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1.3 Introduction

Pipe behavior can be generally classified as flexible or rigid. Both flexible and rigid rely on the backfill structure to transfer overburden loads into the bedding adjacent to the pipe. Prinsco corrugated high density polyethylene (HDPE) and polypropylene (HP) pipe, as well as other flexible pipe materials, is designed to deflect in order to transfer the overburden load to the surrounding soil. Rigid pipe, such as reinforced concrete pipe, is defined as a pipe that does not deflect more than 2% without structural distress and it must be designed to carry a greater amount of the soil load with the pipe wall. Therefore, rigid pipe typically has thicker walls. Regardless of the type of pipe, proper backfill compaction is very important in allowing this load transfer to occur.

This guide explains the recommended design method for Prinsco corrugated HDPE and HP pipe, which is based on AASHTO LRFD Bridge Design Specifications Section 12: Buried Structures and Tunnel Liners (i.e. AASHTO Design Method). This AASHTO Design Method is based on embankment installed conditions. Most pipe installations are trench installation which typically reduces the overburden load on the pipe making the AASHTO Design Method extremely conservative. Recommended changes to the design method to account for trench installations are described throughout this document. This design procedure evaluates wall thrust, bending, buckling, and strain and establishes limits on each condition. The design procedure and resulting minimum and maximum burial depths described herein yields conservative results. Contact your local Prinsco Representative for special installations or additional information.

1.4 Design Criteria

AASHTO first published a design method in 1931, which was based on “Working Stress Design” also known as “Allowable Stress Design.” Generally, for Working Stress Design, a factor of safety is applied to the strength of material to create a cushion. In 2007, AASHTO began to convert to Load-and-Resistance Factor Design (LRFD). Generally, a load reduction factor (γ) is applied to loading conditions and a resistance factor (Φ) is applied to material capacity. Load and resistance factors vary for each type of pipe materials. For the purposes of this design guide, the load and resistance factors used herein will be based on those appropriate for Corrugated HDPE or HP. Further, the design for installed conditions requires the use of specific section properties, and material properties for Prinsco HDPE & HP pipe. This section describes the pipe and backfill properties critical for the design procedure described in this guide.

Pipe Design Section Properties

Critical to performance, section properties of Prinsco Corrugated HDPE & HP pipe include the moment of inertia of the wall profile (I), distance from the inside surface to the neutral axis (c), and the cross-sectional area of a longitudinal section (A_s). Pipe stiffness (PS) is a measure of the flexibility of a prescribed length of pipe and is measured in the laboratory by measuring the force required to deflect the pipe 5% of its inside diameter. Pipe stiffness is primarily a quality check. The section properties for Prinsco's dual wall HDPE pipe, single wall HDPE pipe, and dual wall HP pipe are shown in Tables 1, 2, and 3 respectively.

Table 1: HDPE Dual Wall Pipe Section Properties

Nominal Pipe Diameter		Outside Diameter, OD		Pipe Stiffness, PS		Section Area, A_s		Distance from Inside Diameter to Neutral Axis, c		Moment of Inertia, I	
in.	mm	in.	mm	pii	N/m/mm	$in^2/in.$	mm^2/mm	in.	mm	$in^4/in.$	mm^4/mm
4	100	4.8	122	50	345	0.096	2.44	0.12	3.16	0.0012	20
6	150	7.1	180	50	345	0.116	2.95	0.19	4.88	0.0033	54
8	200	9.6	244	50	345	0.161	4.09	0.30	7.53	0.0098	160
10	250	11.9	302	50	345	0.200	5.08	0.38	9.63	0.0160	262
12	300	14.5	368	50	345	0.211	5.36	0.47	11.98	0.0360	590
15	375	17.6	447	42	290	0.224	5.69	0.47	11.90	0.0452	741
18	450	21.5	546	40	276	0.245	6.22	0.58	14.78	0.0804	1318
24	600	28.1	714	34	235	0.391	9.93	0.89	22.50	0.2745	4498
30	750	34.7	881	30	207	0.392	9.96	0.90	22.95	0.2662	4362
36	900	40.6	1031	22.5	155	0.356	9.04	1.02	25.78	0.3040	4982
42	1050	47.5	1207	21	145	0.378	9.60	1.05	26.66	0.4494	7364
48	1200	54.1	1374	20	138	0.428	10.87	1.29	32.76	0.6315	10348
60	1500	66.9	1699	15	104	0.482	12.24	1.34	34.09	0.7639	12518

Note: Section properties provided above are conservative and result in conservative burial depths. Contact your local Prinsco Representative for additional information.

Table 2: HDPE Single Wall Pipe Section Properties

Nominal Pipe Diameter		Outside Diameter, OD		Pipe Stiffness, PS		Section Area, A_s		Distance from Inside Diameter to Neutral Axis, c		Moment of Inertia, I	
in.	mm	in.	mm	pii	N/m/mm	$in^2/in.$	mm^2/mm	in.	mm	$in^4/in.$	mm^4/mm
3	75	3.6	91	35	345	0.055	1.40	0.16	4.1	0.0006	10
4	100	4.6	117	35	345	0.056	1.42	0.19	4.8	0.0008	13
5	125	5.7	145	35	345	0.073	1.85	0.23	5.8	0.0015	25
6	150	7.1	179	35	345	0.090	2.29	0.26	6.6	0.0028	46
8	200	9.5	241	35	345	0.114	2.90	0.38	9.7	0.007	115
10	250	11.6	295	35	345	0.159	4.04	0.46	11.7	0.012	197
12	300	14.4	366	50	345	0.204	5.18	0.65	16.5	0.031	508
15	375	17.6	447	42	290	0.233	5.92	0.94	23.9	0.076	1245

Note: Section properties provided above are conservative and result in conservative burial depths. Contact your local Prinsco Representative for additional information.

Table 3: GOLDPRO Storm Dual Wall HP Pipe Section Properties

Nominal Pipe Diameter		Outside Diameter, OD		Pipe Stiffness, PS		Section Area, A_s		Distance from Inside Diameter to Neutral Axis, c		Moment of Inertia, I	
in.	mm	in.	mm	pii	N/m/mm	$in^2/in.$	mm^2/mm	in.	mm	$in^4/in.$	mm^4/mm
12	300	14.5	368	70	485	0.222	5.64	0.46	11.8	0.039	639
15	375	17.9	455	60	415	0.262	6.65	0.49	12.5	0.058	950
18	450	21.6	549	56	385	0.299	7.60	0.64	16.2	0.104	1704
24	600	28.2	716	50	345	0.389	9.88	0.74	18.8	0.180	2950
30	750	34.7	881	46	320	0.446	11.33	0.96	24.4	0.332	5441
36	900	40.9	1031	40	275	0.527	13.39	0.99	25.2	0.411	7227
42	1050	47.9	1212	35	240	0.507	12.88	1.13	28.7	0.599	9160
48	1200	54.6	1377	30	205	0.568	14.43	1.31	33.3	0.821	13454
60	1500	67.0	1699	25	175	0.681	17.30	1.49	37.9	1.072	17567

Note: Section properties provided above are conservative and result in conservative burial depths. Contact your local Prinsco Representative for additional information.

Material Time-Dependent Properties

High Density Polyethylene, Polypropylene, and many other thermoplastic pipe materials are viscoelastic materials and exhibit time-dependent relaxations when subjected to stress or strain. Viscoelastic materials exhibit properties called creep and stress relaxation. Creep is a measure of the increase in strain with time under a constant stress. This creep behavior is responsible for thermoplastic pipe's unique ability to circumferentially shorten and transfer the overburden load to the surrounding soil. Stress relaxation is the decay in stress under a constant strain. In other words, a pipe that is held in a deflected position will initially experience relatively high stress levels that then quickly subside. This phenomenon has been studied since the 1950s and more recently for pipe applications has been documented at the University of Massachusetts. Both the creep and stress relaxation properties are well understood and are taken into account in the design calculations described below. The design procedures described later in this document explain how and when to use short term or long-term material properties. See Table 4 for the material properties list for polyethylene and polypropylene.

Table 4: HDPE and Polypropylene Material Properties in AASHTO Section 12

Prinsco Product	Min.Cell Class (ASTM D3350)	Factored Tension Strain Limit ε_{yt} (%)	Factored Comp. Strain Limit ε_{yc} (%)	Initial		50 Year		75 Year		100 Year	
				Fu psi (MPa)	E psi (MPa)	Fu psi (MPa)	E psi (MPa)	Fu psi (MPa)	E psi (MPa)	Fu psi (MPa)	E psi (MPa)
Corrugated HDPE	435400C	5	4.1	3000 (20.7)	110000 (758)	900 (6.21)	22000 (152)	900 (6.21)	21000 (145)	800 (5.52)	20000 (138)
Corrugated PP	Requirements in M330	2.5	3.7	3500 (24.1)	175000 (1206)	1000 (6.89)	29000 (200)	1000 (6.89)	28000 (193)	-	-

Note: 100-year properties are not included in Section 12, but are based on Florida DOT research.

