

deflection. Some pipes manufactured in this manner are sometimes referred to as semirigid. This is simply a misnomer. Many solid wall PVC and ductile iron pipes are actually more rigid and still behave as flexible pipes.

Parallel Pipes and Trenches

When buried pipes are installed in parallel, principles of analysis for single pipes still apply. Soil cover must be greater than minimum. However, the design of parallel buried pipes requires an additional analysis for heavy surface loads. Consider a free-body-diagram of the pipe-clad soil column between two parallel pipes. See Fig. 3.26. Section AA is the minimum cross section. This column must support the full weight of the soil mass, shown crosshatched, plus part of the surface load W shown as a live load pressure diagram. The soil column is critical at its minimum section AA at the spring lines.

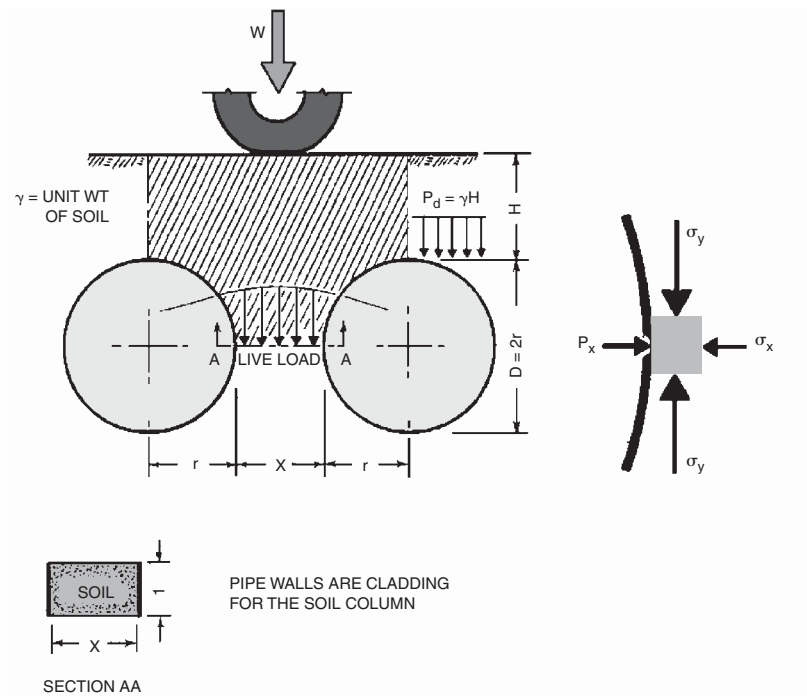


Figure 3.26 Soil column between parallel pipes showing minimum section AA. This section must be able to resist the entire live load plus part of the dead load.

Definitions of terms

D = outside diameter of pipe = $2r$

A = pipe wall area per unit length of pipe

σ_f = ring compression strength of wall

σ_t = ring compression stress in wall

SF = safety factor

W = live load on surface

γ = unit weight of soil

w = load per length of pipe

w_d = dead load per length of pipe

w_l = live load per length of pipe

P = vertical pressure = w/D or V/D

σ_y = vertical soil stress on section AA

S' = vertical soil compression strength

X = width of section AA between pipes

V = total vertical load per length on section AA = $w_d + w_l$

$V' = V - \gamma HD$ = vertical load per length supported by soil at section AA = total vertical load reduced by load that is supported by pipe walls

H = height of soil cover over pipe

For design, the strength of the column at section AA must be greater than the vertical load. Failure (a performance limit) occurs if either of the following happens.

1. Thrust in the pipe wall exceeds the ring compression strength.
2. The vertical soil stress at section AA exceeds the compressive strength (vertical resistance) of the soil.

The ring compression strength of the pipe wall is usually the yield strength. For rigid pipes, σ_f is the crushing strength of the wall. The value for σ_f can be obtained from the pipe manufacturer.

The strength of the soil is found as follows. Assume that the embedment is granular and compacted. Soil strength is vertical stress σ_y at slip. Horizontal soil stress is provided by the pipe walls. Approximate soil strength may be found from triaxial soil tests in which interchamber pressure is equal to the horizontal pressure P_x of the pipe against the soil. For circular, flexible pipes at soil slip, $P_x = P_d = \gamma H$.

