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January 26, 1978

MEMORANDUM TO: G. F. Kemper
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SUBJECT: Research Report 491; "Loads on Box Culverts under High Embankments: Analysis and Design Considerations;" KYHPR-72-68; HPR-PL-1(13), Part II.

Earth pressures on buried structures are understood best in terms of so-called "first principles," which is to say, the simplest and most rational statement of the logic. The dead weight of earth material bearing on a structure is WH, where W is the unit weight or density and H is the depth or height of the overlying material. The first approximation of this pressure is this total weight divided by the area of the structure on which it bears. There usually are affecting factors, often very complex, which may increase or lessen the pressure. On the one hand, the structure may have to support more than its due share of the overburden; and, on the other hand, the pressure may be less than WH.

If a culvert were set on jacks, firmly seated, and an embankment constructed over it, and if the culvert were jacked upward slightly, the resistance would far exceed WH. If, in the same situation, the jacks were lowered slightly, the pressure might be greatly reduced and, for an indefinite time, may be made to approach zero. The latter example illustrates the role of a compressible layer or cushion over or under a culvert during settlement of an embankment. If a culvert having no cushion settles as much as the embankment, the bearing pressure will neither increase nor decrease. However, if the culvert is perched on an unyielding foundation and the embankment at the sides rests on yielding soil, the pressures surely will exceed WH.

If the bedding and foundation vary along the axis of the culvert, pressures will vary because of bridging over soft sections in the foundation and stiffness of the culvert -- that is, if the culvert is not jointed at relatively close intervals. Ideally, box culverts should be segmented (jointed) at relatively close intervals. The grade of the culvert then should be cambered in construction to a height above the straight-grade line equal to any estimated settlement. Jointing will minimize the severe, and very complex, bridging actions arising from differential settlements along the axis and will, in effect, re-simplify the design-load problem. Rationale and criteria for estimating D-loads for RC pipe culverts (circular or oval) could apply as well to box sections.

Two additional sites have been chosen for instrumentation and monitoring. Both sites (adjacent) are in Marion County and near the Taylor County line; they will be on KY 55, the Campbellsville-Lebenon

Road (00F-534-007, SP-078-0292-0001L and SP-109-0028-0006L). There, the foundation soil ranges from about 6 to 20 feet, and the fill heights will be in the order of 50 and 70 feet. An imperfect trench will be constructed over the culvert under the higher embankment. Bending along the axes is expected.

Respectfully submitted,

Jas. H. Havens

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Director of Research

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Enclosure
cc's: Research Committee

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16. Abstract

Pressure and design analyses of five reinforced concrete box culverts are presented. Three of the culverts were constructed with the imperfect trench. Measured pressures on the culverts are compared to pressures predicted by the theories of Marston, Spangler, and Costes. Also, comparisons are made between measured pressures and calculated design pressures using AASHTO's **Standard Specifications for Highway Bridges**, 1.2.2(A). It is noted that the imperfect trench is, apparently, effective in reducing pressures on the top slab. The AASHTO design formulas underestimated the pressure on the top slab of the culverts without the imperfect trench.

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LOADS ON BOX CULVERTS UNDER HIGH EMBANKMENTS: ANALYSIS AND DESIGN CONSIDERATIONS

KYHPR 72-68; HPR-PL-1(13), Part II
Interim Report

by

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in cooperation with
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INTRODUCTION

This report is the third interim report concerning loads on box culverts under high embankments. The first report, in April 1974 *(1)*, described locations, instrumentation, and data collection at three culvert sites in McCreary County. The second report, issued in August of 1975 *(2)*, described site locations and instrumentation associated with two culverts located in Clark County. The purpose of this report is to explicate pertinent analyses of the data.

DETERMINATION OF VERTICAL LOADS ON CULVERTS

There currently is little certainty as to the validity, origin, and accuracy of the current design equations as outlined in **Standard Specifications for Highway Bridges**, 1.2.2(A). Therefore, to check the accuracy of the formulas and to see how well they predicted the load relative to other theories, a comparison was made of the pressures predicted by Marston's, Spangler's, and Costes' formulas. Theories were then compared to measured pressures (Table 1).

Marston's Procedure

Clark County -- Marston's formula *(4)* for determining the vertical load, W_c, in a positive projecting condition is

$$W_c = C_c \gamma B_c^2, \qquad 1$$

where C_c = $\{\exp[\pm 2 K\mu (H/B_c)] - 1\} / \pm 2 K\mu$,

B_c = width of conduit (feet),

μ = Poisson's ratio,

H = height of embankment measured from the top of the conduit (feet),

K = $\tan^2(45° - \phi/2) - 2c/\gamma z\,[\tan(45° - \phi/2)]$ for active earth pressures or

= $\tan^2(45° + \phi/2) - 2c/\gamma z\,[\tan(45° + \phi/2)]$ for passive earth pressures,

ϕ = internal friction angle,

c = cohesion (psf),

z = depth (feet), and

γ = unit weight of soil over the conduit (pcf).

The plus signs are used for the complete projection condition, and the minus signs are used for the complete ditch conditions. For the incomplete trench condition,

$$C_c = \{\exp[\pm 2 K\mu (H_e/B_c)] - 1\} / \pm 2 K\mu + (H/B_c - H_e/B_c) \exp[\pm 2 K\mu (H_e/B_c)], \qquad 2$$

where H_e = height from the top of the conduit to the plane of equal settlement (feet).

The plus signs are used for the incomplete projection condition, and the minus signs are used for the incomplete ditch condition.

Marston's formula for evaluating H_e was derived by equating an expression for the sum of the total strain in the interior prism plus the settlement of the top of the conduit to a similar expression for the sum of the total strain in an exterior prism plus the settlement of the critical plane. The formula is

$$\{1/2\,K\mu \pm [(H/B_c - H_e/B_c] \pm r_{sd}\,P/3]\} \{\exp[\pm 2 K\mu (H_e/B_c)] - 1\} /\pm 2 K\mu \pm (H_e/B_c)^2/2 \pm r_{sd}\,P/3\,[H/B_c]\exp[\pm 2 K\mu (H_e/B_c)] - (1/2K\mu)(H_e/B_c) \pm (H/B_c)(H_e/B_c) = r_{sd}\,P\,H/B_c. \qquad 3$$

where P = unit pressure due to earth backfill (psf),

r_{sd} = settlement ratio = $[S_g - (S_d + d_c + S_f)]/S_d$,

S_g = settlement of original ground (feet),

S_d = settlement of loose material in the ditch above the culvert (feet),

S_f = settlement of the culvert foundation (feet), and

d_c = vertical deflection of the culvert.

The pressure calculated using Marston's method was 10 psi (68.9 kPa) less than the measured pressure at Station 123 + 95. The vertical pressure obtained from Marston's method for the culvert at Station 268 + 30 was 17.9 psi (123.3 kPa) less than the measured pressure.

TABLE 1. COMPARISON OF MEASURED PRESSURES ON THE TOP SLAB WITH COSTES', MARSTON'S, AND SPANGLER'S THEORIES

LOCATION	CURRENT DESIGN PROCEDURES psi (kPa)	MARSTON psi (kPa)	SPANGLER psi (kPa)	COSTES psi (kPa)	PEAK MEASURED PRESSURES[a] psi (kPa)
Sta 123 + 95 Clark County	44.9 (309)	100.0 (689)	NA	118.6 (817)	110.0 (758)
Sta 268 + 30 Clark County	38.3 (264)	44.5 (307)	NA	61.4 (423)	62.4 (430)
Sta 89 + 20 McCreary County	48.7 (335)	NA	26.7 (184)	20.1 (138)	37.5 (258)
Sta 203 + 20 McCreary County	77.3 (533)	NA	37.2 (256)	43.5 (300)	106.0[b] (730)
Sta 210 + 50 McCreary County	103.4 (712)	NA	51.4 (354)	70.7 (487)	48.0 (331)

[a]Readings on 4-2-75

[b]Data obtained from Carlson pressure cells not reliable

Spangler's Procedure

Clark County -- The two culverts located in Clark County were constructed in excavated trenches with the tops of the culverts projecting slightly above the natural ground. This condition would not fit into any of Spangler's cases. The culverts are actually a combination of two classes listed by Spangler, namely a trenched and a positive projecting condition. Therefore, Spangler's procedure *(4)* is not applicable to the culverts in Clark County.

McCreary County -- At Stations 89 + 20 and 210 + 50, Spangler's procedure for imperfect trench conditions overestimated the measured pressure by 1.9 psi (13.1 kPa) and 30.3 psi (208.8 kPa), respectfully. The measured pressure was underestimated at Station 203 + 20 by 50.1 psi (345.2 kPa) (see Table 1).