Installation and Soil Considerations

The structural performance of all pipes depends on proper compaction of backfill around the pipe. In the case of flexible pipe, as the overburden load is applied to the pipe, the resulting deflection and circumferential shortening creates an interaction between the pipe and the adjacent embedment, or backfill envelope. This interaction is commonly referred to as pipe/soil interaction, and is dependent upon the type of backfill material, backfill compaction level, dimensions of the backfill envelope, and native soil conditions. The information presented is substantially consistent with requirements established in ASTM D2321 "*Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications*." Refer to Prinsco's Installation Guides for additional information regarding dimensions of the backfill envelope.

The type of backfill material (sand, gravel, clay, etc.) and compaction level (Standard Proctor Density) is characterized by the soil stiffness or soil strength. The strength of the embedment can be described using different parameters. One way is by describing it in terms of the modulus of soil reaction (E'), which is an empirical value developed by the Bureau of Reclamation to calculate deflection. Another parameter used to describe backfill strength is the secant constrained soil modulus (M_S). While E' and M_S do have similar units, they are not considered interchangeable. For the purposes of this design standard, the constrained soil modulus (M_S) will be used as shown in Table 5 and Table 6.

Table 5: Constrained Soil Modulus (Ms) for Class I Backfill

Aggregate Material	Max Particle Size (in)	Compacted	Dumped
		psi (kPa)	psi (kPa)
Granite	0.75	8500 (58605)	7000 (48263)
	1.5	5000 (34474)	3500 (24132)
Limestone	0.75	5500 (37921)	3500 (24132)
Quartzite	0.75	7500 (51711)	5500 (37921)

Table 6: Constrained Soil Modulus (Ms) for Class II & III Backfill (Standard Proctor Density)

Cover Height	Class II			Class III		
	GW, GP, SW, SP			GM, SM, ML(1) and GC and SC with <20% passing the 200 sieve		
	95%	90%	85%	95%	90%	85%
ft (m)	psi (kPa)	psi (kPa)	psi (kPa)	psi (kPa)	psi (kPa)	psi (kPa)
1 (0.3)	2000 (13790)	1280 (8830)	470 (3240)	1420 (9790)	670 (4620)	360 (2480)
5 (1.5)	2450 (16900)	1440 (9900)	510 (3500)	1610 (11100)	720 (5000)	380 (2600)
10 (3.0)	2840 (19600)	1580 (10900)	550 (3800)	1730 (11900)	750 (5200)	400 (2800)
15 (4.6)	3090 (21300)	1660 (11400)	590 (4100)	1790 (12300)	760 (5200)	410 (2800)
20 (6.1)	3270 (22500)	1730 (11900)	620 (4300)	1840 (12700)	770 (5300)	420 (2900)
25 (7.6)	3450 (23800)	1800 (12400)	650 (4500)	1880 (13000)	790 (5400)	430 (3000)
30 (9.1)	3610 (24900)	1860 (12800)	690 (4800)	1920 (13200)	810 (5600)	450 (3100)
35 (10.7)	3770 (26000)	1920 (13200)	720 (5000)	1960 (13500)	830 (5700)	460 (3200)
40 (12.2)	3930 (27100)	1980 (13700)	780 (5400)	2010 (13900)	860 (5900)	480 (3300)
45 (13.7)	4090 (28200)	2040 (14100)	790 (5400)	2050 (14100)	880 (6100)	490 (3400)
50 (15.2)	4250 (29300)	2100 (14500)	830 (5700)	2090 (1440)	900 (6200)	510 (3500)
55 (16.8)	4400 (30300)	2180 (15000)	860 (5900)			
60 (18.3)	4550 (31400)	2260 (15600)	895 (6200)			
65 (19.8)	4700 (32400)	2340 (16100)	930 (6400)			
70 (21.3)	4850 (33400)	2420 (16700)	965 (6700)			
75 (22.9)	5000 (34500)	2500 (17200)	1000 (6900)			

Notes:

1) M_S values presented in the table assume:
   - The native material is at least as strong as the intended backfill material. If the native material is not adequate, it may be necessary to increase the trench width. Refer to Prinsco's *Installation Guide* for information on over excavation.
   - The density of the backfill is 120 lb/ft3. Contact your local Prinsco Representative for constrained modulus values associated with different density backfills.
2) M_S should be interpolated for intermediate cover heights.
3) For M_S values of Class IVA materials, contact your local Prinsco Representative.

Table 7: ASTM and AASHTO Soil Properties

ASTM D2321[1] Class	ASTM D2321[1] Description	ASTM D2487 Notation	ASTM D2487 Description	AASHTO M43 Notation	AASHTO M145 Notation	ASTM D2321[1] % Passing Sieve Size			Atterberg Limits		Coefficients	
						1.5 in (40 mm)	No.4 (4.75 mm)	No.200 (0.075 mm)	LL	PI	Uniformity Cu	Curvature Cc
IA[2]	Open-graded, clean manufactured aggregates	N/A	Angular crushed stone or rock, crushed gravel, crushed slag; large voids with little or no fines	5		100%	<10%	<5%	Non Plastic		NA	
IB	Dense-graded, clean manufactured, processed aggregates	N/A	Angular crushed stone or other Class IA material and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; little or no fines	56		100%	<50%	<5%	Non Plastic			
II	Clean, coarse-grained soils	GW	Well-graded gravel, gravel-sand mixtures; little or no fines	57	A1, A3	100%	<50% of "Coarse Fraction"	< 5%	Non Plastic		>4	1 – 3
		GP	Poorly-graded gravels, gravel-sand mixtures; little or no fines	6							<4	<1 or >3
		SW	Well-graded sands, gravelly sands; little or no fines	67			>50% of "Coarse Fraction"				>6	1 – 3
		SP	Poorly-graded sands, gravelly sands; little or no fines								<6	<1 or >3
	Coarse-Grained Soils, borderline clean to w/fines	GW-GC, SP-SM	Sands and gravels which are borderline between clean and with fines			100%	Varies	5% - 12%	Non Plastic		Same as for GW, GP, SW, and SP	
III	Coarse-grained soils with fines	GM	Silty gravels, gravel-sand-silt mixtures	Gravel & sand with <10% fines	A-2-4, A-2-5, A-2-6, or A-4 or A-6 soils with more than 30% retained on #200 sieve	100%	<50% of "Coarse Fraction"	12% to 50%	NA	<4 or <"A" Line	NA	
		GC	Clayey gravels, gravel-sand-clay mixtures							<7 & >"A" Line		
		SM	Silty sands, sand-silt mixtures				>50% of "Coarse Fraction"			>4 or <"A" Line		
		SC	Clayey sands, sand-clay mixtures							>7 & >"A" Line		
IVA[3]	Inorganic fine-grained soils	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity		A-2-7 or A-4 or A-6 soils with 30% or less retained on #200 sieve	100%	100%	>50%	<50	<4 or <"A" Line	NA	
		CL	Inorganic clays of low to medium plasticity; gravelly, sandy, or silty clays; lean clays							>7 & >"A" Line		
IVB[4]	Inorganic fine-grained soils	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			100%	100%	>50%	>50	<"A" Line	NA	
		CH	Inorganic clays of high plasticity, fat clays							>"A" Line		
V[4]	Organic soils or Highly organic soils	OL	Organic silts and organic silty clays of low plasticity		A5, A7	100%	100%	>50%	<50	<4 or <"A" Line	NA	
		OH	Organic clays of medium to high plasticity, organic silts						>50	<"A" Line		
		PT	Peat and other high organic soils									

Notes:

1) Refer to ASTM D2321 for more complete soil descriptions.
2) When using open-graded material, additional precaution must be taken to reduce or eliminate the risk of migration of fines from adjacent material. Refer to ASTM D2321 for more complete information.
3) Class IVA material has limited applications and can be difficult to place and compact; use ONLY with the approval of a soil expert. Class IVB and V materials are not permitted as suitable backfill or bedding materials.

If native soil and other locally available materials meet the criteria of Table 7, they may be considered for backfill. Use of locally available materials is a cost-effective way to minimize material and hauling costs. When in doubt about the appropriate material to use in an installation, consult your local Prinsco Representative.

Some backfill materials can be dumped and knifed around the pipe while others require mechanical compaction to meet the necessary constrained soil modulus for a specific design. Additional information regarding the types of mechanical compactors available and the soil types with which they work best is located in Prinsco's *Installation Guide*.

