The Case Against the Ultimate Load Test for Reinforced Concrete Pipe

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•THE American Society for Testing and Materials and other specification writing agencies have, for many years, published standard specifications for the design and fabrication of reinforced concrete pipe for use in culvert and sewer construction and allied fields. From the very first ASTM tentative specification issued in 1930 to the current standard C 76-65 T, these specifications have included requirements relative to wall thickness, amount and disposition of reinforcing steel, quality of materials, manufacturing tolerances and the like. In addition, the structural quality of the manufactured product was specified in terms of required strengths which representative specimens of the pipe must meet when loaded in a laboratory crushing strength test. Up until the current standard, either of two types of prescribed strength tests were permitted—the Sand Bearing Test or the 3-Edge Bearing Test. By action of ASTM Committee C-13 in 1964, the Sand Bearing Test was eliminated from the specification.

Prior to issuance of the earliest strength specification, it was generally required that a reinforced concrete pipe meet both of two separate and distinct minimum test load criteria—the first crack strength and the ultimate strength. First crack strength was defined as the test load at which the first visible crack appeared in the pipe wall, usually a longitudinal crack on the inside of the pipe at the invert, though frequently at the crown and invert simultaneously. Ultimate strength was defined as the maximum or ultimate test load which the pipe could sustain.

The late Professor W. J. Schlick of Iowa State University had extensive experience in conducting tests of reinforced concrete pipe. He observed that some difficulty arose in determining the test load at "first crack." Light conditions in the laboratory, color and surface texture of the test specimen, and even the visual acuity of the observer, all entered into the decision as to when the first crack occurred. In order to provide for a more definite criterion for determining the test strength at an early stage of visible load effect, he suggested that a crack 0.01 in. wide be substituted for the first crack. This provided for a positive criterion which could actually be measured by means of a mechanic's leaf gage, thus eliminating most of the uncertainties associated with the first crack load requirement. His suggested modification was incorporated in the first ASTM tentative standard and has remained in the specification up to the present.

As stated above, the early specifications provided that the pipe must comply with both the 0.01-in. crack strength and the ultimate strength requirements. This provision was modified in 1957 to allow acceptance of the pipe on the basis of the 0.01-in. crack strength alone, or on the basis of both the 0.01-in. crack strength and the ultimate strength, at the option of the purchaser. This modification was made very largely in the light of experience accumulated in the Pacific Coast region, where the design of pipes and pipe installations is primarily based only on the 0.01-in. crack strength. Both strength requirements are still widely used in other regions of the country.

This history of the development of the ASTM standard specifications for reinforced concrete pipe is presented to show that, with the exception noted in the preceding paragraph, the ultimate load test has been a part of the strength requirements for this

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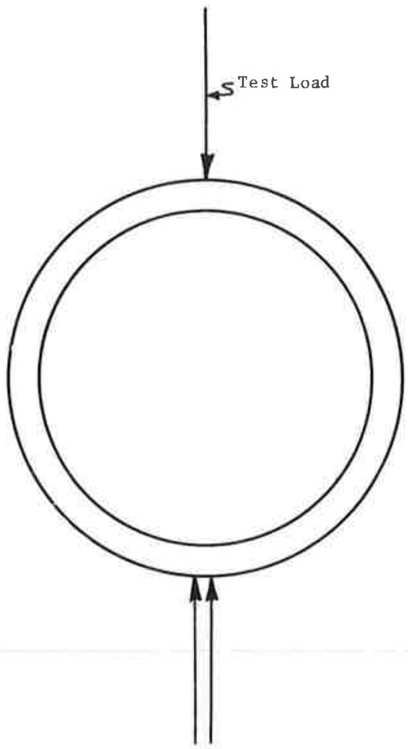

Figure 1. Load system, 3-edge bearing test.

product from the very beginning. The author believes that the test for ultimate strength has outlived its usefulness and recommends that it be eliminated from specifications. The purpose of this paper is to outline the procedure for designing a reinforced concrete pipe installation, and to point out the fact that the ultimate test strength does not serve a useful purpose in connection with that procedure. Nor is it indicative of the load-carrying capacity of a pipe when installed in the ground. Since it is necessarily a test to destruction, it is an expensive test and should be discontinued. This is especially true in the case of large-diameter pipes, which are being used more and more widely, both in highway and sewer construction.

