observation that the peaking amplitudes also happen to be in good agreement. The observation that the peaking amplitude of the averaged data is bracketed by the two slipping conditions is incidental because amplitudes are easily changed by the assumed soil stiffness. However, the peaking range is unaffected by soil stiffness (5).

In summary, it is concluded that FEM models of long-span culvert installations should incorporate slipping interface conditions in order to properly predict deformation histories.

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Deflection of Flexible Culverts due to Backfill Compaction

J.M. DUNCAN AND J.K. JEYAPALAN

A detailed study was made of the effects of compaction on the long-span aluminum culvert structure in Tice Valley, California. Finite-element analyses were performed to simulate field behavior. Measurements showed that the deflections due to compaction were very significant. The analysis procedure appeared to model the effects of compaction reasonably accurately.

Experience with flexible metal culverts has shown that the loads applied by rollers and other heavy construction equipment during compaction of the backfill can cause considerable deflection of a culvert. If heavy construction equipment is permitted to operate close to the sides of a flexible culvert, considerably more "peaking," or upward movement of the crown, may occur than if heavy equipment is kept away from the structure.

In an analytical study of these effects, Katona (1) performed finite-element analyses in which the application of temporary compaction loads was simulated. He found that temporary loads applied to the surface of each new layer of backfill resulted in additional amounts of peaking, and his results were therefore in agreement with field experience.

Thus it is clear from both field experience and analytical studies that compaction loads can induce appreciable deflections in flexible metal culverts. Although the experience and the analytical studies are in qualitative agreement, no data have been available that could serve as a basis for quantitative comparisons of field measurements with analytical results in any particular case.

The purpose of the study described in this paper is to make a detailed evaluation of the effects of compaction for the long-span aluminum culvert structure in Tice Valley, California. Deflections of the haunch, the crown, and the quarter point were measured before and after compaction of each new layer of backfill, and analyses were performed to simulate the field behavior. It was thus possible to determine the magnitude of the compaction load appropriate for the equipment used on the job, and it was also possible to determine the effect of the compaction loads on the bending moments in the culvert.

TICE VALLEY CULVERT STRUCTURE

The Tice Valley culvert structure is located in Walnut Creek, California, about 20 miles east of San Francisco. A cross section through the structure is shown in Figure 1. It is a horizontal ellipse with a span of 25 ft 1 in and a rise of 12 ft 11 in. The culvert is constructed of aluminum structural plate 0.15 in thick and has aluminum bulb angle stiffener ribs spaced at 2 ft 3 in across the crown.

The backfill is a sandy clay classified CL by the Unified Soil Classification System. It was compacted to a minimum of 95 percent of the maximum dry density determined by the standard compaction test of the American Association of State Highway and Transportation Officials (AASHTO) (AASHTO 799, ASTM D698). The final depth of cover over the crown of the structure was 4.0 ft. The backfill was spread in layers about 1.0 ft thick and was compacted by using a 16 500-lb bulldozer and a 3500-lb vibratory roller.

INSTRUMENTATION

Deflection gages were installed at two sections in the culvert located 6 ft 9 in apart. Four measurements were made at each section. As shown in Figure 2, these were (a) change in span, (b) change in rise, (c) deflection of quarter points relative to invert, and (d) vertical movement of quarter points relative to haunch.

The vertical movements of the quarter points were

Figure 1. Cross section of Tice Valley culvert.

