Experimental Investigation of Shear Capacity of Precast Reinforced Concrete Box Culverts

Anil K. Garg[1]; Ali Abolmaali[2]; and Raul Fernandez[3]

Abstract: This study presents an experimental program to investigate the shear capacity of precast reinforced concrete box culverts. Each culvert was subjected to monotonically increasing load through a 254 mm × 508 mm (10 in. × 20 in.) load plate in order to simulate the HS20 truckload per AASHTO 2005. Instrumentation included strain gauges, high-resolution laser deflection sensor, and automated data acquisition. Four tests were conducted on 1.22 m × 1.22 m × 1.22 m (4 ft × 4 ft × 4 ft) box culverts. The location of the load plate was varied to identify the position, which introduces the maximum shear stresses. Laser sensor data and dial gauge readings were recorded to measure the deflection profile of the box culvert. Strain gauges were placed on the steel reinforcement to measure axial strain at locations of maximum positive and negative bending moments. The test results include reporting the loads at which each crack initiated and propagated. The displacement profile of the top slab from the laser instrumentation output along with the load versus maximum deflection for each culvert is also reported.

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Introduction

Culverts are miniversions of bridges for conveyance of natural surface drainage water under roads, taxiways, runways, or railroads. Box culverts have been used increasingly since 1965 to meet drainage requirements where site conditions and the loads acting upon them have been appropriate.

Based on the writers interviewing several pipe manufacturers, through the American Concrete and Pipe Associations, it is believed that 80% of single barrel culvert installations are precast. They are considered to be efficient since they reduce project execution time, in particular, they are ideal when the concrete batch plant is not near the construction site. Precast box culverts are not recommended for areas with excessive settlement where pile foundations are required since pile supported foundations would have to be placed on shorter intervals with the use of precast sections making the installation excessively expensive.

The precast concrete box culverts are manufactured in a range of standard span and rise combinations. A typical cross section of box culverts is shown in Fig. 1. Box culvert sections are typically defined by their span, rise, and design height of fill as 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 individual producers. The standard span per ASTM C 1433 (ASTM 2003) varies from 0.91 to 3.66 m (3–12 ft), the rise varies from 0.61 to 3.66 m (2–12 ft)—both in 0.3 m (1 ft) increments. As mentioned before, the joint lengths vary as a function of the form equipment available to the producer, and they generally vary from 1.22 m (4 ft), as a practical minimum length, up to 2.44 m (8 ft). These sizes are governed by the shipping weights and shipping sizes being the influencing factors. Joint lengths of 3.66 m (12 ft) or even 4.88 m (16 ft) may be available on a regional basis. The inside corners of the wall and slabs are tapered to create a haunch, which has equal dimensions horizontally and vertically (refer to Fig. 1). The haunch dimension is equal to the wall thickness though some producers utilize form equipment, which yields a fixed haunch dimension [usually either 203 mm (8 in.) or 305 mm (12 in.)]. With the exception of special designs, the thickness of culvert walls, top slab, and bottom slab varies from 102 to 305 mm (4–12 in.) and is a function of the span. The box culverts are reinforced with inside and outside layers of plain/deformed steel welded wire reinforcement as per ASTM standards A 185 (ASTM 2001b)/A 497 (ASTM 2001a). These reinforcing layers are proportioned to resist the calculated moments and thrusts in the member's sections. Inside concrete cover (C_i) and outside concrete cover (C_o) is 25 mm (1 in.) except for the unique cases where the height of fill is less than 0.61 m (2 ft), C_o is equal to 51 mm (2 in.) per AASHTO (2005). Precast box sections are designed as per ASTM C 789 for highway loading with earth cover of 0.61 m (2 ft) or more; or as per ASTM C 850 for highway loading with earth cover less than 0.61 m (2 ft). Since 2003, ASTM C 1433 (ASTM 2003) has replaced C 789 and C 850 for both loading conditions. Precast box sections are typically cast with batches designed to yield 34.5 N/mm^2 (5,000 psi). Precast sections are produced by either the drycast or the wetcast method. 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 to slumps in the range of 4–6 in. and using w/c ratios commensurate with wetcast batches.

