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22 Highway Design
23 Highway Drainage
24 Roadside Development

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CONTENTS

FOREWORD

Transportation engineering in the past decade has given impetus to research on the structural design of pipelines and culverts, although these conduits in a wide range of sizes and shapes are the main links in other important facilities. Included in the primary uses of pipeline and culvert technology are the water supply and wastewater management systems and most water resource projects. This RECORD includes 4 structural research papers that bridge the time span between studies begun in 1927 to those that will not be completed until after issuance of this publication.

Spangler documents results of a 21-year period of observation of loads on 3 similar sized culverts of cast iron, concrete, and corrugated steel under a 15-ft embankment. Although there was a large difference in loads on rigid and flexible conduits, no significant changes in load during the test period were noted. He compares measured loads with those calculated by the Marston theory of loads on underground conduits and notes the settlement of the soil at various horizontal planes in the embankment. The measured settlements form the basis for determining the settlement ratio for use in computing loads on the pipe. He also compares results of tests in Iowa with those for highway culverts in other locations. He gives convincing evidence of the value of actual field observation of structural performance of completed pipeline structures to confirm or improve design assumptions.

Taylor reports on successful use in Illinois of the "induced" or imperfect-trench methods of installation of a concrete culvert under 30 ft of fill. The method, first advocated by Marston more than 50 years ago, has come into broader use recently, although some engineers question the explanation given for its effectiveness. The author recommends further validation by others employing this method to help evaluate the theory. Spangler, in his discussion of Taylor's paper, comments on the lower than expected loads as measured, a further evidence of the construction economies afforded to users of the imperfect-trench method as compared with conventional methods of excavation, placement of pipe, and backfill compaction.

Parmelee describes a new long-range research endeavor undertaken to investigate the nature of loading on buried concrete pipelines; both trench and embankment types of installation were used. The study involves extensively instrumented, full-scale, soil-pipe installations at the Ohio Transportation Research Center and a comprehensive finite-element computer program to simulate a wide range of field situations. The stress and strain gauge measurements in the prototype are used in the computer study for verification. Parmelee expects the computer program to indicate the relative significance of various parameters of the soil-structure interaction system. These include nature and distribution of loading around the pipe, inherent structural strength of the pipe, influence of materials, and factor of safety.

LaTona, Bealey, and Heger describe the development of a computer method for the design of precast rectangular reinforced concrete box culverts of various sizes ranging to 12-ft span, 12-ft rise, and 100-ft depths of cover. The objective is to reduce labor costs attendant to conventional cast-in-place box culvert placement and to speed construction, a need cited by the Virginia Department of Highways.

Although this RECORD is not a design manual, it provides timely insights into a wide spectrum of structural design and installation methods for pipeline and culvert placement situations. The project designer, constructor, and operator will note that the state of the art has advanced, and they can have a high degree of confidence in safe, efficient, and practical conduits for transportation use.

—Kenneth S. Eff

LONG-TIME MEASUREMENT OF LOADS ON THREE PIPE CULVERTS

Merlin G. Spangler, Engineering Research Institute, Iowa State University

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 corrcct 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.

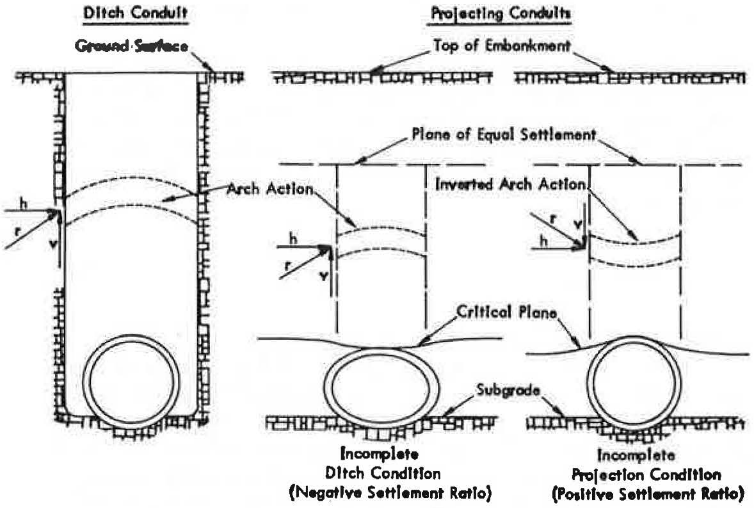


Figure 2. Projecting conduit in incomplete-projection condition—equal additional method.

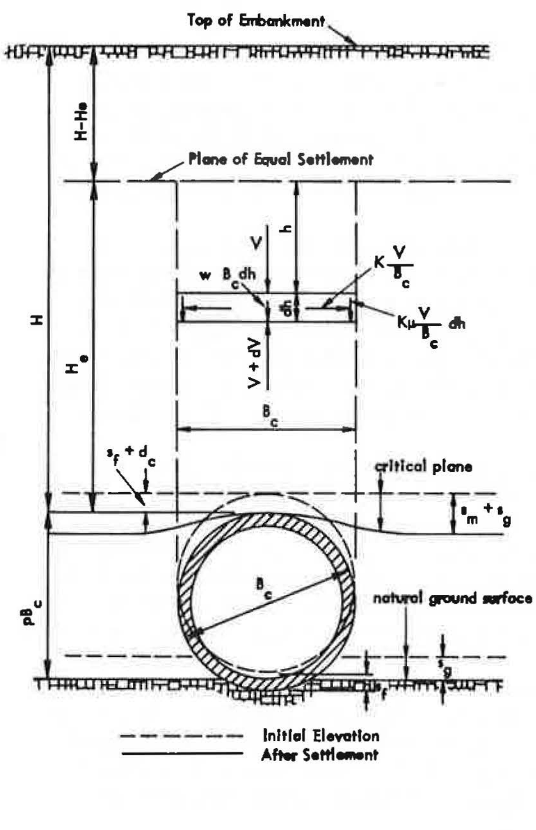
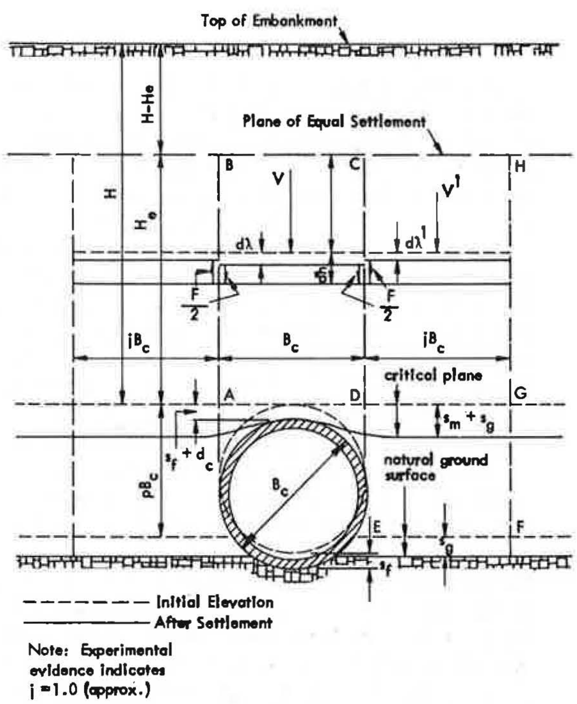


Figure 3. Projecting conduit in incomplete-projection condition—equal total method.



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.

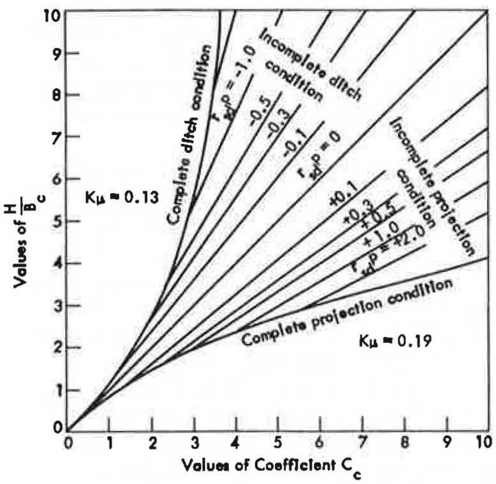


Table 1. Values of constants A and X for extrapolating values of C_c versus H/B_c.

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 $K\mu$.

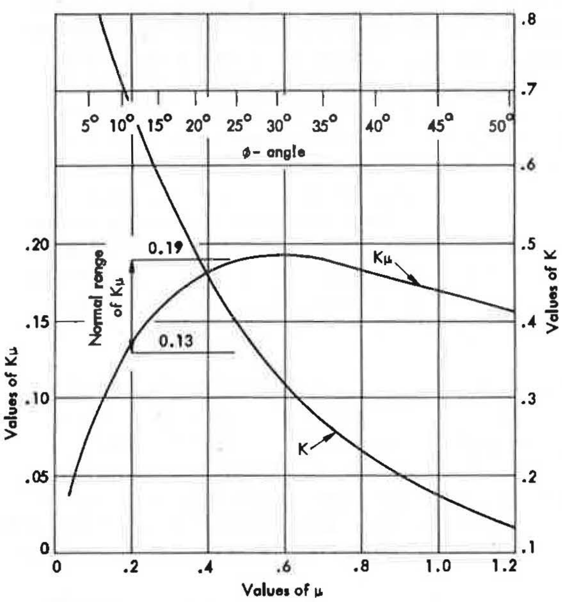
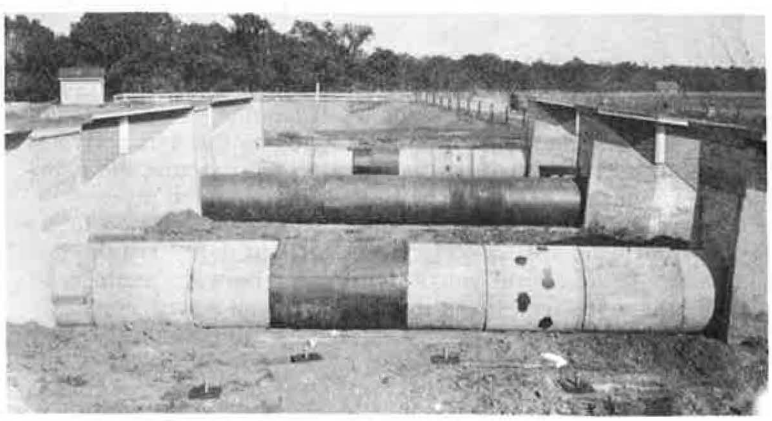


Figure 6. Experimental culverts before fill construction.



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.

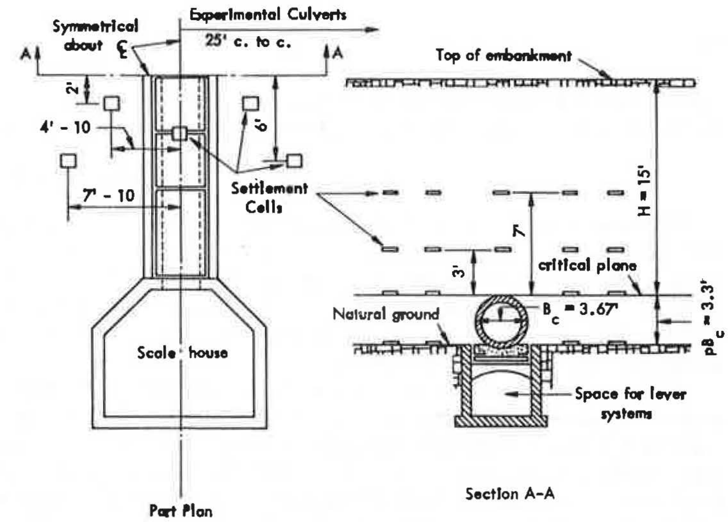


Figure 8. Lever systems.

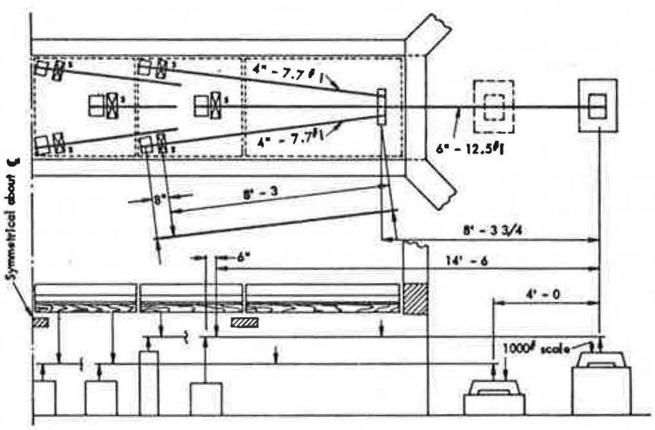


Figure 9. Weighing platforms calibrated by hydraulic jack and spring-bearing arrangement.

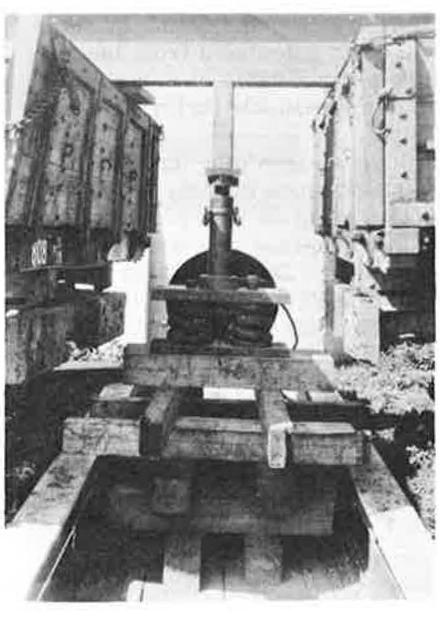


Table 2. Calibration of lever systems (mechanical advantage ratio).

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_s 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_s + 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_s + 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.

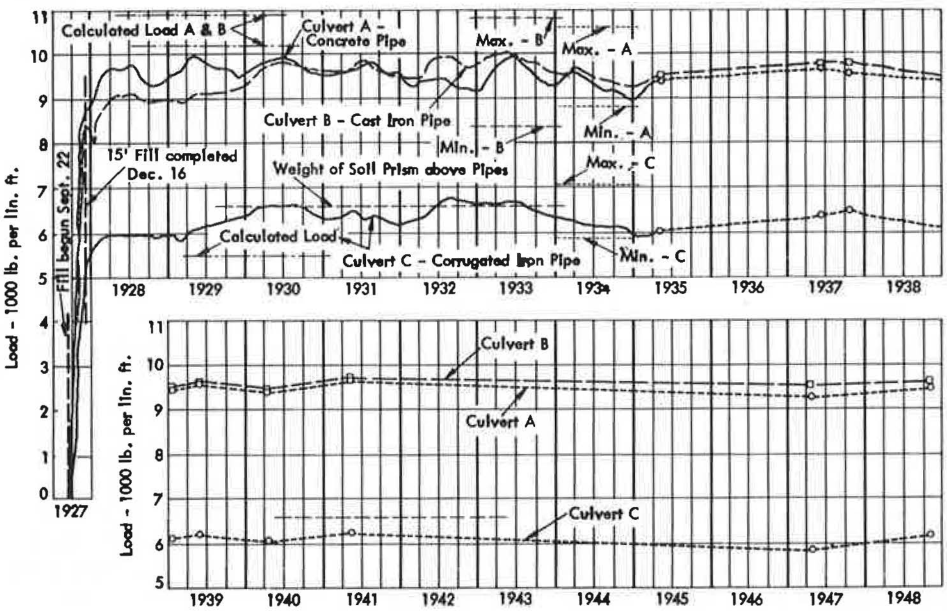


Figure 11. Settlements for concrete culvert A.

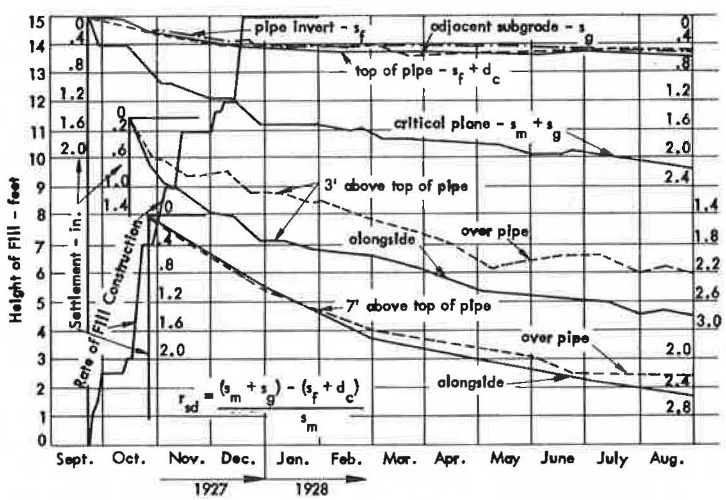
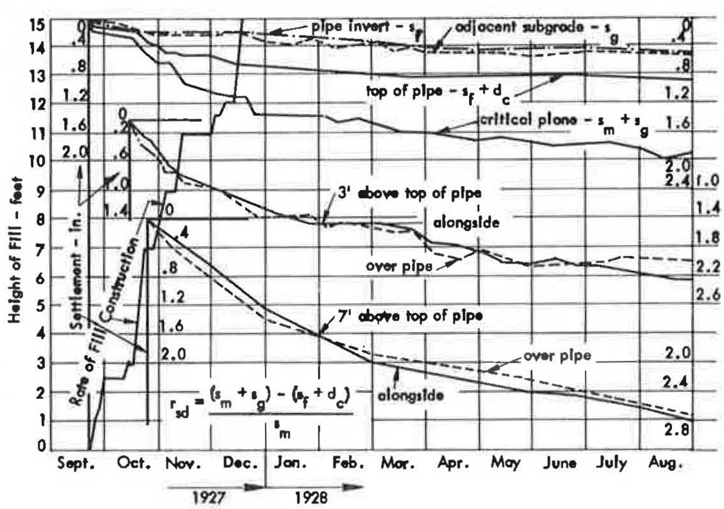


Figure 12. Settlements for cast iron culvert B.



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 whose 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.