Costes' Procedure

McCreary County -- Costes' method *(3)* for determining vertical loads on conduits addresses two cases: Case I, the interior soil prism subsides less than the adjacent masses; and Case II, the interior soil prism subsides more than the adjacent masses. The three culverts in McCreary County fit Costes' second case. Those culverts were designed using the imperfect trench method. The following is Costes' equation for determining the vertical load, W_c, due to overburden material for Case II:

$$W_c = (\gamma B_d^2/2\,K_e \tan\phi_e)\{\exp[-2\,K_e \tan\phi_e)\,(H_e/B_d)]\ [2\,K_e \tan\phi_e\,(H - H_e)/B_d - (1 - 2\,C_e/\gamma B_d)] + (1 - 2\,C_e/\gamma B_d)\} \quad 4$$

where B_d = effective width of the interior prism (feet),

γ = unit weight of the material on top of the conduit (pcf),

ϕ_e = portion of the angle of internal friction of the material that is mobilized along the potential sliding planes,

C_e = portion of the cohesion of the material that is mobilized along the potential sliding planes (psf),

σ_v = vertical principal stress acting on an element of the material along the sliding planes at a distance z from the plane of equal settlement (psf),

σ_h = horizontal principal stress acting on an element of the material along the sliding planes at a distance z below the plane of equal settlement (psf), and

K_e = σ_h/σ_v = equivalent hydrostatic pressure ratio along the sliding planes.

Assuming the soil is cohesionless, C_e is zero, resulting in the following equation:

$$W_c = (\gamma B_d^2/2\ K_e \tan \phi_e) \left\{[\exp(-W' - U')] [V' - U' - 1] + 1\right\} \quad 5$$

where
$W' = (2\ K_e \tan \phi_e)(H_d/B_d)$,
$U' = (2\ K_e \tan \phi_e)(H'_e/B_d)$,
$V' = (2\ K_e \tan \phi_e)(H'/B_d)$,
H_d = height of top of ditch or imperfect trench above the top of the conduit (feet),
$H' = H - H_d$ = height of fill above the top of the imperfect trench (feet), and
$H_e' = H_e - H_d$ = height of the plane of equal settlement above the top of the imperfect trench (feet).

The values W' and V' were determined from the above equations, wherein K_e and ϕ_e were estimated. K_e was assumed to be equal to the value of K_o for loose sand, 0.5. ϕ_e was then determined by equating $(1 - \sin \phi_e)/(1 + \sin \phi_e)$ to 0.5 and solving for ϕ_e. The value U' was obtained by entering a chart with the calculated value of V' and r_{sd} W'/a'; a' is the ratio of the modulus of deformation of the loose material in the imperfect trench within the distance H_d, denoted by E_L, to the modulus of deformation of the remainder of the fill E_f. The value E_f was assumed to equal the tangent modulus obtained from a triaxial, stress-strain curve. The value E_L was assumed to be equal to half the value of E_f.

As shown in Table 1, Costes' pressures differed from measured pressures by as much as 62.5 psi (430.6 kPa) and as little as 17.4 psi (119.9 kPa).

Clark County -- Case I of Costes' procedure gives the vertical load on a conduit when the interior prism subsides less than the exterior prisms. Both culverts in Clark County are positive projecting. This assumption is supported by high pressures on the top and bottom slabs of the culverts located at Stations 123 + 95 and 268 + 30. Consequently, Case I in Costes' procedure was used to determine the vertical load on the top slabs.

Costes' equation for determining the vertical load on top of a positive-projecting conduit due to overburden material is

$$W_c = \gamma B_d^2\ C_n/2\ K_e \tan \phi_e \quad 6$$

where $C_n = \exp[(2\ K_e \tan \phi_e)(H_e/B_d)] [(2\ K_e \tan \phi_e)(H - H_e)/B_d + (1 + 2C_e/\gamma B_d)] - (1 + 2\ C_e/\gamma B_d)$. 7

In calculating the vertical load on the culverts in Clark County, C_e was assumed to be zero after a long period of time. Thus, C_n becomes

$$C_n = \exp[(2\ K_e \tan \phi_e)(H_e/B_d)] [(2\ K_e \tan \phi_e)(H - H_e)/B_d + 1] - 1. \quad 8$$

Costes' method predicted a pressure 8.6 psi (59.3 kPa) greater than the measured pressure at Station 123 + 95 and a pressure of 1.0 psi (6.89 kPa) less than the measured pressure at Station 268 + 30 (see Table 1).

Design Pressure, Using Current Procedures -- The current method of determining the vertical load on the top slab of culverts is outlined in AASHTO's **Standard Specifications for Highway Bridges**, 1.2.2(A). For a structure on a yielding foundation, the weight on the top slab is calculated by

$$P = WH, \quad 9$$

where W is taken as 70 percent of the unit weight of the soil. The pressure on the top slab of an unyielding foundation is calculated from

$$P = W(1.92\ H - 087\ B) \text{ for } H > 1.7\ B \quad 10$$

or

$$P = 2.59\ BW(e^k - 1), \text{ for } H < 1.7\ B, \quad 11$$

where
P = unit pressure due to earth backfill (psf),
B = width of trench, or in case there is no trench, the overall width of the culvert (feet),
H = depth of fill over culvert (feet),
W = effective unit weight of fill material (pcf), and
k = 0.385 H/B.

The vertical pressures on culverts were calculated for each of the five sites using the appropriate equation. The culvert located at Station 123 + 95 was the only structure designed as having a yielding foundation. The remaining four were designed assuming they would have unyielding foundations. As shown in Table 1, the design pressure for Station 123 + 95 was 44.9 psi (309.4 kPa), which was 65.1 psi (448.5 kPa) less than the measured pressure of 110 psi (757.9 kPa). Likewise, a design pressure of 38.3 psi (263.9 kPa) at Station 268 + 30 was 24.1 (166.0 kPa) less than the measured pressure of 62.4 psi (429.9 kPa).

Conversely, pressure readings at the culverts in McCreary County were generally lower than the design pressures. The measured pressures were 11.2 psi (77.2 kPa) and 55.4 psi (381.7 kPa) lower than the design pressure at Stations 89 + 20 and 210 + 50, respectively. The measured pressure was greater than the design pressure at Station 203 + 20; however, the pressure readings at that location were not reliable inasmuch as the pressures exceeded the rated capacity of the pressure cells.

The culverts in both Clark and McCreary Counties were designed using working stress design. The culverts in Clark County were constructed on solid rock foundations rendering it necessary to use Equation 11 to calculate the vertical pressure on the top slab. The culvert at Station 123 + 95 was designed using Equation 10 for a yielding foundation. The three culverts in McCreary County were designed using the imperfect trench method. Equation 10 was used in calculating the vertical design load for each of those three culverts.

CULVERT SURVEYS

An attempt was made to ascertain whether the imperfect trench (B_1) method of design produced a noticeable dip in the roadway surface. This endeavor entailed investigating concrete pipe culverts in the immediate surrounding area. A report by Hughes in 1965 *(5)* listed culvert station numbers and the associated project number, county, diameter, and bedding conditions. Culverts selected had been installed a sufficient time to allow settlement to occur.

The study involved 25 reinforced concrete pipe culverts having the imperfect trench and 10 reinforced concrete pipe culverts with standard bedding. The culverts were primarily Class III structures. The culverts were located in Franklin, Grant, and Shelby Counties. The criterion used to ascertain whether a significant dip existed in the roadway was to determine if a settlement of 0.2 foot (0.06 m) had occurred in a 50-foot (16.2-m) section of roadway above each culvert. This criterion was established by considering the maximum embankment height and maximum culvert diameter encountered in the field. The maximum embankment height in the survey was 53 feet (17.2 m). In determining that the dip should occur in a 50-foot (16.2-m) section, a maximum embankment height of 70 feet (22.7 m) was assumed. The maximum culvert diameter was 6 feet (1.9 m). Using the maximum values, failure lines were drawn first for the active arching condition (Figure 1) *(6)*. Drawing failure lines tangent to a line 45° + $\phi/2$ from a horizontal line, located at the top of the culvert, up to the plane of equal settlement and vertically thereafter, a section 17 feet (5.5 m) long would be affected.

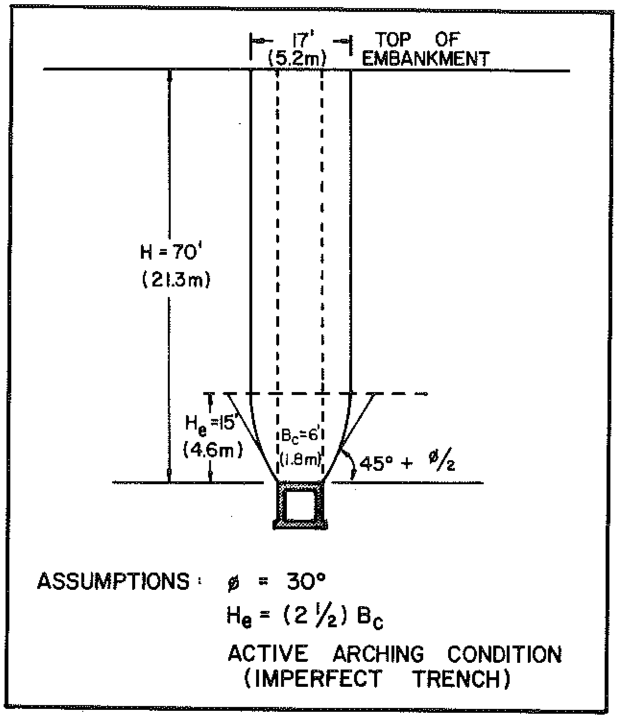

Figure 1. Determination of the Roadway Section Affected by Settlement above a Conduit, Considering the Active Arching Condition.