The 3-edge bearing test is a very severe load test. The load system on the pipe specimen consists of the applied vertical load concentrated along a longitudinal element at the top, and an equal and opposite reaction concentrated along two closely spaced longitudinal elements at the bottom (Fig. 1). There are no lateral pressures applied to the pipe during the test. Bending moments in the pipe wall are relatively high because of the concentrated load; reaction and test strength values, both the 0.01-in. crack and the ultimate, are correspondingly low.

In contrast, when a pipe is installed in the ground, the system of loads acting on it is usually much more favorable. As a generalization, the earth load on top is distributed approximately uniformly over the horizontal width—the outside diameter—of the structure. The bottom reaction is distributed laterally over some fraction of the horizontal diameter, depending on the kind and quality of the bedding in which the pipe is installed. In addition, under favorable circumstances, active lateral earth pressures may act against the sides of the pipe. Lateral pressures tend to produce bending moments in the pipe wall which are in the opposite direction from those induced by vertical loads. Therefore every pound of lateral pressure which reliably can be brought to bear against the sides of a pipe increases its capacity to carry vertical load approximately one for one.

The strength design of a specific pipe installation follows the same classical pattern as that of any other type of structure. First it is necessary to determine the maximum load to which the pipe will be subjected during its functional life. Then the designer selects the materials and the type of installation environment which will insure that the pipe will adequately support this maximum load, with a reasonable factor of safety.

The load-carrying capacity of a reinforced concrete pipe in a field installation may be determined by multiplying its 3-edge laboratory test strength by an appropriate load factor which is defined as the ratio of the pipe supporting strength under any stated condition of loading to its 3-edge bearing strength. Load factors for various installations depend on the distribution of the load on top of the pipe, the distribution of the bottom reaction, and the magnitude and distribution of active lateral pressures on the sides of the pipe. Several field loading systems and corresponding load factors are illustrated in Figure 2.

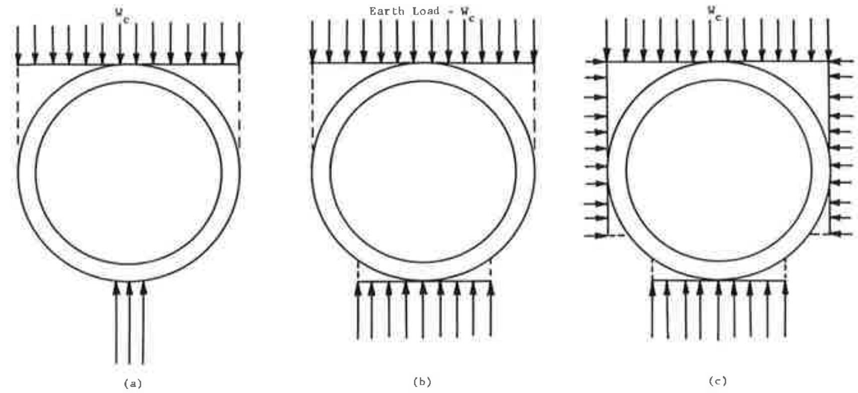

Figure 2. Load systems, various pipe installations: (a) impermissible bedding (Class D), load factor = 1.1; (b) ordinary bedding (Class C), load factor = 1.5; (c) ordinary bedding with active lateral pressure, load factor usually greater than 2.0 (See Eqs. 25-1 and 25-2, p. 424, Ref. 3).

When a reinforced concrete pipe is loaded by earth overburden in the field, the pipe deforms, i.e., the vertical diameter shortens and the horizontal diameter lengthens. The amount of this deformation in early stages of loading is very small because of the inherent rigidity of the pipe. As the load increases, the stress in the reinforcing steel increases, and since the modulus of elasticity of steel is much greater than that of concrete, the protective cover of concrete begins to show fine longitudinal cracks in the tensile zones on the inside of the pipe before the steel is stressed up to its capacity. Such cracks, in the opinion of the author, are not to be considered detrimental to the integrity of the pipe unless or until they approach a width which will permit or promote corrosion of the reinforcing steel. At the present time it is rather widespread practice to consider 0.01 in. as the limiting width of crack which can be tolerated, but there is need for extensive research to determine widths of cracks in the concrete which will effectively inhibit corrosion of the steel reinforcement under various environments.

