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

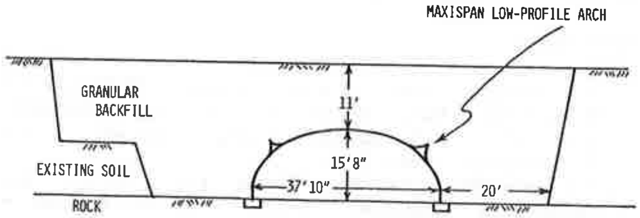
MICHAEL C. McVAY AND ERNEST T. SELIG

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.

Figure 1. Approximate geometry of culvert installation.



FIELD INSTALLATION

The size and shape of the culvert were based on the estimated flow in the creek, the presence of underlying rock, and the desired grade elevation of the roadway. The strength of the culvert wall was based on ring-compression theory, handling criteria, and buckling considerations.

The culvert had approximately a 16-ft rise, a 38-ft span (Figure 1), and 11 ft of backfill cover above the structure when completed. The conduit wall was assembled from 5-gauge structural steel plates that had 2x6-in corrugations. The structure was 88 ft 4 in long and had concrete end walls to protect the backfill against erosion from creek flooding and provide stability to the soil mass adjacent to the culvert opening.

The preexisting embankment soil was classified as a silty loam from visual inspections. The backfill material was all granular, primarily PennDOT 2A material, which is classified GW in the Unified Soil Classification System.

The backfill was placed in horizontal lifts of approximately 4 in or greater thickness. Compaction was supplied by several passes of a 10-ton smooth-wheel roller away from the culvert and a hand-operated vibratory plate compactor near the culvert. Tests of nuclear field density and moisture content were conducted by PennDOT at various elevations throughout the backfill as part of construction control to check the adequacy of compaction. An average field density of 121 pcf and moisture content of 7.6 percent were measured. The difference in fill elevations on the two sides of the culvert was maintained within 1.5 ft during the backfilling process.

The construction began in September 1978 and was completed by January 1979. To evaluate the performance of the structure during construction and provide information for future design and analysis of long-span structures, the culvert and surrounding soil mass were extensively instrumented.

A pair of weldable strain gages was installed at many locations on the structure to determine both bending and thrust stresses in the steel. Displacements of the culvert during backfilling were monitored by using survey targets located around the inside of the structure. In addition, structural extensometers were used to measure displacements of the springline and crown of the structure at three longitudinal cross sections.

Instrumentation was installed in the soil at the springline, compaction wing, and crown elevations. Included were inductance-type soil stress gages, hydraulic soil stress gages, horizontal soil extensometers, vertical soil extensometers, and 11-in-diameter inductance soil strain gages. The extensometers, the inductance stress gages, and the soil strain gages have been described elsewhere (1). The stress gages were laboratory calibrated in soil as described by Selig (2) and placement conditions intended to simulate those in the field were used.

Both laboratory and field tests were performed to document the backfill conditions and obtain soil stress-strain relationships, physical state identification, and strength properties of the soil for the analysis. During the backfilling phase of construction, soil samples were collected from various locations throughout the backfill. Laboratory sieve tests were performed on each of the samples for classification purposes. Laboratory uniaxial strain compression tests and constant confining pressure triaxial compression tests were then conducted at densities and moisture contents representative of field conditions to determine the stress-strain properties of the placed backfill. In addition, during construction six plate-load tests were performed to provide field data representative of the backfill stiffness.

FINITE-ELEMENT APPROACH

The discretization, geometric boundaries, and restraints employed to represent the field installation by a finite-element analysis are shown in Figure 2. Only half the installation was considered on the assumption that the constructed structure and backfill were symmetrical. The soil nodes on the two vertical boundaries were restrained to vertical movements only. The soil nodes on the bottom horizontal boundary were restrained against both horizontal and vertical movement, which allowed in situ shearing stresses to develop between the soil and the rock. The right vertical boundary was placed three culvert spans from the centerline of the soil-culvert system, based on past experience of other researchers (3). The structural nodes were constrained to permit no horizontal or rotational movements at the crown and no horizontal or vertical movements at the footings. The culvert was modeled with straight beam elements and the adjacent soil as either triangular or quadrilateral isoparametric elements. Because no information was available on the initial stress state and stress-strain behavior of the preexisting soil, the soil was modeled in the same manner as the backfill. All soil was given a density of 0.075 lb/in³.