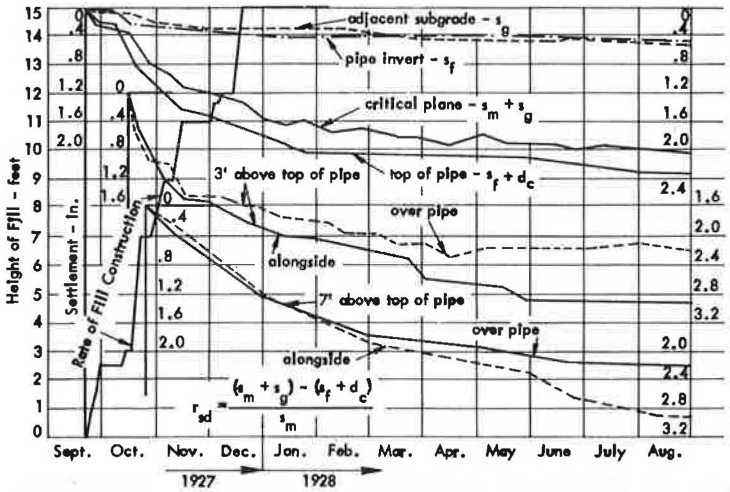


Figure 14. Measured values of settlement ratio on rigid culverts.

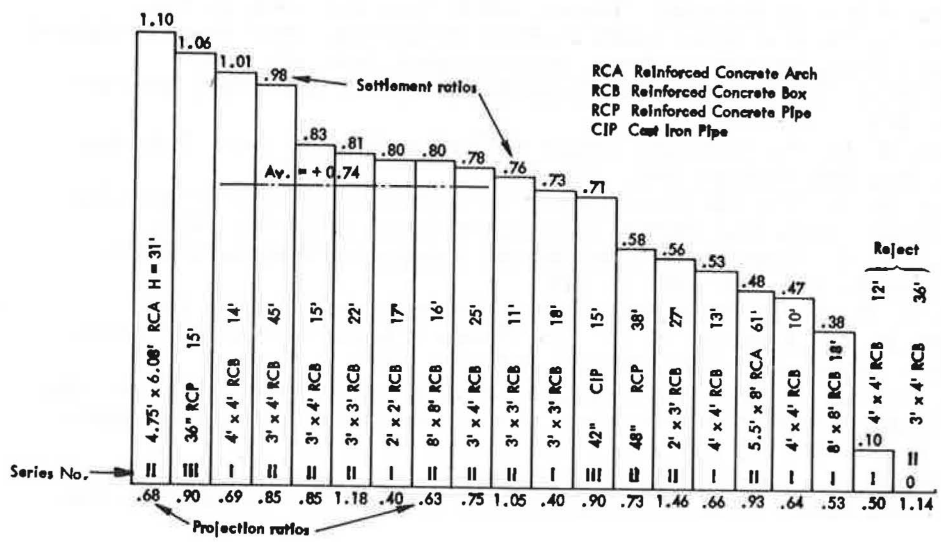
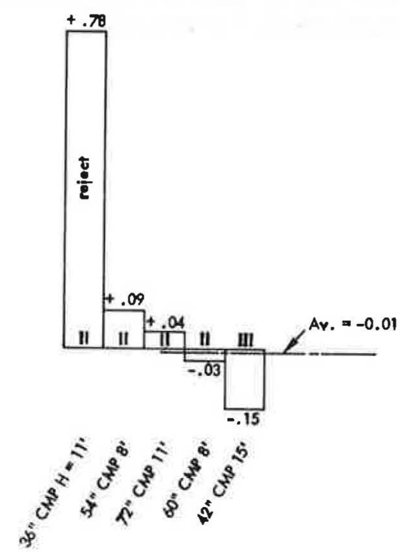


Figure 15. Measured values of settlement ratio on corrugated metal culverts.



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

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INDUCED-TRENCH METHOD OF CULVERT INSTALLATION

R. K. Taylor, Illinois Department of Transportation

The induced-trench (imperfect-trench) method of culvert installation is used to reduce the loads on a culvert under a high fill. Although the method has been used successfully with concrete pipe under some unusually high fills, the magnitude of the reduction in load achieved by the induced trench has not been clearly established. This research project was initiated to evaluate the settlement ratio and to compare the measured loads acting on the culvert with theoretical values. The results of this research indicate that the range of empirical values that have been recommended for the settlement ratio for the induced trench is reasonable for a 48-in. reinforced concrete-pipe culvert under 30 ft of fill. A comparison of the measured loads acting on the culvert with theoretical loads indicates that the load theory is somewhat conservative.

•THE CONSTRUCTION of underground drainage structures in accordance with high safety standards such as those developed for the Interstate Highway System has led to increased costs for culvert installations. The relatively flat highway profiles result in high earth fills, which require longer culverts capable of supporting heavier overburden loads. Ways are continually being sought to reduce the cost of the culverts while adequate structural performance is maintained.

One proposed method of reducing culvert costs is the induced-trench procedure, also known as the imperfect-trench method. Although the induced trench has been successfully used with concrete pipe under some unusually high fills, opportunities to evaluate the settlement ratio under field conditions have been limited. Because current knowledge of the settlement ratio is based on limited experimental proof, an evaluation of the ratio from a number of field installations would greatly help to establish design criteria for the induced-trench method of construction.

The primary objective of this research was to determine the settlement ratio used in estimating the loads on conduits installed by the induced-trench method of construction. From settlement data collected during this study, realistic values of settlement ratios were determined for the induced trench constructed under the specific conditions present at the test site. Those data, in addition to other information from a number of similar installations with varying fill heights and different culvert sizes, will eventually provide the means for more accurately predicting the settlement ratio for the design of culverts constructed by the induced-trench method.

THEORETICAL CONCEPTS

The purpose of the induced trench is achieved as the column of soil above the culvert settles downward relative to the adjacent compacted soil. The relative movement generates shearing forces that act upward on the interior prism of soil as shown in Figure 1. The shearing forces support part of the weight of the column of soil above the conduit, thereby reducing the load on the culvert.

If the embankment is sufficiently high, the shearing forces may terminate within the embankment at a horizontal plane, termed the plane of equal settlement. Above that plane, no relative settlements occur and no transfer of load takes place. If the embankment is not sufficiently high, no plane of equal settlement will develop beneath the top of the embankment. In that case, differential settlement will occur throughout the height of fill above the culvert. That situation, which is termed the complete-ditch condition, could possibly result in a localized sag in the roadway. When considering the use of the induced trench, an engineer is primarily concerned with the possibility of an eventual settlement of the roadway above the culvert.

When the induced trench is analyzed, an important parameter to be considered is the settlement ratio. That ratio is an indication of the magnitude of the relative movements of the prism of soil directly above the conduit and the adjacent soil and is used in computing the design loads on the culvert. The settlement ratio for the induced trench is calculated by the following formula:

$$r_{sd} = \frac{S_g - (S_d + S_f + d_c)}{S_d} \tag{1}$$

where

r_{sd} = settlement ratio,
S_g = settlement of compacted embankment at level of top of trench and adjacent to sides of trench,
S_d = deformation of fill from top of pipe to top of trench,
S_f = settlement of flow line of conduit, and
d_c = shortening of vertical dimension of pipe.

During this study, all factors in the formula were measured directly in the field with the exception of S_d. S_d was determined by subtracting the measured values of S_f and d_c from the total settlement of the critical plane measured as $(S_d + S_f + d_c)$.

Once the settlement ratio is established, charts developed by Spangler (9) facilitate the computation of the theoretical loads on the conduit as determined by the following formula:

$$W_c = C_n w B_d^2$$

where

W_c = load/lin ft of conduit;
C_n = load coefficient, which is a function of ratio of height of fill to width of ditch H/B_d, of projection ratio p', of settlement ratio r_{sd}, and of coefficient of internal friction μ;
w = unit weight of backfill, and
B_d = width of trench.

In his derivation of the load theory for underground conduits, Marston pointed out that the influence of the coefficient of internal friction μ of the fill material is relatively minor, and, therefore, the product of Rankine's lateral pressure ratio K and the coefficient of internal friction may be safely assumed to equal 0.13 for the induced trench. Based on that assumption, Spangler's charts relate the load coefficient to the parameters used to analyze the induced trench (Fig. 2). A different chart is used for each value of the projection ratio. Only the chart for a projection ratio of 1.0 is included in this report because the condition represents the case under study. Once the projection ratio and the H/B_d ratio are determined and the settlement ratio is estimated, the proper value of the load coefficient is found from the chart.

RESEARCH INSTRUMENTATION

The instrumentation used to measure the required settlements consisted of settlement platforms with vertical reference rods located in groups of three beneath the median and under each outside shoulder (Figs. 3 and 4). All settlement plates were placed in the plane of the top of the induced trench 6 ft above the top of the culvert pipe. The platforms consisted of 24-in.-square steel plates $^1/_4$-in. thick and 5-ft lengths of $^1/_2$-in. steel pipe welded to the center of the plates. As the fill height was increased, additional 5-ft extensions of $^1/_2$-in. pipe were added.

Changes in culvert diameter were measured with an extensometer consisting of an Ames dial graduated in 0.001-in. increments and fastened securely to one end of a steel

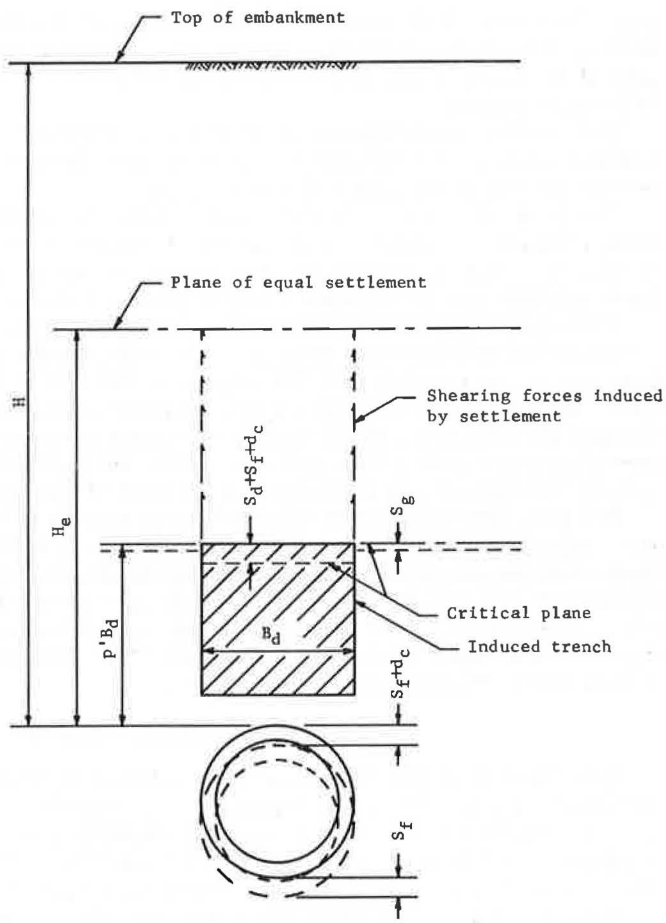


Figure 1. Settlements that influence loads on induced-trench conduits.

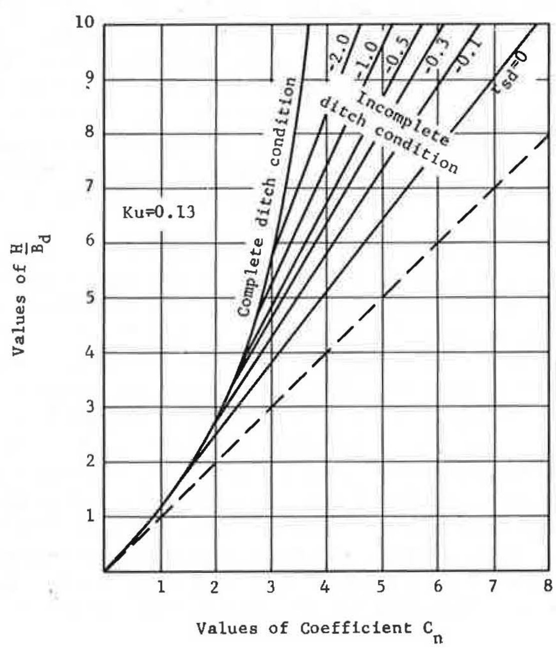


Figure 2. Coefficient C_n for induced-trench conduit when $p' = 1.0$.

rod. To ensure that the extensometer would be at the same precise location each time that the pipe deformation was measured, reference points were established inside the pipe at the same locations along the culvert where the settlement plates and pressure cells were placed.

To measure invert elevations at the 3 locations of instrumentation required that a vertical angle be turned with a transit because the grade of the culvert was too steep to permit the use of a horizontal line of sight.

The pressure cells used to measure the earth pressure against the pipe were originally designed to measure pore pressures under earth dams. Their selection for use in this research was based on their resistance to damage from moisture; that makes them suitable for an extended study of pressure under a high fill.

Each cell is a sealed hollow plastic dish about 8 in. in diameter (Fig. 5). The hollow cell is filled with low-viscosity oil. The sides of the cell are sufficiently flexible so that soil pressure applied to the outside of the cell is transmitted to the oil inside the cell. Within the oil is a thin plastic envelope about 4 in. in diameter. Gas is pumped through the envelope, which is held closed by the oil pressure until sufficient gas pressure develops to open the envelope. Each cell was calibrated to give the external pressure if the applied gas pressure and the rate of flow of gas through the cell are known.

The pressure cells were attached to the outside of the culvert pipe when the top of the compacted embankment was 1 ft above the top of the pipe. Small pits were dug down to the cell locations at the top and spring lines of the pipe. A flat mortar pad was formed at each cell location, and the cell was attached to the flat surface by an epoxy glue. The exposed face of each cell was covered with a 2-in. layer of AM-9 chemical grout. After installation of the pressure cells, the pits were carefully backfilled and compacted with pneumatic hand tampers.

CONSTRUCTION

Construction of the induced-trench installation began in May 1961 and was completed in October 1961. Work was interrupted several times by wet weather. The culvert is a 48-in. ASTM C-76 class 4 reinforced concrete pipe located on a local channel change parallel to and under the base of a 30-ft high ridge. The entire culvert length of 324 ft is in cut except for the extreme downstream end, which meets the natural channel.

The soil material is generally a compact clay till and has a few stone fragments as wide as several inches in diameter. The uphill shoulder of the cut consists of a mottled yellow silty clay loam over gravelly clay till. The downhill shoulder of the cut contains some black organic soils associated with the valley floor.

The maximum depth of cut at the centerline was about 8 ft; the average depth was about 6 ft. The average width of excavation in the plane of the top of the pipe was about 11 ft; the average width at the flow line was about 8 ft.

A 6-in. compacted bed of sand was placed throughout the bottom of the cut to provide a firm base over the slightly muddy and gravelly bottom (Fig. 6). After the pipe was placed, sand backfill was carried up to the spring lines and compacted with pneumatic hand tampers.

The backfill material from the spring lines to the top of the pipe consisted of cut material mixed with sand. The use of pneumatic hand tampers was continued as much as 6 in. below the top of the pipe. From 6 in. below to 6 in. above the top of the pipe, compaction was effected by driving a rubber-tired 4-wheel tractor back and forth along the pipe. After the compacted cover over the pipe reached 6 in., conventional sheepsfoot rollers were used. When the compacted cover reached 1 ft, construction operations were halted to permit installation of the pressure cells.

After the pressure cells were installed and the installation pits were refilled and compacted by pneumatic hand tampers, the fill was completed to a level 6 ft above the top of the pipe. At that point, a 5-ft wide by 5-ft deep by 280-ft long trench was excavated by backhoe directly over the culvert pipe. The trench was refilled by bulldozer with loose topsoil containing some sod and a few cornstalks (Fig. 7). Density of the trench material varied from 51 to 56 percent of the maximum density obtained by AASHO Method T 99. After the trench had been filled loosely, it was blanketed with a foot of

Figure 3. Location of pressure cells and settlement plates with reference rods.

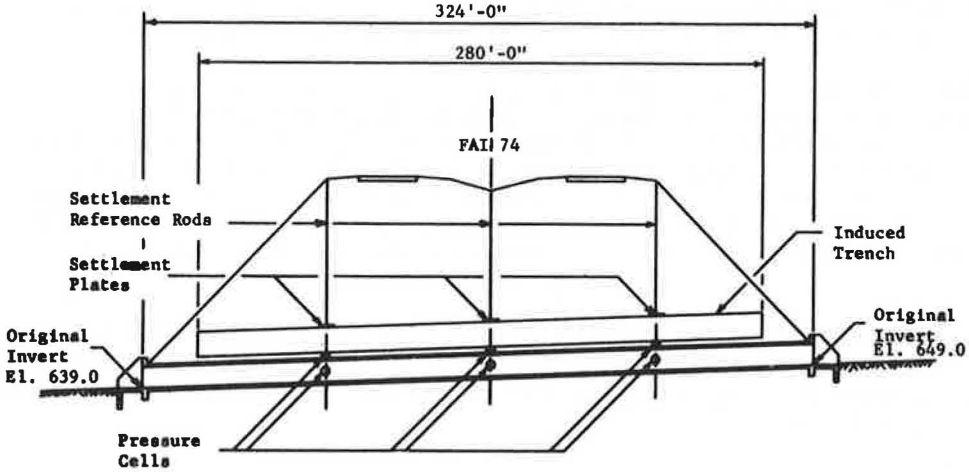


Figure 4. Typical cross section of induced trench.

