

# Failure of Flexible Long-Span Culverts Under Exceptional Live Loads

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Despite recent advances in the ability of nonlinear finite element analysis methods to model soil-structure interaction phenomena associated with buried flexible culverts, there are currently few published full-scale field data available for evaluation of the accuracy and reliability with which nonlinear finite element methods model the influence of live vehicle loads on culvert stability. Presented in this paper are the results of finite element analyses of three full-scale field cases involving culvert failure under exceptional live (vehicle) loads. Field evidence in all three cases suggests that the live loadings applied only barely exceeded the structural capacities of the culvert systems so that these three case studies provide a good basis for evaluation of the accuracy and reliability of the finite element analysis methods employed in these studies. A new empirical procedure is employed in these studies to develop equivalent line loads used to provide a representative plane strain modeling of actual discrete vehicle wheel loads. The results of these case studies provide good support for the accuracy and reliability of the analytical procedures used in these studies.

The past 15 years have seen steady improvement in the ability of nonlinear finite element analysis methods (FEM) to model and analyze soil-structure interaction phenomena associated with buried flexible culvert structures. Full-scale case studies and observed long-term successful performance of culvert designs based on such analysis methods suggest that incremental FEM are now able to model backfill loading and compaction-induced stresses with good accuracy for many types of culvert and backfill configurations and conditions.

Unfortunately, however, there are few published data currently available on the ability of nonlinear FEM to correctly model the influence of surficial live (vehicle) loads on culvert stability. This stems, in part, from the large expense associated with performance of meaningful full-scale live load tests to failure or near-failure conditions, as well as from the understandable reluctance of engineers and culvert manufacturers to disseminate information on field cases involving live-load-induced culvert distress or failure, or both. The study of such field cases involving distress or failure would represent an excellent basis for evaluation of the ability of FEM to accurately model live load effects on culvert stability.

Presented in this paper are the results of finite element analyses of three field cases involving culvert distress or failure resulting from inadvertent application of exceptional live loads, where exceptional live loads are vehicle loads considerably in excess of allowable design loading. Two of the cases consid-

ered involve long-span flexible pipe arch culverts, and the third case involves a 15-ft span flexible box culvert structure. All three of these cases are of particular interest because field evidence suggests that the live loadings applied only barely exceeded the structural capacities of the culvert systems. Back-analysis of these cases, therefore, provides an excellent basis for evaluation of the accuracy and reliability of the nonlinear FEM used to model these live load effects.

## COOPER CITY PIPE ARCH CULVERT STRUCTURES

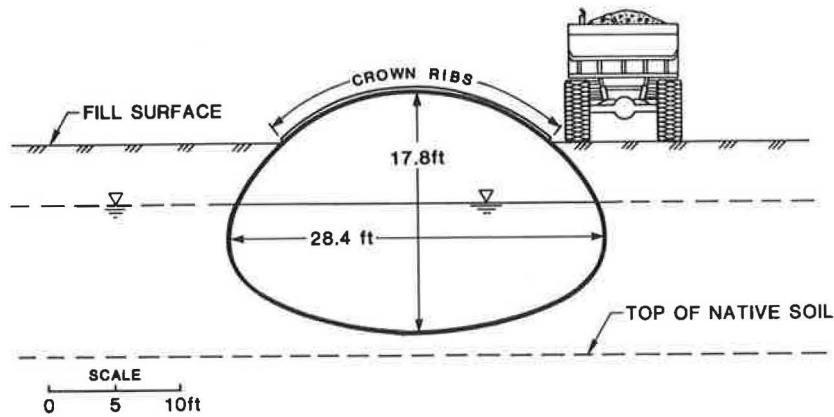
The first two case studies involve a pair of long-span corrugated aluminum pipe arch culverts near Cooper City, Florida, that were damaged by exceptional live loads during construction in 1985 and 1986. Both structures have since been repaired and have performed well under design live loading conditions.

