

Rebuilt Wolf Creek Culvert Behavior

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The failure of an 18.5-ft diameter structural-plate culvert under 83 ft of cover led to its reconstruction using the imperfect trench type of construction as well as other changes. The culvert was instrumented with electrical resistance strain gages placed on the culvert walls at approximately mid-height; Carlson Soil Stress Meters placed on the outside walls of the culvert and in the fill; rubber pressure cells adapted from commercial hot water bottles placed in the fill; and settlement cells, which were placed in the fill on each side of the culvert.

The results correlated well and demonstrated that the vertical load on the culvert was much less than the weight of the overlying column of earth. The strain gages, which measured bending strains in addition to circumferential compression, showed that significant residual bending stresses were induced in the plates during erection. They also monitored bending stress changes that occurred at the strain gage sites during and after the backfilling operation. The vigorous compaction of the backfill on each side of the culvert during the early stages of backfilling produced a bending stress pattern in the side walls that persisted throughout the embankment construction period and thereafter.

•THIS PAPER is a condensation of a final research report (1) that describes the behavior of an 18.5-ft diameter structural plate culvert, built of $\frac{3}{8}$ -in. thick plates, that was rebuilt in 1965 after the failure of the original structure in 1964 under 83 ft of rock fill in Wolf Creek Canyon on Interstate 15 near Helena, Montana.

The failure and the design details of the original culvert were reported by Kraft and Eagle (2). Research on the cause of the failure was reported by Macadam (3). His report supported the hypothesis that the failure was caused by hydrogen embrittlement of the high-strength, $\frac{3}{4}$ -in. A490 bolts used in the original culvert, and prompted a decision that the culvert be reconstructed using $\frac{7}{8}$ -in. A325 bolts of milder steel.

In the reconstruction, a 3-ft thick layer of baled straw was placed 5 ft above the culvert and served as the primary element of an imperfect trench type of load relief detail. Figure 1 shows a typical cross section of the rebuilt culvert.

RECONSTRUCTION OF CULVERT AND INSTALLATION OF INSTRUMENTATION

Following the removal of the rock fill from the damaged culvert in the winter and spring of 1965, disassembly of the failed portion was begun and steps were taken to divert the stream. Unprecedented high water thwarted all stream diversion attempts until September, when a successful diversion was finally effected. Both of the undamaged end sections, with a combined length of over 200 ft, were left intact. Only the 328-ft long central section that had failed was rebuilt.

The culvert bedding was high-quality granular backfill material shaped to fit the culvert curvature. The bedding was completed in the middle of October 1965.

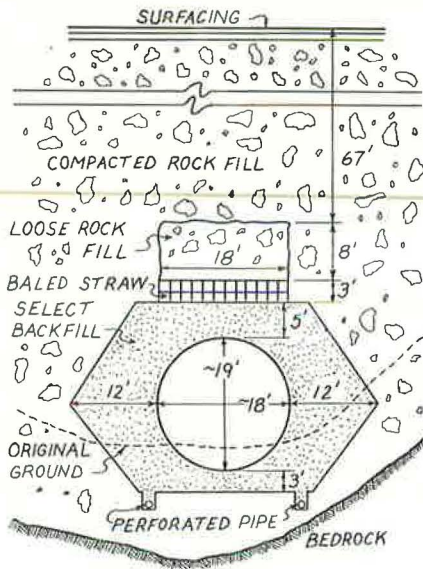


Figure 1. Typical cross section of rebuilt culvert.

The bottom plates were installed first, between the intact end sections. Then the culvert contractor erected plates from each end toward the middle. The culvert was erected in an elliptical shape, approximately 19 ft 5 in. by 17 ft 7 in. Erection of the culvert was essentially complete by the beginning of November and control devices were installed to check movement or rotation of the culvert during backfilling operations. Each control device consisted of a $\frac{3}{4}$ -in. rod, 3 $\frac{1}{2}$ ft in length, welded to a base plate bolted to the culvert floor. A plumb bob was suspended from the roof directly above the rod. The rod was protected from gravel, boulders, and other debris by a section of corrugated metal pipe. This section of pipe was 15 in. in diameter and 3 ft in length and was welded to the base plate. The control devices also served as sites for vertical diameter measurements and floor settlement measurements.

While the culvert was being erected, and with the contractor's cooperation, an interior walkway was constructed on each side of the culvert and crosswalks were placed at each station where vertical and horizontal dimensions and elevations were to be taken.

Backfill work started the second week of November 1965. The backfill adjacent to the culvert was a crushed granular material of base course quality and was compacted by pneumatic-tire rollers, supplemented by hand tamping, in layers approximately 6 in. thick. Minimum specified density was 95 percent of AASHO T-99, Method D. The material was placed to a depth of 5 ft above the culvert and 12 ft on each side, as indicated in Figure 1.