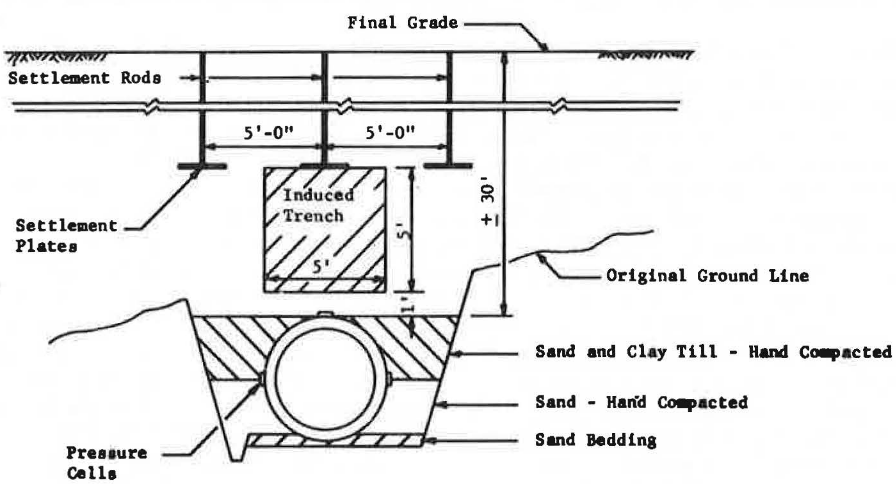


Figure 5. Pressure cell before installation.

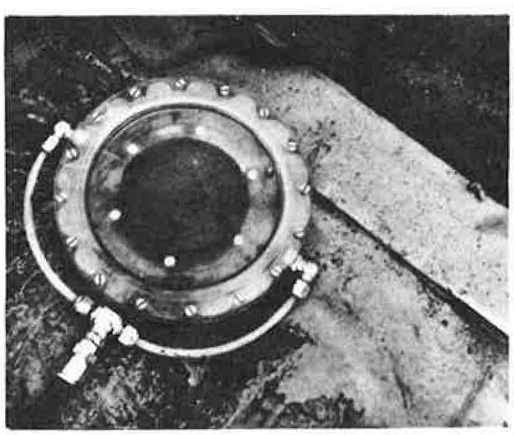


Figure 6. Six-inch compacted sand bed.

silty clay bulldozed from the uphill side of the cut. The settlement plates were then installed, and the remainder of the embankment was constructed in the usual manner. Compaction was by self-propelled scraper haul traffic and crawler-pulled sheepsfoot rollers.

Results of density tests of samples taken near the spring lines, near the top of the pipe, and at approximately 5-ft increments of fill height varied from 97.1 to 105.1 percent of the maximum density obtained by AASHO Method T 99. The percentage of moisture as determined by the same method varied from 46 to 114 percent of the optimum moisture content. Borings taken near each of the 3 transverse instrumentation locations at completion of the embankment indicated that the moisture content varied from 14 to 25 percent, and unconfined compressive strengths varied from 0.9 to 3.8 tons/ft^2.

FIELD TEST RESULTS

The settlement and pressure data were analyzed in this report for the 500 days from May 1, 1961, to September 12, 1962. Data collected beyond that period, although not complete, indicate that little change took place in the settlement ratio or the soil pressures acting on the pipe.

Settlement Ratio

Because the induced trench ensures that the column of soil over the culvert will settle more than the adjacent compacted soil, the settlement ratio for the induced trench will always be a negative quantity. The ratio is currently assumed to lie between -0.3 and -0.5.

Figure 8 shows a plot of the settlement ratios derived from this installation at the north, center, and south locations for the period from May 1961 to September 1962. The settlement ratio after 1 year varied from about -0.25 at the center location to approximately -0.45 at the north location. The ratio at the south location continued to increase in the negative direction and reached a value of -0.80 after 500 days.

Because of the variation in the settlement ratio, the individually measured parameters used in computing the ratio are presented and discussed below.

S_g = settlement of compacted soil adjacent to the trench. The values used for this parameter were the average measured settlement of the side plates installed on the compacted fill at an elevation 6 ft above the top of the pipe. At all 3 locations, the east and west side plates settled different amounts (Figs. 9, 10, and 11). The magnitude of the settlement of the west plates is fairly consistent at the center and the south locations but is about 0.26 ft less at the north location after 500 days. The magnitude of the settlement of the east plates varied from 0.85 at the north location to 0.81 at the center location and 0.52 at the south location after 500 days. The variation in settlement of the side plates at the 3 locations possibly was caused by differences in the natural soil deposits on the uphill and downhill shoulders of the cut as previously described in the construction section of this report.

$(s_d + s_f + d_c)$ = settlement of critical plane. The measured settlement of the center plate located at the top of the trench directly over the culvert centerline was used as the total settlement of the critical plane. The settlement was consistent at the north and center locations but was considerably less at the south location. The variation in settlement is possibly partly due to the varying amount of fill over the different plate locations. The final amount of fill varies from a maximum of 30.5 ft at the north location to a minimum of 27.5 ft at the south location. Measurements indicate unreasonably that the west plate at the south location settled more than the center plate. Only the settlement of the east plate was used in calculating the settlement ratio at the south location.

s_f = settlement of the pipe invert. The pipe invert settlement was consistent at all 3 locations as shown in Figure 12. The average magnitude of the settlement after 500 days was approximately 0.4 ft.

d_c = deformation of culvert pipe. The inside vertical diameter of the conduit decreased an average of about 0.010 ft after 500 days (Fig. 13). The erratic change in pipe deformation during the construction of the embankment is possibly due to temperature changes within the pipe and not due to changes in load. However, data were not collected during this research to confirm the effect of temperature change on the pipe.

Figure 7. Refilling trench with compressible material.

Figure 8. Settlement ratio versus time.

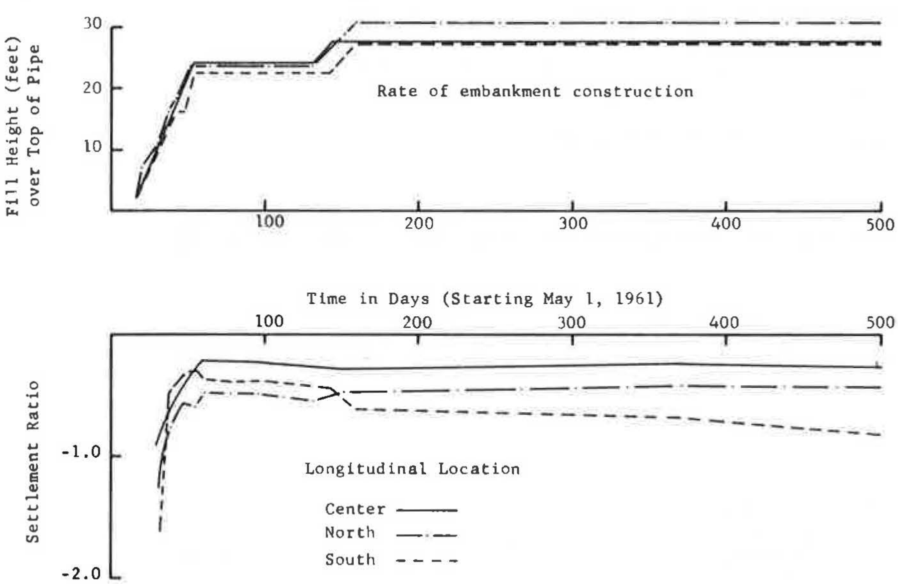
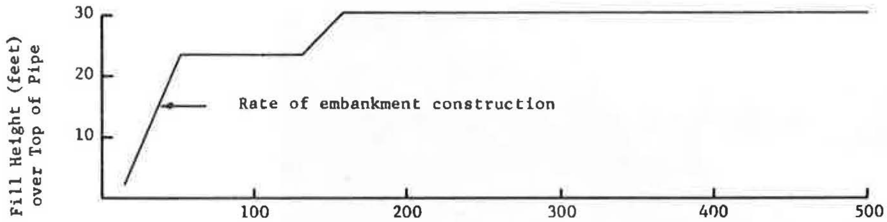


Figure 9. Settlement versus time at north location.



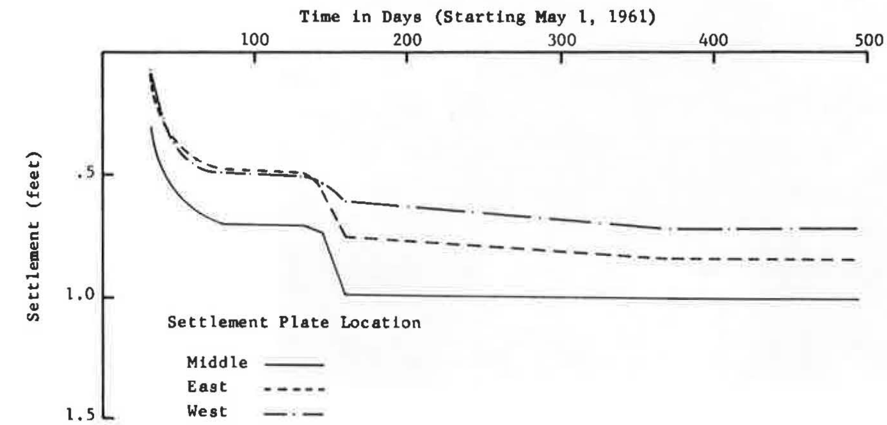
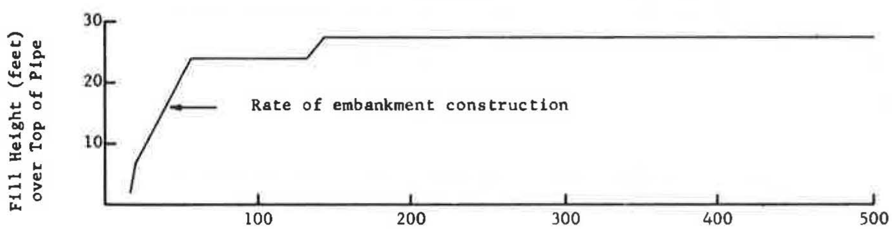


Figure 10. Settlement versus time at center location.



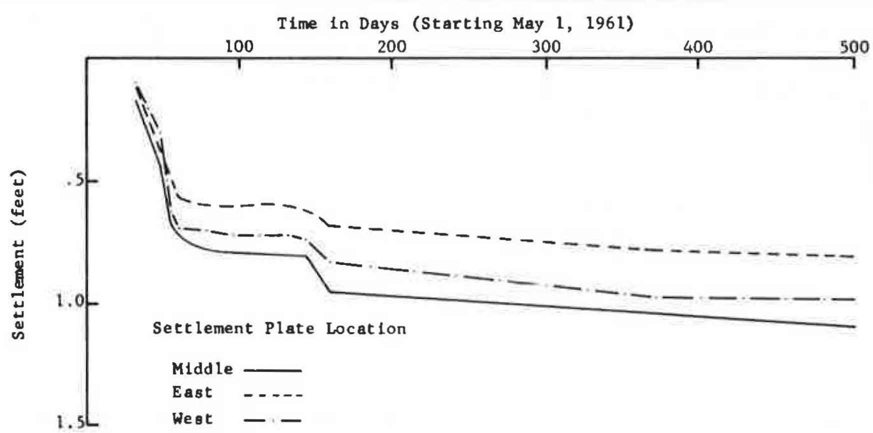


Figure 11. Settlement versus time at south location.

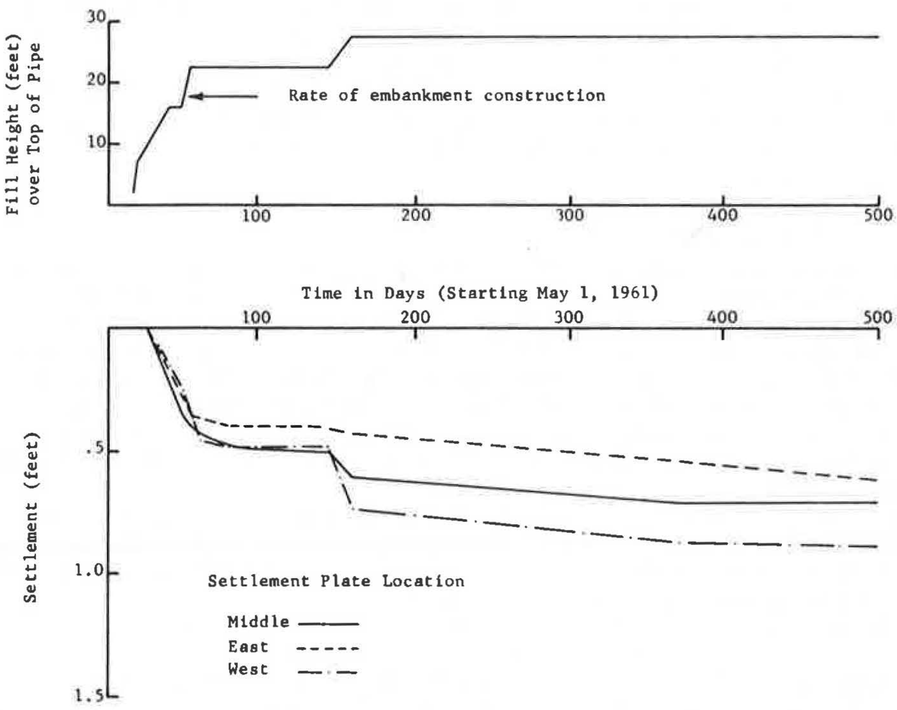
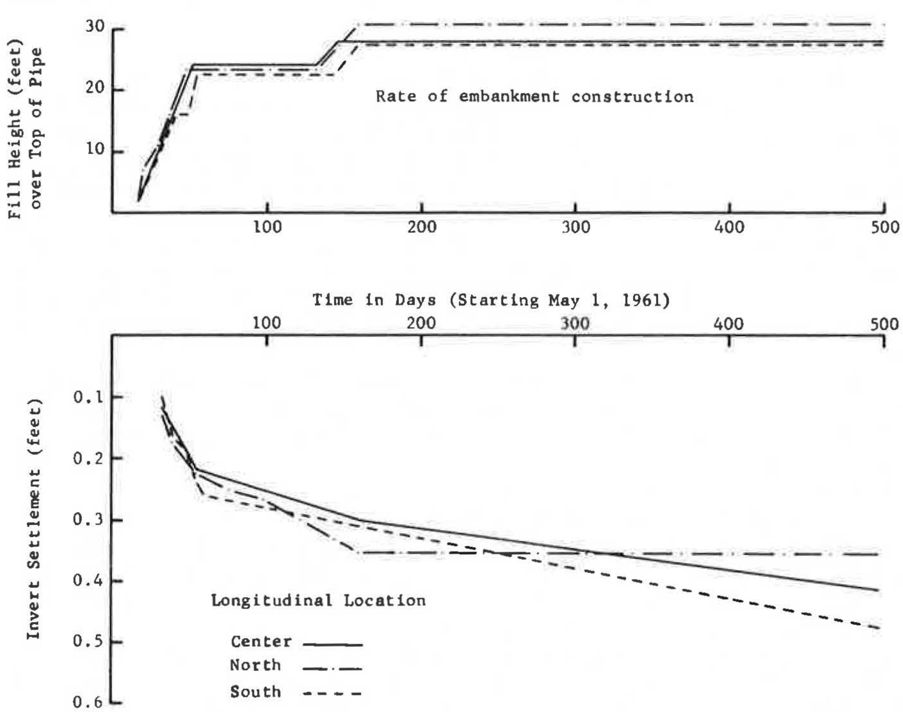


Figure 12. Invert settlement versus time.



In addition to measurements of the change in vertical pipe diameter, changes in the horizontal diameter and each diameter at 45 deg from the vertical were measured. The indicated decrease in all diameters of about 0.01 to 0.02 ft does not appear to be consistent with the loads measured on the sides of the pipe.

Except for the south location, where settlement data were not consistent with data collected at the north and center locations, the values for the settlement ratio were near the limits of the range of values of -0.3 to -0.5 that have been recommended for the induced trench.

Measured and Theoretical Loads on Culvert

Although the theory used to determine the loads on this type of conduit is considered reasonably accurate, the pressures acting on the top and the sides of the culvert were measured during and after construction in order to confirm the theory. The recorded pressures, as shown in Figures 14, 15, and 16, indicate a large variation in pressure on the top cells during the first 150 days after construction began. The reason for the sharp drop in pressure after 60 days at all 3 locations is not apparent, especially since the fill height was constant during this period. It is possible that the heavy rainfall that occurred during the period may have had some effect on the pressures, although the degree of influence is not known.

After 150 days the pressures on the top cells at the north and center locations were fairly consistent and equal to about 7 $lb/in.^2$ The drop in pressure at the south location appears to indicate a possible malfunction in the pressure cell, for the pressure is not consistent with measurements at the other 2 locations.

The measured pressures at the side of the culvert as shown in Figures 14, 15, and 16 were in the order of 1 to 2 $lb/in.^2$ Those values appear to be extremely low for that type of installation, although the arching action of the soil above the culvert could conceivably transmit a large proportion of the load to the sides of the ditch above the pressure cells. Also, it is possible that after the holes were excavated to install the pressure cells, desiccation of the adjacent soil may have formed a hard inflexible crust in front of the cells, and that crust did not permit typical pressures to be transmitted to the cells.

The low recorded pressures also may have been caused by drying out of the AM-9 grout used to fill the spaces between the soil and the pressure cells. The grout may have hardened if the soil became desiccated. Literature from the manufacturer of the chemical grout indicates that the AM-9 gels shrink if they are allowed to dry. Although the shrinking process is understood to be reversible with the addition of water, once the gel had dried, sufficient moisture may not have been present in the soil to swell the dry gel to its original shape.

Pressure cell readings taken in 1963 and in 1967 were consistent with the readings taken in September 1962, except for the west cell at the south location. The pressure recorded at that cell increased from 1.0 psi in 1963 to 6.5 psi in 1967. Clogging of the lines is believed to be responsible for the relatively large increase in the pressure recorded at that location.

The theoretical loads that would act on this conduit were compared with actual measured loads by converting the measured pressures to load per linear foot of pipe. The measured loads corresponding to the rate of embankment construction are shown as curve 1 in Figures 17, 18, and 19.

For a unit weight of soil of 120 lb/ft^3, the theoretical loads that would act on the induced-trench installation based on Marston's formula are as shown by curve 2.

A comparison of curves 1 and 2 at the north and center locations indicates that the measured loads were not greater than 50 percent of the theoretical loads after the embankment was completed. The curves for the south location show the inherent inconsistencies of the pressure charts and are neglected.