### 28-Ft Span Pipe Arch

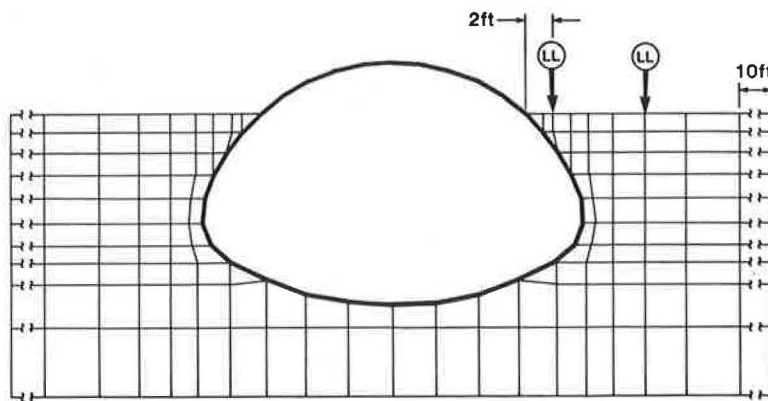
Shown in Figure 1(a) is a schematic cross-section of a long-span pipe arch culvert that was damaged by exceptional live loading in September of 1985. The structure is a corrugated aluminum pipe arch culvert with a span of 28 ft 5 in. and a rise of 17 ft 10 in. installed to provide a canal overpass. The culvert consists of 0.15-in.-thick 9- × 2 1/2-in. corrugated aluminum plate with Type IV aluminum stiffening ribs spaced at 54 in. on center across the crown. The structure was designed to support HS-20 live traffic loads (32 kips on a single axle) at the final backfill configuration, which would consist of 4 ft of backfill cover over the crown with a concrete relieving slab occurring immediately above the crown of the culvert.

Conditions at the time of culvert damage are illustrated in Figure 1(a). The backfill was only partially completed, and had not yet reached the crown of the structure. At this stage of backfill placement, long-span culverts were vulnerable to damage by large vehicle loads occurring above the culvert crown with shallow soil cover, and such large vehicle loads were thus disallowed by design specifications for backfill placement and compaction procedures. Nonetheless, early on the morning of September 3, 1985, a large 16-yd<sup>3</sup> dump truck carrying backfill arrived at the project before routine backfill operations began and passed close alongside the structure, depositing its material. Photographic evidence of tire tracks indicate that the closest rear wheel of the truck travelled parallel to the culvert at a lateral distance of approximately 2 ft from the exposed structure, as indicated in Figure 1(a).

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(a) Schematic Cross-Section Showing Backfill Configuration and Critical Live Vehicle Loading at Failure Conditions



(b) Finite Element Mesh Representing Conditions at Failure

**FIGURE 1** Twenty-eight-ft span pipe-arch culvert structure near Cooper City, Florida (a) Schematic cross-section showing backfill configuration and critical live vehicle loading at failure conditions, (b) Finite element mesh representing conditions at failure.

This resulted in inward flexural failure of the upper quarter region of the culvert along a length of approximately 40 ft of the culvert (whose total length was approximately 200 ft), as illustrated schematically in Figure 2. This "failure" was neither rapid nor catastrophic in nature, as the truck drove off the fill without difficulty and the deformations of the buckled zone did not lead to full culvert collapse with the vehicle loading thus removed. This, along with observations made during subsequent excavation and repair (by plate replacement) of the damaged culvert region, suggests that this exceptional live loading barely induced failure. As a result, this failure represents an excellent opportunity to evaluate the ability of finite element analyses to correctly model live load effects on culverts.

Figure 2 is a schematic illustration of the observed deformation modes of the failed culvert region. The large live load in close proximity to the structure resulted in inward flexure of the upper quarter-point region of the culvert, with formation of two plastic hinges representing failure in flexure at Points *A* and *B*. These hinges were manifested as clearly visible creases in the corrugated structural plate. Careful inspection of the structure revealed no additional plastic hinge in the region between Point *A* and the crown of the structure;

instead, deformations in this upper region were related to smoother curvature spread more evenly over the rib-reinforced crown. There was no sign of flexural failure in this upper crown region.

Finite element analyses were performed to model this failure using the program SSCOMP (1), a plane strain finite element code for incremental nonlinear analysis of soil-structure interaction. The finite element mesh used for these analyses is shown in Figure 1(b). Soil elements were modeled with four node isoparametric elements and the culvert structure was modeled with piecewise linear beam elements. Nonlinear stress-strain and volumetric strain soil behavior was modeled using the familiar hyperbolic formulation proposed by Duncan et al. (2), as modified by Seed and Duncan (3), and structural behavior was modeled as linear elastic. Nodal points at the base boundary were fixed against translation, and nodal points at the right and left boundaries were fixed against lateral translation but were free to translate vertically. The analyses were performed in steps or increments, incrementally modeling layerwise placement of backfill and then, ultimately, the application of live loads representing the 16-yd<sup>3</sup> dump truck.

