# EFFECTS OF STATIC AND REPEATED LOADINGS ON CONCRETE BRIDGE DECKS AND SLABS REINFORCED WITH EPOXY-COATED BARS

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This paper presents findings from an ongoing laboratory and field investigation on the effects of static and repeated loadings on concrete bridge decks and slabs reinforced with epoxy-coated bars in the State of Indiana. Comparisons are presented of the load-deflection behavior, flexural crack pattern and width, and bond strength of companion concrete specimens reinforced with coated and uncoated steel. Current laboratory findings indicate the average concrete crack width is larger in specimens with epoxy-coated reinforcement than in companion specimens with uncoated reinforcement. The findings from one of the five bridge decks in the field investigation are also included.

Keywords: reinforcement bond to concrète, concrete cracking, epoxy coatings, deformed reinforcement, concrete durability, fatigue, lap connections, repeated loading and static loading.

## INTRODUCTION

One cause of concrete bridge deck deterioration is corrosion of the reinforcing steel. Corrosion is often attributed to concentrations of chloride ions from deicing salts in the concrete. These concentrations of ions serve as the electrolyte in the corrosion process. During the corrosion process the volume occupied by the reinforcing steel increases causing pressure on the surrounding concrete leading to eventual spalling and, in extreme cases, loss of structural integrity. Since the early 1970's epoxy-coated reinforcement has been used to minimize rebar corrosion (*1*). An epoxy powder is electrostatically applied to heated reinforcement forming a protective layer that restricts ion contact with the reinforcing steel.

This paper reports on an ongoing HPR-Part II research study, "Behavior of Concrete Bridge Decks and Slabs Reinforced with Epoxy Coated Bars," sponsored by the Indiana Department of Transportation (INDOT) and the Federal Highway Administration. A laboratory and a field phase are being conducted in this research study. The laboratory phase consists of a test program designed to compare the behavior of slab type concrete members containing coated reinforcement with that of otherwise identical companion concrete specimens reinforced with uncoated steel. Comparisons are made of the load-deflection behavior, flexural crack widths and patterns, and the bond strength under static and repeated loading. Test variables include size of the reinforcement bar, ratio of concrete cover to bar diameter, reinforcement splice length, thickness of epoxy coating, number of applied load repetitions, stress range and peak stress. The field phase includes assessment of concrete strength, chloride content, delamination survey, crack patterns, concrete cover, and condition of the reinforcement for five bridge decks reinforced with epoxy-coated steel in the State of Indiana.

## LABORATORY PHASE

The dimensions and loading arrangement for the concrete test specimens with the No. 7 (22 mm, 7/8 in) bars and with No. 11 (35 mm, 1-1/4 in) bars are shown in Figure 1. No. 3 bars (10 mm, 3/8 in) spaced at 152 mm (6 in) on centers were used as transverse reinforcement in all specimens. The physical and mechanical properties of the reinforcement are given in Table I. The concrete test specimens were designed to fail at the lap splices before yielding of the steel, see Figure 1 The specimens were loaded such that the lap splices were placed in a constant moment region. The primary variables of the experimental test program are summarized in Table II. Test sets include identical companion beams, one reinforced with epoxy-coated steel and the other with uncoated reinforcement.

The results of the first 24 tests are reported in this paper. Six sets of companion specimens containing No. 7 bars and six sets containing No. 11 bars were tested under repeated loading. The test specimens were initially cracked by application of 2 to 3 monotonic load cycles up to the peak stress used in the repeated load

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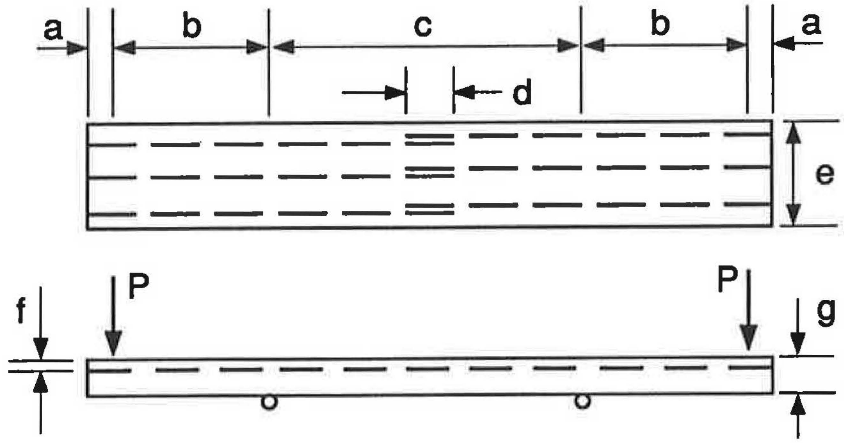


| bar size (mm) | a (mm) | b (mm) | c (mm) | d (mm) | e (mm) | f* (mm) | g (mm) | stirrups |
|---|---|---|---|---|---|---|---|---|
| 22 | 152 | 1219 | 1219 | 305 | 610 | 64 | 203 | 10 mm @152 mm |
| 35 | 152 | 1219 | 1219 | 711 | 711 | 64 | 305 | 10 mm @152 mm |

