

Some Effects of Fabrication Practices on Strength Characteristics of Reinforced Concrete Culvert Pipe

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With the recent increase of highway construction activity, it soon became apparent that there was a definite need for knowledge concerning the manufacture, installation, and field behavior of pipe culverts under the higher and higher fills that are used. The study reported in this paper concerns the fabrication practices and their possible relationship to pipe strength, failure of pipe under embankment loads, and the feasibility of repairing or restoring the structural integrity of in-place pipe that have been damaged.

Thirty-three sections of 54-in. diameter, Class III pipe were fabricated and tested so that the following variables could be studied: (a) type of lap used to fabricate cages of reinforcing steel (lapped and tied or lapped and welded), (b) relative position of joints of inner and outer cages of steel in the pipe, (c) position of joints with respect to points of application of the load in the three-edge bearing test, (d) use of reinforced gunite liner for repair purposes, and (e) effect of lateral restraint on load-carrying capacity of the pipe.

Results of the investigation indicate that: (a) the 0.01-in. crack strengths and ultimate strengths were significantly affected by the three methods used to prepare joints in reinforcing cages; (b) there appears to be no measurable difference in performance of pipe in which joints were spaced 0° apart, 90° and 180° apart; (c) load-carrying capacities were not significantly affected by orientation of joints in the three-edge bearing tests; (d) repairs made on structurally damaged pipe sufficed to restore original load-carrying capacity to the pipe; and (e) rods used to provide lateral restraint appeared to increase load-carrying capacity of the pipe in the same proportion as internal steel.

• **UNDERGROUND CONDUITS** have been used for hundreds of years for drainage and water supply purposes. With the advent of railway and automobile transportation, the use of underground conduits for drainage purposes beneath the roadways increased greatly. It soon became apparent that there was a definite need for knowledge concerning the manufacture, installation, and field behavior of pipe culverts. Accordingly in the first half of the century, much attention and research has been directed by many individuals and organizations toward the development and establishment of acceptable techniques and specifications for the manufacture and installation of pipe culverts.

Factors governing maximum height of fill, and thus maximum load that may be safely placed over reinforced concrete pipe culverts, are (a) pipe strength, (b) character of

fill material over the pipe, (c) character of foundation material beneath the pipe, (d) relative settlement of material over the pipe to that of material on either side of the pipe, and (e) method of bedding and installation. In an attempt to provide a method by which one can easily identify a satisfactory pipe with regard to quality and strength, the rather simple and convenient three-edge bearing test has been proposed as ASTM Designation C 76-59T (AASHTO M170-57). This test method, applicable to concrete pipe up through the 72-in. inside diameter size, is currently being used in Kentucky as well as in many other areas to evaluate the strength of a reinforced concrete pipe. A criterion for the installation design of concrete pipe culverts has been developed by the U. S. Bureau of Public Roads in cooperation with the American Concrete Pipe Association and was distributed to the various highway agencies on April 4, 1957, as Circular Memorandum 22-40, U. S. Bureau of Public Roads. This memorandum is the basis for specifications adopted in 13 states, including Kentucky, to provide for an efficient utilization of rigid pipe by specifying bedding details, methods of installation, and pipe strength required for various heights of fill.

Early experience gained from work on the Interstate Highway System disclosed a need for additional detailed study and understanding of the theory of design, pipe fabrication practices, and construction or installation practices. Investigations of certain aspects of the theory and installation practices have been reported by the Research Division of the Kentucky Department of Highways (1, 2, 3, 4). On November 12, 1959, the Bureau requested the states to furnish annual performance data to evaluate the installation criterion, and according to the Bureau's Circular Memorandum 32-30, dated October 18, 1962, 34 states are participating in this study and are making annual reports. The study reported herein concerns the fabrication practices and their possible relationship to pipe strength, failure of pipe under embankment loads, and feasibility of repairing or restoring the structural integrity of in-place pipe that have damaged during or soon after construction.

THEORETICAL CONSIDERATIONS

The strength of reinforced concrete pipe is commonly stated in terms of D-load strength, the load in pounds per linear foot of internal diameter. An advantage of the D-load designation is that all sizes of pipe of a given D-load, installed under similar conditions of bedding and backfilling, will support the same maximum height of fill. Reinforced concrete pipe are tested by the three-edge bearing test and are classified according to the D-load that will produce a 0.01-in. crack of the D-load that will produce ultimate failure.

The three-edge bearing test is a severe test inasmuch as the load applied to the conduit is in the form of point loading and inasmuch as there is no side support applied to the conduit as there would be in the case of a field installation. Under field conditions of loading, the vertical loads applied to the conduit will be distributed over a portion of the conduit rather than concentrated, and side pressures will be exerted on the conduit; thus, under field conditions the conduit may sustain loads which are greater than those indicated by the three-edge bearing test. This fact is accounted for in design by use of a load factor which is the ratio of the strength of a pipe under any stated condition of loading to its strength when tested by the three-edge bearing method. The value of the load factor is greatly dependent on the method in which the conduit is bedded as well as the nature and density of the backfill material.

It is imperative, therefore, that installation of the pipe culvert be in full accordance with the design theory and with the plans. If damage occurs to the pipe, and if full faith is credited to the design procedure, the fault must arise from either poorly fabricated pipe, or improper installation techniques, or negligence in adhering strictly to plans.

If a pipe is damaged, whether the cause is apparent or not, the question inevitably arises as to what resources are available as an alternative to complete removal and replacement; that is, can the pipe be effectively and satisfactorily repaired in place? With regard to this, Paragraph 30 of ASTM C 76-59T (AASHTO M170-60) is cited in full:

Pipe may be repaired, if necessary, because of occasional imperfections in manufacture or accidental injury during handling

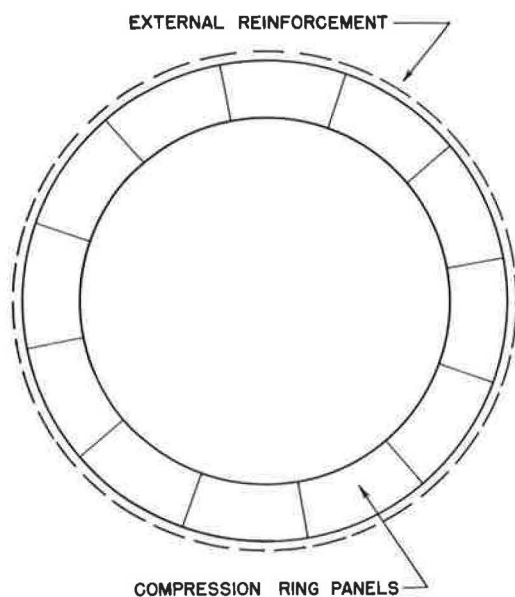


Figure 1. Pipe consisting of compression panels and external reinforcement.

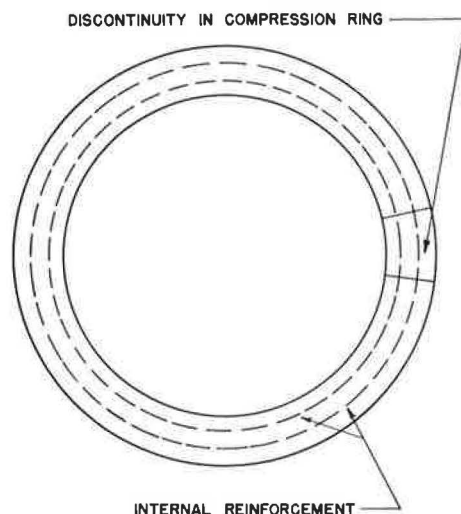


Figure 2. Concrete pipe with discontinuity in compression ring.

and will be acceptable if, in the opinion of the purchaser, the repairs are sound and properly finished and cured and the repaired pipe conforms to the requirements of these specifications.

In a most liberal interpretation, the pipe, even though severely damaged, could be acceptable if repaired and restored sufficiently to meet the requirements of the specification.

The reinforcement steel in a concrete pipe is more or less inactive or passive so long as the pipe is subjected to a uniform radial loading. In this case, the annular shell of the pipe is uniformly in compression. Any unbalance in imposed loads produces tensile stresses in the outer portion of the concrete, and inasmuch as the concrete is inherently weak in tension but strong in compression, the function of the reinforcement is to provide a tensile reactance to the compression borne by the concrete. Thus a pipe could be fabricated from individual concrete panels with external steel bands in much the same manner as wood-stave barrels (Fig. 1). It follows then that one or more damaged panels could be replaced and thereby restore the continuity of bearing and compression in the shell.

However, if the shell were cast monolithically about the steel and were then fractured to such an extent that continuity of compressive bearing in the concrete was lost (Fig. 2), it follows that routing of the damaged concrete and replacement with new concrete, even though a high degree of bond was not achieved, should restore the pipe virtually to its original strength (Fig. 3). This idea assumes, of

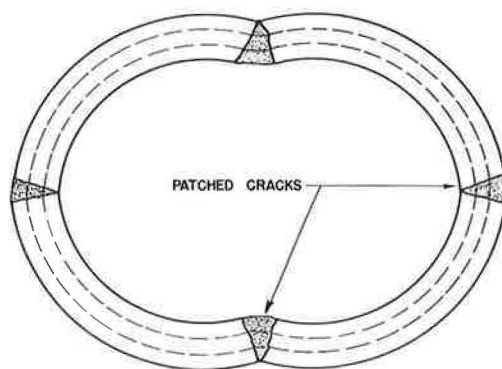


Figure 3. Pipe with compression ring patched and restored.

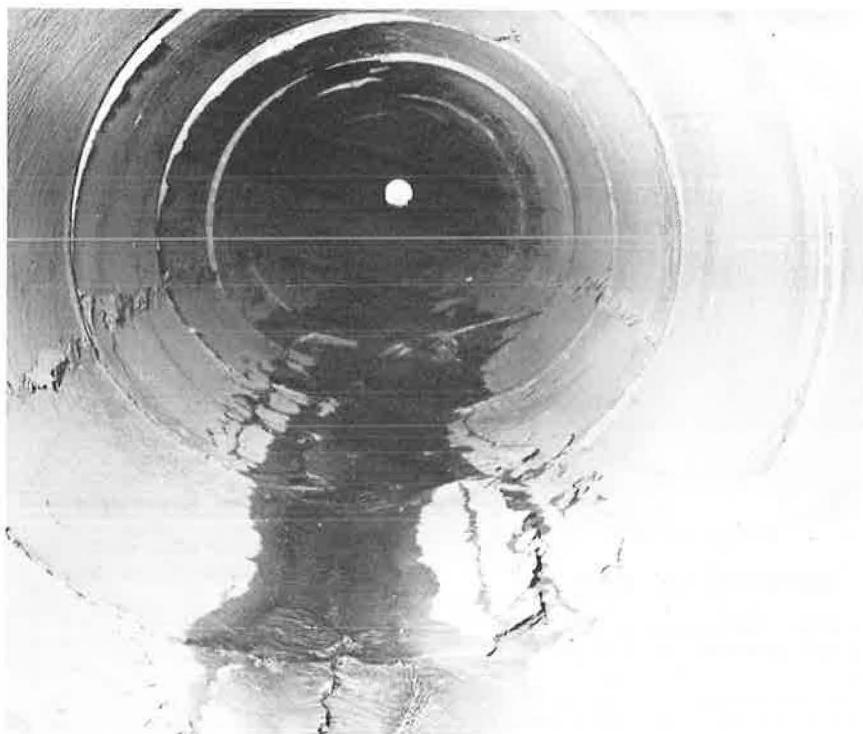


Figure 4. Failure in bottom of 54-in. culvert under a 53-ft fill; Station 1087+50, I-75-7(5)160, Grant County.

course, that the steel is not damaged and that the pipe, when restored, is not critically out-of-round.

Many of the pipe found to be damaged in-place have been deformed considerably—otherwise the concrete shell would have remained intact (Figs. 4 and 5). Rejecting any thought toward jacking in-place pipe back to roundness, the problem at hand involves the degree of collapse to which remedial treatment might be deemed feasible. For instance, if the pipe is elongated horizontally and if the continuity of the shell is restored, the pipe could never be quite as strong as the original round pipe. If the interior of the pipe is also lined to provide an additional compression ring (preferably reinforced), it should be possible to strengthen the pipe to a degree equal to or greater than its original value (Fig. 6).

Of equal importance to the strength of the pipe is the continuity of reinforcement steel. It is assumed that the steel in a damaged pipe may have been stressed to or beyond the yield point, but that it is capable of again withstanding an equal level of stress unless continuity has been impaired. A broken weld or series of welds would therefore be an obvious detriment to strength unless repaired before replacement of concrete in the compression shell. In fact, welds and laps are of considerable interest to this study because of their possible influence on the original strength of the pipe. In manufacture, where two cages of wire are used, it is the usual practice to place the joints 180 deg apart. It follows, therefore, that if the welds or laps are weak and if the pipe is by chance positioned in the three-edge bearing test or in an embankment so that the joints are located at points of greatest bending and highest stresses, strength could be seriously affected—more so than if the joints were randomly orientated.

Aside from the other factors, such as wall thickness and amount and strength of steel which are more closely related to design, the quality of welds or laps in the steel may largely determine D-load strengths. In an older specification ((AASHTO M 41-55), deleted in 1957), the weld was described as follows:



Figure 5. Failure in top of 54-in. culvert under a 53-ft fill; Station 1087+50, I-75-7(5)160, Grant County.

If welded, the member at either a welded splice or intersection shall develop a tensile strength not less than the minimum strength required for the reinforcement...

The specifications (AASHTO M 170-60 and ASTM D 76-59 T) specify: "If the splices are not welded...", and the alternative of welding is not subsequently clarified. AASHTO M 32-60, Cold-Drawn Steel Wire for Concrete Reinforcement, covers the requirements for the wire to be used in Welded Steel Wire Fabric for Concrete Reinforcement, AASHTO M 55-60 (ASTM A 185-58 T), which in turn includes a shear-type test for the quality of welds obtained in the manufacture of the fabric (mesh). Apparently none of the existing specifications covers the particular point about welding the mesh to form the reinforcing cage.