The backfill material was a loosely dumped gravel (GW)

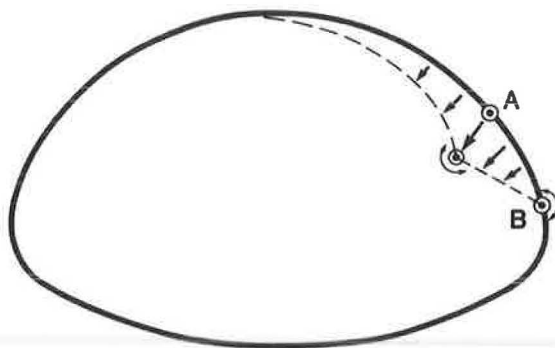


FIGURE 2 Schematic representation of observed failure mode of 28-ft span pipe-arch culvert near Cooper City, Florida.

below the water table, and a fine silty sand (SM) compacted to a minimum of 90 percent relative compaction (modified AASHTO, ASTM D1557) above the water table. The foundation soil was a partially cemented silty clayey sand (SM), which was judged to have properties similar to the upper compacted backfill zone. Listed in Table 1 are the hyperbolic soil model parameters used to model the foundation soils and the upper and lower backfill zones. All parameters are effective stress parameters, and water table effects were modeled by using buoyant unit weights below the water table.

Listed in Table 2 are the properties used to model the various components of the culvert structure. Elastic section moduli for the corrugated aluminum plate sections are based on large-scale flexural test data. Section moduli of the ribbed crown section were modeled as intermediate between the theoretical value for the crown plate and ribs functioning as a composite beam and the theoretical value for the ribs and crown plate each functioning independently. This is again based on large-scale flexural test data and represents the effects of shear slippage at the plate and rib connections. The ultimate plastic moment capacities listed in Table 2 are also based on large-scale flexural test data.

The dashed line in Figure 3 shows the calculated bending moments around the perimeter of the culvert structure resulting from placement of backfill up to the elevation shown in Figure 1, but without additional live load application. The maximum calculated bending moments in the crown and upper haunch (shoulder) regions are approximately 0.92 kip-ft/ft and 0.98 ft/ft, representing factors of safety (FS) of approximately

TABLE 1 HYPERBOLIC SOIL MODEL PARAMETERS USED TO MODEL COOPER CITY CULVERT BACKFILL AND FOUNDATION SOILS

	Foundation Soil	Backfill Below Water Table	Backfill Above Water Table
$\gamma$ (lb/ft <sup>3</sup> )	63	78	125
$K$	450	400	450
$n$	0.25	0.4	0.25
$R_f$	0.7	0.7	0.7
$K_b$	350	120	350
$m$	0	0.2	0
$c$	0	0	0
$\phi$	34°	36°	34°
$\Delta\phi$	6°	4°	6°

TABLE 2 STRUCTURAL PROPERTIES USED TO MODEL 28-FT-SPAN COOPER CITY PIPE-ARCH CULVERT STRUCTURE

Structural Component	Modulus: $E$ (kip/ft <sup>2</sup> )	Area (ft <sup>2</sup> /ft)	$I$ ( $\times 10^{-4}$ ) (ft <sup>4</sup> /ft)	Plastic Moment Capacity: $M_p$ (kip-ft/ft)
Crown with ribs	1,468,800	0.018	1.86	6.82
Invert and haunches	1,468,800	0.015	0.72	3.18

FS  $\approx 7.4$  and FS  $\approx 3.2$  with respect to flexural failure in these two regions, respectively, at this stage of backfill placement.

The plane strain finite element formulation used for these analyses requires that the actual discrete vehicle wheel loads be represented by one or more equivalent (plane strain) line loads. A number of procedures have been employed for developing such representative line loads, but none has gained universal acceptance. The procedure used in this study to develop equivalent line loads providing representative plane strain modeling of discrete vehicle wheel loads is a slightly modified version of an equivalent line load estimation procedure proposed by Duncan and Drawsky (4).

Duncan and Drawsky proposed that the equivalent line load used to model a wheel or an axle with several wheels should be the line load that provides the same maximum vertical stress beneath the wheel(s) on a plane at the depth of the top of the culvert crown as would the actual three-dimensional wheel loading, based on Boussinesq (5) linear elastic vertical stress distribution analysis. This approach is inherently slightly conservative because it provides a line load producing this peak stress along the full length of the culvert, whereas discrete wheel loads provide this stress only beneath the actual wheels, with lesser stresses away from the wheel loading points. This inherent conservatism is minimized, however, in corrugated culverts, which do not have significant flexural stiffness longitudinally along the culvert and which are, therefore, vulnerable to localized overstressing directly beneath discrete wheel loads.

Duncan and Drawsky developed a simple equation and tabulated solution expressing their equivalent line load estimation procedure as

$$LL = AL/K4 \quad (1)$$

where

LL = equivalent line load (kips/ft),  
AL = total axle load (kips), and  
K4 = load factor (ft).

