LONG-TIME MEASUREMENT OF LOADS ON THREE PIPE CULVERTS

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The primary purpose of this paper is to describe and present the results of an experiment in which the loads on three 44-in. OD pipe culverts were measured during a 21-year period from 1927 to 1948. The 3 culverts of concrete, cast iron, and corrugated steel were under a 15-ft embankment. The measured loads are compared with loads calculated by the Marston theory of loads on underground conduits. In addition, settlement measurements of the soil at several horizontal planes in the embankment were made for the purpose of verifying the concept of a "plane of equal settlement," which was discovered by Marston on the basis of pure mathematical reasoning and which plays such an important role in the theory. The measured settlements were also used to determine actual values of the settlement ratio for use in calculating loads on the pipelines. A review of the theory is presented, particularly the mathematical determination of the height of the plane of equal settlement above the top of the conduit. The comparison between measured and calculated loads indicates the general correctness and reliability of the Marston theory. It is also shown that there is a substantial difference between loads on a flexible conduit and those on a rigid conduit because of the difference in values of the settlement ratios that are characteristic of those conduit types. The 21-year measurements of load indicated no substantial increase or decrease of load in that period of time.

•THE FIRST step in the structural design of a culvert under an embankment, after hydraulic and geometrical requirements have been met, is to estimate the probable load to which it will be subjected during its functional life. The most widely used tool for this purpose is the Marston theory of loads on underground conduits. During the development phase of the theory, Marston stated (1, 2), "The only possible real test of the reliability of the new theory of loads on culverts from embankment materials is comparison of the theoretical loads with the loads actually weighed in carefully conducted experiments with culverts." That statement of philosophy was implemented by a number of experiments in which the earth loads on full-scale culvert pipes under actual embankments were measured. Those experiments involved embankments as high as 20 ft and covered relatively short periods of time, generally less than 1 year in duration (1, 3).

To discover what happens to the load on a culvert during a period of time, the Iowa Engineering Experiment Station (now the Iowa Engineering Research Institute) in cooperation with the U.S. Bureau of Public Roads (now the Federal Highway Administration) began an experiment in 1927 in which loads caused by a 15-ft high embankment on three 44-in. OD pipe culverts were measured for a 21-year period. The culverts were of concrete, cast iron, and corrugated metal. The primary purpose of this paper is to present the load measurements and to compare them with loads calculated by the Marston theory. Although this experiment was completed in 1948, the results have never been published, except for brief allusions in other reports (4, 5).

A key discovery in developing the theory was the existence of a plane of equal settlement, which is a horizontal plane in the embankment at and above which the settlement of the prism of soil over the structure is the same as the settlement of prisms of soil at the side adjacent to the central prism. Marston discovered this concept of a plane of equal settlement solely through pure mathematical reasoning. When the theory is incorporated in the load theory, the calculated loads checked closely with measured loads, which was powerful evidence that the concept was correct. Nevertheless, it was desirable to demonstrate by physical measurements whether such a plane actually develops in an embankment. Therefore, a secondary objective of this project was to measure settlements in horizontal planes at several elevations, both over and adjacent to the conduits, to verify this fundamental concept.

LOAD THEORY

The theory provides a mathematical procedure for evaluating the vertical load on a buried conduit. The load is considered to be the resultant of 2 components: (a) the deadweight of the prism of soil that lies directly above the structure and (b) the summation of certain shear or friction forces that are generated by relative movements or tendency for movements along vertical planes rising from the sides of the culvert between the top of the structure and the plane of equal settlement. Those shear forces may be directed upward or downward, depending on the direction of relative movement. The resulting load on the structure may be greater, equal to, or less than the deadweight of the overlying prism of soil.

The Marston theory may be thought of as a means of evaluating arch action in the soil above a culvert. The resultant forces associated with arch action are diagonally oriented and have vertical and horizontal components. The theory deals directly with those components and not with the resultant forces themselves. As shown in Figure 1, the arch action is a bridging action in which the vertical components of the resisting forces are directed upward along the sides of the central prism of soil in the case of ditch conduits and the ditch condition of projecting conduits. In the projection condition, the arch action is inverted and the vertical components act downward.

The load equation is derived by equating the upward and downward forces on a differential horizontal slice of the prism of soil over the conduit, as shown in Figure 2. It is necessary to distinguish between the projection condition, where the shear forces acting on the central prism are directed downward (inverted arch action), and the ditch condition, where the shear forces are directed upward (bridging action) as shown in Figure 1.

The notation for this derivation is

W_c = total load on conduit due to fill materials, lb/unit length;
V = total vertical pressure on any horizontal plane in prism of material directly over conduit, lb/unit length;
B_c = greatest horizontal width of conduit;
p = projection ratio, a ratio of distance that top of conduit projects above subgrade to width of conduit;
pB_c = conduit projection;
H = height of fill above top of conduit;
H_e = vertical distance from top of conduit to plane of equal settlement;
h = distance from top of fill (complete conditions) or plane of equal settlement (incomplete conditions) down to any horizontal plane;
r_{sd} = settlement ratio, relative settlement of top of conduit to that of critical plane, which is horizontal plane through top of conduit at time earth fill is level with top, i.e., when

$$H = 0 = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \tag{1}$$

s_m = compression strain of columns of soil of height pB_c;
s_g = settlement of natural ground or subgrade surface;
s_f = settlement of conduit foundation;
d_c = shortening of vertical dimension of conduit;
C_c = load coefficient for projecting conduits;

w = unit weight of fill materials;
K = ratio of active horizontal pressure to vertical pressure by Rankine's formula

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} \tag{2}$$

μ = coefficient of internal friction ($\tan \varphi$) of fill materials; and
e = base of natural logarithms.

