Load Study of Flexible Pipes under High Fills

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Earth load tests were conducted on three 7-foot-diameter corrugated-metal pipes under 137 feet of fill. Each pipe was 512 feet long on a grade of 2.5 percent with 6 inches of parabolic camber at the center. Pipes were filled strutted to 3 percent elliptical cross-section with long axis vertical.

The fill to a height of 3 feet above the pipe was composed of a granular material of 100 percent Proctor density. At a fill height of 10 feet, a 7-foot-wide and 7-foot-deep trench was cut over each pipe and backfilled with loose uncompacted material before proceeding with normal fill operations. The remainder of the fill was placed in 3-foot lifts compacted by sheepsfoot rollers.

Loads were determined using load cells placed between the sills and the struts, and by attaching SR-4 strain gages to the pipe on the neutral axes of the corrugations. Deflection and subsidence measurements were also made.

Strain-gage data revealed that each increment of fill added its load increment in direct proportion. Deflection and subsidence of the pipe were within design limits.

● WITH the vast improvements in earth-moving equipment and the high cost of labor in construction and maintenance of bridge structures, the use of flexible metal pipe under high fills exceeding 100 feet has become competitive with bridges. While sufficient information has been available to apply corrugated metal pipe safely under high fills, it was recognized that field test data would aid in making such installations more effective.

Load tests were conducted on an installation of three parallel, 84-inch-diameter, Multiplate pipes under 137 feet of embankment which carries US 31 across Hurricane Creek in Cullman County, Alabama. The depth and span of the gorge as well as the volume of the flow in the creek ordinarily would have called for a bridge.

CONSTRUCTION

A section through the center pipe and fill is shown in Figure 1.

The pipes are 512 feet long, laid on a grade of 2.5 percent, and spaced 20 feet, center to center. The pipes were parabolically cambered 6 inches higher at the midpoint than a straight line joining the ends. Pipes were field-strutted to an essentially elliptical cross-section using 8-by-8-inch oak struts and 2-by-8-inch pine compression caps (Figure 2). This caused the vertical axis of the cross-section to be 3 percent greater than original diameter. The struts were placed at 3-foot centers in the middle half of the pipe length and at 6-foot centers at the ends. The pipe was fabricated from 1-gage (0.281-in.) plates at the center, stepping down to 3- to 5- to 8-gage (0.172-in.) at the ends.

After diverting the stream the pipe lines were laid on a 2-foot uniform bed of creek-bed sand. Fill material beneath, between, and to a depth of 3 feet over the pipes was selected granular material compacted to a 100-percent Proctor density value. Springs encountered in preparing the bed were handled with 6-inch corrugated-metal underdrains.

Prior to backfilling, the outside of each pipe was sprayed with an asphalt-base protective coating. Bolts were tightened with pneumatic impact wrenches and checked with a torque wrench to insure a screw-up torque of not less than 150 ft.-lb.

In the imperfect trench method of construction, the conduit is first installed as a positive projecting conduit. Then a fill is constructed over it up to an elevation which is 1 to 1½ times the width of the conduit (Figure 3). The fill is well compacted during construction, and then a trench as wide as the conduit width is dug in the compacted fill directly over the conduit down to its top. The trench is then refilled with loose backfill material to the original level and the embankment continued in conventional manner.

The purpose of the imperfect trench method is to allow the interior prism of fill material to settle downward relative to the exterior prisms so that the settlement will

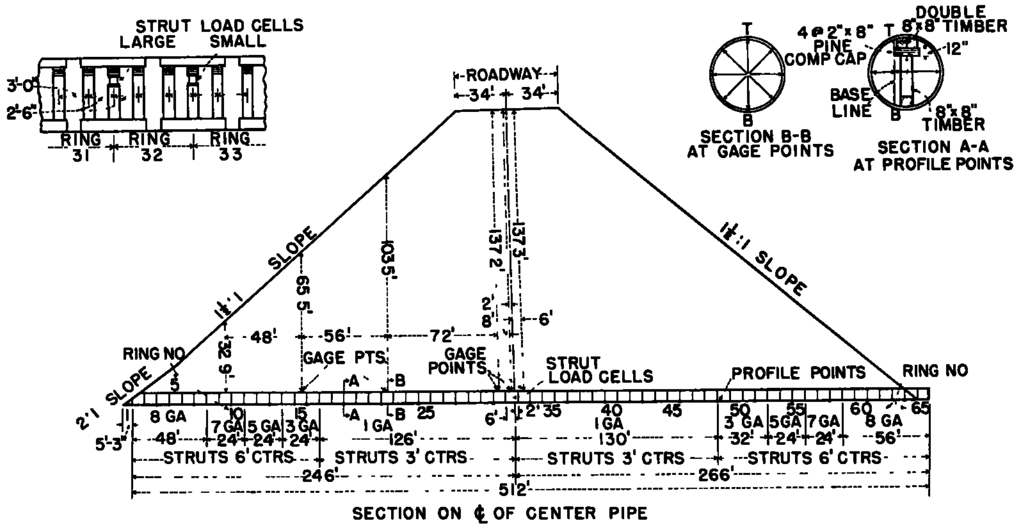

Figure 1. Section through center pipe showing test locations and fill profile.

generate shearing forces which are directed upward, reducing the load that the pipe must support.