Stresses in the pipe and soil are each calculated independently. This is because the bond between soil and pipe can be assumed to be zero. The bond cannot be ensured because of fluctuations in temperature, moisture, loads, etc., all of which tend to break down the bond at the soil-pipe interface.

The pipe must be adequate. Therefore, before the soil column is analyzed, design starts with the ring compression equation

$$\frac{PD}{2A} = \frac{\sigma_f}{SF}$$

For worst-case ring compression, the live load W is directly above the pipe where $P = P_l + P_d$. The live load effect P_l can be found by the Boussinesq equation (see Chap. 2). If W is assumed to be a point load directly over the pipe, the Boussinesq equation reduces to

$$P_l = \frac{0.477W}{H^2} \quad (3.25)$$

If live load W is assumed to be a distributed surface pressure, the Newmark integration can be used. Soil cover must be greater than minimum by the pyramid/cone analysis of Chap. 2. After an adequate pipe has been selected, the soil column can be designed.

The following analysis is for flexible pipes. The vertical load supported by the two flexible pipe walls at section AA is no less than $2PD/2 = PD$. So, in the design of the soil column, it is assumed, conservatively, that the pipe wall cladding takes a vertical load of PD . The remainder of the load must be supported by the soil. The greatest load on the soil occurs when the heavy live load W is centered above section AA—centered between the two pipes. At this location, not only is the live load pressure on the soil maximum, but also the portion supported by the pipe walls is minimum. Pipe walls carry dead load $P_dD = \gamma HD$. The live load P_l on the pipes is small enough to be neglected. The live load on section AA cannot be neglected. This is the Boussinesq soil stress σ_y , and it must be less than strength S' . Vertical stress is soil load per length divided by the distance between pipes X :

$$\sigma_y = \frac{V'}{X} = \frac{S'}{SF}$$

Per unit length of pipe, V is the sum of the deadweight of the cross-hatched soil mass w_d and that portion w_l of the surface live load W that reaches section AA. See Fig. 3.26. The dead load w_d per unit

length of pipe is the soil unit weight times the crosshatched area, i.e.,

$$w_d = \left[(X + 2r)(H + r) - \frac{\pi r^2}{2} \right] \gamma \quad (3.26)$$

The live load w_l is the volume under the live load pressure diagram of Fig. 3.26 at section AA. It is calculated by means of the Boussinesq live load w_l per unit length.

$$w_l = \frac{0.477WX}{(H + r)^2}$$

Example Problem 3.3 What is the vertical soil stress at section AA of Fig. 3.26? The pipes are 24-in-diameter DR 35 PVC. There is 12 in of soil between the two at the spring line. Soil cover is 1.5 ft of soil at a unit weight of 120 lb/ft³. A surface wheel load of $W = 20$ kips is anticipated. Thickness $t = 24/35 = 0.686$ in.

1. Find the maximum ring compression stress in the pipe wall.
2. Evaluate the soil stress at section AA.

The pressure on pipe due to dead load is

$$P_d = \gamma H = 120 \times 1.5 = 180 \text{ lb/ft}^2$$

or

$$P_d D = 180 (2) = 360 \text{ lb/ft}$$

For live load on pipe, use the Boussinesq equation:

$$w_l = \frac{0.477WD}{H^2} = \frac{0.477 \times 20,000 \times 2}{(1.5)^2} = 8480 \text{ lb/ft}$$

$$P_l = \frac{w_l}{D} = \frac{8480}{2} = 4240 \text{ lb/ft}^2$$

$$P = P_d + P_l = 180 + 4240 = 4420 \text{ lb/ft}^2$$

Total vertical load (per length) on pipe $V = w_d + w_l = 360 + 8480 = 8840$ lb/ft = 737 lb/in. The ring compression stress is

$$\sigma_t = \frac{PD}{2t} = \frac{V}{2t} = \frac{737}{2(0.686)} = 537 \text{ lb/in}^2$$

The pipe wall will not crush. It is interesting to note that the total load of 4420 lb/ft² is equivalent to a static load of about 37 ft of cover for soil weighing 120 lb/ft³. Thus, the soil in the pipe zone should be compacted to a density required for 37 ft of cover.