In addition to a comparison of the theoretical and measured loads acting on the induced trench, Figures 17, 18, and 19 also show as curves 3 and 4 the theoretical loads that would act on this culvert if the induced-trench method of construction were not used. Two hypothetical conditions were used for comparison with the induced-trench

Figure 13. Change in vertical pipe diameter versus time.

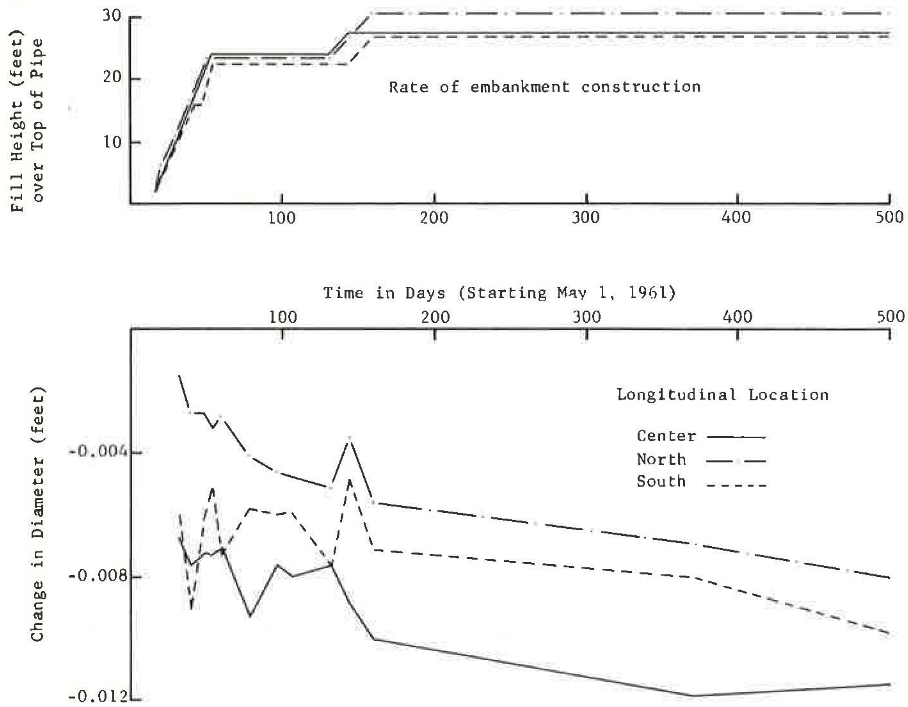


Figure 14. Pressure cell readings versus time at north location.

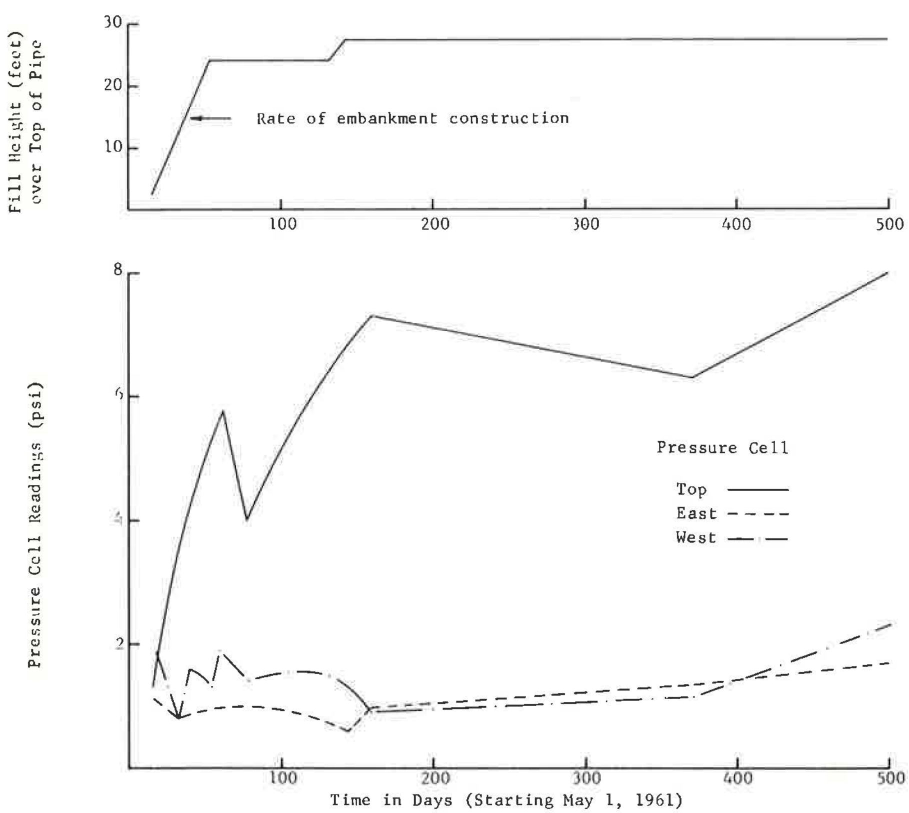


Figure 15. Pressure cell readings versus time at center location.

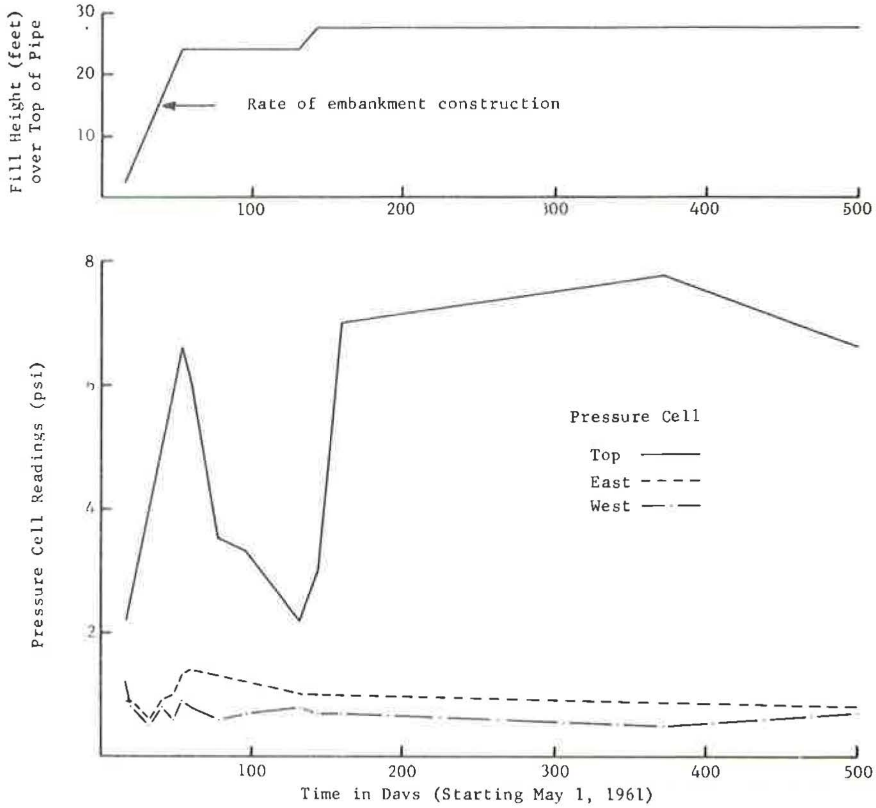


Figure 16. Pressure cell readings versus time at south location.

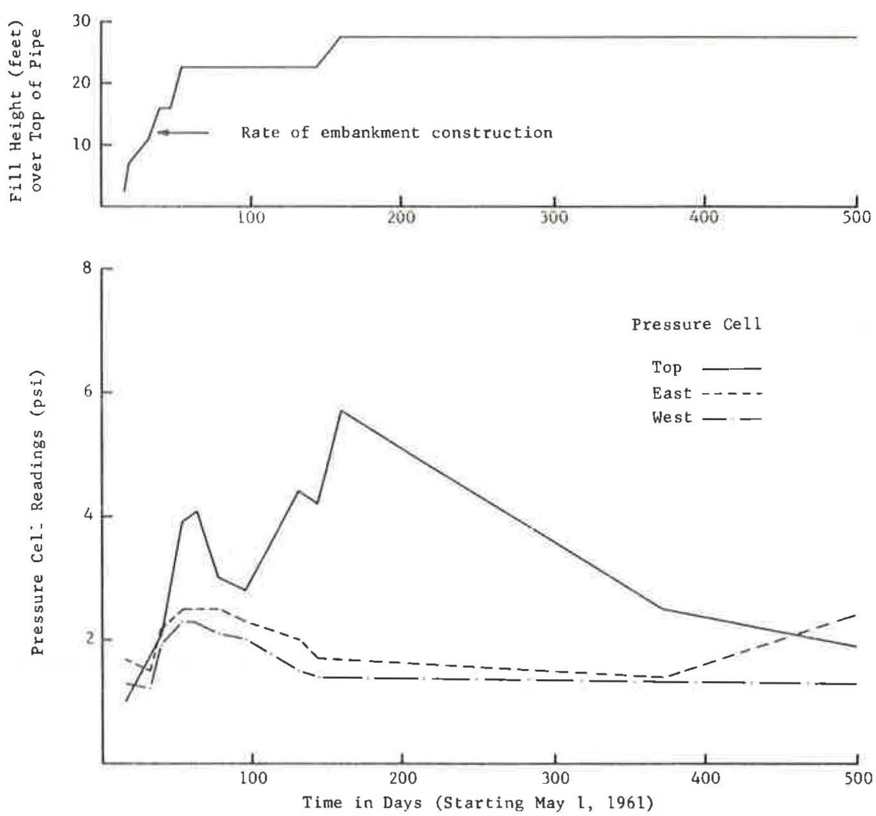


Figure 17. Theoretical and measured vertical loads at north location.

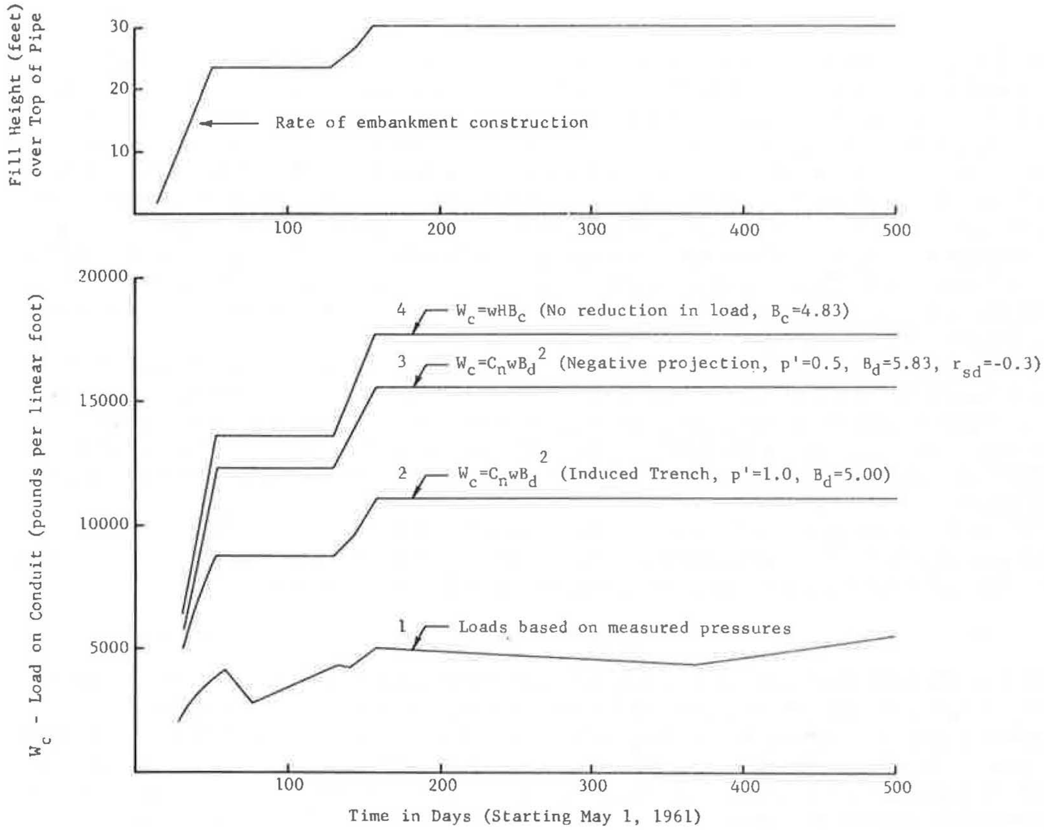
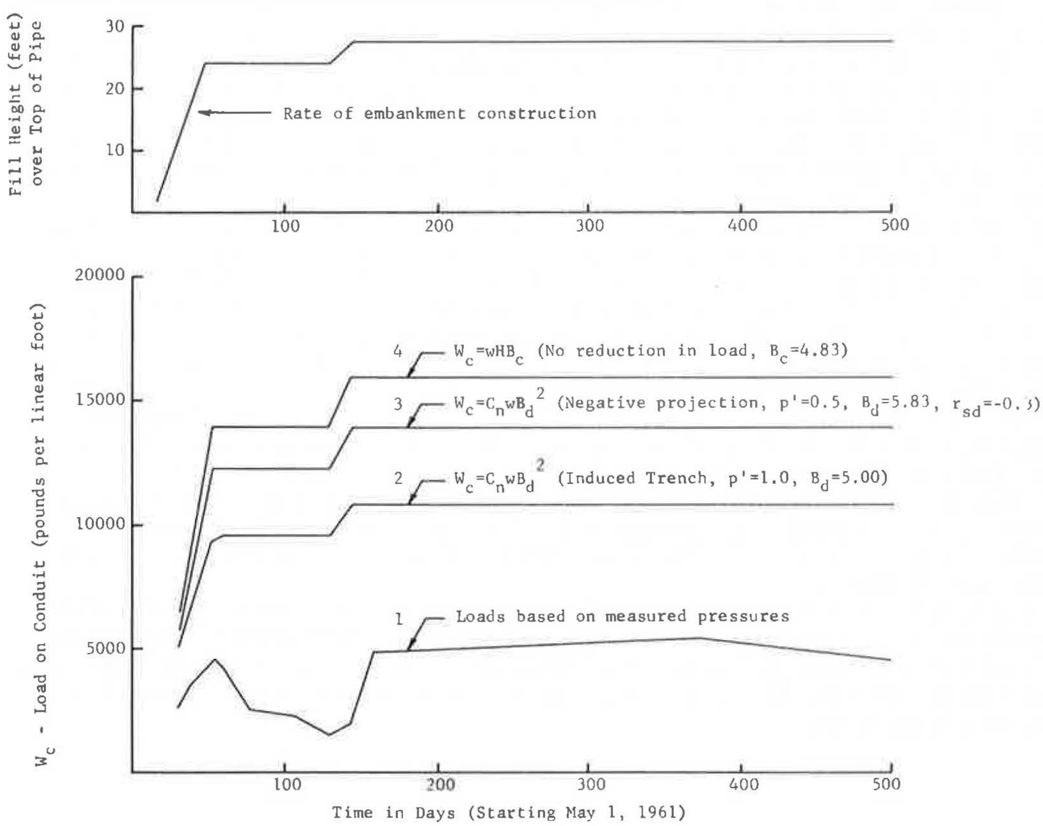


Figure 18. Theoretical and measured vertical loads at center location.



installation. Because the depth of the trench varied from about 4 to 8 ft along the length of the culvert, cases I and II in Figure 20 represent the approximate range in trench depth and were used to estimate the range in loads acting along the length of the culvert.

In case I the width of the trench was assumed to be equal to the width of the conduit plus 1 ft or 5.83 ft, and the depth of the trench below natural ground was assumed to be equal to 8 ft. Because the top of the pipe is placed below natural ground, this case corresponds to a negative projecting conduit and has a projection ratio of about 0.5. The recommended range of values for the settlement ratio for a negative projecting conduit and an induced trench is about the same. For the settlement ratio value of -0.3, the theoretical loads on the culvert computed by Marston's formula are as shown by curve 3 in Figures 17, 18, and 19.

For case II the top of the conduit was assumed to be level with natural ground; that would result in a projection ratio of zero and correspond to a trench depth of about 5 ft. If no relative movement takes place between the soil prism above the pipe and the adjacent soil, i.e., $r_{sd} = 0$, there would be no reduction in load on the conduit, and it would support the total weight of the above column of soil. That situation is shown by curve 4 in Figures 17, 18, and 19.

The unit weight of the fill material above the culvert for curves 2, 3, and 4 was assumed to be 120 $lb/ft.^3$ Comparing curves 3 and 4 with curve 2 reveals the advantage of using the induced-trench method of construction at this installation.

CONCLUSIONS

The settlement ratios for the induced-trench method of installation as determined by this research project correlate well with the range of values that have been recommended by others. Empirical values of the settlement ratio recommended for use with the induced trench range from -0.3 to -0.5. After initial variations during construction of the induced-trench installation, values of the settlement ratio at 2 of the 3 locations where settlement measurements were made ranged from -0.25 to -0.45. At the third location, a settlement ratio of -0.8 was considered unreliable because of discrepancies in the settlement data.

No change from current recommendations regarding values to be assumed for the settlement ratio is proposed on the basis of the research described in this report. Before definite conclusions are reached concerning the precise value of the settlement ratio for this type of construction, other similar tests must be conducted on culverts of different sizes placed under various fill heights.

At this installation there has been no indication of any settlement of pavement over the top of the induced trench. That indicates that a plane of equal settlement has formed beneath the top of the embankment.

At all 3 locations where pressure cells were located at the top and sides of the culvert, the measured pressures appeared to be low for this type of installation. After initial variations, loads based on measured pressures level off about 5,000 lb/lin ft. That represents a load level equal to about 50 percent of the theoretical loads. The sharp drop in load after initial variations at the south location apparently is due to a malfunction of the pressure cell because the load is not consistent with measurements at the other 2 locations.