The finite-element modeling of the installation began at the stage where the soil excavation and construction of the structure were completed, but no backfill was placed. Thus the height of fill is referenced to the top of the footings.

Soil Models

Four different models were considered for simulation of the constitutive relationships of the soil in the finite-element analysis: the constant-modulus linear-elastic model, the stress-dependent bulk-modulus and hyperbolic Young's-modulus model, the bilinear modulus model with constant Poisson's ratio, and the overburden stress-dependent modulus model with constant Poisson's ratio. The soil parameters required by each model were determined from laboratory and field tests performed on the backfill material.

The characterization of soil as a linear-elastic material uses a constant Young's modulus (E) and a constant Poisson's ratio (ν). Based on a synthesis of the results of the field plate-load tests, one-dimensional compression tests, and triaxial compression laboratory tests, a lower-bound estimate of 3000 psi was selected for the Young's modulus of the backfill. The Poisson's ratio value was assumed to be 0.25 based on the measured value of angle of internal friction for the backfill soil. This model does not incorporate a failure condition.

The model represented by a stress-dependent hyperbolic Young's modulus and a bulk modulus was

Figure 2. Finite-element mesh used in computer analysis.

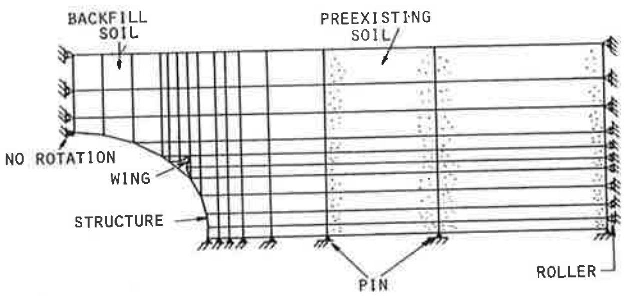


Figure 3. Hyperbolic E-B soil model.

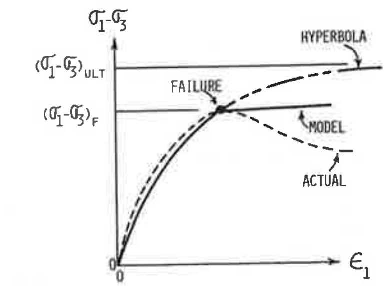
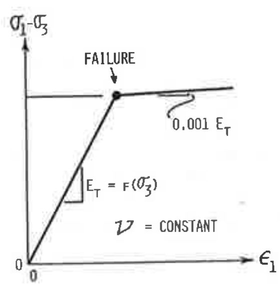


Figure 4. Bilinear soil model.



proposed by Duncan and others (4). This model, which will be designated the E-B model, assumes that the triaxial stress-strain curve up to the failure point can be represented by a hyperbola (see Figure 3). Beyond failure a small constant stiffness is assumed for the model instead of the reduction in stress usually exhibited by actual soils. The tangent Young's modulus (E_t) is the tangent to the hyperbola below failure. It is expressed by (4) the following:

$$E_t = [1 - R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)/(2c\cos\phi + 2\sigma_3 \sin\phi)]\,KP_a(\sigma_3/P_a)^n \tag{1}$$

where

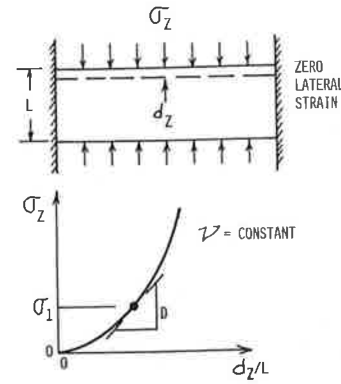
- R_f = failure ratio = $(\sigma_1 - \sigma_3)_f/(\sigma_1 - \sigma_3)_{ult}$,
- σ_1 = axial stress,
- σ_3 = confining pressure,
- c = cohesion,
- ϕ = angle of internal friction,
- P_a = atmospheric pressure, and
- K,n = experimentally determined parameters defining stress dependency of soil stiffness.

The variation of ϕ with confining pressure is represented by the following:

$$\phi = \phi_0 - \Delta\phi \log_{10}(\sigma_3/P_a) \tag{2}$$

The bulk modulus (B) is assumed dependent only on the confining pressure (σ_3) as represented by the following:

Figure 5. Overburden-dependent soil model.



$$B = K_b P_a(\sigma_3/P_a)^m \tag{3}$$

where K_b and m are experimentally determined parameters. The predicted volumetric strains will always be compressive because dilation is not represented by the model.