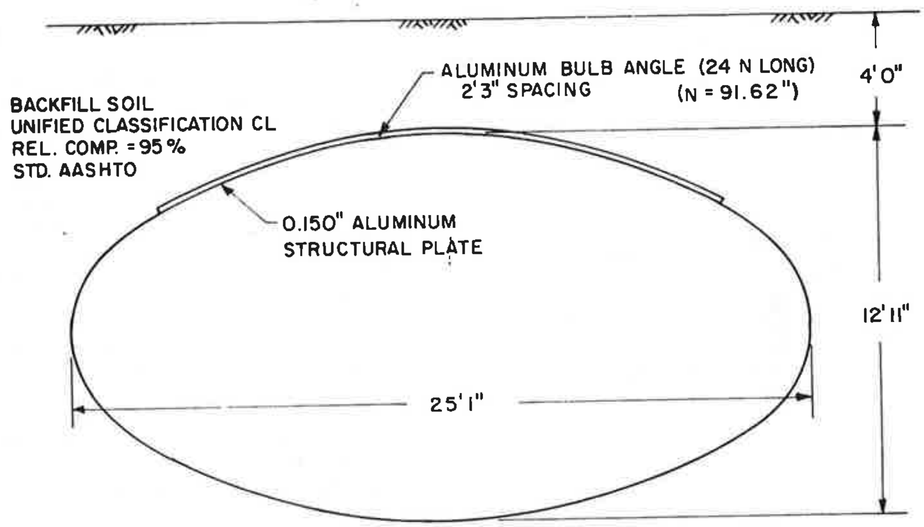
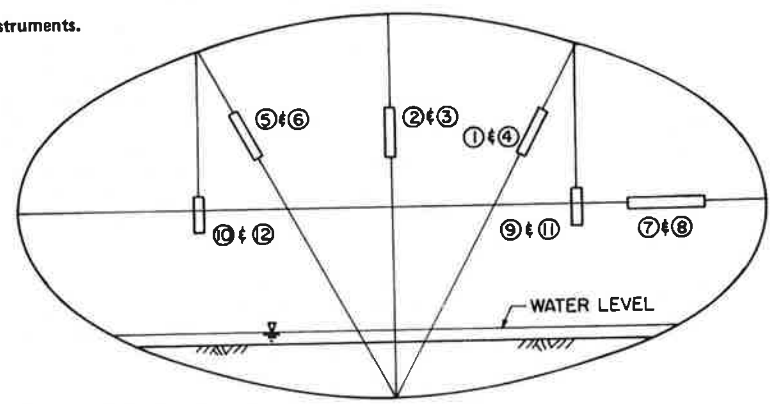


Figure 2. Location of instruments.



NOTE: GAGES WERE INSTALLED AT TWO SECTIONS (A and B) LOCATED 6'9" APART

measured by using steel weights with scales attached that were suspended from the quarter points on bead chains (instruments 9-12 in Figure 2). All other deflections (1-8 in Figure 2) were measured by using deflection gages of the type shown in Figure 3. These consist of a looped cable with a spring to maintain tension as the length changed. A block attached to one side of the loop slid within a tube attached to the other side of the loop, and a scale was attached to the outside of the tube. Changes in length between the ends of the deflection gage produced twice as great a change in the scale reading.

The accuracy of the deflection gages was checked under laboratory conditions and was found to be about ±0.03 in. The hanging plumb bobs used to measure vertical movements of the quarter points could be read with an accuracy of about ±0.05 in. However, to prevent damage by vandalism, it was necessary to remove the instruments from the culvert at the end of each day, and this reduced the accuracy of the readings. All things considered, it is estimated that the accuracy of the readings is within ±0.1 in, and the tolerance is probably less in most cases.

MEASURED DEFLECTIONS

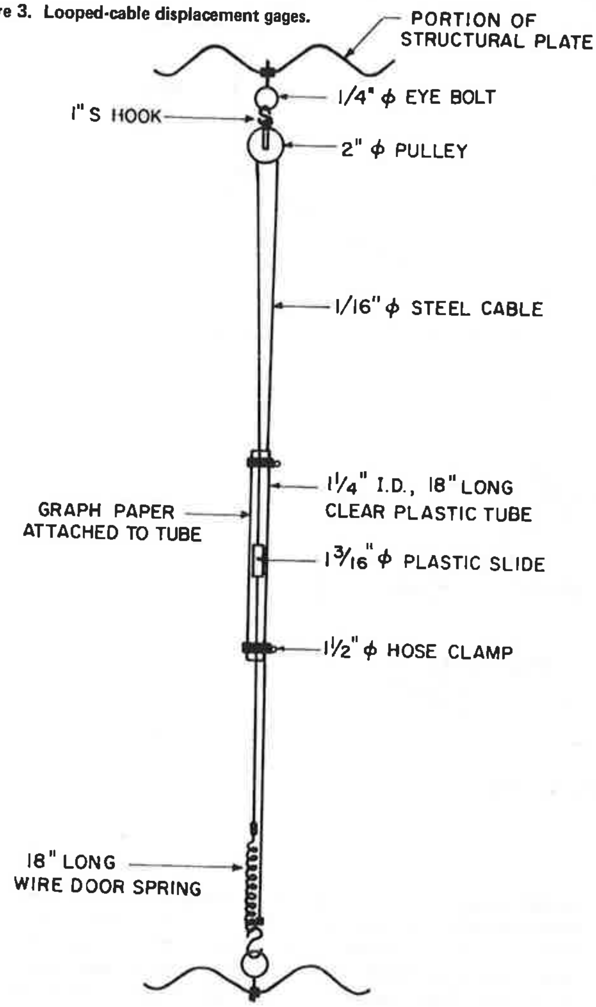
The measured movement of the haunches and the crown is plotted against the level of the fill in Figures 4 and 5. In these figures the fill height is measured from the crown of the structure, so that H = 0 corresponds to the condition in which the top of the fill is level with the crown. The value of H is negative for fill levels below the crown and positive for fill levels above the crown. Measurements were begun with H = -5.5 ft, or fill level about 1 ft above the haunch of the structure.

