Load Test Report and Evaluation of a Precast Concrete Arch Culvert

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A report on and an evaluation of a full-scale load test performed on a Con/Span culvert are presented in this paper. Because of its intended use, it was important that this three-sided box-arch shape's field performance be evaluated extensively, in addition to rigorous theoretical analysis. A load test procedure was devised to evaluate the structural integrity of this unit and to examine to what extent its field performance compared with its predicted behavior. The finite element program CANDE was used for the detailed analysis of the structure. CANDE is a program especially written for the evaluation of soil-structure interaction conditions, and is ideally suited to evaluate the effect of the conditions that exist in field testing. The evaluation revealed a good correlation between the performance of the culvert and the finite element analysis used. Perhaps more impressive was the culvert's capacity to sustain an extreme overload.

Con/Span culverts were developed to meet a need for precast reinforced concrete culverts with large cross-sectional areas for water conveyance at sites with limited vertical clearance. Because of their great width compared with their height and because of the inherent durability characteristics of concrete, these culverts provide an economical design solution for short-span bridge replacements. The box-arch culvert can be made in a number of span and rise combinations. The geometric properties of the 19-ft span culvert studied is shown in Figure 1.

DESCRIPTION

The unique combination of vertical sidewalls and the arch top not only enhance the hydraulic and aesthetic values of the culvert but also greatly increase its load-carrying capacity. This increase in load-carrying capacity is perhaps most effectively shown by examining Figure 2.

In the arch-box when the culvert begins to deflect, thrust is developed by the passive pressure of the earth backfill counteracting the efforts of the applied loads to deflect the top of the structure. In a state of extreme overload, the arch-box cannot collapse without pushing the block of soil behind the sidewalls far enough to allow the arch to collapse. Hinges will form in the culvert but the units will still be a viable structure with the pressure from the backfill providing the necessary support. The dependence on the backfill is not nearly so critical under normal design conditions. For an actual installation, the units are designed according to American Association of State Highway and Transportation Officials (AASHTO) specifications, which require that the culvert be loaded using ultimate loads and the reinforcing steel not be permitted to yield. Based on this practice and the structure's inherent strength, it is easy to see the large reserve capacity built into these culverts.

Another significant contribution to the structural advantages of the box-arch shape is its resistance to shear. Because of the thrust and the arch shape, shear from the vertical loading is greatly reduced in a section. This allows the unit to maintain its standard 10-in. thickness under much deeper fills than normally considered for such a lightweight section. This issue is illustrated in Figure 3.

A flat slab with the same span and loading would have a shear value ($V = WL/2$). Obviously the effectiveness of this shear reduction relies on the radius of the arch, but even with a flat arch the reduced shear values and thinner sections are quite advantageous.

The behavior of the culvert is therefore dependent to a limited degree on its interaction with the surrounding backfill. The backfill restrains the tendency of the sides of the culvert to flex outward. This restraint develops a thrust in the curved top of the unit that creates arch action to increase its capacity to carry vertical loads. This interaction of the structure and soil can be simulated with a computer model to allow a reliable and realistic basis for design.

The design of the reinforced concrete culvert is based on the concept of soil-structure interaction and is modeled by using the finite element method of analysis. The finite element computer program called CANDE (Culvert Analysis and Design) provides the computer model to analyze the behavior of the arch structure during various loading situations. CANDE permits analysis of the culvert beyond conventional elastic analysis into the plastic range. The analysis is performed in an iterative manner, beginning with the structure resting on its foundation with no backfill. Placement of the first layer of backfill alongside the culvert is modeled by adding the first layer of soil elements and loads to the finite element mesh. Through their interaction, the soil elements load the structure. Subsequent steps of the analysis are performed in the same way, adding one layer of elements at a time, simulating the process of backfilling around and over the culvert. After the final layer of fill has been placed over the top of the structure, loads are applied to the surface of the fill to simulate vehicular traffic loads.

Purpose

The purpose of the load test was to verify the validity of the modified CANDE computer program of analysis to model the actual behavior of field-installed box-arch culverts.