Another backfill material that is used in special applications is flowable fill. This material is similar to a very low strength concrete. It is poured around the pipe and hardens to form a solid backfill structure. The final cured strength of this material is dependent on mix design and should be determined by the design engineer. If flowable fill is used, precautions must be taken to prevent flotation and special design considerations, such as the strength of in-situ soils, must be taken into consideration. The major disadvantage of flowable fill is that it can be very costly. However, when properly designed and installed, it can be used as an alternative to typical granular backfill. Contact your local Prinsco Representative for additional guidance in the use of this material.

Another soil property used in design is the shape factor (D_f) which is a function of pipe stiffness, type of backfill material, and the compaction level. The shape factor relates deflection and bending behaviors. Table 8 lists shape factors for a variety of typical installation conditions.

Table 8: Shape Factor (D_f) Based on Pipe Stiffness, Backfill, and Compaction Level

	Gravel – GW, GP, GW-GC, GW-GM, GP-GC, GP-GM (includes crushed stone)		**Sand – SW, SP, SM, SC, GM, GC, or mixtures**	
Pipe Stiffness, pii (kPa)	**Dumped to Slight (<85% SPD)**	**Moderate to High (>85%SPD)**	**Dumped to Slight (<85% SPD)**	**Moderate to High (>85%SPD)**
14 (97)	4.9	6.2	5.4	7.2
16 (110)	4.7	5.8	5.2	6.8
18 (125)	4.5	5.5	5.0	6.5
20 (140)	4.4	5.4	4.9	6.4
22 (150)	4.3	5.3	4.8	6.3
28 (195)	4.1	4.9	4.4	5.9
34 (235)	3.9	4.6	4.1	5.6
35 (240)	3.8	4.6	4.1	5.6
40 (275)	3.7	4.4	3.9	5.4
42 (290)	3.7	4.4	3.9	5.3
46 (320)	3.7	4.3	3.9	5.2
50 (345)	3.6	4.2	3.8	5.1
72 (496)	3.3	3.8	3.5	4.5

Notes:

1) Interpolate for intermediate pipe stiffness values.
2) For other backfill materials, use the highest shape factor for the pipe stiffness.
3) Modified from AASHTO LRFD Section 12, 2010, Table 12.12.3.10.2b-1.

Load and Resistance Factors

In Load and Resistance Factor Design (LRFD), the loads applied to the structure and the resistance of a given structure or element to resist the load are multiplied by modification factors to introduce a factor of safety to each criterion. While modification factors are generally provided in the design method, it is left up to the user to choose between a range of factors for a given application. As stated by AASHTO, "Factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance." These factors should be chosen based on the criterion they are applied to and the severity of the application.

Load and resistance factors published by AASHTO are developed for embankment conditions. Because the loading conditions in trench installations is different than in embankment conditions, a separate load factor is warranted. Therefore, the load factors can be modified based on empirical data found for the increased strength of the in-situ soil in the trench walls. It is important to note that the trench width should be as narrow as possible to take advantage of the natural strength of the consolidated in-situ soils, while still allowing enough room for proper backfill placement and use of compaction equipment. Refer to Prinsco's *Installation Guide* for recommended trench widths.

Tables 9 through 11 below provide modification factors which are used throughout this design method for either embankment or trench installations. Within each equation that follows, references to these tables will be provided with a recommended modification factor where appropriate.

Table 9: Load Factors Derived from AASHTO Section 3

	Vertical Earth Pressure		Water Load		Vehicular Live Load	
Load Combination Limit State	**Trench (γ_{EV})**	**Embankment (γ_{EV})**	**Trench (γ_{WA})**	**Embankment (γ_{WA})**	**Trench (γ_{LL})**	**Embankment (γ_{LL})**
Strength Limit I	0.9-1.1	0.9-1.3	1.0-1.15	1.0-1.3	1.75	1.75
Strength Limit II	0.9-1.1	0.9-1.3	1.0-1.15	1.0-1.3	1.35	1.35

Table 10: Resistance Factors from AASHTO Section 12

Structure Type	Φ
Buckling (Φ_{bck})	0.7
Flexure (Φ_f)	1.0
Pipe	1.0
Soil (Φ_s)	0.9

Table 11: Load Modifiers from AASHTO Section 1

Load Combination	η	Redundancy
Earth Fill	1.0	Non-redundant
Live Load	1.0	Redundant
Construction Load	1.0	Redundant

In addition to the load and resistance factors, the LRFD design method utilizes a number of design factors, which includes the installation factor ($K_{\gamma E}$). The installation factor is used to calculate the factored thrust load (T_u) in Equation 14 below and may vary from a value of 1.5 to 1.0, depending upon the level of inspection and monitoring of the backfill material and compaction levels for embankment installations. However, for trench conditions with good backfill material, it is reasonable to expect a more uniform installation assuming trench dimension requirements are met. Therefore, guidance for the appropriate design installation factor is shown in Table 12. For additional guidance, contact your local Prinsco Representative.

Table 12: Installation Factors for Backfill Materials

Installation Type/Backfill Material	
Trench Installation	**$K_{\gamma E}$**
Class I	1.35
Class II	1.40
Class III	1.45
Embankment Installation	**$K_{\gamma E}$**
All Backfill Materials	1.5 - 1.0

Effective Area (A_{eff})

AASHTO LRFD design also allows two options for determining the effective area. The effective area of a profile wall flexible pipe is the amount of total area which is "effective" in withstanding a given compressive force in the pipe wall. Under this principal, it is assumed only a portion of the pipe wall resists compressive forces. One method is to determine the effective area experimentally by performing the stub compression test in accordance with AASHTO T341-10. The alternative method is to determine the effective area analytically. While the experimental method is typically used, for completeness this guide will describe the analytical method herein.

In order to determine the effective area analytically, AASHTO LRFD design method reduces the actual pipe profile to an idealized profile, both shown in Figure 1 below. The idealized profile is a representation of the actual profile but with straight sides and sharp corners. The thin straight elements that make up the idealized profile are analyzed to determine their effective width and resistance to buckling. Once the effective width of each element is calculated, a reduced effective area is calculated and used to analyze the structural integrity of the pipe section.

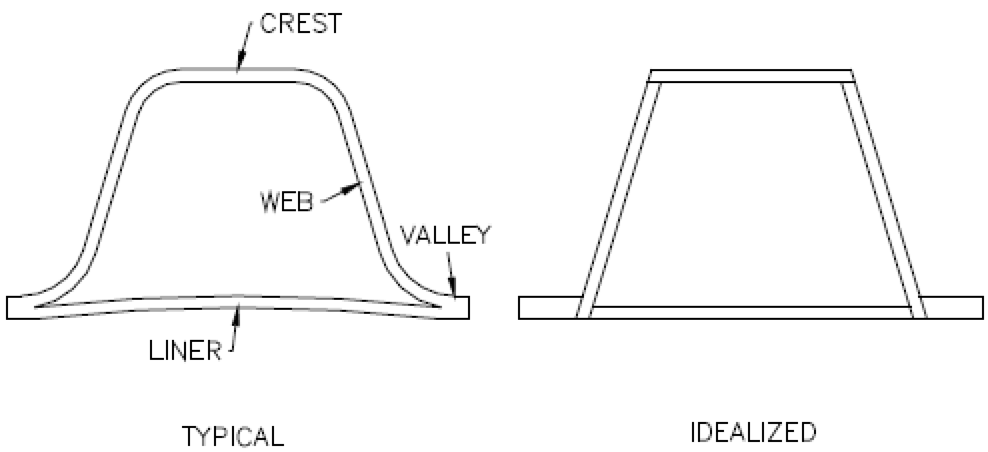


Figure 1: Typical Idealized Profile

In order to determine the effective area, the width (w) and thickness (t) of each component of the idealized profile geometry is measured and analyzed. These measured values are used as input data for the analysis. Equation 1 (from LRFD Eq.12.12.3.10.1b-1) below is the basic equation used to determine effective area.