The values of the parameters for the Republic Steel culvert installation were determined from triaxial and one-dimensional compression tests. The results are listed below. The one-dimensional compression tests were particularly useful in estimating the bulk modulus.

Parameter	Value
K	660
n	0.81
R_f	0.45
K_b	240
m	0.56
c	0
ϕ_0	52 degrees
$\Delta\phi$	18 degrees

The observed stress-strain behavior of the backfill up to failure in the triaxial tests was almost linear; thus a bilinear soil model was established by using a Young's modulus that increases with confining pressure, a failure criterion, and a constant Poisson's ratio (Figure 4). Above failure the modulus is assumed to be a very low value equal to 0.001 of the value (E_t) below failure for the same σ_3.

The relationship between the constant average slope (E_t) and the confining pressure (σ_3) employed in the model is of the following form:

$$E_t = K_1 P_a(\sigma_3/P_a)^{n_1} \tag{4}$$

where

- E_t = tangent modulus,
- σ_3 = minor principal stress,
- P_a = atmospheric pressure expressed in same pressure units as E_t and σ_3, and
- K_1,n_1 = experimentally determined constants.

From triaxial tests on the backfill soil in the Republic Steel culvert installation, the values of the parameters were K_1 = 392 and n_1 = 0.567. Poisson's ratio was assumed to be 0.25 as for the linear-elastic model. The strength parameters c, ϕ, and $\Delta\phi$ defining failure were taken to be the same as those given above for the E-B model.

The overburden-dependent soil model assumes that the soil elements are confined in a state of uniaxial strain, and hence the stiffness increases with stress as indicated in Figure 5. The slope of the curve in Figure 5 at any axial stress is the constrained modulus D. Since σ_z is the major principal stress, it is designated σ_1. Poisson's ratio is assumed to be constant.

The finite-element program uses Young's modulus as the stiffness parameter. To represent the overburden-dependent model, Young's modulus (E) corresponding to the constrained modulus (D) is obtained from elasticity theory by the following:

$$E = \left\{ [(1 + \nu)(1 - 2\nu)]/(1 - \nu) \right\} D \quad (5)$$

where ν is Poisson's ratio. The confined compression test gives values of D as a function of maximum principal stress σ_1. Then Equation 5 gives E as a function of σ_1. Since D increases with σ_1, it is obvious that E will increase with σ_1. This is quite different than indicated by the E-B and bilinear models. The values of E used in the Republic Steel culvert analysis are given below. These are based on one-dimensional compression tests performed in the laboratory on backfill soil samples. (The model incorporates a constant Poisson's ratio of 0.25.)

Vertical Stress (psi)	Young's Modulus (psi)
0.0	44
0.5	623
1.0	840
3.0	1100
5.0	1170
10.0	1720
15.0	2200
20.0	2700
25.0	3200
30.0	3600
40.0	4600

For all the soil models the initial stresses from placement of the soil layers were defined as geostatic. Thus the vertical stress in the center of a placed element was set equal to the product of the soil unit weight and one-half the element height. The corresponding horizontal stress was set equal to K_0 times the vertical. The selected value of K_0 was 0.35 based on a ϕ of 44 degrees. However, the results are not very sensitive to this value.

Structural Plate Representation

The corrugated-steel culvert was represented by 13 plane strain linear-elastic beam elements (Figure 2). The properties of the corrugated steel plates are listed below (plates are 5-gauge steel with 2x6-in corrugations):

Parameter	Value
Actual area	0.267 in²/in
Reduced area (A)	0.0444 in²/in
Modulus of elasticity (E)	30×10^6 psi
Poisson's ratio (ν)	0.33
Moment of inertia (I)	0.127 in⁴/in
Section modulus (S)	0.115 in³/in
Plate thickness (t)	0.218 in

However, the bolted seams in the structure cause the circumferential stiffness (EA) of the culvert wall under compressive loads to be lower than that of a continuous corrugated-steel plate. To model the reduction in this thrust stiffness (EA) and maintain the same bending stiffness (EI) of the bolted joints, the cross-sectional area of the beam element (A) was artificially lowered for all the beam elements. An area reduction of six times the original value was selected based on previous research (4).

