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D. M. BEACH, *Editor*

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THE STRUCTURAL DESIGN OF FLEXIBLE PIPE CULVERTS[a]

Reported by M. G. SPANGLER, Associate Structural Engineer, Iowa Engineering Experiment Station

CORRUGATED METAL PIPES have been utilized in the construction of culverts and other underground conduits for a period of about 40 years. Their use in highway and railway culvert construction has grown from a modest beginning to a place of major importance as a cross-drainage structure. The relatively light weight of this type of culvert pipe, with the attendant advantages in ease of transportation and placement, has been a powerful factor in the highly competitive field of culvert pipe manufacture and installation.

In spite of this long record of use, however, there is no rational method available to the engineering profession for predicting the structural performance of this flexible type of culvert pipe before installation, and there is need for a valid design procedure based upon recognized principles of mechanics.

This paper is a progress report on a research project being conducted cooperatively by the Iowa Engineering Experiment Station and the United States Bureau of Public Roads for the purpose of (1) supplying information relative to the structural performance of flexible pipe culverts under earth embankments, and (2) developing a rational theory of design for this type of structure.

A hypothesis of the magnitude and distribution of the various forces to which a flexible pipe culvert is subjected when installed as a projecting conduit without struts or other prestressing devices, and of the behavior of the structure in response to these forces, will be offered. A description of some extensive experimental work that has been conducted for the purpose of verifying the hypothesis will also be given, together with a comparison between the actual and hypothetical performance of the experimental culverts. The paper will deal wholly with structural aspects of corrugated metal pipe culverts and will not go into the matter of durability of the various metals of which they are made.

Culvert pipes placed under earth embankments derive their ability to support the earth above them from two sources: First the inherent strength of the pipe to resist external pressures; and second, the lateral pressure of the earth at the sides of the pipe, which produces stresses in the pipe ring in opposite directions to those produced by the vertical loads and therefore assists the pipe in supporting the vertical loads. In rigid pipes such as those made of concrete, cast iron, burned clay, etc., the inherent strength of the pipe is the predominant source of supporting ability. The only lateral pressure that can be safely depended upon to augment the load-carrying capacity of the pipe is the active lateral pressure of the earth, since the rigid pipes deform very little under the vertical load and consequently the sides do not move outward enough to develop any appreciable passive pressure in the enveloping earth.

In flexible pipes, however, the situation is reversed. The pipe itself has relatively little inherent strength, and a large part of its ability to support vertical load must be derived from the passive pressures induced as the sides move outward against the earth. The ability of a flexible pipe to deform readily and thus utilize the passive pressure of the earth on each side of the pipe is its principal distinguishing structural characteristic and accounts for the fact that such a relatively light-weight, low-strength pipe can support earth fills of considerable height without showing evidence of structural distress. It is apparent from these considerations that any attempt to analyze the structural behavior of this type of culvert pipe under a fill must take into account the earth at the sides as an integral part of the structure, since so much of the total supporting strength depends upon the side material.

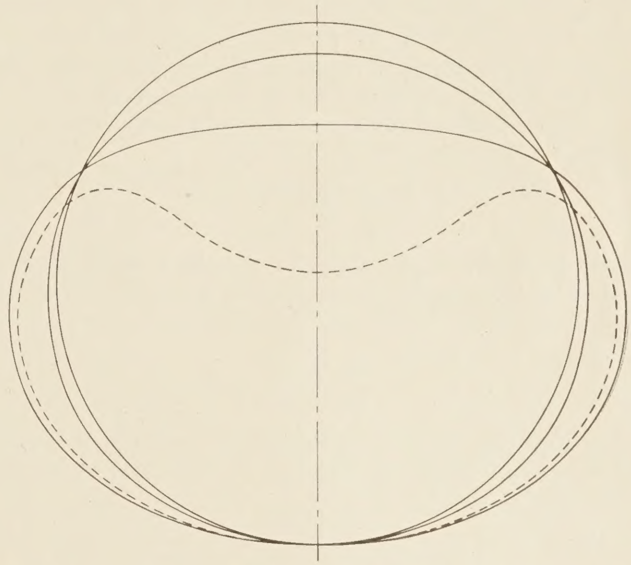

FIGURE 1.—SHAPES TAKEN BY A FLEXIBLE CULVERT PIPE DURING DEFLECTION UNDER VERTICAL LOAD.

The normal sequence of deflection of a flexible pipe culvert installed in the ordinary manner under an earth embankment that is built high enough to cause the pipe to collapse may be summarized as follows: The first layers of the fill cause the pipe to deflect appreciable amounts, the vertical diameter becoming less and the horizontal diameter greater. The outward movement of the sides of the pipe against the enveloping earth brings into play the passive resistance of the earth, which acts horizontally against the pipe and materially reduces the amount that the ring would deflect if acted upon by the vertical earth loads alone.

This action continues as the embankment is built higher until the top of the pipe becomes approximately flat, when additional load will cause the curvature of the top portion of the pipe to reverse direction, becoming concave upward (see fig. 1). The sides of the pipe will then pull inward. This eliminates the side supports of the pipe, since they are passive forces that cannot follow the inward movement, and the pipe will proceed rapidly to collapse and fail completely. The whole action is one of deflection change unaccompanied by rupture or buckling of the metal ring, although the material in certain parts of the ring may be stressed well beyond its elastic limit during the process.

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[a] Paper presented at the Seventeenth Annual Meeting of the Highway Research Board, Washington, D. C., 1937.

It seems evident, therefore, that any attempt to rationalize the structural design of flexible pipe culverts should be directed toward the development of a method for predetermining the deflection of the pipe under specified conditions of installation.

ELASTIC THEORY AS APPLIED TO THIN RINGS OUTLINED

In the elastic theory of flexure as applied to thin rings it is shown that

$$\frac{1}{\rho}-\frac{1}{r}=\frac{M}{EI} \qquad (1)$$

where

ρ=the radius of curvature of the elastic curve of the ring when loaded.
r=the mean radius of the unloaded ring.
$\frac{1}{\rho}-\frac{1}{r}$=the change in curvature of the elastic curve, caused by the load.
M=the bending moment at any point on the ring.
E=modulus of elasticity of the material.
I=moment of inertia of the ring cross section.

Also, by calculus it is known that

$$\frac{1}{\rho}-\frac{1}{r}=\frac{d\theta}{ds} \qquad (2)$$

where $d\theta$ is the angle through which a plane section normal to the elastic curve rotates with respect to its original position, within a length of arc ds.

Then

$$\frac{d\theta}{ds}=\frac{M}{EI} \qquad (3)$$

But $ds=\rho d\phi$, and it is assumed that the difference between ρ and r will be small so that likewise $ds=rd\phi$.[1]

Therefore

$$\frac{rM}{EI}=\frac{d\theta}{d\phi} \qquad (4)$$

In this expression, $\frac{d\theta}{d\phi}$ represents the rate of change of the angular displacement of a normal section with respect to the central angle ϕ.

When the external loads on the ring are symmetrical about the vertical axis, the normal sections at the top and bottom of the ring will remain vertical regardless of the character and magnitude of the angular displacements at intermediate points, and the sum of all the elementary displacements as ϕ varies from 0 to π will be

$$\frac{r}{EI}\int_0^{\pi} Md\phi=0 \qquad (5)$$

or, more simply

$$\int_0^{\pi} Md\phi=0 \qquad (6)$$

From this equation, the moment and thrust caused by any system of loads that is symmetrical about the vertical axis[2] can be obtained for either the top or bottom point of the ring. Having the moment and thrust at either of these points, the moment, tangential thrust, and radial shear can be obtained for any point on the half circle by the equations of equilibrium. Likewise, the vertical and horizontal deflections of the pipe ring can be derived from this basic relationship.

[1] This is an assumption employed generally in thin ring analysis. Its applicability to flexible rings is shown by laboratory experiments described later in this paper.
[2] If the loads are unsymmetrical about the vertical axis, the equation is integrated between the limits 0 and 2π.

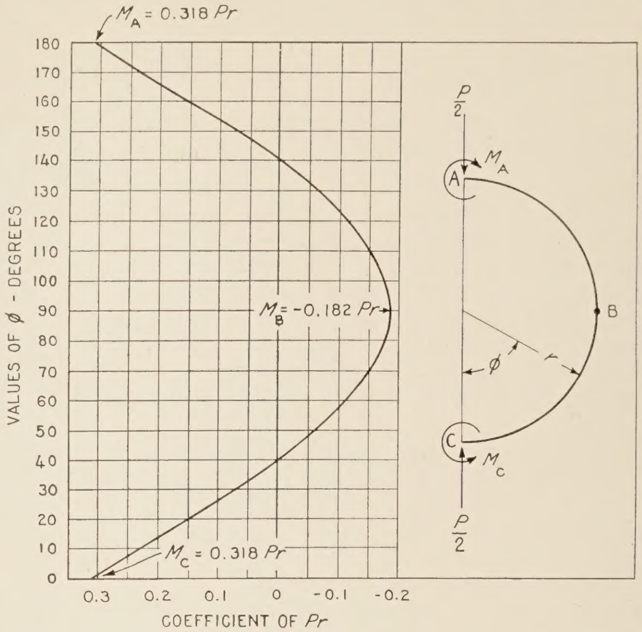

FIGURE 2.—VARIATION IN MOMENT AROUND A PIPE RING RESULTING FROM LOADS AT DIAMETRICALLY OPPOSITE POINTS.

For loads applied at diametrically opposite points on the ring, the expression for moment at the top and bottom is

$$M_A=M_C=0.318\ Pr \qquad (7)$$

and at the sides

$$M_B=0.182\ Pr \qquad (8)$$

in which

P=the load.
r=mean radius of pipe.

The variation in moment around the pipe ring is shown in figure 2. The expressions for deflections caused by this loading are:

For vertical deflection,

$$\Delta y=0.149\frac{Pr^3}{EI} \qquad (9)$$

For horizontal deflection,

$$\Delta x=0.136\frac{Pr^3}{EI} \qquad (10)$$

It will be noted that in the application of the elastic theory to a closed ring, the assumption is made that the radius of curvature of the loaded ring is nearly equal to the radius of the unloaded ring and r is substituted for ρ at one step in the analysis. For rigid rings, this assumption is easily accepted, but for flexible rings such as corrugated metal pipes, where the deflections and changes in the radius of curvature are relatively large, there was considerable doubt whether this assumption is valid. In order to obtain evidence on this point, some flexible pipe sections of various diameters were loaded in the laboratory at diametrically opposite points and the measured deflections were compared with those calculated by formulas (9) and (10). The physical properties of the pipe specimens were determined as outlined below.

During the laboratory loading experiments, the unit strain of the inner fiber of the pipe wall was measured at the top directly under the applied load by means of a Huggenberger tensometer having a gage length of ½ inch. The measured strains are shown in the load-strain diagrams in figures 3 to 6. The modulus of elasticity of the metal in each pipe was calculated by dividing these measured strains into the unit stress obtained by the moment formula (7) and the flexure formula $\frac{Mc}{I}$. Thus

$$E=\frac{s}{\delta}=\frac{0.318\ Prc}{\delta I} \qquad (11)$$

in which

E=modulus of elasticity, pounds per square inch.
δ=unit strain, inches per inch.
P=load per inch length of pipe.
r=mean radius of pipe, inches.
c=distance from neutral axis of the pipe wall to the outer fiber, inches. For standard corrugated sheets with corrugations ½ inch deep and 2⅔-inch pitch, $c=0.25+\frac{t}{2}$, where t=the thickness of the metal.
I=moment of inertia per inch length of pipe.

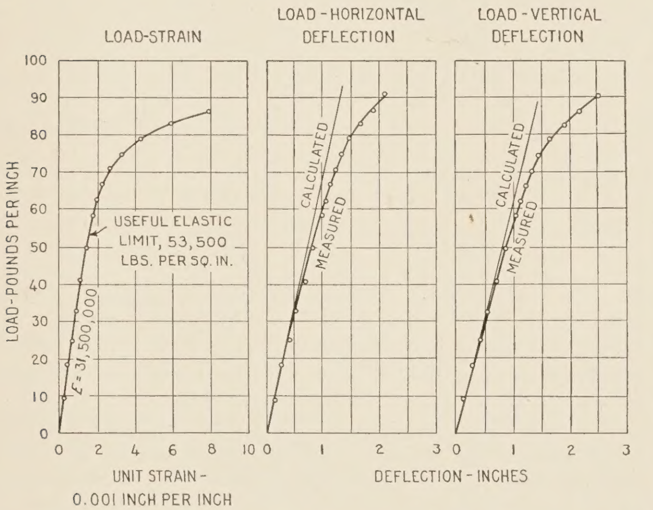

FIGURE 3.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 36-INCH PIPE.

CULVERT PIPE SPECIMENS TESTED IN THE LABORATORY

The moduli of elasticity of the metal in four specimens tested in the laboratory, as determined from measured strains by means of formula (11), were as follows:

Nominal pipe diameter (inches):	*Modulus of elasticity, lb. per sq. in.*
36	31, 500, 000
42	33, 200, 000
48	32, 000, 000
60	26, 800, 000

Each of these laboratory specimens was made from sheets rolled from the same heat of metal as were the test specimens of the same size used in the field-laboratory experiments to be described later, and these values of the modulus of elasticity may be used in analyses of the performance of the field test specimens.

The mean diameter of 8 test specimens of each size averaged approximately 1 inch greater than the nominal diameter and this mean diameter has been used in calculating the deflections of the pipes.

The value of the moment of inertia of the cross section of a standard corrugated sheet having corrugations ½ inch deep and spaced 2⅔ inches center-to-center has been determined by means of a formula developed several years ago by E. T. Jensen, when employed by

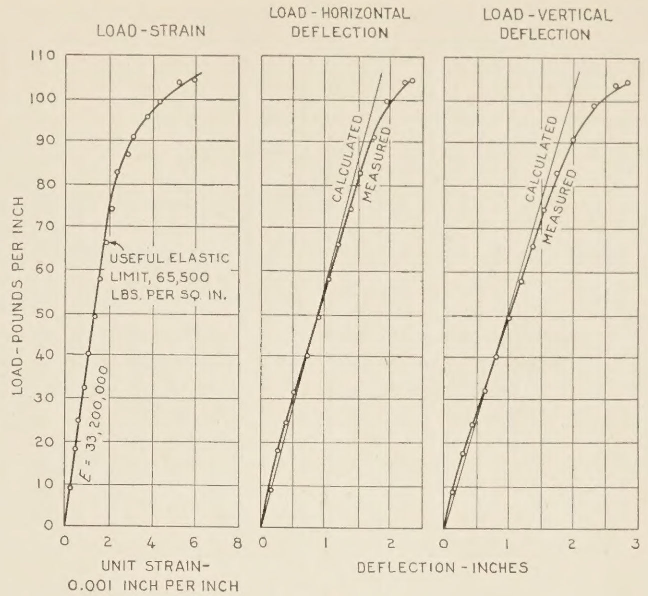

FIGURE 4.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 42-INCH PIPE.

