3. J.M. Duncan, P.M. Byrne, K.S. Wong, and P.N. Mabry. Hyperbolic Volume Change Parameters for Nonlinear Finite Element Analyses of Stresses and Movements in Soil Masses. Univ. of California, Berkeley, Geotechnical Engineering Rept., 1979.

4. J.M. Duncan. Soil-Culvert Interaction Method for Design of Metal Culverts. TRB, Transportation Research Record 678, 1978, pp. 53-59.

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Finite-Element Modeling of Buried Concrete Pipe Installations

ERNEST T. SELIG, MICHAEL C. McVAY, AND CHING S. CHANG

A finite-element computer program, Soil-Pipe Interaction Design and Analysis (SPIDA), was developed for buried concrete pipe. The purpose of the program is to update the current concrete pipe design methods based on the Marston-Spangler approach with the expectation of reducing the cost of the installations and providing a more accurate representation of field conditions. Separate computer models were prepared for a positive-projecting embankment installation and a vertical-sided trench installation to optimize the representation of these two different cases. A wide range of bedding and soil conditions can be simulated. Experience with SPIDA indicates that it gives reasonable results. The program has advanced to the stage where it is ready for trial applications in pipe design.

Research was undertaken to develop a finite-element computer program for design and analysis of buried concrete pipe. The resulting program, Soil-Pipe Interaction Design and Analysis (SPIDA), is described and some examples are given of results obtained with it. Some of the details of the supporting research have been given elsewhere (1).

The purpose of the program is to update the current concrete pipe design methods based on the Marston-Spangler approach. This effort is expected both to reduce the cost of the installations and to provide a more accurate representation of the wide range of conditions experienced in the field.

The guidelines for the development of the computer program were to

1. Achieve low computer cost,
2. Be able to adapt to a wide range of field conditions,
3. Accurately represent characteristics of both trench and embankment installations,
4. Accurately model behavior of reinforced concrete pipe,
5. Permit easy extension to new conditions as experience develops, and
6. Provide simplicity of use by pipe designers.

In spite of the many past efforts to develop finite-element models for soil-structure interaction of buried pipe and conduits, none of the available programs were able to adequately satisfy all of the above guidelines. For example, neither the Northwestern University nor the Culvert Analysis and Design (CANDE) program had incorporated the desired soil models or a suitable reinforced concrete pipe model. These inadequacies together with mesh deficiencies and high computer cost that also exist with NUPIPE precluded the use or modification of that program to achieve the desired objectives. Recent changes in CANDE have improved the suitability of that program, but CANDE was not so easy to modify to meet the specific objectives of this study as the development of SPIDA, and it costs more to run. Only after a careful consideration of all other options was the decision made to begin with a new approach.

GENERAL MODEL FEATURES

The problem was divided into two subgroups, one for embankment installations and one for trench installations. This independent treatment permitted the development of the optimum computer model for each subgroup.

The basic features of the trench model are shown in Figure 1. The sides of the trench are vertical, and no slip is permitted between the backfill soil and the sides of the trench. The backfill is placed in layers to simulate construction. Bedding conditions are represented by the choice of properties of the material under the pipe and the geometry of the bedding zone. The trench depth and width are each variable independent of the pipe diameter. Tight sheeting for trench support is not considered.

The basic features of the embankment model are shown in Figure 2. The soil above the existing ground represents an embankment constructed of placed backfill. The main emphasis in the embankment model development was devoted to the positive-projecting conduit case (Figure 2a). However, the induced-trench condition produced by a compressible zone above the pipe can be simulated by placing soft material in the designated zone in the computer model (Figure 2b). The negative-projecting case (Figure 2c) can be simulated by changing the location of preexisting ground from that in the positive-projecting case.

The following general features apply to both the trench and the embankment models:

1. Two-dimensional plane strain representation of the installation,
2. Symmetry about the vertical pipe centerline,
3. Circular reinforced-concrete pipe,
4. No slip between the soil and the pipe and between the placed backfill and the existing ground,
5. Incremental placement of the soil in layers,
6. Bedding conditions variable to fit actual field conditions, and
7. Representation of any soil type and compaction state in any location.

Time-dependent properties of the soil and pipe have not yet been incorporated nor has a procedure for handling live loads been developed.

Figure 1. Basic features of trench model.

