RESPONSE OF CORRUGATED STEEL PIPE TO EXTERNAL SOIL PRESSURES

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Full-scale external load testing of buried corrugated steel pipes shows the structural performance limits of the soil-pipe system. The tests, sponsored by the American Iron and Steel Institute, indicate that the 3 most important factors influencing performance are the yield point strength of the pipe wall, the soil compressibility (determined primarily by soil density), and the ring flexibility of the pipe. The empirical relationship of these 3 factors is plotted on a graph from which it is possible to design buried corrugated steel pipes.

• SINCE corrugated steel pipes first appeared on the market about 70 years ago, their use as buried conduits has increased phenomenally. The structural success of these pipes is due to the flexibility of the pipe ring. As soil is placed over the pipe the ring is flattened; i.e., the ring is depressed vertically and expanded horizontally. The horizontal expansion develops lateral soil support on the sides, and this gives rigidity to the ring and strengthens it. The vertical depression of the ring relieves the ring of soil pressure concentrations and forces the soil to take part of the vertical load in arching action over the pipe. Thus the soil protects the pipe.

The discovery of this complementary soil-structure interaction has led to innumerable tests and continual observation. Four independent design criteria have evolved from these tests and observations (the Appendix contains a further discussion of these criteria): (a) excessive ring deflection (flattening of the pipe), (b) longitudinal seam strength, (c) ring compression strength (crushing or buckling of the pipe wall), and (d) handling strength (for shipping and installation) sometimes called flexibility factor.

All 4 design criteria have been used successfully. Proponents of each have provided a method of analysis and have suggested allowable limits. The first three are conservative enough so that, if design is within the limit specified for any one criterion, adequate performance is ensured. Most designers check more than one design criterion, however, and base design on the worst case. For the vast majority of all installations, such design is too conservative. However, a few exceptions suggest the need for reevaluation of design methods.

There are other reasons for reevaluation. These 4 design criteria do not include the interaction of 2 or more criteria. They are based on many assumed properties of materials that cannot be measured by the designer. One recourse is to pull a recommended value out of a graph or table based on other unmeasured properties. Soil properties are particularly troublesome. Consequently, they are often rounded up by guess or are abandoned within the comfortable confines of a large safety factor. This is really abandonment of the problem because soil is generally the most important factor in the soil-pipe interaction phenomenon. The allowable limits of design are unreasonably restrictive under some circumstances such as excellent backfill. Every user has to decide which design criteria to adopt. Often the appropriate criterion for design is unknown.

Sponsored by Committee on Subsurface Structures Design and presented at the 50th Annual Meeting.

For these reasons the 4 design criteria have been reevaluated within the past 3 years by the Federal Highway Administration, the American Association of State Highway Officials, and the American Iron and Steel Institute. With considerable cooperation and effort, design criteria, pertinent properties of materials, and design limits were agreed on. All 3 agencies have since published similar design procedures. In general the procedures are easy to use. However, they should be checked for precision and for design limits. Pertinent properties of materials should be verified and the significance of their influence ascertained—especially soil properties. The interaction of the design criteria should be determined. These are the objectives of this study. A test program at Utah State University was funded from 1967 to 1971 by the American Iron and Steel Institute to accomplish these objectives.

The study does not include live loads with minimum soil cover. It does not include longitudinal phenomena such as beam deflection or shearing stresses due to longitudinally nonuniform soil settlement.

TEST CELL

Basically the project comprises the testing to failure of full-scale corrugated steel pipes by applying vertical soil pressure. Sections of pipe 20 ft long and up to 5 ft in diameter are buried in soil within a large test cell (Fig. 1). The vertical soil pressure is applied by 50 hydraulic rams—5 on each of 10 load beams. The total load capacity is about 5 million lb. The maximum vertical soil stress at the level of the top of the test pipe is 20,000 lb/ft^2.

The height of soil cover over the top of the pipe is 1 pipe diameter. Consequently, failure is not a localized "punching through" of surface loads (like wheel loads).

The test cell is a horizontal cylinder of $^5/_8$-in. steel plate 22 ft long, 15 ft wide, and 18 ft high. The basic cross section is approximately elliptical with horizontal radius of curvature 3 times the vertical. This minimizes boundary effect. As an embankment of average soil is increased in height, the ratio of vertical to horizontal stress is roughly 3:1. The 3:1 stress ratio is maintained by the 3:1 ratio of radii in the steel shell. Moreover, the flexible plate does adjust to different stress ratios by changing curvature during loading. Tests on different pipe diameters from 1 to 5 ft prove that boundary effect is of secondary importance in determining ring strength. Even when combined with other secondary (extraneous) variables, the probable deviation is at most 10 percent and that only in loose soil. (Other variables include the nonuniformity of load under the hydraulic jacks and wedging out of soil on either end of the test cell.) Such a deviation is statistically insignificant compared with combined deviations in ring stiffness, yield point, soil density, and soil placement techniques. In field installations the deviations due to boundary effect and soil placement techniques are even greater (viz., trench wall slopes, trench wall compressibility, types of compactors, and proximity to pipe).

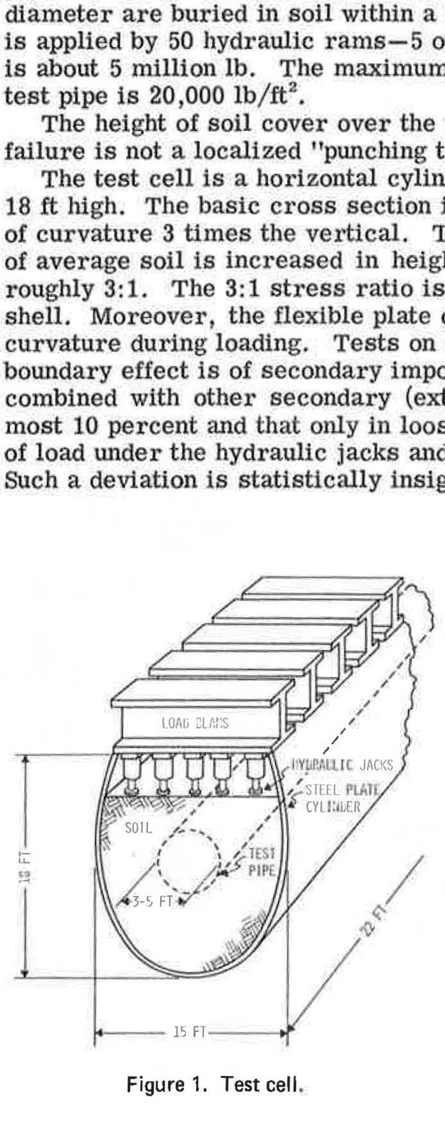

Figure 1. Test cell.

Test sections of pipe are buried as in average embankment installations. Embankment installation was selected over trench installation because ordinarily failure in an embankment occurs at less soil cover than does failure in a trench. Therefore the embankment tests are conservative for design. Tested in the series were 130 sections of pipe.

Probably the worst soil that could be selected for testing is liquid (viscous mud). This was not considered in these tests because (a) the phenomenon is understood [the design of cylinders subjected to external fluid pressure is classical (1)], and (b) for most installations "liquid" soil is not acceptable as a fill. For example, clay would be avoided even though placed dry (dry side of optimum moisture content) if it could become saturated later. Organic material (humus or peat) is not to be included as soil in this report.

The next worst soil that could be selected for testing is highly compressible soil. Such was selected for these tests. It is basically fine sand with 18 percent silt, a trace of clay, and a small fraction of sand. The 100 percent dense unit weight is 130 lb/ft^3. (Density is based on modified AASHO T-180.) It bulks easily and can be placed as loosely as 65 percent standard density. When loose it is highly compressible. If used as backfill in field installations, it should be carefully compacted. Placement techniques such as washing into place or end dumping are not adequate for this soil. Because of the high compressibility when loose, it provides a broad range of soil compressibility for testing purposes.

Figure 2. Loading the cell.

TEST PROCEDURE

The pipe to be tested was instrumented with several fluid-filled pressure gages around the circumference to measure soil pressure against the pipe during the test. A soil bedding was prepared, and the pipe was located in the cell. Soil was then carefully compacted about the pipe in approximately 1-ft lifts (Fig. 2). If loose soil was desired, no compaction was employed. If medium dense soil was required, a Wacker vibroplate made 1 pass over each lift. If dense soil was required, compaction was by Wacker rammer compactors at optimum moisture content (Fig. 3). In-place density tests were conducted by the sand displacement method in several of the lifts.

After the cell was loaded with soil, steel plates were placed on the top of the soil. The beams were then lowered and locked into place (Fig. 4). The hydraulic rams

Figure 3. Technique for compacting the backfill in 1-ft lifts.

Figure 4. Loading beams being lowered into position where they are pinned before load is applied to cell (top), and test cell loaded and ready for test to begin (bottom).

exerted force on the steel plates. This procedure prevented penetration of the hydraulic rams into the soil and produced a more uniform loading.

A profile instrument was mounted inside the pipe to determine the ring profile at any time during the test. During each test the following readings were noted at various intervals: (a) hydraulic ram pressure—this was later converted to vertical soil pressure, (b) pressure readings from pressure gages around pipe circumference, (c) vertical ring deflection, (d) horizontal ring deflection, and (e) ring profile. In addition to these readings any pipe distress noted by technicians inside the pipe was recorded. Also, photographs of most of the tests were made by using both elapsed time photography and still shots.

Most of the pipes tested were in diameters of 3, 4, and 5 ft. Corrugation depths of 1, 0.5, and 0.25 in. were tested. Both annular and helical corrugations were tested. The seams of the annular pipes were of 2 types: spot-welded and riveted. The helical pipes had a lock seam joint.

