Influence of Compression and Shearing Strains In Soil Foundations on Structures Under Earth Embankments

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When an earth embankment is constructed on a soil foundation, the weight of the embankment produces compression and shearing stresses in the foundation. The strains induced by these stresses cause settlement of the natural ground surface and an outward movement of the foundation soil from the region of the centerline of the embankment toward the toes of slope.

These settlements and outward movements have important implications in connection with the design and performance of the embankment and of structures under the embankment, such as culverts and sluiceways. Vertical settlements effect the shrinkage allowance in embankment design and influence the choice of an appropriate amount of camber in the flow line of a culvert. The outward movements due to horizontal shearing strains must be taken into account in the design of transverse joints in the barrel of culvert.

The amount of settlement and lengthening of 46 culverts and sluiceways are recorded in the paper.

● IT is axiomatic that engineering materials undergo strain when they are subjected to stress, and the soil, which is the most widely encountered of all engineering materials, is no exception to this general rule. Therefore, when a highway or railway embankment or an earth dam or levee is constructed on an earth foundation, we may expect the foundation soil to compress under the influence of the weight of the embankment. This compression strain manifests itself as settlement of the natural ground surface. Thus in Figure 1(a), if an embankment ABCD is constructed on a soil foundation, the original ground surface AED will settle to a new elevation such as AFD.

In addition to compressive stresses and strains which account for settlement of the natural ground surface, the foundation soil is subjected to certain horizontal shearing stresses and the strains resulting from these stresses cause an outward movement of the undersoil from the region of the centerline toward the toes of the embankment.

These shearing stresses are illustrated in Figure 1(b). The shear on a horizontal plane in the undersoil is essentially zero under the center of the embankment. It increases in magnitude to a maximum value at points approximately under the mid-points of the side slopes and then gradually decreases to zero at points some distance beyond the toes of the slopes.

A typical shearing stress-strain diagram for soil is shown in Figure 2. It is characteristically a curved line beginning at the origin. As the stress is increased the strain increases at an increasing rate. Finally, the strain increases an unlimited amount without further increase in stress. This stress which produces unlimited strain, i. e. shear failure, is the shearing strength of the soil.

It will be noted that a substantial amount of shear strain may develop in the soil even though the shearing stress is at a value considerably less than the shearing strength. It is these shearing strains which primarily account for outward movement of the foundation soil under an embankment even though the shearing stresses may be well below the strength of the soil. Of course, if the shearing stresses become large enough to equal or exceed the strength, the soil will flow outward from beneath the embankment and form mounds parallel to each side and beyond the toes of slope. This action will continue until the mounds are high enough and heavy enough to produce sufficient "back pressure" to stabilize the undersoil. Such failure conditions are not unheard of in engineering practice by any means, but they are not the subject of this discussion.

Movements in embankment foundation soils due to stresses which do not cause failure — compression strains and shear strains — may have important implications in

connection with the design of the embankment and particularly in connection with the design and performance of structures under the embankment, such as culverts and sluiceways.

With reference to the design of the embankment, it is widespread practice in earthwork to apply a shrinkage factor to the excavation quantities. Experience indicates that in practically all cases it takes more than 100 cu. yd. of cut to make 100 cu. yd. of fill. Therefore a designer must estimate a shrinkage factor by which the calculated fill quantities are increased when grades are being laid and balance points established. The shrinkage between cut and fill quantities is attributable to at least three different sources. First, and probably of least importance, there may be an actual loss of material from hauling equipment as the soil is transported from cut or borrow to fill areas. Second, the soil is usually compacted in the fill to a greater density than its natural density before being excavated. A unit volume of soil consists of both solid particles and void spaces or pores and the relationship between the volumes of these two phases