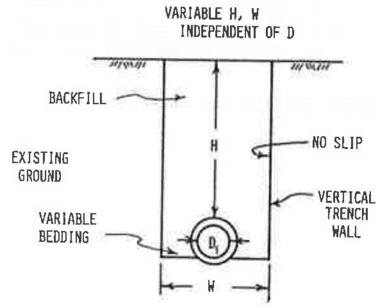
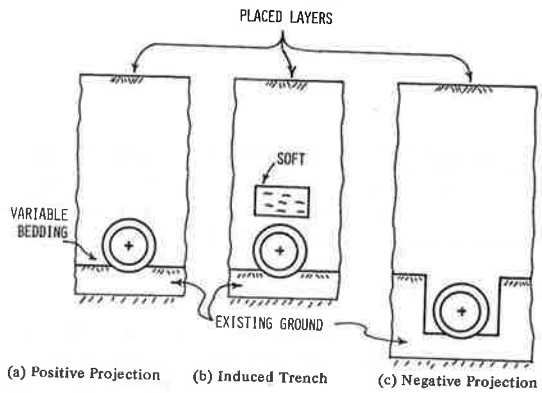


Figure 2. Basic features of embankment model.



CONCRETE PIPE MODEL

The concrete pipe model was developed by Simpson Gumpertz and Heger, Inc. It consists of a half ring with 17 finite elements. Each element is a straight line between nodes located at the mid-thickness of the pipe wall. The element lengths are sufficiently short so that the beam properties may be taken constant with distance along the element. When the wall has cracked, the moment of inertia of the beam element that represents the portion of the wall with the crack is reduced according to an expression for effective moment of inertia similar to an equation given in an American Concrete Institute publication (2) but with coefficients determined as described below.

The model represents a circular pipe with two circular cages of reinforcing steel. The variables that define the reinforcement cages are the wire diameters of the inside and outside steel, the inside and outside concrete cover depths, the number of wires per foot of pipe length, and the type of bond between the steel and concrete. The concrete is described by its compressive strength (f_c') and a bilinear stress-strain relation; the knee in the stress-strain curve is at $f_c = 0.5f_c'$. The remaining pipe variables are the inside diameter and wall thickness.

The pipe model is capable of simulating the nonlinear load-deflection relation observed in three-edge bearing tests. Three conditions of the pipe wall are included in the model: an uncracked wall, a cracked wall in which the maximum compressive stress of the concrete is less than $0.5f_c'$, and a cracked wall in which the maximum compressive stress exceeds $0.5f_c'$.

The coefficients in the equations for cracked section moment of inertia were obtained by using three-edge bearing load-deflection plots. They vary with type of reinforcement: (a) smooth wires or rods, (b) welded smooth wire fabric with cross wires spaced at 6 or 8 in, and (c) deformed wire, wire fabric, or bar reinforcements or any reinforcement type with radial ties (transverse stirrups). The latter types provide better crack control and thus greater stiffness than the reinforcement with poorer bond characteristics.

The following assumptions were used:

1. The displacements in the pipe are small compared with its wall thickness and the strains are small compared with unity;
2. Shear deformation is negligible so that longitudinal planes before deformation remain plane after deformation in cracked or uncracked sections;
3. The stress-strain relation for steel is linear up to the yield point;
4. The stress-strain relation for concrete is bilinear up to f_c' with a knee at 0.5 f_c', as described above; and
5. The effect of the dead weight of the pipe can be included as an initial elastic stress.

The pipe model is valid for stresses in steel that are below the yield stress and for stresses in concrete that are below f_c'.

SOIL MODEL

Considerable research effort was devoted to determining the requirements for properly representing the soil behavior in the finite-element model (1). The research indicated that the following soil-model features were the most appropriate for the placed backfill soil:

1. Nonlinear relationship of stress to strain relevant to the stress conditions in culvert installations,
2. Parameters dependent on both shear-stress and confining-stress states,
3. Parameters that can be related to compaction or density state,
4. Yielding as stress state approaches strength limits, and
5. Parameters that can be estimated for design purposes without individual tests.

The most suitable model currently available that incorporates these features is the model that combines a hyperbolic Young's modulus with bulk modulus (3). This will be termed the E-B model. The basic features of this model are illustrated in Figure 3. For a given confining stress (σ_3), the relationship between deviator stress and axial strain up to failure is represented by a hyperbolic function in which the tangent soil stiffness decreases with increasing deviator stress. This stiffness increases with increasing confining stress and degree of compaction. Bulk modulus is assumed to be dependent only on σ_3. Thus volumetric strain also increases hyperbolically with axial strain up to failure. Increases in amount of compaction or decreases in σ_3 decrease the volumetric strain. At failure, the soil is given a very small constant stiffness.

