No. 3 of 6

Progress Report

AN INVESTIGATION OF THE LOADS ON THREE CORRUGATED CAST IRON PIPE CULVERTS UNDER ROCK EMBANKMENTS

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ACKNOWLEDGEMENT

This investigation was undertaken upon the suggestion and invitation of Mr. Mark Morris, Research Assistant of the Materials and Tests Department of the Iowa Highway Commission. It has been conducted jointly by the author and Mr. J. W. Johnson, of the same department. Mr. Johnson organized the study and took the original data on culvert No. 1 and assisted materially with the data on No. 2. Acknowledgement is also due to Mr. K. M. Bennett, resident engineer on the job, who aided in taking the original data and whose courtesy and willingness to cooperate are appreciated.

SYNOPSIS

In all of the many experiments which have been conducted by the Iowa Engineering Experiment Station to determine loads on culverts under earth fills, gravel, clay or loam materials have been used for the embankments. It was not known, therefore, what the action of an embankment constructed of solid rock excavation would be, and there was considerable question whether loads would be produced on culverts under a fill of this kind in accordance with Marston's theory of loads on conduits which had been checked repeatedly with the finer, more granular materials. There is a feeling among engineers that a rock fill would "arch over" a culvert and the full theoretical load would not be effective on the structure. When, therefore, it was learned that the Iowa Highway Commission contemplated constructing three cast iron pipe culverts under rock embankments in Clayton County, it was decided to make a study of the loads onthese culverts with a view to answering this question,if possible.

The investigation has been productive of results. Conclusive proof exists that the rock fills acted in a manner quite analogous to an earth fill and that loads were produced on the various sections of culvert pipes in accordance with Marston's theory. Concrete evidence is presented to show that the load on a culvert is not a direct function of the height of the embankment and the degree of flexibility of the culvert only, but the relative settlements of the culvert and various planes of the embankment must be taken into account if the loads are to be determined. The importance of the character of the bedding of a culvert is also clearly demonstrated.

Although two of the three culverts studied have failed structurally, they failed solely because the pipes were overloaded. There is no evidence to indicate that the pipe used in the culverts were of an inferior quality or that they did not support the full load which they might have been expected to safely support.

Extensive experimental observations of loads on culverts have shown that maximum loads do not develop until considerable time has elapsed after the embankment has been placed. There is evidence that the specimens in this study are no exception. It is proposed, therefore, to observe these culverts throughout another season before a final report is prepared.

INTRODUCTION

During the winter of 1928-29, the Iowa State Highway Commission permitted the contractor on their Primary Road Grading Project No. 586 to install three 36 " spiral corrugated cast iron pipe culverts as substitutes for three 3'x3' reinforced concrete box culverts originally designed for this project. This substitution seemed desirable because of the fact that the contractor wished to work all winter and the concrete culverts could not conveniently be built until warmer weather, thus holding up the completion of the road for some two or three months.

This project is located near the southern boundary of Clayton County on United States Highway No. 55 just south of Millville, a small settlement about half way between Guttenberg and Luxemburg. The road, which, for three or four miles is newly located, passes through some very rough, rocky, territory and necessitates some heavy solid rock excavation with high embankments constructed of the material thus excavated. These three cast iron culverts were, therefore, located under quite high rock embankments.

The Iowa Engineering Experiment Station, under Director Anson Marston, has been studying loads on conduits in trenches and under embankments for nearly a quarter of a century and Director Marston has discovered and developed the "True Theory of External Loads on Conduits in Ditches and Embankments due to Fill Materials." Parts of this theory, together with the

experimental work supporting it, have been published * from time to time as various steps have been developed and a manuscript is in preparation at the present time in which Marston will present the latest extension of the theory which makes it generally applicable to all conditions of conduit loading. Practically all of the experimental work relating to this subject has been conducted using granular materials such as loam, clay, or gravel or combinations of these soils for the embankments, and the conduit theory assumes the existence of a definite, though not determined, modulus of compression for the filling material. It is apparent that an embankment constructed of solid rock excavation would be of radically different character from one constructed of those materials just mentioned and conjecture fails to give a satisfactory answer to the question of whether such an embankment would act similar to gravel or loam fills and whether loads would be produced on culverts in accordance with Marston's theory.

These three installations in Clayton County seemed to afford an excellent opportunity for observation of the action of rock fills, and upon invitation of Mr. Morris and Mr. Johnson of the Materials and Tests Department of the Iowa State Highway Commission, a study of the loads on the three culverts was undertaken in the spring of 1929.

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*Bulletins Nos. 31, 36, 47, 49, 57, 2nd Progress Report, 76, 79, 80, 93, Iowa Engineering Experiment Station.
Also paper by Anson Marston presented to Advisory Board on Highway Research, Washington, D.C., Dec. 12, 1929.

It should be emphasized that these culverts were not in any sense experimental specimens. They were installed by the earth contractor, and while, to all appearances, the work was well and carefully done, there was not the care and control exercised which characterized the experimental culverts constructed and studied by the Experiment Station in its research work. They were what might be termed practical field installations.

LOCATION AND DESCRIPTION OF CULVERTS

As stated above, these culverts were located on Primary Road, Project No. 586. The following table gives some general information of interest.

Table 1.

Culvert No.	Station	Skew Angle	Grade of Invert	No. of Sections	Length (feet)	Max. Fill over top of pipe ft.
1	108+00	20°R*	23%	33	99	19.3
2	113+55	None	25%	39	117	27.4
3	167+00	45°R*	5%	46	138	20.2

*R=Right when facing downstream (West).

The material used in the construction of the culverts was spirally corrugated cast iron pipe. The pipes were of two diameters and the spiral corrugations were of the same pitch in each so that one pipe section could be threaded into a larger one for about 3 inches, to form the joint between pipes. Then another small pipe was threaded into the other end of the larger pipe and thus the culvert was built, alternating a large and a small pipe until the desired length was attained. Each section of

Fig. 1

Photograph of Culvert No. 2 taken on April 30, 1929 just after completion. The character of the subgrade of the embankment, and the bedding of the pipes are well indicated by the view.

pipe is 3'-3 long and screws into the preceeding pipe 3 inches, so that each section lays up 3'-0 of length of the culvert.

There are 10 corrugations in each section. These are 13/16" deep in the outside sections and 11/16" deep in the inside sections. The diameter of the inside pipes is 36 inches in the clear, while the outside pipes have a diameter about 1-3/8" larger to permit them to screw on to the smaller ones.

The Materials and Tests Department of the Highway Commission conducted three-edge bearing tests on a group of similar pipe submitted by the contractor, previous to the installation of the culverts. The results of these tests are as follows.

Table 2
3-Edge Bearing Strength Tests

Specimen No.	Wt. lbs.	Inside Diam. Inches	Length Ft.	Total Load Pounds	Strength lbs. per lin. ft.	Strength lbs. per. lin. ft. per ft. diam.
1	646	36.0	3.22	17,050	5300	1767
2	663	36.0	do	18,700	5820	1940
3	635	36.2	do	19,950	6200	2055
4	752	37.5	do	17,700	5510	1763
5	739	37.5	do	22,500	6990	2230
Av.				19,180	5960	1950

BEDDING OF PIPES

The ground in the vicinity of these culverts is underlain with solid limestone rock. In fact, the soil on top of this rock is very scanty and varies from nothing to several inches or perhaps a foot in depth. It is impossible to sink a spade for any depth into this soil covering without encountering rock,

which may be quite friable and disintegrated, but nevertheless quite solid. The photograph, in figure 1, gives a good idea of the nature of this soil.

It was considered impractical to excavate into this soil to form a bedding for these pipe culverts, so that a layer of earth was placed on the surface and the pipes bedded into this material to give them a fairly uniform bearing on the bottom side. This bedding varied in thickness from several inches to a foot depending upon the contour of the ground.

An approximate sketch of the bedding conditions of these culverts is shown in figure 2. Sufficient care was taken to shape the earth cushion to fit the pipe that the bedding may be classed as "Ordinary Projection Bedding",* at least at the time the pipe were placed. However, some sections of culvert No. 2 were apparently placed on relatively thin layers of earth over the rock substrata, so that when the embankment was being placed and the pipes settled downward, the bedding condition became materially modified and approached the "Impermissible Bedding" condition. This undoubtedly affected the supporting strength of these pipes and will be discussed in more detail later.

EMBANKMENT MATERIALS

The embankment materials over these culverts consisted of limestone rock taken from solid rock cuts adjacent to the fills. These cuts were blasted and excavated with a steam

