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A STUDY OF
THE EXTERNAL LOADS APPLIED ON BURIED CONDUITS

by

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NOTATION

B_c = horizontal breadth (outside) of conduit in feet.

B_d = horizontal width of ditch at top of conduit in feet.

C_c = load coefficient for projecting conduits.

C_d = load coefficient for ditch conduits.

C_t = load coefficient.

d_c = deflection of the vertical height of the conduit.

D_l = deflection lag factor.

E = modulus of elasticity of pipe metal in psi.

E' = er = modulus of soil reaction in psi.

e = modulus of passive resistance of the enveloping soil, in pounds per square inch per inch.

F_i = impact factor.

h = distance from ground surface down to any horizontal plane in backfill in feet.

H = height of fill above top of conduit in feet.

H_e = height of plane of equal settlement in feet.

I = moment of inertia per unit length of cross-section of the pipe wall in inches^4 per inch.

k = a bedding constant, its value depending on the bedding angle.

K = Rankine's ratio, ratio of active lateral unit pressure to vertical unit pressure;

$$K = \frac{\sqrt{\mu^2 + 1} - \mu}{\sqrt{\mu^2 + 1} + \mu}$$

L = length of conduit section on which load is computed in feet.

P_o = concentrated wheel load on surface of fill in pounds.

r = mean radius of the pipe in inches.

r_{sd} = settlement ratio.

S_f = settlement of the conduit into its foundation.

S_g = settlement of the natural ground surface adjacent to the conduit.

S_m = compression strain of the side columns of soil of height pB_c.

V = vertical pressure on any horizontal plane in backfill in pounds per linear foot of ditch.

w = unit weight of filling material in pounds per cubic foot.

W_c = load on conduit in pounds per linear foot.

W_p = average load per unit length of conduit, due to wheel load in pounds per foot.

e = base of natural logarithms = 2.718218.

μ = coefficient of internal friction of fill material.

μ' = coefficient of friction between fill material and sides of ditch.

INTRODUCTION

The theory of external loads on buried conduit was suggested by Anson Marston (1) in 1913. The term conduit includes culverts, drains, sewers, aqueducts, water pipes, gas mains, telephone conduits, and underground steam mains. Conduits of this general type have been used by mankind for at least 3,000 years, but not until Marston started his work about 1910 was serious effort directed toward the development of a rational method for determining the magnitude and character of the loads to which underground conduits were subjected in service due to the soil overburden and loads superimposed on the surface. The research (2)(3)(4) of Marston and Spangler resulted in papers that developed a complete mathematical theory of external load on closed conduits and of the supporting strengths of pipe conduits. These theories applied to all types of closed conduits and to all classes of field conditions.

The load of the conduit is a function of the depth of burial, shape, and rigidity of the pipe, the soil, and the method and efficiency of construction. The fill load is usually assumed to be uniformly distributed over the width of the conduit and along the length under consideration. Its value depends on the saturated density, the frictional characteristics of the fill, the depth of cover over the top of the conduit, the relative density of the trench wall and fill, and the effective width of the trench. The effective width depends on the method of installation which affects the direction of the frictional forces acting on the shear planes within the fill and on the rigid or flexible conduit.

PURPOSE OF THE STUDY

The purpose of this report is to review Marston's theories of external loads on closed conduits and all the available literature pertinent to this type of structure published since Marston's work. From this the physical conditions of the conduits and the forces to which they are subjected will be analyzed, several kinds of bedding conditions of buried conduit will be shown, and the relation of the external load to the bedding conditions will be shown. The supporting strength of conduits is relatively important in the design of conduits and this will be analyzed in this report. Typical examples of analysis from given data will be shown.

REVIEW OF LITERATURE

From the standpoint of rigidity, closed conduits were classified by Marston (4) as rigid, semi-rigid, or flexible.

Rigid conduits: The cross sectional shapes cannot be distorted sufficiently to change the vertical or horizontal dimensions more than 0.1 per cent without causing materially injurious cracks.

Semi-rigid conduits: The cross sectional shapes can be distorted sufficiently to change the vertical or horizontal dimensions more than 0.1 per cent, but not more than 3.0 per cent without causing materially injurious cracks.

Flexible conduits: The cross sectional shapes can be distorted sufficiently to change the vertical or horizontal dimensions more than 3.0 per cent before causing materially injurious cracks.

On the basis of construction conditions, conduits may be classified by Marston (4) as ditch conduits or projecting conduits.

Ditch conduits: Conduits installed in relatively narrow ditches dug in undisturbed soil and then covered with earth backfill.

Projecting conduits: Conduits installed in shallow earth bedding with the top of the conduit projecting above the surface of the natural ground, and then covered with an embankment.

For the purpose of calculating the external vertical loads on projecting conduits, the field conditions affecting the loads are conveniently grouped into four sub-classifications based on (1) the magnitude of the settlement of the interior prism relative to the exterior prism, and therefore, the direction of the shearing stresses, and (2) the height of the embankment in relation to the height of equal settlement. These four conditions are classified by Spangler (5) as follows.

Complete projection condition: The top of the conduit settles less than the critical plane and the height of the embankment is less than the theoretical height of equal settlement.

Incomplete projection condition: The top of the conduit settles less than the critical plane and the height of the embankment is greater than the height of equal settlement.

Complete ditch condition: The top of the conduit settles more than the critical plane and the height of the embankment is less than the height of equal settlement.

Incomplete ditch condition: The top of the conduit settles more than the critical plane and the height of the embankment is more than the height of equal settlement.

The more important external loads on closed conduits, according to Marston (1), result from the weight of the soil above the pipe. The load at any point on the exterior surface is sufficiently expressed by its vertical and horizontal components acting perpendicularly to the cross sectional plane at the point. The vertical components of such loads are usually much greater and more important than the horizontal components, and the latter are most conveniently expressed in terms of their ratios to the vertical components.

Hence, the mathematical formulas for computing external loads on closed conduits is most conveniently expressed in the vertical load components only, leaving the horizontal pressures to be calculated from their ratio to the vertical loads.

The external loads on closed conduits comprise the downward loads applied usually to the upper portions of the conduit exteriors and the resultant upward soil foundation pressures, applied usually to the lower portions of the conduit exteriors. The total foundation pressure upward must be equal to the total load downward for static equilibrium.

The external loads on closed conduits are of two classes. First is the load due to the fill materials placed over and around the conduits. Second is the load transmitted through the fill materials due to extraneous superimposed loads, applied at the upper surface of the fill.

The mathematical theory of loads on closed conduits due to fill materials was developed by Spangler (5)(6) as outlined below.

Load on ditch conduits

When a conduit is placed in a ditch not wider than two or three times its outside diameter and covered with earth, the backfill material has a tendency to compact and settle downward. This downward movement of the soil in the ditch above the pipe produces vertical frictional forces or shearing stresses along the sides of the ditch which act upward on the prism of soil within the ditch and thus partially support the backfill material. The difference between the weight of the backfill and these upward shearing stresses is the load which must be supported by the conduit at the bottom of the ditch. Assuming the cohesion between the backfill material and the sides of the ditch is negligible, the magnitude of the vertical shearing stresses is equal to the active lateral pressure exerted by the soil backfill against the sides of the ditch multiplied by the tangent of the angle of friction between the two materials.

This assumption of negligible cohesion by Spangler (7) is justified because: (1) A ditch filled with cohesive material cannot develop cohesion between backfill and the sides of the ditch for a long period of time; and (2) The assumption of no cohesion yields the maximum probable load on the conduit. This maximum load may develop at any time during the life of the conduit as a result of heavy rainfall or some other action which may eliminate or greatly reduce cohesion between the backfill and the sides of the ditch.

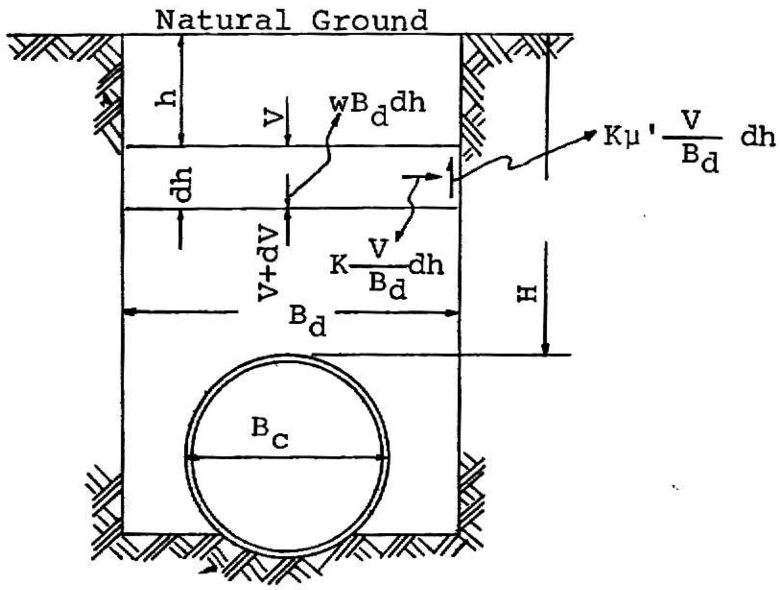


Fig. 1. Free-body Diagram for Ditch Conduit

Let Fig. 1 represent a cross section of a unit length of ditch conduit and consider a horizontal element of the fill material of height dh at a distance h below the ground surface.

