Chapter 7 - Structures

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7.1 Introduction to Soil-Structure Interaction for Thermoplastic Pipe Design

This section describes the main concepts behind the design of buried corrugated-wall thermoplastic pipe, including the interaction of the pipe with surrounding soil and the conceptual behavior of the pipe material. The pipe behavior forms the basis for the calculation of design loads (demand), and the soil-structure interaction (SSI) forms the basis for calculation of structural design capacity (resistance), which will be discussed in subsequent sections.

7.1.1 Buried Thermoplastic Pipe Behavior and Vertical Arching

Thermoplastic pipes are considered as flexible pipe, and they are designed to deflect and develop lateral soil support to carry external loads. Figure 7.1 shows that as a buried flexible pipe is subjected to vertical load, the vertical diameter decreases and the horizontal diameter increases. As the horizontal diameter increases, the supporting soil provides passive pressure resistance which prevents excessive deflection of the pipe and helps to carry the vertical load. As the pipe deflects, the vertical loads then arch around the pipe and are carried through the relatively stiffer soil (based on a typical good quality installation). This concept is known as vertical arching. The deflection of the pipe, and forces in the pipe wall, depend more on the stiffness of the surrounding soil than on properties of the pipe itself.

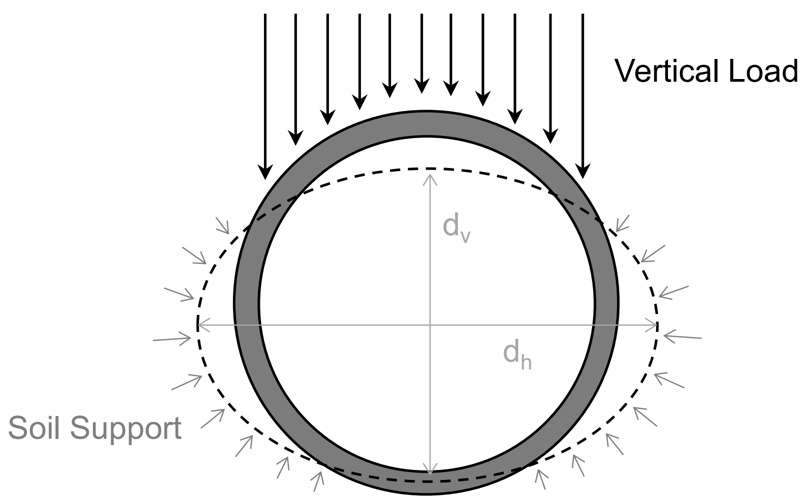

Figure 7.1: Deflection of a flexible pipe under vertical soil load

As seen in Figure 7.2, the pipe and surrounding soil can be thought of as parallel vertical springs. For thermoplastic pipe embedded in compacted structural backfill, the surrounding soil on each side of the pipe is stiffer than the pipe itself, thereby attracting the load. The vertical arching factor is used to estimate the reduction in vertical load in a flexible pipe, as related to the soil prism load introduced later in Section 7.4.

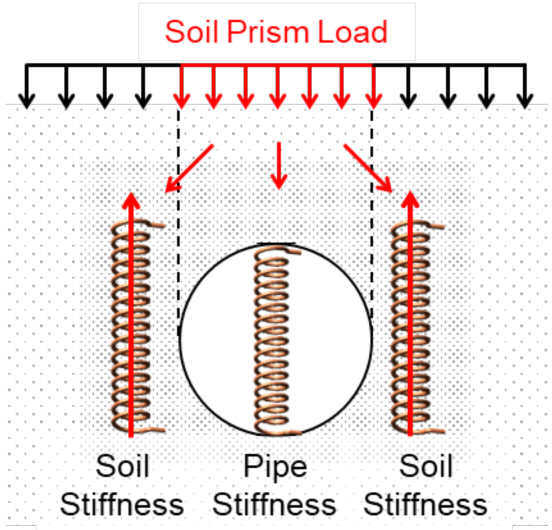

Figure 7.2: Schematic illustration of vertical arching

The theoretical solutions developed by Burns and Richard (1964) can be used to calculate the vertical arching factor (VAF) based on the hoop stiffness factor and the soil prism load. The dimensionless hoop stiffness factor is a ratio of the soil lateral stiffness to the pipe corrugation hoop stiffness (which will be introduced in Section 7.5). Burns and Richard assumed both full-slip and no-slip conditions between the pipe and the surrounding soil. It should be noted that these solutions are mathematically complex and do not account for variations in installation conditions.

Research by McGrath (1999) compared the VAF from many specific installation cases simulated by extensive finite element analysis (FEA) to the Burns and Richard solutions. The FEA models were designed to capture variations in installation conditions to evaluate pipe deflection and load in the pipe, relative to soil prism load. The FEA results were validated through comparison to field test data. Figure 7.3 presents the plots of the VAF from the FEA and from the theoretical solutions for a range of hoop stiffness factors. The range of hoop stiffness factors for typical corrugated wall thermoplastic pipe embedded in compacted structural backfill is shown in the shaded area of the plot.

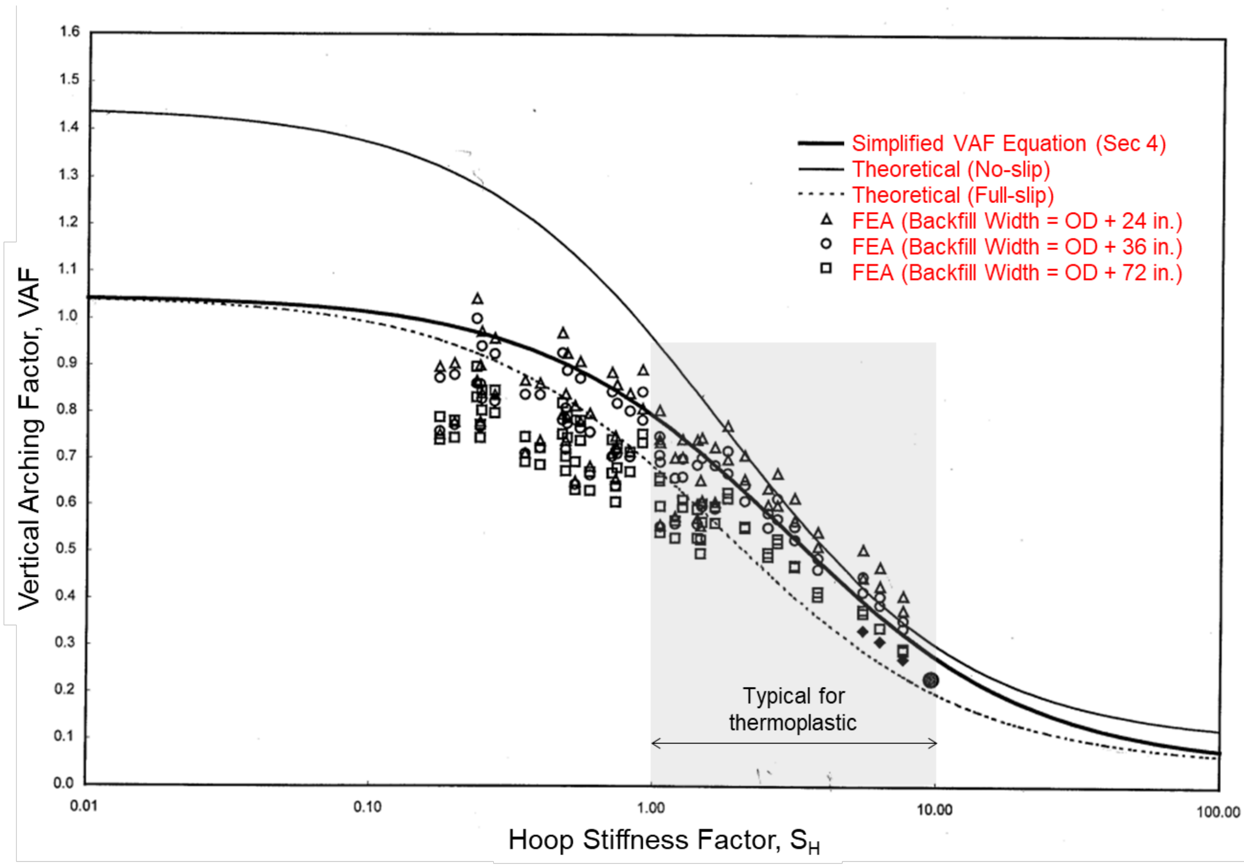

Figure 7.3: Vertical arching factor and hoop stiffness factor for various slip and backfill conditions

Based on the McGrath FEA models, the Burns and Richard theoretical solutions, and the field test results, a simplified equation was developed for VAF based on the hoop stiffness factor (discussed in Section 7.4). This equation is included in the AASHTO LRFD Bridge Design Specifications (2014) thermoplastic pipe design method, hereafter referred to as AASHTO LRFD. The VAF determines the portion of the vertical soil prism load carried by the pipe and is less than 1.0 for corrugated plastic pipe.

Pipe Wall Forces

External soil load on buried pipe results in thrust (i.e., hoop compression) in the wall of the pipe and bending deflection. The magnitude of thrust is the lowest at the top and bottom of the pipe and the greatest at the springline, due to additional vertical load that enters the pipe through the shoulders and through friction. Thrust is carried from the top of the pipe to the springline, and then is transferred back to the supporting soil at the haunch and invert. The external vertical and lateral soil load, and the resulting thrust distribution around the pipe, are shown schematically in Figure 7.4.

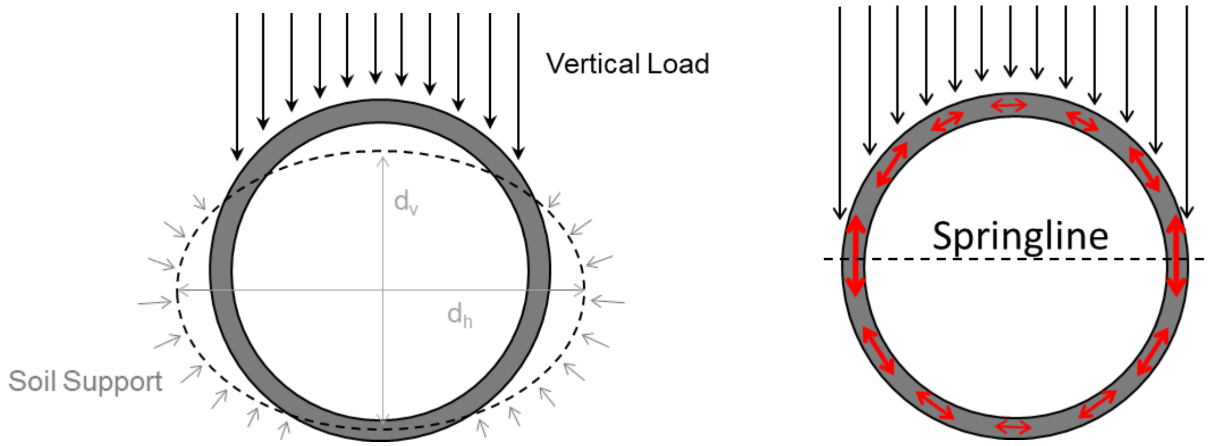

Figure 7.4: Soil loads and thrust distribution in pipe

In HDPE and PP pipe, hoop compression around the circumference of the pipe, and the low hoop stiffness of the corrugation, lead to a reduction in the overall pipe circumference (i.e., circumferential shortening). Vertical deflection results from circumferential shortening and bending deflection (i.e., flexure), which is also known as ovalling of the pipe shape. Bending deflection leads to a flattening of the curvature at the top and bottom of the pipe and to an increased curvature at the springline. This change in shape generates a positive bending moment at the top and bottom of the pipe and a negative bending moment at the springline. The positive bending moment induces compression at the outside surface of the pipe and tension at the inside surface of the pipe. This flexure force distribution around the pipe is shown schematically in Figure 7.5.

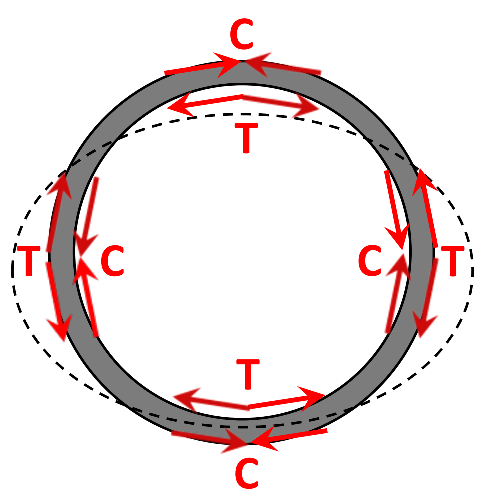

Figure 7.5: Typical flexure force distribution around pipe from vertical deflection (T=tension, C=compression)

Flexure force and thrust demands occur simultaneously and then combine, as shown in Figure 7.6. If flexure forces have a higher magnitude than the thrust forces, the pipe wall will have tension on one surface and compression on the other surface. This may occur with shallow fill depths and live loads. If flexural force effects have a smaller magnitude than the thrust forces, the pipe wall will have compression throughout the cross-section for the entire circumference. This condition is typical for deeper fill installations.

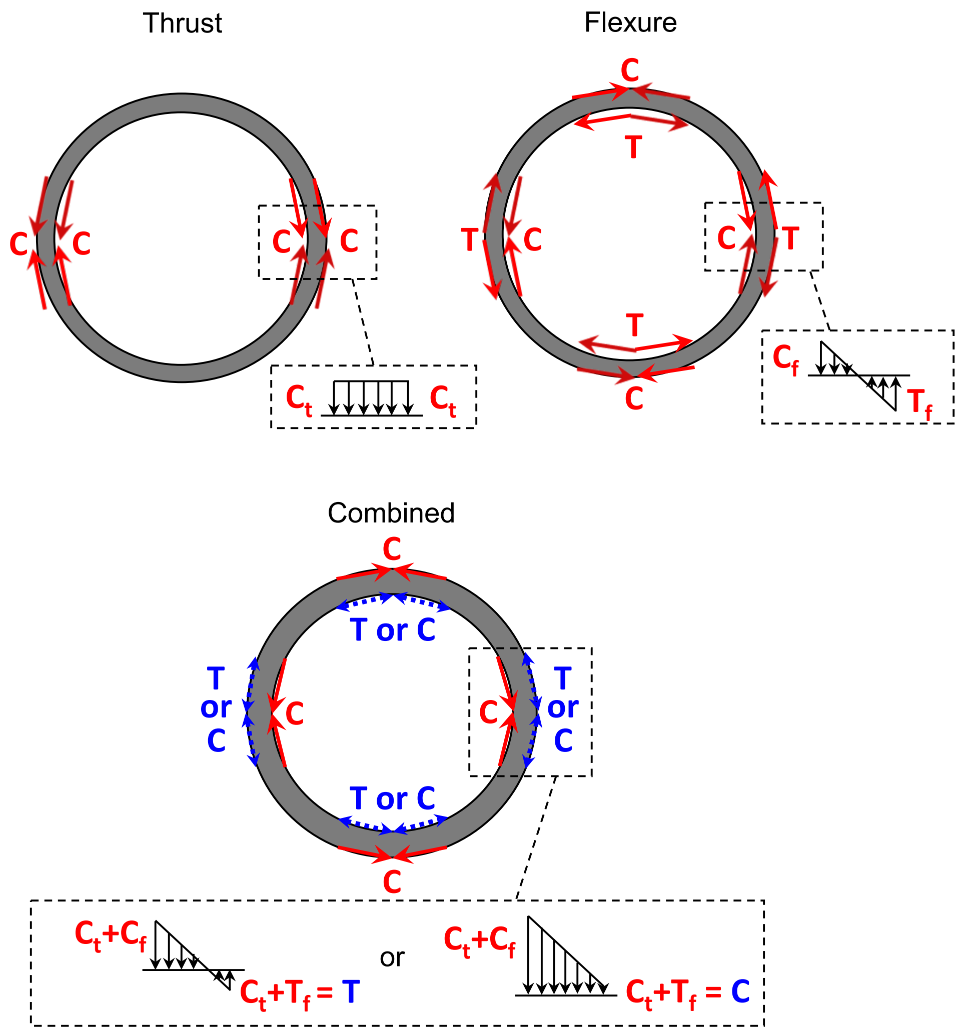

Figure 7.6: Combined thrust and flexural force demands

Compression in the pipe wall may also lead to local or global buckling as shown in the schematic in Figure 7.7. Local buckling will occur if individual elements of the pipe wall profile are overloaded. The capacity to resist local buckling is based on the corrugation element slenderness and the pipe material properties. Local bucking is accounted for in design by using the corrugation effective area concept. Global buckling can occur across the full wall thickness (i.e., corrugation depth) based on the thrust demand in the wall. The capacity to resist global buckling is based on the moment of inertia of the pipe wall and the stiffness of the supporting embedment soil. The design method includes checks to prevent global buckling.

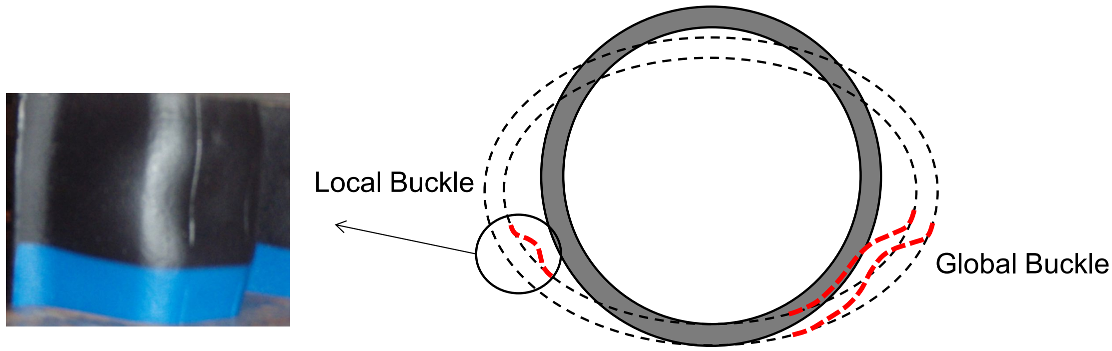

Figure 7.7: Local and global buckling of pipe wall

The strength limit states design the pipe wall for thrust, combined thrust and flexure, and buckling.