The measured pressures of 1 to 2 psi at the sides of the pipe at all 3 locations also appear to be low for this type of installation, although the arching action of the soil above the pipe could conceivably transmit a large portion of the load to the sides of the ditch above the lateral pressure cells. Also, it is possible that, after the holes were excavated to install the pressure cells, desiccation of the adjacent soil in front of the cells may have formed a hard inflexible crust that did not permit typical pressures to be transmitted.

Although the loads based on measured pressures were fairly consistent at 2 of the 3 instrumented locations, much more data from this type of installation are required before final conclusions are drawn on the accuracy of the theory. At the present time, it is recommended that the theory, which appears to be conservative, continue to be used without adjustment.

Figure 19. Theoretical and measured vertical loads at south location.

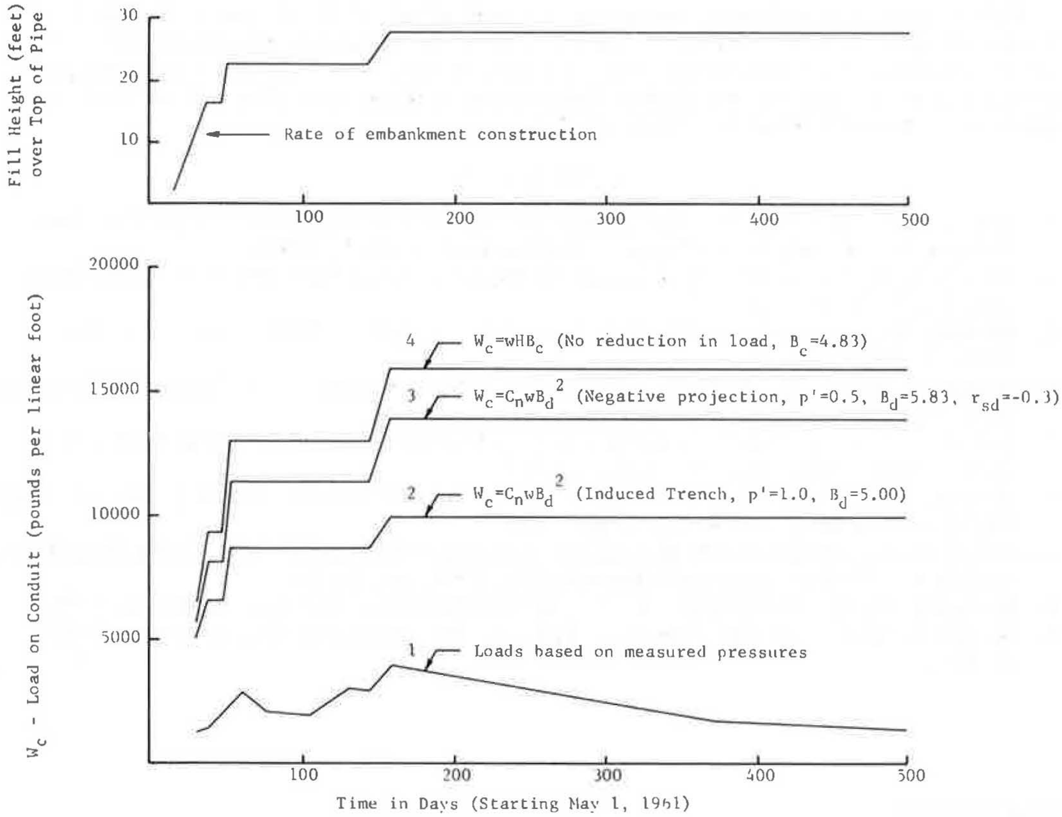
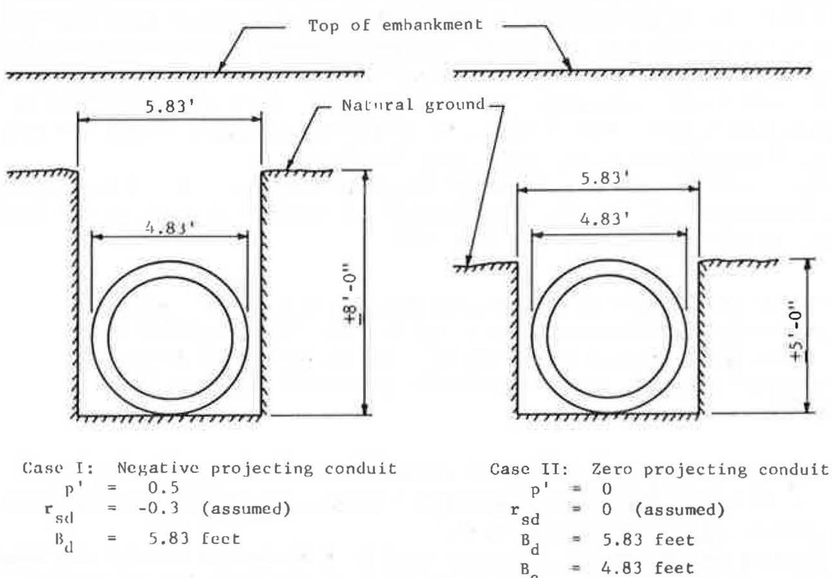


Figure 20. Limits of culvert projection without induced trench.



ACKNOWLEDGMENTS

This research project was conducted as a joint effort of the Illinois Department of Transportation and the American Concrete Pipe Association in cooperation with the Federal Highway Administration. The opinions, findings, and conclusions expressed in this report are those of the Illinois Department of Transportation and not necessarily those of the Federal Highway Administration.

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DISCUSSION

M. G. Spangler, Engineering Research Institute, Iowa State University

This paper constitutes an excellent addition to the literature on the technology of culvert design and construction. It serves to underline and reemphasize the possibilities for greater economy in the installation of cross-drainage conduits under medium to high fills by the imperfect-ditch procedure, which was first introduced and recommended by Marston more than 50 years ago. No matter how reliable laboratory and analytical developments may appear to be, the value of evidence obtained in connection with the performance of actual field installations cannot be overestimated.

The measurements of settlements at several points in the critical plane of the Illinois embankment, which indicated values of the settlement ratio ranging from -0.25 to -0.45, are of special interest to this writer who stated in 1960:

> Research directed toward the determination of loads on negative projecting (and imperfect ditch) conduits has not progressed so far as it has in connection with the other classes of conduits. In the absence of factual data relative to probable values of the settlement ratio for conduits of this class, it is tentatively recommended that this ratio be assumed to lie between -0.3 and -0.5 for the purpose of estimating loads.

The author's findings go a long way toward eliminating the word "tentatively" from the above quotation. It is hoped that other state highway departments will conduct similar studies to add to basic knowledge in this field.

In the matter of load on the structure, pressure-cell measurements indicated a load equal to approximately half of the load calculated by the Marston-Spangler procedure. This, of course, is in the right direction from the standpoint of structural design and safety. Nevertheless, it is of interest to speculate on possible causes of this divergence. One influential factor may have been a difference between the actual coefficient of friction of the embankment soil and the value used to calculate the load.

It is a basic principle of all the conduit-load analyses of the Marston type that the load on the structure is considered to be the resultant of 2 forces: (a) the weight of the prism of soil that lies above the conduit plus or minus the frictional shearing forces that are generated along the sides of this central prism by differential movement or tendency for movement between the central prism and (b) the adjacent soil masses. If the adjacent soil settles more than the central prism, as in the case of the projection condition of positive-projecting conduits, the shear forces are directed downward and are additive to the weight of the central prism to produce the resultant load. If the reverse is true, that is, if the central prism settles more than the adjacent soil, as in the case of ditch conduits, negative-projecting and imperfect-ditch conduits, and the ditch condition of positive-projecting conduits, the shear forces are directed upward and are subtractive from the weight of the central prism.

The magnitude of the unit shear force is a function of the product $K\mu$, where K is Rankine's lateral pressure ratio and μ is the coefficient of friction of the soil (tangent of angle φ). Because K is a function of μ, it develops that the product $K\mu$ varies over a relatively narrow range for all soils: from about 0.13 for $\varphi = 10$ deg to a maximum of 0.19 for $\varphi = 30$ deg or more. It is not considered to be practical to measure the friction angle for the embankment soil of a specific proposed culvert installation. Therefore, in accordance with the principle that the estimated design load should be the probable maximum to which the culvert may be subjected in service, the load calculation diagrams have been constructed by using the value of $K\mu$ that gives the maximum load on the structure. Thus, for conditions wherein the shear forces are directed upward, the calculation diagrams (such as shown in Fig. 2) are based on $K\mu = 0.13$; whereas, for the opposite case of shear directed downward, the diagrams are based on $K\mu = 0.19$.

If the soil of the embankment in the author's research actually had a $K\mu$ value near the maximum of 0.19, the calculated load indicated by the C_n value taken from Figure 2 could have been approximately 40 percent greater than the measured load. That would go a long way toward accounting for the observed divergence between calculated and actual load.

Another circumstance that might have influenced the divergence is the statistical relation between the area of the pressure cells by which the load was measured and the total area of the structure. The pressure cells were 0.67 ft in diameter, and there were 2 cells that gave results considered to be reliable. Thus, the total area over which pressures were measured was approximately 0.7 ft^2. The total area of the pipe projected on a horizontal plane through its top and for the length between the shoulders of the 4-lane roadway was approximately 750 ft^2, or about a thousand times greater than the area of the pressure cells. If one considers the heterogeneous character of soil, it is not surprising that the measured pressures may not have equaled the theoretical loads.

In view of the above possibilities, the author's recommendation that the theoretical approach for estimating design loads on imperfect-ditch conduits continue to be used appears to be justified, even though the results may be somewhat conservative.

INVESTIGATION OF SOIL-STRUCTURE INTERACTION OF BURIED CONCRETE PIPE

Richard A. Parmelee, Department of Civil Engineering, Northwestern University

A long-range research project has been undertaken to investigate the nature of the loading imposed on a concrete pipe when it is buried at a significant depth below the surface of the ground. The project consists of (a) a series of field installations and (b) development of a comprehensive finite-element program. The initial field tests consisted of a trench installation and an embankment installation. Pipe sections were instrumented with strain gauges, fittings for taking diameter and chord measurements, and surface pressure meters. Stress cells were installed in the soil at various locations in the vicinity of the pipe. Data will be used to verify the accuracy of the computer simulation. The plane strain computer model will permit the use of nonlinear mechanical properties for both concrete and soil materials. A cracking mechanism has been developed that accurately models the behavior of concrete pipe at the initiation of, and following the development of, cracking in the concrete. The validity of the finite-element model of the pipe has been established on the basis of the results of a program of laboratory tests on sections of a full-size concrete pipe under controlled loading conditions. The proven computer model of the soil-structure system will help to make it possible to identify the relative significance of various parameters of the interaction system.

•THE ANALYSIS and the design of buried pipes are essentially problems of soil-structure interaction, and full cognizance must be given to that fundamental coupling phenomenon when any design procedures are formulated or evaluated (1). The methods of analysis that are in use today (2, 3) attempt to account for the relative stiffness between the soil and the pipe by a variety of parameters that are largely empirical in nature and appear to have no rational basis within the context of today's engineering knowledge. Although such parameters may achieve their intended goal when used with sound engineering judgment within the limited ranges of applicability for which experience is available, very often their use cannot be extended or generalized. In an effort to clarify this matter, the American Concrete Pipe Association has undertaken a long-range research program to study the problem and develop a reliable design method to more accurately quantify the loads imposed on a buried pipe. The research program is being conducted under a contract with Northwestern University.

The initial step toward achieving improved design methods for determining the distribution of forces and the resulting stresses and deformations in a buried concrete pipe involved an extensive in-depth study of current practices used in the fabrication and installation of pipe. Concurrently, work was started on the basic research plan, which calls for a 2-pronged attack on the problem: experimental and analytical.

The experimental portion of the project involves an actual full-scale installation rather than small-scale models in the laboratory. The latest experimental techniques and instrumentation are being employed; and it was necessary to develop and manufacture some of the instrumentation for this specific application so as to record the data in the most sensitive and reliable manner. Recently developed mathematical methods are being used in conjunction with high-speed, digital electronic computers to study the analytical phase of the problem.

The primary objective of the investigation during the first year was to record the behavior of a full-scale buried pipe and then use those data to assist in the development of a generalized computer simulation of the soil-structure system. The particular mathematical model employed for this purpose is a finite-element idealization. The assembly represents the buried pipe and the entire soil system adjacent to, above, and below the pipe. A finite-element idealization can be compared to an ordinary picture or jig-saw puzzle; i. e., it consists of an assembly of individual parts that, when properly pieced together, present a clear picture of a particular subject.

RESEARCH PLAN

The overall plan for conducting the research program consists of 4 interdependent phases as shown in Figure 1. Phases I and III are primarily analytical, and phases II and IV provide the all-important interfacing with the results from full-scale testing programs.

The objective of phase I is the development of a theoretical mathematical model of a reinforced concrete pipe. That model is based on a plane strain, finite-element idealization in which the elements are quadrilateral and can have nonlinear material properties (4). The stress-strain relation proposed by Hognestad (5) is used for the concrete elements. The model that was developed for phase I of the project will be described in a subsequent section of this paper.

In phase II the validity of this mathematical model of the pipe is established by comparison of the predicted response with data measured during full-scale testing of sections of pipe. The output of phase II will be a valid mathematical model that will accurately predict the behavior of a wide range of sites and classes of concrete pipe.

The soil parameters of the interaction system are the input for phase III. The material properties for the quadrilateral and triangular finite elements representing the soil will be nonlinear and will be established on the basis of an extensive laboratory testing program. The objective of this program will be to define the types of simplified tests that are necessary to perform on any soil to evaluate appropriate material properties to be used for the finite-element idealization of the particular construction site under consideration.

In phase IV the adequacy of the finite-element program to simulate the measured behavior of various full-scale installations will be investigated. The results from the full-scale tests will be considered as the "exact solution" to the problem, and they will serve as a basis for making suitable adjustments in the finite-element mathematical model in order that the 2 solutions will be in agreement. The end product of phase IV will be a valid mathematical model of a soil-pipe interaction system.

This model will make it possible to readily identify the relative significance of various parameters of the interaction of the system, such as nature and distribution of loading around the pipe, inherent structural strength of the pipe, influence of materials, and realistic factor of safety. Whenever possible, the results of recently developed innovations in the manufacture and installation of pipe will be included in the parametric studies undertaken during this phase of the program in order to check their overall effect on the soil-pipe system.

Based on the information generated from detailed study, interpretation, and evaluation of the significant parameters, a set of rational design approaches for concrete pipe will be developed. The resulting design method will be aimed toward permitting the accurate prediction of the distribution of forces on the buried pipe and of the factor of safety against each possible mode of failure from a knowledge of the measurable properties of the pipe, bedding, and fill material.

FULL-SCALE INSTALLATION

Two full-scale installations of instrumented sections of concrete pipe were completed during the initial year of the project. Each of the installations is identical with respect to the size and class of pipe and also the height of fill. The major difference between the test sites is that one pipe system was constructed as an embankment installation and the other as a trench installation. The pipe size is 60 in., and the height of fill is 25 ft.

Studies of the effects of shallow fill heights or live loading or both will be considered in detail during later phases of the project.

The full-scale concrete pipe test installations are located at the Transportation Research Center (TRC) in East Liberty, Ohio (approximately 50 miles northwest of Columbus). The embankment test pipe is part of a utility tunnel under the 7½-mile oval that is being constructed as the center's high-speed test track. The trench test pipe is a "blind" installation located in a corn field in a remote area on the campus; no proposed facility at the center afforded the required depth of cover.

The pipe for each installation was designed on the basis of current design methods (using Marston-Spangler criteria), and a class B bedding was assumed for both the trench and the embankment conditions. All sections of the test pipe (five per installation) were manufactured by means of the wet-casting process, and standard production techniques were employed to ensure that the units would be representative of typical concrete pipe as fabricated for today's market.

The installation of the pipe sections was accomplished through a change order on an existing contract at the TRC; that contract was under the jurisdiction of the Ohio Department of Highways. The pipe sections were installed in accordance with the standard specifications of the Ohio Department of Highways, and the construction sequence was inspected by representatives from the highway department. Thus, the pipe was installed by contractors who used equipment and procedures that are used on typical construction jobs, and the research team did not exercise any control over the installation techniques.

Therefore, each installation can be considered to be representative of an average buried pipeline as constructed today. The data from those test sites will provide valuable insight into the true behavior of buried concrete pipe, and the important effect of soil-structure interaction can be studied. Data will be obtained from those installations for at least 3 years so that the influence of time effects on the behavior of the systems can be evaluated.

Both installations were instrumented in a similar manner. Three types of instrumentation were affixed to the pipe sections: strain gauges, diameter- and chord-measuring devices, and pressure cells mounted flush with the outside surface of the pipe.

The strain gauges are located at points every 22½ deg around the circumference of the pipe. Those gauges are positioned at 4 points through the thickness of the pipe: on the inside surface of the concrete, on the circumferential reinforcement of the inner cage, on the circumferential reinforcement of the outer cage, and on the exterior surface of the concrete. In addition, strain gauges have been applied in a longitudinal configuration at several locations along the length of each test site.

Reference points are located at 45-deg intervals around the pipe sections in order that diameter and chord measurements can be made between the points. That arrangement makes it possible to monitor the changes that occur in the internal geometry of the pipe during the backfilling operation and with respect to time.

The pressure cells on the exterior surface of the pipe are 6 in. in diameter and measure the total normal stress at the soil-pipe interface. Cells of a similar design, but 10 in. in diameter, are buried at various locations in the soil in the vicinity of the pipe and also in the "free-field."

Soil settlement plates were installed at various positions in close proximity to the pipe and also at different elevations of the backfill. During the backfilling operation, numerous soil samples were taken, and an extensive laboratory testing program of those samples is now nearing completion.

Readings were taken of all the instrumentation at various heights of fill during the backfilling operations. Those data were then analyzed by means of statistical techniques, and the results will be used in conjunction with the finite-element computer simulation to construct mathematical models of the soil-structure system at each test site.