In this particular installation at a fill height (above the top of the pipe) of 10 feet, a 7-foot-wide and 7-foot-deep trench was cut over each pipe and backfilled with loose material before proceeding with normal fill operations.

The fill was placed in layers of 3-foot maximum depth. For the first 10-foot depth of cover, rock was kept away from the area directly over the pipes to keep from disturbing the banks when the imperfect trench was excavated.

The fill was mainly a crumbly sandstone and rock for the first 25 feet. Choking was accomplished by alternately placing layers of rock and earth, except for the first 10 feet immediately over the pipes. The lower fill was handled with Euclid wagons over steep haul roads, however as the fill progressed it was largely built using self-loading scrapers. Compaction was accomplished by dual-tandem sheepsfoot rollers.

Upon completion of the fill the slopes were dressed, apron beddings cleared, and the struts pulled from the structure. The aprons and boulder deflector were constructed and the toe of the slopes paved with grouted riprap. The invert of the pipe was paved for a third of the circumference with bituminous concrete.

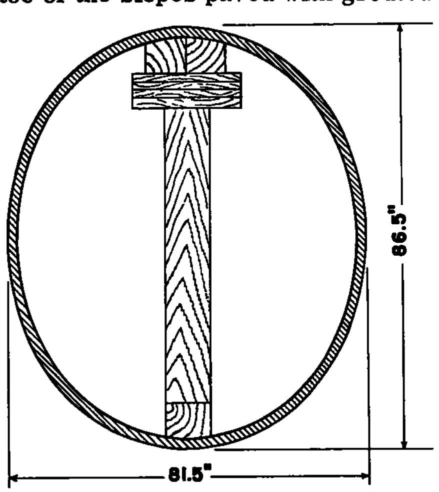

Figure 2. Schematic diagram showing 3% vertical elongation of pipe by use of timber struts.

TEST MEASUREMENTS

Plate stresses, vertical and horizontal deflections, invert elevation and side shift, compression-cap deformation, and strut loads were measured at various intervals during the grading operation. The height of the fill at the time of the test measurements is shown by the fill cross-sections of Figure 4. Location of various test equipment and reference points are shown in Figure 1.

Strain gages were applied on the inside of the pipe at the neutral axis of the corrugation at vertical and horizontal diameters. The gages were located at the center of Rings 31, 32, and 33 under the center of the fill and at the center of Rings 9,

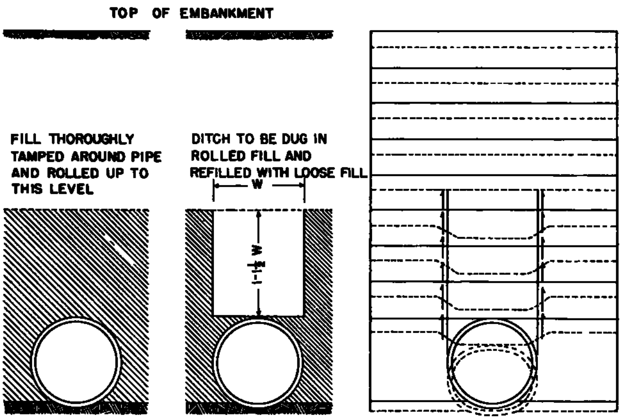

Figure 3. Diagram showing imperfect trench construction and subsequent settlement of interior prism with resulting upward shear forces.

15, and 22, which are 65 feet, 113 feet, and 173 feet from the upstream end of the pipe.

Readings of vertical and horizontal deflection were made at every third plate (24 feet) and at every strain-gage location. Vertical deflections were made on a chord parallel to the vertical diameter 12 inches to one side of the vertical diameter. Vertical, horizontal, and also 45-deg. deflection readings were made at the strain-gage stations.

Invert settlement readings and side-shift readings were made every 24 feet along the complete length of the pipe by means of level measurements.

The deformation of the four 2-by-8-inch pine compression caps was measured to a nail point on the oak strut 4 inches below the top of the strut. A set of these pine compression caps was calibrated under similar loading conditions in a universal testing machine.

Two strut load cells were placed between the struts and compression caps at the center of the pipe under the maximum fill height. These load cells were constructed from 6-inch steel pipe each of a different wall thickness to insure that the load range would be covered by a cell of suitable sensitivity. Wire strain gages were installed on the inside of the load cells before top and bottom bearing plates were assembled. Extreme care was taken to waterproof all strain gage installations. These load dynamometers were calibrated in a universal testing machine.

Test measurements were made at fill heights (above the top of the pipes) of 3 feet, 10 feet, (before and after trenching), 25 feet, 45 feet, 60 feet, 100 feet, and 137 feet (before and after removing struts).

