Analysis of Loads and Settlements for Reinforced Concrete Culverts

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FOR REINFORCED CONCRETE CULVERTS

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Three primary factors in considering the loads on, and therefore the design of, underground conduits are: bedding conditions, trench configuration, and use of the "imperfect trench." Models of varied conditions are analyzed using a finite element program and the results compared to data obtained at instrumented sites of similar conditions. Presented herein are charts developed from these analyses which will facilitate future design of culverts. Also presented are recommendations for design criteria and construction procedures.
INTRODUCTION

The structural design of reinforced concrete culverts is dependent upon a realistic estimate of loads to be supported. In years past, embankments over culverts seldom exceeded 40 feet. Now, embankment heights of 100 feet or more are common. The simplest but not necessarily most accurate estimate of loads or bearing pressures is the weight of earth above the structure. In the early 1900's, Anson Marston (1), Director of the Iowa State Engineering Experiment Station, encountered the problem of determining loads on large diameter pipes in deeper cuts than had previously been used. When failures occurred, he initiated experiments to determine why the recognized practices failed.

Marston theorized that loads bearing on underground structures were influenced by an arching effect wherein a portion of the soil weight above the structure is transferred through frictional forces to the side fill material. Depending upon the relative settlement of adjacent soil prisms, the arching effect could either increase or decrease the load to be borne by the structure (1). This arching effect has made it difficult for designers to estimate actual loads. Therefore, the objectives of this study were:

1. to evaluate factors affecting load configurations on culverts under high fills and devise a method of predicting these loads,

2. to study the height of equal settlement,

3. to compare calculated values of load determined by Marston's and other analytical methods to measured loads.
A total of eight test structures at four sites were involved in this study. These structures were chosen to include varied foundation types, fill heights, and construction methods. Foundations were yielding and non-yielding, fill heights varied from 37.5 feet to 180 feet, and four structures involved the use of the imperfect trench. The test sites chosen were on US 27 in McCreary County, KY 627 in Clark County, KY 55 in Marion County, and KY 80 in Laurel County (Figure 1).

All sites were instrumented to monitor differential settlement throughout the fill and pressures on the structure induced by the fill. Some sites also were instrumented to monitor internal stresses and strains of the structure and to monitor hydrostatic pressures in the soil around the structure. Flowline elevations were obtained periodically to determine vertical movements.

One factor in consideration of loads on culverts is the effectiveness of the "imperfect trench" in reducing those loads. This method of constructing the fill over a conduit was a result of efforts by Marston (1) to predict loads on conduits under high fills.

The basis for Marston's theory is the load on a buried conduit is not necessarily equal to the weight of material over the conduit but is influenced by an "arching" action of the overlying material. That arching action is a result of differential settlement of the overlying material and may increase or decrease the load on the conduit. If the soil prism directly over the conduit (Interior Prism) settles more than the adjacent said prisms (Exterior Prism), friction forces would be mobilized that would reduce the load on the conduit. Loading on the conduit would be increased if the adjacent soil prisms settled more than the interior soil prism. Use of the "imperfect trench" is an attempt to insure the interior prism
settles more than the adjacent prisms. This is accomplished by constructing a portion of the fill immediately over the top of the culvert (interior prism) of a highly compressible material (Figure 2).

McCREARY COUNTY

SITE DESCRIPTION

Three test structures are located on US 27 in McCreary County between Whitley City and the Tennessee state line (Figure 3). The structures are at Station 210+50, Laurel Creek; Station 203+20, Laurel Creek; and Station 89+20, Bridge Fork. The structures are cast-in-place, reinforced concrete, box culverts. The imperfect trench method of construction with an unyielding foundation was used at each site. Fills over the culverts were designated as high fills. At Station 89+20, an 8-foot by 8-foot culvert was placed under 48 feet of fill (Figure 4). At Station 203+20, a 5-foot by 5-foot culvert was placed under 72 feet of fill (Figure 5). At Station 210+50, a 5-foot by 6-foot culvert was placed under 96 feet of fill (Figure 6). Projection conditions for those sites may be seen in Figures 4 through 6.

CONSTRUCTION

Construction of the structures began in May 1972 and was completed in the fall of that year. The imperfect trench was constructed by completing the fill to a height above the culvert equal to the culvert height plus 1 foot. Then, a trench equal to the culvert height and width was excavated directly above the structure. The bottom third of the trench was filled with loose straw. The remaining two thirds were filled with lightly compacted fill material (Figures 4 through 6). Embankment construction then was continued in the normal manner.
GEOLGY

The sites are located in an area where the Breathitt and Lee Formations of Pennsylvanian age outcrop. The area around Whitley City is dominated by the Corbin Sandstone, a member of the Lee Formation (2). This formation is characterized by coarse- to medium-grained sandstone, light gray to brown in color, occurring in massive formations around Stearns and Whitley City. Beds of shale and siltstone 10 to 90 feet thick underlie the Corbin Sandstone. Near Pine Knot, the massive sandstone of the Corbin member joins with carbonaceous shale and siltstone of the Lee Formation. This lies in the River Gem coal bed and is characterized by shale and siltstone with discontinuous beds of sandstone. The structure foundations were of shale common to the area.

SOILS DATA

Soils used in the embankments were sands with some shale intermixed. Particle-size analyses of samples from all three sites indicated the average sand content of the soils was 74 percent, except at Station 203+20. At that site, the sand content increased with depth in the fill and ranged from 32 percent to 65 percent. Clay contents were low, ranging from 11 percent at Station 89+20 to 31 percent at Station 203+20. Results of particle-size analysis, Atterburg limits tests, and CIU (consolidated-isotropic-undrained) triaxial tests are listed in Table 1.

INSTRUMENTATION

Instrumentation at the sites included Carlson earth stress cells, strain gages, settlement plates, multipoint settlement gages, and pneumatic piezometers. Most instrumentation proved reliable and was monitored for several years. However, the strain gages did not perform reliably and were
abandoned soon after construction was completed. A total of 42 earth pressure cells, ten settlement plates, seven settlement gages, and four piezometers were installed.

Fourteen Carlson earth stress cells were placed at each culvert. Eleven cells were placed at the intersection of the culvert and centerline of the roadway. Three were placed in the top slab, three in each sidewall, and one on either side of the culvert on the foundation. Between the culvert inlet and the roadway centerline, two additional cells were placed in the top slab and one in the sidewall. All electrical cables were brought to the inside of the culvert and run through metal guttering to an external monitoring station. Locations of stress cells are shown in Figures 4 through 9. Earth stress cells were monitored for several years with 32 of 42 cells functioning properly for at least six years. Several of the ten remaining cells functioned properly in excess of three years.

Settlement monitoring instrumentation consisted of inverted settlement plates and multipoint settlement gages. Inverted settlement plates were placed at the top of the imperfect trench at various distances along each culvert (Figures 7 through 9). Three plates were placed at Station 89+20 and Station 203+20. Four were placed at Station 210+50. Casings for each plate extended through the top slab of the culvert. This permitted monitoring of the plates from inside the culverts. Figure 10 illustrates the typical placement of earth stress cells and inverted settlement plates at these sites. The vertical pipes shown in that figure are the casings for the inverted settlement plates and three earth stress cells may be seen in a diagonal line across the culvert, near the casing in the foreground.
Settlement gages were installed immediately above the imperfect trench at Stations 89+20 and 210+50. Additional gages were placed at varying elevations throughout the fills, with a total of three gages at Station 89+20 and four gages at Station 210+50. Locations of settlement gages at those sites are shown in Figures 7 and 9.

After construction was completed, holes were drilled on both sides of the culverts at Station 89+20 and Station 210+50. The holes were drilled to depths of the bottoms of the culverts. Piezometers were placed at the bottoms of the holes and the holes were sealed above the piezometers with bentonite plugs (Figure 11).