Generally, the initial stress state of existing ground is unknown, particularly after trench excavation, and its stress-strain properties during backfilling cannot be accurately estimated from available information. Thus the existing ground is represented in the finite-element model only as a linear elastic material with constant Young's modulus and constant Poisson's ratio. However, a more precise model can be used for this material if conditions warrant it.

TRENCH MESH

The geometry of the finite-element mesh for the trench case is shown in Figure 4. The soil elements are all of the four-node, isoparametric type.

The trench wall can be located at either of the vertical lines shown. In addition, the distances C and H can be varied independently of the pipe average diameter D. These features permit flexibility in establishing the trench width. Values assigned to dimensions A and B establish the height of soil cover above the pipe.

The bottom of the trench may be located at the top of any of the elements directly beneath the pipe invert. The dimensions E, F, and G also can be independently varied to obtain the proper thickness of layers beneath the pipe. By the choice of these dimensions and the soil properties assigned to the elements directly beneath the pipe, a wide range of bedding conditions can be represented.

The investigations showed that the trench mesh must be extended to the full height of soil above the pipe, and hence surcharge pressure should not be substituted for finite elements, regardless of the height of soil cover. Computer costs are less if the mesh is stopped at one to two diameters of soil cover over the crown and the remaining soil is represented by surcharge pressure. However, the arching action of the backfill in the trench is lost in the zones where surcharge is used instead of elements. The number and heights of the elements above the pipe are selected by the computer program based on the value of dimension A and constraints on the element-aspect ratios. Also, as A increases, the dimension I automatically increases, so that the right boundary of the mesh remains far enough from the pipe to avoid influencing the response.

Any soil type and compaction condition can be assigned to any of the elements in the backfill zone and any values for the linear elastic properties to any of the existing ground elements. For convenience in computer input, a variety of element combinations have been designated as standard zones that each require only one set of soil properties. Elements within a zone can have different properties by an override feature.

The backfill soil is added in sequential layers in the computer analysis to simulate the placement of backfill in the field. The research showed that the layers must be kept small and begin below the pipe as shown, for example, in Figure 5.

The mesh in Figure 4 is designed primarily for cover heights of more than two to three times the pipe diameter. For cover heights less than this, a

Figure 3. Soil-model characteristics.

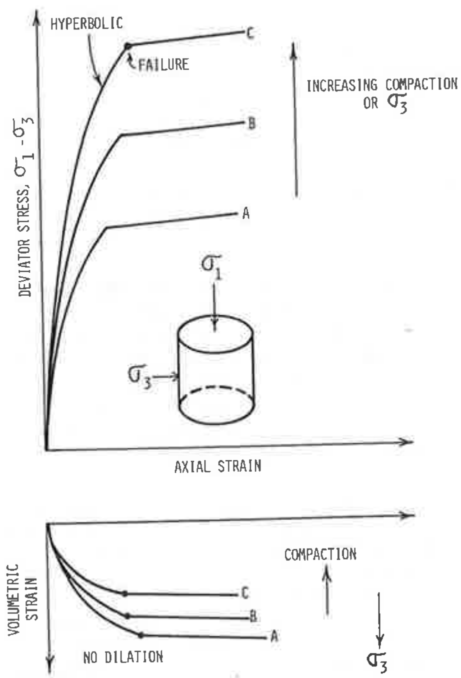


Figure 4. Trench finite-element mesh.

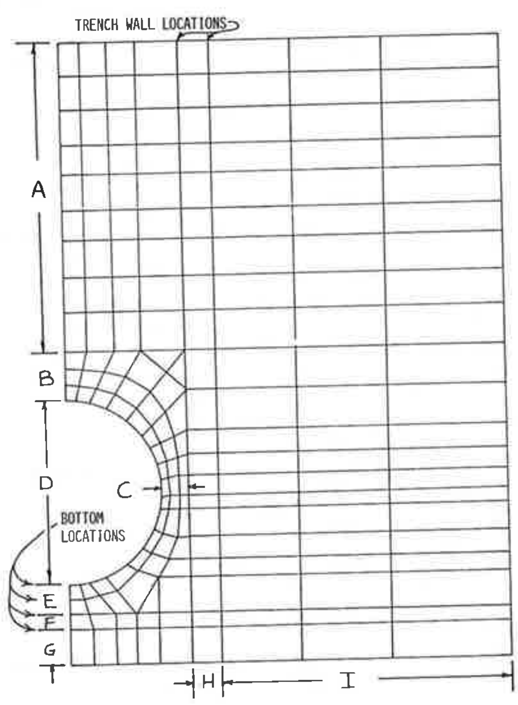


Figure 5. Backfill layers used in trench model.