For the dead load on section AA, use Eq. (3.26),

$$\begin{aligned} w_d &= \left[(X + 2r)(H + r) - \frac{\pi r^2}{2} \right] \gamma \\ &= \left[(1 + 2)(1.5 + 1) - \frac{\pi}{2} \right] 120 \\ &= 712 \text{ lb/ft (dead load on section AA)} \end{aligned}$$

or
$$P_d = \frac{w_d}{X} = \frac{712}{1} = 712 \text{ lb/ft}^2$$

For live load on section AA, the live load w_l can be evaluated by the Boussinesq equation, (3.25), where

$$P_l = 0.477 \frac{W}{H^2} = \text{total live load on section AA}$$

where W = wheel load on surface and $H = 2.5$ ft, which is the depth to section AA = 1.5 ft + 1 ft. So

$$P_l = 0.477 \frac{W}{H^2} = \frac{0.477 (20,000)}{(2.5)^2} = 1526 \text{ lb/ft}^2$$

The soil pressure on section AA is

$$P' = P - \gamma H = P_d + P_l - \gamma H = 712 + 1526 - 120(1.5) = 2058 \text{ lb/ft}^2$$

where γH is the load supported by the pipe walls. Vertical soil stress on section AA is $\sigma_y = P' = 2058 \text{ lb/ft}^2$.

The maximum pressure σ_x at the side of the pipe should be no greater than the vertical load at the top of the pipe. That is, σ_x should be less than or equal to $\gamma H = 180 \text{ lb/ft}^2$. If σ_x is greater than γH , the pipes may collapse inward from the sides. And σ_x is related to σ_y by the following equation:

$$\sigma_x = \frac{\sigma_y}{K}$$

where

$$K = \frac{1 + \sin \phi}{1 - \phi} = 3$$

where K is Rankine's lateral pressure ratio and ϕ is the soil friction angle, and for this soil $\phi = 30^\circ$.

Thus, the pipe must be able to support a horizontal load of $\sigma_y/3 = 2058/3 = 686 \text{ lb/ft}^2$, but it will only support 180 lb/ft^2 (vertical dead load on top of pipe). To remedy the situation, one could:

1. More than triple the space between the parallel pipes.

2. Place the pipes deeper to diminish the live load, and increase the vertical dead load.
3. Place a concrete slab on the soil surface to distribute the live load.

Example Problem 3.4 What is the vertical soil stress at section AA of Fig. 3.26? The pipes are 72-in-diameter HDPE profile-wall pipe. There is 24 in of soil between the two at the spring line. Soil cover is 1.5 ft of soil at a unit weight of 120 lb/ft³. A surface wheel load of $W = 20$ kips is anticipated. This is similar to the last.

$$\text{Area per unit length} = \text{effective thickness} = 0.675 \text{ in}$$

$$\text{Stiffness } F/\Delta y = 18 \text{ lb/in}^2$$

$$\text{Tensile strength} = 1000 \text{ lb/in}^2$$

$$\text{Compression strength} = 3000 \text{ lb/in}^2$$

1. Find the maximum ring compression stress in the pipe wall.
2. Evaluate the soil stress at section AA.

The pressure on pipe due to dead load is

$$P_d = \gamma H = 120 (1.5) = 180 \text{ lb/ft}^2$$

or

$$P_d D = 180(6) = 1080 \text{ lb/ft}$$

For live load on pipe, use the Boussinesq equation:

$$w_l = \frac{0.477WD}{H^2} = \frac{0.477 \times 20,000 \times 6}{(1.5)^2} = 25,440 \text{ lb/ft}$$

$$P_l = \frac{w_l}{D} = \frac{25,440}{6} = 4240 \text{ lb/ft}^2$$

Total vertical load on pipe

$$V = w_d + w_l = 1080 + 25,440 = 26,520 \text{ lb/ft} = 2210 \text{ lb/in}$$

or

$$P = \frac{V}{D} = \frac{26,520}{6} = 4420 \text{ lb/ft}^2$$

Ring compression stress

$$\sigma_t = \frac{PD}{2t} = \frac{V}{2t} = \frac{2210}{2(0.675)} = 1637 \text{ lb/in}^2$$

As in the previous example, the pipe wall will not crush, but this stress is more than one-half of the compressive strength. There may be local buckling. It is interesting to note that the total load of 4420 lb/ft² is equivalent to a static load of about 37 ft of cover for soil weighing 120 lb/ft². Thus the soil in the pipe zone should be compacted to a density required for 37 ft of cover.