Then we may write

$$V + dV = V + wB_c dh \pm 2K\mu \frac{V}{B_c} dh \; , \tag{3}$$

There are 2 cases to consider in the solution of Eq. 3. The first is the complete condition where the plane of equal settlement is imaginary and lies at or above the top of the embankment. In this case the shear forces extend all the way to the top of the embankment (hence the term "complete"). The boundary conditions for this case are $V = 0$ when $h = 0$ and, at the top of the conduit, $V = W_c$ when $h = H$. Then,

$$W_c = C_c w B_c^2 \tag{4}$$

in which

$$C_c = \frac{e^{\pm 2K\mu \frac{H}{B_c}} - 1}{\pm 2K\mu} \tag{5}$$

Upper signs are used for a complete-projection condition, and lower signs are used for a complete-ditch condition.

For the incomplete conditions (Figs. 1 and 2) where the plane of equal settlement is below the top of the embankment, the shear forces are effective only through the distance H_e. The boundary conditions for the solution of Eq. 3 are $V = (H - H_e)wB_c$ when $h = 0$ and $V = W_c$ when $h = H_e$. Then the coefficient C_c in Eq. 4 becomes

$$C_c = \frac{e^{\pm 2K\mu \frac{H_e}{B_c}} - 1}{\pm 2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2K\mu \frac{H_e}{B_c}} \tag{6}$$

Upper signs are used for an incomplete-projection condition, and lower signs are used for an incomplete-ditch condition.

The height of the plane of equal settlement, H_e, is a function of the product of the settlement ratio and the projection ratio, $r_{sd}p$. When the settlement ratio is positive, the incomplete-projection condition prevails and the shearing forces are directed downward, as shown in Figure 1. A negative value indicates the incomplete-ditch condition, and the shear forces are directed upward. A settlement ratio of 0 indicates that the critical plane and the top of the conduit settle equally. In this transition case, the plane of equal settlement coincides with the critical plane, there are no shear forces generated, and the shear force component of load is 0. Therefore, the load on the conduit is equal to the weight of the prism of soil directly over the structure.

An expression for evaluating H_e is derived by equating an expression for the settlement at H_e of the prism of fill material over the conduit to the settlement at the same height of the prisms of material adjacent to the conduit. In Marston's original development (1), the expressions for settlements of the interior and exterior prisms of soil caused only by the weight of soil above the plane of equal settlement, $(H - H_e)wB_c$, were equated. He referred to this as the "plane of equal additional settlement" (2, 6). Later the author developed an expression for H_e by equating settlements of the central prism of soil and of the adjacent soil prisms caused by the total height of fill, H. This has been called the "plane of equal total settlements" (6). The difference between these 2 approaches is purely academic. The calculated loads by both methods are sufficiently close that, on the basis of available experimental evidence, it is impossible to say that one is more nearly correct than the other. Superficially, the principal difference is that the "equal additional" method yields a value of H_e that is constant for all heights of fill, and the $r_{sd}p$ ray lines in the C_c diagram are tangent to the complete condition envelope curves (7, Fig. 8, p. 327). In the "equal total" method, the value of H_e decreases as H increases and the ray lines depart from the envelopes at the angle shown in Figure 3. This results in a somewhat lesser load at higher values of H in the incomplete-projection condition and a somewhat greater load in the incomplete-ditch condition.

In the equal total procedure (as well as in the derivation of Eq. 4), it is assumed that the shear force increment or decrement transferred to the central prism of soil is uniformly distributed over the width of the prism, B_c. Also it is assumed that the shear force decrements or increments are transferred to the adjacent soil prisms in such a manner that the effect on settlement of those columns is the same as if they were uniformly distributed over a width equal to jB_c. No direct physical evidence of the value of j is available. However, calculated values of load using $j = 1$ agree closely with measured loads.

Referring to Figure 3, we may write

$$\lambda + s_f + d_c = \lambda' + s_m + s_g \tag{7}$$

in which

λ = compression strain at H_e of the prism of soil ABCD, and
λ' = compression strain at H_e of the prisms of soil DCHG.

Substituting Eq. 1 in Eq. 7, we obtain

$$\lambda = \lambda' + r_{sd}s_m \tag{8}$$

Again referring to Figure 3 and assuming $j = 1$, we may write

$$d\lambda = \frac{V}{B_cE}\,dh \tag{9}$$

$$d\lambda' = \frac{V'}{B_cE}\,dh \tag{10}$$

$$s_m = \frac{\left(wHB_c - \frac{F}{2}\right)pB_c}{B_cE} \tag{11}$$

in which E = modulus of compression of the soil prisms, and

$$F = W_c - wHB_c \tag{12}$$

The expression in Eq. 11 for s_m neglects any friction that may exist along the vertical plane DE in the height pB_c. However, because p is always numerically small, rarely being greater than 1.0, this assumption does not materially affect results. It is employed as a simplifying procedure.

Evaluating Eqs. 9, 10, and 11 and substituting in Eq. 8 give

$$\left[\frac{1}{2K\mu} \pm \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) \pm \frac{r_{sd}p}{3}\right]\frac{e^{\pm 2K\mu\frac{H_e}{B_c}} - 1}{\pm 2K\mu} \pm \tfrac{1}{2}\left(\frac{H_e}{B_c}\right)^2$$
$$\pm \frac{r_{sd}p}{3}\left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)e^{\pm 2K\mu\frac{H_e}{B_c}} - \frac{1}{2K\mu}\cdot\frac{H_e}{B_c} \pm \frac{H}{B_c}\cdot\frac{H_e}{B_c} = \pm r_{sd}p\,\frac{H}{B_c} \tag{13}$$

Figure 1. Arch action over underground conduits.