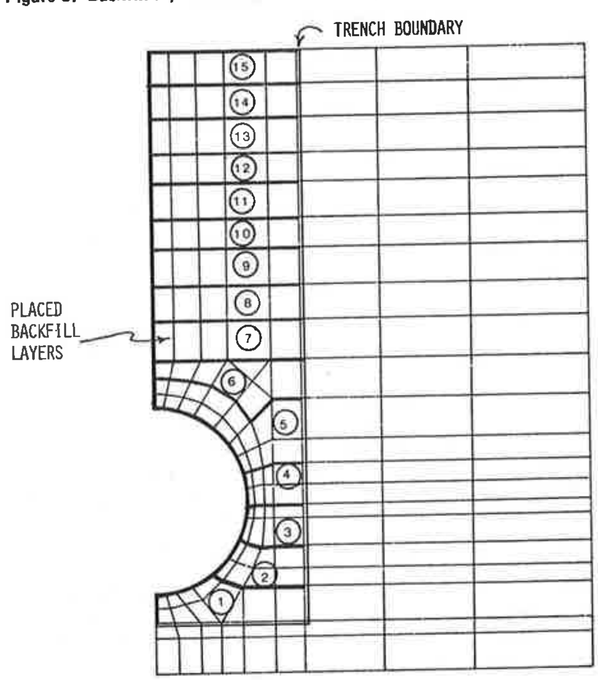


Figure 6. Embankment finite-element mesh.

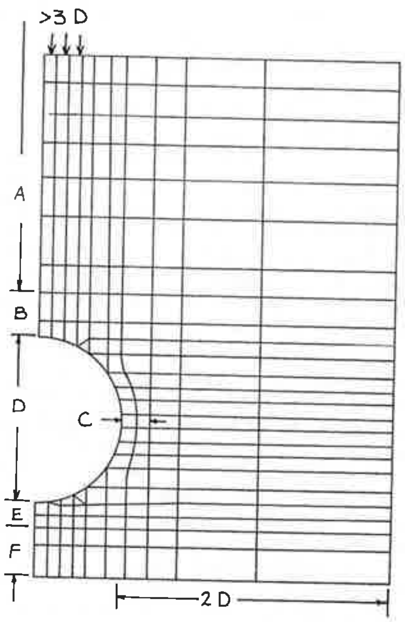
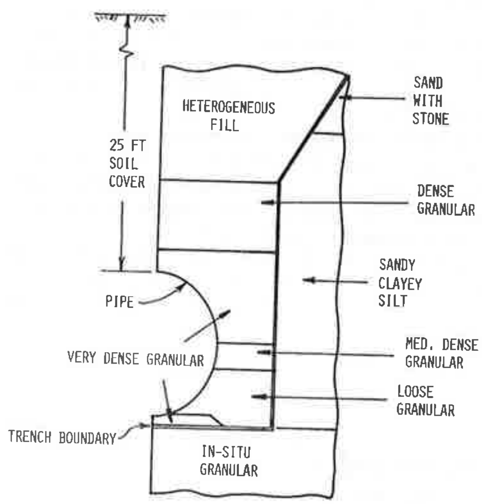


Figure 7. Bedding conditions for trench installation in East Liberty, Ohio.



mesh with fewer elements was developed to provide a lower computer cost. This mesh is similar to the one in Figure 4; the primary difference is the elimination of the outer vertical column of existing ground soil elements.

EMBANKMENT MESH

The geometry of the finite-element mesh for the embankment case is shown in Figure 6. The soil elements are the same as those in the trench case.

The average pipe diameter is D. The width of soil adjacent to the pipe springline is set at twice the pipe diameter. The existing ground can be specified anywhere below and beside the pipe to establish the proper projection ratio. All the soil above the existing ground is placed in horizontal layers.

Dimensions B, C, E, and F are selected to represent the geometry of the zone immediately around the pipe. The choice of soil properties for these completes the bedding definition.

The height of soil cover over the crown defines dimensions A and B. However, above a soil cover equal to three pipe diameters, the elements are replaced with surcharge pressure to reduce computer cost. This is possible with the embankment case, because above the pipe crown the full width of the layer is placed simultaneously and no existing ground is present to develop arching.