For the dead load on section AA, use Eq. (3.26):

$$\begin{aligned} w_d &= \left[(X + 2r)(H + r) - \frac{\pi r^2}{2} \right] \gamma \\ &= \left[(2 + 6)(1.5 + 3) - \frac{9\pi}{2} \right] 120 \\ &= 2624 \text{ lb/ft (dead load on section AA)} \end{aligned}$$

or

$$P_d = \frac{w_d}{X} = \frac{2624}{2} = 1312 \text{ lb/ft}^2$$

Live load on section AA can be evaluated by the Boussinesq equation (3.25):

$$P_l = \frac{0.477W}{H^2} = \text{total live load on section AA}$$

W = wheel load on surface

$$H = 4.5 \text{ ft (which is the depth to section AA)} = 1.5 \text{ ft} + 3 \text{ ft}$$

$$\text{so } P_l = \frac{0.477W}{H^2} = \frac{0.477(20,000)}{(4.5)^2} = 471 \text{ lb/ft}^2$$

The soil pressure at section AA is

$$P' = P - \gamma H = P_d + P_l - \gamma H = 1312 + 471 - 120(1.5) = 1603 \text{ lb/ft}^2$$

And γH is the load supported by the pipe walls. Vertical soil stress on section AA is $\sigma_y = P' = 1603 \text{ lb/ft}^2$.

The maximum pressure σ_x at the side of the pipe should be no greater than the vertical load at the top of the pipe. That is, σ_x should be less than or equal to $\gamma H = 180 \text{ lb/ft}^2$. If σ_x is greater than γH , the pipes may collapse inward from the sides. And σ_x is related to σ_y by the following equation:

$$\sigma_x = \frac{\sigma_y}{K} \quad \text{where} \quad K = \frac{1 + \sin \phi}{1 - \sin \phi} = 3$$

Thus, the pipe must be able to support a horizontal load of $\sigma_y/3 = 1603/3 = 534 \text{ lb/ft}^2$, but it will only support 180 lb/ft^2 . To remedy the situation one could

1. Increase the space between the parallel pipes.

2. Place the pipes deeper to diminish the live load, and increase the vertical dead load.
3. Place a concrete slab on the soil surface to distribute the live load.

Rigid pipes

For a rigid pipe, the pipe wall will take almost the entire load because of the great difference between the modulus of elasticity of the pipe wall and the modulus of elasticity (compressibility) of the soil. Unlike flexible pipes, rigid pipes do not exert pressure $P_x = P$ against the soil. Total load Q is supported by the pipe walls in ring compression and by the soil in vertical passive resistance.

Safety factors

Safety factors for live load analysis may be close to unity. In the above analyses, the arching action of the soil cover was neglected, and thus the analyses are very conservative.

Parallel trench

In 1968, parallel-trench research was conducted at the Buried Structure Laboratory at Utah State University. This research was conducted under the direction Reynold K. Watkins with the primary objective to answer questions that had arisen concerning embedment stability when a trench is excavated parallel to an existing buried flexible pipe. Questions such as these were posed:

1. What is the stability of the trench?
2. At what minimum separation between the pipe and the parallel trench will the pipe collapse?
3. Compacted soil at the sides supports and stiffens the top arch. What happens to a buried flexible pipe when some of or all the side support is removed in a parallel excavation?
4. What are the variables that influence collapse?

Answers to these questions as determined by Watkins are summarized here.

In order to reduce the number of variables, ring stiffness was assumed to be zero. Results were conservative because no pipe has zero ring stiffness. For the most flexible plain steel pipes, D/t is less than 300. For the test pipes, D/t was 600 in an attempt to approach zero stiffness. It was necessary to hold the pipes in shape on mandrels during placement of the backfill.

Vertical trench walls

Figure 3.27 shows the cross section of a buried flexible pipe with an open-cut vertical trench wall parallel to it. If trench wall AB is cut back closer and closer to the buried pipe, side cover X decreases to the point where the sidefill soil is no longer able to provide the lateral support required to retain the flexible ring. The ring deflects, thrusting out a soil wedge, as indicated in Fig. 3.28. As the ring deflects, a soil prism breaks loose directly over the ring. The soil prism collapses the flexible ring. In order to write pi terms to investigate this phenomenon, the pertinent fundamental variables must be identified. Ring stiffness is ignored because the ring is flexible. In fact, at zero ring deflection, the ring stiffness has no effect anyway. The remaining fundamental variables are as follows:

Fundamental variables	Basic dimensions
X = minimum side cover (minimum horizontal separation between pipe and trench at collapse)	L
D = pipe diameter	L
H = height of soil cover over top of pipe	L
Z = critical depth of trench in vertical cut (vertical sidewalls)	L

Critical depth Z is a convenient measure of soil strength. It is defined as the maximum depth of a trench at which the walls stand in vertical cut. At greater depths the trench walls slip or cave in.