In late November and early December the straw was placed on the backfill material to form the imperfect trench. Loose rock fill was then placed on the straw and not compacted, nor was any equipment allowed to cross this section until the loose material reached a height of 8 ft above the straw. The remaining fill, consisting of material that was dumped by various hauling equipment, was then pushed into place and continually worked by a D-9 Caterpillar.

The rock fill was completed early in January of 1966 and a gravel surface was placed as a temporary measure to open the highway to traffic. The final surfacing of asphaltic concrete was placed in August of 1966.

Strain Gage Installation

In August of 1965, while stream diversion attempts were still under way, six of the new $\frac{3}{8}$ -in. thick corrugated structural plates were placed in an easily accessible outdoor work area so that the SR-4 strain gages could be applied, waterproofed, and checked before the plates were installed.

The strain gages selected for this project were epoxy-backed type FA-100-12. This type of gage is a metallic-foil strain gage, which offers a number of advantages over the older paper-backed wire strain gage. One primary advantage, because of its epoxy backing, is its superior long-term stability under adverse environmental conditions.

The gages were installed and their functional behavior checked according to the manufacturer's recommendations, after which they were waterproofed and protected by two layers of epoxy cement and a layer of silicone rubber sealant.

Six strain gages were installed to measure circumferential strains on each instrumented plate, as shown in Figure 2. The gages were placed in pairs, on opposite sides of the plate, at three locations: at the neutral axis of the corrugations and at the bends on each side of the neutral axis. By placing the gages in these positions, the bending stresses and bending moment could be computed and the average compression determined as indicated in Figure 2.

The six plates, containing a total of 36 gages, were identified as plates A, B, C, D, E, and F. The strain gages located on the inside of the culvert, using plate A as an example, were numbered A1, A2, and A3, while those on the outside were A11, A22, and A33. Figure 3 shows the locations of the plates on which strain gages were attached. They were placed in the side walls of the culvert, at mid-height. The rings were numbered from 1 through 70 from downstream to upstream end, as they were in the original culvert, for identification purposes. Ring number 39 is under the centerline of the Interstate roadway. Notice from Figure 3 that plates A and B were installed in adjacent rings, in approximately diametrically opposed positions, as were plates C and D.

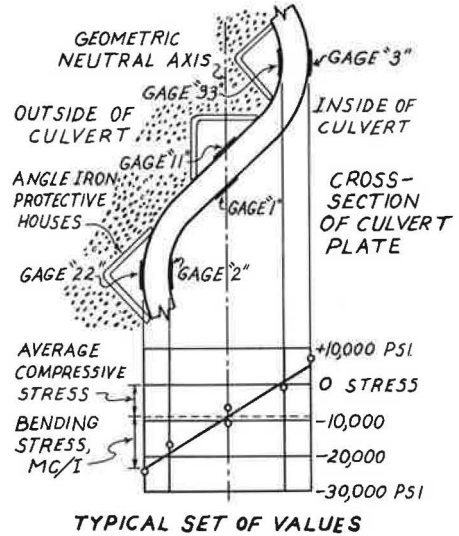


Figure 2. Position of SR-4 strain gages to measure circumferential strain in the corrugated plates.

Pressure Cell Installation

The pressure cell selected to measure earth pressure on the Wolf Creek culvert was the Carlson soil stress meter. The meter selected measures pressures up to 150 psi, is 7 1/4 in. in diameter, 1 in. thick, and has stem on the back that houses the wire resistance coils.

The shape of the meter, particularly the protrusion on the back, created difficulties in mounting it on the corrugated metal culvert wall. The Carlson meter is normally mounted in a rigid wall or footing with its sensitive face flush with the wall surface and against the soil. In this case a mounting frame was made to provide a rigid backing for the cell and to allow it to be mounted essentially flush with the outside surface of the culvert.

Installation of the stress meters was started as soon as the necessary plates were erected by the culvert contractor. The holes for mounting frame bolts were drilled and the holes for the cell stems were cut with an acetylene torch. Eighteen mounting frames were bolted to the culvert. In every case the bottom edge of the frame was butted against a bolted seam. Mounting frames were not used for

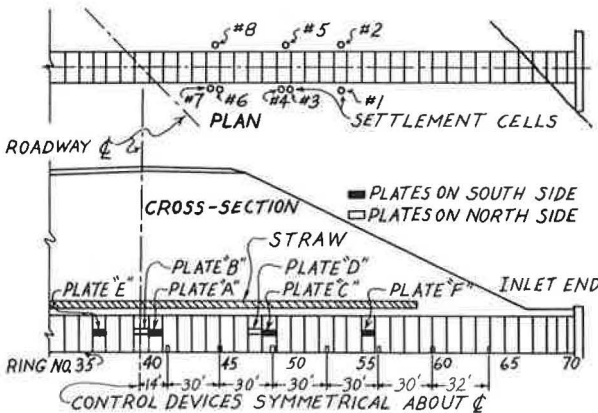


Figure 3. Location of SR-4 strain gages, settlement cells, and control devices.

