FIELD PERFORMANCE OF REINFORCED CONCRETE PIPE

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The field performance of a full-scale reinforced concrete pipe in an embankment installation is described. Normal stresses were measured by specially designed stress cells placed in the soil and at the soil-pipe interface. Displacements in the soil were obtained by settlement plates, and the resulting data were used to calculate a settlement ratio that is in agreement with that anticipated for such an installation. In the pipe itself, diameter changes and strains in the concrete and reinforcing steel were monitored. Data were taken to investigate the response of the soil-pipe system to incremental increases in the height of cover and to the application of a live load under conditions of shallow cover. In general, the experimental measurements are mutually consistent, but they exhibit some differences from results predicted by a plane strain finite element model that utilizes soil parameters obtained primarily from uniaxial strain tests.

•DESPITE the availability of high-speed digital computers to provide essentially exact numerical solutions to idealized problems, the practicing engineer is often unable to use this information to evaluate with any degree of assurance the anticipated response of a given soil-structure system; this is due in large part to incorrect modeling of the physical phenomenon and inadequate determination of the proper values for the input parameters (particularly the stress-strain behavior of the soil). This unfortunate situation exists because, as a consequence of the costs involved, the profession has seldom been provided with the opportunity to adequately instrument and monitor the field performance of a full-scale installation. In recent years, however, the importance of measuring field performance and comparing it with results predicted from a mathematical model has been increasingly recognized, and several significant contributions have emanated from such investigations.

The field performance of a full-scale reinforced concrete pipe buried in an embankment installation is described here. Instrumentation was provided to measure the stresses at the soil-pipe interface and in the adjacent soil, the displacements in the soil above and below the pipe and in the free field, shape changes of the pipe, and strains along the inner and outer faces of the pipe and in the reinforcing cages. Experimental measurements are shown to be mutually compatible and in qualitative agreement with intuitive expectations based on engineering judgment. Typical values at discrete points in the soil-pipe system are compared quantitatively with results calculated by use of a plane strain finite element model, and soil parameters are determined from uniaxial strain tests and triaxial tests on the actual soils from the field installation. Although not yet fully realized, the goals of this study are to develop reliable procedures for predicting the field response of coupled soil-pipe systems.

FIELD EXPERIMENT

As shown in Figure 1, the experimental installation is located on the grounds of the Ohio Highway Transportation Research Center in East Liberty. A 60-in. inside diameter, Class IV, B wall concrete pipe (manufactured by the wet cast method) was installed in a positive projecting embankment condition with a cover of 25 ft. The required strength of the pipe was determined by means of the Marston-Spangler theory, and the pipe was installed in accordance with the specifications of the Ohio Department of Highways. The selected pipe size is the result of a compromise between the

Figure 1. Location of field site.

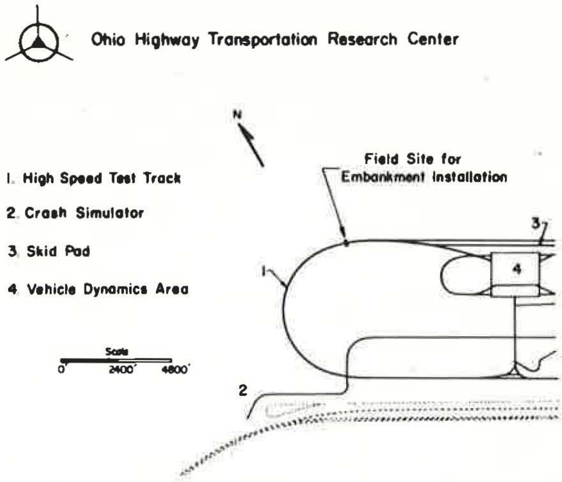


Figure 2. Log of soil boring at field site.

State of Ohio
Department of Highways
Testing Laboratory

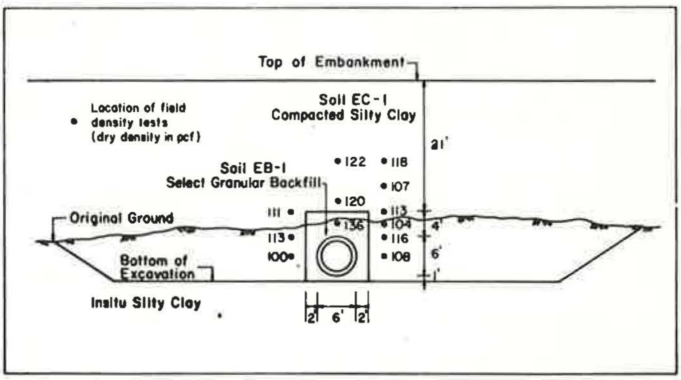
Log of Boring

Date Started 4-5-71 Sampler Type Truck Auger Water Elev. Immediate None Project Identification Logan-Union
Date Completed 4-5-71 After ___ Hours ___ Log-Uni-Ohio Hwy. Trans. Res. Center
Boring No ___ Station & Offset 360-00 50' Rt. Large Loop Surface Elev. ___ Concrete Pipe Res. Installation Site

Location of Pipe	Depth	Description	Fld No.	Lab No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	SHTL Class.
	0–6	Brownish-Gray Silty Clay	1	28794	2	1	5	31	61	53	27	29	A-7-6
	6–12	Brown Silty Clay	2	28795	3	2	5	27	63	42	79	23	A-7-6
	12–16	Gray Silty Clay	3	28796	4	4	7	33	52	38	16	21	A-6b
	16–20	Gray Silty Clay	4	28797	2	4	8	32	54	39	20	21	A-6b
	20	Bottom of Boring											

Depth scale: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36

Figure 3. Cross section of field installation.



smallest pipe for reasonable access of personnel and instruments and the largest pipe consistent with available height of cover, which was dictated by topography and economics. The installation consists of five 8-ft-long instrumented pipe sections (the middle one of which is most heavily instrumented) and several buffer sections at either side. It is located under a section of the 7.5-mile high-speed test loop at the Research Center and forms part of an access tunnel through the embankment. To achieve a 25-ft height of cover, it was necessary to excavate the existing ground prior to installation of the pipe; the width of this excavation at the base was more than 70 ft, thereby giving reasonable assurance that true embankment conditions were achieved. Figure 2 shows the log of one nearby soil boring, which indicates that (a) the soil conditions beneath the pipe were reasonably uniform and consisted essentially of silty clay and (b) the existing water table was below the proposed elevation of the pipe.

The field instrumentation was designed to measure stresses and displacements in the soil and in the pipe as well as at the soil-pipe interface. Total stress cells were employed to measure the normal stresses acting on the soil-pipe interface, at certain discrete points in the soil adjacent to the pipe, and in the free field; a detailed description of these cells and an assessment of their performance are given by Krizek et al. (2). In the pipe itself, horizontal and vertical diameter changes as well as variations in a number of chords were measured to an accuracy of 0.002 in. by mechanical extensometers, and an extensive set of strain readings was collected. Total displacements of the pipe and the soil and relative displacements between the soil and the pipe were monitored by means of settlement plates and the use of ordinary surveying methods. In-place unit weights of the soil were determined to provide information regarding the magnitude of the load increments and the density condition that controls the stiffness of the soil.