The determining case was the passive arching condition (Figure 2) because the failure lines are tangent to lines 45° - $\phi/2$ from the horizontal to the elevation of the plane of equal settlement and vertically thereafter. A 40-foot (13.0-m) section would be affected in this case. The value of 50 feet (16.2 m) was chosen in order that a dip would be more discernible. A settlement of 0.2 foot (0.06 m) was an arbitrary value.

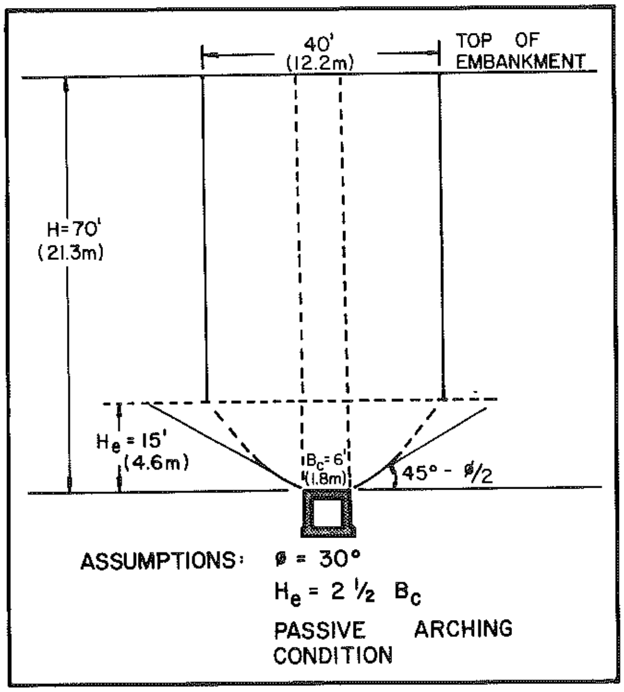

Figure 2. Determination of the Roadway Section Affected by Settlement above a Conduit, Considering the Passive Arching Condition.

As Table 2 illustrates, only one of 25 culverts with the imperfect trench surveyed had a significant dip in the roadway. Likewise, only one of the 10 having standard bedding had a significant dip. No general trend was observed in either the imperfect trench or standard bedding as being associated with roadway settlement above culverts.

Two factors which would affect the measured dip are overlays on the roadway directly above the culvert and whether the roadway above the culvert in question was in a vertical curve. The first factor was not taken into consideration in the analysis as there was no way of determining the overlays for a particular section of roadway. The second factor was recognized in the analysis and compensation for the roadway being in a vertical curve was made where appropriate.

TABLE 2. PIPE CULVERT SURVEY RESULTS INDICATING WHETHER A SIGNIFICANT DIP OCCURRED OVER THE VARIOUS CONDUITS

STATION	COUNTY	SIGNIFICANT DIP
Imperfect Trench		
1458 + 35L	Shelby	No
1536 + 51R	Shelby	No
1552 + 10R	Shelby	No
1604 + 04R	Shelby	No
1633 + 30L	Shelby	No
1635 + 82R	Shelby	No
1637 + 32L	Shelby	No
1653 + 30L	Shelby	No
2233 + 50R	Franklin	No
566 + 65	Grant	No
902 + 60	Grant	No
902 + 60	Grant	No
963 + 26	Grant	No
963 + 26	Grant	No
978 + 12(SB)	Grant	No
978 + 12(NB)	Grant	No
988 + 18(SB)	Grant	Yes
988 + 18(NB)	Grant	No
1085 + 44	Grant	No
1085 + 44	Grant	No
1087 + 50	Grant	No
1146 + 04(SB)	Grant	No
1146 + 04(NB)	Grant	No
794 + 60	Grant	No
794 + 60	Grant	No
Standard Bedding		
1255 + 25 WB	Shelby	No
1255 + 25 EB	Shelby	No
1403 + 10 WB	Shelby	No
1403 + 10 EB	Shelby	Yes
1456 + 90R EB	Shelby	No
1552 + 10R EB	Shelby	No
1653 + 30L WB	Shelby	No
2059 + 00R EB	Franklin	No
2060 + 85L WB	Franklin	No
2154 + 30L WB	Franklin	No

PHOTOELASTICITY

A photoelastic culvert model was constructed to simulate a positive projecting box culvert under a high fill. The lucite model was 2 feet (0.61 m) wide by 1 foot (0.31 m) high by 1 inch (25 mm) thick. A rectangular section 2 inches (50 mm) wide and 1 inch (25 mm) in depth was cut in the center of the bottom portion of the model so that a wooden block 1 1/2 inches (38 mm) high could be pressed gradually up into a gelatin material. The block protruding into the gelatin would, in effect, represent a case where the exterior soil prisms settle more than the interior prism for a positive-projecting box culvert.

The culvert model containing a gelatin solution proportioned approximately three to four percent gelatin by weight, was viewed with a polariscope. Photographs were taken using a polarizing lens (Figure 3). The colored lines in the photograph are isochromatic lines or fringes. Isochromatic fringes represent the loci of points of equal, relative retardation *(7)*. Polarized light, upon passing through a stressed transparent material, is split into two components which vibrate at right angles to one another *(8)*. One of these planes will coincide with the plane containing the greatest tensile stress, and the other plane must coincide with the plane containing the minimum stress. Light travels at slightly different speeds in the two planes; consequently, the two-component rays become out of phase. Relative retardation is the lag of one-component ray behind the other one.

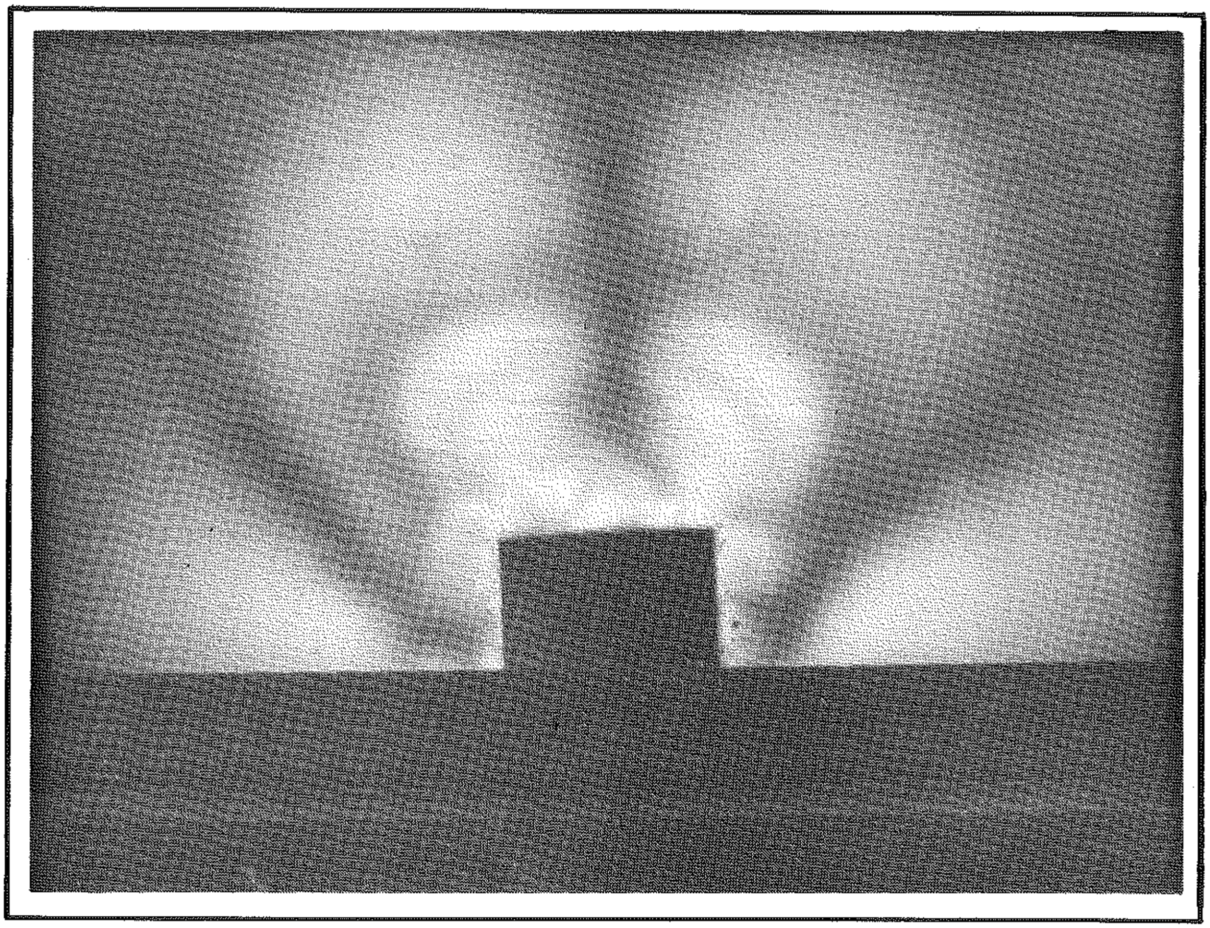

Figure 3. Photoelastic Modeling of Arching and Stress Bulbs above Culvert Model.