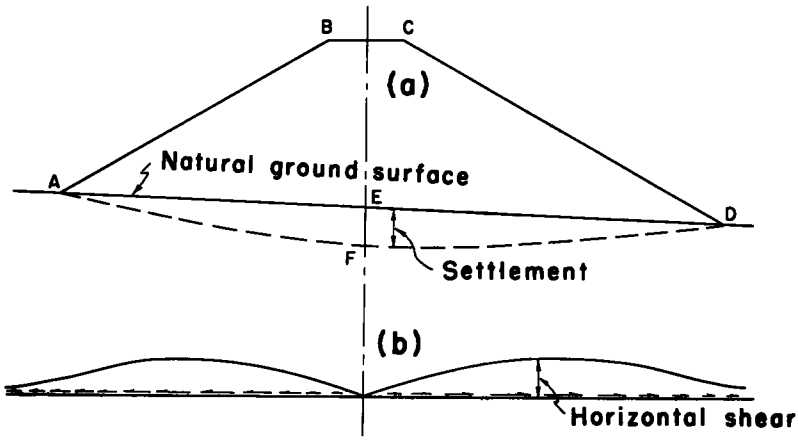

Figure 1.

influences the density of the soil. When a cubic yard is excavated and transported to a fill, the volume of the solid phase remains constant, but the volume of the voids is usually decreased in the fill, either by rolling, by traffic, by the weight of additional fill, by climatic influences or by any or all of these causes combined. Therefore, a greater volume of excavation is required to produce a given volume of fill.

A third source of shrinkage is the settlement of the natural ground surface as a fill is constructed and its weight causes the foundation soil to compress. Referring again to Figure 1(a), the calculated volume of fill is based upon the cross sectional area ABCDE. But the actual volume of soil required to construct the fill is a function of the area ABCDF. In some cases where settlement is considerable, this excess of the actual volume of fill over the calculated volume represents a substantial part of the shrinkage allowance.

When a culvert or sluiceway is constructed on or near the natural ground surface and covered by an embankment, the structure will settle downward and conform to the new contour of the surface, as indicated in Figure 3. In addition, because of the shearing strains in the foundation soil directed outward toward the toes of slopes, the structure has a tendency to lengthen. Also, the structure may

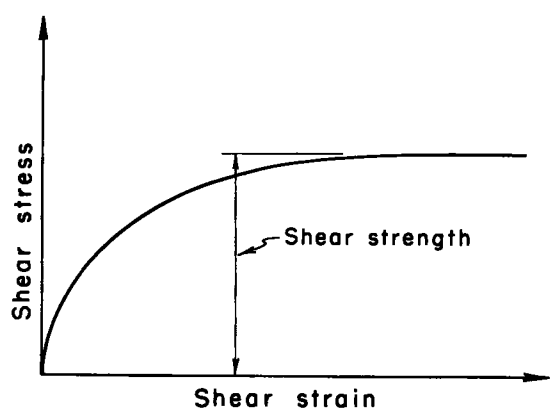

Figure 2. Typical shear stress-strain diagram for soil.

lengthen somewhat simply because the distance around the curve AFD is greater than the chord distance AED, but this source of increase in length is thought to be much less influential in most cases than that due to the outward shearing strains.

As a result of this tendency to settle and to lengthen, it is necessary to design the conduit structure to accommodate these movements without damage to the structure and without impairment of its function as a waterway. The settlement of the flow line of the conduit can be provided for by building the structure on a camber of sufficient amount that there will not be a sag in the flow line after the embankment is completed and settlements have developed. If sufficient camber is not provided and sag develops, water and silt will collect in the conduit. This prevents adequate inspection of the structure, restricts the waterway, and may hasten the deterioration of the conduit material. It may also provide a breeding ground for mosquitoes and be generally unsatisfactory from a number of viewpoints.