MEASURED RESPONSE AND PREDICTIONS

Culvert Displacements

One of the most important but most difficult responses of the culvert installation to predict by finite-element analysis is the displacement of the culvert during the backfilling process. Accurate simulation of the resulting deflection pattern is needed to obtain the correct moment distribution in the culvert wall, but it is also important for evaluating the appropriateness of the soil model and the soil properties.

A comparison of the measured crown vertical displacement as a function of backfill height above the footings with predicted values for the four soil models investigated is shown in Figure 6. The measured crown displacements are combined for three different longitudinal cross sections of the culvert, based on results from the structural extensometers. From the measured response, the crown of the culvert appears to have risen approximately 3-5 in at its midsection from backfilling to the crown and then to have settled back about 2 in as a result of placing the backfilling cover above the crown.

Only the two confining stress-dependent models gave reasonable agreement with the measured values. Even though the linear-elastic and overburden-dependent models show the peaking of the structure at the same fill height as the other two models, both underpredict the crown vertical movements by a sizable amount. They also result in final crown movements that are much lower than the original position before backfilling.

Other finite-element studies of culvert crown vertical movements incorporating the overburden-dependent soil model show similar trends (3,5,6). This displacement discrepancy has suggested the need to include the modeling of the compaction process itself. However, Figure 6 suggests that the choice of soil model could be the problem instead.

Soil Stress

The measured and predicted horizontal stress (σ_H) at the springline elevation at the completion of backfilling are shown in Figure 7. Agreement is quite good for all the soil models studied and the computed stress is not very sensitive to the choice of soil model. The horizontal stress is greatest near the culvert and decreases rapidly with distance from the culvert wall. The stresses continue to decrease until the geostatic or free-field value is reached.

The measured vertical soil stresses (σ_v) above the crown for the final fill height are shown in Figure 8 together with the predicted vertical stresses. The stress is lowest above the crown and highest above the compaction wings. The free-field values lie between the crown and wing values. Measured values show a greater reduction directly over the crown than the predicted values but agree at their maximum values directly over the compaction wing-tip location.

Soil Strain

The measured and predicted average horizontal strain at the springline elevation is shown in Figure 9 as a function of fill height above the footings. The average strain was defined as the horizontal differential movement between the culvert and a position about 10 ft away from the culvert divided by this distance. Only the bilinear and E-B models agreed with the measured values. The other two models predicted strains that were much too small.

Figure 6. Measured and predicted crown vertical displacements with height of backfill above footings.

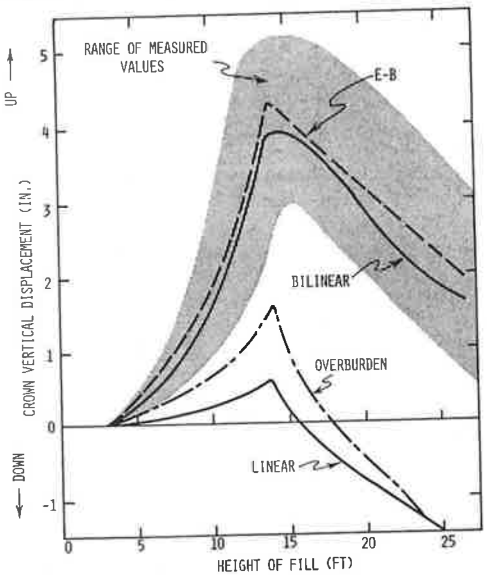


Figure 7. Measured and predicted horizontal stresses at springline elevation at final fill height.

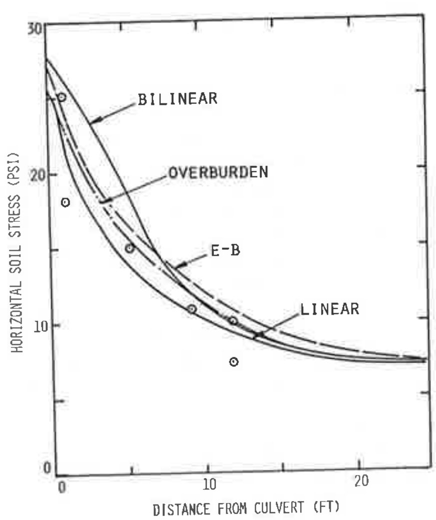


Figure 8. Measured and predicted vertical soil stresses over crown for final fill height.