RESULTS OF TESTS

The most significant results of the tests are shown in Figure 5. The ordinate is apparent ring compression strength f_c. It is defined as the apparent ring compression stress at performance limit; i.e.,

$$f_c = PD/2A \text{ at performance limit} \tag{1}$$

where (Fig. 6)

- P = apparent vertical soil pressure, i.e., calculated pressure at the level of the top of the pipe if no pipe were in place,
- D = nominal diameter of the pipe, and
- A = cross-sectional area of the pipe wall per unit length of pipe.

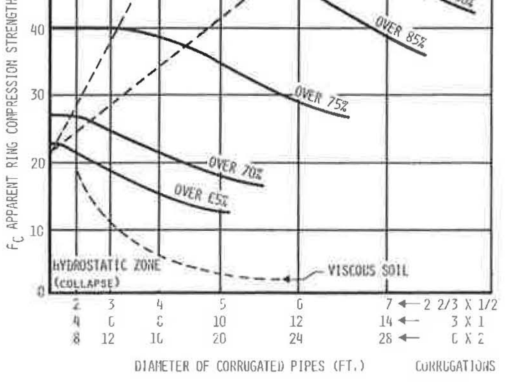

Figure 5. Apparent ring compression strength as a function of ring flexibility and soil compressibility (density) based on performance limit of incipient ring failure (probable deviation, i.e., 50 percent uncertainty, is about half the spacing between curves; curves apply to corrugated steel pipes with yield point of 40 ksi).

Performance limit is ring deformation beyond which the soil-pipe system does not perform adequately. It is discussed in the Appendix.

To design the pipe ring, one can employ the well-known, universal design criterion STRESS < STRENGTH, i.e.,

$$PD/2A = f_c/N \tag{2}$$

where

- P = apparent vertical soil pressure, (i.e., calculated pressure at the level of the top of the pipe if no pipe were in place) that compromises dead load DL and live load LL, i.e., P = DL + LL;
- DL = γH or unit weight of soil γ times the height of fill H over the top of the pipe;
- LL = vertical soil pressure at the level of the top of the pipe due to surface loads;
- f_c = apparent ring compression strength that can be simply picked off the plots (Fig. 5); and
- N = safety factor.

The ordinates f_c shown in Figure 5 are values of apparent ring compression strength to be used in design. The abscissas shown in Figure 5 are values of ring flexibility of the pipe. Ring flexibility is determined almost entirely by the depth of corrugation and the nominal diameter as shown. The parameter distinguishing the various curves is soil density in percentage of modified AASHO T-180. Soil compressibility is the single most important soil property. Soil density is the single most important factor determining compressibility. In fact, other soil properties become secondary (insignificant) within the average deviations of soil density and boundary conditions (trench, zone of compaction, and bedding) resulting from present-day installation techniques.

The data shown in Figure 5 are based on a performance limit referred to as incipient ring failure. Incipient ring failure is defined as a ring deformation beyond which the ring would continue to deform (to collapse) if loads on it were not relieved by soil-arching action. Actually incipient ring failure is not complete failure of the soil-pipe system. The pipe does not collapse. Any increase in external pressure is supported by the soil in arching action. Incipient ring failure is recognized by completely developed plastic hinges on the sides of the pipe or by reversal of curvature of the wall or by seam separation. The Appendix contains a further discussion.

The minimum specified yield strength of culvert steel is 33,000 lb/in.^{2} with values being about 40,000 lb/in.^{2} (40 ksi) for these tests. Figure 5 shows a maximum apparent ring compression strength higher than the 40 ksi yield point. Actually the well-compacted soil is supporting part of the vertical pressure in arching action (a low-grade masonry arch). As the pipe ring begins to be distressed, it deforms and relieves itself of part of the external pressure so actual stress in the pipe wall does not exceed yield point. The ring compression strength (ordinate) shown in Figure 5 is called apparent for this reason.

When the soil is relatively incompressible (densely compacted), the apparent ring compression strength is essentially constant depending on yield point strength or longitudinal joint strength or (more probably) the interaction of both. In Figure 5 this is shown as the wall-crushing zone.

When the soil is relatively compressible (loose or poorly compacted), the apparent ring compression strength is reduced significantly because of pressure concentrations on the ring and because of ring deflection that causes flexural stress in addition to compression stress in the wall. Figure 5 shows that strength envelopes drop down as soil density decreases.

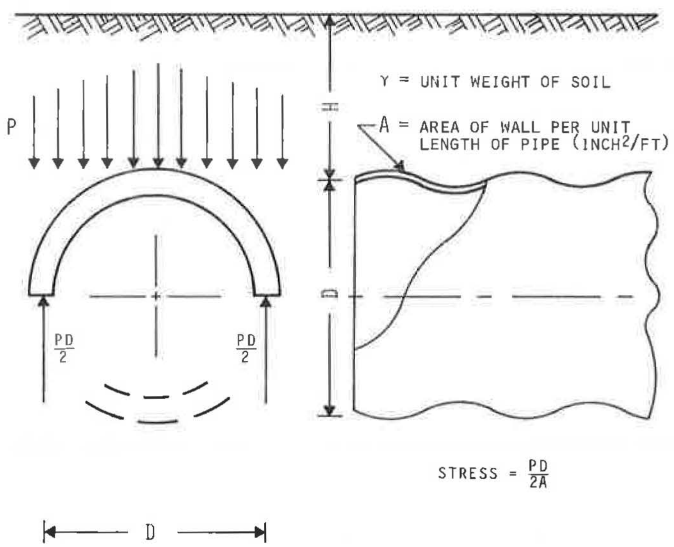

Figure 6. Free body diagram of pipe ring with vertical soil pressure.

It is noteworthy that the strength envelopes dip down to the right with increasing ring flexibility. This is due to the increased sensitivity of the very flexible ring to nonuniform soil density. If the soil could be placed particle by particle, the strength envelopes would not dip down so much (especially in well-compacted soil). However, present soil-placement techniques result in nonhomogeneous soil that causes pressure spots and precipitates wall-buckling in the very flexible rings. This is shown as the ring-buckling zone.

STRUCTURAL DESIGN OF PIPE RING

Suppose that a 48-in. diameter $2\frac{2}{3}$- by $\frac{1}{2}$-in. corrugated steel pipe is to be installed under 120 ft of soil embankment. The soil about the pipe is to be compacted to 90 percent modified density (found to have a unit weight of about 120 lb/ft^3). Determine the pipe wall thickness (gage) if the performance limit is defined as incipient ring failure (Fig. 5). Suppose that H20 loading will pass over the surface. If control of the installation is dubious, a safety factor of N = 2 will be assumed.

The apparent vertical soil pressure on the pipe ring is

$$P = DL + LL = 14.4 \text{ kip/ft}^{-2}$$

where

$DL = \gamma H = 120 \text{ lb/ft}^{-3} \times 120$ ft, and
LL = negligible (2).

The apparent ring compression stress is

$$PD/2A = \left[14.4 \text{ kips } (4.0 \text{ ft})\right]/\text{ft}^2 2A = 28.8 \text{ kips/A ft}$$

The apparent ring compression strength is (based on 40 ksi yield point)

$$f_c = 60 \text{ kips/in.}^{-2}$$

which is the ordinate to the strength envelope shown in Figure 5 corresponding to soil density of 90 percent and a pipe diameter of 4.0 ft in a $2\frac{2}{3}$ by $\frac{1}{2}$ corrugation. (Where the yield point is something other than 40 ksi, the apparent ring compression strength f_c would be modified proportionally.) Equating stress to strength divided by safety factor yields

$$PD/2A = f_c/N$$

or

$$28.8 \text{ kips/A ft} = 60 \text{ kips/2 in.}^2$$

Solving for the area yields A = 0.96 in.^{2}/ft. One should use 12-gage steel that has an area of 1.356 in.^{2}/ft (3).

A check on ring deflection would predict a ring deflection of less than 3 percent at a soil density of 90 percent (4).

The ring flexibility factor (handling factor) is adequate.

CONCLUSIONS

The most pertinent criteria for the structural design of the soil-ring system for buried corrugated steel pipes are included in the empirical strength envelopes shown in Figure 5. Design is simply the equating of stress to allowable strength (design limit), i.e., $PD/2A = f_c/N$.

The stress PD/2A is apparent ring compression stress in the pipe wall; i.e., it is based on a vertical soil pressure P at the level of the top of the pipe if no pipe were in place. f_c/N is the design limit. N is an appropriate safety factor. The apparent strength

f_c is apparent ring compression strength; i.e., it is treated as ring compression stress at failure. Actually f_c includes more than just ring compression. It takes into account the soil-arching action in dense soil and the soil pressure concentrations. It also takes into account ring deflection in compressible soil and wall-buckling of very flexible pipes. It accounts for the formation of plastic hinges and seam failures.

In general the soil is the most important factor in determining f_c. The single most important soil property is soil compressibility, which is basically a function of soil density. Density is the most basic soil property or index for relating the performance of different soil types.

The second most important factor in determining f_c is the yield point of the steel. This is usually within fairly narrow limits (33 to 40 ksi).

The third factor is ring stiffness.

All other factors are of lesser importance and for most installations are secondary and not significant compared to soil density, yield point of steel, and ring stiffness. The total range of influence of secondary factors is less than the probable deviation of the 3 important factors.

A reasonable performance limit of buried corrugated steel pipes for most installations is incipient ring failure. It is defined as that ring deformation beyond which the ring would continue to deform if external loads on it were not relieved by arching action of the soil. If this performance limit is used, the arching action of soil becomes an additional safety factor. Thus protection against collapse is ensured.

