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Number 262

Pipes and Culverts

5 Reports

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Foreword

This RECORD contains five papers on the design and performance of buried conduits under varied site and installation conditions. Designers and constructors of underground storm drain systems, culverts, and tunnels will gain an insight into problems and solutions in the selection of materials for normal and complex situations. The aim, as for all construction, is the use of the most economical conduits that will meet operational requirements and serve effectively for the design life of the project.

The first two papers review problems in the structural design and installation of large circular culverts on Interstate highway projects. One is an 18.5-ft diameter structural plate "flexible" pipe with up to 83 ft of cover in Montana, the other a 4-ft diameter reinforced concrete "rigid" pipe under a maximum cover of 36 ft in Kentucky. Both of these installations evidenced structural distress, necessitating critical review of the original designs and methods of installation. Changes in design and construction followed.

The structural failure of the Wolf Creek, Montana, culvert and research as to its probable causes were described in two reports published in RECORD 144 in 1966. Scheer and Willett have made a thorough study of the reconstruction and subsequent performance of this culvert using strain, pressure, and settlement measuring devices. It is noteworthy that the gages revealed bending strains in addition to circumferential compression in the plates during and after erection. The reconstruction successfully used the "imperfect trench" type of construction with a 3-ft layer of baled straw above the culvert to reduce the vertical load on the structure to about half the weight of the overlying soil prism.

Deen reports on the concrete pipe placed with Kentucky's standard B (positive projection) bedding under rock embankment. With only one-fourth of the 560 sections installed, pipe cracking was noted even before fill reached the full 36-ft design height. The remaining sections were placed using the imperfect trench method of backfill, successfully relieving the load and eliminating cracking.

Spangler, in discussions of the Scheer-Willett and Deen papers, further reviews the conditions attendant to the distress experienced in the Montana and Kentucky projects as originally installed. He points out that the 18.5-ft diameter Wolf Creek reconstruction is the largest use of the imperfect ditch method to date and that the data obtained permit the extension of flexible pipe design criteria previously validated only up to 84-in. diameter. The Deen paper showed that good results are obtained using the Marston theory of loads on underground conduits under rock embankments as well as earth embankments.

Karadi and Krizek describe culvert design practices in several northern and central European countries, including national standards as well as individual designs. The report covers types and sizes of material in use, hydraulic and structural design

methods, bedding and backfilling procedures, and durability considerations. About 85 percent of the culverts used are of concrete, since steel pipe is a critical item, particularly in eastern countries. Newer plastic pipe is only in limited use.

The two other papers in the RECORD report on field, laboratory, and analytical studies to evaluate and predict the durability of aluminum alloy culvert, which has been in use only since 1960. The mechanics of abrasion, erosion, and corrosion are difficult to separate when examining the field use of metal culverts subjected to various soils and waters of different physical and chemical characteristics, varying flow and bedload, and other environmental factors. These studies extend the experience record, geographic coverage, and number of installations reported, thus affording better predictions of serviceability.

Koepf in his paper looks into rock size and flow velocity as the keys to the amount of abrasion. About 200 aluminum alloy culverts, in service an average of about 5 years in abrasive exposures, were examined. Based on his field observations, the author rated the performance of each installation in one of five zones, from "no surface effect" with life expectancy of 50 or more years to "abusive" with pipe life less than 25 years. These field ratings have also been correlated analytically to give comparable ratings of predicted performance for culverts depending on the culvert size and slope and prevailing rock size. Time alone will reveal the accuracy of these predictions.

Lowe et al report on detailed examination of 583 aluminum culverts and their soil and water environments in 30 states. The authors conclude that corrosion behavior closely follows that expected for aluminum in normal aqueous and atmospheric environments. Principal clues are the pH of soil and water and the soil resistivity. The general range of acceptability for uncoated aluminum alloy culverts is a pH of 4 to 9 and soil resistivity not less than 500 ohm-cm. Exceptions to the resistivity limit occur in granular soils subject to seawater (30 ohm-cm) where aluminum performs satisfactorily.

—Kenneth S. Eff

Contents

Rebuilt Wolf Creek Culvert Behavior

ALFRED C. SCHEER, Associate Professor, Montana State University, and
GERALD A. WILLETT, JR., Civil Engineer, Morrison-Maierle, Inc.

The failure of an 18.5-ft diameter structural-plate culvert under 83 ft of cover led to its reconstruction using the imperfect trench type of construction as well as other changes. The culvert was instrumented with electrical resistance strain gages placed on the culvert walls at approximately mid-height; Carlson Soil Stress Meters placed on the outside walls of the culvert and in the fill; rubber pressure cells adapted from commercial hot water bottles placed in the fill; and settlement cells, which were placed in the fill on each side of the culvert.

The results correlated well and demonstrated that the vertical load on the culvert was much less than the weight of the overlying column of earth. The strain gages, which measured bending strains in addition to circumferential compression, showed that significant residual bending stresses were induced in the plates during erection. They also monitored bending stress changes that occurred at the strain gage sites during and after the backfilling operation. The vigorous compaction of the backfill on each side of the culvert during the early stages of backfilling produced a bending stress pattern in the side walls that persisted throughout the embankment construction period and thereafter.

•THIS PAPER is a condensation of a final research report (1) that describes the behavior of an 18.5-ft diameter structural plate culvert, built of $^3/_8$-in. thick plates, that was rebuilt in 1965 after the failure of the original structure in 1964 under 83 ft of rock fill in Wolf Creek Canyon on Interstate 15 near Helena, Montana.

The failure and the design details of the original culvert were reported by Kraft and Eagle (2). Research on the cause of the failure was reported by Macadam (3). His report supported the hypothesis that the failure was caused by hydrogen embrittlement of the high-strength, $^3/_4$-in. A490 bolts used in the original culvert, and prompted a decision that the culvert be reconstructed using $^7/_8$-in. A325 bolts of milder steel.

In the reconstruction, a 3-ft thick layer of baled straw was placed 5 ft above the culvert and served as the primary element of an imperfect trench type of load relief detail. Figure 1 shows a typical cross section of the rebuilt culvert.

RECONSTRUCTION OF CULVERT AND INSTALLATION OF INSTRUMENTATION

Following the removal of the rock fill from the damaged culvert in the winter and spring of 1965, disassembly of the failed portion was begun and steps were taken to divert the stream. Unprecedented high water thwarted all stream diversion attempts until September, when a successful diversion was finally effected. Both of the undamaged end sections, with a combined length of over 200 ft, were left intact. Only the 328-ft long central section that had failed was rebuilt.

The culvert bedding was high-quality granular backfill material shaped to fit the culvert curvature. The bedding was completed in the middle of October 1965.

Paper sponsored by Committee on Culverts and Culvert Pipe and presented at the 48th Annual Meeting.

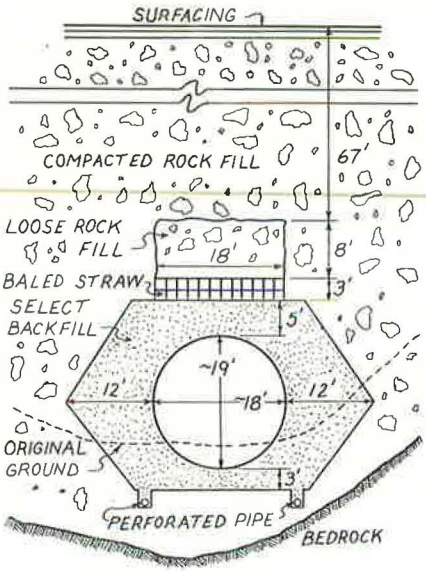

Figure 1. Typical cross section of rebuilt culvert.

The bottom plates were installed first, between the intact end sections. Then the culvert contractor erected plates from each end toward the middle. The culvert was erected in an elliptical shape, approximately 19 ft 5 in. by 17 ft 7 in. Erection of the culvert was essentially complete by the beginning of November and control devices were installed to check movement or rotation of the culvert during backfilling operations. Each control device consisted of a $^3/_4$-in. rod, $3^1/_2$ ft in length, welded to a base plate bolted to the culvert floor. A plumb bob was suspended from the roof directly above the rod. The rod was protected from gravel, boulders, and other debris by a section of corrugated metal pipe. This section of pipe was 15 in. in diameter and 3 ft in length and was welded to the base plate. The control devices also served as sites for vertical diameter measurements and floor settlement measurements.

While the culvert was being erected, and with the contractor's cooperation, an interior walkway was constructed on each side of the culvert and crosswalks were placed at each station where vertical and horizontal dimensions and elevations were to be taken.

Backfill work started the second week of November 1965. The backfill adjacent to the culvert was a crushed granular material of base course quality and was compacted by pneumatic-tire rollers, supplemented by hand tamping, in layers approximately 6 in. thick. Minimum specified density was 95 percent of AASHO T-99, Method D. The material was placed to a depth of 5 ft above the culvert and 12 ft on each side, as indicated in Figure 1.

In late November and early December the straw was placed on the backfill material to form the imperfect trench. Loose rock fill was then placed on the straw and not compacted, nor was any equipment allowed to cross this section until the loose material reached a height of 8 ft above the straw. The remaining fill, consisting of material that was dumped by various hauling equipment, was then pushed into place and continually worked by a D-9 Caterpillar.

The rock fill was completed early in January of 1966 and a gravel surface was placed as a temporary measure to open the highway to traffic. The final surfacing of asphaltic concrete was placed in August of 1966.

Strain Gage Installation

In August of 1965, while stream diversion attempts were still under way, six of the new $^3/_8$-in. thick corrugated structural plates were placed in an easily accessible outdoor work area so that the SR-4 strain gages could be applied, waterproofed, and checked before the plates were installed.

The strain gages selected for this project were epoxy-backed type FA-100-12. This type of gage is a metallic-foil strain gage, which offers a number of advantages over the older paper-backed wire strain gage. One primary advantage, because of its epoxy backing, is its superior long-term stability under adverse environmental conditions.

The gages were installed and their functional behavior checked according to the manufacturer's recommendations, after which they were waterproofed and protected by two layers of epoxy cement and a layer of silicone rubber sealant.

Six strain gages were installed to measure circumferential strains on each instrumented plate, as shown in Figure 2. The gages were placed in pairs, on opposite sides of the plate, at three locations: at the neutral axis of the corrugations and at the bends on each side of the neutral axis. By placing the gages in these positions, the bending stresses and bending moment could be computed and the average compression determined as indicated in Figure 2.

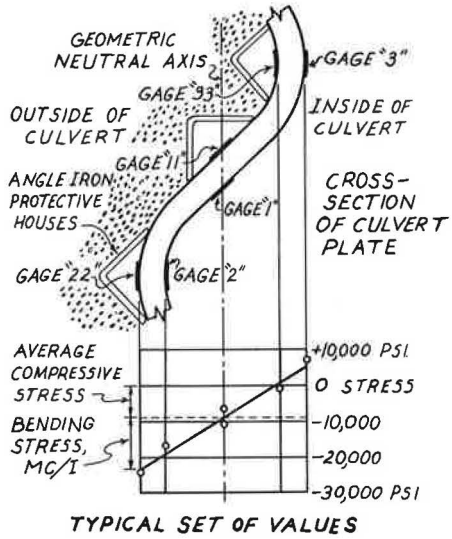

Figure 2. Position of SR-4 strain gages to measure circumferential strain in the corrugated plates.

The six plates, containing a total of 36 gages, were indentified as plates A, B, C, D, E, and F. The strain gages located on the inside of the culvert, using plate A as an example, were numbered A1, A2, and A3, while those on the outside were A11, A22, and A33. Figure 3 shows the locations of the plates on which strain gages were attached. They were placed in the side walls of the culvert, at mid-height. The rings were numbered from 1 through 70 from downstream to upstream end, as they were in the original culvert, for identification purposes. Ring number 39 is under the centerline of the Interstate roadway. Notice from Figure 3 that plates A and B were installed in adjacent rings, in approximately diametrically opposed positions, as were plates C and D.

Pressure Cell Installation

The pressure cell selected to measure earth pressure on the Wolf Creek culvert was the Carlson soil stress meter. The meter selected measures pressures up to 150 psi, is $7\frac{1}{4}$ in. in diameter, 1 in. thick, and has stem on the back that houses the wire resistance coils.

The shape of the meter, particularly the protrusion on the back, created difficulties in mounting it on the corrugated metal culvert wall. The Carlson meter is normally mounted in a rigid wall or footing with its sensitive face flush with the wall surface and against the soil. In this case a mounting frame was made to provide a rigid backing for the cell and to allow it to be mounted essentially flush with the outside surface of the culvert.

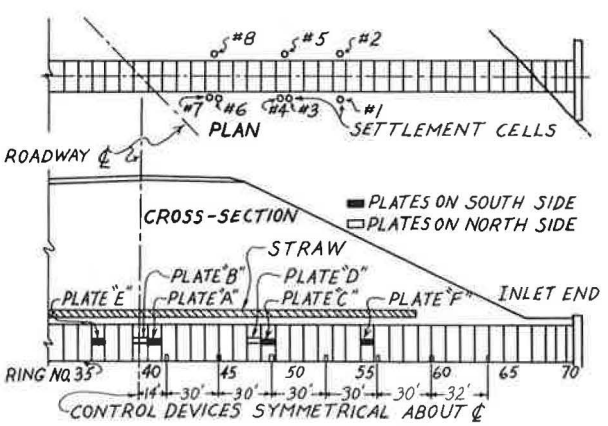

Figure 3. Location of SR-4 strain gages, settlement cells, and control devices.

Installation of the stress meters was started as soon as the necessary plates were erected by the culvert contractor. The holes for mounting frame bolts were drilled and the holes for the cell stems were cut with an acetylene torch. Eighteen mounting frames were bolted to the culvert. In every case the bottom edge of the frame was butted against a bolted seam. Mounting frames were not used for

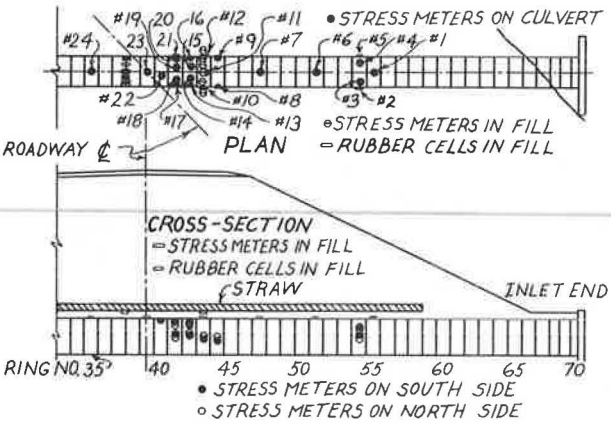

Figure 4. Location of Carlson stress meters and rubber pressure cells.

those meters placed on the top centerline of the structure. These meters were placed in a bedding of sand-cement mortar.

Six stress meters were also placed in mounting frames and buried in the fill on a plane level with the top of the culvert, 1 ft above ring 43, to measure vertical earth pressure above the culvert.

In addition to the stress meters, six rubber bladder pressure cells of the hot water bottle type, using a mixture of water and antifreeze, were used to measure pressure under the baled straw above rings 37 and 38. Holes were drilled in the bottle stoppers and copper fittings attached with epoxy cement. Flexible copper tubing ran from each bottle to a station inside the culvert where the tubing terminated in a bleeder valve, a shut-off valve, and a 100-psi capacity Bourdon gage.

The locations of the stress meters and rubber pressure cells are shown in Figure 4.

Settlement Cell Installation

Settlement of the soil adjacent to the Wolf Creek culvert was determined by the use of eight liquid level settlement cells similar to a type used by Schlick (5). The settlement cells were connected to a manometer board with tubing containing water. Any change in elevation of the cell was indicated directly by the fluid level in the manometer standpipe. The location of the cells is shown in Figure 3. The cells on the south side were placed about a foot above the top of the culvert. The cells on the north side of the culvert were placed about a foot higher than those on the south side to insure a gradient from the cell to the low point located below the manometer board at the culvert inlet.

DATA RECORDED

Strain gage readings were taken before the plates were installed in the culvert to establish a basis for zero strain.

After the culvert was erected and before any backfill was placed, readings were taken to determine the strains caused by erection. Readings were then taken several times weekly until the full fill height was reached. The usual interval between readings was increased to several weeks after the fill was completed.

Stress meter readings were taken for all meters at a central switchboard and were corrected for effects of cable length and temperature. Stress meter readings were taken at the same time that the strain gage readings were taken.

After the rubber pressure bladders were connected to the Bourdon gages and the air removed, the gage pointers were adjusted to read zero. This gave an initial zero reading for measuring earth pressure corrected for the pressure caused by the difference in elevation between the Bourdon gage and the rubber cell. Readings of the Bourdon gages were taken at the same time as those of the strain gages and the stress meters.

Each settlement cell was made operational by filling the system under pressure with a garden spray pump. The pump was connected to the valve at the low point in the system and fluid was forced into the line, with the top of the standpipe plugged to prevent overflow, until the lines were completely filled. At this time the valve was closed, the pump disconnected and the plug at the top of the standpipe removed. After the fluid settled in the standpipe to the level of the cell, its elevation was recorded. It was

found that readings were erratic and unreliable unless the system was drained and refilled each time a settlement reading was desired. Reproducibility was generally better than plus or minus 0.20 in. by this method.

Measurements of the horizontal and vertical culvert diameter were taken with a telescoping rod at 13 stations inside the culvert at the control devices, as soon as the culvert was erected and before any backfill was placed. Periodic re-measurements were made at the same stations during placement of the backfill and thereafter.

Floor settlement was obtained from periodic observations of the elevation at the top of each control device rod, using an engineer's level near the culvert inlet. Level readings were also taken periodically on the roadway surface above the culvert to determine if there was excessive settlement. These readings were started in September of 1966, as soon as the asphaltic concrete surfacing was completed.

RESULTS AND INTERPRETATION

Strain Gages

Strain gage measurements were taken before and immediately after the culvert was erected in order to determine circumferential erection stresses. The maximum extreme fiber erection stresses ranged from 3,800 psi in plate C to 21,900 psi in plate A. Plates B, D, and E might be considered typical, with erection stresses between 9,000 and 10,000 psi in the extreme fibers. Plate A was one of the last plates to be installed and had to be forced vigorously into place, which may account for its high stress of 21,900 psi.

The erection stresses are shown graphically in Figure 5. The excellent linearity of the stress diagrams indicates that the gages performed consistently.

The placement and compaction of the backfill at the sides of the culvert caused a compressive bending stress in the outside fibers and tensile bending stress in the inside fibers of the sidewalls. For plates A, B, D, and E, these bending stresses were directly additive to erection stresses and therefore produced relatively large compressive stresses in the outside fibers. The strain gages showed a steady increase in direct wall compression but little additional change in bending as the embankment was

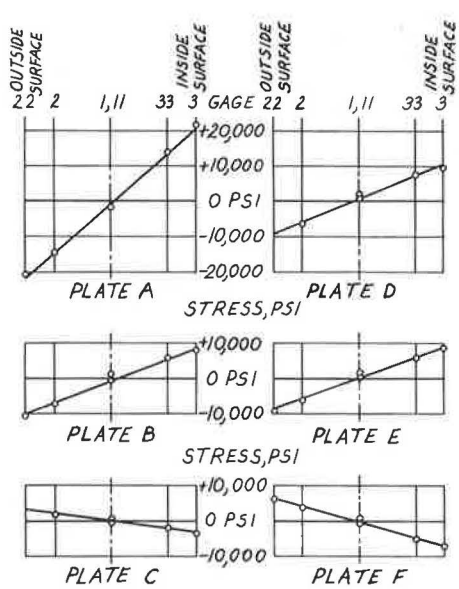

Figure 5. Erection stress in the six instrumented plates.

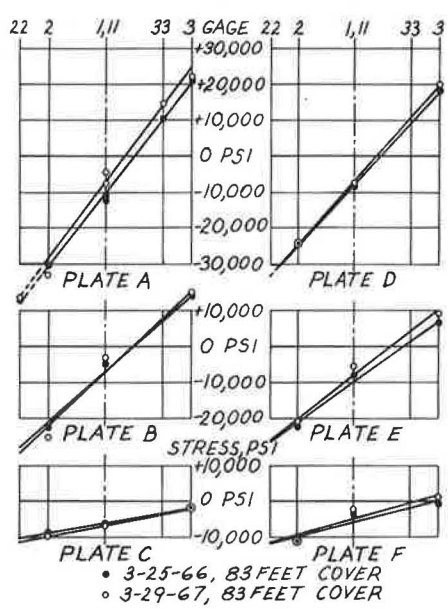

Figure 6. Combined stresses on March 25, 1966, and March 29, 1967.

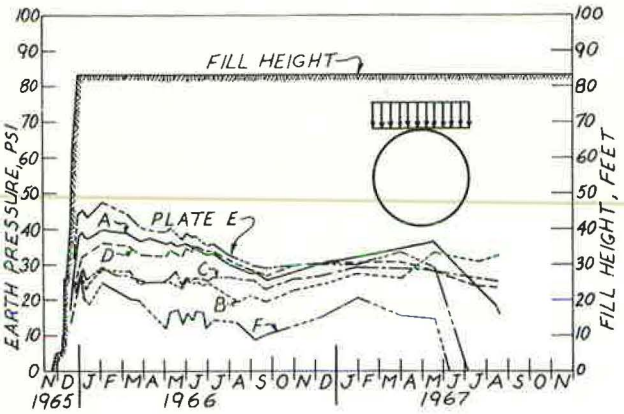

Figure 7. Equivalent uniform vertical earth pressures on culvert, deduced from SR-4 strain gages.

constructed to its full depth. Thereafter the changes were relatively small, as indicated by Figure 6, which shows the stresses approximately 3 months and 15 months following completion of the embankment. By the end of the first winter, many of the gages on the outside walls had deteriorated and no longer furnished usable readings. The performance of the gages on the inside walls did not deteriorate until the summer of 1967.

Figure 7 shows the average vertical earth pressure on the culvert as deduced from the strain gage results by multiplying the average wall compressive stress by the wall sectional area per unit of culvert length and dividing by one-half the culvert width. The pressure was highest shortly after completion of the embankment and it eventually stabilized at a value near 30 psi, which is less than half of the nominal pressure of 72 psi that could be expected from an 83-ft depth of overburden weighing 125 pcf.

Plate F should be considered separately from the other five plates because it was under the sloping side of the embankment at a place where the depth of cover was only 45 ft.

Stress Meters

The earth pressures on the culvert walls were also determined directly with the Carlson soil stress meters. Figure 8 shows only the results from those meters that were mounted directly on the top centerline of the culvert. The stress meter results show generally good agreement with the strain gage results of Figure 7.

Detailed results from the stress meters on the sides of the culvert have been omitted from this report for the sake of brevity. In general, the meters on the sides registered higher and more variable pressures than those at the crown. The overall stres meter results for a group of 15 meters clustered near the center of the culvert are summarized in Figure 9 for the single date of January 4, 1966, when pressures were near maximum. The figure shows an average lateral pressure near 50 psi, or greater, as compared with an average vertical pressure near 35 psi.

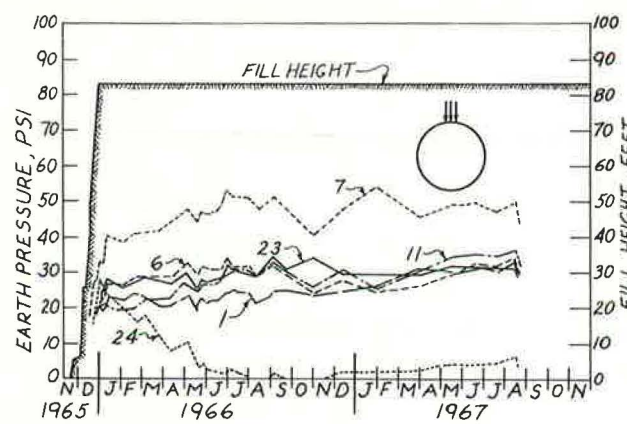

Figure 8. Earth pressure vs time—stress meters No. 1, 6, 7, 11, 23, and 24.

In Figure 9, the minimum and maximum pressure curves are curves that were drawn to mark the boundaries of the zone within which all 15 of the observed pressures fell on January 4, 1966. No special sig-

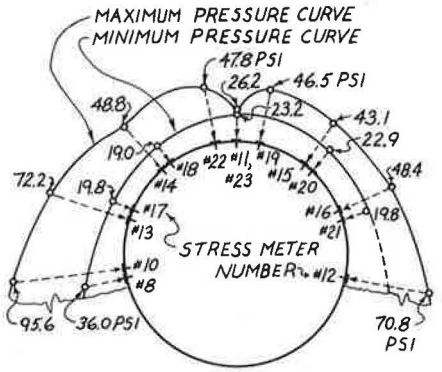

Figure 9. Earth pressures acting on the culvert walls on January 4, 1966.