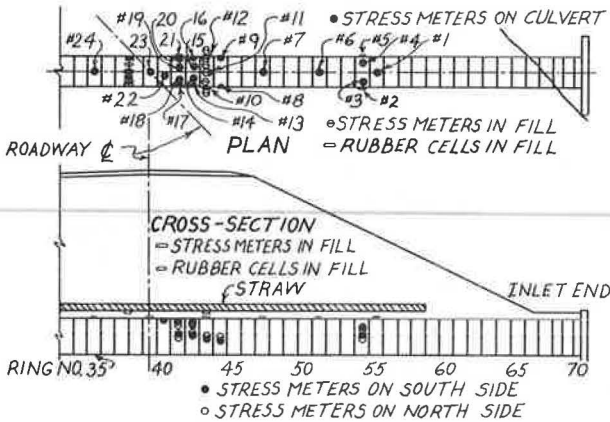


Figure 4. Location of Carlson stress meters and rubber pressure cells.

Flexible copper tubing ran from each bottle to a station inside the culvert where the tubing terminated in a bleeder valve, a shut-off valve, and a 100-psi capacity Bourdon gage.

The locations of the stress meters and rubber pressure cells are shown in Figure 4.

Settlement Cell Installation

Settlement of the soil adjacent to the Wolf Creek culvert was determined by the use of eight liquid level settlement cells similar to a type used by Schlick (5). The settlement cells were connected to a manometer board with tubing containing water. Any change in elevation of the cell was indicated directly by the fluid level in the manometer standpipe. The location of the cells is shown in Figure 3. The cells on the south side were placed about a foot above the top of the culvert. The cells on the north side of the culvert were placed about a foot higher than those on the south side to insure a gradient from the cell to the low point located below the manometer board at the culvert inlet.

DATA RECORDED

Strain gage readings were taken before the plates were installed in the culvert to establish a basis for zero strain.

After the culvert was erected and before any backfill was placed, readings were taken to determine the strains caused by erection. Readings were then taken several times weekly until the full fill height was reached. The usual interval between readings was increased to several weeks after the fill was completed.

Stress meter readings were taken for all meters at a central switchboard and were corrected for effects of cable length and temperature. Stress meter readings were taken at the same time that the strain gage readings were taken.

After the rubber pressure bladders were connected to the Bourdon gages and the air removed, the gage pointers were adjusted to read zero. This gave an initial zero reading for measuring earth pressure corrected for the pressure caused by the difference in elevation between the Bourdon gage and the rubber cell. Readings of the Bourdon gages were taken at the same time as those of the strain gages and the stress meters.

Each settlement cell was made operational by filling the system under pressure with a garden spray pump. The pump was connected to the valve at the low point in the system and fluid was forced into the line, with the top of the standpipe plugged to prevent overflow, until the lines were completely filled. At this time the valve was closed, the pump disconnected and the plug at the top of the standpipe removed. After the fluid settled in the standpipe to the level of the cell, its elevation was recorded. It was

those meters placed on the top centerline of the structure. These meters were placed in a bedding of sand-cement mortar.

Six stress meters were also placed in mounting frames and buried in the fill on a plane level with the top of the culvert, 1 ft above ring 43, to measure vertical earth pressure above the culvert.

In addition to the stress meters, six rubber bladder pressure cells of the hot water bottle type, using a mixture of water and antifreeze, were used to measure pressure under the baled straw above rings 37 and 38. Holes were drilled in the bottle stoppers and copper fittings attached with epoxy cement.

found that readings were erratic and unreliable unless the system was drained and re-filled each time a settlement reading was desired. Reproducibility was generally better than plus or minus 0.20 in. by this method.

Measurements of the horizontal and vertical culvert diameter were taken with a telescoping rod at 13 stations inside the culvert at the control devices, as soon as the culvert was erected and before any backfill was placed. Periodic re-measurements were made at the same stations during placement of the backfill and thereafter.

Floor settlement was obtained from periodic observations of the elevation at the top of each control device rod, using an engineer's level near the culvert inlet. Level readings were also taken periodically on the roadway surface above the culvert to determine if there was excessive settlement. These readings were started in September of 1966, as soon as the asphaltic concrete surfacing was completed.

RESULTS AND INTERPRETATION

Strain Gages

Strain gage measurements were taken before and immediately after the culvert was erected in order to determine circumferential erection stresses. The maximum extreme fiber erection stresses ranged from 3,800 psi in plate C to 21,900 psi in plate A. Plates B, D, and E might be considered typical, with erection stresses between 9,000 and 10,000 psi in the extreme fibers. Plate A was one of the last plates to be installed and had to be forced vigorously into place, which may account for its high stress of 21,900 psi.