*f=clear cover above the reinforcement

**Figure 1** Specimen dimensions.

**Table I** PHYSICAL AND MECHANICAL PROPERTIES OF REINFORCEMENT

| Reinforcement Property | No. 7 bars (22 mm) | | No. 11 bars (35 mm) | |
|---|---|---|---|---|
| | Uncoated | Epoxy-Coated | Uncoated | Epoxy-Coated |
| Average Gap (mm) | 6.94 | 6.53 | 7.39 | 7.19 |
| Average Spacing (mm) | 13.85 | 13.85 | 21.77 | 21.77 |
| Average Height (mm) | 1.15 | 1.13 | 1.96 | 1.79 |
| Variation in Weight (%) | -3.9 | -4.1 | -4.4 | -4.4 |
| Yield Stress (MPa) | 454 | 471 | 515 | 477 |
| Tensile Stress (MPa) | 759 | 707 | 747 | 747 |
| % Elongation in 200 mm | 14 | 13 | 11 | 14 |
| Rib Bearing Area ($mm^2$/mm) | 3.54 | 3.71 | 7.50 | 6.05 |
| Related Rib Area | 0.051 | 0.053 | 0.068 | 0.055 |
| Rib Bearing Area Ratio (1/mm) | 0.0092 | 0.0096 | 0.0075 | 0.0060 |

**Table II** TEST VARIABLES

| Specimen Designation | Bar Size | Cover (mm) | Splice Length (mm) | Concrete Strength (MPa) | Peak Stress (MPa) | Stress Range Below Peak (MPa) | # Cycles |
|---|---|---|---|---|---|---|---|
| U7241 | No. 7 | 64 | 305 | 20.7 | 166 | 60 | 1,000,000 |
| E7241 | No. 7 | 64 | 305 | 20.7 | 166 | 60 | 1,000,000 |
| U7242 | No. 7 | 64 | 305 | 32.4 | 166 | 60 | 1,000,000 |
| E7242 | No. 7 | 64 | 305 | 32.4 | 166 | 60 | 1,000,000 |
| U7361 | No. 7 | 64 | 305 | 35.9 | 248 | 60 | 1,000,000 |
| E7361 | No. 7 | 64 | 305 | 35.9 | 248 | 60 | 1,000,000 |
| U7362 | No. 7 | 64 | 305 | 36.6 | 248 | 60 | 1,000,000 |
| *E7362 | No. 7 | 64 | 305 | 36.6 | 248 | 60 | 600,000 |
| U7363 | No. 7 | 64 | 305 | 27.6 | 248 | 103 | 1,000,000 |
| E7363 | No. 7 | 64 | 305 | 27.6 | 248 | 103 | 1,000,000 |
| U7301 | No. 7 | 64 | 305 | 20.7 | 207 | 60 | 1,000,000 |
| E7301 | No. 7 | 64 | 305 | 20.7 | 207 | 60 | 1,000,000 |
| U11241 | No. 11 | 64 | 711 | 20.7 | 166 | 60 | 1,000,000 |
| E11241 | No. 11 | 64 | 711 | 20.7 | 166 | 60 | 1,000,000 |
| U11242 | No. 11 | 64 | 711 | 32.4 | 166 | 60 | 1,000,000 |
| E11242 | No. 11 | 64 | 711 | 32.4 | 166 | 60 | 1,000,000 |
| U11243 | No. 11 | 64 | 711 | 20.7 | 166 | 103 | 1,000,000 |
| E11243 | No. 11 | 64 | 711 | 20.7 | 166 | 103 | 1,000,000 |
| U11301 | No. 11 | 64 | 711 | 35.9 | 207 | 60 | 1,000,000 |
| E11301 | No. 11 | 64 | 711 | 35.9 | 207 | 60 | 1,000,000 |
| U11302 | No. 11 | 64 | 711 | 36.6 | 207 | 60 | 1,000,000 |
| E11302 | No. 11 | 64 | 711 | 36.6 | 207 | 60 | 1,000,000 |
| U11303 | No. 11 | 64 | 711 | 27.6 | 207 | 103 | 1,000,000 |
| *E11303 | No. 11 | 64 | 711 | 27.6 | 207 | 103 | 336,000 |

* Specimen failed in fatigue.

tests (described later). After the initial loading, the beams were subjected to cycles of load between the maximum and minimum stress levels with a hydraulic pulsator at a rate of 260 cycles per minute in blocks of approximately 100,000 cycles up to a total of 1,000,000 cycles. If failure did not occur during the repeated load phase of the test, the specimen was unloaded and tested monotonically to failure.

The number of cracks in the concrete in the constant moment region and the total width of these cracks are given in Table III. The data presented are from the second and one millionth load cycles. For beams E7362 and E11303 the data are the last measured values before failure in fatigue after 600,000 and 336,000 cycles respectively. The data were recorded at the peak repeated load. This load was selected for comparison because all of the flexural cracks had formed at this level. The values presented in Table III show that the beams with uncoated reinforcement had more flexural cracks than the beams with epoxy-coated reinforcement. The cracks were more widely spaced with epoxy-coated reinforcement which implies a longer transfer length