The data shown in Figure 5 apply to all types of soil that can be compacted and held at a specified density. This would exclude expansive soils such as humus and highly expansive clay. Clay may be more compressible when saturated than when dry. If wet clay were conceivably used as backfill, the lower curve shown in Figure 5 could be helpful as a conservative limit for design. The dotted hydrostatic curve applies to liquid (viscous) soil. Saturated clay with any shearing strength will fall above the hydrostatic curve. Ordinarily if clay has been placed at a density well above critical void ratio (85 percent density), it is so impervious that saturation is too slow to be a problem (if saturation can proceed at all). If saturation proceeds slowly enough, soil cohesion develops and decreases compressibility enough to offset the increase of compressibility due to the water. In other words, the compressibility of densely compacted, confined clay is usually not increased as moisture moves into it if the load is constant.

After the ring is designed by use of curves shown in Figure 5, the ring deflection may be checked if there is any question about excessive ring deflection. The ring flexibility factor (handling factor) should be checked if there is any question.

Seam strength seems to be independent of the type of seam used in these tests. Welded, riveted, and lock seam seams were tested. All performed adequately. It is recommended that welded and riveted longitudinal seams not be placed in the 10 and 2 o'clock positions in the pipe if design is near performance limit. This recommendation is made because riveted or welded seams in these positions can trigger premature reversal of curvature. However, the loss of strength due to improper seam placement was observed to be less than 10 percent.

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Appendix

CRITERIA USED TO DESIGN BURIED CORRUGATED STEEL PIPES

Four independent criteria used in the past to design buried corrugated steel pipes grew out of a need for design criteria and represent 4 structural limitations in corrugated steel pipes.

1. Excessive ring deflection (flattening of the pipe) in some installations led to research at Iowa State University that resulted in the development of the Iowa ring deflection formula (5).
2. The strength of longitudinal seams proved to be important in design. The soil pressure on the pipe (2) determines the ring compression that can be equated to the seam strength (3) reduced by an appropriate safety factor.
3. The ring compression stress in large pipes may not necessarily exceed the seam strength, yet it can be high enough to cause elastic buckling of the pipe wall (6). This has led to a method of design based on reversal of curvature of the ring.
4. A pipe must be handled during transport to the job, and it must withstand distorting pressures during backfilling. This consideration requires a reasonably stiff pipe as quantified by a handling and installation factor called a ring flexibility factor (3).

PERFORMANCE LIMITS

The performance limit of a buried corrugated steel pipe ring is deformation—that deformation of the ring beyond which the system can no longer perform the purpose for which it was designed. If an unacceptable hump or dip or crack develops in the soil surface above the pipe, performance limit is exceeded. If the flow characteristics of the pipe are reduced below designed values because of ring deformation, performance limit is exceeded. The final definition of performance limit must be left up to the design engineer.

For most installations the definition of performance limit is incipient ring failure as shown in Figure 5. Incipient ring failure is defined as some deformation of the ring beyond which the ring would continue to deform (to collapse) if loads on it were not relieved by arching action of the soil. This is an arbitrary performance limit. It does not mean collapse. The proposed strength envelopes shown in Figure 5 become a design chart for this performance limit. The strength envelope for dense soil exceeds the yield point for steel because part of the vertical soil pressure is supported by the soil in arching action. An additional safety factor is "built in" because the ring does not collapse even though it is deformed to incipient ring failure.

The performance limit for buried corrugated steel pipes is not a single phenomenon, but the interaction of a number of phenomena. For example, performance limit is not simply crushing of the wall or buckling of the wall or shearing of the longitudinal seam or ring deflection. Each of these influences one another, and all are interrelated to varying degrees under varying circumstances. As might be anticipated, the crushing strength of the wall is less if the ring deflection is large. This is due to flexural stresses. A longitudinal seam in one panel causes a stress concentration in the wall of the adjacent panel and triggers wall-crushing. Of course, as wall-crushing develops, wall-buckling is initiated, and buckling near seams causes seam failure—truly an interaction phenomenon.

In every case, performance limit is a ring deformation, observable inside the pipe. The probable deviation in observing performance limits may be as much as 10 percent of vertical soil pressure—especially near critical void ratio. The following are some deformations identified as performance limits in these tests.

Wall-Crushing

When the pipe is buried in densely compacted soil (denser than critical void ratio), wall-crushing is often the first indication that performance limit has been reached. Slight dimpling of the corrugations is the first visual indication of distress. Dimpling is not a performance limit, but dimpling portends the location of general wall-crushing

(Fig. 7). This crushing usually occurs between 10 and 2 o'clock in the ring. Deep corrugations dimple as soon as or sooner than shallow corrugations, but general wall-crushing shows up at equal or slightly higher pressures. In general, wall-crushing develops as shown in Figure 8. It starts with a dimpling of the corrugations and progresses into an accordion effect.

Reversal of Curvature

As the load increases, a section of the ring may tend to flatten and then reverse curvature (Fig. 9). There are 2 general types of reversal of curvature. In the case of very loose soil (density less than critical void ratio), as the soil is compressed downward the pipe tends to form an ellipse but, in so doing, high flexural stresses develop at the sides. These stresses combined with some ring compression cause plastic hinges. If this deformation is carried to the extreme, the top of the pipe comes down in a reversal of curvature and ultimately a third plastic hinge forms in the top center.

The other type of reversal occurs in dense soil and may be referred to as localized buckling. This is not confined to top center. It usually forms between 10 and 2 o'clock, but not necessarily so. Occasionally the reversal occurs in the bottom between 5 and 7 o'clock (Fig. 10). None has been seen in the sides between about 2 and 5 o'clock or 7 and 10 o'clock.

Performance limit for deep corrugations tends to be plastic hinges at the sides rather than reversed curvature. For shallow corrugation, plastic hinges at the sides form only if the soil is very compressible; otherwise, performance limit is reversal of curvature. The difference is insignificant in light of uncertainties in soil placement, density, or boundaries.

Seam Separation

Seam separation is complex shearing, tearing, pulling through, and bearing all at once. There is no question about identifying seam separation; the question is usually what triggers seam separation. For example, in the case of the helical

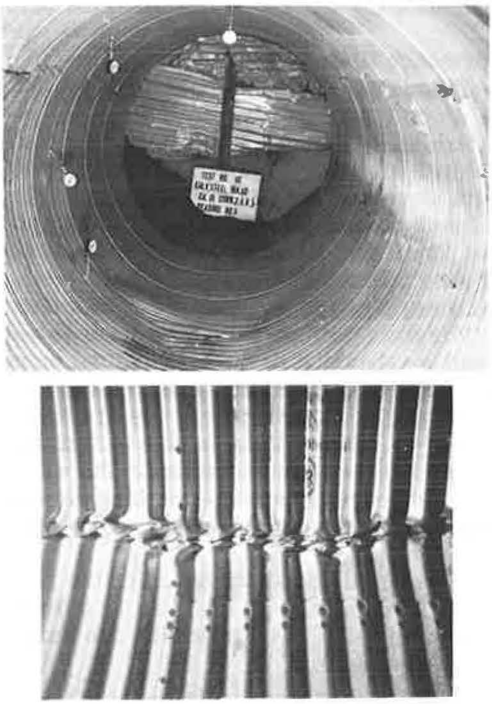

Figure 7. General wall-crushing at 10 and 2 o'clock in test with dense soil where tendency to wall-crushing is visible at 4 o'clock (top) and wall-crushing at 2 o'clock is shown in close-up (bottom), but integrity as a pipe is still maintained.

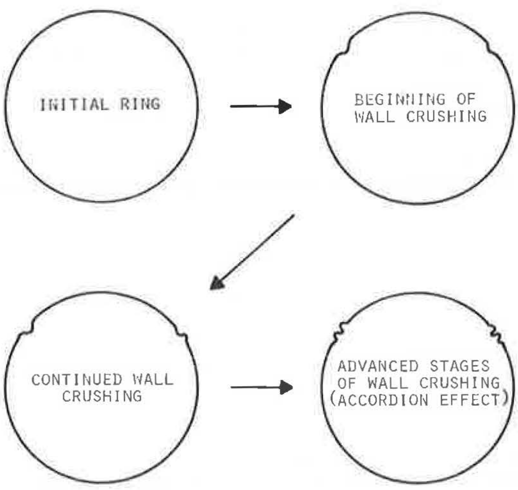

Figure 8. Mechanism of wall-crushing.

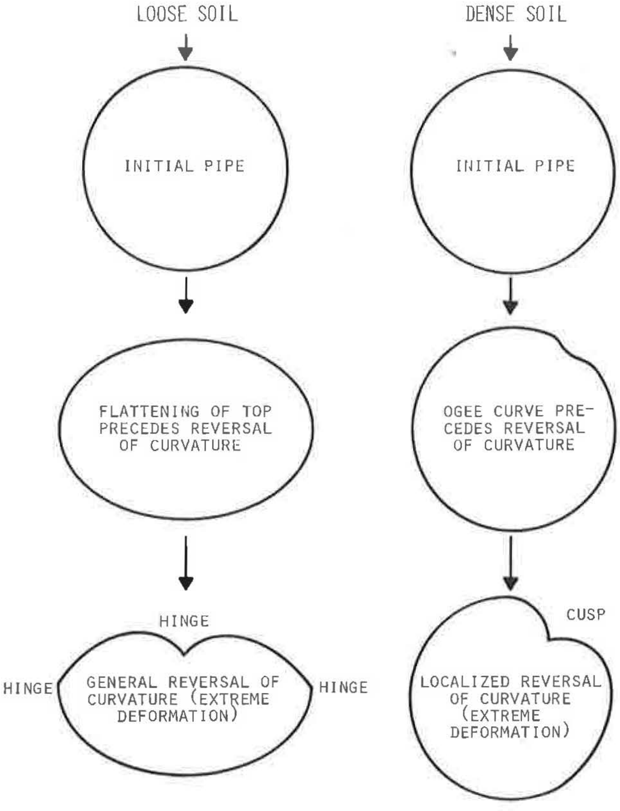

Figure 9. Comparison of types of reversal of curvature observed in dense and loose soil.

