Behavior of ASTM C 850 Concrete Box Culverts Without Shear Connectors

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ABSTRACT

Two ASTM C 850 7 x 5 reinforced-concrete box culvert sections were fabricated, assembled without shear connectors, and loaded on both the 5- and 7-ft spans with simulated service and ultimate wheel loads. The load was applied at three points along the culvert centerline on either side of the joint and at one unsupported edge. Measured deflections and reinforcing steel strains are compared with predicted deflections and moments. Crack patterns are also reported. It was found that the 7 x 5 box design is conservative, and that live-load stresses and deflections caused by design service wheel loads are acceptable without shear connectors.

Precast box culverts have been extensively used to economically span smaller drainage channels. Currently, design standards exist for two categories of box culverts: ASTM C 850 establishes standard designs for boxes with less than 2 ft of cover subject to highway loadings, whereas ASTM C 789 establishes standard designs for other precast box sections. The C 850 standard requires shear connectors between top slabs of adjacent box sections, a requirement that is also adopted by AASHTO.

CURRENT DESIGN STANDARDS

ASTM C 850

ASTM Standard C 850 (1) establishes design standards for precast concrete box culverts to be used with less than 2 ft of cover. The standard specifies minimum steel and concrete strengths, areas, and geometries for 42 standard culvert sizes subject to two loadings. The design criteria, computer programs, and standard designs are based on studies and tests sponsored by the American Concrete Pipe Association, the Virginia Department of Highways and Transportation, and the Wire Reinforcement Institute (2,3). The required transverse steel areas are based on two-dimensional computer solutions that use several simplifying assumptions, including the following assumptions regarding wheel load distribution:

1. Wheel loads are distributed parallel to span over a length equal to 8 in. + 1.75 H, where H is the height of soil cover (in.); and
2. The effective width of the top slab resisting wheel load is taken to be 48 in. + 0.06 (SPAN-HAUNCH).

Figure 1 shows the reinforcement detail and cross-section geometry for a C 850 box section. Figure 2 shows the assumed simplified wheel loading specified for the two-dimensional analysis and design method.

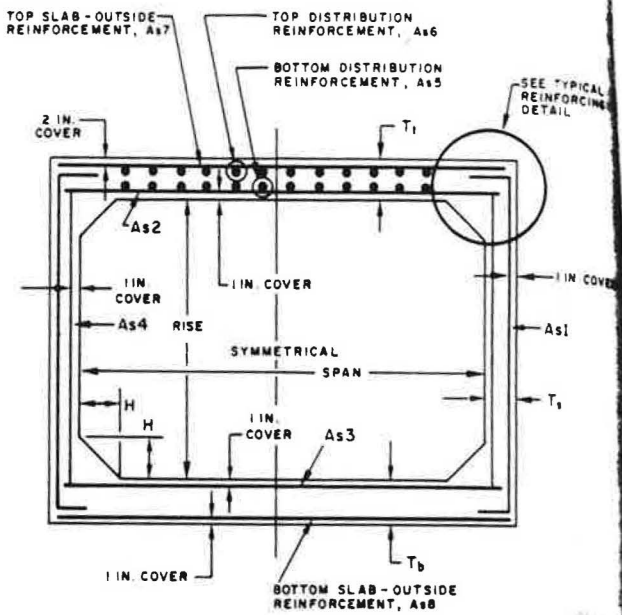


FIGURE 1 ASTM C 850 concrete box section reinforcing steel.

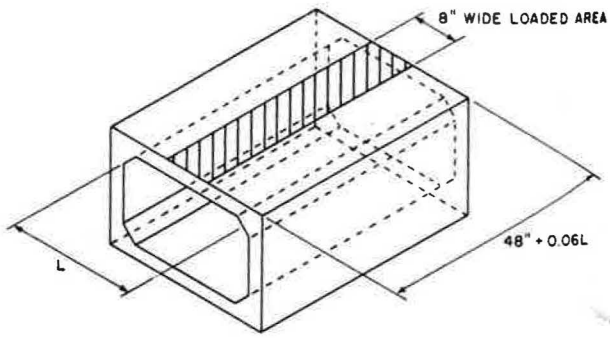


FIGURE 2 ASTM C 850 design wheel load distribution, no cover.

The design procedure used to develop the standard box sections (2) limits the crack widths at service load to 0.010 in. by limiting the design service steel stress to a value given by

$$f_s = (65/3\sqrt{t_b^2\, s_\ell}) + 5(\text{ksi}) \quad (1)$$

where t_b is the distance from the centroid of the tension steel to the outermost concrete tension fiber (in.), and s_ℓ is the spacing of the longitudinal reinforcing steel wires (in.). This equation is based on studies by Lloyd et al. (4) and subsequent criteria developed by Gergely and Lutz (5), and is a more conservative limitation than the American Concrete Institute (ACI) crack control criteria (6) and the AASHTO crack control criteria (7); but the stress allowed by this limitation may be greater than the AASHTO fatigue stress limitation of 21 ksi (8) or the ACI allowable service load stress of 36 ksi (7). For example, the two 7 x 5 boxes tested here have t_b = 1.16 in. and s_ℓ = 2.0 in.; thus the maximum steel stress to limit cracking is approximately f_s = 52 ksi.