The installation of the test pipe was undertaken in June 1971 and was completed about 4 months later. The existing ground surface in the vicinity of the installation was excavated to a depth of about 1½ ft below the designated elevation of the pipe invert, as shown in Figure 3. Then, a select granular bedding material, designated as EB-1 and described in Figure 4, was placed under the pipe with an average thickness of about 1 ft and compacted by use of a small vibratory compactor. Instrumentation to measure stresses and displacements under the pipe was installed before the pipe sections were laid, and every effort was made to ensure that intimate contact was achieved between the pipe and the bedding. The backfilling operation started after all pipe sections were positioned, and, with occasional delays at various stages to install instrumentation, it continued up to a height of 4 ft above the top of the pipe. The same soil used for the bedding was used for the sidefill (about 1½ or 2 ft to either side of the pipe) and the fill above the pipe; each lift was about 1 ft thick before being compacted.

After this stage of the backfilling was completed, the embankment was constructed of the natural excavated soil, a silty clay designated as EC-1 and described in Figure 5. Heavy construction equipment was used to move and compact the soil in approximately 1-ft lifts until a cover of 25 ft was obtained. Densities of the compacted fill were measured frequently during the construction period, and typical data are shown in Figure 3. The operation was stopped every 4 ft to allow readings to be taken from the various types of instrumentation. In addition, the effect of a live load was studied when the pipe was under 4 ft of cover. During the first part of the installation, including most of the instrumentation and backfilling operations up to a height of cover of 12 ft, the weather was generally fair. A cover height of 16 ft was attained after a few weeks, but the remaining 9 ft of cover was not placed until about 4 months thereafter. This delay, although unanticipated, provided the opportunity to evaluate time effects to a limited extent.

STRESS DISTRIBUTIONS DUE TO EARTH LOADS

Ten 6-in.-diameter and eleven 10-in.-diameter total stress cells were installed at discrete points within the soil adjacent to the pipe and at the soil-pipe interface to obtain direct measurements of the average normal total stress (effective stress plus pore water pressure) acting at these points. Nine 6-in.-diameter cells were placed in the

Figure 4. Summary of test data for soil EB-1.

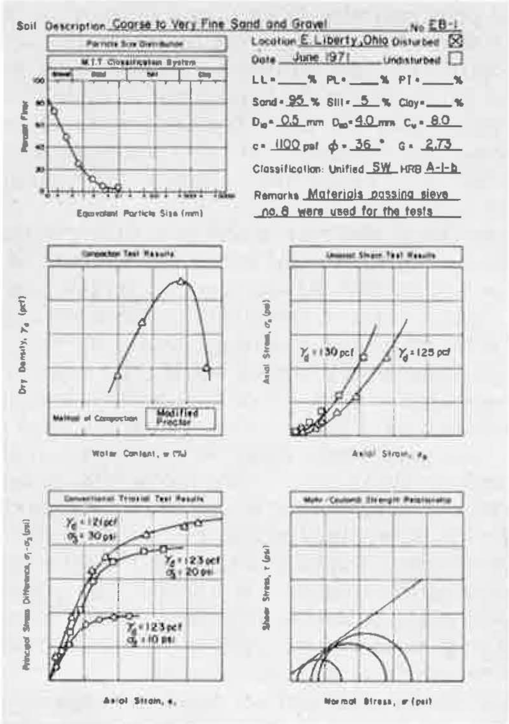
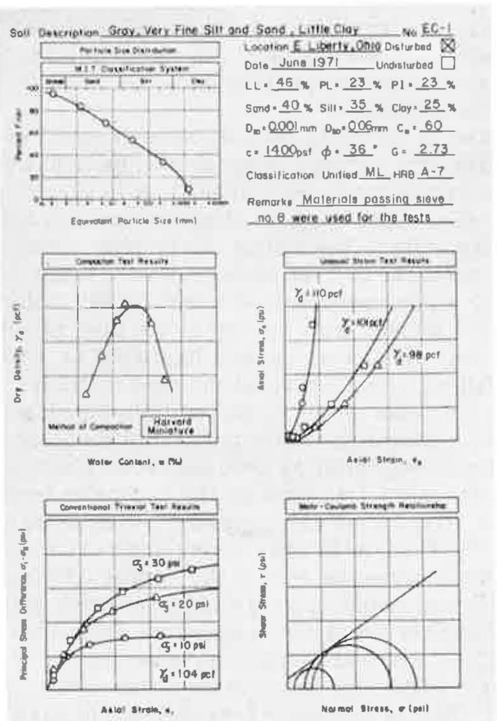


Figure 5. Summary of test data for soil EC-1.



wall of the principal pipe (E3); single cells were installed every 45 deg (alternately in two planes spaced at +30 in. from the midplane) around the pipe with two cells at the top. All cells were placed in recesses provided at the time the pipe was cast, and the surfaces of all cells are flush with the surface of the pipe. In addition, one 6-in.-diameter and two 10-in.-diameter cells were placed in the free field at the elevation of the pipe spring line and 15 ft from the pipe, and nine 10-in.-diameter cells were installed in the soil immediately adjacent to (within about $2\frac{1}{2}$ ft of) the top, bottom, and spring line of the pipe.

Data concerning the stress and temperature measurements were obtained from a digital readout voltmeter and converted to units of gauge pressure, and the resulting distributions of normal stresses acting on the pipe for various heights of cover are shown in Figure 6; these distributions have been drawn by fitting curves to the symmetrized data, which were obtained every 45 deg. The narrowness of the peak at the invert represents an attempt to satisfy the observed fact that the template used to shape the bedding was oversize, thereby providing for pipe contact with the compacted bedding over approximately a 10-deg arc. Since the total stress cells measured only normal stresses, no experimental data were obtained on shear stresses along the soil-pipe interface; because these shear stresses would, in general, not be zero, it follows that the complete stress distribution at the soil-pipe interface has not been obtained. Although the readings from symmetrically placed cells are reasonably similar, the resulting stress distributions shown in Figure 6 are different from that suggested in the classical Marston-Spangler approach. In general, the effect of time (4 months at 16 ft of fill) on the stresses at the soil-pipe interface caused increases on the order of 1 to 3 psi; variations in all the other cells ranged from about -5 psi to +3 psi, and there was no apparent pattern to the increases and decreases.