Haunch movement is shown in Figure 4. Those points labeled "before compaction" are the sum of the incremental movements due to placement of each layer of fill. Those labeled "after compaction" represent the movement due to both the weight of fill and the compaction effects. It may be seen that by the time the fill had reached a level 2 ft below the crown (H = -2 ft) the decrease in span due to compaction effects was about twice as great as that due to the weight of the fill. The span increased slightly (more so at section B than at section A) as the fill was raised above the crown.

Crown movement is shown in Figure 5 by using the same convention as that for haunch movement. It may be seen that the movement of the crown due to compaction was about 1 1/3 times as large as that due to the weight of the fill.

Thus it is evident that compaction of the clay backfill around the Tice Valley culvert to 95 percent of the standard AASHTO maximum dry density with a 16 500-lb bulldozer and a 3500-lb vibratory roller caused considerable deflection of the culvert structure. This was true even though the roller was not operated closer than about 2 ft from the side of the culvert and the fill immediately adjacent to the culvert was compacted with a light hand tamper. At both the haunch and the crown, the movement due to the effects of compaction exceeded that due to the weight of the fill.

Figure 3. Looped-cable displacement gages.



FINITE-ELEMENT ANALYSES

A number of finite-element analyses of the Tice Valley culvert were performed to calculate deflections of the culvert before and after compaction. Previous experience with finite-element analyses of flexible metal culverts has shown that it is essential in such analyses to model the nonlinear and stress-dependent stress-strain behavior of the backfill soil and to simulate the actual sequence of events during construction in order to achieve correspondence with the behavior of actual structures. Accordingly, the analyses of the Tice Valley culvert were performed in a series of increments by using hyperbolic stress-strain relationships for the backfill soil. These relationships, which have been described in detail by Duncan and Chang (2) and by Duncan and others (3), model the nonlinear stress-strain behavior of soils incrementally by varying the values of Young's modulus and bulk modulus in each element in accordance with the calculated stresses.

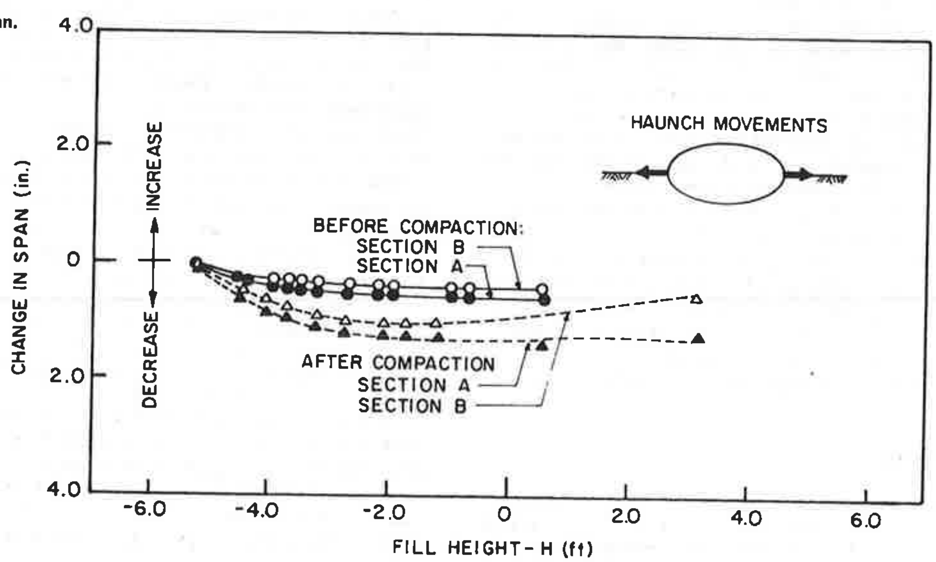
The analyses were performed incrementally, which simulated the placement of backfill around and over the culvert one layer at a time and the application of compaction loads to each layer of fill. The computer program used in these analyses (SSTIPN) models the culvert as a number of straight beam elements connected at common nodes. The backfill is modeled by two-dimensional isoparametric elements with compatible and incompatible deformation modes. The soil elements can be firmly attached to the beam elements at their common nodes or interface elements can be used to allow slip at the culvert-backfill interface.