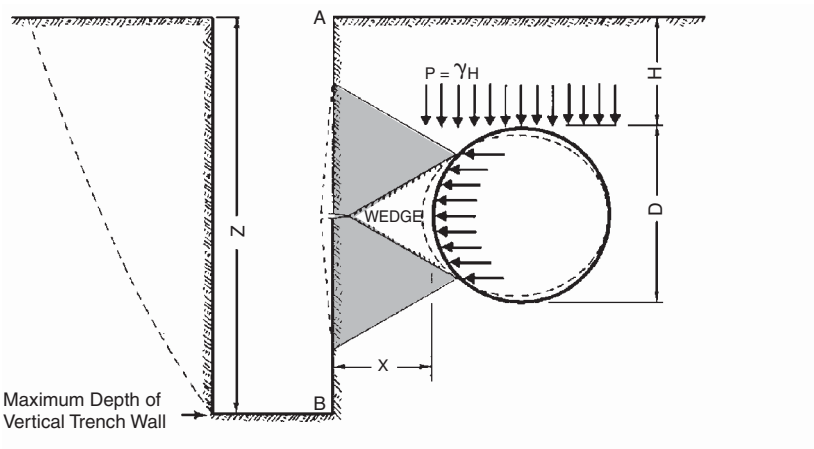


Figure 3.27 Vertical trench wall parallel to a buried flexible pipe showing the soil wedge and shear planes that form as the pipe collapses.

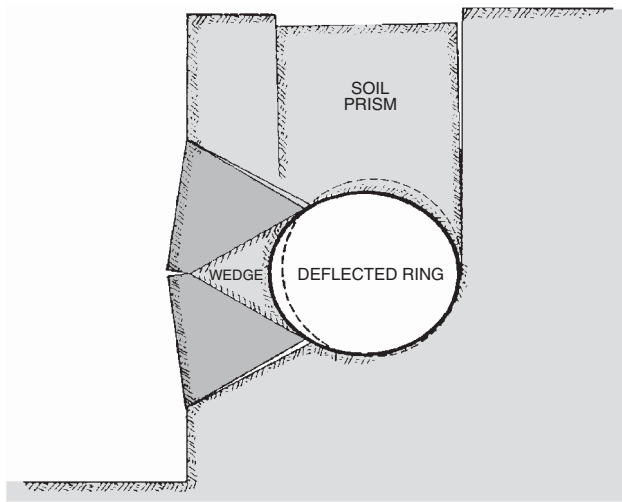


Figure 3.28 Formation of a soil prism on the pipe as a collapse mechanism forms.

Critical depth Z may be determined by excavation in the field, or it may be calculated from the dimensionless stability number $Z\gamma/C$. See Fig. 3.31.

$$\frac{2C}{\gamma Z} = \tan \left(45^\circ - \frac{\phi}{2} \right) \quad (3.27)$$

where Z = critical depth of trench in vertical cut

γ = unit weight of soil, lb/ft³

C = soil cohesion, lb/ft²

ϕ = soil friction angle of trench wall

Depth Z can be found from Eq. (3.27) if soil properties γ , C , and ϕ are provided by laboratory tests.

To investigate the four fundamental variables, three pi terms are required. One possible set is X/D , H/D , and H/Z . Tests show that H/D is not pertinent. Only X/D and H/Z remain as pertinent pi terms.

For a vertical trench wall excavated parallel to a flexible pipe:

1. If the ring has some stiffness, and if soil cover H is not great enough to collapse the ring, soil may slough off the pipe into the trench. This is not considered failure because the soil can be replaced during backfilling.