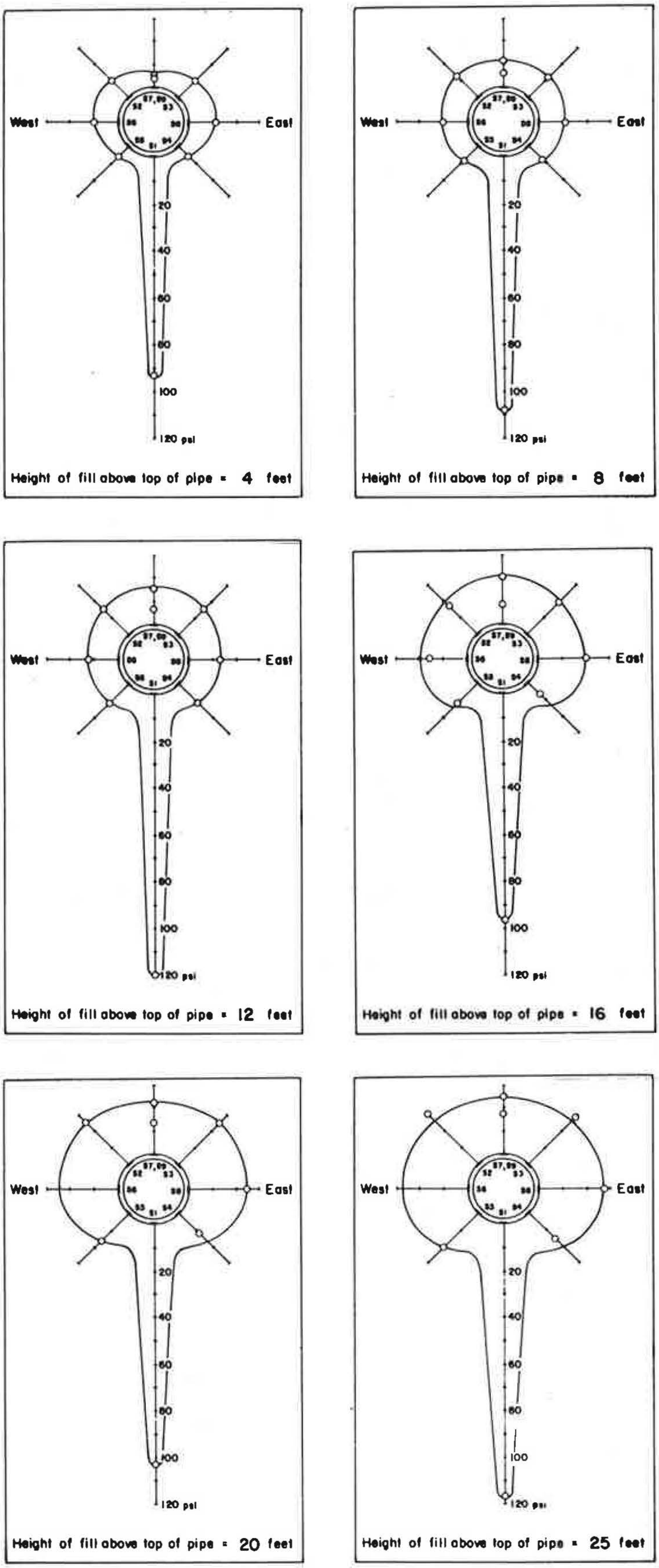
Only 2 of the 21 cells did not function properly. The failure of the cell 6 in. below the pipe was due to a damaged transducer, which was replaced in the field when the fill was about 4 ft above the top of the pipe; although this replacement was accomplished satisfactorily, the reference for this cell was lost, and all subsequent readings represent only a change in stress rather than absolute stress. After approximately 3 weeks (16 ft of cover) of satisfactory performance, one cell (S6) at the spring line ceased to function; the reason is not known.

In the case of the cells (S7 and S9) embedded in the crown of the pipe, there is a serious difference between supposedly identical readings; although no definite reason for this discrepancy can be advanced, it is probably due to the heterogeneity of the fill above the cells. The large readings of the cell (S1) at the bottom of the pipe are probably due primarily to the weight of the pipe acting over a small strip; for example, if the pipe is assumed to rest uniformly on a strip 6 in. wide, the weight of the pipe alone accounts for a normal stress of about 20 psi. Since the actual reading for the situation where the fill is even with the top of the pipe is somewhat greater than 50 psi, it is likely that the cell is resting on a high spot or a hard spot in the bedding material because the fill up to the crown of the pipe could not possibly account for the additional 30 psi. The normal stresses on the pipe increased more or less in proportion to the height of fill. The three cells in the soil directly above the pipe gave stresses that were approximately midway between those given by cells S7 and S9 at the top of the pipe. The stresses recorded by the two cells beneath the pipe attenuate with distance, as expected, and values are consistent with those registered by cell S1 at the bottom of the pipe. The four cells (two measuring vertical stresses and two measuring horizontal stresses) in the adjacent soil at the spring line of the pipe yield stresses that are both qualitatively and quantitatively consistent with those measured at the soil-pipe interface. The horizontal stress at the spring line decreases with distance from the pipe, as expected, and the vertical stress increases with distance from the pipe, probably because the relatively rigid pipe is carrying some of the load that would normally be distributed to the soil. Two of the three free field cells gave stresses that are very close to those calculated by use of the actual overburden of soil, but the third registers somewhat low.

STRESS DISTRIBUTIONS DUE TO LIVE LOADS

The effects of live loads due to heavy equipment passing over the pipe during the

Figure 6. Stress distribution at soil-pipe interface due to earth loads.



construction phase may be of considerable importance, especially when the depth of cover is relatively shallow; however, the effect of a surface load decreases rapidly with increasing depth. To evaluate the influence of construction loads on a pipe with a relatively shallow cover, a fully loaded (heaped) Caterpillar 631B 30-yd^3, rubber-tired tractor-scraper was used to load the pipe when it was covered with 4 ft of fill. Based on manufacturer's specifications, the load on each front wheel of the tractor-scraper was estimated to be about 39,000 lb; this load, in conjunction with a measured 3.0- by 1.5-ft wheel contact area, yields an assumed uniformly distributed static load of 8,700 psf or about 60 psi. The tractor-scraper was parked over the pipe at the two different locations shown in Figure 7, and stresses and deformations were recorded at four stages. The first measurements were made shortly before application of the load; the second and third recordings correspond to the load positioned directly above and 5 ft to the east of the test pipe; and a fourth reading of stresses and deformations was made after the load was removed from the vicinity of the pipe. The normal stress distributions corresponding to these stages are shown in Figure 8.

These data suggest three important points of interest. First, the surface load has a relatively small effect on the overall distribution of stresses around the pipe; except at the bottom of the pipe, the maximum stress increase due to application of the load is less than 5 psi in the extreme case, and it occurs at the pipe crown when the front wheels of the tractor-scraper are directly above the pipe. In general, the stress changes are almost uniformly distributed around the pipe; this is particularly important because a hydrostatic compression load at the soil-pipe interface is a very desirable type of loading from the viewpoint of minimizing the shear stresses that often cause failure of the pipe. This effect was also observed in the deformation measurements taken during these stages of loading; that is, there were no appreciable differences in the diameter changes in the various directions. The situation is a little different in the case where the load was offset by 5 ft from the axis of the pipe; the change in the distribution of stresses around the pipe is not as uniform as in the previous case, but the magnitudes of the changes are relatively small.

A second interesting point can be observed when the stress increase at the crown of the pipe is examined for these case where the load is directly above the pipe. A simple calculation based on the assumption of a homogeneous, isotropic, linearly elastic half-space subjected to the same surface load indicates that the vertical stress increase at a depth of 4 ft is about 7.3 psi, whereas the average normal stress increase measured by cells S7 and S9 in the crown of the pipe is approximately 4.6 psi. Furthermore, since the pipe is presumably stiffer than the volume of soil it replaces, this difference of 2.7 psi seems to be in the wrong direction—that is, the stress is transferred to the surrounding soil instead of being carried by the pipe. The decrease in the vertical diameter was measured to be about 0.03 in. when the tractor-scraper wheels were above the pipe. On the other hand, when the wheels were offset, the diagonal diameter in line with the tractor wheels decreased by only 0.01 in. In both cases it is difficult to evaluate the load transfer mechanism because the shear stresses at the soil-pipe interface are unknown.