The erection stresses are shown graphically in Figure 5. The excellent linearity of the stress diagrams indicates that the gages performed consistently.

The placement and compaction of the backfill at the sides of the culvert caused a compressive bending stress in the outside fibers and tensile bending stress in the inside fibers of the sidewalls. For plates A, B, D, and E, these bending stresses were directly additive to erection stresses and therefore produced relatively large compressive stresses in the outside fibers. The strain gages showed a steady increase in direct wall compression but little additional change in bending as the embankment was

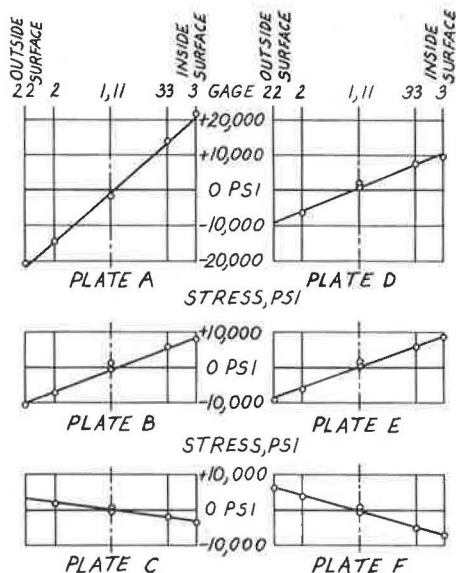


Figure 5. Erection stress in the six instrumented plates.

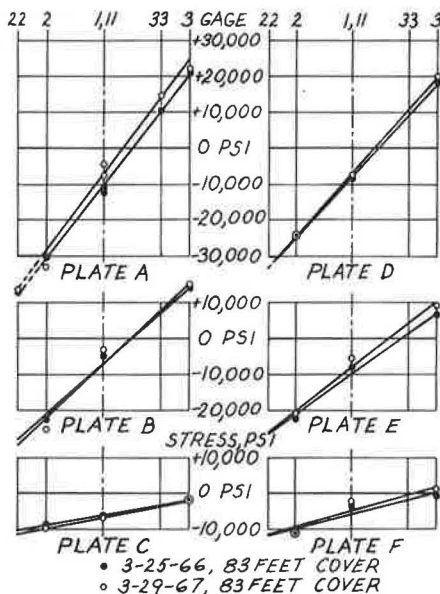


Figure 6. Combined stresses on March 25, 1966, and March 29, 1967.

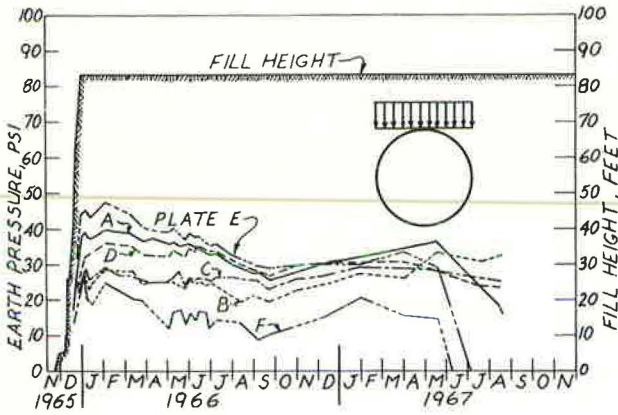


Figure 7. Equivalent uniform vertical earth pressures on culvert, deduced from SR-4 strain gages.

compressive stress by the wall sectional area per unit of culvert length and dividing by one-half the culvert width. The pressure was highest shortly after completion of the embankment and it eventually stabilized at a value near 30 psi, which is less than half of the nominal pressure of 72 psi that could be expected from an 83-ft depth of overburden weighing 125 pcf.

Plate F should be considered separately from the other five plates because it was under the sloping side of the embankment at a place where the depth of cover was only 45 ft.

Stress Meters

The earth pressures on the culvert walls were also determined directly with the Carlson soil stress meters. Figure 8 shows only the results from those meters that were mounted directly on the top centerline of the culvert. The stress meter results show generally good agreement with the strain gage results of Figure 7.