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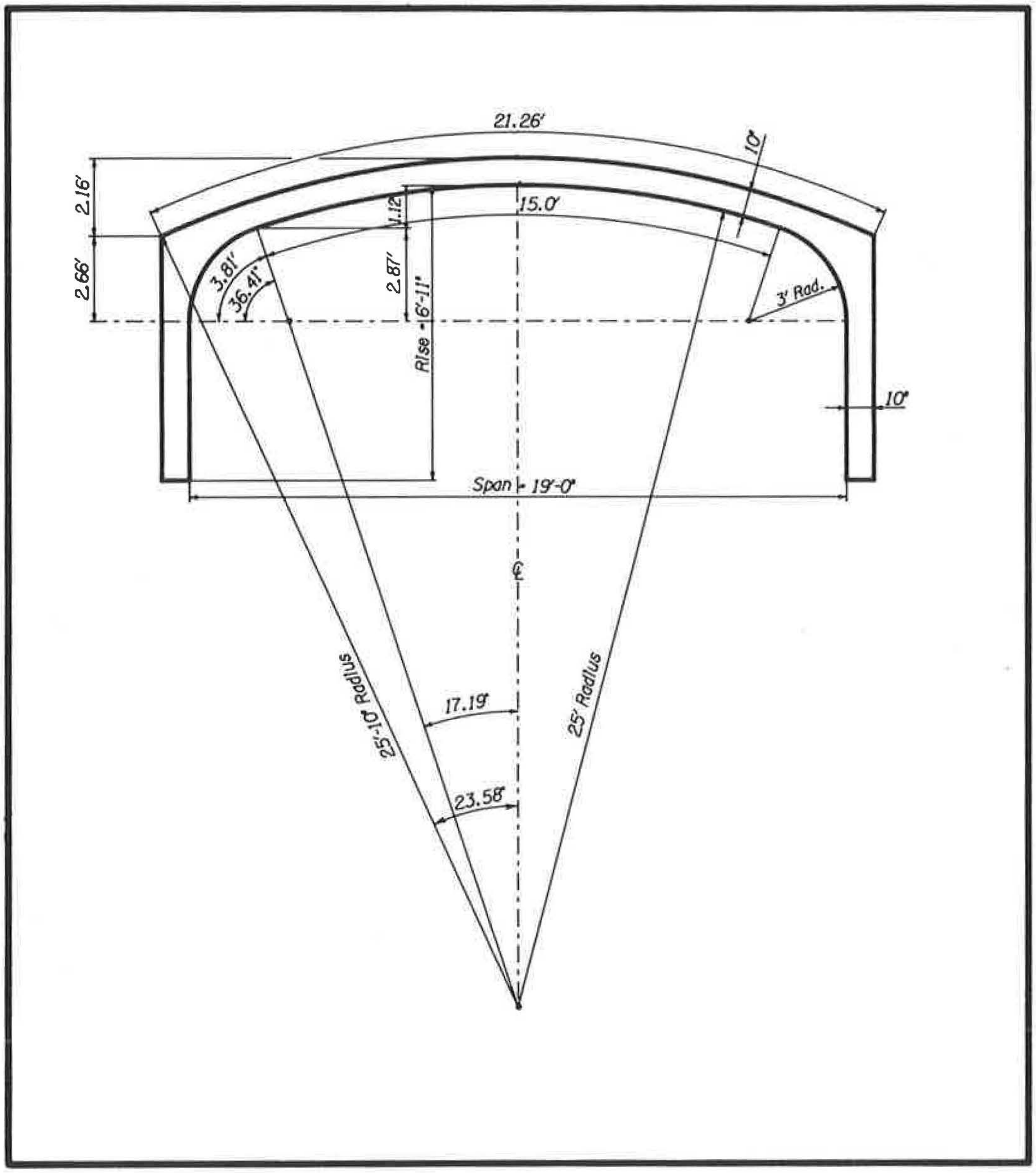

FIGURE 1 Culvert dimensions.

Scope

The test involved installing three 8-ft laying length culverts on a cast-in-place slab. After the backfilling process was complete, the middle culvert was tested. The test culvert was instrumented with deflection gauges. External loads were applied evenly (at mid-span) across the 8-ft laying length of the test culvert. As loads were applied, deflection readings were recorded. The loads applied were as much as five times the HS20 design service loading without impact.

By applying loads that greatly exceed the design loading, appreciable deflections and cracks occurred. After the test was complete, the test unit's actual section properties (com-

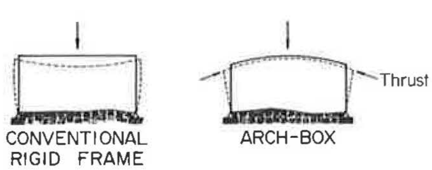

FIGURE 2 Theoretical deflected shapes.

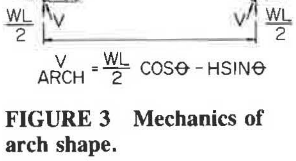

FIGURE 3 Mechanics of arch shape.