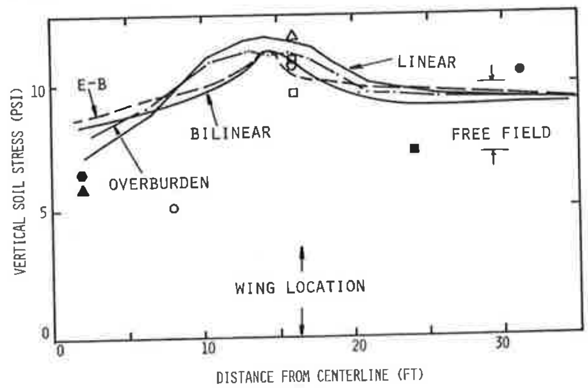
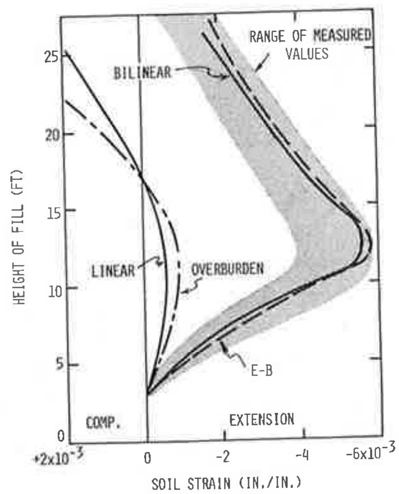


Figure 9. Measured and predicted average horizontal soil strain at springline elevation adjacent to culvert as function of backfill height above footing.



Culvert Thrust and Bending Stress

The measured and predicted thrust compression stresses around the barrel of the structure for the final fill height are compared in Figure 10. Both the measured and the predicted values gradually increase from the crown to the springline. This is due partly to soil-steel friction and partly to the keying action of the compaction wings. The keying action is illustrated by the step changes in thrust stress at the compaction-wing connections. The predicted thrust stresses are in general agreement with the measured values except for those from the linear-elastic model, which are lower by 40 percent at the crown location.

Although not shown in Figure 10, the measured thrust stresses were almost zero for the top wing plates and small in magnitude for the side wing plates. The extremely low thrust stress in the top plates suggests that little if any thrust load is being transferred from the barrel of the culvert to the surrounding backfill through the compaction wing. Both the measured and the predicted thrust stresses in the compaction wings were in close agreement for all the soil models investigated.

Also shown in Figure 10 is the thrust stress at the springline location calculated by the ring-compression theory. The magnitude of the measured springline thrust force in the culvert wall was found to be approximately equal to the overlying weight of backfill above the culvert half-span. However, the calculated ring-compression values considered only the soil weight above the crown, as is the conventional practice, and hence excluded the soil weight between the crown and springline of the culvert. This accounts for most of the difference between the ring-compression theory and measured thrust stresses.

A plot of the measured bending stresses in the culvert at the final fill elevation is shown in Figure 11. As a result of the final displacement patterns of the culvert, at the crown and springline the outer wall is in tension and the inner wall is in compression (positive moment). The maximum bending stresses were larger than the thrust compression stresses. Although not shown, the measured bending stresses in the compaction wing were of the same order of magnitude as those in the culvert barrel, which indicates that the surrounding soil is pushing the wing toward the barrel of the culvert.

Both the bilinear and E-B model results agree quite well with the measured bending stresses. However, the predicted linear-elastic and overburden-dependent bending stresses at the crown of the culvert disagree with the measured values as a result

Figure 10. Measured and predicted thrust compression stresses at final fill height.

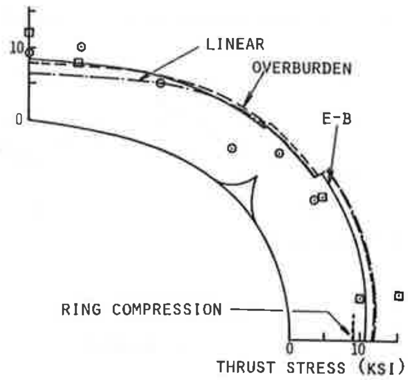


Figure 11. Bending stress in culvert at final fill height.