In addition, ASTM C 850 specifies that the joint provide a smooth interior free of appreciable irregularities, and that the joint be designed or modified to transmit a minimum of 3,000 lb of vertical shear force per foot of top slab joint. Shear connectors used to satisfy this requirement must be spaced no more than 30 in. on center and with a minimum of two connectors per joint. These requirements are intended to provide continuity of shears and deflections across joints to reduce culvert stresses when loaded near a joint and minimize relative displacements of culvert and cover.

AASHTO Standard Specification for Highway Bridges

The AASHTO specifications (7,8) include minimum requirements for design and methods of analysis for highway bridge structures, including culverts. These requirements are essentially satisfied by the ASTM C 850 standard. At longitudinal edges of reinforced-concrete slabs, AASHTO 1.3.2(D) (8) requires an edge beam, additional reinforcement in the slab, or an integral reinforced section of slab and curb. Because edge beams and curbs are not acceptable, and to provide continuity of deflections as well as shears, ASTM C 850 8.2 specifies that shear connectors be used to transmit the calculated shear across joints between culvert segments. This requirement is also adopted by AASHTO 1.15.7(D)(4) (7).

THEORETICAL ANALYSIS

Predicted Internal Moments

The FORTRAN code SLAB 49 (9) was used to predict the internal moments in the top slab of the model shown in Figure 3. For simplicity, the slab is assumed to be isotropic, neglecting the difference in distribution and flexural steel areas. SLAB 49 uses discrete elements that simulate linear, small deformation plate behavior. A 2.0 x 2.0-in. mesh size was used. The symmetric boundary conditions along the centerline were approximated with zero vertical restraint and essentially infinite rotational restraint along the centerline. Edge support at the side wall was approximated by a simple support and an elastic rotational restraint simulating the rotational stiffness of the uncracked side wall. Membrane reactions and forces were neglected, which is consistent with linear plate theory simplifications.

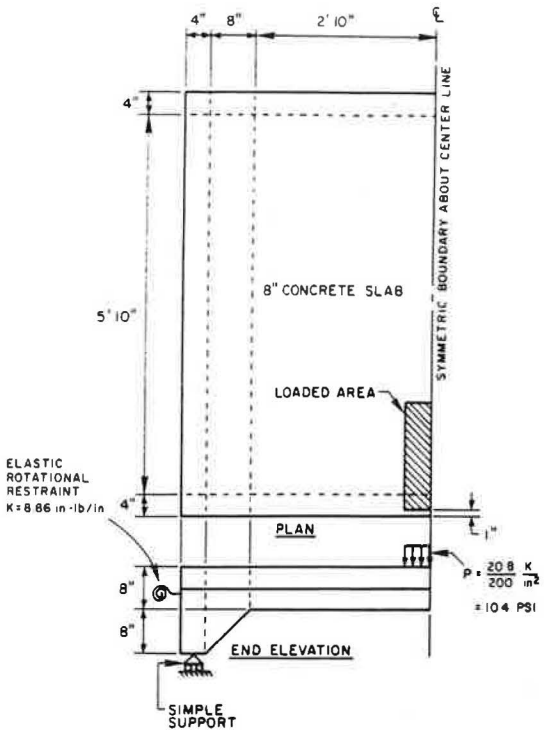


FIGURE 3 SLAB 49 model of 7-ft top slab.

The predicted internal moments are shown in Figure 4 for two basic plate stiffnesses. The result labeled "uncracked" is the predicted moment distribution, which assumes that the stiffness is equal to that of an uncracked 8-in.-thick concrete plate, thereby neglecting the reinforcing steel that would change the stiffness by only approximately 6 percent. The result labeled "cracked" is the predicted moment distribution neglecting the contribution of concrete in the tensile region. The stiffness of the concrete is reduced in the span between the haunches because the moment capacity of the haunched region is greater than the capacity of the 8-in. slab, and because joint rotations will reduce the stress at the haunches below that in the slab. The change in wall stiffness caused by cracking or interaction with adjacent sections was not modeled. It is noteworthy that although the cracked stiffness is only 23 percent of the uncracked stiffness, the predicted maximum flexural moments are not significantly different. In addition to predicted moments, vertical deflections are also predicted, and Figure 5 pre-

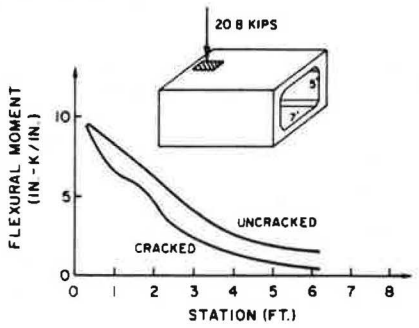


FIGURE 4 Predicted top slab flexural moments, centerline of 7-ft slab.