Presented in Table 3 are the K4 values developed by Duncan and Drawsky based on Boussinesq elastic analyses for various depths of soil cover, where depth of soil cover ( $H_c$ ) is defined as the vertical distance from the base of the wheel(s) to the top of the culvert.

This equivalent base of the wheel(s) to the top of the culvert.

This equivalent line load estimation procedure was modified in this study by defining equivalent depth of soil cover ( $H_c$ ) as the shortest radial distance from the base of a wheel load to the culvert plate, and then by using the same K4 values as shown in Table 3. This modification provides improved

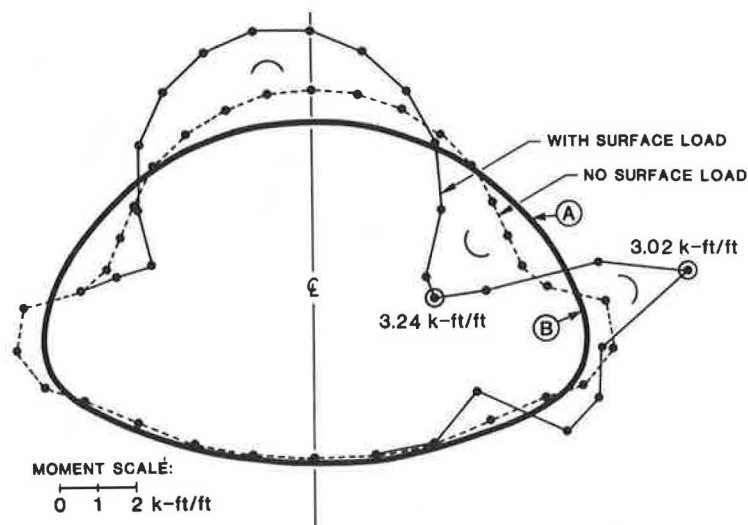


FIGURE 3 Calculated bending moments with and without live load application: 28-ft span pipe-arch culvert.

modeling of live loads not centrally located above the culvert crown.

The fully loaded 16-yd<sup>3</sup> dump truck that precipitated the first Cooper City culvert failure had a rear axle load of approximately 46 kips. Allowing for several inches of observed rutting or tire indentation into the backfill surface, the equivalent depth of cover for the rear wheel of the truck closest to the culvert was approximately 2 ft, corresponding to a value of  $K_4 = 5.3$  ft. This rear wheel was therefore modeled with an equivalent line load of  $LL \approx 46 \text{ kips}/5.3 \text{ ft} = 8.68 \text{ kips/ft}$  applied at a lateral distance of 2 ft from the culvert at the fill surface, as shown in Figure 1(b). The front and right-hand side wheels of the truck were sufficiently far from the loading region of interest to have had a negligible additional effect on culvert stresses in the region of interest (this was verified by means of additional three-dimensional stress distribution analyses considering all four wheels).

The solid line in Figure 3 indicates the calculated bending moments around the perimeter of the culvert following modeling of application of an equivalent surface line load of 8.68 kip-ft/ft 2 ft from the culvert, as shown in Figure 1(b). This live load application caused a large increase in bending moments in the upper haunch (shoulder) and crown regions, as shown in Figure 3, and resulted in a calculated moment of 3.24 kip-ft/ft at a point in the upper haunch region shortly below the crown stiffening ribs. This barely exceeds the plastic moment

capacity of 3.18 kip-ft/ft at this point (as listed in Table 2) and would be expected to result in formation of a plastic hinge. The location of this hinge closely corresponds to the actual observed hinge location at Point A in Figure 2. A second large bending moment occurs approximately at the location of the second observed plastic hinge at Point B.

The calculated factor of safety (FS) at this location is  $\approx 3.18/3.02 = 1.05$ , and this would appear to provide satisfactory agreement with field observations, suggesting that flexural failure barely occurred. The maximum calculated crown moment is 2.64 kip-ft/ft, and thus represents a  $FS \approx 6.82/2.64 = 2.6$  in the rib-reinforced crown region, and so agrees with the observation of some bending but no plastic hinge formation in this region.

These analyses provide surprisingly good agreement with field observations inasmuch as (a) they correctly predict the approximate location and occurrence of the two observed plastic hinges at Points A and B, (b) they correctly predict no third plastic hinge formation in the crown region, and (c) they concur with field observations that the distressed culvert region barely failed. Nonlinear finite element analyses employing the modified Duncan-type live load modeling assumption thus appear to provide excellent prediction of observed field behavior for this case study.

