Investigations of Shear Behavior in Reinforced Concrete Boxes

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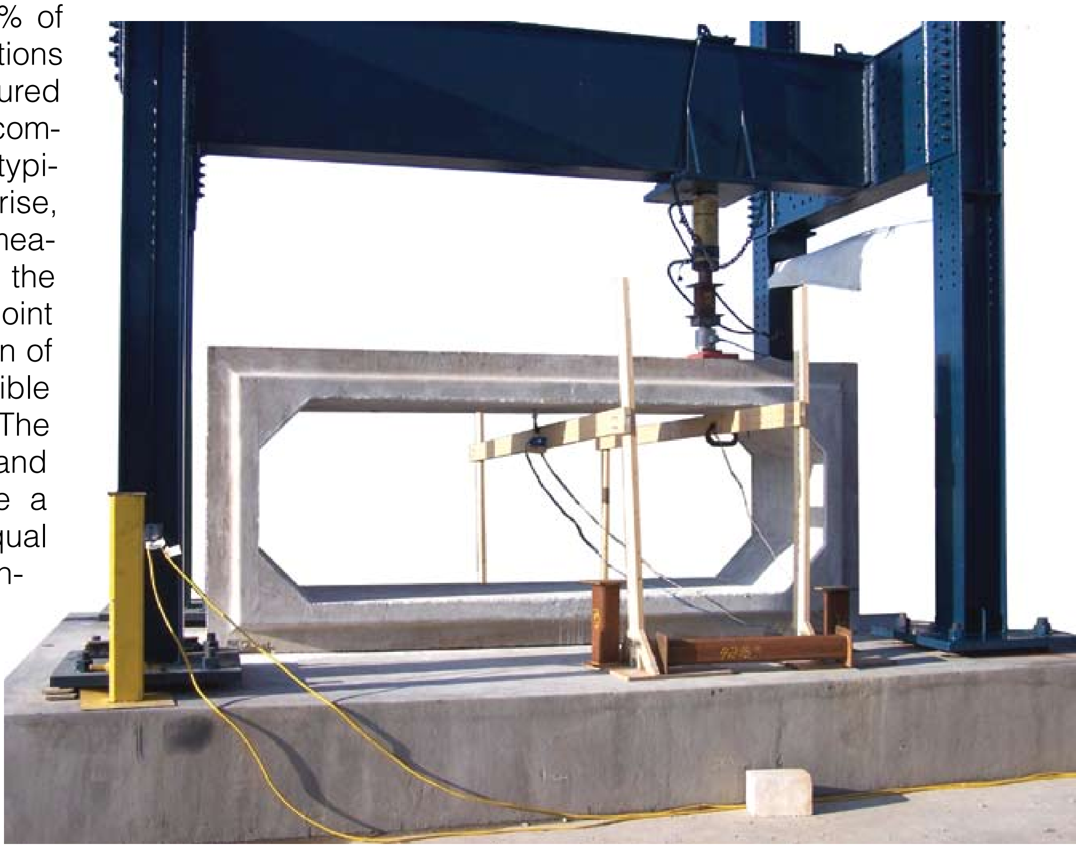
Introduction

Box culverts have been used increasingly since 1965 to meet drainage requirements where the site conditions and loads acting upon them have been appropriate. It is believed that 80% of single barrel culvert installations are precast, and manufactured in a range of span and rise combinations. Box sections are typically defined by their span, rise, and design height of fill measured from finished grade to the top of the box section. The joint or "laying" length is a function of the form equipment accessible to the individual producer. The inside corners of the wall and slabs are tapered to create a haunch, which usually has equal horizontal and vertical dimensions. The haunch dimensions are equal to the wall thickness though some producers utilize form equipment, which yields a fixed haunch dimension. With the exception of the special design cases, the thickness of culvert walls, top slab and bottom slab varies from 4 inches to 12 inches (10 cm to 30 cm) and is a function of the span. Boxes are reinforced with the inside and the outside layers of plain or deformed steel welded wire reinforcement per ASTM A 185 (2001) and A 497 (2001). These reinforcing layers are proportioned to resist the calculated moments and thrusts in the member's sections.