DATA

Flowline elevations were recorded at each culvert site as a baseline for settlement determinations. At Stations 203+20 and 210+50, initial elevations were obtained from construction plans and compared to subsequent surveyed elevations. At Station 89+20, initial flowline elevations were obtained approximately ten months after completion of the culvert.

Average settlement throughout the length of the culvert at Station 89+20 was 0.035 foot. The maximum settlement was 0.072 foot. Flowline elevations at Stations 203+20 and 210+50 indicated average settlements of 0.348 foot and 0.284 foot, respectively. Maximum settlement at Stations 203+20 and 210+50 were 0.389 foot and 0.354 foot, respectively. Flowline elevations were not monitored during construction of the fill, but indications are that approximately 90 percent of the settlement of the structures occurred during the first year after construction.

Settlement plates and settlement gages were monitored frequently during construction of the fills at each site. Settlement plate data indicated an average settlement of 1.192 feet at Station 89+20 with settlements ranging
from 0.958 foot to 1.325 feet. Average settlement at Station 203+20 was 1.067 feet with a range from 0.900 foot to 1.150 feet. Average settlement at Station 210+50 was 0.742 foot with a range from 0.600 foot to 0.825 foot. Approximately 84 percent of the settlement occurred within 30 days after placement of the settlement plates, as may be noted in Figures 12 through 14.

Settlement gage data at Station 89+20 indicated the greatest settlement occurred immediately above the imperfect trench, and directly over the center of the trench. Monitoring points on Gage 1 (Figures 7 and 15), which were over the center of the trench, averaged 0.853 foot of settlement (Figure 16). Monitoring points adjacent to those, but not directly over the trench, averaged 0.383 foot of settlement. As shown in Figures 17 and 18, settlement decreased at higher elevations in the fill, but monitoring points directly over the imperfect trench (Figure 15) continued to settle more than other points.

At Station 210+50, monitoring points on Gage 1, which were directly over the trench (Figures 9 and 19), averaged 0.645 foot of settlement (Figure 20). Monitoring points adjacent to those, but not directly over the trench, averaged 0.328 foot of settlement. As shown in Figures 21 and 22, settlement decreased at higher elevations; however, monitoring points directly over the trench (Figure 19) settled more than other points. Approximately 70 percent of the settlement of the fill immediately above the imperfect trenches (Gage 1 at each site) occurred within 30 days after construction of the trench (Figures 16 and 20).

Piezometers installed at Stations 89+20 and 210+50 were monitored for several years. At no time was an appreciable hydrostatic pressure detected.

For the purpose of comparing pressure on culverts at different sites
and under different conditions, all measured pressures were related to
design loads of the culvert at that point. Design loads were determined
using the AASHTO Standard Specifications for Highway Bridges, Section
1.2.2(A). Converted to compatible units and simply stated, these are:

\[ P_C = \frac{wh}{144} \quad \text{for vertical pressure and} \]
\[ P_C = \frac{wh}{576} \quad \text{for lateral pressure} \]

where \( P_C \) = calculated pressure (pounds per square inch),
\( w \) = unit weight of fill material (pounds per cubic foot), and
\( h \) = height of fill (feet).

To more conveniently illustrate the relationship between measured and
calculated pressures, effective density of the fill material, \( D_E \), was used.
Effective density was defined to be the following relationship:

\[ D_E = \frac{P_M}{P_C} \]

where \( D_E \) = effective density of the fill,
\( P_M \) = measured pressure, and
\( P_C \) = calculated pressure.

Measured pressures (\( P_M \)) for the sites are plotted versus calculated
pressures (\( P_C \)) in Figures 23 through 25. The line of equality indicates a
\( D_E \) of 1.000.

At Station 89+20, \( D_E \) ranged from 0.269 to 5.871 with an average of
1.421. In general, higher values were recorded for cells at locations other
than the intersection of the culvert and roadway centerline. Two cells at
the central instrumentation location indicated high \( D_E \) values. Those
cells, Numbers 4 and 11 (Figure 4), were located adjacent to each other. \( D_E \)
for cells located in the top slab of the culvert averaged 1.371. Pressure
data recorded at Station 89+20 are plotted versus time in Figure 26.

At Stations 203+20, $D_E$ ranged from 0.252 to 3.730 with an average of 1.249. As at Station 89+20, the cell having the highest $D_E$ value, Cell 15, was located away from the central instrumentation location (Figure 8). $D_E$ for cells located in the top slab of the culvert averaged 0.935. Pressure data recorded at Station 203+20 are plotted versus time in Figure 27.

At Station 210+50, $D_E$ ranged from 0.328 to 2.860 with an average of 1.151. $D_E$ for cells located in the top slab of the culvert averaged 0.576. Pressure data recorded at Station 210+50 are plotted versus time in Figure 28.

CLARK COUNTY

SITE DESCRIPTION

Two test structures are located on KY 627 between Winchester and the Kentucky River in Clark County (Figure 29). At Station 123+95, a 4-foot by 4-foot, cast-in-place reinforced concrete box culvert was placed under 77 feet of fill on a yielding foundation (Figure 30). At Station 268+30, a 4-foot by 5-foot cast-in-place reinforced concrete box culvert was placed under 37.5 feet of fill on an unyielding foundation (Figure 31). Those sites were chosen because they were positive projecting and were not designed for the imperfect trench. The structures at Station 123+95 and Station 268+30 project 1.0 foot and 3.5 feet, respectively, above the natural ground.

CONSTRUCTION

Construction of the structures began in the spring of 1974. The fill was completed to grade at both sites in October of 1974. The original soil
at both sites was excavated approximately 15 feet wide and 4 feet deep. Bedrock encountered at Station 123+95 was excavated to 1 foot below foundation level and backfilled with dense-graded aggregate. Soil encountered also was undercut and backfilled with dense-graded aggregate. The trench was backfilled using the excavated material. A layer of the original soil, approximately 3 feet deep, was placed over the culvert. The remainder of the fill was constructed in alternating layers of shot limestone rock and soil.

At Station 268+30, bedrock was excavated to slightly below foundation level, and soil encountered was removed to bedrock. The site was prepared to foundation level with crushed rock. The culvert was constructed on the crushed rock, and the trench was backfilled with excavated soil. Soil was used to construct the fill to approximately 15 feet above the culvert. The remainder of the fill was shot limestone. To facilitate drainage of the surrounding fill, 4-inch weepholes were constructed in the culvert sidewalls at both sites. Weepholes were placed on 8-inch centers and vertical drains were placed at each weephole (Figure 32).

GEOLOGY

The culvert at Station 123+95 lies in the Tanglewood Member of the Lexington Limestone Formation. The formation is of Ordovician Age (3). The Tanglewood is a light gray to brownish-gray fine- to coarse-grained limestone. The culvert at Station 268+30 lies in the Brannon and Grier Members of the Lexington Limestone (4). The Brannon Member is a medium gray to light brownish-gray limestone interbedded with gray clay shale. The Grier Member is a medium light gray to medium gray, very finely to coarsely bioelastic limestone.
SOILS DATA

Soils used in the embankments at Stations 123+95 and 268+30 were classified as MH and ML-CL, respectively. Soils at both locations contained a large percentage of clay and silty material. The soil at Station 123+95 was composed of 43 percent clay and 40 percent silt. At Station 268+30, the soil was composed of 31 percent clay and 31 percent silt. Other pertinent soil characteristics are listed in Table 2.

INSTRUMENTATION

Instrumentation at the sites included Carlson earth stress cells, multipoint settlement gages, and piezometers. Ten earth stress cells were installed at each culvert location. Two cells were placed in each sidewall, in the top slab, in the bottom slab, and on the foundation (Figures 30 and 31). The cells in the bottom slab were placed beneath the sidewalls and the foundation cells were placed within the trench approximately 2 feet from the bottom slab. Electrical cables were either tied to reinforcing bars and concreted in place or extended through electrical conduit inside the culvert to an external monitoring station. Of twenty cells installed at the sites, nineteen continue to operate properly as of this reporting date. The remaining cell, Number 57 at Station 123+95, operated properly in excess of 1,300 days.