The third relatively important conclusion concerns the apparent elastic behavior of the soil-pipe system. Figures 8a and 8d show that the stress distribution after the removal of the load is very much the same as that existing before the load was applied; that is, there are no residual stresses in the system. Although this behavior does not prove that the soil-pipe system is elastic, since strains were not measured, it does lend some support to the use of elastic theory, at least for short-term applications of static loads.

DISTRIBUTION OF DISPLACEMENTS

Information concerning the vertical displacements at discrete points in the soil-pipe system was obtained by use of settlement plates. A total of 22 settlement plates were installed at points in planes parallel and perpendicular to the longitudinal axis of the pipe, as shown in Figure 9. The extreme three plates to either side of the pipe in the transverse plane were intended to measure the free field response. All plates were

Figure 7. Position of tractor-scraper relative to pipe.

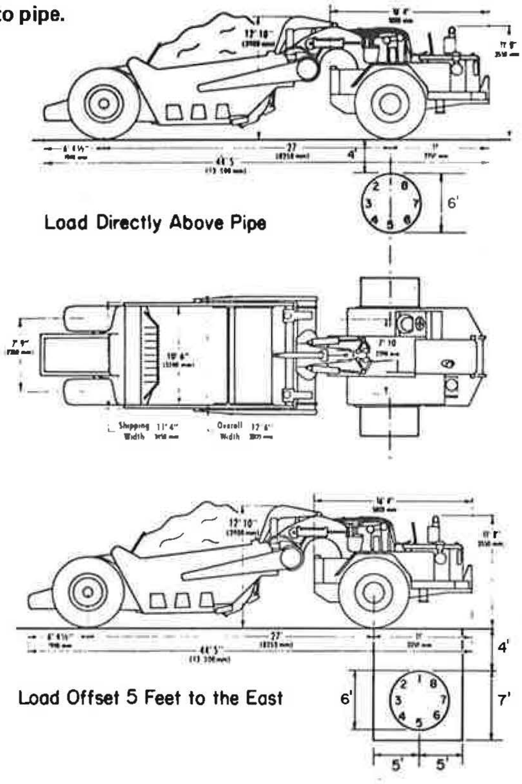
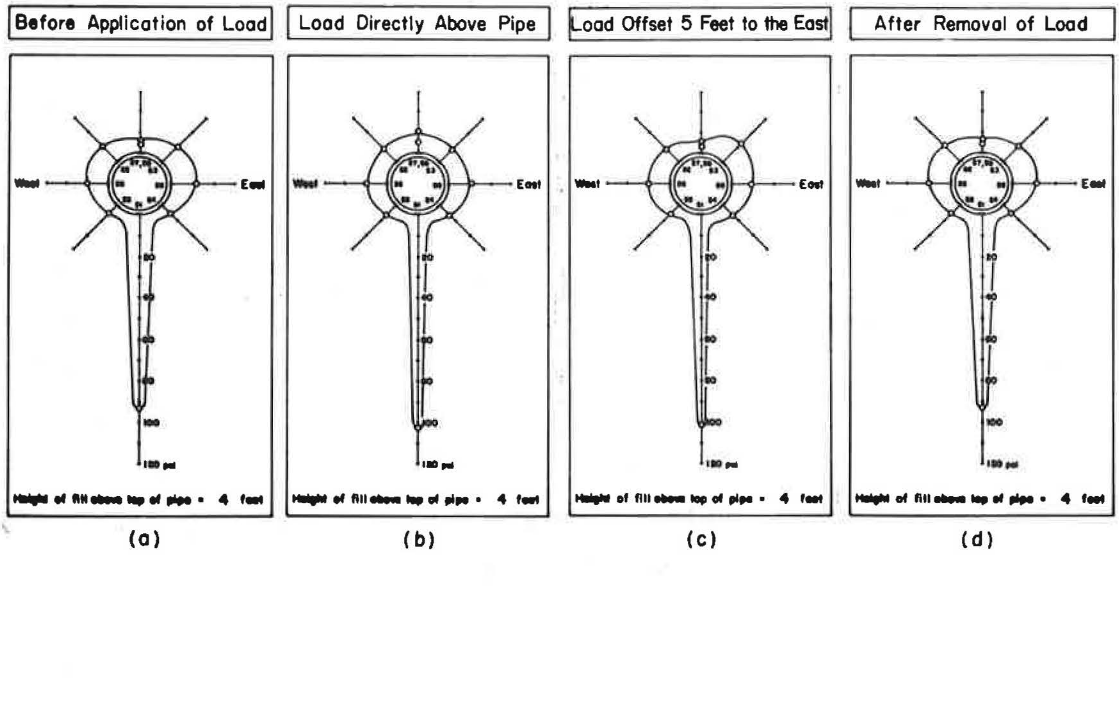


Figure 8. Stress distribution at soil-pipe interface with addition of live load.



placed in recesses that were excavated and filled with a few inches of a uniform sand to ensure proper seating; then the plates were covered with several inches of soil to hold them in position as the adjacent fill was placed. The vertical rods attached to the plates in the longitudinal plane passed through sleeves in the pipe wall, and these plates gave relative displacements between the plate and the pipe wall. The vertical rods fixed to the plates in the transverse plane passed through a casing to eliminate the frictional resistance of the soil on the rods and extended to the surface; ideally, these plates should measure absolute displacements, but the lack of a suitable long-term benchmark diminished the reliability of these measurements. A short-term benchmark was established by driving a steel pipe a few feet deep and several hundred feet from the installation, and some of the variations in the results are probably due to the use of this type of benchmark. Despite attempts to flag the area, the construction equipment may have caused some disturbances to the exposed extension rods of the settlement plates in the transverse plane; in several cases the extension rods were bent somewhat, and undetermined movements of the plates may have occurred. In one isolated instance, a large tractor-scraper apparently ran over an extension rod (a 1-in.-diameter pipe) and cut through its rubber tire, thereby disturbing the plate by some unknown amount.

Longitudinal Plane

Average relative displacements (taken after 25 ft of cover had been placed) of the settlement plates in the longitudinal plane are shown in Figure 10 as a function of the distance of the plates from the outer wall of the pipe. Overall trends and magnitudes indicate that (a) the relative displacements in the soil below the pipe are considerably higher than those above the pipe and (b) these relative displacements appear to attain essentially their maximum values at small distances from the pipe wall (on the order of a few feet). The former observation is consistent with the measured stresses shown in Figure 6. At 25 ft of cover the vertical stresses at the soil-pipe interface above and below the pipe are about 20 psi and 120 psi respectively; hence, it is logical to anticipate displacements below the pipe that are 5 to 10 times those above the pipe (due to the nonlinear behavior of the soil), and this is generally the order of magnitude shown in Figure 10. The latter observation regarding the rapid attenuation of relative displacements has been suggested by virtually all continuum models of soil-pipe interaction, although there admittedly has been little substantiating experimental evidence.