In order to accommodate the tendency to lengthen, it is rather common practice to construct monolithic concrete structures, such as box or arch culverts, in independent sections about 20 to 40 feet long. The construction joints between sections are designed to permit them to open up several inches as the sections pull apart due to the

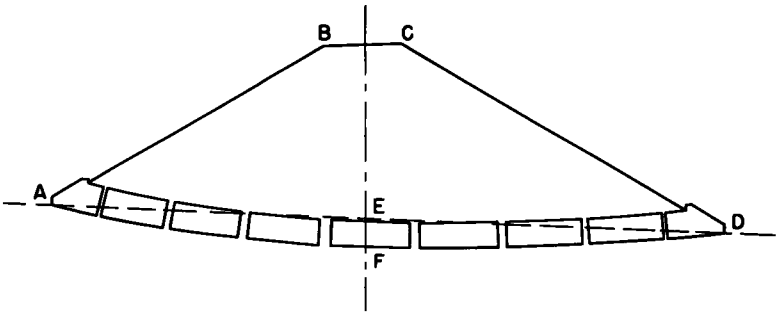

Figure 3. Culverts tend to lengthen as they settle.

outward shearing strains in the foundation soil. One type of design which has been successfully used is to construct a bell on the upstream end of each independent section into which the downstream end of the adjacent section extends in much the same fashion as bell and spigot sewer pipe are laid. This "bell and spigot" or "ship lap" design permits the sections to pull apart at the joints without vertical or horizontal faulting and prevents the intrusion of soil at the opened joint. In the case of a sluiceway through an earth dam, where water-tightness is a prime requisite, it may be advisable to install an accordion-type copper seal to prevent leakage.

Pre-cast concrete pipe with bell and spigot or tongue and groove joints may open up at the joints as the outward strains develop, but the amount of opening at each joint is usually small because of the relatively short length of sections and large number of joints. In most cases, satisfactory results are obtained by simply grouting the opened joints on the inside of the pipe after the embankment is completed and the shearing strains have developed.

Corrugated metal pipe culverts, because of their accordion-like fabrication are able to accommodate themselves to the lengthening tendency without special treatment. The author has heard infrequent rumors to the effect that even corrugated metal pipes have been pulled in two under severe conditions, but he has never actually seen such a case or received a factual report of one. Apparently this phenomenon, if it occurs at all, is very rare.

The first question which confronts a designer in his consideration of an appropriate amount of camber and the design of joints is "how much will the culvert settle and how much will it lengthen?" Presumably it would be possible to estimate these quantities in advance of construction by modern methods in soil mechanics. However, such a procedure would require extensive analysis of the compression and shearing stresses and their distribution in the soil foundation. Also, extensive soundings to establish the geological profile of the undersoil would have to be made. Many undisturbed samples