The first analyses were performed to calculate deflections due to the weight of the backfill, with no compaction loads. The results showed that the best correspondence between the measured and the calculated deflections was achieved when interface elements were used to permit slip between the bottom half of the culvert and the underlying soil. This same condition was used in all subsequent analyses. A number of different procedures were employed to simulate the effects of compaction. These are discussed below.

Procedure 1

The first procedure was the same as that used by Katona (1). After the placement of each new layer

Figure 4. Changes in span.



of backfill, a uniform pressure was applied on the upper surface of the layer to simulate the pressure imposed by the compaction equipment. Subsequently, when the next layer of fill was added, this pressure was removed and a pressure of equal magnitude was applied at the upper surface of the newly placed layer. This process was repeated up to the last layer, where the compaction pressure was applied and then removed. It was found that compaction pressure of 1 lbf•ft (7 psi) produced very little net deflection because the deflections induced when the pressure was applied were diminished to much smaller values when the pressure was removed.

Procedure 2

The next procedure was the same except that the compaction pressure was not removed when the next layer was added. This procedure produced very large deflections with a compaction pressure of 1 lbf•ft, and the variations of the deflections with fill height did not conform well with the measured values. Furthermore, this procedure does not satisfy equilibrium because the extra compaction load is not removed, and it thus does not constitute a rational means of simulating compaction effects.

Procedure 3

A complete analysis was performed, which included the placement of all layers of fill on the structure with no compaction load. The results calculated after addition of each new layer were punched onto cards. Then the conditions after placement of the first layer were used as the initial conditions for analysis of deflections due to compaction of the first layer, and these deflections were punched onto cards. Then these deflections together with the stresses and structural forces calculated previously with no compaction loads were used as the initial conditions for analysis of deflections due to compaction of the second layer. Subsequent stages were analyzed in the same way, up to H = 0.

This process may be described as follows: It is assumed that the deflections due to compaction are completely inelastic until the fill reaches crown level and completely elastic thereafter. Although the actual field behavior would not be expected to be quite so clear-cut, these assumptions do conform reasonably with expected field behavior. Before the crown of a culvert is covered, the wedging action of the fill at the sides of the structure tends to push the crown up and hold it there because soils rebound when unloaded by a fairly small percentage of the amount they deform on first loading. After the culvert has been completely surrounded by fill, the downward compaction loads on the crown tend to force the haunches out. However, the soil at the sides of the structure has been previously loaded in the same mode, and so it responds much more nearly elastically than during its first load-unload cycle. Therefore, when the compaction load is removed from the fill over the crown of the structure, the soil alongside the structure pushes the haunches back in, and the structure rebounds to nearly its original shape.

Procedure 3, though perhaps a somewhat oversimplified representation of the actual behavior, does satisfy equilibrium, and it results in deflections that agree well with those measured in the field. By using a compaction pressure of 0.8 lbf•ft, this procedure produced calculated deflections in substantial agreement with those measured in the Tice Valley culvert.

The calculated variation of crown deflection with fill height is shown in Figure 6. Each increment of deflection due to fill placement is followed by an increment of deflection due to compaction up to crown level. Subsequently, the compaction loads cause no permanent deflection of the structure.