The isochromatic lines also indicate the difference in the principal stresses since the following relationship is true for the relative retardation and principal stresses:

$$R = Ct(P - Q) \qquad 12$$

where R = relative retardation in inches between the two phases from the model,
C = a constant of the material, termed the stress-optical ($in.^2/lb$),
t = thickness of model (in.), and
P and Q = the two principal stresses acting at right angles to the direction of propagation of the light (psi).

The isochromatic fringes seen in Figure 3 also represent lines of equal shear stress. The difference in the principal stresses equals twice the maximum shear stress (maximum shear stress = (P - Q)/2).

The concentric, isochromatic fringes or lines above the culvert model in Figure 3 indicate that, in an elastic material, arching does occur; and, for this case, pressures on the top of the culvert would be greater than the weight of material above the culvert. The isochromatic fringes immediately above the top of the culvert and along the sides are "broken up" or distorted because of "tearing" in the gelatin and are not a function of strain.

The darkest areas of the photograph are areas of zero strain. From that observation, it is immediately apparent that two, large strain "bulbs" formed at the upper corners of the culvert and reached to the surface. If straining were allowed to continue, it appears that failure surfaces similar to those shown in Figures 1 and 2 would develop. This would tend to support present failure theories.

ANALYSIS

Differential Settlement

Settlement gages at the sites in McCreary County were positioned in such a manner that settlement points were located in both the interior and exterior soil prisms *(1)*. Thus, correlations between the differential settlement between the soil prisms and pressure on the top slab could be made. The data were limited because of damage to instrumentation at Station 203 + 20.

The criterion used in determining whether data were acceptable was that the pressure readings had to exemplify the characteristic of an imperfect trench by the pressure on the top slab being less than that on the adjacent exterior prisms. This relationship occurred at all the study sites, even though at Station 89 + 20 the interval between the initial reading and the time at which the pressure in the exterior soil prism exceeded the pressure on the top slab was over two years. This, apparently, could be attributed to arching between the culvert and the original soil.

The design pressure less the measured pressure was used in the comparison rather than the measured pressure to account for differences in fill heights at Stations 89 + 20 and 210 + 50. The remaining, aforementioned variables, with the exception of culvert widths, were assumed to be constant for both locations. The width of the culvert at Station 89 + 20 is 10 feet (3.05 m); the width of the culvert at Station 210 + 50 is 8.167 feet (2.49 m).

From the plot in Figure 4, the relationship between the design pressure minus the measured pressure on the top slab and the differential settlement between the interior and exterior soil prisms for culverts designed using the imperfect trench method is given by

$$\log y = 0.1663\ x \qquad 13$$

where y = design pressure less measured pressure (psi) and

x = differential settlement between interior and exterior soil prisms (inches).

This relationship should be further verified since only five data points were used to obtain the curve.

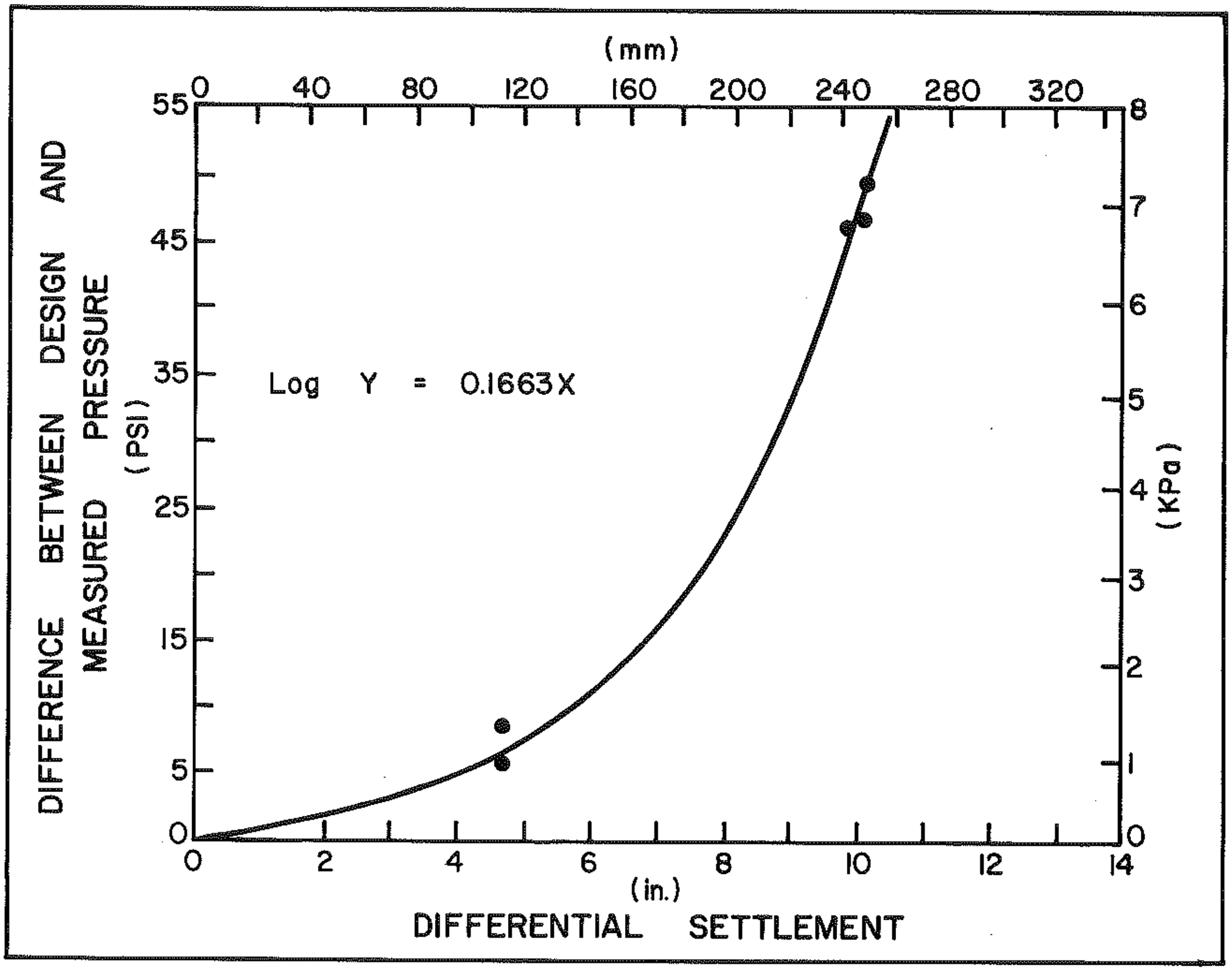

Figure 4. **Difference between Design and Measured Pressures on Top Slab versus Differential Settlement between the Interior and Exterior Prisms: Culverts Designed for the Imperfect Trench Condition.**

Pressure Distributions

The pressure distributions on the culverts at the five sites at various times are shown in Figures 5 through 9. Pressure cell readings at intermittent dates were plotted and lines drawn to connect the data points where possible.

Initially, at Station 268 + 30, the pressure on the top slab was greater on the left side than the right side; greater pressure was measured on the right side of the bottom slab (Figure 5). The pressure distribution also shows that greater pressure initially existed on the left sidewall. As the fill height increased, pressures on the top, bottom, and sidewalls equalized. When the fill height reached 15 feet (4.58 m), a reversal in the trend occurred; higher pressures occurred on the left bottom and on the right top. Then, there was greater pressure acting on the right sidewall. Cell PE-58 set in bedrock to the right of the culvert always measured higher pressure than Cell PE-52 also set in bedrock to the left of the structure. As time progressed, a greater difference was observed between pressures acting on the left side and the right side of the culvert. Approximately five months after construction, Cell PE-55, located in the right side of the top slab, measured 34 psi (4.93 kPa) greater than the pressure measured at PE-60. Likewise, pressure differences of approximately 21 psi (3.05 kPa) and 9 psi (1.31 kPa) were measured at the bottom slab and on the sidewalls, respectivly.