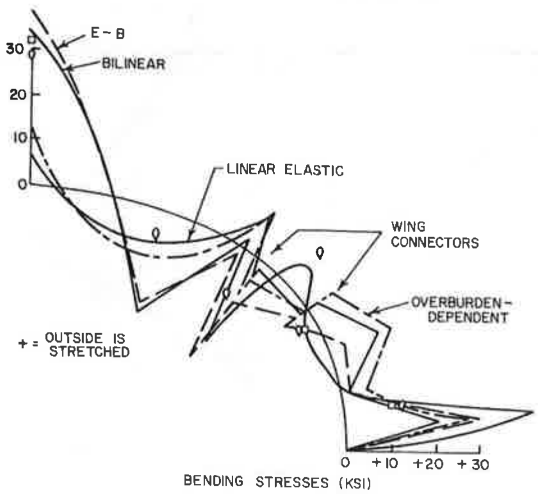
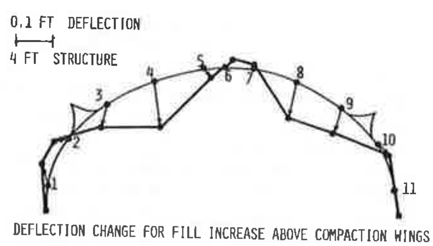


Figure 12. Culvert deflection change for fill increase above compaction wings based on survey target data.



of the error in their predicted displacement patterns.

EFFECT OF COMPACTION WINGS

As part of this investigation of the soil-structure interaction of flexible conduits, the role of the compaction wings on the structure was evaluated. The response of the structure was analyzed with and without the compaction wings by use of the finite-element method, incorporating the E-B soil model. The field measurements with the maxispan structure were also examined.

The computer solution showed a larger upward crown movement during backfilling to the crown and smaller downward movement for backfilling from the crown elevation to the final elevation with no compaction wings. This may be attributed to the reduced barrel stiffness with the removal of the compaction wings. However, according to the survey target data, the compaction wings moved inward relative to the soil during backfilling from the compaction-wing elevation to the final fill elevation (Figure 12). Thus the wings did not serve as abutments or stabilize the top arch of the structure as frequently claimed. The same trend was indicated by a culvert with a thrust beam (7).

The predicted thrust stresses in the barrel of the culvert are slightly smaller without the compaction wings than with them. Thus the thrust in the side walls is not reduced but instead is increased by the presence of the compaction wings. This indicates that the compaction wings do not serve to transfer soil load on top of the structure to the adjacent soil as is sometimes claimed.

The compaction wings may provide stiffness to the barrel of the culvert. However, a more efficient approach would be either to supply the structure with struts during construction or to provide rib stiffeners on the structure.

CONCLUSIONS

The investigation of the Republic Steel structure in Pennsylvania described in this paper together with previous research have led to the following conclusions:

1. The choices of soil model and parameter values are critical to the prediction of culvert deformation and bending stress. They are not critical to the prediction of thrust stress, however.
2. The overburden-dependent soil model should not be used. A model that has stress-dependent soil moduli and a yield (failure) criterion is needed. Of those evaluated, the most suitable is the E-B model. Among its advantages is the ability to estimate parameter values from available data or to determine them from conventional soil-property tests.
3. Incremental placement of the backfill is essential for good predictions. The effects of other construction-related factors such as compaction, bolt tightening, and temporary supports are difficult to predict. This will limit the accuracy of the computed results, particularly deflection and bending stresses.
4. The need for simulation of compaction should be further evaluated. Simulation by surcharge pressure is not representative of the actual process and may partly be compensating for errors from other causes such as inadequate soil modeling.
5. Compaction wings appear to be unnecessary. They do not appear to perform the intended functions such as supporting the top arch. In addition, they have disadvantages, which include increasing cost and increasing springline thrust. Although evidence to support this conclusion already exists, some demonstration installations are desired for confirmation.
6. Satisfactory predictions have been obtained by using modeling techniques described in this paper. However, the finite-element models might be improved by better representing seam slip, soil-culvert interface slip, interface tension release, wall yielding, and wall buckling.
7. Rational criteria have not yet been established for the design of the structural backfill zone, particularly the size of the zone. A better understanding of the role of this zone should result in better economy and an extension of the range of conditions in which long-span structures can be constructed.
8. The bending flexibility of long-span structures does not lead to positive arching. Positive arching can come from seam slip, however. Thus

further study of the influence of seam yielding appears beneficial.