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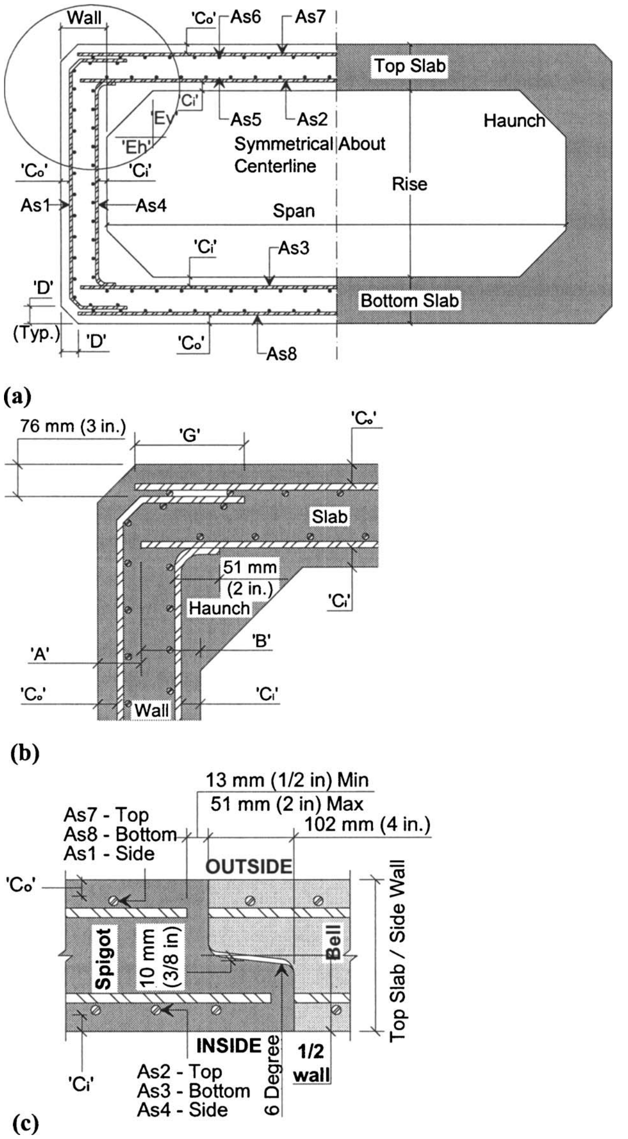

Fig. 1. (a) Typical cross section of box culvert; (b) details of reinforcement at haunch; and (c) joint detail of box culvert

To join the boxes, ends are formed at bell and spigot ends. The spigot end goes into the bell end while placing in position as shown in Fig. 1.

Among previous studies, McGrath et al. (2004) conducted a study for the Pennsylvania Department of Transportation to investigate the live load distribution widths for reinforced concrete box culverts. This study established equations to calculate the distribution of axle loads to the top slabs of box culverts, with 0–0.61 m (0–2 ft) of fill depth, by using finite-element analysis. This study compared in detail the design equations for AASHTO (2005), which concluded that the LRFD specifications are more conservative for box culverts, particularly with spans less than 4.57 m (15 ft). Based on finite-element method (FEM) analyses, they reported that the distribution widths for shear are less that the widths for bending moment. Thus, they concluded the shear design for box culverts bears some scrutiny.

Yee et al. (2004) investigated and performed tests on shear behavior of precast reinforced concrete box culverts. This study concluded that all the designs based on *Canadian Highway and Bridge Design Code* (CHBDC) and AASHTO were conservative. Test results showed that the analysis tools used generated reasonably reliable predictions of moment distributions.

Smeltzer and Bentz (2004) conducted a study to determine the safety of precast concrete box culverts subjected to brittle shear failures. They recommended that more research and investigation should be performed to outline the adequacy of the box culvert design in shear, and to identify the need for shear reinforcement requirement.

Thus, due to the ambiguity with regard to the shear behavior and capacity of the precast box culverts, this study was undertaken. The culverts tested have a span, rise, and joint length of 1.22 m (4 ft). The thicknesses of the top and bottom slab are 191 mm (7.5 in.) and 152 mm (6.0 in.) respectively. The walls and haunches have a thickness of 127 mm (5.0 in.).

The reinforcement cages consist of plain welded wires as per ASTM standards A 185. The size of steel wires used for the reinforcement cages ranges from $W2.0$ to $W8.0$ with their areas varying from 12.90 mm^2 (0.02 in.^{2}) to 51.61 mm^2 (0.08 in.^{2}) per reinforced wire. The nominal diameter of each wire ranges from 4 to 8 mm (0.159–0.319 in.). Typical spacing of wires was 51 mm (2 in.)/102 mm (4 in.)/152 mm (6 in.).