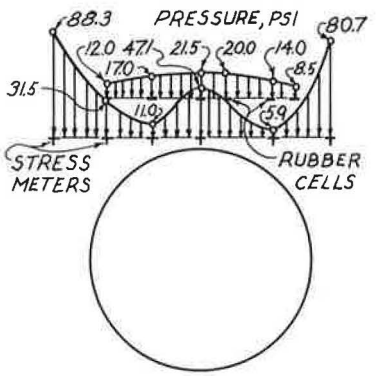

Figure 10. Earth pressures on January 4, 1966, determined from the stress meters and rubber pressure cells located in the fill.

nificance is attributed to the fact that all 15 of the plotted points happen to lie on one or the other of the two boundary curves.

Figure 10 shows the vertical earth pressures registered on the same date by the stress meters buried in the fill 1 ft above the culvert, and by the rubber pressure cells buried in the fill 6 in. below the straw, $4\frac{1}{2}$ ft above the culvert. The rubber cell results shown indicate an average vertical pressure of 17 psi at the base of the straw and are typical of the highly consistent results furnished by those cells. Also notice in the figure that the buried stress meters registered high vertical pressures, in excess of 80 psi, in the soil to the sides of the culvert—this being consistent with (and actually a necessary consequence of) the reduced pressure directly above the culvert resulting from the imperfect trench effect.

Settlements

Figure 11 shows the average settlement of the culvert crown, near the center of the culvert, and the average settlement of the north-side and south-side settlement cells in the same vicinity. Notice that the top of the culvert settled about 2.3 in. while the north-side cells settled 2.7 in. and the south-side cells settled 4.5 in. The lesser amount of settlement experienced by the north-side settlement cells is perhaps related to the fact that the depth to bedrock is somewhat less on the north side of the culvert than on the south side. However, the top of the culvert settled less than any of the settlement cells, which shows that the prism of soil "sandwiched" between the straw and the culvert is subjected to a "positive projection" condition, with the adjacent prisms exerting downward frictional drag stresses on each of the two side faces.

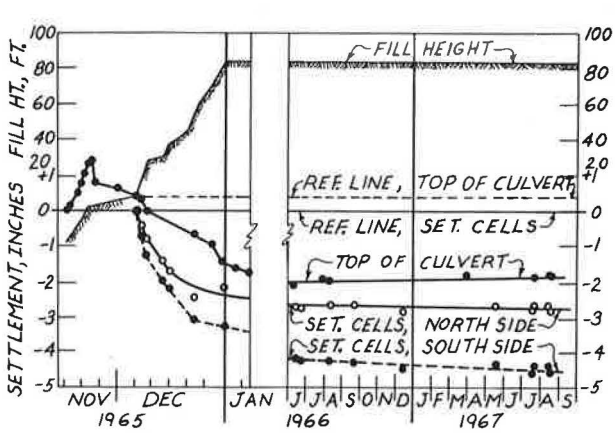

Figure 11. Settlement of the settlement cells and top of culvert.

Figure 12 shows a free-body diagram of the $4\frac{1}{2}$-ft thick central block below the straw with the 17-psi average pressure (from the rubber cells) acting on its top face. The normal pressure on the sides is estimated from Figure 9 as 50 psi and the

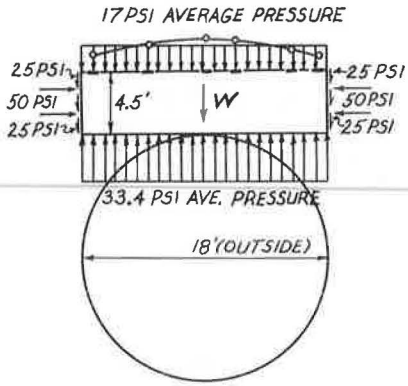

Figure 12. Pressure at top of culvert estimated from rubber pressure cells.

developed internal friction coefficient is roughly estimated as 0.5, which yields an estimated downward shearing stress of 25 psi on the two sides.

For soil weighing 125 pcf, the pressure increase between the top and bottom of the block due to its own weight is 3.9 psi. The additional pressure increase, attributable to the 25-psi shear stresses on the sides, is 12.5 psi. The average estimated pressure at the level of the culvert crown is therefore 33.4 psi. This is a very rough estimate, but it is in good agreement with the vertical pressure on the culvert as deduced from strain gage and stress meter results shown in Figures 7, 8, and 9. It is also in fair general agreement with the four stress meters of Figure 10 that were buried in the fill 1 ft above the top of the culvert, which registered pressures of 31.5, 11.0, 47.1, and 5.9 psi on that date. The 5.9-psi reading should be heavily discounted because the meter from which it was taken proved its unreliability by registering tension a few weeks later.

General Observations

As the backfill material was compacted on each side of the culvert, the vertical diameter increased by 1.4 in. and the horizontal diameter decreased by 1.9 in. Then, as the embankment was constructed to its full height, the vertical diameter decreased until it was approximately 1 in. less than its original value, and the horizontal diameter increased until it was approximately ½ in. less than its original value. Subsequent diameter changes have been very small but they show a slight increase in vertical diameter, amounting to approximately 0.2 in. between June 1966 and August 1967. This is equal to the negative settlement of the top of the culvert, which may be read from Figure 11 for the same time interval. The average settlement of the culvert invert was nearly 1 in. and practically all of this had occurred by the time the embankment was completed.

Periodic profile levels of the roadway pavement have shown that the pavement above the culvert has settled 0.11 ft, which is slightly less than the maximum pavement settlement of 0.14 ft that occurred 100 ft south of the culvert. Thus, the "imperfect trench" caused no differential pavement settlement, but it is not known whether any differential surface settlement occurred before the pavement was placed.

In August 1967 a hole was drilled from the culvert roof up through the straw to determine the compressed thickness of the straw layer that was nearly 36 in. thick originally. At the site of this one exploration hole, the thickness of the compressed straw was only 11 in.

Application of Marston's Theory

Marston's conduit theory (4) was applied to the installation, with the top of the uncompressed straw layer taken as the "critical plane", and with an "equivalent conduit" (18 ft wide and 27 ft high) visualized as consisting of the culvert, the straw, and the intervening earth, all three considered as a single unit. Details are shown in Figure 13.

The "settlement ratio", r_{sd}, was calculated from the following relationship:

$$r_{sd} = \frac{(s_m + s_g) - (s_f + d_c)}{s_m}$$

in which

s_m = the vertical compression of the side columns of soil of height pB_c. B_c is the outside width of the culvert (18 ft) and p is the "projection ratio", which is the ratio of the height of the equivalent conduit to its width. In this case p is 1.5. The numerical value of s_m was estimated, from settlement cell results, as $3\frac{1}{2}$ in.

s_f = settlement of conduit into its foundation. This was measured as approximately 1 in.

s_g = settlement of the natural ground surface adjacent to the conduit. This was estimated as 1 in.

d_c = shortening of the vertical height of the conduit. For the "equivalent conduit" this would be equal to the straw compression plus the vertical diameter change of the culvert, i.e., $d_c = 25 + 2 = 27$ in.

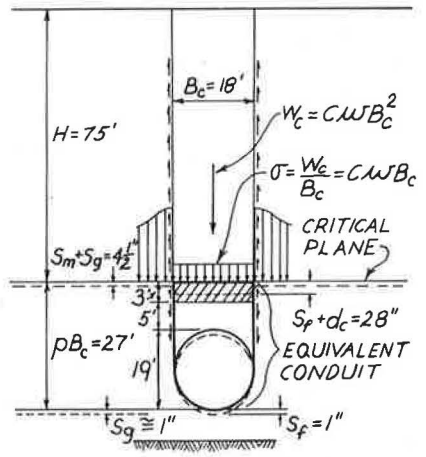

Figure 13. Application of Marston's theory to the Wolf Creek culvert.

The calculated value of r_{sd} is then

$$r_{sd} = \frac{(3.5 + 1) - (1 + 27)}{3.5} = \frac{-23.5}{3.5} = -6.7$$

In this case, with p being equal to 1.5, the product of the settlement ratio and the projection ratio is then

$$r_{sd}p = -6.7(1.5) = -10.0$$

For all practical purposes this is equivalent to a complete ditch condition, so the diagram for ditch conduits may be used to calculate the pressure on the straw. The curve for minimum pressure, in Figure 24-3 of Spangler's text (4), would come the closest to representing the correct condition for the cohesionless rock fill of the Wolf Creek culvert embankment. Then $H/B_c = 75/18 = 4.2$, which yields a value of C = 2.1 from the ditch conduit diagram. Estimating the embankment unit weight, w, as 125 pcf, we may then estimate the pressure on the straw, according to Marston's theory, as

$$\sigma = CwB_c = 2.1\ (125)\ (18) = 4700 \text{ psf or } 33 \text{ psi}$$

This result is almost double the average measured pressure of 17 psi on the base of the straw, as registered by the rubber diaphragm pressure cells. Thus, in this case, Marston's theory overestimates the vertical load on the straw by a factor of 2.

An error as large as a foot or more in the measured thickness of the compressed straw would not have had any effect on the answer obtained from Marston's theory because a value of $r_{sd}p$ as small (numerically) as minus 2 would still have indicated a situation equivalent to the complete ditch condition.

CLOSURE

The investigation showed that the imperfect trench functioned in essentially the anticipated manner and limited the vertical load on the rebuilt Wolf Creek culvert to approximately half the weight of the overburden. The load relief is expected to be permanent because of the highly nonplastic nature of the granular rock embankment.

Significantly, the lateral load on the culvert exceeds the vertical. The stress meter results in Figure 9, the bending stress pattern in the sidewalls, and the slight increase in vertical diameter observed during the final year of the study, are all indicative of dominant lateral pressure.

The strain gage installations were particularly effective in monitoring both the bending stresses and the direct compressive stresses in the sidewalls resulting from erection and backfilling.

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Discussion

M. G. SPANGLER, Iowa State University, Ames—The research described in this paper constitutes a valuable contribution to the developing arts and sciences pertaining to the design of buried conduits of the flexible type. The authors have made an in-depth study of the performance of the reconstructed Wolf Creek culvert, using modern strain and pressure measuring devices, which they skillfully placed on the structure and in the fill adjacent to it. They have interpreted and presented the results of the measurements in a clear, concise, and logical manner that deserves the close attention of those interested in the structural behavior of buried pipelines. The writer wishes to comment on several features of the study that deserve special emphasis.

First, there is no room for doubt about the nature of the stress situation in the pipe wall at the springline. Figure 5 of the paper shows substantial bending moment stresses in the outer fibers of the plates that developed during erection of the conduit. Then in Figure 6 it is indicated that the stress situation changed from bending moment alone to combined bending moment and direct compressive stress when the fill was completed to the finished height of 83 ft above the top of the pipe. This is an important consideration in connection with the design of bolted longitudinal seams in field-erected structural plate pipes. The bending moment component of the combined stress causes a prying action at the seam that throws the outer row of bolts into direct tension. The stress in the bolts is, therefore, a composite of shear and axial tension, not shear alone.

It is currently a widespread practice to design the bolted seams solely on the basis of the tangential shear at the lapped joint. A more realistic procedure, in the light of both theoretical analysis and the field measurements reported in the paper, would be to design the bolts to carry the composite shear and tensile stress.

Strain gages were placed on the structure at the springline of the pipe and stresses were calculated from the measured strains at this location. Theoretical considerations indicate that the maximum bending moment stresses caused by the fill probably occurred at or near the lower quarter points. This is due to the probability that the bottom reaction on the pipe was distributed over a narrower width than was the load on top. This would cause an unsymmetrical distortion of the moment diagram around

the perimeter of the pipe, and bending stresses would be greater at points in the vicinity of the lower quarter points than at the springline and above. The writer believes that this consideration helps to explain the fact that practically all the failures of bolted seams in the original Wolf Creek culvert (6) occurred in the lower quadrants of the pipe.

A second important fact revealed in this study is the demonstration of the effectiveness of the imperfect ditch method of construction. The authors conclude, on the basis of their measurements and observations, that the load on the structure equaled only about half the dead weight of the prism of soil extending upward from the pipe. This is a truly remarkable result. Others have arrived at similar conclusions, but their observations have been on pipes of considerably smaller diameter. Here is a structure about 18 ft wide that experienced substantial relief in load by the application of this method of construction. Also, the authors were unable to detect any abnormal subsidence of the top of the embankment over the culvert, in spite of the fact that the baled straw placed in the bottom of the imperfect trench compressed about 2 ft or two-thirds of its initial height. This demonstrates the validity of the concept of a plane of equal settlement, above which all horizontal elements in the embankment are postulated to settle equally, both over the conduit and adjacent thereto.

In 1950, the writer (7) developed and published a theory of loads on imperfect ditch and negative projecting conduits. This theory involved a settlement ratio that, while completely rational in the theoretical development, is practically impossible to predict on a rational basis for any proposed culvert. Therefore, it is an empirical factor, usable values of which can best be determined by the observation of actual culverts in service.

There have been relatively few opportunities to observe empirical values of this ratio, but on the basis of limited information, the writer has used and recommended values in the range of -0.3 to -0.5. Although the geometry of the reconstructed Wolf Creek culvert imperfect ditch does not comply exactly with the situation for which the load theory was developed, the basic principle of creating favorable differential settlements between the central prism of soil and the adjacent side prisms is the same, and it is of interest to calculate a load by the theory and compare the result with the present authors' measured loads. The main difference between the actual and the theoretical case is the existence of a 5-ft layer of highly compacted granular soil between the top of the pipe and the bottom of the imperfect ditch where the layers of straw bales were placed, as shown in Figure 1 of the paper.

The following data are used in the load calculation: $H = 83$ ft, $B_c = 18$ ft, $H/B_c = 4.6$, $w = 125$ pcf, $p' = 0.6$, and $r_{sd} = -0.5$. Interpolating between Figures 8 and 9 of Ref. 7, $C_n = 2.9$.
Subsituting in the load formula

$$W_c = c_n w B_c^2$$

$$W_c = 2.9 \times 125 \times 18^2 = 117{,}500 \text{ plf} = 45.3 \text{ psi.}$$

The data in Figure 7 of the paper indicate that the maximum equivalent unifrom vertical earth pressures on the culvert as deduced from the strain gage measurements varied from 48 psi on plate E down to 26 psi on plate F. The average maximum on the six plates on which strain gages were mounted was 34.7 psi. Thus the calculated load using a settlement ratio of -0.5 agrees fairly well with the real situation in this case. The same diagrams in Ref. 7 indicate that the plane of equal settlement is about 50 ft above the top of the conduit, or approximately 33 ft below the embankment grade. This is compatible with the authors' observation that there was no differential settlement of the roadway pavement in the vicinity of the culvert.

A third item of interest to be derived from the data presented in the paper is the determination of the modulus of soil reaction, E', of the sidefill material that surrounded the pipe. In 1941 the writer (8, 9) presented a hypothesis of the system of earth loads acting on flexible conduits and developed an equation for estimating the deflection of such conduits under load. Briefly, this hypothesis is based on the concept that each increment of vertical load on the pipe causes an increment of deflection;

the vertical diameter shortens and the horizontal diameter lengthens. As the horizontal diameter increases, the sides of the pipe push outward against the enveloping soil, mobilizing the passive resistance characteristics of the soil. At the present time this passive resistance pressure appears to be a linear function of the amount of movement of the sides of the pipe and therefore a function of pipe deflection.

The deflection equation is

$$\Delta X = D_1 \frac{KW_c r^3}{EI + 0.061E'r^3}$$

in which

ΔX = pipe deflection, in. (vertical and horizontal deflections are nearly the same);
D_1 = deflection lag factor;
K = bedding factor;
W_c = load on pipe per unit length, pli;
r = mean radius, in.;
E = modulus of elasticity of pipe material, psi;
I = moment of inertia of cross section of unit length of pipe wall, in.^{4} per in.; and
E' = modulus of soil reaction, psi.

This is a rational development, considering the pipe and surrounding soil to be a composite elastic body, except for the deflection lag factor, D_1, which is purely empirical. It appears to vary inversely with the modulus of soil reaction, being greater for weak soils with low values of E'. In this study, the deflection lag factor was essentially 1.0, because of the high quality of the select backfill soil. In fact, the authors state that the vertical diameter of the pipe actually increased slightly after completion of the embankment.

In early experiments carried out in the 1930's, the value of E' was definitely indicated to be a function of the kind and quality of the sidefill soil, but the range of soil types and conditions observed was very limited. Later, as the deflection equation has grown in use, it has become apparent that E' can vary over a very wide range. The writer has attempted to accumulate data on values of E' and to correlate them with soil properties, particularly texture and density. The study provides all the data needed to estimate the value of the passive resistance modulus that prevailed in this installation.

The following values of terms in the deflection equation are available from the data presented in the paper:

ΔX = 1.9 in. (vertical deflection = 2.4 in., horizontal = 1.4 in.)
D_1 = 1.0
K = 0.1 (shaped bedding)
W_c = 7500 pli (average maximum vertical pressure = 34.7 psi)
r = 9 ft = 108 in.
E = 30,000,000 psi
I = 0.225 in.^{4} per in.

Solving for E' yields 6300 psi. This is a high value of the modulus of soil reaction, but is consistent with the nature of the sidefill material, which is described as "crushed granular material of base-course quality." Sieve analysis indicates a well-graded gravel passing a 1½-in. sieve. It was compacted in 6-in. layers and at optimum moisture content to 95 percent of standard AASHO density—a very high quality material.

Since original publication of the deflection formula for flexible pipes, efforts have been made to devise laboratory methods for estimating the modulus of soil reaction for a prospective culvert installation, but without conspicuous success. Probably the greatest progress will be made in this regard by considering E' as a semi-empirical constant whose values may best be determined by observation and measurements on actual culvert installations. The writer has made a number of such determinations and in 1958 published (9) a table showing values of E' for various kinds of soil in vari-

TABLE 1

VALUES OF E′ FOR 18 FLEXIBLE PIPE CULVERTS

Item	Location	Pipe Diam. (in.)	Soil Type[a]	Fill Height (ft)	Mod. of Passive Resist., e (psi/in.)	Value of E′ (er) (psi)
1[b]	Ames, Iowa	42	Loam top soil (U)	15	14	294
2[b]	Ames, Iowa	42	Well-graded gravel (U)	16	32	672
3[b]	Ames, Iowa	36	Sandy clay loam (T)	15	28	502
4[b]	Ames, Iowa	36	Sandy clay loam (U)	15	13	234
5[b]	Ames, Iowa	42	Sandy clay loam (T)	15	25	525
6[b]	Ames, Iowa	42	Sandy clay loam (U)	15	15	315
7[b]	Ames, Iowa	48	Sandy clay loam (T)	15	29	696
8[b]	Ames, Iowa	48	Sandy clay loam (U)	15	14	336
9[b]	Ames, Iowa	60	Sandy clay loam (T)	15	26	780
10[b]	Ames, Iowa	60	Sandy clay loam (U)	15	12	360
11[c]	Chapel Hill, N.C.	30	Sand	12	25	375
12[c]	Chapel Hill, N.C.	31.5	Sand	12	56	882
13[c]	Chapel Hill, N.C.	30	Sand	12	80	1200
14[c]	Chapel Hill, N.C.	20	Sand	12	35	350
15[c]	Chapel Hill, N.C.	21	Sand	12	82	861
16[c]	Culman Co., Ala.	84	Crushed sandstone (C)	137	190	7980
17[c]	McDowell Co., N.C.	66	Clayey sandy silt (C)	170	40	1320
18[d]	Wolf Creek, Mont. (reconstructed)	216	Graded crushed gravel (C)	83	58	6300

[a]U = untamped; T = tamped, C = compacted.
[b]Side pressure and pipe deflections measured.
[c]Side pressures estimated, pipe deflections measured.
[d]Load and pipe deflections measured.

ous states of compaction as deduced from 17 actual culverts. This table may now be extended to include the reconstructed Wolf Creek culvert, as given in Table 1.

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Performance of a Reinforced Concrete Pipe Culvert Under Rock Embankment

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A 48-in. diameter reinforced concrete pipe installation has been studied to evaluate the performance of both types of bedding conditions currently used by the Kentucky Department of Highways. A portion of the pipe was laid using Kentucky's standard B bedding. The remainder of the pipe culvert was laid using the B_1 bedding (imperfect trench); the design height of the fill (36 ft) was sufficient to require the imperfect trench construction. The embankment was primarily of a rock fill material, with the largest particle size limited to a maximum of 3 ft. The portion of the pipe with the standard B bedding exhibited stress in relation to each additional height of embankment placed. For every fill increment, there was a corresponding change in the vertical and horizontal diameters of the pipe and in the pipe distress as evidenced by cracking. The absence of signs of distress in the B_1 bedding portion indicates that the imperfect trench performed its purpose of relieving load on the pipe.

•WHEN taken as a general classification, underground conduits in some form have been used by mankind for at least 3000 years for drainage or water supply purposes. In recent years Interstate highways with their flatter grades and longer radius curves have required the construction of deeper cuts and higher fills. Concrete pipe culverts under these higher fills require either special methods of installation, such as the imperfect trench, or stronger pipe in order to carry the heavier loadings.

An extensive research program on pipe problems was inaugurated at Iowa State College in 1908, and it has been carried on practically continuously since. Through this research, a comprehensive theory widely known as Marston's Theory of Loads on Underground Conduits (1, 2, 3) has been developed.

Conceivably, it would be possible to compute by modern methods of soil mechanics the various factors that affect the load on a conduit, but such a procedure would involve the expenditure of both time and money that would be prohibitive in relation to the total cost of the structure. Therefore, many of these factors may be evaluated by an accumulation of data obtained from measurement and observation of actual installation performances. Many states have specifications or standard plans permitting the use of the imperfect trench as proposed by Marston, or with modifications. Most of these states have adopted the imperfect trench method of construction within the past two decades.

Advancement of the present knowledge of loads on pipe culverts depends in part on an accumulation of field experience and supporting data. One particular pipe has been chosen for this special study because of its installation conditions and because it may add to the knowledge of the performance of the imperfect trench. This pipe has been

Paper sponsored by Committee on Culverts and Culvert Pipe and presented at the 48th Annual Meeting.

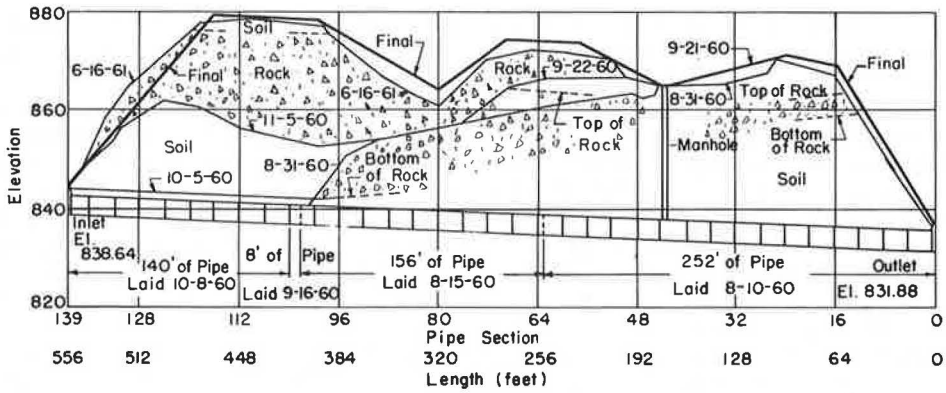


Figure 1. Fill profiles.

constructed using both type beddings that were specified in the 1956 edition of the "Kentucky Department of Highways Standard Specifications for Road and Bridge Construction" and amendments thereto (see Appendix A). Observation of the performance of this installation during and after construction was desirable to check the validity of the imperfect trench theory as applied to rockfill material, which was used in this particular installation.

PIPE INSTALLATION AND PERFORMANCE

A 48-in. diameter pipe installation on Interstate 75 near Georgetown, Kentucky, was selected for special study. This study was inaugurated to take advantage of an opportunity to observe at one location the performance of both bedding conditions as used in Kentucky. Distress was first noted in the pipe in the latter part of August 1960, before the full design length of pipe (560 ft, 140 sections) had been laid. At that time, the fill had not reached full design height (36 ft) and the Class III pipe had hardly been in place a month. This first portion of the pipe was laid using Kentucky's standard B bedding; after a review and check of plans and specifications revealed that Class III pipe with B bedding was an inadequate design, the bedding for the remainder of the pipe was changed to B_1 (imperfect trench) (Fig. 4, Appendix A). Kentucky's standard pipe classes are the same as ASTM designations with reference to strength requirements. Pipe used in this installation was selected from lots accepted as adequate with respect to 3-edge bearing test results.