The only pressure distribution reversal associated with the structure at Station 123 + 95 occurred in the bottom slab (Figure 6). Initially, greater pressure was measured on the left side of the bottom slab than on the right side. Approximately twenty days later, a reversal occurred indicating greater pressure on the right side. The two Carlson cells positioned in bedrock adjacent to the culvert indicated the same reversal. The higher pressure measured by the cell located in the bedrock on the right side could be attributed to active arching between the culvert and the original ground. At Station 123 + 95, the maximum pressure occurred on the side of the culvert with the largest trench width.

The pressure distributions at various times for the three culverts in McCreary County did not exhibit pressure reversals (Figures 7 through 9). However, there were large differences in pressures acting on the right and left side of the top slab of each culvert with the exception of the culvert located at Station 210 + 50.

Finite Element Analysis

In 1973, Duncan and Ozawa *(9)* wrote a computer program called ISBILD for analysis of static stresses and movements in embankments. The program was developed for analysis of stress and movements in dams and is applicable for any soil embankment analysis. The program takes into account the non-linearity of the soil by using hyperbolic stress-strain relationships. In addition, the program utilizes iso-parametric elements with incompatible displacement modes for greater accuracy. This program was used for each of the five culverts. The soil-culvert systems were assumed to be symmetrical, thus making it possible to use half the soil-culvert system in the grid system. The grid system was supported by pin-type supports on the bottom, which would simulate the structures being constructed on solid rock. Support of the nodal points located on the sides of the grid system consisted of rollers providing horizontal support only.

Three material types (rockfill, compacted clay backfill, and granular foundation) were used in analyses of the two soil-culvert systems in Clark County. Values of the parameters associated with the different material types were obtained from a list of parameters for 42 soils tested under drained conditions *(10)*. Two material types were used for the study site in McCreary County. One type was used for the soil (dense sand) fill material and another type (very loose clay) for the material used in the imperfect trenches. Very low modulus values were assigned to the imperfect trench material to represent the low degree of compaction in the trench and the large settlement therein.

A comparison of the pressures obtained from Duncan's program with measured and design pressures is shown in Table 3. The pressures obtained from Duncan's program are, as a general rule, less than measured pressures; design pressures calculated in accordance with 1.2.2(A), **Standard Specifications for Highway Bridges**, were generally larger than the measured pressures. Another observation from Table 3 is that Duncan's program gave pressures closer to the measured pressures at the top slabs than did the design formula.

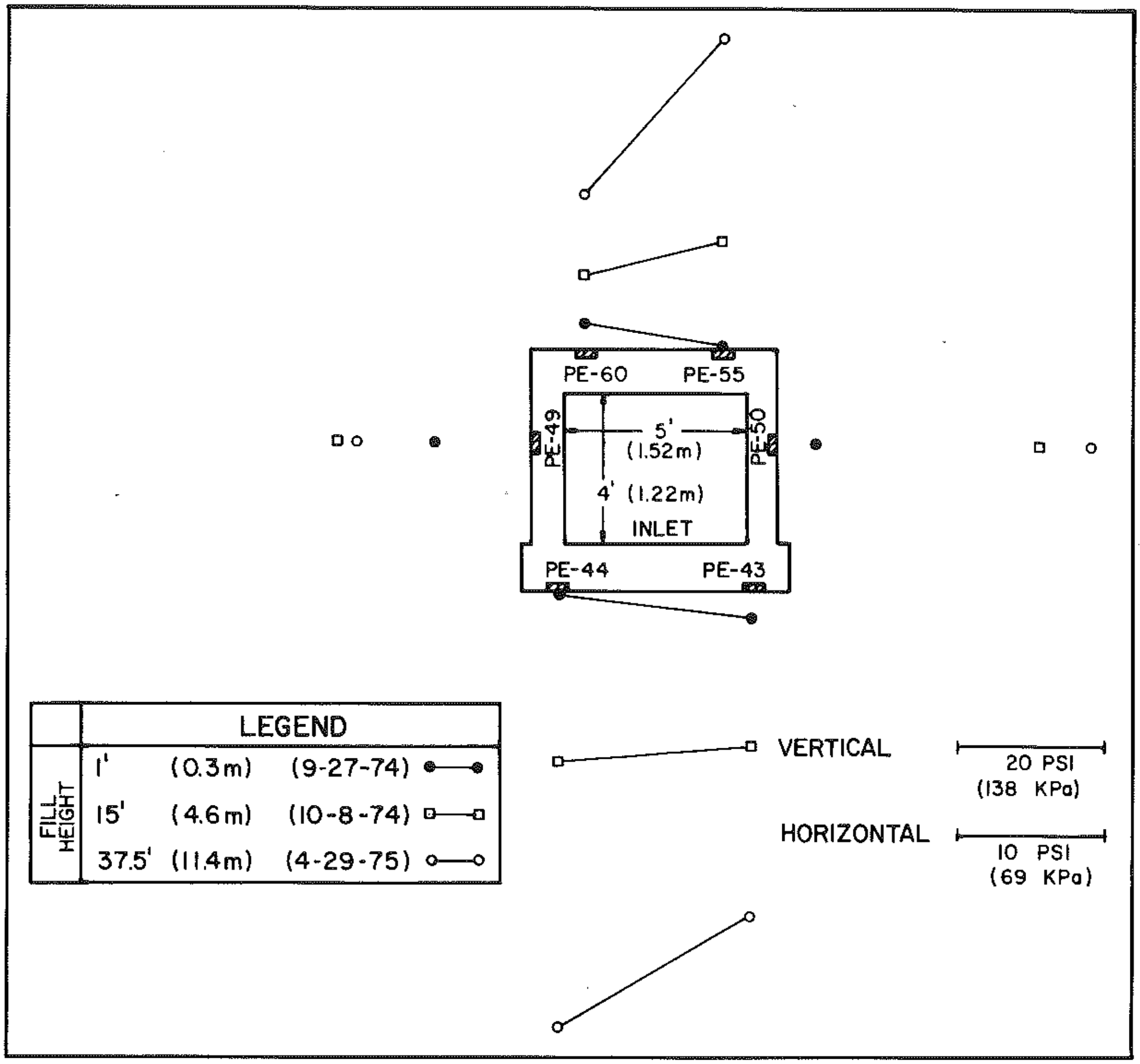

Figure 5. Pressure Distributions and Lapsed Time; Culvert at Station 268 + 30, Clark County.