Observations of in-place pipe which were damaged structurally to an extent that the welded joints in the steel cage were exposed (in the bottom of the pipe), revealed that some welds (lapped about $1\frac{1}{2}$ in., welded on one side) had broken. Welded joints were then clipped from normal production cages and these were tested in tension. Invariably the splice

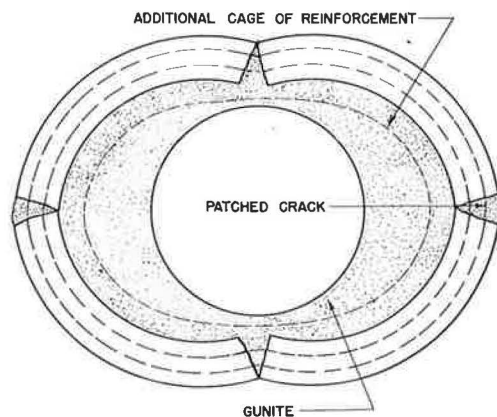


Figure 6. Repaired pipe with reinforced, gunite liner.

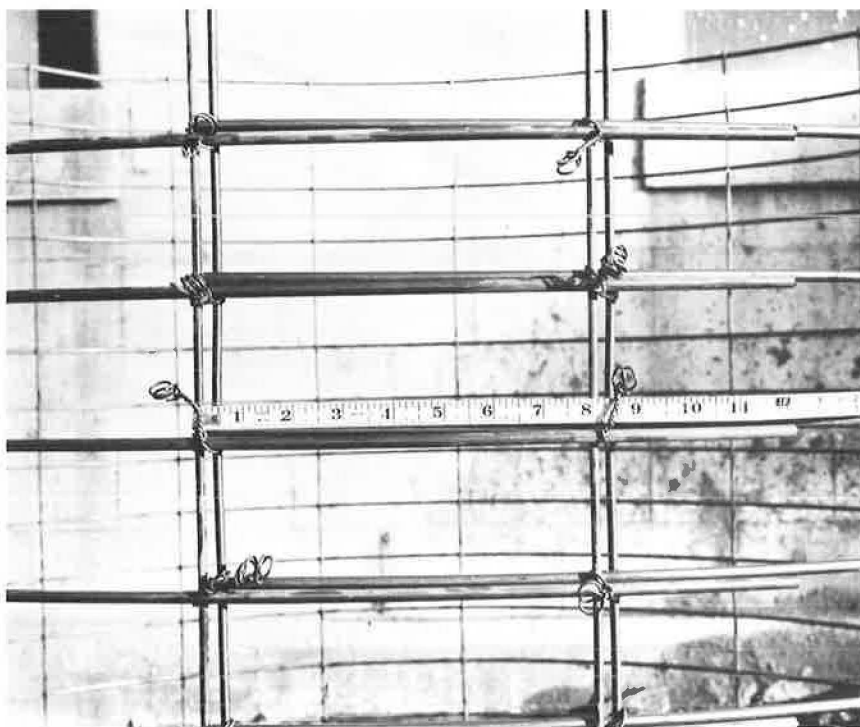


Figure 7. Joint of inner cages of reinforcement used in Series A and B.

was weaker than the wire. It was noted, however, that the break usually occurred at the juncture of the wire and fillet where the weld cut into the wire and reduced the cross-section. Also, due to the eccentricity of the pull about the weld, there was bending of the wire at its juncture with the weld. Although these pull-tests were not realistic or representative of the strength of the joint when encased in concrete, it was apparent that the welds were incapable of withstanding the full tension of the wire.

MATERIALS AND APPARATUS

In an effort to obtain additional information concerning the effects of various pipe-fabrication practices on pipe performances, the Kentucky Concrete Pipe Company was most cooperative in manufacturing and testing several sections of pipe at the Louisville plant.

Thirty-three sections of Class III pipe, 54 in. in inside diameter by 4 ft in length, were prepared according to the following schedule:

Series A—Four sections, inner and outer cages of reinforcement lapped at least 40 diameters (Fig. 7), laps 180 deg apart.

Series B—Four sections, inner and outer cages of reinforcement lapped at least 40 diameters, laps 90 deg apart.

Series C—Four sections, inner and outer cages lapped approximately 3 in. and fastened with two spot welds (Fig. 8), welds placed 180 deg apart.

Series D—Four sections, inner and outer cages lapped approximately 3 in. and fastened with two spot welds, welds placed 90 deg apart.

Series E—Four sections, inner and outer cages lapped approximately 3 in. and fastened with two spot welds, welds 0 deg apart.

Series F—Four sections, non-reinforced.

Series G—Three sections, inner and outer cages lapped approximately $1\frac{1}{2}$ in. and fastened with one spot weld (Fig. 9), welds spaced 180 deg apart.

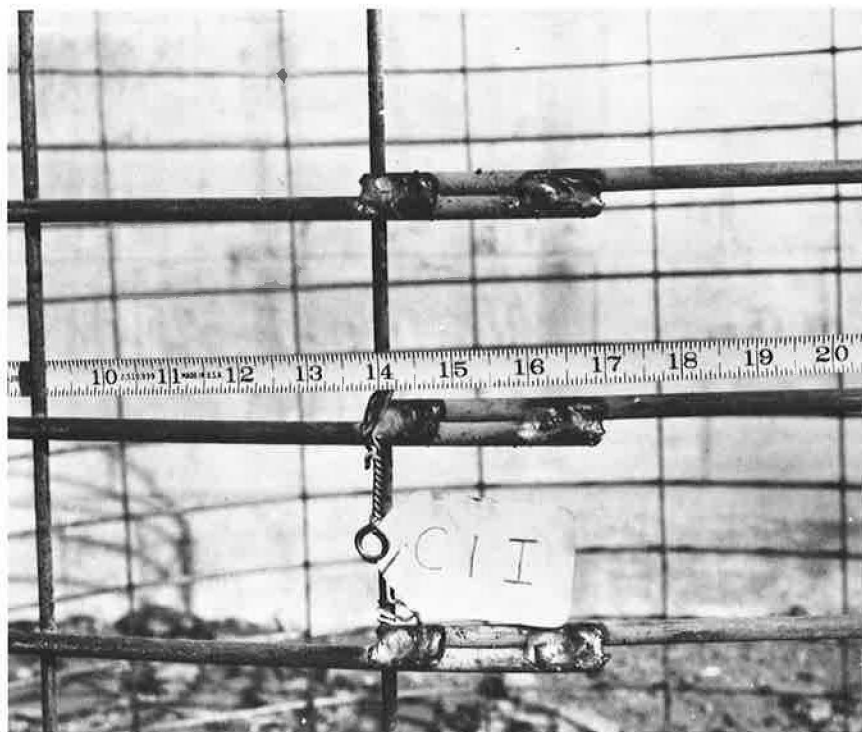


Figure 8. Joint of inner cages of reinforcement used in Series C, D and F.

Series H—Three sections, inner and outer cages lapped approximately $1\frac{1}{2}$ in. and fastened with one spot weld, welds spaced 90 deg apart.

Series J—Three sections, inner and outer cages lapped approximately $1\frac{1}{2}$ in. and fastened with one spot weld, welds placed 0 deg apart.

The inner cages of reinforcement were made from 3- × 8-in., No. 0 + 5, welded wire fabric. The No. 0 wire (0.3065-in. diameter) was the longitudinal wire in the fabric and was spaced 3 in., center-to-center. This is the wire which provides the reinforcement for the concrete pipe. The No. 5 wire (0.2070-in. diameter) was spaced at 8 in. The outer cage of reinforcement was made from 3- × 8-in., No. 2 + 7, welded-wire fabric; the No. 2 wire (0.2625-in. diameter), on 3-in. spacing, provided the reinforcement for the pipe.

All sections of pipe used in this investigation were machine-made, tamped pipe. The four sections of pipe in Series F were manufactured on November 9, 1960, and the 20 sections in Series A through E were made November 10, 1960. The last nine sections, Series G, H and J, were prepared on November 14, 1960. Test cylinders, 6 × 12 in., and beams, 3 × 5 × 20 in., were fabricated from the same concrete used in the pipe.

A Forney testing machine having a capacity of 500,000 lb, located in the yard of the Louisville plant of the Kentucky Concrete Pipe Company, was made available for loading the pipe. All of the tests were made in this machine.

Figure 10 shows a yoke assembly which was used to restrain selected sections of pipe along their horizontal diameters during the three-edge bearing test. The four different sizes of tie rods used in this portion of the study were (a) uniform diameter of $\frac{3}{4}$ in.; (b) uniform diameter of $\frac{3}{8}$ in.; (c) a $\frac{3}{4}$ -in. rod with a reduced section, 8.0 in. long × 0.30 in. in diameter, at the middle; and (d) a $\frac{3}{4}$ -in. rod with a reduced section, 2.6 in. long × 0.30 in. in diameter, at the middle. Selected pipe were tested in three-

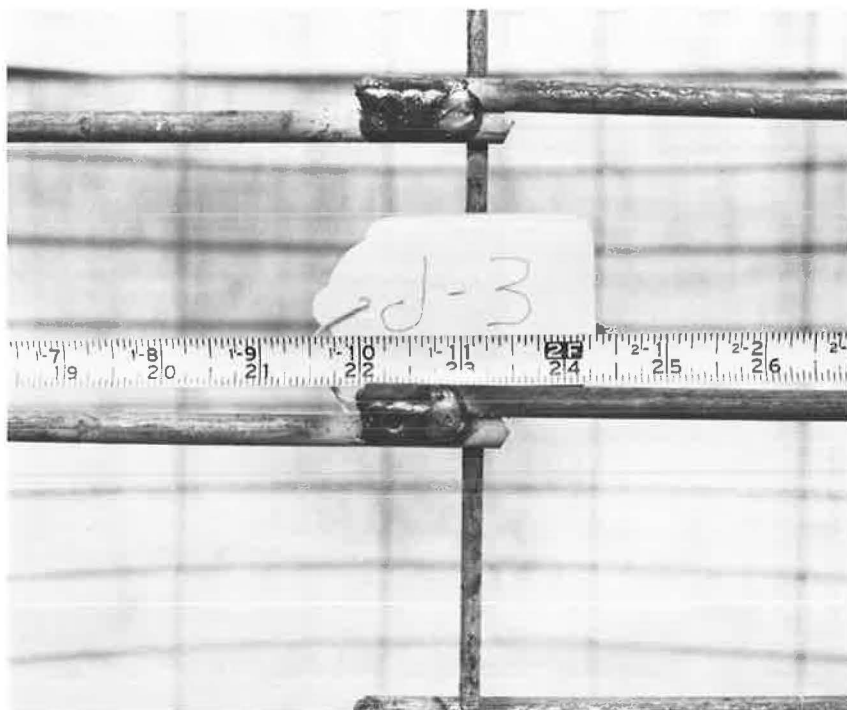


Figure 9. Joint of reinforced cages used in Series G, H and J.

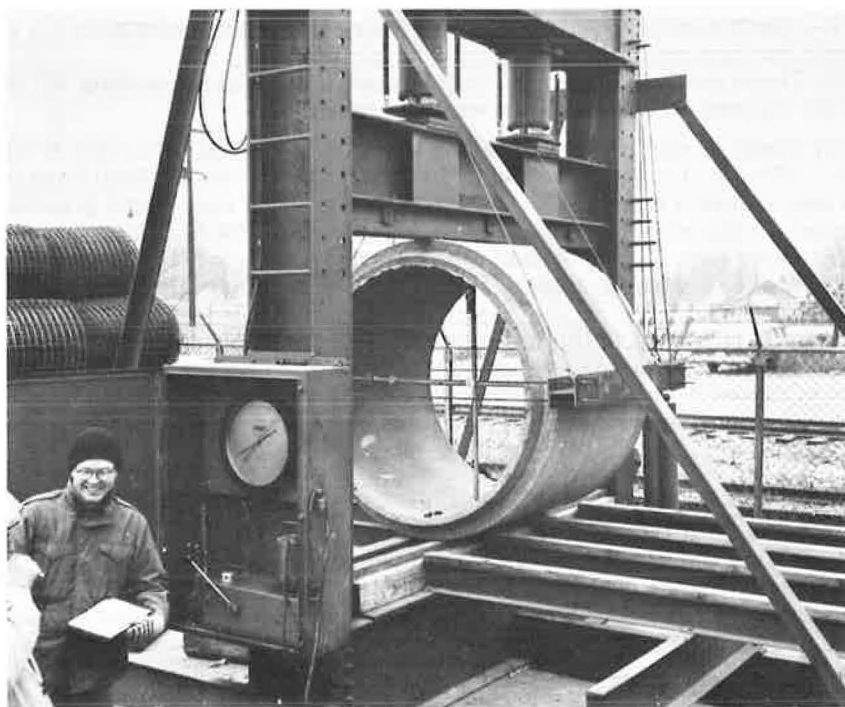


Figure 10. Yoke assembly for providing lateral restraint.