Test Setup and Instrumentation

The test setup consists of a reaction frame with four ($W12$) steel columns welded to 902 mm × 902 mm (35 1/2 in. × 35 1/2 in.) base plates with a thickness of 51 mm (2 in.). The base plates were bolted to the heavily reinforced concrete reaction floor using 51 mm (2 in.) diameter bolts. Two cross beams were bolted to the reaction frame. A mild steel plate 610 mm × 749 mm × 51 mm (24 in. × 29.5 in. × 2 in.) was bolted in soffit/underneath the pair of beams and a hydraulic cylinder is bolted underneath this plate. A steel t-stub column is fixed to the bottom of the cylinder as a filler to transfer the load to the box culvert through the load cell as shown in Figs. 2(a–c).

A 254 mm × 508 mm × 25.4 mm (10 in. × 20 in. × 1 in.) mild steel load plate with a 12.7 mm (1/2 in.) thick rubber sheet was placed on top of the box culvert. This was done to simulate the contact area of the wheel of a HS20 truck or tandem; having an axle load of 142 kN (32 kip) and wheel load of 71 kN (16 kip). An 890 kN (200 kip) capacity precalibrated load cell was placed on the top of the load plate. A load leveler was also used on the top of the load cell to ensure the application of concentrated load to the culvert [Fig. 2(b)].

For one of the tests 76 mm (3 in.) thick 20 mm (3/4 in.) graded stone aggregate bedding was used to simulate real site conditions (Fig. 3). The other tests were performed without the bedding material to isolate the response of the box culvert from the bedding material.

A computerized instrumentation and data acquisition system was custom developed for this study. The measured variables were applied load (electronic load cell), reinforcement strains at five locations (strain gauges), and culvert top slab deflection (laser system). In addition, manual dial gauges were separately used in certain tests to record bedding compaction/sidewall deflections. The load cell used is a standard 450 Ω (single) full-bridge axial unit with 890 kN (200 kip) capacity with a combined error of 0.15% of full scale. It features a stainless-steel housing and hermetical seal design.

Culvert steel reinforcement strains were recorded with general-purpose uniaxial gauges. This basic gauging configuration was selected on the expectation that reinforcements would