A total of 102 four-foot pipe sections manufactured on different dates, from a design length of 140 sections, was placed with standard B bedding before the change. Plans specified the standard B bedding and it must be assumed that this was obtained. Field data substantiate that the backfill depth requirement above rock (Fig. 3, Appendix A) was obtained before placing the pipe; therefore, one of the factors considered to be a major contributing cause to pipe failure—ledge rock in the foundation too close to the bottom of the pipe—is nonexistent at this location.

For the remaining 37 sections actually placed, a B_1 bedding was used. At least 3 ft of unsuitable material was removed from the foundation area below the bottom of the pipe before the backfill was placed (Fig. 5, Appendix B); the resident engineer estimated a 7-ft soil foundation for camber calculations (4). Plans placed the backfill in approximately 1-ft lifts and a rubber-tire dozer was used to compact it. A sand cushion bedding was prepared in a satisfactory manner.

The pipe was laid in the sand cushion bedding (Fig. 8, Appendix B) and the backfill brought up uniformly in layers not exceeding 6 in. to a minimum height of 18 in. (0.30 times the outside pipe diameter). Each layer was compacted thoroughly by single-action pneumatic tampers under the haunches of the pipe to insure that the backfill was in intimate contact with the sides of the pipe. The embankment was then extended

upward in a normal manner to such an elevation that there was a soil cover equal to the overall height of the pipe plus 12 in. in preparation for construction of the imperfect trench (Fig. 9, Appendix B).

A trench equal in width and height to the outside width and height of the pipe was dug in the compacted embankment with a backhoe (Fig. 10, Appendix B). At this time the alignment of the pipe and the trench were noted to be off approximately 1 ft in some places; this was corrected by refilling and compacting the trench in 6-in. layers and redigging the trench over the centerline of the pipe. The trench was then filled with loose straw to a depth of approximately 24 in. Loose backfill in the remainder of the trench compressed the straw to a thickness of about 16 in. The trench was then covered and bridged by a 2-ft layer of compacted soil to a width of 20 ft on each side of the pipe (Fig. 11, Appendix B).

By this time it was late in the season and construction equipment had been moved from the general construction area. Consequently, the rockfill over the pipe was not completed until the spring of 1961. However, a small volume of earth fill (Fig. 1) was placed the last part of the year as an approach for construction of a nearby overpass pier.

The embankment material placed in 1960, other than that used in the immediate backfill near the pipe and imperfect trench, consisted for the most part of rock. With the placement of the rock material (Figs. 1 and 2) during the latter part of August 1960 over the 102 sections laid with standard B bedding, there developed a series of hairline cracks. A survey of pipe conditions revealed that nearly every section, beginning at section 13 through section 71, had at least one hairline crack in the top and bottom by the last of August. Hairline cracks were visible in section 13 through section 86 by the last of September, with a progressive increase in the size and number of cracks in sections 33 to 76 since the survey made the last part of August.

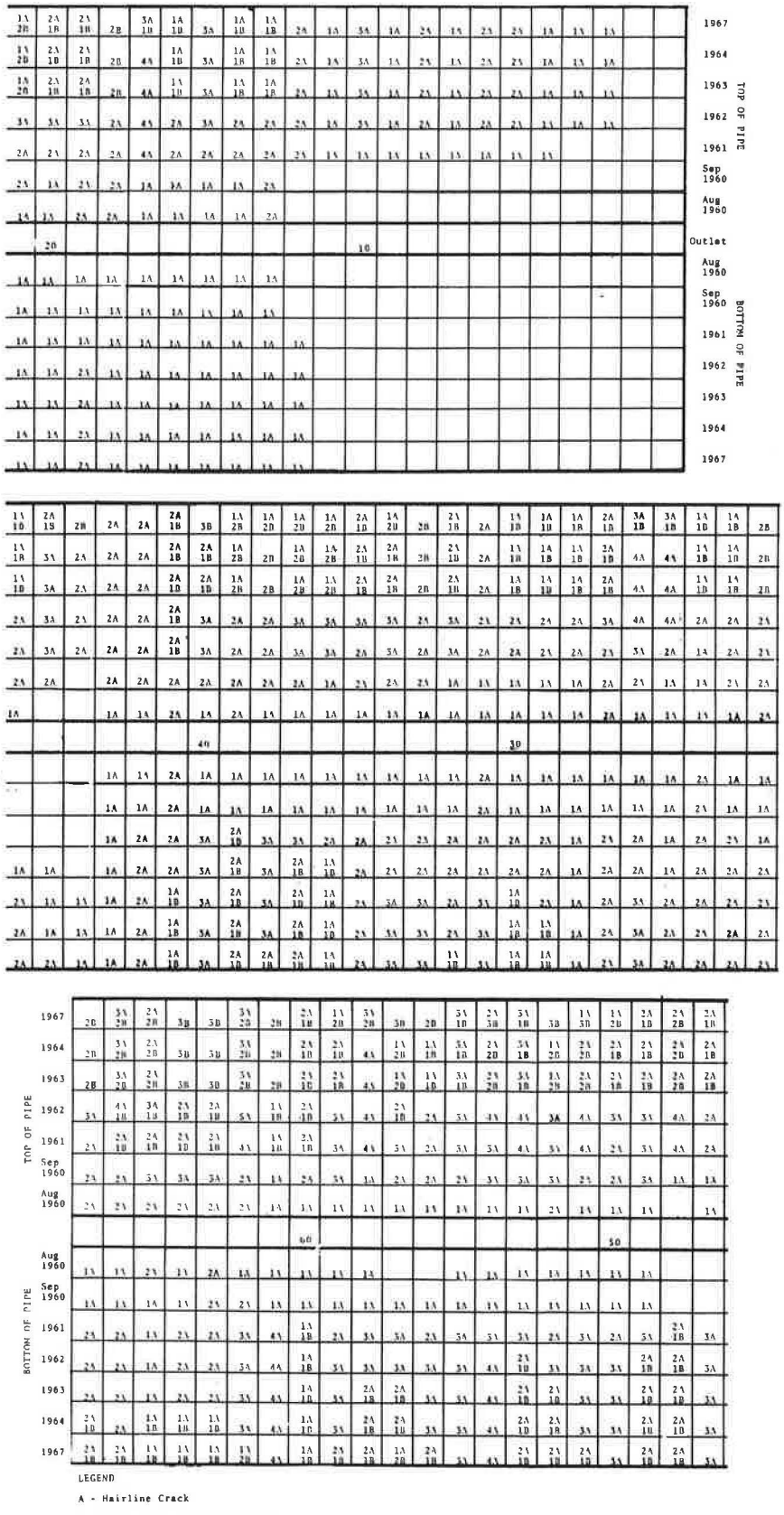
In January 1961 the sections constructed with standard B bedding still exhibited progressive development of hairline cracks under the partial fill load but seemed to be approaching an equilibrium condition with respect to the load. By May 1961 the approximate maximum load appeared to have been reached, since progressive distress was no longer noted in the form of increased size or number of cracks.

Construction of the rock embankment over the pipe was completed the last of May 1961. The additional load on the pipe sections with standard B bedding resulted in a renewed increase of signs of distress. The amount of additional fill may be seen in Figure 1. Increased distress in the B bedding sections due to the additional embankment was most noticeable in sections 75 through 100. The sections with B_1 bedding, now with nearly full embankment design height, remained in good condition and exhibited no distress.

Changes in the inside vertical and horizontal pipe diameters were measured at established checkpoints located in selected sections as the embankment load developed. The ends of the initial vertical and horizontal diameters were marked by punch-marks in the inner surface of the pipe structure. All subsequent measurements were made at these same points. Changes of the vertical and horizontal dimensions measured were less than 1 percent, a change sometimes considered sufficient to cause materially injurious cracks in rigid pipes (5). The maximum change measured over the observational period occurred in section 63 and was only ¼-in. vertical and ⅜-in. horizontal. The pipe diameter changes in section 38 were ⅛-in. vertical decrease and ¼-in. horizontal increase.

RESULTS AND CONCLUSIONS

The development of distress in the sections laid with standard B bedding can be attributed to the 36-ft high fill constructed over the pipe. Formation of a hairline crack, which has been defined as a crack 0.01 in. or less, does not constitute a pipe failure; nevertheless, the existence of hairline cracks was an indication of pipe overload. The 102 sections with standard B bedding have been overloaded and distress may progress to a point that will constitute a failure, but observations to date indicate that equilibrium is being approached. Cracks as wide as ¼ in. were often observed.

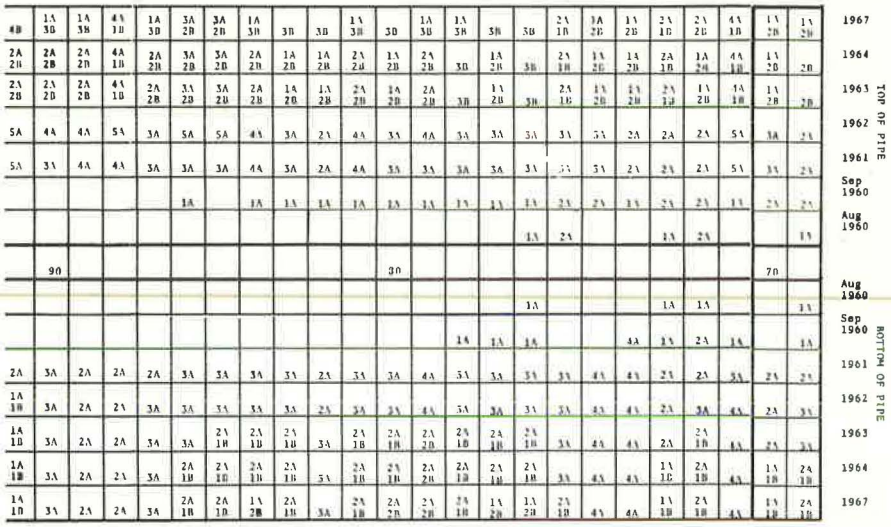


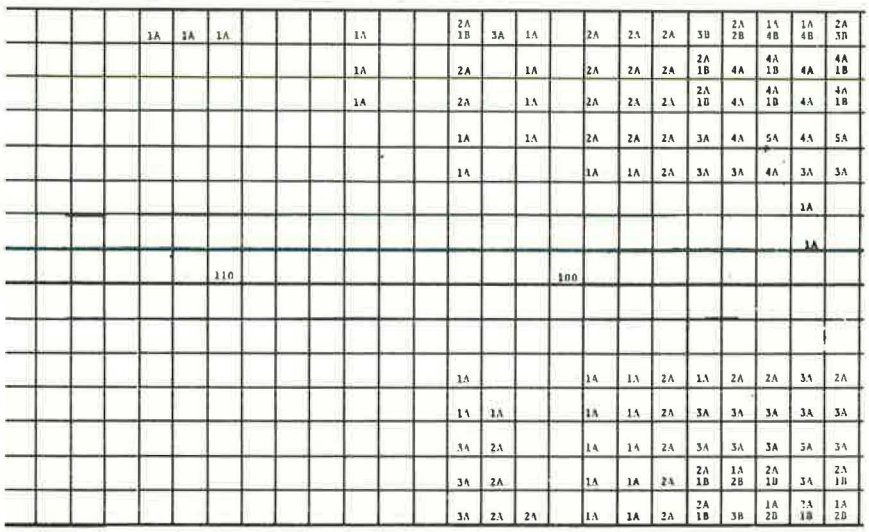
LEGEND

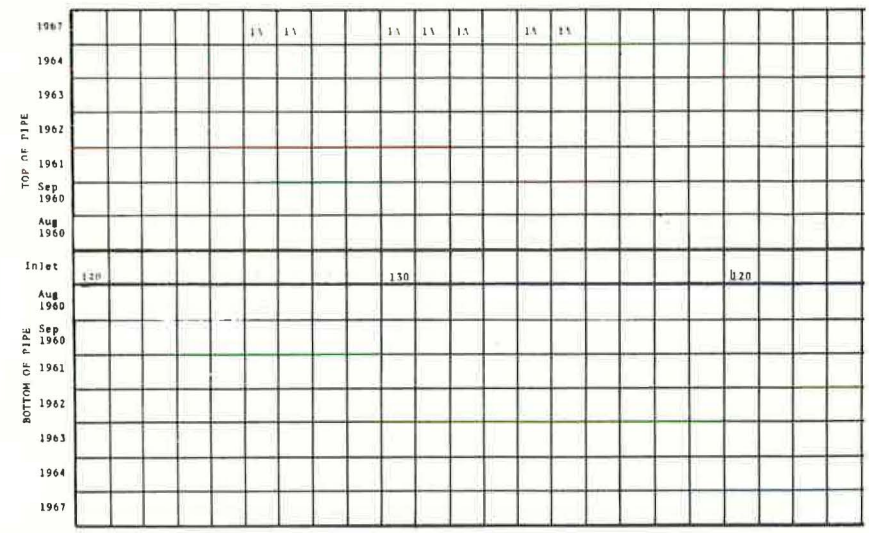
A - Hairline Crack

B - Crack (0.01 inch or Greater)

Figure 2. Summary of pipe distress.







LEGEND

A - Hairline Crack

B - Crack (0.01 inch or Greater)

Figure. 2 (continued).

No indications of autogenous healing of cracks have been noted. Field data do indicate a significant difference in performance of pipe placed with standard B bedding and B_1 bedding (imperfrect trench).

Compaction under the haunches of the pipe to insure development of lateral support and prevent settlement of the embankment material in the exterior prisms is most important. Care in compacting the backfill area is important since the stiffer the side supporting material is, the greater will be the tendency of the mass directly above the conduit to arch over the adjacent bodies, thus relieving the top of the structure from part of the overburden pressure.

Specifications now used by the Kentucky Department of Highways give requirements pertaining to the nature of the side supporting material, method of placing the side supporting material, and size of the surrounding mass. The requirements now used appear to be satisfactory, but control presents a problem. Inspection should be continuous during construction to verify that the desired results are being obtained. Pipe bedding must be obtained that is the same as that assumed in design; otherwise, there is no criterion on which to base design methods.

The compressibility of the imperfect trench material must be greater than that of the adjacent compacted mass and should be in the loosest possible state when placed in the trench. The lower one-third of the excavated trench should be filled with loose hay or straw, even when the soil is very compressible, to insure settlement of the interior prism. Settlement of the interior prism was not noticeable at the surface of the embankment at the test site at any time during the observational period.

When placing the last of the rockfill material in May of 1961, hairline cracks formed in the portion with standard B bedding right up to sections with B_1 bedding. The standard B bedding portion exhibited distress in relation to the additional embankment added. For every fill increment there was a corresponding change in the length of the vertical and horizontal diameters of the pipe and in pipe distress as the load developed. The sections that had complete design fill heights over them before May, and required no additional embankment material, remained in an equilibrium condition of distress. This was expected to be true but could not be verified until this time. The performance of this pipe installation was successful in proving the value of B_1 bedding over the standard B bedding for construction of high fills under rock embankments. Previous work had assumed this to be true but actual verification was desired. The formation of cracks right up to the B_1 bedding portion suggests that the imperfect trench performed its purpose of relieving load on the pipe. Therefore, the rock embankment material appears to act much as soil material does in arching over the pipe.

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Appendix A

KENTUCKY DEPARTMENT OF HIGHWAYS STANDARD DRAWINGS SHOWING PIPE BEDDING DETAILS AND ALLOWABLE FILL HEIGHTS

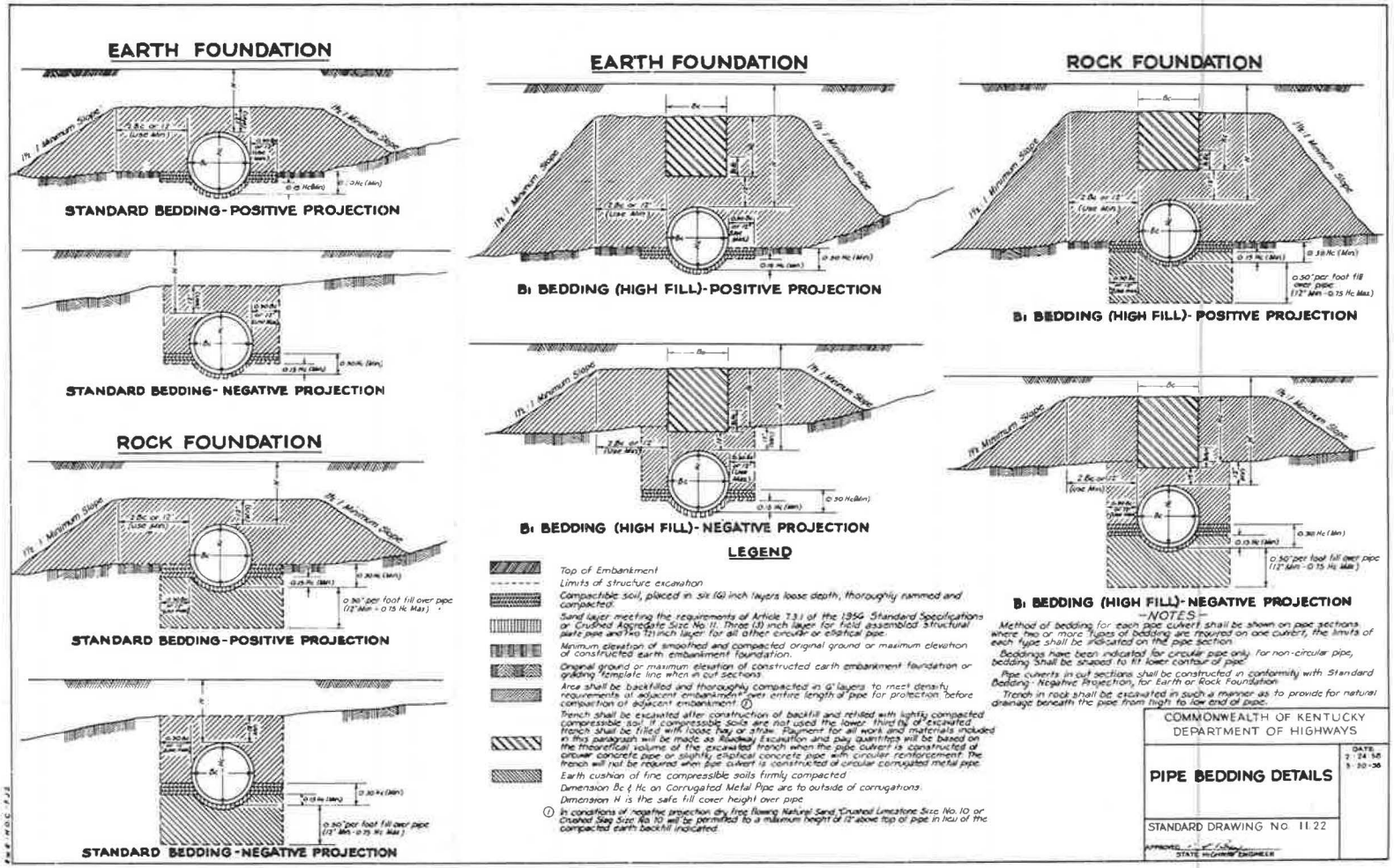

Figure 3. Kentucky pipe bedding details.

TABLE FOR SAFE FILL COVER HEIGHTS AND GAGES FOR CORRUGATED METAL CIRCULAR PIPE

SIZE OF PIPE	Height of Fill Cover Over Top of Pipe (Dimension H of Standard Drawing No. 11.22)											
	Standard Bedding			B₁ (High Fill) Bedding								
	1.5' to 10'	11' to 15'	16' to 20'	21' to 25'	26' to 30'	31' to 35'	36' to 40'	41' to 45'	46' to 50'	51' to 55'	56' to 60'	61' to 65'
	GAGE											
15"	16	16	16	16	16							
18"	16	16	16	16	16							
24"	14	14	14	14	14	14	14	14	12	12	12	12
30"	14	14	14	14	14	12	12	12	10	10	10	10
36"	12	12	12	12	12	10	10	10	8	8	8	8
42"	12	12	12	12	10	10	8	8	8	8	8	8
48"	12	12	12	10	8	8	8	8	8	8	8	8
54"	12	12	10	8	8	8	8	8	8	8	8	8
60"	10	10	8	8	8	8	8	8	8	8	8	8
66"	10	10	8	8	8	8	8	8	8	10	8	8
72"	10	8	8	8	8	8	8	8	8	8	8	7
78"	8	8	8	12	12	12	10	10	8	8	8	5
84"	8	12	12	12	12	12	10	10	8	8	7	5

Factory Riveted Corrugated Metal Pipe - 5% Ellipsed - Held with wire struts

Field Assembled Structural Plate Pipe, Factory Fabricated with 5% Ellipse

CORRUGATED METAL PIPE ARCH

Nominal End Area (Sq Ft)	Span (Inches)	Rise (Inches)	Gage	Diameter of Circular Pipe of Equal Periphery (Inches)	Dimension "B" (Inches) ②	Minimum Radius of Curvature (Inches)
1.1	18	11	16	15	3.5	2.5
1.6	22	13	16	18	4	3
2.8	29	18	14	24	5.5	4
4.4	36	22	14	30	6.5	4.5
6.4	43	27	12	36	8	5.5
8.7	50	31	12	42	9	6.5
11.4	58	36	12	48	10.5	7.5
14.3	65	40	12	54	12	8.5
17.6	72	44	10	60	13	9.5

② Dimension "B" is the vertical distance from the lowest portion of the base to a horizontal line drawn across the widest portion of the arch. Permissible variations in dimension "B" of plus one inch for 18" x 11" and 22" x 13" pipe arch and plus or minus one inch for 29" x 18" to 72" x 44" pipe arch. Permissible variation in span or rise of one inch for 18" x 11" to 36" x 22" pipe arch and two inches for 43" x 27" to 72" x 44" pipe arch.

TABLE FOR SAFE FILL COVER HEIGHTS AND CLASSES FOR REINFORCED CONCRETE CIRCULAR PIPE.

SIZE OF PIPE	Height of Fill Cover Over Top of Pipe (Dimension H of Standard Drawing #22)			
	Standard Bedding	B₁ (High Fill) Bedding		
	1.5' to 20'	21' to 30'	31' to 40'	41' to 65'
	CLASS OF PIPE			
15"	III	III		
18"	II	II		
24"	II	II	II	II
30"	II	II	II	II
36"	II	III	III	II
42"	II	III	III	II
48"	II	II	III	II
54"	III	II	III	II
60"	II	III	III	II
66"	III	III	III	II
72"	III	III	III	II
78"	III	III	III	II
84"	III	III	III	II

GENERAL NOTES

See Division V Sections 11 & 12 of the 1956 Standard Specifications as amended for more detailed information as to construction, basis for payment, etc.

When fill cover height over circular pipe, including slightly elliptical concrete pipe with circular reinforcement, is 30 feet or more the minimum size of pipe shall be 24 inches.

The maximum safe fill cover height for non-circular pipe, excluding slightly elliptical concrete pipe with circular reinforcement, is 15 feet.

Pipe shown within the shaded portion of the chart for "Table for Safe Fill Cover Heights and Gages for Corrugated Metal Circular Pipe" require a 5% ellipse. Both inlet and outlet ends of the ellipsed pipe shall be fabricated circular. Transition from the circular ends to the ellipsed portion of the pipe shall be accomplished in a distance of not more than 18 feet.

Reinforced Concrete Elliptical Pipe (exclusive of slightly elliptical concrete pipe with circular reinforcement) shall meet the same strength requirements specified for Class III Reinforced Concrete Circular Pipe of equivalent nominal end area.

REINFORCED CONCRETE ELLIPTICAL PIPE

Nominal End Area (Sq Ft)	Span (Inches)	Rise (Inches)	Wall Thickness (Inches)	Nominal Weight (Pounds per Lineal foot)	1.0 Round Pipe Equivalent (Inches)
1.8	23	14	2½	195	18
3.3	30	19	3¼	300	24
4.1	34	22	3½	365	27
5.1	38	24	3¾	430	30
6.3	42	27	3¾	475	33
7.4	45	29	4¼	625	36
8.8	49	32	4¾	720	39
10.2	53	34	5	815	42
12.9	60	38	5⅝	1000	48
16.6	68	43	6	1235	54

① Includes Slightly Elliptical Concrete Pipe with Circular Reinforcement.