Nine multipoint settlement gages were installed. Five were installed at Station 123+95 and four at Station 268+30. At each location, one gage (Gage No. 1) was placed in the bottom slab. The second gage (Gage No. 2) was placed approximately 2 feet above the culvert and as near the centerline of the culvert as possible. The remaining gages were spaced throughout the fills (Figures 33 and 34) while maintaining the positions relative to the centerline of the culvert. Gages 1 and 3, at Station
123+95, were damaged during construction and rendered inoperative. Figure 35 shows the culvert inlet at Station 268+30 with Settlement Gage 1 and the electrical cables for the earth stress cells exiting the concrete.

During construction of the fills, two piezometers were installed at each culvert location. Holes were drilled on both sides of each culvert approximately 15 feet from the centerline of the road. At Station 123+95, piezometers were installed approximately 6 feet above the top of the culvert and approximately 9 feet from either sidewall. At Station 268+30, piezometers were installed at approximately the elevation of the bottom slab of the culvert and approximately 1 foot from either sidewall (Figure 36). After the piezometers were in place, the holes were sealed with bentonite clay.

DATA

Placement of settlement gages at the sites precluded obtaining settlement data in the exterior soil prisms. However, settlement data obtained did reflect the differing foundation conditions of the culverts. Data recorded at Station 268+30, Gages 1 and 2 (Figure 34), indicated very little settlement of the culvert or of the gage immediately over the culvert. Gages placed higher in the fill, Gages 3 and 4, indicated settlement averaging less than 1 inch.

Data recorded at Station 123+95, where the culvert was constructed on a yielding foundation, indicated significantly more settlement. Gage 2 (Figure 33) indicated settlement ranging from 0.332 foot at Point 1, approximately 75 feet from the slope surface, to 0.520 foot beneath the centerline of the roadway (Figure 37). All gages at this site indicated increasing settlement toward the centerline of the roadway with Gage 4 ranging from 0.151 foot to 0.542 foot (Figure 38) and Gage 5 ranging from
0.188 foot to 0.263 foot (Figure 39).

As in the case of settlement data, pressure data at the sites reflected the difference in foundation conditions. Probably as a result of greater settlement of the interior soil prism due to the yielding foundation, calculated effective densities, $D_E$, at Station 123+95 were significantly less than at Station 268+30. Average $D_E$ at Station 123+95 was 1.513 with a range from 0.430 to 2.275 (Figure 40). Average $D_E$ at Station 268+30 was 2.832 with a range from 1.702 to 4.722 (Figure 41). $D_E$ for cells located in the top slab of these culverts averaged 1.488 for Station 123+95 and 2.152 for Station 268+30. Pressure data recorded at these sites are plotted versus time in Figures 42 and 43.

Piezometers installed at the sites were monitored for several years. At no time was a significant hydrostatic pressure detected.

MARION COUNTY

SITE DESCRIPTION

Two test structures are located on KY 55 in Marion County between Campbellsville and Lebanon (Figure 44) at Station 448+90 and Station 459+50. Both are cast in place reinforced concrete box culverts and were designated as having yielding foundations. At Station 448+90, the imperfect trench method of construction was used with a 5-foot by 5-foot culvert under 69 feet of fill (Figure 45). At Station 459+50, a 6-foot by 6-foot culvert was placed under 50 feet of fill (Figure 46).

CONSTRUCTION

Construction of the structure at Station 448+90 began in September 1980 and the fill was completed to grade in May of 1981. At Station 448+90, the
trench was undercut approximately 3 feet below the foundation elevation throughout the length of the culvert. An additional 3 to 4 feet were excavated approximately 70 feet either side of the centerline of the culvert. The trench was then backfilled to foundation elevation with shale common to the area. Solid, unweathered shale was encountered toward either end of the culvert with the middle portion of the excavation containing only weathered shale. The trench, as excavated, resulted in a negative projection of approximately 4 feet below the original groundline for the culvert.

The imperfect trench was constructed by backfilling the trench around the culvert and completing the fill to a height above the culvert equal to the height of the culvert plus 2 feet. The trench was then excavated to a depth and width equal to the dimensions of the culvert. The bottom third of the trench was filled with loose straw and the remaining two thirds were filled with uncompacted earth. Construction of the fill then proceeded in a normal manner.

Construction of the structure at Station 459+50 began in July 1980 and the fill was completed to grade in July 1981. At Station 459+50, a trench was excavated until moderately weathered shale was exposed. This varied from 2 inches to 8 inches below foundation elevation. The trench was backfilled with limestone gravel to foundation elevation. The culvert was then constructed with a positive projection of approximately 1.5 feet above the original groundline. The fill then was constructed with soil and shale common to the area. The completed structure before backfilling began is shown in Figure 47.

GEOLGY

Bedrock in the area is the Mississippian age Borden Formation (5). The
culverts were constructed on the New Providence Shale Member of that Formation. The New Providence is a light gray to greenish-gray fissile clay shale having a low slake durability index. This shale, as evidenced by Figure 48, is unstable when saturated.

SOILS DATA

Tests indicated the soil characteristics were essentially the same for both sites. The soil was classified as ML-CL by the Unified Soil Classification system or A-6 by the AASHTO system. Specific gravity was 2.93 with a liquid limit of 38.0 and a plasticity index of 13.9. This soil was high in clay and silt content with approximately 24 percent clay and 54 percent silt. Parameters determined by triaxial tests were $\phi'$ = 35.6 degrees and $c'$ = 1.09 pounds per square inch.

INSTRUMENTATION

Instrumentation at the sites included Carlson earth stress cells, multipoint settlement gages, and horizontal slope indicators. A total of 29 earth stress cells, six settlement gages, and six slope indicators were installed.

Horizontal slope indicators were installed at the sites as a supplementary settlement monitoring system. The system consisted of 3-inch PVC conduit placed in a continuous line through the fill and exiting on the side slopes at both ends (Figure 49). A rope or cable was placed inside the conduit and was used to draw a sensor through the conduit. Accelerometers, housed in the sensor and sensitive to changes in vertical alignment, allow the determination of a continuous profile of the conduit. Protruding ends of the slope indicators were referenced to bench marks to provide vertical control.
At Station 448+90, sixteen earth stress cells were installed. At the intersection of the culvert and roadway centerline, three cells were placed in the top slab, three in each sidewall, one in the footer, and one on either side of the culvert on the foundation. Between the culvert inlet and the roadway centerline, four additional cells were placed in the footer (Figure 45). Pressure cell placement at Station 459+50 was similar, except the four additional cells were omitted and one was placed in each side of the footer for a total of 13 cells (Figure 46). All stress cell electrical cables were tied to reinforcing steel inside the forms for the culvert sidewall. Cables exited the concrete at the inlet wingwall where a monitoring station was established.

Horizontal slope indicators and settlement gages were installed at the same elevation so that the settlement gages would provide backup service in case the relatively untested slope indicators performed poorly. The first two slope indicators at Station 448+90 and the first indicator at Station 459+50 were not functional due to poor installation procedures. Slope indicators installed later functioned properly. At Station 459+50, only Gage 2 of the settlement gages operated properly. Location of settlement instrumentation at Station 448+90 is shown in Figures 50 and 51. Settlement instrumentation locations for Station 459+50 are shown in Figures 52 and 53.

DATA

Earth stress cells at Station 448+90 indicated an average effective density, $D_E^*$, of 1.316 with a range from 0.486 to 1.997 (Figure 54). At Station 459+50, $D_E^*$ averaged 1.371 with a range from 0.558 to 1.741 (Figure 55). Meters away from the central instrumentation location (Figure 51) ranged from 1.469 to 1.997. $D_E$ for cells located in the top slab at these
culverts averaged 1.331 at Station 448+90 and 1.551 at Station 459+50. When compared to other sites not employing the imperfect trench, average $D_E$ at Station 459+50 was low. This could be a result of the foundation of weathered shale on which the culvert was constructed. Pressure data recorded at these sites are plotted versus time in Figures 56 and 57.