TEST RESULTS

The maximum vertical deflection upon removal of the struts was 1. 84 inches or approximately 2 percent of the original diameter. This occurred at the strain-gage location at Plate 15 (Figure 5). The deflection measurements made on the vertical chords at every third circumferential seam indicate a maximum deflection of about 1. 2 inches at Plate 15 (Figure 6). This point of measurement is located approximately 4 feet from the aforementioned strain-gage location. In no case did the pipe return to its original round shape. The pipe remains from ½ to 2½ percent vertically elongated.

The deflection, in general, followed the pattern that would be expected by examining the fill cross-section, with the exception that, at Plate 15 (about 117 feet from the inlet end), the pipe deflection was slightly greater than the deflection under the full height of the fill. While the reason for this cannot be conclusively established, certain factors which could have contributed to this result can be identified. First, the strut spacing at this point changed from 6 feet to 3 feet, and the plate changed from 3-gage to 1-gage material. Second, this part of the pipe was continually under the haul road, until about 80 feet of fill had been placed. This, of course, meant that the fill and pipe under this ramp were being continually subjected to the

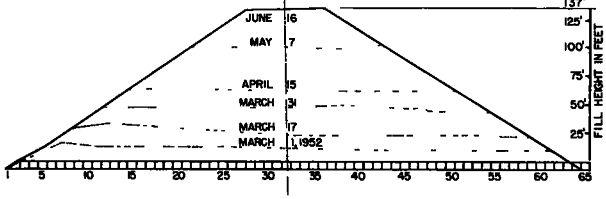

Figure 4. Fill cross-section.

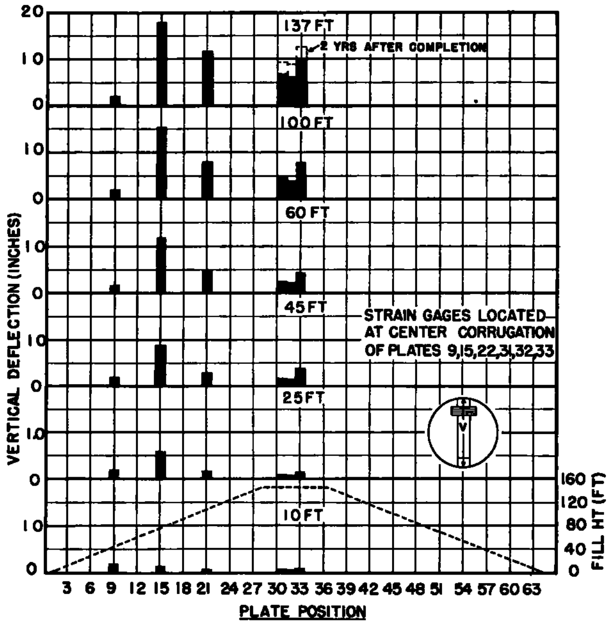

Figure 5. Vertical deflection at strain gage positions.

compaction of heavy construction equipment. Thirdly, since all struts and sills were only approximately the same length, a shorter strut (smaller installation strut load) could have resulted in a greater deflection.

Since the increase in horizontal diameter is practically the same as the decrease in vertical diameter, the graphs of horizontal deflection have been omitted. The average diameter of the pipe appears to remain practically a constant. This illustrates the complementary effect of changes in horizontal and vertical diameters.

The invert settlement, as shown in Figure 7, indicates a maximum settlement of 4.44 inches. As originally installed, the pipe had 6 inches of camber at the center. From the invert settlement curve as shown in Figure 7, the average settlement at the ends was approximately 2 inches. Deducting this from the invert settlement at the center of the pipe, it appears that about $2\frac{1}{2}$ inches of camber were lost (about $3\frac{1}{2}$ inches of camber retained.)

The settlement curve approximates the fill weight distribution. To a certain extent the deflection and invert settlement measurements are complementary. That is, positions that show relatively higher deflections tend to show relatively less invert settlement, and conversely. This probably means that where the invert fill was well compacted greater pipe deflections resulted than where the invert fill was softer. This, of course, assumes that the side fill material was tamped to a similar degree of compaction.

One of the design criteria for corrugated-metal structures is the bolt load at the longitudinal seam. Thrust determinations based on plate loads developed by a unit length of plate lead to evaluation of joint load. For ease of computation, all strut load and plate stress measurements were converted to load per foot of seam.

The strut load per foot of pipe per side is shown in Tables 1 and 2 and Figures 8 and 9. The strut load as obtained by measuring compression of the compression caps is shown in Table 1 and Figure 8. Although the points obtained from readings of the strut load cells at 137 feet were unstable, from the curve in Figure 9, it is evident that the strut load increased proportionately to the fill height up to approximately 60 feet. Above this fill height there is evidence that the strut load increased at a slower rate.