② Excludes Slightly Elliptical Concrete Pipe with Circular Reinforcement.

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS

FILL COVER HEIGHTS GAGES AND DIMENSIONS FOR ① CIRCULAR PIPE AND ② NON CIRCULAR PIPE

DATE 2-24-58

STANDARD DRAWING NO. 11.23

APPROVED [illegible] STATE HIGHWAY ENGINEER

Figure 4. Kentucky allowable fill heights.

Appendix B

PHOTOGRAPHIC SEQUENCE OF CONSTRUCTION OF PIPE INSTALLED WITH B_1 BEDDING (IMPERFECT TRENCH)

Figure 5. Pipe bedding prepared: Undesirable material removed and replaced with selected compacted backfill.

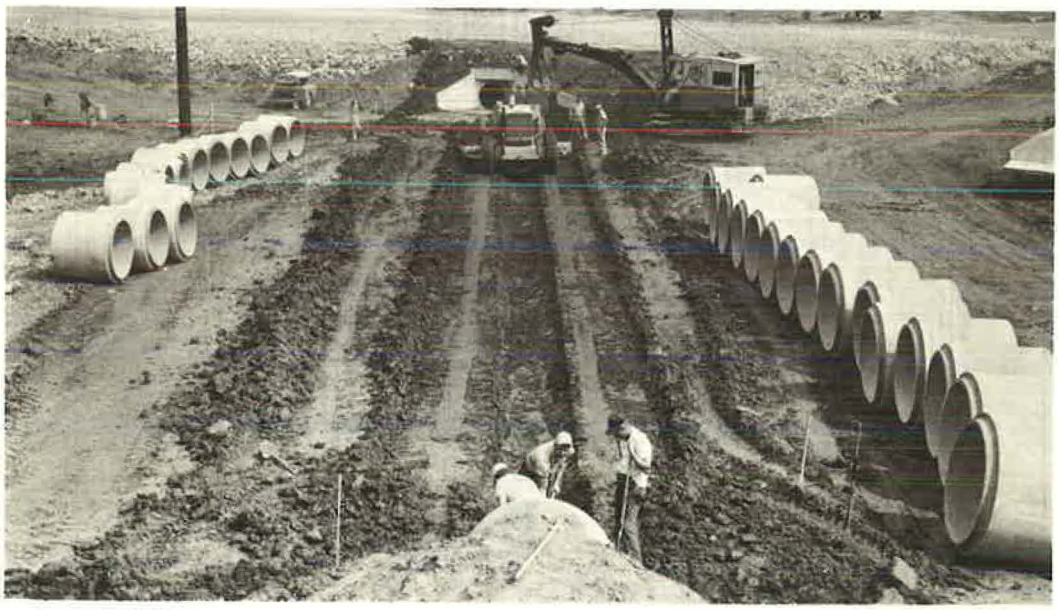

Figure 6. Patrol grader with special blade preparing soil for laying pipe.

Figure 7. Close-up of special blade on patrol grader.

Figure 8. Placing pipe in sand cushion bedding.

Figure 9. Placing backfill before construction of the imperfect trench (when specified).

Figure 10. Construction of the imperfect trench (when specified).

Figure 11. Loose backfilling of imperfect trench: Straw placed to a depth of one-third trench depth.

Figure 12. Completion of the imperfect trench: Embankment placed 2 ft above top of trench.

Figure 13. Embankment material placed over culvert: Line between soil and rock material shown.

Discussion

M. G. SPANGLER, Iowa State University, Ames—John Tyndal, a famous British physicist of the 19th century, once said: "The brightest flashes in the world of thought are incomplete until they have been proved to have their counterparts in the world of fact." In other words, a theory, to be proved valid, must yield results that agree with the facts of a situation.

The writer has had a number of opportunities to compare the results obtained by the Marston theory of loads on underground conduits with the structural performance of pipe culverts under earth embankments, and in all cases it has been demonstrated that the theory yields reasonably good and reliable results. However, opportunities to judge the applicability of the theory to pipe culverts under rock embankments are relatively rare. Mr. Deen's paper is a welcome addition to the literature because it provides an opportunity for a comparison of this kind.

The subject culvert was constructed in accordance with two of the Marston classifications relative to installation environment. The downstream 408 ft (102 four-foot sections) was installed as a positive projecting conduit. This portion of the pipeline suffered extensive damage in the form of longitudinal cracks, some as wide as ¼ in., and deflections of the pipes were excessive. The remaining 38 pipe sections at the upstream end of the culvert were installed by the imperfect ditch method of construction, and, although the height of fill over this portion of the structure was slightly greater than on the downstream section, these pipes did not suffer any structural damage or distress.

It is of interest to calculate loads and supporting strengths of the pipes in the two installation environments (7). For the downstream section load calculation, $H = 35$ ft, $B_c = 4.67$, $H/B_c = 35/4.67 = 7.5$. Assume $p = 0.7$, $r_{sd} = +0.7$, $r_{sd}p = +0.5$, $w = 120$ pcf, $C_c = 11.25$. Then

$$W_c = 11.25 \times 120 \times (4.67)^2 = 29{,}500 \text{ plf}$$

For calculation of supporting strength, assume $K = 0.33$, $m = 0.7$, $x = 0.594$, and class B bedding $N = 0.707$. Then

$$q = \frac{0.7 \times 0.33}{11.25}(7.5 + 0.35) = 0.161$$

$$\text{Load factor} = \frac{1.43}{0.707 - (0.594 \times 0.161)} = 2.34$$

$$\text{Required 3-edge bearing strength} = \frac{29{,}500}{2.34} = 12{,}600 \text{ plf}$$

$$\text{Required D-load strength} = \frac{12{,}600}{4} = 3{,}150\text{D}$$

The pipe used was Class III, which has a D-load strength at 0.01-in. crack of 1,350D. Thus it is indicated that the theoretical load on the pipe was about 2⅓ times the minimum strength of the pipe. This accounts for the extensive damage in the form of longitudinal cracking in this portion of the culvert.

Similar computations of load and supporting strength for the imperfect ditch portion of the culvert have been made. For the load calculation, $H = 38$ ft, $B_c = 4.67$, and $H/B = 8.14$. Assume $p' = 1.0$, $r_{sd} = -0.5$, $C_n = 4.8$, and $w = 120$ pcf. Then

$$W_c = 4.8 \times 120 \times (4.67)^2 = 12{,}600 \text{ plf}$$

For calculation of supporting strength, assume K = 0.33, m = 0.7, x = 0.594, and Class B bedding N = 0.707. Then

$$q = \frac{0.7 \times 0.33}{4.8} (0.14 + 0.35) = 0.408$$

$$\text{Load factor} = \frac{1.43}{0.707 - (0.594 \times 0.408)} = 3.08$$

$$\text{Required 3-edge bearing strength} = \frac{12,600}{3.08} = 4,100 \text{ plf}$$

$$\text{Required D-load strength} = \frac{4,100}{4} = 1,025\text{D}$$

It is indicated that the theoretical required strength of pipe in this section of the culvert is only about three-fourths of the minimum strength of the pipe at 0.01-in. crack. This result agrees with the observed fact that the pipes installed by the imperfect ditch method did not crack.

The comparison between theoretical and actual performance of the two portions of this culvert also indicates that the Marston theory is applicable to rock embankments as well as to those constructed of soil. About 40 years ago the writer (6) investigated the structural performance of three cast iron pipe culverts under rock embankments in northeastern Iowa and arrived at the following conclusion, among others: "There is every evidence that the rock embankments acted in a manner entirely analogous to the action of ordinary earth embankments." These observations are in complete harmony with the conclusion: "Therefore, the rock embankment material appears to act much as soil material does in arching over the pipe."

References

6. Spangler, M. G. Investigation of Loads on Three Cast Iron Pipe Culverts Under Rock Fills. Iowa Eng. Expt. Station Bull. 104, 1931.
7. Spangler, M. G. Soil Engineering, 2nd ed. International Textbook Co., Scranton, Pa., 1960.

Culvert Design in Some European Countries

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Current culvert design practices in several European countries are discussed; these practices include the hydraulic and structural aspects as well as materials used, installation procedures, and durability considerations. The majority of the culverts in Europe are constructed of prefabricated unreinforced and reinforced concrete sections. The design discharge is determined largely by empirical methods, and the culvert dimensions are often obtained from charts supplied by manufacturers. Except in the Soviet Union, structural design of culverts is based primarily on the Marston-Spangler theory; however, extensive research is under way in an effort to establish a more rational basis for structural calculations. Concrete bedding for culverts is generally preferred and frequently used. Provided that the necessary protective measures are taken, the durability of concrete culverts is not of great concern in Europe; properly applied bituminous coatings have been found to be very effective in reducing corrosion of steel culverts. The durability of a steel culvert has been observed to be influenced by the frequency and intensity of applied dynamic loads.

•IN RECENT years, both researchers and practicing engineers in the United States and abroad have been directing more attention to culvert design procedures. The impetus for this increasing interest lies in the introduction of new materials, the complete change in traffic loads, the improvement of construction methods, the economic advantage of culverts as compared to bridges, etc., all of which call for an extensive research effort with the aim of overcoming the technological and economic problems brought about by these new conditions. A number of European countries, including England, West and East Germany, the Soviet Union, Poland, and Czechoslovakia, have started research work in this field and have developed various standards, specifications, and criteria for the design of highway culverts. It is the purpose of this paper to report the current state of the art and the trends in culvert design in these countries.

GENERAL CONSIDERATIONS

Procedure

The culvert design procedure is controlled by different regulations that in general can be classified as national standards and individual design.

National Standards—National standards are issued to specify the quality of materials and the size and quality of prefabricated elements to be used in culvert construction.

Individual Design—The individual design of culverts must follow certain regulations and instructions stated in national specifications (1) or design codes (2). These instructions give details as to the materials to be used, quality control, and safety regulations; in certain countries (Poland, the Soviet Union, and Hungary), the methods of structural calculations are also specified. In order to standardize and simplify the design of culverts, consulting firms (3) and state agencies have prepared "standard projects" for highway culverts (4, 5). Although many of these are "suggested" standard

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projects, the Soviet standard projects are "compulsory," i.e., state-owned design offices are bound to adopt these projects if actual conditions permit their use.

Materials

The usual materials used in culvert construction are unreinforced concrete, reinforced concrete, steel, cast iron, and plastic; in addition, stone culverts are often constructed in the Soviet Union. The most common culvert material is concrete; although no official statistics are available, it is believed that more than 85 percent of the culverts in Europe are built of unreinforced and reinforced concrete. Steel culverts are seldom used; in particular, they are strictly prohibited in many eastern European countries because of the shortage in steel. The use of plastic culverts is still in the experimental stage.

Unreinforced Concrete—Unreinforced concrete is used primarily for prefabricated elements. In recent years, concrete pipes manufactured by special technological processes, such as centrifugal pipes (Germany and England) and compressed pipes (Denmark), have gained a considerable portion of the market because their overall performance characteristics are better than those of regular concrete pipes. The size, shape, and quality of these pipes are specified by national standards. In general, circular and oval elements are prefabricated (5, 6) in lengths of 1 meter. Their use is restricted to relatively simple situations where the loading conditions are favorable; the permissible loads are specified in the standards.

Reinforced Concrete—Reinforced concrete is used for individually designed cast-in-place culverts and for prefabricated culvert elements with normal or prestressed reinforcement. In recent years, prestressed concrete and centrifugal reinforced concrete pipes have gained in popularity because of their economy. The size, shape, and quality of these pipes are specified by standards (7). In general, circular sections with lengths of 4 to 5 meters are used; in the case of prestressed concrete elements, only circular sections are used. These pipes are designed to resist outside pressures, but they are not reinforced for inside pressures. In Germany, Denmark, Sweden, and Switzerland, so-called prestressed reinforced concrete pressure pipes are manufactured in lengths of 8 meters with maximum diameters of 1 meter; these pipes can resist inside pressure, but they are rarely used for culverts. Cast-in-place culverts are individually designed according to specifications, and they are constructed with different shapes, including circular, oval, crescent, lemon, parabolic, quadratic, and rectangular cross sections.

In western Europe (England, Germany, France, Denmark, etc.), culverts built of prefabricated elements (mostly reinforced concrete sections) are preferred to cast-in-place culverts, which are designed only if local conditions do not permit the use of standard elements. As a rough estimate, about 80 percent of the concrete culverts are built of prefabricated elements; this value is obtained by proportioning the cost of prefabricated culverts to the total cost of concrete culverts. In the Soviet Union, Poland, Hungary, and Czechoslovakia, however, this percentage is somewhat lower, probably about 70 percent; this can be explained by the fact that the requirement for prefabricated culvert elements is far beyond the number produced. Efforts are being made to increase the production of prefabricated elements and to build cast-in-place culverts only if it is absolutely necessary. In the Soviet Union, for instance, highway culverts are often built of elements prefabricated at the construction site. In order to maintain the required quality of construction, the field-prefabricated reinforced concrete elements are manufactured according to standard project specifications. As an example, the drawings for a standard circular culvert are shown in Figure 1; this standard culvert is constructed with diameters of 0.50, 0.75, 1.00, 1.25, and 1.50 meters.

Steel—Although steel culverts are highly elastic and have considerable structural strength, their use is limited because of their sensitivity to corrosion and abrasion; when they are used, it is necessary to apply a protective coating to resist these phenomena. The most common coating (8) is three layers of bitumen with two intermediate layers of asbestos tape. The total thickness of this protective layer is about 2 to 3 inches. Experimental work is currently being conducted in most European countries

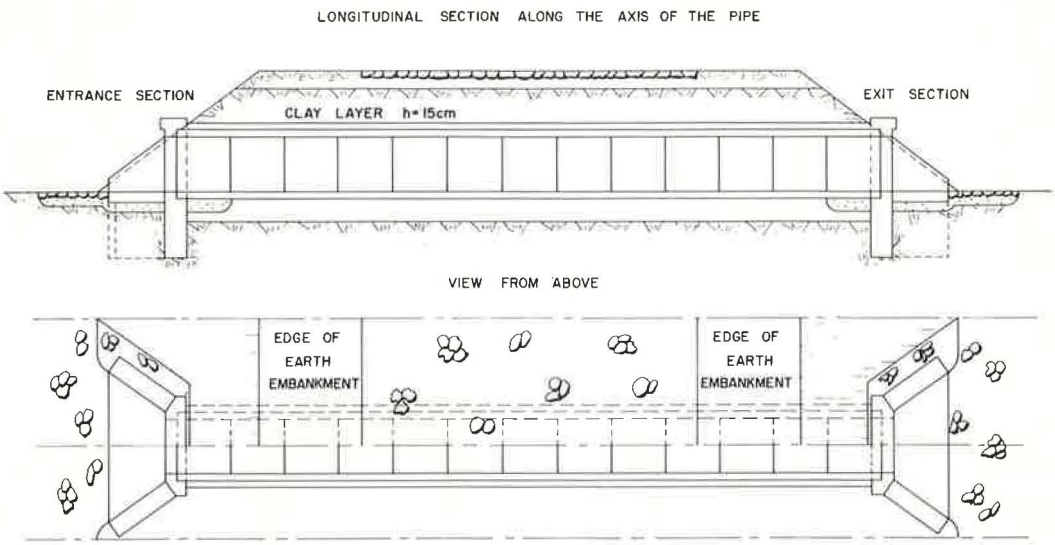

Figure 1. Typical standard reinforced concrete culvert.

to study the feasibility of replacing the bituminous coating with plastic sheets glued to the pipe by plastic glue; however, this latter process is presently too expensive.

In western European countries, corrugated steel is used for culvert construction (8) more often than in eastern European countries. After the corrugated plates are bent to the required shape, a zinc coating is applied to provide corrosion protection; the minimum amount of zinc to be applied is 500 grams per square meter (8, 9). Recently, an additional layer of epoxy-resin or plastic has sometimes been applied to provide more protection. The pipe cross section is usually circular or elliptical, with a maximum diameter of 6.50 meters, or frogmouth-shaped, with a maximum span of 8 meters. The thickness of the corrugated steel plates varies from 2.7 to 7.0 millimeters, depending on the loading conditions, and the plates are manufactured in different lengths (from 1.93 to 2.54 meters) and widths (from 0.85 to 1.83 meters). Corrugated steel plates are often used to repair old brick or stone arch culverts by replacing or strengthening the vault.

It might be of interest to mention that more than 5,000 corrugated steel plate culverts were built in Russia from 1887 to 1914 (10). In 1915, however, serious failures occurred along the Ohrenburg-Taskent railroad, and it was decided to abandon completely the use of corrugated steel plates for culvert construction. Later it was discovered that these failures had been caused by negligence on the part of the contractor and by errors in the structural calculations. In 1950 the use of corrugated steel plates for culvert construction was reconsidered, and the Design Institute for Transportation in Leningrad prepared a set of drawings, as shown in Figure 2, for corrugated steel culverts with diameters of 1.00, 1.25, and 1.50 meters to be used under embankments with heights of 12 meters and less; no information is available regarding the use of these culverts.

Plastic—The plastic pipes manufactured in Germany, Holland, France, England, and the Soviet Union consist mainly of polyvinylchloride (PVC) or polyethylene (PE). Because of the limited strength and the relatively high cost of plastic pipes, their use as culverts is not promising for the time being. In Germany, however, a new kind of plastic pipe, the so-called Wickelrohr, will soon be available in diameters up to 1.6 meters. There is hope that this plastic pipe will be economically and structurally competitive with currently used concrete pipe.

In the Soviet Union, attempts have been made to introduce plastic materials into culvert construction as "plastic concrete" (11), which is a mixture of sand (80 to 90 percent), furfurol-acetone monomer (6 to 20 percent), hardening agent (benzo-sulphuric

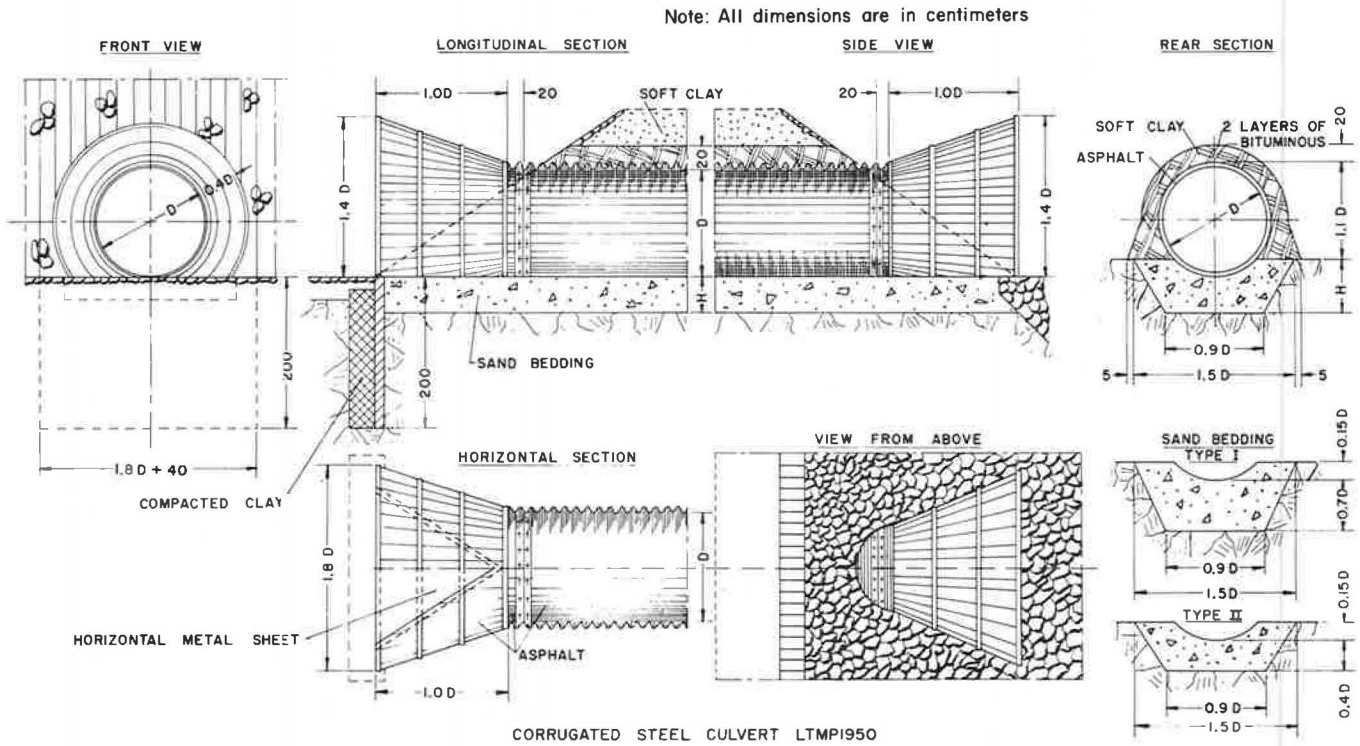

Figure 2. Typical standard corrugated steel culvert.

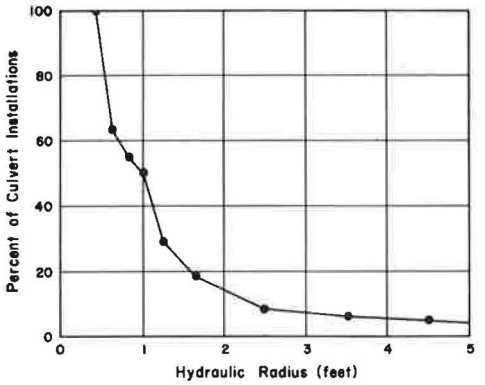

Figure 3. Distribution of culverts according to size.

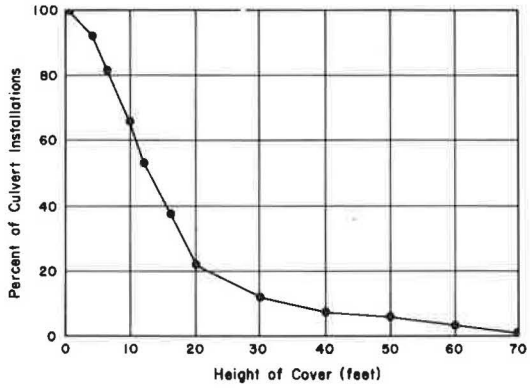

Figure 4. Distribution of culverts according to height of cover.

acid, 2.5 to 3.5 percent), and furfurol (0.2 to 1.5 percent). Experimental culverts with circular cross sections 1 meter in diameter were built, and simultaneous laboratory and field experiments were conducted to compare the plastic concrete culvert with an unreinforced concrete culvert. According to the results of these experiments, the plastic concrete culverts were superior in every respect to the concrete culverts. The only disadvantage of the plastic concrete appeared to be its sensitivity to water. If the mixture is allowed to absorb water, the hardening process is slowed down considerably, or the plastic concrete does not harden at all; this, of course, seriously limits its use as a culvert material.

Miscellaneous—Cast iron and stone were the most common culvert materials before the introduction of plain and reinforced concrete. Although these materials have not been used in most European countries since the mid-1910's, stone culverts were still built to a limited extent in the Soviet Union in the late 1940's.

Distribution

In order to compare culverts with different cross-sectional shapes, their hydraulic radius, defined as the cross-sectional area divided by the wetted perimeter, was used as a characteristic dimension, and data from several European countries (Russia, Germany, Hungary, Czechoslovakia, etc.) are plotted in Figure 3. As can be seen, culverts with a hydraulic radius of 1 foot or less represent about 50 percent of the total number of culverts; this hydraulic radius corresponds to a diameter of 4 feet. Approximately 85 percent of the culverts have a hydraulic radius of 2 feet or less, corresponding to a diameter of 8 feet. Statistical data on the height of cover above the culvert are shown in Figure 4. Approximately 50 percent of the culverts have a height of cover less than 12.5 feet and only about 10 percent have a height of cover greater than 35 feet. Although culverts with various different shapes (oval, crescent, lemon, parabolic, etc.) have been built, circular ones are most common; statistical data indicate that 95 percent of the culverts built in the last 15 years have circular cross sections.

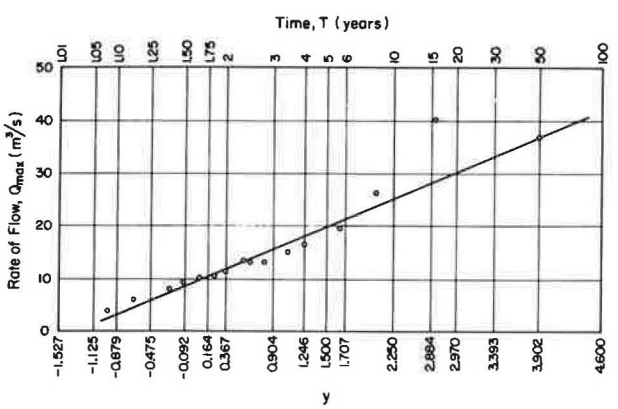

Figure 5. Peak flows of the Bühlot River (West Germany).