Figure 10. Pipe wall buckled at invert in dense soil (this is exceptional, for buckling is usually between 10 and 2 o'clock).

lock seam when a reversal of curvature commences, and more especially as it develops into a cusp, the seam at the cusp tends to open. All of the standard seams tested performed adequately. Differences were insignificant.

In all cases, it is important to note that dimpling of the crests of the corrugations is not a performance limit. Neither is slipping of joints. These should be accepted as stress relievers.

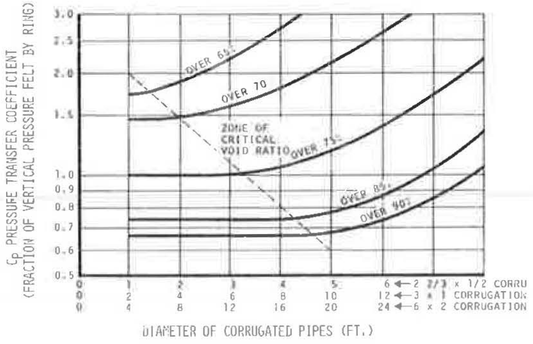

Figure 11. Test values of pressure transfer coefficient C_p as a function of soil density, diameter, and corrugation configuration.

PRESSURE TRANSFER COEFFICIENT

Figure 5 can be redrawn in the form shown in Figure 11. The basic difference is in the definition of apparent ring compression stress and strength. Figure 11 shows that the strength is the nominal yield point strength of the steel (such as 33 ksi). The stress is $C_p(PD/2A)$ where C_p is the pressure transfer coefficient. C_p is read from Figure 11. Design proceeds as before; i.e., stress = strength/N. This design method has one advantage in that the nominal yield point of steel can vary. For Figure 5, the nominal yield point of steel is fixed at 40 ksi. For extreme ranges of yield point, Figure 11 should be checked empirically.

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Discussion

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When laboratory procedures are employed to study the performance of a field prototype structure, it is essential that the experimental specimen be mounted and loaded in such a manner and in such an environment that its action, in all respects, will be analogous to the normal action of its counterpart under service conditions. It is the opinion of this writer that the Utah tests, described by Watkins and Moser, fail to comply with this essential criterion, and the reported results are, therefore, applicable only to those specific pipe specimens that were used in the tests. The results cannot be generalized and validly applied to the vast majority of installations of corrugated steel pipes under normal service conditions. Any similarity between the structural action of the Utah test pipes and that of actual pipes under earth embankments is purely coincidental.

Normally a buried flexible pipe is bedded on soil of some degree of compressibility. Soil fill is then placed on each side of the pipe up to its top, after which a soil embankment is constructed up to a finished grade. The basic structural action of the pipe is as follows: As the side fills are placed, the active lateral earth pressures cause a limited amount of negative pipe deflection; that is, the horizontal diameter decreases and the vertical diameter increases. When the side fills reach the horizontal plane through the top of the pipe, called the critical plane, additional increments of fill cause the pipe to deform positively, the horizontal diameter increasing and the vertical diameter decreasing. As the horizontal diameter increases, the sides of the pipe push outward against the side fills, and this movement mobilizes the passive resistance pressure of the soil. These passive pressures, acting on each side of the pipe, very greatly influence the deformation or change in shape of the pipe and the magnitude and character of stresses in the pipe wall. An essential characteristic of the installation of this type of structure is that the interaction between the soil and the pipe be free and uninhibited by extraneous forces or unnatural barriers to the operation of the flexible character of the pipe. The pipe should "float free" in its surrounding soil environment.

In the Utah tests, corrugated steel pipes were artificially loaded in a cell consisting of heavy steel plates located 1 pipe diameter out from each side of the pipe. These plates were backed up by concrete retaining walls that contributed to the rigidity of the sides of the cell. There is little doubt in this writer's mind that confinement of the pipes between these side walls served to inhibit the normal flexible action characteristics of corrugated steel pipe under an earth embankment. It is as though a 60-in. flexible culvert pipe were installed in a canyon with vertical rock side walls only 15 ft apart. Such an installation certainly would not qualify as typical of the vast majority of corrugated steel pipe culverts in highway or railway construction.

In addition to the lateral restraining influence of the steel plate and concrete side walls of the load cell, the pipes were further restrained in a vertical direction by the action of hydraulic jacks reacting against heavy steel beams that extended transversely across the top of the cell and that were anchored to the steel plates of the side walls. The vertical loads from the hydraulic rams were transmitted to the soil backfill surface, 1 diameter above the top of the pipe, through steel plates. It is doubtful whether such a load system would adequately simulate the load of an actual soil embankment. Furthermore, because of the concave configuration of the steel plates, it is probable that the vertical upward reactions of the cross beams caused horizontal components of force to be directed toward the specimen pipes. All in all, it appears that the pipes were encased in a straitjacket that prevented their normal action as flexible structures in which pipe deflections bring about redistribution of stresses and deformations within the soil. Such interaction and redistribution are the hallmarks of flexible pipe action, and they contribute greatly to the efficiency of this type of structure.

Another circumstance that polarizes and restricts the applicability of the test results is the kind of soil with which the pipe was surrounded in the test cell. Only one soil type was used, a "fine sand with about 18 percent silt, a slight fraction of sand, and a trace of clay." Such a soil, when compacted, would be very stiff and strain resistant. It would contribute to the rigidity of the pipe environment that prevailed during the tests.

The authors show a free body diagram of the top half of a vertically loaded corrugated steel pipe (Fig. 6). This free body diagram is incomplete. When a statically indeterminate structure is cut on any section, the stresses acting on the section are a thrust, a moment, and a shear, as shown in Figure 12b. Equations for these stresses around the periphery of a corrugated metal pipe are given at the end of this discussion. The radial shear is a finite stress but is of minor magnitude and can be neglected in consideration of the action of the pipe, but the bending moment is very real and cannot be ignored. It would appear that in the construction of their free-body diagram the authors have fallen into the same error as the proponents of White and Layer's compression ring theory (8), in which bending moment in the pipe wall is completely ignored and left out of consideration. The authors give an expression for stress in the pipe wall in the form

$$\sigma = pD/2A \qquad (3)$$

where

σ = stress in pipe wall,
p = unit load on pipe,

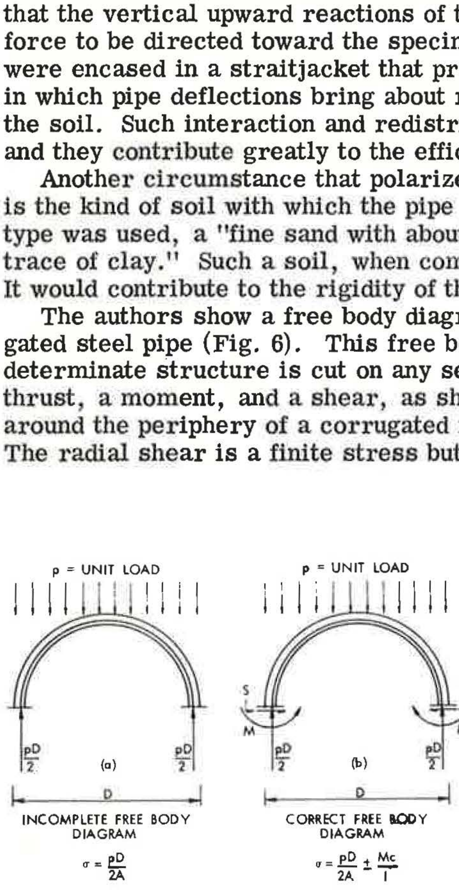


Figure 12. Free body diagrams of pipe.

D = diameter of pipe, and
A = cross-sectional area of pipe wall.

The correct expression for stress is

$$\sigma = pD/2A \pm Mc/I \tag{4}$$

where

M = bending moment in pipe wall,
c = distance from neutral axis to outer fiber, and
I = moment of inertia of cross section through pipe wall.

The validity of the principle of combined direct stress and bending stress is well documented by the strain measurements made by Scheer and Willett (9) in connection with the reconstruction of the Wolf Creek culvert.

It is important to take into account the bending moment in a flexible pipe for 2 principal reasons. First, it is associated with deflection of a pipe. Anyone who has walked through a corrugated steel pipe and observed the deformation of the pipe in relation to its original shape will recognize that the pipe wall is subjected to bending moment. Also it is important in connection with the design of bolted longitudinal seams. Such seams are subjected to a combination of tangential thrust and moment. The thrust causes stress in bolts or rivets at the seam in single shear, while the moment generates a prying action that may throw some of the fasteners into direct tension, so that the stress in the bolts is a combination of shear and tension. The present widespread practice of designing a bolted seam on the basis of its strength in single shear alone is workable only because of the application of a high factor of safety that masks the effect of combined shear and tension. But sometimes this procedure is not successful, as witness the extensive seam failures of the Wolf Creek culvert in Montana (10).

Bending moment in the pipe wall causes outer fiber stresses at the peaks of the corrugations, and these may very readily exceed the yield stress of the metal, even at moderate pipe deflections. The writer agrees with the authors that this circumstance is not detrimental to the performance of the pipe, unless, of course, the stress increases to the ultimate. The regions of such overstress are limited to short sections of the pipe perimeter at the top, bottom, and 2 sides. The plastic strains in these regions serve only to augment the deflection of the pipe. This, in turn, augments the development of passive resistance pressures and contributes to the overall strength of the pipe-soil system. It is also important to note that the load on a buried conduit is a one-shot affair that does not fluctuate widely, except possibly when the earth cover is very shallow and a substantial part of the load is attributable to surface traffic.