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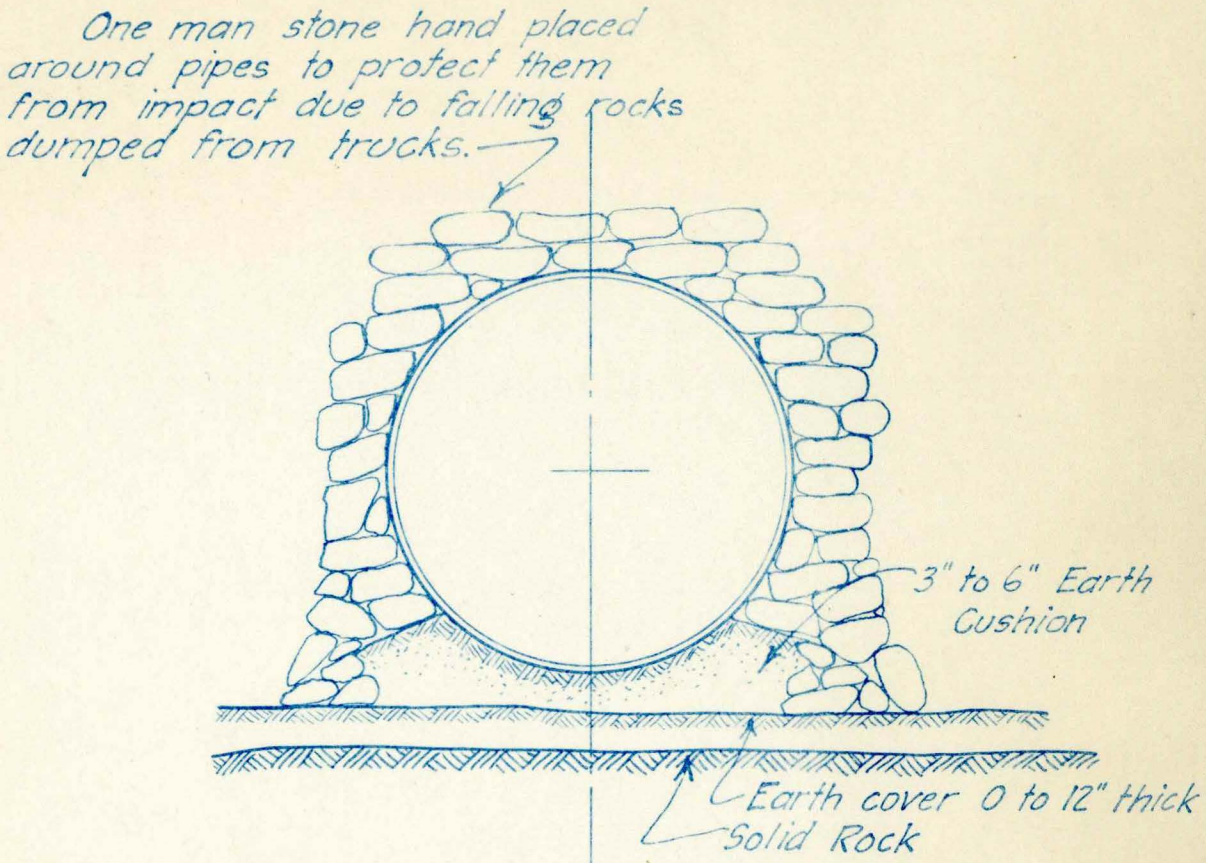
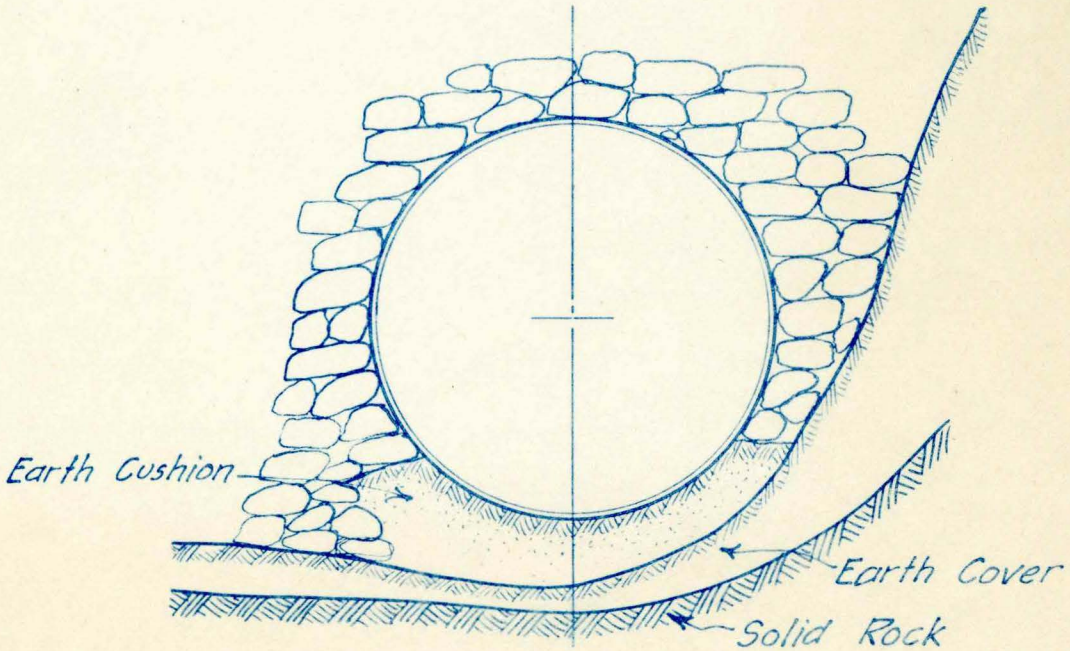
*See Anson Marston's paper presented to Advisory Board on Highway Research, Washington D. C. Dec. 12, 1929.



APPROXIMATE BEDDING CONDITION
Culverts 2 & 3



APPROXIMATE BEDDING CONDITION
Culvert No. 1

Fig. 3

Photograph of outlet end of culvert No. 1 taken on April 30, 1929. Note character of rock fill and method of placing hand stone over crown of pipe. Also the sharp rise of the natural ground to the south (right) of the culvert should be noted.

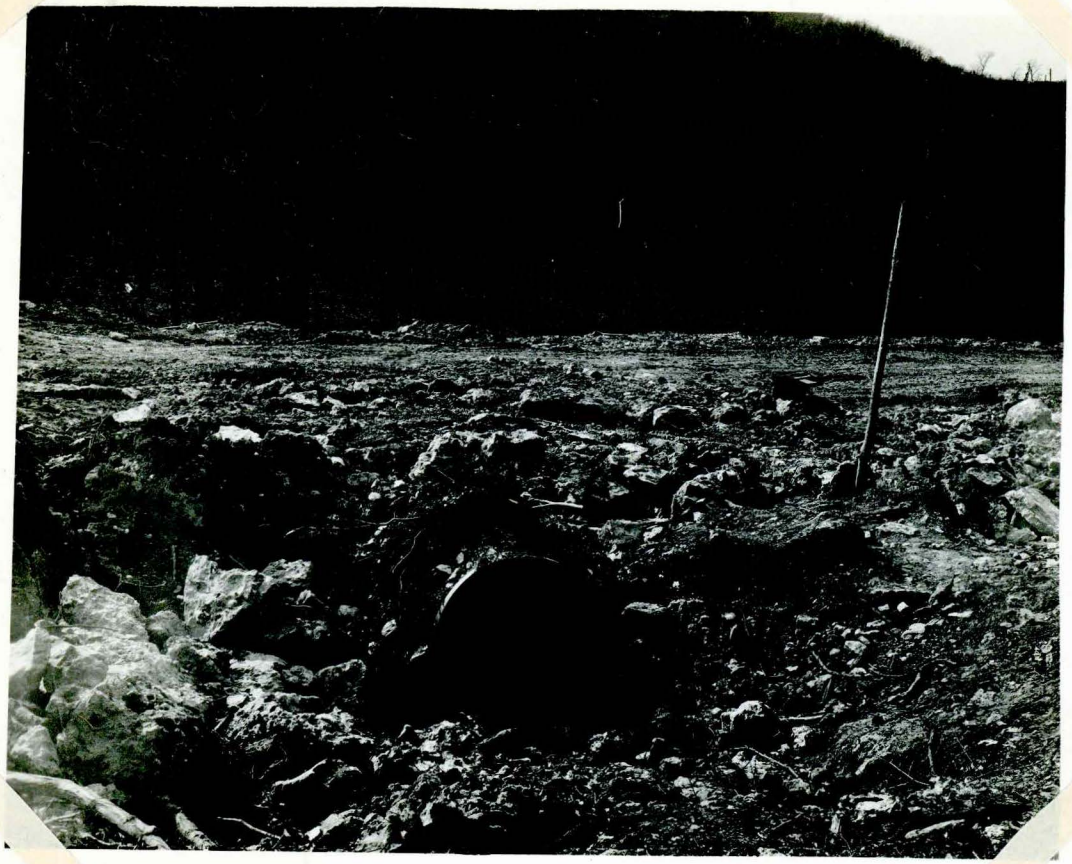


Fig. 4

Photograph of inlet of culvert No. 1. taken on April 30, 1929. Top of pole stuck in ground at right of culvert is the final grade of the embankment.

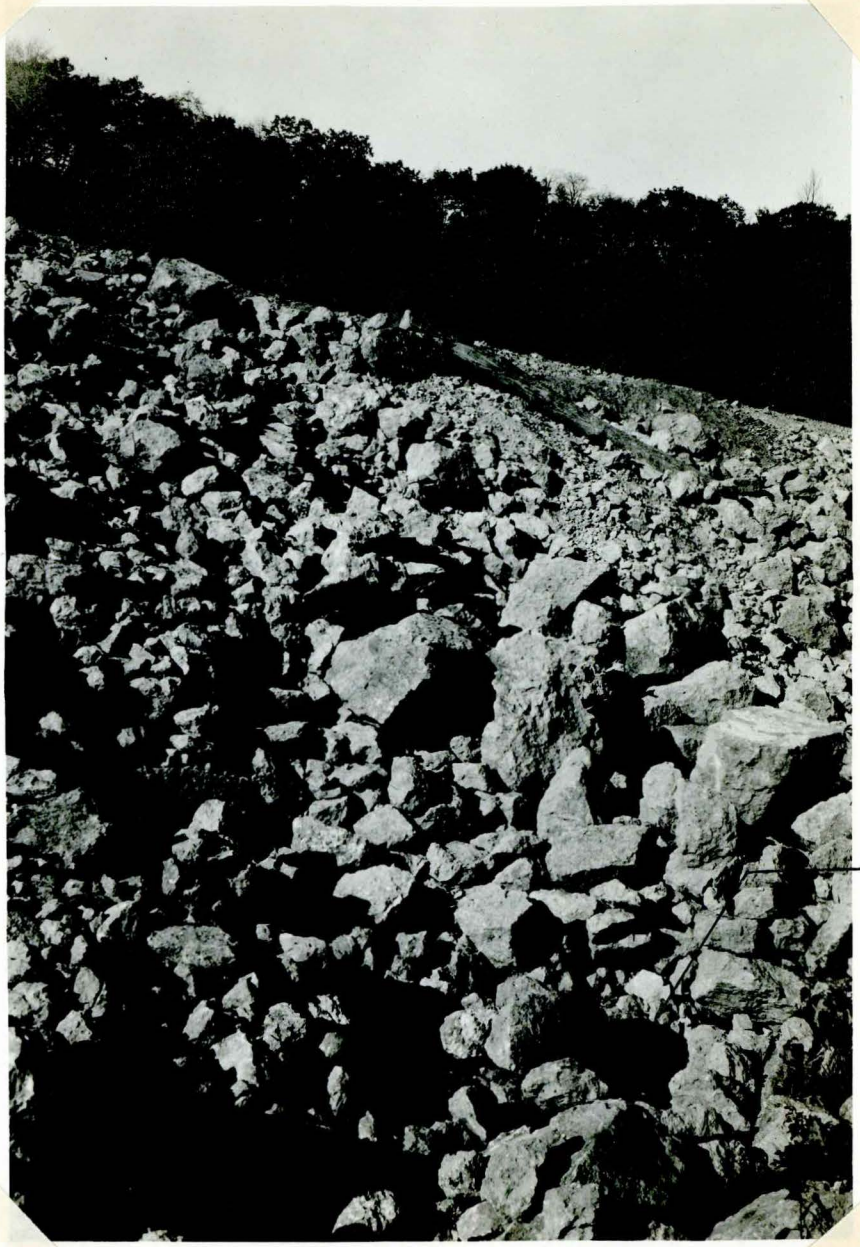


Fig. 5

Photograph of inlet of culvert No. 3 (lower right) The rocky character of the embankment is well illustrated by this view. Taken Oct. 16, 1929.

shovel so that the rocks in the fills varied in size from chats up to individual rocks 3 to 4 feet in dimension, all of them jagged and irregular in shape. A very good idea of the character of these embankments may be gained from the photographs of figures 3, 4, and 5. This material was carried up to within 1.5 to 2 ft of the finished grade. The balance of the fill was a yellow clay soil used for topping the rock and providing a workable surface.