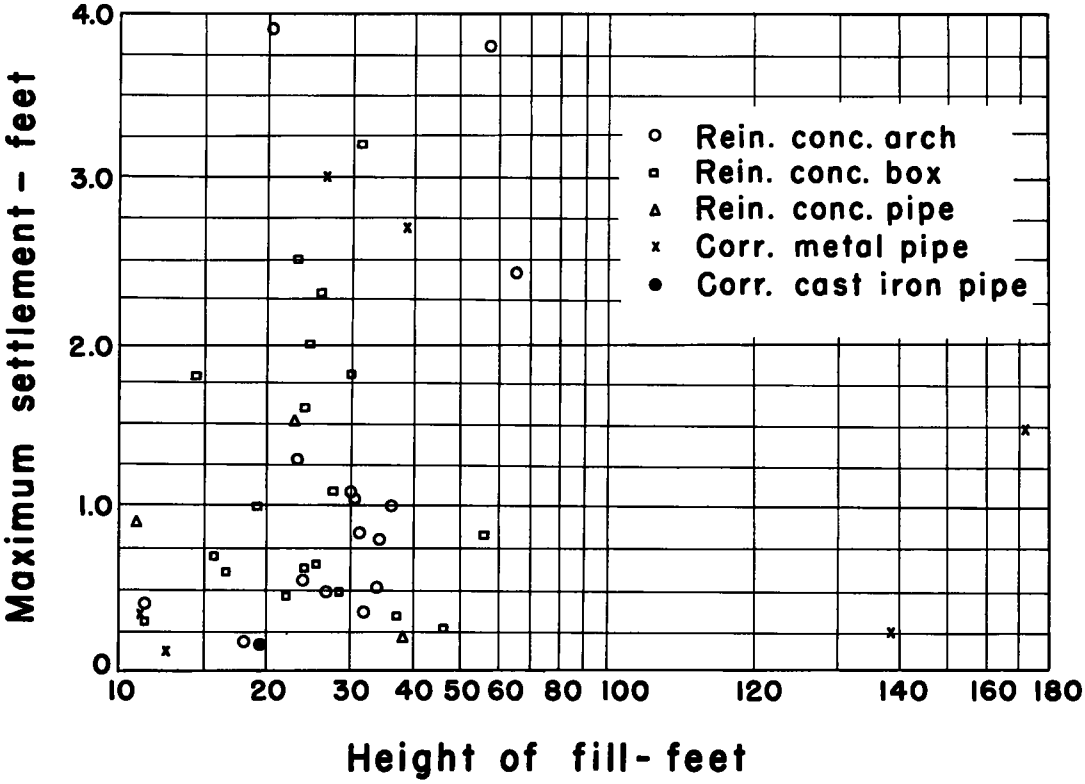

Figure 4.

of the undersoil would have to be taken and the samples tested in the laboratory to determine their stress-strain characteristics. Altogether, this would be a very expensive and time-consuming process. Since culverts under embankments are relatively small structures it is extremely doubtful whether a scientific procedure of this kind would ever be economically justified. Rather it appears that an empirical approach to the problem is the more feasible. A study of the actual settlements and joint opening of culverts and sluiceways of various kinds, under various heights of fill and in various localities, will, it is believed, yield sufficient information for a disigner to make an adequate estimate of the probable settlement and lengthening of a proposed structure of this kind.

The author has been collecting such information for a number of years, both by personal observation and from other sources. He is indebted to the Iowa Highway Commission, the North Carolina Highway Commission, the Rock Island Railroad, the Armco Steel Corporation, the Stanley Engineering Company and the Brown Engineering Company for supplying valuable information concerning the actual settlement and lengthening behavior of culverts and sluiceways.

Information relative to 46 culverts and sluiceways is given in Table 1 including maximum fill height, maximum settlement, and where available, the amount which the structure lengthened and the maximum joint opening.

The maximum settlement versus height of fill for the 46 structures is shown graphically in Figure 4. The magnitude of settlement varied over a very wide range; from 3 inches under 137 feet of fill in the case of an 84 inch corrugated pipe culvert in Cullman County, Alabama to 3.9 feet under 20.2 feet of fill in the case of a 9-by11-foot arch culvert in Woodbury County, Iowa. In connection with this diagram it is pointed out that plotted heights of fill are the design distance from the top of the structure to the top of the embankment. The final distance between the top of the structure and the top of the embankment is essentially equal to the sum of the plotted height of fill plus the amount of settlement. Also, the height of embankment above the natural