Deflection

Deflection measurement should be used to verify the performance of in-service buried flexible pipe. Deflection is defined as the change in vertical diameter of the pipe as a percentage of the undeformed (unloaded or nominal) inside diameter of the pipe. Flexible pipes are designed to deflect in order to mobilize the passive resistance of the surrounding soil. The largest amount of deflection typically occurs during installation. If the deflection during installation is monitored and controlled to be within the established limits after construction is complete, the pipe will perform as designed. If deflections exceed the established limits, the force distributions in the pipe will be different than the design assumptions. These unanticipated forces could lead to a reduced safety factor, a reduced service life, or even failure.

For example, if a soil of poor quality or with inadequate compaction is used to embed the pipe, the passive resistance provided by the soil will be reduced and will result in increased deflections in the pipe. Increased deflections will in turn increase the flexural forces. Additionally, the reduced soil stiffness will result in the pipe carrying more load by leading to a reduction in soil arching (or, increased VAF).

The pipe is checked in the service limit state for deflections. The total predicted service deflection of the pipe is checked against an allowable deflection limit. While certain jurisdictions may have different deflection limits, AASHTO LRFD limits the deflections to 5% of the inside diameter. The AASHTO LRFD Bridge Construction Specifications (2016) also require an evaluation by a professional engineer if measured deflections in the field exceed 5%. If the deflections exceed 7.5% of the inside diameter, the pipe is required to be remediated or replaced. A value other than 5% of the inside diameter may be used as the deflection limit where permitted for certain projects, if the pipe is shown to meet all of the strength limit state design requirements and any relevant product or material test requirements.

While deflection can serve as a measure of pipe performance, it should be noted that not all poor installations will exhibit deflections in excess of 5% of the inside diameter. For shallow burial installations, the majority of the deflections may be generated by live loads, and it is likely the conditions under which the deflections were measured did not include live loads.

7.1.2 Strain-Based Design

As described in Chapter 2, thermoplastics are viscoelastic materials. For stresses and strains that are maintained in the material's linear viscoelastic range, the material behavior is assumed to be governed by a material creep (or, relaxation) modulus, also known as an apparent modulus. This modulus depends only on the duration of the load and not on the magnitude of load. This allows strain demands from loads of very different durations to be combined, based on the Boltzmann Superposition Principle. Since the failure strain is independent of the load rate, thermoplastic structures in the viscoelastic range can be designed for a limiting strain. When combining demands for loadings of different durations, the stress due to each load is divided by the appropriate material creep modulus for the load duration and then the resulting strains are combined and compared to the material-specific strain capacity.

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7.2 Corrugated Thermoplastic Pipe Material, Testing, and Geometric Properties for Structural Design

7.2.1 Pipe Material Properties for Design

Material properties for structural design of buried thermoplastic pipe include creep modulus, tensile strength, and tension and compression strain limits. These properties describe the performance of the pipe material and are independent of pipe geometry. Material properties may be determined by physical testing, provided by the manufacturer, or can be taken from published values based on the cell class of the resin used for the pipe. Chapter 2 provided additional information on HDPE and PP material characteristics.

Creep (Apparent) Modulus

For pipe structural design, the modulus of the pipe material must reflect the duration of the load being analyzed. Dead load effects are evaluated using a long-term creep modulus, E_{lt}, corresponding to the specified design life of the pipe, which is typically 50, 75, or 100 years. The effects of passing vehicle live loads are evaluated using an initial short-term elastic modulus, E_{st}. For loads of other durations (e.g., long-term parked vehicle), an appropriate short-term creep modulus matching the duration of the load may be determined from creep test results or may be provided by the manufacturer. Table 7.1 presents creep modulus values for use in the design of corrugated HDPE and PP pipes which correspond to short-term passing vehicle live loads and long-term soil dead loads.

Table 7.1: Typical design values for corrugated thermoplastic pipe creep modulus

Material	Pipe Specification	Minimum Cell Class(2)	Modulus, ksi (MPa)			
			Short-Term, E_{st}	Long-Term, E_{lt}		
			Initial	50-Yr	75-Yr	100-Yr
HDPE	AASHTO M 294 (2015), ASTM F2306 (2015)	ASTM D3350 (2014) 435400C	110 (758)	22 (151)	21 (144)	20 (138)
PP	AASHTO M 330 (2013), ASTM F2881 (2015)	See requirements in AASHTO M 330 (2013)	175 (1206)	29 (200)	28 (193)	27 (186)

The values shown in Table 7.1 are from AASHTO LRFD for the short-term modulus through the 75-year creep modulus. The 100-year creep modulus for HDPE pipe is based on round robin laboratory creep test results on HDPE pipe samples, reported by Hsuan (2012), using the Stepped Isothermal Method (SIM) in ASTM D6992 (2015). The 100-year creep modulus for PP pipe is based on SIM test results on PP pipe resins reported by Bass et al (2012).

Table 7.1 modulus values are for the standard reference temperature of 73 deg. F (23 deg. C) and all long-term values were obtained from creep testing at a 500-psi load (3447 kPa), which is assumed to be within the material linear viscoelastic range and is established as an upper bound service stress for typical installations. Long-term sustained stresses greater than 500 psi (3447 kPa) may be outside of the material's linear viscoelastic range and would require project-specific materials testing for design. Also, if anticipated service temperatures deviate significantly from 73 deg. F (23 deg. C), the manufacturer should be consulted for any potential effects on modulus values.

Design Strength

The pipe material strength or design strength is defined as the minimum tensile yield stress that corresponds to physical testing at the duration or simulated duration of the design load. Table 7.2 presents the minimum design strengths for corrugated thermoplastic pipes that correspond to short-term duration live loads and long-term duration sustained dead loads.

Table 7.2: Design values for corrugated thermoplastic pipe material tensile strength

Material	Pipe Specification	Minimum Cell Class	Design Strength, F_y, ksi (MPa)			
			Short-Term	Long-Term		
			Initial	50-Yr	75-Yr	100-Yr
HDPE	AASHTO M 294 (2015), ASTM F2306(2015)	ASTM D3350 (2014) 435400C	3.0 (20)	0.9 (6.2)	0.9 (6.2)	0.8 (5.5)
PP	AASHTO M 330 (2013), ASTM F2881 (2015)	See requirements in AASHTO M 330 (2013)	3.5 (24)	1.0 (6.9)	1.0 (6.9)	1.0 (6.9)

The material strength values in Table 7.2 includes the short-term strength through the 75-year strength from the AASHTO LRFD, while the 100-year values are as recommended by Hsuan for HDPE (2012) and Bass et al. for PP (2012).

Strain Limits

The thermoplastic strain limit is the minimum specified yield strain of the material, identified as the strain at which stress-strain curves begin to show significant nonlinearity, as described in NCHRP Report 631 (McGrath et al. 2009). HDPE and PP thermoplastics are ductile materials, with relatively flat post-yield stress-strain curves, as discussed in Chapter 2. This property can make the selection of strain limits from test results somewhat challenging. NCHRP 631 suggested and AASHTO LRFD adopted fixed strain limits for each thermoplastic material. The fixed strain limits are provided in Table 7.3 for the design of corrugated thermoplastic pipes.

Table 7.3: Strain limits for corrugated thermoplastic pipe design

Material	Pipe Specification	Minimum Cell Class	Strain Limit (%)	
			Tension, ε_{yt}	Compression, ε_{yc}
HDPE	AASHTO M 294 (2015), ASTM F2306 (2015)	ASTM D3350 435400C (2014)	5.0	4.1
PP	AASHTO M 330 (2013), ASTM F2881 (2015)	See requirements in AASHTO M 330 (2013)	2.5	3.7

7.2.2 Finished Pipe Laboratory Tests Related to Structural Design

Pipe Stiffness

Pipe stiffness is used to determine the shape factor for calculating flexural strain demand in the structural design process. The pipe stiffness can be theoretically calculated or can be determined by physical testing in the parallel plate test. Product standards typically establish minimum values for pipe stiffness.

Pipe stiffness, PS, is theoretically calculated according to Eqn. 7.1.

$$PS = \frac{E_{st} I_p}{0.149 R^3} \qquad \text{(Eqn. 7.1)}$$

where:

PS = theoretical pipe stiffness, psi (kPa);
E_{st} = short-term (elastic) modulus of pipe material, psi (kPa);
I_p = moment of inertia of pipe wall, in.^{4}/in. (cm^4/cm); and,
R = radius of pipe to centroid of wall, in. (mm).

ASTM D2412 (2011) is the standard method for the parallel plate test. In the test, a ring of pipe is placed between two rigid parallel plates. The two plates are forced toward each other at a constant rate of displacement, and the applied force and displacement are recorded. The pipe stiffness is calculated as the force per unit length of the pipe specimen divided by the vertical displacement at 5% deflection for thermoplastic pipes, typically specified in pounds per inch per inch (psi). This test is used to demonstrate the finished product performance and can be used to confirm that theoretical design properties are physically achieved in the manufactured pipe.

Stub Compression Test

AASHTO T 341 (2014) is the standard method for the stub compression test. In the test, a short section (stub) of the pipe wall is compressed between two rigid plates at a controlled rate of approach. The specimen is three corrugation periods long in the pipe longitudinal direction and has a chord length equal to 1.5 times the corrugation depth. The specimen is oriented such that the plates apply compressive load to the corrugation in its hoop (axial) direction to simulate pure thrust in the pipe wall. The load-bearing ends of the specimen are machined to be parallel to evenly distribute the applied load and the displacement and applied load are recorded. The maximum load achieved in the test divided by the specimen length in the pipe's longitudinal direction, is calculated to be the stub compression capacity, P_{st}.

The stub compression test is used to evaluate failure strain, to characterize the susceptibility to local buckling (i.e., profile compression capacity), and can be used to estimate the corrugation effective area. The stub compression test is used as a finished product wall design qualification test in product standards.

7.2.3 Corrugated Pipe Geometric Properties for Design

The following sections provide corrugated wall terminology and section properties used in design.

Terminology

Figure 7.8 presents some of the standard terminology for round corrugated pipe geometry, illustrated for a triple wall pipe.

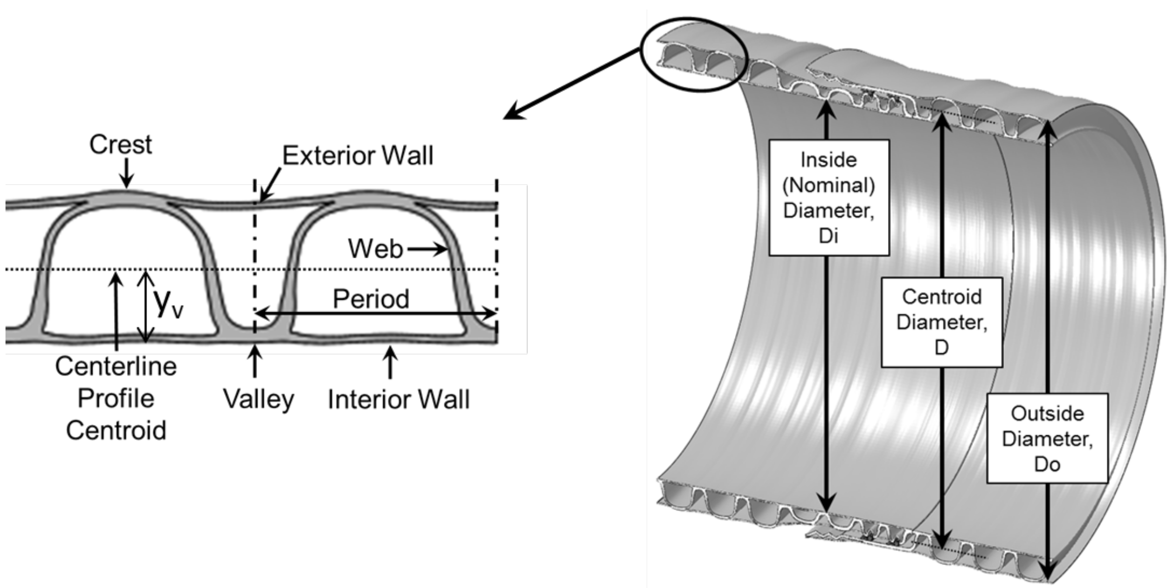

Figure 7.8: Pipe geometric terminology

The valley is the primary element at the interior surface of the corrugation that spans between two web elements. The crest is the primary element at the exterior surface of the corrugation spanning between two web elements. The web elements connect the valley and crest and have a primary orientation nearly perpendicular to the interior and exterior walls. The corrugation period or pitch is the length equal to one repetition of the corrugation, typically taken from the centerline of the valley to the next centerline of valley. The centroid of the profile is at the center of mass of the corrugated wall.

Dual-wall pipes include an interior wall or liner which is typically thinner than the valley and spans between the two webs at the interior surface of the pipe, opposite the crest. Triple wall pipes include both an interior and exterior wall, where the exterior wall is located at the outside surface of the pipe opposite the valley, and is usually thinner than the crest. The exterior wall spans between the two junctions of the crests and webs or between two crests.

The inside diameter, D_i, is the minimum distance between two diametrically opposite points on the inside surfaces of the valley or inner wall. The nominal inside diameter of the pipe, used for reference purposes, is typically the inside diameter rounded to the nearest whole number. The outside diameter, D_o, is the maximum distance between the outside surfaces of the crest or outer wall elements diametrically across the pipe. The centroid diameter, D, is the diameter at the pipe corrugation geometric centroid and is determined from the geometric centroid of one corrugation period, calculated as $D_i + 2y_v$.

Determining Pipe and Corrugation Geometries

Pipe dimensions and corrugation geometric properties can be calculated from measurements on cut sections of the pipe or can be based on manufacturer mold designs. The pipe cross-sections that are considered in design should be representative of the final manufactured part.

The gross area and gross moment of inertia of one period of the corrugated wall are divided by the period length to normalize them per unit length of pipe. The area and moment of inertia may be determined from cut sections of pipe or estimated from the manufacturing mold dimensions.

To determine the local buckling effective area by calculation, the geometries of individual corrugation elements (i.e., the valley, crest, webs, and interior and exterior walls, if present) are required for element idealization. The idealized elements are then used to calculate the corrugation effective area and the capacity of the corrugation to resist both thrust and combined thrust and flexure.

Corrugation Effective Area

The primary structural behavior of concern for design of buried corrugated thermoplastic pipes is thrust, which represents compressive stress acting on the corrugation wall. The resistance of corrugation wall elements to thrust is controlled by local buckling, which requires determination of corrugation effective area. The effective area is a reduction of the gross area to account for the center (unsupported) sections of corrugation elements that become ineffective from local buckling when under compressive load. Even though local buckling is initiated at the center of the corrugation element, effective area theory assumes that ends of elements, where other elements adjoin, provide sufficient strength to carry the compressive load (AISC) to the design failure strain. This concept is illustrated in Figure 7.9.

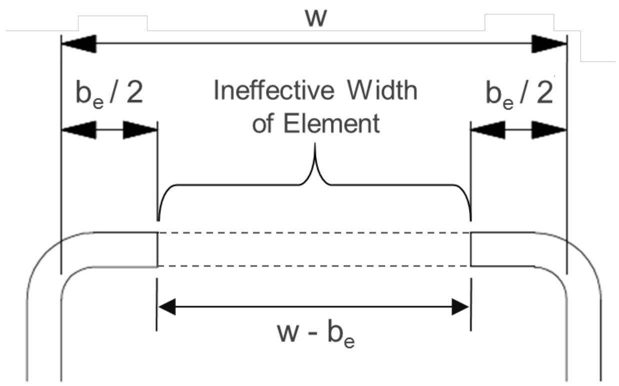

Figure 7.9: Effective width (b_e) and ineffective width (w - b_e) of corrugation element

Slender elements with high width-to-thickness ratios are more likely to buckle than stout elements with low width-to-thickness ratios. The compression force acting on the effective area results in greater strain in the remaining effective section than if the same force were acting on the gross area, effectively reducing the capacity of the pipe to account for local buckling.

The effective area of a corrugation profile is calculated theoretically and may be verified on manufactured pipe using the stub compression test. Determining the effective area by theoretical calculations requires idealization of the corrugation cross-sectional elements.

The effective area calculations are based on methodology that was originally developed for cold-formed steel and then adapted to corrugated thermoplastic pipe wall sections in AASHTO LRFD (2014). The approach determines a clear width and thickness of each element in the section, calculates an ineffective width for each element, sums the total ineffective area across all elements in the section, then subtracts the ineffective area from the gross area in order to give the effective area. Pipe corrugations are idealized into a series of straight elements of uniform thickness with clear widths representative of the physical corrugation to perform the calculations.

The effective area is calculated according to Eqn. 7.2.

$$A_{eff} = A_g - \frac{\sum(w_i - b_{e,i})t_i}{\omega} \qquad \text{(Eqn. 7.2)}$$

where:

A_{eff} = corrugation effective area per unit length of pipe for local buckling resistance in.^{2}/in. (cm^2/cm);

A_g = corrugation gross area per unit length of pipe,in.^{2}/in. (cm^2/cm);

w_i = clear width between adjoining elements of corrugation element i, in. (mm);

$b_{e,I}$ = effective width of corrugation element, i, calculated according to Eqn. 7.4 in. (mm);

t_i = thickness of corrugation element, i in. (mm); and,

ω = period of corrugation in. (mm).

$$b_{e,i} = \rho_i w_i \quad \text{(Eqn. 7.3)}$$

where:

ρ_i = effective width factor of corrugation element, i, calculated according to Eqn. 7.4.

$$\rho_i = \frac{1 - \frac{0.22}{\lambda_i}}{\lambda_i} \leq 1 \quad \text{(Eqn. 7.4)}$$

where:

λ_I = slenderness factor of corrugation element, i, calculated according to Eqn. 7.5.

$$\lambda_i = \left(\frac{w_i}{t_i}\right)\sqrt{\frac{\varepsilon_{yc}}{k_i}} \geq 0.673 \quad \text{(Eqn. 7.5)}$$

where:

ε_{yc} = pipe material compression strain limit, in./in. (cm/cm), and

k_i = plate buckling edge support coefficient for corrugation element, i.