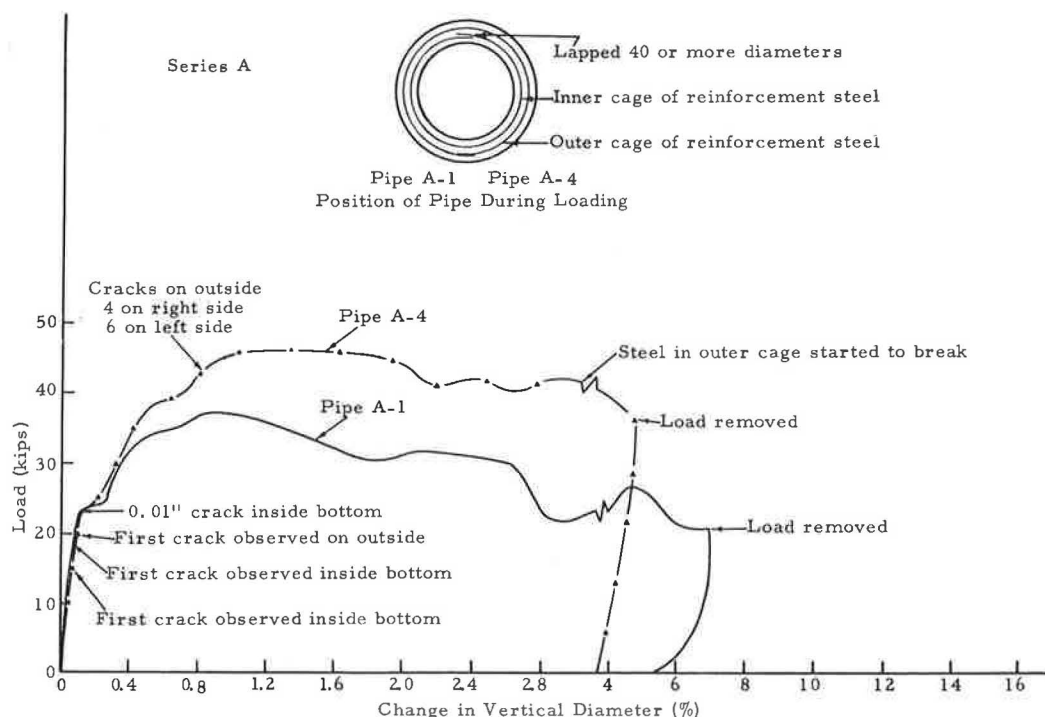


Figure 11. Load-deformation curves for Series A.

edge bearing and with horizontal restraint to obtain some knowledge of the horizontal reactance to vertical loading.

From the group of pipe which was loaded to failure, a limited number was selected for repair. To effect the repair, a cage of reinforcement steel was prepared from 3- x 8-in., No. 0 + 4 wire fabric and placed in the broken pipe sections before gunite was applied to a thickness of approximately 2½ in.

Reinforced Concrete Pipe

Twenty-five pipe were loaded by the three-edge bearing method in accordance with ASTM C 76-59 T. Pipe E-2, as will be shown subsequently (Fig. 26), was tested under a two-point bearing condition. In addition to recording the load required to produce a 0.01-in. crack and the ultimate load, provisions were also made to record load and changes in diameters throughout the entire test. The changes in the vertical and horizontal diameters were measured by means of extension dials (Fig. 10). The load-deformation curves obtained from these tests are shown in Figures 11 through 18. At the top of each figure are sketches showing the position of the pipe and the relative locations of the laps in the two cages of reinforcement steel during the test. An examination of these load-deformation curves indicates certain characteristics (Fig. 19) which are essentially the same for all pipe regardless of fabrication and (or) placement of the steel or position of the pipe in the testing machine.

The load-deformation curves are essentially linear up to the occurrence of the 0.01-in. crack. There is a very slight tendency for the deformation to increase, with little attendant increase in load, when the first crack appears—at something less than 0.1 percent change in vertical diameter and a load of 13 to 18 kips. This tendency for the curve to flatten seems to be more pronounced after the 0.01-in. crack appeared—apparently, there was a slight redistribution of stress from concrete to steel. The 0.01-in. crack typically occurred at a strain of 0.1 to 0.2 percent and at a load of 18 to 25 kips. The slopes of the curves become flatter as the ultimate load (40 to 50 kips) is reached

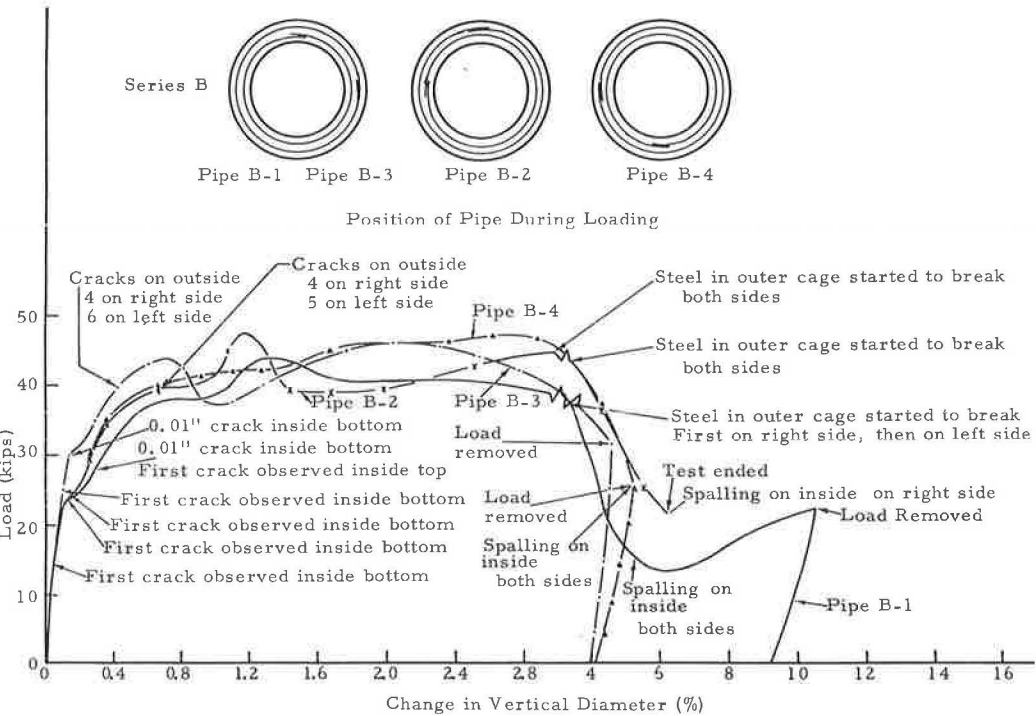


Figure 12. Load-deformation curves for Series B.

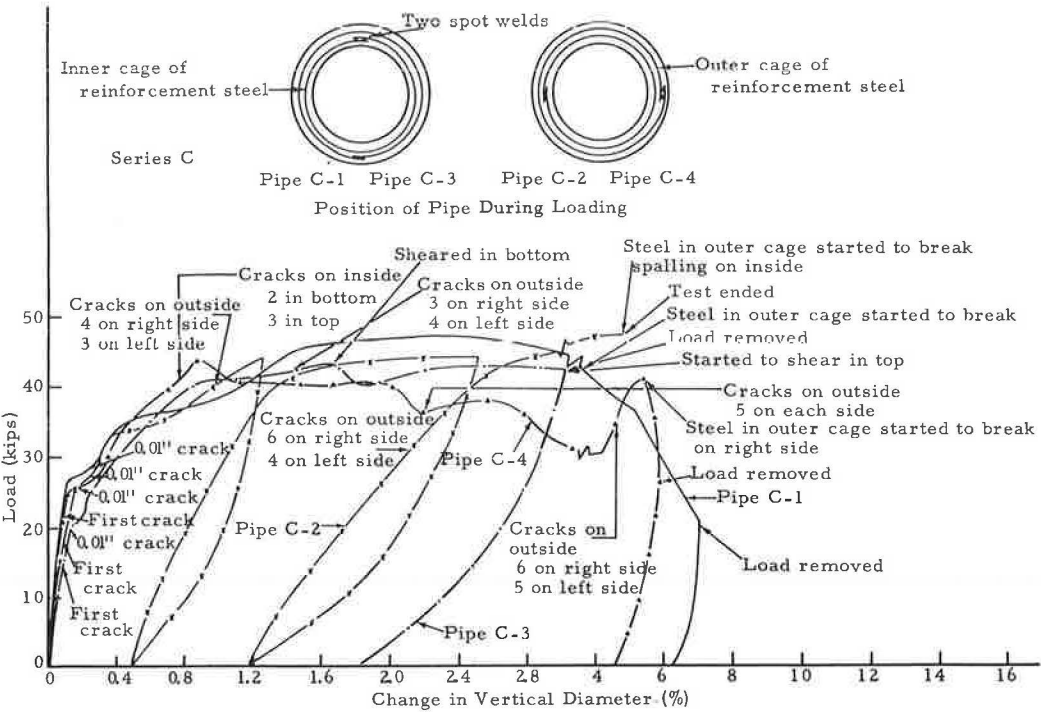


Figure 13. Load-deformation curves for Series C.

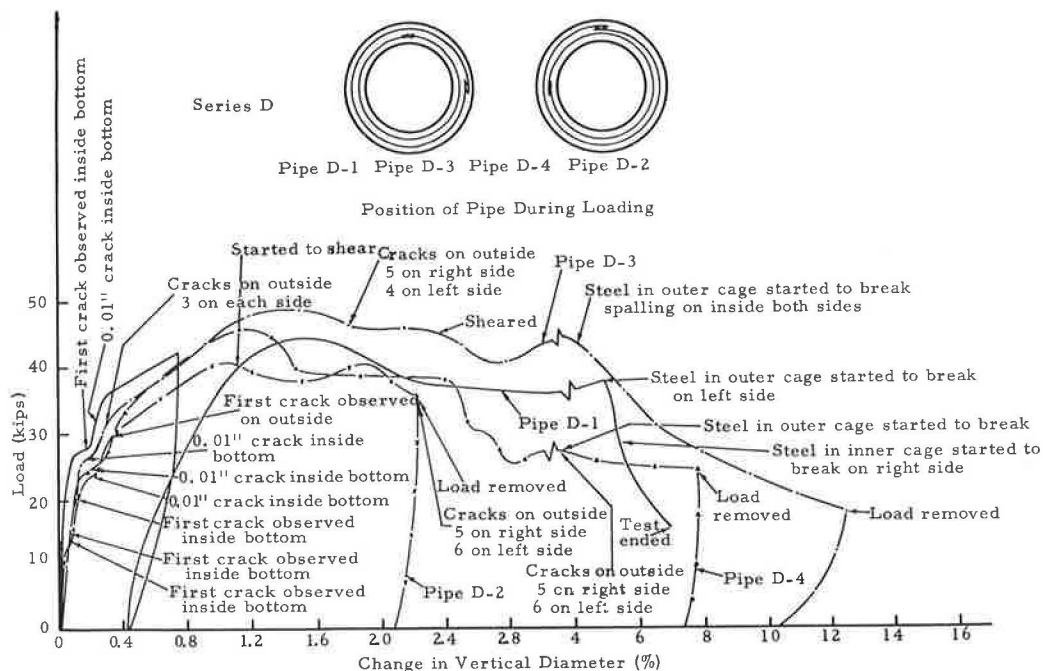


Figure 14. Load-deformation curves for Series D.

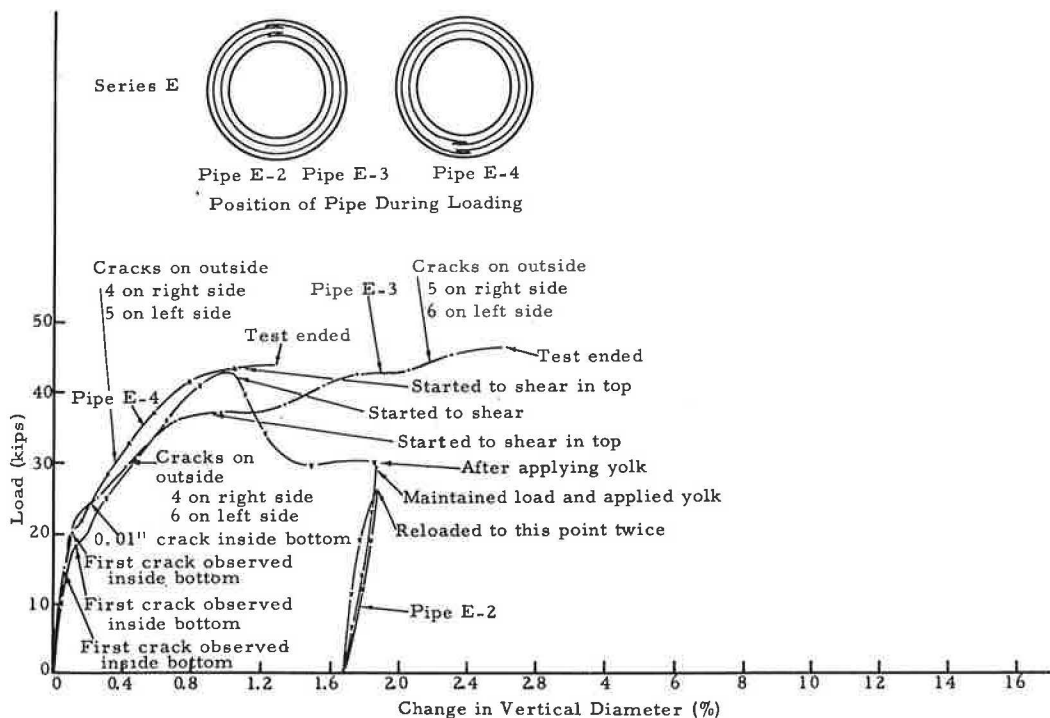


Figure 15. Load-deformation curves for Series E.