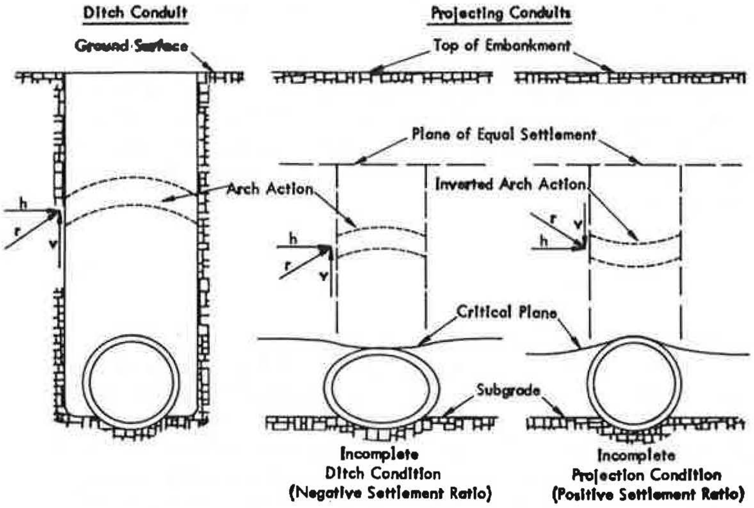


Figure 2. Projecting conduit in incomplete-projection condition—equal additional method.

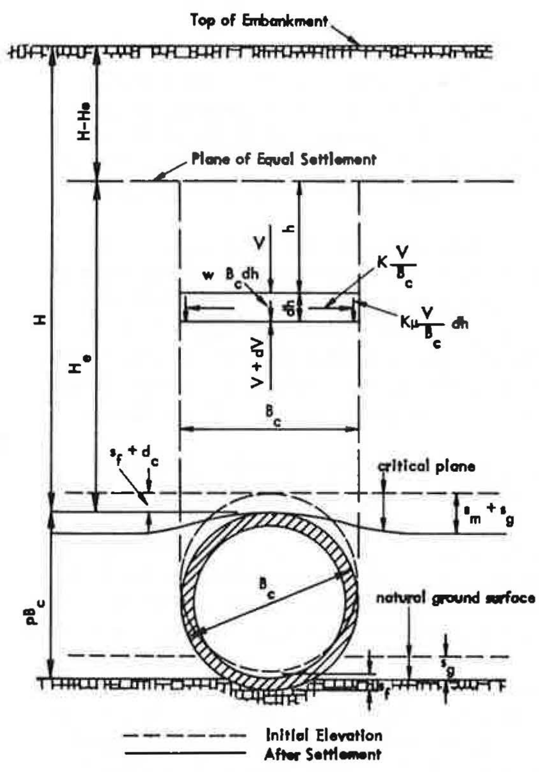
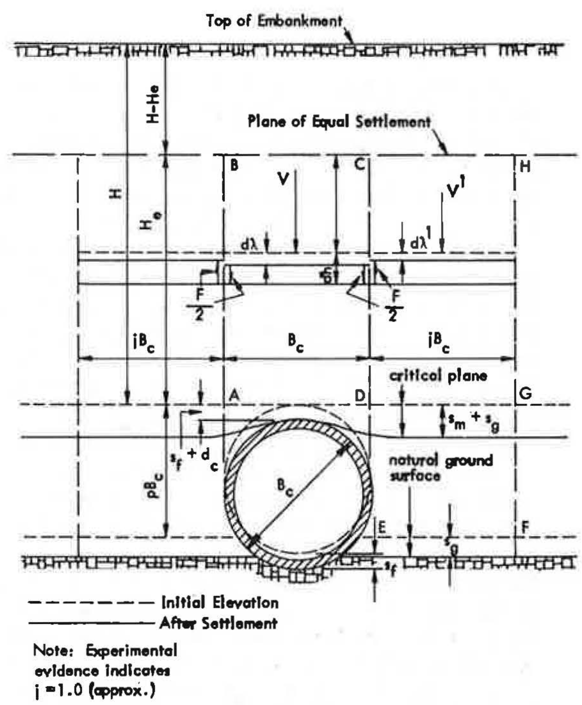


Figure 3. Projecting conduit in incomplete-projection condition—equal total method.



The upper signs are used for the incomplete-projection condition (positive settlement ratio), and the lower signs are used for the incomplete-ditch condition (negative settlement ratio).

Equation 13 is formidable, though it can be programmed. However, its solution for practical problems is made easy by the diagram shown in Figure 4. The envelope curves are plots of Eq. 5, and the ray lines are based on Eqs. 6 and 13. The points of departure of the ray lines from the envelope curves are values at which $H_e = H$. Because the ray lines for various values of the product $r_{sd}p$ are straight lines, they can be extrapolated by an equation of the form

$$C_c = A \frac{H}{B_c} + X \tag{14}$$

Values of the constants A and X have been published (8) and are given in Table 1.

The load equations (Eqs. 5, 6, and 13) are functions of $K\mu$, which is dependent on the coefficient of internal friction of the embankment soil. Theoretically, therefore, it would appear to be necessary to measure this property of the soil in order to calculate the load. In practice, however, it is believed that this refinement is not justified except possibly in research. The coefficient of friction may vary over a wide range for different soils, but the product $K\mu$ varies over a much narrower range—from about 0.13 to 0.19 as shown in Figure 5 (9). Therefore, for design purposes it is recommended that $K\mu = 0.19$ be used for the projection conditions and $K\mu = 0.13$ be used for the ditch conditions. Those values give maximum loads for the respective conditions and are the values used in construction of Figure 4 and Table 1.

WEIGHED LOADS

In the experimental phase of this project, loads produced by a 15-ft earth embankment on three 44-in. OD pipe culverts were measured. The culverts were installed parallel and spaced 25 ft center-to-center. Each culvert consisted of 4 independent sections 4 ft long, on which loads were measured, plus a 6-ft long extender section at each end under the side slopes of the embankment. Figure 6 shows the culverts as installed and before construction of the fill.