Transverse Plane

The settlement readings taken in a transverse plane are shown in Figure 11, which has been plotted by assuming that the original position of each individual settlement plate is at the position given by the curve of the preceding settlement plate for the appropriate height of fill; then, the changes in the settlement of each plate are plotted with reference to the established datum for each plate. For example, plate T3 was installed at the crown of the pipe; then, 5 ft of fill was placed over the crown and the plate settled approximately 0.08 ft, at which time plate T5 was installed with the 0.08-ft reading as a datum. Then, for instance, since the total cumulative settlement of plate T5 was measured to be 0.20 ft after 20 ft of fill was placed, the corresponding value plotted in Figure 11 is 0.28 ft (that is, 0.08 + 0.20 ft). Although there is some scatter in these data, they nevertheless give a general appreciation for the overall response of the system of settlement plates. As a consequence of the benchmark problems previously discussed, there is a degree of uncertainty regarding the absolute settlements of these plates; however, the relative displacements between plates T1 and T3 and the other plates in the installation are considered to be quite accurate. The difference between any two curves in Figure 11 represents the relative displacement between the corresponding two plates due to the soil placed above the higher plate.

The settlement ratio, r_{sd}, as defined by Spangler and Handy (3), is given by

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m} \qquad (1)$$

Figure 9. Locations of settlement plates.

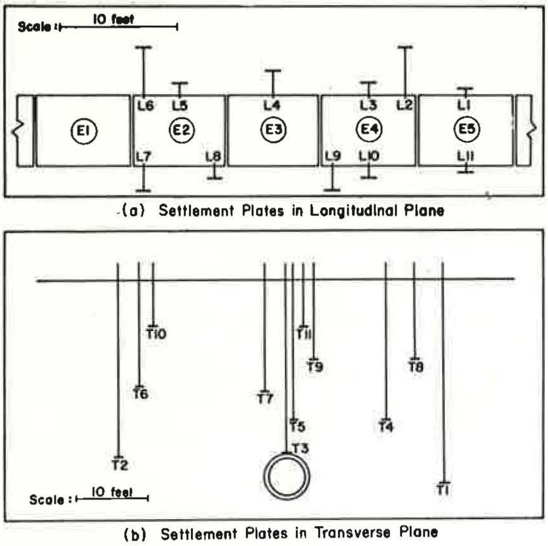


Figure 10. Relative displacements of settlement plates in longitudinal plane.

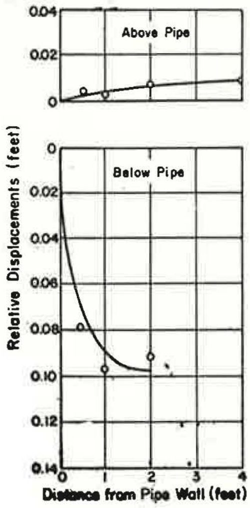


Figure 11. Cumulative displacements of settlement plates in transverse plane.

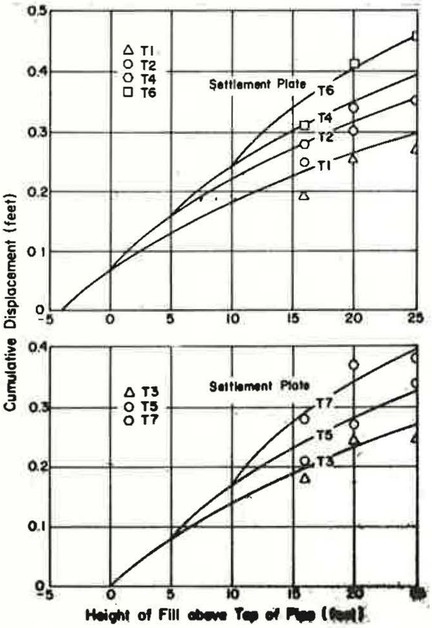
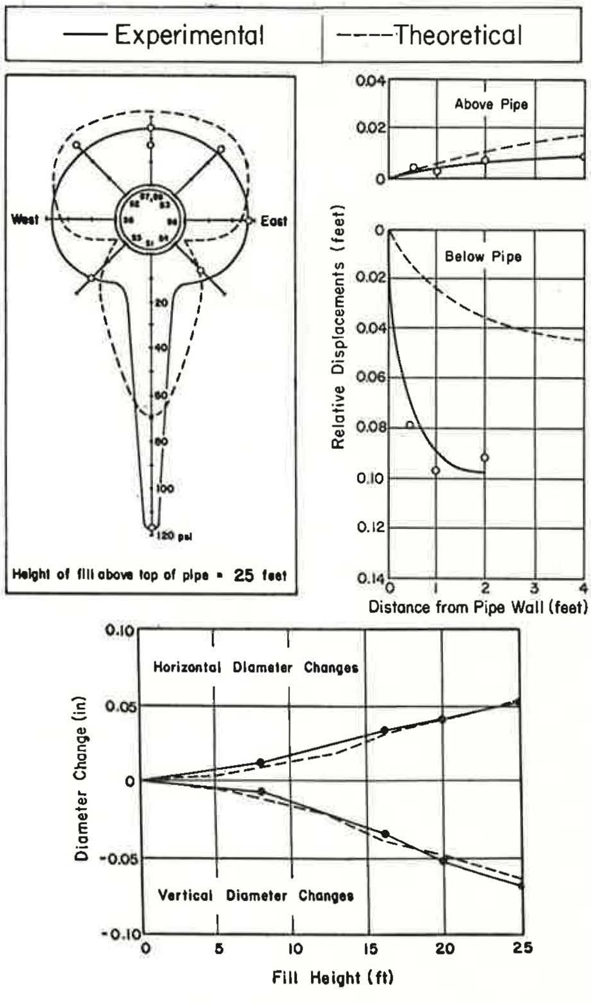


Figure 12. Comparisons between experimental and theoretical results.



where

s_m = compression of the exterior prisms of soil adjacent to the pipe,
s_g = settlement of the natural ground or compacted fill surface adjacent to the pipe,
s_f = settlement of the pipe into its bedding foundation, and
d_c = change in the vertical diameter of the pipe.

For the curves shown in Figure 11, based on the measurement of all settlements due to the fill above the crown of the pipe only, the magnitudes of the foregoing parameters for the completed installation are s_m = 0.09 ft, s_g = 0.23 ft, and (s_f + d_c) = 0.27 ft. The value of s_m is determined by taking the difference between the settlements of T1 and T2 caused by 25 ft of fill (0.06 ft) and multiplying by 1.5, because T1 is located 4 ft below the crown instead of 6 ft. The value of 0.23 ft for s_g is obtained by assuming that the absolute settlement of T1 caused by 25 ft of fill is approximately the same as it would be if it were located at the elevation of the pipe bottom. Finally, (s_f + d_c) is taken directly from the settlement of T3 under 25 ft of fill. Accordingly, the settlement ratio can be computed as

$$r_{sd} = \frac{(0.23 + 0.09) - 0.27}{0.09} = \frac{0.05}{0.09} = 0.56 \qquad (2)$$

which is reasonably representative and consistent with the soil and pipe conditions at this installation.