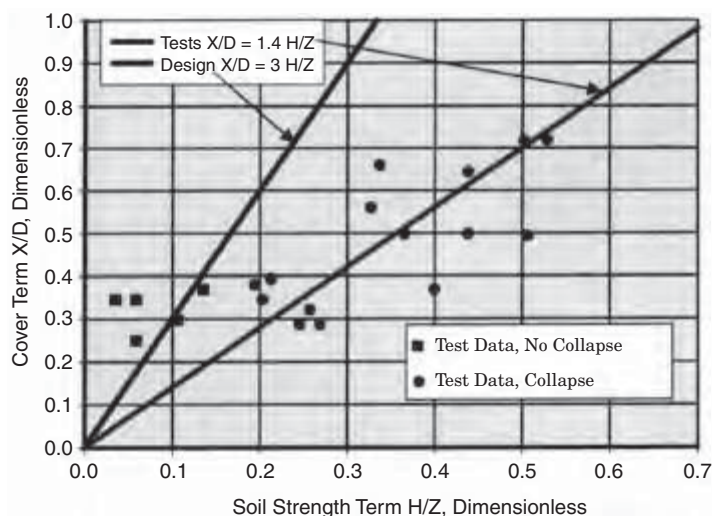


Figure 3.29 Cover term X/D as a function of the soil strength term H/Z for a vertical trench wall excavated parallel to a very flexible pipe.

2. The ring collapses under a free-standing prism of soil that breaks loose on top of the pipe.
3. Failure is sudden and complete collapse.

Test data are plotted in Fig. 3.29, which shows X/D as a function of H/Z . The best-fit straight-line equation is $X/D = 1.4H/Z$. The probable error in X/D is ± 0.1 , so the probable error in side soil cover X is roughly $\pm D/10$. Because field conditions may be less reliable than laboratory conditions, the safety factor should be 2. Therefore, the minimum side cover X might be specified as

$$\frac{X}{D} = \frac{3H}{Z} \quad (3.28)$$

Of interest in Fig. 3.29 are the data points indicated by squares. These do not represent collapse. The ring stiffness for the test pipes was great enough that part of the shallow soil cover merely sloughed off the pipes after the soil wedge fell into the trench. If ring stiffness were to be included as a fundamental variable, ring deflection would have to be included. Then the coefficient of friction between pipe and soil should also be included as a fundamental variable.

If the pipe has significant ring stiffness, the height of soil cover that it can support without collapse can be found for uniform vertical pressure with no side support.

$$\text{Moment } M = \frac{Pr^2}{4} \quad \text{Stress } \sigma = \frac{Mc}{I}$$

For pipes, at yield stress, based on elastic theory,

$$P_y = \frac{4\sigma_y}{r^2} \frac{I}{c} = \frac{16\sigma_y}{D^2} \frac{I}{c} \quad (3.29)$$

where P = vertical soil pressure

I = moment of inertia of wall cross section

D = pipe diameter = $2r$

c = distance to wall surface from neutral surface = $t/2$ for plain pipes

t = wall thickness of plain pipes

σ_y = yield strength of pipe

DR = dimension ratio D/t for plane pipes

$$P_y = \frac{8\sigma_y}{3} \left(\frac{t}{D} \right)^3 \quad \text{for plain pipes} \quad (3.30)$$

For plain pipes, the moment that produces a fully plastic hinge is 1.5 times the moment that produces yielding. Therefore the fully plastic load is

$$P_{fp} = 4\sigma_y \left(\frac{t}{D} \right)^3 \quad (3.31)$$

The approximate vertical ring deflection at plastic hinging can be calculated as follows:

$$\frac{\Delta y}{D} \approx \frac{0.02P_{fp}D^3}{EI}$$

where $\Delta y/D$ = ring deflection

P = vertical pressure on ring

D = circular pipe diameter

EI = wall stiffness per unit length of pipe

Sloped trench walls

Figure 3.30 shows a flexible pipe in cohesionless soil for which the slope is the angle of repose $\approx \phi$. Pressure distribution on the ring is triangular, as shown. Maximum moment at A can be found by

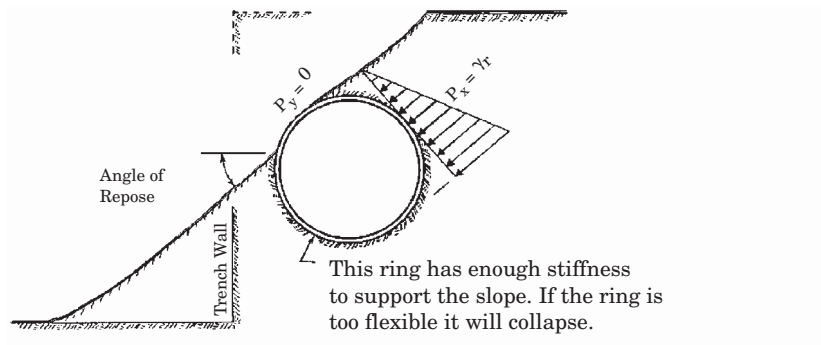


Figure 3.30 Trench wall sloped at the angle of repose (soil is stable). The flexible pipe requires sufficient pipe stiffness or minimum cover to prevent collapse.