V = the vertical pressure on the top of the element.

$V + dV$ = the vertical pressure on the bottom of the element.

$wB_d dh$ = the weight of the element.

$K\frac{V}{B_d}dh$ = the lateral pressure on each side of the element.

Since the element has a tendency to move downward in relation to the sides of the ditch, the lateral pressures include upward shearing forces equal to $K\mu'\frac{V}{B_d}dh$. It may be equated as follows:

$$V + dV + 2K\mu'\frac{V}{B_d}dh = V + wB_d dh \quad - - - - - - - - - - - - (1)$$

The solution of this differential equation is:

$$V = wB_d^{\,2}\ \frac{1 - e^{-2K\mu' \frac{h}{B_d}}}{2K\mu'} \qquad (2)$$

The portion of this total pressure which is carried by the conduit depends on the rigidity of the conduit in comparison with that of the fill material between the sides of the conduit and the sides of the ditch. In the case of a very rigid pipe, such as a burned clay, concrete, or heavy-walled cast iron pipe, the side fills may be relatively compressible and the superimposed load can safely be carried by the conduit. The pipe that is relatively flexible and thin-walled must have soil that is thoroughly tamped at the sides of the pipe, so the stiffness of the side fills support the sides of the conduit and the load on the pipe will be transmitted to the side fills that must be capable of carrying the transmitted pressure.

For the case of rigid ditch conduits with relatively compressible side fills, the load will be:

$$W = C_d w B_d^{\,2} \qquad (3a)$$

For the case of flexible pipe and thoroughly compacted side fills having the same degree of stiffness as the pipes, the load will be:

$$W = C_d w B_c B_d \qquad (3b)$$

in which for both equations:

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K} \quad \text{- - - - - - - - - - - - - - -} \quad (4)$$

Evaluation of these formulas may be simplified by the use of the computation diagram in Fig. 2, in which values of C_d for various values of $\frac{H}{B_d}$ have been plotted for several kinds of fill materials having different coefficients of internal friction.

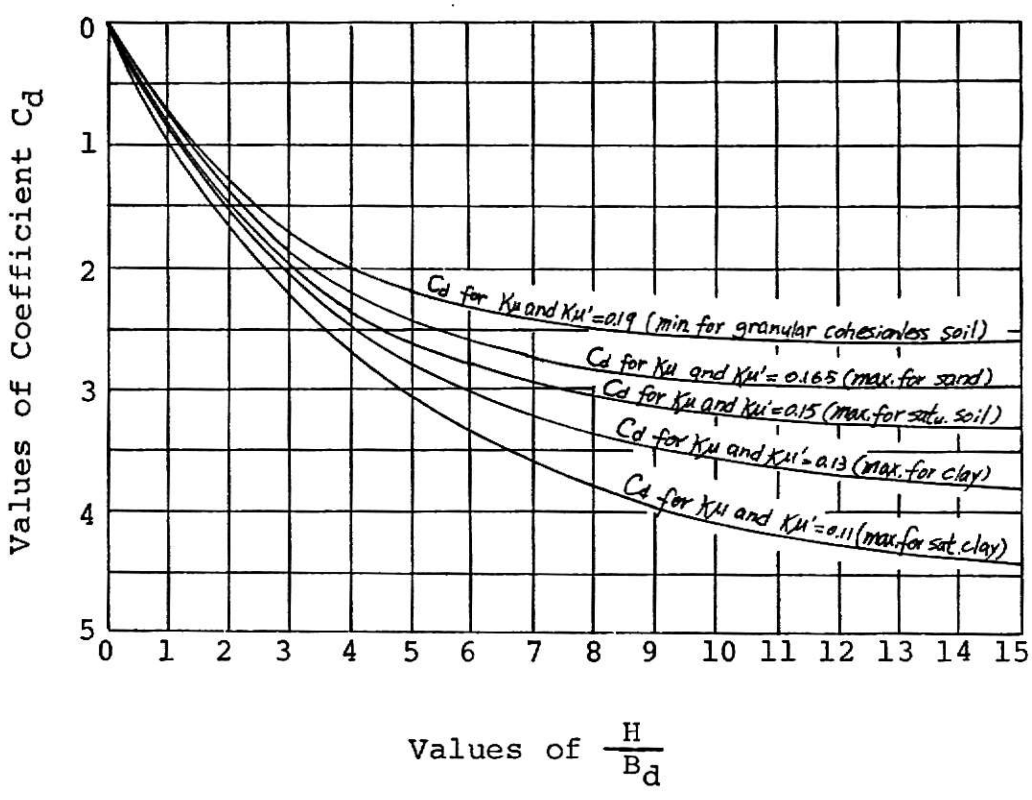


Fig. 2. Diagram for Coefficient C_d for Ditch Conduits

The width of ditch, B_d, is the actual width of the parallel sided ditch. In case the ditch is constructed with sloping sides, the width of ditch, studied by Schick (8), at or slightly below the top of the pipe is the proper width to use in the load formula.

The theoretical loads are, therefore, safe working loads which should be used in the design of ditch conduits to prevent cracking of the pipe. Long unusually wet periods may so weaken the soil that it will no longer support the sides of the pipe. The soil at the sides of the pipe may be dangerously softened or even washed away by water forced through the cracks of the pipe, especially if the pipe is forced at times to operate under head.

Loads on projecting conduits

It is evident that the height of the interior prism, shown in Fig. 3, will be less than that of the exterior prisms by the amount which the conduit projects above the natural ground. This is in accordance with the well-known phenomenon that a high prism of soil will settle more than a lower prism of the same soil at equal density. There is a tendency for the exterior prisms to settle more than the interior prism and for friction forces or shearing stresses to be exerted along the vertical planes bounding the interior prism. These shearing stresses will be equal to the active lateral pressure at these planes, multiplied by the coefficient of internal friction of the fill material. It is known that definite shearing planes between the interior and exterior prisms of soil do not actually exist in an earth embankment. The shearing stresses are transferred from one prism to another through more or less narrow zones of the fill material. Nevertheless, the assumption of actual vertical shearing planes is employed for convenience in developing the theory.

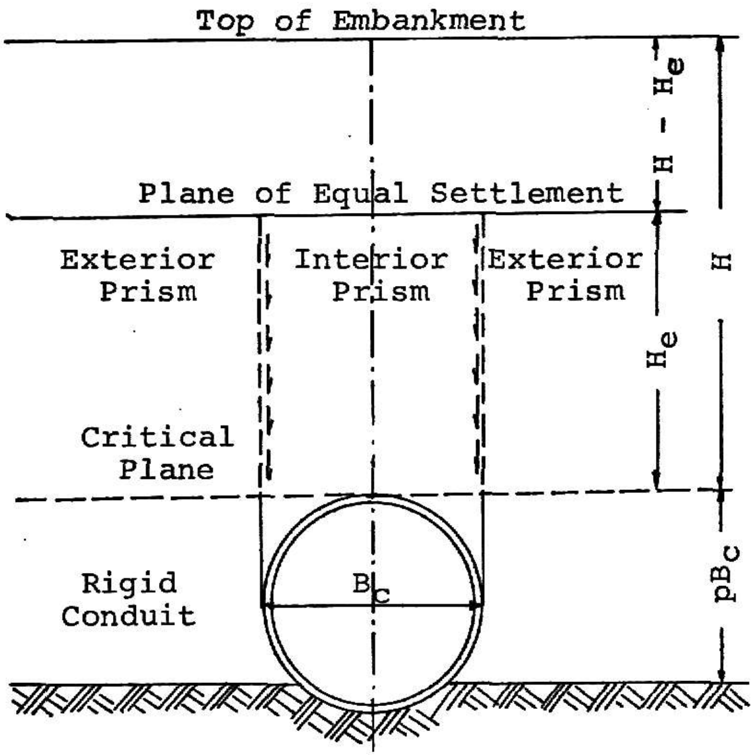


Fig. 3. Projecting Conduit

The plane of equal settlement is defined, by Spangler (5), as the horizontal plane in the embankment at and above which the settlements of the interior and exterior prisms of soil are equal. The fact of a plane of equal settlement in this basic case is brought about by the transfer of pressure, by shear, from the exterior prisms to the interior prism. The vertical deformation of the prism of soil material is due to its own weight which is a function of its height and the unit weight of the material. Normally, the summation of deformations from the bottom of a prism upward will be at a greater rate in a high prism than in a lower one if they act independently of one another. In the case of projecting conduits, however, the exterior prisms of soil transfer a part of vertical pressures to the interior prism. As a result, because of this stress transfer, the rate of summation of vertical deformations will be reduced in the exterior prisms and increased in the interior prism. Therefore, the total summation of deformation in the interior prism will approach that in the exterior prisms, and the height at which they become equal is the height of equal settlement. The existence of shearing stresses below a plane of equal settlement was studied in one of Marston's early experiments.