The authors devote considerable space to a discussion of performance limits for corrugated steel pipe, and the importance of this subject is obvious. Much of their discussion appears to be based on the performance of the experimental pipes of the Utah tests. The writer believes that the place to look for evidence on which to base design limits is in the field, by examination of the performance of actual pipes in service under actual soil embankments. Some field observations have revealed 2 major types of phenomena that contribute to failure of this kind of structure, although the term "failure" has not been defined completely or satisfactorily. These phenomena are (a) excessive deflection of the pipe ring, or (b) distress in longitudinal seams either by failure of the bolt fasteners or by failure of the pipe metal due to bending moment stress in the vicinity of a seam, or (c) both of these. A secondary type of distress may develop at transverse joints when adjacent rings deflect differentially.

Photographs of corrugated steel pipes in actual service are shown in Figures 13, 14, 15, and 16. Deflections in these pipes were excessive. Figures 17, 18, and 19 show excessive distress at the longitudinal seams, due both to bolt failure and to tension failure of the pipe metal adjacent to a seam. In the case of another pipe in which a seam failure was observed, many of the bolts pulled apart; whether they did so by shear or tension or a combination of these stresses could not be determined. In this case, the seam failures were relatively short in length and the pipe retained its essentially circular shape. The separated plates were jacked back together and welded. These

Figure 13. Complete collapse of 96-in. pipe.

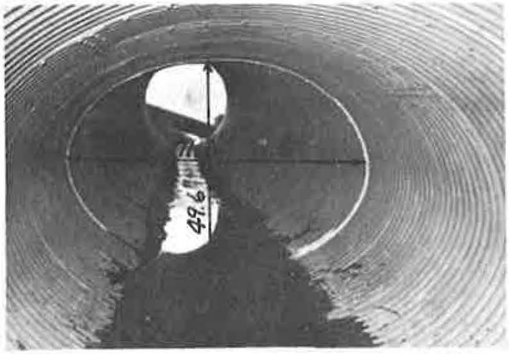


Figure 14. Excessive deflection of 60-in. pipe.

Figure 15. Excessive deflection of 84-in. storm sewer pipe.

Figure 16. Failure of transverse joint in pipe shown in Figure 15.

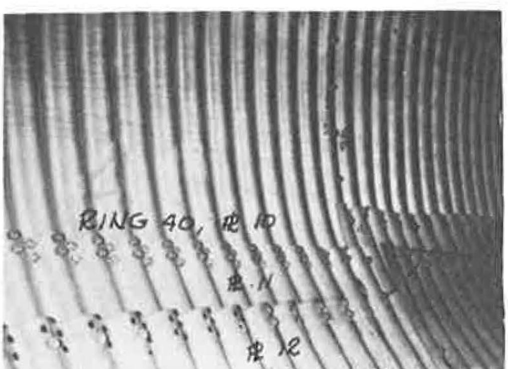


Figure 17. Longitudinal seam failure and circumferential deformation.

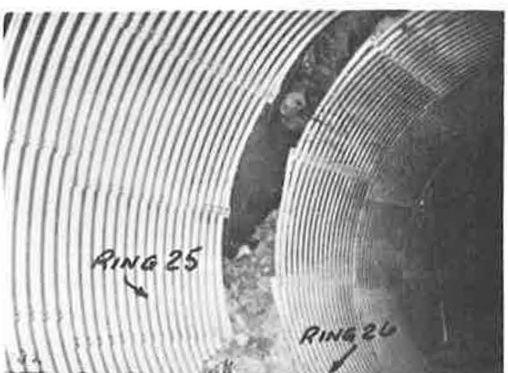


Figure 18. Failure of transverse joint due to differential deformation of adjacent rings.

repairs were satisfactory from a structural standpoint but, of course, the spelter coating in the vicinity of the welds was ruined.

The authors place considerable emphasis on the phenomenon of wall-crushing. In their tests it began with a "dimpling" of the wall, usually at 10 and 2 o'clock, then proceeded to an advanced stage of an accordion-like folding effect. The writer has neither seen nor heard of a similar phenomenon in the case of a structure under field loading. It is the writer's opinion that this type of effect was primarily induced by the unnatural and excessive confinement of the pipe test specimens provided by the shape and rigidity of the side walls of the load cell, by the hydraulic jacks, and by the unyielding type of soil backfill. Such confinement and restraint inhibited ring deflection and greatly increased tangential thrust that led to the crushing phenomenon.

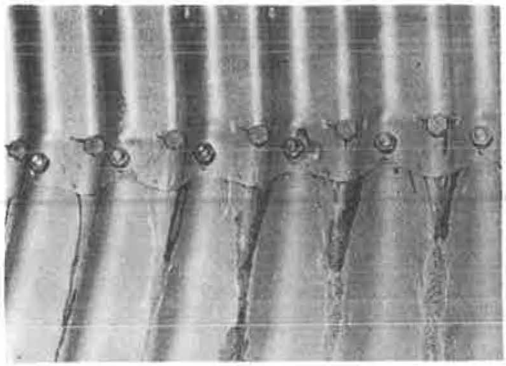


Figure 19. Tension failure due to bending moment at longitudinal seam.

The authors introduce a factor called the "pressure transfer coefficient" designated by the symbol, C_p. This coefficient is multiplied by the weight of a soil column above the horizontal plane through the top of the pipe in order to obtain the load on the pipe. It is a purely empirical factor, whose values have been determined from the test results with a single soil type in several states of density. The experimental results are shown in Figure 11, which is purported to be a widely applicable design diagram.

It is the writer's opinion that the Marston Theory (5, ch. 24; 11) of loads on buried conduits provides a more appropriate means of determining the earth load on a buried pipeline, regardless of whether it is a rigid or flexible type. Marston developed a theoretically sound method of evaluating the load transfer by arch action from or to the column of soil above the pipe, to or from the columns of soil immediately adjacent thereto. As shown in Figure 20, if the side columns of soil settle less than the interior column, that is, if the top of the pipe moves downward more than the critical plane, a part of the weight of the interior column is transferred by arch action to the exterior columns, and the load on the pipe is less than the weight of the interior column. If the reverse situation prevails, that is, if the critical plane settles more than the top of the pipe, as shown in Figure 21, an inverted arch action takes place, and additional load

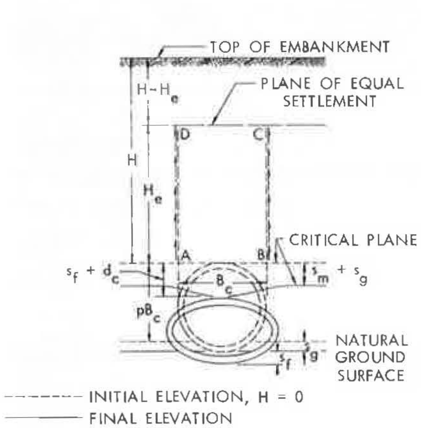


Figure 20. Negative settlement ratio for incomplete ditch condition.

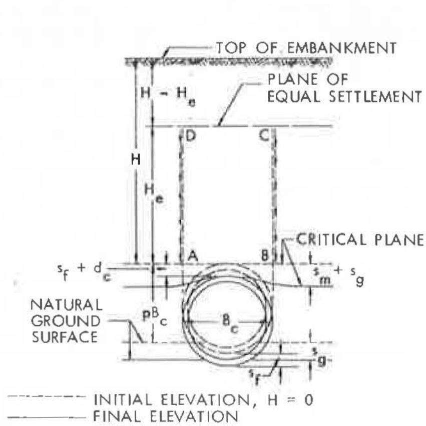


Figure 21. Positive settlement ratio for incomplete projection condition.

is transferred to the interior column and to the pipe. A neutral or transition case occurs when the critical plane and the top of the pipe settle downward the same amount, as shown in Figure 22. In this case no arch action develops and the load on the pipe is equal to the weight of the interior column.

The Marston load equation is

$$W_c = C_c w B_c^2 \tag{5}$$

where

W_c = load on conduit per unit of length,
C_c = calculation coefficient,
w = unit weight of soil, and
B_c = outside width of conduit.

The value of the coefficient C_c is a function of the ratio of the height of embankment to the width of the conduit, H/B_c, and of the product of the settlement ratio times the projection ratio. The projection ratio is equal to the distance from the natural ground surface to the critical plane, divided by B_c, as indicated in Figures 20, 21, and 22. It can be determined from the geometry of a proposed pipe installation. Values of C_c may be taken from data shown in Figure 23.