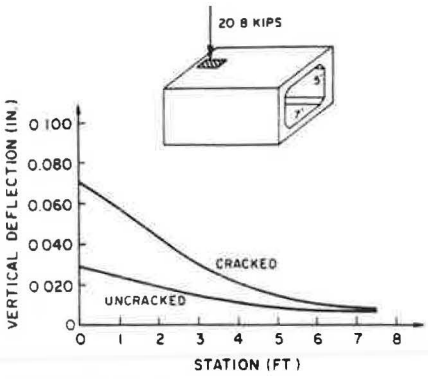


FIGURE 5 Predicted vertical deflections, centerline of 7-ft slab.

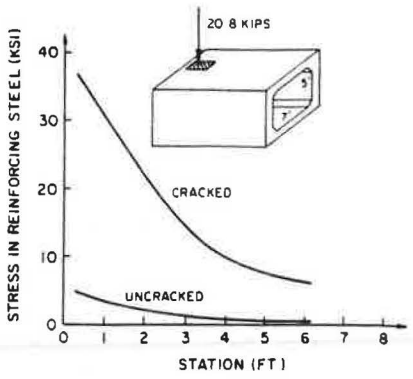


FIGURE 6 Predicted reinforcing steel stresses, centerline of 7-ft slab.

sents the predicted top slab vertical deflection along the culvert centerline.

Predicted Steel Stresses

Steel stresses are calculated from predicted slab flexural moments as follows: By assuming isotropic elastic plate behavior, the flexural stress in the reinforcing steel is

$$f_s = nM_1 y/I_1 \quad (2)$$

where

$n = E_s/E_c$ = modular ratio;
M_1 = flexural moment per unit length;
y = distance of the tension steel from the neutral surface, and
$I_1 = h^3/[12(1 - \nu^2)]$ = moment of inertia per unit width for a slab of thickness h and Poisson's ratio ν.

By using $n = 7.44$, $h = 8.0$ in., $\nu = 0.15$, and $y = 3.0$ in., the moment of inertia per unit width becomes $I_1 = 43.6$ in.^{3}, and the steel stress is given by $f_s = M_1/1.96$ in.^{2}. This calculation is based on the assumptions that the stresses are linearly distributed, and that the reinforcing steel areas may be neglected in computing neutral surface location and moment of inertia.

If cracking occurs the concrete stresses can be assumed to be nonzero in the compression region only, with the resultant tensile force provided entirely by the tension steel. By using this assumption, the calculated equivalent concrete section has a depth of $c = 1.765$ in., and the equivalent concrete section moment of inertia is approximately $I_1 = 10.22$ in.^{3}. The distance from the neutral surface to the tension steel is approximately $y = 5.235$ in., and the resulting relation between flexural stress and moment becomes $f_s = M_1/0.262$ in.^{2}.

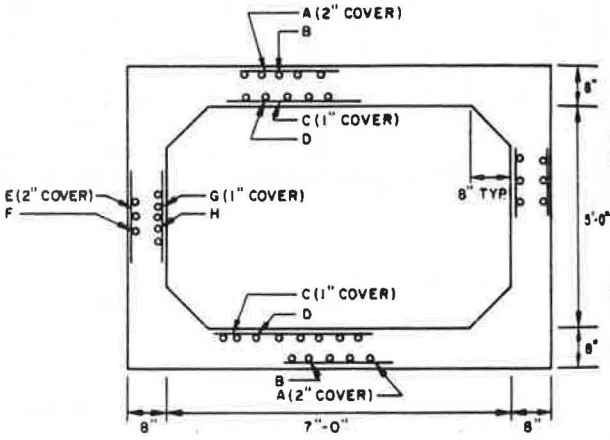
Steel stresses calculated from predicted top slab moments are shown in Figure 6. Although the predicted maximum moments in the uncracked and cracked section models differ by only approximately 2 percent, the predicted maximum steel stress in the cracked section is approximately 7.5 times the predicted steel stress in the uncracked section.

The maximum predicted steel stress of 37 ksi in the cracked section exceeds the 21 ksi AASHTO fatigue limit stress and the 36 ksi service load limit stress of ACI, but is less than the crack control limit stress for this geometry used in the ASTM C 850 design procedure.

EXPERIMENTAL PROCEDURE

Test Sections

Two 7 x 5 precast concrete box culverts were fabricated at the Gifford-Hill and Company plant in Ft. Worth, Texas. The geometry is described in Figure 7. The design and materials met ASTM C 850 minimum requirements, with the exception of reinforcement area A_{s1}. Standard 5 x 5 box requirements for A_{s1} were given precedence over 7 x 5 requirements for A_{s1}. The steel areas were in accordance with C 850, except for the 5-ft slabs, which were more heavily reinforced in order to approximately simulate behavior of standard 5 x 5 box culverts by using the same 7 x 5 specimens rolled 90 degrees (see Table 1). The measured concrete compressive strength was 5,725 which exceeds the design compressive strengt 5,000 psi. The reinforcing mesh is grade 65 (65 ksi yield strength). According to ASTM C 850, the yield



WIRE	SIZE	AREA (in^2/ft.)
A	W5.0 x 3	0.20
B	W4.8 x 3	0.19
C*	W8.0 x 2	0.48
D	W4.8 x 3	0.19
E	W4.8 x 3	0.19
F	W4.8 x 3	0.19
G*	W8.0 x 2	0.48
H	W5.5 x 3	0.22

*INSTRUMENTED WIRES

FIGURE 7 Geometry and reinforcement schedule for test specimens.

