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Improving the Design of U-Beams for Indiana

Introduction

Many state agencies are faced with the challenge of improving deteriorating bridges while increasing roadway capacity and meeting aesthetic requirements of adjacent communities. These challenges have led to increased interest in bridges utilizing U-beams. The U-beam is a pretensioned concrete, open top, trapezoidal-shaped beam that was developed as an economical and aesthetic alternative to traditional I-beams. Bridges designed with U-beams typically require only one-half to two-thirds as many beams as compared to a traditional I-beam bridge. Additionally, U-beams have fewer horizontal break lines per beam. This combination of reduction in the number of girders and break lines leads to a greatly streamlined aesthetic.

While the use of the U-beam section is becoming more common, there are some design concerns and code limitations restricting their efficiency. Questions have arisen regarding how live load is distributed in a U-beam bridge as well as how the bridge deck behaves in flexure. Additionally, code limits on debonding of prestressing strand have been found in practice to limit both the efficiency and economy of this girder section.

The objective of this research program is to develop design strategies to improve the efficiency and optimize the design of the Indiana modified U-beam with a focus on the concerns related to the design of U-beams. In particular, this research program evaluated the live load distribution appropriate for the design of U-beams, assessed the behavior and design of the bridge deck when supported by U-beams, and evaluated both the shear strength and shear design of the composite U-beam system. It is important that the strength of pretensioned concrete beams with debonded strand be fully evaluated.

The research was completed in five major phases. Phase 1 consisted of the field instrumentation of the 21st Street Bridge, which is the first U-beam bridge to be built in Indiana. Phase 2 consisted of an experimental investigation of the effectiveness of debond sheathing. Phase 3 evaluated the influence of strand debonding on the shear strength of pretensioned beams. Phase 4 evaluated the effect of different concrete strengths in a composite section on shear strength. This is important considering that pretensioned girders are typically constructed as composite members using different concrete strengths. Finally, Phase 5 combined the results of Phases 2 through 4 to test scaled U-beams with and without transverse reinforcement to evaluate the applicability of the previous conclusions on this section shape and access overall system behavior.

Findings

Field Evaluation

The load test of the 21st Street Bridge allowed for measurement of the live load distribution factors for this bridge. Upon comparison of the measured live load distribution factors with those calculated based on the AASHTO LRFD Bridge Design Specifications, it appears that the expression for interior girder distribution factors is slightly nonconservative, but reasonable. The exterior girder live load distribution factor computed based on AASHTO was extremely conservative based on the testing performed. While the measured live load distribution factors are only applicable to this bridge deck and girder configuration, the results of this study indicate that a simple spring beam model can be used to closely and conservatively determine the live load distribution factors for interior and exterior girders.

The flexural behavior of the bridge deck between the interior and exterior girder lines exhibited a moment distribution with positive moment in the middle of the span and negative moment over the girder lines. The development of negative moment over the exterior girder lines is expected due to the continuity of the deck over the girder lines. The results of both a simple beam and shell model of the bridge deck indicate that the strains in the bridge deck can be accurately determined using simple finite element models. The shell model also indicates that the strip width values calculated according to the AASHTO LRFD Bridge Design Specifications are both reasonable and conservative.

Debond Sheathing Effectiveness

The type of debonding product used can have a significant impact on the effectiveness of strand debonding. While some of the split sheathing types tested showed acceptable performance, effective debonding can be ensured through the use of un-split sheathing or by sealing slit sheathing along its entire length. It was discovered that paste infiltration as a result of openings in the sheathing reduced the effectiveness of the un-taped, split sheathing, allowing for force transfer inside the debonded region. The amount of overlap the sheathing provides did not influence the results; however, tight-fitting split sheathing products perform better than looser-fitting products. To ensure effective debonding, sealing of split sheathing is strongly recommended.