All 12 of the 4-ft sections were supported on a creosoted-timber platform, which was supported on a system of weighing levers so that the reaction from the load on the platform was transmitted to a scale at the end of the culvert. Each timber platform was equal in size to the horizontal projection of the pipe section that it supported, i.e., 44 in. wide by 48 in. long. Thus, all of the vertical load to which the pipe section was subjected was transmitted to the platform and then to the scale. Because the mechanical advantage of the lever system was known, the scale reading was readily converted into the load on the pipe.

Each platform was fitted with steel plate and angle sideboards, which retained a sand fill in which the pipe section was bedded. The tops of the sideboards and of the sand fill were mounted at a level even with the adjacent natural ground surface. The pipes were placed in a 4.4-in. deep bedding in the sand. Thus, the projection ratio of the pipes was 0.9 as shown in Figure 7. The minimum depth of sand between the bottom of the pipe and the timber platform was 6 in. Two steel flats measuring 1½ in. by ¼ in. by 20 ft were bolted to the fixed extender platforms and fastened loosely to the center of each weighing platform to inhibit end play.

Each platform was supported on its lever system at 3 points, as shown in Figure 8 by the letter S. The lever systems were made of structural steel I-beams. The loads and reactions were transmitted to the beams through hardened tool-steel knife edges, which bore on cast-iron fittings bolted to the beams. Those knife edges and fittings were designed so that loads and reactions were applied in the horizontal axes of the beams and symmetrically about the vertical axes.

The platform and lever assemblies were calibrated prior to installation of the experimental pipes by means of a hydraulic jack and spring-bearing arrangement as shown in Figure 9. The jack reaction was carried by a transverse beam anchored to

Figure 4. Calculation diagram.

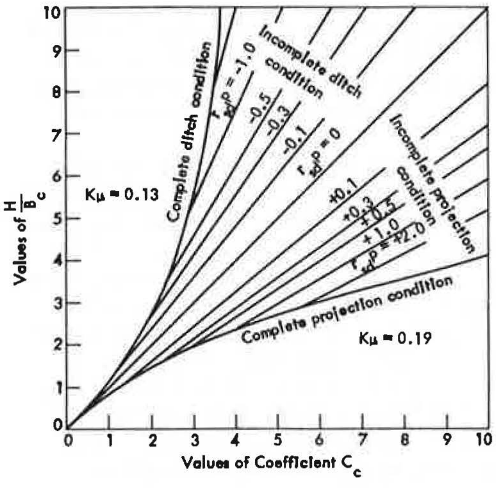


Table 1. Values of constants A and X for extrapolating values of C_c versus H/B_c.

Incomplete Ditch Condition ($K\mu = 0.13$)			Incomplete Projection Condition ($K\mu = 0.19$)		
$r_{sd}p$	A	X	$r_{sd}p$	A	X
0	1.00	0	+0.1	1.23	-0.02
-0.1	0.82	+0.05	+0.3	1.39	-0.05
-0.3	0.69	+0.11	+0.5	1.50	-0.07
-0.5	0.61	+0.20	+0.7	1.59	-0.09
-0.7	0.55	+0.25	+1.0	1.69	-0.12
-1.0	0.47	+0.40	+2.0	1.93	-0.17
-2.0	0.30	+0.91	+3.0	2.08	-0.20

Figure 5. Relation of μ, K, and $K\mu$.

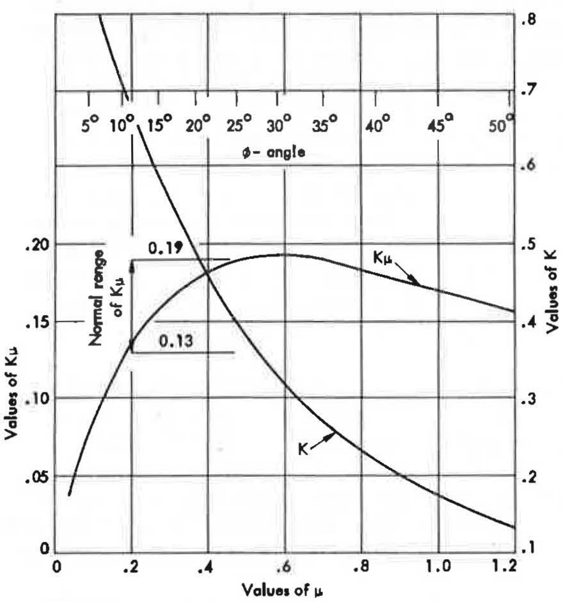
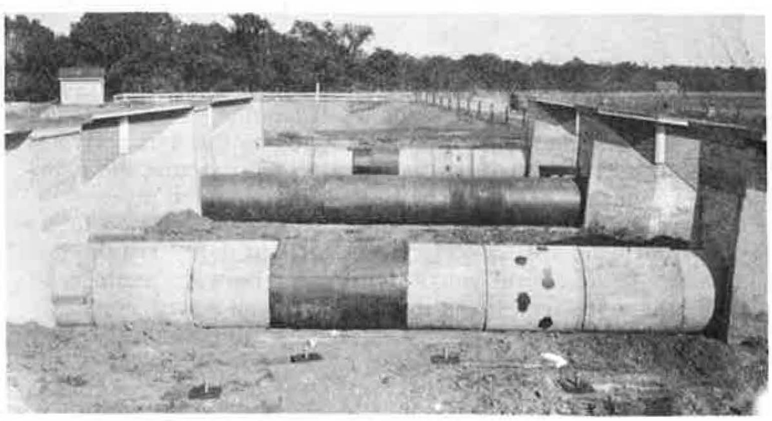


Figure 6. Experimental culverts before fill construction.



2 heavily loaded trucks. Loads were applied to the platforms in increments of 5,000 lb to a maximum of 25,000 lb. The design lever ratio of the weighing systems was 30 to 1; that is, 30 lb on the pipe and platform produced 1 lb on the scale. The average calibrated ratio was 30.4 to 1, as given in Table 2, which exceeded the design ratio by slightly more than 1 percent.