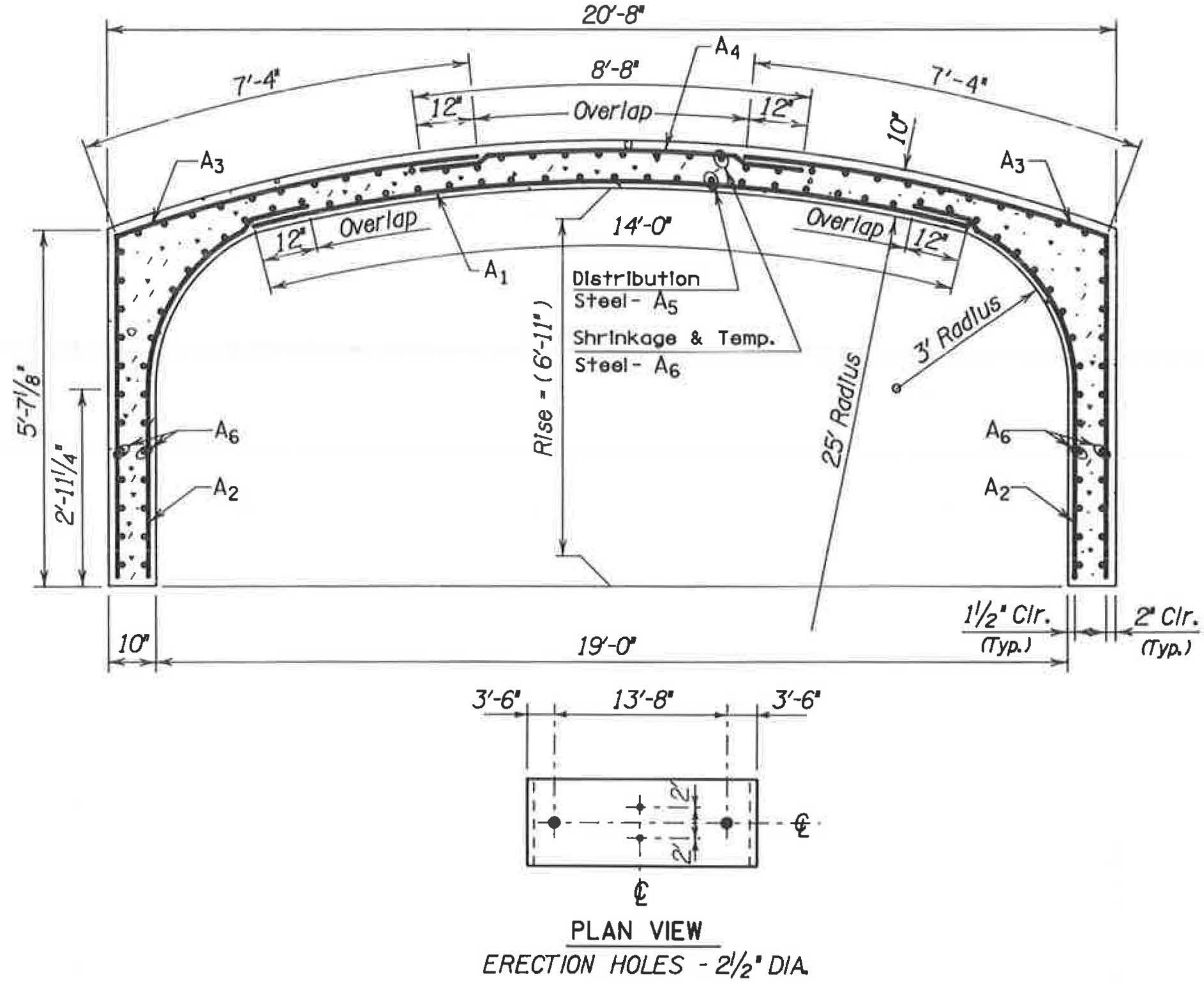

FIGURE 4 Cross section of test unit.

TABLE 1 REINFORCING AREAS FOR TEST UNITS

SPAN: (19'-0")	RISE: (6'-11")		COVER: (1'-0")	
Sheet No.	Area Req'd. (in²/ft)	Mesh Size	Length (ft)	Area Supl'd. (in²/ft)
1	A_1 - .62	2x4 - W10.5xW5.0	14'-0"	.63
2	A_2 - .155	2x8 - W2.5xW2.5	8'-8"	.15
3	A_3 - .32	2x8 - W5.5xW2.5	12'-8"	.33
4	A_4 - .125	2x8 - W2.5xW2.5	8'-8"	.15
5	A_5 - .125	Provided by #1 Transverse steel	7'-8"	.15
6	A_6 - .125	#3(s) @ 12"c.c. GR. 60	7'-8"	.15
Design Loading: HS20				

pressive strength, steel areas, etc.) were determined and used as input for the CANDE computer analysis. The evaluation of the computer model's deflection and crack behavior output and the actual field test data is the subject of this report.

Load Test Installation and Procedure

For the load test, a standard 8-ft laying length of a 19-ft span by 6-ft 11-in. rise culvert was used. The test culvert was made according to the manufacturer's specifications (*1*) and the 19-ft by 6-ft 1-in. box-arch shop drawing (Figure 4) with the steel areas called for in Table 1.

As shown in Figures 5 and 6, three culverts were installed end to end on a cast-in-place base slab. The width of the excavation between the culvert leg cast-in-place base slab. The width of the excavation between the culvert leg and the existing soil was a minimum of 3 ft. After the culverts were plumbed and leveled on shims, the space between the bottom of the culvert's legs and the footer was grouted with a cement grout. A 12-in.-wide strip of filter fabric was placed over the joints. The connection plates normally installed between the units were omitted to allow the test unit to function independently.

With the culvert units set, the backfilling process began. The backfill material met the requirements of the Ohio Department of Transportation (ODOT) 310.02 Grading *B* and was constructed according to the manufacturer's specifications (*1*). This required compaction of soil determined per standard proctor that was 95 percent of the maximum laboratory dry weight. Compaction tests were performed by ODOT's Bureau of Testing to ensure proper compaction. The difference in the backfill elevation on each side of the culvert during placement did not exceed 1 ft. The backfill process was to proceed until 1 ft of cover above the outside of the unit at the centerline of the span was achieved.

After the backfilling was complete, the deflection test frame was attached. Deflection gauges supplied by CTL Testing Laboratory were mounted on the frame. The gauges were zeroed before the application of the load.

The test load was applied by the use of a 100-ton hydraulic hand-operated jack supplied by CTL Testing Laboratory. A calibration chart correlating the hydraulic pressure to the load increments was developed by CTL and is included in the CTL report (*2*). The hydraulic jack reacted between a beam tied to the base slab and a beam centered on the culvert that distributed the load uniformly across its length.