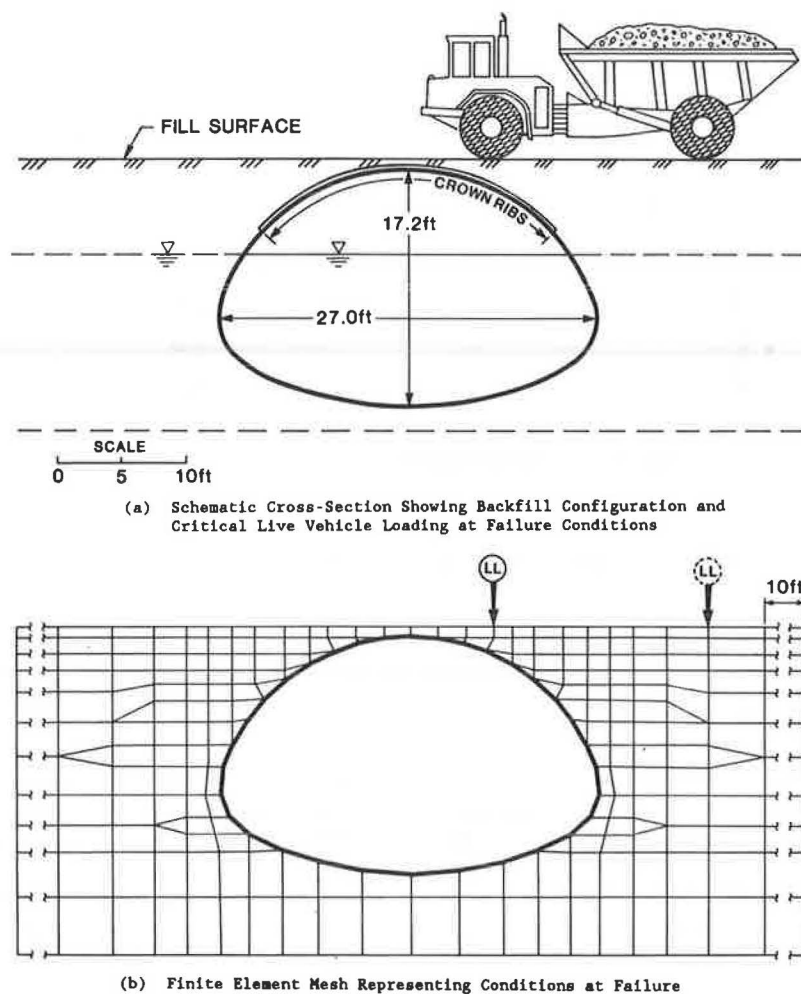
Subsequent to this live-load-induced failure, the pipe-arch culvert structure was excavated and the damaged corrugated structural plate was replaced. The remaining backfill and relieving slab were then placed. The structure has since performed successfully under its original design live loading conditions, including overpassage by the same type of fully loaded 16-yd<sup>3</sup> dump trucks as those that caused the initial culvert distress.

TABLE 3 VALUES OF  $K_4$  FOR CALCULATION OF EQUIVALENT LINE LOADS REPRESENTING SURFACE VEHICLE LOADS (after Duncan and Drawsky, 1983)

Cover Depth $H_c$ (ft.)	Values of $K_4$ (ft.)		
	2 Wheels/Axle	4 Wheels/Axle	8 Wheels/Axle
1	4.3	5.0	8.5
2	5.3	6.4	9.2
3	7.9	8.7	10.6
5	12.3	12.5	13.5
7	14.4	14.5	14.6
10	16.0	16.0	16.0
20	28.0	28.0	28.0

### 27-Ft Span Pipe Arch

A second, similar pipe-arch culvert structure also in a canal near Cooper City, Florida, was damaged by inadvertent exceptional live loading in August 1986. A schematic cross-section of this second culvert is shown in Figure 4(a). The



**FIGURE 4** Twenty-seven-ft span pipe-arch culvert structure near Cooper City, Florida (a) Schematic cross-section showing backfill configuration and critical live vehicle loading at failure conditions, (b) Finite element mesh representing conditions at failure.

structure has a span of 27 ft and a rise of 17 ft 2 in. The culvert consists of 0.125-in.-thick  $9 \times 2 \frac{1}{2}$ -in. corrugated aluminum plate with Type II aluminum stiffening ribs spaced at 54 in. on center across the crown. Backfill and foundation soils are essentially the same as those described previously for the 28-ft span pipe arch culvert, and the structure had a 10-in.-thick concrete relieving slab immediately above the crown of the structure, overlain by an additional 8 to 10 in. of compacted crushed shell and stone intended as a road base.