**Table III** CRACK WIDTHS IN THE CONSTANT MOMENT REGION

| Specimen Designation | Number of Cracks | Total Crack Width (mm) | | E/U Ratio of Total Crack Width | | E/U Ratio of Average Crack Width | |
|---|---|---|---|---|---|---|---|
| | | 2nd Cycle | $10^6$ Cycle | 2nd Cycle | $10^6$ Cycle | 2nd Cycle | $10^6$ Cycle |
| U7241 | 8 | 1.75 | 2.24 | 1.20 | 1.05 | 1.37 | 1.21 |
| E7241 | 7 | 2.11 | 2.36 | | | | |
| U7242 | 6 | 2.08 | 2.41 | 0.94 | 0.95 | 1.41 | 1.42 |
| E7242 | 4 | 1.96 | 2.29 | | | | |
| U7361 | 6 | 3.40 | 4.09 | 0.96 | 0.99 | 0.96 | 1.00 |
| E7361 | 6 | 3.28 | 4.06 | | | | |
| U7362 | 6 | 3.68 | 4.01 | 0.71 | 0.87 | 1.07 | 1.30 |
| *E7362 | 4 | 2.62 | 3.48 | | | | |
| U7363 | 6 | 2.90 | 3.45 | 1.57 | 1.49 | 1.57 | 1.49 |
| E7363 | 6 | 4.55 | 5.16 | | | | |
| U7301 | 6 | 3.40 | 4.06 | 1.13 | 1.12 | 1.13 | 1.12 |
| E7301 | 6 | 3.86 | 4.55 | | | | |
| U11241 | 7 | 1.35 | 1.80 | 1.08 | 0.93 | 1.25 | 1.10 |
| E11241 | 6 | 1.45 | 1.68 | | | | |
| U11242 | 5 | 1.57 | 2.06 | 1.16 | 1.04 | 0.97 | 1.03 |
| E11242 | 6 | 1.83 | 2.13 | | | | |
| U11243 | 7 | 1.47 | 1.98 | 1.24 | 1.17 | 1.45 | 1.37 |
| E11243 | 6 | 1.83 | 2.31 | | | | |
| U11301 | 7 | 1.75 | 2.01 | 1.28 | 1.32 | 1.12 | 1.15 |
| E11301 | 8 | 2.24 | 2.64 | | | | |
| U11302 | 6 | 1.40 | 1.88 | 1.22 | 1.12 | 1.46 | 1.35 |
| E11302 | 5 | 1.70 | 2.11 | | | | |
| U11303 | 7 | 2.29 | 2.57 | 1.11 | 1.21 | 1.11 | 1.21 |
| *E11303 | 7 | 2.54 | 3.10 | | | | |

* Specimen failed in fatigue.

existed with epoxy-coated bars. The ratio of the total crack width of the epoxy-coated to uncoated (E/U) was 1.09 for the second cycle and 1.08 for the one millionth cycle in the specimens reinforced with No. 7 bars. The ratio was 1.18 for the second cycle and 1.13 for the one millionth cycle in the specimens with No. 11 bars. The E/U ratio of average crack width for the second cycle was 1.2 for No. 7 bars and 1.25 for No. 11 bars. After one million cycles the change in the ratio of average crack width was negligible. Although there were fewer cracks in the specimens reinforced with epoxy-coated bars, the width of the individual cracks was larger than with uncoated steel.

The total deflection for each specimen was recorded at the same loads used in the crack width comparisons. The total deflection is defined as the sum of the upward movement at the centerline and the downward movement at the ends of the specimen. At the end of the second cycle, the specimens reinforced with epoxy-coated steel averaged total deflections 5% greater than the beams with uncoated reinforcement. After one million cycles, the E/U ratio for deflections remained at 1.05 for the specimens containing No. 7 bars and decreased to 0.98 in the specimens with No. 11 bars.

Shown in Table IV are the failure load, failure stress and deflections for the 24 specimens reported in this paper. The failure stress was calculated assuming a linear stress distribution in the concrete compression zone and neglecting the tensile strength of the concrete. The bond ratio given in Table IV, is the ratio of the average stress to failure for the specimen reinforced with epoxy-coated steel to its companion specimen reinforced with uncoated steel. All the specimens reported failed before yielding of the tensile reinforcement. Specimen E7362 failed during the repeated loading phase after 600,000 and E11303 after 336,000 cycles. For these specimens, the reported failure stress is the peak stress in the repeated load cycle phase. The average bond ratio for the specimens with No. 7 bars was 0.80, and 0.76 for No. 11 bars.

Two factors that significantly effected the splitting phenomena associated with bond strength reduction in epoxy-coated reinforcement were the concrete cover to bar diameter ratio and the rib bearing