Equation 1

$$A_{eff} = A_s - \frac{\sum(w - b_e)t}{\omega}$$

Where:

A_{eff} = effective wall area, in^2/inch of pipe

A_s = wall area, in^2/in

w = length of each individual profile element, in

t = thickness of each individual profile element, in

ω = profile pitch, in

Equation 2

$$b_e = \rho\ w$$

Where:

b_e = element effective width, in

w = length of each individual profile element, in

Equation 3

$$\rho = \frac{1 - \frac{0.22}{\lambda}}{\lambda} \leq 1$$

Where:

ρ = effective width factor

Equation 4

$$\lambda = \frac{w}{t}\sqrt{\frac{\varepsilon_{yc}}{k}} \geq 0.673$$

Where:

λ = slenderness factor

w = length of each individual profile element, in

t = thickness of each individual profile element, in

k = edge support coefficient, 4.0 for elements with both edges supported

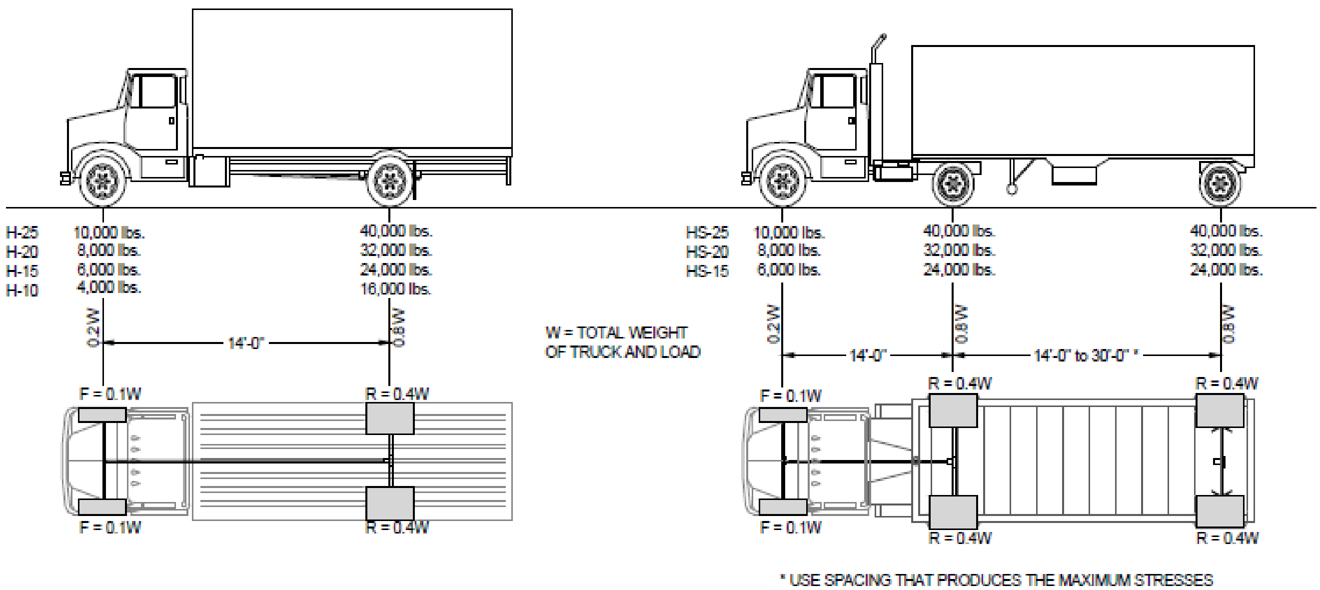
ε_{yc} = material strain limit, in/in (Table 4)

1.5 Design Loads

Two classifications of loads (i.e. live and dead loads) are considered in the design procedure outlined in this guide. Live loads (i.e. dynamic loads) change in position or magnitude, whereas dead loads (i.e. static loads) remain constant throughout the design life of the drainage system. Typical live loads include vehicular loads, usually from trucks, railroads, construction equipment, and aircraft. Typically the only dead load is the soil load; however, forces from high groundwater, surcharge, and foundations are also types of dead loads that should be taken into consideration when appropriate.

Live Loads

Vehicular loads are based on the AASHTO H- or HS- vehicle configurations. Figure 2 represents the two types of design truck configurations and the associated loading distribution. Table 13 provides the critical controlling load that is exerted at each wheel set or tire area, from the design truck configurations represented in Figure 2 or a design tandem rear axle truck (not shown). In railroad applications, the standard load is represented by the Cooper E-80 configuration at 80,000 lbs/ft (1167 kN/m) of track.



Source: AASHTO Standard Specifications for Highway Bridges

Figure 2: AASHTO Highway Loads

Table 13: AASHTO Highway Loads Carried by Wheel Set

Load Type	H-10 lbs (kN)	H-15 or HS-15 lbs (kN)	H-20 or HS-20 lbs (kN)	H-25 or HS-25 lbs (kN)
W(1)	20,000 (89.0)	30,000 (133.4)	40,000 (178.0)	50,000 (222.4)
F(2)	2,000 (8.9)	3,000 (13.3)	4,000 (17.8)	5,000 (22.2)
R(2)	8,000 (35.6)	12,000 (53.4)	16,000 (71.2)	20,000 (89.0)
R_{axle}(3)	16,000 (71.1)	24,000 (106.7)	32,000 (142.3)	40,000 (177.9)

Notes:
1) W is defined as the total vehicle weight (see Figure 2)
2) F is defined as the front tire load and R is defined as the rear tire configuration load (see Figure 2)
3) R_{axle} represents the truck's rear axle load (see Figure 2)

In applications where the pipe is buried relatively shallow it can experience an additional force from the rolling motion of the vehicle. To account for this additional force, the stationary vehicular load is multiplied by an "impact factor." To determine the impact factor for highway loads, the following AASHTO equation is provided:

Equation 5

$$IM = 33 \times (1.0 - 0.125H) \geq 0\%$$

Where:

IM = impact factor, %

H = burial depth, ft

Table 14 provides information about the resultant H-25 and E-80 vehicular forces at various cover heights with impact included in the shallow cover situations. Resultant loads for H-20 vehicles can be estimated by decreasing the values in Table 14 by 20%. These values are widely used throughout the industry, although values based on alternative computation methods can be substituted. The intensity of the vehicular load decreases as the depth increases, conversely, the area over which the force acts increases. As shown in Table 14, for H-25 loading, live load is negligible beyond 8-feet of fill. Table 14 also lists the live load distribution width showing this relationship for an AASHTO H-25 or HS-25 load. This width is based on AASHTO information and assumes that the pipe is installed perpendicular to the direction of traffic. Other AASHTO H or HS loads would have identical live load distribution widths. If desired, alternative ways of calculating this value may be used.

Table 14: Resultant Vehicular Forces Based on Burial Depth

Cover		AASHTO H-25 or HS-25(1)				Cooper E-80(1)	
		Live Load Transferred to Pipe (P_L)		Live Load Distribution Width (L_W)		Live Load Transferred to Pipe	
ft	m	psi	MPa	in	mm	psi	MPa
1	0.3	32.0	0.220	34	860	N/R	N/R
2	0.6	13.9	0.958	48	1210	26.39	0.182
3	0.9	7.6	0.524	61	1561	23.61	0.163
4	1.2	4.9	0.338	147	3740	18.40	0.127
5	1.5	3.5	0.241	161	4090	16.67	0.115
6	1.8	2.7	0.186	175	4441	15.63	0.108
7	2.1	2.1	0.145	189	4791	12.15	0.838
8	2.4	1.6	0.110	202	5142	11.11	0.766
10	3.0	Negligible	Negligible	N/A	N/A	7.64	0.527
12	3.7	Negligible	Negligible	N/A	N/A	5.56	0.383
14	4.3	Negligible	Negligible	N/A	N/A	4.17	0.288
16	4.9	Negligible	Negligible	N/A	N/A	3.47	0.239
18	5.5	Negligible	Negligible	N/A	N/A	2.78	0.192
20	6.1	Negligible	Negligible	N/A	N/A	2.08	0.143
22	6.7	Negligible	Negligible	N/A	N/A	1.91	0.132
24	7.3	Negligible	Negligible	N/A	N/A	1.74	0.120
26	7.9	Negligible	Negligible	N/A	N/A	1.39	0.095
28	8.5	Negligible	Negligible	N/A	N/A	1.04	0.072
30	9.1	Negligible	Negligible	N/A	N/A	0.69	0.048
35	10.7	Negligible	Negligible	N/A	N/A	Negligible	Negligible

Notes:
1) Includes impact.
2) N/R indicates that the cover height is not recommended.
3) N/A indicates that the information is not applicable.

Loads from aircraft vary widely in magnitude and landing gear configuration. The FAA Design Manual should be referenced for more specific information. Construction vehicles may pose a temporary, severe live load or a live load substantially less than the design load for some paving applications. Refer to Prinsco's *Installation Guide* for additional details.

Dead Loads

The overburden soil load on the pipe is calculated using the soil arch load (W_{sp}) design procedure. The soil arch load is considered a more accurate method of determining the overburden load when compared to the soil column or prism load method. The actual soil load is more closely approximated by the soil arch load than the soil column load method, which does not take into consideration the support associated by adjacent soil columns. Generally the soil arch load is the actual load on the pipe as a result of the dead load. In order to determine the soil arch load, it is necessary to determine the vertical arching factor (VAF).