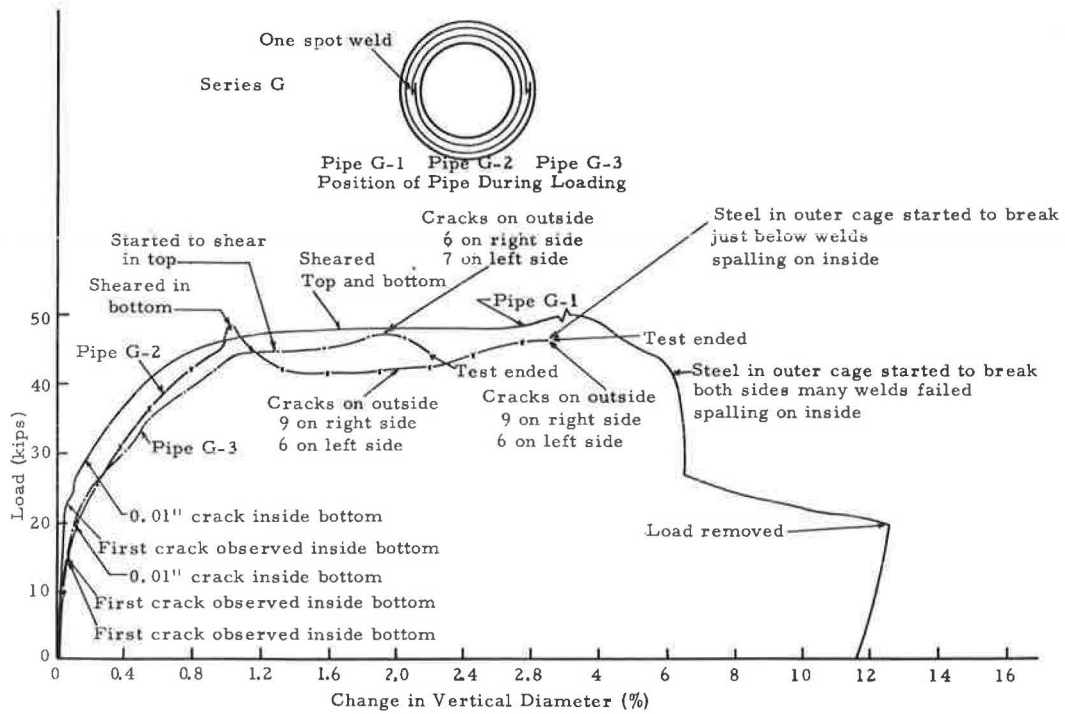


Figure 16. Load-deformation curves for Series G.

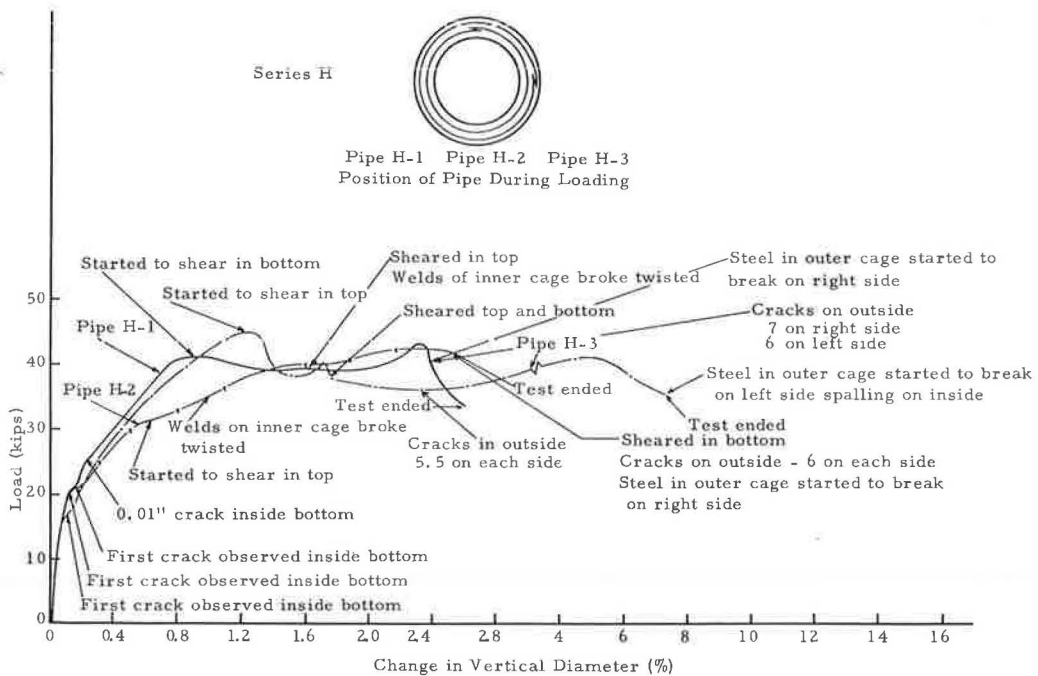


Figure 17. Load-deformation curves for Series H.

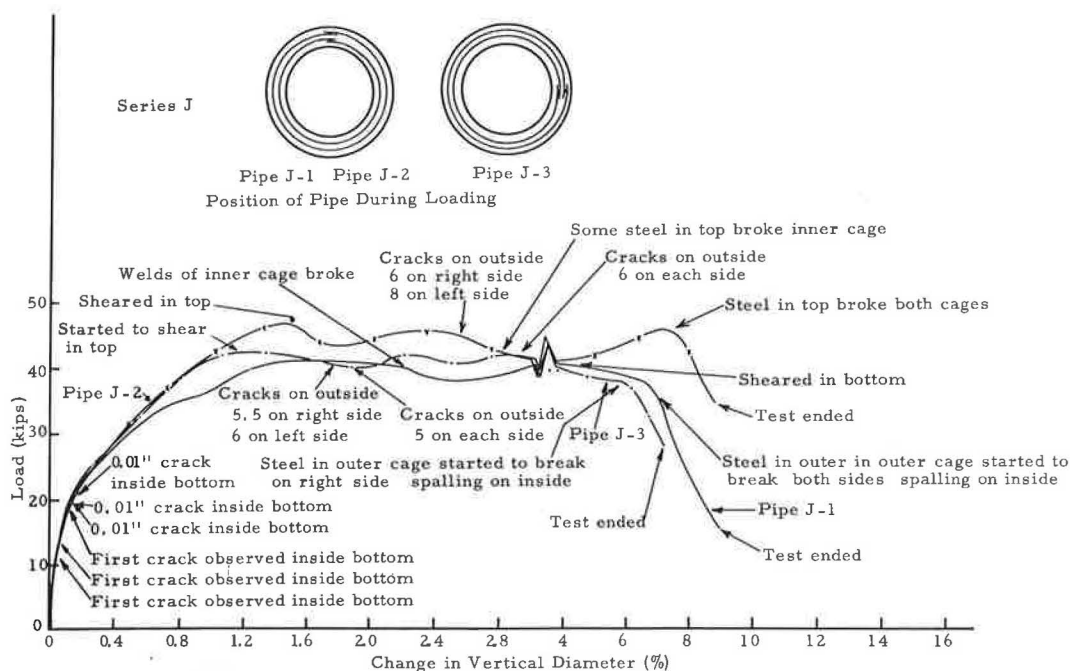


Figure 18. Load-deformation curves for Series J.

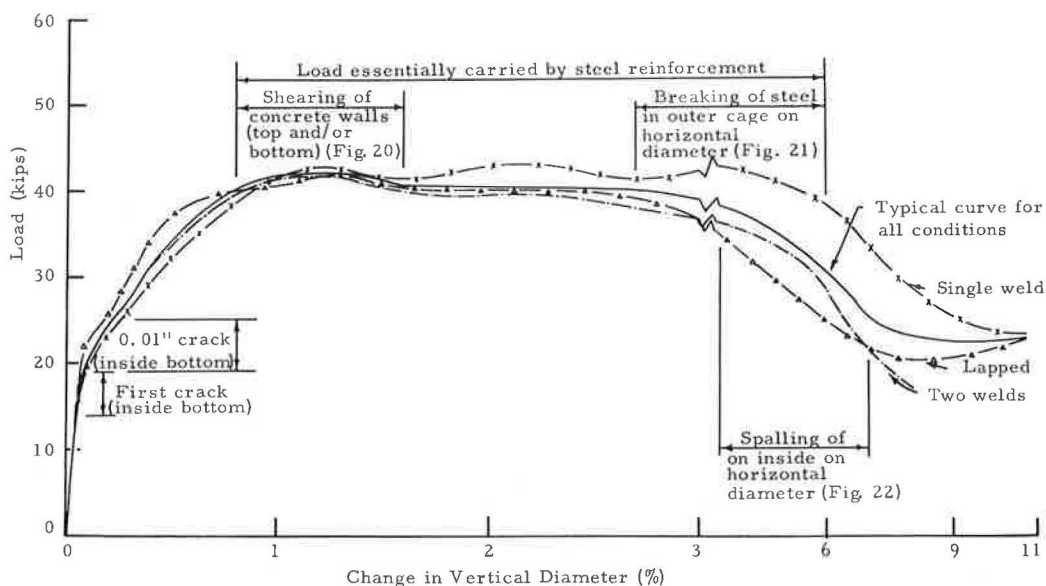


Figure 19. Typical load-deformation curves.

at strains of 0.6 to 1.5 percent. The pipe in Series A and B (steel lapped 40 diameters and tied) appear to have reached the ultimate load at smaller deformations, 0.6 to 1.0 percent, and therefore gave steeper curves than pipe in which the steel was welded.

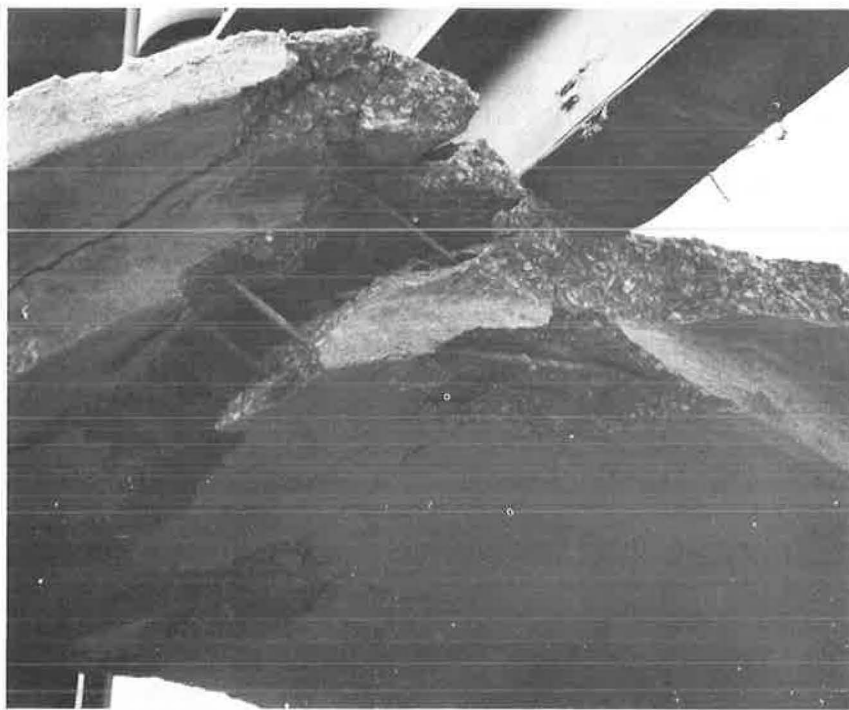


Figure 20. Shear-type failure in top of pipe.

Soon after the ultimate load was reached, the concrete at the top and (or) bottom began to shear (Figs. 4, 5 and 20). There was a slight decrease in the load at this point

and thereafter the load remained rather constant to strains of 2.7 to 6 percent. At deformations of this magnitude the steel started to break in the outer cage about midway up the sidewalls (Fig. 21). Very soon after the steel began to fail, the load decreased rapidly as the stresses transferred to the inside portion of the wall and caused compressive failure of the concrete (Figs. 4 and 22). It may be noted from the load-deformation graphs that pipe not sufficiently loaded to break the steel recovered approximately 50 percent of the strain upon unloading, and that those in which the steel had been broken regained only about 15 percent of the strain.

Other pertinent data from these tests are summarized in Table 1.

TABLE 1
SUMMARY OF TEST RESULTS ON PIPE TESTED IN THREE-EDGE BEARING

Pipe No.	0.01-In. Crack		Ultimate		Date	
	Load (kips)	D-Load (lb/sq ft)	Load (kips)	D-Load (lb/sq ft)	Manufactured	Tested
A-1	23	1,278	37	2,056	11-10-60	12-8-60
A-4	--	--	46	2,556	11-10-60	5-11-61
B-1	28	1,556	44	2,444	11-10-60	12-8-60
B-2	--	--	47	2,611	11-10-60	5-11-61
B-3	30	1,667	46	2,556	11-10-60	5-11-61
B-4	--	--	47	2,611	11-10-60	5-11-61
C-1	28	1,556	47	2,611	11-10-60	12-8-60
C-2	27	1,500	47	2,611	11-10-60	5-11-61
C-3	20	1,111	43	2,389	11-10-60	5-11-61
C-4	25	1,389	44	2,444	11-10-60	5-11-61
D-1	32	1,778	45	2,500	11-10-60	12-8-60
D-2	24	1,333	41	2,278	11-10-60	5-11-61
D-3	26	1,444	48	2,667	11-10-60	5-11-61
D-4	25	1,389	46	2,556	11-10-60	5-11-61
E-2	--	--	43	2,389	11-10-60	5-25-61
E-3	24	1,333	46	2,556	11-10-60	5-25-61
E-4	--	--	44	2,444	11-10-60	5-25-61
G-1	29	1,611	50	2,778	11-14-60	12-8-60
G-2	--	--	43	2,389	11-14-60	5-25-61
G-3	20	1,111	47	2,611	11-14-60	5-25-61
H-1	25	1,389	43	2,389	11-14-60	5-25-61
H-2	--	--	43	2,389	11-14-60	5-25-61
H-3	--	--	45	2,500	11-14-60	5-25-61
J-1	21	1,167	41	2,278	11-14-60	5-25-61
J-2	21	1,167	47	2,611	11-14-60	5-25-61
J-3	21	1,167	43	2,389	11-14-60	5-25-61



Figure 21. Broken steel in outer cage of reinforcement.