**Table IV** FAILURE LOADS, STRESSES AND DEFLECTIONS

| Specimen Designation | Failure Stress (MPa) | Bond Ratio | Load (kN) | | | Deflection (mm) | | |
|---|---|---|---|---|---|---|---|---|
| | | | Split | Failure | Difference | Split | Failure | Difference |
| U7241 | 269 | .95 | 24.5 | 26.6 | 2.1 | 31.4 | 35.7 | 4.3 |
| E7241 | 256 | | 24.5 | 25.3 | 0.8 | 34.4 | 34.4 | 0.0 |
| U7242 | 310 | .82 | 31.1 | 31.4 | 0.3 | 35.8 | 35.8 | 0.0 |
| E7242 | 256 | | 25.6 | 25.6 | 0.0 | 30.4 | 30.4 | 0.0 |
| U7361 | 397 | .73 | 40.9 | 40.9 | 0.0 | 46.2 | 46.2 | 0.0 |
| E7361 | 290 | | 29.4 | 29.4 | 0.0 | 36.4 | 36.4 | 0.0 |
| U7362 | 381 | .65 | 38.3 | 39.1 | 0.8 | 42.8 | 42.8 | 0.0 |
| *E7362 | 248 | | 24.5 | 24.5 | 0.0 | 32.2 | 32.2 | 0.0 |
| U7363 | 392 | .84 | 37.8 | 40.0 | 2.2 | 43.6 | 47.0 | 3.4 |
| E7363 | 330 | | 30.0 | 33.4 | 3.4 | 35.3 | 38.3 | 3.0 |
| U7301 | 301 | .81 | 24.5 | 30.0 | 5.5 | 31.9 | 37.2 | 5.3 |
| E7301 | 244 | | 24.0 | 24.0 | 0.0 | 34.5 | 34.5 | 0.0 |
| U11241 | 290 | .85 | 102.3 | 133.4 | 31.1 | 16.5 | 22.9 | 6.4 |
| E11241 | 248 | | 89.0 | 113.7 | 24.7 | 14.9 | 20.0 | 5.1 |
| U11242 | 339 | .79 | 124.5 | 163.0 | 38.5 | 16.2 | 21.2 | 5.0 |
| E11242 | 267 | | 122.3 | 124.8 | 2.5 | 17.9 | 20.2 | 2.3 |
| U11243 | 311 | .82 | 89.0 | 143.4 | 54.4 | 16.9 | 27.4 | 10.5 |
| E11243 | 254 | | 103.1 | 116.2 | 13.1 | 17.5 | 19.7 | 2.2 |
| U11301 | 379 | .75 | 106.8 | 177.9 | 71.1 | 16.3 | 28.4 | 12.1 |
| E11301 | 284 | | 97.9 | 132.9 | 35.0 | 15.1 | 21.7 | 6.6 |
| U11302 | 360 | .76 | 112.2 | 169.1 | 57.9 | 16.4 | 27.8 | 11.4 |
| E11303 | 274 | | 97.9 | 127.8 | 29.9 | 16.6 | 21.5 | 4.9 |
| U11303 | 353 | .59 | 121.4 | 164.6 | 43.2 | 19.4 | 27.8 | 8.4 |
| *E11303 | 207 | | 94.5 | 94.5 | 0.0 | 13.4 | 15.6 | 2.2 |

* Specimen failed in fatigue.

area ratio. The ratio of concrete cover to bar diameter was 2.86 for the No. 7 bars and 1.77 for No. 11 bars. The rib bearing area ratio is the ratio of the rib bearing area per unit of bar length minus the area of the longitudinal rib to the nominal cross sectional area of the bar. As the bar size increased in the test specimens, the concrete cover to bar diameter ratio and the rib bearing area ratio decreased.

The peak repeated stress influenced the bond ratio for both No. 7 and No. 11 bar specimens. For the No. 7 bar specimens subjected to a stress range of 60 MPa (8.7 ksi) below the peak stress, the average bond ratio was 0.89 for a peak stress of 166 MPa (24 ksi), 0.81 with a peak stress of 207 MPa (30 ksi), and 0.69 for a peak stress of 248 MPa (36 ksi). The specimens with No. 7 bar subjected to a stress range of 103 MPa (15 ksi) below the peak stress of 248 MPa (36 ksi) had a bond ratio of 0.84. For the No. 11 bar specimens subjected to a stress range of 60 MPa (8.7 ksi) below the peak stress, the average bond ratio was 0.82 with a peak stress of 166 MPa (24 ksi), and 0.76 with a peak stress of 207 MPa (30 ksi). The specimens reinforced with No. 11 bars and subjected to a stress range of 103 MPa (15 ksi) below the peak stress, had bond ratio of 0.82 for a peak stress of 166 MPa (24 ksi) and 0.59 for a peak stress of 207 MPa (30 ksi).

Shown in Table IV are the load and corresponding deflection at first sign of splitting and at failure. The average additional load carrying capacity after splitting in the specimens reinforced with No. 7 bars was negligible for both coated and uncoated reinforcement. For specimens reinforced with No. 11 bars, the average additional load beyond splitting was 49.4 kN (11.1 kips) with uncoated steel and 17.6 kN (4.0 kips) with coated steel. In regards to additional deflection beyond splitting the specimens with No. 7 bars showed little increase in the deflection for either type of reinforcement. In the specimens reinforced with No. 11 bars the average post-splitting deflection was 9 mm (0.35 in) in the specimens with uncoated steel and 3.9 mm (0.15 in) with coated steel.

The work in the laboratory phase is continuing with future tests of specimens at a maximum peak stress of 248 MPa (36.0 ksi), monotonic single cycle baseline tests, coating thickness tests, and bar deformation patterns tests.

## FIELD PHASE

This section describes the structures being evaluated in the field phase of the research study and the results of the field evaluation of one structure. The remaining structures are scheduled for evaluation in the future. The field phase is aimed at the condition assessment of concrete bridge decks and slabs reinforced with epoxy-coated steel in Indiana. The field evaluation includes structures throughout the state reflecting a cross section of environmental conditions, traffic and intensity of salt application. It also addresses the performance of decks supported on flexible systems (steel girders) as well as more rigid support conditions (precast prestressed girders) and concrete slabs. A total of five sites have been selected for evaluation. The site selection has been fully coordinated with personnel from the INDOT.