For elements supported by an adjoining element at both ends, k_i = 4. For elements supported by an adjoining element at only one end (e.g., free-standing ribs), k_i = 0.43. For elements with stiffeners, *k*i should be calculated as described herein.

Corrugation Section Idealization

To determine corrugation effective area by calculation, the corrugation geometry is idealized into a series of flat plate elements following the guidance described in this section.

General Requirements for Determining Element Properties

A single corrugation period is divided into a trapezoidal shape with crest, valley, and web elements of constant thickness as shown in Figure 7.10. The interior and exterior wall elements are included if present in the pipe wall. Idealization methods are based upon procedures provided in the AASHTO LRFD (2014) and updated in Bass and Beaver (2018).

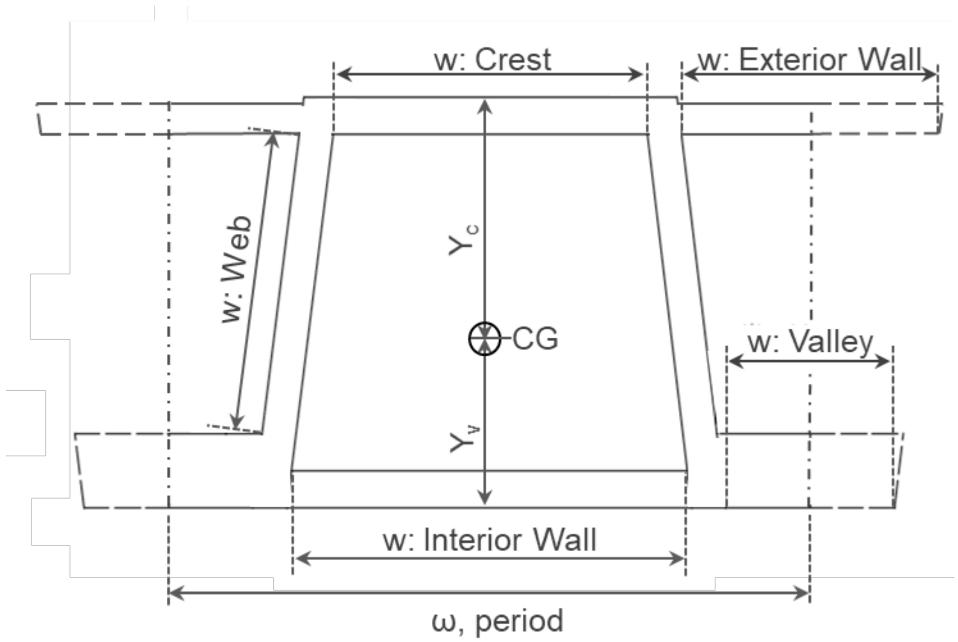

Figure 7.10: Example idealization showing element clear widths

Idealization of Box-Shaped Corrugation

The clear width of each element, *w*, is set as the clear distance between adjoining elements in the idealization. The thickness of each idealized element is set as the thickness at the center of the non-idealized element from the gross cross-section. If the thickness varies by more than 10% along the width of the element, the minimum thickness should be used. The thickness of curved elements may be increased to account for curvature.

The following procedure is provided to consistently determine the clear widths. The procedure is illustrated for the typical dual-wall pipe included in Figure 7.11:

1. Set web alignment in Figure 7.11 (a) - Set the inside surface of the web to intersect the center of the arc between the web and crest (i1) and the center of the arc between the web and valley (i2);
2. Set web clear width in Figure 7.11 (b) - Extend the ends of the inside surface of the web so they intersect the (horizontally extended) inside surface of the crest (i3) and the (horizontally extended) inside surface of the valley (i4). Web clear length is measured between (i3) and (i4);
3. Set web area in Figure 7.11 (b) - Offset the inside surface of the web by the thickness of the nominal web (tw), trim or extend offset line accordingly to meet the inside surfaces of the crest and valley;
4. Set other element clear widths in Figure 7.11 (c) - Set the crest and valley idealized element widths such that they touch the corners of the webs. Set interior wall width so it touches the valley element. When present, set exterior wall width so it touches the crest element. Set all element thicknesses as the thickness at the center of the nominal elements; and,
5. Idealized cross-section elements in Figure 7.11 (d) – Note that the full idealized area is not calculated because it is not used in the effective area calculations.

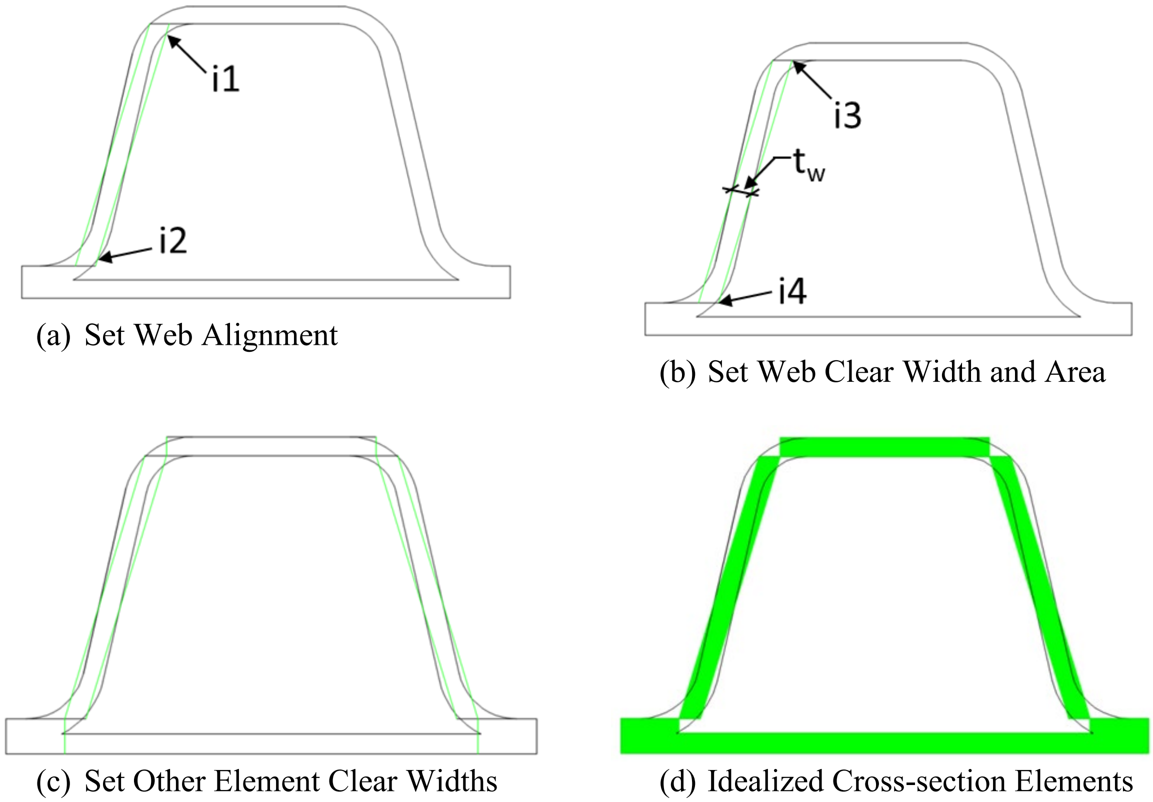

Figure 7.11: Determine cross-section element clear widths and areas (a) web alignment, (b) web clear width, (c) other element clear widths, (d) idealized cross-section

This method determines the element clear widths and thicknesses, but does not calculate an overall idealized area. The idealized area is not used in the calculations to determine effective area for structural design. The element clear widths and thickness are the critical dimensions in an idealization and should be as representative of the gross cross-section as possible. The effective area calculations are formulated such that the total ineffective area of all elements is subtracted from the gross area of the cross-section. This means that the lack of continuity at the corners and between idealized elements does not impact the effective area calculations.

Idealization of Corrugation with Curved Elements

Figure 7.12 illustrates the element idealization for a corrugation cross-section with a curved crest element.

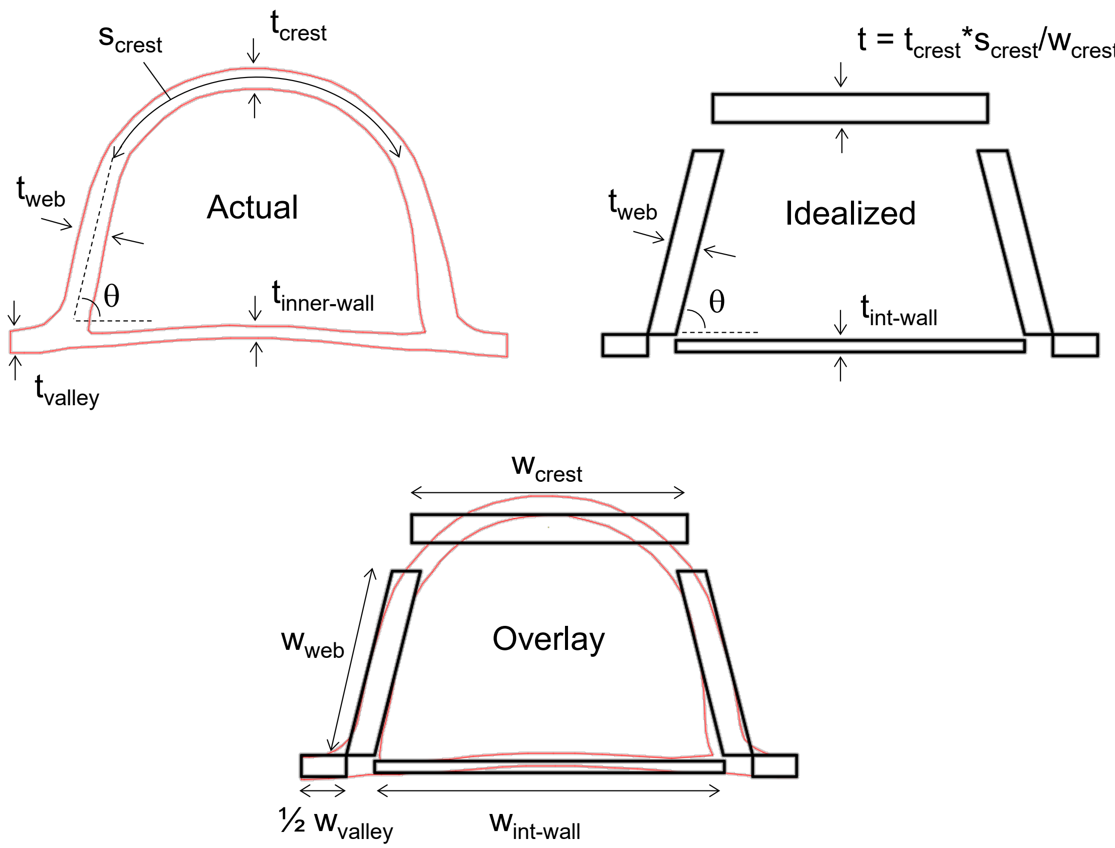

Figure 7.12: Example idealization of corrugation with curved crest

The inside (bottom on Figure 7.12) end of the web is set using the same procedure as described for the box-shaped corrugation. The outside (top in Figure 7.12) end of the web is set where the crest curve starts.

Finite element analysis (Bass and Beaver 2018) has shown that curved elements, such as the crest element in Figure 7.12, have greater local buckling capacity than flat (straight) elements of the same thickness through the same two end points as the curved elements. To account for the additional capacity from the curvature, the thickness of the idealized element is increased such that the area of the idealized crest element matches the area of the full curved element, as shown in the upper right of Figure 7.12. This is a conservative approximation to account for the greater local buckling capacity of the curved element.

The clear width of the curved element should be taken as a straight distance between the ends of the curve rather than the arc length.

Stiffener Evaluation

Some corrugation elements may have intermediate stiffeners, such as the crest of the corrugation profile shown in Figure 7.13. Stiffeners, if present, are intended to add local buckling capacity to wide thin elements.

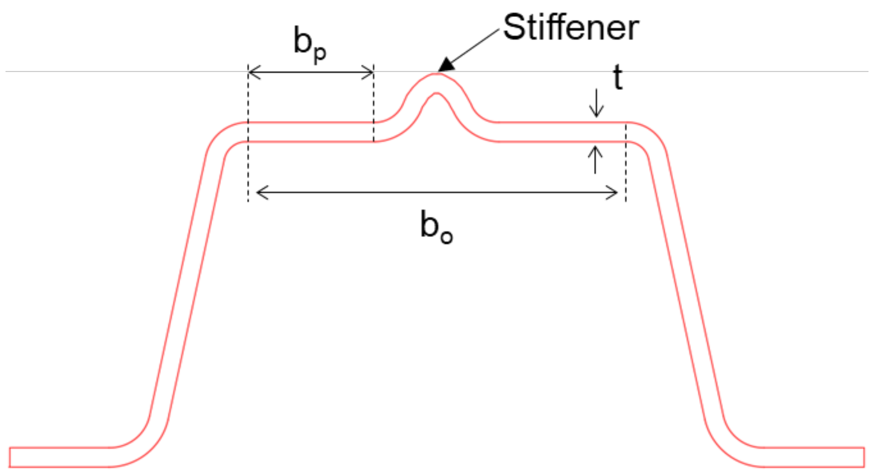

Figure 7.13: Corrugation stiffener evaluation

The effectiveness of stiffeners on parent corrugation elements has traditionally been ignored in thermoplastic pipes design because no published method for design originally existed. The AISI Cold Formed Steel Design Specification (2007) first presented a method to quantify the effectiveness of the stiffener to brace the larger parent corrugation element to resist local buckling.

The stiffener's effectiveness is evaluated by determining a plate buckling edge support coefficient, k, of the stiffener based on its geometry and that of its parent element. This k-value can be input directly into the effective area calculations provided in Eqns. 7.2 to 7.5. A fully effective stiffener will provide a simply supported condition ($k = 4$) to the two sub-elements adjacent to the stiffener, dividing the full width, b_o, of the element into two smaller sub-elements each with width, b_p. This is similar to having the stiffener act as an intersecting element, such as a web element. A fully ineffective (or non-existent) stiffener will not provide restraint to the parent element, and the parent element will resist local buckling based on its full element width, b_o. The full stiffener evaluation and associated equations may be found in the latest edition of ASIC Cold Formed Steel Codes.

Stiffeners with sharp edges or abrupt changes in curvature should be evaluated for susceptibility to stress cracking.

Effective Area by Stub Compression Test

As an alternate method to determining the effective area by section idealization and effective width calculations, the results of stub compression tests can be used to approximate the effective area.

The effective area from stub compression test results is calculated according to Eqn. 7.6.

$$A_{eff} = \frac{P_{st} K_t}{F_y} \leq A_g \qquad \text{(Eqn. 7.6)}$$

where:

A_{eff} = corrugation effective area per unit length of pipe for local buckling resistance, in.2/in. (cm2/cm);
P_{st} = stub compression test capacity, lbf/in. (kgm/cm);
K_t = time factor to account for design load duration from Table 7.4;
F_y = material compression yield strength for design load duration, psi (kPa); and,
A_g = corrugation gross area per unit length of pipe, in.^{2}/in. (cm^2/cm).

Table 7.4: Time factor, K_t, for effective area from stub compression tests

Time Period	Time Factor, K_t
Short-Term, Initial	0.9
50 Year	0.3
75 Year	0.25

7.3 Soil Properties

The accurate classification of soils which will surround buried thermoplastic pipes is instrumental to understanding both the demand loads and capacity of the buried pipe soil-structure interaction system. For buried flexible thermoplastic pipes, the surrounding (i.e., embedment) soil is stiffer than the pipe, which promotes vertical arching (i.e., reduction of load carried by the pipe) through soil-structure interaction. The decrease in the load on the pipe (as related to demand) and the level of soil support for the pipe (as related to capacity) both depend on the soil stiffness. Soil stiffness varies with the embedment type, native soil classification, degree of compaction, trench width, and fill depth.

Soil properties used in design must be representative of the actual soils used in installation. Embedment soil type and compaction, from the bottom of the pipe bedding to the top of initial backfill, are typically selected during the pipe's structural design phase and specified for installation. The quality and stiffness of native soils outside the trench may or may not be known. In cases in which low stiffness soils outside the embedment zone may influence pipe behavior, it is necessary to include their effect in design through use of a composite constrained modulus. This calculation may require an increase in trench width and in structural backfill width.

7.3.1 Site Soil Exploration

Native soil characteristics are generally determined through site soils exploration prior to the design phase of the project. Typical information gathered during site soils exploration includes elevations of various strata and the groundwater table (if present), blow counts to advance a hammer through the strata (i.e. standard penetration test), classification of excavated soils based on the Unified Soils Classification System (USCS), and estimates of shear or unconfined compressive strengths from vane shear or pocket penetrometers. Laboratory test results of samples collected during site soil exploration are also typically provided.

In the absence of project-specific data, available data from nearby sites should be used along with the consultation of a qualified geotechnical engineer familiar with local conditions. In addition, one can reference published data that identify native soils regions.

7.3.2 Embedment Soil Classes

ASTM D2321 (2014) defines soil classes specific to thermoplastic pipe installations based on the USCS classification, plus some additional gradation or other material requirements. The ASTM D2321 thermoplastic pipe soil classes are provided in Table 9.1 along with the USCS descriptions and their relationship to AASTHO M 145 (1995) soil groups and the AASHTO M 43 (2005) or ASTM D448 (2012) standard aggregate sizes based on grain size distribution.

General descriptions of soil classes for thermoplastic pipe installation include the following five classes, and their detailed requirements are provided in Chapter 9.

- Class I: crushed rock with little or no fines;
- Class II: clean, coarse-grained gravel or sand (GW, GP, SW, and SP soils);

- Class III: coarse-grained soil with fines (GM, GC, SM, SC, some CL and ML soils with low fines content);

- Class IV: fine-grained soils (CL and ML soils with ≥ 30% fines); and,

- Class V: unsuitable fines (MH, CH, OL, OH, and PT soils that pass the #200 sieve).