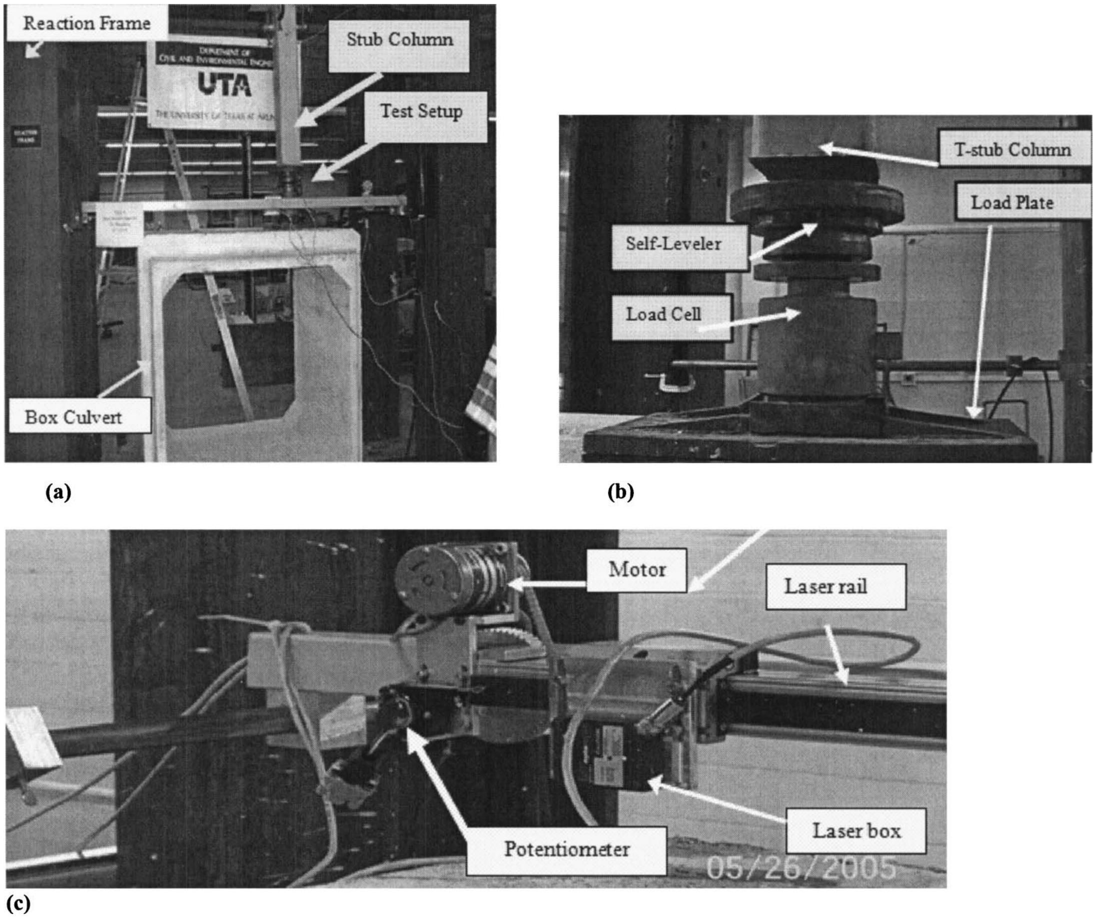

Fig. 2. (a) Test setup and instrumentation; (b) details of load cell and load plate assembly; and (c) details of laser instrumentation

experience a pure axial strain; further commentary on strain gauge locations and purpose is found in the next section. The gauges have a constantan 350 Ω grid with polyimide encapsulation and large-area copper soldering tabs. Fatigue life and operating temperature of the gauges were not important considerations for the testing conditions, and their advertised strain range of 3% was well in excess of that expected. The grid size was selected to be as large as possible for ease of application, while occupying a small area in the circumference of the reinforcement bar. Gauges were applied following standard steel surface preparation practice, and were encapsulated with a special M-COAT J-3 compound to maximize survival during the concrete pouring and casting. Standard three-conductor lead wire connections to each gauge were made.

A moving laser-based optoelectronic displacement sensor was used to accurately measure the deflection profile of the top slab of the culvert. This device measures distances within a 100 mm range (minimum standoff of 50 mm) with 20 μm resolution at 1 kHz; it operates by projecting a laser beam on the target, which is reflected and imaged on a charge coupled device (CCD)-array element such that the target distance may be computed by triangulation. This laser displacement unit (model MicroEpsilon IDL 1400-100) was then mounted on a single-axis motorized stage powered by a Compumotor AX step motor indexer/driver, allowing it to record a continuous stream of top slab deflection measurements along a 1.2 m (48 in.) track. To ensure minimal skewing of span position and deflection measurements, and to minimize acquisition system complexity, a retractable draw-wire potentiometer was attached to the track shuttle to measure span position as an additional analog instrumentation channel. Finally, the draw-wire potentiometer was directly wired into the data acquisition board as a ratio metric input channel.

Strain gauges were placed on the steel wires of the box culverts as shown in Fig. 4. Wires were soldered to the strain gauges and fed to a central point in the box culvert where they would be protected from the impact of pouring concrete as shown in Fig. 5. The strain gauge locations were at midspan, directly under the load and position of maximum negative bending moment.

Testing Procedure

The load was applied to the box culvert through the load plate by using a hydraulic pump. The load was applied at the increments

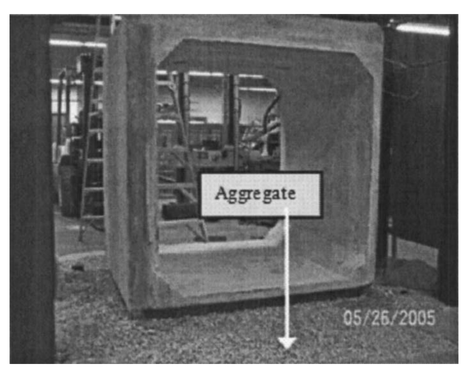

Fig. 3. Setup with bedding materials

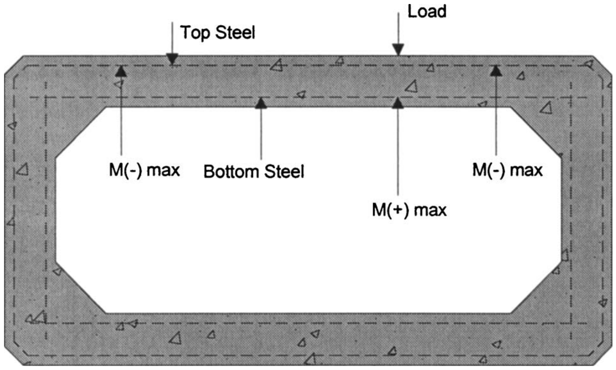

Fig. 4. Location of strain gauges

of 4.45 kN (1 kip) initially up to 44.5 kN (10 kip) after which the load increment was changed to 22.25 kN (5 kip). After each increment was reached, the laser automatically traversed through the culvert's span to measure the deflection.

Crack Monitoring Process

While testing, to ensure consistency between tests, a method was used for identifying crack initiations. After a crack appeared, a black marker was used to draw a line parallel to the crack. When the crack was no longer visible in the culvert, a bar was drawn across the previously drawn line indicating the end of the crack for that load. The load was written in kips next to the line for ease of referencing during testing and in future evaluations (Fig. 6).