TABLE 1 Comparison of Test Specimen Reinforcing Steel Schedule with ASTM C 850 Specification

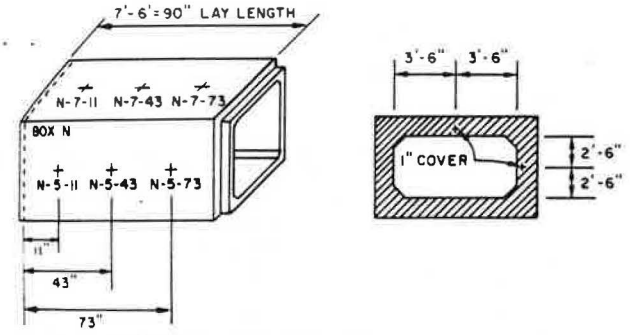
Reinforcement	C 850 7 x 5 HS20 (in.²/ft)	Required Area, Interstate (in.²/ft)	As Built Area (in.²/ft)
7 x 5 Test Configuration (7-ft Span)			
A_{s1}(E)	0.29	0.30	0.19
A_{s2}(C)	0.48	0.48	0.48
A_{s3}(C)	0.27	0.34	0.48
A_{s4}(G)	0.19[a]	0.19[a]	0.48
A_{s5}(A)	0.20	0.20	0.20
A_{s6}(a)	0.19[a]	0.19[a]	0.20
A_{s5}(D)	0.19[a]	0.19[a]	0.19
A_{s6}(B)	0.19[a]	0.19[a]	0.19
5 x 7 Test Configuration (5-ft Span)			
A_{s1}(A)	0.16	0.21	0.20
A_{s2}(G)	0.46	0.46	0.48
A_{s3}(G)	0.26	0.34	0.48
A_{s4}(C)	0.14[a]	0.14[a]	0.48
A_{s7}(E)	0.19[a]	0.19[a]	0.19
A_{s8}(E)	0.17[a]	0.17[a]	0.19
A_{s5}(H)	0.22	0.22	0.22
A_{s6}(F)	0.19[a]	0.19[a]	0.19

[a]Minimum reinforcement area is specified.

strength is taken to be 60 ksi for purposes of analysis.

Instrumentation

The culverts were instrumented with strain gauges bonded to the 8 gauge main transverse reinforcing steel wires in theoretical maximum tensile stress regions. Strain gauge locations are described in Figure 8. Six gauges were installed in each culvert; however, 2 gauges were damaged during placement of concrete, leaving 10 serviceable strain gauges.



NOTE: "N" DENOTES BOX NUMBER 1 OR 2.
GAUGES 1-5-11 AND 1-7-11 WERE DAMAGED DURING CONCRETE PLACEMENT

FIGURE 8 Strain gauge locations and nomenclature.

Installed gauge resistance was checked at the time of installation, and gauge isolation resistance was measured after testing had been completed and after semi-destructive measurements of concrete cover had been made. Measured gauge isolation resistances were approximately 150 M Ω or greater, which indicates acceptable isolation (10,11) at all but three gauges (see Table 2). Gauges at stations 1-7-42, 2-7-11, and 2-7-43 all indicated unacceptably low gauge isolation resistances. Strain gauges at critical stations 1-5-73, 1-7-73, 2-5-73, and 2-7-73 all indicated open circuit gauge isolation resistance with the analog ohmmeter used, which can detect resistances less than 150 M Ω. The strain gauge at station 1-5-42 had a marginal isolation resistance.

TABLE 2 Measured Strain Gauge Isolation Resistance to Reinforcing Steel

Gauge Designation	Gauge Isolation Resistance[a]
1-5-11	NA
1-5-42	>100 M Ω
1-5-73	∞[b]
1-7-11	NA
1-7-42	20 M Ω
1-7-73	∞
2-5-11	∞
2-5-43	∞
2-5-73	∞
2-7-11	50 M Ω
2-7-43	11 M Ω
2-7-73	∞

Note: Data taken in March 1983, 6 months after testing.

[a]Using analog ohmmeter, resolution 100 M Ω.

[b]∞ reported resistance can be interpreted as greater than approximately 500 M Ω.

In addition to the resistance strain gauges installed on the reinforcing steel, the culverts were instrumented with deflection dial indicators to measure vertical deflection at three of the top surface strain gauge locations.

Test Procedure

After curing, the culverts were transported to the test site and assembled in the fixture, as shown schematically in Figure 9. Concentrated loads were applied to the top surface of the culvert through a 1 x 10 x 20-in. steel bearing plate and 0.5-in. neoprene pad (Figure 9). The lower surfaces of the culverts rested on 0.5-in. plywood sheets over doubled 0.75-in. rigid foam thermal insulating panels that rested on the steel reaction frame bed.