7.3.3 Constrained Modulus

The soil stiffness parameter used in buried thermoplastic pipe structural design is the one-dimensional constrained modulus, M_s. The constrained modulus is the soil stiffness determined in uniaxial strain testing on a sample of soil compacted to the field-specified density. Constrained modulus is computed as the slope of the secant from the origin to the stress level on the curve that represents the free field vertical soil stress at the elevation of the installed pipe, typically taken as the springline depth.

The constrained modulus of backfill embedment materials, M_{sb}, and of native soil, M_{sn}, are described in the following sections along with a method to determine a composite constrained modulus that includes the stiffness of the embedment soil, embedment width, and stiffness of the native soil. The composite constrained modulus can be used in the design of trench or embankment installations. For pipes with a fill depth less than or equal to 10 ft (3 m), the composite constrained modulus should be representative of a width of one-half diameter each side of the pipe, but never less than 18 in. (50 cm) each side. For pipes with fill depth greater than 10 ft (3 m), the composite constrained modulus should be representative of a width of one diameter each side of the pipe. The constrained modulus of the embedment material can be used directly in design when permanent solid trench wall sheeting, designed to last for the entire service life of the pipe, is installed over the height of the embedment zone.

Constrained Modulus of Embedment Materials, M_{sb}

Constrained modulus values for selected Class I materials are provided in Table 7.5. These values are based on physical tests performed by Gemperline et al. (2011), and are provided for use in design in which the embedment consists of the type of aggregate and particle sizes given in the table.

Table 7.5: Constrained modulus of selected Class I embedment materials

<table>
<tr><th rowspan="2">Aggregate</th><th rowspan="2">Maximum Particle Size, in. (mm)</th><th colspan="2">Constrained Modulus, M_{sb}, psi (kPa)</th></tr>
<tr><th>Dumped</th><th>Compacted</th></tr>
<tr><td rowspan="2">Granite</td><td>0.75 (1.9)</td><td>7,000 (98,265)</td><td>8,500 (58,607)</td></tr>
<tr><td>1.50 (2.8)</td><td>3,500 (24,132)</td><td>5,000 (34,475)</td></tr>
<tr><td>Limestone</td><td>0.75 (1.9)</td><td>3,500 (24,132)</td><td>5,500 (37,922)</td></tr>
<tr><td>Quartzite</td><td>0.75 (1.9)</td><td>5,500 (37,922)</td><td>7,500 (51,712)</td></tr>
</table>

The constrained modulus for these crushed aggregate materials was found to be relatively constant at typical pipe burial depths; however, the modulus values did depend upon the methods of aggregate placement and compaction. When compacted, the materials exhibited higher stiffness than when they were more loosely placed or when dumped without compaction. For the purposes of Table 7.5, the term dumped refers to aggregate that is dropped into place from some height above the pipe without additional compaction, such as from an excavator bucket, front end loader, conveyor, or aggregate boom. Compacted refers to a minimum of two passes with a vibratory compactor over a maximum 12 in. (30 cm) backfill lift height.

For dumped placement of Class I material not meeting the aggregate type and particle size limits of Table 7.5, the constrained modulus of Class II material at SPD 90, or 90% of the maximum dry density from the standard Proctor test, shall be used as provided in Table 7.6 for Class II materials. For compacted Class I materials not meeting the aggregate type and particle size limits of Table 7.5, the constrained modulus of Class II material at SPD 100 from Table 7.6 shall be used.

Constrained modulus values for Class II, Class III, and Class IV materials are provided in Table 7.6, Table 7.7 and Table 7.8, respectively. The constrained modulus of these materials varies with compaction and burial depth to the pipe springline (i.e., confining pressure). The M_s values are given by both the vertical soil prism pressure, P_{sp}, and the depth of fill (assuming a unit weight of 120 pcf (1922 kg/m^3) for soil. Linear interpolation may be used between the values provided. The compaction levels are given in terms of the Standard Proctor Density test (SPD). Compaction of Class III and Class IV materials to SPD 100 is not considered reliable; as a result, a maximum value of SPD 95 should be used for design with these materials.

Table 7.6: Constrained modulus of Class II embedment

Psp, psi (kPa)	Depth of Fill Plus OD/2, ft (m) 120 pcf soil (1922kg/m^3)	Constrained Modulus, Msb psi (kPa)			
		SPD 100	SPD 95	SPD 90	SPD 85
1 (7)	1.2 (0.4)	2,350 (16202)	2,000 (13790)	1,275 (8791)	470 (3240)
5 (34)	6 (1.8)	3,450 (23787)	2,600 (17,927)	1,500 (10,392)	520 (3585)
10 (68)	12 (3.6)	4,200 (28,959)	3,000 (20,685)	1,625 (11,204)	570 (3,930)
20 (138)	24 (7.3)	5,500 (37,922)	3,450 (23,787)	1,800 (12,411)	650 (4481)
40 (275)	48 (14.6)	7,500 (51,712)	4,250 (29,303)	2,100 (14,479)	825 (5688)
60 (413)	72 (21.9)	9,300 (64,123)	5,000 (34,475)	2,500 (12,237)	1,000 (6895)

Table 7.7: Constrained modulus of Class III embedment

Psp, psi (kPa)	Depth of Fill Plus OD/2, ft (m) 120 pcf soil (1922kg/m^3)	Constrained Modulus, Msb psi (kPa)			
		SPD 100	SPD 95	SPD 90	SPD 85
1 (7)	1.2 (0.4)	N/A	1,415 (9756)	670 (4619)	360 (2482)
5 (34)	6 (1.8)		1,670 (11,514)	740 (5102)	390 (2689)
10 (68)	12 (3.6)		1,770 (12,204)	750 (5171)	400 (2758)
20 (138)	24 (7.3)		1,880 (12,962)	790 (5447)	430 (2964)
40 (275)	48 (14.6)		2,090 (14,410)	900 (6205)	510 (3516)
60 (413)	72 (21.9)		2,300 (15,858)	1,025 (7067)	600 (4137)

Table 7.8: Constrained modulus of Class IV embedment

Psp, psi (kPa)	Depth of Fill Plus OD/2, ft (m) 120 pcf soil (1922kg/m^3)	Constrained Modulus, Msb psi (kPa)			
		SPD 100	SPD 95	SPD 90	SPD 85
1 (7)	1.2 (0.4)	N/A	530 (3654)	255 (1758)	130 (896)
5 (34)	6 (1.8)		625 (4309)	320 (2206)	175 (1206)
10 (68)	12 (3.6)		690 (4757)	355 (2447)	200 (1379)
20 (138)	24 (7.3)		740 (5102)	395 (2723)	230 (1585)
40 (275)	48 (14.6)		815 (5619)	460 (3171)	285 (1965)
60 (413)	72 (21.9)		895 (6171)	525 (3619)	345 (2378)

When flowable fill, or a controlled low-strength material (CLSM), is used as embedment material, the constrained modulus should be based on the mix design. For a typical mix ratio of 1.5 sacks of cement per cubic yard of flowable fill, a constrained modulus of 25,000 psi (172,368 kPa) can be used in the absence of project-specific information.

Constrained Modulus of Native Soils, Msn

Approximate constrained modulus values for native soils are provided in Table 7.9 that are based on soil type, blow count, and unconfined compressive strength and were determined during site soil exploration. For granular soils, values are based on blow count, N, at the pipe elevation in accordance with ASTM D1586 (2011). For cohesive soils, values are based on unconfined compressive strength, qu, in accordance with ASTM D2166 (2016).

Table 7.9: Constrained modulus of native soils

Granular Soils		Cohesive Soils		Constrained Modulus, Msn, psi, (kPa)
Blow Count, N blows/ft (blows/m)	Description	Unconfined Compression Strength, qu, psi (kPa)	Description	
0 to 1 (0 to 3)	Very, very loose	0 to 0.4 (0 to 2.7)	Very, very soft	50 (344)
1 to 2 (3 to 6)	Very loose	0.4 to 0.9 (2.7 to 6.2)	Very soft	200 (1379)
2 to 4 (6 to 12)	Loose	0.9 to 1.7 (6.2 to 11.7)	Soft	700 (4826)
4 to 8 (12 to 24)	Loose	1.7 to 3.5 (11.7 to 24.1)	Medium	1,500 (10,342)
8 to 15 (24 to 45)	Slightly compact	3.5 to 7.0 (24.1 to 48.2)	Stiff	3,000 (20,684)
15 to 30 (45 to 90)	Compact	7.0 to 14.0 (48.2 to 96.5)	Very stiff	5,000 (34,473)
30 to 50 (90 to 150)	Dense	14.0 to 21.0 (96.5 to 144.8)	Hard	10,000 (68,947)
> 50 (>150)	Very dense	> 21.0 (> 144.8)	Vary hard	20,000 (137,895)

Constrained modulus for rock can be taken as 50,000 psi (344,737 kPa). For poor native soils, a geotextile wrap of the pipe zone may increase the constrained modulus values for those soils beyond those provided in Table 7.9. An evaluation of the strength increase provided by a geotextile wrap should be performed by a qualified engineer.

Composite Constrained Modulus of Soil

The composite constrained modulus of soil is used when it is necessary to consider the stiffness of the native soil outside the trench zone. It may also be used to evaluate the effect of embankment fill outside the embedment zone for installations in which the pipe embedment zone soil and embankment sidefill have different stiffnesses, due to either material or compaction.

The stiffness contribution of the native soil (i.e., the embankment soil to the sides of the embedment) is determined using the soil support combining factor, Sc. The values for Sc are provided in Table 7.10 as a function of the ratio of Msn to Msb and the ratio of trench (or embedment zone) width at the pipe springline, Bd, to the pipe's outside diameter, Do. Linear interpolation may be used between the values provided.

Table 7.10: Soil support combining factor, Sc

Ratio of Msn/Msb	Ratio of Total Trench (or Embedment Zone) Width at Pipe Springline to Mean Pipe Diameter, Bd/Do						
	1.25	1.50	1.75	2.0	2.5	3.0	4.0
0.005	0.02	0.05	0.08	0.12	0.23	0.43	0.72
0.01	0.03	0.07	0.11	0.15	0.27	0.47	0.74
0.02	0.05	0.10	0.15	0.20	0.32	0.52	0.77
0.05	0.10	0.15	0.20	0.27	0.38	0.58	0.80
0.1	0.15	0.20	0.27	0.35	0.46	0.65	0.84
0.2	0.25	0.30	0.38	0.47	0.58	0.75	0.88
0.4	0.45	0.50	0.56	0.64	0.75	0.85	0.93
0.6	0.65	0.70	0.75	0.81	0.87	0.94	0.98
0.8	0.84	0.87	0.9	0.93	0.96	0.98	1.00
1	1.00	1.00	1.00	1.00	1.00	1.00	1.00
1.5	1.40	1.30	1.20	1.12	1.06	1.03	1.00
2	1.70	1.50	1.40	1.30	1.20	1.10	1.05
3	2.20	1.81	1.65	1.50	1.35	1.20	1.10
≥5.0	3.00	2.20	1.90	1.70	1.50	1.30	1.15

The soil support combing factor is multiplied by the constrained modulus of the embedment material to calculate the combined constrained modulus for use in design in Eqn. 7.7.

$$M_s = S_c M_{sb} \qquad \text{(Eqn. 7.7)}$$

where:

M_s = composite constrained modulus, psi (kPa);
S_c = soil support combining factor; and,
M_{sb} = constrained modulus of embedment material, psi (kPa).

When the constrained modulus of the native soil is less than that of the embedment material, the soil support combining factor is less than 1.0. This gives a composite constrained modulus less than the constrained modulus of the embedment material, which accounts for the reduced stiffness and support from weaker native soil. The role of native soil stiffness is minimized as the width of the embedment material increases.

7.4 Structural Design Loads

Structural loads on buried thermoplastic pipes broadly fall into two categories: permanent loads and transient loads.

Permanent loads, or dead loads, are assumed to remain constant throughout the design life of the pipe. Permanent loads for buried thermoplastic pipe consist of vertical soil load and hydrostatic loads from groundwater, but can also include horizontal soil load (e.g., from unbalanced fill), sustained ground surface surcharges (e.g., stockpiled soils or other materials), adjacent foundation reactions, and other long-term special loads.

Transient loads, typically considered as live loads, may vary in position and magnitude and are not permanently in-place over the buried pipe. Live loads typically consist of vehicular loads, such as trucks, trains, aircraft, construction equipment, or agricultural equipment. These loads can be in-place over the pipe for durations that range from less than a second (e.g., passing vehicle) to a week or more (e.g., parked trucks or staged construction equipment).

7.4.1 Dead Load (DL)

Standard design dead loads are comprised of vertical and lateral soil loads. The soil load is based on the vertical soil prism pressure above the pipe. The prism load accounts for fill depth and the unit weight of the soil is determined along with the maximum expected groundwater elevation above the pipe, if present.

The self-weight of the pipe is typically excluded from design due to the relatively small weight and its negligible effect on structural design.

Soil Unit Weight

A moist unit weight of soil, γ_s, of 120 pcf (1922 kg/m^3) shall be assumed unless otherwise required in the project documents or unless determined by site soil exploration for a specific project. This unit weight is considered reasonable and conservative for typical compacted soils. When the maximum groundwater elevation is expected to be above the springline of the pipe, the buoyant (i.e., submerged) unit weight of soil,γ_b, shall be used for the portion of the soil volume between the pipe springline elevation and the maximum expected groundwater elevation (i.e., when determining the vertical soil prism pressure).

The buoyant unit weight of soil,γ_b, can be determined by Eqn. 7.8.

$$\gamma_b = \gamma_{sat} - \gamma_w \qquad \text{(Eqn. 7.8)}$$

where:

γ_b = buoyant unit weight of soil, pcf (kg/m^3);
γ_{sat} = saturated unit weight of soil, pcf (kg/m^3); and,
γ_w = unit weight of water = 62.4 pcf (998 kg/m^3).

The saturated unit weight of soil, $\gamma_{sat,}$ is determined by using soil unit weight relationships. A saturated unit weight of soil, γ_{sat} = 136 pcf (2178 kg/m^3), may be used in the absence of site soil data. This corresponds to a 120 pcf (1922 kg/m^3) soil with a specific gravity of solids of 2.65 and a water content of 15%. In this case, the calculated buoyant weight of soil, γ_b, is 74 pcf (1185 kg/m^3).

Vertical Soil Prism Pressure

The dead load on the pipe is referred to as the vertical soil prism pressure, P_{sp}, that represents the weight of the soil above the pipe from the pipe springline to the ground surface.

The unit weight of soil is adjusted for buoyant effects if the maximum groundwater elevation is expected to be above the springline of the pipe. The vertical soil prism pressure is calculated according to Eqns. 7.9 through 7.11 for the cases in which the groundwater is below the top of the pipe Eqn. 7.9, where the groundwater is above the top of pipe and below the ground surface Eqn. 7.10, or where the groundwater is at the ground surface Eqn. 7.11.

$$P_{sp} = (H + 0.11D_o)\gamma_s \text{ for } H_w \le 0.5D_o, \quad \text{(Eqn. 7.9)}$$

$$P_{sp} = [H - (H_w - 0.5D_o)]\gamma_s + (H_w - 0.5D_o + 0.11D_o)\gamma_b \text{ for } 0.5D_o \le H_w < H + 0.5D_o, \quad \text{(Eqn.7.10)}$$

$$P_{sp} = (H + 0.11D_o)\gamma_b \text{ for } H + 0.5D_o \le H_w \quad \text{(Eqn. 7.11)}$$

where:

P_{sp} = vertical soil prism pressure at springline of pipe, psf (kg/m^2);
H = height of fill above top of pipe, ft (m);
D_o = outside diameter of pipe, ft (m);
γ_s = moist unit weight of soil, pcf (kg/m^3);
H_w = maximum expected height of groundwater table relative to pipe springline, ft (m); and,
γ_b = buoyant unit weight of soil, pcf (kg/m^3).

The factor of 0.11 times the pipe outside diameter, D_o, accounts for the soil between the springlines and the top of pipe. The portion of the vertical soil prism pressure carried by the pipe for design is determined by the vertical arching factor.

Lateral Load from Horizontal Soil Pressure

For most flexible pipe installations, the fill depth over the pipe is approximately uniform across the width of the pipe and surrounding ground surface, and the effect of horizontal soil pressure as a load on the pipe is minimal. Soil to the sides of the pipe provides passive resistance as the pipe springlines move outward under the predominantly vertical loads that act on the pipe.

For some installations soil may exert a horizontal component of pressure on a pipe, such as the case in which there is unbalanced loading from uneven backfill or a surcharge load to one side of the vertical soil prism. Such unbalanced loading will change the thrust and bending moment distribution around the pipe and will result in different force demands from the typical assumptions considered in this design method. These special conditions can be addressed through soil-structure interaction finite element modeling by qualified engineers using a program such as Culvert Analysis and Design (CANDE). CANDE-2007 is a publicly available finite element analysis and design program specializing in soil-structure interaction for buried structures (2019).

Temporary Surcharge Loads

In locations where soil or other materials are temporarily stockpiled over the pipe, temporary loads similar to uniform surcharge loads will affect the pipe. These loads deviate from the final installation conditions that are assumed for design and the ability of the pipe to resist such temporary loads should be evaluated.

As a first estimate, the load magnitude can be approximated as an equivalent depth of fill based on the soil unit weight assumed in design. This depth of fill can then be compared to the maximum allowable cover depth for the pipe, considering long-term installation conditions (e.g., the typical maximum allowable depth of fill). If the calculated equivalent depth of fill is less than the maximum allowable depth of fill, the surcharge load is likely acceptable.

If the calculated equivalent depth of fill is greater than the maximum allowable long-term fill depth for the pipe, more refined calculations can be performed. The vertical prism pressure from the temporary load may be calculated, and the Strength I Limit State Design calculations may be performed, by following modifications for special evaluation that include:

1) The pipe is evaluated for short-term live load and long-term dead load that correspond to the long-term design depth of fill at the site (Strength I limit state design, but at a depth of fill that is representative of conditions at the specific site).
2) An additional component of dead load strain is based on the vertical prism pressure from the temporary load. The strain is calculated assuming a creep modulus provided by the manufacturer, or one that is determined by testing, and is representative of the anticipated duration of the surcharge; and,
3) The long-term dead load strains and short-term live load strains are combined using Strength I load factors, summed to the additional strain from the surcharge as multiplied by a 1.95 load factor. The total factored strain is compared to the material strain limits and global buckling strain resistance.

