EFFECTS OF METHODS A AND B BACKFILL ON FLEXIBLE CULVERTS UNDER HIGH FILLS

David W. Spannagel, Raymond E. Davis, and Alfred E. Bacher,
California Department of Transportation

Two large-diameter, structural steel plate pipes embedded in deep embankments were instrumented and tested to assess circumferential soil stress distributions, deformations, and internal strains. Construction techniques included the "imperfect trench" method (method B backfill) and positive projection (method A backfill). Method B uses layers of baled straw over a 114-in. (290-cm) pipe under 89 ft (27 m) of overfill. Method A consists of ordinary embankment material surrounding twin, 108-in. (274-cm) pipes under 160 feet (49 m) of overfill. Method B soil stress-fill height functions were nonlinear; strains and strain gradients in the pipe walls were larger than those observed for the method A installation. Radial displacements were smaller than those observed for the method A installation. Method A soil stress-fill height functions were essentially linear. Observed deformations and stresses were compared with theoretical values obtained from Marston's theory, the Iowa deflection formula, and the ring compression method. The ring compression method provided correlations that were sufficient for design purposes. Internal strains were correlated with external, measured pressures by neutral point and finite element methods. Baled straw inclusions are not recommended for future designs of flexible pipe culverts. Design can be based on ring compression with a safety factor of 4, but a 70 percent increase in soil densities may be anticipated over a period of time after fill completion.

TWO techniques are commonly employed in backfilling culverts. The first, method A backfill, consists entirely of soil backfill from the foundation to the final grade. A second, originally suggested by Anson Marston of Iowa State University, is called the "imperfect trench" method or method B backfill; it uses a compressible inclusion in the embankment directly above the crown to reduce overburden pressure. This paper presents the results of 2 research projects conducted in California to determine the effects of these backfill types on large-diameter flexible pipe culverts embedded in deep embankments.

PURPOSE

The objectives of these projects were threefold. The first was to assess the behavior of a structural plate pipe culvert under a high fill by measuring (a) the magnitude of deformations, lateral movements, longitudinal dilations, and settlements occurring at each installation; (b) the distribution of soil pressure around the culvert periphery and in the embankment; and (c) the strain distribution within the pipe walls. The second was to assess current analytical and design techniques and compare them to observed pipe behavior. Among the design theories evaluated were Marston's theory of loads on buried conduits, Spangler's Iowa deflection formula, and White and Layer's ring compression theory. Two analytical techniques for determining internal stresses and moments from external applied loads, the neutral point and finite element methods, were also assessed. The third objective was to study the behavioral similarities and differences

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between the 2 prototypes. Recommendations concerning the design of flexible culverts were developed from this comparison.

DESCRIPTION OF THE PROTOTYPES

Method B backfill was employed in a prototype culvert in Chadd Creek canyon in Humboldt County, California, during the fall of 1965 and spring of 1966. The culvert was a 114-in.- (290-cm-) diameter, number 1 gauge, structural steel plate pipe having 6- by 2-in. (15.2- by 5.0-cm) corrugations. An initial ellipticity was produced by a 5 percent vertical diameter elongation. The culvert periphery comprised 6 segments of 60-deg arc each with longitudinal seams at the horizontal diameter. The pipe was installed in a 7-ft- (2.1-m-) deep trench, having shaped bedding; it was backfilled with well-graded, granular backfill to a height of 1 to 2 ft (0.3 to 0.6 m) above the pipe crown. Baled straw was placed in layers 3 to 5 ft (0.9 to 1.5 m) thick, above the structure backfill. The maximum fill height, measured from the culvert crown, was 89 ft (27.1 m). Figure 1 shows the Chadd Creek installation where 3 stations were instrumented extensively to determine the effects of the following parameters (1 ft = 0.3 m):

Station	Max. Fill Height Over Crown (ft)	Structure Backfill Depth (ft)	Baled Straw Depth (ft)
A (0-96)	81	1	5
B (0+44)	89	2	5
C (1+00)	76	2	3

Method A backfill was used in the second prototype culvert, which was constructed at Apple Canyon in Los Angeles County, California, during the spring of 1966. This culvert comprised twin, 108-in.- (274-cm-) nominal-diameter, structural steel plate pipes, which were elongated 5 percent in the vertical dimension. Both pipes were constructed from six 6- by 2-in. (15.2- by 5.0-cm) corrugated plates formed into 60-deg arcs. However, various plate thicknesses, ranging from 0.109 in. [2.77 mm (number 12 gauge)] to $\frac{3}{8}$ in. (9.5 mm), were used along the culvert axis. The twin pipes were placed 4 ft (1.2 m) apart on shaped bedding in an 8-ft- (2.4-m-) deep by 24-ft- (7.3-m-) wide trench with sloping sides. Structure backfill surrounding the pipes was well-graded, granular material placed to a height of 1 ft (0.3 m) above the culvert crowns. One of the pipes was instrumented at 2 stations. Station D, at the centerline of the embankment, was covered by 160 ft (488 m) of overfill; station E, under the sideslope, was covered by 68 ft (20.7 m). Figure 2 shows the Apple Canyon installation.

INSTRUMENTATION

The instrumentation layout was essentially the same at each test station and is shown in Figures 3 and 4. Relative shape changes and wall movements were assessed by measuring the lengths of 14 chords as defined by steel spheres affixed to each pipe's internal periphery at the octant points. Assuming that the vertical diameter remained vertical, chord lengths were used to calculate the position of each sphere in a Cartesian coordinate system with origin at the pipe inversion. No attempt was made to determine the degree of rigid body rotation at either location. A second set of spheres, placed at the quadrant points [2 corrugations or 12 in. (30.5 cm) away from the primary set] was used to assess local, longitudinal elongations.

Soil pressures acting normal to the pipe periphery were measured by modified Carlson soil stressmeters made up of standard, oil-filled stressmeter metal discs fitted with linear variable differential transformers as strain transducers. Each meter was calibrated before installation by placing it between steel blocks and applying a uniaxial compression. Stressmeters were carefully embedded at pipe octant points in the structure backfill, 6 in. (15 cm) from the pipe and tangent to it. Three stressmeters were placed in the embankment material above the pipe as shown in Figure 3.