The assignment of soil properties by zone is the same in the embankment mesh as that described for the trench mesh.

COMPUTER OUTPUT

The computer program SPIDA permits a detailed output after each layer of the pipe and soil response, which includes the following:

1. Deflections at all node points in the soil and on the structure;
2. Stresses and strains in the center of all soil elements; and
3. Moment, thrust, and shear at all pipe nodes.

From this information, the following were obtained:

1. Distribution of normal stress and shear stress around the pipe from the surrounding soil;
2. Pipe vertical and horizontal diameter change;
3. Weight of prism of soil above crown of pipe and above springline of pipe;
4. Vertical soil geostatic stress at crown; and
5. Arching factor, defined as ratio of springline thrust to one-half the soil prism weight.

The first step of the computer program generates the finite-element mesh from the input dimensions. This mesh can be plotted automatically for inspection at the option of the user. This provides the opportunity to ensure that the geometry of the installation is being satisfactorily represented by the mesh.

An output table is also generated that indicates the occurrence of cracking on the inside or outside of the pipe at any node after each layer of backfill has been placed.

For the convenience of the analyst, a program has been prepared that can plot distributions around the pipe of the computed pipe moment and thrust and the soil normal stress and shear stress on the surface of the pipe.

TRENCH IN EAST LIBERTY, OHIO

To illustrate the SPIDA trench model, results will be shown for the computer simulation of an installation in East Liberty, Ohio. The details of this analysis are available elsewhere (1). This case involved a 60-in inside diameter, Class IV, B wall, reinforced-concrete pipe placed in a trench with 25 ft of backfill cover (4). The bedding conditions for the pipe are shown in Figure 7. Although the trench side wall sloped back beginning a few feet above the pipe, the vertical trench model was used as an approximation. The zones selected for the trench mesh are shown in Figure 8, but the individual elements, as in Figure 4, have been omitted for clarity.

The computer simulation began with the trench empty. The existing soil was assigned a constant Young's modulus and Poisson's ratio based on values suggested by Krizek and McQuade (4). The first layer of backfill shown in Figure 5 was placed, followed by the pipe, and then the remaining layers. The E-B model was used to represent all the backfill zones. The parameters were selected from values

Figure 8. Finite-element mesh zones of trench installation in East Liberty.

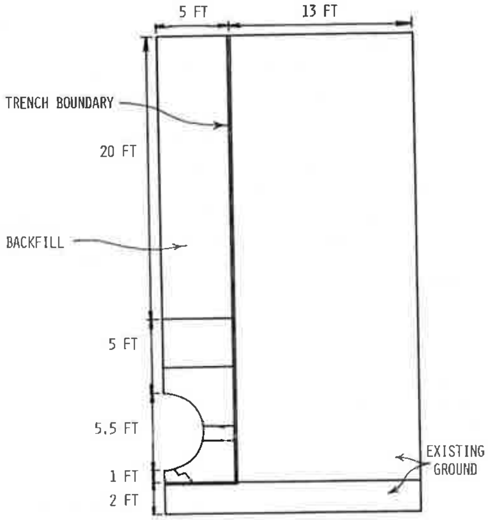


Figure 9. Predicted response for trench installation in East Liberty with vertical-side trench approximation.

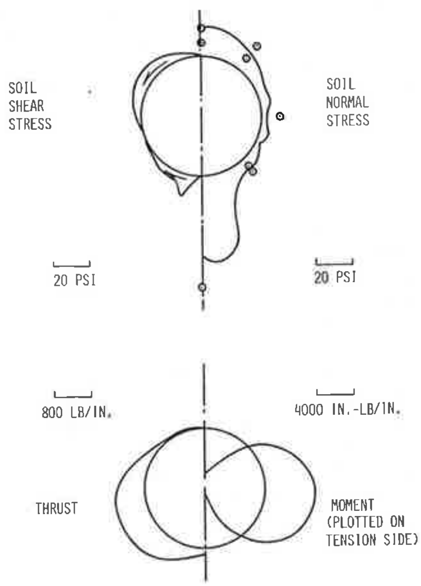
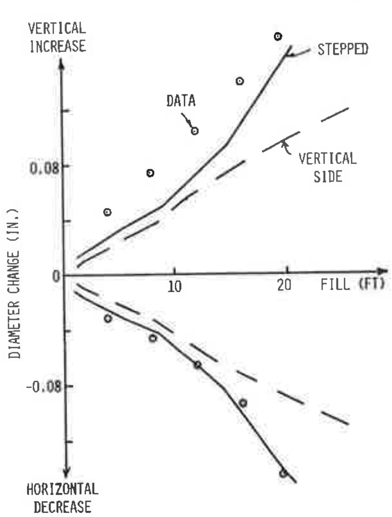


Figure 10. Pipe diameter change for trench installation in East Liberty.



recommended by Duncan and others (3) based on information on material type and compaction state from reports by Krizek and others (4-6). The specific values are given by McVay (1).