Monitoring points on Settlement Gage 1 at Station 448+90 were placed so that Points 1 and 2 would be approximately 3.5 feet from the vertical plane of the imperfect trench. Data from this gage revealed little differential settlement of the points (Figure 58), thus indicating that all points are within the soil prism influenced by the imperfect trench. Average settlement of points on Gage 1 was 1.349 feet.

Settlement gages placed higher in the fill indicated decreasing settlement at higher elevations. Gage 2 monitoring points averaged 1.070 feet of settlement (Figure 59) and Gage 3 monitoring points averaged 0.486 foot of settlement (Figure 60). Settlement data obtained from Gages 2 and 3 did not reveal evidence of differential settlement of soil prisms.

Horizontal Slope Indicator 3 was installed approximately 1.0 foot lower in elevation than Settlement Gage 3 (Figure 50). Maximum settlement of the slope indicator was 0.896 foot as compared to a maximum settlement of 0.692 foot for Settlement Gage 3. As shown in Figure 61, horizontal slope indicator data indicated a possible differential settlement of soil prisms. The area of greater settlement extends approximately 25 feet along the indicator and is directly over the imperfect trench.

At Station 459+50, settlement data again indicated decreasing settlement as elevation in the fill increased. Settlement Gage 2 (Figure 52) indicated an average settlement of 0.388 foot (Figure 62) with monitoring points under the highest fill (Figure 53) settling most.
Horizontal Indicators 2 and 3 settled a maximum of 0.892 foot and 0.579 foot, respectively (Figure 63). Greatest settlement of each indicator occurred beneath the highest part of the fill.

LAUREL COUNTY

SITE DESCRIPTION

The test structure is located on KY 80 in Laurel County between London and Somerset (Figure 64) at Station 1202+12 where KY 80 crosses Pine Creek. KY 80 was constructed as a two-lane road with truck lanes; however, drainage structures were constructed to allow additional lanes in the future. At this site, 180 feet of fill were placed over a culvert constructed on a yielding foundation (Figure 65).

CONSTRUCTION

The culvert differs from other culverts involved in this study in that it is a precast, reinforced concrete pipe. The culvert has twin conduits having an inside diameter of 8 feet and a maximum wall thickness of 23 inches. The conduit was placed in sections of 6- and 8-foot lengths with a minimum of 2 feet of fill between the conduits. The sections were cast in Louisville, Kentucky, and transported to the site by truck.

Bedding conditions were as specified in Kentucky Department of Highways Standard Drawing No. RDI-020-03. A trench was excavated 7.35 feet below the bottom of the conduit. Areas where rock was not encountered were excavated to rock and the additional undercutting refilled with sandstone. Soil was then placed in the trench and compacted. When the soil was at an elevation $H_c/3$ above the bottom of the culvert, a template matching the outside dimensions of the culvert was used to groove the backfill. The
conduit was then placed in the groove and the backfill operation continued
with special attention to the compaction of material below the horizontal
diameter of the conduit.

Construction of this structure began in August 1980 but was halted
temporarily. Initial construction of the culvert bedding did not meet
specifications for soil classification of the bedding material, groove
construction, or compaction of the material around the conduit. All conduit
and bedding was removed and reconstructed in compliance with
specifications. Reconstruction began in September 1980 and the fill was
completed to grade in September 1981. Figure 66 shows the reconstruction of
the outlet end of the culvert.

GEOLOGY

Bedrock in the area is comprised of the Lee Formation of Pennsylvanian
age and the upper portion of the Pennington Formation of Mississippian age
(6). The Pennington Formation, which is the sub-foundation at this site,
contains mudstone, sandstone, and limestone. Mudstone is green or reddish
brown and locally shaly. Sandstone is green to brown, fine- to very fine­
grained and occurs in beds from very thin to 20 feet thick. Limestone has
been cut out locally by the overlying, gray to buff fine- to medium-grained
sandstone of the Lee Formation.

SOILS DATA

Materials used in constructing the embankment were primarily sandstone
and shale common to the area. Culvert bedding material was also soil
common to the area. Tests conducted on the bedding material indicated a
classification of SM by the Unified Soil Classification system and A-2-4 by
the AASHTO system. The material contained 6 percent clay, 16 percent silt,
and 72 percent sand and had a specific gravity of 2.75.

INSTRUMENTATION

Instrumentation included Carlson earth stress cells, multipoint settlement gages, and horizontal slope indicators. A total of eight stress cells, four settlement gages, and four slope indicators had been planned for this site. Due to the distance involved both horizontally and vertically, slope indicators and settlement gages planned for placement in the lower part of the fill were impractical. Settlement Gages 2 and 3 (Figures 67 and 68) were installed, at approximately 95 and 135 feet, respectively, above the culvert. Horizontal Slope Indicators 3 and 4 were installed approximately 135 feet and 165 feet, respectively, above the culvert. All other settlement gages and slope indicators were omitted.

Due to the possibility of lanes being added in the future, earth stress cells were placed at two locations along the culvert. Four cells, one on the top of each conduit and one on the side wall of each conduit (Figure 65), were placed at the intersection of the culvert and roadway centerline (Figures 67 and 68). At the intersection of the culvert and centerline of the proposed additional lanes, four cells were placed in the same manner. All stress cell cables were placed in PVC water line and run through the fill to the inlet wingwall where a monitoring station was established.

All settlement gages and slope indicators functioned properly. Electrical cables on several stress cells were not of sufficient length to reach the monitoring station. They were spliced, and this probably contributed to the failure of two cells to function properly.

DATA

Flowline elevations were established on both conduits when
approximately 75 percent of the fill was in place. Settlement of the conduits from that time was a maximum of 0.124 foot in the south conduit and 0.233 foot in the north conduit. As shown in Figure 69, both conduits settled more near the center of the culvert.

Earth pressure data recorded at this site indicated an average $D_E$ of 0.934 with a range from 0.392 to 1.994 (Figure 70). Meters having the highest $D_E$, Number 98 with a $D_E$ of 1.994 and Number 99 with a $D_E$ of 0.928, were both on the south conduit. Greater settlement of the north conduit, Figure 69, probably relieved some of the pressure on that conduit, resulting in lower $D_E$ values. Average $D_E$ for cells located on top of the conduits was 1.220. Pressure data at this site are plotted versus time in Figure 71.

Settlement data at this site were limited to the upper portions of the fill. Settlement Gage 2 settled an average of 1.184 feet (Figure 72). Settlement Gage 3 settled an average of 0.722 feet (Figure 73). Points of greatest settlement for both gages were points located near the centerline of the roadway.

Horizontal Slope Indicator 3 settled a maximum of 0.848 foot (Figure 74). Maximum settlement occurred near the centerline of the roadway and relates closely to the maximum settlement of Gage 3 of 0.799 foot. Horizontal Slope Indicator 4, which was placed near the top of the fill, provided unusual data (Figure 74). The greater settlement near the ends of Horizontal Slope Indicator 4 is probably the result of sloughing of the embankment.
SUMMARY OF FIELD DATA

In reviewing data obtained at several sites, it becomes apparent that the imperfect trench does significantly influence loading of the culverts. In Figure 75, all $D_E$ values for an individual site are averaged to obtain the overall loading characteristic of that site. The average $D_E$ for a site is then plotted versus fill height. $D_E$ for sites employing the imperfect trench increase slightly as fill height decreases.

As shown in Figure 76, average $D_E$ for sites not employing the imperfect trench are consistently higher. At greater fill heights, average $D_E$ for the two situations approach the same values. At shallow fill heights, average $D_E$ for sites not employing the imperfect trench increase dramatically. In Figure 76, the data point located at a fill height of 50 feet represents the site at Station 448+90 in Marion County. The culvert was constructed on a soft foundation, which could explain the relatively low $D_E$ at that site.