The embankment material was a sandy loam top soil and had considerable gravel and some light clay intermixed. It was composed of the strippings from several gravel pits, which had been stripped from the original position for several years and had been moved and removed 2 or 3 times, so that it was somewhat weathered and worn. The embankment was constructed by teams and wheeled scrapers and was not formally compacted except by the team and scraper traffic. The unit weight of the soil was determined by sinking 2 vertical shafts, 3 by 3.5 ft in cross section, down through the entire 15 ft of fill and by weighing all material removed. The measured unit weight was approximately 120 lb/ft^3. The friction coefficient of the fill material was determined by measuring the force required to pull a bottomless box filled with the soil over a flat surface of the same material. A large number of tests were made, 159 in all. The value of μ ranged from 0.53 to 0.81, the average was 0.69, which yields a value of $K\mu = 0.19$ (Fig. 5).

The measurements of earth loads by means of lever systems and platform scales have been criticized on the basis that vertical movement of the scale platforms might have caused unacceptable movements of the pipe specimens during weighing operations (10). That possibility was studied thoroughly during the experiments. One significant test directed toward this question was to balance the scale beam by moving the rider to the pan end of the beam, then moving it back toward the fulcrum end to a balanced position (11, 12). That tended to raise the balance beam and to lower the scale platform, levers, support platforms, and pipes. If there had been any adverse effect on load measurement, the indicated load would have been less than the actual load as a result of that operation. Next, the rider was moved to the fulcrum end of the balance beam and slowly moved forward to a balanced position. That tended to raise the scale platform and pipe against the soil, and, if there had been any effect on loads, the indicated load would have been greater than the actual. However, there was no difference in the indicated load, no matter which way the rider was moved. The operation was repeated many times on all 12 of the scales, and the indicated results were always the same.

The pan end of the balance beam of the platform scale was dampened in the usual manner so that the vertical throw of the beam was about $^5/_8$ or $^5/_{16}$ in. up or down from a balanced position. The mechanical advantage ratio of the scale was 100 to 1 and that of the weighing lever system was 30.4 to 1, giving a total ratio from the pipe to the scale pan of 3,040 to 1. Therefore, the maximum movement of the test pipe up or down from its position when the rider was at a balance was approximately 0.3125 divided by 3,040 or 0.0001 in. Apparently that amount of potential movement was not sufficient to influence the indicated load on the pipe.

As a further check, the dampening cage was removed from several scales, and the balance beam was permitted to swing vertically through a distance of approximately 4 in. That permitted the pipe to move up or down as much as 0.0007 in., but the indicated load was always the same regardless of the direction of movement of the rider to the balance position.

Figure 10 shows the results of the load-measuring operations from the beginning of fill construction on September 22, 1927, to the final readings taken on October 1, 1948. The graphs indicate the average load per linear foot on the 16-ft lengths on which loads were measured. Also shown are the maximum and minimum loads on the 4-ft active pipe sections of each culvert, the loads calculated by the Marston theory, and the component of load represented by the weight of the soil prism directly above the pipes. Scale readings were taken daily in the early part of the experiment, then reduced to twice weekly after several years, until the spring of 1935. After that date, readings were taken on a hit-and-miss basis, with several years intervening between some readings. In the years prior to 1935, the measured loads fluctuated up and down, roughly between 90 and 100 percent of the maximum, in a poorly defined cyclic pattern. It is

Figure 7. Typical layout of settlement cells.

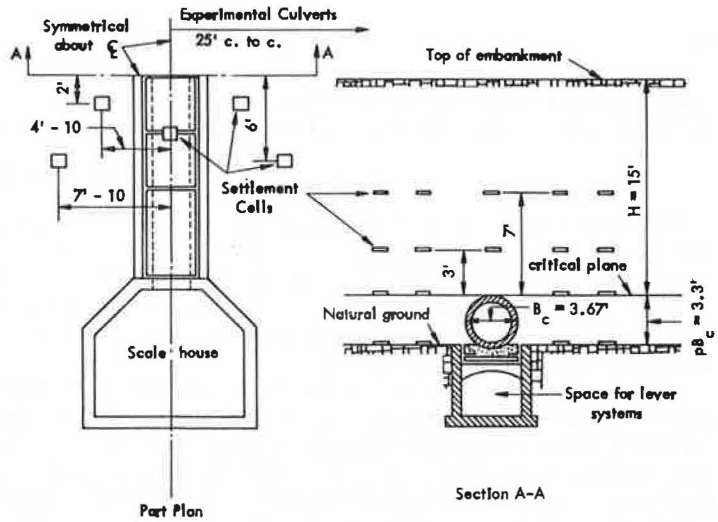


Figure 8. Lever systems.

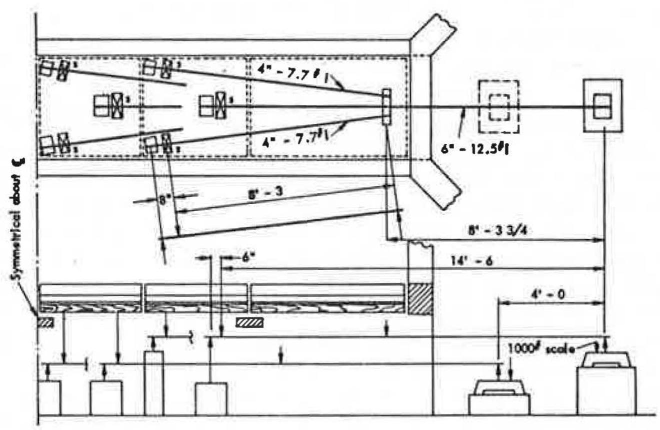


Figure 9. Weighing platforms calibrated by hydraulic jack and spring-bearing arrangement.