Figure 1. Culvert installation and cross section for method B backfill at Chadd Creek.

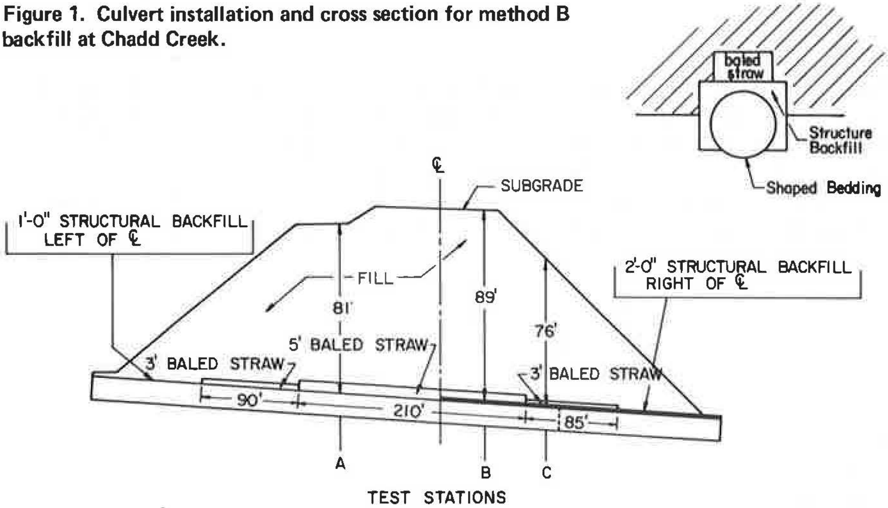


Figure 2. Culvert installation and cross section for method A backfill at Apple Canyon.

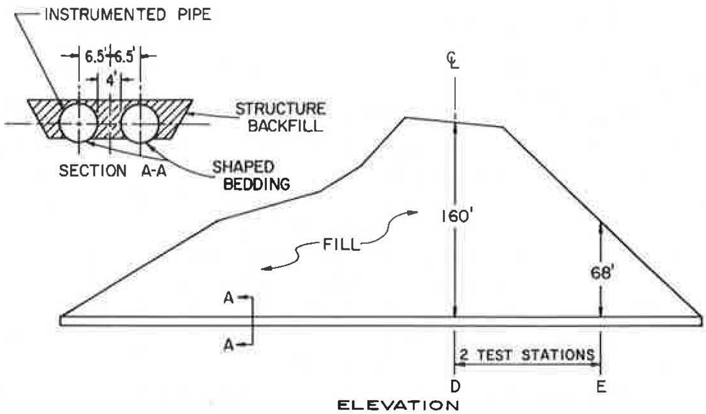


Figure 3. Typical culvert instrumentation.

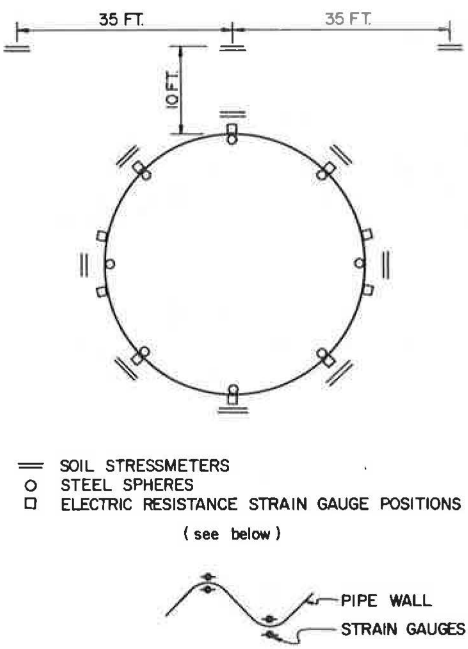
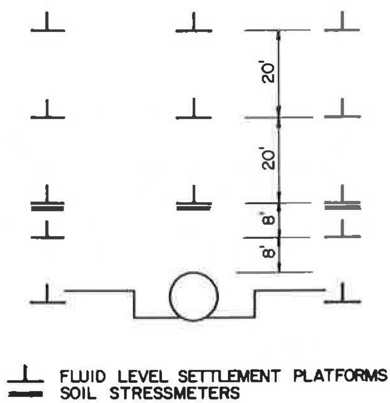


Figure 4. Typical embankment instrumentation.



Electric resistance strain gauges were placed in quadruplets at each of the 3 upper and 3 lower octant points. Four additional quadruplets straddled the longitudinal seams at points 12 in. (30.5 cm) above and below the horizontal diameter to avoid the double plate thickness at the splice. Each quadruplet had gauges with 2 orthogonal elements at the inner and outer corrugation crowns and valleys. Gauge elements were oriented longitudinally and circumferentially. The following equations were employed for calculating stresses from observed strains:

$$\sigma_L = E(\epsilon_L + \nu\epsilon_T)$$

$$\sigma_L = E(\epsilon_T + \nu\epsilon_L)$$

where

σ_L = longitudinal stress,
σ_T = transverse stress,
ϵ_L = longitudinal strain,
ϵ_T = transverse strain,
E = Young's modulus, and
ν = Poisson's ratio.

Soil settlements were assessed by sealed, fluid-level settlement platforms installed within the embankment as shown in Figure 4. Each platform comprised an elongated, water-filled U-tube with a standpipe sealed in a closed pipe of larger diameter and buried in the embankment. The standpipe was filled with water, and an armored plastic tube connected it to a transparent tube mounted on a post on the embankment's side slope. Settlement of the platform lowered the free water surface in the visible tube where levels could be referenced to distance benchmarks. An airline and drain tube were provided to maintain atmospheric pressure at the buried standpipe and to remove overflow water. Culvert lengthening because of embankment dilation, changes in pipe camber, and lateral pipe movements were evaluated by periodic tape and level surveys through the culverts. Survey monuments were placed at intervals along the culverts to determine test station movements.