ANALYTICAL COMPARISONS

Some appreciation for the ability of a fairly sophisticated mathematical model to describe the field performance of this soil-pipe system can be obtained by comparing the experimental data at certain discrete points with calculated results. For this purpose a plane strain finite element program developed by Anderson (1) was used; the plane strain condition was justified on the basis of experimentally measured longitudinal strains in the pipe wall. The mathematical model of the pipe, including the mechanical properties of the concrete and reinforcing steel and an appropriate cracking mechanism, consists of 320 elements, and its ability to characterize the response of the pipe was validated by means of a series of load tests on pipes of different diameter, wall thickness, and reinforcement. This validated pipe model was then incorporated into a soil-pipe model, which consists of up to 257 additional soil elements, depending on the height of cover above the pipe, and requires the specification of appropriate mechanical properties for each element of soil. Idealized boundary conditions (no shear and no normal displacement) were used at the external boundaries (about 1 diameter below the pipe and 3 diameters to the side of the pipe), and the interface condition between the pipe and the soil was assumed to be full bond.

Piece-wise linear values for the modulus and Poisson's ratio of soils EB-1 and EC-1 were determined by a series of uniaxial strain tests and triaxial tests, typical data from which are shown in Figures 4 and 5. Since density was found to exert a significant influence on the stress-strain behavior of these soils, an attempt was made to prepare test specimens at three different densities (the maximum dry density determined from a standard Proctor test, a density 10 percent above this value, and a density 10 percent below this value). The uniaxial strain tests were performed on disc-shaped specimens (2.5 in. in diameter and 1.0 in. thick) in accordance with the standard loading schedule for consolidation tests; all specimens were saturated prior to testing, and each sample susceptible to swelling was subjected to a sufficient load (not included in the stress-strain data) to prevent any swelling. The triaxial tests were conducted by subjecting cylindrical specimens (2.5 in. in diameter and 5.0 in. long) at approximately optimum water content (as determined from a standard Procter compaction test) to a constant confining pressure and increasing the axial load incrementally; radial displacements were measured at seven discrete points on the boundary of each specimen by use of electronic distance-measuring probes. In all tests the stresses were expressed in terms of effective stresses, since specimens were partially saturated in the triaxial tests and pore pressures were allowed to dissipate completely in the uniaxial strain tests.

Although modulus relationships determined from the triaxial tests could have been used in a general case as input information for the mathematical model, results from the simpler uniaxial strain test were incorporated in this study, and the triaxial test data were used only to provide guidance in the selection of appropriate values for Poisson's ratio (as engineering judgment improves, this step can possibly be eliminated). Once the value of Poisson's ratio, ν, corresponding to a given state of stress is chosen, the associated piece-wise linear value of the modulus, E, can be determined from the relationship

$$E = \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)} M \tag{3}$$

where the constrained modulus, M, is the slope of the stress-strain curve obtained from a uniaxial strain test at the stress load of interest. Since the soil in the vicinity of the pipe was subjected to different states of compaction (as observed during the field installation), and since the stress-strain behavior of a soil is strongly dependent on its density, the mechanical properties assigned to each soil element of the mathematical model were selected to reflect the estimated or measured initial density of that element; then, as the state of stress in the element changed due to the increase in the height of cover above the pipe, the mechanical properties were varied incrementally to account for this nonlinear behavior.

Typical comparisons between experimentally measured data and results obtained from the mathematical model of the soil-pipe system for a cover height of 25 ft are given in Figure 12. The respective stress distributions exhibit the greatest discrepancy at the bottom of the pipe, where the mathematical model (averaged every 10 deg) predicts a lower normal stress than was measured experimentally; although impossible to determine with certainty, this is probably related directly to the bedding condition of the pipe. Despite attempts in the field to properly seat the pipe in a shaped bed, it appears that this condition was not achieved, and a considerable stress concentration exists at the bottom of the pipe; this high stress is substantiated by experimental data from other stress cells in the soil below the pipe. In general, the predicted stresses across the top of the pipe are slightly higher than the measured ones, but the respective distributions are quite similar. The apparent variation between experimental and theoretical stresses in the haunch region of the pipe may be simply a consequence of insufficient experimental data points in this area of high stress gradient; if the computed stresses were further smoothed over this region, the experimental and theoretical values would be in very good agreement. Considerable reliability is given to the measured stresses at the spring line, because a stress of 27 psi was measured at the soil-pipe interface and stresses of 22 psi and 18 psi were measured at points 6 in. and 12 in. respectively from the pipe wall. Possible improvements in the computed results may stem from a modification of the assumed condition of perfect bond at the soil-pipe interface, improved stress-strain relationships for the soils, better characterization of nonhomogeneities in the field installation, and use of a finer mesh for the finite element grid. The calculated relative displacements in the soil above the pipe are somewhat higher than the measured values, but the opposite is true below the pipe; however, this is consistent with the stress comparisons. This suggests that the experimental data are mutually compatible and probably quite reliable, although they are not in full agreement with the values determined from the mathematical model. The foregoing comparisons between experimental and theoretical stresses and displacements represent rather severe situations, because the values are obtained at discrete points in the system; in contrast, the diameter changes in the pipe reflect to a greater extent the integrated response of the overall soil-pipe system. As seen, the experimental and theoretical diameter changes in the horizontal and vertical directions are in excellent agreement.

CONCLUSIONS

Within the scope and limitations of the results reported here, certain conclusions can be deduced. Of considerable importance is the fact that the experimentally measured stresses and displacements appear to be mutually consistent, although these values at certain discrete points in the system differ somewhat from values calculated by use of a mathematical model. However, both the experimental and theoretical distributions of stresses around the pipe differ somewhat from those suggested in the Marston-Spangler approach. The experimentally measured and theoretically calculated horizontal and vertical diameter changes due to earth loads only were found to be in excellent agreement. The application of a heavy construction load to the pipe under a depth of cover of 4 ft had little effect on the diameter changes of the pipe or the stress increases and distribution of stresses at the soil-pipe interface, and upon removal of the load the pipe exhibited no residual stresses or displacements.

ACKNOWLEDGMENT

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DISCUSSION

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This paper is a valuable addition to the literature on the structural performance of buried conduits and is of special interest to this writer for two reasons: first, because of the experimental evidence dealing with the technology of soil-structure interaction in this type of structure and, second, because it represents the culmination of a major reversal in policy by the sponsoring agency, the American Concrete Pipe Association.

This fine industrial organization was not always research-minded. The writer well remembers a period about 45 years ago, when the Association did not approve the efforts being made at the Iowa Engineering Experiment Station toward unraveling some of the mysteries of soil-pipe performance. When our bulletin, "The Supporting Strength of Rigid Pipe Culverts" (4), was published in 1933, the managing director at the time was very critical and said, "The mathematics in that bulletin would make any self-respecting concrete pipe blush with shame." This reversal in policy is a welcome development.