TABLE 1

SETTLEMENTS AND LENGTHENING OF CULVERTS

Item No.	Location County	Size and Kind	Max Fill Height ft.	Max Settlement ft	Total Length ft	Increase in Length ft	No of Joints	Max Joint Opening ft	Remarks
1	Tama	3'-8 x 5'-6 RC Arch	35 7	1.00	166	0 64	5	0 30	
2	Tama	4'-9 x 6'-1 RC Arch	30 8	0 83	161	0 31	6	0 13	
3	Tama	6'-3 x 9'-3 RC Arch	29 8	1.05	152	0 38	7	0 15	
4	Tama	10'-0 x 12'-6 RC Arch	11 1	0 39	67	0.07	3	0 03	
5	Tama	5'-6 x 8'-0 RC Arch	18.0	0 18	92	0.01	4	0 30	
6	Tama	4'-9 x 6'-1 RC Arch	26 6	0 46	110	0.51	4	0 30	
7	Tama	3' x 4' RC Box	23 5	0 64	116	0 38	5	0 15	
8	Tama	4'-9 x 6'-1 RC Arch	29 7	1.08	138	1 04	6	0 36	0 2' vert fault at one joint
9	Tama	10'-0 x 12'-6 RC Arch	21 0	1 00	104	0 25	4	0 12	
10	Tama	12'-0 x 12'-6 RC Arch	22 9	1 30	105	0 65	4	0 34	0 1 to 0 2 vert fault at three joints
11	Tama	3'-8 x 5'-6 RC Arch	33 6	0 52	144	0.47	5	0 19	
12	Tama	3'-8 x 5'-6 RC Arch	33 6	0.80	151	0 28	5	0 15	0 1 vert. fault at one joint
13	Pottawattamie	96" Corr Metal	35 8	2 70	170	-	-	-	Outlet raised 1'. 3' silt deposit at center
14	Pottawattamie	90" Corr. Metal	27.2	3.00	144	-	-	-	Inlet raised 0 4', outlet settled 0 6' 3' silt deposit at center
15	Harrison	5'-6 x 8'-0 RC Arch	57 0	3 80	274	1 28	7	0 45	Inlet settled 0 5' Outlet settled 0 2'
16	Cullman (Alabama)	84" Corr Metal	137 3	0.25		-	-	-	2' sand bedding on ledge rock
17	Buncombe (No Carolina)	66" Corr Metal	172 0	1 46		-	-	-	
18	Fremont	3'-8 x 5'-6 RC Arch	64 0	2 53	267	-	8	0 33	Tongue and groove joints broken Some horizontal faulting
19	Des Moines	4' x 5' RC Box	55 0	0 83	420		14	0 19	Outlet raised 0 35'
20	Union	3' x 5' RC Box	29 0	1 81			6	0 69	
21	Jasper	3' x 5' RC Box	27 0	1 11	481	-	none	-	Several transverse cracks up to 1" + in barrel
22	Ringgold	4'-9 x 6'-1 RC Arch	31.3	0.36	207	-	-	-	
23	Olmsted (Minnesota)	48" RC pipe	37 5	0 22	170	-	-	-	Shallow depth to ledge rock
24	Harrison	3' x 3' RC Box	21.8	0 47	121	-	-	-	
25	Pottawattamie	3' x 4' RC Box	15 4	0 70	111	-	-	-	
26	Pottawattamie	3' x 3' RC Box	11 2	0 30	70	-	-	-	
27	Mills	3' x 4' RC Box	24 8	0 65	121	-	-	-	
28	Mills	8' x 8' RC Box	16.2	0 61	94	-	-	-	
29	Marion	3' x 4' RC Box	45 2	0 25	202	-	-	-	
30	Johnson	3' x 4' RC Box	36 4	0.35	158	-	-	-	
31	Johnson	2' x 3' RC Box	26.8	0 48	138	-	-	-	
32	Linn	60" Corr Metal	8 0	0 27	62	-	-	-	
33	Monroe	36" Corr. Metal	12 3	0 12	74	-	-	-	
34	Monroe	36" Corr Metal	11 0	0.28	64	-	-	-	
35	Woodbury	4' x 4' RC Box	30 5	3 20	223	0 79	5	0 29	Inlet settled 0 5'. Outlet settled 0 7'
36	Woodbury	9' x 11' RC Arch	20 2	3 90	157	1 34	4	0 52	Inlet settled 1 6' Outlet settled 1 5'
37	Woodbury	8' x 8' RC Box	22 9	2.50	177	0 71	4	0 37	Inlet settled 0 6' Outlet settled 0 6'
38	Woodbury	48" RC Pipe	22 1	1 50	186	-	15	-	Inlet settled 0 1' Outlet settled 0 7'
39	Woodbury	6' x 6' RC Box	14 0	1 80	126	0.29	4	0 19	Inlet settled 0 6' Outlet settled 0 4'
40	Woodbury	3' x 4' RC Box	23.4	1 60	191	0 50	5	0 18	Inlet settled 0 3' Outlet settled 0 4'
41	Woodbury	4' x 5' RC Box	25 2	2 30	196	0 23	5	0 17	Inlet settled 0 5' Outlet settled 0 1'
42	Woodbury	4' x 4' RC Box	24 0	2 00	181	0.52	4	0 31	Inlet settled 0 3' Outlet settled 0 2'
43	Woodbury	42" RC pipe	10 6	0 90	132	-	10	-	Inlet settled 0 3' Outlet settled 0 3'
44	Woodbury	4' x 4' RC Box	18.7	1 00	154	0 18	4	0 12	Inlet settled 0 3' Outlet settled 0 3'
45	Clayton	36" Corr Cast Iron	19 3	0 16	99	-	32	-	Shallow earth bedding on ledge
46	Clayton	36" Corr Cast Iron	23.7	0 58	117	-	38	-	Shallow earth bedding on ledge