Compressive strains in the baled straw layers over the Chadd Creek culvert were measured by inverted riser settlement platforms that were made up of plates placed at the upper and lower layer surfaces and of attached reference rods that extended into the culvert below. Deformation of the straw layer was determined by measuring the distance between the pipe crown and the end of each rod.

RESULTS OF OBSERVATION

Soils Data

Physical properties of embankment soils at the 2 culvert installations were determined. Triaxial tests, the results of which are shown in Figures 5 and 6, were made by using specimens compacted in the laboratory to field-measured densities. The properties for Chadd Creek, which was sampled at 10-ft (3-m) intervals of fill depth, and Apple Canyon, which was sampled once at the surface, were as follows (1 pcf = 16 kg/m^3; 1 psf = 4.9 kg/m^2; and 1 deg = 0.02 rad):

Item	Chadd Creek	Apple Canyon
Description	Silty, sandy clay with sandstone and shale fragments	Clayey, sandy gravel derived from soft shale
Density		
Average	137 pcf	
90 percent compaction		109 pcf
Range	126 to 142 pcf	
Avg. moisture content	12 percent	
Avg. cohesion	1,690 psf	1,430 psf
Avg. internal friction angle	26 deg	28 deg

Figure 5. Triaxial test data for embankment soil at Chadd Creek.

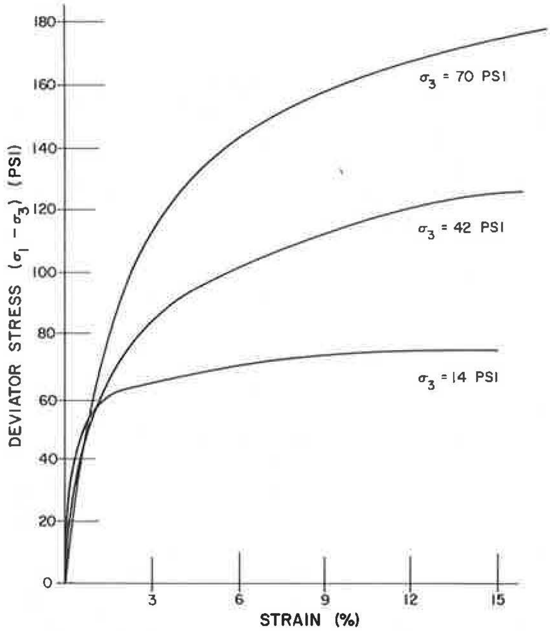


Figure 6. Triaxial test data for embankment soil at Apple Canyon.

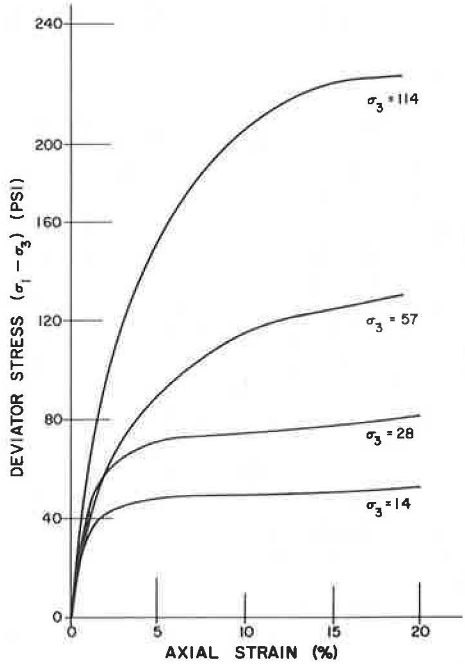
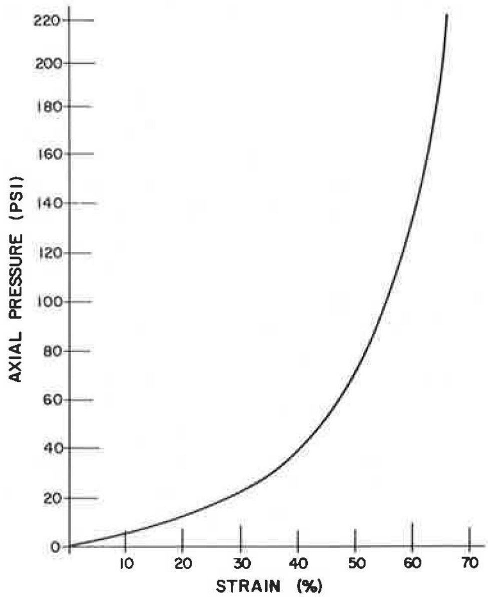


Figure 7. Axial compression tests for baled straw at Chadd Creek.



A modulus of elasticity for baled straw was assessed by uniaxial compression tests on 18- by 24- by 48-in. (46- by 61- by 122-cm) bales confined in an open-ended, rigid wooden box. A stress-strain curve is shown in Figure 7. Total compaction amounted to 12 in. (30 cm) under a 250-kip (1 112-kN) load.

Displacements

Typical, measured changes in pipe dimensions are shown in Figures 8 and 9. Horizontal diameters decreased and vertical diameters increased at both installations until fill material reached the crown level. Thereafter the trend reversed; at 8 ft (2.5 m) of overfill at Chadd Creek and 15 to 20 ft (5 to 6 m) at Apple Canyon, the measured diameters assumed their original lengths. The trend again reversed at 20 to 40 ft (6 to 12 m) of overfill at all 3 stations at Chadd Creek, but no such reversal was observed at Apple Canyon.

The final displaced shape of the Chadd Creek culvert indicated that the vertical diameter had shortened at 2 stations and had returned to its initial length at the third. Horizontal displacements ranged from an increase of 0.4 in. (1.0 cm) to a decrease of 0.5 in. (1.3 cm). These changes were relatively small; the largest occurred at station C under the thinnest overfill and straw layers. Final displacements at Apple Canyon were much larger than those at Chadd Creek. Under 160 ft (49 m) of overfill the vertical diameter at station D decreased 2 in. (5 cm) and the horizontal diameter increased 1.5 in. (4 cm). Station E diameter changes were smaller.