A critical plane is defined as the horizontal plane tangent to the top of the conduit after installation. When the interior prism settles more than the exterior prism, the shearing resistance between the exterior and interior prisms greatly reduces the pressure on the pipe and when the reverse is true, the pressure on the pipe is greatly increased. The resultant load is less than the weight of the soil above the structure when the shearing forces act upward. This is called the ditch condition.

In the mathematical analysis of loads on projecting conduits, the net effect of settlement factors, both as to magnitude and direction of the relative movements of the three prisms of soil, is combined into an abstract ratio known as the "settlement ratio". A settlement ratio is defined, by Spangler (5), as the ratio of the difference between the settlement of the critical plane and the top of the conduit to the settlement of the fill material and the top of the conduit.

$$r_{sd} = \frac{(S_m + S_g) - (S_f + d_c)}{S_m} \quad \text{- - - - - - - - -} \quad (5)$$

Conditions	Settlement ratio
Rigid culvert on foundation of rock or unyielding soil.	+1.0
Rigid culvert on foundation of ordinary soil.	+0.5 to +0.8
Rigid culvert on foundation of material that yields with respect to adjacent natural ground.	0 to +0.5
Flexible culvert with poorly compact side fills.	-0.4 to 0
Flexible culvert with well-compacted side fills.	-0.2 to +0.8

Table 1. Design values of settlement ratio

It is more practical to consider the settlement ratio as an empirical quantity and to determine working values for design purposes from observations of the performance of actual culverts under embankments. These values, based on observation, are shown in Table 1 (7).

Considering first the complete ditch and projection conditions, Fig. 4, in which $H < H_e$ (H_e is imaginary in this case), the vertical forces on any thin horizontal element of the interior prism may be equated (5) as follows:

$$V + dV = V + wB_c dh \pm 2K\mu \frac{V}{B_c} dh \qquad (6)$$

the solution of this differential equation is:

$$V = wB_c{}^2 \frac{e^{\pm 2K\mu \frac{h}{B_c}} - 1}{\pm 2K\mu} \qquad (7)$$

At the top of the conduit, $V = W_c$, $h = H$; therefore,

$$W_c = C_c w B_c{}^2 \qquad (8)$$

in which

$$C_c = \frac{e^{\pm 2K\mu \frac{H}{B_c}} - 1}{\pm 2K\mu} \qquad (9)$$

In this formula the plus sign is used for the complete projection condition and the minus sign for the complete ditch condition.

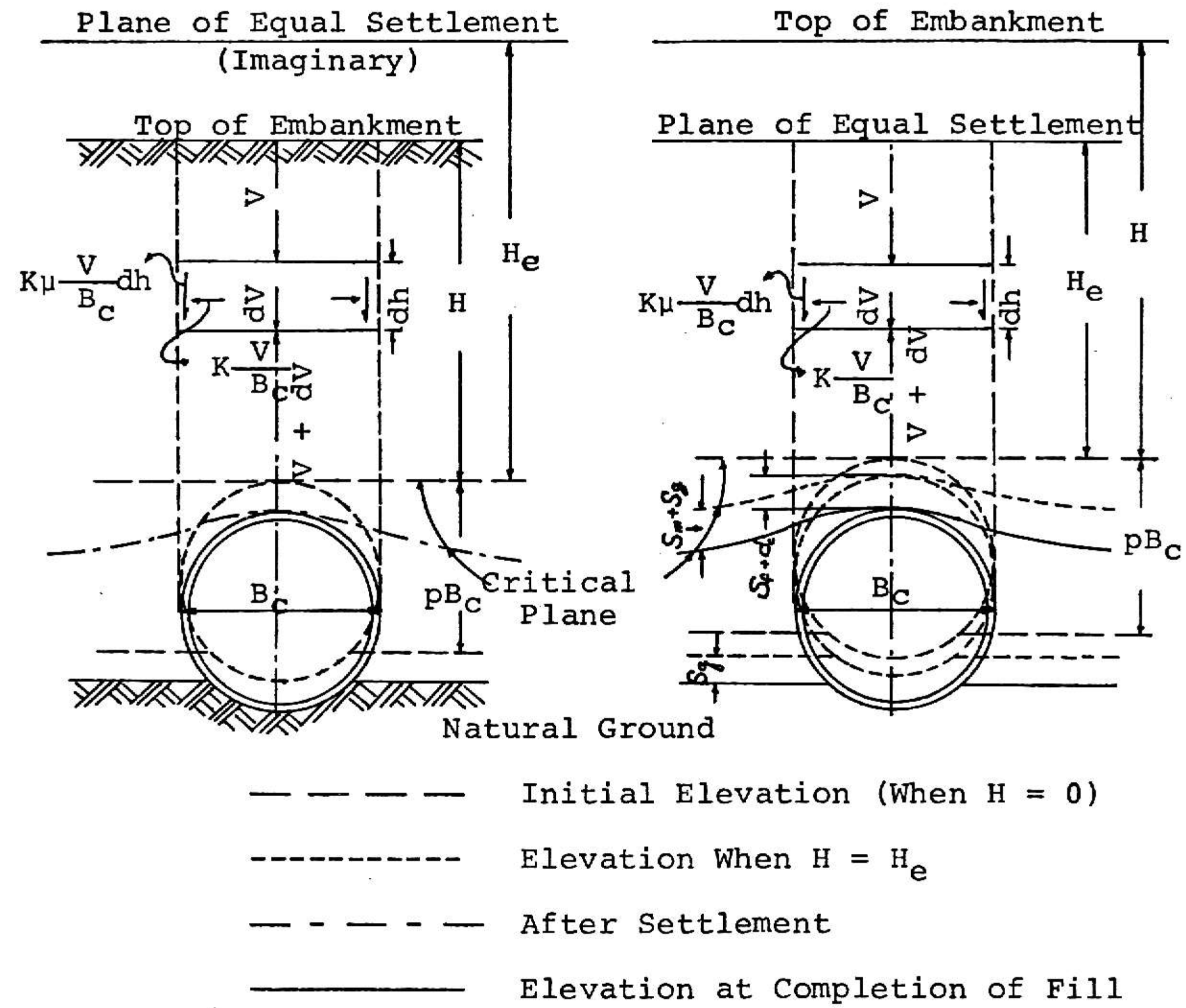


Fig. 4. Complete Projection Condition

Fig. 5. Incomplete Projection Condition

Considering the incomplete ditch and incomplete projection conditions (Fig. 5), in which $H > H_e$, and again equating the vertical forces on a thin horizontal element, we obtain:

$$V + dV = V + wB_c dh \pm 2K\mu \frac{V}{B_c} dh \quad (10)$$

When $h = 0$, $V = (H - H_e)wB_c$, and the solution of this differential equation is

$$V = \frac{wB_c^2}{2K\mu} \pm \frac{wB_c^2}{2K\mu} e^{\pm 2K\mu \frac{h}{B_c}} + (H - H_e)wB_c e^{\pm 2K\mu \frac{h}{B_c}} \quad (11)$$

At the top of the conduit, $V = W_c$, and $h = H_e$, and we have

$$W_c = C_c w B_c^2 \text{ - } (12)$$

in which

$$C_c = \frac{e^{\pm 2K\mu \frac{H_e}{B_c}} - 1}{\pm 2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c}\right) e^{\pm 2K\mu \frac{H_e}{B_c}} \text{ - - - - } (13)$$

As before, the plus signs are applicable to the incomplete projection condition and the minus signs apply to the incomplete ditch condition.

As in the case of ditch conduits, the solution of these various expressions for loads on projecting conduits is made easier by the construction of a computation diagram (4)(9), as shown in Fig. 6, from which values of the load coefficient C_c can be obtained for substitution in Equations (9) and (13). It will be noted that C_c is a function of the ratio of the height of fill to the width of the conduit, $\frac{H}{B_c}$, and of the product of the settlement ratio and the projection ratio, $r_{sd}p$, as well as of the friction characteristic of the soil.

However, the influence of the coefficient of internal friction, μ, is relatively minor in this case, and it is not considered. Therefore, in Fig. 6, it was assumed that Kμ = 0.19 for the projection condition, and that Kμ = 0.13 for ditch condition.

