the team to conclude (a) all specimens became cracked transverse to the prestressing direction at the time of prestress release, and (b) epoxy injection for these specimens was ineffective in restoring them to a pre-cracked condition

The epoxy injection testing demonstrated that (1) cutting coupons from the web end of a pretensioned I-beam was not an effective method of testing for structural tensile capacity, (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 experience 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) bottom flange confinement reinforcement and 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.

TASK (D): DURABILITY TESTING

The durability testing consisted of two stages. The objective of the first stage was to investigate which sealant material should be used if repair is required. The objective of the second stage was to investigate whether it is required that end zone cracks be filled with a patching material before a surface sealant is applied.

Stage I

The test procedure was a slightly modified version of the ASTM D6489 Standard: *Test Method for Determining the Water Absorption of Hardened Concrete Treated with a Water Repellant Coating*.[9] Five sealants were selected:
Product (A) Low viscosity methacrylate resin, Product (B) Water based epoxy modified portland cement bonding agent with anti corrosion coating, Product (C) Cement based filler, Product (D) High molecular weight methacrylate resin, and Product (E) Concrete Waterproofing by Crystallization.

These sealants were chosen based on the responses of the national survey and the recommendations received from the precast concrete producers who fabricated the full-scale girder specimens. Sixty 4 in. × 8 in. (102 mm × 203 mm) concrete cylinders were produced, ten cylinders for every sealant and ten cylinders for a control group, which received no sealant coating. After the cylinders were cured for 28 days, they were washed and cleared of debris and then heated in a draft oven for 24 hours. They were then coated with the selected sealants. All of the specimens were then immersed completely in water and left to soak. At 24 hours, and again at 96 hours, 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. The test results showed that the best performing sealants were those that were able to bond with the concrete and did not run off the cylinder. This shows that that a good sealant has to maintain some degree of viscosity.

Stage II

In the second stage of the durability test, the authors observed how assorted sealers performed in preventing water penetrating into concrete specimens exhibiting various sizes of cracks. The procedure of the test was modified from the 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*[10] and D6489-99 *Standard Test Method for Determining the Water Absorption of Hardened Concrete Treated with a Water Repellent Coating*.[9]

The concrete specimens were made in the form of 3 in. × 3 in. × 12 in. (76 mm × 76 mm × 300 mm) rectangular prisms. The design concrete strength was 5000 psi (34,000 kPa). Although this concrete mixture is relatively more porous than the concrete normally used in precast concrete girders, it was used to amplify the amount of water absorbed if the sealers failed. Artificial cracks were formed with metal and plastic shims, penetrating down 2.25 in. (57.2 mm) from the top surface of the specimens and measuring 9 in. (230 mm) in length (Figure 5). 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 in. to 0.054 in. (0.2 mm to 1.4 mm). After all specimens were fabricated, they were placed in a draft oven for 24 hours and then the dry weight W_A was recorded. When cooled, the selected sealants were used to cover the four sides and bottom face of each specimen, leaving only the top surface containing the crack not coated. These sides were covered to prevent moisture from either entering or escaping the surfaces not being tested.

Figure 5. Durability Test Specimens with Metal Shims

There were two sets of specimens for each sealant, with each set containing prisms with cracks of each available size. The top surface of the specimens, which had the crack, of the first set was sealed only with the specified sealant. The top surface of the specimens of the second set had a hydraulic cementitious material rubbed into the cracks by hand, and then sealed with the same sealant as the first set. The hydraulic cementitious material was rubbed into the cracks by hand (Figure 6), while the sealants were applied with a roller. The specimens were placed on their sides and the selected sealants were applied to their specific sets. This orientation mimics the orientation of the cracks on the webs of production girders.

Figure 6. Hand application of the hydraulic cementitious material

Once the specimens dried, they were turned upright and a 3-in.-tall (76 mm) rectangular plastic dike was built on the top 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 dikes in place (Figure 7).

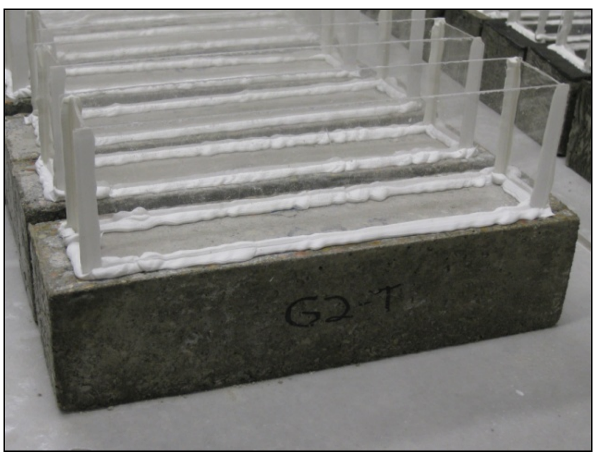


Figure 7. Specimens with water dikes

The specimens were then weighed and the data recorded as W_1. The specimens were all placed face up in an area where they would not be disturbed. Each dike was then filled to the top with water. The specimens were given the opportunity to absorb water for 24 hours. Every effort was made to ensure that the dike remained filled with water at all times. At 24 hour, the water in each dike 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 *A%* by each sample was calculated using Eq. (1):

$$A\% = \frac{100\left(W_2 - W_1\right)}{W_A} \qquad (1)$$

Stage II of the durability test was conducted on 112 specimens using five sealants, five crack sizes 0.007 in., 0.012 in., 0.016 in., 0.033 in., and 0.054 in. (0.2 mm, 0.30 mm, 0.41 mm, 0.84 mm, and 1.4 mm), and a control group that did not receive any sealant coating. Readings were taken at both 24 hours and 48 hours, and the percent absorption *A%* for each specimen was determined.