Foundation conditions, and therefore structure settlement, also influence culvert loading but to a lesser degree. As evidenced by data obtained at the Clark County sites, a yielding foundation results in lower $D_E$ values. At another site without the imperfect trench, Laurel County, the average $D_E$ was relatively low and there was a significant amount of structure settlement; however, the points having the highest $D_E$ values were points located on the conduit, which had less settlement.

All sites utilizing the imperfect trench were instrumented with earth stress cells at the center of the culverts and at points on the culverts nearer the inlets. $D_E$ values at the points nearer the inlets would tend to be greater because of the two factors previously mentioned (less height of fill and less structure settlement). As shown in Table 3, $D_E$ values at these points verify the point.
FINITE ELEMENT ANALYSES

COMPARISON TO FIELD DATA

A finite element program entitled ISBILD, written by Ozawa and Duncan (7), was used in an attempt to predict loads on and settlements of the culverts. A finite element grid was constructed for each site to model the geometry of the site as accurately as possible. Soil parameters used in the analysis are listed in Table 4. Parameters listed in that table for original ground were obtained from triaxial tests on materials from the two Marion County sites. Those same parameters were used for the analyses of the McCreary and Clark County sites. It is recognized that some error would be introduced by this procedure; however, it was considered to be the best procedure because the triaxial data from the other sites were not as extensive as from the Marion County site. At all sites, the embankments were of "shot rock". The parameters for that material were obtained from a report by Wong and Duncan (8). All soil properties were obtained by converting stress-strain data from triaxial tests to hyperbolic parameters as reported by Wong and Duncan (8). The method for converting stress-strain data to hyperbolic parameters is detailed in that report and will not be repeated here. An analysis was not made on the Laurel County site due to problems encountered in modeling the twin barrels.

Pressure

Figure 77 shows the relationship between pressure as predicted from the finite element program and pressures measured in the field. The data points represent top slab and sidewall earth pressures for the sites listed in that figure. The general trend is good, although there is a slight "skew" in data, as indicated by the regression line. In general, the finite
element program (with the soil parameters used in this study) tended to slightly over predict pressures that were less than 50 pounds per square inch and under predict pressures greater than 50 pounds per square inch.

Part of this skew is due to the parameters assigned to the material in the imperfect trench. Table 4 shows that very low values of density and shear strength were assigned to the material. In actuality, those values were probably somewhat higher. Higher values would reduce the magnitude of settlement and increase the magnitude of pressure. That would cause some points that are greater than 50 pounds per square inch to fall closer to the line of equality.

Figure 78 compares measured pressures on the top slabs at all the sites (except Laurel County) with pressures predicted by Marston (1), Spangler (9), and Costes (10). Considerable scatter is noted in the data. Regression analysis indicates there also is a skew in the data.

Comparing Figures 77 and 78 indicates the finite element method more accurately predicts pressures on box culverts than do the three analytical methods. Also, the finite element method would appear to be more versatile in types of problems that may be analyzed.

Settlement

Figures 79 through 82 show measured settlement and predicted settlement for two culverts in McCreary County and two culverts in Marion County. The plots show settlement across the top of the box. It is readily apparent that the finite element program over predicted settlement on those culverts having an imperfect trench. Again, this is related to material parameters assumed for the material in the imperfect trench. The material in the trench was considerably stiffer than was assumed in the analysis.
ANALYSIS OF EFFECTIVE DENSITY

To determine the effects that geometry, culvert bedding, and soil parameters may have upon load magnitude, a finite element grid similar to the one shown in Figure 83 was used. Soil properties used in these analyses are the same as listed in Table 4.

The relative height of the top of the culvert to the surrounding natural ground is often referred to as projection ratio. Figure 84 illustrates three typical projection conditions. Figure 84(a) illustrates a positive projection condition. This occurs when the top of the culvert is above the natural ground. In Figure 84(b), the top of the culvert and the natural ground coincide. This is referred to as zero projection. Finally, in Figure 84(c), the natural ground is above the top of the culvert, and this is defined as negative projection. Each projection condition is discussed separately in the following analyses.

Positive Projection

In analyses of positive projection culverts, three fill heights, \( H \) (34 feet, 74 feet, and 114 feet), three box heights, \( H_c \) (4 feet, 8 feet, and 10 feet), and three box widths, \( B_c \) (5 feet, 8 feet, and 12 feet) were used. In those analyses, the foundation was assumed to be rigid (unyielding). All calculated pressures were converted to effective densities.

Figure 85 shows the relationship between fill height and the peak effective density, \( D_{EV} \), on the top slab of an 8-foot by 8-foot box culvert. \( D_{EV} \) ranges from 2.45 at a 1-foot fill height to 2.21 at a 200-foot fill height. The curve indicates that beyond approximately 80 feet, the fill height does not appreciably affect \( D_{EV} \). It appears this may be related to
the location of the plane of equal settlement (to be discussed more fully later).

Because the curve in Figure 85 was developed for an 8-foot by 8-foot box, it must be modified to accommodate other box sizes. Figure 86 shows the effect of box width, \( B_C \), on effective density. The normalized coefficient, \( C_B^P \), is arbitrarily set equal to 1.0 for an 8-foot box, and \( D_{EV} \) for any other box width may be calculated by multiplying \( D_{EV} \) from Figure 85 by \( C_B^P \) in Figure 86. It is clear from Figure 86 that effective density decreases as box width increases. The relationship between \( B_C \) and \( C_B^P \) may be described by the following equation:

\[
C_B^P = 1.30 - 0.038B_C + 0.0006B_C^2.
\]

In a similar manner, \( D_{EV} \) from Figure 85 must be modified for various box heights, \( H_C \). In Figure 87, \( C_{HC}^P \) is the normalized coefficient that describes the effect of box height on \( D_{EV} \). Again, the coefficient for a box height of 8 feet has been set equal to 1.0. From Figure 87, it may be noted that an increase in box height produces a corresponding increase in \( D_{EV} \). Coefficient \( C_{HC}^P \) varies as \( H_C \) in the following manner:

\[
C_{HC}^P = 0.84 + 0.02H_C.
\]

To estimate pressure on the top slab of a complete positive projecting culvert on an unyielding foundation (Figure 84(a)), obtain a value for \( D_{EV} \) from Figure 85 and use in the following equation:

\[
P_{CV}^P = (wH)(D_{EV})(C_B^P)(C_{HC}^P).
\]

The relationship between effective density on the sidewall, \( D_{EH} \), and fill height, \( H \), is shown in Figure 88 (8-foot by 8-foot box). As \( H \)
increases from zero to 60 feet, $D_{EH}$ decreases, but remains essentially the same as $H$ increases beyond 60 feet. Again, values of $D_{EH}$ in Figure 88 must be corrected for box width and box height. Figure 89 indicates that $D_{EH}$ decreases as $B_C$ increases according to the following relationship:

$$C_{SB}^P = 1.90 - 0.15B_C + 0.0044B_C^2,$$

where $C_{SB}^P$ = sidewall coefficient relating $D_{EH}$ to box width. From Figure 90, it may be noted that the coefficient $C_{SHC}^P$ relating box height, $H_C$, to $D_{EH}$ increases as $H_C$ increases in the following manner:

$$C_{SHC}^P = 0.53 + 0.058H_C.$$

As in the case of top slab pressure, the expected sidewall pressure for a positive projecting culvert on an unyielding foundation may now be calculated,

$$P_{CH} = (wH)(D_{EH})(C_{SB}^P)(C_{SHC}^P).$$

For complete positive projecting box culverts on rigid foundations, a wide and low cross section is most effective in keeping pressures low (for both slab and sidewall).