Transverse embankment dilation resulted in elongation of prototype barrels at both sites. Local elongations, measured at each station, were as large as 0.10 in./ft (0.833 cm/m) at Chadd Creek. Tape surveys made between stations A and C indicated an average elongation of 0.025 in./ft (0.208 cm/m) over a 140-ft (43-m) distance.

Local elongation at Apple Canyon amounted to 0.01 in./ft (0.083 cm/m) at station D and 0.02 in./ft (0.167 cm/m) at station E. The average elongation determined by surveys was 0.004 in./ft (0.033 cm/m). These apparent contradictions may be explained by the fact that the overall elongations were measured across the center of embankment dilation where no stretching would be expected.

Soil Pressure Data

Typical soil pressure-fill heights for Chadd Creek and Apple Canyon are shown in Figures 10 and 11. Chadd Creek pressure functions were nonlinear during the fill construction period. For 19 months after fill completion, most stressmeters exhibited temporary stabilization or slight decreases. The majority of meters showed higher pressures after 28 months. Soil pressures at station C under the side slope evidenced distinct longitudinal overburden distribution as they continued to increase at nearly the same rate during the 3-month interval between reaching maximum overfill at that station and fill completion at the roadway centerline.

Pressure-fill height functions at Apple Canyon were nearly linear (Fig. 11) during fill construction. Pressure increases that occurred at station E side slope during the placement of additional fill at station D are evidenced clearly in the center portion of the plot. Pressures rose rapidly for 6 months after fill completion and then at a slower rate to an average level 70 percent higher 36 months after fill completion.

Figures 10 and 11 also depict typical variations in soil effective density as a fraction of fill height. Effective density is the equivalent fill density required to produce a given measured pressure under hydrostatic conditions. The relationship between pressure and effective density is

$$ED = \frac{\Delta P}{\Delta H} \times C$$

where

ED = effective density,
ΔP = change in pressure in psi for a given change in fill height,

Figure 8. Typical measured changes in culvert diameter for method B backfill at Chadd Creek.

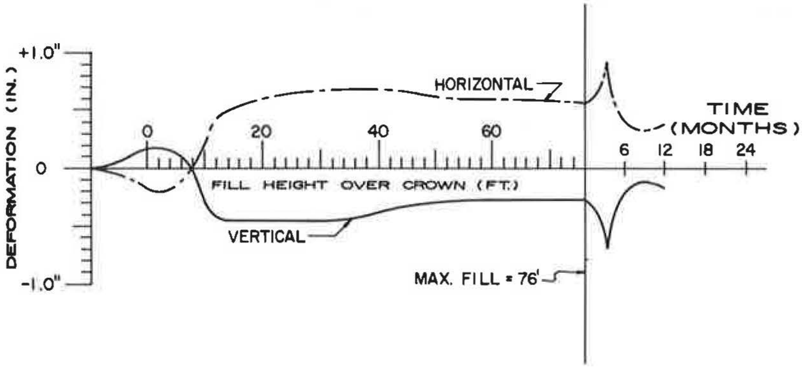


Figure 9. Measured changes in culvert diameter for Method A backfill at Apple Canyon.

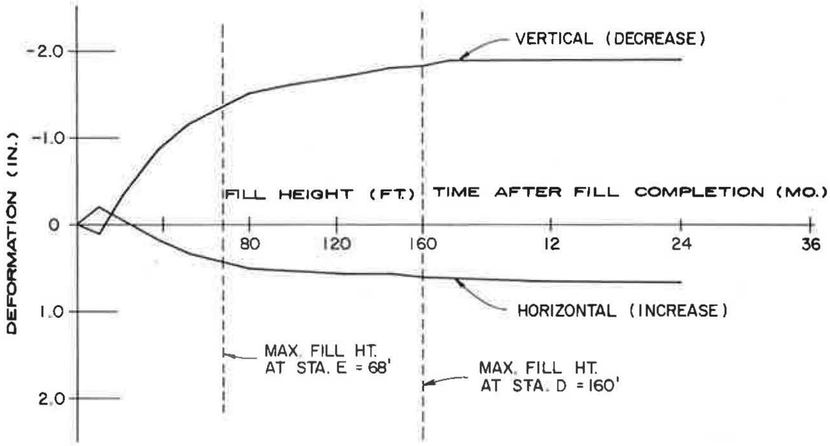


Figure 10. Soil pressure and effective density as functions of fill height and time at Chadd Creek.

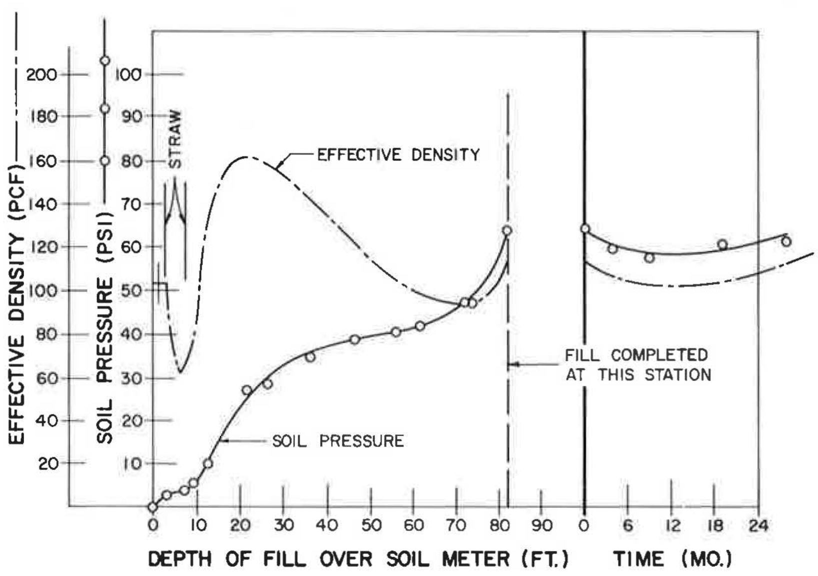


Figure 11. Soil pressure and effective density as functions of fill height and time at Apple Canyon.