A comparison of the strut load

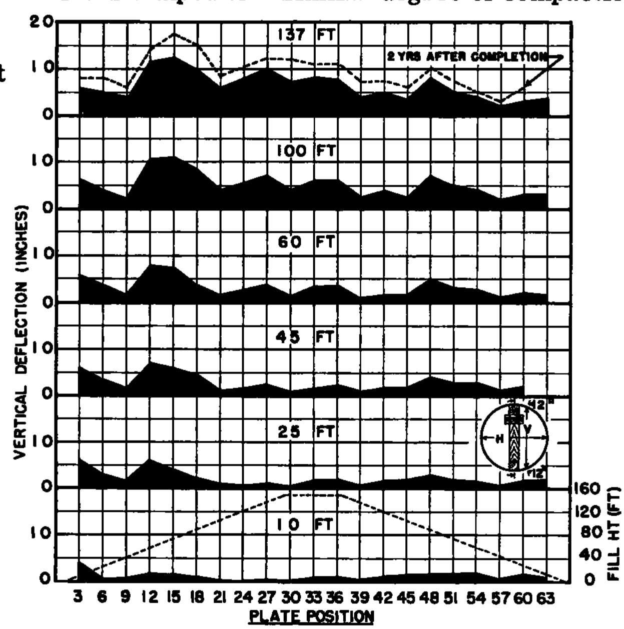

Figure 6. Vertical deflection at every third plate junction (24-ft.).

with the load carried by the plate on the basis of 1 foot of seam length indicates that at low fill heights (low loads) the struts carried an appreciable amount of the total load, while at high fill heights (higher loads) the struts carried a decreasing proportion of the total load (Figure 10), less than 20 percent of total load at final grade.

The plate loads as measured by the strain gages are shown in Table 3. These results were taken from the readings of the gages on the north and south side only. This practice was followed because the bottom gages,

TABLE 1

STRUT LOAD PER FOOT BASED ON STRUT LOAD CELLS

Fill Height ft.	Load: Large Cell lb.	Load: Small Cell lb.	Load per Ft. per Side: Large Cell lb.	Load per Ft. per Side: Small Cell lb.	Load per Ft. per Side: Ave.
12	7,000	8,000	1,400	1,600	1,500
25	12,000	16,000	2,400	3,200	2,800
45	22,000	27,000	4,400	5,400	4,900
60	30,000	34,000	6,000	6,800	6,400
100	40,000	44,000	8,000	8,800	8,400
137	52,000	39,500	10,400	7,900	9,150

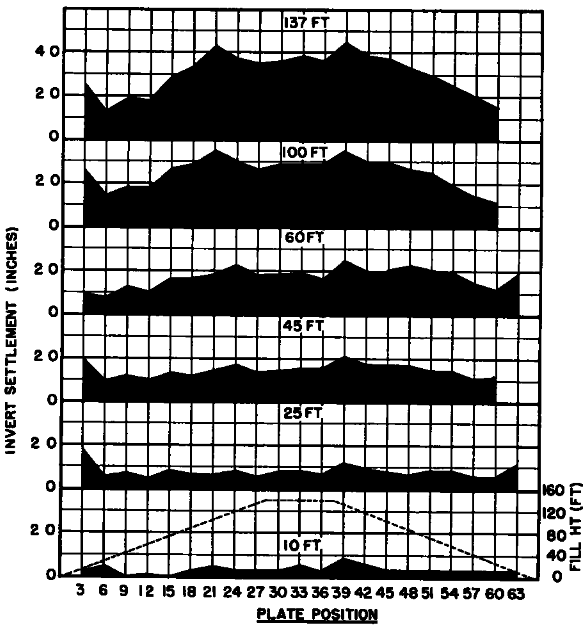

Figure 7. Invert settlement.

which were subjected to water and debris continually after installation, gave inaccurate readings shortly after installation. While the top gages were operative throughout the tests, some high strains were noted, particularly at the higher fills. It is believed that this was due to the fact that local bending of the plate occurred over the top sill.

The total load per foot of seam was computed as the sum of the strut load per foot per side plus the seam load as determined by strain gages (see appendix). The results of these calculations are shown in Figure 11 and Table 4. A comparison of the total load per foot of seam as determined in the test with the theoretical total load per foot of seam appears in Table 4. The theoretical seam load is based on a projected column of earth above the pipe with a density of 118 pcf. as averaged from the dry weight per cubic foot of embankment as determined by the State Highway Department of Alabama.

Figure 12 shows a plot of the total seam load per foot versus fill height based on the test results and on the weight of the projected column of earth. This plot indicates the agreement of the test results with the design calculations and shows that each increment of fill added its load in almost direct proportion to fill height.

The test results are of the same order of magnitude as the theoretical, except in the case of Plate 15 (which was under the haul road) where the

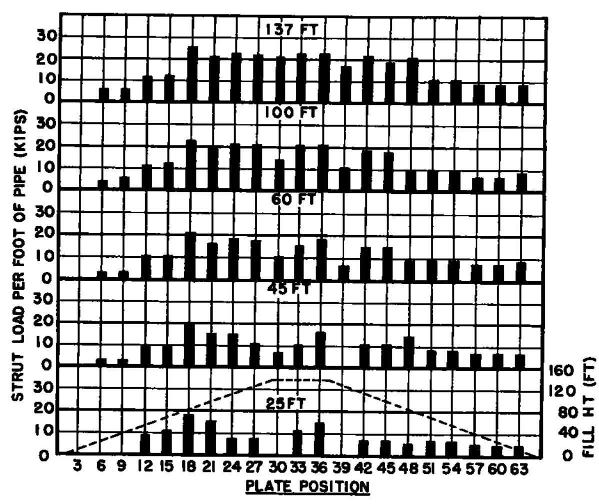

Figure 8. Strut load per foot of pipe.