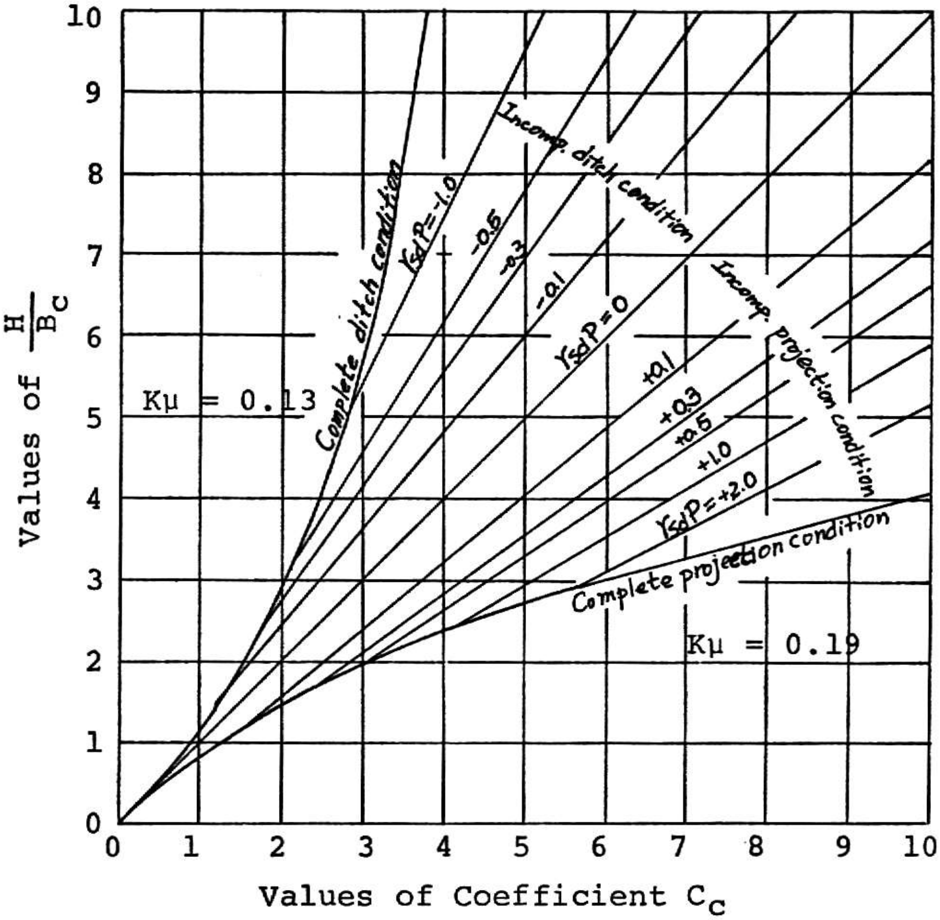


Fig. 6. Diagram for Coefficient C_c for Projecting Conduits

<u>The mathematical theory of loads on closed conduits due to super-load</u>

In addition to external loads imposed by the filling material around and above underground conduits, these structures are also subjected to loads resulting from highway, railway, or airplane traffic or from other types of loads applied at the surface and transmitted through the soil to the underground structure. Such loads are of major importance when a conduit is placed under a trafficway with a relatively shallow covering of earth.

John H. Griffith, (10), was among the first to suggest the applicability of the Boussinesq solution for the distribution of stress in a semi-infinite elastic solid to various problems of stress distribution in soils. Subsequent experiments on both ditch and projecting conduits have shown that a concentrated surface load, such as a truck wheel, is transmitted through the soil covering to the underground structure substantially in accordance with the Boussinesq solution.

From these facts, the load on an underground conduit due to a concentrated surface load may be expressed by Spangler (5)(11) as:

$$W_p = \frac{1}{L} F_i C_t P_o \text{ - - - - - - - - - - - - - - - - } (14)$$

Values of C_t for conduit 3 ft. long of various widths, B_c, are shown in Fig. 7 for various heights of fill up to 10 ft. When fill loads and surface loads are combined to obtain the design load on an underground conduit, the minimum load will be found to occur when the height of fill is relatively thin over the top of the structure.

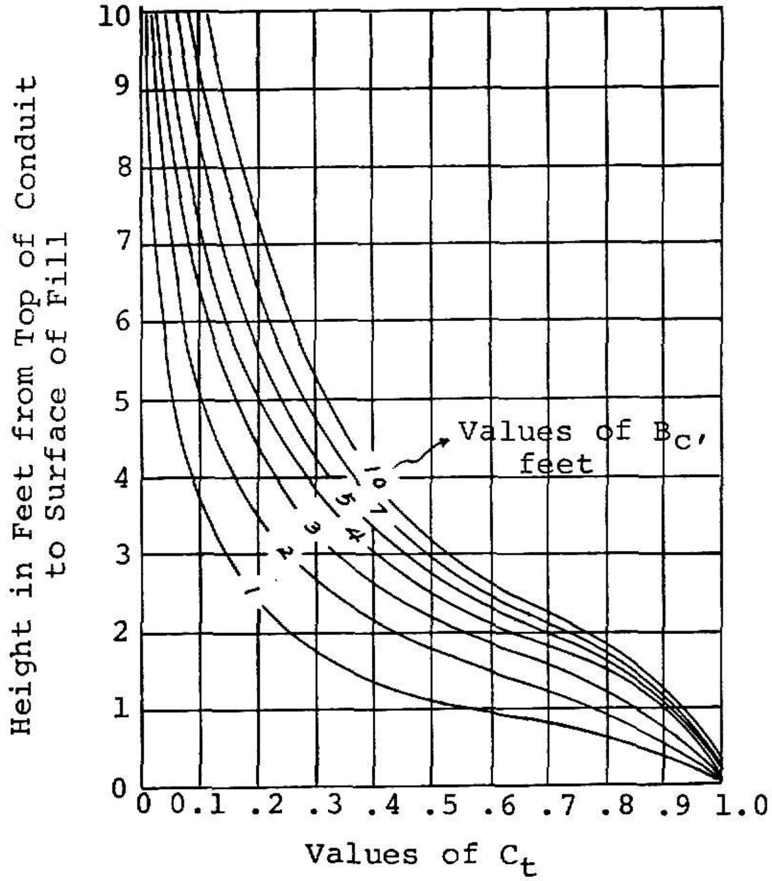


Fig. 7. Concentrated Surface Load Coefficients for Underground (L = 3 Ft.)

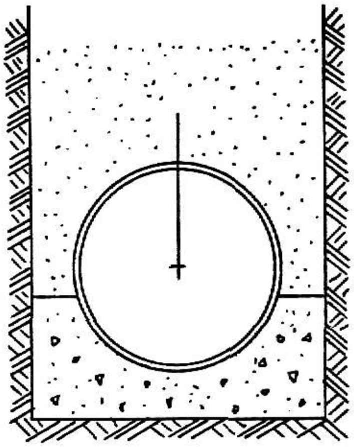
The impact factor, F_i, is equal to unity when the surface load is static. When it is moving, as in the case of truck or airplane wheels, the value of F_i may vary widely depending on the speed of the vehicle, vibratory action, wing uplift, and most importantly, the roughness characteristics of the roadway surface. The design values of F_i range from 1.5 to 2.0 for highway traffic on an unsurfaced roadway.

<u>The bedding conditions of buried conduits</u>

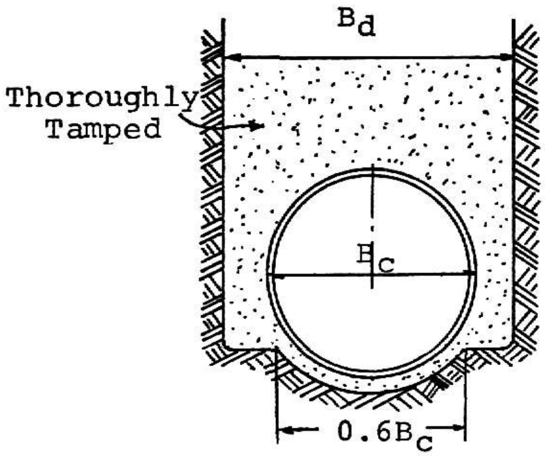
A wide variety of bedding conditions affecting the distribution of the bottom reaction and of backfilling conditions affecting lateral pressure on a pipe may be encountered in practice. The distribution of the bottom reaction depends upon the quality of the pipe bedding. For instance, if a pipe rests on a flat concrete floor, the reaction is concentrated along a line and stresses in the pipe wall are very high, just as the stresses in a beam are greatly increased if the load is concentrated at the center instead of being distributed over the span length of the beam. The definite supporting strength which sewer pipe will develop in actual ditches with different pipe-laying methods is essential to the rational design of pipe conduits. The bedding classes were developed a number of years ago, when hand labor was extensively employed in the installation of conduits in trenches. More recently, practicing engineers have developed methods of bedding pipe conduits which require the use of selected granular materials, but much less hand labor.