Precast box sections used to be designed as per ASTM C 789 for highway loading with earth cover of 2 feet (61 cm) or more or as per ASTM C 850 for highway loading with earth cover less than 2 feet (61 cm). Since 2003, ASTM C 1433 (2003) has replaced C 789 and C 850 for both loading conditions.

Precast box sections are typically cast by either the drycast



or wetcast method with batches designed to yield 5000 psi (34.5 MPa). Drycasting is characterized by the use of very low water/cement (w/c) ratios (0.35 or less) while wetcast uses standard mix designs yielding slumps in the range of 4 inches (10 cm) to 6 inches (15 cm).

Research Need

Box culverts are typically designed similar to bridges, and the new design concepts for bridges are based on the Load and Resistance Factor Design (LRFD) developed by AASHTO 1998. These specifications introduced new provisions for distributing live loads to the reinforced concrete bridge decks, which also apply to the design of reinforced concrete boxes with depths of fill less than 2 feet (61 cm).

The AASHTO (1998) provisions introduced three separate equations for the height of fill less than 2 feet (61 cm) based on axle load for distributing live load to the top slab of box culverts. These equations include one equation for spans greater than 15 feet (4.6 m) and two equations for spans less than 15 feet (4.6 m), depending on the sign of the bending moment. McGrath et al. (2004) reported that the distribution width equation for spans greater than 15 feet (4.6 m) were developed based on the National Cooperative Highway Research Program (NCHRP) Project 12-26, while the distribution width for spans less than 15 feet (4.6 m) were based on a study conducted by Modjeski and Masters (2003).

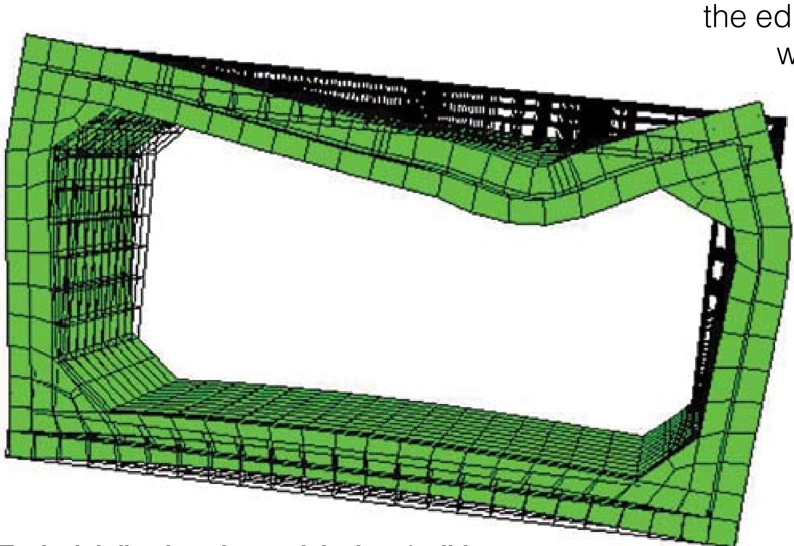
AASHTO (2002) provisions provided a single equation for distribution width for heights of fill less than 2 feet (61 cm), based on a single axle load on the top slab of boxes. This distribution applies to all span lengths for both positive and negative bending moments and shear force. Compatible comparisons of the distribution width for depth of fill less than 2 feet (61 cm), calculated based on AASHTO (1998) and AASHTO (2002) indicate noticeable differences.

To address these differences and develop a more simple distribution, McGrath et al. (2004) used the element method (FEM) to investigate the live load distribution widths for reinforced concrete boxes. This study concluded that the distribution width for shear in general was narrower than that of positive and negative bending moments, and it governed the behavior. The results of this study are implemented in the Interim AASHTO (2005) specification, which provides new distribution width equations based on shear force distribution. The provisions suggest a means of transfer should be provided across the joint, if the calculated distribution width exceeds the length between the two adjacent joints.

AASHTO (1998) specifications require design check for shear at all depths of fills, while the AASHTO (2002) specifications only require it for depth of fill more than 2 feet (61 cm). This is because the shear strength characteristic of boxes with depth of the fill less than 2 feet (61 cm) is controversial, since boxes are constructed with spans as small as 3 feet (91 cm) with slabs thinner than typical bridge decks. Prior to Interim AASHTO (2005) specifications, boxes were not required to be designed with joints to transfer direct shear across the joints. This concept was based on the research studies conducted by James (1984) and Frederick et al. (1988), which reported that shear transfer was not critical with zero fill depth across the joint due to the small deflections and strains that caused no cracks at service load. However, both the aforementioned studies placed the wheel live load at the edge of the bell or spigot ends, at the middle of the culvert's span during their experimental testing and/or modeling. Therefore, this raised concerns that the wheel load location may not have produced the critical shear stresses, since it was placed away from the vicinity of box's wall (support).