The resident engineer and the contractor say that every precaution was taken to prevent any of these large rocks from falling onto the culvert pipe and striking an impact blow which might have been injurious to the pipe. They state that the culverts were first surrounded with one man stone, (see figures 2 and 3) hand placed in a more or less horizontal position with the idea that this method of placing would protect the pipes from injury by later dumping operations, and that the rocks so placed would tend to arch over the pipes and relieve them of some of the load due to the embankment. The photograph of the outlet of culvert No. 1 shown in figure 3 indicates the care with which this material was placed around the pipe.

It has been very difficult to determine the unit weight of these fills. Some measurements of this property were made by tamping small stones and clay as solidly as possible into a hemispherical vessel of known volume and weighing the contents. These measurements indicate a unit weight of about 140 lbs. per cu. ft. This figure may be somewhat in error, but is about as close as could be determined with the facilities at hand.

The coefficient of internal friction of an embankment of this character is likewise very difficult to determine by ordinary means. However, a fairly accurate idea of this property may be gained from the natural slope of the embankment, which seemed to run about 1.4 to 1. This slope corresponds to a coefficient of internal friction of about .7.

OBSERVATIONS AND MEASUREMENTS

In the course of the investigation of the loads produced on these culverts, two distinct phenomena were observed and measured, viz. the distortion of the pipes and the settlement of the invert and the top of the pipes. The distortion of the pipes was measured by noting the shortening of the vertical diameter and the lengthening of the horizontal diameter, by means of an internal expanding caliper which could be read directly to .1 inch and easily estimated to .05 inch. It is probable that these measurements are not accurate closer than .02 or .03 inch.

The settlement of the invert of the pipes was noted by setting up a transit at the outlet end of the culverts and reading a short rod held perpendicular to the grade of the invert at the center of each pipe section. Two hub stakes, one at the outlet and one at the inlet end were driven when the original elevations were taken and all subsequent readings were referred to the line between these stakes. This method of taking elevations gave a measurement of the settlement of the pipes in a direction perpendicular to the barrel of the culvert and not in a vertical direction. The set-

tlement of the top of the pipes was obtained by adding the deformation of the vertical diameter of the pipes to the settlement of the invert.

Culvert No. 3 was constructed during the month of February, 1929, and the embankment built over it shortly thereafter. No measurements or elevations on the invert were made on this culvert so that no detailed data is available for a study of the loads on it. On October 16, 1929, this culvert was visited by the writer and found to be in a very bad state of failure. As shown in figure 9 it appears to be entirely collapsed throughout about one-half of its length and most of the remaining sections are badly deformed.

The author crawled into this culvert from the inlet end about as far as section No. 6 and inspected the interior of the pipes by means of an automobile spotlight. The havoc wrought by the embankment loads was surprising. Only the first two sections had escaped cracking at this end. Section No. 3 was cracked on the bottom and two sides. Sections 4 to 19 inclusive were cracked top, bottom and sides and the deformation of these pipes became increasingly great as the interior of the embankment was reached. Section No. 20 was collapsed to such an extent that the crown and invert were probably less than 6 inches apart. Through the narrow spaces between the segments of this section it was possible to see the jagged edges of subsequent sections which were apparently collapsed for some distance beyond No. 20.

A similar inspection from the outlet end also revealed only two sound pipes. No. 43 and 44 were cracked top, bottom

and sides and No. 42 had collapsed in a manner similar to No. 20. Whether or not all the sections from 20 to 42 are collapsed, it was not possible to determine, but there is little doubt in the writer's mind that such is the case.

The engineer and the contractor state that this culvert was bedded on an earth cushion throughout its length in much the same manner as culverts No. 1 and No. 2. It has been included in this study because of some very interesting features affecting the loads upon it, even though no measurements of the various phenomena were made.

The original data on culvert No. 1 was taken on April 16, 1929. At that time a fill of about 4 or 5 feet had been placed over a short stretch of the culvert. Original data on culvert No. 2 was taken on April 30, 1929, after the pipes were all in place but no fill had been deposited over them.

The next inspection of culverts No. 1 and 2 was made on August 2, 1929. At this time the rock embankment was completed over these two culverts, but the 2 feet of clay topping had not been placed. No. 1 was found to be in good condition, and showed no effects from the load on it except an apparent settlement under the higher portions of the embankment.

Five sections of culvert No. 2 were found to be cracked on this date; sections 11, 12, 13, and 14 in one group and section 23 in another. The invert of the culvert had settled very materially under the higher portions of the fill and, where the two groups of cracked pipe were located, the invert showed a distinct upward bulge as compared to the rest of the culvert indicating that at these places the pipes

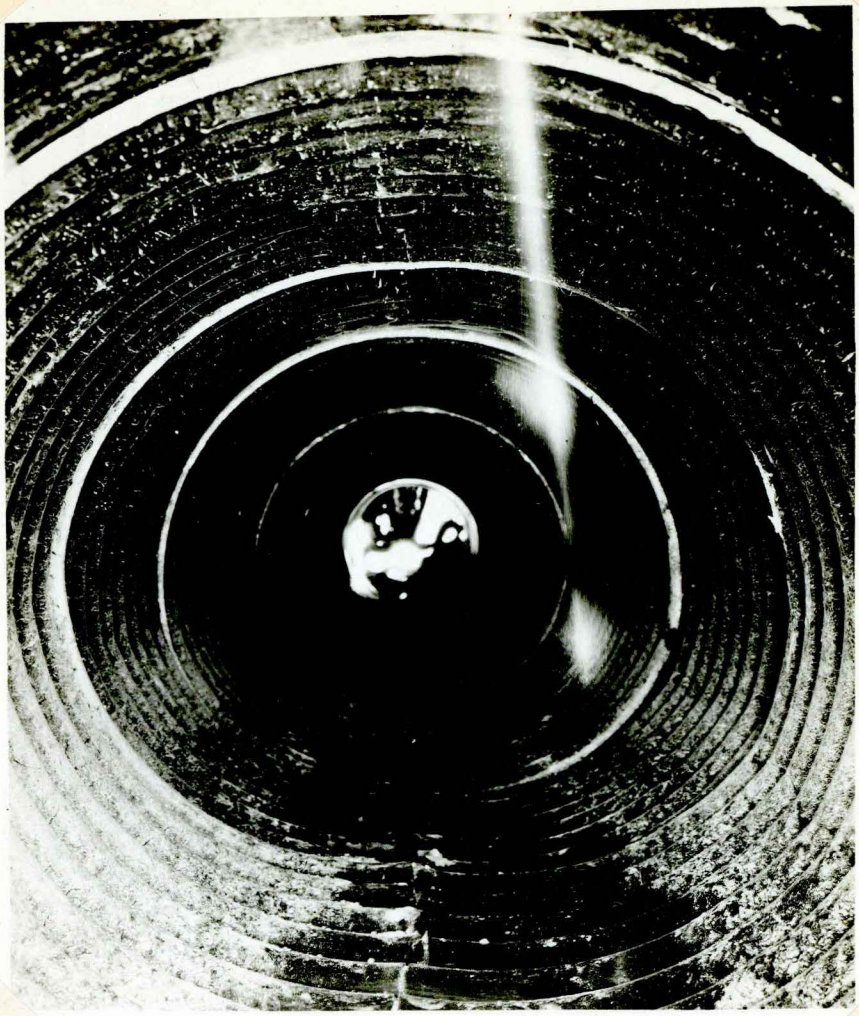


Fig. 6

Interior photograph of culvert No. 2. The section in the immediate foreground is No. 14 and the view is toward the inlet. The bottom and top cracks may be seen plainly. The side cracks are not so clear, but may be discerned in the two nearest sections just above the lower right corner of the picture.

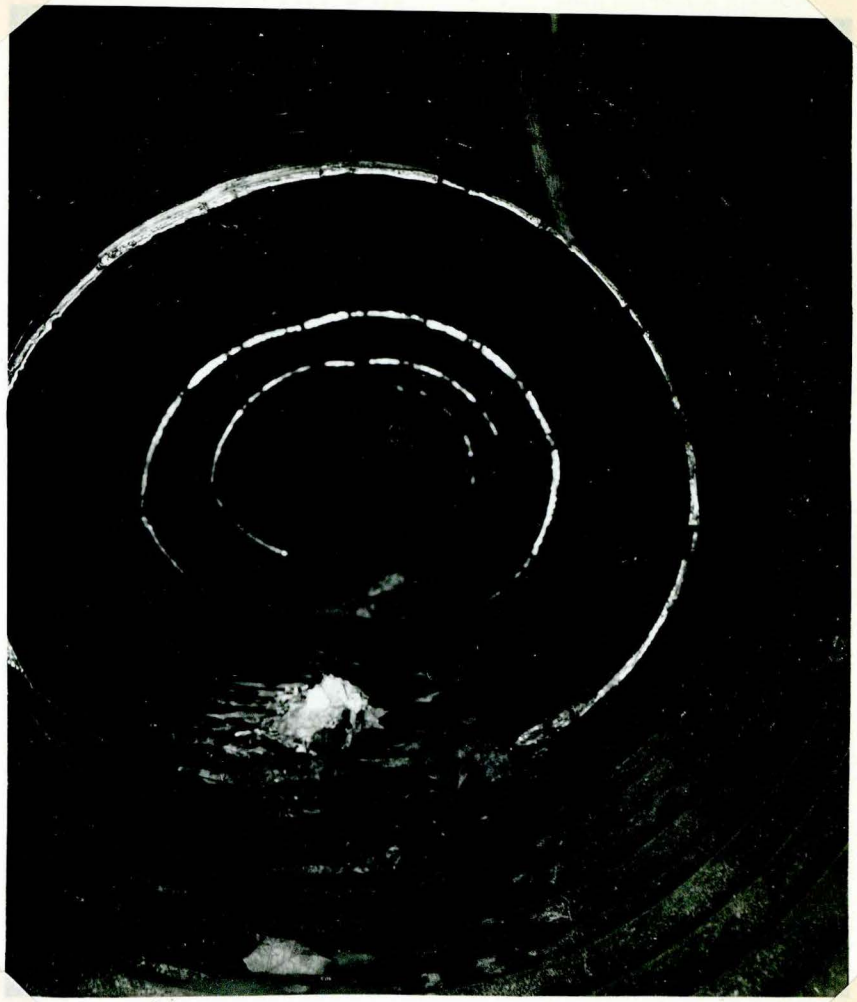


Fig. 7

Photograph of the interior of culvert No. 3. This picture was taken from section No. 4 looking downstream. The high lights on the edge of several inside pipes indicates the deformation of Sections 5, 7, 9, and 11. Top, bottom, and side cracks may be seen in section No. 5.

had approached the solid rock sub-strata in the course of settlement and could not recede from the load to the same extent as adjacent pipes did.