Bedding conditions for ditch conditions

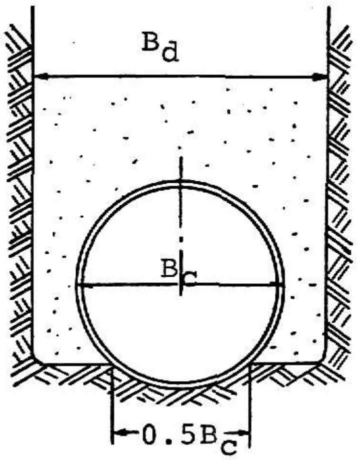
The following bedding classifications have been defined by Marston (2) and are illustrated in Fig. 8.



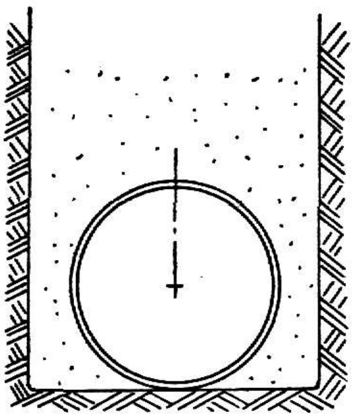
(a) Concrete Cradle Bedding



(b) First Class Bedding



(c) Ordinary Bedding



(d) Impermissible Bedding

Fig. 8. Ditch Conduit Bedding

(a) Concrete cradle bedding. This is a method of bedding a ditch conduit in which the lower part of the conduit is bedded in a cradle constructed of plain or reinforced concrete of suitable thickness under the lowest part of the conduit and extending upward on each side of the conduit for a half of its height. This cradle must be poured as a unit without horizontal construction joints.

(b) First class bedding. This is a method of bedding a ditch conduit in which the pipe is carefully bedded on fine granular materials in an earth foundation carefully shaped to fit the lower part of the conduit exterior for a width of at least 60% of the conduit breadth. The remainder of the conduit is entirely surrounded to a height of at least 1.0 ft. above its top by granular materials that are carefully placed to completely fill all spaces under and adjacent to the pipe and that are thoroughly tamped on each side and under the pipe as far as practical in layers not exceeding 0.5 ft. in thickness.

(c) Ordinary bedding. This is a method of bedding a ditch conduit in which the pipe is bedded with "ordinary" care in an earth foundation shaped to fit the lower part of the pipe with reasonable closeness for a width of at least 50 per cent of its outside breadth, and in which the remainder of the pipe is surrounded to a height of at least 0.5 ft. above its top by granular materials that are shovel-placed and shovel-tamped to completely fill all spaces under and adjacent to the pipe.

(d) Impermissible bedding (bedding that should not be used in any case). This is a method of bedding a ditch conduit in which little or no care is exercised to shape the foundation to fit the lower part of the conduit or to fill all spaces under and around the conduit with granular materials.

Bedding conditions for projecting conduits

As in the case of ditch conduits, it is convenient to name and define several classes of bedding conditions for projecting conduits. These are illustrated in Fig. 9 and defined (2)(7) below.

(a) Concrete cradle projection bedding. This is a method of bedding a positive projecting conduit in which the lower part of the conduit is bedded in a cradle constructed of a minimum 2,000 psi compressive strength concrete having a minimum thickness under the pipe of one-fourth its inside diameter and extending up the sides of the pipe for a height equal to one-fourth its outside diameter.

(b) First class projection bedding. This is a method of bedding a positive projecting conduit, having a projection ratio not greater than 0.7, in which the conduits carefully bedded on fine granular materials in an earth foundation that is carefully shaped to fit the lower part of the pipe for at least 10 per cent of its over-all height and in which earth-filling material is thoroughly rammed and tamped, in layers not exceeding 0.5 ft. in depth, around the pipe for the remainder of the lower 30 per cent of its height.

(c) Ordinary projection bedding. This is a method of bedding a positive projecting conduit in which the conduit is bedded with "ordinary" care in an earth foundation shaped to fit the pipe with reasonable closeness for at least 10 per cent of its over-all height and in which the remainder of the pipe is surrounded by granular materials that are shovel-placed to completely fill all spaces under and adjacent to the conduit.

(d) Impermissible projection bedding. This is a method of bedding a positive projecting conduit in which little or no care is exercised to shape the foundation to fit the lower part of the pipe or to fill all spaces under and around the pipe with fill materials. This type of bedding also includes the case of a conduit on a rock foundation in which an earth cushion is provided under the conduit but which is so shallow that the conduit, as it settles under the influcnce of vertical load, approaches contact with the rock.

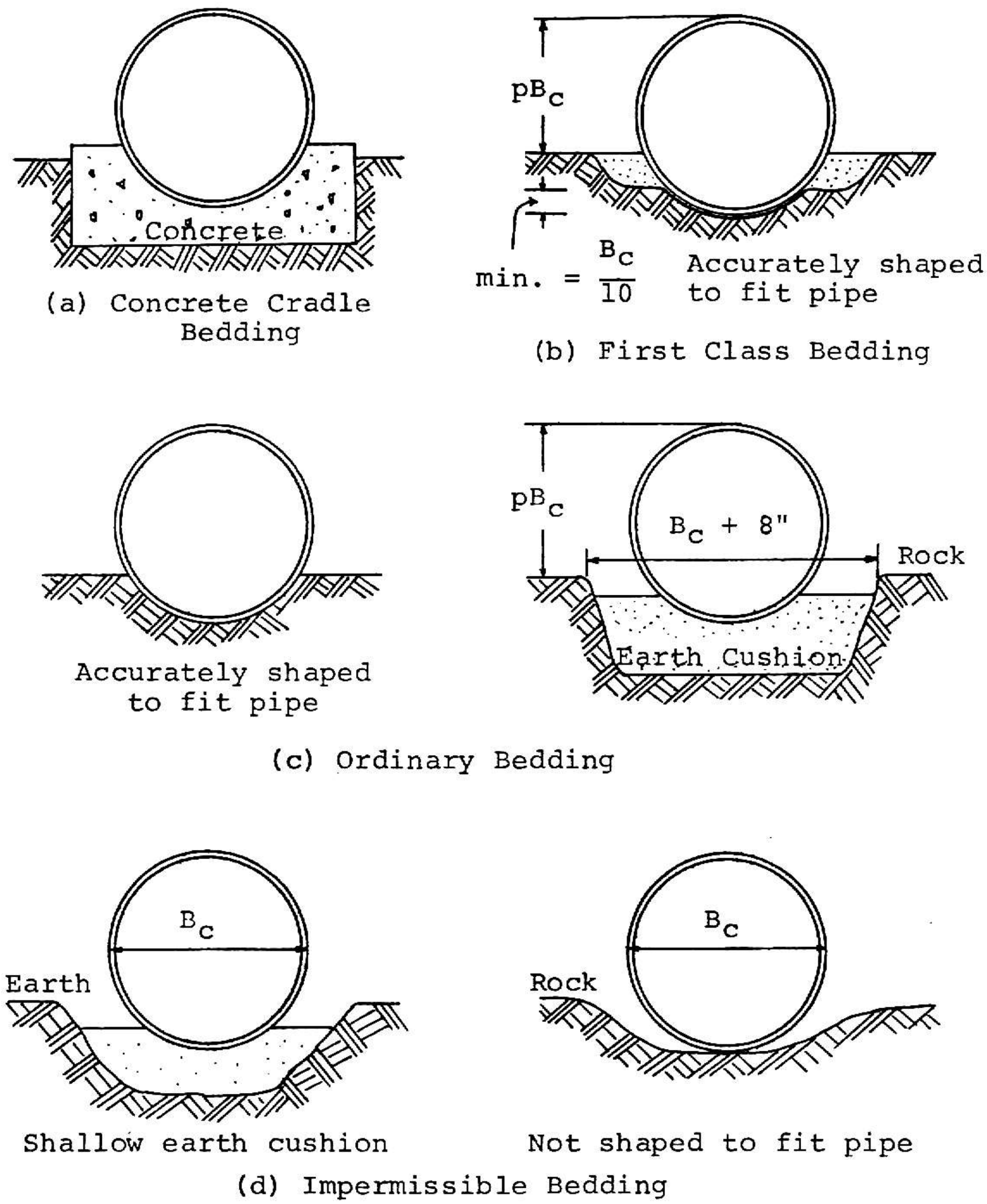


Fig. 9. Beddings for Projecting Conduits

The supporting strength of the buried conduits

The design of pipe conduits depends upon rules-of-thumb methods to determine whether or not the pipe to be used in each conduit would give the supporting strength required to support safely the loads due to, or transmitted through, the ditch filling materials. This practice has resulted necessarily in the construction of many conduits which later cracked, and often collapsed, because of insufficient supporting strength.

In general, the load on a conduit is independent of its shape and the material of which it is made, except for the effect these properties may have on the settlement of the top of the conduit. On the other hand, the supporting strength or load-carrying capacity of a conduit is intimately dependent on its shape and the kind and quality of material of which it is made.