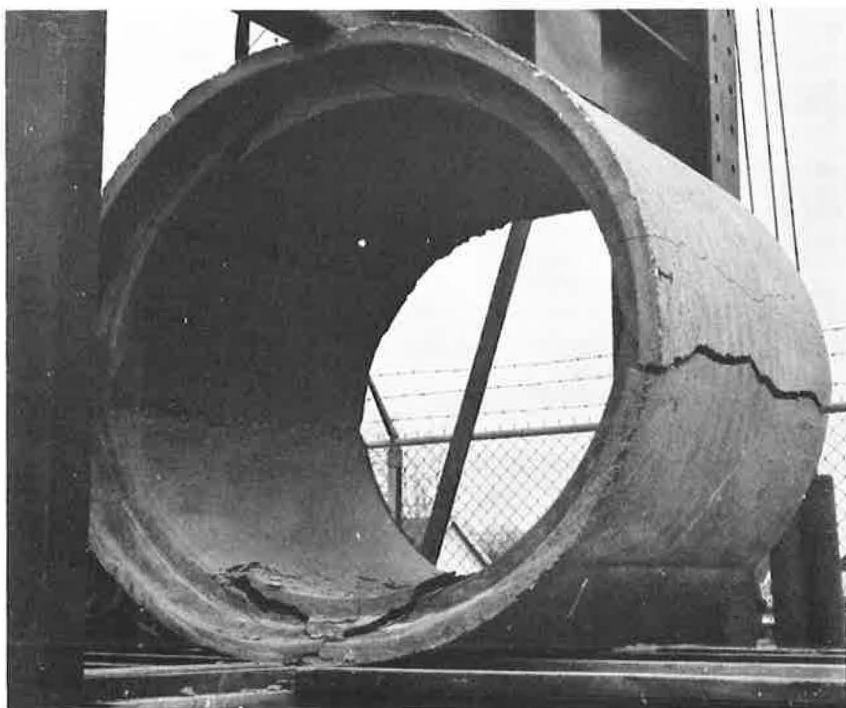


Figure 22. Compressive failure of inner portion of concrete wall.



Figure 23. Non-reinforced concrete pipe under test.

TABLE 2

SUMMARY OF TEST RESULTS ON NON-REINFORCED CONCRETE PIPE (Series F)

Pipe No.	Ultimate Load (kips)	D-Load	Date	
			Manufactured	Tested
F-1	16.5	919	11-9-60	12-6-60
F-2	26.0	1,440	11-9-60	7-18-61
F-3 ¹	23.0	1,280	11-9-60	7-18-61
F-4	22.0	1,220	11-9-60	7-18-61

¹Maximum vertical deflection less than $\frac{1}{8}$ in.

Non-Reinforced Concrete Pipe

Four pipe (Series F) fabricated with no reinforcing steel were broken in the three-edge bearing test (Fig. 23). The results of these tests are summarized in Table 2.

Repaired Pipe

Three pipe (C-3, E-2, and J-2) were used for this portion of the study. These pipe were first loaded in the three-edge bearing, as described previously, to obtain damaged pipe. The results of these loading

tests were given in Figures 13, 15 and 18—which indicate the extent of structural damage before repair was attempted. To effect the repair, a cage of reinforcement was placed in the damaged pipe and gunite was applied and permitted to cure. After repairs had been completed, the pipe were again loaded by the three-edge bearing method.

The cages of reinforcing steel used in the repair work were fabricated from 3- x 8-in., No. 0 to 4 wire fabric. The cages for pipe C-3 and pipe J-2 were prepared from a single section of the wire fabric; the joints were lapped approximately 3 in. and were joined by a single weld on each strand of wire. The cages were positioned in the damaged pipe and a clearance of about 1 in. was maintained between the wire and the wall of the pipe by clipping and bending portions of the No. 4 wire. The cage of pipe E-2 consisted of two semicircular sections of wire fabric lapped approximately 3 in. and spliced together with two spot welds on each strand of wire. The clearance between the wire and the pipe wall was maintained with 1-in. mortar cubes.



Figure 24. Close-up of gunite repair.

TABLE 3
SUMMARY OF TEST RESULTS ON REPAIRED PIPE

Pipe No.	Ultimate		Date			
	Load (kips)	D- Load (lb/sq ft)	Manu- factured	Initial Testing	Repaired	Tested
C-3	90	5,000	11-10-60	5-11-61	6-13-61	7-18-61
E-2	112 ¹	6,222	11-10-60	5-25-61	6-13-61	7-18-61
J-2	94	5,222	11-14-60	5-25-61	6-13-61	7-18-61

¹Yoked to prevent rebound.

The proportioning of the material used to prepare the gunite was three parts sand, two parts cement, and water as needed to obtain the desired consistency. The total quantity of materials used for repair of the three pipe included seven bags of portland cement and 30 gal of water. Approximately 300 lb of material was lost through rebound. The gunite was applied to a thickness ranging between $2\frac{1}{4}$ in. and $3\frac{1}{2}$ in. The gunite was applied first to the sides of the pipe, then to the top and finally to the bottom after removing rebound material with compressed air (Fig. 24). Gunite work was done on June 13, 1961, by a Department of Highways' maintenance crew.



Figure 25. Repaired pipe E-2.

Special provision was made to prevent pipe E-2 from rebounding or recovering its circular shape after it had been loaded beyond its ultimate strength. At the conclusion of the initial loading test, the yoke assembly (Fig. 10) was placed on the pipe to hold it in its collapsed shape until it could be repaired and retested. Figure 25 shows pipe E-2 after it had been repaired and at the time of retesting.

The load-deformation curves for the gunite-repaired pipe are shown in Figures 26 through 28. It is noted that this type of repair is more than adequate to restore structural integrity to the pipe. Table 3 summarizes some data obtained in this portion of the study. Figures 29 and 30 show a repaired pipe before and after the loading test.

Laterally Restrained Pipe

To determine the effects of lateral restraint (lateral resistance) on the load-carrying capacity of pipe, pipe A-2, A-3 and E-1 were laterally restrained by the yoke assembly (Fig. 10) and were then loaded. The load-deformation curves resulting from these tests are shown in Figures 31 to 33.

Even though the lateral restraint was applied by a rather critical, two-point bearing, the load-carrying capacity was increased 150 to 260 percent. When the $\frac{3}{8}$ -in. diameter tie rods or the tie rods having the reduced section were used, this increase was 150 to 180 percent, and when the $\frac{3}{4}$ -in. diameter tie rods were used in the yoke assembly, the increase in ultimate load was 230 to 260 percent.

The overall slope of the load-deformation curves was somewhat steeper when the lateral restraint was applied than when the pipe was not restrained. The first crack observed in these restrained pipe was at a load of approximately 20 kips and a strain of less than 0.1 percent; the 0.01-in. crack occurred at loads of 25 to 30 kips and strains of 0.1 to 0.15 percent. Table 4 summarizes some of the data obtained from these tests.

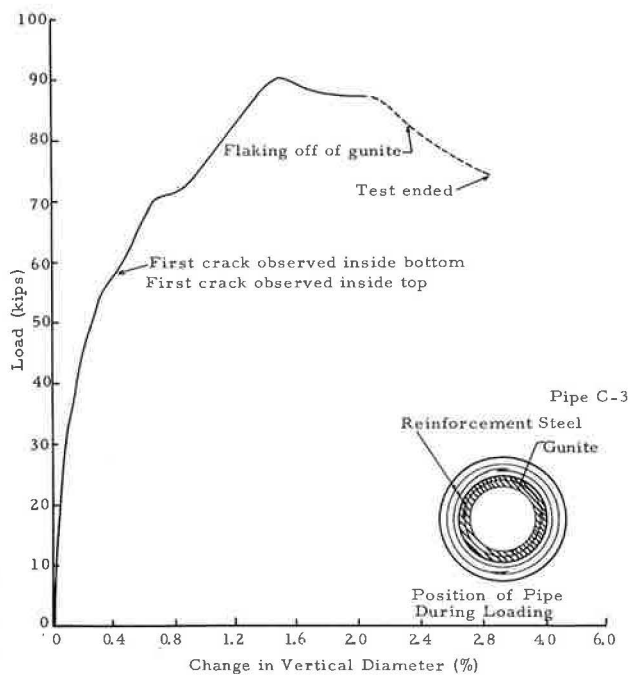


Figure 26. Load-deformation curve for repaired pipe C-3.

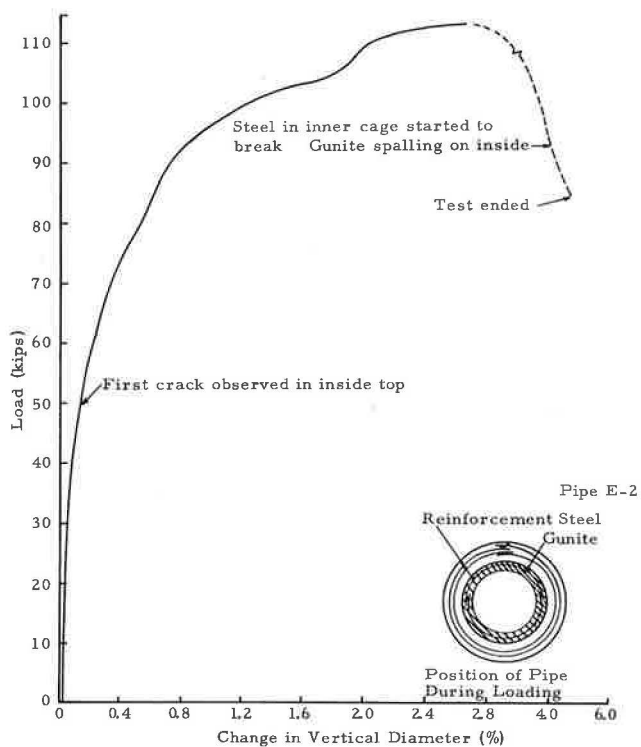


Figure 27. Load-deformation curve for repaired pipe E-2.

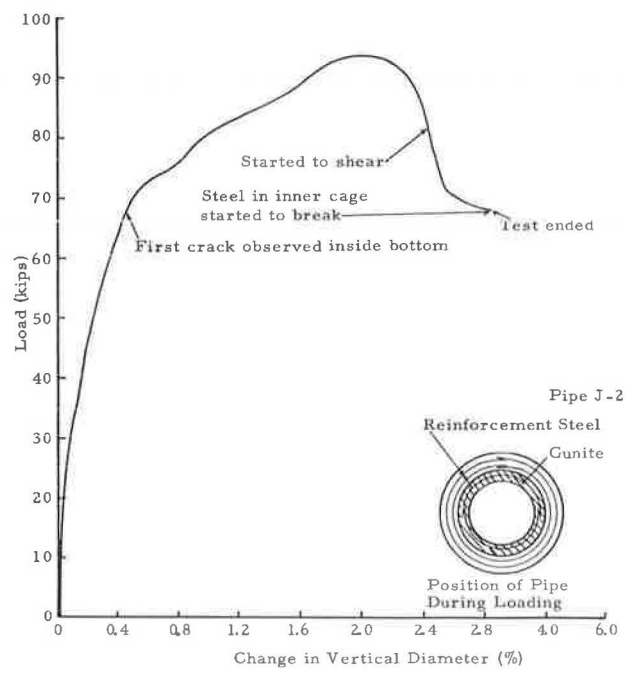


Figure 28. Load-deformation curve for repaired pipe J-2.

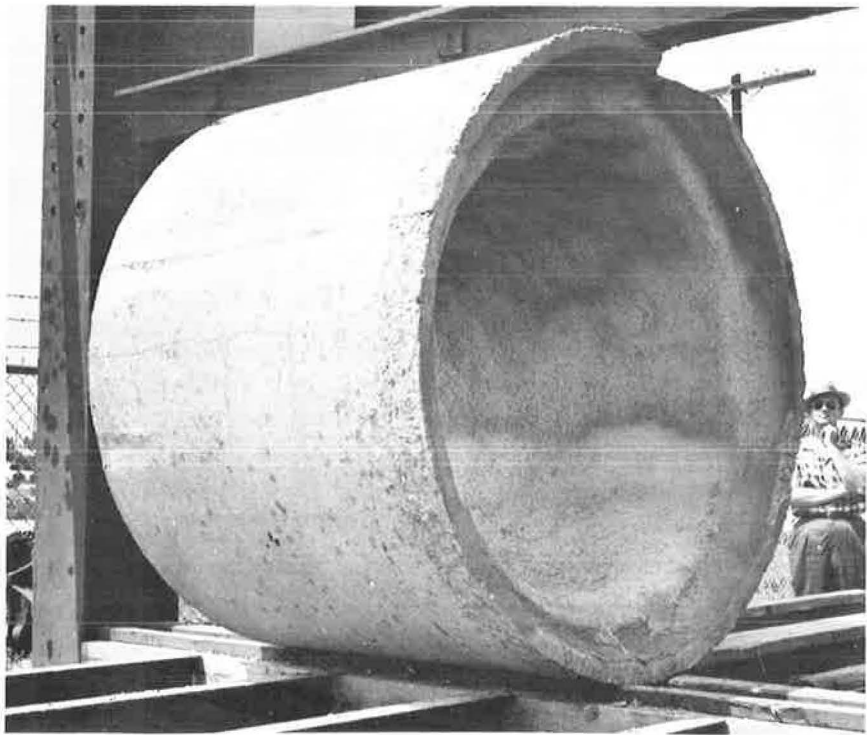


Figure 29. Repaired pipe before testing.