Zero Projection

For zero projecting culverts (Figure 84(b)), an additional variable becomes extremely important in determining loads on the box. This variable is the width of the side trench. Figure 91 illustrates the effect of trench width, $W_T$, on $D_{EV}$ for a hypothetical 1-foot fill. As in the case of positive projecting culverts, $D_{EV}$ decreases as fill height increases. Coefficient $C_{H}^0$ is the reduction factor that relates fill height to $D_{EV}$ (Figure 92). Again, fill height does not appreciably affect $D_{EV}$ above 80
feet. The curve in Figure 92 may be approximated by the following equation:

$$C_H^0 = 0.96 - 0.09(\log H).$$

As in the case of positive projection, box width and height also influence the magnitude of $D_{EV}$. Figure 93 illustrates the effect of box width on $D_{EV}$ for a zero projecting culvert (coefficient $C_B^0$). As box width increases to approximately 16 feet, $C_B^0$ decreases. For culverts wider than 16 feet, $C_B^0$ remains relatively constant. $C_B^0$ may be described by the following equation:

$$C_B^0 = 1.60 - 0.10B_C + 0.0031B_C^2.$$

As box height increases, $D_{EV}$ also increases. This is shown by coefficient $C_{HC}^0$ in Figure 94. A linear function approximates the relationship between $D_{EV}$ and $C_{HC}^0$:

$$C_{HC}^0 = 0.9 + 0.013H_C.$$  

Therefore, to estimate pressures on the top slab of zero projecting culverts on a rigid foundation, $D_{EV}$ is obtained from Figure 91, is multiplied by $H$, and then is modified by the proper coefficients:

$$P_{CV}^0 = (D_{EV})(WH)(C_H^0)(C_B^0)(C_{HC}^0).$$

Figure 95 shows the effect of $W_T$ on effective density, $D_{EH}$, for the sidewall of a zero projecting culvert. As in the case for the top slab, an increase in trench width produces a corresponding increase in effective density. In this particular case, $B_C$ had no significant effect on $D_{EH}$.  

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Figures 96 and 97 display the effects of fill height and box height on $D_{EH}$, respectively. $D_{EH}$ decreases as fill height increases as described by coefficient $C^0_{SH}$:

$$C^0_{SH} = 0.95 - 0.24(\log H).$$

Coefficient $C^0_{SHC}$ describes the decrease in $D_{EH}$ as box height increases:

$$C^0_{SHC} = 1.80 - 0.13H_G + 0.0043H_G^2.$$

Sidewall pressures for zero projecting culverts on rigid foundations may now be estimated according to the following:

$$P^0_{CH} = (D_{EH})(wH)(C^0_{SH})(C^0_{SHC}).$$

Summarizing, it is clear that narrow side trenches are very effective in reducing pressures on the top slab and sidewalls of zero projecting culverts. Low and wide box cross sections also reduce pressures on top slabs of zero projecting culverts. However, unlike positive projecting culverts, taller boxes (up to 10 feet) reduce sidewall pressures on zero projecting culverts, while box width appears to have no significant effect on sidewall pressures.

Negative Projection

For the case of negative projecting culverts, the height of the trench above the top of the culvert, $H_T$, is an additional variable that must be considered along with those discussed in the two previous cases. Figure 98 shows the variation in $D_{EV}$ with $H_T$ and $W_T$. The relationship between these variables is rather complex. It should be noted when $H_T$ equals 1 foot, the curve that represents the relationship between $D_{EV}$ and $W_T$ is very similar to the zero projection curve (the condition it most closely approximates).
However, as $H_T$ increases, it becomes very effective at reducing $D_{EV}$ for $W_T$ values of 5 feet or less. From $W_T$ from 6 feet to 12 feet, higher values of $H_T$ lose their effectiveness and produce $D_{EV}$ values that are greater than those for $H_T$ equal to 1 foot. At first glance, this would appear to be unusual. However, the reported values of $D_{EV}$ are peak values. Furthermore, the distribution of stress across the top slab is not uniform, and most of the time the highest stress is near the edge of the slab.

Figure 99 is an example illustrating the effect of $W_T$ on the distribution of stress across the top slab of a negative projecting culvert, with $H_T$ greater than 1 foot. From that figure, it is evident the stress distribution becomes more uniform as $W_T$ approaches 10 or 12 feet. If the average stress across the top slab were plotted in Figure 99, the curves representing $H_T$ values of 3, 5, and 9 would have a shape similar to the curve representing an $H_T$ value equal to 1 foot, but they would have lower $D_{EV}$ values. Therefore, from the analyses, deeper trenches appear to be effective in reducing the average stress on the top slab of a negative projecting culvert, but this does not hold true when considering peak stress.

As in the first two projection cases, the values of $D_{EV}$ in Figure 98 must be modified to account for the effects of fill height, box height, and box width. The coefficients for these variables are shown in Figures 100 through 102, respectively. Coefficient $C^N_H$ relating $D_{EV}$ to fill height is described by the following function:

$$C^N_H = 1.62 - 0.0087H + 0.000024H^2.$$ 

Unlike the first two projection cases, fill heights well beyond 80 feet continue to be a significant factor in the magnitude of $D_{EV}$. Also, for
negative projection, a wider box experiences smaller values of $D_{EV}$ (see Figure 102). Coefficient $C_B^N$ varies linearly with $B_C$ as follows:

$$C_B^N = 1.46 - 0.0583B_C.$$  

As in the two previous projection cases, $D_{EV}$ increases with box height. Coefficient $C_{HC}^N$ modifies the values of $D_{EV}$ in Figure 98 to account for $H_C$ (see Figure 101):

$$C_{HC}^N = 0.80 + 0.024H_C.$$  

The estimated pressure on the top slab of a negative projecting culvert on a rigid foundation now may be calculated:

$$P_{CV}^N = (D_{EV})(wH)(C_H^N)(C_B^N)(C_{HC}^N).$$  

Figure 103 illustrates the effects of side trench width, $W_T$, and trench depth, $H_T$, on $D_{EH}$. As in all previous cases, the values of $D_{EH}$ are modified for fill height (coefficient $C_{SH}^N$), box height (coefficient $C_{SB}^N$), and box width (coefficient $C_{SHC}^N$) using the following equations:

$$C_{SH}^N = 1.40 - 0.0052H + 0.0000148H^2,$$

$$C_{SB}^N = 1.16 - 0.021B_C,$$  

and

$$C_{SHC}^N = 1.15 - 0.00264H_C^2.$$  

These relationships are illustrated in Figures 104 through 106.

From the previous equations, the estimated sidewall pressure on a negative projecting culvert on a rigid foundation is calculated from the following equations:

$$P_{HC}^N = (D_{EH})(wH)(C_{SH}^N)(C_{SB}^N)(C_{SHC}^N).$$  

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Imperfect Trench

The effects of various widths of the imperfect trench were also analyzed. Figure 107 shows a box culvert with an imperfect trench whose width is $rB_c$, where $r$ is some ratio of $B_c$. In Figure 108, $D_{EV}$ is plotted as a function of $r$. It may be seen that increasing the width of the trench to a value twice the width of the box ($r = 2.0$) is very effective in reducing $D_{EV}$. However, as the width of the trench increases further, little or no further reduction in $D_{EV}$ occurs.

One additional case was analyzed with the imperfect trench modified as shown in Figure 109. The width of the trench was 1.5 times $B_c$. However, the loose material in the trench also was placed along side the top half of the culvert. The $D_{EV}$ on the top slab for this case was 0.24. This appears to be the most effective configuration for the imperfect trench.