A recent study at the University of Texas at Arlington (UTA) was undertaken to enhance the knowledge gained from the McGrath (2004) research which utilized linear elastic modeling without experimental verification of the model. In addition, a better grasp of the behavior of boxes was needed. The UTA research report is based on the findings of a major and comprehensive full-scale experimental and finite element study that considers all practical culvert span sizes with and without distribution steel (As6) in the top slab. Twenty-four full-scale experimental tests were conducted on the common ASTM C1433 boxes with varying sizes. Several tests were also conducted to identify the location of the wheel load, that produces the maximum shear effects. A comprehensive nonlinear inelastic three dimensional finite element model was developed with capabilities to predict crack initiation and propagation that is validated with the conducted full-scale tests. Finally, the developed models were used to obtain the distribution width values for shear, which were then used to calculate the shear capacity of the ASTM C1433 precast boxes.

Conclusion



Typical deflection shape of the box (solid – Deformed, and wireframe – undeformed)

The full-scale experimental tests indicated that flexure governed the behavior for all the test specimens up to and beyond the AASHTO factored live load. For all the test specimens the flexural cracks formed initially on the inside face of the top or bottom slab, which extended to the spigot or bell toward the middle of the load plate. No flexural cracks were observed at loads below the AASHTO service load.

Another series of cracks, for all the test specimens, were negative moment cracks which formed on the wall closest to the load plate along the joint length at a distance equal to approximately one-third from the top slab. These cracks normally extended to the spigot and bell ends.

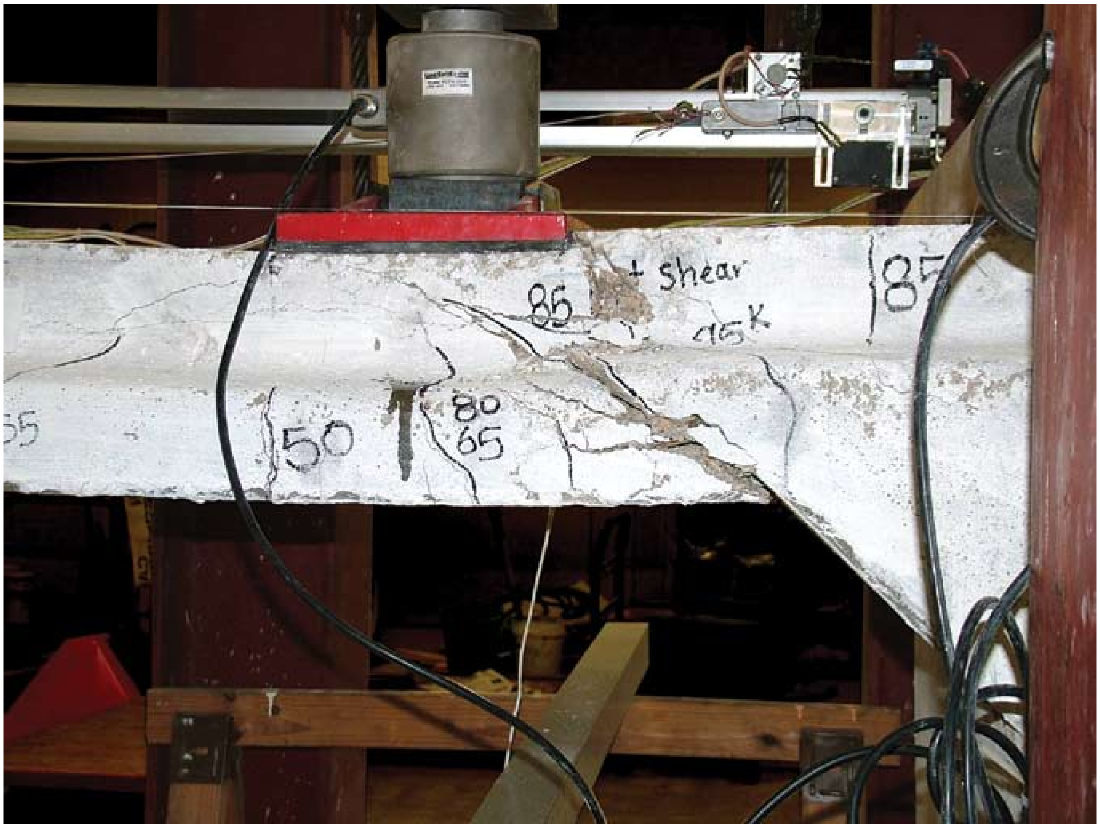
The shear cracks were among the final cracks observed. For all of the test specimens, shear cracks formed at approximately 72 kip (320 kN) of load (almost twice the AASHTO factored load). These cracks initiated independently from the tip of the haunch (on the spigot or bell testing end) and extended toward the edge of the load plate. By independent shear crack, we mean that it did not initiate at the tip of the flexural cracks. No shear crack was observed before flexural cracks in any of the specimens tested.