Detailed results from the stress meters on the sides of the culvert have been omitted from this report for the sake of brevity. In general, the meters on the sides regis-

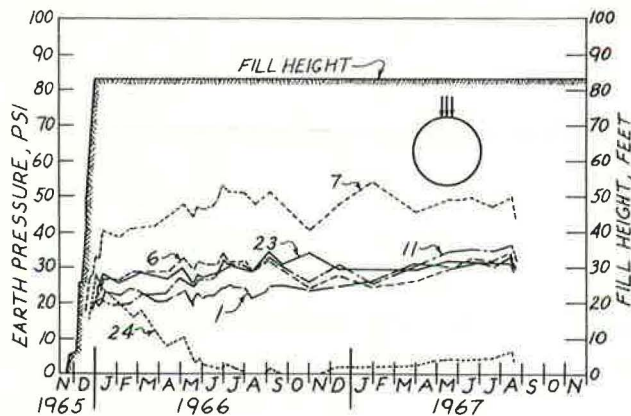


Figure 8. Earth pressure vs time—stress meters No. 1, 6, 7, 11, 23, and 24.

constructed to its full depth. Thereafter the changes were relatively small, as indicated by Figure 6, which shows the stresses approximately 3 months and 15 months following completion of the embankment. By the end of the first winter, many of the gages on the outside walls had deteriorated and no longer furnished usable readings. The performance of the gages on the inside walls did not deteriorate until the summer of 1967.

Figure 7 shows the average vertical earth pressure on the culvert as deduced from the strain gage results by multiplying the average wall com-

puted higher and more variable pressures than those at the crown. The overall stress meter results for a group of 15 meters clustered near the center of the culvert are summarized in Figure 9 for the single date of January 4, 1966, when pressures were near maximum. The figure shows an average lateral pressure near 50 psi, or greater, as compared with an average vertical pressure near 35 psi.

In Figure 9, the minimum and maximum pressure curves are curves that were drawn to mark the boundaries of the zone within which all 15 of the observed pressures fell on January 4, 1966. No special sig-

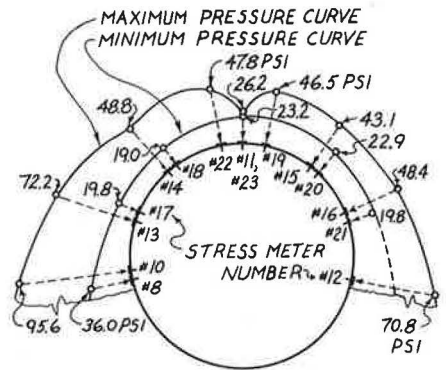


Figure 9. Earth pressures acting on the culvert walls on January 4, 1966.

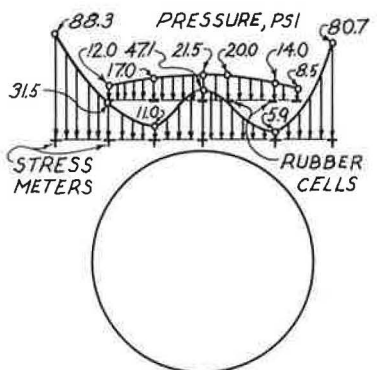


Figure 10. Earth pressures on January 4, 1966, determined from the stress meters and rubber pressure cells located in the fill.

nificance is attributed to the fact that all 15 of the plotted points happen to lie on one or the other of the two boundary curves.

Figure 10 shows the vertical earth pressures registered on the same date by the stress meters buried in the fill 1 ft above the culvert, and by the rubber pressure cells buried in the fill 6 in. below the straw, 4½ ft above the culvert. The rubber cell results shown indicate an average vertical pressure of 17 psi at the base of the straw and are typical of the highly consistent results furnished by those cells. Also notice in the figure that the buried stress meters registered high vertical pressures, in excess of 80 psi, in the soil to the sides of the culvert—this being consistent with (and actually a necessary consequence of) the reduced pressure directly above the culvert resulting from the imperfect trench effect.

Settlements

Figure 11 shows the average settlement of the culvert crown, near the center of the culvert, and the average settlement of the north-side and south-side settlement cells in the same vicinity. Notice that the top of the culvert settled about 2.3 in. while the north-side cells settled 2.7 in. and the south-side cells settled 4.5 in. The lesser amount of settlement experienced by the north-side settlement cells is perhaps related to the fact that the depth to bedrock is somewhat less on the north side of the culvert than on the south side. However, the top of the culvert settled less than any of the settlement cells, which shows that the prism of soil "sandwiched" between the straw and the culvert is subjected to a "positive projection" condition, with the adjacent prisms exerting downward frictional drag stresses on each of the two side faces.

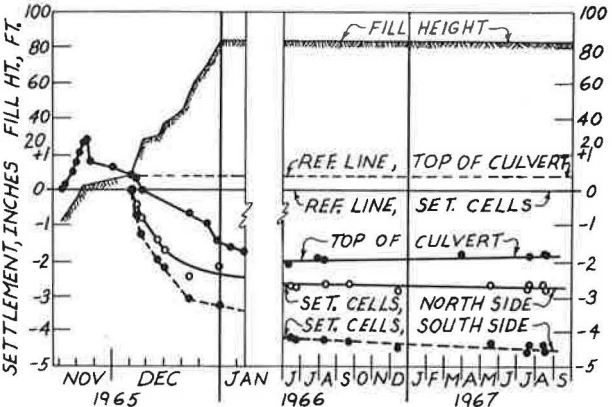


Figure 11. Settlement of the settlement cells and top of culvert.