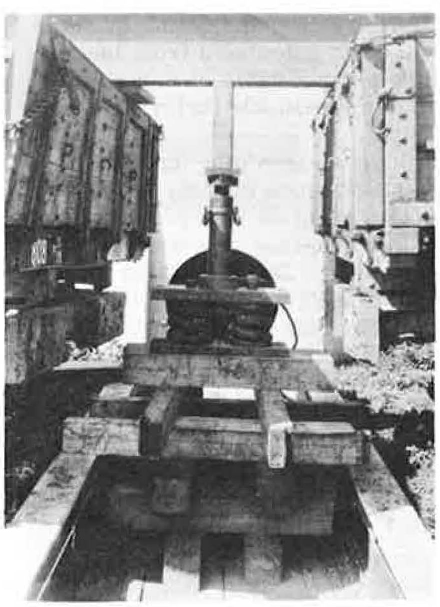


Table 2. Calibration of lever systems (mechanical advantage ratio).

Section	Concrete Culvert A	Cast-Iron Culvert B	Corrugated-Metal Culvert C
1	30.7	30.6	30.2
2	29.7	30.6	30.4
3	30.8	30.8	30.4
4	30.6	29.8	30.3
Avg	30.4	30.4	30.3

speculated that this pattern may have been due to temperature changes in the weighing systems, but no specific information is available in this regard.

SETTLEMENT MEASUREMENTS

The settlements of various horizontal planes in the embankment were measured by 108 Ames settlement cells placed on the embankment subgrade, in the critical plane level with the top of the culvert, and in the horizontal planes 3 and 7 ft above the top. In those latter 2 planes, settlement cells were placed both over the pipes and at 3 and 6 ft outside the pipes, as shown in Figure 7. Several settlement cells in place on the subgrade are visible in Figure 6.

The Ames settlement cell operates on the principle that, when water is free to act under the influence of gravity, the water level in 2 vessels connected by a tube will rise to the same elevation in each vessel. The cell consists of a 12-in.-square steel plate with a small chamber attached at the center (4, p. 313). Two water pipes extend from that chamber out through the fill. One is connected to a stationary glass gauge tube, which is attached firmly to a post or headwall, and the other pipe acts simply as an outlet at some lower elevation. When the cell is installed at a point in an embankment under construction, water is introduced into the system through the gauge tube. When the system is full, as evidenced by water spilling through the outlet pipe, water rises in the gauge tube to the level of water in the cell chamber. A zero mark is made on the gauge tube to indicate the initial elevation of the cell. As the embankment is constructed, the cell settles with the soil and the amount of settlement can be measured at the gauge tube. Those settlement cells operated satisfactorily, and there is little doubt that they correctly indicated the settlement of the soil at the specific points at which they were installed. However, it is recognized that the area of soil represented by an individual cell is very small in relation to the whole area whose settlement influences load development. That fact must be considered in appraising the precision of the settlement data.

The settlements measured by the cells on the adjacent subgrade and in the critical plane were used to estimate values of the s_m and s_g terms of the settlement ratio (Eq. 1). The s_f term was determined by level readings on the pipe inverts, and the d_c term was determined by shortening of the vertical diameters, which were measured by means of micrometer calipers. The settlement-cell measurements for the concrete pipe, the cast-iron pipe, and the corrugated-metal pipe are shown in Figures 11, 12, and 13 respectively. Figure 11 shows that, in the case of the concrete pipe, the critical plane ($s_m + s_g$) settled considerably more than the top of the pipe ($s_f + d_c$), which clearly indicates that the incomplete-projection condition prevailed. The plane 3 ft above the pipe settled more alongside than it did directly above the pipe; but at 7 ft above, the settlements over and alongside were nearly the same, indicating this was close to the plane of equal settlement. The settlement ratio calculated from the settlement measurements was approximately +1.06, which gives a value of $r_{sd}p$ = +0.95. The calculated load on the concrete pipe, using this value, is 10,900 lb/lin ft, which is near the upper limit of the measured load on this pipe.

The measured settlements of various elements of the cast-iron pipe installation are shown in Figure 12. In this case, the critical plane settled more than the top of the pipe, but the spread between those 2 elements was not so great as the spread in the concrete pipe. That is primarily due to the fact that the cast-iron pipe was somewhat less rigid than the concrete, and the value of d_c was greater. The calculated values were +0.71 for the settlement ratio and +0.64 for $r_{sd}p$. This gives a calculated load on the cast-iron pipe of 10,200 lb/lin ft, which is about 6 percent less than the calculated load on the concrete pipe and is in harmony with the fact that the measured loads on those 2 pipes were nearly the same, as shown in Figure 10.

The settlement measurements adjacent to the corrugated-metal pipe (Fig. 13) show that the top of the pipe ($s_f + d_c$) settled slightly more than the critical plane ($s_m + s_g$). That indicates a negative settlement ratio and is typical of the incomplete-ditch condition. The approximate values were -0.15 for the settlement ratio and -0.13 for $r_{sd}p$, which yield a calculated load of 5,500 lb/lin ft. That is less than the measured load

Figure 10. Time-load curves.