As in any structure, the strength of a pipe is dependent upon the distribution of the load, being much smaller when the load is concentrated than when widely distributed.

Culvert pipe placed under earth embankments derive their ability to support the loads placed upon them from the inherent strength of the pipe to resist external pressures, and from the lateral pressure of the earth on the sides of the pipe which causes stresses in the pipe ring in opposite directions to those produced by the vertical loads. In a rigid pipe the inherent strength of the pipe is the predominant source of supporting ability. The only lateral pressure which can be relied upon to augment the load-carrying of a rigid pipe is the active lateral pressure of the earth or, at the most, the lateral earth pressure at rest, since the pipe deforms very little under vertical load and the sides do not move outward enough to develop any appreciable passive pressure in the enveloping earth.

With a flexible pipe, however, the situation is reversed. Here the pipe itself has relatively little inherent strength and a large part of its ability to support vertical loads must be derived from the passive pressures induced as the sides move outward against the earth. The ability of a flexible pipe to deform readily and thus utilize the passive pressure of the earth on each side of the pipe is its principal distinguishing structural characteristic and accounts for the fact that such a relatively light-weight pipe can carry heavy loads since much of the total supporting strength depends upon the sidefill material, we must consider the earth at the sides to be an integral part of the structure. The greater the sidefill density, the greater the modulus of passive pressure which results in less deflection of the pipe.

The supporting strength of a rigid culvert pipe is defined as the maximum vertical load which it will support without rupture of the pipe wall; or more simply, it is the vertical load which causes the pipe to crack. This definition is directly applicable to pipe made of plain concrete, cast iron, burned clay, or other brittle substances. In the case of reinforced concrete pipe, where cracks in the concrete open up very slowly and are extremely fine and difficult to see in the early stage of development, there is growing practice of specifying the supporting strength as the vertical load which will produce cracks in the concrete of a definite, measured width, such as 0.01 of an inch.

In producing the elongation of the horizontal diameter, the shape of the pipe is changed from circular to elliptical with the longer diameter horizontal. This change necessitates that, when concrete is placed around the pipe, either the concrete and the pipe must act as a unit in resisting this deformation or that the bond between the pipe and the concrete must be broken so that each acts separately. The investigations, reported by Schick (8), indicate that the pipe and the concrete cradle act as a unit until the load is sufficient to cause a deformation beyond the elastic limit of the pipe. It seemed quite certain that the development of the main failure cracks in the pipe and the breaking of the bond between the pipe and the concrete, on one side or the other, occurred at, or very nearly, the same instant.

The development of the rigid pipe failure cracks was always indicated (9) by a sufficient deformation to cause a marked decrease in the indicated load. In order to produce this deformation it was necessary that the pipe become more elliptical in shape. The fact that this change occurred simultaneously with the development of the main failure cracks, is clearly indicative that the side support furnished by the soil at the sides was very small. The supporting strength of the soil at the sides of the pipe varies with the type of earth or sand bedding. The facts stated above indicate quite clearly that this increase in supporting strength was not due, to any appreciable extent, to the side support furnished by the soil. Rigid circular pipes cannot be analyzed by principles of mechanics, and since they are relatively small structures, their supporting strength can most easily be determined by testing a representative group of specimens in the laboratory. Several types of laboratory tests have been devised for this purpose, such as the three-edge test, the sand bearing test, the Minnesota bearing test and the two-edge bearing test. Of these, the three-edge test is the simplest and the most widely used. In this test the load and reaction are applied to the pipe along very narrow longitudinal elements at the top and bottom of the pipe. The load situation is very severe, as shown in Fig. 10.

The ratio of the strength of a pipe under any stated condition of loading to its three-edge-bearing test strength is called the Load Factor. In a field installation the supporting strength, and therefore the load factor, are mainly influenced by conditions affecting two things: first, the distribution of the bottom reaction of the pipe; and second, the amount and distribution of any lateral soil pressure which may act against the sides of the pipe. The result of research, worked by Spangler (9), presented the working values of the load factor for use in the design of rigid pipe culverts. A working value of 1.5 has been chosen for all cases of loads on pipe due to concentrated surface load. For earth loads, these values are given in Fig. 11, Fig. 12, and Fig. 13.

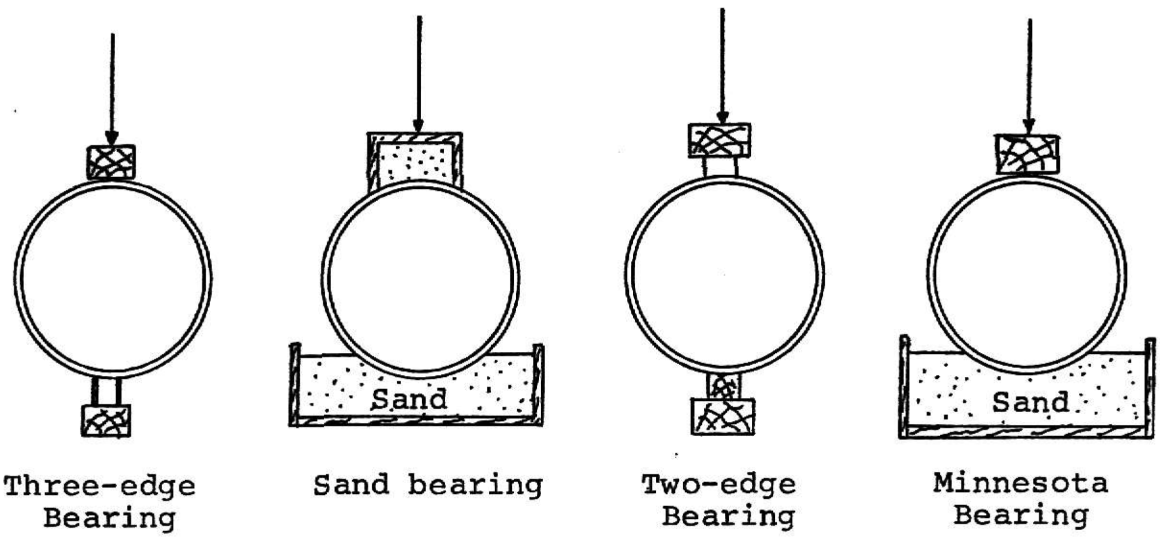


Fig. 10. Four Types of Bearing Tests of Pipe

The structural design of underground conduits requires the application of a reasonable factor of safety. For rigid pipes the following relationship is appropriate:

Field supporting strength

$$= \frac{\text{three-edge bearing strength x load factor}}{\text{factor of safety}}$$