Test Results and Comparisons

Different loading positions were tested to examine the behavior of box culverts. All tests were conducted on the spigot end of each culvert. The focus of this portion of the study was to monitor the initiation and development of the cracks. Each test specimen was identified by a code found in the Notation. The test designations were defined as: S or B, S or D B, SRL, NB, WB, or P. For example, S-SB-444-NB-5 identifies a spigot-end single box culvert test whose dimensions are: span=1.22 m (4 ft); rise =1.22 m (4 ft), and joint length=1.22 m (4 ft) with no bedding. The center of the load plate for this test was located 5 in. (127 mm) from the edge of the haunch.

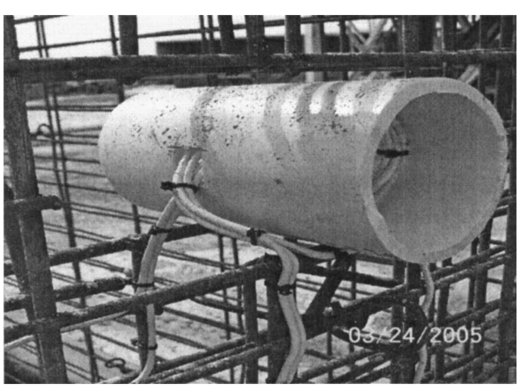

Fig. 5. PVC pipe used to protect wires

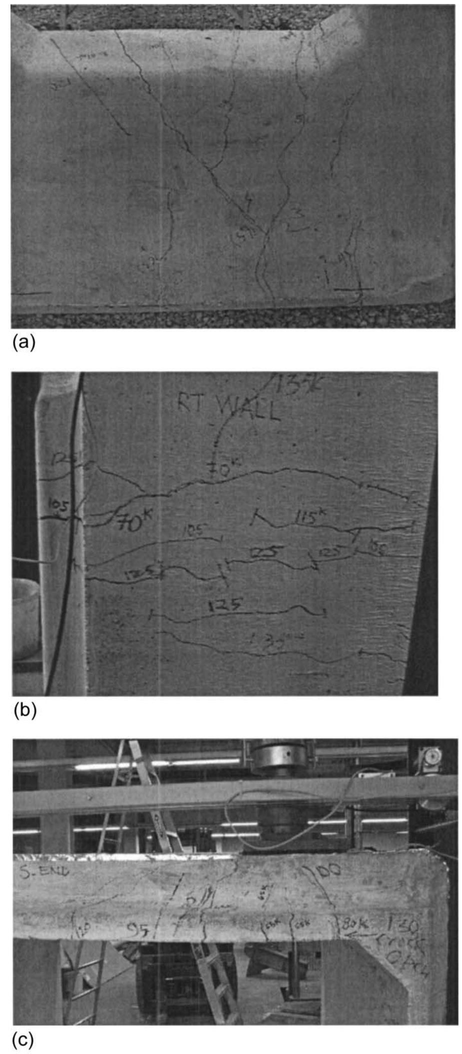

Fig. 6. Typical crack pattern; (a) bottom slab; (b) right-side wall (near load); and (c) at spigot end

Test Results

Test 1: S-SB-444-WB-5

The first superficial (hairline) flexure cracks initiated on the spigot end at the top of the bottom slab at 156 kN (35 kip) directly under the load plate. The second superficial flexure crack appeared on the top of the bottom slab directly under the load at approximately 178 kN (40 kip). The thickness of each crack was less than 0.33 mm (0.013 in.). Formation and propagation of superficial flexure cracks continued on the top of the bottom slab and on the bottom of the top slab up to 356 kN (80 kip). The first shear crack appeared on the tip of the haunch, near the load at 356 kN (80 kip). At approximately 400 kN (90 kip), flexure cracks appeared on both outer sidewalls spanning the entire length of the culvert. A shear crack on the spigot end at midspan appeared at 423 kN (95 kip). Formation, propagation and widening of flexure and shear cracks continued until ultimate failure at 632 kN (142 kip), which occurred along the right sidewall and continued onto the top slab and behind the load plate. After the box culvert was removed from the reaction frame, it was noted that the ag-