On the technical side, this writer has difficulty accepting the normal unit pressure measurements at the spring line of the pipe, as shown in Figure 6. The diagrams indicate a nearly uniform distribution of pressure around the periphery of the structure above the bedding. This quasi-hydrostatic pattern is more typical of the distribution around a flexible pipe in which a relatively large increase in horizontal diameter, as the pipe is loaded, brings into play the passive resistance pressure of the soil at the spring line. In contrast, a reinforced-concrete pipe deflects a negligible amount, and the unit pressure at the spring line would probably be more nearly equal to the active lateral pressure of the soil. According to Figure 12, the horizontal diameter of the pipe increased approximately 0.05 in. under 25 ft of fill. This indicates an outward movement of each side of the pipe of 0.025 in., which is not enough to mobilize any passive resistance pressure of the soil.

The question raised, then, is not concerned with the accuracy of the pressure cells used or the accuracy of observations. Rather, the question is directed toward the validity of measuring the unit pressure on a relatively small area and extrapolating the measurement to apply to a larger area such as the side of a concrete pipe. The soil that constitutes the fill over and around a pipe may appear to be quite uniform. However, the writer's experience has led to the conclusion that the pressure indicated by a pressure cell may not be representative of the pressure on a larger prototype area because of unapparent heterogeneity of the soil. The authors quite properly suggest this possibility in their discussion of the different results indicated by cells S7 and S9, which were embedded in the crown of the pipe at points of supposedly identical pressure.

In an effort to minimize this possible discrepancy between a pressure cell reading and the actual prototype unit pressure on a larger area, this writer developed a pressure-measuring device that consisted of a stainless steel ribbon, ½ in. wide by 0.008 in. thick. This ribbon was mounted on the outside of a pipe along longitudinal elements of theoretically equal pressure. The ribbon was confined between layers of canvas and passed over stainless steel rollers at the ends of a 4-ft-long section of pipe. Then the ends of the ribbon could be pulled in a radial direction from the inside of the pipe. After calibration and during and after construction of an embankment, the pull required to start the ribbon sliding gave a measure of normal pressure on the pipe wall that was mechanically averaged over an element ½ in. wide and 48 in. long.

The pressure distribution measured in this manner on a 44-in. outside diameter concrete pipe is shown in Figure 13 (4). The 15-ft-high embankment in this case was constructed by teams and wheeled scrapers and was not compacted in a formal manner but only by the team and scraper traffic. The reason for the skewness of the vertical load on the pipe is not known, but may be related to the heterogeneity of the soil as noted earlier. It may or may not be significant to note that the direction of the skewness is toward the direction of approach of the team traffic. Attention is directed to the fact that the lateral pressures on the sides of the pipe are considerably less than the vertical and seem to be more compatible with the normal relationship between active lateral pressure and vertical pressure of soil.

The relatively minor load effect on the authors' experimental pipe caused by a heavy live load applied to the surface of the fill at an elevation 4 ft above the pipe essentially agrees with the findings by the writer in an extensive series of tests performed approximately 50 years ago (5). In these tests, a heavily loaded truck wheel was positioned over a culvert section 2 ft long and 3.5 ft in outside diameter. The load transmitted to the culvert was measured and expressed as a fraction of the truck wheel load when applied at the surface of fills ranging in depth from 6 in. to 6 ft. The transmitted loads were compared with loads calculated by the Boussinesq theory of stress transmission in an elastic, isotropic, homogeneous medium of semi-infinite extent (half space). Although soil is neither elastic, nor isotropic, nor homogeneous, it was found that a reasonably good correlation existed between the measured and theoretical loads. The following conclusion on this matter is quoted from Bulletin 79 (5):

> The theoretical formula (Boussinesq) seems to give a locus showing the maximum percent of load transmitted through any thickness of fill. In the experimental work, however, this maximum load generally was not reached, but when conditions were most favorable... the experimental results came very close to the theoretical.

The correspondence between the theoretical and experimental loads on the culvert is shown in Figure 14.

It is of interest to compare the authors' measurements of vertical unit pressures on cells S7 and S9 with calculated pressures by the Boussinesq equation. Using Newmark's integration, unit pressures along the longitudinal centerline of the experimental pipe have been calculated and are shown in Figure 15. The position of the live loads relative to the pressure cells was furnished to this writer by Krizek. The measured pressures are somewhat greater than the theoretical value, which is contrary to the writer's experience noted above, but the divergence is not significant.

In estimating the effect of surface traffic loads on a buried conduit, the possibility of impact loads must be considered. In the writer's experiments referred to earlier (5), impact loads were measured along with static loads. Impact loads varied widely,

Figure 13. Radial earth pressures on concrete pipe measured by pressure ribbons.

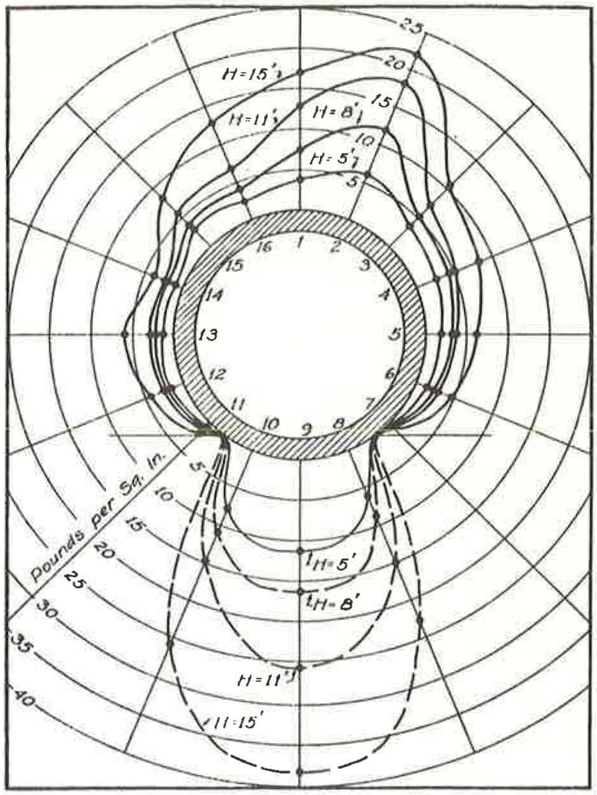


Figure 14. Static wheel loads transmitted to a section of culvert 2 x 3½ ft in outside diameter.

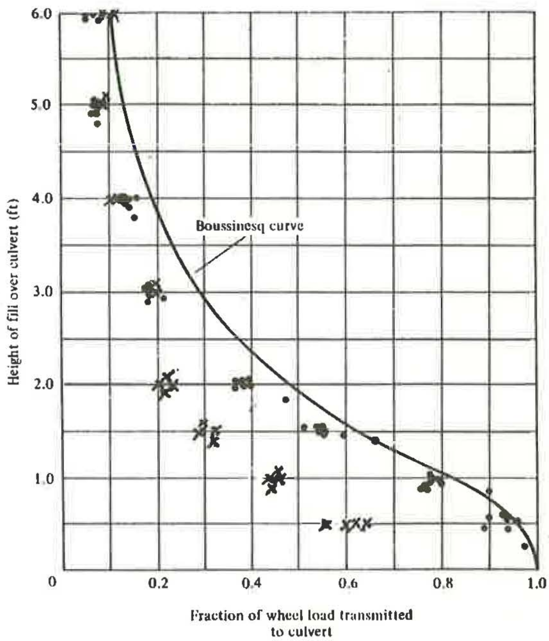


Figure 15. Calculated pressures along centerline of pipe.