The first structure selected for evaluation is located in Indianapolis and consists of a six-span continuous composite steel box girder bridge with a concrete slab. This bridge deck was built in 1985 and has a maximum span length of 62.8 m (206 ft). This bridge represents the case of a deck on a flexible superstructure in the southern part of the state subjected to heavy urban traffic and severe salt exposure. The bridge cross section is shown in Figure 2 and the plan view in Figure 3. The second structure is located in South Bend. The structure is a four span continuous bridge deck supported on precast prestressed AASHTO sections and represents the case of a concrete deck built on a more rigid support system. The structure was built in 1983 and the maximum span length is 27.4 m (90 ft). This bridge is located in the northern part of the state in an urban area with significant traffic and severe salt exposure. The third structure selected is located south of South Bend in the northern part of the state. The structure consists of a three-span continuous welded steel beam with a composite concrete deck. The structure was built in 1980 and has a maximum span length of 18.9 m (62 ft). This structure is subjected to heavy truck traffic and heavy salt application. The fourth structure is located in southern part of the state and consists of three span skewed continuous reinforced concrete slab bridge. The bridge was built in 1985 and has a maximum span length of 14.0 m (46 ft). This structure is subjected to moderate traffic and moderate salt application. The fifth structure is located in the northern part of the state in Gary. The structure is a three span continuous bridge deck supported on a continuous steel beam. This bridge was built in 1980 with a maximum span length of 19.8 m (65 ft). The concrete deck was built using stay-in-place metal forms. The bridge is subjected to heavy industrial traffic with heavy de-icing salt exposure.

The deck evaluation at each of the five sites will include a deck survey for delamination as well as a