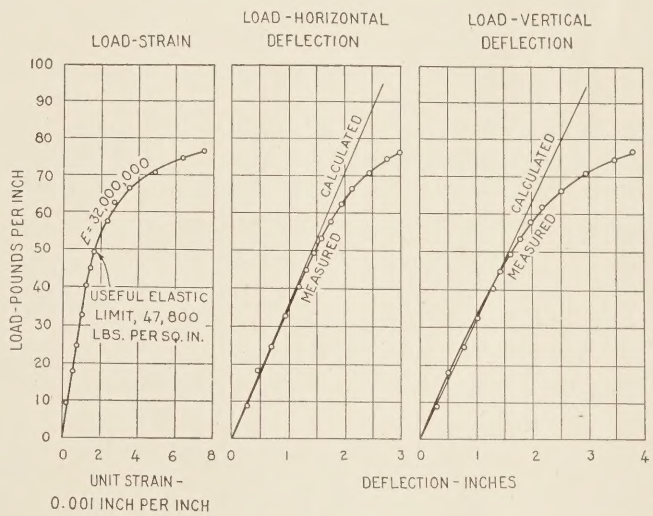

FIGURE 5.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 48-INCH PIPE.

the Iowa Engineering Experiment Station, as follows:

$$I=0.02925t-0.00150t^2+0.10425t^3-0.00225t^4 \qquad (12)$$

G. E. Shafer has devised a simpler straight-line formula for moment of inertia which is

$$I=\frac{t}{30} \qquad (13)$$

This formula gives moments of inertia that are up to 10 percent higher than values given by formula (12) within the range of thicknesses ordinarily encountered in corrugated metal culverts. Values of the moment of inertia and the section modulus for various gages by these two formulas are shown in table 1.

A graphical comparison between the two formulas for moment of inertia is shown in figure 7. The plotted points in this diagram are experimental moments of inertia obtained by Shafer by loading corrugated

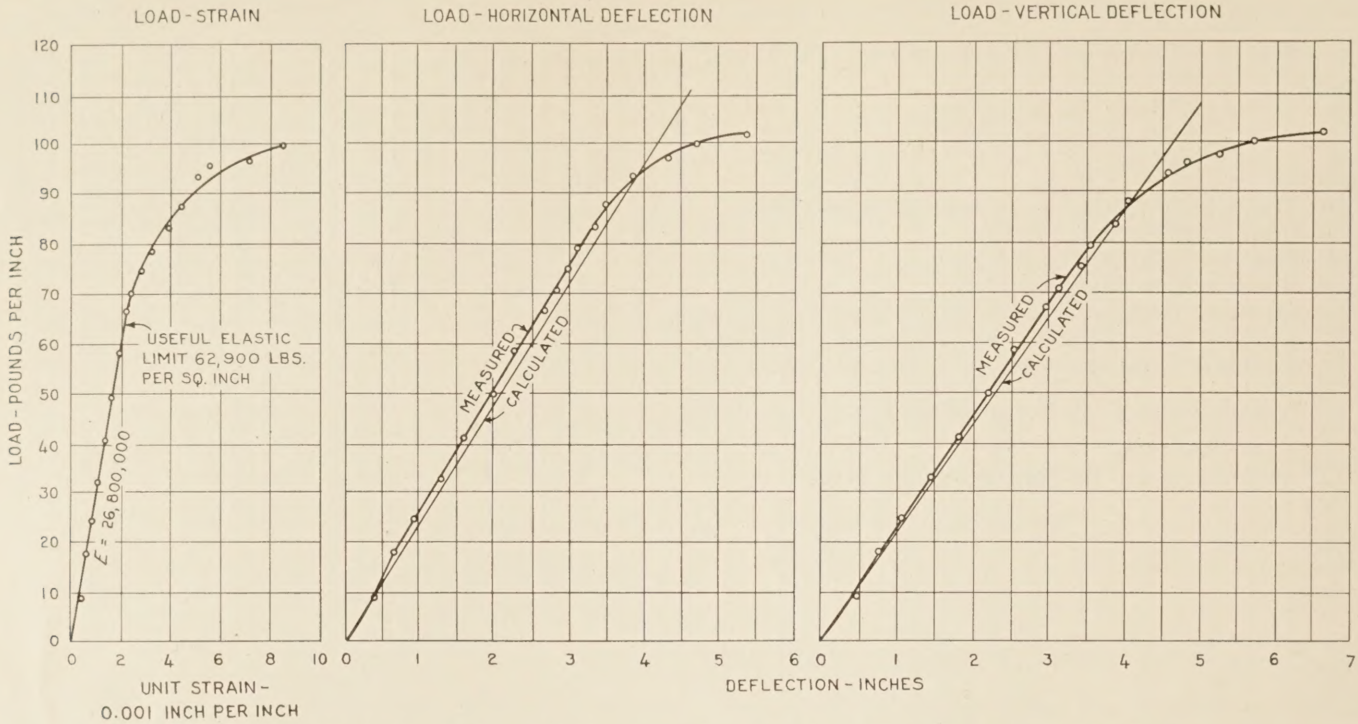

FIGURE 6.—MEASURED STRAINS AND DEFLECTIONS UNDER VARIOUS LOADS FOR A 60-INCH PIPE.

sheets 48 inches long and 8 inches (three corrugations) wide as a beam with concentrated loads at the third points, and calculating I from observed deflections.

TABLE 1.—*Values of moment of inertia and section modulus for various gages of pipes* [1] *by two formulas*

U. S. Gage No.	Thickness	Jensen formula		Shafer formula	
		Moment of inertia, I	Section modulus	Moment of inertia, I	Section modulus
	Inch	*Inch*^{4}	*Inch*^{3}	*Inch*^{4}	*Inch*^{3}
4	0.234375	0.008275	0.022536	0.007813	0.021277
6	.203175	.006744	.019183	.006773	.019263
8	.171875	.005512	.016407	.005729	.017054
10	.140625	.004373	.013651	.004688	.014634
12	.109375	.003317	.010887	.003646	.011966
14	.078125	.002326	.008046	.002604	.009009
16	.062500	.001848	.006570	.002083	.007407
20	.037500	.001104	.004109	.001250	.004651
24	.025000	.000733	.002792	.000833	.003175
30	.012500	.000366	.001428	.000417	.002016

[1] Pipes have standard corrugations ½-inch deep and spaced 2⅔-inches center-to-center.

The measured deflections and those calculated by the thin-ring elastic theory as outlined above are close enough together (figs. 3 to 6) to justify the conclusion that this theory is applicable in the case of corrugated metal pipes under two-point loading. Even though the deflections and accompanying changes in radius of curvature are relatively large, the tolerance is probably no greater than that occasioned by variations in the modulus of elasticity of the metal, variations in thickness, depth, and spacing of corrugations, and other variables inherent in the manufacture of this type of conduit.

It seems tenable, therefore, to assume that the theory will also apply in the case of a corrugated culvert pipe installed under an embankment, since the external pressures on the pipe in the field will be more nearly uniformly distributed around the pipe than they were in the laboratory tests. An investigation of the structural performance of corrugated metal pipe culverts, therefore, becomes mainly a study of the magnitude and distribution of the loads and pressures to which they are subjected in service.

Research has been conducted in the general field of structural analysis and supporting strength of underground conduits of many types during the past 30 years, and some general conclusions have been reached relative to the behavior of such structures and the earth around them. One of these conclusions is that the earth embankment over a projecting conduit [3] produces vertical load on the structure in accordance with Marston's conduit load theory and this load is approximately uniformly distributed over the entire width of the conduit. This seems to be true regardless of the shape of the conduit. Also, in the case of circular pipes the vertical reaction is distributed over some narrower width, depending upon the width of bedding in which the pipe is laid and upon the rigidity or flexibility of the pipe.

In the case of rigid pipes the reaction, which is a passive pressure, will tend to be greater near the center of the pipe because of the circular shape. In the case of flexible pipes this tendency is counteracted partially by the large deflection of the pipe which flattens the bottom segment somewhat, thereby increasing the reaction near the edges of the bedding and bringing about a more nearly uniform distribution of pressure over the width of the bedding.

THEORY OF PRESSURE DISTRIBUTION AROUND FLEXIBLE PIPE CULVERTS DISCUSSED

Little is known concerning the magnitude of the passive resistance pressures that are developed at the sides of a flexible pipe as the pipe deflects and moves outward

[3] A projecting conduit is one that projects above the subgrade on which it is constructed, as opposed to a ditch conduit which is laid in a trench. A projecting conduit, therefore, is laid with its top some distance above the natural ground line, whereas the top of a ditch conduit is below the natural ground line.

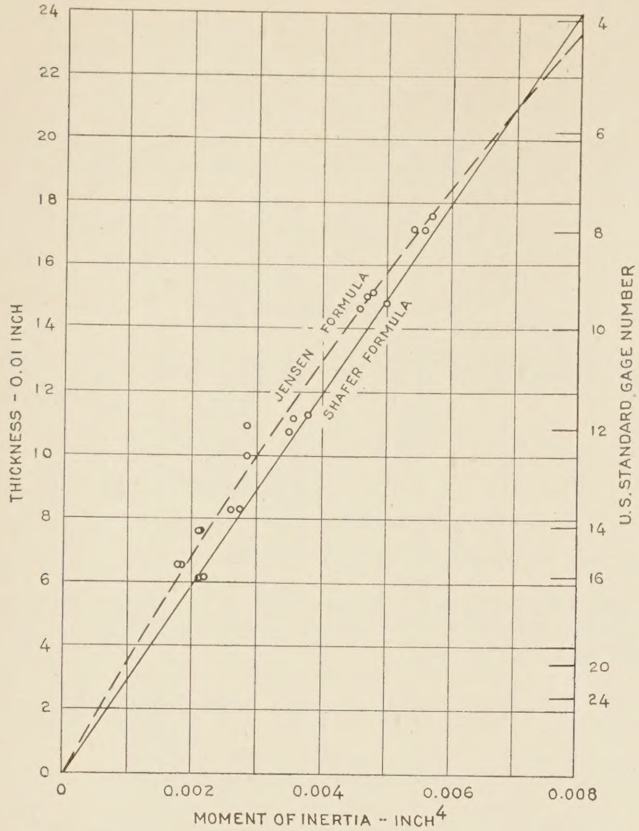

FIGURE 7.—COMPARISON OF MOMENTS OF INERTIA FOR VARIOUS GAGES OF PIPE BY TWO FORMULAS.

against the enveloping earth. In the Rankine theory of lateral earth pressures, the limiting value of the ratio of passive horizontal pressure to vertical pressure that a granular soil without cohesion can develop is shown to be the reciprocal of the active pressure ratio. Since active pressure, according to this theory, is roughly about one-third the vertical pressure, the passive pressure would be three times the vertical pressure or nine times the active lateral pressure. This theory does not give a clue as to the amount of movement required to develop the full limit of passive pressure, however.

At best Rankine's ratio would appear to establish a limiting value of passive pressure for a cohesionless soil, and is of little service when one attempts to assign specific values of pressure to definite movements or pipe deflections. Further, it is evident from the limited amount of work that has been done in this field that the relationship between passive pressure and movement is closely linked with soil characteristics and these characteristics will need to be carefully correlated with movement and vertical soil pressures in any rational treatment of flexible pipe design.

In two preliminary experiments, one begun in 1927 and the other in 1934, in which the structural action of two 42-inch corrugated pipe culverts was observed, measurements were made of the horizontal earth pressure at the ends of the horizontal diameter after the completion of each 1-foot increment of fill over the top of the pipes. Measurements of the horizontal diameter change for each increment of fill were also made. It was found that the ratio of horizontal pressure to diameter change was practically a constant regardless of the height of fill, in both these trials. Also, the values of this ratio were considerably different for the two types of soil of which the embankments were constructed, being approximately three times as great in the case of a gravel fill as it was for one built of loam.

Since the change in the horizontal diameter of a pipe is equal to the sum of the outward movements of the sides of the pipe at the two ends of the horizontal diameter, these preliminary observations have led to the tentative conclusion that, as a fill is built over a flexible culvert, the horizontal pressure on the pipe at any point bears a nearly constant relationship to the horizontal movement of the point. This constant ratio has been called the "modulus of passive resistance" of the fill material and is expressed as units of pressure per unit of movement such as pounds-per-square-inch per inch. Therefore, if the lateral movements of various points on the pipe ring are known, the distribution of lateral pressures on the pipe can be determined by multiplying the movement at each point by this modulus.

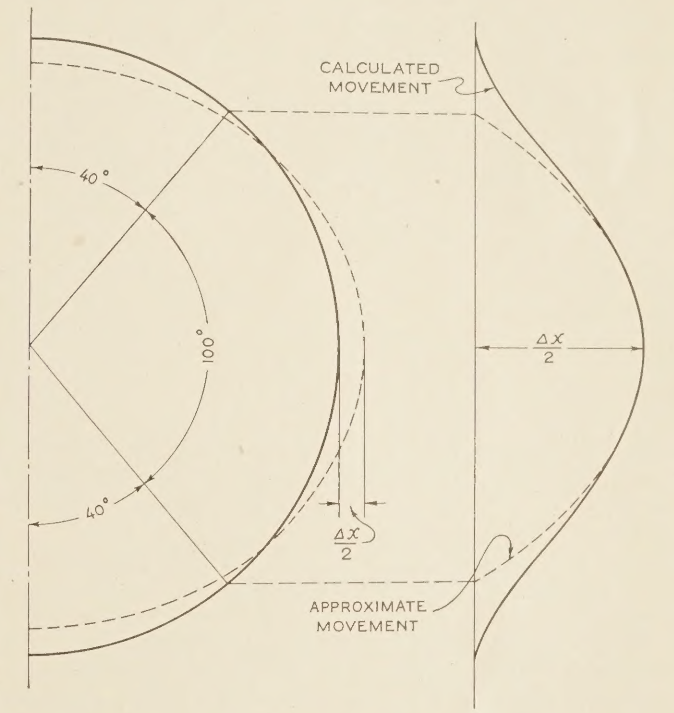

FIGURE 8.—CHANGE IN SHAPE OF A FLEXIBLE CULVERT PIPE UNDER LOAD. LEFT, CHANGE FROM CIRCLE TO ELLIPSE; RIGHT, AMOUNT OF HORIZONTAL MOVEMENT OF THE PIPE WHEN DEFORMED INTO AN ELLIPSE.

In general, it may be said that a corrugated metal culvert pipe under an embankment load deforms from a circular to an elliptical shape, with the minor (vertical) axis of the ellipse less than the diameter of the circle by the amount of the vertical deflection and the major (horizontal) axis greater than the diameter by the amount of the horizontal deflection. Also, for moderate deflections up to 5 or 6 percent of the nominal diameter, the length of the periphery of the original circle and the ellipse may be considered to be the same.