As load on a pipe increases, further evidence of its effect may take the form of a separation of the protective cover of concrete from the body of the pipe wall. This separation occurs at the circumferential surface, which contains the inner layer of reinforcement, and in the tensile regions, which are at the top and bottom of the pipe. It is caused primarily by the fact that the inner cage of steel, being more flexible than the concrete wall in which it is embedded, tends to change shape more rapidly than the more rigid

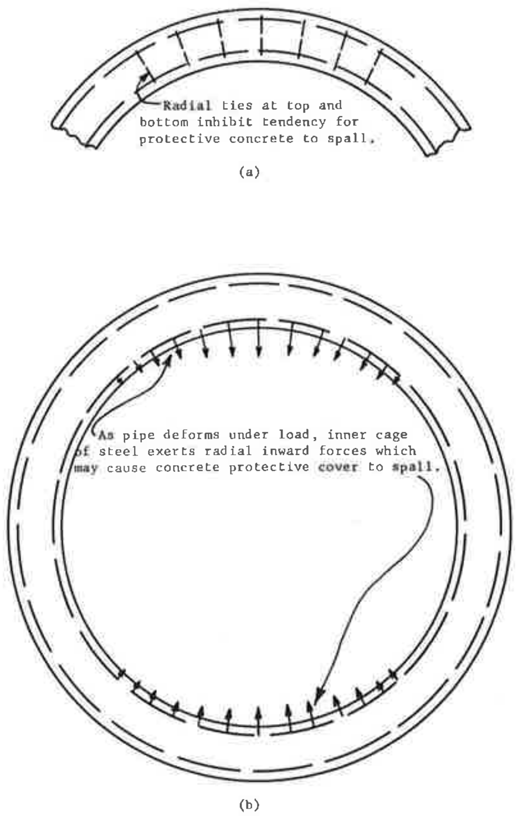

Figure 3.

Figure 4. Longitudinal crack about 1/16 to 1/8 in. wide; easily repaired by chipping and guniting after sufficient passive soil pressures have developed to establish equilibrium.

Figure 5. Concrete protective cover beginning to spall; can usually be repaired by chipping and guniting after sufficient passive soil pressures have developed to establish a state of equilibrium. Extreme cases may require pressure grouting to improve bedding conditions and lateral pressures.

concrete. The inner cage pulls downward at the top and upward at the bottom. This introduces tensile stresses in the concrete which are directed radially inward (Fig. 3b), and the protective cover may break loose. Further increase in load causes this concrete to shatter and "slab off," and the steel is laid bare, as shown in Figures 5 and 6. This type of action can be inhibited and the strength of the pipe increased by installing radial ties between the inner and outer cages of steel at the crown and invert of the pipe (Fig. 3a). The primary function of these radial ties (sometimes referred to as bridging, stirrups, or shear steel) is to hold the inner cage of reinforcement in place and prevent the development of radial tensile stress in the protective cover of concrete.

While the action described is progressing with further increase in load, the pipe loses rigidity and approaches the condition of a flexible or semirigid structure. The horizontal diameter increases under load to such an extent that the passive resistance pressure of the soil is mobilized in much the same manner as in the case of a flexible metal pipe. The primary source of supporting strength of the originally rigid structure gradually shifts from its inherent strength characteristics to dependence upon the passive resistance of the enveloping soil.

The more a pipe deforms the greater the magnitude of the mobilized passive pressure for a given soil, and it is impossible to define an ultimate pipe strength under field loading conditions in the same sense or which is comparable to the ultimate laboratory test strength, wherein no lateral pressures are applied during the test. A pipe under earth loading may have undergone gross deformation, but because of the passive soil pressures developed may still be capable of accepting additional load, and an

Figure 6. Advanced stage of spalling; note how reinforcement has pulled away from concrete wall. This 108-in. pipeline was repaired by pressure grouting through holes drilled at lower quarter points, after which new concrete protective cover was applied. Line has served 18 years since repairs were made and gives promise of a long satisfactory life.

"ultimate" load is practically never reached. Therefore it is impossible to apply a load factor to the ultimate test strength to obtain a field supporting strength, because the two strengths being considered are not comparable. They are radically different and attributable to different sources. Of course, long before the condition of gross deformation referred to is reached, the concrete protective cover will be badly shattered and the reinforcement pulled loose in the crown and invert (Figs. 5 and 6), which further complicates any attempt to define ultimate strength under field load conditions.