Although no measurements of either diameter changes or invert settlements were made during this inspection, the behavior of the pipes as the embankment had been placed, especially in culvert No. 2, was very evident and extremely interesting. Under very heavy and increasingly distressing loads, the pipes had been distorted and forced downward into their earth bedding as the height of fill increased. Those sections which had sufficient soft material under them were able to recede from the load, as the embankment settled, to such an extent that they escaped being distorted enough to cause them to crack. Sections 12, 13, 14, and 15 and section 23 of culvert No. 2, however, apparently had approached the solid rock sub-strata in their downward movement, and when the material under them became very greatly compressed (or perhaps they had even come in actual contact with the rock), they could not settle further and were forced to stand still and pit their inherent structural strength against the forces of the embankment. This phenomena caused very heavy loads to develop on these pipes, so that they were cracked, while sections 16 to 21 incl. remained sound, although under greater heights of fill, and they stand as very eloquent, though mute, evidence of the importance of relative settlements in the production of loads on conduits under embankments.

The exact height of fill at which these five sections cracked is not known definitely. It is estimated, however, that sections 12, 13, and 14 probably cracked under a height

of about 15' to 17'.

These culverts were again visited on October 16, 1929. Between this inspection and the one previous, the clay topping had been placed and the embankments were now up to their final height and the road was open to traffic. Complete measurements of diameters and settlements were made on this date and the inverts were found to have settled as shown in the profiles in figures 8 and 9. All of the pipes in culvert No. 1 were sound, but two additional sections, No. 15 and No.22 were cracked in culvert No. 2. Also, sections 12, 13, and 14 which were cracked only on the bottom and on the two sides on August 2, were found to be cracked in the top on this date, and these cracks were bright and new, indicating that they had occured quite recently. Otherwise, the cast iron thus exposed to the damp air in the culverts would probably have been rusted.

The last inspection, to date, was made on Nov. 13, 1929, and another set of diameter and settlement measurements made. Here again, culvert No. 1 was found to be sound, while three additional pipes Nos. 9, 10, and 16 were cracked in the bottom only in No. 2. These three cracks were very slight and did not extend the full length of the pipe sections involved, and there had been no measurable decrease in the verticle diameter of these pipes since the previous inspections.

These inspections reveal a progressive failure of the pipes in culvert No. 2 which is summarized in Table 3. The changes in vertical and horizontal diameters of both culverts are shown in Tables 4 and 5.

TABLE 3

Section No.	Date of Inspection and State of Embankment			
	April 30. No. Fill	August 2. All but 2' Completed	October 16. Fill Completed	November 13. Fill Completed
9	sound	sound	sound	○
10	sound	sound	sound	○
11	sound	○	○	○
12	sound	○	○	○
13	sound	○	○	○
14	sound	○	○	○
15	sound	sound	○	○
16	sound	sound	sound	○
22	sound	sound	○	○
23	sound	○	○	○

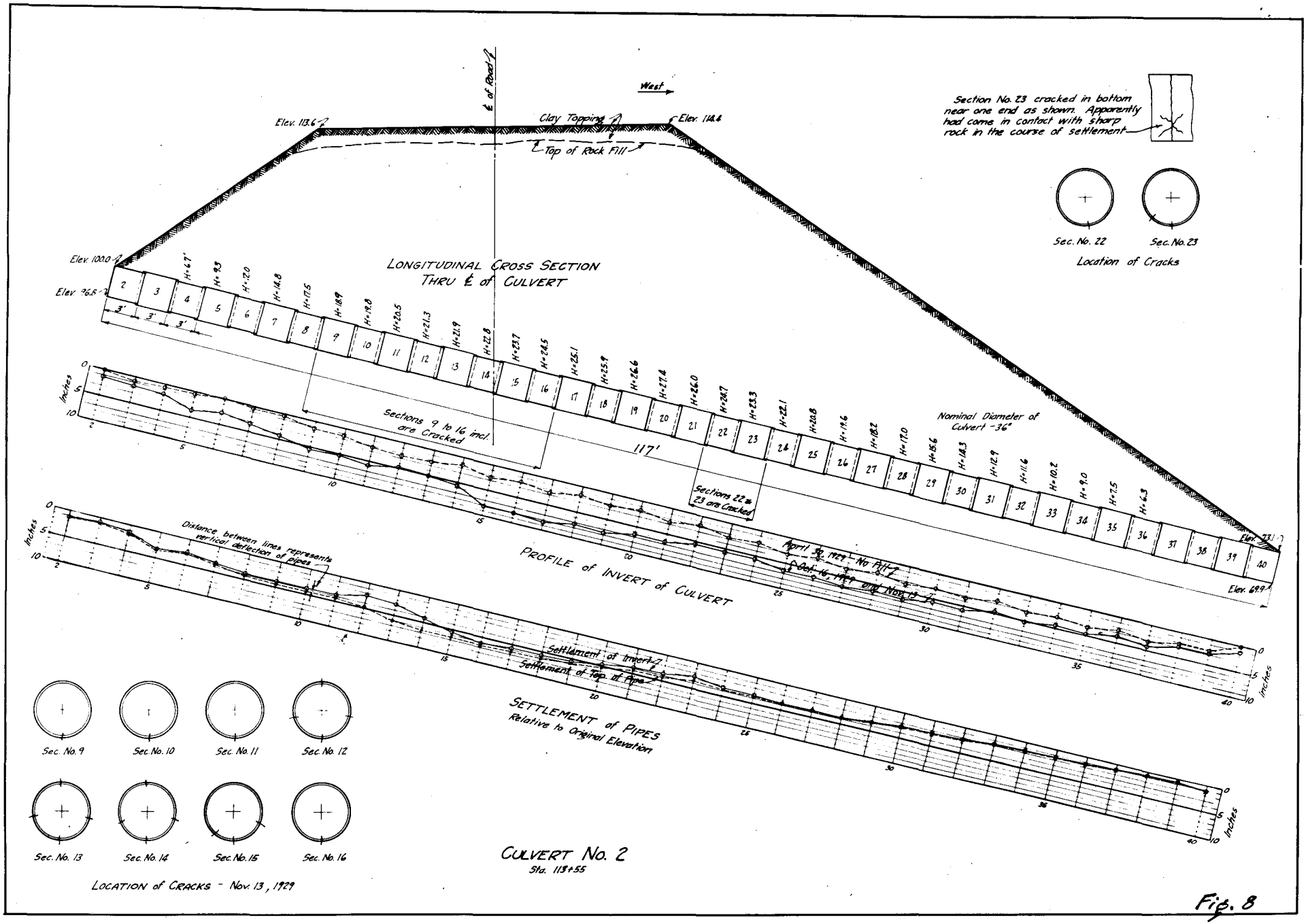
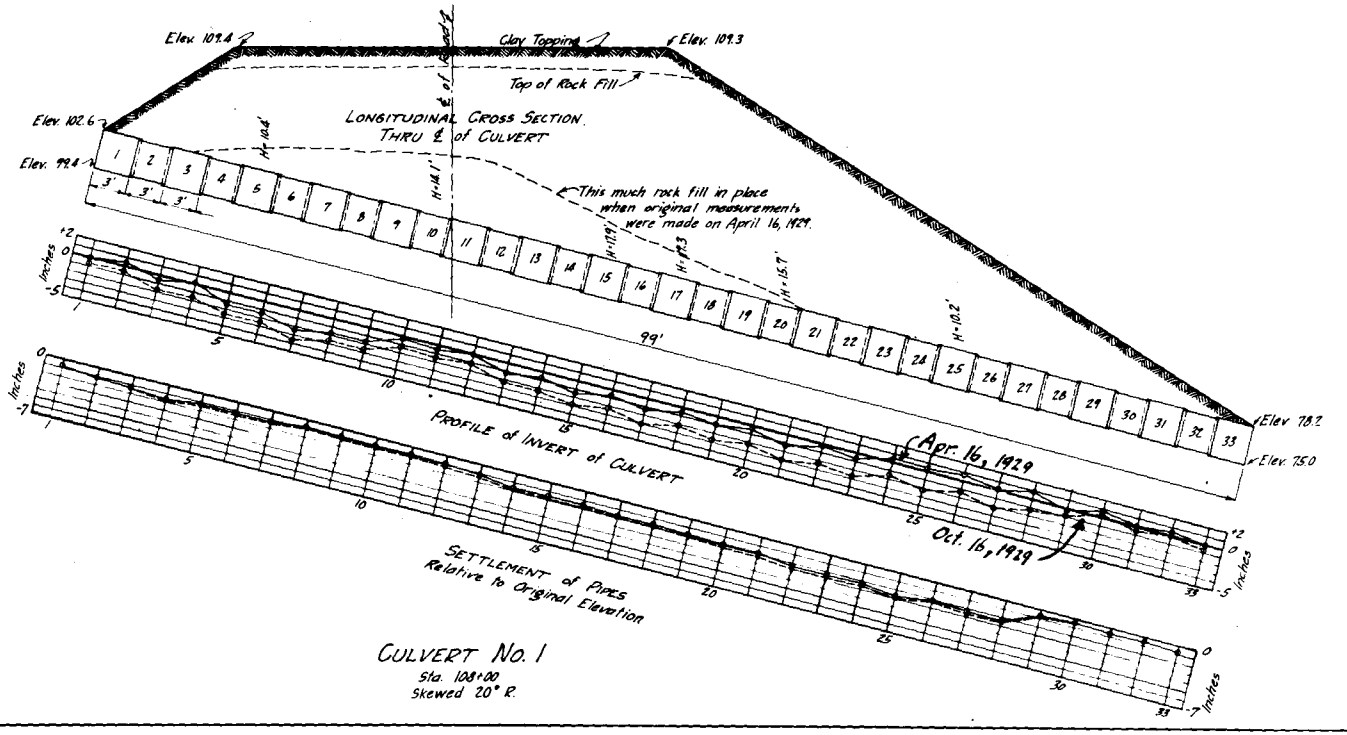
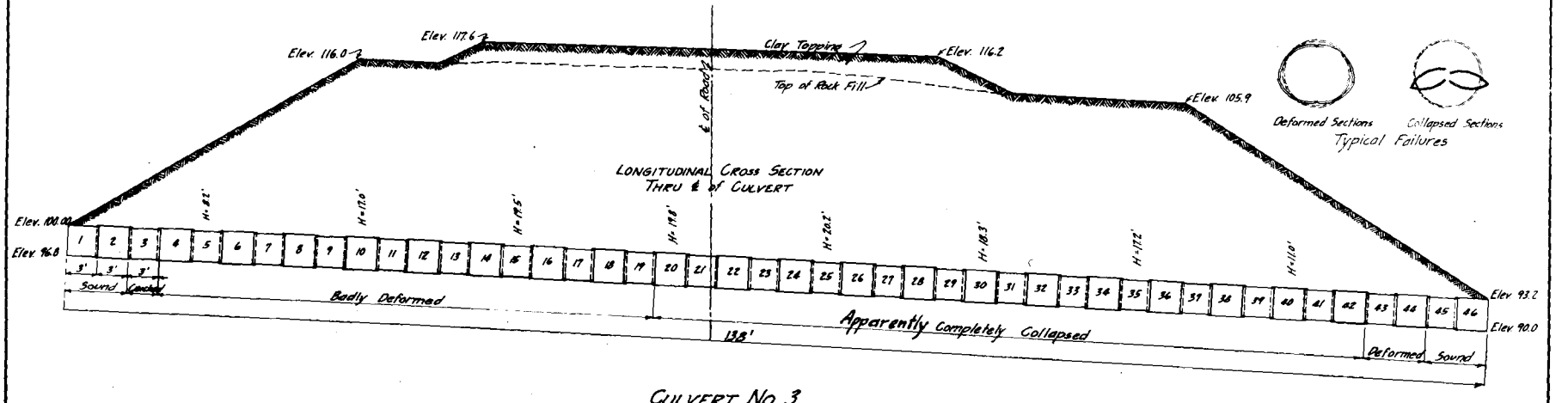