ground surface adjacent to the structures was probably somewhat greater than the plotted height, since many of them were probably installed as projecting conduits. However, factual data in this regard is not available except in a very few cases. Furthermore in some cases the settlement of the ground surface adjacent to the structure may be somewhat different than that of the structure itself, but in general, the structure settles just about the same amount as the settlement of the original ground surface.

The shotgun distribution of the plotted points clearly indicates that the settlement is not a function of height of fill alone, but other factors must be taken into consideration in making an estimate of probable settlement. The most important of these factors are the depth of the foundation soil to ledge rock and the compressibility characteristics of the soil stratum. The influence of this first factor is clearly shown in the case of the 84-inch corrugated pipe in Alabama (Item 16, Table 1). Here the depth to ledge

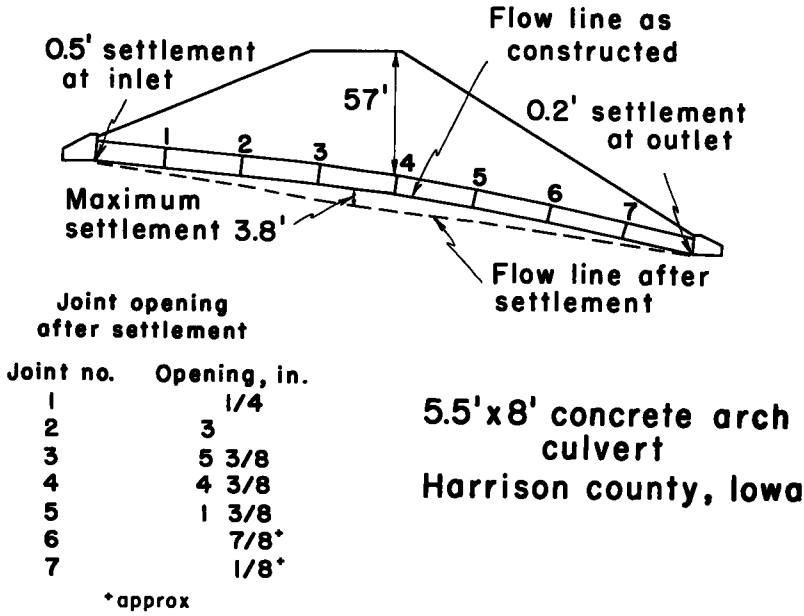

Figure 5.

rock was very shallow. In fact ledge was removed to a depth of 2 feet below the pipe and replaced by a selected sand bedding on which the pipe was laid. The settlement of this pipe was only 3 inches under 137 feet of fill. The 48-inch concrete pipe culvert in Olmsted County, Minnesota (Item 23, Table 1) also illustrates this point. The actual depth to ledge on this job is not known, but the rocky character of the soil in the stream bed and adjacent limestone outcrops indicate that it is very shallow. The settlement in this case was 2½ inches under 37.5 feet of fill.