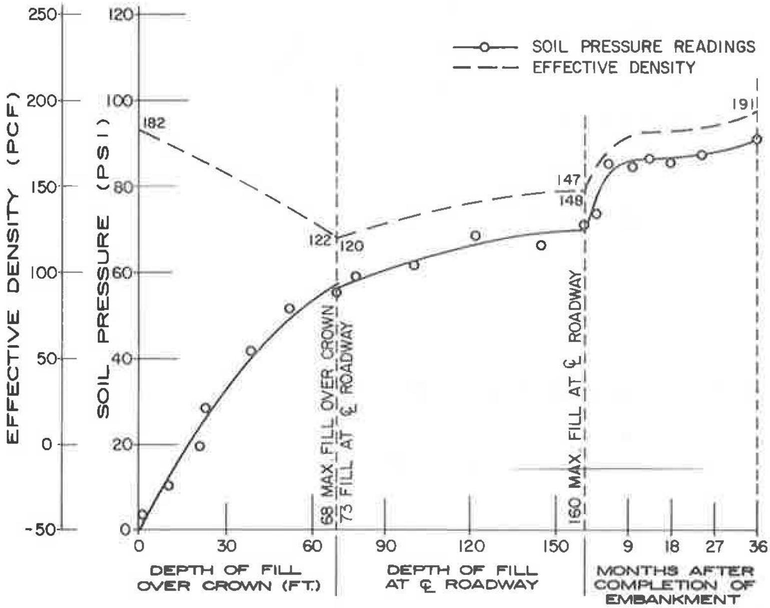


Figure 12. Composite effective density profile for 80 feet of overfill at Chadd Creek.

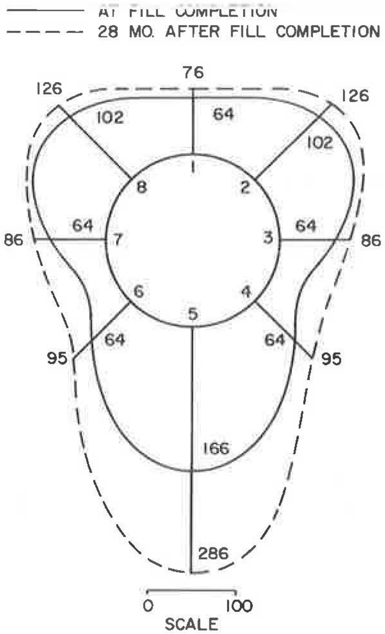
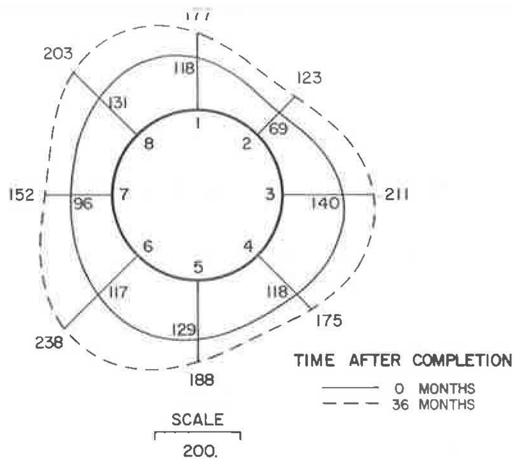


Figure 13. Effective density profile for 160 ft of overfill at Apple Canyon.



ΔH = fill height in feet, and
C = a constant required for units conversion (144 sq in./sq ft in this case).

Typically, the effective densities at Chadd Creek were highly nonlinear and reached relative maxima when the embankment was 10 to 60 ft (3 to 18 m) above the meters and relative minima at fill heights of 70 to 85 ft (21 to 26 m). These extremes occurred in the lower portion of each range for most meters. Apple Canyon exhibited linear effective density functions that decreased slightly during fill construction and showed increases commensurate with pressure changes thereafter.

Composite, circumferential effective density distributions for Chadd Creek are shown in Figure 12. These profiles were constructed by averaging the readings from all test stations to produce a distribution symmetrical about the vertical diameter. During the averaging process, each meter position was characterized by a number of readings that were close to each other in magnitude and 1 reading that differed greatly from the rest. These odd readings, which might be attributed to meter idiosyncrasies and local soil heterogeneity, were eliminated before developing the composite profiles.

The profiles depict the influence of the compressible inclusion in distributing the vertical load across the top of the pipe. Large bulbs of effective density were noted at both upper octant points and the inversion. The density increases that occurred 28 months after fill completion also are shown; the largest increases were observed at the invert.

The effective density profiles for Apple Canyon are shown in Figure 13. Because only 1 station was located under the maximum fill height, averaging to obtain symmetry was not possible. These profiles indicate a tendency toward a circumferential distribution. A small degree of asymmetry may be attributed to the closeness of the second pipe and to relatively large lateral movements. Effective density increases after fill completion averaged 60 to 70 percent.

At Chadd Creek, large effective densities that were twice the actual density as determined by compaction tests were observed in the exterior prisms as a result of transfer of load from the settling interior prism over the straw by shearing forces. Apple Canyon exhibited exterior prism densities that were lower than those over the pipe crown.

Observed Strains

Stretching of both prototype culverts because of embankment dilation as evidenced by the surveys and local longitudinal deformations was also evidenced in strain patterns of those elements of the resistance strain gauges that were oriented longitudinally along the pipe axis. Most of the inner and outer crown elements exhibited compressive strains although the corrugation valley elements manifested tensile strains (as might be expected when pipe is stretched like an accordion). Exceptions to this behavior were noted at each installation at stations B and D near the center of the embankment where a small amount of dilation would be expected. These exceptions probably resulted as differential settlement reduced the culvert camber and produced a longitudinal bending moment in the pipe with its upper half in compression.

Circumferential strain data plotted for cross sections of the pipe wall at both culverts indicated that plane sections remained plane for all gauge quadruplets at each test station. Variations in individual gauge strains as fill height increased were smooth, nonlinear functions; the nonlinearity probably was the result of strain redistribution.