Castigliano's equation. However, it is sufficiently accurate to find the equivalent moment $M_A = Pr^2/4$ for average uniform pressure $P_x = r\gamma$.

Excavation

Depth of the excavation must include *overexcavation* required to remove unstable subbase material. It should be replaced by approved bedding material. Some tank manufacturers consider soil to be unstable if the cohesion is less than $C = 750 \text{ lb/ft}^2$ based on the unconfined compression test or if the bearing capacity is less than 3500 lb/ft^2 . In the field, bearing capacity is adequate if an employee can walk on the excavation floor without leaving footprints. A muddy excavation floor can be choked with gravel until it is stable. These are conservative criteria for soil stability.

Of greater concern are OSHA safety requirements for retaining or sloping the walls of the trench. Excavations for tanks are usually short enough that OSHA trench requirements leave a significant margin of safety. Longitudinal, horizontal soil arching action is significant.

These criteria for bearing capacity and cohesion are equivalent to a vertical trench wall over 20 ft deep. Bearing capacity of 3500 lb/ft^2 can support more than 29 ft of vertical trench depth at soil unit weight of 120 lb/ft^3 . Cohesion of 750 lb/ft^2 can support a vertical open-cut trench wall that is more than 20 ft deep.

Critical depth of vertical trench wall. Granular soil with no cohesion cannot stand in vertical cut. Much of the native soil in which pipes and tanks are buried has cohesion. Therefore, the wall of the excavation can stand in vertical open cut to some critical depth Z . See Fig. 3.31

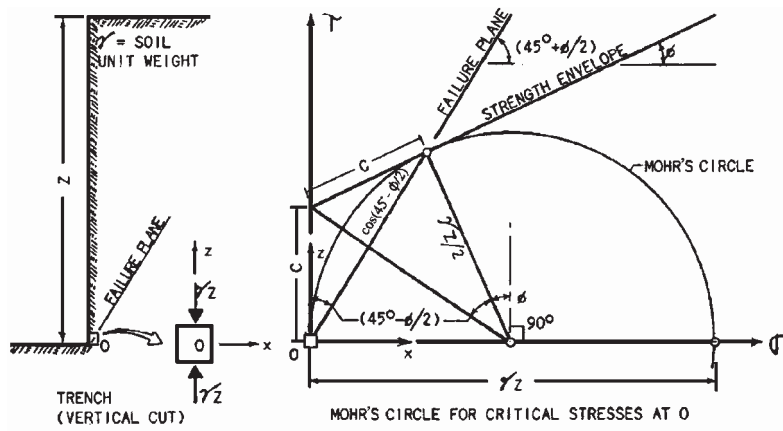


Figure 3.31 Mohr's circle analysis for finding critical depth Z for a vertical trench wall in a brittle soil cohesion C and a soil friction angle ϕ , where $2C/(\gamma Z) = \tan(45^\circ - \phi/2)$.

(left). Greater depth will result in a “cave-in” starting at the bottom corner O , where the slope of the failure plane is $45^\circ + \phi/2$. For a two-dimensional trench analysis, the infinitesimal soil cube O is subjected to vertical stress γZ , where

γ = soil unit weight

Z = critical depth of vertical trench wall

ϕ = soil friction angle

C = soil cohesion

Mohr's circle is shown in Fig. 3.31. The orientation diagram (x - z) of planes on which stresses act is superimposed, showing the location of the origin O . The strength envelope slopes at soil friction angle ϕ from the cohesive strength C . At soil slip, Mohr's stress circle is tangent to the strength envelope. From trigonometry,

$$\tan\left(45^\circ - \frac{\phi}{2}\right) = \frac{2C}{\gamma Z}$$

This is the critical depth, Eq. (3.31).