FIGURE 5 Box-arch culvert load test, August 1986.

FIGURE 6 External view of test unit.

Loadings were applied to the culvert in multiples of an AASHTO HS-20 service load. The load to be applied was determined as follows:

Cover = 1 ft,
S (span) = 19 ft,
P (single wheel load) = 16 kips,

Per Section 3.24.3.2 AASHTO specifications:

E = distribution width for 1 wheel load,
$E = 4 + 0.06S = 4 + 0.06\ (19) = 5.14$ ft,
W = live load/unit length, and
$W = P/E = 16.0/5.14 = 3.1$ kips/ft of width.

Therefore, on an 8-ft width of culvert:

Design service load = 3.10 (8) = 24.8 kips.

Actual load increments were 25 percent of a full service load multiple, or a 6.2 kip jack reaction.

On August 7, 1986, the first phase of the load test was performed. The culvert was loaded until the 6.2 kips were applied. After the load was stabilized, the CTL testing engineer recorded the deflection readings. In time periods of approximately 3 to 4 min, the load was increased to the next load increment and deflections were recorded. The last load increment was 49.6 kips, which represents two times the design load. After this load was stabilized for a period of 3 min, the deflection readings were recorded. The load was removed and the deflections were again recorded. The deflections that occurred during the first phase of the load test are shown in Table 2. Maximum deflections were less than $\frac{1}{16}$ in., and the structure rebounded to nearly its original position.

On August 12, 1986, the jack and the deflection gauges were reinstalled. Once the gauges were zeroed, the load was gradually applied in increments of 6.2 kips in time intervals of 3 to 4 min. Deflections were recorded for each load incre-

TABLE 2 19-FT SPAN LOAD TEST, PHASE 1: ACTUAL DEFLECTIONS

Load (kips)	Gauge Readings (in.)									
	No. 1	No. 3	No. 5	No. 7	No. 0	No. 8	No. 6	No. 4	No. 2	No. 9
6.2	0.001	0.001	0.0015	0.000	0.000	0.001	0.002	0.002	0.000	0.000
12.4	0.002	0.003	0.005	0.003	0.000	0.004	0.006	0.004	0.001	0.000
18.6	0.004	0.005	0.008	0.005	0.000	0.006	0.009	0.007	0.002	0.000
24.8	0.004	0.008	0.013	0.008	0.001	0.008	0.014	0.009	0.003	0.000
31.0	0.008	0.010	0.019	0.011	0.001	0.012	0.020	0.012	0.007	0.000
37.2	0.010	0.014	0.025	0.015	0.001	0.015	0.026	0.016	0.009	0.001
43.4	0.013	0.018	0.032	0.019	0.001	0.018	0.033	0.019	0.012	0.001
49.6	0.017	0.022	0.041	0.023	0.001	0.022	0.041	0.023	0.015	0.002

ment. The load increments and the associated deflections are shown in Table 3. The test culvert was loaded until the maximum load was obtained.

The results of the second phase of the load test are included in a CTL Engineering, Inc., report (*2*). The following are the written remarks by CTL included in their report (taken directly from CTL's report dated August 19, 1986).

II. Results

The loading progressed for 21 increments up to a loading of 133,500 lbs (133.5 kips). At this point the concrete span was loaded more than five (5) times the design load. At this loading, operating the 100-ton jack could only produce the constant loading, but the dial indicators began a constant movement indicating the concrete span was in a failure mode. The constant movement of the concrete surface continued to about a 2¾-inch deflection when it suddenly broke through the 8-foot span in the center of the arch and through the two drilled 2½-inch diameter holes made for passage of the two threaded bars for the loading system.

During the test, two of the dial indicators (# 1 and # 2) had withdrawn from contact with the concrete span. This indicated the span was belling out sideways while the span top was being compressed. Additional material was added to the steel test frame to remount these dial indicators. During this interval of time, about 25 minutes, the load was sustained at 133.5 kips. Once the dial indicators were remounted (at 12:55 p.m.) and pumping the jack continued normally again, the failure mode continued until the major break occurred at 1:02 p.m.

It should be noted that the first hairline cracks appeared at 55.8 kips loading, which was more than twice the design load of 24.8 kips.

Additional hairline cracking continued to appear at 66.0 kips, 68.2 kips, 74.4 kips, and the original hairline cracks became noticeably larger. At 80.6 kips, the original cracks had enlarged to about 1/64 inch. At about 111.6 kips, the cracks had increased to 1/32 inch and passed through the 2½-inch drilled holes for the thread bars. At 130.2 kips, the original cracks had enlarged to 1/16 inch.

At failure, 133.5 kips, the major cracks were about ⅜ inch wide all the way across the concrete span, passing through both drilled 2½-inch diameter holes. Some fallout of material, of course, made wider spots.

Most cracks appeared within about a 4-foot width across the span with the most concentration within about a 1-foot width.

The sideways or north-south movement of the span mainly occurred on the north side, where it remained deflected about 1¼ inch after the failure occurred. This was the side to which the deflection gauge frame was fastened.