Figure 12 shows a free-body diagram of the 4½-ft thick central block below the straw with the 17-psi average pressure (from the rubber cells) acting on its top face. The normal pressure on the sides is estimated from Figure 9 as 50 psi and the

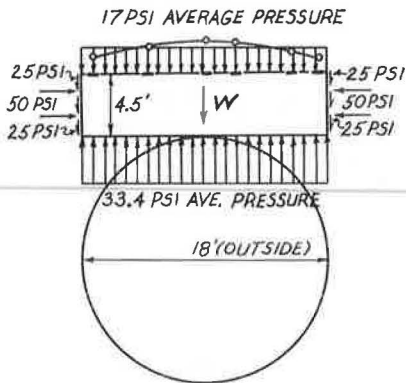


Figure 12. Pressure at top of culvert estimated from rubber pressure cells.

1 ft above the top of the culvert, which registered pressures of 31.5, 11.0, 47.1, and 5.9 psi on that date. The 5.9-psi reading should be heavily discounted because the meter from which it was taken proved its unreliability by registering tension a few weeks later.

General Observations

As the backfill material was compacted on each side of the culvert, the vertical diameter increased by 1.4 in. and the horizontal diameter decreased by 1.9 in. Then, as the embankment was constructed to its full height, the vertical diameter decreased until it was approximately 1 in. less than its original value, and the horizontal diameter increased until it was approximately $\frac{1}{2}$ in. less than its original value. Subsequent diameter changes have been very small but they show a slight increase in vertical diameter, amounting to approximately 0.2 in. between June 1966 and August 1967. This is equal to the negative settlement of the top of the culvert, which may be read from Figure 11 for the same time interval. The average settlement of the culvert invert was nearly 1 in. and practically all of this had occurred by the time the embankment was completed.

Periodic profile levels of the roadway pavement have shown that the pavement above the culvert has settled 0.11 ft, which is slightly less than the maximum pavement settlement of 0.14 ft that occurred 100 ft south of the culvert. Thus, the "imperfect trench" caused no differential pavement settlement, but it is not known whether any differential surface settlement occurred before the pavement was placed.

In August 1967 a hole was drilled from the culvert roof up through the straw to determine the compressed thickness of the straw layer that was nearly 36 in. thick originally. At the site of this one exploration hole, the thickness of the compressed straw was only 11 in.

Application of Marston's Theory

Marston's conduit theory (4) was applied to the installation, with the top of the uncompressed straw layer taken as the "critical plane", and with an "equivalent conduit" (18 ft wide and 27 ft high) visualized as consisting of the culvert, the straw, and the intervening earth, all three considered as a single unit. Details are shown in Figure 13.

The "settlement ratio", r_{sd} , was calculated from the following relationship:

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m}$$

developed internal friction coefficient is roughly estimated as 0.5, which yields an estimated downward shearing stress of 25 psi on the two sides.

For soil weighing 125 pcf, the pressure increase between the top and bottom of the block due to its own weight is 3.9 psi. The additional pressure increase, attributable to the 25-psi shear stresses on the sides, is 12.5 psi. The average estimated pressure at the level of the culvert crown is therefore 33.4 psi. This is a very rough estimate, but it is in good agreement with the vertical pressure on the culvert as deduced from strain gage and stress meter results shown in Figures 7, 8, and 9. It is also in fair general agreement with the four stress meters of Figure 10 that were buried in the fill

Significantly, the lateral load on the culvert exceeds the vertical. The stress meter results in Figure 9, the bending stress pattern in the sidewalls, and the slight increase in vertical diameter observed during the final year of the study, are all indicative of dominant lateral pressure.

The strain gage installations were particularly effective in monitoring both the bending stresses and the direct compressive stresses in the sidewalls resulting from erection and backfilling.

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Discussion

M. G. SPANGLER, Iowa State University, Ames—The research described in this paper constitutes a valuable contribution to the developing arts and sciences pertaining to the design of buried conduits of the flexible type. The authors have made an in-depth study of the performance of the reconstructed Wolf Creek culvert, using modern strain and pressure measuring devices, which they skillfully placed on the structure and in the fill adjacent to it. They have interpreted and presented the results of the measurements in a clear, concise, and logical manner that deserves the close attention of those interested in the structural behavior of buried pipelines. The writer wishes to comment on several features of the study that deserve special emphasis.

First, there is no room for doubt about the nature of the stress situation in the pipe wall at the springline. Figure 5 of the paper shows substantial bending moment stresses in the outer fibers of the plates that developed during erection of the conduit. Then in Figure 6 it is indicated that the stress situation changed from bending moment alone to combined bending moment and direct compressive stress when the fill was completed to the finished height of 83 ft above the top of the pipe. This is an important consideration in connection with the design of bolted longitudinal seams in field-erected structural plate pipes. The bending moment component of the combined stress causes a prying action at the seam that throws the outer row of bolts into direct tension. The stress in the bolts is, therefore, a composite of shear and axial tension, not shear alone.