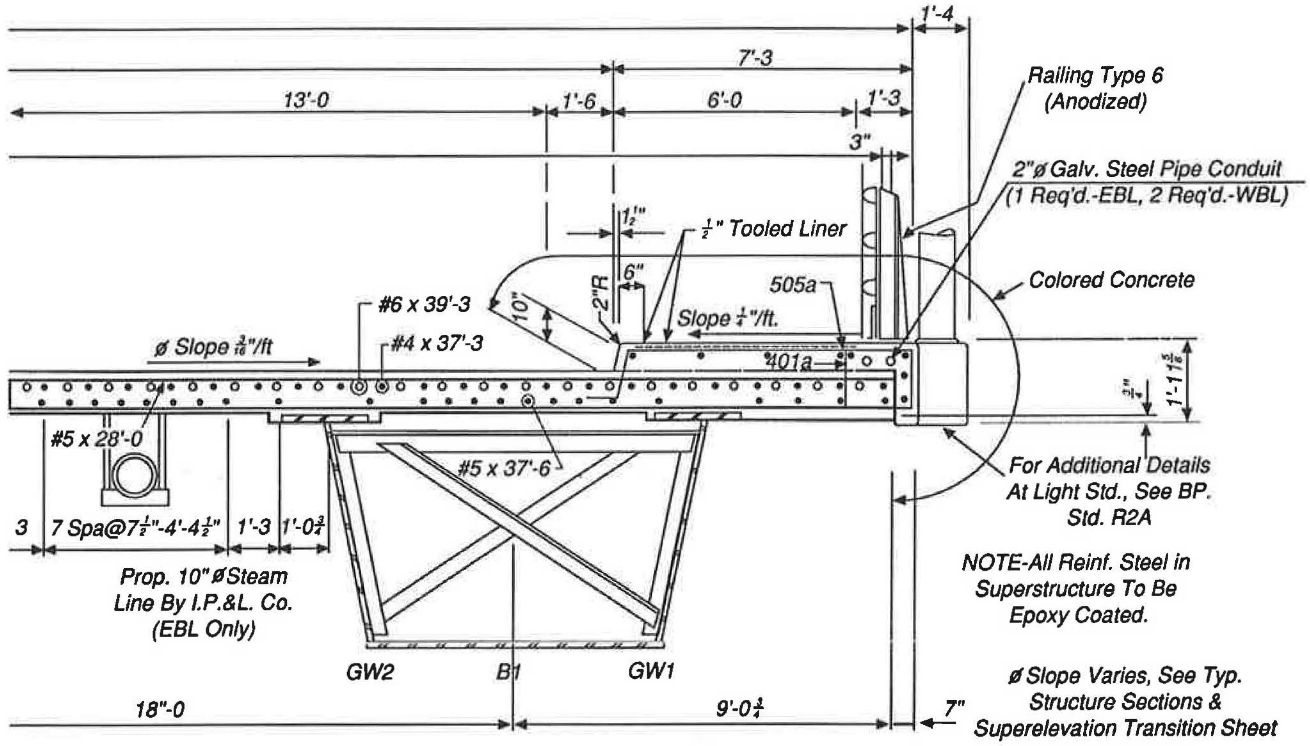


**Figure 2** Bridge cross section.

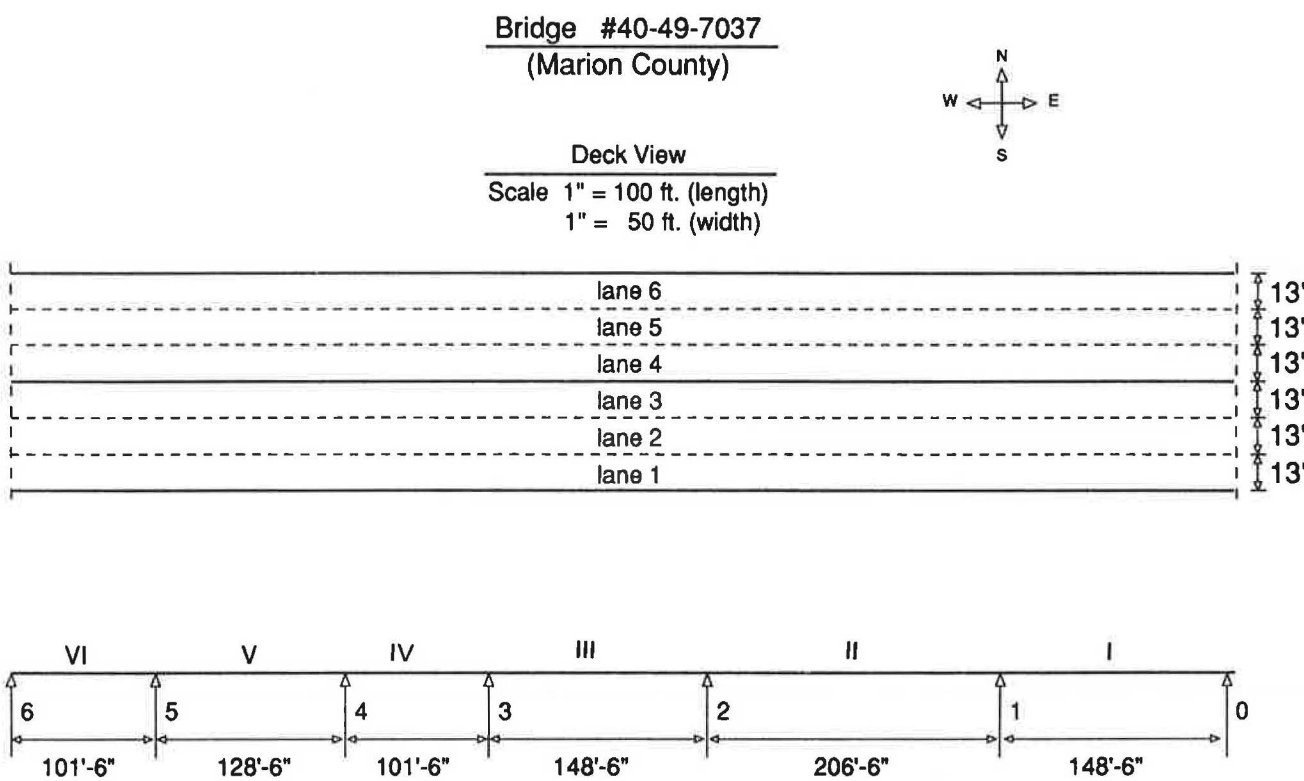


**Figure 3** Continuous composite steel box beam bridge.

detailed mapping of the observed cracking. Core samples and chloride samples will be taken. The concrete cover will be evaluated using the R-meter (focused electromagnetic field) as well as coring. The reinforcement condition evaluation will include coating condition, thickness of coating and deformation pattern. In addition, the evaluation will include factors such as: (a) environment, (b) traffic, (c) degree of salt application, (d) storage methods, (e) local practices, sources and specifications, (f) coating process, and (g) type of epoxy material.

The Division of Materials and Tests of the INDOT conducts a series of tests on samples taken from every 1,360 kg (3,000 lb) of epoxy-coated steel used on bridge decks built in the state. These tests include yield and ultimate strength, elongation, 180 degree ASTM bend test, ASTM-Deformation, epoxy thickness AASHTO M-284, and 120 degree bend test AASHTO M-284. No checks are made for holidays, this is left to an on-site INDOT project engineer walk-through visual survey. Coating thicknesses typically exceed minimum requirements. In general, construction practices depend on the contractor's quality control emphasis and level experience working with epoxy-coated rebars, and the level of State inspections. For most jobs the bars are stored for short periods before placement in the structure.

The findings from the field investigation of the bridge structure located in Indianapolis are described below. The evaluation of the bridge deck was conducted on the outside lanes (1 & 6) as shown in Figure 3. Typical crack patterns are shown in Figure 4. The number of cracks in each span are shown in Figure 5 and the average crack width in Figure 6. The deck concrete compressive strength was determined using 127 mm (5 in) cores. The measured concrete cover, the results of compression tests, and the chloride content at depths of 25.4, 50.8, 76.2 and 101.6 mm (1, 2, 3 and 4 in) are shown in Table V.

The average flexural crack width was less than 0.410 mm (0.016 in). The deck concrete compressive strength adjusted for a test core height/diameter of 1.0 resulted in an average strength of 35 MPa (5.13 ksi). The measured average cover over the top layer of steel was 61 mm (2.4 in) with a maximum of 76 mm (3.0 in) and a minimum of 41 mm (1.6 in). The average chloride content was 1.29 kg/m$^3$ at 51 mm (2.18 lb/yd$^3$ at 2 in) below the surface of the deck and 0.88 kg/m$^3$ at 76 mm (1.48 lb/yd$^3$ at 3 in). No signs of concrete delamination were observed. The epoxy-coating on the rebar sections extracted from the deck showed no signs of damage.