The actual construction sequence for each site will be simulated on the computer by means of solving the appropriate finite-element model for each step of the process (i. e., preparation of the site, placement of bedding, installation of the pipe, and layer-by-layer sequence of backfilling). The data from the solution at each step will be compared with the corresponding quantities from the analysis of the experimental data. Such a comparison will make it possible to introduce appropriate corrections to the input parameters for the soil-structure system such that the output from the computer simulation agrees with the data from the "exact solution" (i. e., the results from the full-scale test).

FINITE-ELEMENT MODEL FOR REINFORCED CONCRETE PIPE

For proper analysis of the behavior of buried concrete pipe, the pipe itself must be modeled in a way that takes into account the influence of progressive cracking of reinforced concrete. The phenomenon of progressive cracking has a significant effect on internal stresses and displacements as well as on external deflection of the pipe structure. Thus, it is imperative that the computer simulation of the soil-structure system be capable of modeling that very important aspect of the behavior of actual installations of buried concrete pipe. A major programming effort was necessary to properly incorporate the phenomenon of progressive cracking (and the subsequent changes in the statics, geometry, and material properties) into the computer simulation of the pipe.

The finite-element representation of a typical cross section of the pipe is shown in Figure 2. The concrete mass and the steel reinforcement are each assumed to be homogeneous. A quadrilateral type of finite element is used for both the steel and the concrete, and separate properties are assigned to each of the 2 materials. For purposes of analysis, each round bar is replaced with an equivalent square bar having the same area. The finite-element representation for plane strain is based on a slice of the member of unit width. Because the bars occupy a substantial part of the section width, the reduced area of concrete must be accounted for at the level of the bars. The effect of such thickness variation is easily included in the analysis because the element stiffness, which is usually based on a unit width, is reduced in direct proportion to the thickness reduction.

The uncracked pipe is loaded incrementally until the principal tensile stress exceeds the tensile strength of the concrete at 1 or more element locations. If the principal tensile stress in 2 adjacent elements exceeds the prescribed tensile strength, then a crack is established to exist along the common edge between those 2 elements. That is done by mathematically disconnecting the elements at their common corners. The modeling of the progressive cracking phenomenon can be improved by reducing the length of the finite elements for the pipe; however, such a decrease in element size means an increase in computational effort.

PRODUCT TESTING PROGRAM

As a part of the long-range research project, tests were conducted to evaluate the performance of full-scale pipe under controlled loading. That experimental program provided data that were used to ensure the validity and accuracy of the finite-element computer simulation for reinforced concrete pipe. The correct mathematical modeling of the pipe is considered to be an important and necessary step to be completed prior to extensive computer runs of the entire soil-structure system.

Most of the pipe sections were tested by using the standard 3-edge bearing test procedure (6), and some were tested in a modified loading rig. The instrumentation for all of the test pipe was applied to the surface of the pipe, and 2 major types of data were collected: surface-strain measurements and horizontal and vertical diameter measurements. Plugs for the mechanical strain gauges were located in a continuous manner around the inside surface of the pipe and also in continuous lines along each side of the pipe. With that orientation, it was possible to obtain a continuous distribution of strain around the inner surface of the pipe and the distribution of strain in the vicinity of the spring lines on the exterior surface of the pipe.

The finite-element model that was used to simulate the 3-edge bearing tests is shown in Figure 2. The radial grid lines are spaced at 3-deg intervals around the pipe, and advantage is taken of symmetry in order to reduce computer time.

Typical deflection data as measured during tests of 2 similar pipes are shown in Figure 3. Readings were taken at predetermined load increments, and those values are

Figure 1. Research plan.

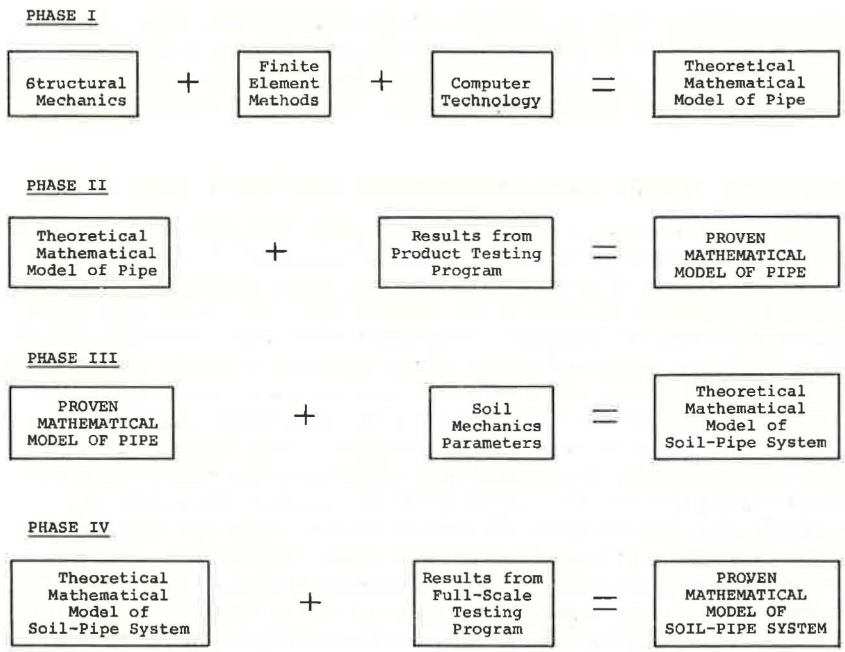


Figure 2. Finite-element representation of 3-edge bearing test.

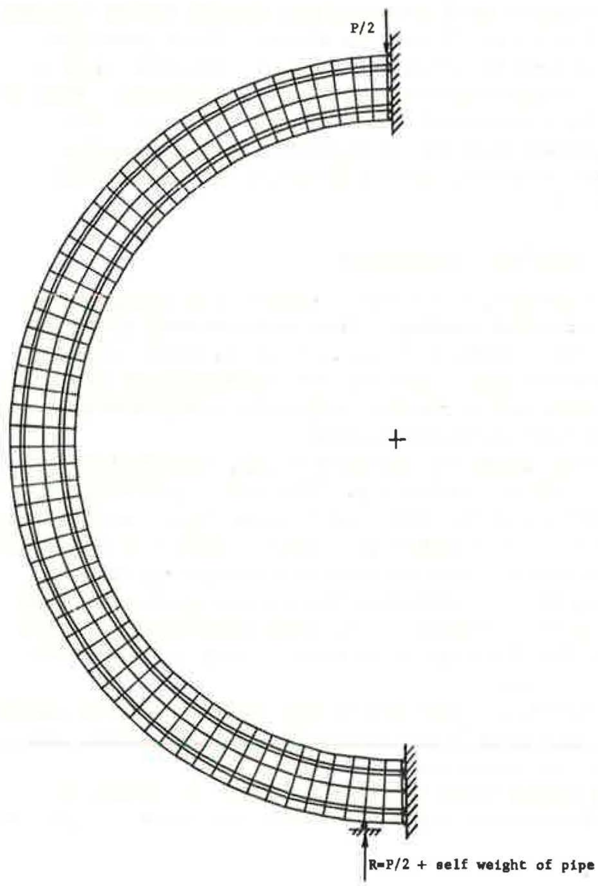


Figure 3. Measured and predicted deflection changes of 60-in. pipe.

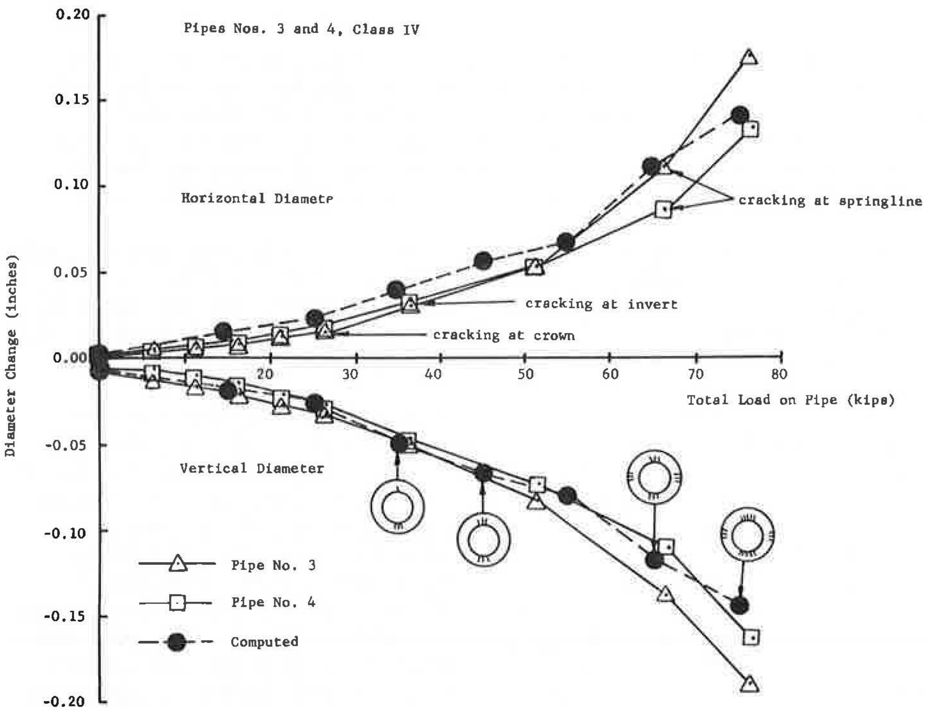
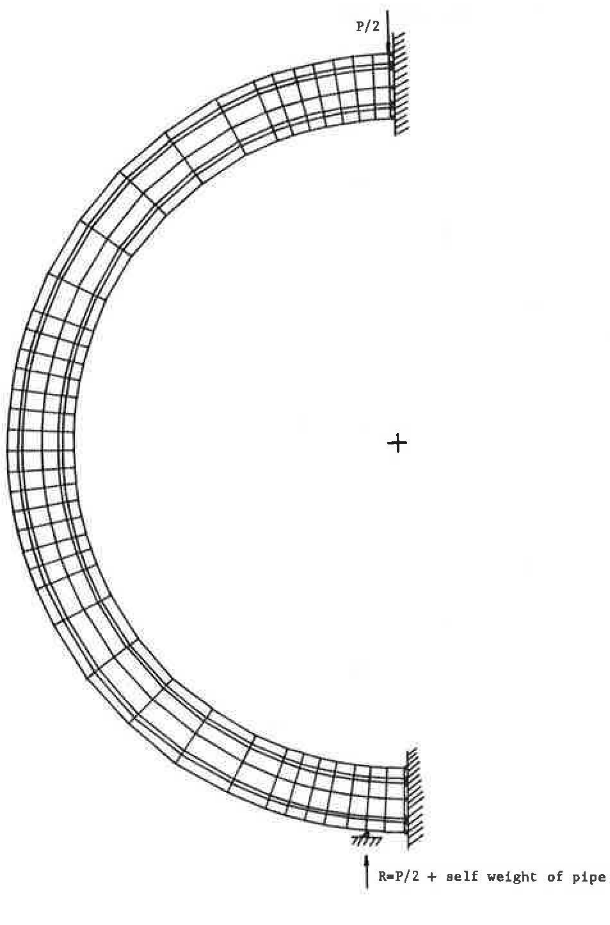


Figure 4. Mixed-element representation of 3-edge bearing test.



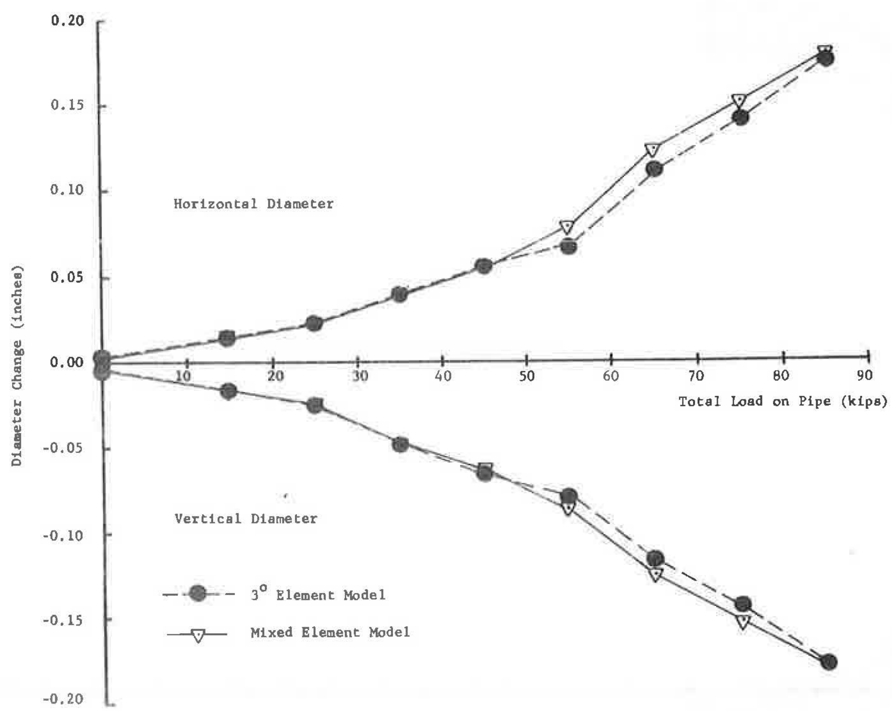
recorded as either triangles or squares in the figure. The data points have been connected with straight lines to denote the progression of deflection with applied load. The load increments at which surface cracking was initially observed are also noted.

The predicted deflection behavior of the mathematical model of a pipe having properties equivalent to the actual pipe are shown as solid circles. Sketches showing the extent of cracking in the model at various loading stages are presented in the lower portion of the figure.

The agreement between the actual and the predicted values of deflection is very good, and similar correlations were obtained for the strain data around the surface of the pipe and also for the changes in geometry the pipe experienced throughout the loading cycle. The computer simulation of the concrete pipe was judged to be very satisfactory for all of the specimens tested, regardless of the size of the pipe, amount of reinforcement, or method of loading.

There is a difficulty in using the 3-deg-element mesh to model the buried pipe because of the large number of elements and node points required. Therefore, the model was modified to reduce the number of nodes and elements. The grid developed is shown in Figure 4. The elements are a mixture of 3-, 6-, and 9-deg elements. The smaller elements are located at positions where the previous analyses indicated cracking will occur. The response of this model for the concrete properties and loading of the previous analysis compared with the response of the 3-deg model is shown in Figure 5. Generally the mixed-element model is a good approximation of the 3-deg-element model for the same concrete properties and loading, and it requires much less computer time. Thus, the mixed-element grid is the mathematical model for the pipe and will be used in the computer simulation of the buried pipe system; that represents the completion of phase II (Fig. 1).

Figure 5. Comparison of computer simulations for tests 3 and 4 using 3-deg- and mixed-element models.



SOIL MECHANICS PARAMETERS

The results of a few preliminary analyses of the full-scale test installations have shown that extreme care must be taken in defining the nonlinear material properties of the soil surrounding the pipe. The necessary theoretical development and associated laboratory testing program for establishing the appropriate input for phase III of the overall program are currently in progress. Those parameters will be incorporated into the computer simulation as soon as they have been evaluated and verified.

CONCLUSION

In general, all of the previous full-scale investigations of buried concrete pipe have recorded only the gross behavior of the pipe itself. In contrast, the present investigation is the most extensive testing program yet undertaken for concrete pipe, and it is the first field study to consider the interaction behavior of the total soil-structure system. A wealth of data have been and will be obtained from the 2 test sites, and all of it will be most useful in assessing the present state of the art in the design, fabrication, and installation of buried concrete pipe.

With the aid of the computer-simulated, finite-element model of the soil-structure system and the data from the full-scale installations, it will be possible to properly evaluate and quantify the degree of accuracy of existing design procedures for buried concrete pipe. The finite-element model will be used to make extensive parametric studies of the composite soil-structure system and investigate the factors influencing the behavior of buried concrete pipe. Those data will make it possible to propose modifications of current design methods or to develop new design procedures, fabrication techniques, and installation methods. Any recommendations will be tested in additional full-scale installations to be carried out in the remaining years of the research program. The ultimate objective of this long-range investigation will be the development of a design tool that will be realistic, yet simple and easy to use.

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COMPUTERIZED DESIGN OF PRECAST REINFORCED CONCRETE BOX CULVERTS

Raymond W. LaTona and Frank J. Heger, Simpson, Gumpertz and Heger, Inc.; and Mike Bealey, American Concrete Pipe Association

This paper describes the development of a general computer method for design of single-cell, precast reinforced concrete box culverts. The method uses the loading requirements of the American Association of State Highway Officials and the ultimate strength design approach of the American Concrete Institute. The user describes geometry and loading conditions, and the program analyzes many loading cases by the stiffness matrix method and determines the design forces by appropriate combinations of the results of those analyses. Based on the design forces, reinforcing steel is selected to provide adequate strength to resist the bending moments and axial forces. Shear stresses are checked to determine whether slab thicknesses are sufficient without shear reinforcement; no shear reinforcement is included in the design. A crack-control provision based on work by Gergely and Lutz is included. Culvert spans of 3 to 12 ft, rises of 2 to 12 ft, and burial depths of 2 to 100 ft are permitted. The top and bottom slabs of the culvert may have different thicknesses, and the side walls of the culvert may be a third as thick. Linear haunches may be specified and are taken into account in both the analysis and the design procedures. The computer program and its applications are discussed, and 2 sample problems are included.

•CAST-IN-PLACE, reinforced concrete box culverts have been designed and used for many years because of special waterway requirements or unusual load conditions at certain locations or because of designer preference. As labor costs continue to rise, so do the costs associated with cast-in-place reinforced concrete. As the volume of highway traffic increases, so does the cost of inconvenience and delay associated with cast-in-place construction methods. Therefore, attempts have been made to develop and specify precast concrete box sections, but they have been unsuccessful because the approach was local in nature or confined to one project.