Soil Arch Load (W_{sp})

The soil arch load (W_{sp}) approximates the actual soil load experienced by a flexible pipe. The soil arch load calculation uses a vertical arching factor (VAF) to reduce the prism load in order to account for the support provided by adjacent soil columns. The arch load is determined using the three-step procedure described below.

Step 1: The soil prism load is calculated by determining the weight of soil directly above the outside diameter of the pipe. Three equations are used to determine prism load. Each equation is used for different water table height scenarios. Equation 6 below (from LRFD Eq.12.12.3.7-3), is used to describe the prism load, P_{sp}, when the water table is at or below the spring line.

Equation 6

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_s}{144}$$

Equation 7 below (from LRFD Eq.12.12.3.7-1), is used to describe the soil prism load, P_{sp}, when the water table is at or above the finished grade elevation.

Equation 7

$$P_{sp} = \frac{\left(H + 0.11\frac{D_o}{12}\right)\gamma_b}{144}$$

Equation 8 below (from LRFD Eq.12.12.3.7-2), is used to describe the soil prism load, P_{sp}, when the water table is above the top of the pipe and below the finished grade elevation.

Equation 8

$$P_{sp} = \frac{\left(\left(H_W - \frac{D_o}{24}\right) + 0.11\frac{D_o}{12}\right)\gamma_b}{144} + \frac{H - \left(H_W - \frac{D_o}{24}\right)\gamma_s}{144}$$

Where:

P_{sp} = soil-prism pressure, psi

D_o = outside diameter of pipe, in (Table 1, 2, or 3)

γ_b = unit weight of buoyant soil, pcf

H = burial depth over top of pipe, ft

H_w = depth of water above springline of pipe, ft

γ_s = unit weight of soil, pcf

Step 2: The vertical arching factor (VAF) is determined. This factor is a function of the hoop stiffness of the pipe. The vertical arching factor is computed as shown in Equation 9 (from LRFD Eq.12.12.3.5-3).

Equation 9

$$VAF = 0.76 - 0.71 \times \left(\frac{S_h - 1.17}{S_h + 2.92}\right)$$

Where:

VAF= vertical arching factor, unitless

S_h = hoop stiffness factor (from LRFD Eq.12.12.3.5-4) = $(\Phi_s)(M_s)R / (E_p A_s)$

Φ_s = capacity modification factor for soil, (Table 10)

M_s = constrained soil modulus, psi (Table 5 or 6)

R = radius from center of pipe to centroid of pipe profile, in = $D_i/2+c$

D_i = inside diameter of pipe, in

c = distance from inside diameter to neutral axis, in (Table 1, 2, or 3)

E_p = modulus of elasticity of pipe, psi initial and long term (Table 4)

A_s = section area, in^2/in (Table 1, 2, or 3)

<u>Step 3:</u> After the geostatic load, P_{sp}, and the VAF have been determined, the soil arch load can be found as shown in Equation 10.

Equation 10

$$W_{sp} = (P_{sp}) \times (VAF)$$

Where:

W_{sp} = soil arch load, psi

P_{sp} = geostatic load, psi

VAF= vertical arching factor, unitless

It should be noted that the vertical arching factor (VAF) is a function of the pipe diameter and the pipe's hoop stiffness. Hoop stiffness is governed by the modulus of elasticity of the pipe material and the cross-sectional area of the pipe. Therefore rigid pipe material like RCP pipe, which has a very high modulus and cross sectional area, will have a larger VAF than a flexible pipe. In general, the following relationships hold for flexible and rigid pipes.

Flexible pipe: VAF < 1.0
Rigid pipe: VAF > 1.0

Hydrostatic Loads

If ground water is present at or above the springline (i.e. midpoint) of the pipe, its load must be taken into consideration. Equation 11 (from LRFD Eq.12.12.3.8-1) provides the method to calculate hydrostatic pressure. Where hydrostatic pressure is present, the geostatic load (P_{sp}) should be adjusted to account for the buoyant weight of the soil in the saturated zone.

Equation 11

$$P_W = \frac{\gamma_W K_{wa} H_W}{144}$$

Where:

P_w = hydrostatic pressure at springline of pipe, psi

γ_w = unit weight of water, 62.4 lb/ft^3

H_w = height of groundwater above springline of pipe, ft

K_{wa}= factor for uncertainty in ground water table, 1.0 – 1.3

Foundation Loads

Prinsco does not recommend installing drainage pipe under a foundation. However, in some cases where the pipe is installed adjacent to the foundation, the projection of the foundation load may influence pipe behavior. In these cases, the foundation load contribution must be added to the dead load before proceeding with the design process. Contact your local Prinsco Representative for additional foundation load design guidance.

1.6 Maximum and Minimum Cover Limitations

The design procedure described in the section below is provided for completeness and may provide an unnecessarily high level of detail for many installations. The information in this section is intended to provide quick access to many cover height questions with a high degree of conservatism. The most common concerns are minimum cover in trafficked areas, and maximum burial depths. Deeper burial depths or more shallow burial depths (for live loads) may be possible for special design and installation conditions.

Maximum Cover for Deep Burial

The maximum burial depth is significantly influenced by the type of backfill and level of compaction. Other factors influencing the burial depth includes the pipe diameter and pipe section properties. For Prinsco GOLDFLO® or ECOFLO 100® (corrugated dual wall HDPE) and for GOLDPRO Storm™ (corrugated dual wall HP), Tables 15 through 18, summarize the maximum allowable burial depths for the backfill, compaction, and diameters listed in the table. For maximum burial depths for Prinsco GOLDLINE® (single wall corrugated HDPE), contact your local Prinsco Representative.

Table 15: Maximum Burial Depth for HDPE Dual Wall with Class I Backfill

Maximum Burial Depth – Class I Backfill, ft (m)								
Diameter in. (mm)	Compacted				Uncompacted			
	Granite .75"	Granite 1.5"	Limestone	Quartzite	Granite .75"	Granite 1.5"	Limestone	Quartzite
4 (100)	65+ (19.8+)*	50 (15.2)	53 (16.2)	64 (19.5)	61 (18.6)	42 (12.8)	42 (12.8)	53 (16.2)
6 (150	65+ (19.8+)*	46 (14.0)	49 (14.9)	59 (18.0)	57 (17.4)	38 (11.6)	38 (11.6)	49 (14.9)
8 (200)	65+ (19.8+)*	47 (14.3)	49 (14.9)	60 (18.3)	57 (17.4)	38 (11.6)	38 (11.6)	49 (14.9)
10 (250)	63 (19.2)	46 (14.0)	48 (14.6)	59 (18.0)	56 (17.1)	37 (11.3)	37 (11.3)	48 (14.6)
12 (300)	54 (16.5)	39 (11.9)	41 (12.5)	50 (15.2)	48 (14.6)	31 (9.4)	31 (9.4)	41 (12.5)
15 (375)	52 (15.8)	37 (11.3)	40 (12.2)	48 (14.6)	46 (14.0)	30 (9.1)	30 (9.1)	40 (12.2)
18 (450)	47 (14.3)	33 (10.1)	35 (10.7)	43 (13.1)	41 (12.5)	27 (8.2)	27 (8.2)	35 (10.7)
24 (600)	53 (16.2)	38 (11.6)	40 (12.2)	49 (14.9)	46 (14.0)	30 (9.1)	30 (9.1)	40 (12.2)
30 (750)	50 (15.2)	36 (11.0)	38 (11.6)	46 (14.0)	44 (13.4)	28 (8.5)	28 (8.5)	38 (11.6)
36 (900)	40 (12.2)	28 (8.5)	30 (9.1)	37 (11.3)	35 (10.7)	22 (6.7)	22 (6.7)	30 (9.1)
42 (1050)	35 (10.7)	24 (7.3)	26 (7.9)	32 (9.8)	31 (9.4)	19 (5.8)	19 (5.8)	26 (7.9)
48 (1200)	36 (11.0)	25 (7.6)	27 (8.2)	33 (10.1)	31 (9.4)	20 (6.1)	20 (6.1)	27 (8.2)
60 (1500)	38 (11.6)	27 (8.2)	28 (8.5)	35 (10.7)	33 (10.1)	21 (6.4)	21 (6.4)	28 (8.5)