If the combined factored strain demand from a special evaluation according to Steps 1 to 3 above exceeds the material strain limits or global buckling strain resistance, it may be possible to further refine evaluation of the demands through the use of finite element analysis with staged construction performed by a qualified engineer. In all cases, stress levels should be checked against the material's viscoelastic limit and the modulus values used should be derived from creep testing of the pipe material at an appropriate stress level and over an appropriate duration of time for the loads being evaluated.

7.4.2 Hydrostatic Load (W)

The hydrostatic load on the pipe is calculated as the external hydrostatic pressure, P_W, from the maximum expected height of groundwater relative to the pipe springline and accounting for normal seasonal variation. Hydrostatic pressure will contribute to hoop thrust, but has a negligible effect on the deflection and is not considered in deflection calculations. Hydrostatic pressure on the pipe is determined in accordance with Eqn. 7.12.

$$P_w = \gamma_w K_w H_w \leq \gamma_w (H + \frac{D_o}{2}) \qquad \text{(Eqn. 7.12)}$$

where:

P_W = hydrostatic groundwater pressure at springline of pipe, psf (kg/m^2);
γ_W = unit weight of water = 62.4 pcf (998 kg/m^3);
K_W = factor for uncertainty in level of groundwater table, $1.0 \leq K_W \leq 1.3$;
H_W = maximum expected height of groundwater table relative to pipe springline, ft (m);
H = height of fill above top of pipe, ft (m); and,
D_o = outside diameter of pipe, ft (m).

There is often uncertainty in the level of the groundwater table and its seasonal variations. The factor, K_W, is applied to account for this uncertainty. A value of $K_W = 1.3$ is typically a conservative value, where H_W is estimated or determined through short-term monitoring like observations in boreholes during the site soil investigation. Values as low as $K_W = 1.0$ may be used along with conservative values of H_W, or in instances when there has been long-term monitoring of the groundwater elevation. The total design groundwater elevation (K_W x H_W) is typically limited to $H + D_o/2$, unless standing floodwater is anticipated and is a required design condition.

In some cases, the owner may specify a height of groundwater above the top of fill to represent seasonal floods. In such cases, the expression to the right of the inequality in Eqn. 7.12 is ignored, and an appropriate value of K_W shall be used based on the reliability of the data from which the groundwater height is derived. For the design of pipes that are subjected to floodwater elevations greater than those expected during seasonal variations, such as floods with an expected 100-year return period, adjustments will be necessary.

7.4.3 Live Load (LL)

Live load typically consists of vehicular loads, such as trucks, trains, aircraft, construction equipment, or agricultural equipment. For pipes below roadways, the standard US highway vehicular live load is specified as the AASHTO HL-93 live load. The AASHTO HL-93 live load, its components, and application are described in the following sections. The effects of other specific design or construction vehicles can typically be evaluated using a similar approach, depending on the particular vehicle wheel or axle loads and configurations.

AASHTO HL-93 Live Load

Buried pipe that is installed in a location subjected to general traffic or vehicular loads is typically designed according to the AASHTO LRFD procedures for design of highway culverts (2014). The AASHTO LRFD requires design for the HL-93 live load. The AASHTO HL-93 consists of the AASHTO Design Truck or AASHTO Design Tandem in combination with the AASHTO Lane Load.

AASHTO Design Truck (HS-20)

The AASHTO Design Truck (formerly referred to as HS-20) is the notional representation of a heavy tractor-trailer vehicle. The Design Truck consists of one 8000 lb (35 kN) steering front axle and two 32,000 lb (142 kN) rear axles. The front and first rear axle are spaced at 14 ft (m) apart. The two rear axles have a variable spacing between 14 and 30 ft, (and m) with the designer required to select the rear axle spacing that provides the most severe loading on the structure being designed. For typical (diameter less than 6 ft) buried corrugated thermoplastic pipes, with diameters less than 6 ft (1.8 m), only one Design Truck axle can load the pipe at a time. In the rare case where multiple Design Truck axles must be considered, the minimum 14 ft (4.3 m) rear axle spacing is selected.

Axle loads are applied on two wheel patches per axle with each wheel load patch applied uniformly over a 10-in. (25-cm) long (in the truck's travel direction) by 20-in. (50-cm) wide ground surface contact area. The contact areas are centered 6 ft (1.8 m) apart on the axle. The Design Truck configuration is shown in Figure 7.14.

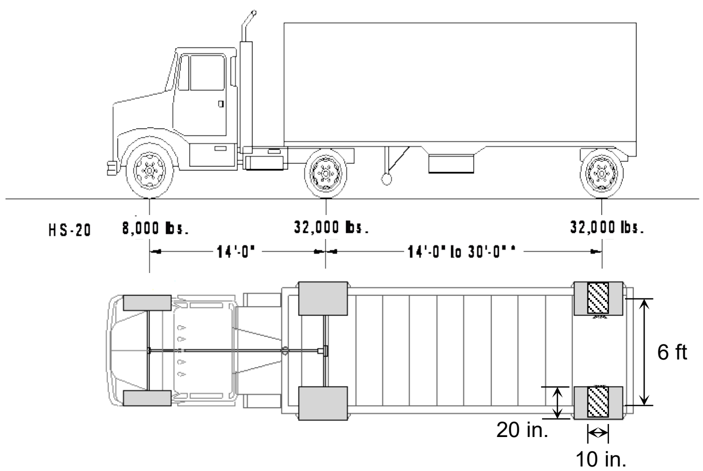

Figure 7.14: AASHTO design truck configuration

AASHTO Design Tandem

The AASHTO Design Tandem, formerly referred to as the Alternate Military Vehicle, is the second AASHTO HL-93 live load axle configuration. The AASHTO Design Tandem consists of two 25,000 lb (111 kN) axle loads spaced 4 ft (1.2 m) apart. Each axle has two wheels whose loads act uniformly on the 10-in. long by 20 in. wide, (25-cm by 50-cm) ground surface contact areas. The Design Tandem configuration is shown in Figure 7.15.

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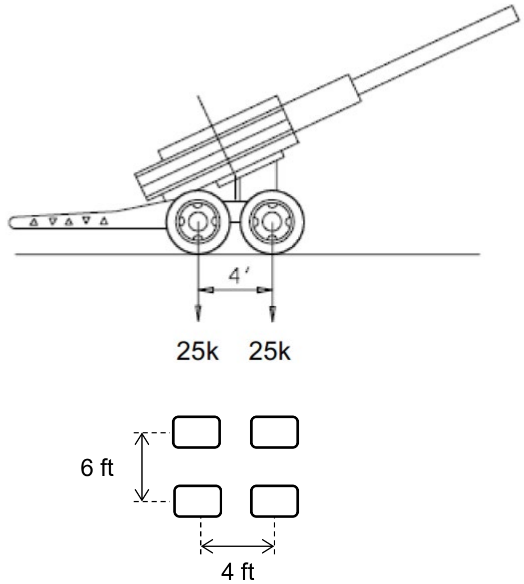


Figure 7.15: AASHTO design tandem configuration

AASHTO Design Lane Load

The AASHTO Design Lane Load simulates the effect of a series of smaller vehicles acting simultaneously on the roadway surface with the Design Truck or Tandem. The Design Lane Load consists of a 640 lbf/ft (868 N-m) uniformly distributed load acting over a 10-ft (3-m) lane width, equivalent to a 64 psf (3 kPa) pressure acting on the ground surface over the 10-ft (3-m) wide lane. The Design Lane Load is continuous and not interrupted where the Design Truck or Tandem is present.

Application of the AASHTO HL-93 Live Load to Buried Pipe

For typical buried pipe and culverts crossing under a roadway, the AASHTO HL-93 live load is applied on the ground surface to a single loaded lane crossing the span of the pipe. For other applications such as storm drains, where the pipes may be running longitudinally under the roadway, it is possible that multiple lanes of traffic may load the pipe simultaneously. Other considerations specific to the application of live load onto buried structures include the dynamic force effects (impact) that vary with burial depth and the distribution of the live load force effects through the soil fill. The AASHTO HL-93 live load is applied to buried pipe and culverts considering these topics.

Multiple Presence of Live Load

The AASHTO HL-93 live load was developed considering the statistical calibration of live load based on pairs of design vehicles acting together. Therefore, when a single design vehicle is on a bridge, it may be heavier than each one of a pair of vehicles on the bridge and can still have the same probability of occurrence. To account for the statistical probability that a single truck could be overloaded, the static vehicle loads are adjusted by the multiple presence factor shown in Table 7.11.

Table 7.11: Multiple presence factor

Number of Loaded Lanes	Multiple Presence Factor, m
1	1.2
2	1.0

In the situation when two loaded lanes are considered, the ground surface contact areas between wheels from the vehicles in adjacent lanes are spaced at a clear distance of 4 ft (1.2 m). Typical designs use a single loaded lane with m = 1.2.

Dynamic Load Allowance (Impact)

The static effect of the Design Truck and Design Tandem vehicle loads shall be increased to account for dynamic amplification associated with moving loads. Dynamic effects on buried structures are produced as vehicles pass over pavement imperfections (e.g., joints, cracks, potholes, undulations, etc.). Dynamic effects are mitigated (dampened) by soil fill as the fill depth increases. Dynamic load allowance (IM) is calculated using Eqn. 7.13.

$$IM = 1 + 0.33(1 - 0.125H) \qquad \text{(Eqn. 7.13)}$$

where:

IM = dynamic load allowance, and
H = height of fill above top of pipe, ft (m).

Dynamic load allowance shall not be applied to the design lane load or other effects that already include dynamic considerations such as vehicle breaking force or centrifugal forces, or for loads with longer durations like parked vehicles.

Live Load Distribution through Soil

The vertical pressure at top of the pipe, PL, due to Design Truck or Design Tandem live loads is calculated by distributing the live loads from the ground surface contact areas through the depth of soil fill to the top of the pipe using a live load distribution factor. The live load distribution factor for buried thermoplastic pipes is 1.15.

Conceptually, the 10 in. by 20 in. (25 cm by 50 cm) ground surface contact area for each wheel is increased in each direction by 1.15 times the distance from the ground surface through the soil fill. This approximation is similar to the 60° rule found in many texts on soil mechanics. Live load distribution through soil fill is ignored for fill depths 1 ft (0.3 m) and less.

As fill depths increase, contact areas from adjacent wheels (including those on separate vehicles, if present) and axles may overlap as they are distributed through the soil fill. In areas of overlap, the total load from overlapping wheels (including dynamic load allowance and multiple presence factors) is summed and distributed uniformly over the length and width of the overlapping area at depth.

The interaction depths and live load distribution equations assume that traffic is driving across the span (diameter) of the buried pipe. As such, the live load distribution length is in the span direction of the pipe, similar to the direction of the wheel contact length as a wheel passes over the buried pipe. The live load distribution width is in the longitudinal direction of the pipe similar to the wheel contact width and vehicle width. Another approach may be taken to distribute the wheel and axle loads through fill for traffic that is running parallel to the longitudinal alignment of the pipe.

The depth at which wheel loads on a single axle (across the width of the design vehicle) interact, $H_{int\text{-}w}$, is calculated according to Eqn. 7.14. The depth at which wheel loads on consecutive axles (along the length of a design vehicle) interact, $H_{int\text{-}a}$, is calculated according to Eqn. 7.15.

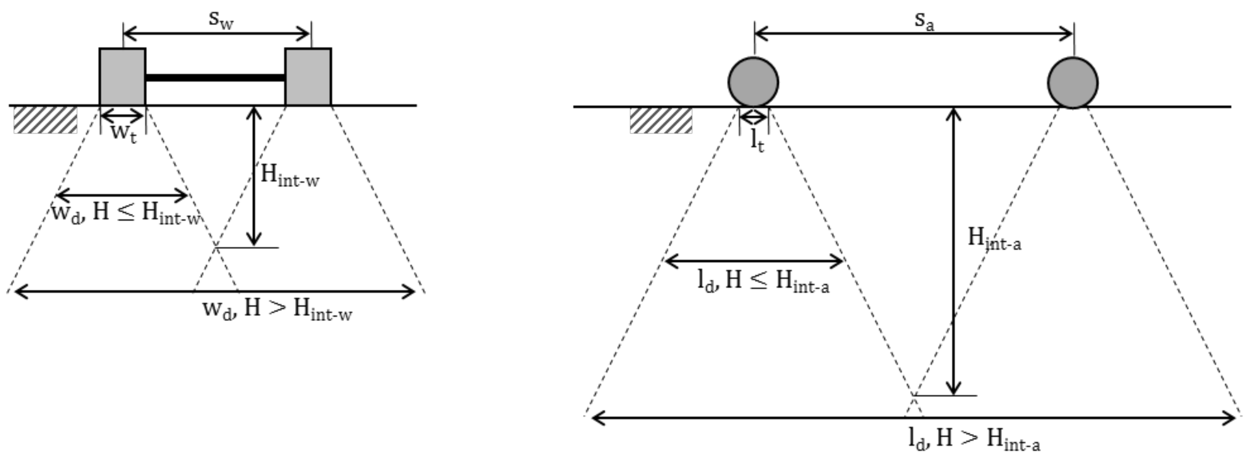
$$H_{int-w} = \frac{s_w - w_t - 0.06D_i}{LLDF}, \quad \text{(Eqn. 7.14)}$$

$$H_{int-a} = \frac{s_a - l_t}{LLDF} \quad \text{(Eqn. 7.15)}$$

where:

$H_{int\text{-}w}$ = interaction depth of wheels on single axle, ft (m);
$H_{int\text{-}a}$ = interaction depth of consecutive heavy axles on a single truck, ft (m);
s_w = wheel center spacing on single axle (across vehicle) = 6 ft (2 m) for AASHTO vehicles;
s_a = axle center spacing (along vehicle) = 14 ft (4.3 m) for AASHTO Design Truck or 4 ft (1.2 m) for AASHTO Design Tandem; and,
w_t = width of ground surface tire contact area = 1.67 ft (0.5 m) for AASHTO vehicles;
l_t = length of ground surface tire contact area, = 0.83 ft (0.25 m) for AASHTO vehicles;
D_i = inside diameter of pipe, ft (m); and,
$LLDF$ = live load distribution factor = 1.15.

The 0.06 term multiplied by the pipe's inside diameter allows for a modest increase in live load distribution area over the traditional distribution of 1.15 times the depth of fill, as discussed in NCHRP Report 647 (Petersen et al. 2010). Figure 7.16 conceptually shows the live load distribution for (a) two wheels on a single axle and (b) on two consecutive axles.



(a) (b)

Figure 7.16: Live load distribution through soil fill for wheels on a (a) single axle, and for (b) two axles

The distributed width, w_d, and length, l_d, of the live load at depth are calculated as based on the depth of fill. When the fill depth, H, is less than the interaction depths, H_{int-w} and H_{int-a}, the length and width of the live load distribution area at depth, l_d and w_d, respectively, are calculated according to Eqns. 7.16 and 7.17.

$$l_d = l_t + LLDF * H, \quad \text{(Eqn.7.16)}$$

$$w_d = w_t + LLDF * H + 0.06D_i \quad \text{(Eqn.7.17)}$$

where:

w_d = distributed width of live load pressure at top of pipe, ft (m);
l_d = distributed length of live load pressure at top of pipe, ft (m); and,
H = height of fill above top of pipe, ft (m).

When the fill depth, H, is greater than the wheel interaction depth, H_{int-w}, the width of the live load distribution area at depth is calculated according to Eqn. 7.18.

$$w_d = w_t + s_w + LLDF * H + 0.06D_i \quad \text{(Eqn.7.18)}$$

Where the fill depth, H, is greater than the axle interaction depth, H_{int-a}, the length of the live load distribution area at depth is calculated according to Eqn. 7.19.

$$l_d = l_t + s_a + LLDF * H \quad \text{(Eqn.7.19)}$$

The vertical pressure at top of the pipe due to live load, P_L, is calculated accounting for the dynamic load and multiple presence factors applied to the vehicular live load as shown in Eqn. 7.20.

$$P_L = \frac{P_{surf} * IM * m}{w_d l_d} + P_{lane} \qquad \text{(Eqn. 7.20)}$$

where:

P_L = vertical pressure at top of pipe from Design Truck or Tandem plus Lane Load, psf (kg/m^2)

P_{surf} = live load from ground surface summed across all intersecting wheels and axles, lbf (metric ton); and,

IM = dynamic load allowance;

m = multiple presence factor; and,

P_{lane} = vertical pressure from Design Lane Load, psf (kg/m^2).

Table 7.12 provides vertical pressures at top of pipe, P_L, from the Design Truck and Design Lane Load, including dynamic load allowance and multiple presence factor for a single-loaded lane (AASHTO HL-93 live load) at fill depths from 1 to 8 ft (0.3 to 2.4 m).

Table 7.12: Vertical pressure at top of pipe under AASHTO HL-93 live load

Fill Depth (ft)	Pipe Diameter (in.)									
	12	15	18	24	30	36	42	48	54	60
1	30.6	30.4	30.3	29.9	29.6	29.4	29.1	28.8	28.5	28.2
1.5	19.6	19.5	19.4	19.3	19.1	18.9	18.8	18.6	18.5	18.3
2	13.6	13.6	13.5	13.4	13.3	13.2	13.2	13.1	13.0	12.9
2.5	10.0	10.0	10.0	9.9	9.8	9.8	9.7	9.7	9.6	9.6
3	7.7	7.7	7.7	7.6	7.6	7.5	7.5	7.5	7.4	7.4
4	5.1	5.1	5.1	5.1	5.0	5.0	5.0	5.0	5.0	5.0
5	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
6	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
7	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
8	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0

AASHTO allows HL-93 live load to be neglected on buried structures where the depth of fill is greater than 8 ft (2.4 m) and where the depth of fill exceeds the pipe diameter. The vertical soil pressure is over four times the vertical pressure at the top of the pipe from live load at a fill depth of 8 ft (2.4 m) assuming 120 pcf (1944 kg/m^3) soil. This difference, along with the effect of the long-term modulus estimated to be 1/6 of the short-term modulus, results in a negligible contribution of live load to total strain at the 8-ft (2.4-m) depth of fill.