The author believes and recommends that the most appropriate and, in fact, the only rational approach to the design of a reinforced concrete pipe installation is to utilize the 0.01-in. crack test strength (or some similar visible and measurable indicator of early load effect), as a basis of selection of pipe strength and specified bedding and backfilling requirements. The ultimate 3-edge bearing test strength has no meaning in terms of field performance, and there is no basis for translating the results of the laboratory test into an ultimate strength under field conditions. Furthermore, since the test to ultimate requires destruction of the test specimen, and is therefore expensive to conduct, it is recommended that the ultimate strength requirement be deleted from specifications for reinforced concrete pipe. In contrast, the 0.01-in. crack strength test (or a similar width criterion) is nondestructive in character, and therefore many more pipe sections could be tested to this criterion for the same expenditure of laboratory funds. This would make laboratory testing more palatable to manufacturers and consumers alike. Many more specimens could be tested and, in the opinion of the author, this would tend to upgrade the whole process of design, manufacture and installation of reinforced concrete pipe structures.

DESIGN CALCULATION

An example of the design of a reinforced concrete pipe culvert installation based on the 0.01-in. crack 3-edge bearing strength of the pipe is presented (see ch. 24 and 25 of Ref. 3).

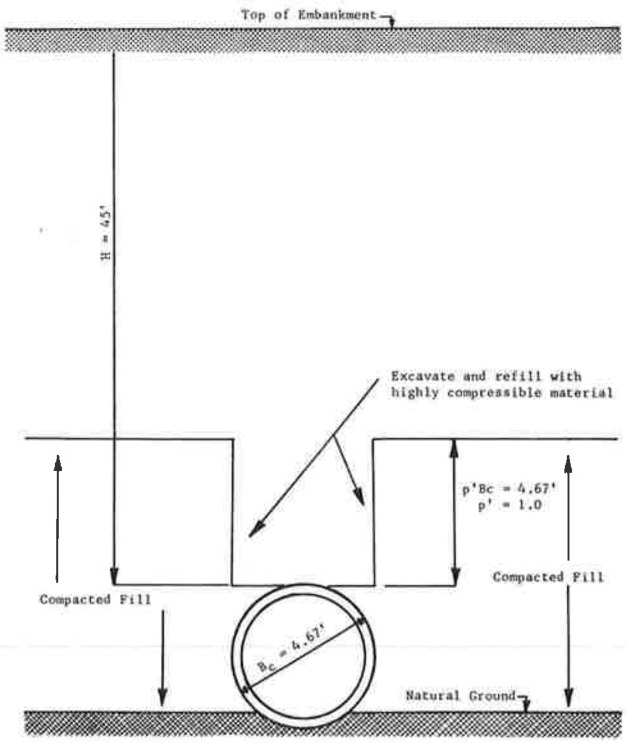

Figure 7. Example culvert, imperfect ditch installation.

Load calculation

Assume $w = 120$ pcf, $K\mu = 0.13$

$p' = 1.0, \; r_{sd} = -0.3$

$B_c = 4.67, \; \dfrac{H}{B_c} = \dfrac{45}{4.67} = 9.6$

$C_n = 5.9$

$W_c = 5.9 \times 120 \times (4.67)^2 = 15,400$ plf

Strength calculation

Assume $m = 0.7, \; x = 0.594, \; K = 0.33$

Class C bedding, $N = 0.840$

$$q = \frac{0.7 \times 0.33}{5.9} (9.64 + 0.35) = 0.390$$

$$L_f = \frac{1.431}{0.840 - (0.594 \times 0.390)} = 2.35$$

Required 0.01-in. 3-edge strength $= \dfrac{15,400}{2.35} = 6600$ plf

Required D-Load $= \dfrac{6600}{4} = 1650$ D

Use Class IV pipe (2000 D at 0.01-in. crack)

Factor of safety based on 0.01-in. crack $= \dfrac{2000}{1650} = 1.2$

The factor of safety based on the 0.01-in. crack strength of the pipe in the above example is 1.2. Since an appropriate factor of safety cannot be determined rationally by principles of mechanics, it remains purely a matter of judgment based on experience and observation. It is the author's opinion that a minimum factor of safety of 1.0 is both adequate and economical for reinforced concrete pipelines designed on the basis of the 0.01-in. crack strength. Reasons for this opinion are (a) the failure of this type of structure does not involve the safety of human life; (b) reinforced concrete pipes have a large reservoir of load-carrying capacity beyond the 0.01-in. crack stage, due to inherent strength and the strength imparted by passive soil pressures as the pipe deforms; and (c) a pipe in the ground does not fail suddenly or collapse completely, so there is adequate time and opportunity for making repairs in case of accidental overloading.