Strain data were used in conjunction with material properties to determine the longitudinal and circumferential stresses. Specified yield and ultimate strengths were 28,000 and 42,000 psi (193 000 and 290 000 kPa) respectively for both pipes. Tests of coupons taken from the corrugated and curved pipe plates produced average yield values and ultimate strengths for the Chadd Creek pipe of 45,900 and 56,400 psi (316 500 and 388 900 kPa) respectively. Corresponding test values for the Apple Canyon pipe averaged 57,300 and 69,300 psi (395 100 and 477 800 kPa).

Figures 14 and 15 show typical stress profiles for the Chadd Creek and Apple Canyon pipes respectively. Circumferential strains commensurate with measured yield

Figure 14. Extreme fiber stress profile for 80 ft of overfill at Chadd Creek.

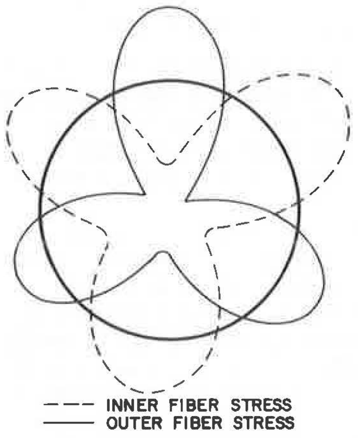


Figure 15. Extreme fiber stress profile for 80 ft of overfill at Apple Canyon.

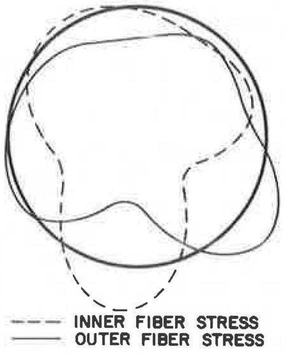


Figure 16. Comparison of moment profiles for method A backfill at Apple Canyon and method B backfill at Chadd Creek.

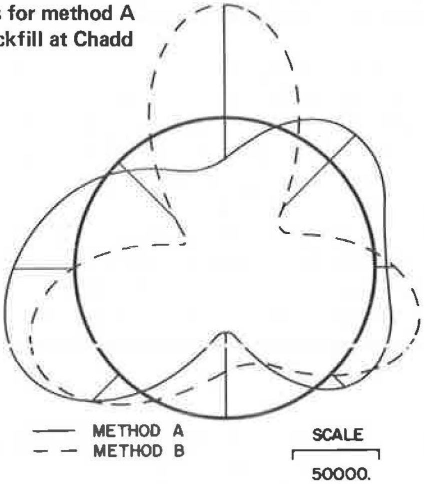
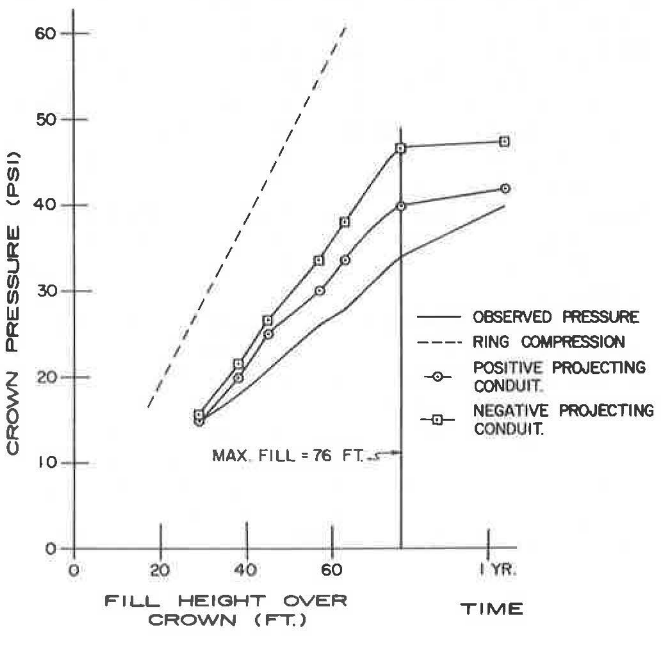


Figure 17. Comparison of crown pressures calculated by various theories and those actually observed for method B backfill.



strength were first observed at Chadd Creek at the crown of each test station with between 63 and 71 ft (19 and 22 m) of overfill. Subsequently, stresses at station A indicated yield along the right side of the pipe from the crown to the inversion. Circumferential yield stresses at the remaining stations were confined to the crown and upper left octant points. Longitudinal yield stresses were observed at the crown and just below the left quadrant point after fill completion at station A only.

Yield strains were measured in the Apple Canyon pipe at station D only. Initial circumferential yield occurred at the inversion under 160 ft (49 m) of overfill and subsequently spread to the lower left octant point, right quadrant point area, and the crown. Longitudinal yield strains were recorded at the crown area at various times following embankment completion.

Station E at Apple Canyon was located in a section of number 1 gauge pipe as were all 3 stations at Chadd Creek. Therefore, stresses, moments, and thrusts observed in the 2 culverts were compared for a fill height of 62 ft (19 m) above each culvert crown, an elevation at which data were collected at all test stations. The maximum tensile stress was found to be about 35 percent higher in the Chadd Creek pipe than in the Apple Canyon pipe, but the maximum compressive stress was only about 4 percent larger. The stress gradient, the difference between stress values at adjacent octant points, averaged 43 percent larger at Chadd Creek.

Circumferential stresses were used in conjunction with pipe section properties to calculate bending moments and thrusts in culvert walls. The influence of curvature on the bending moments was assessed and found to be negligible. At Chadd Creek the largest observed moments and thrusts occurred at the crown at each test station. Maximum values at Apple Canyon were observed at the inversion.

The method used for comparing stresses of the 2 prototypes was also used for comparing bending moments and thrusts. The observed differences between maximum positive and negative moments were negligible, but the moment gradient was about 57 percent higher at Chadd Creek. Figure 16 shows a comparison of moment profiles for both culverts. The maximum thrust and thrust gradient were significantly larger at Apple Canyon.