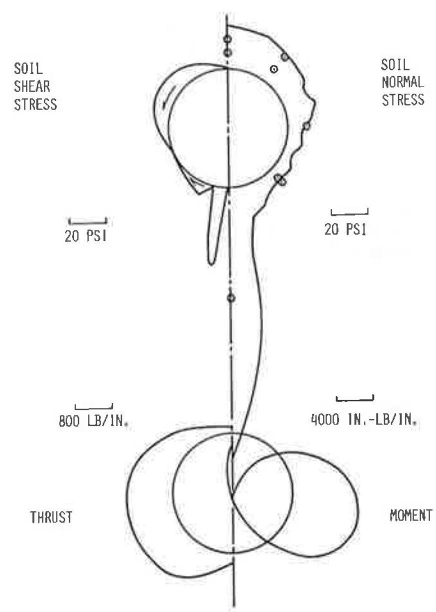
The distributions of predicted soil-pipe interface stresses and pipe moment and thrust are shown in Figure 9. The soil pressures on the pipe are in reasonable agreement with the measured values, although the predicted invert pressure is low. However, the predicted pipe deflections (dashed curves, Figure 10) were lower than those measured, and the computer output did not indicate the cracking observed in the field.

Inadequate representation of the pipe bedding and the sloped trench were considered to be possible reasons for the low deflection predictions. Thus the analysis was rerun with a more concentrated support at the invert and with the top four layers of existing soil elements placed as backfill along with the corresponding trench layers 12 through 15 (Figure 5). The resulting distributions are shown in Figure 11. The pipe thrust and moment have increased. The soil pressure on the pipe has also increased, particularly at the invert. However, only the invert pressure is clearly inconsistent with the measured values. Thus the pipe support was too concentrated. However, the pipe-deflection predictions were improved (Figure 10), and cracking was indicated as observed in the field.

MOUNTAINHOUSE CREEK EMBANKMENT

To illustrate the SPIDA embankment model, results will be given for the computer simulation of an installation at Mountainhouse Creek in California. The details of this analysis are available elsewhere (1). In this case, an 84-in inside diameter reinforced-concrete pipe was installed in an induced-trench condition with 78 ft of soil cover (7). The installation configuration is shown in Figure 12. The site was prepared by excavating unsuitable material for 5-10 ft below the original ground level for a width of about 29 ft at the pipe location. This excavated zone was backfilled with compacted embankment soil to an elevation of about 5 ft above the bottom of the culvert. An 11-ft-wide trench was then excavated in this material to a depth of 5 ft to form the existing ground for the start of the pipe installation. The zones assigned to the mesh

Figure 11. Predicted response for trench installation in East Liberty with sloped-trench approximation and stiff support.



in Figure 6 are shown in Figure 13. The existing ground was added in layers simultaneously with the embankment and bedding-material layers for convenience in modeling, on the basis that the site conditions and the method of establishing the existing ground around the pipe would make this approach reasonable. However, the existing ground may be placed before the embankment and bedding layers.

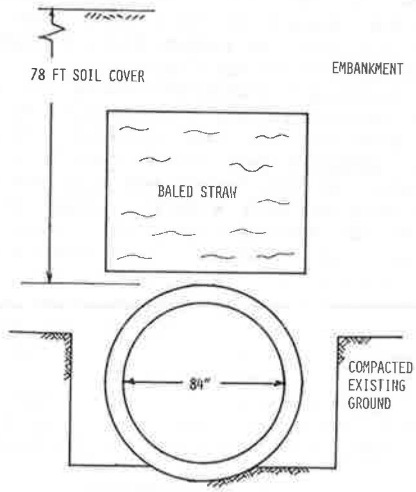


Figure 12. Configuration of Mountainhouse Creek embankment installation.