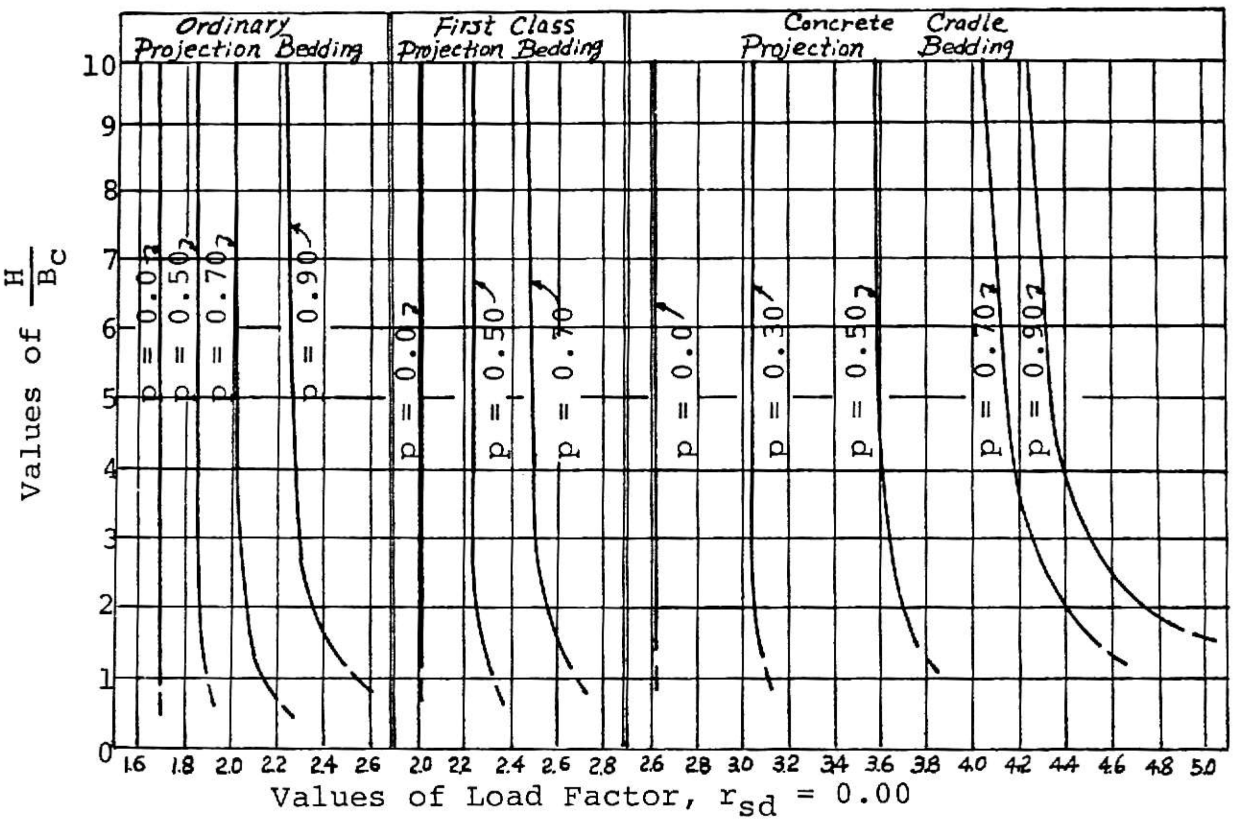


Fig. 11. Working Values of the Load Factor for Projecting Conduits

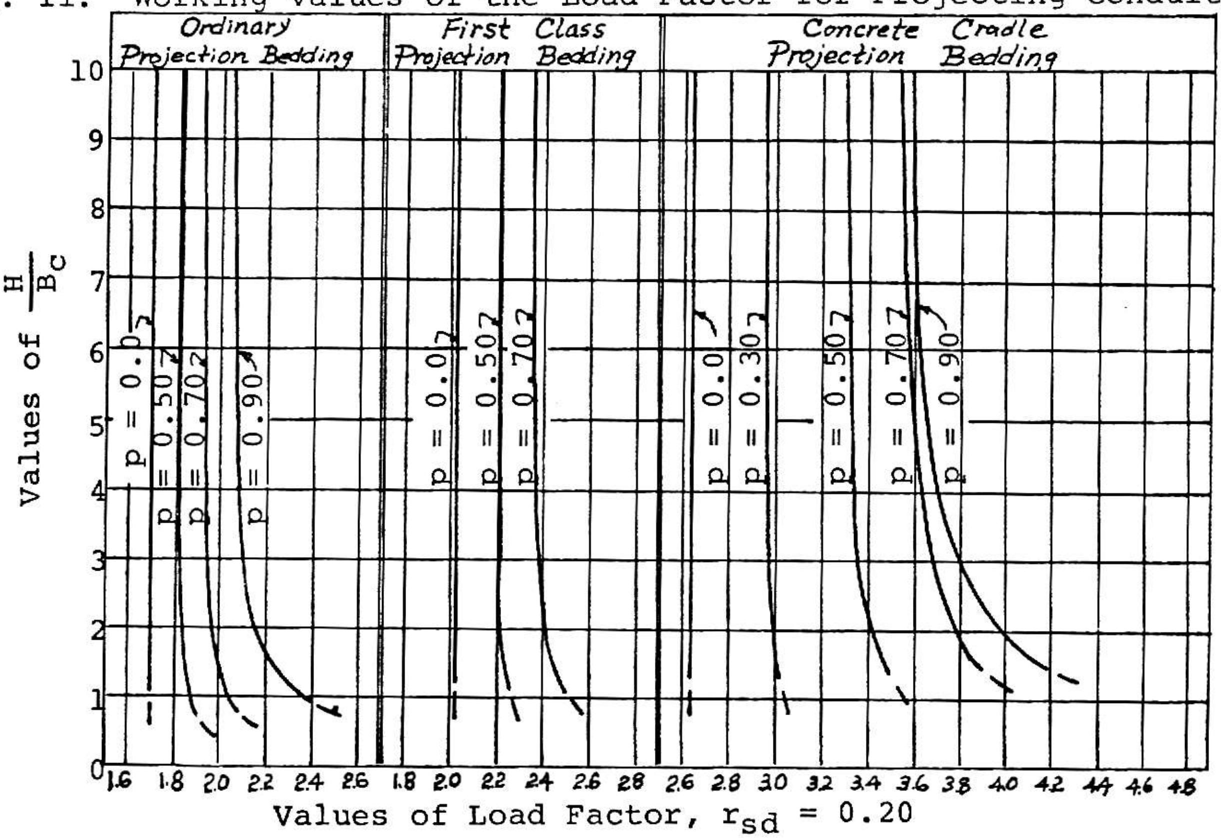


Fig. 12. Working Values of the Load Factor for Projecting Conduits

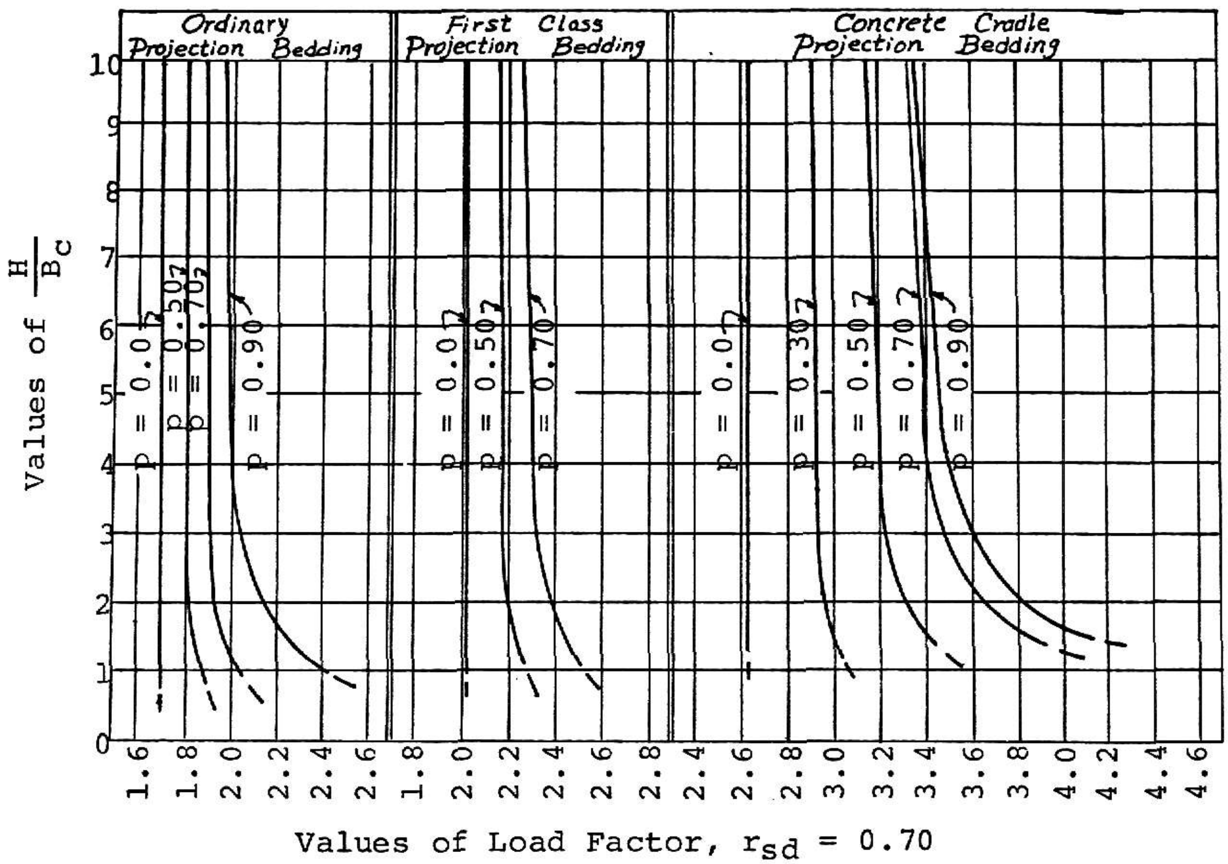


Fig. 13. Working Values of the Load Factor for Projecting Conduits.

The supporting strength of flexible conduits, such as corrugated metal pipe culverts, presents an entirely different type of problem from the rigid pipes since conduits of this kind fail be excessive deflection rather than by rupture of the pipe wall. It is necessary to investigate the deflection for a proposed installation. The basic action of a flexible pipe under earth load is as follows. Increments of vertical load on the pipe cause it to deflect. The vertical diameter shortens, and the horizontal diameter lengthens, causing the pipe to bear laterally with increasing force against the adjacent soil. The greater the lateral bearing resistance of the soil, the less will be the deformation of the pipe and the less the chance of failure by excessive deflection. At the present time a maximum deflection of 5 per cent of the nominal diameter is a widely accepted design limitation, although research is needed to more clearly establish this criterion and to determine its applicability to pipes of various diameters. A formula, derived by Spangler (12), for estimating the deflection of a flexible pipe under an earth fill is

$$X = D_1 \frac{kW_c r^3}{EI + 0.061\ E'r^3} \quad \text{- - - - - - - - - - - -} \quad (15)$$

The bedding angle, α, is defined as one-half the angle subtended by the arc of the pipe ring which is in contact with the pipe bedding, as shown in Fig. 14. The deflection lag factor cannot be less than unity and has been observed to range upward toward a value of 2.0. A normal range of values from 1.25 to 1.50 is suggested for design purposes. The deflection lag factor is the ratio of final deflection to the load-augmented deflection.