Table 16: Maximum Burial Depth for HDPE Dual Wall with Class II & III Backfill

Maximum Burial Depth - Class II & III Backfill, ft (m)				
Diameter in. (mm)	Class 2		Class 3	
	95%	90%	95%	90%**
4 (100)	44 (13.4)	31 (9.4)	30 (9.1)	16 (4.9)
6 (150)	39 (11.9)	27 (8.2)	26 (7.9)	15 (4.6)
8 (200)	39 (11.9)	27 (8.2)	27 (8.2)	15 (4.6)
10 (250)	39 (11.9)	27 (8.2)	26 (7.9)	15 (4.6)
12 (300)	31 (9.4)	22 (6.7)	21 (6.4)	14 (4.3)
15 (375)	30 (9.1)	20 (6.1)	20 (6.1)	13 (4.0)
18 (450)	26 (7.9)	17 (5.2)	17 (5.2)	12 (3.7)
24 (600)	30 (9.1)	21 (6.4)	20 (6.1)	14 (4.3)
30 (750)	28 (8.5)	19 (5.8)	19 (5.8)	13 (4.0)
36 (900)	21 (6.4)	14 (4.3)	14 (4.3)	9 (2.7)
42 (1050)	18 (5.5)	12 (3.7)	12 (3.7)	8 (2.4)
48 (1200)	18 (5.5)	12 (3.7)	12 (3.7)	8 (2.4)
60 (1500)	20 (6.1)	13 (4.0)	13 (4.0)	8 (2.4)

Table 17: Maximum Burial Depth for GOLDPRO Storm Dual Wall HP with Class I Backfill

Maximum Burial Depth – Class I Backfill, ft (m)								
Diameter in. (mm)	Compacted				Uncompacted			
	Granite .75”	Granite 1.5”	Limestone	Quartzite	Granite .75”	Granite 1.5”	Limestone	Quartzite
12 (300)	53 (16.2)	39 (11.9)	41 (12.5)	49 (14.9)	47 (14.3)	32 (9.8)	32 (9.8)	41 (12.5)
15 (375)	54 (16.5)	39 (11.9)	42 (12.8)	50 (15.2)	48 (14.6)	32 (9.8)	32 (9.8)	42 (12.8)
18 (450)	59 (18.0)	43 (13.1)	45 (13.7)	55 (16.8)	52 (15.8)	35 (10.7)	35 (10.7)	45 (13.7)
24 (600)	59 (18.0)	42 (12.8)	45 (13.7)	54 (16.5)	52 (15.8)	35 (10.7)	35 (10.7)	45 (13.7)
30 (750)	53 (16.2)	38 (11.6)	41 (12.5)	49 (14.9)	47 (14.3)	31 (9.4)	31 (9.4)	41 (12.5)
36 (900)	53 (16.2)	38 (11.6)	40 (12.2)	49 (14.9)	47 (14.3)	25 (7.6)	25 (7.6)	40 (12.2)
42 (1050)	38 (11.6)	27 (8.2)	28 (8.5)	35 (10.7)	33 (10.1)	21 (6.4)	21 (6.4)	28 (8.5)
48 (1200)	36 (11.0)	25 (7.6)	27 (8.2)	33 (10.1)	32 (9.8)	20 (6.1)	20 (6.1)	27 (8.2)
60 (1500)	41 (12.5)	29 (8.8)	30 (9.1)	37 (11.3)	32 (9.8)	23 (7.0)	23 (7.0)	30 (9.1)

Table 18: Maximum Burial Depth for GOLDPRO Storm Dual Wall HP with Class II & III Backfill

Maximum Burial Depth – Class II and Class III Backfill, ft (m)				
Diameter in. (mm)	Class 2		Class 3	
	95%	90%	95%	90%**
12 (300)	32 (9.8)	23 (7.0)	23 (7.0)	15 (4.6)
15 (375)	32 (9.8)	23 (7.0)	23 (7.0)	15 (4.6)
18 (450)	36 (11.0)	26 (7.9)	25 (7.6)	16 (4.9)
24 (600)	35 (10.7)	25 (7.6)	24 (7.3)	15 (4.6)
30 (750)	31 (9.4)	22 (6.7)	22 (6.7)	15 (4.6)
36 (900)	31 (9.4)	21 (6.4)	21 (6.4)	14 (4.3)
42 (1050)	20 (6.1)	13 (4.0)	13 (4.0)	9 (2.7)
48 (1200)	19 (5.8)	13 (4.0)	13 (4.0)	9 (2.7)
60 (1500)	21 (6.4)	14 (4.3)	14 (4.3)	10 (3.0)

Notes for Tables 15 through 18:

1) * Special design considerations should be made for these burial depths. Contact your local Prinsco Representative for more information.
2) ** For installations using a lower quality backfill material or lower compaction levels, pipe deflection may exceed the 5% design limit, however with proper control of the installation, the deflection may not be a limiting factor for the pipe. For installations where deflection limits are critical, higher compaction levels and/or a higher quality backfill material is recommended.
3) Calculations are based on a trench installation as described in this guide. Decreased burial depths for embankment installations may be experienced.
4) A design interval of 75 years was used when calculating burial depths.
5) Calculations assume no hydrostatic pressure and a density of 120 pcf (1922 kg/m3) for overburden material.
6) Backfill materials as defined by ASTM D2321 and compaction levels are standard proctor densities. Backfill material must be uniformly distributed around the pipe and between the corrugations.
7) Contact your local Prinsco Representative for special designs or deeper burial depths.

Minimum Cover for Trafficked Conditions

Pipe with diameters of 4- to 48-inch subjected to AASHTO HL-93, H-25, or HS-25 traffic loads must have at least one foot of cover over the pipe crown, while 60-inch diameter pipe must have at least 18 inches of cover. Table 19 below summarizes these minimum burial depth recommendations. These minimum cover heights are measured from the top of the pipe to the bottom of flexible paving or from the top of the pipe to the top of rigid paving. Structural backfill should be placed as directed by the design engineer. However, structural fill should extend (at a minimum) from the top of the pipe to 6-inches above the pipe. See Prinsco's *Installation Guide* for additional information.

Table 19: Minimum Burial Depth for Trafficked Conditions

Inside Diameter, in. (mm)	Minimum Cover, ft. (m)		Inside Diameter, in. (mm)	Minimum Cover, ft. (m)
3 (75)	1 (0.3)		18 (450)	1 (0.3)
4 (100)	1 (0.3)		24 (600)	1 (0.3)
6 (150)	1 (0.3)		30 (750)	1 (0.3)
8 (200)	1 (0.3)		36 (900)	1 (0.3)
10 (250)	1 (0.3)		42 (1050)	1 (0.3)
12 (300)	1 (0.3)		48 (1200)	1 (0.3)
15 (375)	1 (0.3)		60 (1500)	1.5 (0.46)

Note: Minimum covers for AASHTO HL-93, H-25, or HS-25 traffic loads, Class III backfill material compacted to 90% standard Proctor density around the pipe and a minimum of 6-inches over the pipe crown.

In cases where temporary construction traffic is necessary for paving or other special construction operations, Table 20 summarizes the minimum allowable covers for specific ground pressure.

Table 20: Temporary Minimum Cover

Vehicular Load At Surface, psi (kPa)	Temporary Minimum Cover for 4" – 48" Diameters, in. (mm)	Temporary Minimum Cover for 60" Diameter, in. (mm)
75 (517)	9 (230)	12 (300)
50 (345)	6 (150)	9 (230)
25 (172)	3 (80)	6 (150)

Note: Temporary minimum cover should only be employed during construction when the vehicle load is less than 75 psi.

1.7 Thermoplastic Pipe Design Procedure

This section describes the design methodology for corrugated polyethylene & polypropylene pipe based on the AASHTO Design Method. Design of corrugated polyethylene & polypropylene pipe in non-pressure applications involves calculating wall thrust, bending strain, buckling, and strain limits based on combined tension and compressive conditions. Criteria for pipe, installation conditions, and loads from Section 1.5 are required for this procedure; references are made to areas where the required information can be found.

In this design procedure, the pipe is evaluated at various limit states to ensure the objectives of constructability, safety, and serviceability is obtained. The pipe is first analyzed for the service limit state with restrictions on stress and deformation. Next the pipe is evaluated at strength limit states for wall area, buckling, thrust, and combined strain. Each condition is evaluated to ensure that strength and stability, both global and local, are provided to resist the specified load combinations expected.

Minimum and Maximum burial depths depend on the application, product, backfill material, and compaction level. Contact your local Prinsco Representative for example design calculations.

It is important to note that the following design procedure is intended for pipe and does not apply to fittings that are fabricated from Prinsco dual wall pipe. Prinsco recommends a maximum burial depth of 8 feet for fabricated fittings unless special considerations are made. Contact your local Prinsco Representative for additional information.

Thrust Strain Limits

The thrust strain limit must satisfy Equation 12 (from LRFD Eq.12.12.3.10.1d-1) below in order to be considered a structurally "safe" design. The factored compressive strain must be less than or equal to the allowable strain limit as modified by the resistance factor.