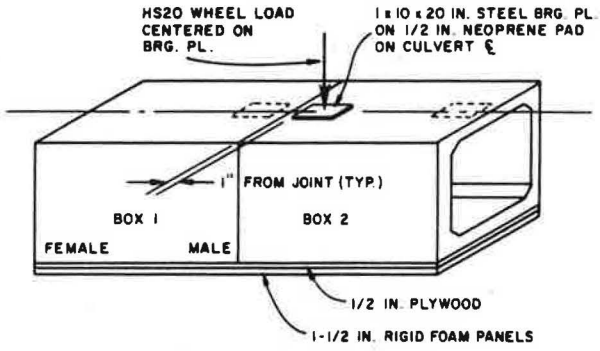


FIGURE 9 Test configuration schematic.

The culvert sections were aligned and fitted together snugly without grout, joint filler material, or shear transfer connectors. The fit of the top slab joint was qualitatively evaluated by inserting a sheet of paper through the top slab joint. The paper could completely penetrate the joint at several places, but interference between the two faces prevented drawing the paper along the length of the joint. The visible joint was generally of uniform width, with no significant variations.

The reported test loads represent HS20-44 16-kip wheel loads multiplied by a 1.3 impact factor specified by AASHTO for a design service load of 20.8 kips and a 16-kip wheel multiplied by factors 1.3 x 1.67 x 1.3 = 2.82 for a design ultimate factored load of 45.2 kips. The 78.0-kip load reached in test 9 represents the limiting load on the test fixture compression members, which indicated impending lateral instability. The testing schedule is given in Table 3.

TABLE 3 Actual Test Schedule

Date	Test No.	Test Configuration Code[a]	Maximum Test Load (kip)	Repetitions
7-29-82	1	7F2	20.8	2
7-29-82	2	7M1	20.8	2
8-17-82	3	5F2	20.8	3
8-17-82	4	5M1	20.8	3
8-17-82	5	5M1	45.2	2[b]
8-18-82	5A	5M1	45.1	3
8-18-82	6	5F2	45.2	3
8-19-82	7	7F2	45.7	3
8-19-82	8	7M1	45.5	3
8-19-82	9	7M1	78.0	1
8-20-82	10	7M2	45.5	3

[a]The first digit in the test configuration code denotes the span in feet, the letter M or F refers to the male or female end, and the second digit refers to box 1 or 2.

[b]Replaced 0.5-in. bearing plate with 1-in. plate.

TEST RESULTS

Measured Stresses

Figure 10 shows measured steel stresses for test configurations 7M1, 7F2, and 7M2 for test loads of 20.8 and 45.2 kips. Critical loading occurs in test 7M1, and maximum steel stress for the design service loading of 20.8 kips is 6.4 ksi in the top slab steel at gauge location 1-7-73. For the design ultimate load of 45.2 kips, the maximum steel stress is 17.2 ksi.

Figure 11 shows measured steel stresses for test configurations 5M1 and 5F2 and for test loads of 20.8 and 45.2 kips. Critical steel stresses occur in test configuration 5M1; however, stresses are lower than measured stresses in tests of the 7-ft span.

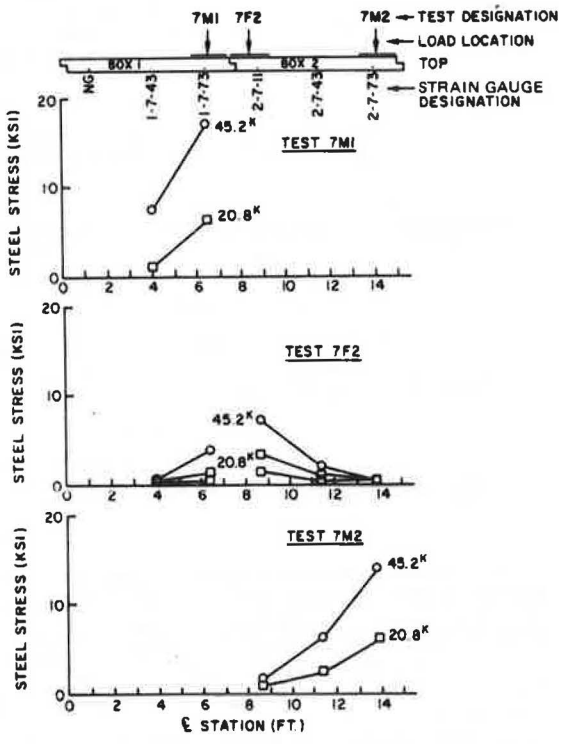


FIGURE 10 Measured reinforcing steel stresses, centerline of 7-ft slab.