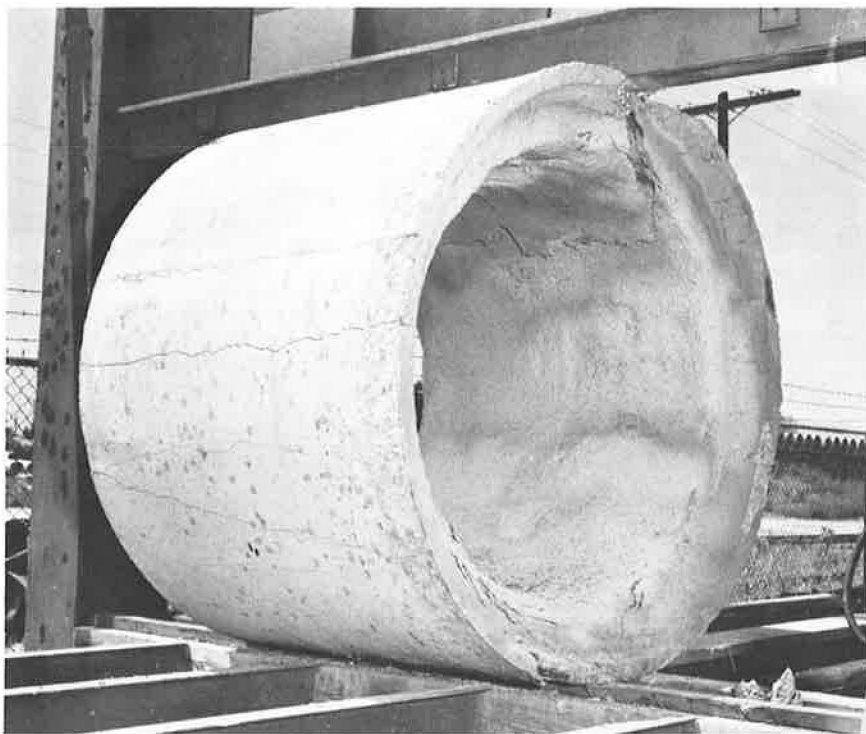


Figure 30. Repaired pipe after testing.

TABLE 4
SUMMARY OF TEST RESULTS ON LATERALLY RESTRAINED PIPE

Pipe No.	0.01-In. Crack		Ultimate		Date	
	Load (kips)	D-Load (lb sq/ft)	Load (kips)	D-Load (lb sq/ft)	Manufactured	Tested
A-2	25	1,389	74 ¹	4,111	11-10-60	5-9-61
A-3	25	1,389	105 ²	5,833	11-10-60	5-9-61
			68 ³	3,778		
			104 ²	5,778		
			53 ⁴	2,944		
E-1	30	1,667	61 ⁵	3,389	11-10-60	12-20-60
			93 ²	5,167		

¹Tie rods having reduced sections 2.6 in. in length, 0.30 in. in diameter.

²Tie rods $\frac{3}{4}$ -in. uniform diameter.

³Tie rods having reduced sections 8 in. in length, 0.30 in. in diameter.

⁴No lateral restraint.

⁵Tie rods $\frac{3}{8}$ -in. uniform diameter.

The ultimate load data summarized in Table 4, as well as those contained in Tables 1 and 2, are presented in graphical form in Figure 34. The sums of the areas of steel in the internal reinforcement and in yoke-assembly tie rods have been plotted as a function of the ultimate loads obtained in the bearing tests. A similarity is noted here in the linear relationships apparent in these data and in those presented by the American Concrete Pipe Association (5); that is, the additional area of steel contained in the yoke-assembly tie rods increases the load-carrying capacity of the pipe in much the same manner as if this steel had been included in the internal reinforcement. Also, the ulti-

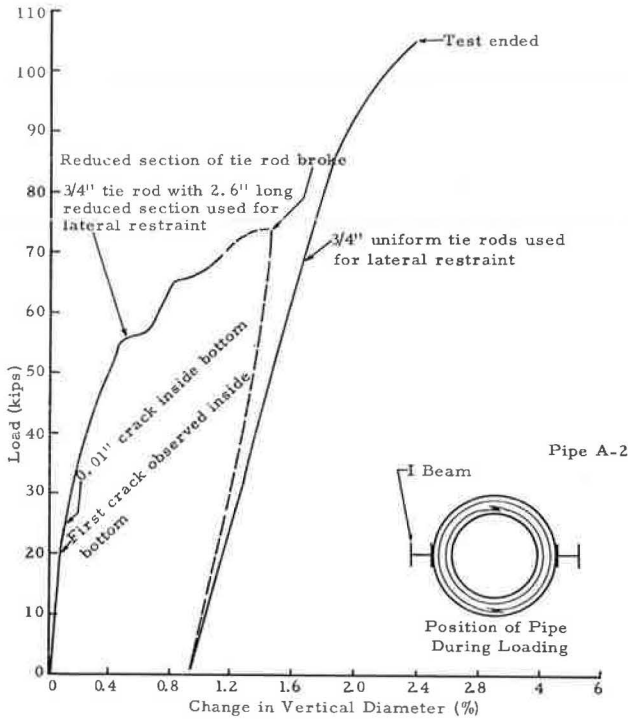


Figure 31. Test results for pipe A-2: (1) restrained by four 0.30-in. diameter, 2.6-in. long, reduced-section tie rods; loaded to failure of reduced section and then unloaded; and (2) restrained by four $\frac{3}{4}$ -in. uniform-diameter tie rods and loaded to ultimate. Both sets of tie rods elongated (yielded); elongation was concentrated in reduced section in first test condition; uniform rods elongated uniformly throughout their full length.

Figure 32. Test results for pipe A-3: (1) restrained by four 0.30-in. diameter, 8-in. long, reduced-section tie rods; loaded to ultimate and then unloaded; (2) restrained by four $\frac{3}{4}$ -in. uniform-diameter tie rods; loaded to ultimate and then unloaded; and (3) loaded to ultimate without lateral restraint.

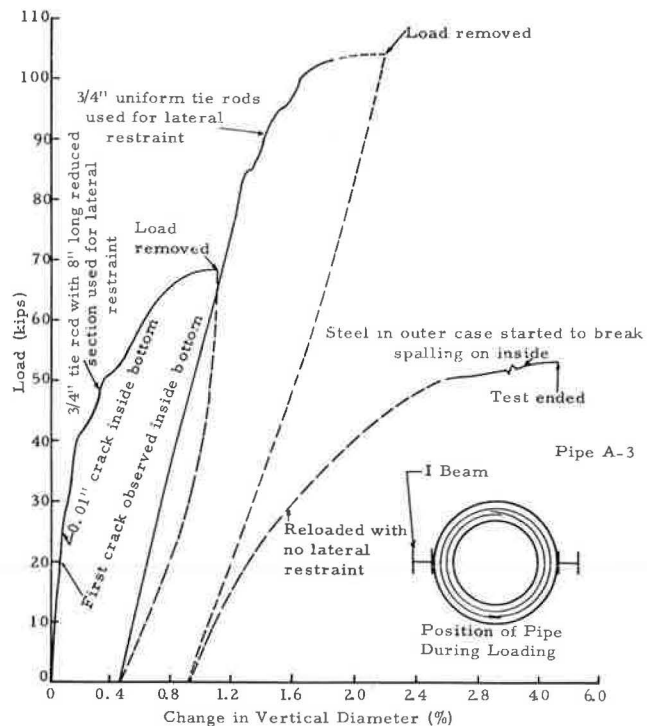


Figure 33. Test results for pipe E-1: (1) restrained by four $\frac{3}{8}$ -in. uniform-diameter tie rods; loaded to ultimate and then unloaded; and (2) restrained by four $\frac{3}{4}$ -in. uniform diameter tie rods; loaded to ultimate.

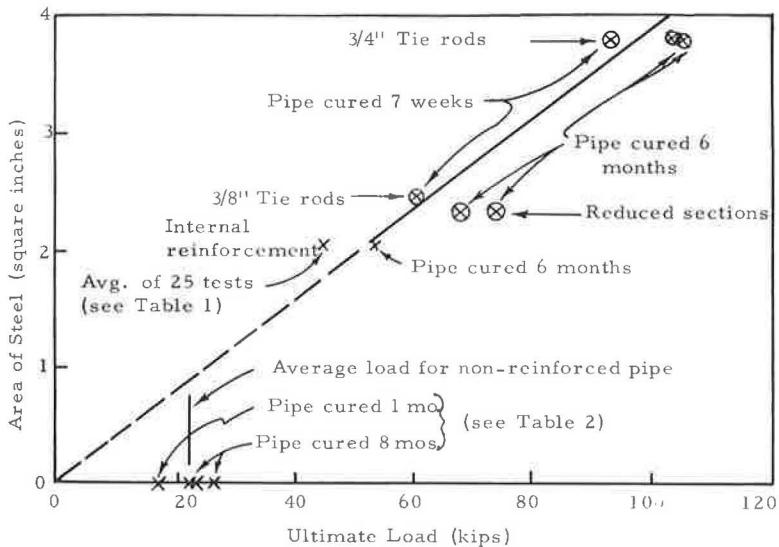
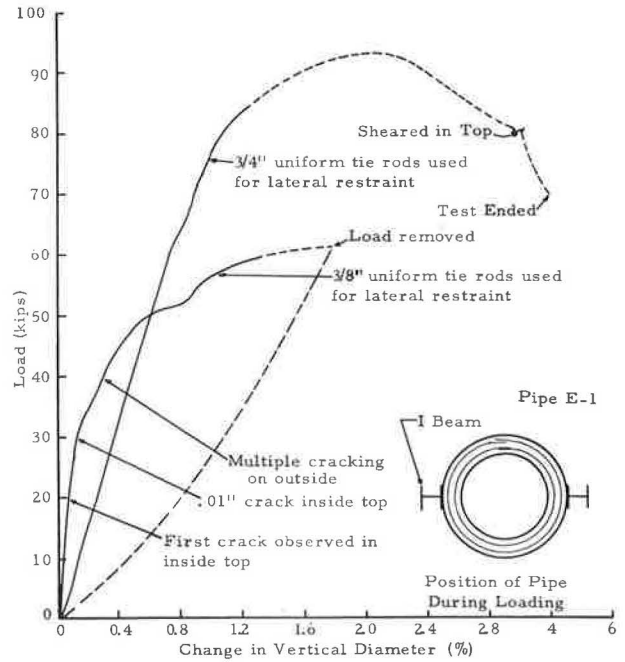


Figure 34. Relationship between ultimate load and total area of steel (internal reinforcement plus external tie rods).

mate load sustained by the non-reinforced concrete pipe (20 to 25 kips) was on the order of the load required to produce the 0.01-in. crack in the reinforced pipe.

The restraint offered by the tie rods of the yoke assembly may be, in a manner, considered to represent the reactance provided by the earth pressure around a culvert in a field installation. Inasmuch as it has been previously suggested that the total cross-sectional area of the tie rods can be equated to a similar increase in the area

of internal reinforcement, it may be inferred that lateral earth pressures on the pipe culvert can also be related to an equivalent area of internal reinforcing steel—that is, the resistance of the soil to lateral deformations is analogous to diametrical reinforcement. Thus, compaction of backfill contributes to the load-carrying capacity of a pipe in much the same way as internal reinforcing steel.

SUMMARY OF SIGNIFICANT FINDINGS

The principal findings regarding the fabrication and location of joints in the reinforcing steel are summarized as follows:

1. The 0.01-in. crack strength and the ultimate strength are not significantly affected by the three methods used to fabricate joints in the reinforcing cages. Data indicate that there is very little difference in the overall performance of pipe prepared with: (a) lapped-and-tied joints (Fig. 7), (b) double-welded joints (Fig. 8), and (c) single-welded joints (Fig. 9).

2. With regard to the relative positions of joints in the outer and inner cages of reinforcement, there appears to be no measurable difference in the performance of pipe in which the joints are spaced: (a) 0 deg apart, (b) 90 deg apart, and (c) 180 deg apart.

3. The load-carrying capacities of the pipe fabricated for this investigation were not significantly affected by location of joints in the three-edge bearing tests. Pipe oriented with their joints in the most critical location (points of maximum bending, that is, four cardinal points) performed as satisfactorily as those pipe in which joints were located in more favorable positions.

The findings from the other phases of study are summarized as follows:

1. The ultimate loads withstood by non-reinforced concrete pipe were approximately equal to the load required to produce the 0.01-in. crack in the reinforced pipe.

2. The repairs, which were made on structurally damaged pipe, with reinforced-gunite inner liners, sufficed to restore original load-carrying capacity to the pipe.

3. Steel in tie rods used to provide lateral resistance (horizontal reactance to vertical loading) appeared to increase the load-carrying capacity of pipe to the same extent that an equal amount of internal reinforcement would have provided.

These findings have already proved to be of value and suggest additional areas of study. Committee C-13 of ASTM approved the following revision to C76 in 1962 (AASHTO Committee on Materials subsequently approved a revision of M170 to conform to ASTM C76-62T):

When splices are welded and are not lapped to the minimum requirements above, pull tests of representative specimens shall develop at least 75 percent of the minimum specified strength of the steel.

The apparent relationships between the action of external, horizontal tie steel and the action of internal reinforcing steel may engender further study and lead, perhaps, to more meaningful relationships between passive earth-pressures and the ability of D-strength pipe to withstand fill loads.

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Discussion

M. G. SPANGLER, Research Professor of Civil Engineering, Iowa State University, Ames—The authors have included some appropriate suggestions relative to the repair in situ of reinforced concrete pipes which have suffered structural damage due to earth overburden load. The writer has had an opportunity to investigate a number of pipelines which have been damaged and would like to add his experiences, opinions, and recommendations for repair work to those expressed by the authors.

When a reinforced concrete pipe is subjected to earth loading and visual evidence of structural effect appears, such evidence can be roughly divided into three stages or categories. The first stage is the appearance of fine hairline longitudinal cracks in the concrete surface in the invert and later in the crown of the pipe. As the load increases, the cracks open wider and become more plainly visible, and additional cracks, roughly parallel to the first, may develop. These cracks are caused by bending moments produced in the pipe wall by vertical load and reaction. Similar cracks may develop in the outside surface of the pipe at the springings, because bending moments at these locations are in the opposite direction from those at the top and bottom. They, of course, are not visible from the inside of the pipe.