Commentary on HL-93 Live Load Applied to Buried Pipe and Culverts

The AASHTO HL-93 live load was developed as a notional representation of a group of design vehicles permitted on highways of various states under previously-specified exclusions to weight limits. The vehicular model is called notional because it does not represent any particular truck, but is based on a probabilistic representation of the vehicles that are considered and the forces imparted on the structures they pass over.

The HL-93 live load has some features that may not necessarily be representative of physical loading on buried pipe and culverts since it was developed for traffic traveling directly on the bridge deck. For example, after distributing the Design Truck and Design Tandem loads through soil and accounting for wheel and axle interaction, the Design Truck induces higher pressures at the top of pipe than the Design Tandem to a fill depth up to about 66 in. (cm), after which the Tandem controls by a negligible amount (up to 0.15 psi (1 kPa) at 8-ft (2.4-m) fill depth). Consequently, typical designs do not need to consider the tandem loads.

The Design Lane Load, equal to a distributed load of 640 lbf/ft (868 N-m) across a 10 ft (3-m) lane width, effectively does not distribute through soil fill since it extends indefinitely along the length of the road and across the lane width at the ground surface. This load is equal to a uniform surface pressure of 64 psf (3 kPa) or approximately 0.5-ft (0.15-m) of additional soil fill (assuming 120 pcf (1920 kg/m^3) soil). However, unlike soil load, the live load from passing vehicles is not sustained, and the contribution of the lane load to demand strains in the pipe wall is negligible, based on the relative short- and long-term creep moduli used to calculate those strains. Also, the lane load, which represents the traffic around the design vehicle(s) cannot physically load a buried pipe simultaneously with a Design Truck heavy axle. In nearly all of the cases for pipes with less than 6-ft (1.8-m) diameters, only a single axle from the Design Truck will load the pipe for a given truck position, as relative to the pipe. The length and axle spacing of the Design Truck precludes a second axle, or additional traffic from loading a buried pipe perpendicular to the travel direction, at the same time as the Design Truck axle under consideration.

As such, it is reasonable to neglect the difference between the Design Tandem and Design Truck, neglect the Design Lane Load, and consider only a single heavy axle of the Design Truck for design of buried flexible pipes.

Figure 7.17 shows the unfactored vertical pressure at the top of a 36-in. (90-cm) diameter pipe due to the Design Truck, Design Tandem, Design Lane Load, soil load at 120 pcf (1920 kg/m^2) and combined loads. Vertical pressure due to the Design Truck is more important than the Design Tandem until the difference between them becomes negligible. Vertical pressures from the Design Truck and Design Tandem will reduce with increased fill depth until they are negligible at about 8 ft (2.4 m) of fill, when the soil load then controls the demand. Vertical pressure from the Design Lane Load is negligible at all fill depths.

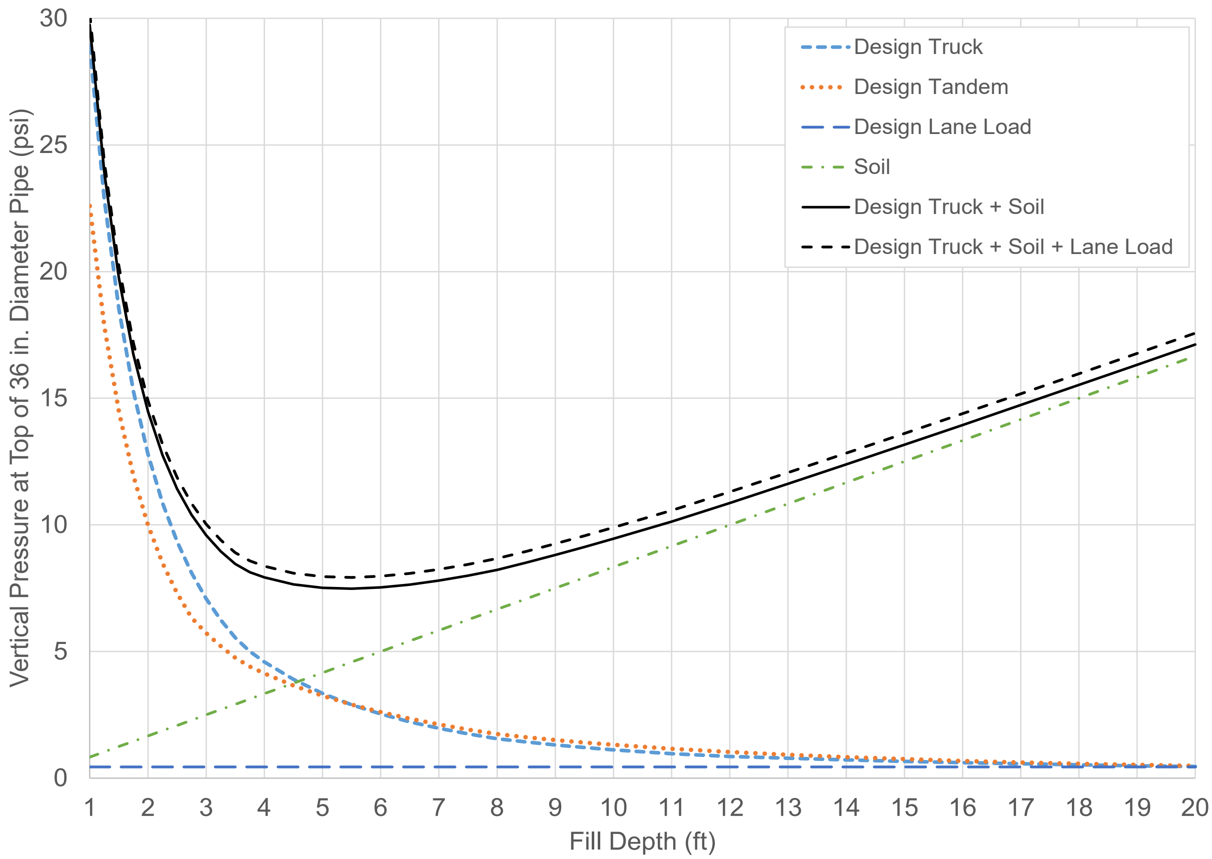


Figure 7.17: Vertical pressures at top of 36 in. (90-cm) diameter pipe

Former AASHTO Design Vehicles

Certain projects may require design for live loads that are not part of the AASHTO HL-93 and may be based on other historic AASHTO design vehicles. Some of these vehicles are shown in Figure 7.18.

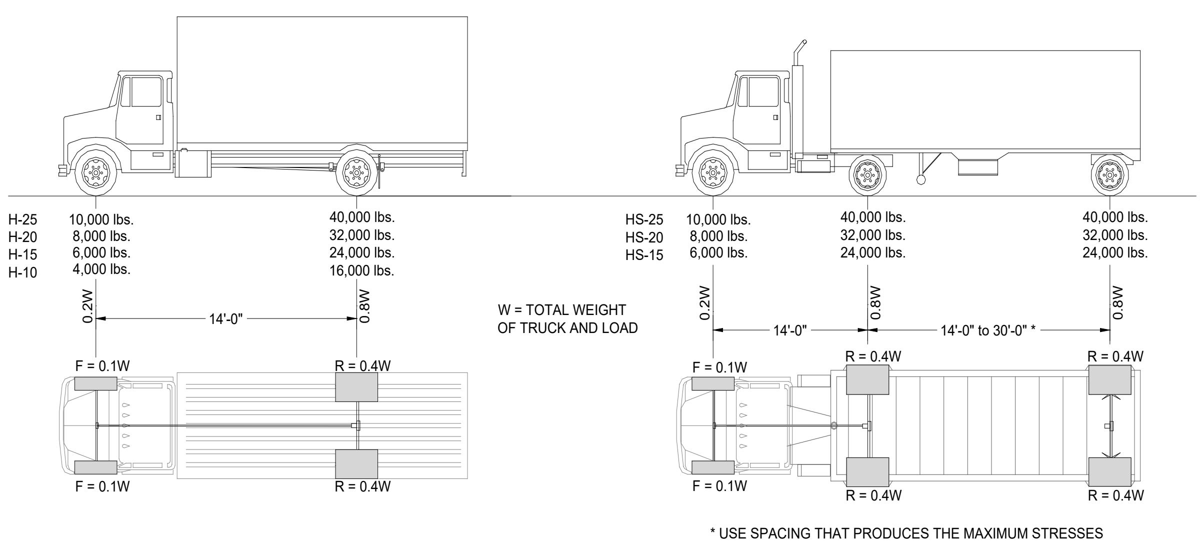


Figure 7.18: Historic AASHTO design vehicles

The AASHTO Design Truck is a three-axle representation of the former H-20 design vehicle, as shown in Figure 7.18. The H-20 vehicle included an 8 kip front axle followed by a single 32 kip rear axle 14 ft away, for a total weight of 40 kips or 20 tons. The H-10 to H-25 vehicles have a similar vehicle configuration as the H-20 with the same axle locations and ratios of front to rear axle loads (1:4), but with total load magnitudes ranging from 10 to 25 tons (9 to 23 metric tons).

The HS-series vehicles are similar to the H-series vehicles but with a third axle spaced 14 to 30 ft (4.6 to 9.1 m) from the second axle. The third axle has the same load magnitude as the heavy (second) axle. The current AASHTO Design Truck is equivalent to the former HS-20 design vehicle.

Guidance on Tire Contact Areas for Non-AASHTO Design Vehicles

For vehicles such as front-end loaders or other rubber-tired construction equipment, the specified tires generally have a defined width and inflation pressure. Manufacturer specifications for these vehicles typically provide the total gross vehicular weights (GVW) and the distributions of that weight on each axle. Based on the wheel load, the specified tire width, and the specified inflation pressure, the tire-to-ground surface contact length may be estimated. A simple approximation is that the tire contact length is typically half the amount of the tire contact width.

AASHTO provides guidance on estimating tire-to-ground surface contact areas for vehicles other than the Design Truck or Tandem and for those with unknown tire dimensions. The ground surface contact width, measured in inches (mm) in the direction across the vehicle, may be taken as $P/0.8$ where P is the design wheel load in kips (kN). The tire-to-ground surface contact length in the vehicle travel direction may be taken as $6.4*\gamma LL*1.33$. For a Strength I live load factor of $\gamma LL = 1.75$, this length is about 14.9 in (37.8 cm).

Cooper E-80 Railroad Live Load

Buried pipe that is installed in a location that is subjected to railway loads should be designed to withstand the Cooper E-80 live load, as specified by AREMA (2017). The Cooper E-80 live load consists of a series of axle loads ranging from 40 to 80 kips (178 to 355 kN) each with variable axle spacing of between 5 and 8 ft (1.5 and 2.4 m), plus a distributed load of 8 kip/ft (0.11 kN/m). An allowance for dynamic effects (i.e., impact) should also be considered.

AREMA (2017) provides a table of Cooper E 80 vertical live load pressures at the top of pipe for buried corrugated metal pipes, including a 50% increase for dynamic effects (impact). The Cooper E-80 vertical live load pressures at the top of a buried corrugated thermoplastic pipe including dynamic effects are given in Table 7.13. The Cooper E-80 live load need not be considered where the soil fill depth, H, is greater than 25 ft (7.6 m) unless otherwise required in project documents. The H in Table 7.13 is measured from the top of the pipe to the bottom of the railroad track tie.

Table 7.13: Cooper E-80 live load on pipe (including dynamic load allowance)

Fill Depth, H (ft)	Cooper E-80 Vertical Live Load Pressure at Top of Pipe, PL	
	(psf)	(psi)
2	3,800	26.4
5	2,400	16.7
8	1,600	11.1
10	1,100	7.6
12	800	5.6
15	600	4.2
20	300	2.1
25	100	1.4

Aircraft Loads

Aircraft loads must be considered in design where they may load a buried pipe, as these loads vary widely in magnitude and landing gear configuration and can be significantly greater than AASHTO HL-93 loading. The Federal Aviation Administration (FAA) Advisory Circular AC-150/5300-13A entitled Airport Design, recommends designing runway and taxiway bridges to support static and dynamic loads imposed by the heaviest aircraft expected to use the structures (2014). The design should include allowance for concentrated loads from the aircraft's main gear configuration. The document references the use of a 20% to 25% increase in load to account for possible future fleet growth. It also notes that overdesign is preferable to the cost and/or operational penalties of replacing or strengthening an under-designed structure.

The FAA AC-150/5300-13A highlights design load considerations unique to airfield bridges that can include runway load factors due to dynamic loading, longitudinal loads due to braking forces, and transverse loads caused by wind on large aircraft. Braking loads as high as 0.7G (for no-slip brakes, in which 0.7G is 70% of the static vertical load, must be anticipated on ground surfaces or bridge decks subjected to direct wheel loads.

The FAA Advisory Circular AC-150/5320-6F Airport Pavement Design and Evaluation, suggests using a point wheel load of 100,000 lbf with 250 psi tire pressure for design of manhole covers (2016). For the 250 psi tire pressure, a ground surface contact area of approximately 16 by 25 in. may be representative of a single landing gear wheel pair. The circular also notes that it may be reasonable to design bridges that will carry aircraft at large hub airports for a 1,500,000 lbf aircraft load.

The Boeing 747 shows a 234,515 lbf static load on each of four main landing gear struts (Boeing Commercial Airplanes 2017). The struts each have four 21-in. wide tires (two tires on each of two axles) with an inflation pressure of 221 psi, resulting in individual tire contact areas of 21 by 12.6 in. The axle spacing is 56.5 in., and center to center spacing of wheels on an axle is 46.8 in. as shown in Figure 7.19.

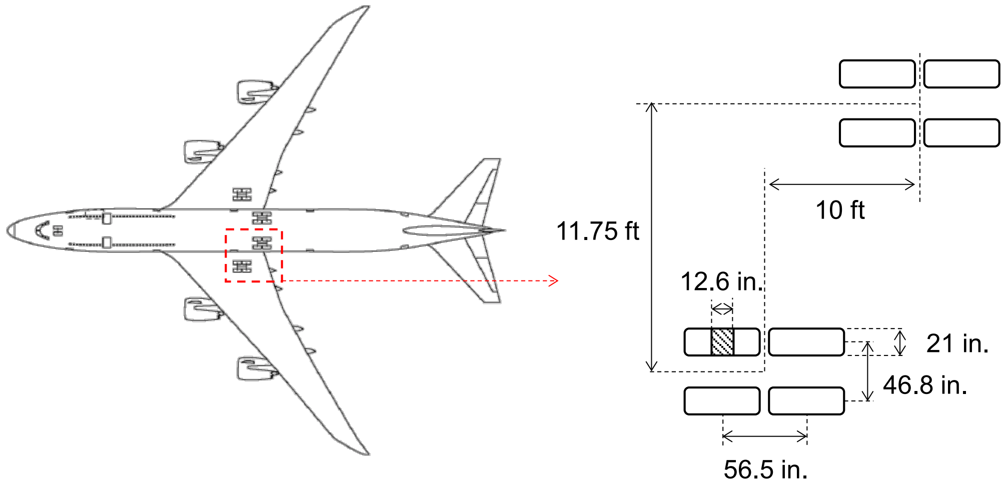


Figure 7.19: Boeing 747 landing gear configuration (2017)

Fire Truck Loads

Certain jurisdictions require the consideration of fire truck loads over buried structures. Specific loads and their associated load durations will vary based on the particular firefighting apparatus, amount of time required to fight the fire, and local regulations.

The notational vehicles of the AASHTO HL-93 live load can cover typical axle loads for fire trucks while traveling on roadways. Certain firefighting equipment, such as ladder trucks with stabilizing outriggers, will have different live load configurations when in operation. Actual outrigger ground contact pressures may vary substantially depending on the configuration of the ladder, its load, and the configuration of the outrigger and the bearing plate. The National Fire Protection Association (NFPA) specifies that fire truck outrigger plates be sized such that their ground contact pressure does not exceed 75 psi (517 kPa). This is less than the AASHTO Design Truck unfactored tire pressure of 80 psi (551 kPa), but may extend over a larger area which could be a more demanding load.

The live load distribution approach may be used to calculate the maximum vertical pressure at the top of the pipe due to fire truck outrigger loads. The duration of a fire truck outrigger load is unknown in many cases. A duration of 24 hours may be reasonable for evaluation when there is no dynamic load allowance (i.e., impact). The live load pressure, PL, from the fire truck load, distributed to the top of the pipe, may be included as a design condition used to determine the fire truck's live load design strain demand while using a creep modulus from testing or provided by the manufacturer that is representative of a 24-hour load at the appropriate magnitude (which may be in excess of 500 psi (3447 kPa)). An additional case to consider may be for an instantaneous fire truck outrigger load and the dynamic load allowance should be found by using the short-term elastic modulus of the pipe material for evaluation.

Owner-Specified Design Vehicles

Owner-specified special design or permit vehicles typically have more specificity in the actual magnitude and location of loads as compared to the variation found with standard highway vehicles. As such, they can typically be considered with the Strength II limit state, which has a lower live load factor than that which is considered in Strength I design for standard highway vehicles. The Strength II limit state may also be appropriate for certain fire truck and aircraft loads, depending on the certainty with which the live load is specified and the likelihood for the design vehicle to be representative of the worst-case loading over the length of time considered.

Live loads of configurations other than the Design Truck or Design Tandem may be evaluated using a similar live load application method. Load magnitudes and ground surface contact areas shall correspond to those of the specific vehicles considered.

Construction Live Loads

Construction live loads that may be applied at fill depths that are less than the design minimum cover or the final design depth of fill, or that have loads greater than those of the AASHTO Design Truck, shall be considered in pipe design. Temporary fill may be added over the pipe to help distribute these construction live loads and to decrease their effect on the pipe.

The construction live loads may be evaluated using a similar application method as the one which is used for the Design Truck. The load magnitudes and ground surface contact areas shall correspond to those of the specific vehicles considered. Project specifications should include limits on construction loads, specifically the size of equipment and the minimum and maximum allowable fill depths.