TABLE 3 19-FT SPAN LOAD TEST: ACTUAL AND PREDICTED DEFLECTIONS

Load (kips)	Actual Gauge Readings (in.)										Predicted Values (in.)			
	No. 1	No. 2	No. 5	No. 7	No. 0	No. 8	No. 6	No. 4	No. 2	No. 9	No. 1	No. 3	No. 5	No. 0
6.2	0	0	0.002	0.001	0	0.002	0.003	0.002	0.001	0	0.001	0.001	0.000	0
12.4	0.001	0.002	0.006	0.004	0	0.004	0.0065	0.004	0.002	0	0.005	0.010	0.008	0.001
18.6	0.003	0.004	0.011	0.006	0	0.0065	0.011	0.007	0.004	0	0.007	0.015	0.015	0.002
24.8	0.005	0.007	0.017	0.010	0	0.010	0.017	0.010	0.006	0	0.010	0.019	0.023	0.003
31.0	0.007	0.010	0.023	0.013	0	0.014	0.022	0.013	0.008	0	0.013	0.024	0.027	0.004
37.2	0.010	0.013	0.029	0.016	0	0.018	0.027	0.016	0.010	0	0.017	0.029	0.038	0.005
43.4	0.013	0.017	0.036	0.020	0.001	0.021	0.034	0.020	0.013	0.0005	0.020	0.035	0.046	0.006
49.6	0.015	0.020	0.043	0.024	0.001	0.023	0.041	0.023	0.015	0.0005	0.023	0.040	0.054	0.007
55.8	0.019	0.024	0.051	0.028	0.0015	0.028	0.050	0.027	0.019	0.001	0.027	0.046	0.063	0.008
62.0	N/A	0.029	0.064	0.033	0.002	0.050	0.062	0.032	0.024	0.001	0.030	0.051	0.072	0.009
68.2	N/A	0.037	0.080	0.041	0.002	0.048	0.078	0.039	0.030	0.0015		0.060	0.082	0.010
74.4	N/A	0.044	0.093	0.048	0.0025	0.049	0.091	0.045	0.034	0.002		0.073	0.098	0.012
80.6	N/A	0.051	0.108	0.056	0.003	0.055	0.105	0.051	N/A	0.0025		0.094	0.120	0.015
86.8	N/A	0.098	0.188	0.107	0.0035	0.105	0.190	0.095	N/A	0.004		0.115	0.154	0.017
93.0	N/A	0.114	0.217	0.124	0.004	0.120	0.219	0.110	N/A	0.005		0.136	0.187	0.020
99.2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A		0.157	0.221	0.023
105.4	N/A	0.131	0.258	0.143	0.005	0.138	0.252	0.128	N/A	0.006		0.184	0.254	0.026
111.6	N/A	0.149	0.289	0.161	0.007	0.156	0.282	0.144	N/A	0.007		0.244	0.296	0.031
117.8	N/A	0.171	0.332	0.186	0.010	0.177	0.322	0.165	N/A	0.008		0.308	0.397	0.035
124.0	N/A	0.240	0.449	0.251	0.015	0.241	0.438	0.229	N/A	0.012		0.391	0.502	0.058
130.2	N/A	0.390	0.735	0.434	0.026	0.417	0.720	0.378	N/A	0.012		0.478	0.621	0.070
133.5	N/A	0.816	1.616	0.924	0.031	0.889	1.604	0.795	N/A	0.012		0.568	0.747	0.080

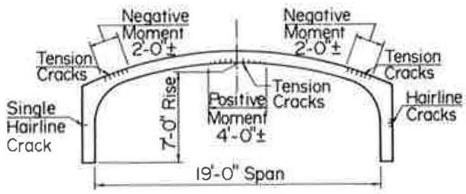

FIGURE 7 Crack patterns.

OBSERVATIONS FOLLOWING TEST

Wide cracks had opened up in the ground surface above both culvert legs indicating horizontal movement of the backfill. Mounding the fill over the culvert in place of complete burial reduced the capacity of the soil to resist the horizontal thrusts. This mounding is apparent in Figures 5 and 6.

Approximately 5 days after the load test, the backfill was removed and the test culvert was exposed. Considerable tension cracking (¼ in. to ⅜ in. wide) occurred in the span at the edge of the haunch section where the wall thickness is 10 in. (see Figure 7 for crack pattern sketches). The backfill at the base of the legs was cleaned away. The grout between the precast leg and the top of the footer was inspected. On the exterior side of the legs, the grout did not crack. No horizontal movement of the precast legs could be detected. The maximum deflection recorded at 1 ft above the foundation was 0.03 in. The precast legs appeared to rotate on the footer.

The tension steel at each of the major cracks was exposed and inspected. At every location the individual wires had elongated, necked down, and cracked. At the mid-span, positive moment area, this feature was characteristic across the full 8-ft cross section. In no instance did the compression face of each high moment area show any signs of distress.

With respect to the load test, the following items are concluded:

1. When the test culvert failed, three distinct hinges formed: two at the haunches and one at the mid-span. At all three locations the tension steel yielded and failed.
2. The legs of the test culvert rotated on top of the footer. The passive pressure resistance of the backfill and the resisting friction force kept the legs from moving horizontally. A pinned connection at the base of the legs can be assumed.
3. After the test culvert formed, three hinges and the tension steel failed at the three hinges, the test unit continued to support the 133.5-kip load. It appeared that the backfill's pressure against the back of the culvert's legs provided the necessary support to sustain this load.