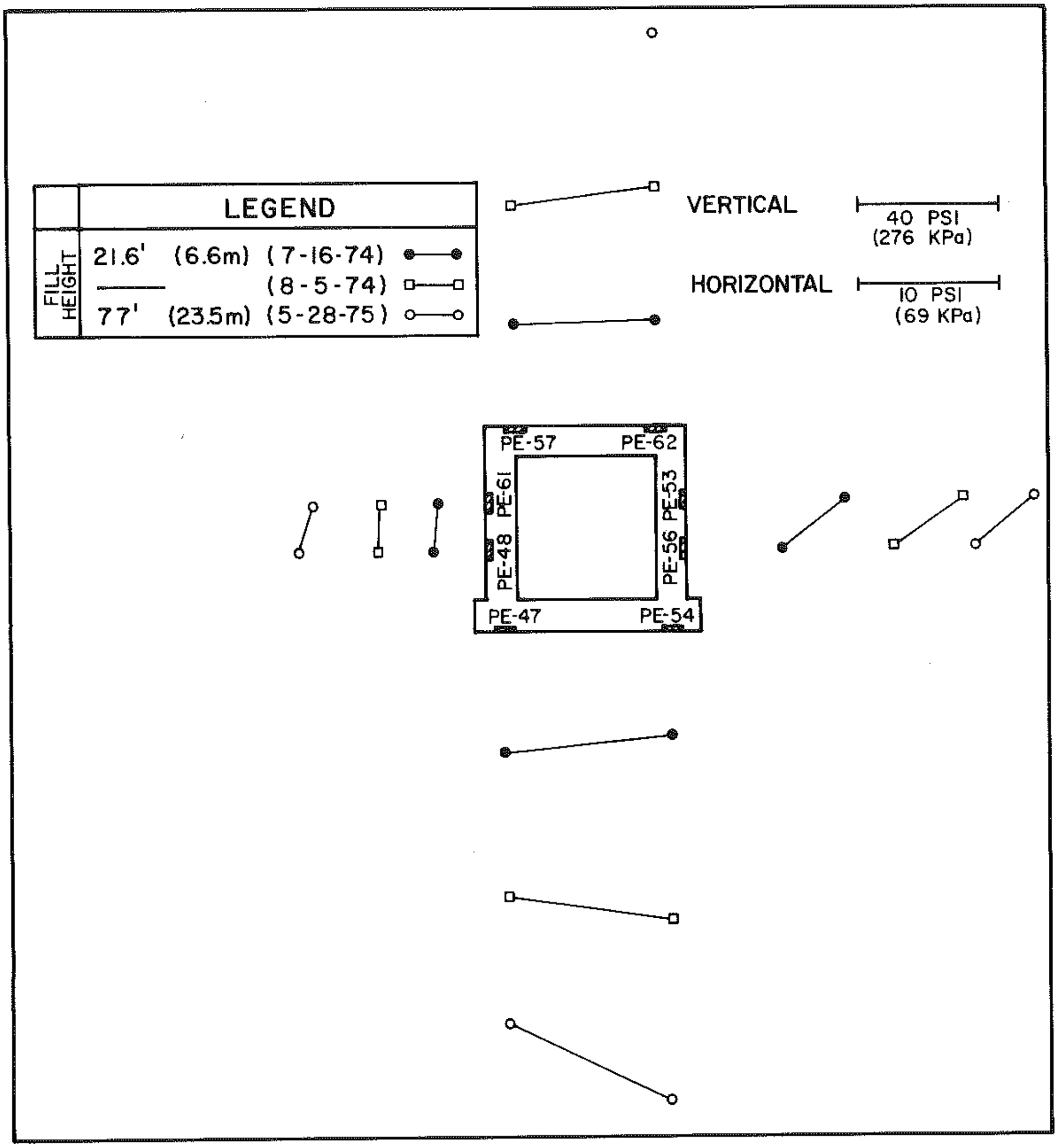

Figure 6. Pressure Distributions and Lapsed Time; Culvert at Station 123 + 95, Clark County.

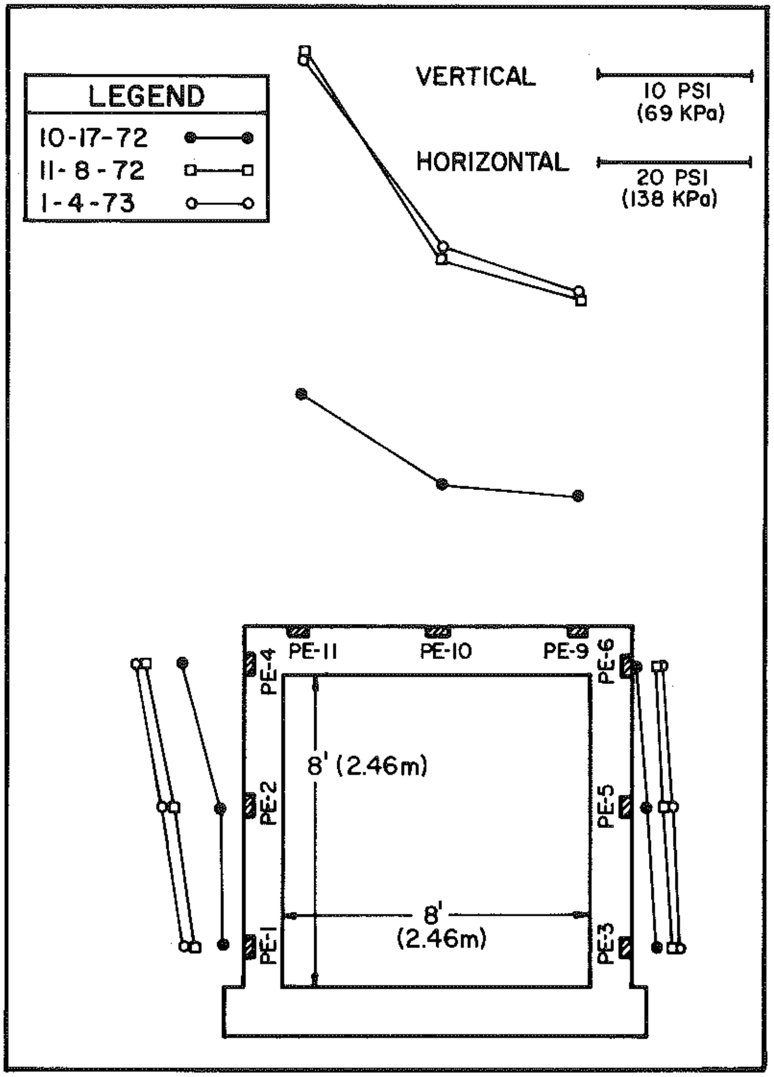

Figure 7. **Pressure Distributions and Lapsed Time; Culvert at Station 89 + 20, McCreary County.**

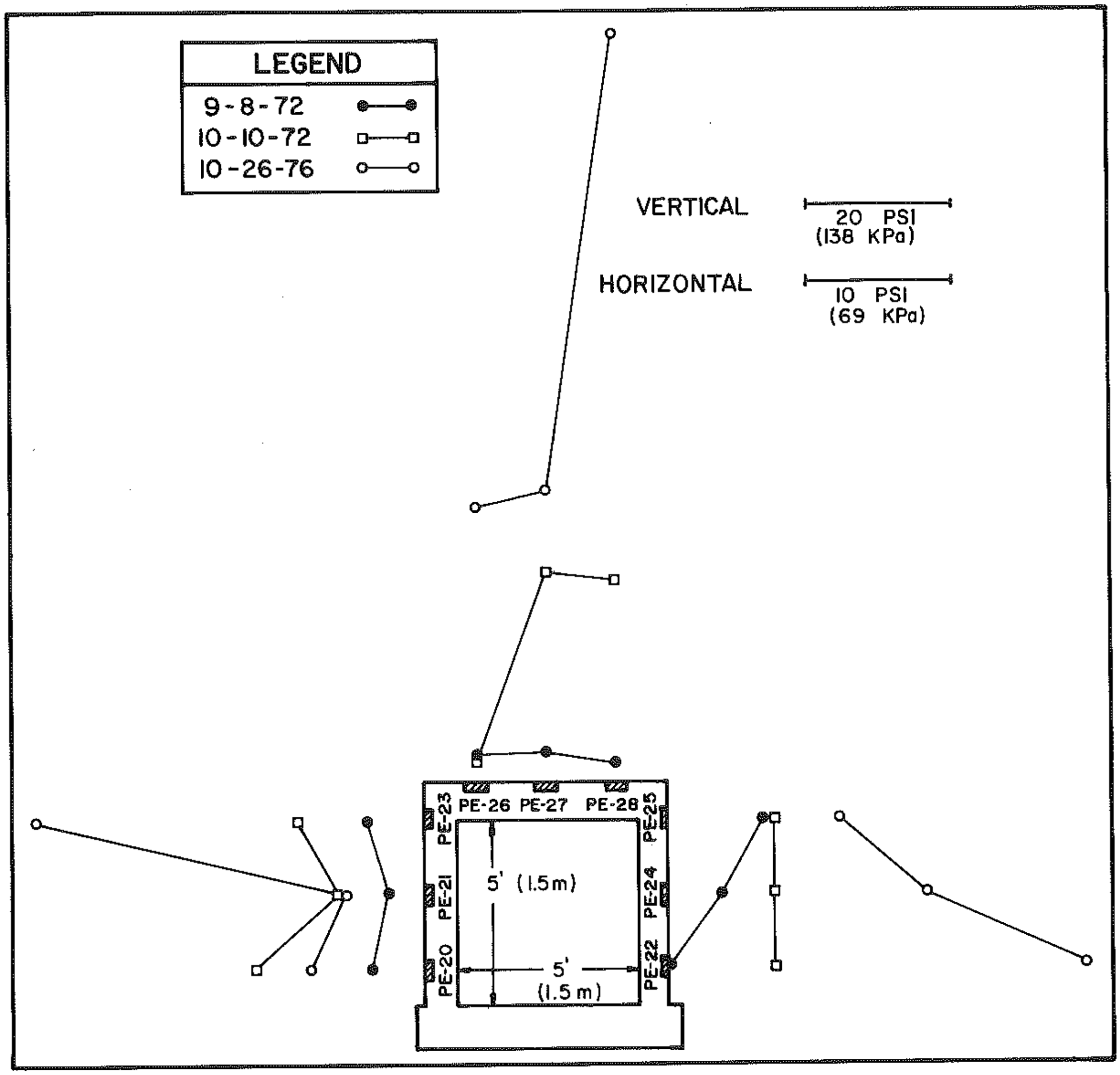

Figure 8. Pressure Distributions and Lapsed Time; Culvert at Station 203 + 20, McCreary County.