In contrast to these cases of minor settlement, culverts constructed in the loess region of western Iowa (Woodbury, Harrison, Pottawattamie and Fremont Counties) have experienced large settlements, frequently in the range from 2 to 4 feet. Although specific information relative to the depth of soil foundation is not available in these instances, it is generally recognized that the soil mantle in this region is very thick; in the order of magnitude of 300 to 500 feet.

As a general rule, the settlement of a culvert is roughly proportional to the height of fill over it throughout its length. That is, the settlement is greatest under the roadway portion of the embankment, where the fill height is greatest, and diminishes toward zero at both the inlet and outlet. This observation is illustrated in Figures 5, 6 and 7. Figure 5 shows the initial flow line profile of a monolithic, 5½-by-8-foot concrete arch culvert and the final profile after construction of the 57-foot embankment. This culvert is located in Harrison County, in the Peorian Loess region of western Iowa. The

maximum settlement of the flow line is 3.80 feet and it occurred between Joints 3 and 4, which are near the centerline of the embankment.

The amount which the various construction joints in this culvert opened up as the structure lengthened under the influence of shearing strains in the foundation soil is shown. There has been no vertical or lateral faulting at the joints and no noticeable infiltration of soil, although the loess in this area is highly erosive in character. Figure 8 is a photograph showing the bell-and-spigot joint construction employed in this design. The bells overlapped the adjacent upstream sections about eighteen inches.

Figure 6 illustrates the settlement of a 96-inch corrugated metal pipe culvert under a 36-foot railroad embankment in Pottawattamie County which is also in the Peorian

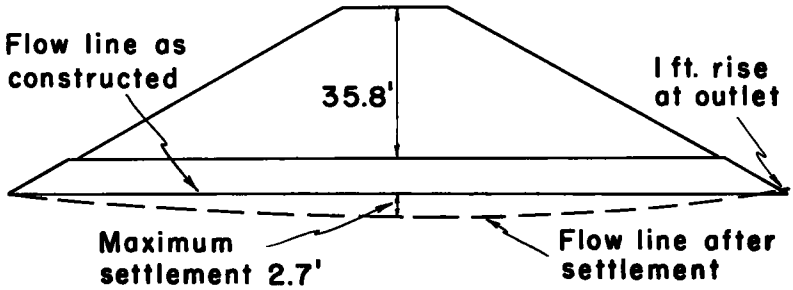

Figure 6. A 96-inch corrugated-metal-pipe culvert, Pottawattamie County, Iowa.

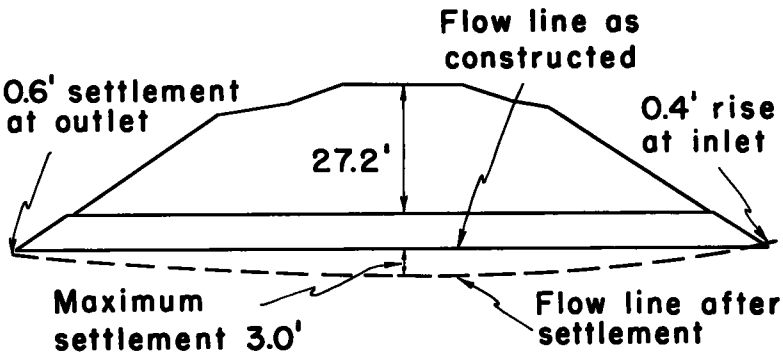

Figure 7. A 90-inch corrugated-metal-pipe culvert, Pottawattamie County, Iowa.

Loess region of western Iowa. The maximum settlement of the flow line of this culvert was approximately 2.7 feet near the centerline of the embankment. The elevation of the inlet was not affected by the construction of the fill, but the outlet raised about a foot.