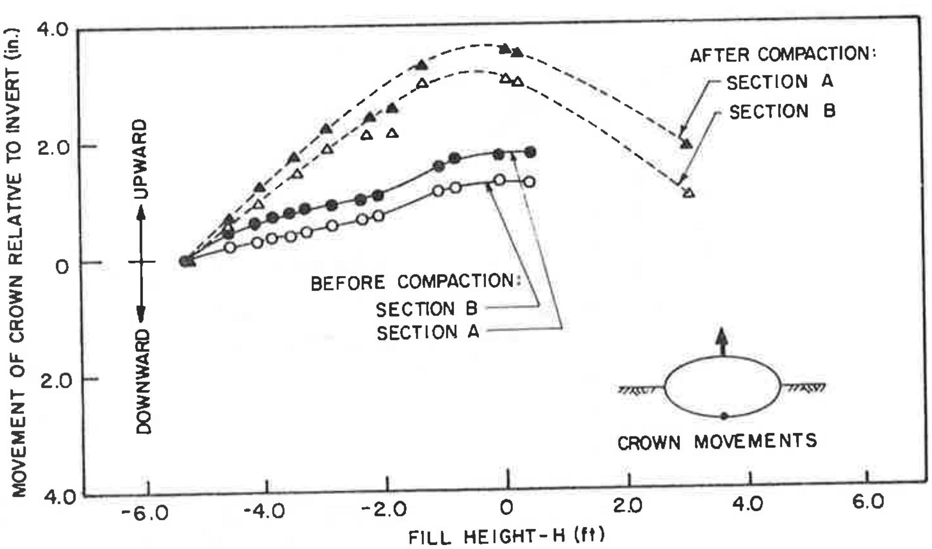
COMPARISON OF CALCULATED AND MEASURED DEFLECTIONS

The measured and the calculated deflections before and after compaction are shown in Figures 7 and 8.

The measured and calculated haunch movements are shown in Figure 7. It may be seen that the calculated values agree quite well with those measured up to H = -2.0 ft. Subsequently, the calculations indicate a considerable increase in span (about 2.0 in), whereas very little occurred in the field. This lack of agreement indicates that the soil alongside the structure was probably considerably stiffer on reloading than was assumed in the analyses. It may be seen that the difference between the measured and the calculated changes in span is as great for conditions before compaction as for conditions after, which indicates that the calculated deflections due to compaction are approximately equal to those measured.

The measured and the calculated crown movements are shown in Figure 8. It may be seen that they are

Figure 5. Movement of crown relative to invert.



in good agreement up to the stage when the crown begins to move down. Subsequently, the calculated downward movements are larger than those measured. It seems likely that the discrepancy is due to the fact that the soil adjacent to the haunches of the structure was actually stiffer than assumed for the analyses, as mentioned previously; restricting outward movement of the haunches, the soil in this zone also inhibits downward movement of the crown. The differences between the measured and the calculated deflections are about equal before and after compaction, which indicates that the calculated deflections due to compaction are reasonably accurate.

BENDING MOMENTS

Calculated distributions of bending moments around the Tice Valley culvert are shown in Figure 9 (for H = 0) and Figure 10 (for H = 4 ft). The dashed lines in these figures represent the distributions of calculated bending moments due to the weight of fill only. The solid lines indicate the range of values calculated assuming two values of percentage rebound on removal of the compaction loads. The actual amount of rebound is unknown; it is considered likely, however, that this value would fall between zero and 30 percent, probably closer to 30 percent. Thus it would be expected that the bending moments would fall somewhere within the shaded bands shown in Figures 9 and 10, most likely near the lower limit of these ranges.

It can be seen that the calculated moment values are very strongly affected by the compaction loads. Values at the crown (point A) and the quarter point (point B) are summarized in Table 1. At H = 0, the calculated bending moments due to both backfill weight and compaction effects are about three to four times as large as those due only to the weight of backfill. At H = 4 ft, the calculated moments at the crown with and without compaction loads are of

Figure 6. Scheme of compaction analyses.