Yielding Foundation

To determine the effects of a yielding cushion under the culvert, a 4-foot thick soil pad, under an 8-foot box under a 115-foot fill, was analyzed. The soil under the culvert was assumed to have the same properties as the soil in the fill above the culvert. The first pad analyzed was 12 feet in width. The maximum $D_{EV}$ on the top slab for this case was 1.83. The second pad analyzed was 16 feet wide, yielding a maximum $D_{EV}$ of 1.87 because $W_T$ was wider for that case. When these values are compared with the imperfect trench (assuming a trench 8 feet in width), it appears the imperfect trench is more effective in reducing $D_{EV}$ ($D_{EV} = 1.0$ from Figure 108). Without the imperfect trench or soil pad, the value of $D_{EV}$ was 2.23 for the 8-foot by 8-foot box.
Sloping Trench Walls

Spangler and Handy (11) have stated that the effective width of a trench with sloping sides (similar to that shown in Figure 110) is generally equal to the width of the trench at or slightly below the top of the culvert. Finite element analyses generally support that conclusion when considering the load on the top slab. However, the location of the maximum sidewall pressure appears to change. In a trench with vertical sides, the maximum sidewall pressure generally occurs somewhere in the top half of the wall. When the trench walls are sloping, the maximum sidewall pressure generally shifts to the lower portion of the wall.

Soil Properties

Marston (1) and Spangler and Handy (11) have indicated the angle of internal friction, $\phi$, is of minor importance when calculating loads on positive projecting culverts. An analysis was made of a positive projecting 8-foot by 8-foot box on a rigid foundation. When $\phi$ was varied from a high of 34 to a low of 20 degrees, the load on the top slab decreased by less than 5 percent. This appears to lend support to the statements of the previously mentioned authors.

ANALYSIS OF SETTLEMENT

Finite element analyses indicated the primary factor in determining the height of equal settlement, $H_E$, for positive projecting culverts on rigid foundations was fill height. Box height had a small and insignificant effect on $H_E$ (particularly within the range of box heights analyzed in this study). Figure 111 shows the effect of fill height on $H_E$ for a positive projecting culvert. At fill heights less than 35 feet, the height of equal
settlement becomes imaginary \( \frac{H_E}{H} > 1 \) and the complete projection condition prevails (11).

For negative and zero projecting culverts, trench width is also important in determining \( H_E \). Figure 112 shows the effects of trench width on \( H_E \) for negative projecting culverts. As trench width increases, the height of equal settlement progresses upward in the fill. Although not analyzed in this study, it is expected that this holds true only to some very wide trench width, at which point, any increases in trench width would result in corresponding decreases in \( H_E \) until positive projecting conditions were reached (infinitely wide trench).

**SUMMARY OF CONCLUSIONS**

Finite element analysis provides a reasonable approximation of loads on box culverts.

Effective density decreases as fill height increases to approximately 80 feet, when considering positive and zero projecting culverts. For negative projecting culverts, fill height continues to be an influence on effective density to a height of 200 feet or more.

Low and wide box cross sections experience lower values of effective density than cross sections that are tall and narrow.

For zero and negative projecting culverts, narrow side trench widths are very beneficial in reducing effective density. When constructing a negative projecting box, the benefits of the deep trench may be lost when the side trenches become wider than 4 or 5 feet.

The imperfect trench is effective in reducing effective density. An imperfect trench width of twice the box width appears to be the most
effective width, when considering a normally shaped trench. However, finite element analyses indicate an imperfect trench shaped similar to the one illustrated in Figure 109 is the most effective shape analyzed.

For the limited number of cases analyzed in this study, it appears the imperfect trench is more effective in reducing effective density than a compacted soil cushion placed under the barrel.

Trenches having sloping sides appear to behave as a trench with an equivalent width equal to the width of the trench at the top of the box (when estimating loads on the top slab). This lends support to statements by Marston (1).

Also, in support of Marston, the internal friction angle of the soil appears to have only a small influence on the loads on the top slab.

For positive projecting culverts, the plane of equal settlement is influenced by the height of fill and is not appreciably affected by the shape of the box. For zero and negative projecting culverts, the height of fill and width of side trench are the major factors affecting the location of the plane of equal settlement. A wide side trench will cause the plane of equal settlement to be located higher in the fill, until the trench becomes sufficiently wide for the conduit to act as a positive projecting culvert. All of the statements in this paragraph concerning settlement apply to rigid foundations.

RECOMMENDATIONS

The AASHTO Standard Specifications for Highway Bridges state in Section 1.2.2(A) that culverts untrenched or on unyielding (rigid) foundations require a special analysis to estimate loads. It is noted that finite
element analyses presented in this report more accurately predicted loads than the theoretical methods. Therefore, the charts in this report could be used to estimate both vertical and horizontal loads for boxes on unyielding foundations. An example of how the charts may be applied is shown in the Appendix.

It is recommended that the estimated peak stress be used when designing the top slab, instead of assuming a uniform distribution of load. This recommendation is based on data illustrated in Figure 99. (As stated previously, the charts in this report are based upon peak stress.)

The Kentucky Department of Highways Standard Drawing No. RDI-020-03 (Pipe Bedding for Sewers, Storm Drains, and Their Combinations) and Standard Drawing No. RDI-120 (Bedding for Precast Box Culverts, Sewers, Storm Drains, and Their Combinations) indicate the original groundline should be uniform for a distance of $2B_C$ or 12 feet (use lesser) on both sides of the culvert. This was done in an attempt to attain uniform loads on both sides of the culvert. From field data and finite element analyses, it is recommended that (as a minimum) this standard be retained, and consideration be given to possibly increasing this distance to as much as 20 feet. This recommendation is based upon the shape of the curve in Figure 91 where $D_{EV}$ is still gradually increasing beyond 12 feet. However, it appears the curve may become reasonably "flat" in the range of 16 to 20 feet.

The same two standard drawings have a side trench width requirement of at least 12 inches, but not more than 15 inches. This is an extremely important criterion and should be retained. In addition, it should be rigidly enforced, as much as possible, during construction. This helps to induce smaller loads on the conduit and also keeps the height of the plane.
of equal settlement lower in the fill.

In view of results displayed in Figure 108, it is recommended that a culvert to be constructed under a high fill have an imperfect trench installed that is 1.5 or 2.0 times wider than the culvert. This would help verify or disprove the usefulness of a wider imperfect trench. Also, a culvert with an imperfect trench shaped as illustrated in Figure 109 should be constructed to determine the effectiveness of the modified shape.

It also is recommended that bedding conditions for a culvert be uniform throughout the length of the culvert. In other words, the box should not be placed partially on a rigid foundation and partially on a yielding foundation. This will produce longitudinal stresses in the box for which it was not designed.

It is suggested that the finite element program ISBILD could be implemented for specific sites to obtain loads and settlements when unusual geometry or soil conditions exist.

IMPLEMENTATION

The results of this study have been implemented through changes in the design standards of the Guidance Manual of the Division of Bridges of the Kentucky Department of Highways. These changes are shown in Appendix B.

REFERENCES


2. Pomerene, J. B.; Geologic Map of the Whitley City Quadrangle, GQ-260, Department of the Interior, United States Geological Survey in Cooperation
with the Kentucky Geological Survey, 1964.


TABLE 1. SUMMARY OF SOIL TEST DATA (McCREARY COUNTY)

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TABLE 3. COMPARISON OF AVERAGE $D_E$ FOR SITES WITH $D_E$ VALUES NEAR INLET

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<th>D_E @ POINTS</th>
<th>AVERAGE D_E</th>
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<tr>
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<td>FUNCTION *</td>
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<td>-------------------------------------------</td>
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<tr>
<td>K, K&lt;sub&gt;ur&lt;/sub&gt;</td>
<td>Modulus number</td>
<td>Relates $E_i$ and $E_{ur}$ to $\sigma_3$</td>
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<td>c</td>
<td>Cohesion intercept (psf)</td>
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<tr>
<td>$\phi$</td>
<td>Friction angle (degrees)</td>
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<td>$R_f$</td>
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<tr>
<td>G</td>
<td>Poisson's ratio parameter</td>
<td>Value of $v_1$ at $\sigma_3 = P_a$</td>
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<td>Decrease in $v_1$ for ten-fold increase in $\sigma_3$</td>
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<td>Rate of increase of $v_1$ with strain</td>
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<tr>
<td>w</td>
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* $E_i$ = initial elastic modulus

$E_{ur}$ = hysteresis modulus

$(\sigma_1 - \sigma_3)_f$ = deviator stress at failure

$(\sigma_1 - \sigma_3)_{ult}$ = ultimate deviator stress

$P_a$ = atmospheric pressure

$v_1$ = initial Poisson's ratio

$v_t$ = tangent Poisson's ratio
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NOTE: HSI 2 DOES NOT READ IN THIS AREA

- Pressure Meters
- Monitoring Points
  (Settlement Gage 2)

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APPENDIX A

EXAMPLE PROBLEM
The following is an example of the use of the charts in this report to estimate pressures on the top slab and sidewall of a positive projecting culvert.