Values of the bedding constant, K, for various values of the bedding angle are shown in Table 2. The stiffness factor, EI, may be determined by testing the pipe metal to determine its modulus of elasticity and by calculating the moment of inertia of the shape of the cross-section of the pipe wall.

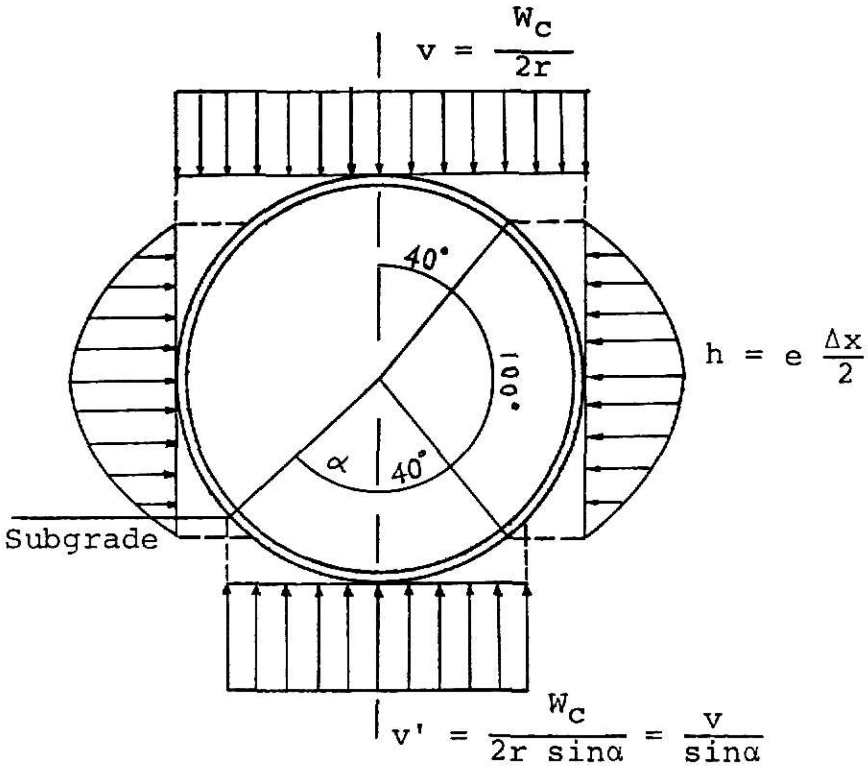


Fig. 14. Assumed Distribution of Pressure on Flexible Conduit

Bedding Angle Degrees	Bedding Constant k
0	0.110
15	0.108
22.5	0.105
30	0.102
45	0.096
60	0.090
90	0.083

Table 2. Values of Bedding Constant

NUMERICAL EXAMPLES

Example 1. Suppose that it is desired to determine the load on 36 in. diameter sewer pipe under a 18 ft. fill which has a unit weight of 120 pcf. Assume that the projection ratio is 0.7, and the settlement ratio is +0.7.

Solution:

In this case,

$H = 18$ ft. $\qquad p = 0.7$

$B_c = 3$ ft. $\qquad r_{sd} = +0.7$

$\frac{H}{B_c} = 6 \qquad r_{sd}p = +0.49 = +0.5$

From Fig. 6, the value of C_c is 9. Substituting in Eq. 8, we obtain:

$$W_c = 9 \times 120 \times 3^2 = 9720 \text{ lb. per lin. ft.}$$

Example 2. An 18 in. nonreinforced sewer pipe is to be installed under 18 ft. of cover in a ditch which is 36 in. wide at the elevation of the top of the pipe. Assume a clay soil weighing 120 pcf, Ordinary Bedding, and factor of safety = 1.25. Determine the required three-edge bearing strength of the pipe.

Solution:

$H = 18$ ft. $\qquad B_d = 3$ ft.

$\frac{H}{B_d} = 6$

From Fig. 2, $C_d = 3$

From Eq. 3a, $W_c = 3 \times 120 \times 3^2 = 3{,}240$ plf.

Load factor = 1.5 (Ordinary Bedding)

Required three-edge bearing strength $= \frac{3{,}240 \times 1.25}{1.5}$

$= 2{,}700$ plf.

Example 3. Determine the three-edge bearing strength required of 36 in. clay pipe with a wall thickness of 2 5/8 in., for use in a culvert under an embankment 3 ft. high and subjected to traffic loads equal to a 15-ton truck with 12,000 pounds on each rear wheel, spaced 6 ft. center to center.

$H = 3$ feet $\qquad p = 0.80$

$w = 120$ pcf $\qquad r_{sd} = 0.50$

$L = 3$ feet (assumed effective length)

Impact = 50 percent of live load

Solution:

$B_c = 3.44$ ft., $\frac{H}{B_c} = 0.87$, and $r_{sd}p = 0.4$

From Fig. 6, $C_c = 1.05$, then the vertical earth load is

$$W_c = 1.05 \times 120 \times 3.44^2 = 1{,}500 \text{ plf.}$$

Applying a factor of safety of 1.5 gives 2,250 plf. which is that part of the total load due to the earth fill. From Fig. 13, the load factor for ordinary bedding is 2.2. Dividing 2,250 by 2.2, gives 1,000 plf, the three-edge bearing strength required for that part of the total load due to the earth fill.

According to the Formula (15), the combined effect of the two rear wheels on a pipe 3 feet long directly under one of the wheels is 4,650 pounds, or 1,550 pounds per linear foot. Adding 50 per cent for impact gives 2,325 pounds per linear foot. Applying a factor of safety of 1.5 gives about 3,500 pounds per linear foot. Since the load factor for super-imposed loads is 1.5, the three-edge bearing strength required for that part of the total load which is due to the truck wheels is about 2,300 pounds per linear foot.

The required average three-edge bearing strength is then

1,000 plf	(earth load)
2,300 plf	(traffic load)
3,300 plf	(total)

Example 4. A 60 inch corrugated metal pipe is to be installed as a projecting conduit with a 60 degree bedding (bedding angle = 30 degrees) and covered with an embankment 20 ft. high. Assume the projection ratio = 0.7, the settlement ratio = 0, the unit weight of soil = 120 pcf, and the value of E' = 700 psi. Determine the deflection of the pipe with the deflection lag factor = 1.25.

Solution:

$$B_c = 5 \text{ ft.}, \; H = 20 \text{ ft.}, \; \frac{H}{B_c} = 4.0, \; r_{sd}p = 0$$

From Fig. 6, $C_c = 4.0$

By Eq. 8, $W_c = 4.0 \times 120 \times 5^2 = 12{,}000$ plf or 1,000 pli

From Table 2, $K = 0.102$

$I = 0.0045$ in.^{4} per in.

Also $E = 30{,}000{,}000$ psi, $EI = 135{,}00$ lb.-in.

By Eq. 15,
$$X = \frac{1.25 \times 0.102 \times 1{,}000 \times 30^2}{135{,}000 + 0.061 \times 700 \times 30^2} = 2.68 \text{ in.}$$

CONCLUSION

In designing conduits to carry safely the load to which they will be subjected in use, it is necessary to know, in advance, both the load and the supporting strength of the conduit. Marston's theories of loads on conduits are applied to all types of underground conduits. These essential design formulas, graphs and other data for rigid and flexible conduits may be used to compute the safe load carrying capacity of a conduit. This theory can also be used to compute the earth forces on the pipe. In summary the theories of Marson and Spangler have not been appreciably altered and these theories have been found to have excellent practical application. The field supporting strength of rigid pipe is materially affected by the character of the bedding of the pipe. The design of flexible conduits is very dependent upon the side support of the lateral soil mass.

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A STUDY OF

THE EXTERNAL LOADS APPLIED ON BURIED CONDUITS

by

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AN ABSTRACT OF A MASTER'S REPORT

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ABSTRACT

The underground conduits of the types used for sewers, drains, water mains, gas lines, and the like, have served to improve the standard of living of mankind since the dawn of civilization. Marston's theory is one of the most rational theories for computing the loads on the underground conduits. The field supporting strength of rigid pipe is dependent upon the distribution of the applied vertical loads and the strength of the pipe as well as the bedding conditions. The passive pressures on the sides of the flexible pipe is the principal structural characteristic and accounts for the loading that the flexible pipe can support without structural distress.

Typical examples of the computations for each of these conditions are shown as a guide to learning the technique required for the design of conduits.