The relieving slab covered the entire central portion of the culvert, which was designed to carry HS-20 live traffic loading. Unfortunately, after completion of slab construction, backfill placement, and even road paving before installation of road curbs, a fully-loaded 16-yd<sup>3</sup> dump truck traveled off the edge of the paved zone and collapsed the crown of the end of the culvert outside the zone protected by the concrete relieving slab. A schematic illustration of conditions at the time of this failure is shown in Figure 4 (a). The large truck halted (parked) with its front wheels approximately 6 ft from the center line of the culvert, and the collapse proceeded fairly slowly with the front axle of the truck descending slowly into the canal and the driver sustaining no injury.

This failure was modeled with the program SSCOMP using the same procedures as described previously. Figure 4(b) shows the finite element mesh used for these analyses. Backfill and foundation soils were similar to those described previously for the 28-ft span pipe arch culvert, and the soil model parameters listed in Table 1 were again used to model the foundation and backfill soils. Structural properties used to model the culvert itself differ slightly from those of the previous study; the structural properties modeled in this analysis are listed in Table 4.

**TABLE 4** STRUCTURAL PROPERTIES USED TO MODEL 27-FT-SPAN COOPER CITY PIPE-ARCH CULVERT STRUCTURE

Structural Component	Modulus: $E$ (kip/ft <sup>2</sup> )	Area (ft <sup>2</sup> /ft)	$I (\times 10^{-4})$ (ft <sup>4</sup> /ft)	Plastic Moment Capacity: $M_p$ (kip-ft/ft)
Crown with ribs	1,468,800	0.015	1.20	4.62
Invert and haunches	1,468,800	0.012	0.60	2.65

The equivalent line load used to provide a representative plane strain modeling of the actual discrete vehicle loading was again based on the modified Duncan empirical load estimation procedure. The front axle of the 16-yd<sup>3</sup> dump truck carried a load of approximately 42 kips on two wheels. After allowing for observed rutting or tire indentation of approximately 8 in. in the unpaved backfill surface (the truck traveled off the paved surface underlain by the relieving slab), the remaining depth of soil cover between the base of the wheels and the top of the culvert plate was approximately 12 in., and the corresponding live load factor from Table 3 is  $K_4 = 4.3$  ft. This led to modeling of the front wheel loads with an equivalent line load of  $LL = 42 \text{ kips}/4.3 \text{ ft} = 9.8 \text{ kips/ft}$ . The rear wheels of the truck were far enough from the structure so that they contributed negligibly to the culvert stresses. This was verified by performing analyses with and without modeling the rear axle as a line load of 10 kips/ft. Inclusion or omission of the rear axle line load changed the calculated bending moments in the culvert by less than 5 percent.

Calculated bending moments in the culvert before and after application of the 9.8-kip/ft. line load modeling the front axle of the truck is shown in Figure 5. The dashed line represents moments resulting from backfill loads only, and corresponds to an FS with respect to flexural failure of more than 3.0 throughout the crown region. The solid line represents moments with the live vehicle loading applied. The two points circled on this figure indicate bending moments of 4.64 kip-ft/ft and 8.31 kip-ft/ft, both of which equal or exceed the flexural plastic moment capacity of the culvert crown region, which is approximately 4.62 kip-ft/ft. This analysis, therefore, also appears to provide good agreement with observed field behavior, which in this case involved failure of the crown region in flexure.

### RANCHO FLEXIBLE BOX CULVERT STRUCTURE

The third case study considered a 15-ft span corrugated aluminum box culvert structure near Rancho, California, that was crushed by a 95-kip scraper that passed over the crown of the structure, greatly exceeding the allowable design live loading, in March 1985. The structure has since been replaced and has performed well under design loading conditions.

A schematic cross-section of the box culvert and live load conditions at the time of failure is shown in Figure 6. The

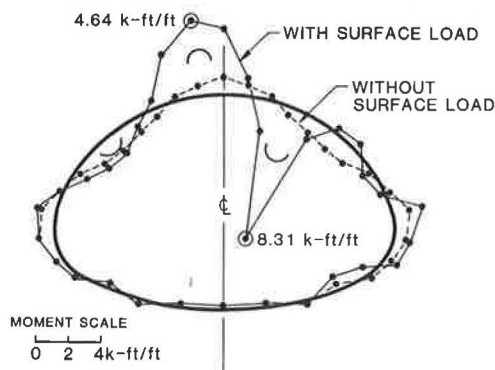
structure was a corrugated aluminum box culvert with a span of 14 ft 8 in. and a rise of 4 ft 1 in., installed to provide a roadway bridge over a stream. The culvert consisted of a 0.175-in.-thick  $9 \times 2 \frac{1}{2}$ -in. corrugated aluminum plate with Type IV aluminum bulb angle stiffening ribs at 27-in. spacing reinforcing the crown and upper haunch regions, and Type II aluminum bulb angle stiffening ribs at 27-in. spacing reinforcing the lower haunch regions. The culvert was cast into a full reinforced concrete invert slab. The structure was designed to support HS-20 live traffic loads at the final backfill configuration, which would consist of 24 in. of soil cover above the crown of the culvert overlain by a paved road surface.