On the basis of these assumptions it is possible to calculate the horizontal movement of any point on the periphery of a circular pipe as it deforms to an elliptical shape. The curve obtained by plotting the horizontal movement of each point on the vertical diameter of the pipe is a witch-shaped curve as shown in figure 8.

However, since the horizontal movements of points within the top and bottom 40° segments are small, a simpler parabolic curve embracing the middle 100° of the semicircle may be substituted for the witch without serious error. Since pressures are assumed to be proportional to movement, the passive resistance pressures on each side of the pipe are distributed parabolically over the middle 100° of the pipe; and the maximum unit pressure, which will occur at the ends of the horizontal diameter, is equal to the modulus of passive resistance of the fill material times one-half the horizontal deflection of the pipe.

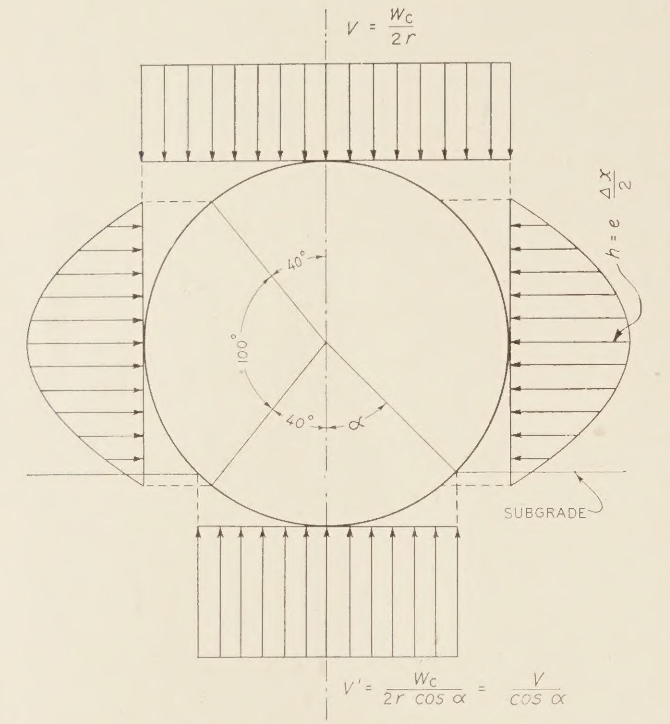

Figure 9.—Hypothetical Distribution of Pressure on a Flexible Culvert Pipe Under an Earth Fill.

This hypothesis may be summarized as follows:

1. The vertical load may be determined by Marston's theory of loads on conduits and is distributed approximately uniformly over the width of the pipe.

2. The vertical reaction is equal to the vertical load and is distributed approximately uniformly over the width of bedding of the pipe.

3. The horizontal pressure on each side of the pipe is distributed parabolically over the middle 100° of the pipe and the maximum unit pressure is equal to the modulus of passive resistance of the fill material multiplied by one-half the horizontal deflection of the pipe.

The distribution of pressures around a flexible pipe under an earth fill in accordance with this hypothesis is shown graphically in figure 9.

Having set up this hypothesis it is possible to develop mathematical expressions for the moments, thrusts, shears, and deflections of a pipe, in terms of the properties of the pipe and the earth of which the embankment is constructed. Referring to figure 10 and calling clockwise moments negative, the equation for the moment at any point is

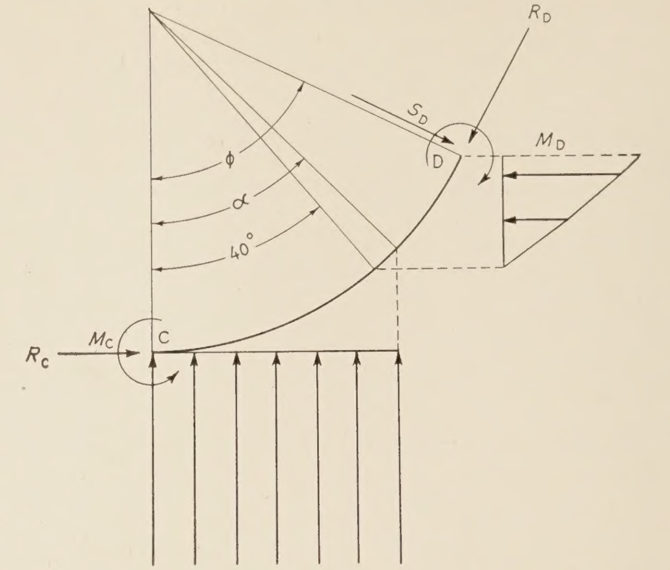

Figure 10.—Free Body Diagram for Determining Moment, Thrust, and Shear at Any Point D on a Flexible Culvert Pipe.

$$M=M_C+R_Cr(1-\cos\phi)\Big|_0^{\pi}-0.5v'r^2\sin^2\phi\Big|_0^{\alpha}$$
$$-\sin\alpha r^2v'\left(\sin\phi-\frac{\sin\alpha}{2}\right)\Big|_{\alpha}^{\pi}$$
$$-hr^2(0.147-0.51\cos\phi+0.5\cos^2\phi-0.143\cos^4\phi)\Big|_{40^\circ}^{140^\circ}$$
$$+1.021hr^2\cos\phi\Big|_{140^\circ}^{\pi}-0.500vr^2(1-\sin\phi)^2\Big|_{\frac{\pi}{2}}^{\pi}\text{------}(14)$$

in which

M = moment at any point.
M_C = moment at bottom of pipe.
R_C = tangential thrust at bottom of pipe.
r = mean radius of pipe.
ϕ = the central angle.
v = unit pressure on top of pipe.
v' = unit pressure on bottom of pipe.
h = maximum unit pressure on side of pipe.
α = one-half the bedding angle.

Substituting equation (14) in equation (6) and integrating yields

$$M_C=-R_Cr+0.057vr^2+0.345hr^2$$
$$+v'r^2[0.08\alpha-0.04\sin 2\alpha-0.159\sin^2\alpha(\pi-\alpha)$$
$$+0.318\sin\alpha(1+\cos\alpha)]\text{-----------------}(15)$$

FORMULA FOR USE IN DESIGN OF FLEXIBLE CULVERT PIPES DEVELOPED

This is the expression for the bending moment at the bottom of the pipe, but contains R_C, the thrust at that point, which is as yet unknown. To evaluate R_C, the displacement theory of arches may be utilized as was done by Cain.[4] In this theory the bottom of the pipe is assumed to be fixed. Then the displacement of any point on the ring relative to the bottom will equal $\frac{1}{EI}$ times the integral of moments at all intermediate points multiplied by the ordinates to the points that are perpendicular to the supposed displacement. Thus the

[4] Stresses and Deflections of Pipe Culverts, by William Cain. Public Roads, vol. 10, no. 9, November 1929.

horizontal movement of A relative to C will equal

$$\frac{r^2}{EI}\int_0^\pi M(1-\cos\phi)d\phi \quad (16)$$

Since the loads are symmetrical about the vertical axis, this expression may be equated to zero, and since

$$\int_0^\pi M d\phi=0, \text{ reduces to}$$

$$\int_0^\pi M\cos\phi d\phi=0 \quad (17)$$

By substituting equation (14) in this expression and integrating,

$$R_C=-0.106v'r\sin^3\alpha+0.511hr+0.106vr \quad (18)$$

Then from equation (15)

$$M_C=-0.049vr^2-0.166hr^2+v'r^2[0.106\sin^3\alpha+0.080\alpha-0.040\sin 2\alpha-0.159\sin^2\alpha(\pi-\alpha)+0.318\sin\alpha(1+\cos\alpha)] \quad (19)$$

Since $v=\frac{W_c}{2r}$ and $v'=\frac{W_c}{2r\sin\alpha}$, where W_c is the total vertical load on the pipe per unit of length,

$$M_C=-0.166\,hr^2+W_cr\left[0.053\sin^2\alpha+0.040\frac{\alpha}{\sin\alpha}-0.020\frac{\sin 2\alpha}{\sin\alpha}-0.080\sin\alpha(\pi-\alpha)+0.159\cos\alpha+0.135\right] \quad (20)$$

$$R_C=0.053W_c(1-\sin^2\alpha)+0.511\,hr \quad (21)$$

Again using the displacement theory of arches, Cain has obtained an equation for the horizontal deflection of a pipe:

$$\Delta X=\frac{2r^2}{EI}\int_0^{\frac{\pi}{2}}M\cos\phi d\phi \quad (22)$$

Substituting the general value of M as given in equation (14) and integrating

$$\Delta X=\frac{W_cr^3}{EI}\left[0.500\sin\alpha-0.082\sin^2\alpha+0.080\frac{\alpha}{\sin\alpha}-0.160\sin\alpha(\pi-\alpha)-0.040\frac{\sin 2\alpha}{\sin\alpha}+0.318\cos\alpha-0.208\right]-\frac{0.122hr^4}{EI} \quad (23)$$

Now according to the hypothesis of this paper, the maximum horizontal unit pressure on the sides of the pipe is equal to the modulus of passive resistance of the earth at the sides of the pipe multiplied by one half the horizontal deflection. That is

$$h=\frac{e\Delta x}{2} \quad (24)$$

Substituting this expression for h in equation (23)

$$\Delta X=\frac{KW_cr^3}{EI+0.061er^4} \quad (25)$$

in which

ΔX=horizontal deflection of flexible culvert pipe.

$K=0.500\sin\alpha-0.082\sin^2\alpha+0.080\frac{\alpha}{\sin\alpha}-0.160\sin\alpha(\pi-\alpha)-0.040\frac{\sin 2\alpha}{\sin\alpha}+0.318\cos\alpha-0.208.$

W_c=vertical load per unit length of pipe.

r=mean radius of pipe.

E=modulus of elasticity of pipe metal.

I=moment of inertia per unit length of cross section of pipe wall.

e=modulus of passive resistance of the enveloping earth.

This equation is offered tentatively for use in the design of flexible culvert pipes when the conditions of installation are sufficiently well known to justify the calculation of the vertical load on the pipe by means of Marston's conduit load theory, and the modulus of passive resistance of the filling material is known or can be estimated within a reasonable tolerance. Although the equation gives the horizontal deflection of the pipe, both theory and field observations indicate that the vertical and horizontal deflections are nearly the same under most conditions.

Equation (25) is general for all widths of bedding if the reaction is uniformly distributed over the full width of bedding as assumed. Values of the bedding constant K for widths from $\alpha=0$ to $\alpha=90°$ are as follows:

α Degrees	Bedding constant K
0	0.110
15	.108
22½	.105
30	.102
45	.096
60	.090
90	.083

The value of $K=0.083$ for $\alpha=90°$ agrees with the value obtained by Talbott for a pipe loaded uniformly over the top and bottom.[5]

FOUR CULVERTS INSTALLED AND TESTED

In order to test the applicability of this design formula, an experimental program was conducted in which four corrugated metal-pipe culverts were installed at the Iowa Engineering Experiment Station field laboratory at Ames, Iowa, and loaded with a clay embankment 15 feet high above the top of the culverts. The pipes for these experiments and the laboratory studies previously referred to were of four different diameters and United States gage thicknesses, namely: 36-inch, 16-gage; 42-inch, 14-gage; 48-inch, 14-gage; and 60-inch, 12-gage. Lightweight pipes were deliberately chosen so that the diameter changes under the fill loads would be relatively high. All of the pipes tested were furnished without charge by several manufacturers of corrugated pipe.

The embankment was 17 feet wide on top with side slopes about 1.2 to 1. The center 17 feet of the culverts under the full-height portion of the embankment consisted of eight sections each 25½ inches long, and constituted the test sections on which observations were made. The 20 feet of culvert at each end was merely filler pipe under the side slopes of the embankment. The culverts were spaced about 20 feet center-to-center. Figure 11 shows the culverts in place before the fill was constructed.

[5] Tests of Cast-Iron and Reinforced Concrete Culvert Pipe. Bulletin 22, Engineering Experiment Station, University of Illinois, p. 18. Reprinted 1922.

FIGURE 11.—CULVERTS IN PLACE BEFORE THE FILL WAS CONSTRUCTED.

The site chosen for the experimental embankment was the bottom of an old gravel pit which had been partially filled in with strippings of the pit. A subgrade was first prepared by hauling in clay from a borrow pit and making a fill which varied in thickness from about 6 inches on one side to about 1 foot 9 inches on the other. This subgrade was built level for a width of 17 feet in the center and sloped downward 6 inches in 20 feet at each side. The culverts were thus placed higher in the center than at the ends in order to make sure that no surface water would get to the center test sections. The center line of the subgrade ran east and west, the longitudinal axis of each culvert thus being in a north and south line.

After completion of this subgrade, the bedding for the culverts was prepared by cutting a trench having a circular-shaped cross section for the full length of each culvert. The radius of this trench was made 2 inches greater than the pipe to be laid in it and then refilled with sand, which was struck off with a template of the same radius as the pipe. The depth and width of each trench was such that when the pipes were laid they were in contact with the sand bedding for the bottom 90° of the circumference. Thus the pipes projected above the subgrade a distance equal to 0.85 of their diameter, making the projection ratio of Marston's theory, $p=0.85$.

The clay filling material was hauled in two-wheeled scrapers drawn by teams. The fill on each side of the south half of each culvert was hand tamped in 6-inch layers for a distance out from the sides equal to the diameter of the pipe, and for a depth equal to three-fourths the distance the pipe projected above the subgrade. The fill at the sides of the north half of each culvert and at all other places outside this tamped area was simply dumped from the scrapers and placed by shovels without any special manipulation affecting the density or other properties of the embankment. It was expected that the pipes with the untamped side fills would deflect more than those with the tamped side fills and this proved to be the case, as will be shown later. Figure 12 shows the layout of the culvert installation and testing apparatus.

The measurements and observations made during the experiments were directed toward three principal objectives.

1. The settlements of various elements of the pipes and adjacent embankment material was observed in order that the settlement ratio in Marston's theory could be calculated. Also, the unit weight of the fill material was measured and all other necessary data obtained so that the load on the culverts could be calculated according to this theory.

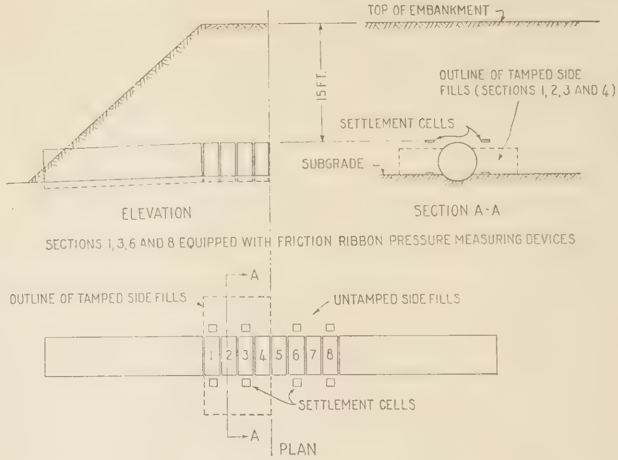

FIGURE 12.—LAYOUT OF CULVERT INSTALLATION AND TESTING APPARATUS.