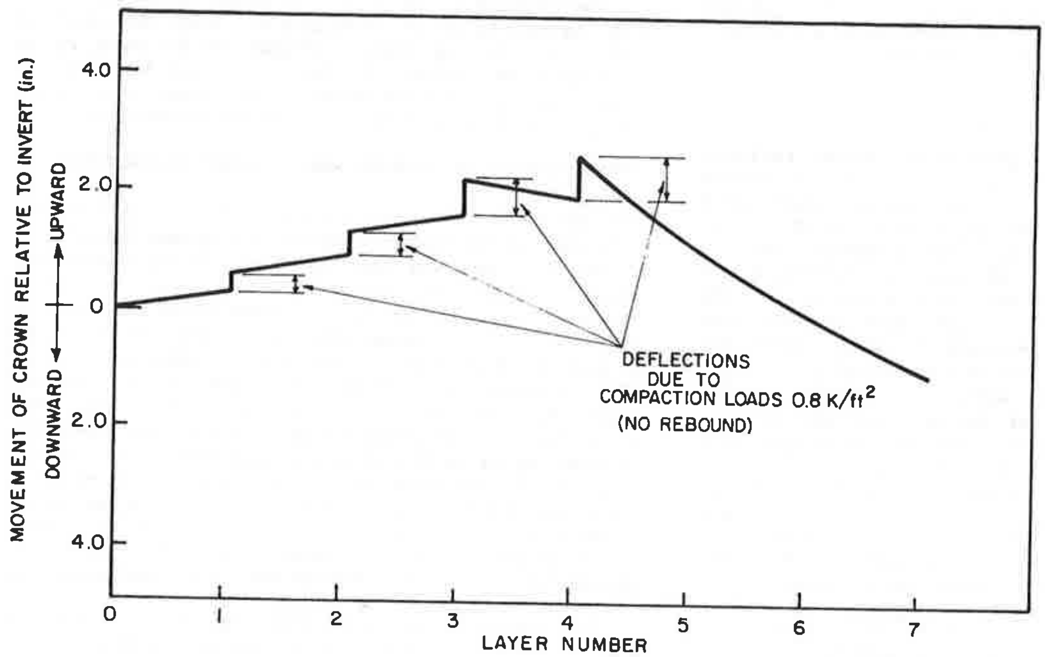
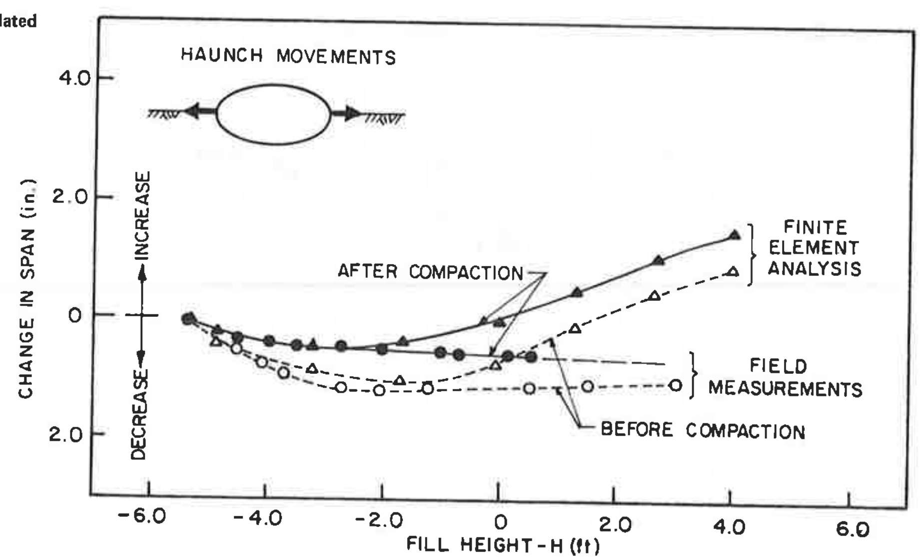


Figure 7. Comparison of measured and calculated haunch movements.



opposite sign: The calculations indicated that the crown would bend down without compaction loads and up with compaction loads. At the quarter point, the calculated moments with compaction loads are about three times as large as those due only to the backfill.

These values of bending moment determined from the finite-element analyses compare reasonably well with values calculated by using the simple soil-culvert interaction (SCI) method of structural design described by Duncan (4). The SCI design procedure can be used to estimate the bending moment at the quarter point (point B). At H = 0 the SCI method indicates a bending moment at B equal to -1.97 lbf•ft. At H = 4 ft, the SCI method indicates a bending moment at B equal to -0.88 lbf•ft. Thus at H = 0, the bending moment from the SCI procedure is 91 percent of the value calculated from finite-element analyses by assuming no rebound and 115 percent of the value calculated by assuming 30 percent rebound. At H = 4 ft, the bending moment from the SCI procedure is 41 percent of the value calculated by assuming no rebound and 51 percent of the value calculated by assuming 30 percent rebound.