In early 1971, the Virginia Department of Highways and the American Concrete Pipe Association (ACPA), with financial support of the Wire Reinforcement Institute (WRI), began a cooperative venture to develop a manufacturing specification including standard designs for precast reinforced concrete box sections that would be used primarily by the Virginia highway department but could be adaptable as a national specification under the auspices of AASHO or ASTM. From the beginning, both groups believed that the same production and construction methods used with precast concrete pipe could be successfully applied to precast concrete box sections; in other words, these could be considered as precast concrete pipe of rectangular cross section. The proposal was that standard box culverts be plant-produced, be manufactured under strict quality control procedures and subject to inspection, and be installed by rapid cut-and-fill procedures. The venture quickly evolved into 2 efforts—the manufacturing specification and the standard designs—although certain parameters were important and common to both.

PRELIMINARY STUDY

The objectives of the preliminary study were to determine the effect of parameter variation and to give some indication as to what sizes should be selected for publication as standard designs. The infinite number of cross-sectional dimensions and of designs possible with box sections was the main problem. In plant production, the capital cost and inventory of forms are critical items in determining product costs. Obviously a producer cannot be expected to maintain infinite numbers of forms and sizes or forms for sizes rarely used in his area.

The initial sizes selected for study were a compromise reached by interested producers representing all parts of the United States and Canada. The slab and wall thicknesses and the steel design stress were varied to produce 384 designs that were analyzed by the ACPA Technical Committee.

After the analysis was reviewed, it was evident that, although final design parameters could be selected, the existing computer design program was inadequate for designing precast reinforced concrete box sections for several reasons. The existing program could not properly handle the high-strength, welded-wire fabric considered for use in the manufacture of the box section; the program was set up for covers over the steel as normally used in cast-in-place design and not the lesser covers that could be maintained through plant production as evidenced by those used in precast concrete pipe; and the existing program did not include haunches in the design and analysis procedures.

It was necessary, therefore, to develop a new program. It was proposed that a general computer method be developed for the design of single-cell, precast reinforced concrete box sections. The method would take into account the close tolerances, the quality and high concrete strength capabilities of plant production, and the characteristics of high-strength, welded-wire fabric and would include haunches in the design and analysis procedures. The remainder of this paper describes the development, criteria, and applications of the computer program.

GENERAL CAPABILITIES AND LIMITATIONS OF THE PROGRAM

Application

The program designs buried precast reinforced concrete box culverts in accordance with the loading requirements of AASHO (1) and ultimate strength design provisions of ACI (2). The program is general, can be used to design any rectangular culvert with or without haunches, and gives the designer the capability of specifying the following information:

1. Culvert geometry—span, rise, wall thicknesses, and haunch dimensions;
2. Loading data—depth of fill, density of fill, lateral pressure and effective height coefficients for soil, truck loading, and internal pressure loading;
3. Material properties—steel strength, concrete strength, and concrete density; and
4. Design data—concrete cover over reinforcement, diameter of reinforcement, and minimum spacing of reinforcement.

Only the span, rise, and depth of fill have to be given as input data. Specification of additional input data is optional with the user. Standard values are used when specific input data are omitted.

The program has the following limitations:

1. Only single-cell culverts can be considered;
2. The range of burial depth permitted is 2.0 to 100.0 ft;
3. The range of spans permitted is 3.0 to 12.0 ft;
4. The range of rises permitted is 2.0 to 12.0 ft; and
5. Only those loading cases that are discussed below can be considered.

The limitations on the range of culvert sizes and maximum burial depth are arbitrary and easily modified, but modification of the other limitations listed above would require major programming changes.

Design

The design capabilities of the program are based on the ACI ultimate strength design method. The area of required tension steel is selected by taking bending moment, axial forces, and cracking control into account, and the shearing stresses are checked. Welded-wire fabric will be used in the standard designs; therefore, in addition to the area of steel that is required, the maximum wire spacing that is consistent with controlled cracking is computed. However, the design produced by the program is also valid for culverts reinforced with bar reinforcing, provided the correct yield strength is input.

The following limitations apply to the design in the program:

1. Only transverse reinforcing is selected;
2. Anchorage lengths are not computed;
3. The program does not design wall thicknesses;
4. The present version of the program does not design shear reinforcement, but it does print a message when shear reinforcement is required; and
5. Maximum wire spacing is determined based on the assumption that a single layer of reinforcing is to be used for each of the reinforcing locations.

Cost

When the design of a culvert is complete, the volume of concrete and steel used in the design is computed. The cost per unit length of culvert is determined based on input unit costs for materials. Only material costs are considered; consequently, other costs such as transportation and installation must be added to determine the cost in place.

STRUCTURAL CRITERIA FOR ANALYSIS AND DESIGN

Notation

The notation used in this section is defined below.

A_s = area of steel;
b = width of unit strip (12 in.);
d = depth from extreme compression fiber to centroid of tension reinforcement;
d_f = depth of fluid in culvert;
f'_c = compressive strength of concrete;
f_{DL} = load factor for dead load;
f_{LL} = load factor for live load;
f_s = stress in reinforcement at service loads;
f_y = yield stress of reinforcing steel;
h = height of fill;
H_h = horizontal length of haunch;
H_v = vertical length of haunch;
L_l = length of distributed wheel load along span;
m = $f_y/0.85\, f'_c$;
M_u = ultimate design moment;
P_H = $0.5\, H_v\, H_h\, \gamma_c$;
P_u = ultimate design axial force (positive for compression);
R = rise;
s_ℓ = spacing of longitudinal wires;
S = span;
S' = $S + t_S$;
t_b = distance from centroid of tension steel to outermost concrete tension fiber;
t_B = thickness of bottom slab;
t_c = thickness of concrete cover over reinforcing steel;
t_S = thickness of side wall;
t_T = thickness of top slab;
V_u = ultimate design shear;

w_T = load intensity on top slab;
w_S = load intensity on side wall;
w_B = load intensity on bottom slab;
w_{ST} = load intensity on side wall at top;
w_{SB} = load intensity on side wall at bottom;
w_{rL} = reaction intensity at left;
w_{rR} = reaction intensity at right;
α = coefficient for lateral soil pressure;
β = effective height coefficient;
γ_c = density of concrete;
γ_f = density of fluid (water);
γ_s = density of soil;
ρ = pressure head;
ξ = nondimensional fraction of S'; and
φ = capacity reduction factor.

Structural Arrangement

Figure 1 shows the structural arrangement. The top and bottom slabs may be different thicknesses, and the side walls may be a third as thick. At the user's option, linear haunches of arbitrary dimensions may be specified. The steel arrangement is shown, and the steel areas designated AS1, AS2, AS3, and AS4 are to be designed as well as the cutoff lengths L_T and L_B. Design forces are evaluated at the cross sections indicated; the design based on those forces is discussed in another section of this report.

Loadings

The loading cases that are analyzed are shown in Figure 2. The loadings are separated into 3 groups: permanent dead loads, additional dead loads, and live loads. Load cases 1, 2, and 3 are the permanent dead loads; load cases 4 and 5 are the additional dead loads; and load cases 6 through 19 are live loads. The distinction between permanent and additional dead loads is made so that maximum force effects may be evaluated. Additional dead loads are considered to be acting only when they tend to increase the particular design force under consideration.

In load cases 1, 2, and 4, the soil reaction is assumed to be uniformly distributed across the width of the culvert. Load cases 3 and 5 have no soil reaction on the bottom slab. In load cases 6 through 19, the soil reaction on the bottom slab is assumed to vary linearly across the width of the culvert. It is assumed that no soil reactions are imposed on the sides of the culvert.

Load cases 6 through 19 are truck loadings. Load cases 6 through 12 are for an AASHO truck, and load cases 13 through 19 are for the truck loading required on Interstate Highways. Depending on the culvert span and depth of burial, as many as 7 load cases are used to simulate different positions of a wheel load as a truck traverses the culvert. The 7 cases are obtained by selecting different values of the parameter ξ in Figure 2f. The truck load design force that is selected at each section is the maximum force that occurs at that section under any of the truck loadings.

The length of the distribution of the wheel load in the direction of the span (length L_1 in Fig. 2f) is determined in accordance with the AASHO standards (1); however, a modification of the width of the AASHO distribution is used in the direction of the axis of the culvert. The maximum width over which the load from a truck is distributed is the width of 1 lane, i.e., 10 ft. This modification is made because distribution of loads along the length of the culvert will be discontinuous at the joints between culvert segments, and, with multiple traffic lanes over more than 1 culvert segment, the modified load intensity represents a maximum design condition. Thus, the length of the culvert does not affect the design requirements.

The AASHO standards (1) allow the use of 70 percent of the soil weight in culvert design and allow the designer to neglect truck loads when the depth of overburden exceeds 8.0 ft. However, they allow this reduction in load on the presumption that the concrete design will be in accordance with the AASHO working stress design approach, which leads to conservative steel stresses. In the method described here, the ACI ultimate strength design approach is used rather than the AASHO working stress design approach; consequently, 100 percent of the weight of the soil is used, and truck loads are considered for all overburden depths.

Method of Analysis

Structural analysis is performed by using the stiffness matrix method. A 1-ft slice of the culvert is analyzed as a 4-member frame (Fig. 1). For each member of the frame, the flexibility matrix is determined and inverted to obtain the member stiffness matrix. The member stiffness matrices are then assembled into a structure joint stiffness matrix, a joint load matrix is assembled, and conventional methods of matrix analysis are employed.

For simplicity, the fixed-end force terms and flexibility coefficients for a member with linearly varying haunches are determined by numerical integrations. Analytic integration is possible, but the algebraic expressions that result are cumbersome. The trapezoidal rule with 50 integration points per member is used, and a sufficiently high degree of accuracy is obtained.

Method of Design

The design procedure consists of selecting the steel that is required to resist the design bending moment and axial force, checking for crack control, and checking shear stresses. The wall thicknesses and haunch geometry are input parameters that are selected by the designer. The equation that is used for steel selection is based on the ACI ultimate strength design approach for combined bending and axial compression where the cross section is proportioned so that its ultimate strength is governed by the tension steel. Three-quarters of the steel corresponding to balanced conditions for bending alone is the maximum percentage of steel that is permitted.

Design forces resulting from the design loads multiplied by load factors are evaluated at the cross sections shown in Figure 1. The load factors are input parameters that may be specified by the designer; if they are not specified, load factors of 1.5 and 2.2 are used for dead loads and live loads respectively. The maximum design forces are obtained by summing the permanent dead load forces, the additional dead load forces when they tend to increase the design force, and the maximum force resulting from the live load cases.

The four steel areas designated AS1, AS2, AS3, and AS4 in Figure 1 are designed. The area AS1 is the maximum of the steel areas required to resist M_4 (i. e., the moment at the cross section labeled M_4, Fig. 1), M_5, M_7, or M_8. AS2 is designed to resist M_1, AS3 is designed to resist M_{11}, and AS4 is designed to resist M_6. V_3 (i. e., the shear at the cross section labeled V_3, Fig. 1) is used to check the shear stress in the top slab, the maximum of V_5 and V_7 is used to check the shear stress in the side wall, and V_9 is used to check the shear stress in the bottom slab. Moments M_2, M_3, and M_4 are used to determine the theoretical cutoff length L_T for AS1 in the top slab; and moments M_8, M_9, and M_{10} are used to determine the theoretical cutoff length L_B for AS1 in the bottom slab. Linear interpolation or extrapolation is used to determine the point at which the negative moment envelope is zero.

The following ultimate strength design formula is used to select the bending reinforcement:

$$A_s = \frac{bd}{m} - \sqrt{\left(\frac{bd}{m}\right)^2 - \left[\frac{2b\ M_u}{\varphi\ f_y\ m} - \frac{P_u\ bd}{\varphi\ f_y\ m} + \left(\frac{P_u}{\varphi\ f_y}\right)^2\right]} \tag{1}$$

The derivation of Eq. 1 is given below:

1. Figure 3 shows the forces acting on the cross section of a reinforced concrete flexural member at ultimate strength conditions when subjected to flexure plus axial compression.

2. Writing equilibrium of the forces in the vertical direction and imposing a capacity reduction factor leads to

$$P_u = \varphi(0.85\, f_c'\, ba - A_s f_y) \tag{2}$$

3. Writing moment equilibrium about the point x = a/2 leads to

$$M_u - P_u\left(\frac{d - a}{2}\right) = \varphi A_s f_y\left(d - \frac{a}{2}\right) \tag{3}$$

4. Solving Eq. 2 for a, substituting the result into Eq. 3, and rearranging terms give

$$A_s^2 - \frac{2bd}{m} A_s + \left[\frac{2b\,M_u}{\varphi f_y m} - \frac{P_u\, bd}{\varphi f_y m} + \left(\frac{P_u}{\varphi f_y}\right)^2\right] = 0 \tag{4}$$

5. Solving Eq. 4 for A_s gives Eq. 1.

The crack-control criterion that is used is somewhat more conservative than the crack-control provision given in the ACI code. It is based on tests by Lloyd, Rejali, and Kesler (3) of slabs reinforced with welded-wire fabric. Essentially, the research determined that the semi-empirical equation presented by Gergely and Lutz (4) may be used for slabs reinforced with smooth and deformed welded-wire fabric.

Using the Gergely and Lutz equation leads to the following requirement for the stress in the reinforcement when a single layer of reinforcement is used and the maximum permissible crack width at service load levels is 0.01 in.:

$$f_s \leq \frac{65}{\sqrt[3]{t_b^2\, s_\ell}} + 5 \tag{5}$$

The derivation of Eq. 5 is given below:

1. The semi-empirical equation proposed by Gergely and Lutz (3) for relating maximum crack width to other design parameters is

$$w_b = 0.091 \times 10^{-3} \sqrt[3]{t_b A}\, (f_s - 5)\, R \tag{6}$$

where $R = (h - kd)/[(1 - k)d]$; f_s = reinforcing steel stress, in ksi; t = thickness of slab; t_b = distance from bottom of slab to centroid of tension reinforcing; and A = area of concrete surrounding one bar or wire. For slabs with a single layer of reinforcing, $A = 2t_b s_\ell$.

2. Maximum crack width is limited to 0.01 in. at working stress; thus, w_b = 0.01 in. when $f_s = f_y\varphi/\text{avg load factor}$.

3. R_{max} = 1.34 is used for typical culvert slabs.

4. Then,

$$0.01 = 0.091 \times 10^{-3} \sqrt[3]{t_b\, 2t_b\, s_\ell}\, (f_s - 5)\,(1.34) \tag{7}$$

$$f_s - 5 = \frac{65}{\sqrt[3]{t_b^2\, s_\ell}}$$

which is the same as Eq. 5.

5. Equation 5 is compared to ACI crack-control criteria for ordinary exterior exposure (w_b = 0.012).

$$\max f_s = \frac{145}{\sqrt[3]{d_c A}}$$

$$\max f_s = \frac{145}{\sqrt[3]{2t_b^2 s_\ell}} \qquad (8)$$

$$\max f_s = \frac{115}{\sqrt[3]{t_b^2 s_\ell}}$$

Correction is made for reduction of maximum crack width from 0.012 to 0.01 in.

$$\max f_s = \frac{96}{\sqrt[3]{t_b^2 s_\ell}} \qquad (9)$$

The conclusion is that max f_s obtained by ACI criteria is significantly higher than max f_s obtained from the Gergely and Lutz equations for use with typical slabs.

Shear reinforcing is not required if

$$\frac{V_u}{\varphi bd} \le 2\sqrt{f_c'} \qquad (10)$$

where $\varphi = 0.85$ and b and d are as given above. Equation 10 is obtained from the requirements of ACI 318-71 (2, paragraphs 11.2.1 and 11.4.1).

Assumptions and Limitations

Although the design and analysis procedures that were developed in this work are intended to be as general as possible, there are inherent limitations to the applicability of the design program due to the assumptions that were made in developing the design procedure. The major design assumptions are given below:

1. The moments M_1 and M_{11} (Fig. 1) always cause tension on the inside face of the culvert wall, and the moments M_4, M_5, M_7, and M_8 always cause tension on the outside face of the culvert wall.
2. Critical sections for shear and moment do not occur within the haunch.
3. Based on the notation in Figure 1, SPAN $\ge 4(t_B - t_c) + 2H_h$, SPAN $\ge 4(t_T - t_c) + 2H_h$, and RISE $\ge 2(t_s - t_c) + 2H_v$.
4. A single layer of reinforcement is used.
5. For welded-wire fabric made of smooth wire, the maximum cross-wire spacing is 6 in. For welded-wire fabric made of deformed wire, there is no cross-wire spacing limitation.

Assumptions 1, 2, and 3 are valid for culverts with "normal" proportions; however, for unusual conditions where some of the assumptions are violated, application of the design procedure may give erroneous results. For example, if very flat haunches are used, the critical section for shear or moment or both may lie within the haunch, and unconservative results would be obtained. The designer-user should be aware of those assumptions so that the design program is not used for cases where the assumptions are violated.

Assumptions 4 and 5 affect the crack-control criterion. If more than one layer of reinforcing is used, the wire spacing computed by the program is overly conservative. If smooth welded-wire fabric is used with cross-wire spacing greater than 6 in., the longitudinal wire spacing that is computed may be unconservative.

The conclusions of Lloyd, Rejali, and Kesler (3) state that welded smooth-wire fabric and welded deformed-wire fabric are equally effective for crack control in slabs. However, it is well established that cross-wire spacing influences the effectiveness of welded smooth-wire fabric for crack control. Because no limits for cross-wire spacing are given by Lloyd, Rejali, and Kesler, the above limitation restricting the maximum cross-wire spacing to 6 in. requires further confirmation.

COMPUTER PROGRAM

General Description

The program using the design method presented in this report was written in FORTRAN IV and implemented on an IBM 360 model 65 computer. The input data requirements for the program are flexible because many data are optional. The amount of input data for the design of a particular culvert ranges from a minimum of 3 cards to a maximum of 15 cards; standard values for optional input data are assumed if specific data are not input by the user.

The output data consist of an echo print of the input data and a 1-page summary of the design. (Figs. 5 and 7 show typical designs obtained from the program.) The first line of output identifies the culvert size and the depth of overburden. These are required data items and must be supplied to the program by the user. The material properties, soil data, loading data, and concrete data are optional data items; when they are not supplied by the user, values are assigned by the program. The reinforcing steel data and the weight and cost data are generated by the program.