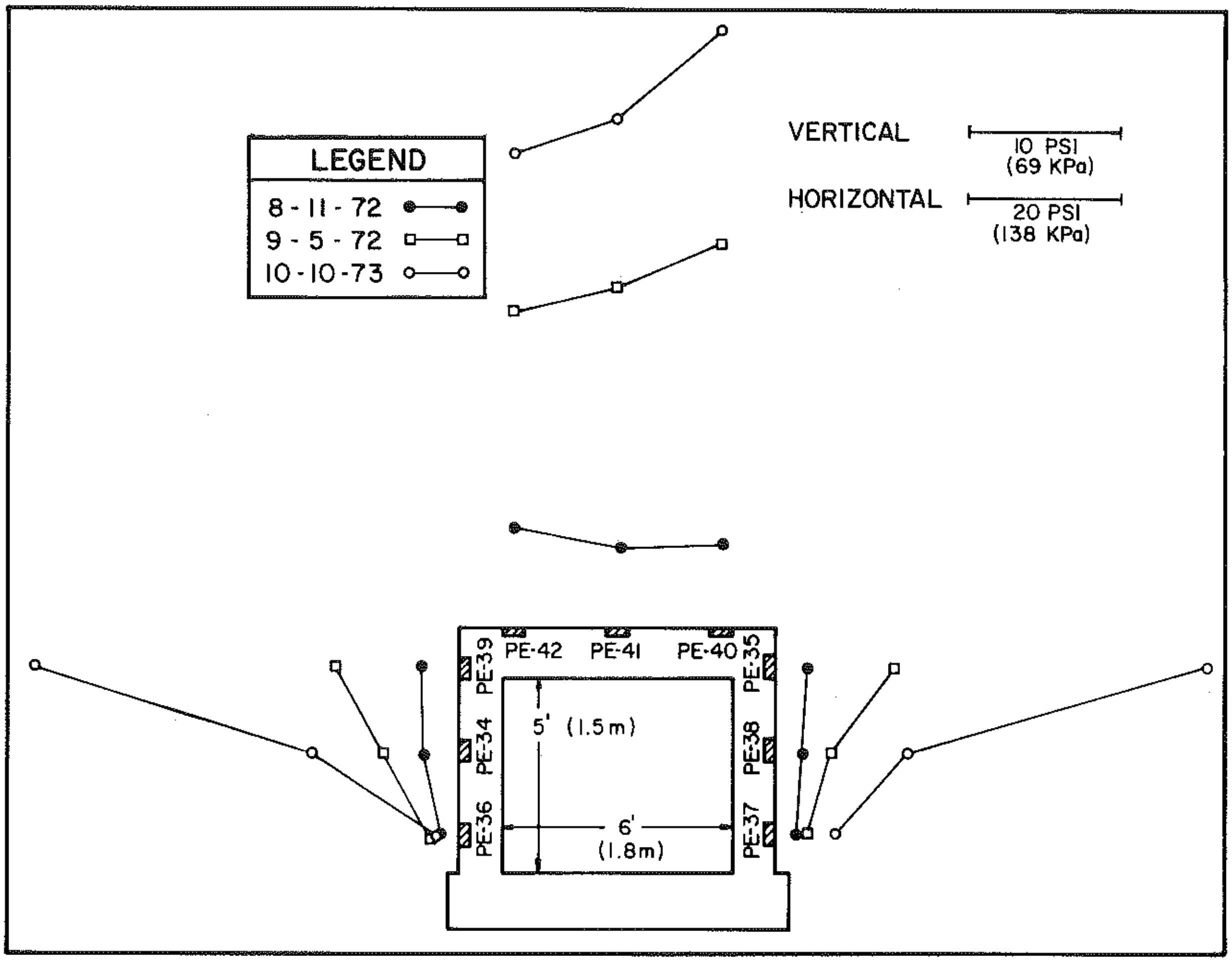

Figure 9. Pressure Distributions and Lapsed Time; Culvert at Station 210 + 50, McCreary County.

TABLE 3. COMPARISON OF PRESSURES CALCULATED BY DUNCAN'S FINITE ELEMENT PROGRAM WITH MEASURED AND DESIGN PRESSURES

LOCATION	SLAB	CARLSON CELL NUMBER	(A) DUNCAN'S FINITE ELEMENT PROGRAM psi (kPa)	(B) CARLSON CELL PROGRAM (MEASURED) psi (kPa)	(C) DESIGN PRESSURE psi (kPa)	(B - A)/B DIFFERENCE (%)	(B - C)/B DIFFERENCE (%)
Sta 123 + 95 Clark County	Top Slab	62	85.1 (586)	110.6 (762)	44.9 (309)	26	59
		57	85.1 (586)		44.9 (309)		
	Left SW	61	21.1 (145)	12.4 (85)	21.8 (150)	-70	-76
		48	30.7 (212)	13.4 (92)	22.3 (154)	-129	-66
	Right SW	53	21.1 (145)	24.3 (167)	21.6 (149)	13	11
		56	30.7 (212)	20.2 (139)	22.0 (152)	-60	1
Sta 268 + 30 Clark County	Top Slab	55	25.8 (178)	63.2 (435)	38.3 (264)	59	39
		60	25.8 (178)	31.5 (217)	38.3 (264)	18	-22
	Left SW	50	45.3 (312)	21.3 (147)	14.2 (98)	-113	33
		51	45.3 (312)	15.2 (105)	14.2 (98)	-66	7
	Right SW	49	45.3 (312)	12.0 (83)	14.2 (98)	-74	-18
		45	45.3 (312)	8.8 (61)	14.2 (98)	-415	-61
Sta 203 + 20 McCreary County	Top Slab	26	26.3 (181)	37.2 (256)	103.4 (712)	29	-178
		28	26.3 (181)	101.3 (698)	103.4 (712)	74	-2
		27	25.9 (178)	39.5 (272)	103.4 (712)	34	-162
	Left SW	23	7.5 (52)	26.2 (181)	35.8 (247)	71	-37
		21	7.5 (52)	5.4 (37)	36.8 (254)	-39	-582
		20	7.5 (52)	7.7 (53)	37.9 (261)	3	-392
	Right SW	25	7.5 (52)	11.5 (79)	35.8 (247)	35	-211
		24	7.5 (52)	17.4 (120)	36.8 (254)	57	-112
		22	7.5 (52)	0.8 (6)	37.9 (261)	-838	-4,638
Sta 210 + 50 McCreary County	Top Slab	42	12.1 (83)	30.4 (209)	77.3 (533)	60	-154
		40	12.1 (83)	38.4 (265)	77.3 (533)	69	-101
		41	9.6 (66)	32.8 (226)	77.3 (533)	71	-136
	Left SW	39	8.3 (57)	54.4 (375)	37.0 (255)	85	32
		34	8.3 (57)	18.9 (130)	37.9 (261)	56	-101
		36	0.8 (6)	2.6 (18)	38.8 (267)	69	-1,392
	Right SW	35	8.3 (57)	55.8 (384)	37.0 (255)	85	34
		38	8.3 (57)	17.0 (117)	37.9 (261)	51	-123
		37	0.8 (6)	7.7 (53)	38.8 (267)	90	-404

FACTOR OF SAFETY ANALYSIS

Factors of safety were calculated by dividing the moment capacity of the top slab by the maximum moment actually acting on the top slabs. The maximum moment acting in the top slab was obtained in the following manner. Pressures measured by the Carlson cells were used to draw pressure distributions around each culvert. Since the pressure distributions on the sidewalls were unequal, a sidesway correction was incorporated.

The moment capacity of the top slab was calculated from

$$M = f_s A_s jd \qquad 14$$

where f_s = actual unit stress in the steel (psi),
A_s = area of steel (in.^{2}),
j = $(1 - \sqrt{2pn + pn^2} - pn)/3$,
d = distance from edge of concrete in compression to tension steel (in.),
p = percentage of steel in section, and
n = ratio of moduli.

The moment capacity was divided by the actual moment to obtain a factor of safety. The factors for Stations 123 + 95 and 268 + 30 in Clark County were 6.1 and 6.8, respectively. The factors of safety were 12.9, 12.3, and 20.8, respectively, for the three culverts at Stations 89 + 20, 203 + 20, and 210 + 50 in McCreary County.

The reason the factors of safety are so high at all five sites is primarily due to the method of design. The working-stress design method automatically gives some margin of safety. Also, only a small percentage of the actual strength of the steel and concrete was allowed in design, i.e.; 20-ksi (138-MPa) strength was assumed for 60-ksi (414-MPa) steel and 1.2-ksi (8.3-MPa) strength was assumed for 4.5-ksi (31.0-MPa) concrete. This alone would introduce large factors of safety. Also, in McCreary County, the culverts were designed without the imperfect trench but were constructed with the trench, resulting in pressures considerably less than the design pressures.

Additionally, the magnitudes of the factors of safety are affected somewhat by the method of calculation used in this report. Moment diagrams were drawn from measured pressures, and the maximum moment could have occurred at any point on the top slab. However, in design, the maximum moment is assumed to occur at the center of the slab.

DISCUSSION AND CONCLUSIONS

The data to date indicate that the imperfect trench method of design is effective in reducing vertical loads on culverts with a sandy soil or a soil composed of silt and sand as the fill material. After five years, the two culverts in McCreary County having reliable pressure cell data have pressures less than expected on the top slab. In one of the two cases, the measured horizontal pressure was greater than the design pressure.