The greatest moment in the structure will occur under the H-20 (design load) vehicle. The SCI procedure indicates that this vehicle will induce a bending moment of -1.53 lbf•ft at the quarter point, with tension on the inside of the culvert. Superimposing this moment with the quarter-point moments calculated by the finite-element method (Table 1) results in maximum bending moments from 3.24 lbf•ft to 3.69 lbf•ft. The maximum moment determined from the SCI method is 2.41 lbf•ft, or 66 to 74 percent of the values determined by adding the live load moment to the finite-element values.

Figure 8. Comparison of measured and calculated crown movements.

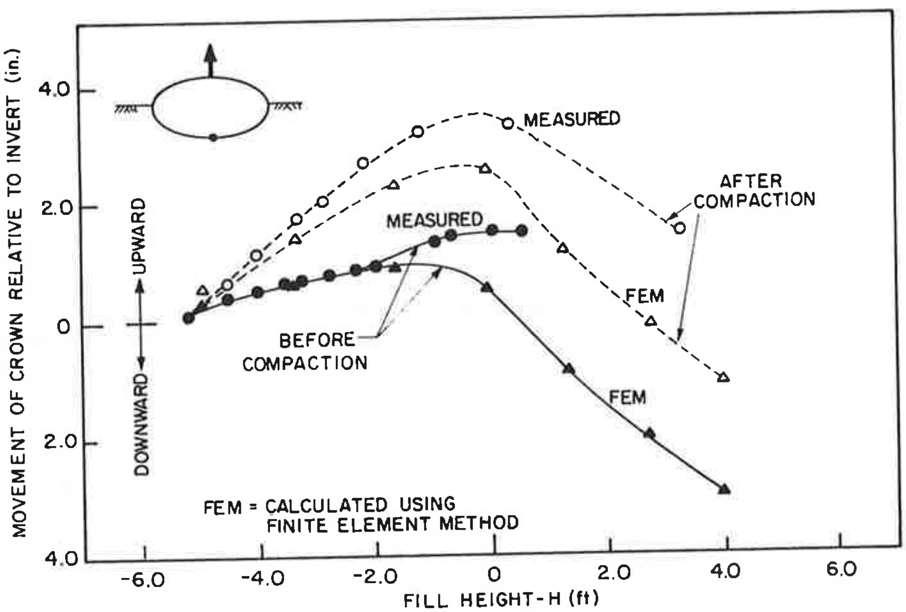


Figure 9. Moments due to backfilling and compaction loads (H = 0).

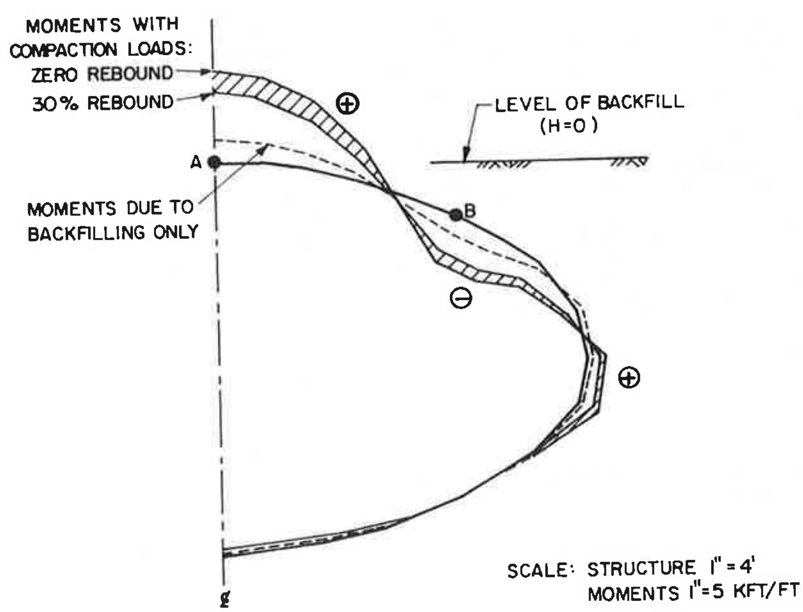


Figure 10. Moments due to backfilling and compaction loads (H = 4 ft).