In March 1985 the structure failed during inadvertent crossing by a fully loaded scraper with a total weight of approximately 95 kips. This crossing was the result of operator error, as such large vehicle loads were proscribed by both design and operating regulations.

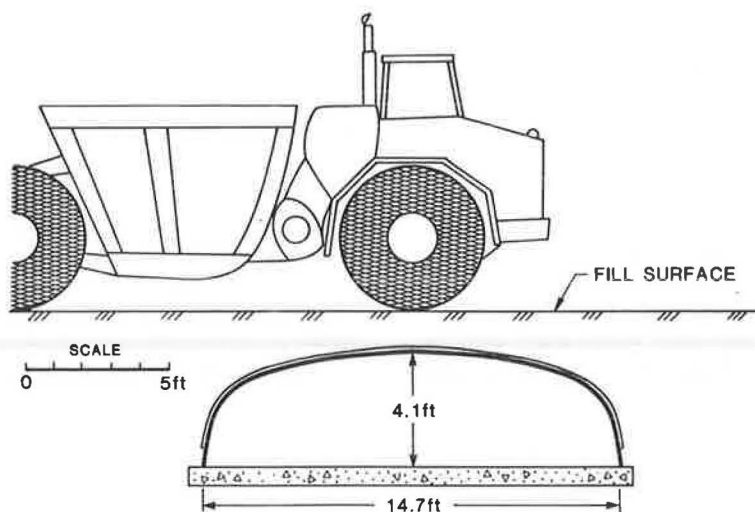
This failure was modeled with the program SSCOMP using the same procedures as those described previously. The finite element mesh used for these analyses is shown in Figure 7. As the rear wheels of the scraper contributed negligibly to the failure, the problem was symmetric about the box culvert center line and only a half mesh was used. Nodal points out that the right-hand boundaries of this mesh were free to translate vertically but were fixed against both lateral translation and rotation. Backfill and foundation soils were a silty clay of low plasticity (CL) compacted to a minimum of 90 percent relative compaction (modified AASHTO, ASTM D1557), and were modeled with the following hyperbolic soil model parameters:  $\lambda_m = 125/\text{ft}^3$ ,  $K = 150$ ,  $n = 0.45$ ,  $R_f = 0.7$ ,  $K_b = 110$ ,  $m = 0.25$ ,  $c = 400 \text{ ft}^2$  and  $\phi = 30$  degrees. The structural properties used to model the box culvert and invert slab are listed in Table 5.

Examination after the failure showed that the box culvert structure failed when the front axle of the scraper was essentially directly over the center line of the box culvert. The surface of the fill had not yet been paved and the scraper wheels rutted or indented into the fill surface so that the effective backfill cover depth between the bases of the front wheels and the crown of the structure was approximately 18 or 19 in. The front axle load was approximately 60 percent of the total vehicle load, or about 57 kips, which greatly exceeded the 32 kip HS-20 single axle design load intended for a paved fill surface. This front axle load was again modeled as a representative equivalent line load based on the modified Duncan empirical load estimation procedure, with  $K_4 = 4.85$  ft. The 57-kip front axle load on two wheels with 19 in. of soil cover was thus modeled as an equivalent line load of  $57 \text{ kips}/4.85 \text{ ft} = 11.8 \text{ kips/ft}$ . One-half of this line load (5.9 kips/ft) was applied to the surface of the backfill above the culvert center line (at the extreme right edge of the finite element mesh shown in Figure 7).