From tests, Eq. (3.27) provides a reasonable analysis for brittle soil. If the soil is plastic, soil slip does not occur until shearing stresses reach shearing strength C . Consequently, in plastic soil, the critical depth equation is $2C/(\gamma Z) = 1$. Below the water table, critical depth is essentially doubled.

Example Problem 3.5 What is the critical depth Z of a vertical, open-cut trench wall if $C = 600 \text{ lb/ft}^2$, $\gamma = 120 \text{ lb/ft}^3$, and $\phi = 30^\circ$?

Substituting into Eq. (3.27) gives $Z = 17.3 \text{ ft}$. This is a lower limit if the soil has some plasticity (is not brittle).

Example Problem 3.6 Suppose that a sloped trench wall exposes a pipe as shown in Fig. 3.30. Pressure P_x must be resisted by ring stiffness. What is the required pipe stiffness for a 72-in HDPE pipe to prevent buckling?

Assume the soil is granular with unit weight of 120 lb/ft^3 . Since there is no soil on one side of the pipe, assume the pipe is unsupported and must be able to withstand the pressure $P_x = \gamma r$. From Eq. (3.14),

$$P_{cr} = \frac{3EI}{(1 - \nu^2) R^3} = \gamma r$$

Assume the following:

$$E = 110,000 \text{ lb/in}^2 \quad \nu = 0.4$$

and solve for EI/R^3 .

$$\frac{EI}{R^3} = \frac{\gamma r (1 - \nu^2)}{3} = \left(\frac{120 \text{ lb/ft}^3}{1728 \text{ in}^3} \right) 36 (0.84) = 2.1 \text{ lb/in}^2$$

Pipe stiffness is

$$\frac{f}{\Delta y} = 6.7 \frac{EI}{R^3} = 14 \text{ lb/in}^2$$

It is possible the bending stress will exceed the yield strength before buckling takes place. The maximum moment in the ring is $M = (P_x r^2)/4$, and bending stress is given by

$$\sigma_y = \frac{Mc}{I} = \frac{M}{I/c}$$

Solve for I/c .

$$\frac{I}{c} = \frac{M}{\sigma_y}$$

$$M = \frac{(\gamma r) r^2}{4} = \frac{\gamma r^3}{4} = \left(\frac{120}{1728} \right) \frac{(36)^3}{4} = 810 \text{ in} \cdot \text{lb/in}$$

Assume the following:

$$\sigma_y = 110,000 \text{ lb/in}^2$$

c = distance from neutral axis to outer fiber = 1.7 in

$$\frac{I}{c} = \frac{810}{110,000} = 7.36 \times 10^{-3}$$

Therefore,

$$I = 7.36 \times 10^{-3} (1.6) = 1.18 \times 10^{-2} \frac{\text{in}^4}{\text{in}}$$

Thus, a little ring stiffness makes a big difference in the stability of a flexible ring on a sloped trench wall (sidehill).

For most pipes, the ring stiffness required for installation is adequate if it complies with the familiar rule of thumb according to which minimum cover is $D/2$. For a pipe parallel to a sloped trench wall in cohesionless soil, *minimum cover is one-half a diameter to the sloped surface of the trench wall.*

Analytical Methods for Predicting Performance of Buried Flexible Pipes

Introduction

There are various methods for predicting the structural behavior of flexible conduits. Included here is an in-depth analysis of the various methods, pointing out strengths and weaknesses with emphasis on large-diameter profile-wall HDPE pipes. Comparisons of test data with predictions from the various theoretical methods are made. Methods discussed include (1) full-scale testing, (2) semiempirical equations (such as the Iowa formula), (3) closed-form analytical method (such as the Burns and Richard elastic solution), (4) finite element methods, and (5) model testing (with dimensional analysis).

The Burns and Richard solution and the Iowa formula are both linear elastic theories. Both assume the soil and the pipe structure to be linear elastic materials. The assumption that the soil is elastic can lead to large errors. The Burns and Richard solution allows for a nonlinear soil modulus correction to account for overburden pressure. With the same soil modulus or the same modulus correction, these two methods are shown to produce almost identical results. For large-diameter PE pipes, the Burns and Richard method, although still in error, offers some advantages over the Iowa formula. It produces results such as strain, horizontal deflection, and thrust that are not directly available from the Iowa formula. The presently used soil modulus correction in the Burns and Richard solution is shown to be incorrect.