2. The distribution of vertical load, vertical reaction, and horizontal pressures were measured in order to check the hypothetical distributions and to determine the value of the modulus of passive resistance of the clay filling material, both in the tamped and untamped condition.

3. The vertical and horizontal deflections of the pipes were measured for comparison with the hypothetical deflections computed by equation (25).

Marston's theory of loads on conduits[6] involves certain physical characteristics of both the conduit and the embankment and also a settlement ratio

$$r_{sd}=\frac{(s_m+s_g)-(s_f+d_c)}{s_m} \qquad (26)$$

in which

r_{sd}= the settlement ratio.
s_m= the settlement of the columns of earth at the sides of the culvert which extend from the subgrade to the level of the top of the pipe.
s_g= the settlement of the subgrade.
(s_m+s_g)= the settlement of the horizontal plane of the embankment which was level with the top of the pipe at zero fill.
s_f= the settlement of the pipe invert.
d_c= the vertical deflection of the pipe.
(s_f+d_c)= the settlement of the top of the pipe, due to its flexibility plus yielding of its foundation.

The settlements s_m and s_g were measured by means of Ames settlement cells, which are fully described in Bulletin 112 of the Iowa Engineering Experiment Station. There were 64 cells used in all. Half of these were placed on the subgrade at points 1 foot out from the sides of sections 1, 3, 6, and 8 of each culvert and these gave a measure of s_g. The other half were placed at the level of the top of the pipe and directly over the first group. These gave a measure of (s_m+s_g) and by subtracting the value of s_g, the value of s_m was obtained.

DISTRIBUTION OF PRESSURES ON PIPES MEASURED BY FRICTION RIBBONS

The settlements of the pipe inverts, s_f, were measured by means of an ordinary level and rod. By taking short sights, it was possible to read the rod to 0.001 foot with confidence. The shortening of the vertical

[6] The Theory of External Loads on Closed Conduits in the Light of the Latest Experiments, by Anson Marston. Bulletin 96, Iowa Engineering Experiment Station.

diameter, d_c, was measured by means of an inside caliper equipped with a sliding pointer and a steel scale graduated to 0.01 inch. The horizontal deflections were measured with the same instruments. All the deflections were measured between gage points drilled in a valley of the corrugations at the center of each individual pipe section.

The settlement ratios for the various culverts were readily determined from these data and formula (26), and are directly applicable for determining the loads on the pipes with untamped side fills. It is necessary, however, to apply a conversion factor to the settlement ratios for the sections with tamped side fills, since Marston's theory assumes that the modulus of compression of the fill material is uniform throughout the height of the embankment from the subgrade to the top. Where the side fills were tamped, this assumption does not apply, since the compression per unit of load for the tamped material is much less than for untamped material.

It can be shown that the settlement ratios obtained from test data and applicable to the tamped sections must be multiplied by the ratio of the modulus of compression of the tamped fill material to that of the untamped material before they can validly be used to compute the load on the tamped sections by Marston's load formula. Further, the value of s_m is proportional to the modulus of compression of the column of material within the height pB_c, and since this material was tamped adjacent to half of each culvert and untamped at the other half, the ratio of s_m for the tamped and untamped material will be equal to the ratio of the moduli of compression for tamped and untamped material; that is

$$\frac{s'_m}{s_m}=\frac{C'}{C}$$

The values of s_m and s'_m are readily obtainable from the test data. Settlements of the 60-inch culvert with tamped side fills are shown in figure 13, and settlements for the 60-inch culvert with untamped side fills are shown in figure 14.

The distribution of the vertical and horizontal pressures on the pipes was measured by means of stainless steel friction ribbons that were made to slide between two layers of canvas and that passed radially inward over a stainless steel roller at each end of the 25½-inch pipe sections. After the fill was placed, these ribbons were pulled and the pressure normal to the ribbon was assumed to be proportional to the pull required to start the ribbon in motion. The ribbons were ½ inch wide and 0.008 inch thick and very uniform in surface texture. Twenty ribbons were placed on sections 1, 3, 6, and 8 of each culvert, 10 being placed horizontally to measure vertical pressures and the other 10 placed vertically to measure horizontal pressures.

Each ribbon was mounted on either a strip of steel 1 inch wide or on a 1-inch angle that was welded to the pipe sections on top of the corrugations. The steel strips were placed at the top, bottom, and sides of the pipe, while short-legged angles were used at the 22½-degree points and equal-legged angles at the 45-degree points. The inner canvas surface was cemented directly to the steel strip or angle and the outer canvas was sewed around these shapes after the ribbon was placed. After all of the ribbons and canvases were in place, the section was installed in the culvert and surrounded with a sheet of pure gum rubber about one-sixteenth inch

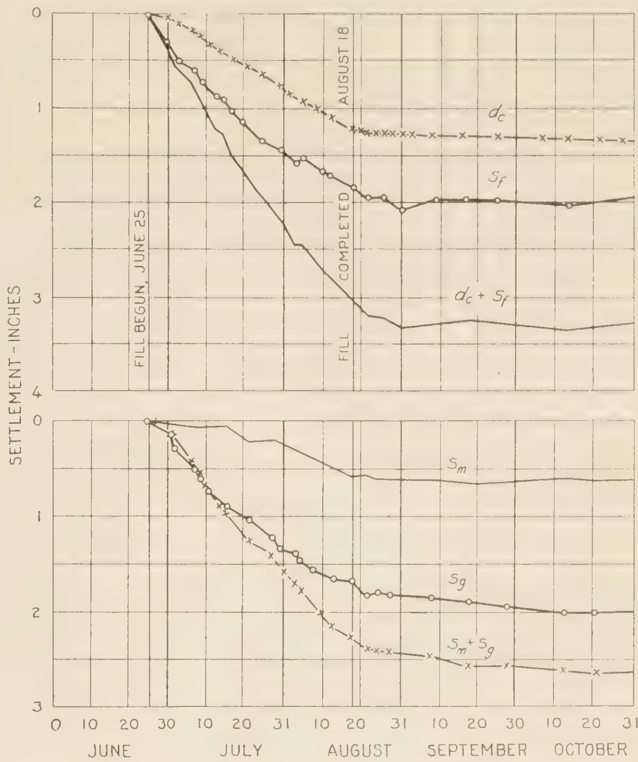

FIGURE 13.—SETTLEMENT OF THE 60-INCH CULVERT WITH TAMPED SIDE FILLS.

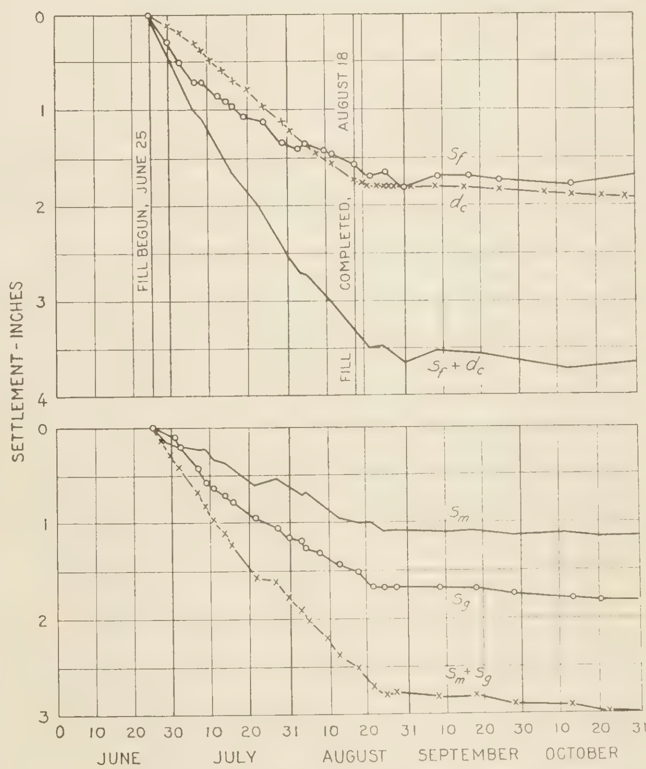

FIGURE 14.—SETTLEMENT OF THE 60-INCH CULVERT WITH UNTAMPED SIDE FILLS.

thick. These sheets were of sufficient length to go completely around the pipe with a generous overlap, and the ends were securely cemented. Each sheet was 3 feet wide, so the edges overlapped enough to permit

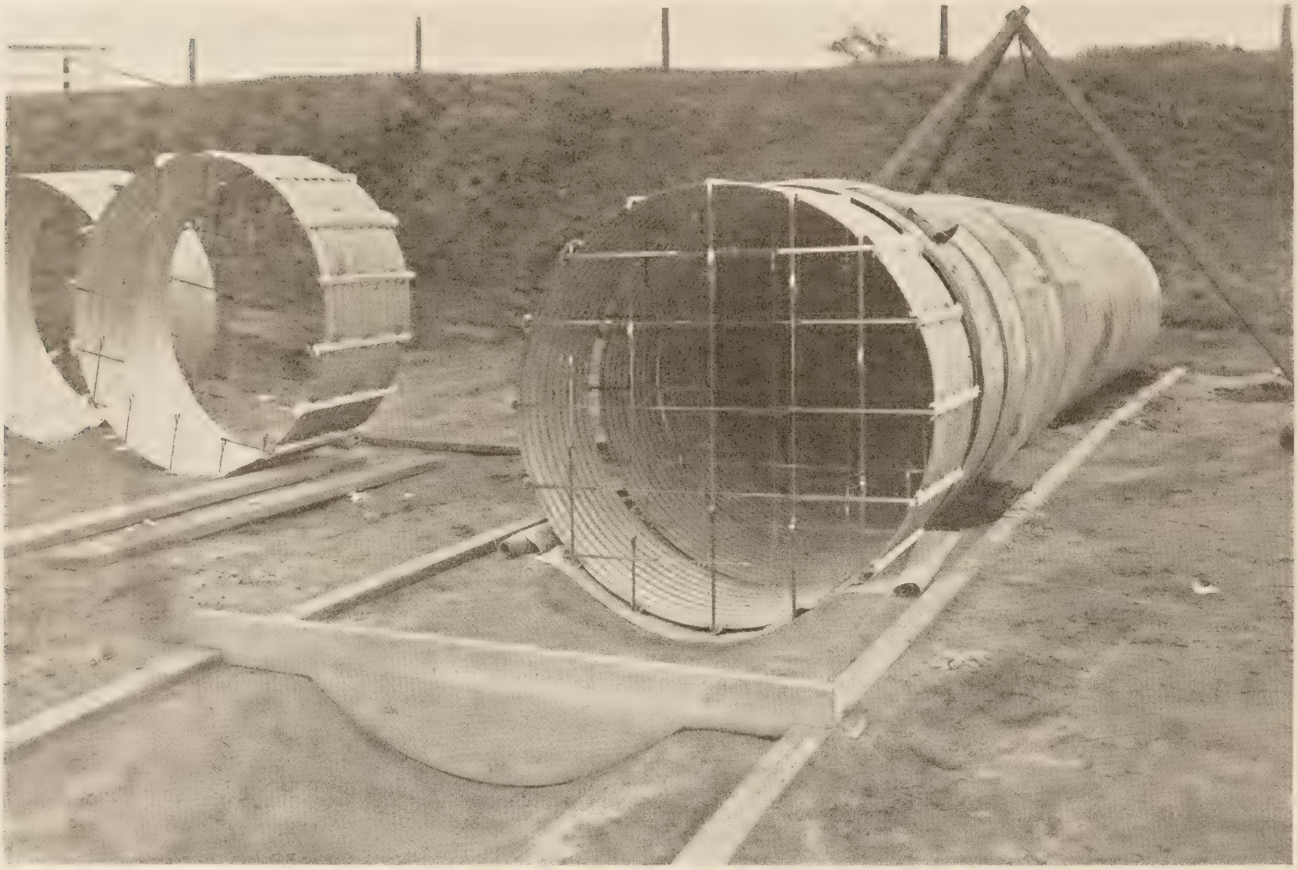

FIGURE 15.—SECTIONS OF CULVERT PIPE ON WHICH RIBBONS ARE BEING INSTALLED TO MEASURE VERTICAL AND HORIZONTAL EARTH PRESSURES.

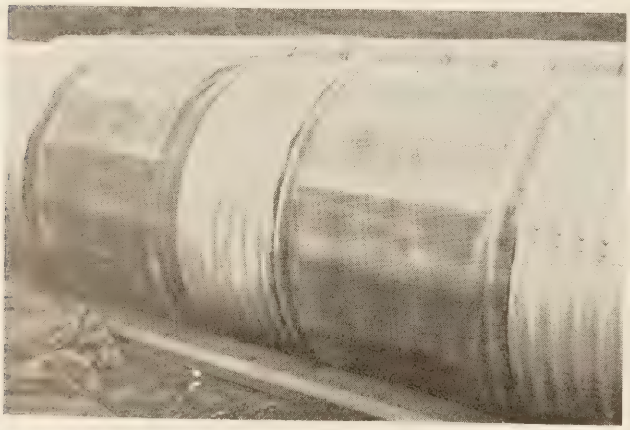

FIGURE 16.—SECTIONS OF CULVERT PIPE ON WHICH RIBBONS HAVE BEEN INSTALLED AND COVERED WITH A PROTECTIVE RUBBER JACKET.

them to be cemented to adjacent culvert sections. The purpose of the rubber jackets was to keep the dirt and moisture of the fill from entering the ribbon spaces. Ribbon-equipped sections during and after installation are shown in figures 15 and 16.

The ribbons have been of great value in these studies and have yielded much information in regard to the distribution of the various pressures, particularly the horizontal pressures, about which so little is known. They are not precision instruments, however, and the results obtained from many tests must be studied before specific statistical conclusions are justifiable.

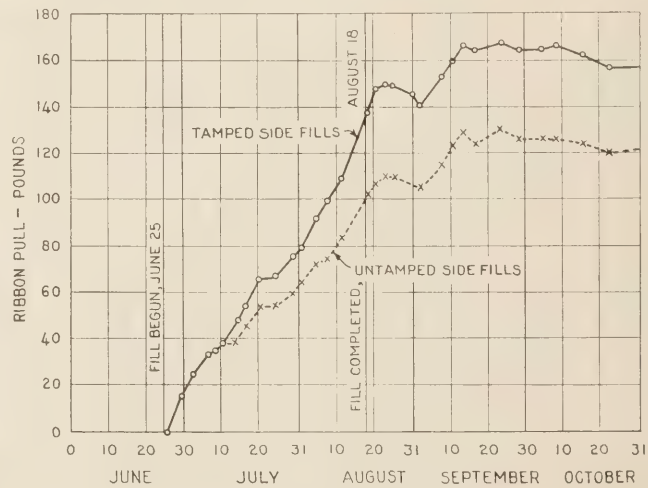

FIGURE 17.—AVERAGE PULL REQUIRED TO MOVE TOP RIBBONS ON 48-INCH PIPE.