The ASCE Manual of Practice No. 37, "Design and Construction of Sanitary and Storm Sewers" (WPCF Manual No. 9), recommends the use of a factor of safety of 1.5 based on the ultimate test strength of the pipe. It is pointed out that this value gives exactly the same result as the value of 1.0 based on the 0.01-in. crack strength in the case of ASTM Classes I, II, III, and IV pipe, since the required ultimate strength for these classes is 1.5 times the crack strength. For Class V pipe the required test strengths are 3750 D and 3000 D respectively. Therefore, a factor of safety of 1.5 based on ultimate is the equivalent of 1.2 based on 0.01-in. crack strength. However, since the ultimate test strength of a reinforced concrete pipe has no equivalent or comparable counterpart when the pipe is installed in the ground, factors of safety based on ultimate test strength have no numerical meaning.

REPAIR METHODS

A matter of collateral interest in connection with reinforced concrete pipes which have developed structural difficulty (Figs. 4, 5, and 6) has to do with methods of repair. Some engineers and contractors have resorted to threading a metal pipe liner

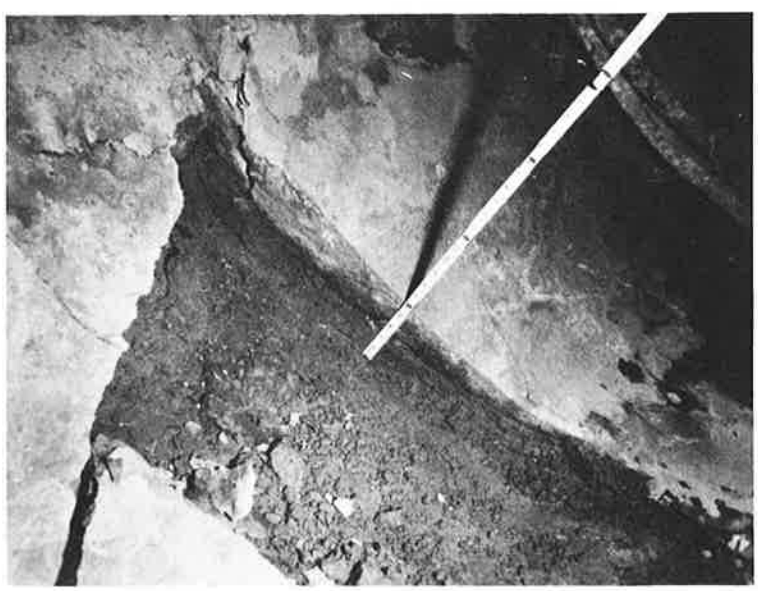

Figure 8. An 84-in. pipeline repaired by pressure grouting; several pipe sections were removed to observe grout distribution. Line has served 16 years since repairs were made and is in good condition.

of smaller diameter through the distressed pipe and grouting the annular space between the liner and the concrete. This procedure is expensive and adds the further disadvantage of reducing the hydraulic capacity of the pipeline.

Another procedure which may often prove to be effective and economical is to take advantage of passive soil pressures at the sides of the pipe which develop in response to horizontal movement against the soil as the pipe deforms, as described earlier. If the damage to the pipe is not too extensive, it may deform to a state of equilibrium wherein passive pressures build up sufficiently to prevent further deformation. This state can be determined by measuring the horizontal and vertical diameters at a number of places, marking the points between which the measurements are made, then repeating such measurements at weekly or monthly intervals. When a state of equilibrium is indicated, cracks can be reamed out with an air chisel, damaged concrete removed, and the areas patched with gunite or a suitable epoxy cement. Patches of this kind will not add to the strength of the pipe, but will protect the steel from corrosion. Two types of damage for which this procedure may be appropriate are shown in Figures 4 and 5.

Pipes which are more extensively damaged may be strengthened by drilling holes through the pipe walls at approximately the lower quarter points and injecting grout under pressure between the pipe and the bedding and backfill soil. This effectively increases lateral pressure and improves the bedding to such an extent that the supporting strength of the pipe is made adequate to carry the vertical load without further deformation. After this operation the loose and shattered concrete may be removed and a protective cover applied over the steel, as in the preceding paragraph. This method of repair was employed in the case of the damaged 108-in. pipeline shown in Figure 6. The repaired structure has served satisfactorily for nearly 18 years since repairs were made, and gives promise of much longer service. Figure 8 shows an 84-in. pipeline in which the bedding was improved by pressure grouting, after which several pipe sections were removed for observation of the distribution of grout. The balance of the pipeline was repaired by grouting and guniting and has served satisfactorily for about 16 years.

REFERENCES

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