The settlement ratio is equal to the difference between settlement of the top of the conduit and the adjacent critical plane, divided by the compression strain of the pB_c column of soil. It is indicated by the formula

$$r_{sd} = \left[(s_g + s_m) - (s_f + d_c)\right]/s_m \tag{6}$$

where

r_{sd} = settlement ratio,
s_m = compression strain of columns of soil pB_c,
s_g = settlement of the natural ground surface adjacent to the conduit,
$(s_m + s_g)$ = settlement of the critical plane,

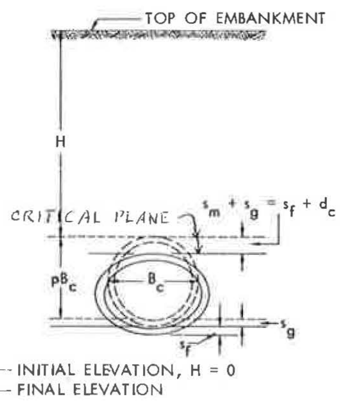
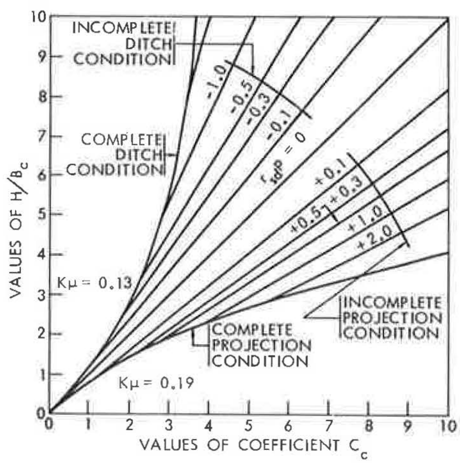


Figure 22. Zero settlement ratio for equal settlements.



Values of C_c in terms of H/B_c

Incomplete Projection Condition Kμ = 0.19		Incomplete Ditch Condition Kμ = 0.13	
$r_{sd}p$	Equation	$r_{sd}p$	Equation
+ 0.1	$C_c = 1.23H/B_c - 0.02$	- 0.1	$C_c = 0.82H/B_c + 0.05$
+ 0.3	$C_c = 1.39H/B_c - 0.05$	- 0.3	$C_c = 0.69H/B_c + 0.11$
+ 0.5	$C_c = 1.50H/B_c - 0.07$	- 0.5	$C_c = 0.61H/B_c + 0.20$
+ 0.7	$C_c = 1.59H/B_c - 0.09$	- 0.7	$C_c = 0.55H/B_c + 0.25$
+ 1.0	$C_c = 1.69H/B_c - 0.12$	- 1.0	$C_c = 0.47H/B_c + 0.40$
+ 2.0	$C_c = 1.93H/B_c - 0.17$		

Figure 23. Diagram for coefficient calculation.

s_f = settlement of the conduit foundation,
d_c = shortening of the vertical height of the conduit, and
$(s_f + d_c)$ = settlement of the top of the conduit.

Examination of Eq. 5 and Figure 23 reveals that, when r_{sd} is negative, the load on the pipe is less than the weight of the overlying column of soil; when it is positive, the load is greater than the weight of the soil column; and when it is zero, the load is equal to this weight.

Although the settlement ratio is completely rational in the development of the Marston Theory, it cannot be evaluated for a particular culvert in advance of construction without extensive soil tests and computations, which are expensive and impractical. It is, therefore, considered to be a semi-empirical constant, usable values of which can best be determined by observation of the settlements and environmental characteristics of actual conduits. In this respect it is similar to many such semi-empirical constants that are prevalent in engineering practice, as, for example, the coefficient of roughness in the Manning Formula for hydraulic flow. A very few field measurements (11) of the settlement ratio for flexible conduit installation have been made. Those few indicate support for the currently widespread practice of designing flexible culverts to carry the weight of the overlying column of soil, that is, assuming that the settlement ratio is equal to zero. Many more field measurements are needed before the determination of a design load on this type of conduit can be considered to be on a reliable basis.

When the load to which a flexible conduit will be subjected in service has been determined, it is possible to estimate the probable deflection of the pipe and stresses in the pipe wall by means of the Iowa formula and the associated stress formulas (1, 5). The Iowa formula is

$$\Delta X = D_1\left[(KW_c r^3)/(EI + 0.061\ E' r^3)\right] \tag{7}$$

where

ΔX = horizontal deflection (vertical deflection is essentially the same),
D_1 = deflection lag factor,
W_c = load on pipe per unit length,
K = bedding width factor,
r = radius of pipe,
E = modulus of elasticity of pipe material,
I = moment of inertia of cross section of pipe wall,
E' = er, modulus of soil reaction, and
e = modulus of passive resistance of soil.

The deflection lag factor is an empirical quantity that was introduced into the deflection equation as a result of observations of the fact that pipe deflection sometimes continues to develop for a substantial period of time after the maximum load is applied. It results from a yielding of the soil at the sides of the pipe in response to continuing pressure between the pipe and the soil. Values of this factor are related to the strain-resistant characteristics of the side fill soil. For loose soil the lag factor is relatively high. For dense well-graded soil it is essentially unity and can be ignored.

Although Watkins, the principal author of the paper, played a major role in the refinement and revision of the Iowa formula in 1957 (12, 13), he has since repudiated it. In a document dated February 21, 1970, he stated (14): "In fact, I don't accept the Iowa Formula as an adequate method for predicting deflection of pipes. . .because the Iowa Formula has not predicted precisely the deflection in pipes in the field and generally speaking the Iowa Formula has predicted more deflection than has been measured. . . ." Presumably he has field data on which this repudiation is based, but the writer has not seen it. The extent to which the statement is true merely reflects the state of uncertainty relative to design values of E' and D_1, coupled with the desire of most designers to be conservative in a situation in which specific information is scarce. The writer does not share the author's lack of confidence in the deflection formula, when appropriate values of the various factors are employed, although it is known that some people in the corrugated steel pipe industry agree with his statement.

When the Iowa Formula was developed, field experiments with flexible pipes under actual embankments indicated the validity of the concept of a modulus of soil reaction, and they yielded some specific values of this factor for a very limited number of soils. Since publication of the equation, its application to actual situations (9) has revealed that this modulus varies over a very wide range—from as little as 234 psi to as much as 8,000 psi, a 34-fold variation. The soil properties that influence this factor are somewhat obscure although qualitatively it is certain that texture and density characteristics are of prime importance. Probably moisture content is also influential.

Several investigators have attempted to determine the modulus of soil reaction E′ by direct laboratory measurements, but without success. Spangler and Donovan (15) tried in 1957. Watkins and Nielsen (16) later developed the Modpares device (acronym for modulus of passive resistance) for this purpose in 1964. Nielsen (17) developed a correlation between modulus of soil reaction and soil properties, particularly the CBR, but this correlation has not been widely tested. The writer's conclusion from these attempts is that E′, like the settlement ratio, should be treated as a semi-empirical constant.

The writer's appraisal of the current situation with respect to the design of flexible culvert pipes is that we have available theoretically sound procedures for estimating loads and predicting deflections and stresses in a proposed structure by means of the Marston Theory, Spangler's Iowa Formula, and associated stress equations. The practical application of these theories is hampered at the present time by lack of reliable values of certain semi-empirical constants, such as the settlement ratio, the deflection lag factor, and the modulus of soil reaction.

The best way and, in fact, the only reliable way to obtain usable values of these constants is to mount a massive program of study and measurement of a large number of actual flexible pipe conduits in the field at the time they are being constructed and subsequent thereto. It is recommended that the American Iron and Steel Institute, the Federal Highway Administration, the Corps of Engineers, the Aluminum Pipe Industry, the various state highway departments, and other interested parties undertake such a program. The data for each installation should include—in addition to the height of fill, the size of pipe, the gage of metal, and the type, depth, and spacing of corrugations—(a) the pipe bedding; (b) the projection ratio; (c) a complete description of the soil, particularly its texture, density, and moisture content; (d) settlements of the top of the conduit; (e) settlements of the critical plane and the natural ground surface; (f) deflections of the pipe both during construction and for a period of time after completion; (g) load on the pipe either reliably estimated or measured; and (h) all additional pertinent data that may become available during and after construction. With a complete record of the environment and performance of a large number of individual installations, encompassing many soil types at various densities, good reliable values of the semi-empirical constants needed for design of this type of structure will become available.

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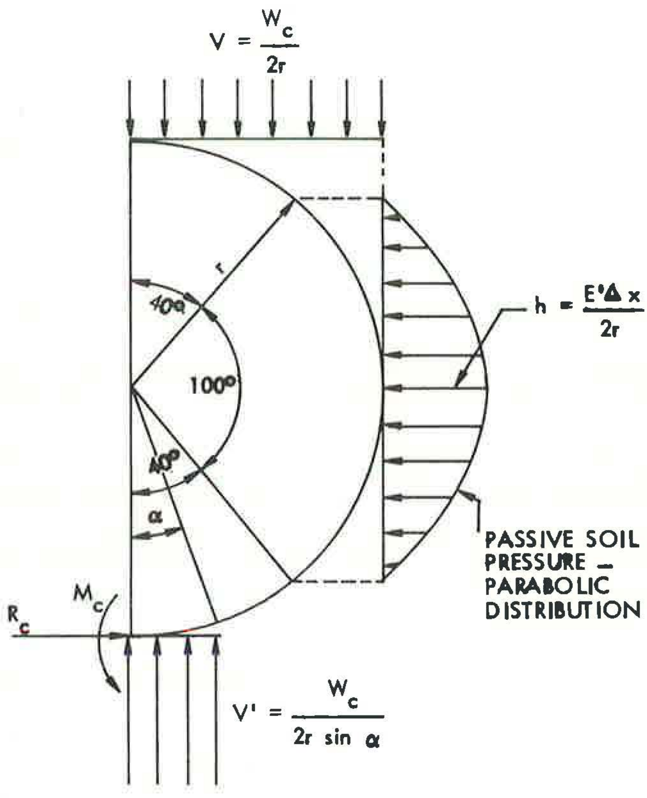
Appendix



LOAD SYSTEM

Deflection Formula

$$\Delta x = D_1 \frac{KW_c r^3}{EI + .061E' r^3}$$

α	K
0	0.110
35^o	0.100
60^o	0.090

Moment and thrust at bottom of pipe due to horizontal load:

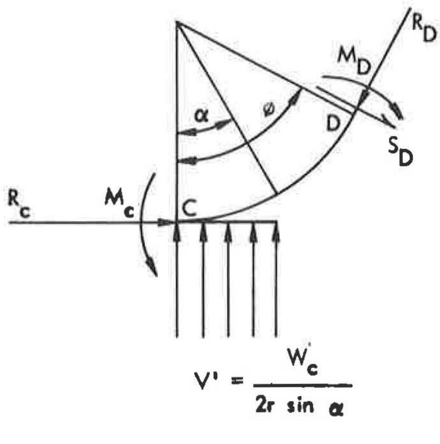
$M_c = -0.166\ hr^2$ $\qquad\qquad$ $R_c = 0.511\ hr$

Moment and thrust at bottom of pipe due to vertical load:

$M_c = AW_c r$ $\qquad\qquad$ $R_c = BW_c$

α	A	B	sin α
0	0.294	0.053	0
15	0.234	0.050	0.259
30	0.189	0.040	0.500
45	0.157	0.026	0.707
60	0.138	0.014	0.866

Moment, thrust and shear due to vertical load:



$$0 \leqq \phi \leqq \alpha$$

$$M_D = W_c r \left[A + B\,(1 - \cos\phi) - 0.250\,\frac{\sin^2\phi}{\sin\alpha} \right]$$

$$R_D = W_c \left(0.500\,\frac{\sin^2\phi}{\sin\alpha} - B\cos\phi \right)$$

$$S_D = W_c \left(0.500\,\frac{\sin\phi\cos\phi}{\sin\alpha} - B\sin\phi \right)$$

$$\alpha \leqq \phi \leqq 90^{\circ}$$

$$M_D = W_c r\,[A + B(1 - \cos\phi) - 0.50\sin\phi + 0.25\sin\alpha]$$

$$R_D = W_c\,(0.500\sin\phi + B\cos\phi)$$

$$S_D = W_c\,(0.500\cos\phi - B\sin\phi)$$

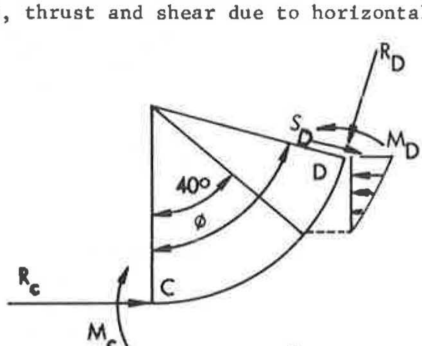
$$90^{\circ} \leqq \phi \leqq 180^{\circ}$$

$$M_D = W_c r\,[A + B(1 - \cos\phi) - 0.25\,(1 + \sin^2\phi - \sin\alpha)]$$

$$R_D = W_c\,(0.50\sin^2\phi + B\cos\phi)$$

$$S_D = W_c\,(0.50\sin\phi\cos\phi - B\sin\phi)$$

Moment, thrust and shear due to horizontal load:



$$0 \leqq \phi \leqq 40^{\circ}$$

$M_D = hr^2(0.345 - 0.511 \cos \phi)$

$R_D = 0.511 \; hr \cos \phi$

$S_D = 0.511 \; hr \sin \phi$

$$40^\circ \leqq \phi \leqq 140^\circ$$

$M_D = hr^2(0.199 - 0.500 \cos^2\phi + 0.143 \cos^4\phi)$

$R_D = hr(\cos^2\phi - 0.568 \cos^4\phi)$

$S_D = hr(\sin \phi \cos \phi - 0.568 \sin \phi \cos^3\phi)$

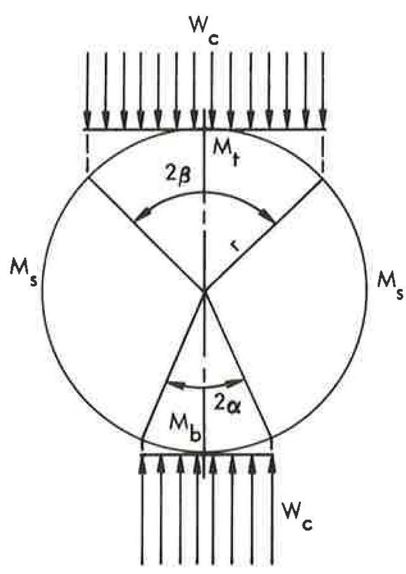
$$140^\circ \leqq \phi \leqq 180^\circ$$

$M_D = hr^2(0.345 + 0.511 \cos \phi)$

$R_D = - 0.511 \; hr \cos \phi$

$S_D = 0.511 \; hr \sin \phi$

Combine stresses due to vertical and horizontal loads algebraically.



MOMENTS: $M = KW_c r$

DEFLECTIONS: $\Delta = K \dfrac{W_c r^3}{EI}$

USE: K_b FOR MOMENT AT BOTTOM
K_t FOR MOMENT AT TOP
K_s FOR MOMENT AT SIDES
K_x FOR HORIZONTAL DEFLECTION
K_y FOR VERTICAL DEFLECTION

2β, deg.	2α, deg.	K_b	K_t	K_s (neg.)	K_x	K_y
	0	0.318	0.318	0.182	0.137	0.149
	30	0.259	0.317	0.180	0.135	0.146
	60	0.213	0.312	0.175	0.130	0.138
0	90	0.182	0.305	0.168	0.122	0.129
	120	0.162	0.299	0.161	0.116	0.122
	150	0.153	0.295	0.156	0.111	0.117
	180	0.150	0.294	0.153	0.110	0.116
	0	0.317	0.259	0.180	0.135	0.146
	30	0.257	0.257	0.178	0.133	0.143
	60	0.211	0.252	0.173	0.127	0.135
30	90	0.180	0.246	0.166	0.120	0.127
	120	0.160	0.240	0.159	0.114	0.119
	150	0.151	0.236	0.154	0.109	0.115
	180	0.148	0.235	0.152	0.108	0.113
	0	0.312	0.213	0.175	0.129	0.138
	30	0.252	0.211	0.173	0.127	0.135
	60	0.207	0.207	0.168	0.122	0.127
60	90	0.175	0.201	0.161	0.115	0.118
	120	0.156	0.194	0.154	0.109	0.111
	150	0.146	0.190	0.149	0.104	0.107
	180	0.143	0.189	0.147	0.103	0.105
	0	0.306	0.182	0.168	0.122	0.129
	30	0.246	0.180	0.166	0.120	0.127
	60	0.201	0.175	0.161	0.115	0.118
90	90	0.169	0.169	0.154	0.108	0.110
	120	0.150	0.163	0.147	0.101	0.103
	150	0.140	0.158	0.142	0.097	0.098
	180	0.137	0.157	0.140	0.096	0.096
	0	0.299	0.162	0.161	0.116	0.122
	30	0.240	0.160	0.159	0.114	0.119
	60	0.194	0.156	0.154	0.109	0.111
120	90	0.163	0.150	0.147	0.101	0.103
	120	0.143	0.143	0.140	0.095	0.096
	150	0.134	0.139	0.135	0.091	0.091
	180	0.131	0.138	0.133	0.089	0.089

2β, deg.	2α, deg.	K_b	K_t	K_s (neg.)	K_x	K_y
	0	0.295	0.153	0.156	0.111	0.117
	30	0.236	0.151	0.154	0.109	0.115
	60	0.190	0.146	0.149	0.104	0.107
150	90	0.158	0.140	0.142	0.097	0.098
	120	0.139	0.134	0.135	0.091	0.091
	150	0.129	0.129	0.129	0.086	0.086
	180	0.126	0.128	0.128	0.085	0.085
	0	0.294	0.150	0.153	0.110	0.116
	30	0.235	0.148	0.152	0.108	0.113
	60	0.189	0.143	0.147	0.103	0.105
180	90	0.157	0.137	0.140	0.096	0.096
	120	0.138	0.131	0.133	0.089	0.089
	150	0.128	0.126	0.127	0.085	0.085
	180	0.125	0.125	0.125	0.083	0.083

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The basis and keystone of the highly simplified method for the design of corrugated steel pipe as presented by the authors is the apparent ring compression strength f_c. The values of this empirical parameter were obtained from full-scale tests utilizing 1 type of soil and 130 pipe sections ranging in size from 3- to 5-ft diameters, and for 3 different corrugation configurations. These f_c values are shown in Figure 5 and are related to the curves shown in Figure 11.

Because these curves assume such a major role in the application of the proposed design method, the writer would like to inquire about the rationale of their construction. The validity of the curves is strongly dependent on the distribution of the 130 data points from the test results. However, in the absence of a graphical display or a discussion and statistical description of the dispersion of these points, the implications and significance of the curves shown in Figures 5 and 11 become suspect. The proper significance of the curves could be easily evaluated by the reader if the authors would present information concerning the distribution of the data points with respect to soil densities, corrugation configurations, and pipe diameters.

For purposes of this discussion the essential features shown in Figures 5 and 11 have been reproduced and are shown in Figures 24 and 25 respectively. Each figure has been subdivided into 4 zones as noted along the top of the figure. The upper bounds for zones I, II, and III are determined on the basis of the scale value for a 5-ft diameter pipe for each of the 3 corrugation configurations tested. The significance of these bounds is that they correspond to the maximum diameter of pipe tested in the investigation. Thus, it appears that no test data were obtained for establishing the shape of the design curves in zone IV. Consequently, these curved portions of the diagrams are shown as dashed lines in Figures 24 and 25.