From the data obtained, the current design equations used in determining the loads on culverts appear to be inadequate. The design pressure using current procedures underestimated the measured pressure on the top slab at Stations 123 + 95 and 268 + 30 by 59 percent and 39 percent, respectively. However, for the same two stations, Costes' method overestimated the design pressure by 7.8 percent and underestimated the design pressure by 1.6 percent, respectively (Table 1). Likewise, the design pressure overestimated the measured pressure at Stations 89 + 20 and 210 + 50 by 30 percent and 115 percent, respectively. Costes' procedure again gave better results than the design pressure; however, Spangler's theory predicted pressures which were closer to the measured pressures than did Costes'. Using Spangler's theory, pressures 29 percent less than design pressures were predicted at Station 89 + 20; pressures 7 percent greater than design pressures were predicted at Station 210 + 50. Thus, for pressure predictions, Costes' method is more accurate for culverts on unyielding foundations and Spangler's theory gives best results for culverts with the imperfect trench.

From the culvert surveys, there was no conclusive evidence that the imperfect trench method of design causes a dip to form in the roadway surface. As stated earlier, only one of 25 pipe culverts designed using imperfect trench had a significant dip in the roadway surface; one of ten pipe culverts with standard bedding conditions had a significant dip. After five years, there are no dips in the roadway surface at any of the three culverts in McCreary County.

All box culverts used as study sites appear to be overdesigned. This is attributed to the use of working stress rather than ultimate stress in design. For instance, factors of safety calculated for the culverts located at Stations 123 + 95 and 268 + 30 by dividing the ultimate moment capacity of the top slab by the maximum moment actually on the slab were 6.1 and 6.8, respectively. These factors of safety resulted even though measured pressures on the culverts were considerably greater than the design pressures.

In excavating for a box culvert, there is a possibility that a dip in the bedrock will occur (Figure 10a). Two alternatives are suggested to prevent the structure from developing exaggerated bending stresses at this location. The first alternative would be to excavate the bedrock to an elevation below the dip (Figure 10b); to ensure uniform bedding conditions and reduce the load on the top of the culvert, a uniform layer of soil should be placed under the structure (Figure 10b). The other alternative would entail filling the dip with DGA or some suitable material to prevent downward movement of the structure at this location (Figure 10a). However, Section A-A, Figure 10c, illustrates a problem area in this case. Viewing the culvert from the inlet, it is apparent that the dip might extend beyond the width of the trench and the exterior prism. It is suggested that the excavation and backfill extend the width of the exterior prism in order to prevent an unequal amount of settlement between the interior and exterior soil prisms on either side of the culvert.

If there is a sudden change in bedrock elevation ("break") along the longitudinal axis of the culvert, it is possible that severe bending stresses would be induced in the culvert. That portion of the culvert having the thickest compressible foundation material (soil or gravel) will be subject to considerably greater settlement, due to overburden pressures. This would cause the culvert to act as a box-beam, and the distribution of stresses and pressures along the culvert barrel and in the footer would not have been considered in design. For example, as shown in Figure 10a, at the transition point between the thin foundation and the thick foundation, tremendous amounts of pressure would be concentrated in that portion of the footer resting on the higher bedrock. Also, large shearing stresses would occur in the culvert barrel at the same point.

Most problems of this nature could be eliminated by simply constructing stress relief joints in box culverts. This would allow each individual segment of the culvert to act somewhat independently of the others. Also, it has been shown from a previous study *(11)* that a culvert on a uniform, compressible foundation will settle more in the center (under the highest portion of the embankment) than at the ends. Therefore, cambering and the use of stress relief joints would also eliminate most of the problems associated with this type of settlement.

The foundations for the two culverts in Clark County are comprised of very uniform limestone with no sharp differences in elevation along the length of the culverts. In addition, the culvert footers are resting on approximately 12 inches (300 mm) of uniformly compacted, dense-graded aggregate. Flowline elevations taken periodically since construction have indicated little or no settlement. It appears, therefore, those culverts are not being subjected to significant bending stresses, and the measured pressures are probably true earth pressures. However, the foundation conditions for the three culverts in McCreary County are not known; therefore, the same cannot be said for the pressure readings at those sites.

RECOMMENDATIONS

Following is a list of recommendations resulting thus far from the study:

1. Use the imperfect trench method of design in soils composed largely of sand and silt.
2. Attain uniform bedding conditions by eliminating dips in the bedrock in the foundation and either or both of the exterior prisms.
3. Use Costes' method of predicting the load on top of culverts designed without the imperfect trench and constructed on bedrock. If Costes' method yields pressures that are higher than pressures found by current design procedures, use the pressure resulting from Costes' method.
4. Use Spangler's method for culverts designed using the imperfect trench or on a yielding foundation as a check on the pressure calculated using the current design formula.

These recommendations should ensure more economical designs with lower loads acting on top of the structure. A more uniform pressure distribution should also result.

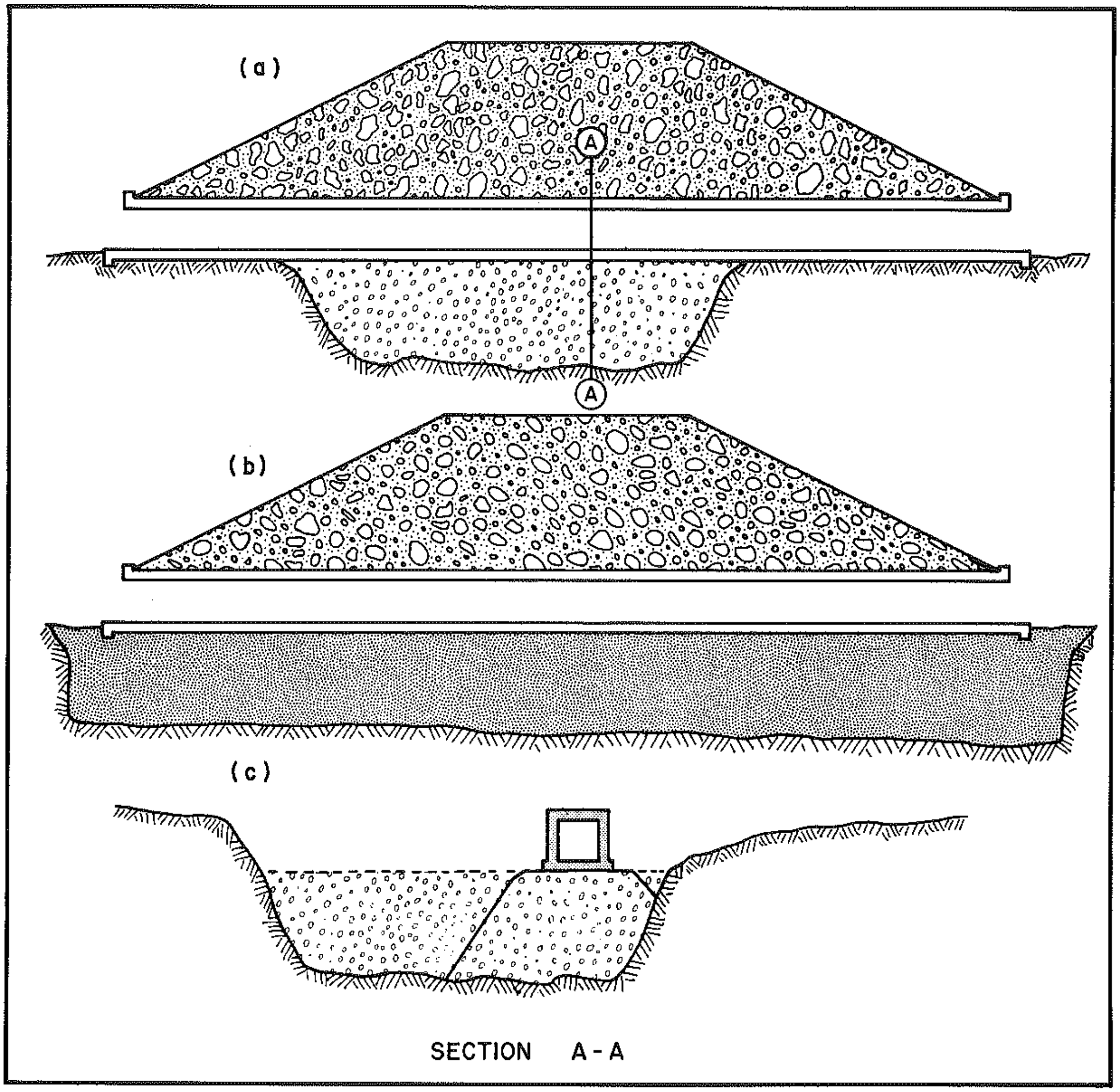

Figure 10. Illustration of Possible Bedding Problems and the Recommended Methods of Construction.

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