HYDRAULIC DESIGN

The basic problem in the hydraulic design of culverts is to determine the design discharge. Because of the complexity of this problem and the lack of experimental data, there is no generally accepted method for making this determination. Usually, the local highway department specifies the spacing of culverts along a highway and the frequency of the flood the culverts must be designed to handle. In the eastern European countries (Hungary, the Soviet Union, Czechoslovakia, etc.), "suggested" standards are available to determine the design discharge; however, these standards are based on formulas that are practically the same as those used in other European countries. As soon as the frequency of the design discharge is specified (for example, the maximum discharge occurring once in five years), the calculation of its actual value depends on data of actual discharge measurements and/or empirical procedures.

Discharge Measurements—If data of actual discharge measurements are available, which is usually not the case, the evaluation of the design discharge employs various statistical methods that are well known in the field of hydrology. One of the methods often used in Europe to establish the flood frequency is that suggested by Powell (12); according to this method, the maximum discharge, Q_{max}, during a period of T years can be calculated from the relationship

$$Q_{max} = Q_{avg} + \sigma\,(0.780\,y - 0.450) \tag{1}$$

where Q_{avg} is the average discharge for a period of n years, σ is the standard deviation of the maximum discharge for a period of n years, and y is given by

$$y = -\ln\left[-\ln\left(1 - \frac{1}{T}\right)\right] \tag{2}$$

The standard deviation, σ, is obtained from

$$\sigma = \sqrt{\frac{n}{n-1}\left(\frac{Q_i^2}{n} - Q_{avg}^2\right)} \tag{3}$$

where n is the number of years of observation and Q_i is the maximum yearly discharge. As an example of this technique, the frequency of the maximum yearly discharge of the Bühlot River (West Germany) is shown in Figure 5. The standard deviation is 9.186, so that for a period of five years, we have y equal to 1.5 and Q_{max} equal to 19.9 cubic meters per second.

Since the recording gage is normally located at some point away from the culvert, the corresponding catchment areas will usually not be the same. The discharge, Q, for the actual catchment area of the culvert, A_a, can be calculated from data obtained from the catchment area of the recording gage, A_g, by use of the formula

$$Q = Q_g \sqrt{\frac{A_a}{A_g}} \tag{4}$$

where Q_g is the discharge determined for the catchment area of the recording gage.

Empirical Methods—Since actual discharge measurements are seldom available, the design discharge must be calculated by other methods, most of which are empirical. One of the best known procedures is the isochrone method, whereby the runoff is predicted from the genetic runoff formula using data on precipitation in the catchment basin. However, this method is generally not used in culvert design because a number of factors, such as the runoff coefficient and characteristic precipitation, cause calculated results to be completely unreliable. For this reason, empirical methods are

TABLE 1

TYPICAL VALUES FOR SPECIFIC DISCHARGE

Catchment Area (square kilometers)	Specific Discharge (liters per second per square kilometer)			
	Lowland		Hills	
	Bare	Wooded	Bare	Wooded
1	5000	3000	10000	6000
5	3700	2400	6000	3800
10	2900	1700	4000	2500
50	1500	700	2000	1300
100	600	400	1000	600
200	300	160	800	400

usually used in practice to give guidance in choosing a design discharge. In Germany, for example, Table 1, given by Kirwald (13), is often used to estimate the "average maximum discharge." The main weakness of this method is that it gives little allowance to the actual geographical and meteorological conditions, and it does not specify the meaning of the "average maximum discharge." The eastern European countries generally follow the practice of the Soviet Union, whereby the maximum discharge of 3 percent probability is calculated by the simple formula

$$Q_{max} = \alpha \sqrt{A} \tag{5}$$

where α is a factor that depends on the climatic and geographical conditions and A is the catchment area. Values for α are specified according to geography and are usually

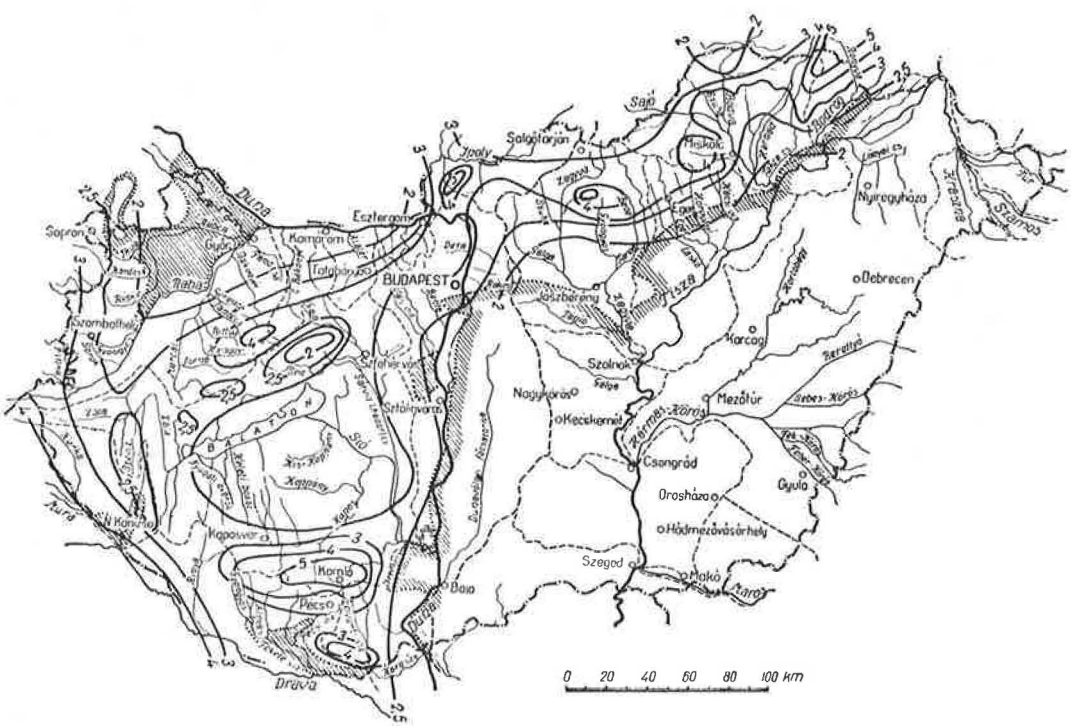

Figure 6. Typical designation for specific discharge factor α.

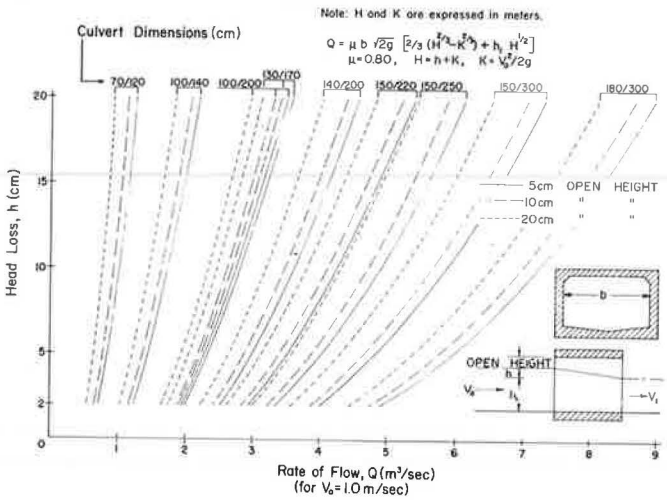

Figure 7. Chart for hydraulic design of rectangular culverts.

given in a map, as shown in Figure 6, which illustrates isochrones established by the Water Resources Research Institute of Hungary.

As soon as the design discharge has been determined, the dimensions of the culvert can be individually designed according to the hydraulic conditions of its operation. The criterion for the hydraulic design is based on providing free-surface flow conditions in the culvert at the given design discharge; and allowable design velocity and an allowable head loss are specified along the culvert. The manufacturer usually provides charts or tables to determine the required dimensions of the culvert; one typical chart for a rectangular culvert produced by the Wiweg firm (14) is shown in Figure 7. For example, if the design discharge is 4 cubic meters per second and the total head loss must not exceed 10 centimeters, culvert sections 1.5 meters deep and 2.2 meters wide must be installed to provide a clearance of about 17 centimeters between the water surface and crown of the section. These curves were constructed by assuming a mean velocity of 1 meter per second. Besides this chart, other similar ones, such as that suggested by Rowe (15), are also used in Germany, England, Holland, and elsewhere. Similar practices are followed in eastern Europe except that the charts and tables are specified in national standards.

There is a definite trend favoring the installation of a single culvert unit at a particular location; multiple unit culverts account for not more than 5 percent of the total number of culverts constructed. In addition to minimum height of cover requirements, the basis for this trend can readily be explained by the fact that the discharge capacity of a given culvert varies approximately as the 8/3 power of the diameter; therefore, it is more economical to use one culvert unit instead of two or more to convey a given rate of flow.

STRUCTURAL DESIGN

In most European countries, there are no special standards or regulations for the structural design of culverts; such design procedures are usually included in general specifications and standards for roads, bridges, reinforced concrete and steel structures, etc. In Germany, however, the load on a pipe, installed as shown in Figure 8, is specified by a standard (16). The dead and live loads are usually determined by the methods of Frühling (17) or Marston (18). Although the Marston method is almost universally used, some doubts have arisen as to the acceptability of this method in view of changing construction methods, the complete change in live loads, etc. Recognizing the needs in this field, studies have been undertaken in England, Germany, Czechoslovakia, and several other European countries.

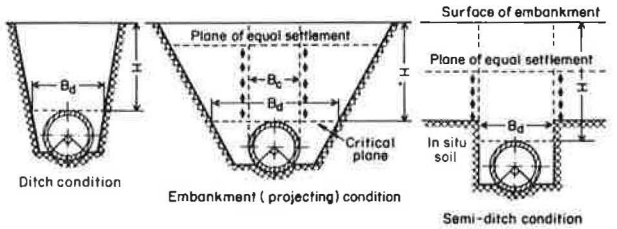

Figure 8. Classes of conduits according to DIN 4033.

Rigid Culverts

After performing a series of experiments under field conditions, Wetzorke (19) concluded that the earth load at the level of the pipe crown can be calculated with sufficient accuracy by the silo theory of Janssen (20). Instead of employing the active earth pressure to compute the frictional force, as suggested by Marston, an at-rest coefficient of 0.5 is used. The vehicle wheel (live) loads are determined from the well-known Boussinesq theory (21) for an elastic, isotropic, semi-infinite body. The results of these experiments have verified that, in the case of rigid pipes, consideration must be given to a load concentration factor. Also, from the stress analysis point of view, these results indicate that the cracking resistance of the pipe, instead of the crushing load, should be used as the design criterion.

In the Soviet Union, the Norms and Technical Conditions NiTu-57 (2) and the Standard SN-200-62 specify the load conditions and the method of structural calculations. Circular culvert sections are designed for bending moment (without considering the normal and tangential forces) by the formula

$$M = \nu r^2 (p + q) \left[1 - \tan^2 \left(45° - \frac{\phi_s}{2}\right)\right] \tag{6}$$

where p and q are the vertical pressures due to permanent (dead) and temporary (live) loading, respectively, r is the average radius of the section, and ν is a coefficient determined by the type of foundation. Values for ν are not specified in the standard, but Artamonov et al (22) have proposed a chart, shown in Figure 9, which gives ν as a function of the angle of bedding, α. The values of p and q are given by

$$p = C \gamma_s H \tag{7}$$

and

$$q = \frac{19}{H+3} \quad (H > 1 \text{ meter}) \tag{8}$$

where, in Eq. 8, H is expressed in meters and q in metric tons per square meter, and in Eq. 7,

$$C = 1 + A \tan \phi_s \tan^2 \left(45° - \frac{\phi_s}{2}\right) \tag{9}$$

and

$$A = mhH^{-3} (2H^2 - mB_c h) \tag{10}$$

H is the height of the embankment measured from the crown of the culvert to the top of the pavement; h is the distance between the plane of the foundation and the crown of the culvert; B_c is the outer width of the culvert; m is a coefficient to be determined from soil characteristics; and ϕ_s and γ_s are the standard values for the angle of internal

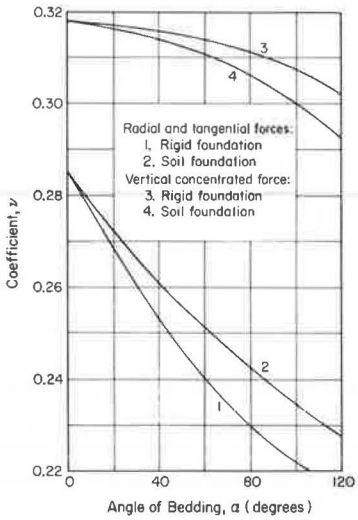

Figure 9. Effect of foundation type and angle of bedding on coefficient ν.

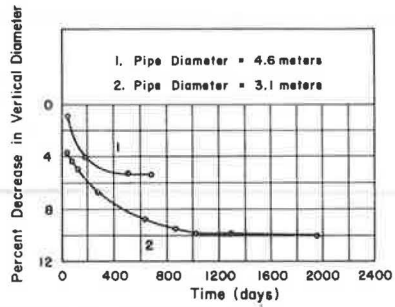

Figure 10. Change in vertical diameter with time.

friction and the weight density of soil, respectively. These latter values are determined by test if the culvert is individually designed, but, in lieu of this, values of 35 degrees and 1800 kilograms/meter^3, respectively, are used if standard projects are adopted. Values for m are chosen according to the type of soil; some typical values are 15 for very hard soil, 10 for compacted soil (medium to dense sand, sandy clay, and hard clay), and 5 for loose soil (loose sand and clay with low plasticity). According to the Soviet standard, culverts of rectangular cross section are designed as closed frames, and the walls are checked as frames with rigidly embedded supports.

Flexible Culverts

Although flexible culverts are not currently common in Europe, especially in the eastern countries, increasing attention is being devoted to their design. The distinction between flexible and rigid pipes is usually made by use of the following criterion proposed by Klein (23) and substantiated experimentally by Wetzorke (19):

$$\frac{E_p}{E_s}\left(\frac{t}{r}\right)^3 = k \tag{11}$$

where E_p is the modulus of elasticity of the pipe material, E_s is the modulus of elasticity of the soil, t is the wall thickness of the pipe, and r is the average radius of the pipe. If k is less than 1, the pipe is termed flexible, while k values greater than 1 characterize rigid pipes.

In contrast with many others, Yaroshenko et al (10) emphasize that the structural design of a flexible pipe must not be based on conditions that exist after all deformations have taken place; in addition, they do not accept the assumptions that the bending moments in the cross section of a corrugated steel culvert are negligibly small or that a uniform distribution of normal forces exists initially around the perimeter of a culvert. In fact, they contend that the unequal horizontal and vertical pressures existing soon after completion of a culvert cause large bending moments that ultimately lead to the development of plastic hinges; following the formation of these plastic hinges, the pressure distribution around the perimeter gradually equalizes to an approximately hydrostatic one. Figure 10 shows experimental data representing the shortening of the vertical diameter of a culvert with time. The ratio between the horizontal and vertical pressures changes sharply with the rigidity of the culvert and the mechanical properties of the surrounding soil.

The theoretical and experimental evidence supporting the existence of plastic hinges in flexible culverts led to the idea of designing plastic hinges in rigid culverts to mobilize more lateral earth support. Tests on 30 pipe sections were conducted in the

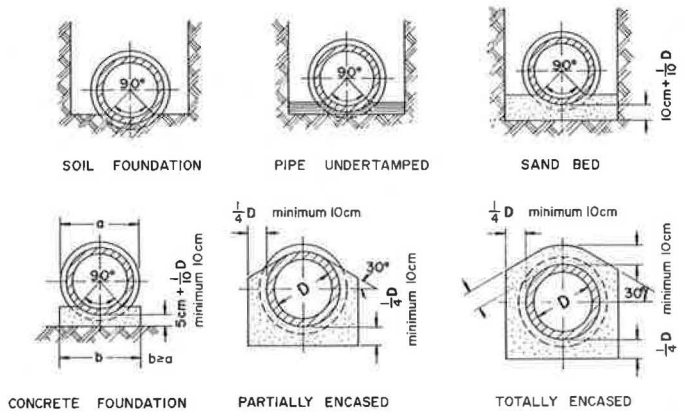

Fig. 11 Typical Bedding Conditions

Central Research Institute for Transportation in the Soviet Union during the period from 1949 to 1951 to compare the behavior of reinforced concrete cross sections with and without built-in plastic hinges; the diameters of the experimental sections varied between 1.0 and 2.0 meters and the wall thicknesses were 8 to 11 centimeters. For the pipe sections with built-in plastic hinges (these were produced by considerably weakening the reinforcement of the cross section at that point), it was found that the horizontal to vertical pressure ratio rapidly approached 1, indicating that the expected lateral support and the corresponding uniform pressure distribution around the pipe did develop. The rigid culvert models failed at a unit vertical load of 2.0 kg/cm^2, while those with built-in plastic hinges sustained a load of 3.5 to 5.0 kg/cm^2. Initially, no destruction of the concrete was noticed, except for cracks of 1 to 3 millimeters at the plastic hinges; these cracks closed by the time the deformations had developed completely, and the width of the remaining fine cracks did not exceed 0.10 to 0.15 millimeters, which was considered as safe for corrosion. The results of these experiments led to the conclusion that built-in plastic hinges considerably increase the structural strength of reinforced concrete culverts. However, there is currently no available information as to how many culverts with plastic hinges have actually been built in the Soviet Union or what practical advantages have been experienced.

INSTALLATION

Bedding

Since the structural behavior of a culvert is greatly affected by bedding conditions, these are often specified in standards (16, 24). According to German standard DIN 4033 (16), six different types of bedding are suggested, as shown in Figure 11, but concrete beddings are generally preferred. Five years ago, a survey in Hungary led to the conclusion that 60 percent of the pipe culverts laid directly on natural soil or on non-concrete bedding suffered failure to some extent. Even in cases where the pipes were bedded on compacted soil, no definite improvement in structural performance was noticed; this may be attributed to the difficulty in achieving uniform compaction in the field in order to prevent irregular settlements. Concrete bedding, however, has proved to be very effective; this has also been emphasized by Wetzorke (19). If the pipe is laid on a concrete base, the angle of support is usually taken to be 90 degrees, as shown in DIN 4033. For pipe culverts of large diameter, definite angles of support are often prescribed. Various formulas, such as that of Marquardt (25), for the bedding of pipes on preformed beds with various angles of support are based on the assumption that the supporting forces are distributed uniformly or according to the cosine law. In the Soviet Union, the structural calculations take into account the different angles of bedding, but they do not assume a uniformly distributed reaction (22). The experiments by Wetzorke (19) showed that considerable settlements of culverts can occur due to a fluctuating water

TABLE 2

THICKNESS OF COVER FOR VARIOUS COMPACTORS

Compactor	Thickness of Cover (cm)
1500 kg vibro-plate	120
1700 vibro-roller	50
100 kg rammer	60
500 kg rammer	120
1000 kg rammer	200

table. In order to avoid this problem, he suggests that the drain area be covered with concrete and the lower part of the pipe be bedded in concrete. Similar suggestions are found in certain standard projects (24).

Backfilling

As with bedding conditions, the structural performance of a pipe culvert is significantly influenced by the method of backfilling. A definite distinction is made between embedding the culvert and refilling the pit. Embedding is the first part of the backfilling operation and it generally includes filling the space between the pipe and the sides of the trench up to 30 centimeters above the crown of the pipe. It is universally accepted that only approved material containing no large stones can be used. Different standards and specifications allow the embedding work to be placed in layers not exceeding 15 to 20 centimeters in loose depth; according to the English specification, the maximum loose depth is 9 inches. The compaction of this material must be very carefully controlled because this soil adjacent to the pipe greatly affects the pressure distribution on the pipe, as shown by experiments (19). If the soil in this zone is well-compacted, the pipe will carry less vertical load. Proper compaction is most important in the case of corrugated steel culverts. Although the second part of the backfilling operation is less sensitive, proper precautions associated with each particular item of compaction equipment must be taken. DIN 4033 specifies that a minimum soil cover above the crown of the pipe must be reached before a certain type of compaction equipment may be used; some typical values are given in Table 2.

DURABILITY

Concrete

Measures to be taken in order to enhance the durability of unreinforced and reinforced concrete culverts are specified in national standards. It is generally accepted that special protective measures are needed if the sulfate content of the water exceeds 0.1 percent. Depending on the actual amount of sulfate present, different methods of protection are specified; in the order of increasing effectiveness, some of the commonly used methods in Europe are (a) use of a sulfate-resisting (high alumina) cement, such as bauxite cement; (b) admixing sulfate-resisting ingredients, such as sodium silicate in the amount of 5 to 10 percent; (c) decreasing the porosity of the concrete by special methods (centrifugal pipes); and (d) applying a protective bituminous or epoxy-tar coating. A multilayered plaster protective coating is still commonly used in many countries (Hungary, Poland), while it is almost completely unknown in western European countries. Bonded-type coatings are common in culvert construction; the application of 3 or 4 hot bituminous layers is a standard procedure, but cold bituminous surface treatment is also used. PVC sheets have been successfully used in Czechoslovakia and are considered promising. Concrete used for culverts must possess high abrasion resistance; to achieve this, the mix is usually designed for high compressive strength. A low water/cement ratio is also very important; this has even more effect on abrasion resistance than on the compressive strength. The use of tough aggregate, such as crushed igneous rock, gives good results; flint aggregates are too brittle and are generally ineffective. Good densification of the concrete improves its resistance to abrasion. Experience (8) indicates that few durability problems arise if appropriate protective measures corresponding to the activity of the water and soil are taken. The lifetime of a concrete culvert is considered to be about 90 years.

Steel

The deterioration of steel culverts due to corrosion has reached alarming proportions in many European countries and very intensive research activity was started about

TABLE 3

CORROSIVITY-RESISTANCE RELATIONSHIP FOR SOIL

Corrosivity	Resistance (ohm-cm)	Time To Achieve Total Corrosion of a 1-Centimeter Thick Plate (years)
Low	>10000	>25
Normal	2000 to 10000	10 to 25
Aggressive	1000 to 2000	5 to 10
High	500 to 1000	3 to 5
Very high	0 to 500	1 to 2

ten years ago. Although much work to find more reliable methods of protection is currently going on, no definite results have been reported so far. Strong emphasis is placed on the problem of errant currents, which are often considered to be the main reason for steel corrosion. In general, corrosion of steel culverts is caused by both the soil and the groundwater; the action of the soil in the corrosion process is accelerated by the water because it stimulates the exchange of ions induced by the electric potential gradient (26). Since wet soil is a heterogeneous, colloidal system, it may be considered an electrolyte, and corrosion is primarily an electrochemical process. It has long been believed that the corrosivity of the soil is characterized by its resistance, as indicated in Table 3 (27). Recent investigations, however, have shown that the corrosion process is much more complicated. According to Tomashov (28), ohmic resistivity of soil is not a principal factor except for very dry soils. Although the majority of soils have a predominantly cathodic control, the anodic process may become predominant in dry soils. As shown in Table 4, however, no relationship exists between soil resistance and corrosivity.

Bituminous coatings, if properly applied, provide the necessary protection for steel culverts; however, dynamic loads may cause cracks to develop in the coating, and corrosion is initiated at these cracks. Somewhat related to this situation, it was noted in Hungary that the durability of a steel culvert is related to the frequency of its dynamic loading; pipes exposed to frequent dynamic loads corroded much faster than those acted upon by infrequent dynamic loads. It is generally accepted in Europe that the lifetime of a properly coated steel or corrugated steel culvert is about 50 years, while an uncoated culvert or one exposed to frequent dynamic loads may last for only 10 to 15 years. Abrasion of a culvert is not serious compared with corrosion, but it ruins the protective coating, which in turn results in excessive corrosion.