Influence of Debonding on Shear Strength

As the percentage of debonding increased from 0% to 75%, shear strengths decreased. For V_{ci} at the end of the debonded region,

a 35% reduction in shear strength (at formation of primary shear crack) was observed in the specimen with 50% debonding relative to the specimen with 0% debonding. In increasing the debonded strand to 75%, a 61% reduction in shear strength (at formation of primary shear crack) occurred. For V_{cw} within the debonded region, a 16% reduction in shear strength (at formation of primary shear crack) was observed in the specimen with 50% debonding relative to the specimen with 0% debonding. Where V_{ci} cracks formed outside the debonded region, an 8% reduction in shear strength (at formation of primary shear crack) was observed in the specimen with 50% debonding relative to the specimen with 0% debonding. This minor reduction is within the scatter expected in shear test results.

The modulus of rupture was observed to be lower at the end of the debonded region than at midspan (fully bonded region). Values as low as $3\sqrt{f_c'}$ were observed for the Series II specimens with 75% debonding. This corresponds to a 53% reduction in the modulus of rupture relative to midspan. It is theorized that these reduced modulus of rupture values result from damage sustained at the end of the debonded region at transfer (when the strands were cut). As the number of debonded strands increased, the modulus of rupture at the end of the debonded region decreased.

Composite Section Shear Strength

The concrete compressive strength was observed to have almost no impact on the shear strength of the specimens tested in this experimental program. The small differences in test results (8% for the specimens with 0.48% reinforcement and 12% for the specimens with 2.40% reinforcement) are within the scatter expected in the shear test results.

U-Beam Shear Strength

Overall, shear strengths developed by these specimens were as expected based on the test results from the previous phases. Low modulus of rupture values were also observed for these specimens, especially at the end of the debonded region. On average, the modulus of rupture was $4.6\sqrt{f_c'}$ at midspan and $2.1\sqrt{f_c'}$ at the end of debonding. Therefore, debonding 50% of the strand resulted in a 54% reduction in the modulus. This was higher than observed for the rectangular section where 50% debonding resulted in a 26% reduction in the modulus. Adding transverse reinforcement in the debonded region provided additional shear capacity as well as improved ductility. With the addition of #3 at 12 in., the shear capacity was increased 20% beyond the shear at the formation of the primary shear crack. Shear crack widths were controlled and failure was not brittle. The transverse reinforcement also forced the shear failure to occur outside of the debonded region.

Implementation

The following recommendations are provided for implementation by INDOT to improve the efficiency and economy of girders utilizing debonded strands. These recommendations can be incorporated into the INDOT Design Manual as well as the standard construction specifications.

1. The percentage of debonded strands should not be limited. However, debonding of strands can have a significant influence on shear strength. Therefore, the concrete contribution to shear strength (V_c) must be calculated in the debonded region. Web-shear strength (V_{cw}) can control throughout the debonded region while flexure-shear strength (V_{ci}) will control at the end of debonding. For the calculation of shear strength of beams with debonded strand, the modulus of rupture should be assumed as zero ($f_r = 0\sqrt{f_c'}$) in the debonded regions to account for the reduced moduli that can occur at the ends of debonded regions. The lower limits for flexure-shear strength in AASHTO as well as ACI 318 are not appropriate and should not be used. In considering the composite section, the concrete strength in the compression zone of composite beams should be used to calculate flexure-shear strength. Conservatively, the section can be assumed as homogenous using the lower strength concrete.

2. Debond sheathing should be staggered so that all debonded strands do not begin transfer at the same location. A significant reduction (54%) in the modulus of rupture was observed when increased numbers of strands were transferred at the same location. The AASHTO LRFD Bridge Design Specifications provide recommendations regarding staggering that are considered reasonable to assist in minimizing this phenomenon.

3. All openings in debonding sheathing should be sealed to ensure effective debonding. Sealing can be achieved using a flexible adhesive tape such as duct tape. Alternately, un-split sheathing should be used.

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