West
℄ of Road
Elev. 113.6
Clay Topping
Elev. 114.4
Top of Rock Fill
Elev. 100.0
Elev. 96.8
LONGITUDINAL CROSS SECTION
THRU ℄ of CULVERT
Sections 9 to 16 incl. are Cracked
117'
Sections 22 & 23 are Cracked
Nominal Diameter of Culvert - 36"
Elev. 73.1
Elev. 69.9
Section No. 23 cracked in bottom near one end as shown. Apparently had come in contact with sharp rock in the course of settlement
Sec. No. 22
Sec. No. 23
Location of Cracks
PROFILE of INVERT of CULVERT
Distance between lines represents vertical deflection of pipes
April 30, 1929 - No Fill
Oct. 16, 1929 and Nov. 13
Settlement of Invert
Settlement of Top of Pipe
SETTLEMENT of PIPES
Relative to Original Elevation
Inches
Sec. No. 9
Sec. No. 10
Sec. No. 11
Sec. No. 12
Sec. No. 13
Sec. No. 14
Sec. No. 15
Sec. No. 16
LOCATION of CRACKS - Nov. 13, 1929
CULVERT No. 2
Sta. 113+55

Fig. 8



CULVERT No. 1
Sta. 108+00
Skewed 20° R.



CULVERT No. 3
Sta. 167+00
Skewed 45° R.

Fig. 9

TABLE 4

Diameter Change - Culvert No. 1

Sec. No.	Vertical Diameter-In. Apr. 16	Oct. 16	Diff.	Horizontal Diameter-In. Apr. 16	Oct. 16	Diff.	Ht. of Fill-Ft.
1	38.08	38.10	+.02	37.97	38.00	.03	1.4
2	36.61	36.62	+.01	36.67	36.75	.08	4.0
3	38.05	38.00	.05	38.07	38.20	.13	6.9
4	36.67	36.58	.09	36.71	36.90	.19	9.5
5	37.98	37.83	.15	38.09	38.32	.23	10.4
6	36.62	36.50	.12	36.75	36.93	.18	11.1
7	38.07	38.00	.07	38.10	38.25	.15	12.1
8	36.61	36.42	.19	36.77	36.98	.21	12.6
9	37.94	37.78	.16	38.16	38.40	.24	13.3
10	36.44	36.20	.24	36.76	37.05	.29	14.1
11	37.80	37.62	.18	38.24	38.48	.24	14.7
12	36.55	36.45	.10	36.66	36.83	.17	15.4
13	38.01	37.92	.09	38.14	38.30	.16	16.2
14	36.68	36.60	.18	36.72	36.90	.18	16.9
15	38.04	37.90	.14	38.13	38.35	.22	17.9
16	36.63	36.45	.18	36.59	36.85	.26	18.4
17	38.05	37.90	.15	38.07	38.32	.25	19.3
18	36.64	36.45	.19	36.57	36.85	.28	17.8
19	38.00	37.73	.27	38.16	38.48	.32	16.7
20	36.72	36.40	.32	36.59	37.00	.41	15.7
21	38.09	37.91	.18	38.07	38.30	.23	14.8
22	36.65	36.48	.17	36.60	36.86	.26	13.9
23	38.07	37.90	.17	38.02	38.27	.25	12.6
24	36.67	36.50	.17	36.51	36.80	.29	11.4
25	38.06	38.00	.06	38.03	38.15	.12	10.2
26	36.59	36.38	.21	36.69	36.90	.21	9.0
27	38.00	37.95	.05	38.08	38.20	.12	7.8
28	36.71	36.65	.06	36.67	36.83	.16	6.5
29	38.03	38.02	.01	38.11	38.19	.08	5.2
30	36.77	36.78	+.01	36.72	36.78	.06	4.0
31	38.11	38.14	+.03	38.07	38.08	.01	2.7
32	36.69	36.75	+.04	36.72	36.75	.03	1.5
33	38.11	-----	----	38.06	38.05	-.01	.4

Sections 30 to 33 incl. added to culvert April 30, 1929

*Diameter change measurements on Nov. 13, 1929 revealed no measurable change had taken place since Oct. 16 in this culvert.

TABLE 5

Diameter Changes - Culvert No. 2

Vertical Diameter, Inches					Horizontal Diameter, Inches					Ht. of Fill Feet
Apr. 30	Oct. 16	Change	Nov. 13	Change	Apr. 30	Oct. 16	Change	Nov. 13	Change	
36.66	36.70	4.04	36.70	4.04	36.78	36.90	.12	36.90	.12	1.5
38.06	37.87	.19	37.86	.20	38.14	38.32	.18	38.30	.16	3.6
36.69	36.55	.14	36.55	.14	36.80	37.00	.20	37.00	.20	6.7
38.14	37.82	.32	37.83	.31	38.08	38.39	.31	38.40	.32	9.3
36.69	36.50	.19	36.50	.19	36.72	37.00	.28	37.00	.28	12.0
38.11	37.79	.32	37.80	.31	38.07	38.41	.34	38.40	.33	14.8
36.85	36.45	.40	36.45	.40	36.72	37.18	.46	37.18	.46	17.5
38.11	37.55	.56	37.55	.56	38.02	38.60	.58	38.61	.59	18.9
36.62	36.00	.62	36.00	.62	36.71	37.37	.66	37.40	.69	19.8
38.07	37.49	.58	37.45	.62	38.09	38.68	.58	38.70	.60	20.5
36.68	34.20	2.48	34.11	2.57	36.68	38.02	1.34	38.12	1.44	21.3
38.10	35.00	3.10	34.90	3.20	38.06	40.40	2.34	40.40	2.34	21.9
36.61	34.20	2.41	34.20	2.41	36.81	38.70	1.89	38.80	1.99	22.8
38.11	37.20	.91	37.18	.93	38.03	38.82	.79	38.90	.87	23.7
36.62	36.00	.62	36.00	.62	36.69	37.36	.67	37.38	.69	24.5
38.17	37.60	.57	37.60	.57	38.04	38.65	.61	38.65	.61	25.1
36.71	36.10	.61	36.08	.63	36.74	37.30	.56	37.33	.59	25.9
38.02	37.55	.47	37.55	.47	37.99	38.45	.46	38.48	.49	26.6
36.67	36.20	.47	36.20	.47	36.63	37.12	.49	37.15	.52	27.4
38.11	37.72	.39	37.65	.46	38.09	38.45	.36	38.58	.49	26.0
36.67	36.08	.59	36.06	.61	36.67	37.30	.63	37.33	.66	24.7
38.14	37.45	.69	37.42	.72	38.00	38.65	.65	38.68	.68	23.3
36.71	36.30	.41	36.30	.41	36.77	37.20	.43	37.21	.44	22.1
38.11	37.82	.29	37.82	.29	38.11	38.39	.28	38.40	.29	20.8
36.62	36.45	.17	36.45	.17	36.73	36.93	.20	36.92	.19	19.6
37.99	37.94	.05	37.95	.04	38.14	38.22	.08	38.22	.08	18.2
36.62	36.46	.16	36.45	.17	36.72	36.90	.18	36.90	.18	17.0
38.10	37.96	.14	37.95	.15	38.09	38.22	.13	38.22	.13	15.6
36.70	36.62	.08	36.60	.10	36.72	36.85	.13	36.85	.13	14.3
38.09	37.98	.11	37.97	.12	38.02	38.15	.13	38.15	.13	12.9
36.69	36.67	.02	36.65	.04	36.71	36.80	.09	36.80	.09	11.6
38.12	38.03	.09	38.02	.10	38.03	38.12	.09	38.12	.09	10.2
36.69	36.66	.03	36.65	.04	36.69	36.80	.11	36.80	.11	9.0
38.04	38.00	.04	38.00	.04	37.96	38.04	.08	38.05	.09	7.5
36.68	36.70	4.02	36.70	4.02	36.71	36.80	.09	36.80	.09	6.3
38.10	38.13	4.03	38.11	4.01	38.09	38.10	.01	38.10	.01	4.6
36.65	36.68	4.03	36.65	*.00	36.65	36.70	.05	36.70	.05	3.5
38.15	38.10	.05	38.10	.05	38.03	38.12	.09	38.12	.09	2.0
36.66	36.70	4.04	36.67	4.01	36.75	36.80	.05	36.80	.05	0.8

*Sections cracked due to fill load.