SUMMARY

The general trend in culvert construction in Europe is to use prefabricated elements, especially unreinforced and reinforced concrete pipe sections. Steel pipes are not

TABLE 4

ANODIC AND CATHODIC POLARIZATION AND CONTROL FOR CORROSION OF STEEL IN VARIOUS SOILS

No.	Soil Characteristic	Anodic Polarization, P_A (V/a)	Portion of Anodic Control, $\frac{100 \times P_A}{P_A + P_C}$ (percent)	Cathodic Polarization, P_C (V/a)	Portion of Cathodic Control, $\frac{100 \times P_C}{P_A + P_C}$ (percent)	Rate of Steel Corrosion (g-yr/m^2)	Specific Resistance of the Soil (ohm-cm)
1	Very wet sand-clay base	10	4	220	96	252	900
2	Salty wet sand	3.3	6	50	94	1,572	1,800
3	Gray clay, with 7 to 10 percent moisture	50	9	500	91	120	40,800
4	Very wet clayey-sandy soil with pebbles	40	12	300	88	345	180,000
5	Very wet gray clay	36	13	235	87	252	720
6	Slime from a sewer ditch	40	19	170	81	270	30,000
7	Dry sand-clay base	125	53	110	47	92.5	240,000
8	Dry loam with lime	500	56	380	44	26	3,900

common because of their sensitivity to corrosion and because of the shortage in steel production in many European countries. Corrugated steel plates are sometimes used for repairing old culverts. Although plastic pipes are being used on an experimental basis, they are not economically competitive at the present time.

The calculation of the design discharge of culverts is based primarily on empirical methods, the reliability of which is questionable. The dimensions of the culverts are determined individually according to local hydraulic conditions. If prefabricated sections are adopted, charts supplied by manufacturing firms are used.

The structural design of culverts in most European countries is based primarily on the Marston method, but a different approach is followed in the Soviet Union. Research work has been started in many countries in an effort to establish a rational basis for structural calculations that would take into account the changing conditions of operation and installation. Concrete bedding for culverts is generally preferred and frequently used.

Durability of unreinforced or reinforced concrete culverts is not of great concern in Europe, provided the necessary protective measures are taken. Steel culverts are much more sensitive to corrosion, as well as abrasion, even if a protective coating is applied; however, bituminous coatings have been found to be very effective in reducing corrosion and prolonging the life of steel culverts. A relationship has been observed between the frequency and intensity of dynamic loadings and the durability of steel culverts; intensive dynamic loads significantly shorten the life of steel culverts.

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The Mechanisms of Abrasion of Aluminum Alloy Culvert, Related Field Experiences, and a Method to Predict Culvert Performance

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•THE STRUCTURAL design (1, 2) and corrosion performance (3) of aluminum alloy culverts have been investigated and reported in a number of papers. There is also a need to investigate aluminum alloy culvert service in conditions of an abrasive nature to complete the basic studies. Several years of exposure of selected aluminum alloy culverts now provide data that may be used as a basis for substantiation of a mathematical hypothesis of abrasion performance. The purpose of this paper is to present an analytic approach to the behavior of aluminum alloy culvert under abrasive flow conditions as a prelude to methods of prediction of culvert performance.

BACKGROUND

Since the introduction of aluminum alloy culvert in 1961, the author has had the opportunity to personally observe the abrasion performance of well over 1,000 aluminum alloy culverts and a like number of galvanized steel culverts. These observations were further supplemented by observations by the author and other engineers of additional applications of aluminum.

Several preliminary conclusions were possible after reviewing the observations gathered. There was general uniformity of performance for similar terrains, bed load, and cumulative flow times. While there was some surface peening, there was no observed metal loss due to sand or very small rock bed loads, even when the flows were high. No significant evidence of corrosion was found on aluminum culverts in abrasive-type flow installations due to nearly neutral pH conditions. The abrasion rate was observed to be linear in nature.

Preliminary evaluation suggested that abrasion might be related to bed load flow energy, mass, and velocity, with contributions from the effect of the repetitious corrugations.

EROSION-CORROSION CYCLE

The mechanics of wastage of aluminum alloy culverts under abrasive flow conditions needs to be reviewed. This is particularly important because the total cycle causing metal loss of aluminum due to erosion-corrosion has been observed by the author to differ markedly from the cycle causing metal loss from galvanized steel culvert.

Galvanized steel exposed to an abrasive flow follows an erosion-corrosion-erosion cycle. The abrasive material removes the relatively soft zinc and exposes the steel surface below at a rate that depends on the severity of flow; iron oxide is then formed, only to be removed by further abrasive flow. This mechanism of corrosion-erosion attack causes steady rates of wastage of steel culvert inverts. The surface hardness of steel was observed to resist the abusive action of larger rocks under severe conditions of flow moderately well. The wastage rate appears to be governed more by the corrosion portion of the erosion-corrosion couple than by abrasion only. The progressive erosion-corrosion cycle proceeds in all types of abrasive flow including sand and

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gravel. In cases where water may be corrosive, wastage would increase. A generalized cumulative erosion-corrosion curve is shown in Figure 1 to illustrate the characteristic metal wastage patterns of steel culvert.

The corrosion resistance of aluminum alloys eliminates the type of corrosion attack pattern that produces and continues rust on steel. The cumulative curve of erosion-corrosion across the full range of flows as developed for aluminum alloys may be simulated on Figure 1. When superimposed the divergent performance of steel and aluminum becomes clear, even though they appear to ultimately perform comparably over extended periods.

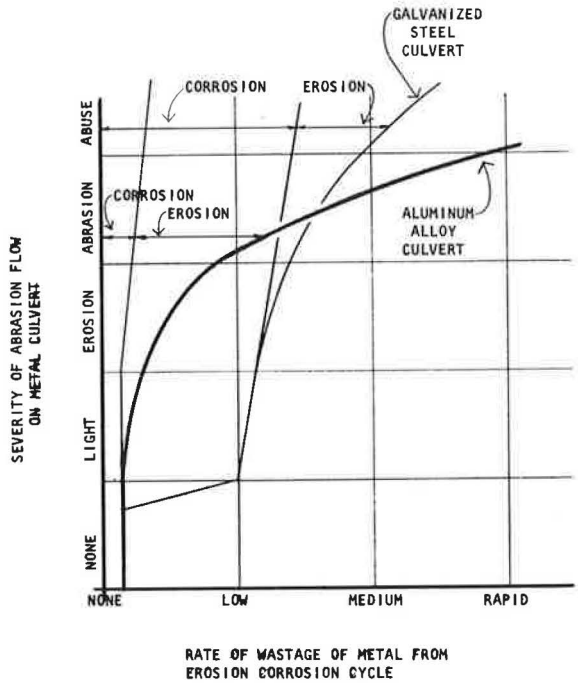


Figure 1. Cumulative curve of erosion-corrosion.

MECHANICS OF ABRASION

Nomenclature

P = Pressure force of water, lb
W = Weight of rock, lb
F = Friction resistance, lb
N = Normal force on surface, lb
I = Mass moment of inertia of rock, ft lb sec^2
u = Rock velocity, ft/sec
V_W = Mean pipe velocity, water ft/sec
V_e = Culvert water entrance velocity, Q/A, ft/sec
β = Incline slope of culvert, 27 deg for $2\frac{2}{3} \times \frac{1}{2}$ shape
ϕ = Slope of culvert
θ = $\beta - \phi$
ρ = Density of rock in water, lb/ft^3
r = Spherical radius of equivalent rock, ft
a_L = Linear acceleration of rock, ft/sec^2
a_R = Rotational acceleration
a = Total acceleration, ft/sec^2
s = Distance of free flow, ft
e = Impact velocity restoration factor
μ = Friction coefficient
ΔKE = Mean impact kinetic energy, ft lb
KE_u = Peak kinetic energy, ft lb
x = Horizontal distance, ft
y = Vertical distance, ft

Mathematical Hypothesis

The source of surface abrasion on the invert of metal culvert is the cumulative result of impact or rubbing action by particles of hardness equal to or greater than the metal surface. To describe rates and predict abrasion in its varying parameters satisfactorily it is desirable that abrasion action be reduced to a mathematical hypothesis. Observations showed that culverts subjected to active flow are self-scrubbing (i.e., self-cleaning) including corrugation valleys. Observations also have consistently shown no evidence of abrasive action by sand or very small rocks, while flows containing

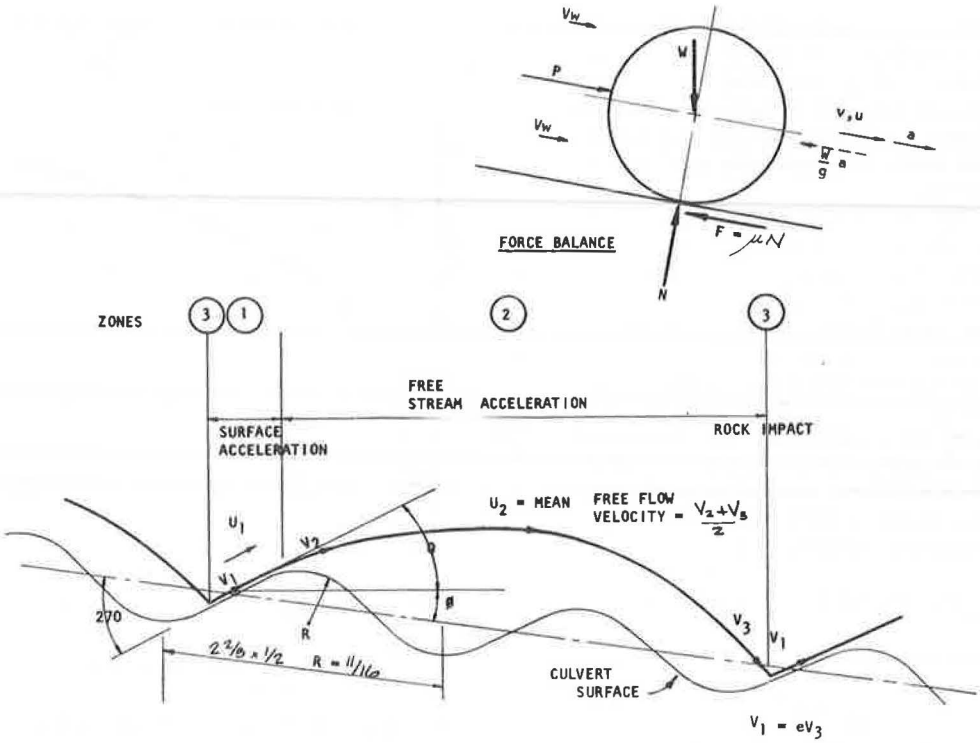


Figure 2. Rock cycle path.

increasing quantities and sizes of rock show increasing surface abrasion. From this, abrasion may be related to the cumulative kinetic energy of the rocks that flow through, i.e., to mass and velocity.

The energy theory permits the use of conventional force, velocity, and acceleration equations as a framework for analysis (4). Figure 2 shows the forces acting on a single rock, represented by a sphere. When the driving force, water pressure, exceeds the net resistance of gravity and friction the rock will move and continue to accelerate to a terminal velocity, u. Since culvert inverts are not smooth but corrugated, the actual path of a rock (particularly a smaller rock) does not follow a constant path but rather a series of cycles as shown in Figure 2 (5). Observation indicated that the rocks move by translation. During the initial portion of a flow cycle the rock is accelerated up the corrugation incline, proceeds by free flow over one or several corrugations, then drops to impact on the incline, which reduces the velocity and starts the repeat of the cycle.

Using basic equations of kinetic and potential energy and the flow path of Figure 2, the analysis may be developed for each zone as follows:

Zone 1—Surface flow

$$a_L = 0.075 \frac{(V_W - u_1)^2}{r} - 32.2 (\sin \theta + \mu \cos \theta) \tag{1}$$

Zone 2—Linear flow

$$a_2 = 0.075 \frac{(V_W - u_2)^2}{r} - 32.2 \sin \theta_{mean} \tag{2}$$

$$\approx 0.075 \frac{(V_W - u_2)^2}{r} - 32.2 \tan \phi \tag{3}$$

$$S_2 = \frac{2u_2^2}{g} \tan \beta \tag{4}$$

$$V_3 - V_2 = \frac{2u_2}{g} \tan \beta \left(0.075 \frac{(V_W - u_2)^2}{r} - 32.2 \tan \phi\right) \tag{5}$$

Zone 3

$$V_1 = eV_3 \tag{6}$$

Equations 1 through 6 are derived from the basic equations of dynamics (4).

$$\Sigma F = \frac{w}{g} a \quad \text{Force balance} \tag{7}$$

$$F = \mu n \quad \text{Friction} \tag{8}$$

$$V = eV_0 \quad \text{Impact restoration} \tag{9}$$

The solution includes a number of assumptions. For example, a rock is considered a sphere of uniform density, rotation is assumed minimal, culvert corrugations are 2⅔ by ½ in. and the free flow velocity, u_2, is taken as the arithmetic mean between start and impact. Such a method is considered reasonable in dimensional analysis provided the forms of the equations are substantiated by experimental data. Further, Zone 1 has little effect on the overall solution and since it is of the same form as zone 2 it may readily be combined with Zone 2 to eliminate one set of equations.

Two alternate solution forms to the foregoing equations are possible. The first solution is based on calculations using actual flow paths to determine velocities, accelerations, and distances. It solves for the magnitude of the velocity reduction at impact (1 - e), and hence a measure of the energy lost on each impact ΔKE. The second solution is based on solving for a constant or mean rock flow velocity, u, and an equivalent friction factor, μ, resulting in peak energy available for any impact KE_u. It is recognized that rock flow in culverts is at best an inexact science and reproducibility of results depends on similarity of a host of variables such as rock size, shape, frequency, hardness, site geometry, mean flow patterns, short-term higher flow rates, and rock availability. To account for and minimize these variables a field test program was undertaken to study the apparent effect of these variables on the dimensional analysis or mathematical hypothesis. The overall effect on aluminum alloy was also cataloged so that the calculated performance energy levels could be rated against field performance.

Rock Size, Shape, Availability, and Hardness

A basic assumption in the analysis was that progressive abrasion will follow a generally linear pattern. This has been confirmed to date by several investigations of the selected culverts over a period of years. If anything, abrasion rates in average installations appear to diminish somewhat.

The field investigation program also confirmed that the incidence of abrasion varies somewhat due principally to differences in terrain and rainfall patterns. It is interesting to note, however, that ranges observed as a result of the variations were not excessive. For example:

1. Areas with considerable ground cover will restrict rock flow and thus cause less abrasion. Increased abrasion will occur in areas with little ground cover and loose rock in the slopes. A steep, bare, loose rock slope subjected to sudden and repeated heavy flows would represent the most severe exposure possible for culvert abrasion.

These appear to be more likely to occur on the west slopes of the North Pacific Coast. Because of these conditions a substantial number of the sites used for study are concentrated in the Pacific Northwest.

2. Where few rocks are present, such as in the central United States, abrasion of aluminum alloys is not expected and was not observed.

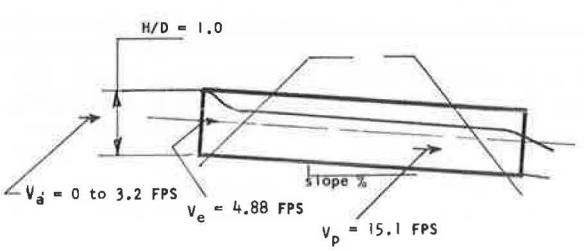
3. Sharp, hard rocks cause more relative abrasion than less angular, rounded glacial or soft rock when all other conditions are equal.

Velocity

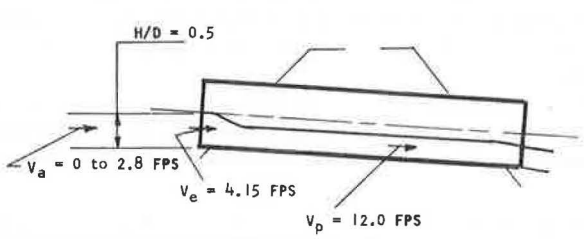
The analysis is particularly dependent on the velocity of the rock, since any velocity developed is a squared term. Consequently, it is necessary to review the different velocities that may occur in a typical culvert installation (6):

1. Approach velocity, V_a, is the velocity in the approach channel. This velocity is low, usually less than 5 ft per sec, and provides limited information to the designer on expected performance. It furnishes no insight into rock flow.

2. Entrance velocity, V_e, is the velocity at the entrance to a culvert. Designers frequently use the entrance flow as a guide to selection of culvert size. Typically, the 10-year flood flow, Q_{10}, with a fully submerged entrance is used as a basis of design. Such velocities are a direct function of culvert diameter and entrance geometry. Designers frequently attempt to define abrasion with a limit on entrance velocity, as in the Bureau of Public Roads publication, "Corrugated Metal Pipe Culverts" (7), which suggests a value of 5 ft per sec. Although the flood flow method appears reasonable for culvert sizing, it has very limited effectiveness as a method of defining abrasion. Since V_e is directly proportional to culvert diameter, an entrance velocity of 5 ft per sec merely indicates that a 42-in. maximum size culvert had been selected. On the other hand, entrance velocity can serve as a guide to describe probability of number and size of rocks that may be inducted into the culvert. Field observations of rock flow indicate that a Q_{10} flooded entrance velocity, V_e, of less than 4 ft per sec will not induct a significant number of 2-in. and larger rocks, and where V_e exceeds 5 ft per sec, it must be combined with frequent induction of rocks over 3 in. in size before significant abrasion can be expected.



3A Conditions at Submerged Entrance, Typical Q_{10} = 47 CFS



3B Conditions at Half Submerged Entrance, Typical Q = 20 CFS

Figure 3. Velocities experienced in typical culvert having 42-in. diameter, projecting entrance, 10 percent slope.

3. Pipe velocity, V_p, is the velocity of the water in the culvert once clear of the entrance. It is a function of flow (Q_{10} for example), culvert diameter, culvert slope, and corrugation shape. As an example, a 42-in. culvert with a Q_{10} of 47 cu ft per sec at the flooded entrance will develop a pipe velocity of 11 ft per sec on 5 percent slope or 22 ft per sec on 30 percent slope. Pipe velocity describes the degree of abrasion from velocity when combined with rock flow. However, the use of V_p as the sole basis of degree for abrasion is not recommended because by definition it occurs only once in 10 years and poses an unrealistic level.

4. Mean pipe velocity, V_w, is a defined velocity in the culvert as a result of flow through

a projecting entrance filled half the depth of the culvert, selected to represent a reasonable combination of frequency of occurrence and velocity magnitude. Mean pipe velocity is used for this analysis.

5. Rock velocity, u, is the net velocity of a rock as it passes through the culvert when subjected to a forcing flow due to V_W. The rock velocity may vary from zero to V_W and will of course vary between impacts.

An example of the several velocities that may be experienced in a typical culvert are shown in Figure 3 (8). For the analysis a 42-in. diameter, projecting entrance, 10 percent slope culvert is shown.

Mean Impact Energy

The mean energy loss to the surface at each impact of a single rock (4) as it passes through the culvert is shown as

$$\Delta KE = \frac{W}{2g} V_W^2 (1 - e)^2 \tag{10}$$

When the rock is considered as an equivalent sphere, then

$$\Delta KE = 5.21\, r^3 V_W^2 (1 - e)^2 \tag{11}$$

The total energy to which a surface may be subjected over a long period of time is proportional to these energy equations.

This solution requires analysis of the rock flow cycle (Fig. 2). A series of rock flow tests were run in culverts. From these tests calculation curves of both the velocity impact factor (1 - e) and the energy impact factors $(1 - e)^2$ were prepared as shown in Figure 4.

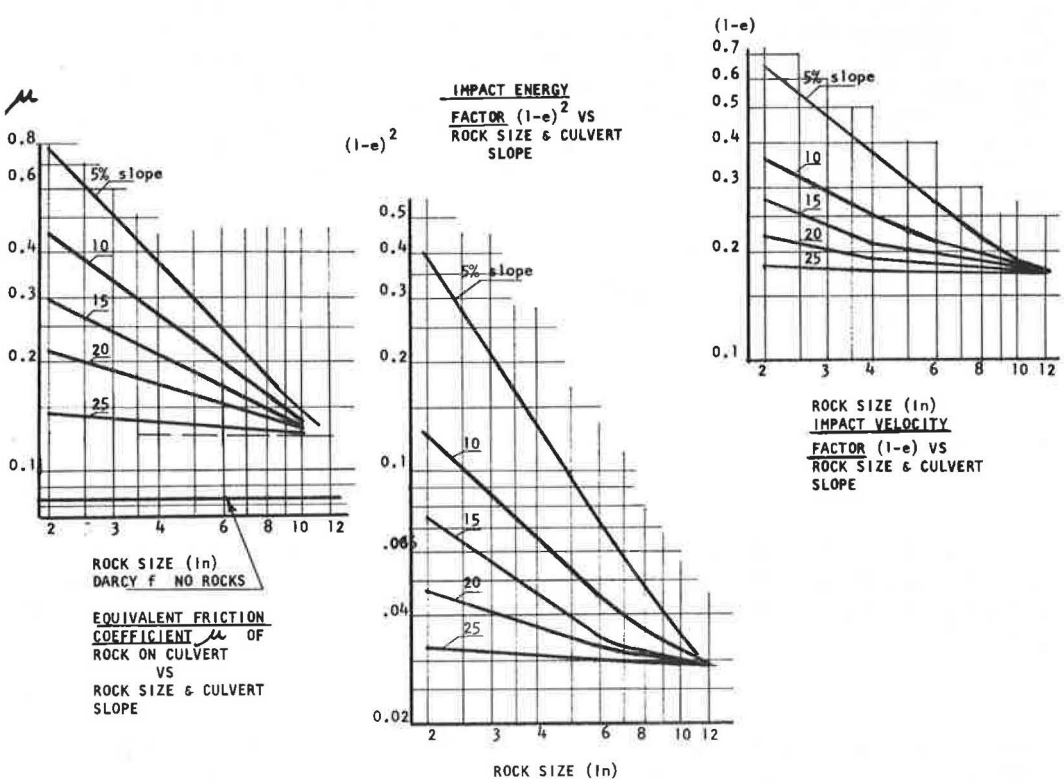


Figure 4. Curves of friction and energy factors—aluminum alloy culvert.

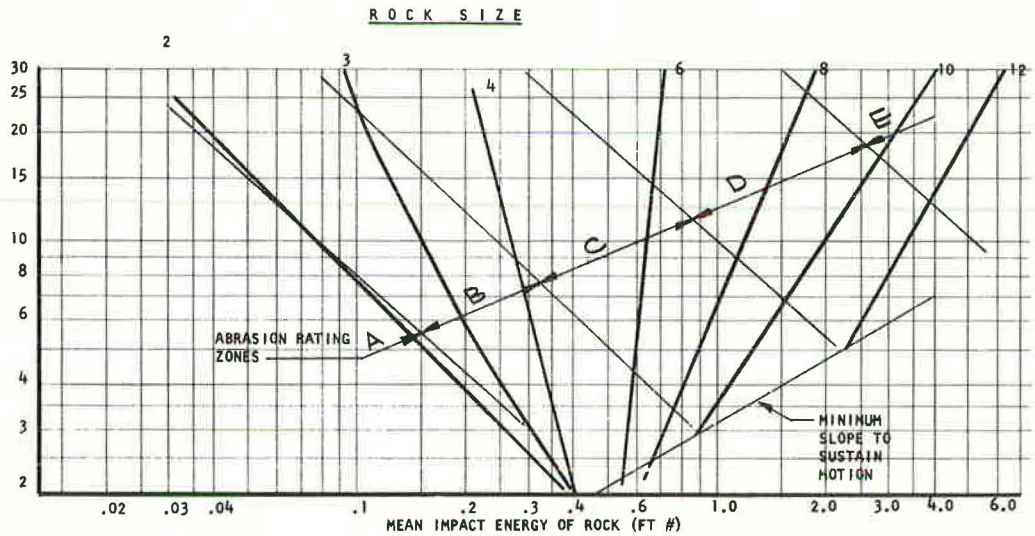


Figure 5. Typical mean impact energy curves for 48-in. culvert.

The magnitude of the water flow in the pipe at the precise time the mathematical rock passes through is certainly highly variable. A substantial minimum flow must exist to permit rocks to move at all. Combining the mean energy loss in Eq. 10 or 11 and Figure 4, a mean impact energy curve may be calculated for a representative range of slopes, rock sizes, and culvert sizes. Figure 5 shows a typical set of curves for a given culvert size. The lower cutoff line approximates the minimum flow and slope conditions necessary to commence rock movement. It is interesting to note that impact energy actually increases for small rocks as the culvert slopes are reduced, due to drastic increases in (1 - e) resulting from the steeper impact angles with the inclined corrugated surface. As the rocks become larger thay tend to pass across the corrugations, causing the energy line to increase with increased slopes.

Figure 5 includes a series of abrasion rating lines developed from field observations that are discussed later.