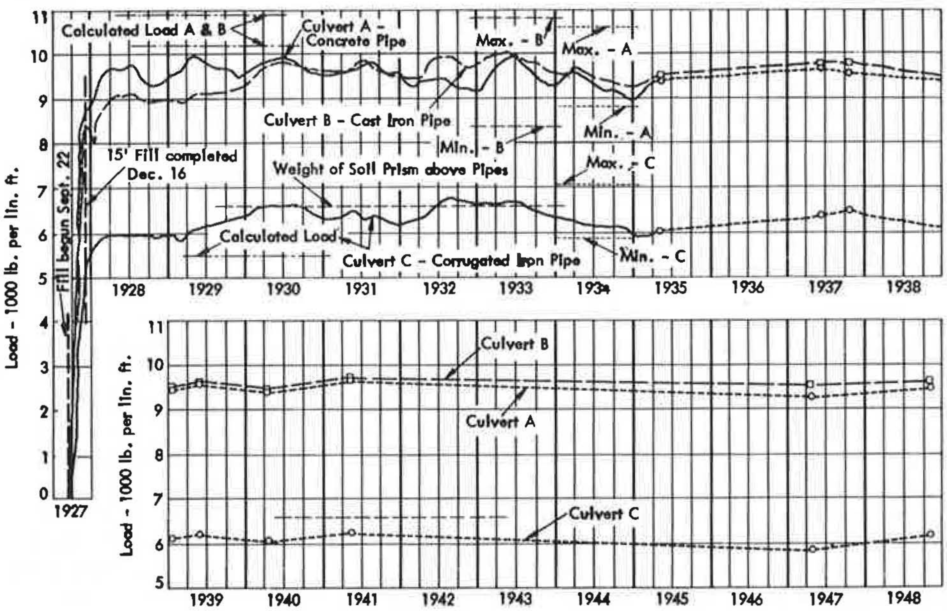


Figure 11. Settlements for concrete culvert A.

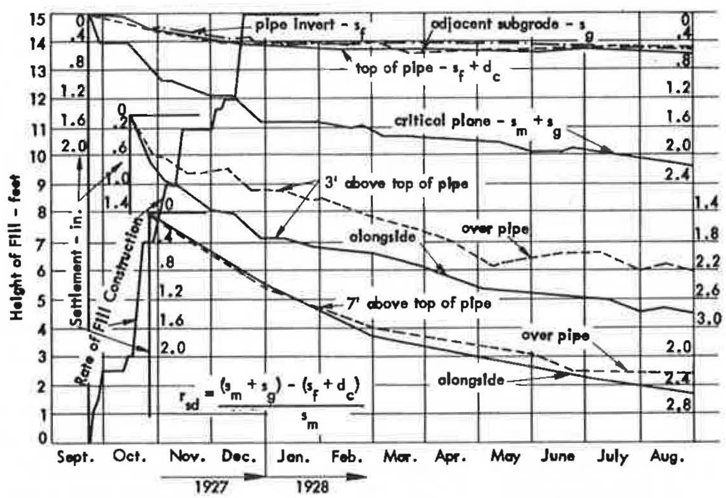
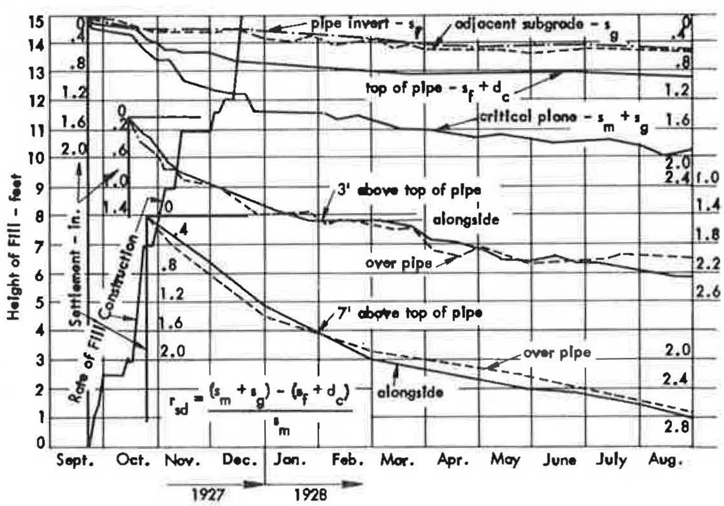


Figure 12. Settlements for cast iron culvert B.



and the weight of the prism of soil above the pipe, both of which were 6,600 lb/lin ft. However, the theoretical load is very sensitive to minor changes in the settlement ratio when values are in the neighborhood of zero (Fig. 4). That, coupled with the relatively small area of soil whosc settlement is measured by the cell, may readily account for the wider discrepancy between measured and calculated load in this case. If the actual effective settlement of the critical plane had been only 0.2 or 0.3 in. more than that indicated by the cells, the settlement ratio would have been zero and the calculated load would have been equal to the measured load.

CONCLUSIONS

The measurements of settlements and long-time measurements of loads on 3 pipe culverts confirm the essential correctness of the Marston theory of loads on underground conduits. The calculated loads and measured loads are in substantial agreement, probably as close as can be expected in this kind of work. The size of pipes, the projection ratio, the height of fill, and the soil were the same for all the pipes. The only difference among them was their rigidity or the amount of deflection under load. That difference brought about a lower value of the settlement ratio in the case of the corrugated-metal pipe, resulting in a much lower load, which is strictly in accordance with theory. The greater load on the rigid pipes compared to that on the flexible pipe can only be attributed to the fact that the side prisms of soil settled more than the central prism, thereby generating downward friction forces that were additive to the deadweight of the overlying central prism.

It is significant also to note that the spread between the load on the rigid pipes and that on the flexible pipe persisted undiminished throughout the 21 years of load measurement. Some engineers contend that, in order for the down-drag shear or friction force increments to exist, there must be finite and continuing relative movement between the interior and the exterior masses of soil (10). It is this author's belief that such shear forces develop as a result of a tendency for movement as well as actual relative movement. Surely in the 21 years covered by this experiment, all finite movement between the adjacent prisms of soil had ceased, and the persistent transfer of load by shear, as evidenced by the greater load on the more rigid pipes, can only be attributed to a tendency for relative movement.

It is of some interest to compare the values of the settlement ratios that prevailed in this experiment with those measured on 22 actual culverts under highway embankments in Iowa and Minnesota and reported in another paper (6). Of those 22 structures, 15 were reinforced concrete box culverts, 2 were reinforced concrete arches, 1 was a reinforced concrete pipe, and 4 were corrugated steel pipes. The results of the 18 rigid culverts are shown in Figure 14. The 2 rigid pipes of this experiment have been incorporated in that graph and are designated as series III.