THEORETICAL ANALYSES

Marston's Theory

The theory of loads on buried conduits, developed by Anson Marston, is 1 method used to determine the vertical load on a culvert crown. The theory and nomenclature are well documented (1, 3). The analysis considers the relationship of settlement of the mass of fill directly over the culvert to that of adjacent fill masses and the resulting transfer of load by shear forces from 1 mass to the other.

The data obtained from the sealed, fluid-level settlement platforms; level surveys; and riser settlement platforms were used to determine the crown load at each tested station by Marston's theory. The settlement ratio, an abstract quantity representing the relative settlement of the soil masses above and to the sides of the conduit, normally is assumed in design but, in this case, was calculated from the settlement data at both culverts. The plane of equal settlement, the horizontal plane above which the settlements of the interior and exterior prisms are equal, was also calculated and compared to the observed embankment settlements.

The Chadd Creek culvert with its baled straw inclusion was classified as an imperfect trench conduit according to Marston's theory. The imperfect trench condition was analyzed as a negative projecting conduit. However, because the pipe actually was placed on original ground, and a trench with sloping sides then was artificially constructed around the barrel, the installation also resembled a positive projecting conduit. For comparison, crown loads were calculated for both classifications.

The observed plane of equal settlement lay between 30 and 32 ft (9 and 10 m) above the crown at Chadd Creek. The negative projecting conduit equations predicted the location of this plane more accurately than did the positive projecting conduit equations. Agreement between the theoretical and observed pressures was poor. Positive projecting conduit crown pressures averaged 17 percent too high at the time of fill completion and ranged from 5 percent low to 50 percent high after 1 year. Negative projecting conduit pressures were high by 8 to 37 percent at the time of fill completion and were high by 13 to 36 percent 1 year later. Figure 17 shows the comparison between theoretical and observed crown pressures for Chadd Creek.

Placement of the twin pipe culverts at Apple Canyon in a wide, shallow trench also produced conditions resembling both positive and negative projecting conduits. Accordingly, crown loads were calculated for both conditions. During fill placement, excellent correlations were obtained between the theoretical and observed crown pressures for both test stations. The average difference between the negative projecting conduit pressures and the measured values was only about 3.3 percent during fill construction, but, as shown in Figure 18, the theory failed to predict the pressure increases after completion.

Spangler's Iowa Deflection Formula

Spangler developed an expression for the anticipated pipe deformations known as the Iowa deflection formula (2, 3). The formula is based on Marston's crown pressures and an assumed peripheral pressure distribution and pipe shape. This formula was applied to both culvert prototypes even though the measured pressure distributions and initial pipe shapes did not correspond to the theory's assumptions.

Design based on the formula requires a knowledge of the installation geometry and the use of assumed constants based on soil compaction (modulus of soil reaction, E') and ratio of final to instantaneous deflections (deflection lag factor, D_L). Observed moduli of soil reaction have been reported to range from 230 to 8,000 psi (1 590 to 55 160 kPa). AASHO specifications suggest a value of 1,400 psi (9 650 kPa) for E', and 1.25 for D_L, when backfill is placed at 95 percent compaction.

At Chadd Creek use of the AASHO design values resulted in calculated deflections that ranged from 10 to 16 times larger than the measured vertical displacements. The appropriate value for E' at Chadd Creek would be from 16,000 to 20,500 psi (110 300 to 141 300 kPa).

Application of the Iowa deflection formula at Apple Canyon using specified E' and D_L values resulted in theoretical deformations 8 to 11 times larger than those observed. Calculated values for E' were about 16,400 psi (113 100 kPa). The extremely large E' values at both culverts can probably be attributed to the confinement afforded by the trenches in which the culverts were placed. This confinement limits the horizontal displacement and thus reduces vertical movements.

Ring Compression Theory

White and Layer's ring compression theory (3) is a method for determining required pipe wall thickness. The theory assumes that the total load on a culvert, the hydrostatic weight of soil above the pipe, is resisted by the wall thrusts. A safety factor of 4 is normally applied to the loads. The crown pressures as calculated for hydrostatic loading are compared with the observed crown pressures at each prototype as shown in Figures 17 and 18.

Ring compression thrust at Chadd Creek was about 6 percent larger than actual wall thrusts when a safety factor of 1 was used; this indicates that design based on the normal safety factor would be adequate in this case. For Apple Canyon, calculated thrusts were only 52 percent of the measured thrusts when the safety factor was 1. A safety factor of 4 yielded thrusts slightly above twice the observed value.

Neutral Point Analysis

Correlations between external applied loads and internal stresses and displacements were established using the neutral point method of analysis. This method is an extension of the general method of indeterminate structures, the theoretical basis of which is described by Grintner (4). For analysis, a culvert subjected to an arbitrary external pressure distribution is made statically determinate by assuming the pipe to be cut at its invert. Loads are applied to a number of small segments of equal

Figure 18. Comparison of crown pressures calculated by various theories and those actually observed for method A backfill.

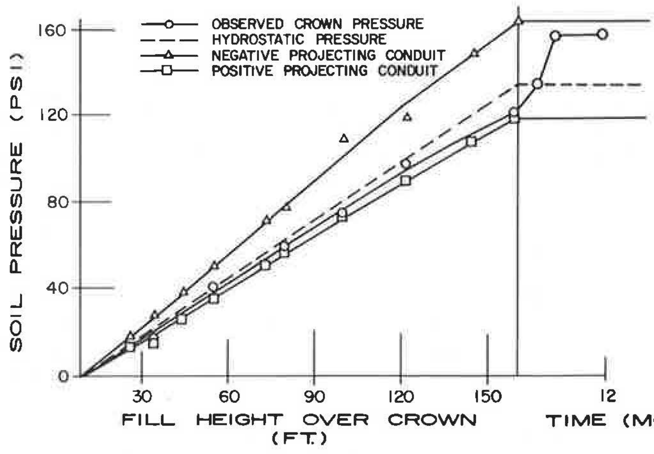


Figure 19. Comparison of observed bending moments and those calculated by the neutral point method for method B backfill.