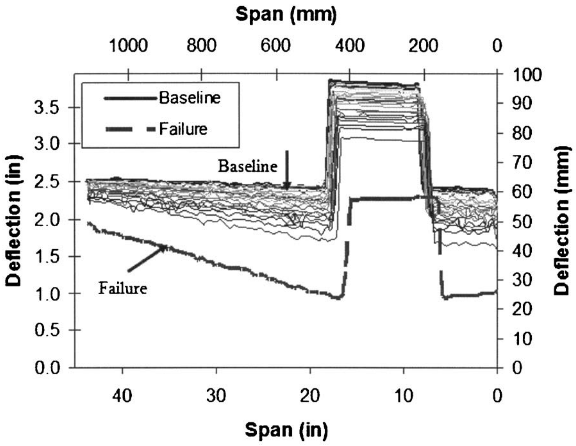

Fig. 7. Typical automated laser sensor deflections

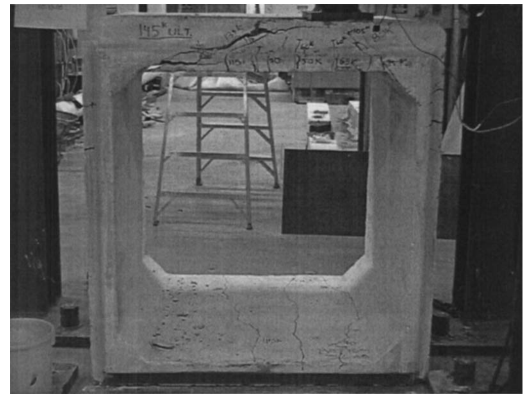

Fig. 8. Typical ultimate failure view at spigot end

gregate bedding had settled up to 48 mm (1.875 in.) directly under the load. It was also noted that some of the aggregate was crushed by the loaded box culvert.

The data from the laser sensor system were analyzed. Fig. 7 shows a typical contour of the box culvert during testing. The load plate appears as an elevated section of the graph which is 254 mm (10 in.) along the span. For measuring deflection, a baseline contour has been taken before beginning the test. For each loading, contours have been plotted. These contours were then compared to the baseline contour to give deflections at specific loads.

Alternatively, four dial gauges on the four corners of the bottom slab were stationed to measure settlement of the bedding material. Fig. 8 shows a typical ultimate failure view at the spigot end.

Test Two: S-SB-444-NB-5

The second test was conducted similarly to Test 1, but without bedding. The first superficial flexure crack [thickness less than 0.25 mm (0.01 in.)] appeared on the top of the bottom slab and the bottom of the top slab at 133 kN (30 kip) simultaneously. Formation and propagation of these cracks continued on the top of bottom slab and on the bottom of the top slab up to 267 kN (60 kip). Flexure cracks along the sidewalls first appeared at 267 kN (60 kip) with their thickness being less than 0.33 mm (0.013 in.). The first shear crack appeared on the tip of the haunch, near the load, at 267 kN (60 kip) which was widened at 334 kN (75 kip). Up to 512 kN (115 kip) new flexure cracks developed in both side walls, in full joint length. A shear crack appeared at midspan in the spigot end at 512 kN (115 kip). Formation, propagation, and widening of the flexure and shear cracks continued until ultimate failure at 712 kN (160 kip), which occurred as shear failure joining the haunch and load plate.

Test Three: S-SB-444-NB-6 1/2

The difference between this and the second test (S-SB-444-NB-5) was that the load plate was moved 38 mm (1.5 in.) toward the center of the culvert, to make the center of the load at distance d from the tip of the haunch, where d=effective depth of the top slab of the box for positive reinforcement. The first superficial crack appeared at 107 kN (24 kip) near the spigot end at the top of the bottom slab and propagated further at 156 kN (35 kip). Also, a small crack at the bottom of the top slab under the load plate appeared at 156 kN (35 kip). Formation and propagation of superficial flexure cracks continued on the top of the bottom slab and on the bottom of the top slab up to 289 kN (65 kip). Flexural cracks along the sidewall near the load and away from the load appeared at 312 kN (70 kip) and 334 kN (75 kip), respectively, with their thickness being less than 0.33 mm (0.013 in.). The first shear crack appeared on the tip of the haunch near the load at 334 kN (75 kip). The first shear crack at the midspan developed on the spigot end at 512 kN (115 kip). Formation, propagation, and widening of the flexure and shear cracks continued until ultimate failure at 645 kN (145 kip), which occurred as shear failure joining the haunch and load plate (Fig. 8).