Series I includes 7 box culverts on which loads were measured by stainless-steel friction tapes. Knowing the load, the width of the conduit, the height of fill, and the projection ratio and assuming a unit weight of soil, one can work backward through the load formula to obtain the settlement ratio. In the 11 rigid structures and 4 flexible pipes labeled series II, the elements that constitute the settlement ratio were measured by settlement cells and by leveling operations to obtain data for calculating settlement ratios.

The average settlement ratio on 18 rigid conduits (after 2 anomalous measurements were rejected) was +0.74. That confirms the author's practice, based on experience gained in investigations of culvert failures, to the effect that a settlement ratio of about +0.7 represents a satisfactory value for design purposes. Of course, if specific circumstances in individual cases can be identified that indicate a need to raise or lower this figure, such modifications should be made.

In connection with Figure 14, a very large proportion of the culverts included are flat-bottomed structures, such as arch and box culverts. Also, the 2 pipes of series III were supported on weighing platforms, which probably inhibited their downward settlement to some extent. Those circumstances may have caused the measured settlement ratios to be somewhat on the high side, as compared with rigid pipe culverts

Figure 13. Settlements for corrugated metal culvert C.

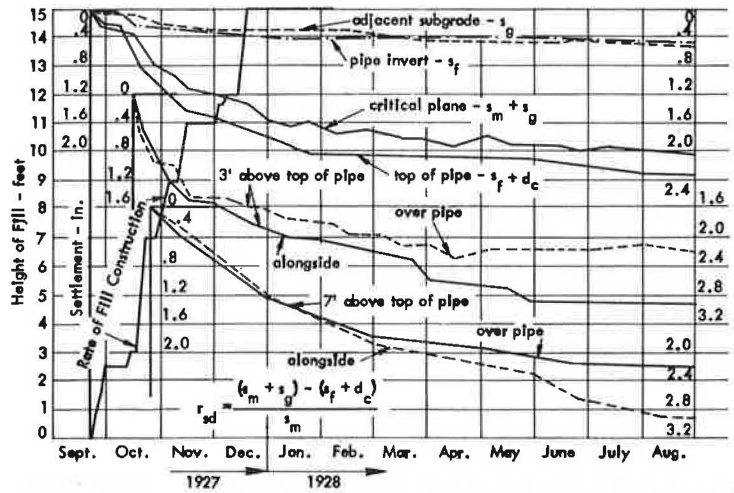


Figure 14. Measured values of settlement ratio on rigid culverts.

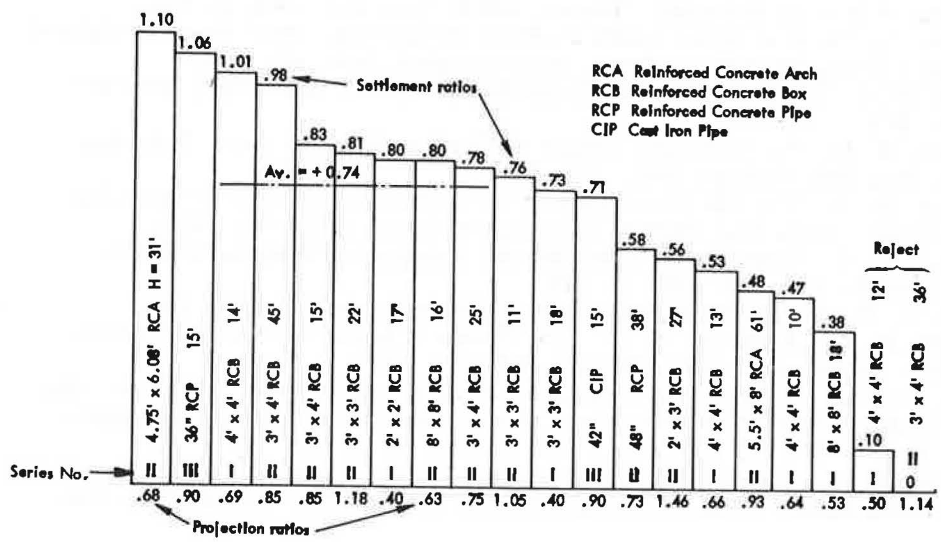
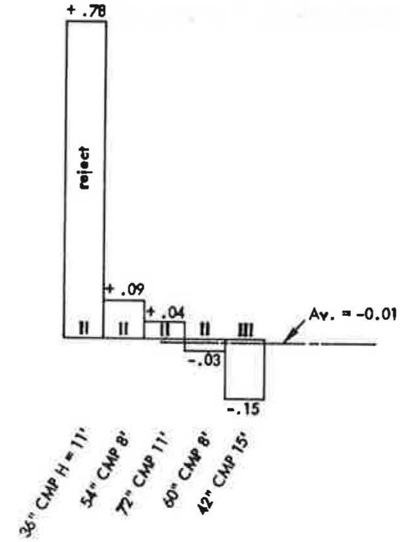


Figure 15. Measured values of settlement ratio on corrugated metal culverts.



under normal field conditions, but the extent of such influence, if any, is not determinable.

In the case of corrugated-steel pipe (Fig. 15), the number of settlement ratio measurements is pitifully small; but on the basis of 4 actual cases (after 1 anomalous result was rejected), a value of $r_{sd} = 0$ appears to be justified. In other words, the load on a corrugated pipe under an embankment is usually equal to the weight of the prism of soil above. That conclusion coincides with rather widespread practice at the present time, but more confirmatory data are needed on that subject.

ACKNOWLEDGMENTS

The work reported in this paper is supported by the Engineering Research Institute at Iowa State University.

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