After the testing was complete, the test culvert's actual material properties were determined and used as input for the computer analysis of the test section (Table 4). Samples of the welded wire fabric representative of the steel used in the test unit were submitted to the ODOT Bureau of Testing. A concrete core from the test unit was obtained and submitted to CTL for testing. Samples of the backfill material were also submitted to CTL to determine the density, sieve analysis, and the California bearing ratio value of the granular backfill. The mesh layout (Figure 8) for the computer model was revised to simulate the actual ground surface.

COMPARISON OF ACTUAL AND PREDICTED DEFLECTIONS

As stated, the primary purpose of the test was to compare results obtained from the load test with the results predicted from a CANDE computer analysis. Shown in Figure 9 is a graph of center-line deflections from the load test and the corresponding values determined from the analysis using the actual material properties as input for the program. Also shown is a single curve representing the predicted mid-span deflection based on the original design specifications.

TABLE 4 MATERIAL PROPERTIES

		Actual	Original Specifications
Concrete compressive strength (f'_c)		7,275 lb/in^2	4,000 lb/in^2
Steel yield stress (fy)	*W*5.5	78,610 lb/in^2	60,000 lb/in^2
	*W*10.0	71,360 lb/in^2 [a]	60,000 lb/in^2
Steel areas	*W*5.5	0.32 in^2/ft [b]	0.33 in^2/ft
	*W*10.0	0.62 in^2/ft	0.63 in^2/ft
Backfill material			
Compaction		95 percent maximum dry weight	
Classification		*GW*, *GP*	
"*d*" distance from centroid of the steel reinforcement to the compressive surface			
	*W*5.5	7.75 in.	8.0 in.
	*W*10.0	8.25 in.	8.5 in.

[a] Value used as input for analysis.
[b] Because the *W*5.5 mesh has a higher yield stress than the *W*10.0, the steel area was increased to 0.36 in./ft to compensate for the use of the lower yield stress as the input value in the analysis.

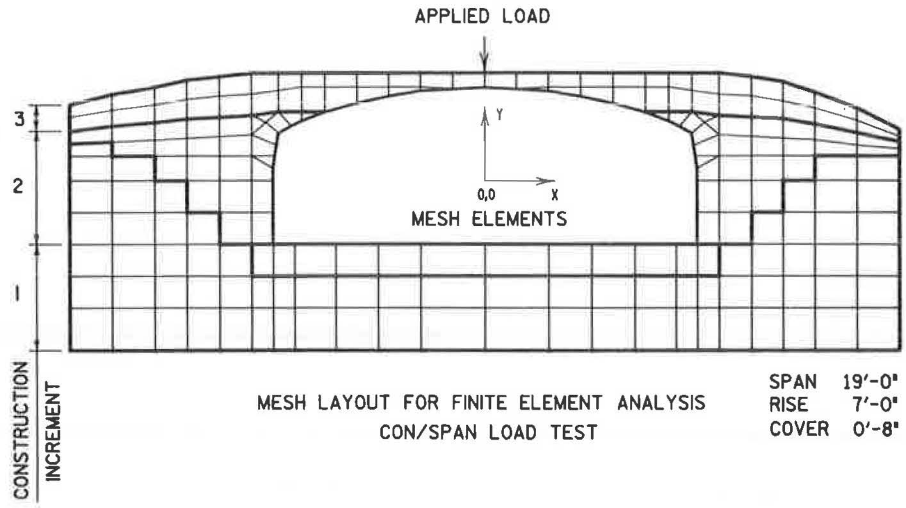

FIGURE 8 Mesh layout for finite element analysis: Con/Span load test.

The comparative values of deflection are in good correlation with each other, although the theoretical values are slightly larger. These larger theoretical deflections result from initial differences in the stiffness of an uncracked section and cracking strain. The theoretical value used for the cracking strain is based on the American Concrete Institute (ACI) recommendation for the tensile rupture strength equal to 7.5 f'_c. Of all the parameters, the concrete cracking strain influences the shape of the load-deformation curve the most significantly. The actual value of the tensile rupture strength is normally higher, which would have resulted in a closer correlation through the elastic range.

The sharp deflection occurring at 3½ times the service load on the actual deflection curves is the transfer of loading from the concrete to the steel as the cracking propagates throughout the section. The analysis models the cracking, progressing gradually through the section and producing a smooth stress-strain curve. Also causing this sudden deflection would be the initial cracking at the section just above the haunch, which theoretically occurs two load steps earlier. However, the pattern has been that the theoretical action occurred slightly before the actual action and would have occurred almost simultaneously if a higher value for the cracking strain had been used.

The CANDE analysis assumes plastic flow occurs when the stress in the steel is equal to its yield strength, and the analysis does not identify ultimate tensile or rupture strength of the steel. The actual stress in the steel can increase above the yield stress to its ultimate tensile strength. This is evident from the test in the area of 4 times the service load. The theoretical load-deformation curve flattens out when yielding occurs at midspan, indicating the beginning of the plastic range. The actual curve does not flatten out until two load increments later, indicating its additional load-carrying capacity above initial yield. The test culvert stopped taking additional load once the steel reached its ultimate tensile strength, and the load test was stopped when the steel necked down and cracked, producing excessive deflections. This point on the theoretical curve cannot be identified because the analysis cannot predict the steel's ultimate strain.