It is currently a widespread practice to design the bolted seams solely on the basis of the tangential shear at the lapped joint. A more realistic procedure, in the light of both theoretical analysis and the field measurements reported in the paper, would be to design the bolts to carry the composite shear and tensile stress.

Strain gages were placed on the structure at the springline of the pipe and stresses were calculated from the measured strains at this location. Theoretical considerations indicate that the maximum bending moment stresses caused by the fill probably occurred at or near the lower quarter points. This is due to the probability that the bottom reaction on the pipe was distributed over a narrower width than was the load on top. This would cause an unsymmetrical distortion of the moment diagram around

the perimeter of the pipe, and bending stresses would be greater at points in the vicinity of the lower quarter points than at the springline and above. The writer believes that this consideration helps to explain the fact that practically all the failures of bolted seams in the original Wolf Creek culvert (6) occurred in the lower quadrants of the pipe.

A second important fact revealed in this study is the demonstration of the effectiveness of the imperfect ditch method of construction. The authors conclude, on the basis of their measurements and observations, that the load on the structure equaled only about half the dead weight of the prism of soil extending upward from the pipe. This is a truly remarkable result. Others have arrived at similar conclusions, but their observations have been on pipes of considerably smaller diameter. Here is a structure about 18 ft wide that experienced substantial relief in load by the application of this method of construction. Also, the authors were unable to detect any abnormal subsidence of the top of the embankment over the culvert, in spite of the fact that the baled straw placed in the bottom of the imperfect trench compressed about 2 ft or two-thirds of its initial height. This demonstrates the validity of the concept of a plane of equal settlement, above which all horizontal elements in the embankment are postulated to settle equally, both over the conduit and adjacent thereto.

In 1950, the writer (7) developed and published a theory of loads on imperfect ditch and negative projecting conduits. This theory involved a settlement ratio that, while completely rational in the theoretical development, is practically impossible to predict on a rational basis for any proposed culvert. Therefore, it is an empirical factor, usable values of which can best be determined by the observation of actual culverts in service.

There have been relatively few opportunities to observe empirical values of this ratio, but on the basis of limited information, the writer has used and recommended values in the range of -0.3 to -0.5. Although the geometry of the reconstructed Wolf Creek culvert imperfect ditch does not comply exactly with the situation for which the load theory was developed, the basic principle of creating favorable differential settlements between the central prism of soil and the adjacent side prisms is the same, and it is of interest to calculate a load by the theory and compare the result with the present authors' measured loads. The main difference between the actual and the theoretical case is the existence of a 5-ft layer of highly compacted granular soil between the top of the pipe and the bottom of the imperfect ditch where the layers of straw bales were placed, as shown in Figure 1 of the paper.

The following data are used in the load calculation: $H = 83$ ft, $B_C = 18$ ft, $H/B_C = 4.6$, $w = 125$ pcf, $p' = 0.6$, and $r_{sd} = -0.5$. Interpolating between Figures 8 and 9 of Ref. 7, $C_N = 2.9$.

Substituting in the load formula

$$W_C = c_n w B_C^2$$

$$W_C = 2.9 \times 125 \times 18^2 = 117,500 \text{ plf} = 45.3 \text{ psi.}$$

The data in Figure 7 of the paper indicate that the maximum equivalent uniform vertical earth pressures on the culvert as deduced from the strain gage measurements varied from 48 psi on plate E down to 26 psi on plate F. The average maximum on the six plates on which strain gages were mounted was 34.7 psi. Thus the calculated load using a settlement ratio of -0.5 agrees fairly well with the real situation in this case. The same diagrams in Ref. 7 indicate that the plane of equal settlement is about 50 ft above the top of the conduit, or approximately 33 ft below the embankment grade. This is compatible with the authors' observation that there was no differential settlement of the roadway pavement in the vicinity of the culvert.

A third item of interest to be derived from the data presented in the paper is the determination of the modulus of soil reaction, E' , of the sidefill material that surrounded the pipe. In 1941 the writer (8, 9) presented a hypothesis of the system of earth loads acting on flexible conduits and developed an equation for estimating the deflection of such conduits under load. Briefly, this hypothesis is based on the concept that each increment of vertical load on the pipe causes an increment of deflection;

the vertical diameter shortens and the horizontal diameter lengthens. As the horizontal diameter increases, the sides of the pipe push outward against the enveloping soil, mobilizing the passive resistance characteristics of the soil. At the present time this passive resistance pressure appears to be a linear function of the amount of movement of the sides of the pipe and therefore a function of pipe deflection.