7.5 Structural Design Method

The structural design method is applicable to gravity flow, buried, corrugated wall, thermoplastic pipes that are subject to dead (including surcharge), live, and hydrostatic loads. An overview of buried flexible pipe behavior is presented first, followed by detailed equations for the general structural design of the pipe wall. Hydraulic design was covered separately in Chapter 6.

Special topics following the detailed structural design method include the evaluation at the Strength II limit state (for special design vehicles) and the Extreme Event limit state (for rare flood occurrences), as well as other design parameters such as minimum cover, trench width, bend radius, and minimum spacing between multiple runs of pipe.

7.5.1 Service Limit State – Deflection

The total deflection of the pipe under service or unfactored loads is checked against an allowable deflection limit, typically taken as 5% of the undeformed inside diameter of the pipe.

The deflection calculation is a modified version of the Spangler deflection equation (Spangler 1941). Spangler's equation has been refined since its initial introduction and has been validated through extensive field testing. The calculation considers the stiffness of the pipe (i.e., through the creep modulus, moment of inertia, and diameter), the support of the surrounding soil (by the bedding factor and constrained modulus), and the loading conditions (i.e., the dead and live load, and the deflection lag, and the full soil prism load).

The total service vertical deflection, Δ_t, due to bending from long-term dead and short-term live loads and due to circumferential shortening is calculated according to Eqn. 7.21.

$$\Delta_t = \frac{K_B D_L P_{sp} D_o}{\frac{E_{lt} I_p}{R^3} + 0.061 M_s} + \frac{K_B C_L P_L D_o}{\frac{E_{st} I_p}{R^3} + 0.061 M_s} + 2R\varepsilon_{sc} \quad \text{(Eqn. 7.21)}$$

where:

Δ_t = total pipe service vertical deflection, in. (mm);
K_B = bedding coefficient, $0.083 \leq K_B \leq 0.110$, typically taken as $K_B = 0.10$;
D_L = deflection lag factor, $1.0 \leq D_L \leq 6.0$, typically taken as $D_L = 1.5$;
P_{sp} = vertical soil prism pressure, psi (kPa);
D_o = outside diameter of pipe, in. (mm);
E_{lt} = long-term creep modulus of pipe material, psi (kPa);
E_{st} = short-term elastic modulus of pipe material, psi (kPa);
I_p = moment of inertia of pipe wall per unit length of pipe, in.^{4}/in. (cm^4/cm);
R = radius of pipe to centroid of wall, in. (mm);
M_s = constrained modulus of soil at springline of pipe, psi (kPa);
C_L = live load coefficient, = $l_d/D_o \leq 1.0$;
l_d = distributed length of live load pressure at top of pipe, in. (mm);
P_L = vertical pressure at top of pipe due to live load, psi (kPa); and,
ε_{sc} = service compression strain, calculated in accordance with the hoop thrust strain demand with all load and redundancy factors set to a
value of 1.0.

The bedding coefficient, K_B, varies from a value of 0.083 for full invert and haunch support to a value of 0.110 for line load support under the pipe invert. Project specifications should always require good haunch support; however, a value of $K_B = 0.10$ is typically used in design to account for inconsistent haunch support.

The deflection lag factor, D_L, accounts for an increase in deflection with time throughout the life of the pipe. It can account for an increase in deflection from a variety of causes including increases in loading and consolidation of embedment and native soils. A typical value of 1.5 is used for long-term deflection estimates assuming standard contractor installation practices. Lower values may be justified for installations with relatively stiff native soils and dense granular embedment placed with a high level of certainty.

The live load coefficient, C_L, adjusts the total applied live load to account for shallow fill cases in which the distributed width of the live load may be less than the pipe diameter.

The total pipe deflection is compared to the allowable deflection according to Eqn. 7.22.

$$\Delta_t \leq \delta D_i \qquad \text{(Eqn. 7.22)}$$

where:

δ = design allowable pipe vertical deflection, typically taken as δ = 5%, and

D_i = inside diameter of pipe, in. (mm).

7.5.2 Strength I Limit State Design

Strength I limit state design assesses the pipe under dead loads and standard live loads such as AASHTO HL-93, Cooper E-80, or construction loads. Designs in which the live loads consist of specific design vehicles such as special permit vehicles or other well-defined owner-specified vehicles are checked by Strength II limit state design.

The pipe is checked for strains due to hoop thrust, combined thrust and bending, and global buckling using factored loads and resistance.

Load Factors

Load factors for Strength I limit state design are shown in Table 7.14.

Table 7.14: Strength I limit state load factors

Type of Load	Strength I Limit State Load Factor
Soil (Vertical Earth)	$\gamma_{EV} = 1.3K_{\gamma E}$ (maximum), or = 0.9 (minimum)
Live	$\gamma_{LL} = 1.75$
Hydrostatic	$\gamma_{WA} = 1.0$

The installation factor, $K_{\gamma E}$, is a coefficient of the soil dead load factor used in design that is directly linked to the level of inspection required during pipe installation. The values for $K_{\gamma E}$ shall be 1.15, 1.35, or 1.5. These values represent the need for continuous special inspection, periodic special inspection, or standard installation inspection, respectively, during a pipe installation and backfill as described in Chapter 9. A value of $K_{\gamma E}$ = 1.5 provides a traditional level of safety with a resulting total dead load factor of 1.95. A value of $K_{\gamma E}$ = 1.15 may be used in scenarios when agencies require that 100% of the pipe installation is subjected to continuous special inspection for 100-year service life projects. At no time should $K_{\gamma E}$ have a value less than 1.15.

In cases in which soil load effects are beneficial to the design, the minimum soil load factor of γ_{EV} = 0.9 should be considered. Examples of cases in which the soil load may be beneficial to the design include when checking the combined flexural and compressive strain in shallow fill installations and for buoyancy calculations.

The load factor for hydrostatic loads of $\gamma_{WA} = 1.0$ reflects the certainty in hydrostatic load from water with a known unit weight. The uncertainty associated with the elevation of the groundwater table is addressed with the K_W.
For the Strength I and II limit states, buried thermoplastic pipes are considered to be non-redundant under dead loads of soil fill and redundant under live loads of soil fill. The soil load redundancy factor, η_{EV}, is taken as 1.00. The live load redundancy factor, η_{LL}, is taken as 1.00.

Resistance Factors

The resistance factors, that are applicable for Strength I and Strength II limit states are shown in Table 7.15.

Table 7.15: Strength I and Strength II limit state resistance factors

Behavior	Resistance Factor
Hoop Thrust	$\phi_t = 1.0$
Flexure (Combined Strain Results in Tension)	$\phi_f = 1.0$
Global Buckling	$\phi_{bck} = 0.7$
Soil Stiffness	$\phi_s = 0.9$
Buoyancy	ϕb = 0.75

Hoop Thrust Design

The hoop thrust force is calculated from dead and live loads, and the dead loads are reduced by the vertical arching factor to account for the soil-structure interaction and pipe material creep with load duration.

The hoop thrust strain is calculated from the hoop thrust force by using the appropriate creep modulus values for the pipe material. The corrugation effective area is also used to account for local buckling. The hoop thrust strain is checked against the compressive strain limit.

Hoop thrust is checked at the top of the pipe and at the springline, and the hoop thrust at the springline is the maximum.

Vertical Arching Factor

The relative stiffness of the pipe and surrounding soil is quantified by the hoop stiffness factor, S_H. The hoop stiffness factor is the ratio of the soil stiffness, based on the constrained modulus of soil, to the pipe hoop (axial) stiffness in Eqn. 7.23.

$$S_H = \frac{\phi_s M_s R}{E_{lt} A_g} \qquad \text{(Eqn. 7.23)}$$

where:

$S_H =$ hoop stiffness factor;
$\phi_s =$ resistance factor for soil support;
$M_s =$ constrained modulus of soil at springline of pipe, psi (kPa);

$R =$ radius to centroid of pipe wall, in. (mm);
$E_{lt} =$ long-term creep modulus of pipe material, psi (kPa); and,
$A_g =$ gross area of pipe corrugation per unit length of pipe, in.^{2}/in., (cm^2/cm).

The resistance factor for soil support is used to account for potentially-reduced soil properties, relative to design values. The long-term creep modulus of the pipe is used to estimate the stiffness over the duration of the pipe design life, rather than at initial placement. The hoop stiffness factor is used to determine the vertical arching factor, *VAF*, according to Eqn. 7.24.

$$VAF = 0.76 - 0.71\left[\frac{S_H - 1.17}{S_H + 2.92}\right] \qquad \text{(Eqn. 7.24)}$$

The VAF equation was developed for embankment installations. The calculation of vertical arching factors specific to trench installations has not been developed. The embankment-based VAF is considered to be conservative for most trench installations.

Hoop Thrust Demand

The calculation for hoop thrust demand takes the dead and live load vertical pressures, plus hydrostatic load, and converts them into a hoop thrust force at the pipe springline. The hoop thrust demand from long-term dead and hydrostatic loads, T_D, and the hoop thrust demand from short-term live load, T_L, are calculated separately in Eqns. 7.25 and 7.26. This is because they have different pipe material creep modulus values for the strain calculation. It should be noted that the VAF applies to soil load only.

$$T_D = \eta_{EV}\left(\gamma_{EV}K_2(VAF)P_{sp} + \gamma_{WA}P_w\right)\frac{D_o}{2}, \qquad \text{(Eqn. 7.25)}$$

$$T_L = \eta_{LL}\gamma_{LL}C_L F_1 F_2 P_L \frac{D_o}{2} \qquad \text{(Eqn. 7.26)}$$

where:

$T_D =$ factored long-term dead and hydrostatic load hoop thrust per unit length of pipe, lbf/in. (kgm/cm);
$T_L =$ factored live load hoop thrust per unit length of pipe, lbf/in. (kgm/cm);
$\eta_{EV} =$ soil load redundancy factor;
$\eta_{LL} =$ live load redundancy factor;
$\gamma_{EV} =$ vertical earth dead load factor;
$\gamma_{WA} =$ hydrostatic load factor;
$\gamma_{LL} =$ live load factor;
$K_2 =$ coefficient to account for variation of thrust around pipe circumference, = 1.0 for thrust at springline, = 0.6 for thrust when evaluated at the crown;
$VAF =$ vertical arching factor;
$P_{sp} =$ vertical soil prism pressure, psi (kPa);
$P_w =$ hydrostatic groundwater pressure at springline of pipe, psi (kPa);

D_o = outside diameter of pipe, in. (mm);

C_L = live load coefficient, = $l_d/D_o \leq 1.0$;

F_1 = live load distribution adjustment factor calculated according to Eqn. 7.27;

F_2 = soil type live load thrust correction factor calculated according to Eqn. 7.28; and,

P_L = vertical pressure at top of pipe from live load, psi (kPa).

$$F_1 = \max\left(\frac{0.75D_o}{l_d}, \frac{15}{D_i}, 1.0\right) \qquad \text{(Eqn. 7.27)}$$

$$F_2 = \frac{0.95}{1 + 0.6S_H} \quad \text{(Eqn. 7.28)}$$

where:

D_i = inside diameter of pipe, in. (mm);
l_d = distributed length of live load pressure at top of pipe, in. (mm); and,
S_H = hoop stiffness factor.

Verification of Material Properties through Hoop Thrust Stress Check

Pipe material creep modulus values are calibrated to a service level stress of 500 psi (3447 kPa). Service stresses that exceed 500 psi (3447 kPa) may invalidate these properties and require additional testing to characterize the creep modulus.

Therefore, the service stress from dead and hydrostatic load hoop thrust, σ_D, which typically controls design of thermoplastic pipes, is limited to 500 psi (3447 kPa) according to Eqn. 7.29 without further engineering assessment:

$$\sigma_D = \frac{(K_2(VAF)P_{sp} + P_w)D_o}{2A_g} \leq 500psi \quad \text{(Eqn. 7.29)}$$

where:

σ_D = service stress from dead and hydrostatic load hoop thrust, psi (kPa); and,
A_{eff} = corrugation gross area per unit length of pipe, in.^{2}/in. (cm^2/cm).

Hoop Thrust Strain Demand

The factored hoop thrust strain demand, ε_c, is calculated as the sum of the hoop thrust strain demands from long-term dead load and hydrostatic load hoop thrust, and the live load hoop thrust according to Eqn. 7.30, while accounting for the corrugation effective area:

$$\varepsilon_c = \frac{T_D}{A_{eff}E_{lt}} + \frac{T_L}{A_{eff}E_{st}} \quad \text{(Eqn. 7.30)}$$

where:

ε_c = factored hoop thrust compression strain demand, in./in. (cm/cm);
T_D = factored hoop thrust from long-term dead load and hydrostatic load per unit length of pipe (lbf/in.),
T_L = factored hoop thrust from live load per unit length of pipe, lbf/in. (kgm/cm);
A_{eff} = effective area of pipe corrugation per unit length of pipe, in.^{2}/in. (cm^2/cm);
E_{lt} = long-term creep modulus of pipe material, psi (kPa); and,
E_{st} = short-term elastic modulus of pipe material, psi (kPa).

Hoop Thrust Strain Check

The factored hoop thrust compression strain demand is checked against the factored compression strain limit, according to Eqn. 7.31:

$$\varepsilon_c \leq \phi_t \varepsilon_{yc} \qquad \text{(Eqn. 7.31)}$$

where:

ε_c = factored hoop thrust compression strain demand, in./in. (cm/cm);
ϕ_t = resistance factor for hoop thrust; and,
ε_{yc} = pipe material compression strain limit, in./in. (cm/cm).

Combined Hoop and Flexural Strain Design

In the absence of a more refined approach such as finite element analysis, the flexural strain demand may be calculated from deflections and an empirical shape factor. The flexural strain is combined with the compressive strain from hoop thrust and checked for the possibility of net tension and for net compression.

Shape Factor

The shape factor, D_f, is an empirical factor that is used to directly relate the deflection to bending moment in the pipe wall, and therefore flexural strain. Table 7.16 presents shape factors based on pipe stiffness, approximate structural backfill type, and backfill compaction for corrugated HDPE and PP pipe. The values may be interpolated or extrapolated as needed.

Table 7.16: Shape factors, D_f, for corrugated thermoplastic pipe (1)

Pipe Stiffness, PS psi (kPa)	Shape Factor, D_f, for Given Pipe Embedment Material and Compaction Effort			
	Gravel		Sand	
	Dumped to Slight Compaction	Moderate to High Compaction	Dumped to Slight Compaction	Moderate to High Compaction
9 (62)	4.5	6.0	5.0	7.0
18 (124)	3.5	4.5	4.0	5.5
36 (248)	2.8	3.5	3.0	4.5
72 (496)	2.3	2.8	2.5	3.5

The values presented in this table have been adapted to corrugated thermoplastic by reducing the traditional values, developed for stiffer fiberglass pipe, by 1.0 as allowed by AASHTO.

In Table 7.16, the gravel refers to GW, GP, GW-GC, GW-GM, GP-GC, and GP-GM soil classifications in Classes II and III, while sand refers to SW, SP, SM, SC, GM, and GC soil classifications in Class III or mixtures. Dumped to slight compaction is representative of less than 85% of maximum dry density per the standard Proctor test. Moderate to high compaction represents greater than or equal to 85% of maximum dry density per the standard Proctor test.

Flexural Strain Demand

The factored flexural strain demand, ε_f, may be estimated based on the empirical shape factor, the pipe geometry, and the bending deflection in accordance with Eqn. 7.32.

$$\varepsilon_f = \gamma_{EV} D_f \frac{c}{R}\left(\frac{\delta D_i - \varepsilon_{SC} D}{D}\right) \quad \text{(Eqn. 7.32)}$$

where the terms R and ε_{SC} are as defined in Eqn. 7.21, and δ and D_i are as defined in Eqn. 7.22, and:

ε_f = factored flexural strain, in//in. (cm/cm);
γ_{EV} = vertical earth dead load factor (maximum);
Df = shape factor;
c = the larger of the distances from the centroid of the pipe wall to the innermost fiber of the valley or outermost fiber of the crest, in. (mm);
R = radius to centroid of pipe wall, in. (mm); and,
D = diameter to centroid of pipe wall, in. (mm).

Combined Strain Check – Net Tension

If the flexural strain is greater than the compressive strain, it will induce tension on one side of the pipe wall at certain locations around the circumference. For installed pipe, this would be most likely to occur at the top of the pipe for shallow burial cases under live loads, where bending moments are large and the thrust is minimal.

In the case of combined strain which results in net tension at the inside or outside surface, the factored flexural strain, ε_f, is combined with the factored hoop thrust strain, ε_c, and checked against the factored tension strain limit in accordance with Eqn. 7.33.

$$\left|\varepsilon_f - \varepsilon_c\right| \leq \phi_f \varepsilon_{yt} \quad \text{(Eqn. 7.33)}$$

where:

ε_f = factored flexural strain, in./in. (cm/cm);
ε_c = factored hoop thrust compression strain, in./in. (cm/cm);
ϕ_f = resistance factor for flexure; and,
ε_{yt} = pipe material tension strain limit, in./in. (cm/cm).

Since the dead loads primarily cause hoop thrust (i.e., compressive strain) and live loads at shallow fill primarily cause bending moments (i.e., flexural strain), this check should be performed twice: once using the maximum dead load factor, and a second time using the minimum dead load factor.

Combined Strain Check – Net Compression

In cases when the flexure causes compression strain, it will combine with the hoop thrust strain on one side of the pipe wall at certain locations around the circumference. This effect is typically most significant at the inside fiber of the springline, where the bending moment results in compression and the hoop thrust demand is at its maximum. Factored flexural strain, ε_f, which acts in compression and the factored hoop strain, ε_c, are combined and checked against an increased compression strain limit according to Eqn. 7.34.

$$\varepsilon_f + \varepsilon_c \leq \phi_t 1.5\varepsilon_{yc} \qquad \text{(Eqn. 7.34)}$$

where:

ε_f = factored flexural strain, in./in. (cm/cm);
ε_c = factored hoop thrust compression strain, in./in. (cm/cm);
ϕ_t = resistance factor for hoop thrust; and,
ε_{yc} = pipe material compression strain limit, in./in. (cm/cm).