If the load continues to increase, a second stage of visible damage frequently develops. This is a "slabbing" or "spalling" of the protective cover of concrete over the reinforcing steel at the invert and/or the top of the pipe. This action is caused by the fact that as the pipe deforms under load—the vertical diameter shortens and the horizontal diameter lengthens—the inner cage of steel has a tendency to change shape faster than the more rigid concrete wall in which it is embedded. This causes radial inward forces to be exerted against the protective cover of concrete, which is thereby stressed in direct tension (Fig. 35a). These tensile stresses produce the spalling effect. Similar spalling in the tensile zones on the outside of the pipe at the springings does not develop, inasmuch as the radial stresses in these areas are directed inward toward the central core of the wall rather than toward the relatively thin protective cover, as is the case at the bottom and top on the inside of the pipe.

A preliminary or incipient phase of spalling is the development of rupture of the protective cover of concrete along a circumferential surface at the inner cage of reinforcement. This rupture surface may not be visible from the inside of the pipe, but can be identified by tapping lightly on the pipe wall with a ball peen hammer. Areas thus affected will yield a hollow sound under the hammer. A circumferential rupture surface at the end of a pipe is shown in Figure 36.

This spalling action can be prevented or greatly inhibited by incorporating radial ties in the pipe wall at the top and bottom during manufacture as shown in Figure 35b. The function of these ties is to hold the inner cage of steel in position and prevent its displacement toward the center of the pipe, thus eliminating the tendency for radial stress damage to the concrete cover.

A third stage of structural damage may develop in extreme cases of overloading. This stage is characterized by relatively

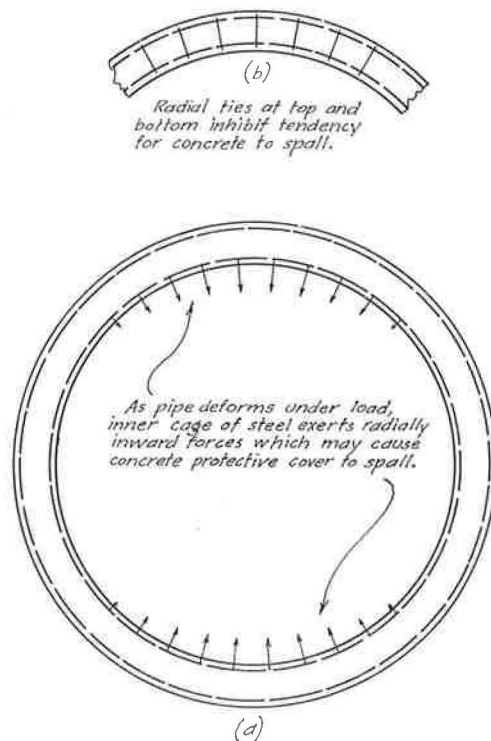


Figure 35.

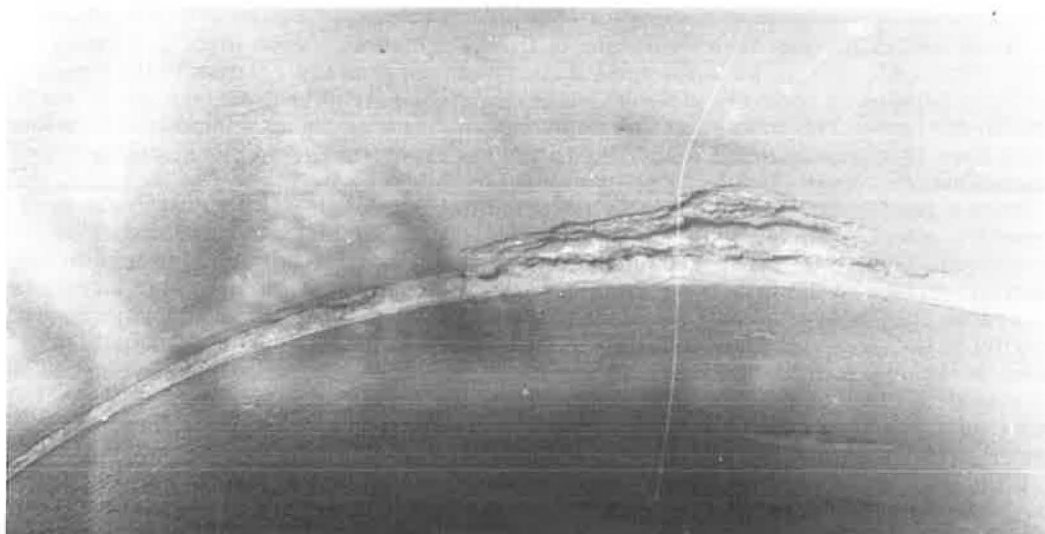


Figure 36. Incipient spalling stage II damage. End view of circumferential crack in protective cover over reinforcement.

large changes in the vertical and horizontal diameters; by the development of shear and diagonal tension failures in the concrete, especially in the bottom of the pipe; and by compression failure of the concrete on the inside surface along longitudinal elements at the springlines. A pipe which has reached this stage has lost much of its inherent strength as an elastic ring. It has become essentially a 4-hinged ring which depends almost wholly on the passive resistance pressure of the enveloping soil for maintenance of its shape and ability to carry vertical load.

Cause of Structural Damage

Structural damage to the extent indicated in the several stages previously described is frequently brought about by one or both of two details of construction in connection with installation of the pipes. One of these is the excavation of a trench to a width substantially in excess of that assumed in the design of a ditch conduit. The load on a pipe in a trench is very sensitive to the width of the trench at the elevation of the top of the pipe. If for any reason the installer elects to increase this width, he should make sure that the pipe is sufficiently strong to support the additional load. On one project involving a 24-in. pipeline, the design width and the specified width of trench was 48 in. The trench was actually dug 56 in. wide in some areas and 62 in. in others. This increased the load on the pipe by approximately 25 and 46 percent, respectively. The factor of safety used in design was not large enough to absorb these increases in load, and the sewer was in trouble.

Another frequent cause of structural distress is failure to obtain a good quality of bedding during construction. The function of the pipe bedding is to distribute laterally the upward reaction on the bottom of the pipe. The greater the width of reaction distribution, the less will be the bending moment in the pipe wall and the greater the load-carrying capacity of the pipe, other factors being equal. For example, suppose two similar pipes are subjected to vertical loads which are distributed uniformly over the top 180 deg. If the quality of bedding on one of them is such that the bottom reaction is uniformly distributed over 60 deg of arc, the maximum bending moment in the pipe wall (neglecting lateral pressure on the pipe) is

$$0.189 W_c r$$

(1)

in which

W_c = load on pipe, plf; and
 r = mean radius of pipe, ft.

In contrast, if the bottom reaction is uniformly distributed over a narrower width of 10 deg, the bending moment is

$$0.272 W_c r \quad (2)$$

This represents a reduction of 30 percent in pipe strength due simply to the reduction in width of reaction distribution from 60 to 10 deg. In other words, a pipe on a 60-deg bedding is about 1.4 times as strong—that is, it will carry 1.4 times as much earth load as if it were supported on a bedding 10 deg wide.

Marston recognized the influence of bedding width on the strength of pipes in the ground. His earliest definitions of "Ordinary Bedding"—Class C—and "First Class Bedding"—Class B—called for shaping the soil at the bottom of an excavation to fit the contour of the pipe to get good reaction distribution. "Impermissible Bedding"—Class D—was the term used to designate the situation where a pipe is laid directly on a flat bed of soil with no effort made to shape it to the pipe contour, and as the name implies, this type of installation was not recommended. If pipes are laid on a flat bed of soil, it may be necessary to use a higher strength of pipe to insure that the load will be carried satisfactorily.

Obviously it takes time and requires careful workmanship to shape the soil to obtain a good bedding and good reaction distribution. Sometimes, contractors install a pipe on a flat bed of soil and then attempt to get satisfactory bedding by compacting the soil beneath the lower haunches of the pipe, but this procedure is not always successful. Two pipeline installations in which this method of bedding pipes was employed and which the writer has recently had an opportunity to study are described.

The first of these was in a region of "fairly stiff glacial till." The installation procedure was to blade this natural soil to flat surface at an elevation approximately 1 in. below the grade of the bottom of the pipe. A layer of pit run, fine sand was spread on the soil and brought to grade and the pipe installed on the sand layer. Next, sand was placed alongside the pipe up to the springlines and allowed to take its angle of repose. Job site soil was then bladed up at the sides of the pipe adjacent to the sand and compacted by the wheels of a maintainer operating parallel to the pipeline. The wheels were run as close to the pipe as possible. Laterally beyond the width of the maintainer the soil was compacted by sheepsfoot roller. Figure 37 shows the installation procedure.

In the second case, the natural soil was a sandy material which was sufficiently dense and had enough cohesion to enable it to stand temporarily on a fairly steep slope.

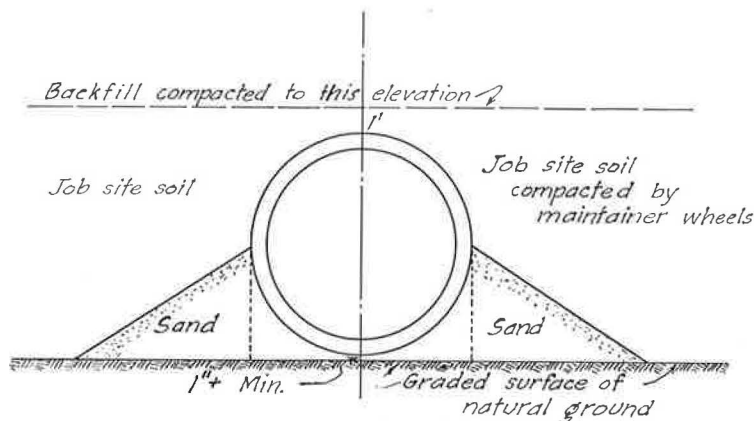


Figure 37. Pipe bedding.

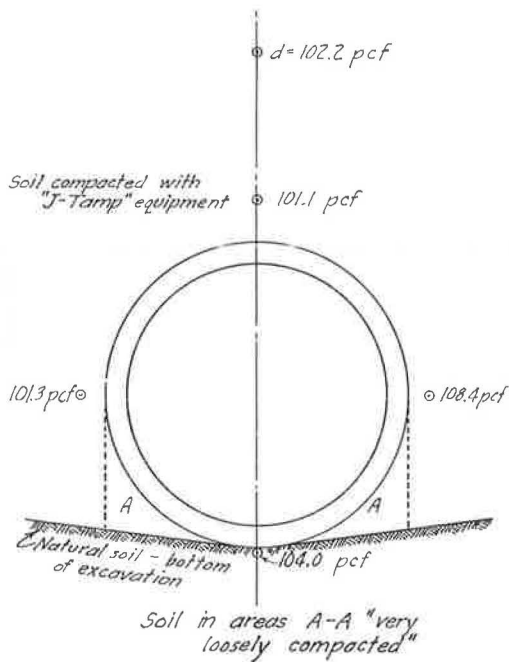


Figure 38. Pipe bedding.

The pipes were installed on this compact material at the bottom of the excavation, which was not shaped to fit the contour of the pipe. Backfill was placed under the haunches and alongside the pipe. The soil at the sides was compacted in layers by means of mechanical equipment which had a tamping face about 12- x 18-in. in area. Tamping was carried out in a vertical direction only, with no special effort made to compact soil in the triangular areas below the springlines.

Both of these pipelines developed evidence of stage I structural effect and a limited amount of stage II effect. In the second case described, several sections of pipes were removed and the backfill soil and bedding of the pipes were examined. Measurements of soil density above and at the sides of the pipes indicated good quality compaction. Also, a density measurement in the bedding directly beneath the longitudinal centerline indicated a compact supporting material in this region (Fig. 38). However, the backfill beneath the lower haunches of the pipe was described by the engineers who observed the removal of the pipe as "very loosely compacted."

Obviously, in both these cases the bottom reaction was concentrated over a relatively narrow longitudinal element. This caused the development of relatively high bending moments and the resulting structural damage to the pipe.

When pipes are installed on a flat bed of soil, as previously described, special care and effort must be exercised to obtain good quality compaction of backfill in the critical areas beneath the lower haunches of the pipes. It must be realized that compaction of soil in the vertical direction does not spread laterally. The soil under the haunches must be specifically and thoroughly compacted; otherwise these areas will not be capable of providing satisfactory distribution of the bottom reaction on the pipe. A much better procedure, one which will produce more positive and reliable results, is to shape the bedding material to fit the contour of the pipe. The objective of obtaining a wide distribution of the upward reaction should be kept constantly in mind during construction.

Another circumstance which may lead to difficulty due to concentration of the reaction, is the presence of bed rock or other highly compact and unyielding material at a shallow depth beneath the bottom of a pipe. There should always be sufficient bedding material of a yielding character to prevent approach to a strain-resistant stratum as the pipe settles normally under vertical load. This bedding material should be such that the pipe can "nestle down" and develop good reaction distribution.

Repairs

It is feasible and usually economical to repair damaged reinforced concrete pipe in situ without removal and replacement. Pipes which are cracked longitudinally to the extent described in stage I, cannot be said to have failed in any reasonable sense of the word. As a rule, such cracks indicate nothing more than that the reinforcement steel is being stressed at a level somewhere near that for which it was designed and which it is capable of carrying. Unless the cracks are wide enough to permit corrosion of the steel, they are not at all damaging to the pipe. Longitudinal cracks typical of stage I structural effect are shown in Figures 39 and 40.



Figure 39. Stage I cracks in crown of pipe— $\frac{1}{100}$ -in. crack center; hairline crack, right; very fine hairline crack, left.