## DESIGN IMPLICATIONS

The surface roughness of uncoated bars and the irregularities along the steel concrete interface caused by adhesion of particles of concrete to the steel, found with uncoated reinforcement provide important components to the bond mechanism between concrete and steel. The absence or the reduction of these features eliminates or reduces the friction contribution in the coated bars. Lack of friction leads to higher rib-bearing forces, larger slip, higher bar strains at flexural crack locations and lower bond strength. Some of these deficiencies could be overcome by using deformation patterns with larger rib-bearing areas and steeper rib angles (*2*).

The stress range as well as the other variables in the laboratory phase of this study were selected to be typical of service conditions in a bridge deck. Repeated loading over the stress ranges, number of cycles and concrete strength evaluated in this study were more detrimental to the specimens with uncoated reinforcement. Although the total crack width in the constant moment region was approximately the same for both types of reinforcement, the average width of a single crack was larger for the specimens with epoxy-coated reinforcement. The wider cracks could lead to increased deterioration due to freeze-thaw action and could be of concern if epoxy coatings do not provide the protective barrier that has been assumed. The inspection of epoxy-coated bars after failure in the laboratory specimens found no visible damage to the coating due to the repeated loading.

Due to the larger crack opening, reinforcement stresses at crack locations will be higher for epoxy-coated bars. Hence radial stresses will be higher as well. Thus, adequate confinement must be provided by sufficient concrete cover. Larger cover to bar diameter ratios are recommended in harsh environments and should not be reduced with the expectation that the epoxy coating will be the sole system of corrosion protection. The extra cover provides improved anchorage for the bars. Furthermore, durability depends on careful design, good construction practices and adequate material selection. Improvements in any of these areas will reduce the problem, but individually will not provide an effective solution. Providing adequate cover is an example of good design strategy. Adequate inspection, finishing and curing represent solid construction practices and will lead to durable concrete. The use, proper manufacturing and handling of epoxy-coated bars are but a few of the aspects related to durable concrete bridge decks.