GIVEN: Culvert Width = 10 feet
Culvert Height = 6 feet
Fill Height = 50 feet
Rigid Foundation
Fully Positive Projecting

FIND: Pressure on Top Slab
Pressure on Sidewall

(1) Effective density on an 8-foot by 8-foot box under a 50-foot fill is 2.26. This was determined from Figure A1.

(2) This value must be corrected for box width. From Figure A2, coefficient $C_B^P$ for a 10-foot-wide box is 0.92.

(3) Also, the $D_{EV}$ value from Figure A1 must be corrected for box height. From Figure A3, coefficient $C_{HC}^P$ for a 6-foot-tall box is 0.95.

(4) To calculate the corrected $D_{EV}$ on the top slab, multiply the $D_{EV}$ from Figure A1 by 0.92 (Figure A2) and by 0.95 (Figure A3):

$$D_{EV} = 2.26$$
$$C_B^P = 0.92$$
$$C_{HC}^P = 0.95$$
and $(2.26)(0.92)(0.95) = 1.98$ (corrected $D_{EV}$).
(5) Estimated top slab pressure equals:

\[(\text{corrected } D_{EV})(\text{assumed unit weight})(\text{fill height})\]
\[\text{or } (1.98)(120 \text{ lb./cu.ft.})(50 \text{ ft.}) = 11,880 \text{ lb./sq.ft.}\]

(6) For estimated sidewall pressure, $D_{EH}$ from Figure A4 equals 0.47.

(7) Correcting the value of $D_{EH}$ from Figure A4 for box width yields a value of 0.85 for coefficient $C_{SB}^P$, from Figure A5.

(8) Also, correcting $D_{EH}$ for box height yields a value of 0.86 for coefficient $C_{SHC}^P$ from Figure A6.

(9) The corrected $D_{EH}$ on the sidewall is now calculated:

\[D_{EH} = 0.47 \text{ (Figure A4)}\]
\[C_{SB}^P = 0.85 \text{ (Figure A5)}\]
\[C_{SHC}^P = 0.86 \text{ (Figure A6)}\]
\[\text{and } (0.47)(0.85)(0.86) = 0.34 \text{ (corrected } D_{EH})\]

(10) Estimated sidewall pressure equals:

\[(\text{corrected } D_{EH})(\text{assumed unit weight})(\text{fill height})\]
\[\text{or } (0.34)(120 \text{ lb./cu.ft.})(50 \text{ ft.}) = 2,040 \text{ lb./sq.ft.}\]
Figure A1

Fill Height, H (Feet)

Figure A2

Box Width, Bc (Feet)

Figure A3

Box Height, Hc (Feet)
APPENDIX B

IMPLEMENTATION
CHAPTER 66-04

SECTION 6 - CULVERTS

6.2 DEAD LOADS

6.2.1 Culvert in trench or culvert untrencched on yielding foundation.

B. REINFORCED CONCRETE BOX CULVERTS

The following formula is recommended for computing the unit dead load fill pressure for culvert on yielding foundation only.

\[ P = WH \]

Where \( P \) = Unit Pressure in pounds per square foot due to earth backfill
\( W \) = 120 pounds per cubic foot
\( H \) = Height of fill over culvert

The same loadings outlined in Guidance Manual Article 66-04.6.2.2 may be used for culverts on unyielding foundations, in a trench with zero projection, except that the value of \( W_1 \) may be reduced to .75\( W_1 \).

The same loadings outlined in Guidance Manual Article 66-04.6.2.2 may be used for culverts on unyielding foundations, in a trench with negative projection, except that the value of \( W_1 \) may be reduced to .67\( W_1 \).

In either condition, the moments and shears calculated shall not be less than those calculated when using a uniform load of 120 lbs. per cubic feet distributed over design length "L".

Refer to Exhibit 66-41 for examples of culverts in a trench, zero projection and positive projection.

6.2.2 CULVERT UNTRENCHED ON UNYIELDING FOUNDATION

When a R.C. Box Culvert is rigidly supported on rock and is untrencched it shall be designed according to the following parameters:

A uniformly distributed load of 84 lbs. per cubic feet shall be distributed over the design span length "L".

Design span length "L" is the distance center to center of exterior sidewalls, refer to Exhibit 66-39. This load shall be supplemented by another uniformly distributed load \( W_1 \) lbs. per cubic feet extending a distance equal to .55+.25L2. Refer to Exhibit 66-39. The value of \( W_1 \) is equal to (120xK_1xK_2xK_3)-84 lbs. per cubic feet. The values of \( K_1 \), \( K_2 \) and \( K_3 \) shall be interpolated from the graphs shown on Guidance Manual Exhibit 66-40.
CHAPTER 66-04

SECTION 6 - CULVERTS (Cont.)

6.2.2 The moments and shears calculated shall not be less than those calculated when using a uniform load of 120 lbs. per cubic feet distributed over design length "L".

The loads shown above are based on Research Report UKTRP-84-22.
DESIGN CRITERIA FOR REINFORCED BOX CULVERTS

CULVERT UNTRENCHED ON UNYIELDING FOUNDATION
When a R.C. Box Culvert is rigidly supported on rock and is untrenched it shall be designed according to the following parameters:

A load \( P_1 \), equal to 84 lbs. per cubic feet \( x H \), in pounds per square feet shall be distributed over design span \( L_1 \). This load \( P_1 \) shall be supplemented by 2 additional loads \( P_2 \). The value of \( P_2 \), in pounds per square feet, is equal to \( \left[ \frac{(120 \text{ lbs. per cubic feet} \times K_1 \times K_2 \times K_3) - 84 \text{ lbs. per cubic feet}}{H} \right] \). Refer to Exhibit 66-39 for locations of load \( P_2 \) and design span \( L_1 \). \( H \) is equal to fill height over culvert. The values of \( K_1 \), \( K_2 \) and \( K_3 \) shall be interpolated from graphs on Exhibit 66-40.

Shear shall be checked at the distance \( \frac{1}{12} \) the clear width of culvert, or \( "d" \) depth of the slab, measured from the inner face of the vertical wall, whichever is closer to the face of the wall.

The moments and shears calculated shall not be less than those calculated when using a uniform load of \( (120 \text{ lbs. per cubic feet} \times H) \) distributed over design length \( "L_1" \).

IMPERFECT TRENCH METHOD
For culverts rigidly supported on rock, with fills greater than 60', the imperfect trench method of construction should be considered.

When the imperfect trench method of construction is utilized, the loads applied to the culvert on rock are the same as the load applied to culverts on yielding foundation.
On multiple barrel culverts: $L_1$: Distance center to center of exterior walls.
$L_2$: Distance from inside of exterior wall to inside of exterior wall.
2.1

2.0

1.4

1.2

1.0

0.1

0.1

0.0

TOP SLAB COEFFICIENT - $K_2$

K as a Function of Fill Height.

TOP SLAB COEFFICIENT - $K_3$

Coefficient $K_3$ as a Function of Box Height.
EXAMPLES OF THREE TYPES OF CULVERT PROJECTIONS

EXHIBIT 06-61

[Diagram showing three types of culvert projections: Positive, Zero, and Negative]

* If W ≤ exceeds 6° the culvert shall be considered to be in a positive projection.

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