BRIEF STATEMENT OF CULVERT LOAD THEORY

Numerous experiments conducted by the Iowa Engineering Experiment Station* have shown quite conclusively that highway traffic has no appreciable load effect on culverts when the embankment exceeds a height of 5' or 6' above the top of the culvert. Since the fill over the top of the culverts in this investigation is nowhere less than about 10', under the traveled roadway surface, the super-imposed traffic may be ignored in the consideration of loads acting upon the culverts.

A brief summary of the theory of loads on culverts due to embankments without any attempt to go into its mathematical derivation, will be given here so that the application of the theory to the cases in point may be better understood. Reference is made to figure 10.

The resultant vertical load on a culvert produced by an embankment is made up of two elements: first, the weight of the prism of earth directly over the culvert, and second, a friction increment which may act downward or upward and tend to increase or decrease the resultant load as compared to the first element. The first element is comparable to the dead load on a bridge or other structure, in its immutability, and may for convenience, be referred to as the dead load. It is readily determinable when the size of culvert, the height of embankment, and the unit weight of the filling material are known. The second element, the friction increment, is dependent both for magnitude and direction of action upon the relative settlement of the prism of material over the culvert and the

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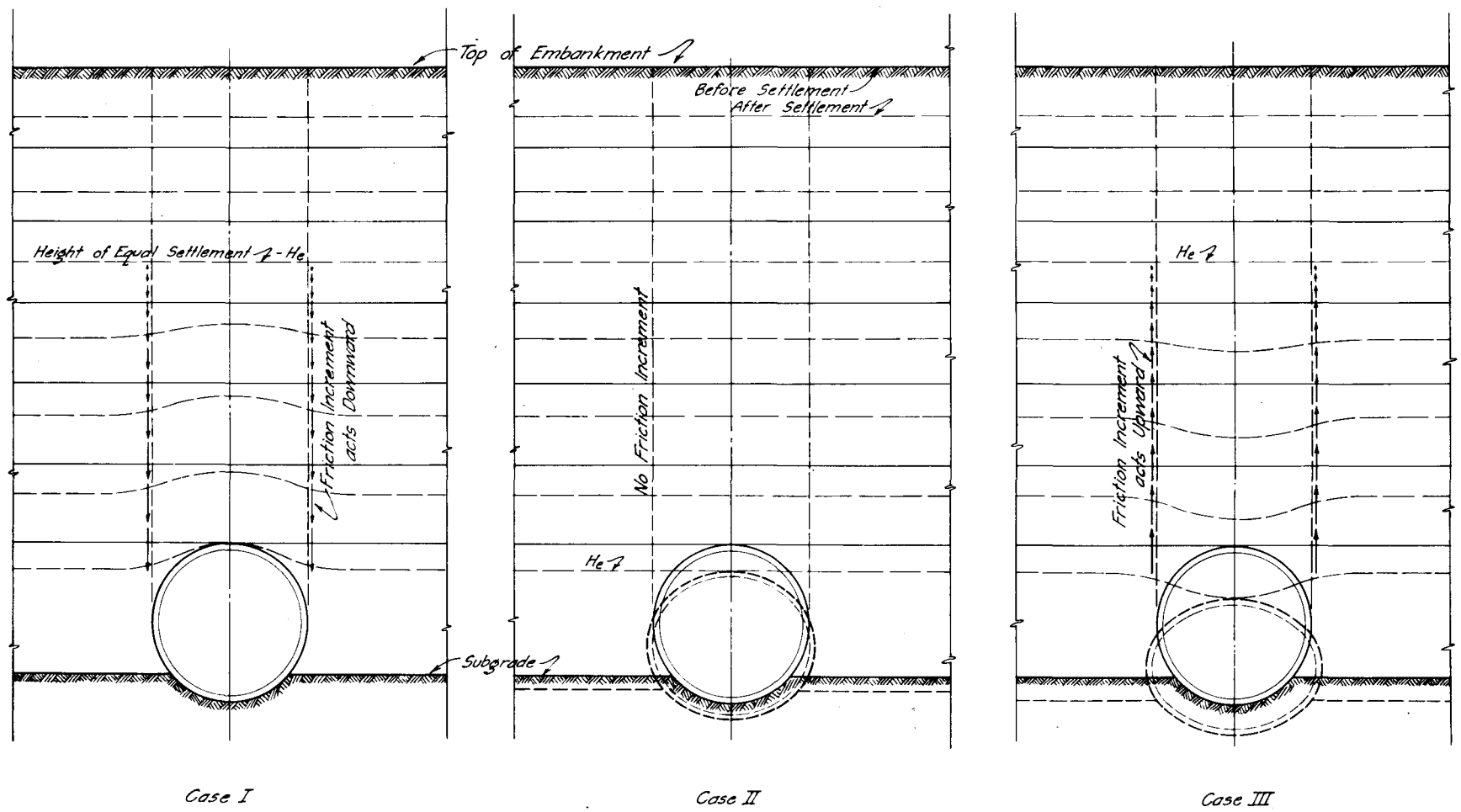
*See Bulletin No. 79



TYPICAL SETTLEMENT CONDITIONS
AFFECTING LOADS ON CULVERTS

Fig. 10

material adjacent to this prism on either side.

The tendency of newly placed embankments to settle is a phenomenon which is recognized by both engineers and laymen and may be accepted as axiomatic. The amount of settlement is dependent upon the modulus of compression of the material and the pressure at a given point. Therefore, assuming the material over a culvert to be homogeneous, and conditions existing as shown in case I of figure 10, where the column of material adjacent to the culvert is greater in height than the column directly over the culvert, a differential movement takes place along the vertical plane or zone (A-A) tangent to the sides of the culvert, which causes a frictional force to be exerted downward and the resultant load on the culvert to be considerably in excess of the dead load. Under these conditions, the product of the settlement-deflection ratio and the projection ratio, ($r_{sd}p$), is a positive quantity. This is a typical example of the "Incomplete" projection condition. (See page 30 for definitions and nomenclature).

The added pressure on the material in the prism directly over the culvert causes this material to compress or settle more per unit height of the prism than does the material alongside, and in every embankment, sufficiently high, there is a horizontal plane at which settlement of the prism over the culvert just equals the settlement of the materials alongside. This condition results when the greater pressure times a lesser height within the prism over the culvert equals the lesser pressure times a greater height of the material alongside. This plane is known as the "Height of Equal Settlement" and is

designated as H_e. Above H_e, the addition of uniform layers of material causes equal settlements over the pipe and adjacent to it at H_e. Hence the load on a culvert increases proportional to the height of fill above H_e, although at a greater rate than the weight of material directly over the pipe, in case of the incomplete projection condition and at a less rate in the incomplete ditch condition

As indicated in the diagram, the friction element of the resultant load is applied to the prism A-A' only below the level H_e, so that it is necessary to know the value of H_e before the load on a conduit can be evaluated. This value is dependent upon and is correctly indicated by $r_{sd}p$ which is the product of the abstract quantities, the settlement -deflection ratio and the projection ratio. (see page 31)

In case I it is assumed that the culvert is a very rigid structure on an unyielding foundation. If, as is illustrated by the diagram of case II, the top of the culvert settles downward, either by pushing into its bedding or by distortion of the vertical diameter, or both, an amount just equal to the settlement of the plane of material originally level with the top of the culvert, the load on the culvert is very materially reduced. It is apparent that under such conditions, the prism of earth over the culvert will move downward the same amount as the material alongside and , since no differential movement takes place along planes A and A', there is no friction increment. The load on the pipe in this case is just equal to the weight of the prism of material over the culvert, previously referred to as the dead load. Also, it will be noted, H_e is equal to zero. The value of $r_{sd}p$ is also zero and this condition constitutes the dividing line between the "Incomplete Projection Condition" and the"Incomplete Ditch Condition" (see diagram in fig. 11)

d_c = the increment of deflection of the conduit (i. e. the shortening of its vertical height),-feet.

S_f = The increment of settlement of the conduit foundation, feet.

S_g = the increment of settlement of the embankment subgrade, feet

S_m = the increment of settlement of materials adjacent to the conduit between the levels of the conduit top and the adjacent embankment subgrade,-feet.

Note: d_c, S_f, S_g, and S_m are increments due to the addition at or above the height of equal settlement of any (the same for all) incremental, uniform layer of fill materials.

$$r_{sd} = \frac{(S_m + S_g) - (d_c + S_f)}{S_m}$$

Note: a negative value of r_{sd} indicates the "Incomplete" ditch condition prevails. A positive value indicates the "Incomplete" projection condition.

W_c = the vertical external load on a closed conduit due to fill materials,-pounds per foot of length

w = the unit weight of fill materials, pounds per cu.ft.

K = the ratio of active horizontal pressure at any point in the fill to the vertical pressure which causes the active horizontal pressure,-abstract number.

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu} \qquad \text{(Rankine's formula)}$$

the "coefficient of internal friction" between the fill materials, abstract number

μ = the "coefficient of sliding friction" between the fill materials, and the sides of the ditch-abstract number. Note: μ' may be less than μ, but cannot be greater.

E 2.7182818 base of natural logarithms, abstract number.