Test 4: S-SB-444-NB-11 1/2

The difference between this and the third test was that the load plate was moved further 127 mm (5 in.) toward the center of the culvert, thus making the center of the load a distance of 292 mm (11.5 in.) from the tip of the haunch. This was done to place the near edge of the load plate at distance d from the tip of the haunch. The first superficial flexural crack initiated at 111 kN (25 kip) and was located on the top of the bottom slab and the bottom of the top slab simultaneously, directly under the load plate. At 133 kN (30 kip), both cracks extended toward the bell end of the culvert. Cracks on the bell end began appearing at 200 kN (45 kip) near the center of the culvert. As the load was increased, more cracks developed away from the center of the culvert on the bell end. A superficial flexural crack appeared along the sidewall near the load at 245 kN (55 kip) and spanned the entire length of the culvert. The superficial flexural crack on the other sidewall appeared at 289 kN (65 kip). The first serviceability flexure crack [wider then 0.33 mm (0.013 in.)] was developed on the bottom of the top slab at 289 kN (65 kip). Serviceability shear cracks developed to join the haunch and the load plate at 534 kN (120 kip). Formation, propagation, and widening of the flexural and shear cracks continued until the ultimate failure load of 578 kN (130 kip), which occurred as shear/bond failure joining the haunch and the load plate. The final failure crack for this test is believed to be due to arching action which resulted in bond failure (Fig. 9).

Test Comparisons

It should be noted that in every test the load plate was placed on the right side of the spigot end of the culvert, thus all the test

Fig. 9. Spigot end top slab at ultimate failure for S-BC-444-NB-11 1/2

specimens exhibited similar crack patterns with respect to initiation and propagation. Generally, the initial superficial flexural cracks occurred on the top of the bottom slab or on the bottom of the top slab directly under the load plate on the spigot end. Flexural cracks continued to occur in these two places throughout testing. At approximately 245–400 kN (55–90 kip), flexure cracks initiated on the sidewalls of the culverts for all four test specimens. The last three tests failed due to arching action along the spigot end to the left of the load plate. The failure cracks were nearly identical extending from the left corner of the load plate diagonally to the edge of the haunch.

The first two tests were compared to evaluate the effects of aggregate on the culvert. For both tests the load plate was placed 127 mm (5 in.) from the haunch. Tests 2, 3, and 4 were compared to evaluate the culvert's response to the different positions of the load plate. For each test, a load versus maximum deflection was plotted. All four tests are shown in the graph of Fig. 10. As per AASHTO (2005) the permissible maximum deflection for a 1.22 m (4 ft) span is 1.52 mm (0.06 in.) (span/800) at the service load of 71 kN (16 kip). Tests 2, 3, and 4 satisfied this parameter. However, the maximum relative deflection recorded for Test 1 was higher than $L/800$. For this test the culvert was supported by the bedding material to simulate the culvert's actual support condition in the field, while the effect of the lateral soil pressure in the field was not simulated in the laboratory, which would reduce

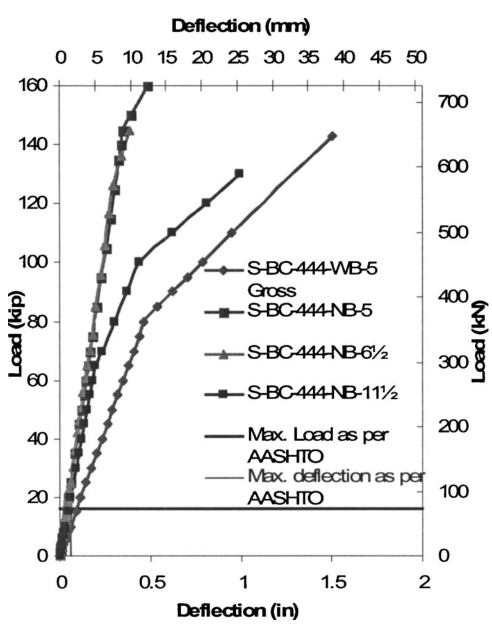

Fig. 10. Load versus deflection plots for all tests

Table 1. Summary of Test Results

Test	First shear crack 0.33 mm (0.013 in) wide: Load [kN(kip)]	First shear crack 0.33 mm (0.013 in) wide: Location	Failure load [kN(kip)]	Failure mode
S-SB-444-WB-5	356 (80)	Spigot end	632 (142)	Shear
S-SB-444-NB-5	267 (60)	Spigot end	712 (160)	Shear
S-SB-444-NB-6 1/2	334 (75)	Spigot end	645 (145)	Shear
S-SB-444-NB-11 1/2	356 (80)	Spigot end	578 (130)	Shear/bond

the overall culvert's transverse deflection. It should be noted that for the test in which bedding material was used, the relative displacement was calculated. However, for the tests without the bedding materials, absolute displacement was measured since the vertical joint deflection was considered negligible. This was done to capture the effect of the support irregularities on the tensile stresses induced in the culvert, which causes crack formation. Thus, the maximum deflection reported for Test 1 does not reflect the true culvert's deflection in the field. Even though one would argue that Tests 2, 3, and 4 also lack the effect of the field lateral soil pressure, for these tests bedding material was not applied to the culverts. Thus, the absence of both bedding material and lateral soil pressure would offset each other.