Figure 17 shows the average pull required to move the top ribbons at various times during and after fill construction for the 48-inch pipe.

An attempt was made to calibrate the relations between pressure and pull on the ribbons by applying pressure to a ribbon in a similar assembly in the laboratory, but without success. For a given set of conditions, the ratio between pull and pressure remains fairly constant; but when other conditions are imposed, such as different moisture content of the canvas surfaces, the results are widely different. It has been impossible to

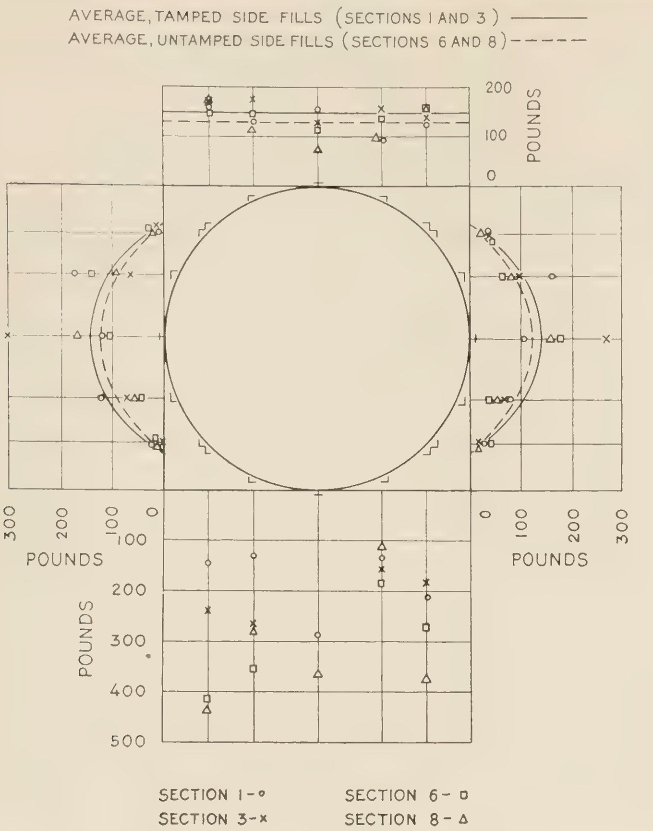

FIGURE 18.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 36-INCH PIPE ON AUGUST 26, 1936.

identify the conditions existing in the field trials with sufficient accuracy to attempt to reproduce them in the laboratory calibration. Instead, the ribbon assemblies on the field experimental culverts have been calibrated by calculating the vertical load on the pipes by means of Martson's load theory, using actual settlement and other data obtained in the experiments. Then, assuming the vertical loads were uniformly distributed over the top 180° of the pipe, the relation between pull on the top ribbons and vertical unit pressure has been obtained.

This assumption of uniform distribution of the vertical load on the pipes is based upon previous studies of loads on rigid pipe and rectangular box culverts, and upon a study of the pulls of top ribbons in these experiments. These pulls, while they varied widely in individual values, showed a definitely uniform distribution trend as indicated by the data shown in figures 18 to 21, inclusive. These data show the pressure distribution patterns on 1 day only, but they are typical of ribbon pulls prior to that date.

MEASUREMENTS MADE FREQUENTLY DURING FILL CONSTRUCTION AND AFTER COMPLETION

The conditions affecting the coefficient of friction between the ribbons and canvas surfaces on the top and the two sides of the pipe were in all probability identical and the calibration of the top ribbons has been applied to the side ribbons. The bottom ribbons, however, presented a different problem. In fact little credence is placed in the results obtained from the bottom ribbons, which were erratic in performance and of questionable reliability. This resulted from the fact that in placing the pipes with the ribbons assembled, it was

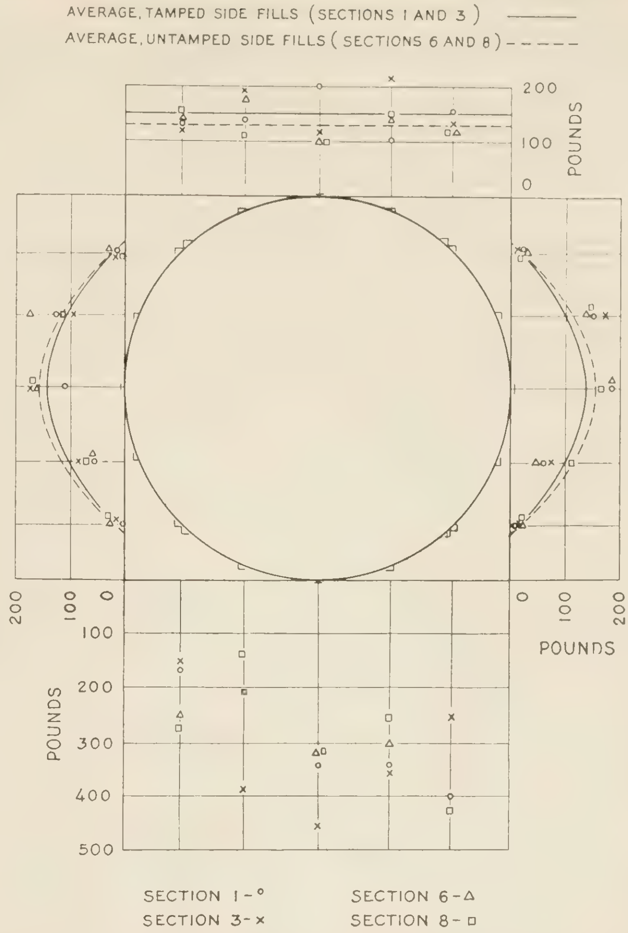

FIGURE 19.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 42-INCH PIPE ON AUGUST 26, 1936.

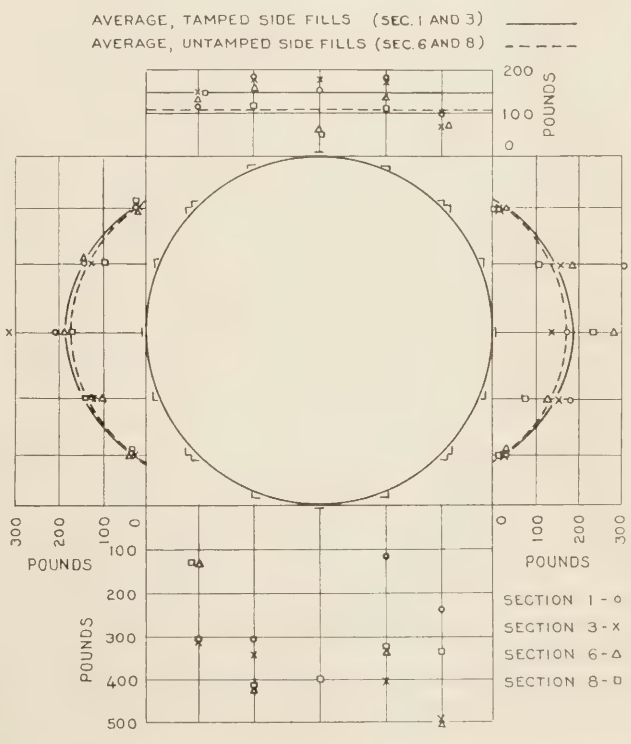

FIGURE 20.—PULLS REQUIRED TO MOVE RIBBONS ON 4 SECTIONS OF THE 48-INCH PIPE ON AUGUST 26, 1936.

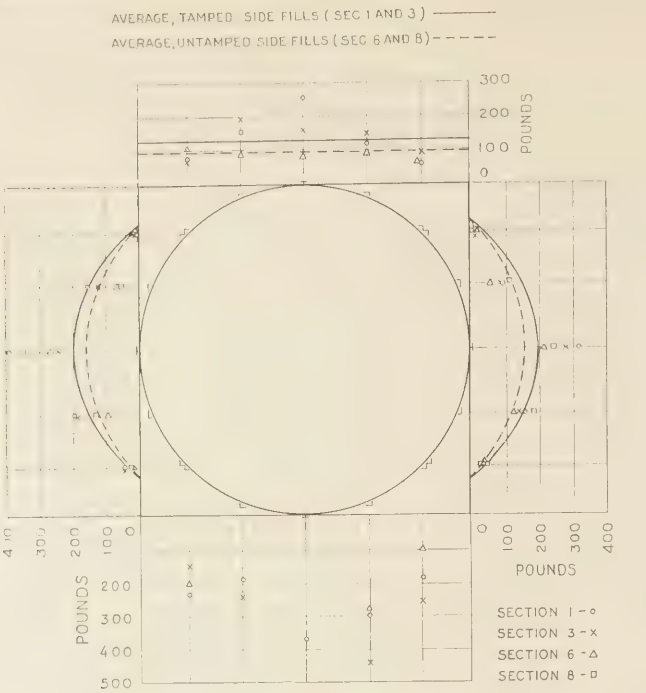

Figure 21.—Pulls Required to Move Ribbons on 4 Sections of the 60-Inch Pipe on August 26, 1936.

virtually impossible to prevent small quantities of the bedding sand from entering the ribbon and roller spaces; and although precautions were taken to prevent this, some particles did get in between the ribbons and canvases. Also, during construction of the embankment, when diameter changes and ribbon pulls were being measured every day, some dust and dirt was carried into the culverts on the shoes of the operators and found its way to the roller spaces. The distribution of the vertical reaction on the pipes must, therefore, be judged empirically rather than wholly on the basis of the bottom-ribbon pulls.

The pipes were placed in their beddings and the fill at the sides between the subgrade and the top of the pipes was placed during the first 3 weeks in June 1936. The fill over the top of the pipes was begun on June 25 and was completed to a height of 15 feet on August 18, 1936. During this loading period the settlements, deflections, and ribbon pulls were observed at least once for every 1-foot increment of fill. The observations were continued after completion of the fill at about weekly intervals until about October 1, and since then at less frequent intervals.

Some of the data that have been selected as typical for the period from June 25 to October 31 are shown graphically in figures 13, 14, and 17 to 23. The settlements of various elements of the embankment and the pipes of the 60-inch culvert with tamped and untamped side fills are shown in figures 13 and 14. The points through which the curves are drawn are the average of 4 settlement-cell readings for (s_m+s_g) and s_g, and vertical diameter change and invert settlement of 4 pipe sections for d_c and s_f. The average pull required to move the top ribbons on the 48-inch culvert is shown in figure 17 both for the tamped and untamped side fills. Here each point is the average of 10 ribbon pulls, 5 on each of 2 sections.

It appears from examination of the deflection and foundation-settlement curves that the maximum load

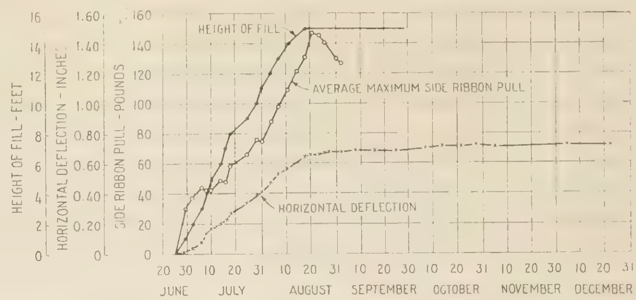

Figure 22.—Horizontal Deflections and Side-Ribbon Pulls for the 36-Inch Pipe With Tamped Side Fills.

on the culverts was reached within a very short time after the fill was completed, probably within less than a week. The ribbon pull, for the sections in untamped fills, however, shows a slight increase early in September. During the summer of 1936 there was a drought in the corn belt and the rainfall at Ames was practically negligible from the middle of June to September 4, accompanied by extremely hot and dry atmospheric conditions. On September 4 more than 4 inches of rain fell and was followed by more nearly normal atmospheric conditions. In the attempts to calibrate the ribbon assemblies in the laboratory, it was found that moisture in the canvas increased the starting pull on the ribbons and in all probability the increased pull on the field ribbons following this heavy rain resulted from increased moisture rather than from an increase in load on the pipes. A portion of the increase in ribbon pull may have been caused by the weight of water absorbed by the fill during this rainfall.

With these conditions in mind, the data taken on August 26 has been chosen as representative of the situation after the maximum load on the culverts was reached and before the ribbon pulls were affected by the change in meteorological conditions following the rain of September 4. The ribbon pulls on the top and side ribbons on August 26 are shown in figures 18 to 21. The patterns of distribution of pressure shown in these figures are typical of the patterns until the rain of September 4. After this date, the distribution of the pulls on the side ribbons became somewhat erratic.

In these ribbon-pull diagrams, lines have been drawn representing the average ribbon pulls for sections having tamped side fills and for sections having untamped fills. In the case of the top ribbon the readings were averaged numerically in accordance with the uniform pressure assumption. For the side ribbons, the readings have been weighted to obtain the average curves in accordance with the assumed parabolic distribution. The values of the maximum ordinates to the weighted average curves for the 36-inch culvert throughout the loading period are shown in figures 22 and 23.

TAMPING SIDE FILLS DOUBLED THEIR CAPACITY TO RESIST CULVERT DEFLECTION

In order to determine the relations between the lateral pressure on the pipes and the horizontal movement of the sides of the pipes, that is, the modulus of passive resistance of the fill material, it is necessary to interpret the pulls required to start the side ribbons in terms of pressure. This has been done in the following manner, since the laboratory calibration operations failed to produce results that could be applied to the field ribbons.

First the load on each experimental culvert was calculated by means of Marston's conduit load theory

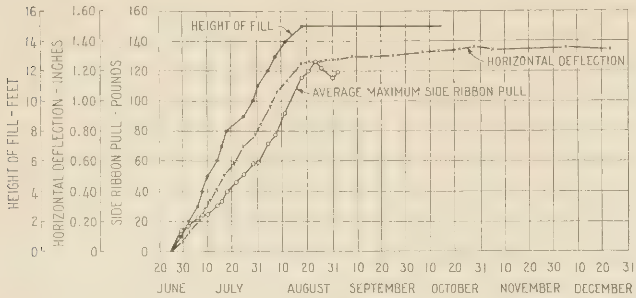

FIGURE 23.—HORIZONTAL DEFLECTIONS AND SIDE-RIBBON PULLS FOR THE 36-INCH PIPE WITH UNTAMPED SIDE FILLS.

using measured settlement data and the measured unit weight of the clay filling material (120 pounds per cubic foot). The load was divided by the area of the horizontal projection of the pipe to obtain the average vertical pressure on the pipe. This unit pressure was then divided by the average ribbon pull on the top ribbons to obtain the pressure on the ribbons per pound of pull required to start the ribbons. The test data and calculated results of these operations for all the culverts are shown in table 2. The average value of this ratio of unit pressure to pull is 0.0673 pound per square inch per pound of pull.