The test results showed that the best performing sealant was the most viscous sealant tested, that was Product (B). It performed well both with and without the hydraulic cementitious packing material, showing almost no measurable absorption of water in either case. For specimens with cracks as wide as 0.016 in. (0.41 mm), this sealant was able to fill the crack without leaving voids for the water to seep into. Based on the results of the durability test, the authors were able to propose that when using low-viscosity sealants, packing cracks with a thick cementitious material allows the cracks to be closed when the sealant alone is not adequate. 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.

TASK (E): FIELD INSPECTION OF BRIDGES

The objectives of the field inspection were to determine whether end zone cracking widens with time and whether unrepaired end zone cracks lead to corrosion of the strands and bars. Two bridges from Nebraska and three bridges from Virginia were selected for field inspection. The inspection process included collection of reports of inspection conducted at the fabrication plant to examine the repair method and material, collection of inspection reports of the bridges in service, and visits to the bridges under study to report on visible signs of crack growth since production and signs of reinforcement corrosion and concrete delamination.

The authors found it difficult to collect the inspection reports conducted at the fabrication plants of most of the selected bridges. The fabrication plants do not retain these reports for an extended period of time and assume that it is the responsibility of the bridge owner to maintain such records. On the other hand, the bridge owners do not keep these records if no serious problems were detected in the girders before shipping them to the bridge site. At the precast concrete plant, the inspectors are usually looking for any imperfections or visible damage to the girders, such as sweep, excessive deflection, chipping of concrete, etc. End zone cracking, if present, is not typically recorded in these reports as long as the cracks do not exceed acceptable limits mandated by the owner and the inspector feels that they are not severe enough to be repaired.

Since the selected bridges were opened to traffic, they were inspected every two years. The field inspectors typically look for visible signs of damage and distress that might affect the bridge durability and service conditions, such as deck cracking and damage to bearing devices. The field inspection reports are typically well maintained by the bridge owner. The authors found that end zone cracking was recorded in the field inspection reports in four out of the five selected bridges, but in an ambiguous way. For example, instead of giving the crack width, the reports stated: "hairline cracks exists." Length and pattern of the cracks were not recorded for future follow up and comparison. As a result, in some cases, it was hard to know from reading these reports whether the recorded cracks were end zone cracks or vertical shear cracks.

The authors found it difficult to trace a specific girder from the precast concrete plant to the construction site because the girder producer and the bridge owner used different identification systems. Therefore, if a girder was repaired in the fabrication plant it would not be easy to locate the girder in the bridge to see if the original damage was causing any problems.

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

- Crack width was in the range from 0.006 in. to 0.020 in. (0.15 mm to 0.50 mm). Comparing the crack widths at the time of our inspection with those documented in the inspection reports revealed no growth.
- Four out of the five bridges were built over water channels, where the ambient air is humid for an extended period in the summer. Field inspection of these bridges did not reveal any visible signs of reinforcement corrosion or concrete delamination, although end zone cracking had existed at the time of prestress release.
- Although girders in some of the selected bridges were repaired at the precast concrete plant, there was no documentation relative to methods and materials used to repair end zone cracking.
- Neither the Nebraska Department of Roads (NDOR) nor Virginia Department of Transportation (VDOT) had a policy to include in the field inspection reports whether end zone bursting cracks had been reported in the plant inspection reports. Also, there was no consistency in girder identification between the producer's and the owner's identification systems.

PROPOSED CRITERIA OF ACCEPTANCE, REPAIR OR REJECTION

Based on the tasks executed in NCHRP 18-14 project, the authors developed a manual for decision criteria for acceptance and repair of web end cracking during production. The proposed manual uses the crack width as the major criterion for acceptance, repair or rejection, as follows:

- Cracks narrower than 0.012 in. (0.30 mm) may be left unrepaired.
- Cracks ranging in width from 0.012 in. to 0.025 in. (0.30 mm to 0.64 mm) should be repaired by filling the cracks with approved specialty cementitious materials and the end four feet of the girder side faces coated with an approved sealant.
- Cracks ranging in width from 0.025 in. to 0.050 in. (0.64 mm to 1.30 mm) should be filled by epoxy injection, and then the surface coated with a sealant.

- For girders exhibiting cracks wider than 0.05 in. (1.3 mm), the research team recommends that the girder be rejected. For such girders, it is believed 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.

CONCLUSIONS

The nature and consequences of end zone cracking are quite different from those of flexural cracking. For example, flexural cracks in beams tend to grow in width and depth with the application of superimposed loads. They may adversely affect 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 work conducted in the NCHRP 18-14 project, the authors have 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. 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.

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