The quantity, $r_{sd}p$, while entirely rational in the theory of loads on conduits, is not capable of being pre-determined when loads on a culvert are being considered. It should therefore, be treated as an empirical constant whose value for any given

MATHEMATICAL FORMULAS for CALCULATING VERTICAL EXTERNAL LOADS on CLOSED CONDUITS

DUE TO FILL MATERIALS

The following general formula applies to all commonly encountered conditions:

$$W_c = C w B^2 \qquad (1)$$

Formulas (2) and (3) apply to "ditch conduits" (completely buried in ditches excavated in comparatively "passive" and solid materials).

$$W_c = C_d w B_d^2 \qquad (2)$$

$$C_d = \frac{1-\varepsilon^{-2k\mu' \frac{H}{B_d}}}{2k\mu'} \qquad (3)$$

Formulas (4), (5), (6), and (7) apply to "projecting conduits" (under embankments and projecting above the embankment sub-grades).

$$W_c = C_c w B_c^2 \qquad (4)$$

For the "complete" ditch and projection conditions:

$$C_c = \frac{\varepsilon^{\pm 2k\mu \frac{H}{B_c}} - 1}{\pm 2k\mu} \qquad (5)$$

For the "incomplete" ditch and projection conditions:

$$C_c = \frac{\varepsilon^{\pm 2k\mu \frac{H_e}{B_c}} - 1}{\pm 2k\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right)\varepsilon^{\pm 2k\mu \frac{H_e}{B_c}} \qquad (6)$$

Note:- In formulas (5) and (6), use the − sign for the ditch conditions and the + sign for the projection conditions.

For calculating H_e in equation (6):

$$\varepsilon^{\pm 2k\mu \frac{H_e}{B_c}} \mp 2k\mu \frac{H_e}{B_c} = \pm 2k\mu r_{sd} p + 1 \qquad (7)$$

Note:- In equation (7),
use the − , + and − signs for the ditch condition.
use the + , − and + signs for the projection condition.

FOR THE "COMPLETE" and "INCOMPLETE" DITCH CONDITIONS WITH "ACTIVE" MATERIALS ADJACENT TO THE CONDUIT

For use in equation (4):

in place of (5) — $C_c = \frac{H}{B_c} - k\mu\left(\frac{H}{B_c}\right)^2$, "complete" ditch condition (8)

in place of (6) — $C_c = \frac{H}{B_c}\left(1 - 2k\mu \frac{H_e}{B_c}\right) + k\mu\left(\frac{H_e}{B_c}\right)^2$, "incomplete" ditch condition (9)

in place of (7) — $\frac{H_e}{B_c} = \sqrt{-\frac{r_{sd} p}{k\mu}}$, "incomplete" ditch condition (10)

proposed culvert must be chosen by the designer after careful consideration of the factors which will affect the relative settlement of various parts of the embankment and the structure itself.

In Culvert No. 2, which affords the best example of the action of the conduit theory of loads, it is apparent that the value of $r_{sd}p$ varied considerably throughout its length, due to the unequal opportunity of some sections to settle as compared to others. If the load on each section were known, it would be possible to calculate the value of this quantity for any section.

PROBABLE LOAD ON CULVERT NO.2

The load on some sections of culvert No. 2 can be estimated fairly accurately, since the three-edge bearing strengths of the pipe are known as given in Table 2 on page 7. The Iowa Engineering Experiment Station has been engaged since 1925 in determining the ratio of the supporting strength of culvert pipes under an embankment to the laboratory strength of such pipe when tested by standard 3-edge bearings. In Bulletin 76, which gives the results of the first experiment in this series, a ratio of 2.3 to 2.6* is announced for rigid pipe with ordinary projection bedding. Subsequent experiments indicate that the smaller value is more nearly the correct average value, although further tests are being conducted to verify this.

All these experiments have been conducted, using a pit run gravel containing a high percentage of clay, so that the material

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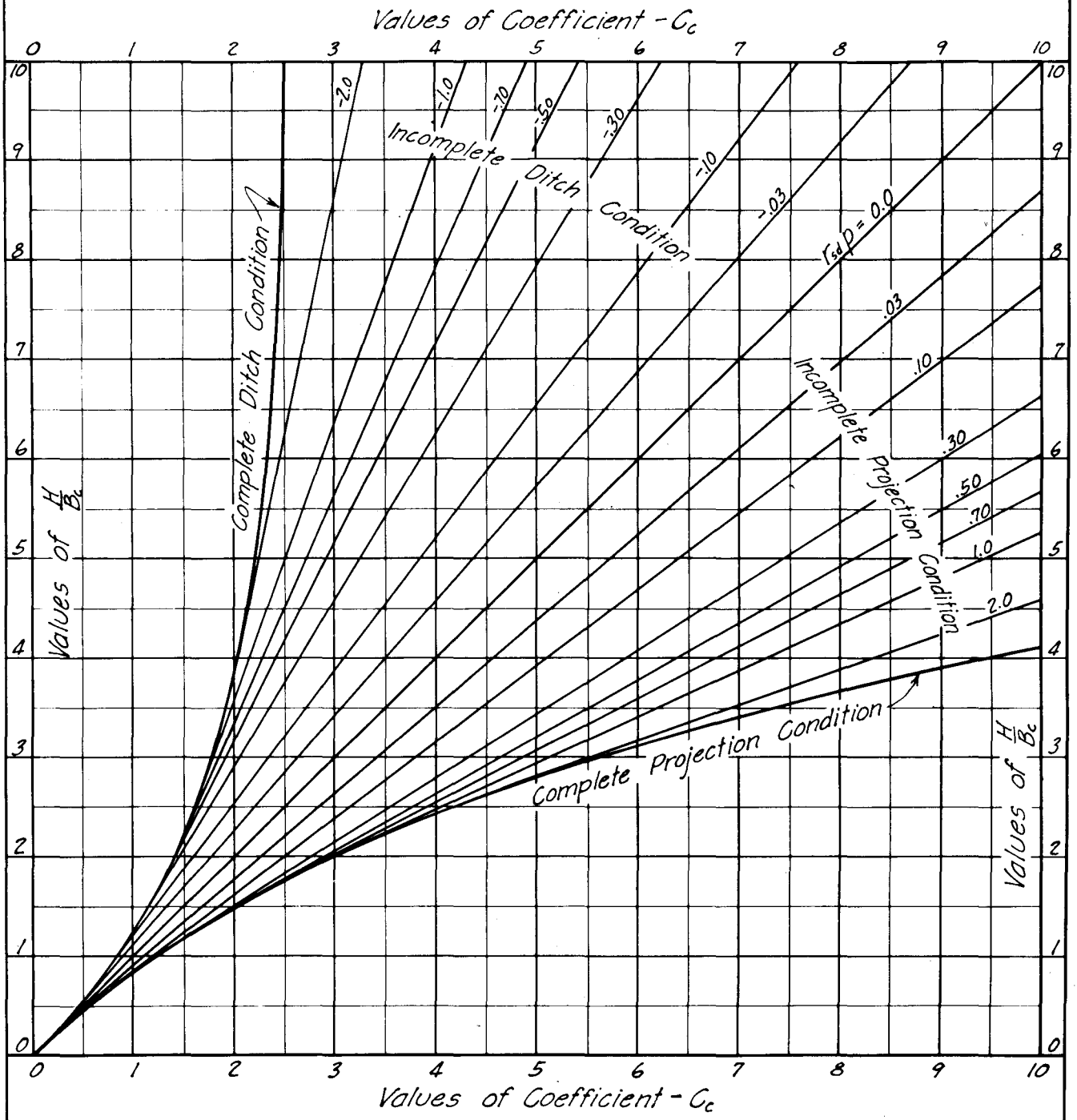
*See table 3, page 22, Bulletin 76

and, therefore, the pressure, was pretty uniformly distributed over the surface of the pipe. It is probable that the rocky character of the fill material over the Clayton County culverts caused the pressures on them to be more or less concentrated, which would tend to reduce the supporting strength of the pipes somewhat. On this basis a ratio varying from 2.0 to 2.3 will be assumed, so that, having a 3-edge bearing strength of 6000 lbs. per lin. ft., these culvert pipes probably supported a load of 12,000 to 14,000 lbs. per lin. ft. at the time they cracked.

Further, Table 3 shows that between October 16 and November 13, sections 9 and 10, and 16, cracked slightly in the bottom, although there had been no measurable decrease inthe vertical diameter between these dates (see Table 5). This indicates that the load on these sections was practically equal to the cracking load on these dates and also indicates that the pipes will not withstand a vertical deformation of more than about .60" (the average for these three sections at the time of cracking) without cracking. Therefore, it is evident from Table 5, that sections 9, 10, and 11 in one group and sections 16, 17, and 18 in another are supporting a load of approximately 12,000 to 14,000 lbs. per lin. ft. at the present time under heights of fill of 19.7' and 25.1' respectively.

It has been estimated that sections 13, 13, and 14 cracked under a height of fill of 15' to 17'. (See page 16). However, since these three sections very evidently approached contact with the solid rock sub-strata in the course of settlement, the

COMPUTATION DIAGRAM
FOR
EXTERNAL LOADS ON CONDUITS



For chosen values of the "settlement-deflection" and "projection" ratios ($r_{sd}p$), and given values of $\frac{H}{B_c}$, find values of the coefficient, C_c, and substitute in the formula $W_c = C_c w B_c^2$. See page for notation.

Fig. 11

reaction on the bottom of the pipe was undoubtedly more or less concentrated on the longitudinal element at the base of the vertical diameter at the time of cracking. Thus the ordinary projection bedding was modified to some extent and approached the impermissible bedding condition,* which without doubt reduced their supporting strength appreciably. There is no experimental data available to indicate the extent of this reduction in strength, but having these considerations in mind, a ratio of 1.75 to 2.00 of field to laboratory supporting strength will be assumed for these three sections, indicating that they probably were supporting a load of 10,500 lbs. to 12,000 lbs. per. lin. ft. at the time they cracked.

PROBABLE VALUES OF THE EMPIRICAL CONSTANT, $r_{sd}p$

With the above assumption, we may calculate approximate values of $r_{sd}p$ for these three groups of pipes, from the formulas on page 32 or take them from a diagram such as is shown in figure 11, page 35. Following are tables showing such values using B_c = 3225; μ = .70; and w : 140 lbs. per cu. ft.