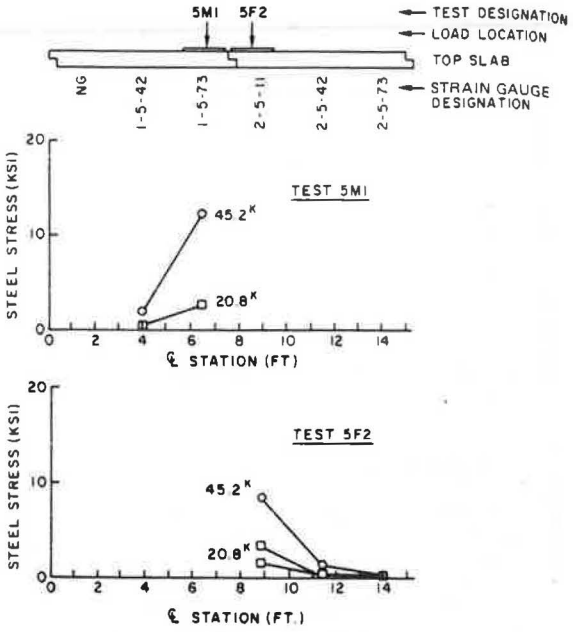


FIGURE 11 Measured reinforcing steel stresses, centerline of 5-ft slab.

Measured Deflections

Measured top slab deflections are shown in Figure 12 for test configurations 7M1, 7F2, and 7M2 and for test loads of 20.8 and 45.2 kips. Test configuration 7M1 is critical with respect to maximum absolute deflection and relative deflection across the joint. Maximum observed absolute deflection in test 7M1 was 0.021 in. at the service load (20.8 kips) and 0.04 in. at the design ultimate load (45.2 kips). Maximum observed relative deflection across the joint in test 7M1 was 0.021 in. at the service load and 0.04 in. at the design ultimate load. (Point 2-7-11 was observed to move upward approximately 0.002 in.)

Measured top slab deflections are shown in Figure 13 for tests 5M1 and 5F2 for test loads of 20.8 and 45.2 kips. Maximum absolute and relative deflections occur in configuration 5F2. In test configuration 5F2 the maximum observed absolute and relative deflection was 0.013 in. at the service load. At the design ultimate load, the observed absolute deflection was 0.036 in., and the observed relative deflection across the joint was 0.036 in.

Discussion of Results

For test loads of 20.8 kips, which represent the design service wheel load, all measured steel stresses are well below the C 850 live-load fatigue stress of 21.0 ksi. The maximum measured stress is 6.4 ksi in configuration 7M1.

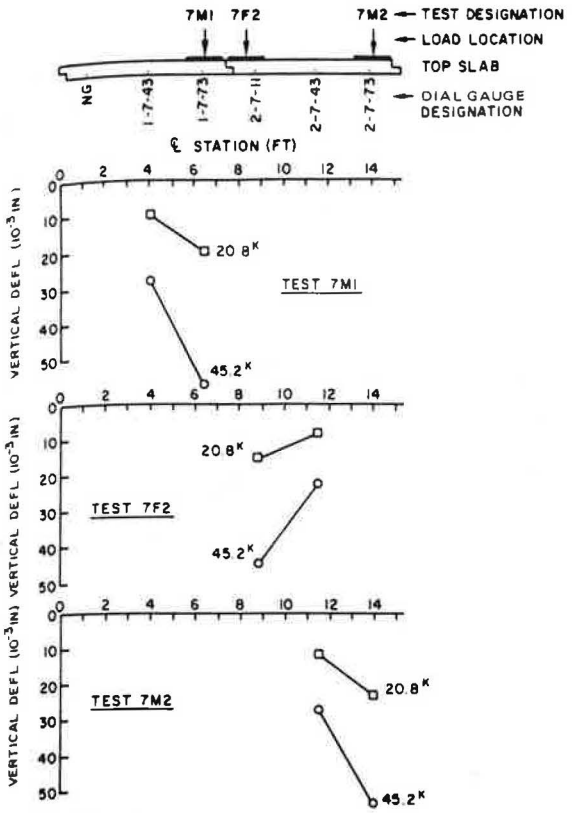


FIGURE 12 Measured vertical deflections, centerline of 7-ft slab.

For test loads of 45.2 kips, which represent the design ultimate wheel load, all measured steel stresses are well below the C 850 ultimate total load yield stress limitation of 60 ksi. The maximum measured steel stress is 17.2 ksi in configuration 7M1. Dead-load stresses have not been calculated here.

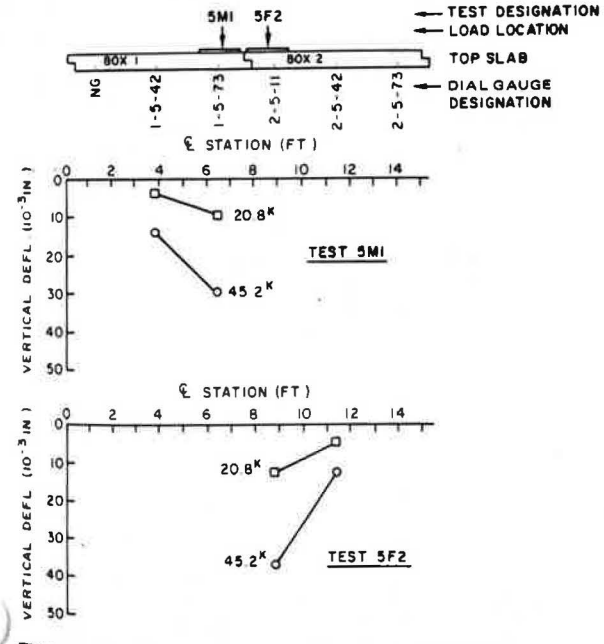


FIGURE 13 Measured vertical deflections, centerline of 5-ft slab.