Peak Available Impact Energy

An alternate approach to analysis comes from the peak available impact energy method (4) where

$$KE_u = \frac{W}{2g} u^2 \tag{12}$$

When the rock is considered as an equivalent sphere, then

$$KE_u = 5.21\, r^3\, u^2 \tag{13}$$

This represents the peak energy available in a rock that becomes suddenly arrested while in the stream. This method of analysis becomes more indicative of energy levels that may cause occasional gouging. The peak energy requires determination of relative rock velocity, u, from

$$(V_w - u)^2 = 430\, r\, (\mu_m - \sin \phi) \tag{14}$$

This equation uses an equivalent constant flow (acceleration zero) and an equivalent friction coefficient, μ_m, which takes into account rock size and culvert slope. It is derived from the flow equations of the previous section.

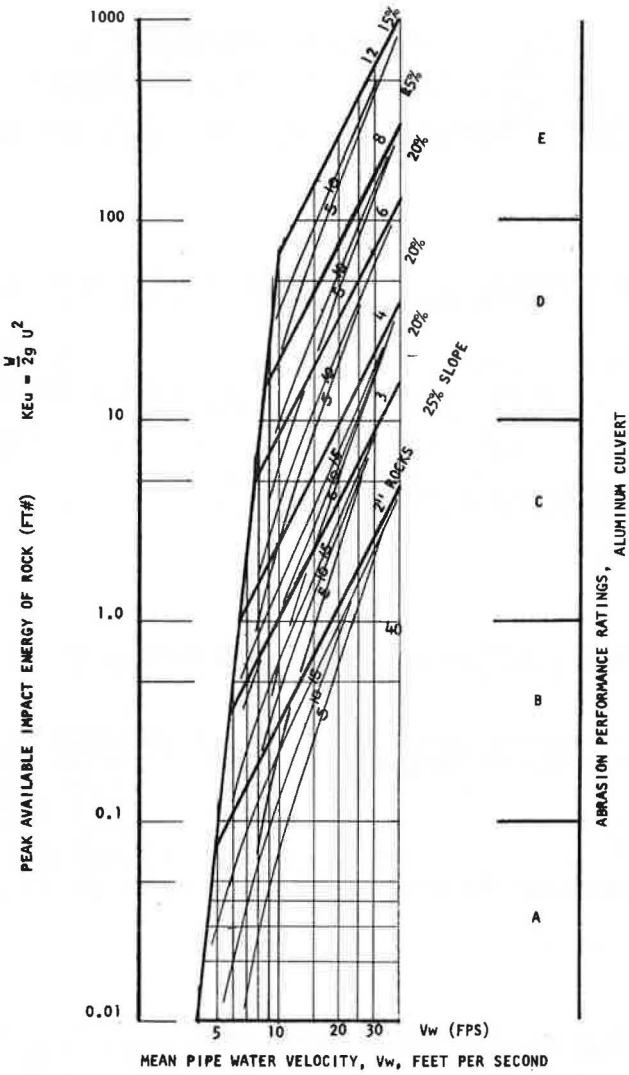


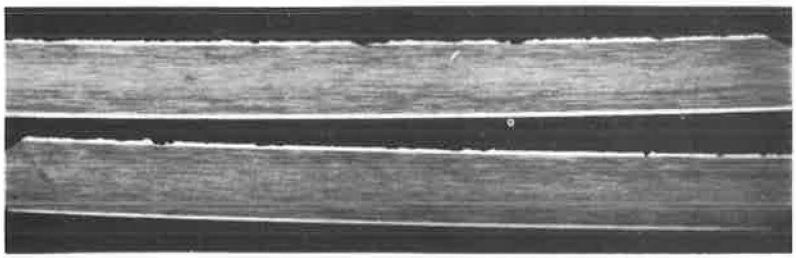
Figure 6.

Peak rock energy KE_u produces a somewhat different set of curves plotting flow vs energy (Fig. 6). The rating system noted previously is also indicated in Figure 6.

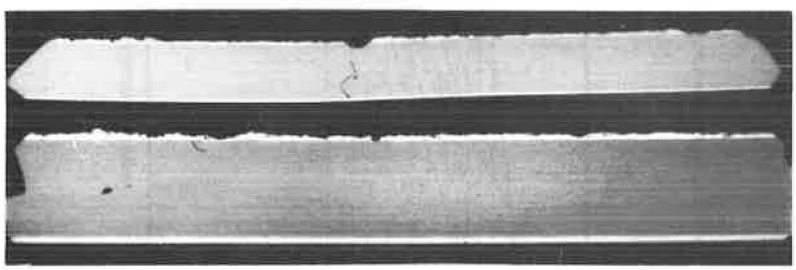
OBSERVED METAL LOSS FROM FIELD SAMPLES

Each culvert inspected was given a visual abrasion rating as described in Table 1. These are tabulated for the control culverts in the field program. The data thus obtained with culvert slope and rock size parameters were plotted and superimposed on both the mean energy curves (Fig. 5) and the peak energy curve (Fig. 6). This resulted in the positioning of the zone division lines shown. A satisfactory degree of reproducibility was attained by this method with very little scatter. Rock size was determined in the field by inspection of the stream bed and the culvert interior, selected to represent not the largest possible rock to pass through, but the estimated largest "statistical" rock that may pass through the culvert repeatedly at a "significant" frequency during periods of high water flow. This size selection is approximate only, but is important and calls for the exercise of judgment to simulate the exposure as expected

ABRASION RATING A 67-330



ABRASION RATING A/B 67-354



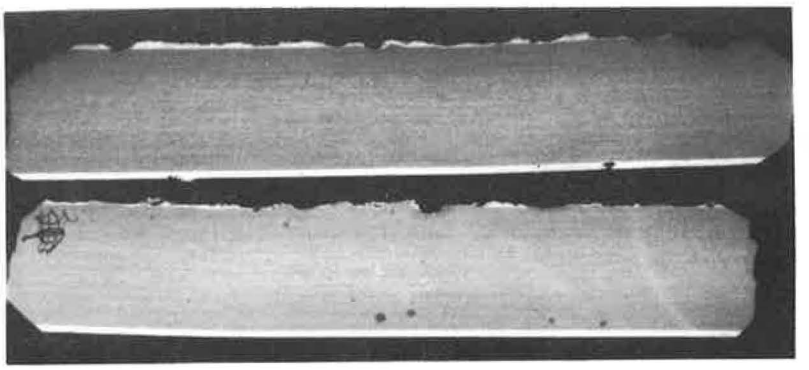
ABRASION RATING B 67-335

ABRASION RATING B/C 67-316

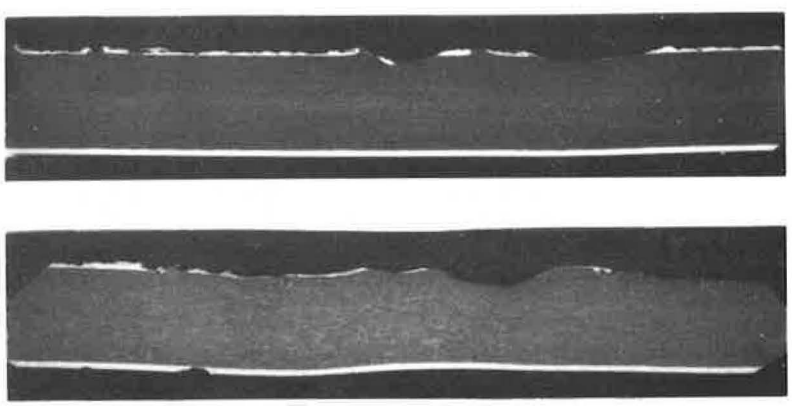
Figure 7. Photomicrographs of culvert sections.

over many years. This approximation is very necessary to the rating and it proved to be manageable with some experience.

Culverts were also sampled by drilling out coupons from the invert crowns and subjecting them to laboratory examination. Detailed surface and cross section inspection was conducted on each culvert to confirm the visual rating. The coupon samples were selected to be representative of the more severe portions of the culverts given a visual abrasion rating. Abrasion occurs on the upstream portion of the crown radius of the corrugations at the invert line. Figure 7 shows selected photomicrographs representing examples of no effect (Rate A), non-erosion (Rate B), erosion (Rate C), and abrasion (Rate D). The sample of no effect is included to show clearly the extent of the



ABRASION RATING C 67-326 - 7



ABRASION RATING D 67-319

Figure 7. (continued).

cladding on the surface—5 percent of the thickness or approximately 0.003 in. The non-erosion (Rate B) sample shows characteristic light surface pebbling but no significant removal of cladding. The cladding is slightly softer than the core so will show surface effects more readily than the core. The sample of erosion (Rate C) shows the same pebbling texture and light random gouging but to a degree sufficient to visibly disturb and gradually remove metal. The abrasion sample (Rate D) represents the cumulative result of pebbling and gouging that more rapidly reduces the metal thickness by removal. The abusive condition (Rate E) is not shown as no instances were observed in this sampling. Such a sample would show more rapid progressive removal by gouging and pounding than shown in example D. Rivet projections are particularly vulnerable in abusive flow and are rapidly worn flush when they project on the crown of the invert. All samples in Figure 7 represent the typical result of 5 years of exposure.

Each coupon was subjected to careful surface and cross section examination, and from these examinations the depth of metal removed was recorded. Dividing this depth by the years of exposure provided an average wastage abrasion rate expressed in inches per year (IPY). The metal culvert gage was then divided by the abrasion rate (IPY) to produce analytical service life estimates in years. This method proved to be a good comparison check on the visual rating system. Reinforced by the laboratory IPY values, the visual abrasion rating and the energy rating performance data could be handled

TABLE 1

ABRASION PERFORMANCE RATING SCHEDULE

Performance Zone Ratings	Effect on Surface of Crown of Corrugation, Invert Only[a]
A	No surface effect. No reduction in service life due to bed load abrasion. Abrasion design prediction life 50 years minimum.
B	Non-erosive. Some slight roughening of the metal surface but no metal removal by erosion action. No reduction in normal service life of aluminum culvert. Abrasion design prediction life 50 years.
C	Erosion. Surface roughening and slight progressive removal of metal from culvert. Estimated maximum average rate on invert crown 0.002 in. per year. Some gouging may be noted if rocks tend to be large. Abrasion design prediction life 25 to 50 years depending on gage of metal.
D	Abrasion. Surface roughening and slow removal of metal from culvert. Estimated maximum average rate on invert crown 0.002 to 0.005 in. per year. Definite reduction in pipe life due to abrasion. Gouging of surface may be expected. Design prediction life 15 to 25 years or more depending on gage of metal.
E	Abusive. Surface roughening and rapid removal of metal from culvert. Estimated maximum average rate on culvert crown greater than 0.005 in. per year. Definite reduction in pipe life. Design prediction life under 25 years depending on gage of metal.

[a]Abrasion affects only this portion of the surface.

TABLE 2

COMPARISON TABLE OF ABRASION RATINGS

Estimated Service Life (years)	Peak Energy (ft lb)	Abrasion Rating Recommended	Wastage of Metal (increase per year)
50+	0.1 and less	A	Nil
50	0.1 to 1.0	B	Nil
25 to 50	1.0 to 10	C	0.002
25+	10 to 100	D	0.002 to 0.005
Under 25	100 and higher	E	0.005

TABLE 3

ROCK SIZES AND CULVERT SLOPE

Abrasion Rating Desired	Rock Size[a] (in.)	Maximum Slopes (percent) for Typical Culvert Sizes		
		24 in.	48 in.	72 in.
A	2	— any slope —		
B	3	25	20	7
	4	12	9	4
	6	6	4	2
C	6	45	14	8
	8	25	9	6
	12	—	5	4
D	8	—	25	18
	12	—	13	10

[a]Statistical peak rock size expected.

with assurance. Having the results, the procedure could then be reversed and be used as a method of prediction of performance on new culverts.

ESTIMATING SERVICE LIFE USING ENERGY CURVES

The knowledge developed and shown in Figures 5 and 6 and the abrasion rating system may be rewritten in the form of a service life chart, Table 2. For convenience the peak energy curve (Fig. 6) is used. A spread in expected service life may be expected when wastage rates are applied, due in large part to the wide differences in metal thickness (from 16 to 8 gage) that are commonly used.

With these recommendations an experienced engineer can predict performance by evaluating the channel to be drained in the immediate proximity to the upstream entrance to a culvert and estimating what maximum rock size can be expected to reach and pass repeatedly through the culvert. He must consider slopes, soil types, and rainfall patterns. Table 3 is included as a guide.

In addition to the size-slope table a number of other possibilities should be considered:

1. Gage of the culvert is normally selected by structural considerations, and the thickness of the metal selected must be combined with the abrasion rates (IPY) to give an accurate picture of anticipated service life.
2. Channel and culvert entrance design is the key to reducing rock flow. This factor is the most important to design because rock size limitation is highly desirable. Reduce the velocity of approach, V_a, wherever possible.
3. Paving of inverts with softer materials such as bitumen or asphalt is ineffective for use as abrasion control. Such coatings cannot resist rock flows and the filling of invert corrugations does not appear to improve resistance to abrasion.
4. Structural plate or deep corrugations will slow rock flows, causing a reduction in rock energies.
5. Where difficult abrasive conditions cannot be avoided, invert gages should be increased and/or liners installed in the invert.
6. The use of oversize culverts will reduce pipe velocities considerably and thus reduce abrasion energies.
7. Flared or apron entrances do not beneficially improve abrasion resistance.

FUTURE

The intent of this paper is to stimulate consideration of the abrasion of aluminum alloy culvert as being within a mathematical system framework supported by field investigations.

It is expected that the program of field observations will continue to improve the accuracy and basis of performance predictions and development of design parameters.

ACKNOWLEDGMENT

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Corrosion Evaluation of Aluminum Culvert Based on Field Performance

T. A. LOWE and R. H. VATERLAUS, Kaiser Aluminum, and
R. I. LINDBERG and L. R. LAWRENCE, Reynolds Metals

Nearly 1000 uncoated aluminum culverts were inspected to develop a broad base for analysis of performance. From this group 583 were examined in detail, with water, soil, and metal sampled for laboratory examination. The service history of all culverts inspected averaged 4.7 years. Approximately 20 percent of those sampled had been in service for 6 years or more. The results are compared with corrosion behavior of aluminum in other environments. Only in a few unusual exposures was significant attack noted, and these cases were predictable from published parameters. From this it can be concluded that aluminum culvert behavior closely follows that expected in normal aqueous and atmospheric environments.

Graphic comparisons are made with water pH and resistivity, as well as with soil pH and minimum resistivity. These comparisons confirm earlier pH recommendations limiting use of uncoated aluminum alloy culvert to soils and waters within a pH range of 4 to 9, but suggest a minimum soil resistivity of no less than 500 ohm-cm. This resistivity value is lower than that recommended in an earlier investigation based on the minimum value of several field measurements made with a Vibroground. The present current recommendation is based on laboratory measurement of minimum soil resistivity. Exceptions to the resistivity limit occur in granular soils contaminated with such low resistivity media as sea water (30 ohm-cm); in such areas, aluminum performs satisfactorily. Two exposure conditions do cause significant corrosion of uncoated aluminum alloy culvert: (a) runoff containing sulfuric acid at pH less than 4, and (b) low minimum resistivity clay mucks. The inconsequential degree of attack represented by the large majority of culverts now in place allows one to conservatively extrapolate to a 50-year service life.

•ALUMINUM CULVERT has been available since 1960. Since that time a number of investigators have reported on selected short-term corrosion results. The number of culverts covered in these evaluations was necessarily limited. Nonetheless the results are reviewed here for later comparison with the present evaluation.

Nordlin and Stratfull made a preliminary study (1) based on 0.8 to 2.4 years' exposure of eight simulated culvert installations. The sites selected represented the most severe exposures available: three represented naturally formed sulfuric acid (pH < 4); two represented aggressive abrasion sites; one represented San Joaquin peat; one represented seawater-contaminated tidal zone clay; and one represented low-resistivity alkaline soil. As expected, the sulfuric acid sites produced aggressive corrosion, abrasion was severe at the abrasion sites, and the remaining sites showed

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TABLE 1

ALUMINUM CULVERT INSPECTION SUMMARY BY STATE

State	No. of Culverts Logged	No. of Culverts Sampled	No. With 6 or More Years Service
Alabama	14	14	–
Arizona	9	9	1
California	297	254	27
Colorado	2	2	–
Florida	42	42	20
Georgia	4	4	3
Illinois	14	14	–
Indiana	9	9	1
Iowa	20	20	16
Kansas	126	15	–
Kentucky	1	1	–
Louisiana	71	8	–
Maryland	26	26	20
Michigan	8	8	–
Mississippi	3	3	–
Missouri	8	8	6
Nebraska	8	8	–
New Mexico	3	3	–
New York	30	30	–
North Carolina	3	3	–
Ohio	7	7	4
Oklahoma	166	10	–
Oregon	28	28	–
South Dakota	3	3	2
Tennessee	2	2	–
Utah	1	1	1
Virginia	4	4	2
Washington	40	32	8
Wisconsin	11	11	6
Wyoming	4	4	–
Totals	965	583	117

varying degrees of corrosion, generally limited to the cladding.

Berg (2) evaluated 16 culverts installed in typical bedding for 2 to 4 years. The most severe corrosion occurred with oyster-shell backfill in a tidal drainage ditch, but in no case did pitting go beyond the cladding.

Haviland, Bellair, and Morrell (3) published results on 34 aluminum uncoated culverts with 3 to 6 years' exposure. Some staining and limited corrosion was observed—attack was confined to the cladding.

Lowe and Koepf (4) discussed performance of 71 selected uncoated aluminum culverts with 0.4 to 3.5 years' exposure. The soil and water environments represented a wide range of exposure conditions, and results reflected this. Where exposures were severe, such as in runoff containing sulfuric acid, aggressive corrosion resulted. Culverts installed under more typical exposures were unaffected or, at worst, showed corrosion confined to the cladding.

The application of aluminum alloy culvert has developed to a point where clearly definitive performance may be observed. To establish this performance, the Aluminum Association undertook an extensive culvert inspection program that included development of location information, installation characteristics, and samples of water, soil, and metal.

A national inspection program was initiated having the distribution given in Table 1. As wide a range of conditions as possible was included; 965 individual uncoated culverts were inspected and from this group 583 culverts were sampled. Service life averaged 4.7 years; 20 percent of all culverts sampled had a service history of 6 or more years. Table 2 gives a breakdown of exposure history.

These exposure periods are the longest yet reported and are considered adequate in time and quantity to provide meaningful data for (a) evaluating performance, (b) establishing recommended limits of environmental conditions, and (c) extrapolating to expected service life. The data are sufficient for comparison with published data on other atmospheric, aqueous, and underground corrosion of aluminum. The underground corrosion characteristics of aluminum are shown to be similar to those in other types of exposures.

TABLE 2

BREAKDOWN OF EXPOSURE HISTORY OF ALL CULVERTS INSPECTED

Years Service to Date of Last Inspection	Number of Culverts
7 and over	19
6 to 7	98
5 to 6	266
4 to 5	335
3 to 4	202
2 to 3	29
Under 2	16
Total inspected	965

THE NATURE OF ALUMINUM CORROSION

A detailed discussion of aluminum corrosion principles is beyond the scope of

this paper. Excellent discussions are published elsewhere (5, 6). Some comment on corrosion does, however, seem appropriate to complement subsequent interpretations.

Oxide Film

Aluminum depends on a thin film of aluminum oxide for its corrosion resistance. The aluminum oxide film is tough and highly tenacious, an excellent dielectric, and inert to a wide range of environments. When damaged it forms again instantaneously.

Since corrosion is an electrochemical phenomenon, these film characteristics play a major role in providing aluminum its corrosion resistance. The oxide film produces high electrical resistance in the circuit between anodes and cathodes, thereby reducing current flow and the amount of corrosion. Thus, in an environment in which the oxide film is chemically stable, corrosion is virtually nil.

Pitting

How does aluminum corrode if the film is so inert? The aluminum that we encounter is not pure aluminum, but an aluminum alloy. Other metals are added primarily to improve strength. Certain elements present as purposeful additions or as impurities are believed to affect the structure of the oxide film. In other words, we have point-site imperfections in the protective film. When attack occurs at these imperfections, the imperfection is enveloped by aluminum oxide resulting from corrosion of the metal substrate, or it is displaced by the oxide. The resulting build-up of oxide protects the underlying metal, and attack is effectively arrested. This, and similar processes occurring as the result of other mechanisms, accounts for the localized attack, or pitting, associated with the corrosion of aluminum. Long-term exposure data affirm the "self-stopping" nature of pitting attack on aluminum alloys in atmospheric and aqueous environments (5, 7).

Control of Corrosion by Cladding

Cladding consists of nothing more than making a "sandwich" of the stronger alloy core and outside layers of an alloy that is anodic to the core material. These layers are metallurgically bonded during the rolling operation. Aluminum alloy culvert sheet is clad in this manner—the core being alloy 3004 and the cladding alloy 7072. Both alloy components of culvert sheet exhibit a high order of corrosion resistance; however, the cladding is galvanically expended should corrosion occur, thereby preventing attack of the core until a substantial area of cladding is removed laterally. Table 3 compares the relative resistance with other widely used aluminum alloys (6).

TABLE 3

RELATIVE CORROSION RATINGS[a]

Type of Alloy	Commercial Examples	General Corrosion	
		Temper	Rating[b]
Unalloyed Al	1100	All	A
Al-Mn	3003	All	A
Al-Mg	5005, 5050, 5052	All	A
Al-Mg	5056, 5154	All	A[c]
Al-Mg-Mn	3004, 5454	All	A
Al-Mg-Mn	5086, 5083, 5456	All	A[c]
Al-Mg-Si-Cu	6061	All	B
Al-Zn	7072	All	A

[a]Modified from Van Horn (6).
[b]Relative ratings A through E (no C through E alloys are shown for the purposes of this comparison) are in decreasing order of merit, based on exposure to sodium chloride solution by intermittent spraying or immersion. Alloys with A and B ratings can be used in industrial and seacoast atmospheres with protection.
[c]Rating may be changed for material held at elevated temperature for long periods.

pH and Resistivity

In chemical media, which includes soils and waters, pH alone does not define corrosivity. Use of aluminum is generally satisfactory within a pH range of 4 to 9. Exceptions occur when heavy metals (usually copper) are present.

Resistivity provides another insight into the potential corrosivity of soils and waters. This property indicates the relative ease with which current can flow in the media.

When used alone, resistivity cannot predict aluminum corrosion performance. For example, the alloys listed in Table 3 show excellent resistance to seawater over long periods of time. Not only do they provide satisfactory service, they display that all-important faculty of corrosion arrestment in such an aggressive environment. The resistivity of seawater is in the range of 35 ohm-cm. Rarely does a soil reach this value.

ABRASION

Culverts are subject to abrasion where flows carry considerable rock. Culvert abrasion has been treated by Koepf (8) and will not be discussed here. Aluminum's abrasion performance should generally be divorced from corrosion since most natural waters are not aggressive toward aluminum. Usually an aluminum culvert installation is of interest for either its corrosion performance or its abrasion performance. No installation was found during this inspection program in which a combination of abrasion and corrosion condition existed for the same pipe.

FIELD INSPECTION PROGRAM

Site Selection

Since the introduction of aluminum culvert, the aluminum industry has maintained an inspection program of selected installations. A listing of culvert locations initiated in 1963 provides a wide geographical representation of soils and waters and was used by the Highway Applications Committee of the Aluminum Association for this study.

A large number of inspections were made in California where a variety of exposure conditions exist. The Pacific Northwest provided mild organic acid sites. The Midwest was represented from Kansas to Ohio, and the Great Plains from Oklahoma to South Dakota. Florida provided saltwater and sand as well as dense clays. Missouri, Indiana, Ohio, and California provided sulfuric acid runoff from either natural strata or mine waste. Wisconsin offered acid peat soil exposures. Northern Michigan waters and soils contained copper in small concentrations. Virginia and Maryland provided organic acid exposures characteristic of the Middle Atlantic and Northeastern region. Thus, a wide range of exposures was accumulated. The large number of samples allows a broadly based evaluation of aluminum culverts.

In Kansas, Oklahoma, and Louisiana, where many culverts are installed on roads having similar soil and water conditions, early inspections demonstrated that performance is also similar; in these cases only representative samples were taken.