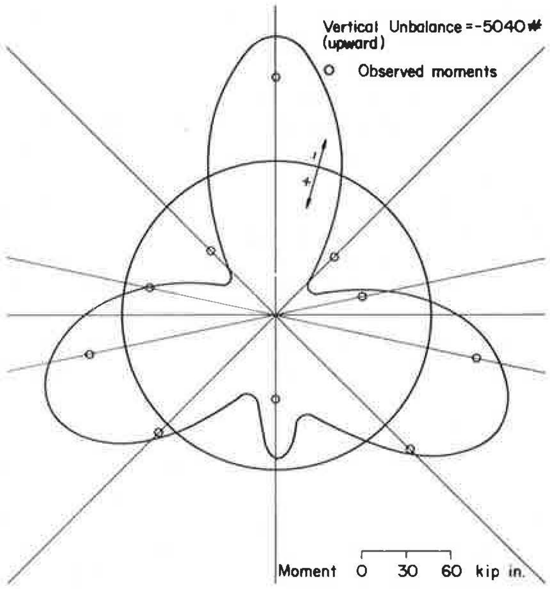
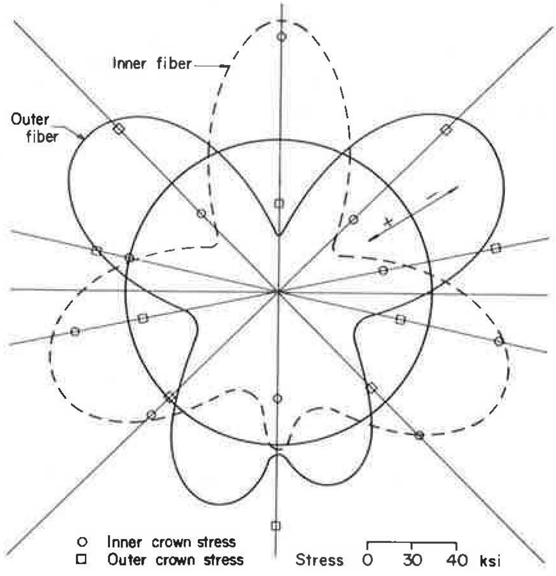


Figure 20. Comparison of observed stresses and those calculated by the neutral point method for method B backfill.



length called voussoirs. One end of the cut ring is assumed to be clamped to a rigid support and the other end is connected by a hypothetical rigid bracket to the centroid of elastic weights or neutral point. Restoration of compatibility after loads are applied is achieved by the application of redundant forces at the neutral point. Moments, thrusts, shears, and displacements are calculated from these forces and the applied loading.

Determining accurate pressure distribution and true pipe shape was a critical phase of the analysis. For low fill heights, the tolerance of the pressure meters was a considerable percentage of the readings, and transducer errors were possible causes for poor correlations between analytical and experimental results. Another problem was the lack of static balance of forces acting on the culvert. The soil shear stresses must be known to produce equilibrium, but, because shear stress transducers were not available, these data were not obtained. Attempts to force static balance by using either a soil friction component or an assumed shear stress distribution reduced the load unbalance but did not improve the correlations with observed quantities.

The analysis was extremely sensitive to small variations in pipe shape. The procedure that best described the shape used 2 vertical semiellipses established from the positions of the octant point steel spheres.

Good correlations were obtained between the theoretical bending moments and stresses and those actually observed at Chadd Creek when a horizontally balanced average pressure distribution and the final displaced shape were used as input. Figure 19 shows the correlation of bending moment. Extreme fiber stress correlations are shown in Figure 20. An analysis using the initial pipe shape also was made, the results of which indicated that stresses and displacements could be predicted with reasonable accuracy if the pipe shape and soil pressures could be predetermined accurately. It was noted, however, that the sensitivity of the method to small errors in either parameter requires that both be predicted precisely.

At Apple Canyon the exact pipe shape was known, but because of the unavailability of soil shear stress data the pipe static equilibrium could not be established. Hence, the neutral point method did not predict pipe behavior accurately.

Finite Element Analysis

The finite element method was used to analyze the Apple Canyon installation, and the output was compared to the measured soil pressures, pipe displacements, moments, thrusts, and embankment settlements. For large embankments, analysis must be based on accurate soil properties and geometric models. At this installation extensive soil property data were not available and no foundation investigation was performed. Soil samples taken from the completed embankment provided the only available data.

Finite element analyses were based on a linear, elastic soil model and a nonlinear, hyperbolic model developed by Kulhawy, Duncan, and Seed (5). Both models resulted in computed peripheral soil pressure distributions that agreed with the observed pressures within 20 percent, but correlations between measured and theoretical displacements, settlements, and pipe stresses were very poor.

CONCLUSIONS

The single pipe installation at Chadd Creek produced nearly symmetric density, stress, and moment profiles for method B backfill. The dual pipe configuration of Apple Canyon was at least partially responsible for the asymmetry observed for method A backfill. We believe, however, that the effects of these construction differences are not of sufficient significance to invalidate any comparisons between the 2 projects.

The nearly uniform pressure profile, linear pressure-fill height functions, and small stress and moment gradients observed at Apple Canyon suggest that method A backfill provides more favorable conditions for flexible culverts than does method B backfill. Control of deflections, a critical consideration in present culvert design, was adequate with either method, although vertical deflections under method B were smaller. Based on the results of these 2 research projects, the use of method B backfill for flexible culverts is not recommended.

Observations at Apple Canyon suggest a uniform peripheral distribution of pressure as a basis for design. However, the large fiber strains demonstrate that significant bending moments may result from small departures from either a circular pipe configuration or a uniform pressure distribution. Such departures are highly probable and should be considered in design.

In general, the design methods discussed in this paper did not predict the observed culvert behavior accurately. Good correlations were obtained between observed crown pressures and theoretical pressures determined by Marston's theory during construction. This method, however, failed to determine the large increases in pressure that occurred after construction was completed, and it provided no facility for predicting the remainder of the peripheral pressure distribution. Moreover, the method requires prediction of soil properties that cannot be accurate. The ring compression theory, including the safety factor, proved conservative in all cases. It is recommended that soil densities used in the ring compression theory be increased by 70 percent to account for long-term effective density increases and to provide a long-term factor of safety of 4.

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