Below the abscissa in the 2 figures are bar scales with tick marks indicating the 3 diameters of the test pipe (3, 4, and 5 ft) for each of the 3 corrugation configurations studied. The scale for the test pipe having the 6 by 2 corrugation extends over only a small portion of the diameter scale of zone I. In contrast, the bar scales for the test pipe having 3 by 1 and 2⅔ by ½ corrugations cover almost the entire range of zones II and III respectively. Thus, the basis for establishing the shape of the design curve within zones I, II, and III is dependent on only 1 corrugation configuration; no overlapping of test data for different corrugations was possible. The authors state, "It is noteworthy that the strength envelopes dip down to the right with increasing flexibility." Figures 24 and 25 show that the greatest amount of "dipping" occurs in the dashed curves in zone IV.

The authors also remark, "Corrugation depths of 1, 0.5, and 0.25 in. were tested." A knowledge of the distribution of the 130 test pipes with respect to corrugation configuration and diameter is of extreme importance. This is especially true for the case of the $2\frac{2}{3}$ by $\frac{1}{2}$ corrugation because these data are used to establish the behavior of the curves in zone III. This 1 zone occupies the major portion of the diagram representing regions for which test data were obtained. Consequently, this zone serves the unique function of establishing the basis for the dramatic changes in the slopes of the design curves and possibly justifying the extrapolation of the curves to larger pipe diameters.

The writer would like to inquire as to the statistical basis of the design curves; i.e., What are the correlation coefficient, standard deviation, and the standard error of the estimate of the f_c curves shown in Figure 5?

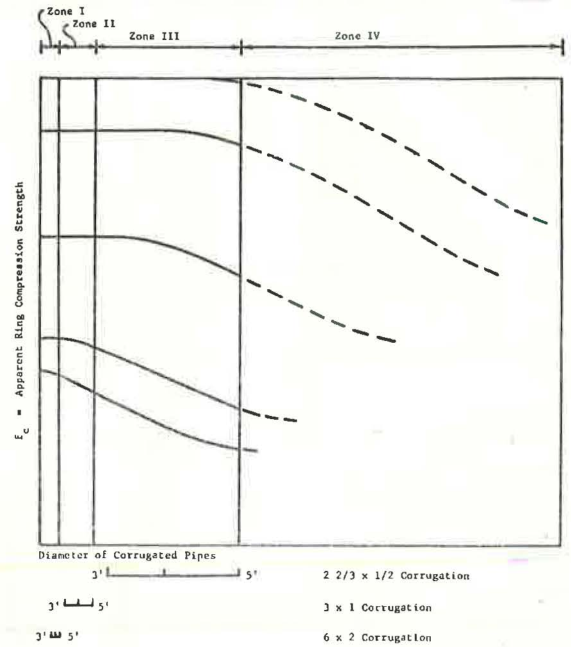

Figure 24. Apparent ring compression strength from Figure 5.

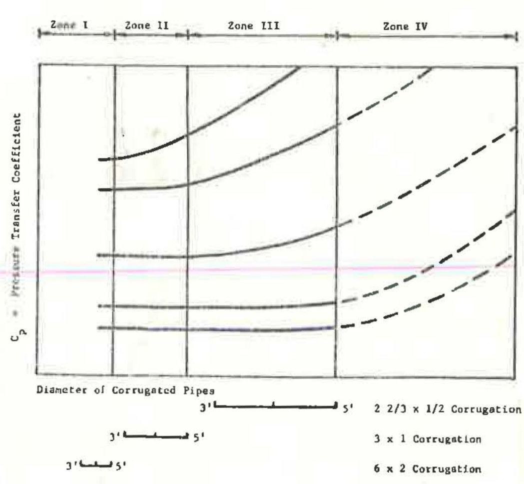

Figure 25. Pressure transfer coefficient from Figure 11.

AUTHORS' CLOSURE

The basic method proposed for designing corrugated steel pipes is the method of apparent ring compression strength. Empirical ring compression strength must exceed the calculated ring compression stress. The method is widely used and understood. It applies to standard culvert steel (33,000 to 40,000-psi yield point) and so does not raise the academic question concerning performance of conduits of much higher or much lower yield points.

A more general design method, suggested in the last section of the Appendix, is based on yield point strength. In this case the ring compression stress is modified by a pressure transfer coefficient. Both methods are simple to use. When corrugated steel culverts of extremely high or extremely low yield point are manufactured, additional charts can be prepared to provide the apparent ring compression strength.

As the paper indicates, the apparent ring compression strength provides automatic correction for flexural stress in the wall, relative compressibility of the soil and pipe, and effect of seams in standard corrugated steel pipes.

The test cell was designed to duplicate field conditions. The elliptical shape of the cell was selected to maintain soil stresses of P vertical and P/3 horizontal. The cell was calibrated by placing soil pressure gages at several locations in the soil and then loading the cell. The calibration gave vertical soil pressure anywhere in the cell as a function of the applied load. Using the calibration data, we presented the apparent ring compression strength envelopes as a function of the pressure at the top of the pipe if no pipe were in place.

The first tests were run without placing steel loading plates on top of the soil. Some penetration resulted, so plates were introduced. However, the load at the performance limit was not significantly affected by the use or absence of loading plates.

The concrete retaining walls were constructed only to hold the flexible cell in its approximate elliptical shape during soil placement. The flexible cell is drawn away from the concrete retaining walls during pressure loading of the test cell. Actual tests to determine the boundary effect of the cell on pipes of different diameters show that boundary effect exists but is not significant compared with other pertinent variables. The most conservative (lowest) strength envelope for various diameters is plotted for each soil density. The conservative test cell boundaries are adequate when one contemplates field boundary conditions. In the field, how compressible is the bedding?

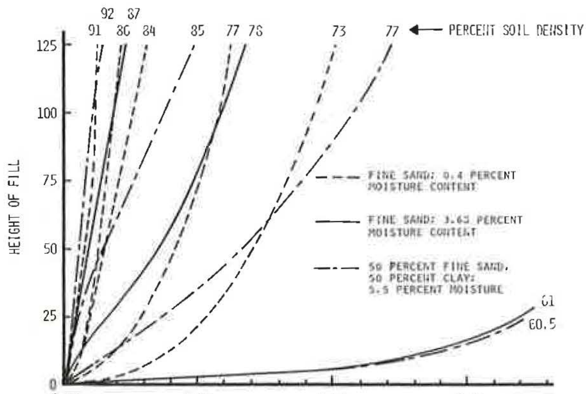

Figure 26. Typical soil compression diagram for different soil types at different densities (tests were modified consolidation tests).

How uniform is the trench cross section? How homogeneous is the boundary soil? How uniform is the fill soil?

In the selection of test pipes, the very flexible rings were achieved by special $2\frac{2}{3}$ by $\frac{1}{4}$ corrugations (up to 5 ft diameter) that extend the ring flexibility values out to the equivalence of a 10-ft diameter pipe in the standard $2\frac{2}{3}$ by $\frac{1}{2}$ corrugation. Similarity is ensured. In this way the full plots are based on actual test data.

The test soil was selected because of the broad range of densities to which it can be compacted. It was found that the most important soil property in the performance of buried flexible conduits is soil compressibility, E′. A better term is soil stiffness. Soil stiffness is affected mostly by soil density. Figure 26 shows a few typical vertical compression diagrams for different soil densities. The 2 most important observations from this figure are that (a) soil density is more important than any other variable (including soil type) in determining the soil stiffness E′ (slope of secant to some curve at some given height of fill) and that (b) the diversity of compression diagrams points out the reality that soil is complex, and different soils do not perform exactly the same. Many variables (soil friction angle, Poisson's ratio, and moisture content in sand) must be handled as secondary soil properties. Fortunately the total range of variation of these secondary soil properties is less significant (has less effect on performance) than the probable deviation due to soil density and soil placement techniques. Moreover, even though the soil is important in buried conduit performance, it is only 1 of 2 components in the system. The conduit also influences performance and contributes to the standard deviation.

Granted that soil stiffness E′ is the most important soil property, soil stiffness is not quickly and easily determined. On the other hand, soil density is understood. It can be determined rapidly by standard techniques in the field as a control during the placement of backfill. Greater sophistication is probably not justified under the variability of common installation techniques. In the future it may become possible to select and place the soil with such homogeniety that E′ and even additional soil properties will become significant.

For soil placed at density greater than critical void ratio, and excluding wet soil with a substantial fraction of fines (such as viscous soil), soil density is the most important criterion of soil stiffness E′. Any exceptions to the density criterion would be a very special type of soil. For example, a spongy soil (high organic content) would be more compressible (less stiff) than granular soil at the same percentage of density because of rebound. However, highly organic soil would be suspect as backfill. If used, a special test would be advisable. Viscous soil (mud) is another exception, but a conduit in viscous soil would be analyzed by classical theories for collapse.

The Marston-Spangler method of ring design is based on many empirical observations (settlement ratio, bedding angle, lag factor, plane of equal settlement, projection ratio, and modulus of passive resistance). Because of the difficulty of obtaining some of the empirical values, the Marston-Spangler method does not lend itself to easily understood and usable design. All of these empirical variables are really functions of more basic variables such as soil stiffness (soil density), ring stiffness, yield point, and soil placement techniques. So why not use the more basic variables—especially when they are measurable?

In the future as the soil properties as well as conduit materials are controlled within close tolerances, highly theoretical computer methods for analysis will take over. However, if performance limit is deformation, the soil does not perform as an elastic medium. Shearing planes develop, and for analysis the inclusion of friction angle and soil cohesion is required. The precise analyses of the future must include Poisson's ratio and the anisotropy and nonhomogeniety that result from soil compaction. Essential also will be the effect of time lag in soil consolidation and the trench or embankment boundary conditions. Near the conduit, where compaction is so difficult, the effect of compaction is most critical. Installations of the future may well include a special compressible backpacking about the conduit.

Until soil control and placement techniques justify such precision, the statistical, empirical design procedure proposed here is the most realistic approach.