Significant load transfer across the joint is obvious in the data from test configuration 7F2 only. Load transfer occurs apparently through contacting irregularities in the dry joint. Stress data for test configuration 5F2, in which load transfer is also possible, does not indicate any significant load transfer, although deflection data does indicate some minor load transfer is occurring; the observed deflection of gauge station 1-5-73 is about 0.001 at design ultimate load (45.2 kips). As expected, no load transfer across the joint is observed in test configurations 7M1 or 5M1. Although maximum measured stresses occurred at gauge locations 1-7-73 and 1-5-73, maximum deflections occurred at stations 1-7-73 and 2-5-11. The deviations in actual cover from the design 1.0-in. cover may be the cause of this observation. The observed cover at station 2-5-11 is 2.24 in., and the corresponding uncracked moment of inertia at that station could be as much as 43 percent less than the uncracked section modulus corresponding to the design 1.0-in. cover. Deflections at this station are therefore expected to be somewhat larger. In addition, the stiffness at stations 2-7-11 and 2-5-11 is expected to be less, neglecting shear interaction across the joint, than the stiffness at stations 1-7-73 and 1-5-73, respectively, because of joint geometry. Both stations are located 11 in. from the joint line, but the male connection of stations 1-5-73 and 1-7-73 has more concrete outboard of the point of load application than does the female connection at stations 1-5-11 and 1-7-11. The vertical deflection in configuration 7F2 is less than the deflection in configuration 7M1 because of the significant shear transfer that occurs in configuration 7F2. Without significant shear transfer in configuration 5F2, the deflection is slightly greater than that of configuration 5M1 for the reasons previously given.

A comparison of the measured stresses shown in Figure 10 and the predicted stresses shown in Figure 6 suggests that the top slab is behaving essentially as an uncracked section. Measured maximum service load steel stresses are approximately 6.1 to 6.4 ksi, approximately 75 percent greater than the predicted steel stress of 3.6 ksi assuming the section is uncracked, and well below the approximately 32 ksi predicted steel stress in the cracked section. Measured vertical deflections shown in Figure 12 more closely agree with predicted uncracked section deflections than predicted cracked section deflections shown in Figure 5.

Interaction from adjacent box sections in the tests, a factor that was not modeled in the numerical solutions, is not thought to be a significant factor in comparison of theory with test results. Test 7M2 was conducted without any contact by an adjacent box, and the predicted deflections for the uncracked slab agree well with measured deflections.

Another potential source of measurement error is the variation in actual cover from the specified 1.0-in. cover (Table 4). With the exception of the strain gauge at station 2-5-11, the deviations in cover do not significantly affect the conclusions. The theoretical section modulus at the tension steel for specified 1.0-in. cover was calculated earlier as 1.96 and 0.262 $in.^3/in.$ for uncracked and cracked sections, respectively. By using the tabulated values for actual cover from Table 4, and excluding the gauge at station 2-5-11, these section moduli can be shown to deviate no more than +2 percent, -21 percent (uncracked) and +8 percent, -1 percent (cracked) from this value. The measured stresses more closely agree with predicted uncracked section stresses.

Cracking was not observed in the top slab at the service load. The first observed crack in the 7-ft slab appeared at a test load of 27 kips. Two other

TABLE 4 Measured Concrete Cover

Strain Gauge Designation	Measured Cover (in.)	Design Cover (in.)
1-5-73	1.387	1.00
1-7-73	0.950	1.00
2-5-11	2.240	1.00
2-7-11	1.375	1.00
2-7-73	1.600	1.00

flexural cracks opened at test loads of 50 and 55 kips, respectively. These three cracks were the only observed cracks that have planes that might intersect the instrumented tension steel. The width of the central crack was measured with a graduated reticle at various loads. The observed crack widths were 0.010 in. at 50 kips and 0.013 in. at 60 kips. The field sketch of observed cracks is shown in Figure 14.

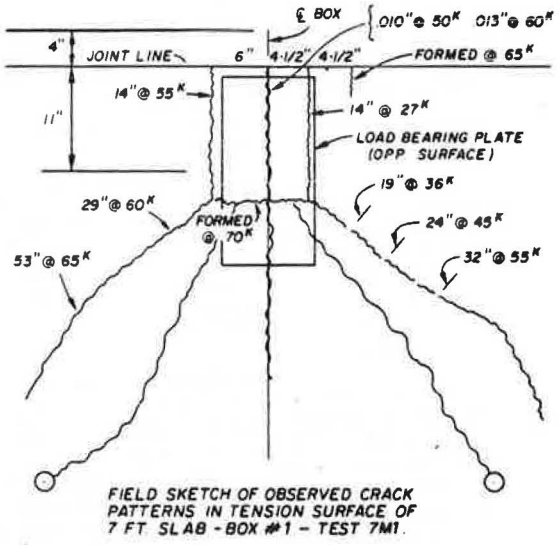


FIGURE 14 Field sketch of observed crack patterns.