TABLE 4

RATING CLASSIFICATIONS FOR ALUMINUM CULVERT INSPECTIONS

Rating	Appearance Description	Description of Corrosion
A	Excellent	1. No observed corrosion or significant metal surface staining.
B	Very Good	1. Superficial corrosion in the form of occasional pits confined to surface and/or cladding. Pits no more than 5 percent of surface; or 2. Extensive surface staining, grey cast in alkaline exposures to orange cast in organic acid exposures.
B/C	Good	1. Significant corrosion confined to cladding. Pit frequency unlimited except that surface etching less than 50 percent of the surface on the worst square foot observed. Usually evidence of corrosion build-up in pits. Staining may accompany attack but will be incidental to the overall effect.
C	Fair	1. Attack covering more than 50 percent of the surface on the worst square foot observed with corrosion limited to cladding. Will give appearance of etched surface. Occasional pit may appear to penetrate into core.
D	Poor	1. Attack, but not perforation, of the core alloy, generally accompanied by extensive surface corrosion.
E	Very Poor	1. Perforation of the metal.

Visual Rating

A rating system was developed to insure uniform reporting of on-site observations. The log sheet, shown in Figure 1, provides for culvert location, size, slope, and flow character.

CULVERT INSPECTION REPORT

SAMPLE NO. 68-

ROAD
STATE/COUNTY/CITY
ROUTE/PROJECT
MILE/STATION/INTERSECTION

REF. LOG NO.
TYPE INSTALLATION
DIAMETER GAUGE JOINT/SEAM
SLOPE LENGTH COATED/PAVED
INLET/OUTLET FILL HEIGHT
TERRAIN & FLOW

DATE SAMPLE BY
FLOW
ROCKS/DEBRIS
ALIGNMENT CONDITION
JOINTS CONDITION
PIPE CONDITION
COMMENT
PIPE MARKINGS

PHOTOS B/W
PHOTOS C
SOIL
WATER
COUPON

RATING OF PIPE (RATE INSIDE AND OUTSIDE SURFACES SEPARATELY)

RATING	DESCRIPTION	CORROSION	ABRASION
A	EXCELLENT	___NO CORROSION/STAINING	___NO EFFECT
B	VERY GOOD	___SUPERFICIAL CORROSION/STAINING	___SLIGHT ROUGHENING NO METAL LOSS
B/C	GOOD	___RANDOM CORROSION/STAINING	___SLIGHT EROSION LITTLE METAL LOSS
C	FAIR	___OVER 50% SURFACE CORROSION NO ATTACK OF CORE METAL	___EROSION-SLIGHT PROGRESSIVE METAL LOSS, SMALL DENTS
D	POOR	___HEAVY CORROSION ENTIRE SURFACE DEEP PITTING INTO CORE METAL	___ABRASION-SLOW PROGRESSIVE METAL LOSS, DENTS & GOUGES
E	VERY POOR	___VISIBLE PERFORATIONS	___ABUSIVE-CONSIDERABLE METAL LOSS, DENTS, GOUGES

RATING OUTSIDE
INSIDE
ABRASION

DATE INSTALLED YEARS SERVICE TO DATE
ESTIMATED LIFE (CORROSION) (ABRASION)

SOIL PH (LAB) r (LAB)

WATER PH (LAB) r (LAB)

Figure 1. Log sheet for on-site observations.

The visual rating system is described in Table 4. Similar ratings applied to the abrasion condition. Visual inspection was the principal means for determining uniformity of results throughout a culvert, or from one culvert to the next.

The interior was examined thoroughly. The invert was of greatest interest and where significant differences appeared between the invert and upper surfaces they were noted. The joints were examined for evidence of attack, cleanliness, and alignment. Where dirt was deposited in the invert a reasonable area was uncovered for inspection. Surface stain, pits, and substantial corrosion were recorded by using the rating scale.

The exterior surface of the culvert was uncovered to the extent practical within the limits of the site and the surface examined and rated for corrosion. When soil adhered to the culvert, it was wiped, washed, or lightly brushed to permit examination.

Sampling

A quart of soil taken adjacent to the culvert and a pint of water (when available) were collected for laboratory evaluation.

The final inspection step was to collect representative specimens of the metal culvert by use of a battery-driven hole saw, which removed a 1-inch diameter coupon. Samples were taken from invert, waterline, and, in some instances, the top of culverts. The orientation of the specimens was marked and coupons shipped with water and soil to the laboratory for examination. It should be noted that coupons were taken in locations where the most severe attack was observed.

Alternate Evaluation Method

It is possible to estimate the rate of corrosion of buried metal culverts by running polarization curves. The method has been published by Schwerdtfeger (9) as well as others. Equipment and methods have been reported by Lindberg (10), whose procedure permits calculation of the rate of corrosion occurring at the time of the test. In order to judge metal loss over a period of time, periodic testing is required so that an average rate may be found. The method has been tried on culverts in a number of areas but not over a long enough period to justify statements on cumulative metal loss. It was found, however, that in all cases the instantaneous corrosion rates indicated very little metal wastage. This was supported by the condition of the pipe sections.

LABORATORY EXAMINATION

The water pH and resistivity were measured using procedures and equipment specified in California Division of Highways Test Method 643-B (11). California Test method 643-B was also used as the basis for laboratory examination of the soil.

In a previous investigation (4), all soil resistivity measurements were taken in the field with a ground resistance tester (Model 263A Vibroground, Associated Research Inc., Chicago) at 2.5, 5.0, and 10-foot depths. This method was suspended in favor of the minimum resistivity technique (3).

The evaluation of metal coupons consisted of the following steps:

1. Examining visually as received from field;
2. Cleaning in chromic phosphoric acid;
3. Re-examining visually and at low magnification, rating for corrosion condition, and marking areas for metallographic sectioning, at least one section representing the most severe attack noted;
4. Preparing and polishing mounts, and then etching in 5 percent HF-10 percent H_2SO_4 for 30 to 40 seconds;
5. Photographing the complete mount at 5x, photographing areas of significant interest, and adjusting the corrosion rating if required.

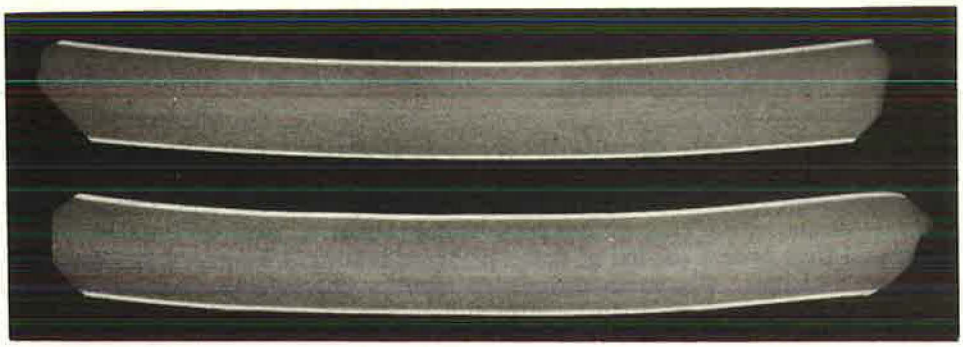
The final rating assigned to a culvert installation represents the worst condition observed on the soil side and the water side for any coupon from that culvert.

Figure 2 shows 5x metallographic mounts illustrating various ratings. Figure 3 further illustrates the condition of a B/C coupon. In Figure 2, the following is noted:

1. The cladding shows clearly as a light layer. The cladding is 5 percent of the total thickness on each side—e.g., 0.003-in. and 0.008-in. for 0.060-in and 0.164-in. thickness metal, respectively.
2. The sequence of B, B/C, and C photographs shows progression of attack on the cladding and the protection that the cladding is giving the core. Figure 3 shows the progression at 100x.
3. Only after the cladding has been expended over large areas does a significant attack of the core appear, usually in small isolated areas in the center of clad-wasted areas (D). Such attack is limited to a negligible percentage of the total culvert cross section.

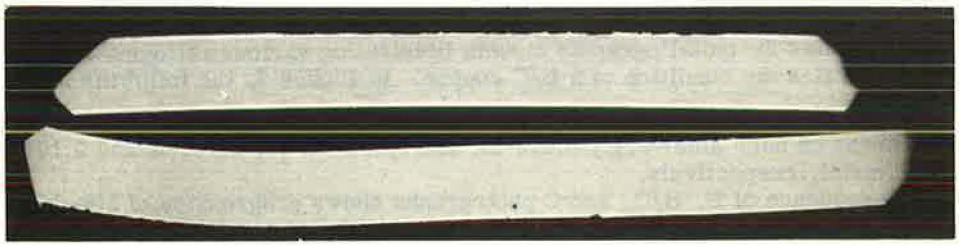
DISCUSSION OF RESULTS

Figures 4 through 7 show the number of culverts in each rating as a function of water pH, water resistivity, soil pH, and minimum soil resistivity. Each bar chart



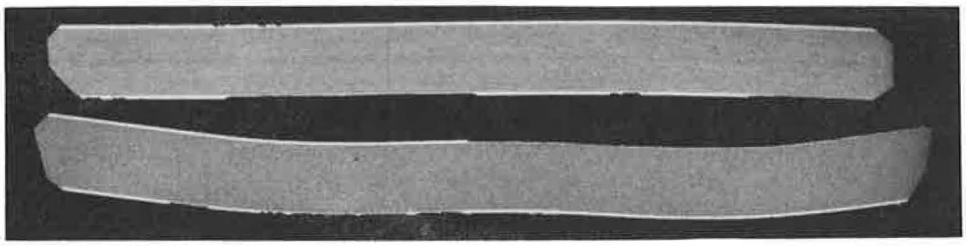
Rating A—No corrosion (Note: Cladding each side 5 percent of thickness).

Rating B—Up to 5 percent of surface attacked.

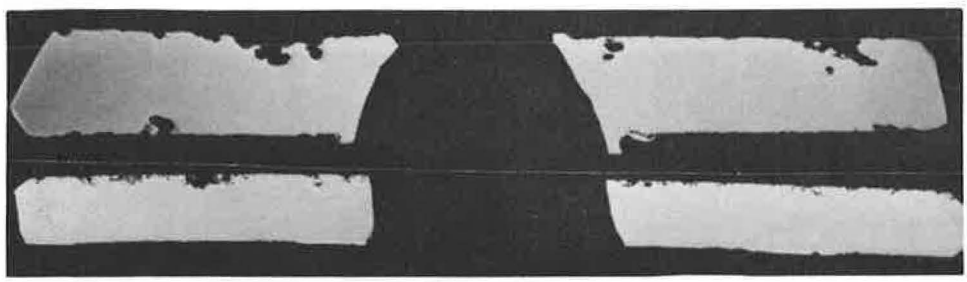


Rating B/C—5 to 50 percent of surface attacked on worst square foot (Note: Corrosion is only half depth on cladding).

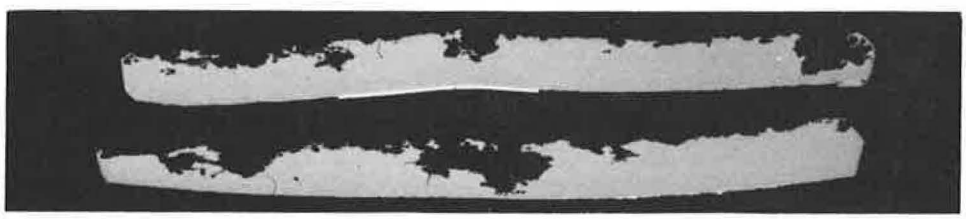
Figure 2. 5x sections showing examples of corrosion ratings.



Rating C—Over 50 percent of surface corroded on worst square foot
(Note: Corrosion confined to cladding).

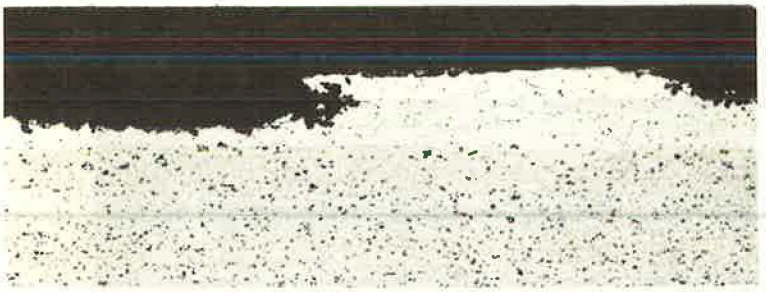


Rating D—Attack of core.

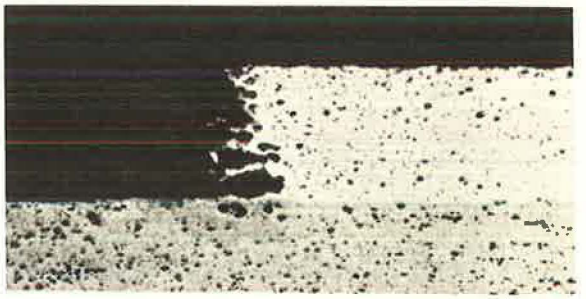


Rating E—Perforation of core.

Figure 2. (continued).



Attack limited to partial cladding thickness (100x).



Attack to full cladding thickness.

General surface appearance (5x) of a B/C-rated coupon.

Figure 3. Nature of cladding corrosion. Evaluation of coupon shows varying degrees of attack, progression of corrosion, and effectiveness of cladding.

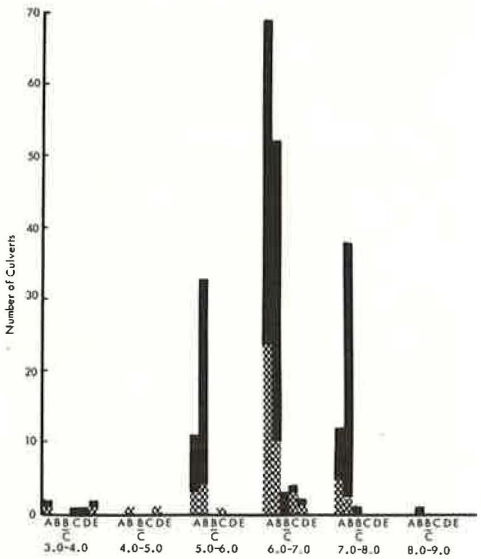


Figure 4. Culvert performance rating as influenced by water pH (cross-hatched bar represents culverts 6 years old and older).

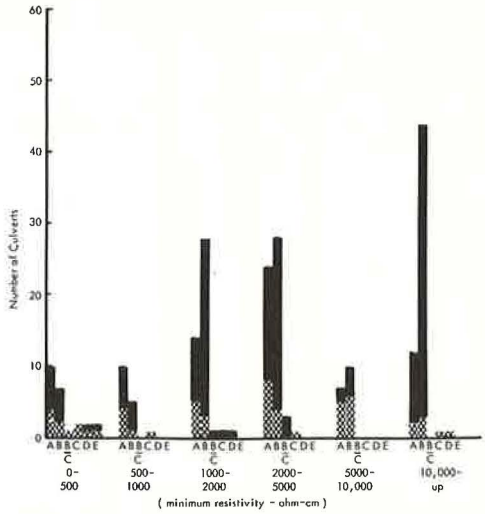


Figure 5. Culvert performance rating as influenced by water resistivity (cross-hatched bar represents culverts 6 years old and older).

designates the number of sites with 6 years or more of service, as well as the total number of sites within a category.

Most aggressive corrosion occurs at a pH below 4.0. As shown in Figure 4, exceptions to the low pH limit can occur where organic acids are responsible for the acidity of the water. The poorer ratings are attributed to sulfuric acid-containing runoff (12, 13) or to chloride-contaminated clays and mucks. Corrosive sites are characterized in Table 5. Limited data available in the pH range of 4.0 to 5.0 neither confirm nor refute the lower pH limit of 4.0, which was based on the response of aluminum to chemical environments. The Aluminum Association plans to seek additional field installations within the 4.0 to 5.0 pH range to better define this lower limit.

TABLE 5

EVIRONMENTAL DATA ON SITES PRODUCING D AND E RATINGS

Sample No.	Location	Water		Soil		Time in Service	Rating
		pH	Resistivity	pH	Min. Resistivity		
67-042	North of Maine Prairie Road, Salano, Calif.	6.9	1700	7.7	1330	5.0	D
67-180	Sweetwater Bridge, National City, Calif.	6.5	35	7.7	10.1	5.8	D
68-415	Severin Road, Port Charlotte, Fla.	6.2	35	6.7	—	6.0	E
68-449	Highway 17, Brunswick, Ga.	6.8	97	7.6	165	7.0	D
68-001	California 17, Santa Cruz County, Calif.	3.1	1440	5.4	1345	5.5	E
68-458	County Road No. 4, Meigs County, Ohio	4.5	45310	4.1	3524	8.0	D
68-461	County Road No. 33, Steudal, Pike County, Ind.	3.4	185	4.8	810	6.0	E
68-471	West of Coe, Pike County, Ind.	2.2	115	2.9	230	5.0	D

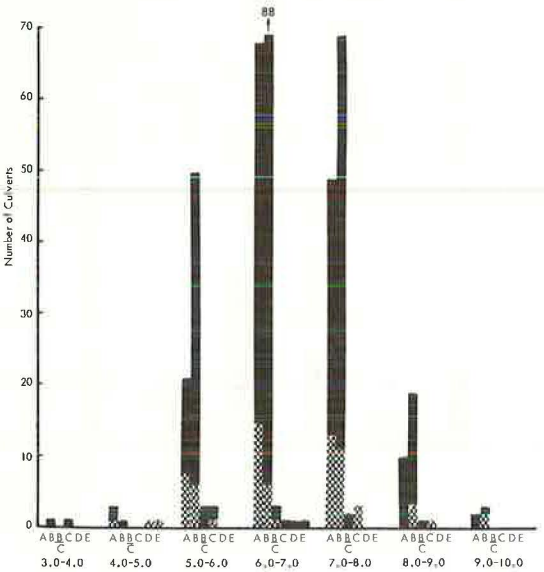


Figure 6. Culvert performance rating as influenced by soil pH (cross-hatched bar represents culverts 6 years old and older).

The data indicate that aluminum is not significantly attacked when the pH is greater than 4 and definitely will be attacked by mineral acids at pH less than 4.

Figure 5 shows lower ratings concentrated in the lowest resistivity range. Those sites responsible for the poorer ratings are listed in Table 5. Considering that low pH or muck conditions are present at these sites, the data indicate that water resistivity is not a factor affecting corrosion of aluminum culvert.

Soil pH is analogous to water pH with relation to its influence on corrosion (Fig. 6). Again the same sites listed in Table 5 are responsible for the D and E ratings throughout the pH ranges. The data support the use of uncoated aluminum culvert within a soil pH range of 4 to 9.

The influence of minimum soil resistivity is shown in Figure 7. Those sites characterized by highly acid runoff or by chloride-contaminated heavy soils (Table 5) caused lower ratings in most minimum resistivity ranges. It is interesting to note the performance of culverts in backfills with minimum resistivities below 1000 ohm-cm. Those represented by A and B ratings are saltwater-saturated granular soils, and are performing very well. Those represented by the C rating are in heavier, salt-saturated soils with some evidence of attack. Generally the attack follows the characteristics of concentration cell corrosion. The poorer ratings at the lower end are once again the acid sites or those with clay muck. The data indicate that a low-resistivity heavy soil such as clay or muck may be corrosive, but granular soils with low resistivity, and all soils with a minimum resistivity greater than 500 ohm-cm, appear to have little effect.

Field observations revealed that aluminum was subject to various degrees of staining. The degree and color of stain are known to be influenced by water chemistry (5, 14). These stains do not contribute to corrosion nor are they an indication of corrosion. The rating system provided for logging of stain as an indicator of surface appearance only.

Concentration cell corrosion has been noted when a combination of low resistivity and dense soil is encountered. The classic example is clay muck and saltwater. In this instance the poor circulation and discontinuous nature of the back-

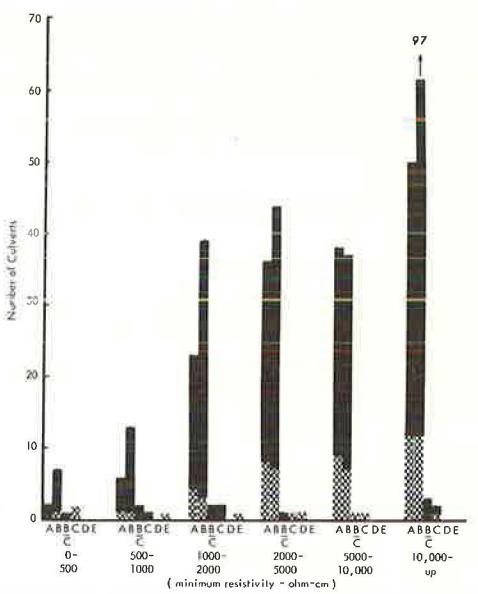


Figure 7. Culvert performance rating as influenced by soil minimum resistivity (cross-hatched bar represents culverts 6 years old and older).

fill does cause concentration cell corrosion, accelerating wastage of metal. Complete bedding and backfill of select granular material is recommended.

The field inspection revealed no selective attack in joints.

PREDICTION OF SERVICE LIFE

The prediction of culvert service life is a difficult, complex, and controversial undertaking. In the past, performance of aluminum culvert was based on one or a combination of the following: (a) service history generally less than 3 years (1, 2, 3), and (b) small sample size with detailed environmental data (2) or relatively large sample size with limited environmental data (4). This study represents the most comprehensive culvert survey undertaken to date, both as to number of culverts studied and detail on each culvert.

Researchers have not arrived at any agreement on a definition of culvert life. Haviland et al (3) suggest that point in time at which the culvert reaches a safety factor of one. Nordlin and Stratfull (1) suggest life to first perforation. It is felt that life to perforation is considered as a basis of comparison rather than an end of culvert usefulness. It must be apparent that culverts have been widely trepanned (as in the present study) to obtain specimens for examination.

For aluminum culvert, cladding and core must be analyzed separately in order to arrive at a meaningful service life figure. It is possible to estimate extent of cladding loss vs time based on coupon examination, polarization techniques, or an occasional relocated pipe. In the present study, which is based on examination of as many as five plugs from over 500 culverts, the greatest bulk of culverts showed no attack or so little attack that the cladding was not penetrated. In cases where cladding was penetrated, and therefore fulfilling its function of sacrificial protection to the core metal, less than 5 percent of the cladding was expended.

The average age of all culverts inspected in this study is 4.7 years. On this basis, the life of the cladding component is 94 years. However, this figure does not take into account the possibility that the limit of cladding effectiveness may, due to nonuniform attack, be reached before all the cladding is expended. If a 50 percent factor is assumed, and in the experience of the authors this is certainly conservative, a cladding life of 47 years is obtained. At this point in time 95 percent of the culvert wall thickness would remain—90 percent if one assumed simultaneous attack on both surfaces.

The second segment contributing to total surface life is the core material. Since core attack was not observed in 98 percent of the installations examined, it is impossible to determine pitting or weight loss rates from this study. However, valuable information may be obtained from burial studies conducted by the National Bureau of Standards, British Non-Ferrous Metals Research Association, ALCOA and ALCAN (15). These studies include the evaluation of alloy 3003, which possesses corrosion behavior very similar to alloy 3004, the core material specified for aluminum culvert. The data indicate an average pitting rate of less than 2 mils/year after a burial period of 10 years. Therefore, based on 16-gage material and again assuming that attack takes place simultaneously from both sides, a service period of 27 years may be anticipated for core material alone before initial perforation occurs. It is interesting to note that comparison of pitting rates between 5- and 10-year burial periods shows a 33 percent reduction for the longer service. This would indicate that a straight-line pitting rate may not be appropriately applied to the aluminum core alloy.

The combined effectiveness of cladding and core material results in an expected service period of 74 years before the first perforation is reached.

Application of weight loss data results in longer average service life. Ten-year burial figures show an average loss of 0.62 percent per year. If present-day design criteria are applied, a safety factor of two must be used, since buckling is controlled by this value. Applying the weight loss rate results in an 81-year period before half the core material is expended.

Again, combining the effectiveness of cladding and core material results in an expected service life of 128 years before structural integrity of the pipe wall is impaired.

CONCLUSIONS

Aluminum alloy culvert can be used where minimum soil resistivity is above 500 ohm-cm, and where pH of water and soil falls within the range of 4 to 9. Strong mineral acids, principally sulfuric, do attack uncoated aluminum culvert when the pH is less than 4.0.

The need for continuing evaluations is well recognized. The tedious portion of conducting such a survey—that of physically locating installations of particular interest—has been accomplished. Periodic inspections will report on performance of pipe in threshold environments, thereby more clearly defining upper and lower limits of water and soil parameters. Space limitations in this report precluded the inclusion of detailed data pertaining to each site. The Aluminum Association will be pleased to make this information available to any State Highway Department or federal agency upon written request.

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