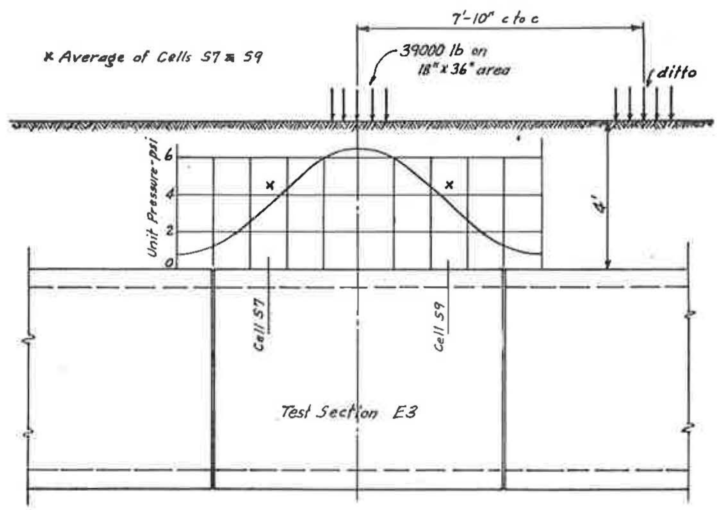


Figure 16. Total load on buried conduit versus height of fill.

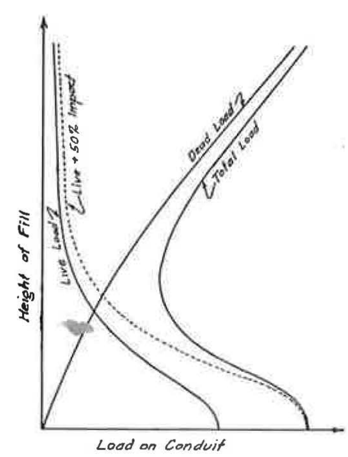
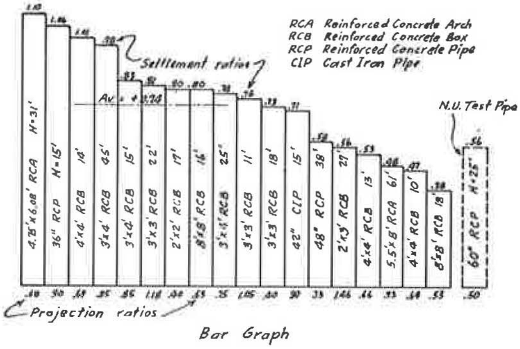


Figure 17. Measured values of settlement ratio for rigid culverts.



mainly depending on the character of the roadway over the culvert. The type of vehicle used in the authors' experiments probably would exert an impact load on a buried conduit under actual field conditions. Such vehicles operate at fairly high speeds and have been observed to operate in a "bouncing" or "porpoising" manner at times, which undoubtedly would cause considerable impact if the downward cycle of a bounce came directly above the conduit. This writer estimates that an appropriate impact factor of 2.0 probably would suffice, if a design were based on such a vehicle operating on the surface at a shallow depth of cover.

A diagrammatic relationship between earth load, static and impact loads, and total load on a buried conduit at various depths of cover is shown in Figure 16. It is indicated that the total load decreases to a minimum at some relatively shallow height of fill, then increases as the height increases.

Another factor of interest in connection with concentrated surface loads is the matter of the "effective length" of conduit to be considered in connection with such loads. The load produced by a concentrated surface load is of varying intensity on the conduit, being a maximum directly under the center of the applied load and decaying rather rapidly in a longitudinal direction, as indicated in Figure 15. Effective length is defined as the length of conduit over which the transmitted load can be considered to be uniformly distributed to produce the same stresses and deflections in the pipe ring as does the actual varying load. To illustrate, suppose the authors' experiments had been conducted on a pipe only 2 ft long. In all probability the measured stresses in the pipe would have been greater than those observed in the 8-ft-long pipe section.

As far as the writer is aware, no research on this matter of effective length has ever been reported.

The authors measured the settlement ratio that prevailed in the experimental installation and found it to be +0.56. This is of great interest and value, as such measurements are very scarce, and every additional reliable measurement adds a great deal to our knowledge in this area. The writer has measured this ratio in connection with 18 actual field structures consisting of reinforced-concrete arch culverts, reinforced-concrete box culverts, reinforced-concrete pipe culverts, and one cast-iron pipe culvert (6). The average of these measurements was +0.74. The authors' measured value fits very nicely with these values, as shown in Figure 17.

The writer has for many years advocated using a value of +0.7 for this factor in design work on positive projecting conduits, particularly in cases where environmental conditions are not sufficiently well defined to permit a rational estimate, which is usually the case. This measurement by the authors lends support to this design practice.

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AUTHORS' CLOSURE

The authors thank Spangler for his interesting and thought-provoking discussion of this work. His vast experience in the area of buried conduits provides the necessary background to place this study in appropriate historical perspective, and his added data and supplemental calculations are most welcome.

The normal pressure measurements at the spring line of the pipe are indeed higher than had been expected from the Marston-Spangler theory, but there are good reasons to believe that these measurements are reasonably accurate. Among these reasons are the facts that (a) the stress cells employed were evaluated quite thoroughly by a coordinated experimental and theoretical study; (b) their sensing area is relatively large (over 25 in^2 per cell and an equivalent linear sensing area for all nine cells equal to one quadrant of the pipe); (c) the readings from nine stress cells symmetrically distributed in two transverse planes were reasonably symmetric and, with the exception of the cell at the bottom of the pipe and one cell at the top, yielded a relatively smooth and continuous normal pressure distribution around the pipe; (d) the magnitudes of the normal stresses at the soil-pipe interface were consistent with other normal stresses (not reported in this paper) measured in the soil immediately adjacent (within a foot or two) to the pipe; (e) the measured normal stresses are consistent with those calculated by use of the stress-strain properties of the soils and simultaneously measured normal strains; and (f) the measured stresses due to an imposed live load are consistent with those anticipated from an engineering approximation of the problem. In brief, Spangler is certainly correct in his statement that the pressure indicated by any one stress cell may, due to installation conditions and/or local heterogeneity of the soil (as was apparently the case with one of the cells at the top of the pipe), not be representative of the pressure on a larger prototype area, but we feel that the foregoing facts tend to substantiate the reasonable overall validity of the stress measurements reported and the interpretations deduced.

It is agreed that the small horizontal displacements of the pipe at its spring line are, in all likelihood, not sufficient to mobilize the full passive resistance of the adjacent soil. However, by the same token, the outward displacements of the pipe at its spring line do not suggest the presence of an active pressure condition. As described in the paper, the approach advocated is based on the stress-strain properties of the various components of a continuum model, and the active or passive resistance of the soil is not incorporated into the formulation.

The excellent supplemental information regarding the effect of a live load and the favorable position of our measured settlement ratio in the context of 18 case histories accumulated by Spangler are sincerely appreciated. Such agreements tend to illustrate and emphasize the tremendous early contributions made by Marston, Spangler, and their coworkers, and they further serve to demonstrate the compatibility between the present analytical approaches to problems of this type and the engineering approaches taken by the Iowa group.