A CANDE analysis was run using the same loading on the culvert without backfill or cover. Its load-deformation curve was nearly identical until the loading reached 4½ times the service load, at which time the effects of the support from the backfill were realized. This comparison reveals the inherent strength in the unit itself.

The deflections predicted for the original specifications also predict a substantial capacity above and beyond the required ultimate strength. Although the CANDE analysis does not clearly indicate a load limit, it can be said that the ultimate load is reached when the slopes of the load-deflection curves approach a flat line. Based on this, it appears that the load-carrying capacity of the culvert would have exceeded the required ultimate strength by 50 percent at approximately 4¼ times the design service load, had it been built according to the original specifications.

COMPARISON OF CANDE ANALYSIS WITH CONVENTIONAL ANALYSIS

A comparison was made of moments at mid-span of the culvert using a conventional stress analysis and the CANDE analysis both with backfill and without. The conventional analysis was used, first, with the culvert legs considered restrained (pin-pin), and second, with horizontal movement allowed (pin-roller) as shown in Figure 10. Assuming a conventional elastic analysis, the maximum load carried by the pin-roller structure would be approximately 48 kips. At this point, using conventional concrete design, the steel would yield at mid-span and no further increase in capacity could be mobilized. The conventional analysis for the pin-pin condition would permit a capacity of approximately 65 kips. At this point the steel yields near the haunches. Some increased loading could be sustained but cannot be predicted by an elastic analysis.

The CANDE analysis uses a more sophisticated method of computing the actual section capacity than the standard Whitney rectangular stress distribution block method used in conventional design. This, together with consideration of the benefits of the extra capacity resulting from the axial compression on the section, yields a higher limit to the elastic range than in conventional design. Beyond this point CANDE analysis continues to model the structural response through the in-

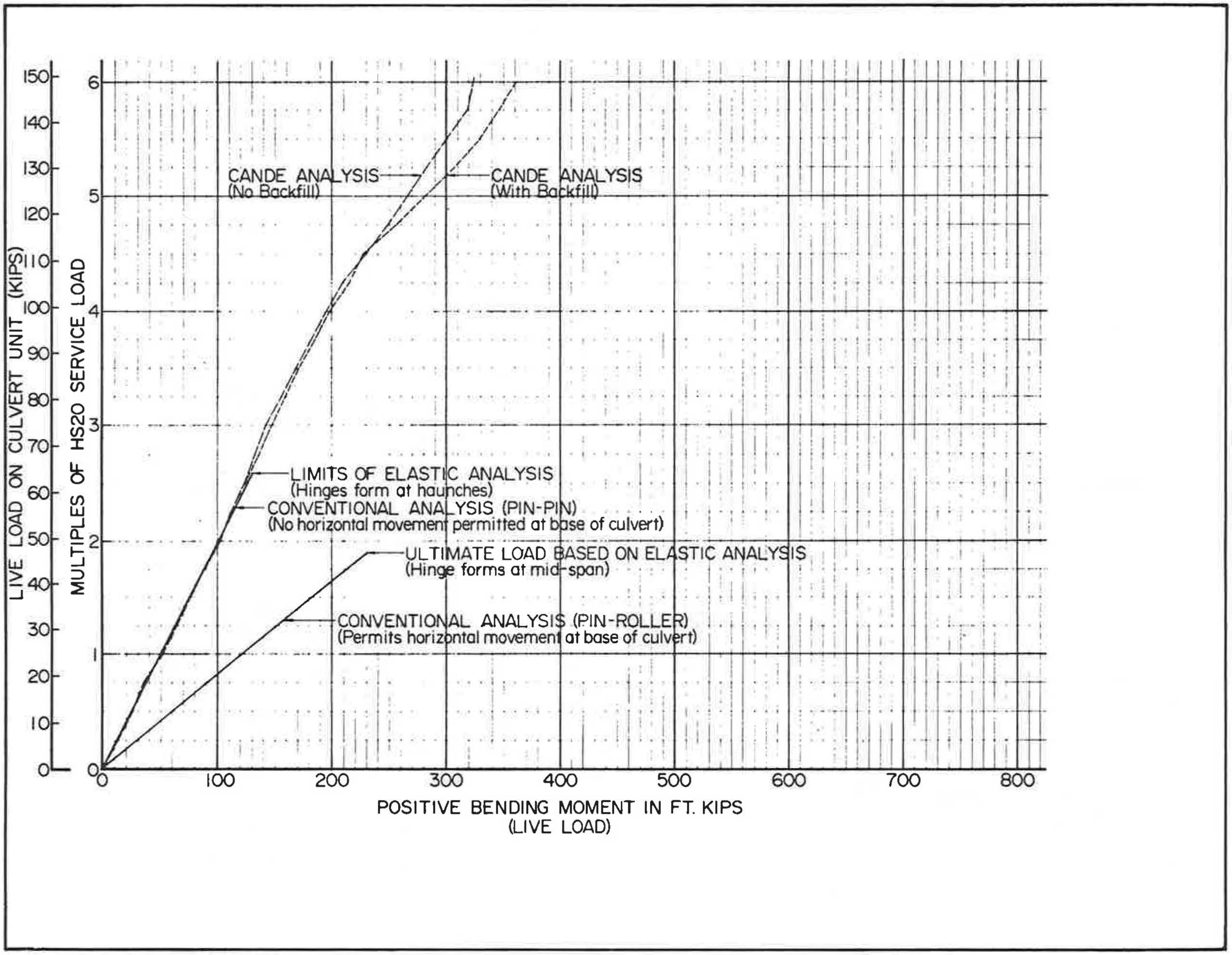

FIGURE 10 Precast box-arch load test: comparison of analyses.