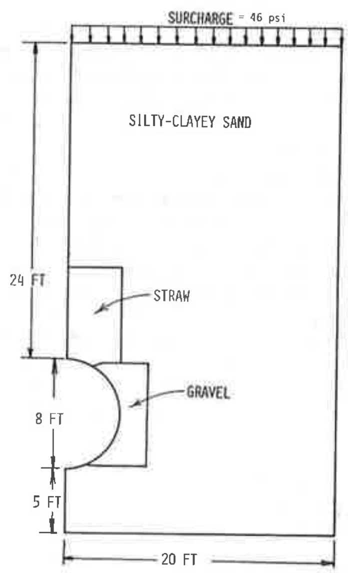


Figure 13. Finite-element mesh zones for Mountainhouse Creek embankment installation.

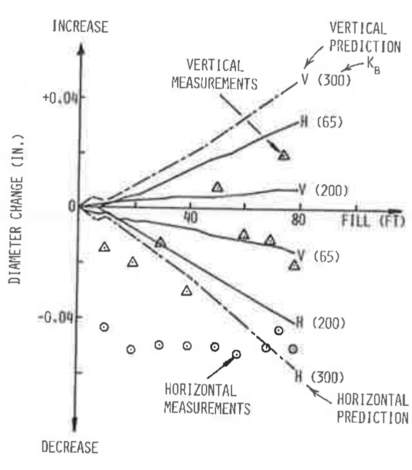


Figure 14. Pipe diameter change for Mountainhouse Creek embankment installation.

One-dimensional compression-test data for the baled straw from a report by Davis (8) were used to establish appropriate parameters for the E-B soil model for this zone. The gravel-zone E-B model properties were obtained from a report by Duncan and others (3) based on compaction and classification descriptions. The Young's modulus parameters for the E-B model for the remainder of the embankment were derived from triaxial test results (7), and the bulk-modulus values were initially estimated (3) based on compaction and classification descriptions. The specific values are given by McVay (1).

The bulk-modulus (B) formulation for the E-B model is as follows:

$$B = K_b P_a (\sigma_3/P_a)^m \tag{1}$$

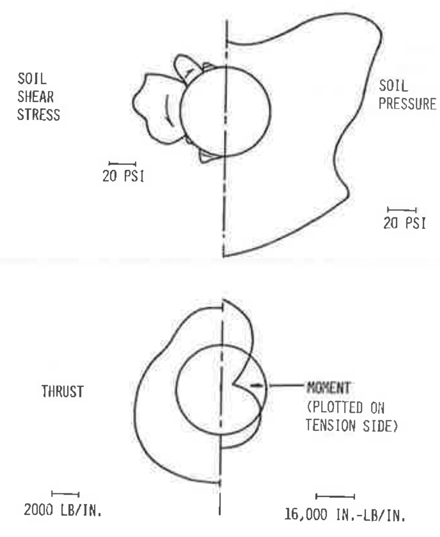
where P_a is atmospheric pressure, σ_3 is the minimum principal stress, and K_b and m are dimensionless parameters. K_b was estimated to be 65 (3,7).

The field observations (7) showed that for the induced-trench test sections, the pipe horizontal diameter decreased and the vertical diameter increased slightly or at least decreased less than the horizontal diameter. For the initially selected K_b of 65, the predicted results show an increase in horizontal diameter and a smaller decrease in vertical diameter (Figure 14). A parameter-sensitivity study with SPIDA indicated that the most likely cause of these contradicting trends between predicted and measured pipe deflections was the value of bulk modulus.

Increasing the bulk modulus causes an increase in pipe lateral confinement, which increases the springline (horizontal) pressure on the pipe, and at the same time a decreasing soil pressure on the pipe crown. This will cause the predicted pipe deflections to change in a manner consistent with the field observations. As shown in Figure 14, a K_b of 300 makes too large a change, but a K_b of 200 gives reasonable results. Further adjustments in all the soil parameters could apparently improve the agreement.

The resulting distributions of soil normal and shear stress and pipe thrust and moment are shown in Figure 15 for K_b = 200. The moment distribution is consistent with an increase in vertical pipe diameter and decrease in horizontal diameter. The soil pressure on the pipe is lowest at the crown as a result of positive arching induced by the straw zone.

Figure 15. Predicted response for Mountainhouse Creek pipe with K_b = 200.



SUMMARY

The research leading to the SPIDA program has indicated that the choice of soil model and values of soil parameters are important for accurately modeling the soil-pipe interaction. The overburden-dependent soil model often used in the past is not satisfactory because the model assumptions are inconsistent with the stress states at many locations in the soil. The hyperbolic model using Young's modulus and bulk modulus was found to be suitable. The value of bulk modulus had a great influence on the computed pipe deflections because it affects the compressibility of the soil adjacent to the pipe and hence the lateral support. Because of its importance, bulk modulus deserves more attention in the future.