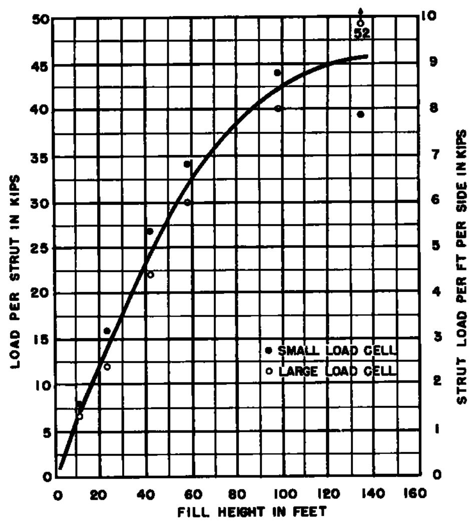

Figure 9. Fill height versus load per strut as measured by strut load cells.

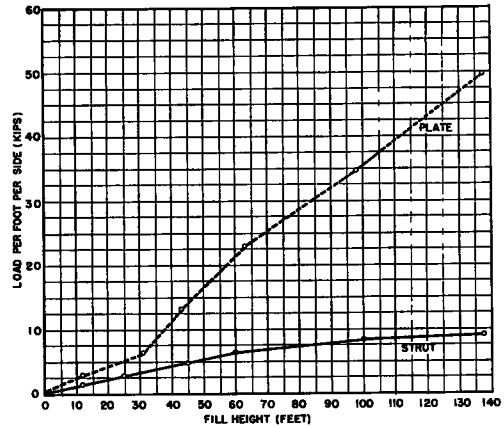

Figure 10. Comparison of strut load and plate load versus fill height for Plate 32 under center of fill.

test results appear over 100 percent higher than the theoretical. While this relatively high load cannot be wholly explained, the high deflection and strut load at this location coupled with the low invert settlement partially substantiate the higher load. At Plate 32 the agreement of the test results with the theoretical are satisfactory, having a maximum disagreement of approximately 18 percent, with most values falling within 10 percent.

The pipe showed a maximum horizontal shift of about 0. 8 inch from a base line established when the pipe was covered with 3 feet of fill.

TABLE 2

STRUT LOAD PER FOOT PER SIDE BASED ON COMPRESSION CAP DEFLECTION

Plate Position	25 Feet	45 Feet	60 Feet	100 Feet	137 Feet
3-4	0	0	0	0	0
6-7	0	1667	1667	1667	2917
9-10	0	1667	1667	2917	2917
12-13	4250	5170	5250	5520	5625
15-16	5170	5250	5520	6333	6625
18-19	9000	9666	10500	11833	12500
21-22	7500	7500	7500	9667	10666
24-25	3333	7500	9083	10667	11833
27-28	3333	5833	8500	10333	11250
30-31	0	3333	5833	7500	10333
33-34	5833	5833	7500	10333	11250
36-37	7500	8500	9083	10667	11833
39-40	0	0	3333	5833	8500
42-43	3333	5833	7500	9083	11250
45-46	3333	5833	7500	8500	9667
48-49	2915	7500	4250	4833	10333
51-52	3750	4250	4530	5000	5250
54-55	3750	4250	4530	5000	5250
57-58	2915	3750	3750	3750	4250
60-61	2915	3750	3750	3750	4250
63-64	2915	3750	4250	4530	4530

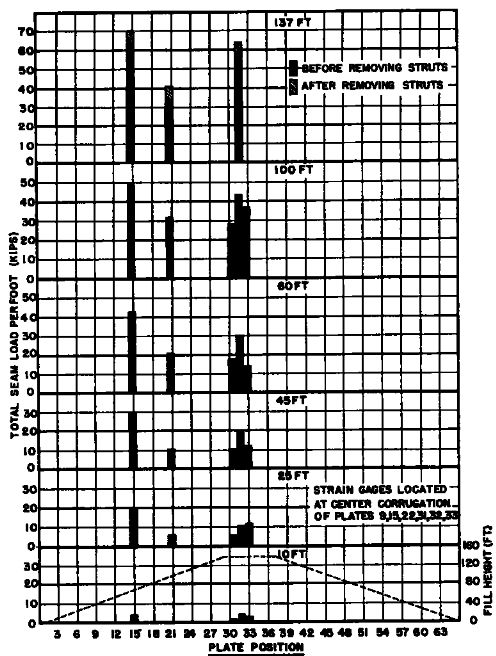

Figure 11. Total load per foot based on strut load plus seam load as determined by strain gages.

The deflection measurements showed little change before and after trenching at the 10-foot level, as did the load measurements. The deflection measurements upon removal of the struts at the completion of the fill showed practically no change, the maximum being 0. 05 inch. The load in the plate, however, did show a considerable increase, as would be expected, since the struts had been carrying part of the load.