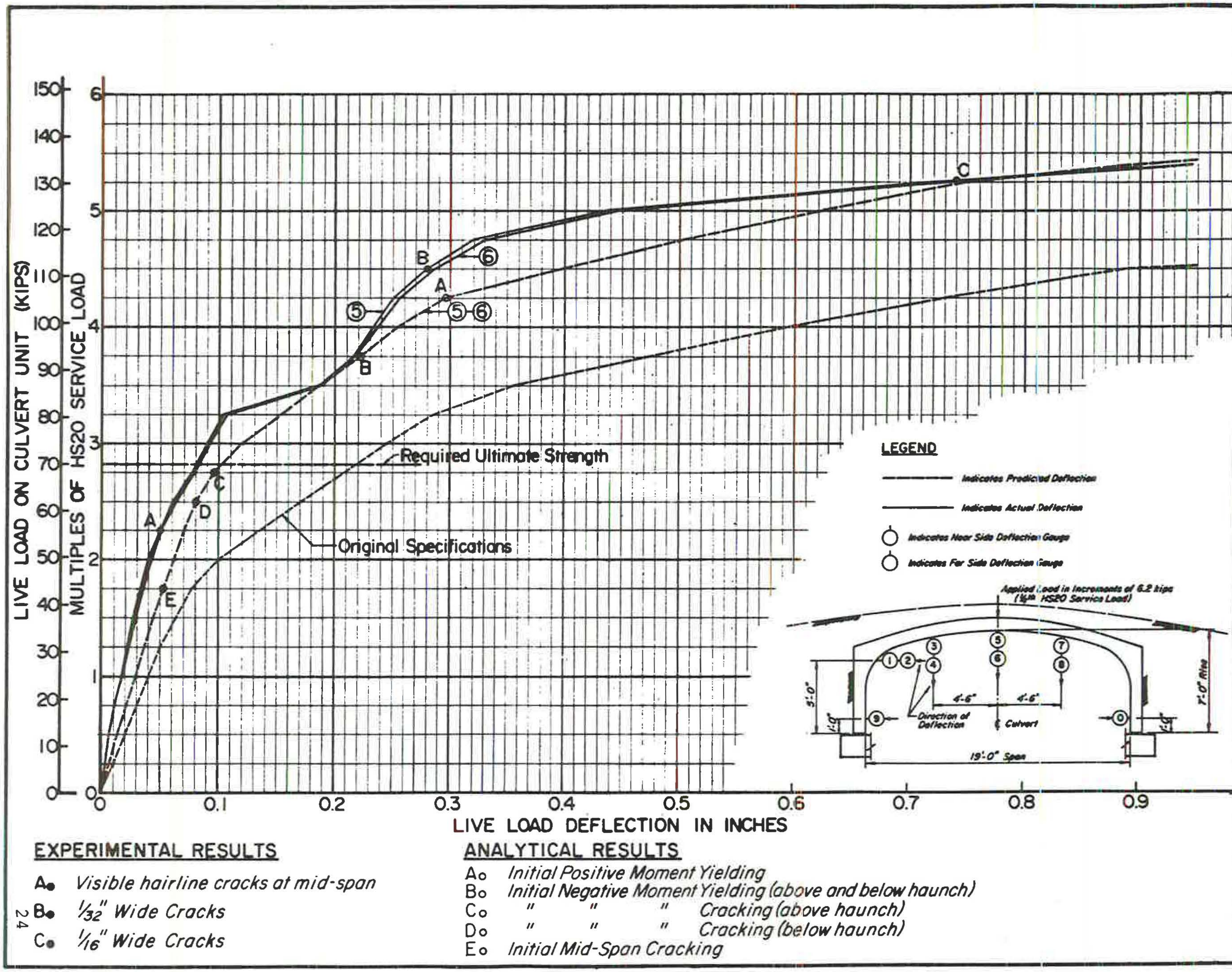

FIGURE 9 Precast box-arch load test: actual and predicted deflections.

elastic range, revealing its true reserve capacity beyond the first incidence of steel yielding.

CONCLUSIONS

The objectives of the load test were to validate the structural integrity of the box-arch culvert section and to verify that the computer model generated by the modified CANDE program provided a valid representation of the structural behavior of the buried culvert.

The structure greatly exceeded all performance requirements for highway loading. It carried a load greater than 5 times an HS20 design service load without impact and sustained the load through succeeding deformations imposed by the loading jack. The required ultimate capacity, including impact, was 2.8 times the service load. Material tests indicated that actual steel and concrete strengths were somewhat higher than the values used in the design. Correcting the analysis for the effect of these higher strength materials still resulted in a conservative design.

The CANDE program has been demonstrated by many others to be a reliable method to use to model the performance of buried structures. Because of the relatively high stiffness of the culvert, the predominant effect in this test was the structural behavior of the precast unit. The soil-structure interaction would have increasing and earlier effects for longer spans, higher fills, and level ground surfaces above the structure.

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