Having this ratio, and assuming that the conditions affecting it were the same for the side ribbons as for the top ribbons, the side-ribbon pulls may be multiplied by it to obtain the magnitude and distribution of the lateral pressure. Further, the modulus of passive resistance of the fill material may be obtained by dividing the maximum lateral unit pressure by ½ the horizontal deflection of the pipe. The data resulting from these calculations are given in table 3, and show an average modulus of passive resistance for the untamped clay in this fill to be 13.4 pounds per square inch per inch of movement, and 27.0 pounds per square inch per inch for the tamped filling material. It appears, therefore, that tamping the side fills in these experiments practically doubled their capacity to assist the pipes to carry the vertical earth load.

Having determined the modulus of passive resistance of the fill material and knowing the physical properties of the pipes, the deflection of the pipes under the calculated load can be determined by means of tentative design formula (25). The results of these calculations are shown in table 4. The measured horizontal and vertical deflections on August 26 are also shown for comparison. The calculated horizontal deflections show fairly close agreement with the actual measured values, thus serving to substantiate the load hypothesis given earlier in this paper.

TABLE 3.—*Moduli of passive resistance for tamped and untamped side fills for the various culverts tested*

Diameter of culvert[1]	Maximum ordinate of weighted average pull curve	Maximum lateral pressure	$\frac{\Delta x}{2}$	Modulus of passive resistance	
				Tamped side fills	Untamped side fills
Inches	*Pounds*	*Lb./sq. in.*	*Inch*	*Lb./sq. in. /in.*	*Lb./sq. in. /in.*
36 T	141	9.49	0.335	28.30	
36 U	122	8.21	.635		12.93
42 T	141	9.49	.385	24.67	
42 U	153	10.30	.695		14.82
48 T	187	12.59	.430	29.30	
48 U	173	11.65	.810		14.39
60 T	193	12.99	.505	25.71	
60 U	154	10.37	.895		11.60
Average				27.00	13.43

[1] T indicates tamped side fills, U indicates untamped side fills.

TABLE 4.—*Calculated horizontal deflections and measured horizontal and vertical deflections of the various pipes*

Diameter of culvert[1]	Calculated horizontal deflection		Measured horizontal deflection[2]		Measured vertical deflection[2]	
	Deflection	Percentage of nominal diameter	Deflection	Percentage of nominal diameter	Deflection	Percentage of nominal diameter
Inches	*Inches*		*Inches*		*Inches*	
36 T	0.83	2.31	0.67	1.86	0.71	1.97
36 U	1.17	3.25	1.27	3.53	1.21	3.36
42 T	.94	2.24	.77	1.83	.84	2.00
42 U	1.41	3.35	1.39	3.31	1.31	3.12
48 T	.97	2.02	.86	1.79	1.02	2.12
48 U	1.56	3.25	1.62	3.38	1.56	3.25
60 T	.87	1.45	1.01	1.68	1.26	2.10
60 U	1.72	2.88	1.79	2.98	1.82	3.03

[1] T indicates tamped side fills, U indicates untamped side fills.
[2] On August 26, 1936.

These calculations reveal the relative influence of the inherent strength of the pipe and the side restraint afforded by the fill upon the deflection the culverts assumed under the fill. In the design formula (25), the term EI in the denominator represents the influence of the pipe strength and the second term $0.061\ er^4$ represents the influence of the passive resistance of the fill material. In these experiments, the second term was 1.65 times the first term for the 36-inch pipe with untamped side fills and this ratio varied up to a

TABLE 2.—*Test data and ratio of calculated unit pressure to pull on ribbons for the various culvert pipes tested*

Diameter of culvert[1]	(s_m+s_g)	s_g	s_m or s'_m	d_c	s_f	(d_c+s_f)	r_{sd}	Ratio $\frac{s'_m}{s_m}$	Adjusted r_{sd}	Load on pipe			Average pull on top ribbon	Ratio of unit pressure to pull
Inches	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*	*Inches*				*Lb. per lin. ft.*	*Lb. per lin. in.*	*Lb. per sq. in.*	*Pounds*	*Lb. per sq. in. per lb. pull*
36 T	2.26	1.74	0.52	1.71	1.71	2.42	−0.31	0.57	−0.18	4,100	342	9.08	144	0.0630
36 U	1.91	1.00	.91	1.21	1.05	2.26	−.38			3,600	300	7.84	127	.0617
42 T	2.29	1.61	.68	.84	1.60	2.45	−.24	.56	−.14	5,100	425	9.71	150	.0647
42 U	2.21	1.00	1.21	1.31	1.21	2.52	−.26			4,500	375	8.45	130	.0650
48 T	2.13	1.42	.71	1.02	1.33	2.35	−.31	.56	−.17	5,500	458	9.18	149	.0616
48 U	2.00	.91	1.09	1.56	.86	2.42	−.38			4,900	408	8.06	110	.0732
60 T	2.41	1.81	.60	1.27	1.94	3.21	−1.33	.57	−.76	5,800	483	7.79	127	.0614
60 U	2.77	1.67	1.10	1.82	1.68	3.50	−.67			6,100	508	8.09	92	.0880
Average														.0673

[1] T indicates tamped side fills; U indicates untamped side fills.

maximum of 16.05 for the 60-inch pipe with tamped side fills. From these facts it seems probable that pipes of greater thicknesses would have deflected but slightly less than did the pipes actually used, especially the larger sizes. On the other hand, greater consolidation of the fill material, by compaction at optimum moisture content or by some artificial stabilization process, might have reduced the deflections materially.

FORMULAS GIVEN FOR MOMENT, THRUST, AND SHEAR

Although the deflection of flexible culvert pipe is the phenomenon of primary interest from the structural standpoint, because failure of flexible pipes occurs by excessive deflection rather than excessive stress, the stress distribution around the periphery of the pipes may also be of some interest. From equations (20) and (21) the moment and thrust at the bottom of the culvert pipes in these experiments ($\alpha=45°$) are

$$M_C=0.157W_cr-0.166hr^2 \quad (27)$$
$$R_C=0.026W_c+0.511hr \quad (28)$$

Having the moment and thrust at the bottom of the pipe, the moment, thrust, and shear at any other point on the ring may be obtained from the equations of equilibrium. It is convenient to write the equations for these quantities for vertical loads and for horizontal loads separately.

For moment, thrust, and shear caused by vertical loads—

When ϕ lies between 0 and 45°:

$$M_D=W_cr(0.183-0.026\cos\phi-0.354\sin^2\phi) \quad (29)$$
$$R_D=W_c(0.026\cos\phi+0.707\sin^2\phi) \quad (30)$$
$$S_D=W_c(0.707\sin\phi\cos\phi-0.026\sin\phi) \quad (31)$$

When ϕ lies between 45° and 90°:

$$M_D=W_cr(0.360-0.026\cos\phi-0.500\sin\phi) \quad (32)$$
$$R_D=W_c(0.026\cos\phi+0.500\sin\phi) \quad (33)$$
$$S_D=W_c(0.500\cos\phi-0.026\sin\phi) \quad (34)$$

When ϕ lies between 90° and 180°:

$$M_D=W_cr(0.110-0.026\cos\phi-0.250\sin^2\phi) \quad (35)$$
$$R_D=W_c(0.026\cos\phi+0.500\sin^2\phi) \quad (36)$$
$$S_D=W_c(0.500\sin\phi\cos\phi-0.026\sin\phi) \quad (37)$$

For moment, thrust, and shear caused by horizontal loads—

When ϕ lies between 0 and 40°:

$$M_D=hr^2(0.345-0.511\cos\phi) \quad (38)$$
$$R_D=0.511hr\cos\phi \quad (39)$$
$$S_D=0.511hr\sin\phi \quad (40)$$

When ϕ lies between 40° and 140°:

$$M_D=hr^2(0.199-0.500\cos^2\phi+0.143\cos^4\phi) \quad (41)$$
$$R_D=hr(\cos^2\phi-0.568\cos^4\phi) \quad (42)$$
$$S_D=hr(\sin\phi\cos\phi-0.568\sin\phi\cos^3\phi) \quad (43)$$

When ϕ lies between 140° and 180°:

$$M_D=hr^2(0.345+0.511\cos\phi) \quad (44)$$
$$R_D=-0.511\,hr\cos\phi \quad (45)$$
$$S_D=0.511hr\sin\phi \quad (46)$$

Moment diagrams for these equations are shown in figure 24 in terms of W_cr for vertical loads and hr^2 or $\frac{e\Delta xr^2}{2}$ for horizontal loads. The net moment is the algebraic sum of the moments caused by the vertical and horizontal loads. Using the calculated loads and deflections and the moduli of passive resistance for tamped and untamped materials, the net flexural stress in the ring of each experimental culvert is as shown in figure 25.

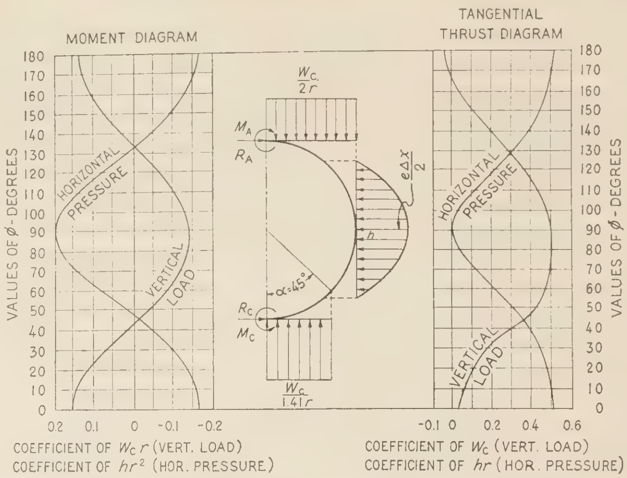

FIGURE 24.—MOMENT AND TANGENTIAL THRUST DIAGRAMS FOR FLEXIBLE CULVERT PIPES.

Since the reaction is less distributed than the vertical load, the greatest bending stress occurs at the bottom of the pipe. This stress diminishes rapidly as ϕ increases, and reverses direction at about 35° to 40° and attains a maximum negative value at about 60°. From 60° to 140° the stresses are relatively low. Beyond this point the stress increases rapidly to the maximum value at the top which is about ⅔ to ¾ the magnitude of the stress at the bottom.

The calculated maximum stresses in the pipes are rather high and in 5 of the 8 culverts they exceed the elastic limit of the metal as determined on companion specimens in the laboratory tests described earlier in this report. However, the length of arc overstressed is relatively short and no appreciable effect on the rate of deflection of the pipe is discernible.

The tangential thrust in the pipe walls is of interest in connection with the design of riveted or bolted longitudinal joints connecting the corrugated sheets of which the pipes are fabricated. The hypothetical loads yield thrust diagrams as shown in figure 24. Using the calculated values of W_c, e, and Δx, the net thrusts per inch length for each experimental pipe have been calculated and are shown in figure 26. The radial shears are small and of little importance in the design of this type of structure.

DEFLECTION CONTINUED AFTER FILL COMPLETION

A phenomenon of considerable interest that has been measured in this and an earlier study of corrugated pipe culverts is that the pipes continue to deflect slowly long after the fill is completed and the maximum vertical load has been attained. In these experiments the pipes have continued to deflect for more than a year after the completion of the fill. The average amount of this additional vertical deflection of the pipes with tamped side fills has been 0.25 percent of the nominal diameter between August 26, 1936 and September 15, 1937, and 0.38 percent for the pipes with untamped side fills.

This same phenomena has been observed in another experiment conducted at Ames in which a 42-inch, 10-gage pipe has been loaded under a 15-foot fill of loam soil for a period of nearly 10 years. This culvert is mounted on a system of weighing platforms and levers so that the vertical load on it can be determined at the time deflections are measured. The load and deflection history of this culvert, which was constructed with untamped side fills, is given in table 5.

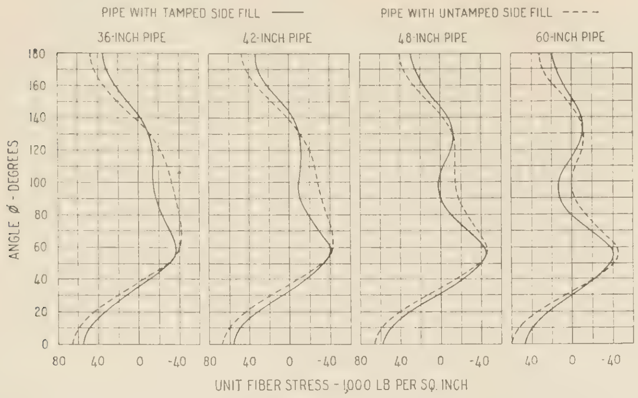

FIGURE 25.—NET FLEXURAL STRESS IN RINGS OF CULVERT PIPE TESTED.

TABLE 5.—*Load and deflection data for a 42-inch culvert pipe under a 15-foot fill of untamped loam soil*

Date	Load	Vertical deflection	
		Deflection	Percentage of nominal diameter
	Lb. per lin. ft.	*Inches*	*Percent*
Dec. 16, 1927 (fill completed)	5,300	1.43	3.41
July 1, 1928	6,000	1.78	4.24
Aug. 15, 1932	6,800	2.44	5.81
Apr. 19, 1935	6,300	2.59	6.17
Sept. 15, 1937	6,400	2.62	6.24

The fact that flexible pipes continue to deform slowly after the fill is completed is in all probability analogous to the widely known fact that all structures resting on earth foundations continue to settle long after the maximum load on the footings is applied. Apparently the fill material at the sides of the pipes slowly recedes in response to the pressure acting between the pipe and the fill and this permits the pipe to deflect even when there is no increase in the vertical load. This phenomenon may be of importance in supplying data upon which specifications for flexible pipe can be based. The deflection at the time of fill completion should possibly be specified for practical reasons; but this specified deflection should be related to the ultimate deflection, which may occur years later and which determines the load capacity of the pipe.

NEED FOR FURTHER STUDIES INDICATED

One of the most important contributions of this research has been the knowledge gained regarding the relationship between the pressure developed at the sides of the pipes and the deflection, but there are many important elements of this relationship that need further study. In these experiments, the straight-line increase of both deflections and side pressures as the fill was constructed justifies the use of a constant ratio between deflection and pressure in this ordinary method of construction.

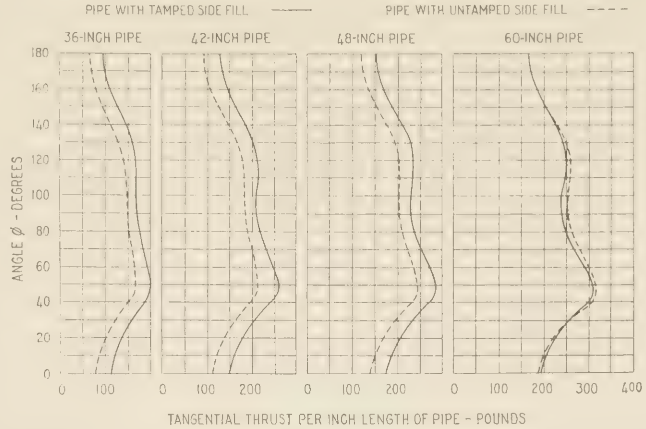

FIGURE 26.—CALCULATED TANGENTIAL THRUSTS IN EACH OF THE PIPES TESTED.