Deflection and invert elevation readings were made two years after completion of the fill. The vertical deflection has increased an average of 0.2 to 0.3 inch, resulting in a maximum deflection of about 2.14 inches as compared with 2.5 inches of original vertical elongation as strutted. The increased invert settlement has been insignificant, generally less than 0.2 inch. The upstream end of the pipe actually indicated a rise of 0.4 to 0.6 inch.

The loads, as determined by the strain gages applied to the plate corrugations, are a direct measure of the load imposed on the plates by the weight of the fill. However, the final plate stresses cannot be determined, because the original tensile stresses due to strutting were not predetermined. Strutting caused the joints and plates to be placed under tension at an unknown tensile stress level. This was the condition of the pipe at the time the strain gages were installed and base readings made.

Vertical and horizontal diameter measurements made at every third circumferential seam on the other two pipes in the fill indicate that these pipes also retained some of their original vertical elongation, except at one location in the south pipe where diameters were approximately equal.

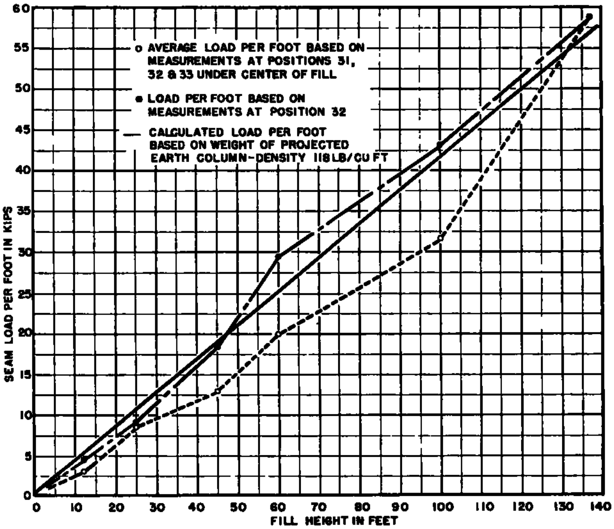

Figure 12. Comparison of measured loads and calculated under center of fill.

TABLE 3

SEAM LOAD PER FOOT BASED ON STRAIN GAGE DATA AVERAGE OF HORIZONTAL DIA. GAGE POSITIONS

Height ft.	Load in Pounds per Foot at Plant Number				
	No. 15	No. 22	No. 31[b]	No. 32	No. 33
12	+545	+3,040	-3,000	-3,000	-2,400
	-3,790	+2,720	-1,800	-3,000	-3,000
31	-14,350	+900	-6,000	-6,400	-4,800
43	-23,560	-2,390	-6,600	-13,500	-4,800
61	-36,200	-11,960	-11,400	-23,100	-5,700
98	-41,100	-20,960	-19,800	-34,800	-14,400
137	[c] -56,250	-22,750[a]		-49,800	
	[d] -68,750	-39,500[a]		-63,900	

[a] Based on reading of N side only
[b] Based on reading of S side only
[c] Before strut removal
[d] After strut removal

TABLE 4

TOTAL LOAD PER FOOT OF SEAM CALCULATED VERSUS MEASURED

Height	Total Load per Foot of Seam in Pounds at Plate Number									
	15		22		31		32		33	
	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.	Calc.	Meas.
12	-4950	+545[a]	-4950	+3040	-4950	-3000	-4950	-4500	-4950	-2400
		-3790[b]		+2720		-1800		-4500		-3000
25	-10330	-19520	-10330	-4270	10330	-6000	-10330	-9200	-10330	-10633
45	18600	-28810	-18600	-9890	-18600	-9933	-18600	-18400	-18600	-10633
60	24800	-41720	-24800	-19460	-24800	-17233	-24800	-29500	-24800	-13200
100	27150	-47433	-41300	-30627	-41300	-27300	-41300	-43200	-41300	-24733
137	27150	-62875	-43300	-33416	-56600		-56600	-58950	-56600	

[a] Before trenching
[b] After trenching

CONCLUSIONS

1. The vertical load on the pipe, as shown by strain readings, agrees substantially with the weight of the vertical column of fill immediately above the pipe. This is the weight ordinarily used to design culverts under high fills.

2. Lack of knowledge of the stresses imposed by the strutting operation prevented complete measurement of the stress on the pipe.

3. Part of the elongation of the vertical diameter still remained in the structure after completion of the fill and removal of the struts. This shows that the amount of elongation by strutting the strength of the pipe, compaction of the fill and the spacing of the struts had all been given proper consideration in the design.

4. The invert camber of 6 inches above straight grade, as originally established by foundation analysis, proved adequate, as about 3 inches of this camber remained after completion of the fill.

5. Analysis of the observed load distribution on the pipe shows that the methods used to construct a fill can affect the amount of load transmitted to the structure. In the construction of this fill, the material was repeatedly brought in over the pipe at one place. Greater loads were measured at this point than the weight of the vertical column immediately above the structure, because of this "hard spot" in the fill. In constructing such large fills, it would appear desirable to place the material in relatively thin horizontal layers and compact them uniformly so as to transmit the load to the original ground and to the structure in a uniform manner.