The settlement of the flow line of a 90-inch corrugated pipe culvert in the same vicinity is illustrated in Figure 7. The maximum settlement of this culvert, under 27 feet of fill was approximately 3 feet near the centerline of the embankment. The inlet raised about 0.4 foot and the outlet settled about 0.6 foot.

Both of these corrugated pipes were constructed on a relatively flat, uniform grade from inlet to outlet. As a result there is an accumulation of silt in the culverts up to 3 feet deep. There is no evidence of the bolted joints pulling apart, although it is probable that rather large outward shearing strains were developed in the soil foundation.

Another matter of considerable interest which is indicated by this study is the fact that the amount which the structure lengthens is roughly proportional to the amount of settlement. No quantitative statement of this relationship can be made, but it seems clear that the greater the settlement, the greater the amount of lengthening of the structure. Also the joint opening is greatest near the center of the embankment and

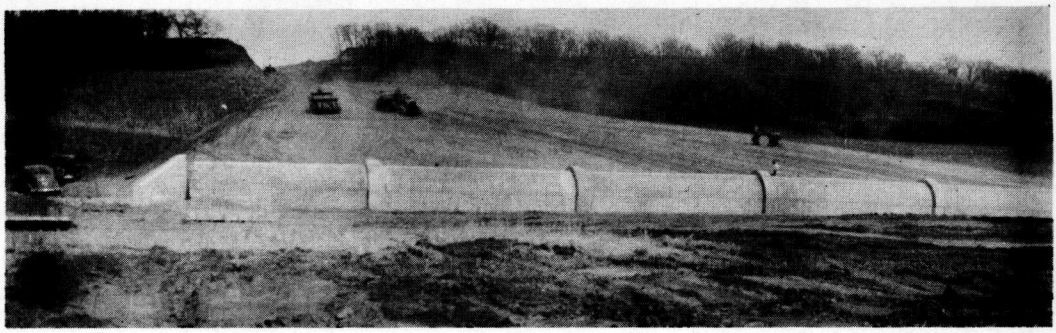

Figure 8. A 5.5' x 8' concrete arch culvert in Harrison County, Iowa, showing bell and spigot type construction joints.

decreases to an insignificant amount toward the ends of the structure, as shown in Figure 5.

Still another matter of interest is the time relationship between construction of the fill and the development of settlement. Although, very little detailed data are available, the indications are that most of the settlements developed during and shortly after construction of the fill. Three culverts in Woodbury County, (Items 35, 36, and 37, Table 1) were built and covered during the 1952 construction season. The settlements of these structures was first measured in February 1953. A re-check of the settlements in January 1954 indicated that no further settlement developed during this intervening period.

In the case of a concrete arch culvert in Fremont County (Item 18, Table 1) the maximum settlement of the flow line was 2.14 feet immediately upon completion of the fill on September 4, 1930. The settlement increased to a maximum of 2.53 feet after a lapse of nearly 9 years, according to the following schedule of observations.

Day of Observation date	Time after Completion of Fill months	Maximum Settlement feet
9-49-1930	0	2.14
10-16-1930	1.5	2.19
5-20-1931	9.5	2.25
6-13-1933	33.5	2.32
4-3-1936	67.0	2.43
6-16-1939	105.5	2.53

In summary, observations show that settlement of the natural ground surface under an earth embankment varies widely depending upon the height of the embankment, the depth of foundation soil to ledge rock and the compression stress-strain characteristics of the foundation soil. Also, it is indicated that there is an outward movement of the foundation from the region of the centerline of the embankment toward the toes of slopes, caused by shearing strains within the undersoil.

These vertical and horizontal movements have important implications in connection with the design of an embankment and of culverts or sluiceways beneath the embankment. It is believed that a study of the settlement and lengthening of a large number of culverts or sluiceways in relation to environmental conditions, such as height of fill and depth and compressibility of the foundation soil layer, will provide a practical means for estimating the probable vertical and horizontal movements of a proposed conduit.

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