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REFERENCES

1. E.T. Selig. Instrumentation of Large Buried Culverts. In Performance Monitoring for Geotechnical Construction. American Society for Testing and Materials, Philadelphia, PA, Special Tech. Publ. 584, Aug. 1975, pp. 159-181.
2. E.T. Selig. Soil Stress Gage Calibration. ASTM Geotechnical Testing Journal, Vol. 3, No. 4, Dec. 1980, pp. 153-158.
3. G.A. Leonards and T.H. Wu. Predicting Performance of Buried Conduits. Engineering Experiment Station, Purdue Univ., Lafayette, IN, March 1981.
4. 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 at Berkeley, Rept. UCB/GT/80-01, Aug. 1980.
5. C.S. Chang, J.M. Espinoza, and E.T. Selig. Computer Analysis of Newton Creek Culvert. Journal of the Geotechnical Engineering Division of ASCE, Vol. 106, No. GT5, May 1980, pp. 531-556.
6. M.G. Katona, J.B. Forrest, R.J. Odello, and J.R. Allgood. Computer Design and Analysis of Pipe Culverts. Civil Engineering Laboratory, Port Hueneme, CA, Interim Tech. Rept. 51-040, Oct. 1974.
7. E.T. Selig, C.W. Lockhart, and R.W. Lautensleger. Measured Performance of Newtown Creek Culvert. Journal of the Geotechnical Engineering Division of ASCE, Vol. 105, No. GT9, Sept. 1979, pp. 1067-1087.

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Design Method for Concrete Pipe Under High Fills

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A method is proposed for design of reinforced concrete pipe installed in embankment conditions under high fills. At the current stage, the method is restricted to pipes bedded in the natural soil. The method widely used at this time has seen little change since its introduction more than 40 years ago, and recent research has indicated that it is highly conservative for present practice. A study of the complex soil-structure interaction system has been made by using the finite-element method. A simplified linear-elastic approach has enabled comparison of three-edge bearing test conditions with those encountered by a pipe in the field in such a way as to provide curves suitable for a rapid design procedure. Justification for such an approach can be obtained from review of field response data for such conditions. For a large-scale field study the method predicts the cover height to produce cracking within about 20 percent, whereas the conventional procedure provides a safety margin in the ratio of 4.6:1.

On the basis of his own work and that of his colleagues, Spangler (1) proposed a method for concrete pipe design. He recognized the interaction effects of the pipe, the underlying soil, the pipe bedding, and the compacted fill and suggested a parameter, the settlement ratio r_{sd}, defined as follows:

$$r_{sd} = [(S_m + S_g) - (S_f + d_c)]/S_m$$

where

S_m = compression of soil on either side of pipe,
S_g = settlement of natural ground adjacent to conduit,
S_f = settlement of conduit into its foundation, and
d_c = change in vertical height of conduit.

The settlement ratio was used in conjunction with the projection ratio, the lateral pressure coefficient, the soil friction coefficient, the soil unit weight, the height of soil cover, and the pipe diameter to obtain an equivalent total load on the pipe (W_c). On the basis of bedding type, a load factor L_c was estimated. The required class of pipe was then determined from the three-edge bearing strength (S_{eb}) as follows:

$$S_{eb} = W_c/L_f$$

The theory was based on the ultimate resistance to movement on boundaries of a rectangular prism of soil above the pipe. Crude guidelines for selection of r_{sd} were suggested by Spangler based on back-calculation from field tests (2), but the basis for proposals for appropriate values of lateral earth pressure coefficient, friction angle, and load factor, all extremely uncertain, is not so evident. Review of the field tests indicates that the proposed values for r_{sd} are questionable, particularly in the light of the other uncertainties.

Nearly five decades and many computer hours of concrete pipe studies later, the Spangler approach remains the conventional approach to the problem, in spite of the fact that capabilities for analysis are now many times greater than those that were available to Spangler. Some finite-element studies of the buried reinforced concrete pipe problem have been completed (3-5), but they either constitute a parametric study of particular aspects of the problem or offer a complex programming procedure for the system that is not usable for ordinary design purposes.

In this work certain simplifications have been made to enable presentation of results from a comprehensive finite-element study in a simple graphical form. Both the pipe and the soil are assumed to deform in a linear manner under load. Some nonlinearity is to be expected for both the pipe and the