Standard Designs

The program has been used to generate data for culverts that will be proposed for standards and incorporated in a specification by the ACPA Technical Committee. Table 1 gives the standard sizes that have been designed.

In Table 1, "span" and "rise" are as shown in Figure 1, and the column headed "thickness" applies to top slab, side walls, and bottom slab. Also, the proposed standard sizes have 45-deg haunches with a leg dimension equal to the wall thickness. Designs were made for each standard size at many burial depths; the depth of overburden was increased from 2.0 to 6.0 ft in 1.0-ft increments, and then increased in 2.0-ft increments until a depth was reached where shear reinforcing was required. Designs were made for culverts with no truck load, AASHO HS20 truck load, and Interstate loading. About 1,200 designs have been generated for the ACPA Technical Committee, and in every design the area of steel designated AS4 in Figure 1 was not required; therefore, the standard culverts that are included in the specification may not have AS4.

Special Designs and Parameter Studies

The program can be used for designing or evaluating special nonstandard culverts. Many geometric quantities may be varied including the span; rise; depth of overburden; thickness of the top, bottom, and side walls; horizontal and vertical haunch dimensions; and thickness of concrete cover over the reinforcement. Also, steel and concrete strengths can be changed and soil parameters can be varied. These freedoms in describing a problem allow the user to design a nonstandard culvert for a special condition or to evaluate the adequacy of a proposed design. Further, by making several runs, the program can be used to evaluate the maximum or minimum burial depths or both that a given culvert design can sustain.

Another application of the program is its use in performing parameter studies. Often, a designer would like to determine how changing one or more parameters affects the final design, particularly the cost of the design. Making several runs and varying a particular parameter allow the impact of that parameter on the design to be evaluated. For example, the program could be used to study how cost is affected by wall thickness, and the designer could readily establish the wall thickness that optimizes the culvert cost.

Sample Problem 1

Sample problem 1 demonstrates the use of the program for the design of a culvert when all input data are specified. Figure 4a shows the culvert geometry, and Figure 4b

Figure 1. Structural arrangement and location of design forces.

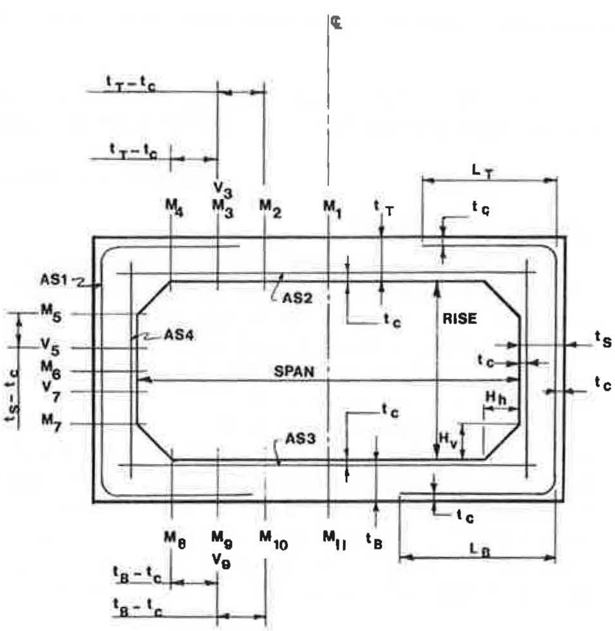


Figure 2. Load cases.

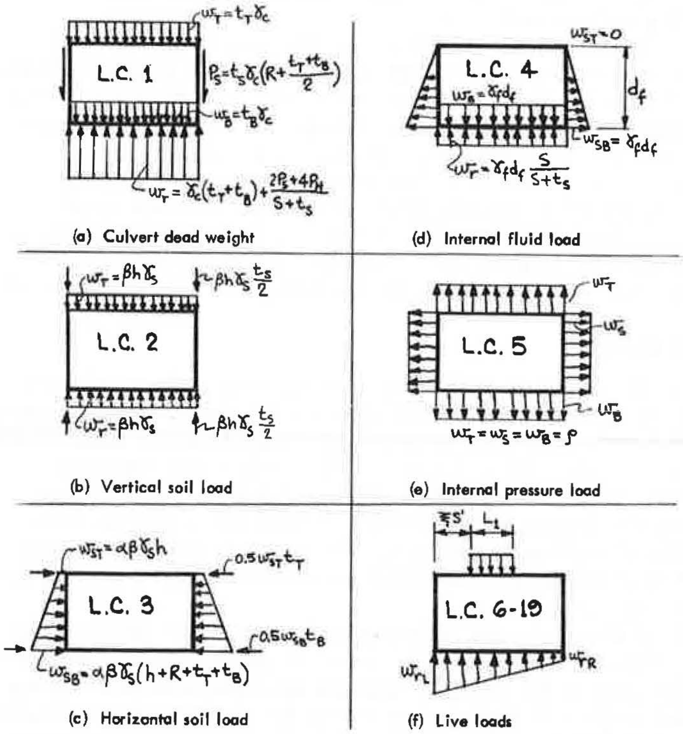


Figure 3. Forces on cross section.

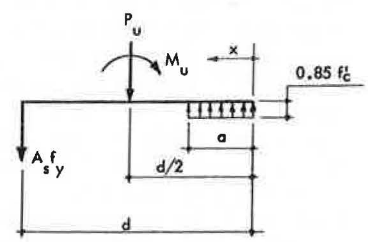
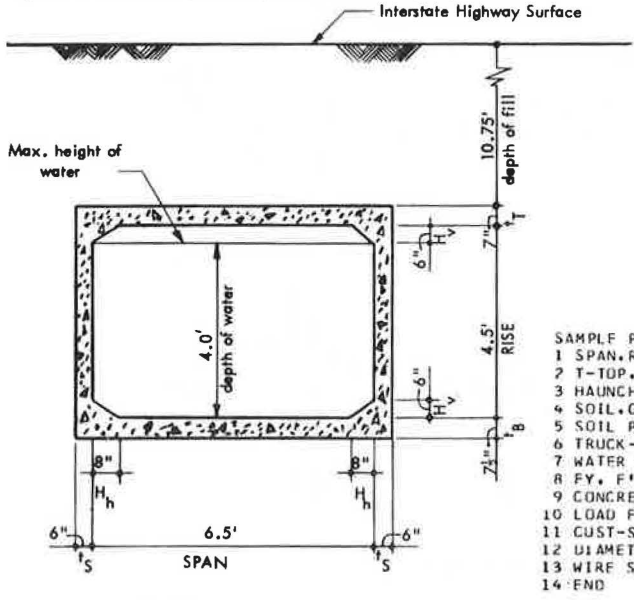


Table 1. Standard sizes.

Span (ft)	Thickness (in.)	Rise (ft) 2	3	4	5	6	7	8	9	10
3	4	X	X							
4	5	X	X	X						
5	6		X	X	X					
6	7		X	X	X	X				
7	8			X	X	X	X			
8	8			X	X	X	X	X		
9	9				X	X	X	X	X	
10	10				X	X	X	X	X	X

Figure 4. Geometry and input data for sample problem 1.



(a) Culvert Geometry

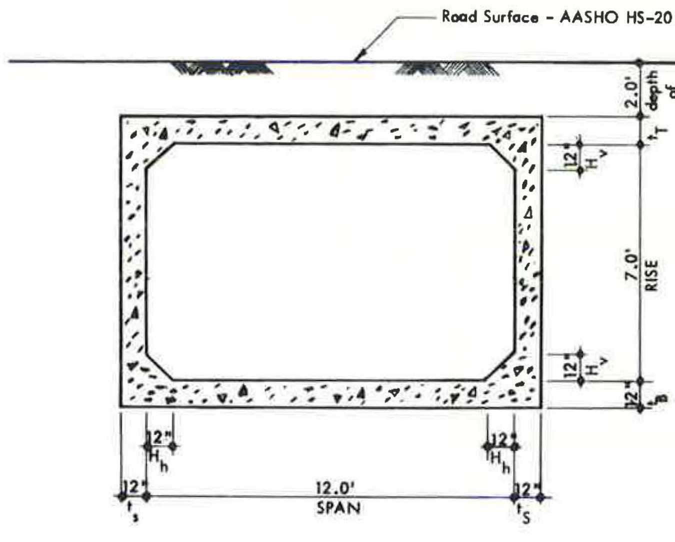
```
SAMPLE PROBLEM 1 - COMPLETE INPUT DATA
 1 SPAN.RISE.DEPTH          6.500      4.500      10.750
 2 T-TOP.T-BOT.T-S          7.000      7.500       6.000
 3 HAUNCH-HH.HV             8.000      6.000
 4 SOIL.CONC.WATER        110.000    145.000      62.400
 5 SOIL PARM                0.400      1.100       0.0
 6 TRUCK-INTERSTATE         4.000
 7 WATER DEPTH.PRESS        4.000      0.0
 8 FY. F'C                 60.000      4.500
 9 CONCRETE COVER           1.250
10 LOAD FACTORS             1.400      1.700
11 COST-STEEL.CONC          0.135      6.250
12 DIAMETERS                0.250      0.375       0.375       0.250
13 WIRE SPACING             3.000      3.000       0.0         0.0
14 END
```

(b) Echo Print of Input Data

Figure 5. Output for sample problem 1.

```
6.5 FT. X  4.5 FT. PRECAST CONCRETE CULVERT WITH 10.750 FT. OF OVERBURDEN
*************************************************************************

M A T E R I A L   P R O P E R T I E S
-------------------------------------------------------------------------
  STEEL - MINIMUM SPECIFIED YIELD STRESS, KSI                      60.000
  CONCRETE - SPECIFIED COMPRESSIVE STRENGTH, KSI                    4.500

S O I L   D A T A
-------------------------------------------------------------------------
  UNIT WEIGHT, PCF                                                110.000
  RATIO OF LATERAL TO VERTICAL PRESSURE                             0.400
  EFFECTIVE HEIGHT COEFFICIENT                                      1.100

L O A D I N G   D A T A
-------------------------------------------------------------------------
  LOAD FACTOR - DEAD LOAD                                           1.400
  LOAD FACTOR - LIVE LOAD                                           1.700
  TRUCK LOAD.                               INTERSTATE OR AASHO HS-20
  UNIFORM INTERNAL PRESSURE, PSI                                    0.0

C O N C R E T E   D A T A
-------------------------------------------------------------------------
  TOP SLAB THICKNESS, IN.                                           7.000
  BOTTOM SLAB THICKNESS, IN.                                        7.500
  SIDE WALL THICKNESS, IN.                                          6.000
  HORIZONTAL HAUNCH DIMENSION, IN.                                  8.000
  VERTICAL HAUNCH DIMENSION, IN.                                    6.000
  CONCRETE COVER OVER STEEL, IN.                                    1.250

R E I N F O R C I N G   S T E E L   D A T A
-------------------------------------------------------------------------
                                                    MIN.          MAX.
                                      AREA          WIRE          WIRE
              LOCATION               SQ. IN.       SPAC'G        SPAC'G
                                     PER FT.        IN.           IN.
  -----------------------------------------------------------------------
     TOP SLAB    - INSIDE  FACE      0.303         3.0           3.6
     BOTTOM SLAB - INSIDE  FACE      0.315         2.0*          3.6
     SIDE WALL   - OUTSIDE FACE      0.185         3.0           5.5
     SIDE WALL   - INSIDE  FACE      0.0           2.0*          0.0
  -----------------------------------------------------------------------
     *PROGRAM ASSIGNED VALUE

  THE SIDE WALL OUTSIDE FACE STEEL IS BENT AT THE CULVERT CORNERS AND
  EXTENDED INTO THE OUTSIDE FACE OF THE TOP AND BOTTOM SLABS.  THE
  THEORETICAL CUT-OFF LENGTHS MEASURED FROM THE BEND POINT
  ARE 13.2 AND 10.6 IN. AT THE TOP AND BOTTOM RESPECTIVELY.
  ANCHORAGE LENGTHS MUST BE ADDED.

W E I G H T   A N D   C O S T   D A T A
-------------------------------------------------------------------------
  WEIGHT OF CULVERT, KIPS/FT.                                       2.090
  WEIGHT OF STEEL, LB./FT. OF CULVERT LENGTH                       38.154
  UNIT COST OF CONCRETE, $/TON                                      6.250
  UNIT COST OF STEEL, $/LB.                                         0.135
  COST OF STEEL, $/FT. OF CULVERT LENGTH                            5.151
  COST OF CONCRETE, $/FT. OF CULVERT LENGTH                         6.412
  TOTAL COST, $/FT. OF CULVERT LENGTH                              11.563
```

Figure 6. Geometry and input data for sample problem 2.



(a) Culvert Geometry

```
SAMPLE PROBLEM 2 - MINIMUM DATA
 1 SPAN.RISE.DEPTH        12.000      7.000      2.000
99 END OF DATA
```

(b) Echo Print of Input

Figure 7. Output for sample problem 2.

```
12.0 FT. X  7.0 FT. PRECAST CONCRETE CULVERT WITH  2.000 FT. OF OVERBURDEN
******************************************************************************

M A T E R I A L   P R O P E R T I E S
------------------------------------------------------------------------------
  STEEL - MINIMUM SPECIFIED YIELD STRESS, KSI                          65.000
  CONCRETE - SPECIFIED COMPRESSIVE STRENGTH, KSI                        5.000

S O I L   D A T A
------------------------------------------------------------------------------
  UNIT WEIGHT, PCF                                                    120.000
  RATIO OF LATERAL TO VERTICAL PRESSURE                                 0.330
  EFFECTIVE HEIGHT COEFFICIENT                                          1.000

L O A D I N G   D A T A
------------------------------------------------------------------------------
  LOAD FACTOR - DEAD LOAD                                               1.500
  LOAD FACTOR - LIVE LOAD                                               2.200
  TRUCK LOAD,                                                     AASHO HS-20
  UNIFORM INTERNAL PRESSURE, PSI                                          0.0

C O N C R E T E   D A T A
------------------------------------------------------------------------------
  TOP SLAB THICKNESS, IN.                                              12.000
  BOTTOM SLAB THICKNESS, IN.                                           12.000
  SIDE WALL THICKNESS, IN.                                             12.000
  HORIZONTAL HAUNCH DIMENSION, IN.                                     12.000
  VERTICAL HAUNCH DIMENSION, IN.                                       12.000
  CONCRETE COVER OVER STEEL, IN.                                        1.000
  WIRE DIAMETER USED FOR COMPUTING DEPTH OF STEEL, IN.                  0.600

R E I N F O R C I N G   S T E E L   D A T A
------------------------------------------------------------------------------
                                                          MIN.          MAX.
                                             AREA         WIRE          WIRE
              LOCATION                      SQ. IN.      SPAC'G        SPAC'G
                                            PER FT.       IN.           IN.
      ------------------------------------------------------------------------
      TOP SLAB     - INSIDE  FACE            0.494        2.0*          9.3
      BOTTOM SLAB  - INSIDE  FACE            0.354        2.0*          7.7
      SIDE WALL    - OUTSIDE FACE            0.362        2.0*          8.3
      SIDE WALL    - INSIDE  FACE            0.0          2.0*          0.0
      ------------------------------------------------------------------------
      *PROGRAM ASSIGNED VALUE

  THE SIDE WALL OUTSIDE FACE STEEL IS BENT AT THE CULVERT CORNERS AND
  EXTENDED INTO THE OUTSIDE FACE OF THE TOP AND BOTTOM SLABS.  THE
  THEORETICAL CUT-OFF LENGTHS MEASURED FROM THE BEND POINT
  ARE 36.9 AND 37.7 IN. AT THE TOP AND BOTTOM RESPECTIVELY.
  ANCHORAGE LENGTHS MUST BE ADDED.

W E I G H T   A N D   C O S T   D A T A
------------------------------------------------------------------------------
  WEIGHT OF CULVERT, KIPS/FT.                                           6.447
  WEIGHT OF STEEL, LB./FT. OF CULVERT LENGTH                           94.776
```

shows the echo print of the input data for the problem. Each line in this echo print gives the information that was input on a data card. The first card is a title card for the problem, and the remainder of the cards are data cards that contain a comment field that is convenient to use to identify the data items on the card. Data cards 1, 2, and 3 define the culvert geometry; the comment field on each card identifies the data items. Data card 4 gives the densities of the soil, concrete, and water. Data card 5 gives the soil parameters to be used for the analysis: 0.400 is the coefficient for lateral soil pressure, 1.100 is the effective height coefficient, and 0.0 is a code that indicates that the lateral earth pressure will be considered as a permanent dead load. Data card 6 gives the code that indicates that the Interstate truck loading is to be considered. Data card 7 gives the depth of water, 4.00 ft, and the internal pressure, 0.0 psi. Data card 8 gives the yield strength of the reinforcing steel and the ultimate strength of the concrete. Data card 9 gives the concrete cover over the reinforcement, and data card 10 gives the load factors for dead load and live load respectively. Data card 11 gives the unit prices for steel and concrete in dollars per pound and in dollars per ton respectively. Data card 12 gives the reinforcement diameters that are to be considered for the design of the steel areas AS1, AS2, AS3, and AS4 in that order. Data card 13 gives the minimum wire spacing that will be allowed for the 4 steel areas; the spacings that are printed as 0.0 indicate that the minimum wire spacing was not specified for those steel areas. Data card 14 indicates that the end of the input stream has been reached.

Figure 5 shows the summary of the design that was obtained for sample problem 1.

Sample Problem 2

Sample problem 2 demonstrates the use of the program with minimum input data. Figure 6a shows the culvert geometry for this design, and Figure 6b shows the echo print of the input data. Only the problem title card and 2 data cards are necessary; the first data card gives the span, rise, and depth of fill, and the second one indicates the end of the input stream. Figure 7 shows the design that was obtained for sample problem 2 and the standard values that are assumed for materials properties, soil data, loading data, and concrete data when those data are not input. All of the concrete data with the exception of the concrete cover over steel are determined as a function of the culvert span. The weight and cost data show only the weight of culvert and the weight of steel; because no unit costs were input, no culvert costs are determined.

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