6. Data taken on strut cells and pipe wall strain gages at successive stages of fill height show that each increment of fill added its increment of load in almost direct proportion.

7. In this installation the use of the "imperfect-trench method" to reduce the loads on the structure was of only temporary value, as the load transmitted to the pipe substantially equaled the weight of the vertical column immediately above the pipe when the fill was completed.

The many factors involved in the construction of a fill of such proportions make it difficult to draw conclusions of other than a general nature. Such conclusions as are made can be specifically applied only to this test. However, general trends can be predicted. The indications are that flexible pipe under reasonably high fills can have the vertical diameter timber strutted in the field and the backfill around the pipe can be so compacted that little or no change in the pipe diameter will occur during the building and consolidation of the fill.

ACKNOWLEDGMENT

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References

1. Spangler, M. G., Soil Engineering, 1951 p. 427-430.
2. Tribble, J. F., Roads and Streets, Volume 95, Number 6, June 1952.

Appendix

Calculations that were used in processing the data appear below:

1. Strut load per foot per side.

$$\text{Strut load per foot per side} = \frac{\text{Strut load}}{\text{Strut spacing x 2}}$$

At Plate 15 for 137-ft. fill height

Compression-cap deformation	- $1\frac{5}{16}$ in.
Strut load	- 79,500 lb.
Strut spacing	- 6-ft.

$$\text{Strut load per foot per side} = \frac{79,500}{6 \times 2} = 6,625 \text{ lb.}$$

2. Plate load based on strain-gage readings.

Horizontal Gage Position	Strain in Micro-inches
North	365
South	675

Stress for north gage = $30 \times 10^6 \times 365$ = 10,950 psi.
Stress for south gage = $30 \times 10^6 \times 675$ = 20,250 psi.
Cross-sectional area of one foot of plate = Formed length x 1.21 x thickness
Cross-sectional area of one foot of plate = 12 x 1.21 x .249 = 3.61 sq. in.
Average load per foot of plate = Average stress x cross-sectional area

$$= \frac{10,950 + 20,250}{2} \times 3.61 = 56,250 \text{ lb.}$$

3. Total load per foot of seam.

$$\text{Total load per foot} = \begin{matrix}\text{Seam load per foot} \\ \text{from strain gages}\end{matrix} + \begin{matrix}\text{Strut load per} \\ \text{foot per side}\end{matrix}$$

$$\begin{matrix}\text{Total load per foot of seam} \\ \text{for Plate 15 at 137 feet fill}\end{matrix} = 56,250 + 6,625 = 62,875 \text{ lb.}$$

4. Load per foot of pipe based on theoretical weight of fill.

(a) $W = W_1 + W_1 + W_S$

where W = load per lineal ft. of pipe
W_1 = load per lineal ft. per side
$\frac{W_S}{2}$ = Strut load per lineal ft. per side

(b) $W = D \times h \times c$

where D = pipe diameter (ft.)
h = fill height (ft.)
c = unit weight of fill (density)

(a) $W = W_1 + W_1 + W_S$
$W = 2W_1 + W_S$

when struts are removed $W_S = 0$

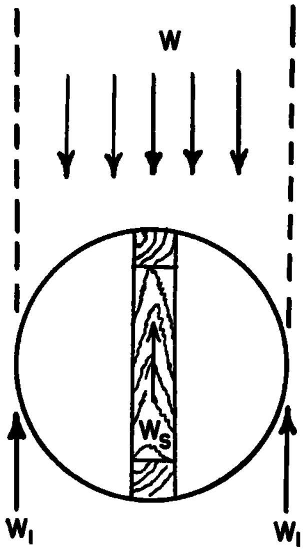
then $W - 2W_1$

(b) $W_1 = \frac{W}{2} = \frac{D \times h \times c}{2}$

$W_1 = \frac{D \times h \times c}{2}$ where D = 7 ft.
h = 65.6 ft.
c = 118 pcf.

$$W_1 = \frac{7 \times 65.5 \times 118}{2}$$

W_1 = 27,150 lb. for fill height of 65.6 ft.



Discussion

M. G. SPANGLER, Department of Civil Engineering, Engineering Experiment Station, Iowa State College — Timmers' paper marks a significant milestone in the advance of our knowledge relative to the loads imposed upon culverts under earth fills. His observations show conclusively that the load on the structure increased with every increment of height of fill up to the maximum of 137 feet which prevailed on this job. This fact is definitely in harmony with the results indicated by Marston's "Theory of Loads on Conduits."

In the experimental research which accompanied the development of Marston's theory, the maximum height of fill employed was about 21 feet. In some concurrent experiments of a similar nature conducted by the University of North Carolina, the maximum height of fill used was about 12 feet. The American Railway Engineering Association, in 1926, published results of some field measurements of loads on pipe culverts under a railway embankment near Farina, Illinois, in which the height of fill was 35 feet. Still later, in 1947, Wilson V. Binger, of the U. S. Corps of Engineers, described measurements of load on a concrete box culvert in Panama under 51 feet of fill. In all of these load measuring projects, the principle of linear relationship between load and height of fill above the plane of equal settlement was demonstrated.