Finally, Table 1 presents a summary of the test results which include the loads at which 0.33 mm (0.013 in.) shear cracks occurred, the ultimate load, and the failure mode for each test specimen. This table shows than shear cracks for all the test specimens formed at 267 kN (60 kip) to 356 kN (80 kip) which are above the strength limit state of 174 kN (39.02 kip) for live load were used in design per AASHTO (2005).

Conclusions

The precast concrete box culverts are manufactured in a range of span and rise combinations. To better understand the shear behavior of box culverts, four tests were conducted on test specimens of 1.22 m (4 ft) span, 1.22 m (4 ft) rise, and 1.22 m (4 ft) joint length. The bedding material was placed only in the first test specimen to simulate the site conditions with regard to the bedding materials. The wheel load of a HS20 truck or tandem was simulated by applying the load on a box through a steel plate representing a footprint of one wheel. To capture the location at which maximum shear stresses are induced, the center of the load plate was placed at three different locations for different tests: (1) at 127 mm (5 in.); (2) at 165 mm (6.5 in.); and (3) 292 mm (115 in.) from the tip of the haunch. The load at which failure cracks for all the test specimens stopped the testing ranged from 578 kN (130 kip) to 712 kN (160 kip). The failure cracks were identified as a combination of bond and shear cracks due to a combination of excessive shear stresses and arching action.

All the test specimens behaved in a similar fashion with initial flexural cracks forming at the top of the bottom slabs and bottom of the top slabs. For all the test specimens, hairline shear cracks initiated at approximately 267 kN (60 kip)–356 kN (80 kip). The aforementioned loads are above the strength limit state of 174 kN (39.02 kip) for live load used in design per AASHTO (2005).

Thus, based on the results of the boxes tested, the shear capacity of box culverts is adequate and shear reinforcements are not required.

McGrath et al. (2004) used linear elastic finite-element analyses (FEA) and concluded that the shear distribution width for the applied live load was smaller than that for flexure, and thus, shear was more critical. Their linear elastic FEA was not calibrated against experimental observations. The fact that linear elastic analyses were used is indicative that their model was unable to predict cracks and the behavior up to failure. This combined with the lack of experimental observations led them to believe that the small distribution width for shear (at service load) compared to that of moment (at service load) indicates that the shear is more critical, and it governs the behavior. Our study, which is based on the full-scale experimental observations for the 1.22 m (4 ft) span, as well as those reported by Abolmaali and Garg (2006) for the 2.44 m (8 ft) span boxes, contradicts the McGrath et al. (2004) findings.

Cylinder tests were conducted for compressive strength of the concrete used in the box culverts under tests, which varied from 6,000 to 7,000 psi. Thus, the ratio between the actual compressive strength tested and those recommended by the ASTM C 1433, which was also used by McGrath et al. (2004), is ($\sqrt{6{,}500/5{,}000}=1.14$). This means that, on average, the boxes tested had 14% more strength than the standard strength of 5,000 psi used, which is practical.

It was found that the precast concrete box culvert of size 1.22 m (4 ft) span and 1.22 m (4 ft) rise with up to 0.62 m (2 ft) height of fill exhibited shear strength between 1.55 and 2 times the factored design live load per AASHTO (2005).

Fig. 10 presents the experimentally obtained load versus maximum deflection plots for the boxes tested. This figure shows that for all the test specimens without the bedding materials, the maximum deflections at the service limit state is less than $L/800$ [i.e., 1.52 mm (0.06 in.)] as prescribed by AASHTO (2005). For the test with bedding material (i.e., Test 1: S-SB-444-WB-5) the value of maximum deflection is greater than $L/800$. This is attributed to the fact that the effect of the lateral soil pressure, which reduces the culvert's deflection, is unaccounted for due to the fact that the culvert was not buried in the soil during the testing, and only the bedding material was placed under the culvert. This was merely done to identify the effect of the culvert's differential settlement on its general crack initiation and propagation.

Acknowledgments

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Notation

The following symbols are used in this paper:

B = bell end;
DB = double box culvert;
NB = no bedding;
P = distance from tip of right haunch to center of load plate (in.);
S = spigot end;
SB = single box culvert;
SRL = dimension of culvert [span (ft), rise (ft), and joint length (ft)]; and
WB = with bedding.

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