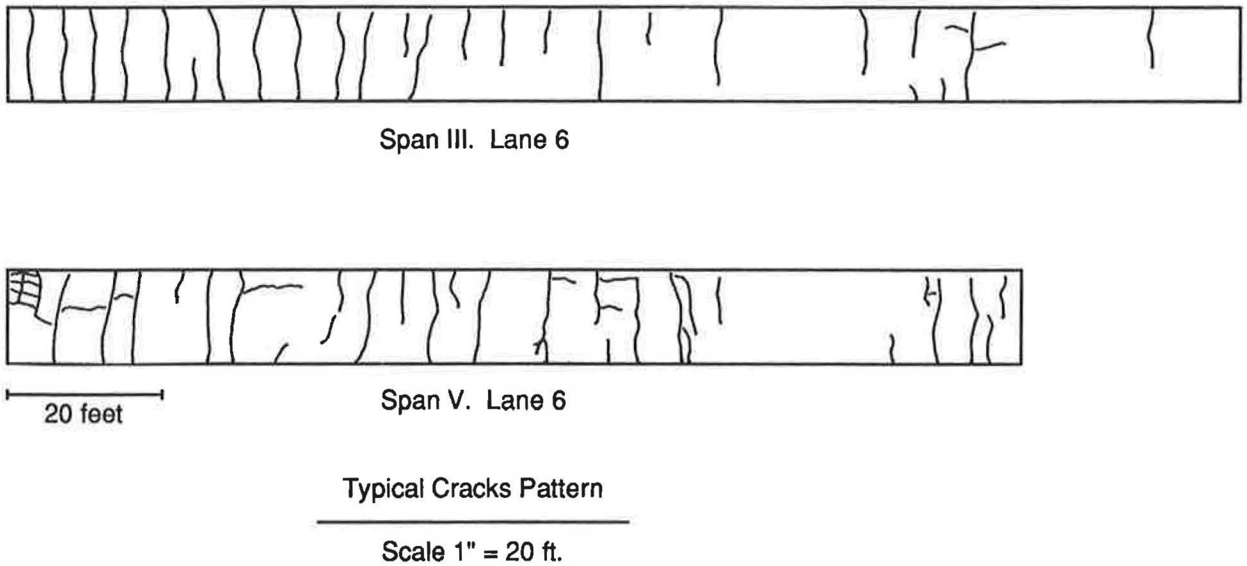


**Figure 4** Typical crack patterns.

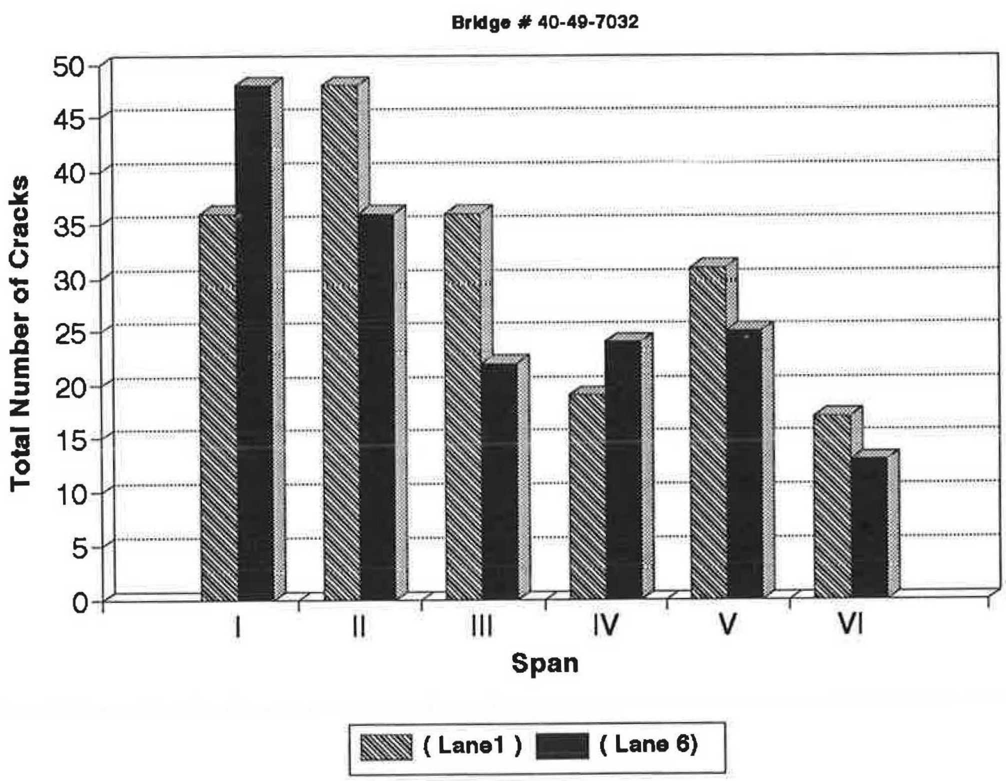


**Figure 5** Total number of cracks.

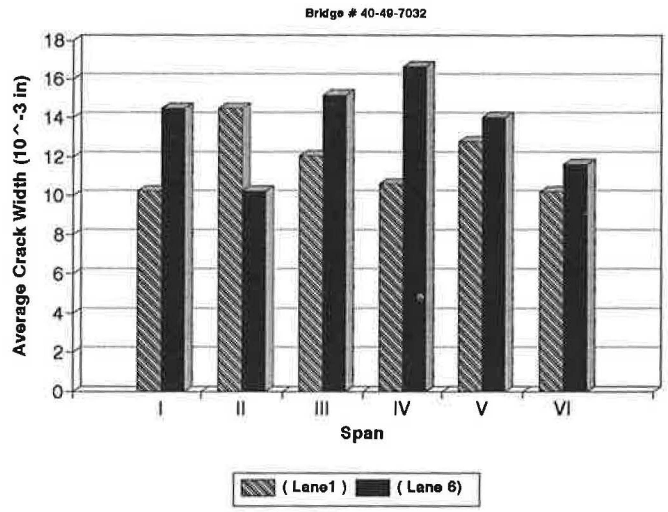


**Figure 6** Average crack width.

**Table V** COMPRESSIVE STRENGTH, UNIT WEIGHT AND CHLORIDE CONTENT OF CONCRETE CORES FROM A BRIDGE IN MARION COUNTY INDIANA

| Core No. | Span No. | Minimum Compression Strength (MPa) | Unit Weight (Kg/m$^3$) | Concrete Cover (mm) | Sample No. | Span No. | Chloride Content (Kg/m$^3$) at Depth (mm) of | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 25.4 | 50.8 | 76.2 | 101.6 |
| 1 | I | -- | -- | -- | 1 | VI | 2.11 | 0.72 | 0.89 | 0.56 |
| 2 | I | 41.92 | 2329 | 68.6 | 2 | V | 2.06 | 1.41 | 0.79 | 0.54 |
| 3 | I | 43.57 | 2372 | 55.9 | 3 | IV | 7.29 | 3.80 | 1.47 | 0.36 |
| 4 | II | 42.06 | 2340 | 55.9 | 4 | III | 5.39 | 1.17 | 0.81 | 0.68 |
| 5 | II | 33.99 | 2283 | 61.0 | 5 | II | 2.72 | 0.58 | 0.74 | 0.65 |
| 6 | IV | 43.64 | 2392 | 71.1 | 6 | I | 4.00 | 1.51 | 0.69 | 0.64 |
| 7 | IV | 43.64 | 2390 | 76.2 | 7 | I | 2.46 | 0.52 | 0.46 | 0.55 |
| 8 | V | 35.16 | 2315 | 68.6 | 8 | II | 2.32 | 0.71 | 0.58 | 0.73 |
| 9 | V | 48.40 | 2356 | 63.5 | 9 | III | 5.54 | 1.98 | 0.53 | 0.78 |
| 10 | VI | 42.82 | 2281 | 50.8 | 10 | IV | 1.33 | 0.64 | 0.75 | 1.00 |
| 11 | VI | 43.92 | 2334 | 48.3 | 11 | V | 4.67 | 0.68 | 0.70 | 0.86 |
| 12 | IV | 45.30 | 2355 | 40.6 | 12 | VI | 2.30 | 1.77 | 2.14 | 0.06 |
| 13 | IV | 42.13 | 2311 | 55.9 | | | | | | |
| 14 | II | 50.26 | 2403 | 76.2 | | | | | | |
| 15 | I | 25.99 | 2311 | 61.0 | | | | | | |

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