The calculated bending moments in the box culvert before and after application of the 5.9 kip/ft line load modeling the front wheels of the scraper are shown in Figure 8. The dashed line represents moments caused by backfill loads only and these moments are small relative to the flexural capacities of the culvert crown and haunch regions. The solid line represents moments with the live vehicle loading applied. The maximum calculated bending moments of 14.0 kip-ft/ft and 12.6 kip-ft/ft at the center line and tops of the two haunches, respectively, both exceed the culvert plastic moment capac-



**FIGURE 5** Calculated bending moments with and without live load application: 27-ft span pipe-arch culvert.



**FIGURE 6** Schematic cross-section of the Rancho box culvert showing critical vehicle loading at the time of failure.

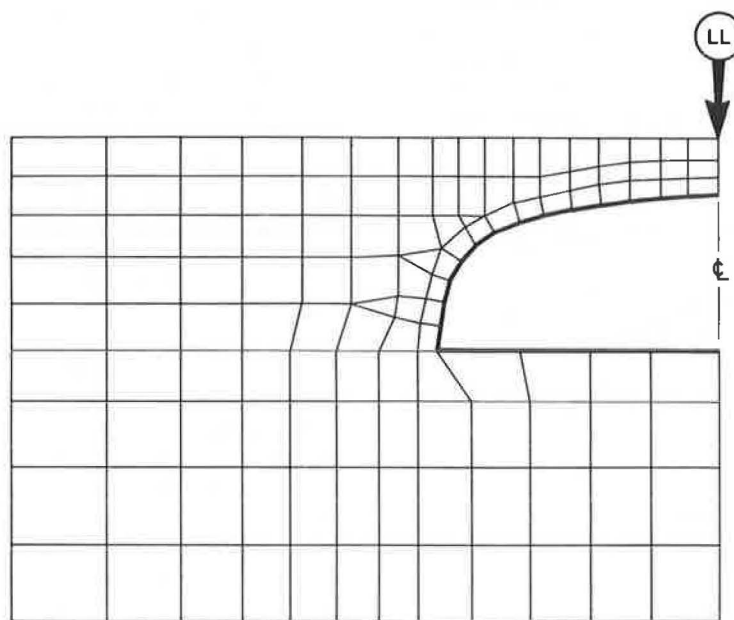
ities of 10.39 and 7.94 kip-ft/ft, respectively (as shown in Table 5). The locations of these calculated maximum moments closely correspond to the observed locations of flexural failure, as manifested by clearly discernible creasing and tearing of the corrugated aluminum structural plate. These analyses, therefore, appear once again to provide excellent agreement with observed field behavior.

### SUMMARY AND CONCLUSIONS

Three full-scale field case studies were considered involving failure of flexible long-span culvert structures caused by

exceptional live loads. Each case was analyzed using nonlinear FEM. An empirical procedure was proposed for developing equivalent line loads used to provide a representative plane strain modeling of actual discrete vehicle wheel loads. This empirical procedure represents a slight modification of an earlier procedure proposed by Duncanson and Drawsky (4).

For all three field cases considered, the nonlinear finite element analyses performed provided excellent agreement with observed field behavior, correctly predicting not only failure conditions but also observed failure modes. This provides good support for the accuracy and reliability of the FEM and equivalent line load modeling procedures employed in these studies. This is particularly so because the case studies con-



**FIGURE 7** Finite element mesh representing the Rancho box culvert structure at the time of failure.

TABLE 5 STRUCTURAL PROPERTIES USED TO MODEL 15-FT-SPAN BOX CULVERT STRUCTURE AT RANCHO CALIFORNIA

Structural Component	Modulus: $E$ (kip/ft <sup>2</sup> )	Area (ft <sup>2</sup> /ft)	$I (\times 10^{-4})$ (ft <sup>4</sup> /ft)	Plastic Moment Capacity: $M_p$ (kip/ft/ft)
Crown with Type IV ribs	1,468,800	0.026	3.70	10.39
Invert and haunches with Type II ribs	1,468,800	0.022	2.72	7.94
Concrete invert slab	520,000	0.67	250	Large

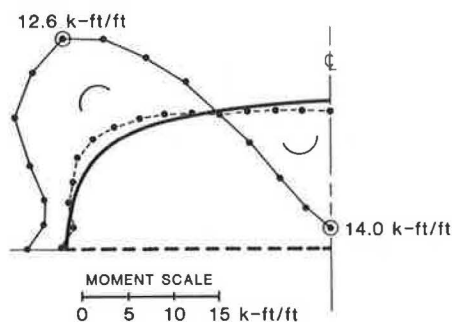


FIGURE 8 Calculated bending moments with and without live load application: 15-ft span Rancho box culvert.

sidered involve situations wherein field evidence suggests that the exceptional live vehicle loadings applied only barely exceeded the structural capacities of the culvert systems (structural capacities were exceeded by less than 10 percent

for three of the six flexural failures predicted by the analyses performed) so that back analysis of these cases provides a good basis for evaluation of the accuracy and reliability of these analytical methods.

## ACKNOWLEDGMENTS

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