Equation 12

$$\varepsilon_{uc} \leq \phi_T \varepsilon_{yc}$$

Where:

ε_{uc} = factored compressive strain, in/in

ϕ_T = resistance factor for thrust effects

ε_{yc} = factored compression strain limit of pipe wall, in/in

The ultimate thrust load on the pipe in its simplest form is determined by multiplying the total factored load on the pipe (which may include soil loads, vehicular loads, and hydrostatic forces) by the outside diameter of the pipe. Thus, the stress in the pipe wall is simply the thrust load divided by the effective cross-sectional area of the pipe wall. The pipe must be able to withstand these forces in both tension and compression in order for it to remain structurally stable. The simplest form of the thrust load is shown in Equation 13 (from LRFD Eq.12.12.3.10.1c-3) below.

Equation 13

$$T_u = \frac{P_u D_o}{2}$$

Where:

T_u = Factored thrust per unit length, lb/in

P_u = Factored load, psi

D_o = outside diameter, in (Table 1, 2, or 3)

The equation for factored thrust (T_u) load when combining the soil load, live loads, and water loads becomes more complex. Additionally the live loads only need to be evaluated with the short term modulus of the HDPE & HP since these loads are dynamic in nature. To determine the thrust strain, it is necessary to account for all loading types and divide the factored thrust by the product of the effective area of the profile and the appropriate modulus as shown in Equation 14 (from LRFD Eq.12.12.3.10.1c-1) below.

Equation 14

$$\varepsilon_{uc} = \frac{T_u}{A_{eff} E_p} = \left[\frac{\eta_{EV} (\gamma_{EV} K_{\gamma E} K_2 VAF (P_{sp}) + \gamma_{WA} P_W)}{A_{eff} E_l} + \frac{\eta_{LL} \gamma_{LL} P_L C_L}{A_{eff} E_s} \right] \left(\frac{D_o}{2} \right)$$

Where:

ε_{uc} = factored compressive strain , in/in

T_u = Factored thrust per unit length, lb/in

A_{eff} = effective wall area, in^2/inch of pipe

W_{sp} = soil arch load, psi (Equation 11)

η_{EV} = load modifier, earth fill, (Table 11)

γ_{EV} = load factor, vertical earth pressure, (Table 9)

γ_{WA} = load factor, water load (Table 9)

η_{LL} = load modifier, live load (Table 11)

γ_{LL} = load factor, live load (Table 9)

P_{sp} = soil-prism pressure, psi

P_L = live load transferred to pipe, psi (Table 14)

C_L = live load distribution coefficient

= the lesser of L_w/D_o or 1.0, where

L_w = live load distribution width at the crown, in (Table 14)

D_o = outside diameter, in (Table 1, 2, or 3)

P_w = hydrostatic pressure at springline of pipe, psi (Equation 11)

E_l =Long-term Modulus, psi (Table 4)

E_s =Short-term Modulus, psi (Table 4)

$K_{\gamma E}$ =Installation Factor, (Table 12)

K_2 = coefficient for thrust variations around the circumference, springline = 1, crown = 0.6

If the condition shown in Equation 12 is satisfied, the system is considered safe relative to the compressive strain associated with thrust loads.

$$\varepsilon_{uc} \leq \phi_T \varepsilon_{yc} \;\; \therefore OK$$

General Buckling Strain Limits

The thrust strain limit must satisfy Equation 15 (from LRFD Eq.12.12.3.10.1e-1) below in order to be considered a structurally "safe" design. The factored compressive strain must be less than or equal to the buckling strain capacity as modified by the resistance factor.

Equation 15

$$\varepsilon_{uc} \leq \phi_{bck} \varepsilon_{bck}$$

Where:

ε_{uc} = factored compressive strain (Equation 14), in/in

Φ_{bck}=resistance factor for buckling effects (Table 10)

ε_{bck} =buckling strain capacity, in/in

The overburden loads on buried pipe can lead to significant compressive hoop thrust around the pipe circumference, which in turn can lead to buckling instability. Therefore, it is necessary to check for the buckling instability with Equation 16 (from LRFD Eq.12.12.3.10.1e-2) below.

Equation 16

$$\varepsilon_{bck} = \frac{1.2C_n\left(E_pI_p\right)^{\frac{1}{3}}}{A_{eff}E_p}\left(\frac{\phi_s M_s(1-2\nu)}{(1-\nu)^2}\right)^{\frac{2}{3}} R_h$$

In which:

Equation 17

$$R_h = \frac{11.4}{11 + \frac{D}{12H}}$$

Where:

Φ_{bck} =resistance factor for buckling effects (Table 10)

ε_{bck} =buckling strain capacity, in/in

R_h = correction factor for backfill geometry

C_n = calibration factor to account for nonlinear effects = 0.55

E_p = short- or long-term modulus of pipe material (Table 4)

I_p = moment of inertia of pipe profile per unit length of pipe (Table 1, 2, or 3), in^4/in

A_{eff} = effective area of pipe profile per unit length of pipe, in^2/in

Φ_s = resistance factor for soil pressure effects (Table 10)

M_s = constrained soil modulus, (Table 5 or 6)

ν = Poisson's ratio of soil

D = diameter to centroid of pipe profile, in

H = depth of fill over top of pipe, ft

If the condition shown in Equation 15 is satisfied, the system is considered safe relative to the general buckling strain limit associated with compressive hoop thrust.

$$\varepsilon_{uc} \leq \phi_{bck}\varepsilon_{bck} \therefore OK$$

Combined Strain

To make sure there is adequate deflection capacity, the combined strain at the extreme fibers of the pipe profile must be evaluated at allowable deflection limits. By way of background as pipe deflects, a bending strain is induced in the pipe wall. At the crown and invert positions the outer surface is in compression and the inner surface is in tension. Similarly, at the spring line positions the outer surface is in tension and the inner surface is in compression. It is noted for live loads the peak bending strain occurs at the crown of the pipe and for deep burial conditions the peak bending strain occurs near the invert of the pipe. In addition to the strain caused by deflection, the pipe experiences a compressive strain in the hoop direction (ε_{uc}). The combined strain formulas in this section combine the worst-case scenarios to ensure the design does not exceed the allowable strain limits.

Combined Bending and Thrust Strain in Tension Conditions

The combined strain in maximum tension locations must satisfy Equation 18 (from LRFD Eq.12.12.3.10.2b-1) below in order to be considered a structurally "safe" design.

Equation 18

$$\varepsilon_f - \varepsilon_{uc} < \phi_f \varepsilon_{yt}$$

In which:

Equation 19

$$\varepsilon_f = \gamma_{ev} D_f \left(\frac{c}{R}\right)\left(\frac{\Delta_f}{D}\right)$$

In which:

Equation 20

$$\Delta_f = \Delta_a - \varepsilon_{sc} D$$

In which:

Equation 21

$$\varepsilon_{sc} = \frac{T_s}{A_{eff} E_P} = \left[\frac{(K_2 VAF(P_{sp}) + P_W)}{E_l A_{eff}} + \frac{C_L P_L}{E_s A_{eff}}\right]\left(\frac{D_o}{2}\right)$$

Where:

ε_f = factored bending strain, in/in

ε_{uc}= factored compressive strain due to thrust, in/in

Φ_f = resistance factor for flexure (Table 10)

ε_{yt} = service long term tension strain limit of pipe (Table 4)

γ_{ev} = load factor for vertical pressure for dead load (Table 9)

D_f = shape factor (Table 8)

c = greatest distance from centroid to extreme fiber of profile, in (Table 1, 2, or 3)

R = radius from center of pipe to centroid of pipe profile, in

Δ_f = reduction in vertical diameter due to flexure, in

D = diameter to centroid of pipe profile, in

Δ_a = total allowable reduction in vertical diameter, in

ε_{sc} = service compressive strain due to thrust, in/in

T_s = service thrust per unit length, lb/in

E_p= short- or long-term modulus of pipe material (Table 4)

A_{eff} = effective wall area, in^2/inch of pipe

P_L = live load transferred to pipe, psi (Table 14)

C_L = live load distribution coefficient

= the lesser of L_w/D_o or 1.0, where

L_w = live load distribution width at the crown, in (Table 14)

D_o = outside diameter, in (Table 1, 2, or 3)

P_w = hydrostatic pressure at springline of pipe, psi (Equation 11)

E_l = long-term modulus, psi (Table 4)

E_s = short-term modulus, psi (Table 4)

P_{sp} = soil-prism pressure, psi

K_2 = coefficient for thrust variations around the circumference, springline = 1, crown = 0.6

VAF = vertical arching factor (Equation 9)

If the condition shown in Equation 18 is satisfied, the system is considered safe relative to the combined strain in maximum tension associated with compressive hoop thrust and deflection.

$$\varepsilon_f - \varepsilon_{uc} < \phi_f \varepsilon_{yt} \therefore OK$$

It should be noted that the service compressive strain and other service conditions should be evaluated for all service conditions.