The effects of the progressive cracking are apparent in Figure 15, which presents steel stresses and vertical deflection histories at gauge station 1-7-73 during repeated tests in configuration 7M1. The steel stress per unit load increases with repeated testing, apparently because crack development causes a change in the neutral surface location and a reduction in the effective moment of inertia. The stiffness of the top slab is also reduced for the same reason. The observed effects of the cracking are still significantly less than would be expected if the section is assumed to be fully cracked, according to the design philosophy of the ACI Building Code Requirements for Reinforced Concrete (6). This is interpreted as an indication that the observed cracking at the strain gauge section is not fully developed, in spite of the large overload applied, and that the fully cracked section design philosophy is overly conservative when applied to the reinforced slab with the concentrated load considered here.

The limiting crack width of 0.10 in. was observed at a test load of 50 kips. The maximum measured steel stress at that load was approximately 21.7 ksi, which is considerably less than the 43 ksi

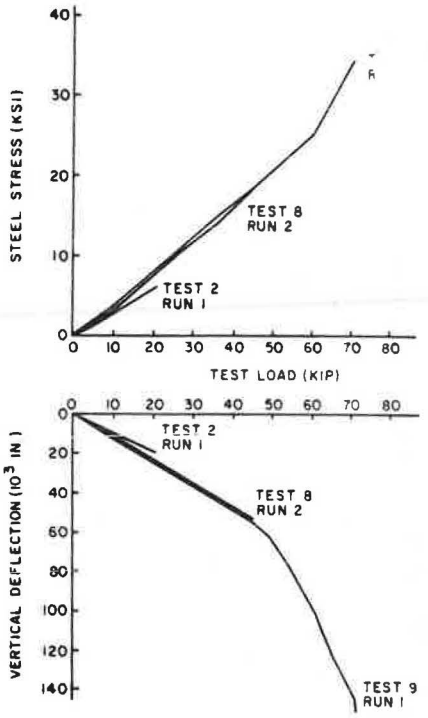


FIGURE 15 Repeated load deflection data, station 1-7-73, test configuration 7M1.

limiting stress given by Equation 1. The strain gauge station is close to the observed crack plane shown in Figure 14.

HS20 AXLE LOAD STRESSES

The live-load stresses caused by two wheels of an HS20 axle can be approximated by the superposition of measured wheel load stresses. Because the load locations of tests 7F2 and 7M2 are approximately 5 ft 8 in. apart, superposition of measured steel stresses in these two tests will allow a conservative approximation of steel stresses caused by two HS20 wheels spaced 6 ft 0 in. By such superposition the maximum steel stress expected under a design 32.0-kip axle with a 1.3 impact factor is approximately

6.1 ksi + 0.3 ksi = 6.4 ksi.

This maximum stress occurs at gauge station 2-7-73 the male joint end of the culvert.

The maximum steel stress due to a design ultimate axle load of 90.4 kips is

14.0 ksi + 0.4 ksi = 14.4 ksi.

CONCLUSIONS

The following conclusions are drawn from the results

1. Maximum reinforcing steel stresses in ASTM C 850 7 x 5 box culverts subjected to a design wheel load of 20.8 kips are significantly less than AASHTO 1.5.26(B) design allowable service stress of 24 ksi. The maximum steel stress measured was approximately 6.4 ksi.
2. Maximum reinforcing steel stresses in ASTM C 850 7 x 5 box culverts subjected to a design ultimate

wate wheel load of 45.2 kips are significantly less than the design yield strength of 60 ksi. The maximum measured steel stress was approximately 17.2 ksi.

3. Cracking caused by the application of the design ultimate wheel load is relatively insignificant with respect to cracking in a fully cracked section condition specified by the ACI design criteria.

4. Relative deflections of adjacent spans, in the absence of shear connectors, are relatively small: less than approximately 0.020 in. at design service wheel loads.

Live-load stresses caused by other forces and dead-load stresses have not been investigated.

RECOMMENDATIONS

The following recommendations are offered.

1. ASTM C 850 size 7 x 5 reinforced-concrete box sections appear to be conservatively designed due in part to design assumptions and simplifications regarding load distribution. A three-dimensional analysis and experimentally measured stresses support the use of these boxes without shear connectors.

2. A field trial of a C 850 box culvert installed without shear connectors is recommended. Sufficient instrumentation should be installed to verify the presented test results for 7 x 5 boxes or, in the case of boxes of other sizes, to extend the test results. Particular attention should be given to absolute and relative deflection measurements and long-term crack pattern observations.

3. The results of the present study, and future test results, should be presented for consideration to the AASHTO Rigid Culvert Liaison Committee, to the AASHTO Bridge Committee, and to ASTM Committee C-13.

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