It should be pointed out, however, that each increment of deflection and side pressure was accompanied by an increment of fill or vertical pressure on both the culvert and the material at the sides of the pipe, and the effect of the vertical pressure on the ratio of deflection to side pressure is not revealed in these studies. Therefore, when a flexible pipe is "strutted" before the fill is placed, as is frequently done when high fills are encountered, and the struts are removed after completion of the fill, a different situation is presented and in all probability the ratio between deflection and side pressure will not be a constant in this case.

Further, there is much to be learned in regard to the characteristics of the side fill materials and their effect on the modulus of passive resistance. In these experiments, it was shown that tamping the side fills approximately doubled the modulus, with a consequent reduction in deflection of the pipe. Probably if the tamping had been done under optimum moisture conditions, a greater density and a greater modulus might have been attained.

The width of bedding of all the pipes in these studies was the same, the value of the angle α being 45 degrees. Further experiments should be conducted to obtain data on the effect of other bedding widths.

The hypothesis advanced in this paper has been sufficiently sustained by the experiment that a program to correlate the hypothesis with actual field performance would seem to be justified. In any such program, the observation of field culverts should begin at the time the subgrade for the pipe is prepared so that bedding conditions could be noted in detail and settlement plates installed on the subgrade and at the level of the top of the pipe for determination of practical design values of the settlement ratio. Then deflections of the pipe should be measured at suitable intervals during construction of the fill and after its completion. Also, the characteristics of the side fills should be observed in detail in an attempt to develop working values of the modulus of passive resistance of various types of fill materials.

EIGHTH INTERNATIONAL ROAD CONGRESS TO BE HELD IN JUNE

The Eighth International Road Congress will be held in June 1938 at The Hague, Netherlands, under the auspices of the Permanent International Association of Road Congresses. The Netherlands Government has invited all members of the Association to attend.

The sessions of the Congress are tentatively scheduled to begin on Monday, June 20, and to extend through Saturday, June 25. All-day excursions are planned for Monday, June 27, through Friday, July 1. A trip through The Hague and its environs and the ceremonial closing session of the Congress are tentatively scheduled for Saturday, July 2. Definite and more detailed information as to the schedule of the Congress will be announced later by the Association.

An exhibition of road equipment and materials, several receptions, and numerous excursions are planned. For the ladies who do not want to attend the sessions, separate excursions or visits will be organized for June 21, 22, and 23.

Persons desiring to attend the Congress can obtain temporary membership in the Association by paying a fee of 150 French francs ($4.90 at the rate of exchange on Jan. 25, 1938). Applications and fees can be sent to Mr. Thos. H. MacDonald, Chairman, American National Committee of the Permanent International Association of Road Congresses, Pan American Building, NW., Washington, D. C.

Association members are entitled to attend all sessions of the Congress as well as the receptions and official excursions taking place during the Congress. They are privileged to use the travel facilities and tariff reductions granted on behalf of the Congress. They receive, before the Congress opens, the official reports on the subjects dealt with at the sessions, and after the Congress a general account of the work accomplished.

The following program of subjects has been announced:

FIRST SECTION (CONSTRUCTION AND MAINTENANCE)

First Question:
- *a.* Progress, since the Congress at Munich, with the use of cement for carriageway surfacings.
- *b.* Brick surfaces.
- *c.* Surfaces in special materials such as cast-iron, steel, rubber.

Second Question:

Progress since the Congress at Munich in the preparation and use of:
- *a.* Tar.
- *b.* Bitumen (asphalt).
- *c.* Emulsions. For the construction and maintenance of carriageways.

SECOND SECTION (USE, REGULATION, AND ADMINISTRATION)

Third Question:

Accidents on roads:
- *a.* Bases of statistical returns and their unification.
- *b.* Methods of investigation into the causes of accidents and means for their prevention.

Fourth Question:

The segregation of the various classes on the highway:
- Carriageways (single and dual).
- Cycle tracks.
- Footways.
- Service roads in connection with "Ribbon Development," parking places.

Road junctions and crossings:
- *a.* A study of the circumstances which make these provisions desirable or undesirable.
- *b.* Application to motor roads.

FIRST AND SECOND SECTIONS (COMBINED)

Fifth Question:

Examination and standardization of carriageway surfacings from the point of their:
- *a.* Slipperiness or rugosity and resistance to skidding.
- *b.* Light value or the degree to which they absorb light (under artificial illumination).

Sixth Question:

Examination of the subsoil of roads:
- *a.* Determination of the properties of subsoils: Methods of testing, and testing apparatus.
- *b.* Influence of the properties of the subsoil on the construction of roads (foundations and surfaces) and their maintenance.

The Association makes no provision for sending the reports and proceedings of its congresses to persons other than members. All persons, libraries, and organizations, desiring to receive these reports and proceedings should therefore apply for membership.

Reductions in fares for travel in the Netherlands have been offered to Congress members by certain railways, shipping companies, and air lines in the Netherlands. The Association is attempting to obtain for Congress members reductions in fares from railroads and shipping companies of various other countries.

Everyone attending the Congress will be able to find suitable hotel accommodations at The Hague. Several hotels will grant special reductions to Congress members. Hotel accommodations will be available at reasonable rates; for room and breakfast, $1.40 to $3.40 per day; for room and full board, $2.25 to $5.60 per day; room with bath will cost an additional 60 to 90 cents per day. Modest but good accommodations can be obtained at lower prices at numerous boarding houses.

The planned excursions will be 1-day trips or less, beginning and ending at The Hague, so the excursionists need not carry luggage. Whole-day excursions can be expected to cost from $2.80 to $3.40, while short excursions will cost from $0.90 to $1.40.

Many places of particular interest to Congress members will be visited, and ample time will be afforded for careful inspection and study of the more important features. Excursion itineraries will include visits to roads built in lowlands on very poor subgrades, inspection of many large bridges and of tunnel construction at Rotterdam, and observations of the work of reclaiming the famed Zuiderzee. An excursion will be made to the big dike that cuts off the Zuiderzee from the outer sea and forms a new motor road between the northeastern provinces and the heart of the country.

Numerous main highways and secondary roads will be inspected, so that Congress members can get a general idea of the progress made in solving road problems in the Netherlands. If possible a trip will be made at night over the highway from Amsterdam to Haarlem, which is lighted the entire distance.

General sightseeing trips will also be made, including visits to the cheese market at Alkmaar, flower and tree nurseries, quaint towns along the Zuiderzee coast, the Dutch moorland, etc.

Exhibits will show what has been done in the Netherlands during the past few years to improve the network of roads as well as to regulate and safeguard traffic.

The Seventh International Road Congress, held in Munich, Germany, in September 1934, was attended by approximately 2,000 delegates from 50 countries.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF JANUARY 31, 1938

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 1,122,820	$ 551,410	64.3	$ 2,710,041	$ 1,355,020	124.6	$ 3,099,020	$ 1,549,055	141.6	$ 4,391,695
Arizona	1,673,415	1,212,650	81.4	1,065,082	704,210	49.5	888,078	597,388	54.1	605,575
Arkansas	2,879,864	2,876,323	183.8	1,281,294	1,277,611	85.9	23,505	23,177		2,221,807
California	6,649,769	3,676,359	149.9	7,085,194	3,720,044	116.0	1,832,467	982,813	32.2	530,038
Colorado	3,172,067	1,771,163	119.8	629,200	341,560	27.6	1,056,150	572,398	37.2	1,710,005
Connecticut	783,198	389,196	9.3	231,520	108,123	.7				1,513,589
Delaware	463,922	231,872	18.1	70,588	35,294	2.8	579,739	265,811	17.7	960,035
Florida	343,682	171,253	14.7	2,763,372	1,381,686	60.2	772,270	386,135	12.2	2,478,109
Georgia	2,113,634	1,024,810	122.5	4,485,023	2,242,507	203.4	2,879,354	1,439,677	126.4	3,921,543
Idaho	2,593,499	1,532,132	198.9	743,890	443,836	56.1	744,295	443,043	43.3	553,955
Illinois	9,986,641	4,930,085	284.4	5,375,161	2,683,473	152.5	6,001,300	3,000,650	86.7	363,321
Indiana	5,688,691	2,823,161	131.9	2,696,415	1,348,207	86.2	1,872,300	936,150	44.3	1,543,176
Iowa	6,554,603	2,968,790	210.2	4,302,663	1,924,718	127.8	2,412,244	1,122,150	68.5	71,630
Kansas	3,838,259	1,916,953	203.9	2,930,392	1,465,097	79.3	1,874,305	937,154	108.1	3,013,055
Kentucky	2,096,616	998,308	64.0	3,611,784	1,805,892	97.3	1,414,098	707,049	80.0	2,173,749
Louisiana	381,379	185,898	9.3	6,555,335	1,430,435	45.2	5,547,243	892,045	23.0	1,688,814
Maine	1,869,761	934,879	52.6	1,908,936	954,468	42.2	549,530	274,760	13.5	118,081
Maryland	930,330	465,145	13.4	1,814,594	905,534	28.9	460,360	230,180	.5	1,493,949
Massachusetts	4,403,913	2,201,955	20.2	1,828,245	914,122	3.3	705,420	352,710	5.0	1,619,545
Michigan	6,459,043	3,167,692	166.9	5,421,010	2,711,005	130.8	1,734,971	792,660	25.3	50,529
Minnesota	6,209,382	3,090,385	289.6	2,783,053	1,380,282	120.1	2,061,662	1,020,831	103.9	1,029,238
Mississippi	2,199,190	1,096,851	102.0	4,319,990	2,204,590	200.0	1,487,740	738,620	58.4	2,593,115
Missouri	8,074,930	4,014,743	443.3	4,462,675	2,127,557	120.5	3,541,856	1,379,471	74.7	1,317,097
Montana	3,824,816	2,144,049	277.7	1,846,412	1,038,231	86.4	571,734	321,594	33.8	1,986,038
Nebraska	2,365,131	1,182,565	239.8	4,418,217	2,196,292	409.2	2,984,508	861,941	118.4	1,902,067
Nevada	2,350,747	2,023,616	106.8	698,358	601,695	62.3	82,190	71,270	5.2	673,602
New Hampshire	360,696	177,963	6.5	480,084	237,150	9.5	311,182	154,753	5.0	864,953
New Jersey	1,674,608	760,009	20.2	1,669,700	834,530	12.5	1,367,360	682,795	7.5	1,209,232
New Mexico	2,940,681	1,802,689	229.0	2,084,983	1,371,538	99.8	352,746	187,444	16.8	199,386
New York	14,161,686	6,506,706	238.5	12,141,954	6,024,460	206.8	2,523,490	1,247,102	42.2	263,327
North Carolina	4,768,607	2,379,740	372.3	4,360,915	2,010,673	141.8	2,682,883	1,306,392	132.0	1,435,123
North Dakota	1,035,479	1,031,251	186.5	1,310,220	1,289,710	77.4	955,709	955,700	59.2	2,445,572
Ohio	3,673,133	1,766,053	51.0	9,122,269	4,524,626	97.4	1,651,515	826,193	19.4	4,987,905
Oklahoma	2,918,620	1,523,667	143.6	2,736,560	1,414,738	114.2	1,492,074	753,183	59.7	2,964,543
Oregon	3,616,277	2,133,704	126.9	1,722,209	1,011,655	83.8	662,848	404,440	16.0	738,817
Pennsylvania	11,604,873	5,774,005	164.8	6,686,538	3,323,829	94.1	3,377,053	1,687,489	58.7	2,089,737
Rhode Island	824,252	403,311	7.1	1,045,330	522,665	14.0	8,930	4,465		774,458
South Carolina	3,295,514	1,377,202	246.4	4,856,415	2,055,705	223.1	1,498,948	652,023	60.2	771,773
South Dakota	2,462,981	1,392,121	247.0	1,638,066	905,760	173.5	467,843	257,723	58.3	2,991,657
Tennessee	1,846,076	922,109	69.4	1,367,384	683,692	40.6	1,778,040	889,020	57.2	4,048,696
Texas	11,039,999	5,506,327	770.5	10,897,080	5,418,952	517.2	3,478,283	1,686,449	221.3	3,805,476
Utah	1,329,852	947,399	130.0	775,530	554,172	61.0	187,080	133,304	1.4	977,044
Vermont	1,016,663	489,925	29.2	1,679,730	739,095	43.1	6,730	3,169		61,552
Virginia	2,930,010	1,452,923	140.7	2,473,147	1,197,158	54.4	3,301,692	1,645,796	96.0	689,929
Washington	1,953,223	1,009,420	75.1	3,323,312	1,743,512	42.3	533,365	280,300	8.7	460,932
West Virginia	1,226,281	616,845	39.3	1,336,886	711,081	30.2	555,241	434,280	15.1	1,930,947
Wisconsin	8,040,395	3,854,287	269.4	4,701,047	2,159,177	107.5	232,942	111,700	4.7	1,076,949
Wyoming	2,447,097	1,416,767	255.6	1,290,812	777,929	141.1	201,870	124,730	24.4	208,179
District of Columbia										
Hawaii	640,163	311,939	11.5	431,329	212,667	6.4	293,360	144,695	4.8	1,158,907
Puerto Rico				947,719	471,880	20.0	130,840	65,420	.3	87,700
TOTALS	174,836,468	91,138,605	7,409.2	155,117,663	77,497,843	4,930.5	73,228,363	34,537,297	2,282.9	76,766,151