Figure 40. Stage I crack in pipe invert—approximately $\frac{1}{16}$ in. wide.

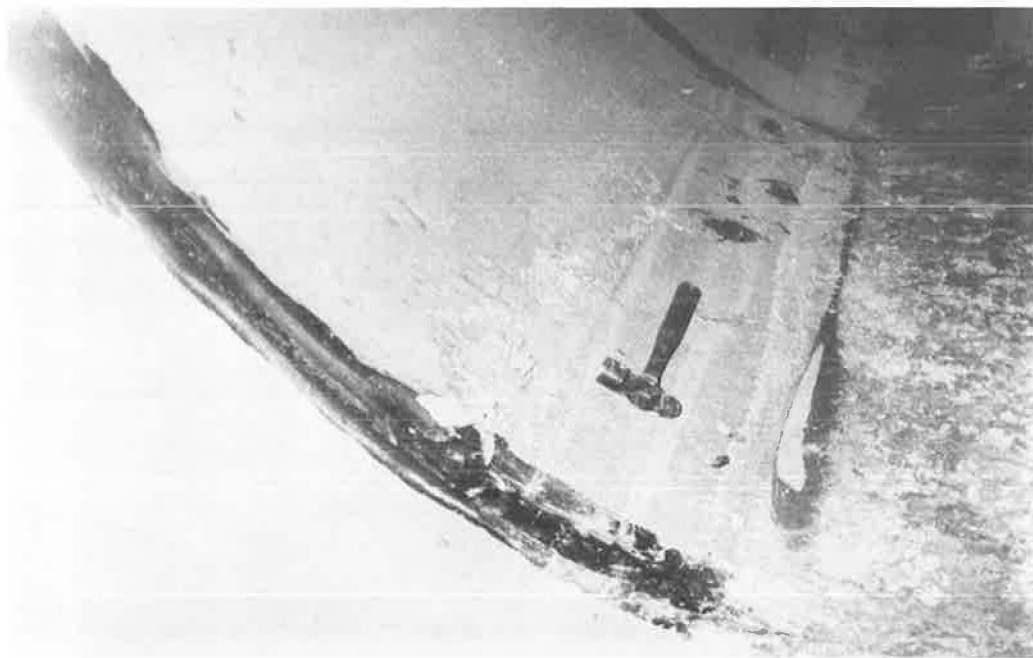


Figure 41. Spalled concrete, stage II damage to left of hammer, ice in pipe invert at right.

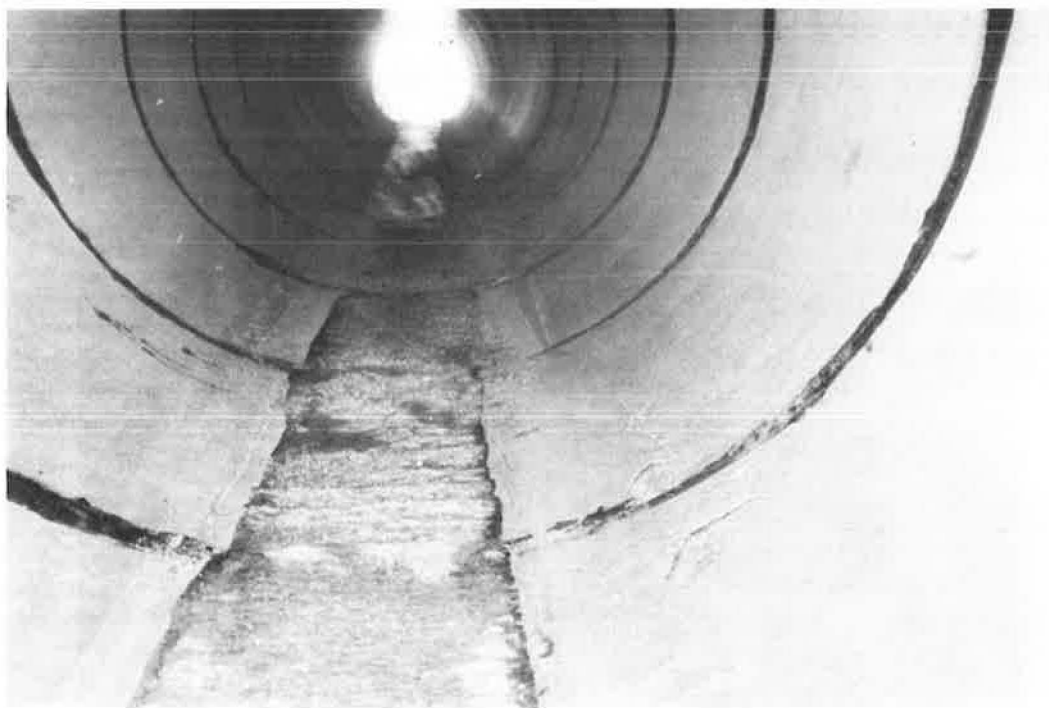


Figure 42. Spalled concrete, stage II damage.



Figure 43. Advanced stage II damage to 108-in. pipe. When damage was first discovered, engineer ordered timber struts installed. Later these were removed and a system of diameter measurements established, which indicated that the pipe had reached state of equilibrium. Beddings were improved by grouting, then pipe walls were repaired with concrete. Pipeline has served satisfactorily for 14 years (at this writing).

Opinions vary relative to the width of a crack which will permit the steel to corrode and deteriorate. It is this writer's opinion that cracks up to about $\frac{1}{16}$ in. in width can be safely tolerated in a reinforced concrete pipe, unless the environment is unusually conducive to corrosion of steel. If cracks greater than $\frac{1}{16}$ in. have developed, they can be repaired by reaming out with an air chisel to the depth of the steel and replacing with gunite concrete or some similar protective material. It is not wise to try to fill the cracks without first reaming them out.

Damage described under stage II (that is, spalling of the protective cover over the steel at the top or bottom) has the appearance of a very severe condition (Figs. 41 and 42). One engineer described it by saying, "It looks like the foundation is coming up through the bottom of the pipe." Actually it is not as severe as it appears to be and can be repaired with complete safety and confidence. When this type of damage occurs, the pipe has deformed enough to develop a substantial amount of passive resistance pressure of the soil at the sides. It has, in effect, become a "semi-rigid" pipe and a state of equilibrium develops in which the vertical load is carried by the residual inherent strength in the pipe plus the passive resistance pressure against the sides. This state of equilibrium develops in early phases of pipe damage and can be verified by measuring the vertical and horizontal diameters of the pipes and marking the points

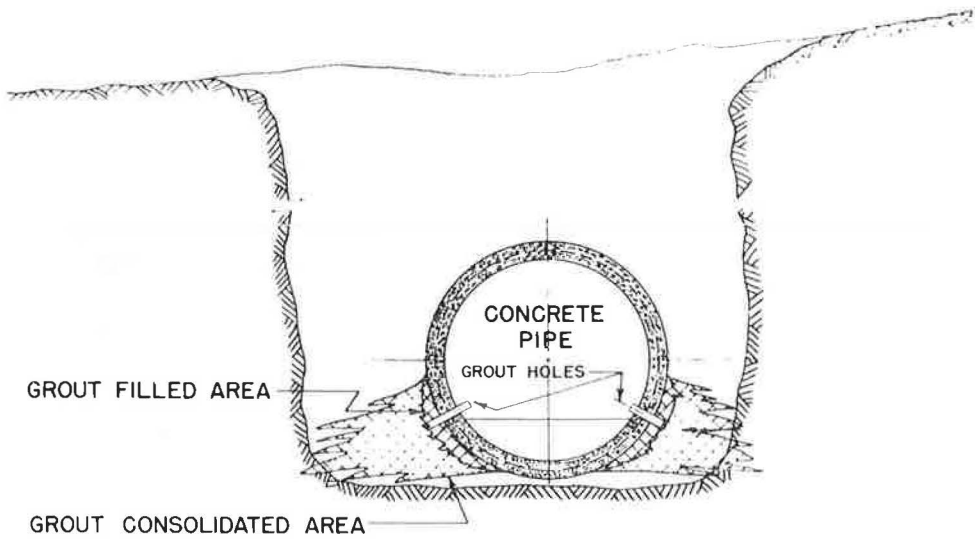


Figure 44. Method of improving pipe bedding by grouting.



Figure 45. Stage II damaged 84-in. pipe removed for inspection of grout injected through holes drilled approximately at lower quarter points. After bedding was improved, pipes were repaired on the inside with gunite concrete. Pipeline has served satisfactorily for 9 yr (at this writing) and repairs appear to be permanent.

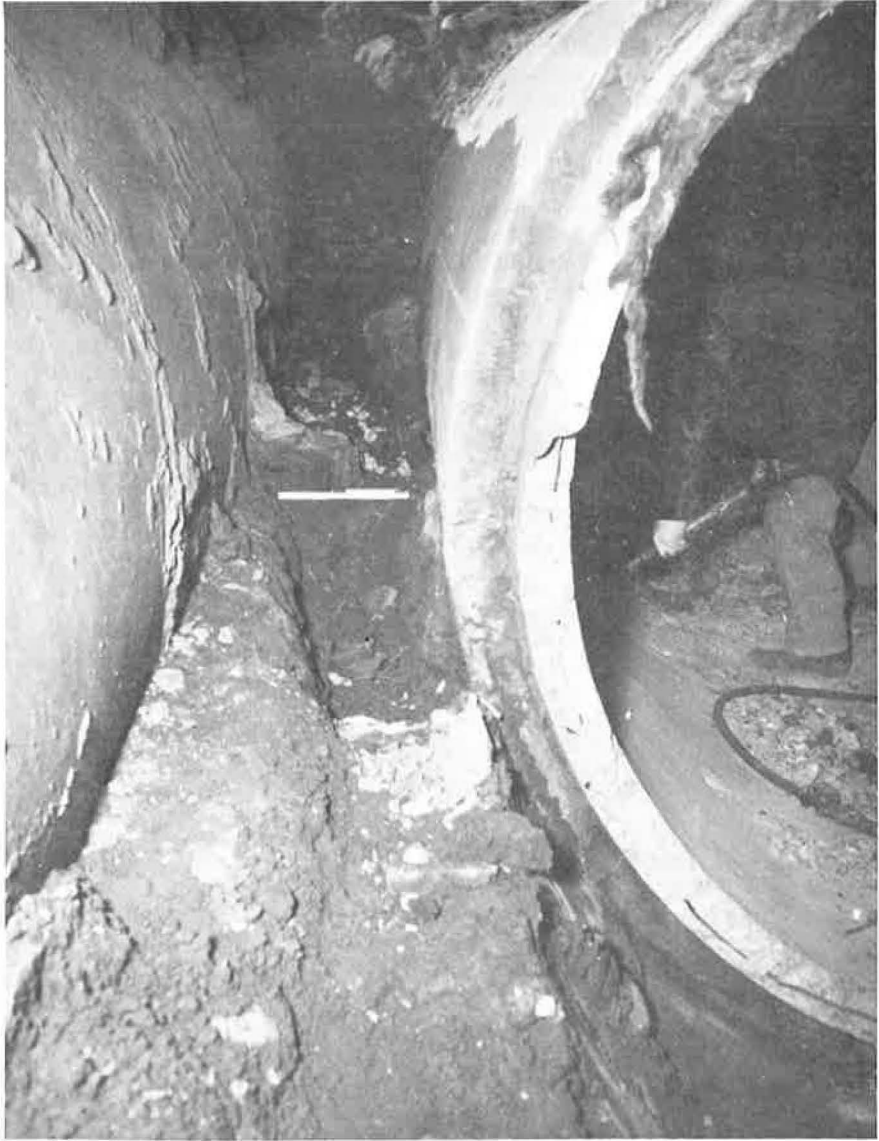


Figure 46. Parallel 84-in. pipelines only 1 ft apart. Impossible to do an adequate job of compacting soil under haunches with pipes so close together.

between which measurements are made. Then repeat the measurements at about 30-day intervals until a steady state is indicated. The pipe may then be repaired by reaming out all cracks wider than $\frac{1}{16}$ in. and removing all spalled and broken concrete with an air chisel. Then replace with gunite concrete to protect the steel against corrosion.

In severe cases of stage II damage (Fig. 43), when it is known that the density of soil beneath the lower haunches is deficient, the pipe can be strengthened by pressure grouting the soil in this region. Holes may be drilled in the pipe wall between the spring line and the lower quarter point and grout forced into the soil (Fig. 44). This improves the bedding situation and increases the lateral pressure and the pipe is greatly strengthened. An 84-in. pipe grouted in this manner and then removed to observe the results is shown in Figure 45. After the grouting operation was completed, gunite repairs of the inside of the pipe were carried out.

When two pipelines are laid parallel, they should be placed sufficiently far apart to permit workmen to do an effective job of compacting the soil beneath the lower haunches of the pipes. Figure 46 shows two parallel 90-in. pipelines with only 1 ft clearance between them. Obviously it was impossible to adequately compact the soil in the triangular spaces beneath the spring lines.

Damage to pipes described under stage III is very severe and the inherent strength of the pipe is, for all practical purposes, completely dissipated. However, the development of very high passive soil pressures at the sides of the pipe holds it in a quasi-circular shape even though the pipe diameters are extensively changed. As a matter of fact, the writer has never seen or heard of a reinforced concrete pipe that has completely collapsed under a fill. Under the severest conditions, there is still a substantial area of waterway and hydraulic capacity remaining, and this should be preserved to the greatest extent possible.

Repairs can be made either by threading a thin-walled steel pipe through the damaged pipe and pressure grouting the annular space with concrete, or by guniting a 2- to 3-in. lining on the inside of the damaged pipe, as shown in Figures 24, 25 and 29. This gunite lining should be reinforced, either by a wire mesh cage or a spirally wound continuous-bar cage. An economic study should be made to determine the proper choice between the liner pipe and the gunite methods of repair. Both are effective and will restore the pipeline to a serviceable condition.