The deflection equation is

$$\Delta X = D_1 \frac{K W_c r^3}{EI + 0.061 E' r^3}$$

in which

ΔX = pipe deflection, in. (vertical and horizontal deflections are nearly the same);

D_1 = deflection lag factor;

K = bedding factor;

W_c = load on pipe per unit length, pli;

r = mean radius, in.;

E = modulus of elasticity of pipe material, psi;

I = moment of inertia of cross section of unit length of pipe wall, in.⁴ per in.;

and
 E' = modulus of soil reaction, psi.

This is a rational development, considering the pipe and surrounding soil to be a composite elastic body, except for the deflection lag factor, D_1 , which is purely empirical. It appears to vary inversely with the modulus of soil reaction, being greater for weak soils with low values of E' . In this study, the deflection lag factor was essentially 1.0, because of the high quality of the select backfill soil. In fact, the authors state that the vertical diameter of the pipe actually increased slightly after completion of the embankment.

In early experiments carried out in the 1930's, the value of E' was definitely indicated to be a function of the kind and quality of the sidefill soil, but the range of soil types and conditions observed was very limited. Later, as the deflection equation has grown in use, it has become apparent that E' can vary over a very wide range. The writer has attempted to accumulate data on values of E' and to correlate them with soil properties, particularly texture and density. The study provides all the data needed to estimate the value of the passive resistance modulus that prevailed in this installation.

The following values of terms in the deflection equation are available from the data presented in the paper:

ΔX = 1.9 in. (vertical deflection = 2.4 in., horizontal = 1.4 in.)

D_1 = 1.0

K = 0.1 (shaped bedding)

W_c = 7500 pli (average maximum vertical pressure = 34.7 psi)

r = 9 ft = 108 in.

E = 30,000,000 psi

I = 0.225 in.⁴ per in.

Solving for E' yields 6300 psi. This is a high value of the modulus of soil reaction, but is consistent with the nature of the sidefill material, which is described as "crushed granular material of base-course quality." Sieve analysis indicates a well-graded gravel passing a 1½-in. sieve. It was compacted in 6-in. layers and at optimum moisture content to 95 percent of standard AASHTO density—a very high quality material.

Since original publication of the deflection formula for flexible pipes, efforts have been made to devise laboratory methods for estimating the modulus of soil reaction for a prospective culvert installation, but without conspicuous success. Probably the greatest progress will be made in this regard by considering E' as a semi-empirical constant whose values may best be determined by observation and measurements on actual culvert installations. The writer has made a number of such determinations and in 1958 published (9) a table showing values of E' for various kinds of soil in vari-

TABLE 1
VALUES OF E' FOR 18 FLEXIBLE PIPE CULVERTS

Item	Location	Pipe Diam. (in.)	Soil Type ^a	Fill Height (ft)	Mod. of Passive Resist., e (psi/in.)	Value of E' (er) (psi)
1 ^b	Ames, Iowa	42	Loam top soil (U)	15	14	294
2 ^b	Ames, Iowa	42	Well-graded gravel (U)	16	32	672
3 ^b	Ames, Iowa	36	Sandy clay loam (T)	15	28	502
4 ^b	Ames, Iowa	36	Sandy clay loam (U)	15	13	234
5 ^b	Ames, Iowa	42	Sandy clay loam (T)	15	25	525
6 ^b	Ames, Iowa	42	Sandy clay loam (U)	15	15	315
7 ^b	Ames, Iowa	48	Sandy clay loam (T)	15	29	696
8 ^b	Ames, Iowa	48	Sandy clay loam (U)	15	14	336
9 ^b	Ames, Iowa	60	Sandy clay loam (T)	15	26	780
10 ^b	Ames, Iowa	60	Sandy clay loam (U)	15	12	360
11 ^c	Chapel Hill, N. C.	30	Sand	12	25	375
12 ^c	Chapel Hill, N. C.	31.5	Sand	12	56	882
13 ^c	Chapel Hill, N. C.	30	Sand	12	80	1200
14 ^c	Chapel Hill, N. C.	20	Sand	12	35	350
15 ^c	Chapel Hill, N. C.	21	Sand	12	82	861
16 ^c	Culman Co., Ala.	84	Crushed sandstone (C)	137	190	7980
17 ^c	McDowell Co., N. C.	66	Clayey sandy silt (C)	170	40	1320
18 ^d	Wolf Creek, Mont. (reconstructed)	216	Graded crushed gravel (C)	83	58	6300

^aU = untamped; T = tamped; C = compacted.

^bSide pressure and pipe deflections measured.

^cSide pressures estimated, pipe deflections measured.

^dLoad and pipe deflections measured.

ous states of compaction as deduced from 17 actual culverts. This table may now be extended to include the reconstructed Wolf Creek culvert, as given in Table 1.

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