CURRENT STATUS OF UNITED STATES WORKS PROGRAM HIGHWAY PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31, 1938

STATE	APPORTIONMENT	COMPLETED			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	Estimated Total Cost	Works Program Funds	Miles	
Alabama	$ 4,151,115	$ 3,923,416	$ 3,874,665	136.9	$ 250,100	$ 243,478	7.8				$ 32,972
Arizona	2,569,841	3,153,137	2,501,555	193.7	38,548	38,548					29,738
Arkansas	3,352,061	3,146,869	3,117,422	350.5	181,216	180,523	9.7				54,115
California	7,747,928	7,950,258	7,534,238	263.9	138,156	138,156					75,534
Colorado	3,395,263	2,382,800	2,290,828	99.4	89,597	89,596	6.0	$ 8,200	$ 8,200		1,006,639
Connecticut	1,418,709	1,190,950	1,098,858	17.9	232,031	206,330	4.5	124,130	64,435	0.2	49,086
Delaware	900,310	871,470	843,920	66.4	10,234	10,234	.2	26,712	26,712	4.4	19,444
Florida	2,597,144	2,603,553	2,529,126	99.1	38,957	38,957					29,061
Georgia	4,988,967	1,903,869	1,860,597	109.2	2,288,924	2,020,544	105.6	442,230	442,230	29.6	665,596
Idaho	2,222,747	2,273,614	2,167,698	185.9	33,341	33,341					21,707
Illinois	8,694,009	8,139,409	7,875,966	465.0	685,010	685,010	23.2				133,033
Indiana	4,941,255	5,175,857	4,841,748	237.9	49,000	49,000					50,508
Iowa	4,991,664	5,241,284	4,885,935	528.3	105,902	104,865	.3				864
Kansas	4,994,975	4,679,497	4,626,032	370.6	251,823	249,651	21.6	38,500	38,500	.5	80,793
Kentucky	3,726,271	3,598,480	3,429,673	355.0	271,221	271,221	3.5				25,377
Louisiana	2,890,429	2,835,498	2,522,062	166.2	300,666	240,049	1.6	97,087	97,070	10.4	31,247
Maine	1,676,799	1,630,250	1,616,398	74.4	60,849	58,250	1.7				2,151
Maryland	1,750,738	773,435	766,648	27.2	467,536	467,536	10.0	168,198	134,366	4.3	382,189
Massachusetts	3,262,885	2,219,079	2,218,903	18.2	802,410	411,640	.2	1,136,726	568,363	.8	63,979
Michigan	6,301,414	6,645,939	5,950,255	288.6	284,921	284,921	3.3	223,938	63,746	.3	2,492
Minnesota	5,277,145	6,388,859	5,170,685	896.0	93,670	85,850	5.9				20,611
Mississippi	3,457,552	3,153,110	3,148,449	220.8	224,146	223,106	14.9	10,800	10,800	.6	75,196
Missouri	6,012,652	5,284,555	5,136,674	776.9	790,784	736,082	.7	34,391	32,294		107,602
Montana	3,676,416	3,595,845	3,532,169	200.7	95,385	95,385	.1	8,462	8,462		40,401
Nebraska	3,870,739	3,432,515	3,322,815	362.3	445,830	445,830	8.3	70,270	70,270	1.8	31,824
Nevada	2,243,074	2,302,820	2,193,967	110.1	84,970	38,146	1.7				10,961
New Hampshire	945,225	875,636	843,253	37.7	121,092	101,301	5.7				671
New Jersey	3,129,805	1,355,077	1,334,220	29.4	1,754,189	1,754,189	6.1	34,468	34,468		6,928
New Mexico	2,871,397	2,810,989	2,805,816	213.7	43,071	43,071		14,681	12,196		10,314
New York	11,046,377	10,741,839	10,276,419	170.1	316,300	276,300	1.9	62,700	62,700	.2	430,958
North Carolina	4,720,173	4,641,280	4,572,513	285.4	108,068	108,068	5.4				39,592
North Dakota	2,867,245	2,488,588	2,440,578	378.4	107,799	107,799	1.2	286,285	286,285	36.1	32,583
Ohio	7,670,815	6,993,649	6,877,800	291.9	691,682	685,682	4.7	25,850	25,850	1.9	81,483
Oklahoma	4,580,670	4,565,484	4,322,708	400.8	239,470	239,470	7.6	8,800	5,000	.1	13,492
Oregon	3,038,642	3,202,434	2,926,575	164.6	45,580	45,580		11,846	11,846	1.0	54,642
Pennsylvania	9,347,797	7,835,325	7,275,288	264.2	2,127,068	1,870,835	21.2	36,737	36,737	.4	164,937
Rhode Island	989,208	1,113,068	989,136	18.8							72
South Carolina	2,702,012	2,356,294	2,150,784	226.2	551,692	498,752	23.4	9,664	9,664	1.5	42,811
South Dakota	2,976,454	2,689,680	2,685,957	482.6	270,437	270,437	22.0				20,060
Tennessee	4,192,460	3,531,595	3,480,417	135.3	679,103	679,103	15.5	35,710	22,060	2.6	10,881
Texas	11,989,350	12,576,411	11,520,967	1,105.8	518,808	406,058	14.9	23,100	23,100	4.3	39,224
Utah	2,067,154	2,152,015	1,928,492	207.7	112,055	112,055	.1				26,607
Vermont	924,306	1,059,845	908,301	23.2	11,100	11,100					4,905
Virginia	3,652,667	3,479,593	3,290,628	941.2	131,731	128,434	13.5				233,605
Washington	3,026,161	3,316,821	2,940,420	164.3	65,824	65,824	.7				19,916
West Virginia	2,231,412	2,009,315	1,913,773	86.0	375,022	317,639	14.5				
Wisconsin	4,823,884	5,229,337	4,727,232	343.4	91,678	90,200	.3				6,452
Wyoming	2,219,155	2,171,792	2,165,171	152.4	33,287	33,287					20,697
District of Columbia	949,496	950,000	949,496	8.8							
Hawaii	926,033	946,320	862,482	17.4	9,533	8,907		62,530	54,644	.6	
TOTALS	195,000,000	183,588,755	173,245,712	12,770.4	16,719,816	15,300,340	389.5	3,002,015	2,149,998	101.6	4,303,950

U. S. GOVERNMENT PRINTING OFFICE: 1938

PUBLICATIONS of the BUREAU OF PUBLIC ROADS

Any of the following publications may be purchased from the Superintendent of Documents, Government Printing Office, Washington, D. C. As his office is not connected with the Department and as the Department does not sell publications, please send no remittance to the United States Department of Agriculture.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1931. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1933. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1934. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1935. 5 cents.

Report of the Chief of the Bureau of Public Roads, 1936. 10 cents.

Report of the Chief of the Bureau of Public Roads, 1937. 10 cents.

DEPARTMENT BULLETINS

No. 1279D. .Rural Highway Mileage, Income, and Expenditures, 1921 and 1922. 15 cents.

No. 1486D. .Highway Bridge Location. 15 cents.

TECHNICAL BULLETINS

No. Highway 55T. . . Bridge Surveys. 20 cents.

No. 265T. . .Electrical Equipment on Movable Bridges. 35 cents.

MISCELLANEOUS PUBLICATIONS

No. 76MP. .The Results of Physical Tests of Road-Building Rock. 25 cents.

No. 191MP.Roadside Improvement. 10 cents.

No. 272MP.Construction of Private Driveways. 10 cents.

No. 279MP.Bibliography on Highway Lighting. 5 cents.

The Taxation of Motor Vehicles in 1932. 35 cents.

Guides to Traffic Safety. 10 cents.

Federal Legislation and Rules and Regulations Relating to Highway Construction. 15 cents.

HOUSE DOCUMENT No. 462:

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.

Part 2 . . . Skilled Investigation at the Scene of the Accident Needed to Develop Causes. 10 cents.

Part 3 . . . Inadequacy of State Motor-Vehicle Accident Reporting. 10 cents.

Part 4 . . . Official Inspection of Vehicles. 10 cents.

Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.

Part 6 . . . The Accident-Prone Driver. 10 cents.

An Economic and Statistical Analysis of Highway-Construction Expenditures. 15 cents.

Highway Bond Calculations. 10 cents.

Single copies of the following publications may be obtained from the Bureau of Public Roads upon request. They cannot be purchased from the Superintendent of Documents.

SEPARATE REPRINT FROM THE YEARBOOK

No. 1036Y. .Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio (1927).

Report of a Survey of Transportation on the State Highways of Vermont (1927).

Report of a Survey of Transportation on the State Highways of New Hampshire (1927).

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio (1928).

Report of a Survey of Transportation on the State Highways of Pennsylvania (1928).

Report of a Survey of Traffic on the Federal-Aid Highway Systems of Eleven Western States (1930).

UNIFORM VEHICLE CODE

Act I.—Uniform Motor Vehicle Administration, Registration, Certificate of Title, and Antitheft Act.

Act II.—Uniform Motor Vehicle Operators' and Chauffeurs' License Act.

Act III.—Uniform Motor Vehicle Civil Liability Act.

Act IV.—Uniform Motor Vehicle Safety Responsibility Act.

Act V.—Uniform Act Regulating Traffic on Highways.

Model Traffic Ordinances.

A complete list of the publications of the Bureau of Public Roads, classified according to subject and including the more important articles in PUBLIC ROADS, may be obtained upon request addressed to the U. S. Bureau of Public Roads, Willard Building, Washington, D. C.

CURRENT STATUS OF UNITED STATES WORKS PROGRAM GRADE CROSSING PROJECTS

(AS PROVIDED BY THE EMERGENCY RELIEF APPROPRIATION ACT OF 1935)

AS OF JANUARY 31, 1938

STATE	APPORTIONMENT	COMPLETED		NUMBER			UNDER CONSTRUCTION		NUMBER			APPROVED FOR CONSTRUCTION		NUMBER			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
		Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Works Program Funds	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 4,034,617	$ 3,546,325	$ 3,530,996	47	1	12	$ 378,519	$ 378,519	2			$ 175,300	$ 109,000	4			$ 16,102
Arizona	1,256,099	1,285,065	1,216,596	15		5	18,841	18,841									20,662
Arkansas	3,574,060	2,880,768	2,873,485	51	6	27	643,751	643,046	5		4	6,541	6,541			2	50,988
California	7,486,362	7,198,713	7,019,499	46	8		414,005	414,004	1			10,000	10,000	5			42,859
Colorado	2,631,567	2,281,830	2,212,475	27	1	18	418,484	418,484	3	2	3						608
Connecticut	1,712,684	479,557	479,463	3	1	1	1,157,071	1,135,710	7								97,511
Delaware	418,239	130,000	130,000	1			279,052	279,052	2								9,187
Florida	2,827,883	2,409,152	2,381,150	30	5	2	119,018	119,018	1			136,010	136,010			74	191,705
Georgia	4,895,949	753,574	751,468	17	6	11	1,491,539	1,491,539	23	7	6	753,110	753,110	10	2	35	1,899,832
Idaho	1,674,479	1,392,801	1,366,439	20	3	12	252,696	252,696	2		6	4,261	4,261			3	51,083
Illinois	10,307,184	8,792,858	8,668,197	65	8		1,500,134	1,493,730	9			101,000	101,000	2			44,257
Indiana	5,111,096	4,368,156	4,235,087	39	13		857,447	857,447	3		162						18,563
Iowa	5,600,679	4,583,906	4,490,835	98	9	8	1,092,875	1,089,000	8								20,844
Kansas	5,246,258	3,946,372	3,923,996	54		5	1,273,119	1,253,704	4	1		115,024	22,020	2		3	46,538
Kentucky	3,672,387	1,358,467	1,339,494	19	5		1,984,219	1,694,487	5	6		612,579	612,579	3	1		25,827
Louisiana	3,213,467	1,466,944	1,466,936	15	1		982,285	982,265	10	1		649,599	600,110	4	1		164,155
Maine	1,426,861	1,266,779	1,263,012	19	1	4	160,788	142,540	1	1							21,309
Maryland	2,061,751	665,089	660,481	5	3	18	878,674	828,674	5	2	10	279,561	257,093	1	1	2	315,503
Massachusetts	4,210,833	2,951,633	2,949,199	22	4		841,150	841,150	4			249,991	249,991	1			170,493
Michigan	6,765,197	6,861,708	6,563,144	44	8		101,477	101,477				167,000	75,730	1	1		24,846
Minnesota	5,395,441	4,753,430	4,645,372	82	13	50	676,480	670,430	4								79,639
Mississippi	3,241,475	2,326,477	2,321,323	51	5	10	448,900	448,900	6	2	3	27,700	27,700			10	443,552
Missouri	6,142,153	4,265,566	4,094,182	39	1		2,029,178	2,029,178	10		1	1,650	1,650			1	17,143
Montana	2,722,327	2,556,515	2,513,626	37	7		245,576	185,931	1	1							22,770
Nebraska	3,556,441	2,893,042	2,849,115	76	3	21	470,813	470,813	5		1	199,683	199,683	3		2	36,831
Nevada	887,260	877,030	845,455	8	3		13,308	13,308				3,630	3,630			5	24,867
New Hampshire	822,484	791,275	791,208	9	4	6	53,297	29,170	1			2,106	2,106			1	
New Jersey	3,983,826	3,051,471	3,038,291	20	5	1	831,570	831,570	4	4		44,730	44,730		1		69,235
New Mexico	1,725,286	1,700,885	1,693,330	19	1		25,879	25,879									6,076
New York	13,577,189	11,566,131	11,236,557	35	45		2,016,770	2,016,770	10	4		94,000	94,000				229,862
North Carolina	4,823,958	3,637,396	3,617,250	50	18		1,081,059	1,081,059	12		115						125,649
North Dakota	3,207,473	2,817,479	2,813,235	56	4		389,012	389,012		1							5,226
Ohio	8,439,897	2,496,894	2,364,902	19	6	5	5,668,434	5,200,288	30	2	6	674,350	659,350	10		3	215,357
Oklahoma	5,004,711	3,692,565	3,677,928	58	9	16	1,358,994	1,225,894	7	3	15	31,050	31,050			10	69,839
Oregon	2,334,204	2,329,684	2,240,979	16	6	2	74,215	74,215	1								19,010
Pennsylvania	11,483,613	8,099,993	7,587,065	68	18	9	3,825,861	3,458,873	18	5		150,473	150,473	5			287,202
Rhode Island	699,691	702,023	694,241	4	3												5,450
South Carolina	3,059,956	1,753,597	1,726,238	37	11	26	859,783	859,783	9	5	10	142,578	142,578	1		12	331,358
South Dakota	3,249,086	2,857,287	2,855,689	59	6	53	175,186	175,186	6			269,350	210,393	17			7,818
Tennessee	3,903,979	1,815,998	1,807,317	30	3	30	1,746,370	1,746,370	15	1	4	114,020	114,020	1		6	236,272
Texas	10,855,982	9,840,502	9,827,676	126	14	97	565,611	565,611	2	1	16	104,519	104,519			36	358,175
Utah	1,230,763	1,203,470	1,196,574	17	1	1	18,461	18,461									15,728
Vermont	729,857	752,107	706,907	10	7	20	10,900	10,900									12,050
Virginia	3,774,287	2,807,710	2,671,298	43	18	21	1,065,684	1,059,059	9	3	1	6,750	6,750			2	37,180
Washington	3,095,041	2,713,500	2,690,310	22	11	11	368,494	368,069	1	1							36,662
West Virginia	2,677,937	1,218,521	1,216,553	11	2	5	1,379,082	1,378,581	15	1	1	78,855	78,855	2		1	3,948
Wisconsin	5,022,683	4,544,529	4,508,123	37	6	7	392,238	392,035		1	10	94,010	26,563			17	95,962
Wyoming	1,360,841	1,220,886	1,203,889	13			111,212	111,212	1								45,740
Dist. of Columbia	410,804	417,779	410,804	3													
Hawaii	453,703	284,891	284,005	3			179,710	169,698	2								
TOTALS	196,000,000	148,608,330	145,651,394	1695	313	514	40,927,221	39,441,638	264	55	374	5,299,430	4,835,495	72	7	225	6,071,473