Combined Bending and Thrust Strain in Compression Conditions

The combined strain in maximum compression locations must satisfy Equation 22 (from LRFD Eq.12.12.3.10.2b-2) below in order to be considered a structurally “safe” design.

Equation 22

$$\varepsilon_f + \varepsilon_{uc} < \phi_T (1.5 \varepsilon_{yc})$$

Where:

ε_f = factored bending strain, in/in (Equation 19)

ε_{uc} = factored compressive strain , in/in (Equation 14)

Φ_T = resistance factor for thrust effects

ε_{yc} = factored compressive strain limit of pipe wall, in/in (Table 4)

If the condition shown in Equation 22 is satisfied, the system is considered safe relative to the combined strain in maximum compression associated with compressive hoop thrust and deflection.

$$\varepsilon_f + \varepsilon_{uc} < \phi_T (1.5 \varepsilon_{yc}) \therefore OK$$

Deflection

Deflection is the change in diameter that results when a load is applied to a flexible pipe. Pipe installations are typically designed for a maximum vertical deflection of 5%, but vertical deflection of up to 7.5% of the base inside diameter are allowable provided the design constraints discussed above are met. It should be noted that the base inside diameter is the nominal diameter less manufacturing and out-of-roundness tolerances inherent to the manufacturing process.

The deflection lag factor (D_L) plays an important role in predicting the deflection. AASHTO has established a range of 1 to 6 for the deflection lag factor and recommends a value of 1.5 for embankment conditions. However, for trench installations, the contribution of the trench side wall and the empirical data of historical usage have proven that a deflection lag factor of 1.0 accurately predicts deflection when installed in accordance with Prinsco’s recommendations. It is noted in conditions where the water table is expected to extend above the top of the pipe, consideration should be given to use a deflection lag factor of 1.2 to 1.5.

Total deflection, as predicted by Equation 23 below (from LRFD Eq.12.12.2.2-2), is the sum of the vertical deflection and the circumferential shortening of the pipe due to soil and live loads.

Equation 23

$$\Delta_t = \left[\frac{K_B(D_L P_{sp} + C_L P_L)D_o}{(\frac{E_p I_p}{R^3} + 0.061M_s)}\right] + \varepsilon_{sc} D = \left[\frac{D_L K_B\, P_{sp}(D_o)}{(\frac{E_l I_p}{R^3} + 0.061M_s)}\right] + \left[\frac{C_L P_L K_B(D_o)}{(\frac{E_s I_p}{R^3} + 0.061M_s)}\right] + \varepsilon_{sc} D$$

Where:

Δ_T = total reduction in vertical diameter, in

D = diameter to centroid of pipe profile, in

ε_{sc} = service compressive strain due to thrust, in/in (Equation 21)

E_p = short- or long-term modulus of pipe material (Table 4)

P_L = live load transferred to pipe, psi (Table 14)

C_L = live load distribution coefficient

= the lesser of L_w/D_o or 1.0, where

D_o = outside diameter, in (Table 1, 2, or 3)

E_l = long-term modulus, psi (Table 4)

E_s = short-term modulus, psi (Table 4)

K_B = bedding coefficient, 0.1 typical

D_L = deflection lag factor, values as defined above

P_{sp} = soil-prism pressure, psi

I_p = moment of inertia for pipe, in^4/in (Table 1, 2, or 3)

R = radius from center of pipe to centroid of pipe profile, in

M_s = constrained soil modulus, (Table 5 or 6)

1.8 Research and Installations

Corrugated polyethylene pipe has been heavily researched in the laboratory and through actual installations. This section summarizes the findings of some of those projects. Additional information and reports about polyethylene and polypropylene can be obtained from the Plastic Pipe Institute.

1. **“Analysis of the Performance of a Buried High Density Polyethylene Pipe.”** Written by Naila Hashash and Ernest Selig, University of Massachusetts, and published in *Structural Performance of Flexible Pipes,* edited by Sargand, Mitchell, and Hurd, October 1990, pp.95 - 103.

In 1988, the Pennsylvania Department of Transportation began a study to evaluate the behavior of corrugated polyethylene pipe backfilled with crushed stone under a 100 foot (30.5m) burial depth. This document, which is a status report of the pipe condition 722 days after installation, summarizes one of the most heavily instrumented pipe installations to date. Measured vertical deflection was 4.6% and horizontal deflection was 0.6%.Much of this was due to a slight (1.6%) circumferential shortening. This amount of deflection is well within the 7.5% generally accepted limit. Soil arching reduced the load on the pipe by 77% which shows that the soil column load is a very conservative method to estimate this load component.

2. **“Field Performance of Corrugated Polyethylene Pipe.”** Written by John Hurd, Ohio Department of Transportation, and published in *Public Works*, October 1987.

This article summarizes the results of a field study conducted in 1985 on 172 culvert installations. These installations represented real-world applications where backfill procedures may or may not have been conducted in accordance with standard ODOT recommendations. Regardless, the primary findings regarding structural integrity were that shallow cover, even with heavy truck traffic, did not appear to cause significant amounts of deflection; what deflection that did occur seemed to be due to installation.

3. **"Laboratory Test of Buried Pipe in Hoop Compression."** Written by Ernest Selig, Leonard DiFrancesco, and Timothy McGrath, and published in *Buried Plastic Pipe Technology* - 2nd Volume, 1994, pp.119 - 132.

The project involved developing a fixture so as to subject the pipe to purely compressive forces. A pressure of 55 psi (379 kPa) was reached at which time equipment problems developed. The authors indicated this pressure was the equivalent of 100 feet (30.5m) of cover in other tests they had performed. At this pressure, the pipe also experienced a 3% circumferential shortening which resulted in a significant beneficial soil arching.

4. **"Pipe Deflections - A Redeemable Asset."** Written by Dr. Lester Gabriel and published in *Structural Performance of Flexible Pipes*, edited by Sargand, Mitchell, and Hurd, October 1990, pp.1 - 6.

This paper provides an easy-to-read description of the role of deflection in properly performing flexible pipe. Deflection is not a liability, but a behavior that forces the backfill material to take on a disproportionate amount of load.

Deflection allows flexible pipe to be installed in applications with surprisingly deep burials.

5. **"Short-term Versus Long-term Pipe Ring Stiffness in the Design of Buried Plastic Sewer Pipes."** Written by Lars-Eric Janson and published in *Pipeline Design and Installation*, proceedings from the International Conference sponsored by the Pipeline Planning Committee of the Pipeline Division of the American Society of Civil Engineers, March 1990, pp.160 - 167.

This report describes the viscoelastic behavior of polyethylene. The author suggests the use of short-term properties when the pipe is backfilled in friction soils or firm silty/clayey soils.

6. **"Stress Relaxation Characteristics of the HDPE Pipe-Soil System."** Written by Larry Petroff and published in *Pipeline Design and Installation*, proceedings from the International Conference sponsored by the Pipeline Planning Committee of the Pipeline Division of the American Society of Civil Engineers, March 1990, pp.280-293.

This is an excellent report on the viscoelastic nature of polyethylene and discusses both creep and stress relaxation behaviors. One of the major points made is how deflection decreases with time; over 80% of the total deflection that a pipe will experience throughout its life will occur within the first 30 days. Petroff also indicated that the highest stresses for polyethylene pipe buried in a compacted granular material occur soon after installation but relax soon thereafter.

7. **"Stiffness of HDPE Pipe in Ring Bending."** Written by Timothy McGrath, Ernest Selig, and Leonard DiFrancesco, and published in *Buried Plastic Pipe Technology- 2nd Volume*, 1994, pp.195 - 205.

This project was conducted to determine how or if the modulus of elasticity changes over time. The pipe was deflected and held in position to generate a stress/strain curve. Although the results gave the appearance that the material was losing strength over time, repeated incremental loads caused the pipe to respond with its short-term modulus.

8. **“Structural Performance of Three-Foot Corrugated Polyethylene Pipe Buried Under High Soil Cover.”** Written by Reynold Watkins and published in *Structural Performance of Flexible Pipes*, edited by Sargand, Mitchell, and Hurd, October 1990, pp.105 - 107.

A three-foot (900mm) diameter corrugated polyethylene pipe was tested in a load cell to determine if it performed as well as the smaller sizes. The author recognizes the effects of stress relaxation. The report concludes “There is no reason why corrugated polyethylene pipes of three-foot diameter cannot perform structurally under high soil cover provided that a good granular pipe zone backfill is carefully placed and compacted.” This is consistent with the backfill and material recommendations set forth in previous sections.