Furthermore, in the North Carolina project, reported at the 1955 Annual Meeting of the Highway Research Board, by Costes and Proudley, preliminary indications are that this same principle held in the case of a culvert under 168 feet of fill, although final digest of the data on this job is not complete.

These qualitative checks on the validity of the Marston theory are of extreme importance at this time, because the requirements of modern highway transportation will undoubtedly result in the need for much higher fills as the highway program goes forward.

The writer would like to comment relative to Timmers' Conclusions 1 and 7, in which it is stated that the vertical load on the pipe was substantially equal to the weight of the prism of soil directly above the pipe and, therefore, that the imperfect-ditch method of construction was of only temporary value as a procedure for reducing load. If we limit consideration of measured loads to Sections 31, 32, and 33, which were under the roadway portion of the embankment and were not influenced by haul roads or by the side slopes of the embankment, it is seen that the average load on the culvert was only 75 percent of the weight of soil above 100 feet of fill (see Figure 12 and Table 4). The next load measurements were made at 137 feet of fill, when it is noted that no data were obtained for Sections 31 and 33. The only load measurement at this height of fill was on Section 32, which had consistently indicated a load approximately a third greater than the average of these three sections throughout the period of observation. A study of the data given in Figure 12 and Table 4 leads this writer to conclude that the average load on Sections 31, 32, and 33 was about 75 percent of the weight of the prism of soil directly above the pipe. Therefore, it is also concluded that the imperfect-ditch method of construction was effective to a substantial degree in accomplishing a reduction in load on the culvert.

As a further comment, although Timmers' paper does not state specifically, it is the writer's understanding that the backfill material in the imperfect trench consisted of the same crumbly, friable, sandstone soil which was removed during construction of the trench. Although this material was replaced in the trench in a loose state without compaction, from the description of the soil, it is probable that it was not very compressible material.

Most effective results of the imperfect-ditch method of construction are obtained when the trench backfill is a highly compressible material. As a matter of fact, when Marston invented this method of construction, he recommended that, in some cases, high compressibility of the backfill material might be achieved by filling the trench part way with straw, hay, cornstalks, or brush to obtain the desired result. The writer does not know of an instance where this recommendation has been followed in actual construction, but it serves to emphasize the desirability of exerting special effort to achieve high compressibility of the trench backfill, which will result in greater reduction in load on the structure.

JOHN H. TIMMERS, Closure — Spangler's comment that the author's paper marks a significant milestone in the advance of knowledge relative to loads imposed upon culverts under earth fills is much appreciated. The author would like to thank him for pointing out that the test observations show results in agreeance with Marston's theory.

The marshalling of substantiating evidence from past tests as stated in Spangler's discussion adds considerably to the value of the author's report.

In discussing the comments relative to Conclusions 1 and 7, the author has no wish to refute Spangler's interpretation of the data. He would like, however, to present his reasons for his interpretation of the data. In regard to Conclusion 1 of the report, in which it is stated that vertical load on the pipe was substantially equal to the weight of the prism of the soil directly above the pipe, the author would like to say that this conclusion was drawn largely from the readings taken on Section 32, which had consistently indicated more reliable action of the strain gages than any of the other sections.

Since this was the initial use of electrical strain gages in an installation subject to inundation of the gages, no precedent existed for waterproofing the gages. Several different methods of waterproofing and protecting against physical damage from water-borne rocks, etc., were made up in sample form and tested in the laboratory. Of these methods, the most promising were selected for use in the test; of these, the gages in Section 32 proved the most reliable. Several of the other gages displayed instability in readings and some became inoperative — among these were the gages of Sections 31 and 33, which had shown sluggish responses as compared to gages of Section 32 and could not be read for the final 137 feet of fill. It should be noted that the test structure had flowed full several times during the construction of the fill.

Hence, from actual close personal contact with the gage readings as the work progressed the author believed the data from Section 32 to be the most reliable and indicative of the loads on the structure. When the weight of the vertical column above the structure was compared to the loads computed from the strain gages of Section 32, the magnitudes were the same and since there was no change in the slope of the curve as the fill height increased the author could see no permanent reduction of load from the imperfect trench construction.

Thus the author arrived at what he considered the best and most-logical interpretation of the data. However, knowing that other interpretations were possible and plausible, the entire data was published.

There could even be argument as to the exact weight of the vertical column of earth above the pipe. The author chose to use the dry density of 118 pcf., as determined from samples taken adjacent to the structure itself. Although the moisture content was 10 percent, it was believed that the dry density of the thin layers well compacted near the structure, most closely approximated the moist weight of the material placed in 3-foot lifts above the structure. Hence, the value of 118 pcf. was used, since no other data was available on the precise weight of the material above the structure.

In concluding the author would like to point out that Spangler's interpretation based on averages of readings is just and plausible and can be as nearly correct as that of the author based on the most-severe readings from the gages considered most reliable. Both interpretations do not differ in magnitude but in detail and both confirm that present design practice is conservative.