The combined compressive strain is checked against 1.5 times the factored compression strain limit. This 50% increase in compression strain capacity is an approximate design assumption to account for the distribution in strain across the pipe wall. The combined flexural and compressive strain is only present at the extreme fiber of the critical compression element (e.g., the inside surface at the springline). The web elements and the opposing element (e.g., the crest element at the springline) experience less compression and are able to brace the critical compression element against buckling. Increasing the compressive strain capacity by 50% has been shown by testing and analysis to be an appropriate conservative design approximation (McGrath and Sagan 2000).

Global Buckling Design Check

Individual corrugation element local buckling is accounted for in the hoop thrust design check by considering the corrugation effective area. Stability of the full corrugated wall, however, must be checked separately through the global buckling design check.

Global buckling strain resistance, ε_{bck}, is calculated according to Eqn. 7.35 using a simplification of the continuum buckling theory approach developed by Moore (1990):

$$\varepsilon_{bck} = \frac{1.2C_n\left(E_{lt}I_p\right)^{\frac{1}{3}}}{A_{eff}E_{lt}}\left[\frac{\phi_s M_s(1-2\nu)}{(1-\nu)^2}\right]^{\frac{2}{3}} R_h \qquad \text{(Eqn. 7.35)}$$

where:

ε_{bck} = nominal global buckling strain resistance, in./in. (cm/cm);
C_n = calibration factor for nonlinear effects = 0.55;
E_{lt} = long-term modulus of pipe material, psi (kPa);
I_p = moment of inertia of pipe wall per unit length of pipe, in.^{4}/in. (cm^4/cm);
A_{eff} = effective area of pipe wall per unit length, in.^{2}/in. (cm^2/cm);
ϕ_s = resistance factor for soil support;
M_s = constrained modulus of soil at springline of pipe, psi (kPa);
ν = Poisson's ratio of soil; and,
R_h = correction factor for backfill soil geometry, determined by Eqn. 7.36.

$$R_h = \frac{11.4}{11 + {}^{D}/_{12H}} \qquad \text{(Eqn. 7.36)}$$

where:

D = diameter to centroid of pipe wall, in. (mm), and
H = depth of soil fill from top of pipe to ground surface, ft (m).

The factored hoop thrust compression strain, ε_c, is checked against the factored global buckling resistance, ε_{bck}, according to Eqn. 7.37.

$$\varepsilon_c \le \phi_{bck}\varepsilon_{bck} \qquad \text{(Eqn. 7.37)}$$

where:

ε_c = factored hoop thrust compression strain, in./in. (cm/cm), from Eqn. 7.30;
ϕ_{bck} = resistance factor for global buckling; and,
ε_{bck} = global buckling strain resistance.

Flexibility Factor

To limit risk to pipe during construction for lifting and handling, the flexibility factor, *FF*, shall be limited to a maximum of 0.095 in./lbf when calculated according to Eqn. 7.38.

$$FF = \frac{D^2}{E_{st} I_p} \leq 0.095\, in./lbf \tag{Eqn. 7.38}$$

where:

- FF = flexibility factor of pipe, in./lbf (cm/kgm);
- D = diameter to centroid of pipe wall, in. (mm);
- E_{st} = short-term modulus of pipe, psi (kPa); and,
- I_p = moment of inertia of pipe wall per unit length, in.^{4}/in. (cm^4/cm).

Thermoplastic pipe will exhibit greater reduced stiffness at higher temperatures and increased stiffness at lower temperatures. The flexibility factor limit of 0.095 in./lbf is evaluated against the short-term modulus of the pipe material at the standard reference temperature of 73.4 deg. F.

Buoyant Force Design Check

Where the maximum expected groundwater elevation is above the bottom of the pipe, the pipe should be checked for buoyant forces (flotation). The buoyant force demand, Fbd, is calculated as the volume of the pipe times the unit weight of the fluid being displaced, considering the outside diameter of the pipe, according to Eqn. 7.39.

$$F_{bd} = \frac{\pi}{4} {D_o}^2 \gamma_w \tag{Eqn. 7.39}$$

where:

- F_{bd} = buoyant force demand, lbf/ft (kgm/cm);
- D_o = pipe outside diameter, ft (m); and,
- γ_w = unit weight of displaced fluid; 62.4 lb/ft^3 (997 kg/m3) for groundwater.

The buoyant force resistance, F_{br}, is the weight of the soil above the pipe. This is calculated as the soil prism load times the outside diameter of the pipe according to Eqn. 7.40.

$$F_{br} = P_{sp} D_o \tag{Eqn. 7.40}$$

where:

- F_{br} = buoyant force resistance, lbf/ft (kgm/m);
- P_{sp} = vertical soil prism pressure at springline of pipe, psf (kPa); and,
- D_o = pipe outside diameter, ft (m).

The factored buoyant force demand is then compared to the factored buoyant force resistance according to Eqn. 7.41. The minimum soil load factor, $\gamma_{EVmin} = 0.9$, is used with the buoyant force resistance factor, ϕ_b, to determine the factored buoyant force resistance.

$$\gamma_{WA} F_{bd} \leq \gamma_{EVmin} \phi_b F_{br} \qquad \text{(Eqn. 7.41)}$$

where:

γ_{WA} = hydrostatic load factor for Strength I limit state;
γ_{EVmin} = minimum vertical earth dead load factor; and,
ϕ_b = resistance factor for buoyancy.

The use of γ_{EVmin} and ϕ_b together gives an effective safety factor of approximately 1.5 for buoyancy.

The pipe should also be checked against flotation using the same methodology during any phases of construction for which it may be subjected to buoyant loads, such as during placement of flowable fill. In these cases, it may be necessary to provide vertical restraint to prevent uplift and flotation, as soil weight will not be present over the pipe. When calculating the buoyant force demand, F_{bd}, for such conditions, the unit weight used in Eqn. 7.39 should be of the fluid displaced, which may be greater than the unit weight of water. In such cases, the volume of fluid displaced may be adjusted for considerations such as the maximum lift height of the fluid during the placement of flowable fill.

7.5.3 Maximum Allowable Fill Height Tables for Strength I Design

Tables 7.17 and 7.18 provide representative maximum allowable fill height values for HDPE and PP pipe, respectively, by pipe diameter, embedment material, and compaction condition. The fill heights are determined, assuming typical thermoplastic dual-wall pipe, groundwater below the bottom of the pipe, soil density of 120 pcf (1922 kg/m^3), and standard installation inspection ($K_{\gamma E}$ = 1.50).

Table 7.17: Representative maximum allowable fill heights (ft) for HDPE pipe (Strength I Design – AASHTO HL-93 Live Load and 100-Year Design Life)

Diameter (in.)	Class I Embedment		Class II Embedment		Class III Embedment	
	Compacted	Dumped	95% SPD	90%SPD	95% SPD	90% SPD
12	29	21	21	15	15	9
15	26	19	19	13	13	9
18	26	19	19	13	13	9
24	24	18	17	12	12	9
30	24	18	17	12	12	8
36	24	17	16	11	10	7
42	23	17	16	10	10	7
48	23	17	16	10	10	7
60	23	17	16	10	10	7

Note: Additional diameters may be available from some manufacturers; consult with the manufacturer for fill heights for any additional diameters. Deeper fill heights may be possible; consult pipe manufacturers to determine fill heights based on specific installation conditions and manufacturer pipe profiles.

Table 7.18: Representative maximum allowable fill heights (ft) for PP pipe (Strength I Design - AASHTO HL-93 Live Load and 100-Year Design Life)

Diameter (in.)	Class I Embedment		Class II Embedment		Class III Embedment	
	Compacted	Dumped	95% SPD	90%SPD	95% SPD	95% SPD
12	32	27	24	17	17	10
15	30	25	23	17	17	10
18	25	23	23	16	17	10
24	25	22	22	15	16	10
30	25	20	20	14	14	10
36	24	18	17	12	12	8
42	23	17	16	10	11	7
48	21	16	15	10	10	6
60	21	16	15	10	10	6

Note: Additional diameters may be available from some manufacturers; consult with the manufacturer for fill heights for any additional diameters. Deeper fill heights may be possible; consult pipe manufacturers to determine fill heights based on specific installation conditions and manufacturer pipe profiles.

Maximum allowable fill heights published by individual manufacturers may vary from the general values presented in Tables 7.17 and 7.18 based on manufacturer-specific corrugation profiles. The values presented are representative values across the industry and may be used for agency-specific fill height tables, with the manufacturer's certification of compliance with the values shown in the table.

7.5.4 Strength II Limit State Design

The Strength II limit state is used to assess the pipe under dead load and owner-specified or permit vehicle live loads. The calculation method matches that of the Strength I limit state presented in Section 7.5.2, but with the live load factor set equal to 1.35. Vertical pressure at the top of the pipe from live load, P_L, shall be determined using a similar approach for application as described previously. Load magnitudes and ground surface contact areas shall correspond to those of the specific vehicles considered. Tire contact areas may be based on Section 7.4.3.4.

The reduction in live load factor is based on reduced uncertainty in the magnitude and position of live loads for a special design vehicle, such as a crane outrigger load or specified model fire truck wheel load, compared to typical highway traffic.

7.5.5 Extreme Event Limit State Design for Flood

The Extreme Event limit state is used to assess the pipe under rare flood conditions based on the AASHTO LRFD Extreme Event II limit state. Extreme events such as major floods are considered to be unique occurrences with return periods that may be significantly greater than the design life of the structure. Design for extreme events is intended to ensure the survival of the structure under such an event. Seasonal, yearly, or other high frequency floods are not considered extreme events and should be evaluated in the maximum expected groundwater elevation for typical hydrostatic load design. Design for the Extreme Event limit state follows the same methodology as design for the Strength I limit state, with the following changes.

The vertical soil prism pressure for a flood event, P_{spf}, shall be determined using the approach in Section 7.4.1, but using the height of groundwater, H_w, equal to the expected height of floodwater above the pipe springline. Hydrostatic pressure demand from floodwater, P_{wf}, shall be calculated as P_w from Section 7.4.2, but with K_w set equal to 1.0 and H_w set equal to the expected height of floodwater above the pipe springline. The vertical live load pressure at the top of pipe, P_L, shall be determined in accordance with Section 7.4.

The live load factor for the Extreme Event limit state, γ_{LLF}, is set to 0.5. Redundancy factor for dead load, η_{EV}, and all resistance factors are set to 1.0.

The factored hoop compression strain demand for an Extreme Event limit state design for flooding, ε_{cf}, is calculated according to Eqn. 7.42.

$$\varepsilon_{cf} = \left[\frac{\gamma_{EV} K_{\gamma E} K_2 (VAF) P_{spf}}{A_{eff} E_{lt}} + \frac{\gamma_{WA} P_{wf}}{A_{eff} 1.5 E_{lt}} + \frac{\gamma_{LLF} C_L F_1 F_2 P_L}{A_{eff} E_{st}}\right] \frac{D_o}{2} \qquad \text{(Eqn. 7.42)}$$

where the terms not previously defined include:

ε_{cf} = factored thrust compression strain under for extreme event flood loads, in./in. (cm/cm);
P_{spf} = vertical soil prism pressure for flood event as described in this section, psi (kPa);
P_{wf} = hydrostatic pressure demand from floodwater as described in this section, psi (kPa); and,
γ_{LLF} = live load factor for the Extreme Event II limit state as described in this section.

The hoop thrust compression strain demand uses a factor of 1.5 times the long-term creep modulus value to estimate the strain from flood loads. This factor assumes a flood duration of approximately one week and is estimated from the ratio of creep modulus values at one week to 100-years by the stepped isothermal method (SIM) creep tests on thermoplastic pipe materials.

7.5.6 Other Design Considerations

Minimum Depth of Fill

Although corrugated thermoplastic pipe may satisfy structural design requirements at very shallow depths of fill, there are serviceability issues such as pavement cracking, allowance for thermal expansion, and maintaining embedment confinement that should be considered when establishing the minimum depth of fill. These issues are not directly considered in the structural design process.

The minimum depth of fill (i.e., cover height) values for corrugated thermoplastic pipes shall be as shown in Table 7.19, unless otherwise specified. The minimum depth of fill, H_{min}, is measured as the distance from the top of the pipe to the bottom of the flexible pavement, or from the top of the pipe to the top of the rigid pavement. The depth of fill, H, used in design to determine the vertical soil prism pressure shall include all fill, from the top of the pipe to the top of the ground surface, regardless of the type of pavement.

Table 7.19: Minimum depth of fill, H_{min}

Condition	Minimum Depth of Fill, H_{min}	
	Pipe Dia ≤ 36 in. (91 cm)	Pipe Dia > 36 in. (91 cm)
Under unpaved areas	12 in. (30 cm)	12 in. (30 cm)
Under lightly trafficked paved areas, such as residential driveways	12 in. (30 cm)	18 in. (45 cm)
Under roadways with standard truck traffic	18 in. (45 cm)	24 in. (60 cm)

Unpaved areas that are subjected to live loading may experience rutting, and in which case the design cover without rutting should be greater than 12 in. (30 cm), and the amount of rutting shall not be permitted above the pipe to reduce the cover to less than 12 in (30 cm). In locations in which the site constraints do not allow for the minimum depth of fill to meet the requirements of Table 7.19, it may be possible to a design rigid pavement that will adequately distribute live loads and provide confinement for the pipe. Such designs should be performed by a qualified engineer in consultation with the pipe manufacturer.

The minimum depth of fill for construction loads was described in Section 7.4.3. The AASHTO Construction Specifications provide guidance on the minimum depth of fill for certain axle loads over a range of pipe diameters, as shown in Table 7.20.

Table 7.20: Minimum depth of fill for construction loads

Nominal Pipe Diameter in. (mm)	Minimum Depth of Fill for Indicated Range of Axle Loads			
	18 kip to 50 kip (80 to 222 kN)	50 kip to 75 kip (222 to 333 kN)	75 kip to 110 kip (333 to 489 kN)	110 kip to 150 kip (489 to 667 kN)
24 to 36 (60 to 91)	24 in. (60 cm)	30 in. (76 cm)	36 in. (91 cm)	36 in. (91 cm)
42 to 48 (106 to 122)	36 in. (91 cm)	36in. (91 cm)	42 in. (106 cm)	48 in. (122 cm)
54 to 60 (137 to 152)	36 in. (91 cm)	36 in. (91 cm)	42 in. (106 cm)	48 in. (122 cm)

The values in Table 7.20 were established based on estimated pipe properties. Designers should verify that these values are appropriate for specific pipe diameters and corrugation geometries. The depths of fill are given as the distance from the top of pipe to the top of the maintained construction roadway and if unpaved, the surface shall be maintained in good condition in order to prevent rutting.

Minimum Trench Width

Trench width must provide sufficient space between the outside of the pipe and the trench wall to safely place and compact the embedment material. This space should include adequate width for compaction equipment and for personnel to insert material into the haunch zone. If the pipe is designed using the composite constrained modulus of soil, the minimum trench width should be greater than or equal to the required embedment zone width from the composite constrained modulus calculations. This requirement should be included in the contract documents and installation cross-sectional details.

In the absence of other criteria, the minimum total trench width shall be as follows and as shown in Figure 7.20:

- At the bottom of the trench, 1.25 times the outside diameter of the pipe plus 12 in. (30 cm); and,
- At the pipe springline, 1.5 times the outside diameter of the pipe plus 12 in. 930 cm) but not less than the diameter plus 16 in. (40 cm).

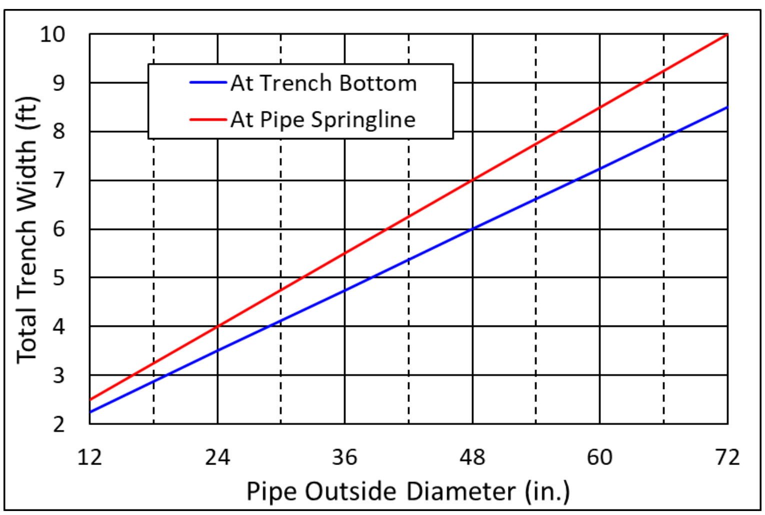


Figure 7.20: Minimum trench widths

Certain installations may have trench widths that are less than the values shown in Figure 7.20. For example, when using flowable fill embedment material in stiff soil, the trench width may be reduced to the pipe outside diameter, plus 12 in. (30 cm), with the pipe centered in the trench. In all cases, the trench width must be sufficient for safe working conditions.

Minimum Spacing between Multiple Runs of Pipe

In locations where multiple runs of pipe are installed parallel to one another, the clear distance between the pipes should be one half the diameter of the larger pipe or 12 in. (30 cm), whichever is greater. Similar to the minimum trench width, the clear distance between multiple runs of pipe must also provide sufficient space to safely place and compact the embedment material.

When the width of the structural embedment zone is dictated by a design that uses a composite constrained modulus, and half of the embedment zone width is greater than one half of the diameter of the larger pipe, a more detailed soil-structure interaction analysis may be required.

Minimum Radius of Bends

The bends along the length of installed buried pipe should be placed in accordance with any manufacturer recommendations. The bend requirements are governed by joint rotation, and the pipe joints must accommodate any deflection that is compatible with the radius of the bend.

Joints and Fittings

Thermoplastic pipe joint performance is assumed to meet the pipe barrel design, provided that all other requirements are met. Standard types of joints include bell and spigot or ring couplers. Thermoplastic pipe joints are not typically subjected to pipe-to-pipe shear across a joint.

In situations when manufactured fittings such as wyes or tees are required, the designer should consult the manufacturer for the minimum and maximum allowable depths of fill for the fittings.

7.6 List of Figures

7.7 List of Tables

7.8 References

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