The installation geometry must also be properly represented. To achieve this, the SPIDA model has two basic subgroups, one for trench installations and one for embankment installations. This permits optimization of the model for each of these two different situations. Considerable flexibility in representing bedding conditions has been incorporated into the models. In addition, a wide variety of soil types and compaction conditions can be designated. Thus a broad range of field conditions can be simulated.

Experience with SPIDA indicates that it gives reasonable results. The program has advanced to the stage where the basic trench and embankment models are ready for trial applications in pipe design. It is through such applications that the benefits of the program will be proved to the profession. Compared with current design methods, the successful use of SPIDA is expected to reduce the cost of the pipe or reduce the risk of failure or some combination of both.

ACKNOWLEDGMENT

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REFERENCES

1. M.C. McVay. Evaluation of Numerical Modeling of Buried Conduits. Department of Civil Engineering, Univ. of Massachusetts, Amherst, Ph.D. dissertation, Feb. 1982.
2. ACI Standard Building Code Requirements for Reinforced Concrete. American Concrete Institute, Detroit, MI, ACI-318-77, 1977.
3. J.M. Duncan, P. Byrne, K.S. Wong, and P. Mabry. Strength, Stress-Strain, and Bulk Modulus Parameters for Finite Element Analysis of Stresses and Movements in Soil Masses. Univ. of California, Berkeley, Rept. UCB/GT/80-01, Aug. 1980.
4. R.J. Krizek and P.V. McQuade. Behavior of Buried Concrete Pipe. Journal of the Geotechnical Engineering Division of ASCE, Vol. 99, No. GT7, July 1978, pp. 815-836.
5. R.J. Krizek, R.B. Corotis, and T.H. Wenzel. Soil-Structure Interaction of Concrete Pipe Systems. Proc., National Structural Engineering Conference on Methods of Structural Analysis, Madison, WI, ASCE, Vol. 2, Aug. 1976, pp. 607-643.
6. R.J. Krizek and R.B. Corotis. Synthesis of Soil Moduli Determined from Different Types of Laboratory and Field Tests. Proc., Specialty Conference on In Situ Measurements of Soil Properties, Raleigh, NC, ASTM, Vol. 1, June 1975, pp. 225-240.
7. R.E. Davis and A.E. Bacher. Structural Behavior of Concrete Pipe Culvert--Mountainhouse Creek (Part 2). California Department of Transportation, Sacramento, Rept. FHWA-CA-ST-4121-75-8, Sept. 1975.
8. R.E. Davis. Structural Behavior of a Reinforced Concrete Arch Culvert. Bridge Department, California Division of Highways, Sacramento, Rept. SSR3-66, Sept. 1966.

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Performance and Analysis of a Long-Span Culvert

MICHAEL C. McVAY AND ERNEST T. SELIG

A low-profile-arch long-span corrugated-steel culvert was installed in Pennsylvania as a bridge-replacement structure. The Republic Steel Company maxispan design was used. Instrumentation was installed in the soil and on the structure to monitor performance during construction. Field and laboratory soil property tests were conducted to characterize the soil behavior. Predictions from finite-element computer analyses were compared with the field results. From this and previous research, a number of conclusions were drawn. The choice of soil model had its most significant influence on the culvert deformation and bending-stress predictions. Overburden-dependent and linear-elastic soil models were shown to be unsatisfactory. Effects of construction procedures are difficult to predict accurately. Further study is needed to evaluate the importance of factors such as compaction-induced deformation, soil-culvert interface conditions, culvert wall yielding, and wall buckling. Seam slip rather than bending flexibility is needed to develop positive arching and hence further study is warranted. A particularly important observation was that special features like compaction wings appear to be both unnecessary and undesirable.

A long-span flexible corrugated-steel low-profile-arch culvert was constructed in Bucks County, Pennsylvania, as a bridge-replacement structure. The owner was the Pennsylvania Department of Transportation (PennDOT). The design used was a Republic Steel Company maxispan with compaction wings. The structure and soil backfill were instrumented to monitor performance during construction. Field and laboratory tests were conducted on the backfill soil to characterize the soil behavior. In addition, a series of finite-element computer analyses were carried out to help evaluate the performance of the structure and to assess the validity of the computer model.

This paper summarizes the field installation and the measurements made. The computer model is then described. This is followed by comparisons between some of the important calculated and measured results. Finally, the computer model is assessed and the culvert design concepts are evaluated.