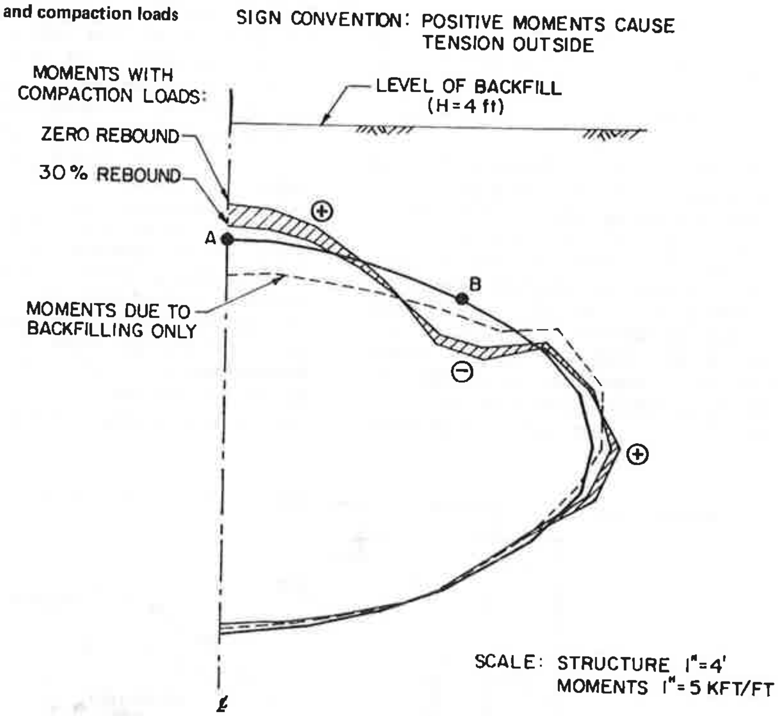


Table 1. Moments in structure due to backfilling and compaction loads.

Fill Level (ft)	Location	Moment (lbf-ft) Backfill Only	With Compaction Zero Rebound	With Compaction 30 Percent Rebound
H = 0	A	+1.04	+3.97	+3.09
	B	-0.67	-2.16	-1.71
H = 4	A	-1.42	+1.51	+0.63
	B	-0.67	-2.16	-1.71

Note: Positive moments cause tension outside the structure; negative moments cause tension inside the structure.

The plastic moment capacity of the Tice Valley culvert at the quarter point is 7.5 lbf•ft and the factor of safety against formation of a plastic hinge is thus about 1.9 as compared with the minimum value of 1.65 conventionally used for design. Thus, although the finite-element analyses including compaction effects indicate moments about 30 percent higher than the simplified SCI procedure, the Tice Valley culvert has more than adequate capacity to withstand these moments.

It should be emphasized that these considerations of the effects of compaction on bending moments are approximate and should be considered speculative until confirmed by more-detailed field studies. More-exact evaluation of bending moments will require strain-gage instrumentation.

CONCLUSIONS

This study of the movement of the Tice Valley long-span aluminum culvert structure during construction shows that the deflections due to compaction were very significant. The upward movement of the crown relative to the invert caused by compaction was about 2.0 in as compared with 1.5 in due to the weight of the fill.

The procedure used to analyze the effects of compaction by the 16 500-lb bulldozer and the 3500-lb vibratory roller appears to model the effects of compaction reasonably accurately. This procedure involved application of a uniform compaction pressure of 0.8 lbf•ft to the surface of each new layer of fill. It was assumed that the backfill behaved completely inelastically during the period when the fill level was below the crown of the culvert (i.e., it was assumed that no rebound occurred during this stage of backfill and compaction) and that the backfill behaved completely elastically when the fill level was above the crown. Although these assumptions are approximations of the actual field behavior, they appear to model the field behavior with a reasonable degree of accuracy.

The bending moments calculated in these finite-element analyses are somewhat larger than values calculated by using the simplified SCI design procedure. Accurate evaluation of the actual moments would require strain-gage instrumentation.

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Joy Taylor, Nancy Hoes, and Rosalind Iiams typed the manuscript.

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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.