Even though the load plate was placed at distance **"d"** from the tip of the haunch to the edge of the load plate, the box's behavior was governed by flexural cracks during the experiment up to high load levels. This was due to the box's joint rotation, which contributed significantly to the box's bending moment. Thus, it was concluded that the behavior of the box was different than that of the bridge slabs. Furthermore, the AASHTO bridge design concept for the distribution width was not justifiable for culverts.

The comparison of the test results, with and without top slab top face distribution steel, showed that the effect of the distribution steel in the top slab is insignificant. This comparison was made with respect to crack initiation and propagation as well as the load-deflection plots. The overall box behaviors during the course of experiments were almost identical for the specimens with and without top slab distribution steel.

The final failure for all the test specimens was due to shear/bond failure at loads ranging from 72 kip (320 kN) to 160 kip (712 kN) or above for all the boxes tested.

The finite element model exhibited close correlation with the experimental results for load-deflection and crack prediction for all the test specimens. The FEM analyses showed that when the load plate was placed at the distance **"d"** from the tip of the haunch to the edge of the load plate, the value of the maxi-



Photograph of a 244 cm (8 foot) Span and 122 cm (4 foot) joint length specimens at failure

mum shear force was located between the edge of the haunch and the edge of the load plate for all the boxes' geometry used in the experimental program. Indeed, for 70% of the boxes tested, the maximum shear force value was at one-half the distance between the tip of the haunch and edge of the load-plate.

It was shown that the maximum value of the shear force increased as the boxes' span increased for the same load. This counter intuitive finding is because the wheel load plate was placed at a distance **"d"** from the tip of the haunch to the edge of the load plate, and since the span increases in a larger magnitude 3 feet (91 cm) to 12 feet (366 cm) compared to the increase in **"d"** 6 inches (15 cm) to 12 inches (30 cm), the **span** /**"d"** ratio was larger for the larger span boxes. This implies that for the shorter span boxes, the load plate was closer to the center of the span which forces more bending than shear behavior.

The values of the distribution width calculated based on the validated FEM analyses in this study were between one-fourth and one-third of those calculated, based on the AASHTO 2005. Since experimental testing of 24 test specimens and FEM analyses of 42 ASTM C1433 box geometries confirmed that shear was not governing the behavioral mode (particularly at service and factored AASHTO loads), it is concluded that there are no relations between the AASHTO 2005 distribution width equations and the box's behavior.

The critical factored shear force for all the ASTM C1433 precast boxes were calculated and compared with the two ACI shear capacity equations: *w c 2b d f ‘* (lower bound) and *w c 3.5b d f ‘* (upper bound). It was shown that the shear capacity exceeds the critical shear force for all the aforementioned cases.

The study recommends that the AASHTO 2005 distribution width for boxes needs to be revisited. It is highly recommended the following statement "shear transfer device should be provided across the joint, if the calculated distribution width exceeds the length between the two adjacent joints" be eliminated from the AASHTO 2005 provisions.

Note: This article is an abstract of the study entitled, *"Experimental and Finite Element Based Investigations of Shear Behavior in Reinforced Concrete Box Culverts."* Contact Dr. Abolmaali at 817-272-3877 for a complete report.