Table 6 - Sections 9, 10, and 11

W_c lbs. per lin. ft.	C_c	Values of $r_{sd}p$		
		for H = 18.9'	for H = 19.8'	for H = 20.5'
		H/B_c = 5.82	H/B_c = 6.10	H/B_c = 6.31
12,000	8.13	.19	.13	.10
13,000	8.81	.32	.23	.19
14,000	9.47	.46	.36	.30

*See paper by Anson Marston presented to Advisory Board Highway Research. Washington, D.C. Dec. 12, 1929.

Table 7 - Sections 16, 17, and 18

W_c lbs. per lin. ft.	C_c	Values of $r_{sd}p$		
		for H 24.5'	for H 25.1'	for H 25.9'
		H/B_c 7.54	H/B_c 7.73	H/B_c 7.97
12,000	8.13	+ .007	+ .004	+ .001
13,000	8.81	+ .03	+ .02	+ .01
14,000	9.47	+ .08	+ .06	+ .04

Table 8 - Sections 12, 13, and 14

W_c lbs. per lin. ft.	C_c	Values of $r_{sd}p$		
		for H 15'	for H 16'	for H 17'
		H/B_c 4.62	H/B_c 4.93	H/B_c [illegible]
10,500	7.12	+ .38	+ .24	+ .16
11,250	7.63	+ .52	+ .36	+ .26
12.000	8.13	+ .75	+ .51	+ .37

It is apparent from these detailed analyses of the loads on culvert No. 2 that the character of the bedding of the various pipe sections played an important part in governing the amount of load each section was called upon to withstand. If sections 12, 13, and 14 had had as much earth cushion underneath them as did the adjacent sections and could have settled downward with the embankment as did the others, they would probably not have broken, at least not more than perhaps a slight crack in the bottom. If, on the other hand, all the pipes had had no more cushioning underneath them then these three sections, analysis indicates that those sections under the greatest height of fill would probably have been called upon to withstand a vertical load of from 21,000 to

24,000 lbs. per lin. ft. without cracking. This corresponds to a 3-edge bearing strength of about 10,000 to 12,000 lbs. per lin. ft.

Stated another way, if these pipes had been free to settle into their foundations only about 3" instead of 5" or 6" they would have been called upon to withstand nearly twice their actual supporting strength in an embankment. These facts indicate the tremendous importance of relative settlements within an embankment over a culvert when the load on the culvert is being considered and show conclusively that the load on a culvert cannot be expressed as a direct function of the height of the embankment and the degree of rigidity of the conduit.

PROBABLE LOADS ON CULVERTS NO. 1 AND 3

Detailed mathematical studies of the load on culvert No. 2 have been given because it was possible to estimate the loads on various parts of the culvert in a logical manner. It is not possible to do this with the same degree of confidence either in the case of culvert No. 1 or No. 3. There are, however, some outstanding general features connected with these two culverts which are worthy of discussion.

The size and quality of the pipes used to construct these culverts were the same. The maximum height of fill over the top of the pipes, as well ad the character of the fill and the manner of placing it, are substantially the same, and the method of bedding the pipes on an earth cushion was the same. Yet we are confronted with the fact that nearly all of the pipes in culvert No. 3 are broken and a very large number of them complete-

ly collapsed as a result of the vertical load due to the embankment, while culvert No. 1 is in good condition and measurements of the diameter changes indicate roughly that the pipes under the highest fill are carying not more than about one-half of their ultimate supporting strength.

There are not enough data available to support a strict mathematical explanation of these phenomena, but there is evidence in each case which leads to a plausible explanation of the facts, as they have been observed.

Culvert No. 1 was placed in a very shallow ravine right at the base of a fairly steep slope. This slope was the transition between a cut and fill section of the new road, (see photograph in figure 3, page 10). The natural ground surface, or embankment subgrade, therefore, rises rapidly to the south (the right side of the picture) of the culvert, and the column of embankment material is less in height on this side of the culvert than the prism of material over the pipe or adjacent to it on the other side, (the left in the picture). This culvert then, is placed in such a manner that the projection condition exists on one side of it, while the ditch condition prevails on the other side, (a ditch with sloping sides). Dean Marston states in Bulletin 31 that, in a sloping sided trench, the backfill will produce a load on a pipe equal to the load produced in a vertical trench whose width is equal to the width of the sloping trench at a point a little below the top of the pipe. Some current experiments being conducted by Mr. W.J. Schlick of the Experiment Station Staff, verify this con-

clusion. From these facts it is reasonable to conclude that the prism of embankment material directly over culvert No. 1 was acted upon by forces on one side in accordance with the ditch condition theory and on the other side by forces in accordance with the projection condition theory.

If we set up a hypothetical case by assuming that this culvert had been placed in a sloping sided trench whose width at the top of the culvert is 5 ft., we may calculate the load on the pipe in accordance with the ditch condition formulas (2 and 3 or 8 page 32, depending upon whether the materials alongside the culvert are active or passive). Then by formulas (4), (6) and (7), we may calculate the load on the culvert if it had been placed with the projection condition existing on both sides. Since it was actually constructed with the ditch condition prevailing on one side and the projection condition on the other, an idea of the actualload on the structure may be gained by averaging the two loads thus calculated.

The following table shows resultant loads calculated, as outlined above for sections 15 to 20 inclusive of culvert No. 1.

Table 9

Average H = 17.6 μ = .70 W = 140 lbs. per cu. ft.

Projection condition load B_c = 3.25' H/B_c = 5.42 formulas (4), (6), (7)		Ditch condition loads Bd 5.0' = H/B_d = 3.52 formula (2)		Average load on sections 15 to 20 incl lbs. per lin. ft.	
Various values of $r_{sd}p$	lbs. per lin. ft.	Passive Materials alongside formula(3)	Active Materials alongside formula(8)	Passive Materials alongside	Active materials alongside
.00	8,000			7,400	6,300
.10	10,300	6,700	4,600	8,500	7,500
.30	12,000			9,300	8,300
.50	13,200			9,900	8,900

These approximate computations indicate that these sections are carrying from 6,000 to 10,000 lbs. per lineal foot.

The average vertical deformation of these pipes is, at present about.20" (see table 4 page 24) which is just 1/3 of the deformation at which they probably would crack, (see page 34). Their load-deformation relationship is not known, but some current experiments on 42" smooth cast iron pipe, which are being conducted by the author, show this relationship to be a curved line; the deformation increasing morerapidly as the ultimate load is approached. Examination of these curves indicate that it takes approximately 1/2 the ultimate load to produce 1/3 the ultimate deformation. Applying this fact to the sections under discussion indicates that they are carrying from 6,000 to 7,000 lbs. per lin. ft. which gives a rough check on the calculations shown in table 9.

In the case of culvert No. 3, which completely collapsed under a 20 foot (or less) embankment, the fact that it was constructed in the winter time was very probably a contributory factor to the cause of failure. According to the engineer and the contractor on the job, this culvert was bedded on an earth cushion over the natural surface of the ground just the same as the other culverts. However, since February, 1929, was a very cold month in this vivinity, this earth bedding probably was in a frozen state when placed or, if not, undoubtedly froze solid soon after being placed. In either case the bedding of the pipe was comparatively rigid and would not permit any appreciable settlement of the pipe. On the other hand, the rock fill would probably not be affected by the freezing weather and

would tend to settle just as much as though it were warmer weather. This situation would produce very high positive values of the "settlement-deflection ratio" and consequently, very high resultant vertical loads on the culvert.

A phenomenon of special interest in connection with culvert No. 3 is the fact that the sections near the ends, under the slopes of the embankment, cracked and deformed under relatively very low heights of fill. Sections 43 and 44, for example, are cracked and deformed considerably, although under only about 4 to 6 feet of ~~the~~ fill, and even the very severe complete projection condition formula (4 and 5) will not account for ~~the~~ loads any where near large enough to crack the pipes.

The explanation is simple enough, however, if we will visualize chronologically the developments after the first pipe section has failed.

A circular pipe of this character loaded by an embankment, will develop considerable supporting strength even after it is cracked at the top, bottom and sides, due to the development of passive resistance in the filling material at the sides of the pipe. The top half of the pipe becomes a 3-hinged arch and will continue to carry a load until the crown is forced down far enough that it cannot longer sustain the horizontal thrust or until, as sometimes happens, one segment slips past the other. When this stage is reached, the pipe is no longer capable of supporting the load on it and the prism material above the section will move downward until it is held in equilibrium

by friction on all four of its vertical faces. This frictional force on the faces above the ends of the pipe section is transfered to the prism of material over the two adjacent pipes and the load on these pipes is greatly increased thereby.

Thus, when any one interior section of culvert No. 3 became distorted enough to be unable to support the material over it, the weight of this material was partially transmitted to the two adjacent sections. These sections, already heavily loaded, were thereby further distorted and rendered incapable of supporting their respective prisms of fill material and this action continued progressively along the culvert each way from the original section which failed.

The sections near the end of this culvert, were therefore, subjected not only to the vertical forces as indicated in case I of figure 10, page 27, but in addition had to sustain the frictional force on the vertical plane of the filling material at the end of the adjacent pipe section, due to the inability of this section to carry its share of the load. This accounts for the fact that some of the pipes cracked and deformed under very low heights of fill.

CONCLUSIONS

A summary of the conclusions derived from the study of these three culverts is as follows:

1. There is every evidence that the rock embankments over these culverts acted in a manner entirely analogous to the act-

ion of an ordinary earth embankment. Settlements of the top of the pipes, and, therefore, the bottomof the prism of material directly over the pipes, were measured and found to be as great as 6½ inches. From these measurements and the estimated loads on the pipes, the settlement of the plane in the embankment originally level with the top of the culvert is calculated to be about 7 inches. These settlements occurred under an embankment about 25 feet high.

2. Marston's theory of loads on conduits is substantiated by these studies. All the observed phenomena are readily explained by means of this theory.

3. The earth cushion on which the pipes were laid proved to be the salvation of culvert No. 2 by allowing the pipes to settle appreciably, thereby reducing the settlement-deflection ratio. In the case of culvert No. 3, the earth cushion probably was not effective because of the freezing weather.

4. The loads on culvert No. 1 were less than might normally be expected, probably because of the sharp rise of the natural ground surface adjacent to the culvert on the south, which produced the ditch condition on one side of the culvert.

5. There is nothing revealed by this study to indicate that the pipes used in culverts No. 2 and 3 were of inferior quality. They were merely loaded beyond their capacity, and any other semi-rigid type of culvert pipe of the same strength would, in all probability, have failed under similar circumstances.