CHAPTER 6

STRUCTURAL DESIGN

INTRODUCTION

Corrugated steel conduits, long recognized for outstanding structural strength under the heaviest of underground loadings, are now understood to be a complex composite – the result of soil and steel interaction.

Soil-steel interaction means that a flexible steel conduit acts with the surrounding soil fill to support the loads. Modern research has shown that the ideal underground structure places much of the load on the soil around and over it. Corrugated steel structures approach this ideal condition.

Design methods for buried corrugated steel pipe are turning more toward the all-important soil component of the composite soil-steel system. While still conservative in reference to the soil structure, the present design procedures recognize it and open the way to future developments.

Structural plate underpass and stream crossing. The sizes of these structures are 11.69 m span x 7.42 m rise horizontal ellipse and 6.94 m diameter round pipe respectively. The round pipe had 28.2 m of cover.

The design of corrugated steel pipe has evolved from the semiempirical Marston-Spangler method and the ring compression method to more sophisticated methods which recognize compressive failure by crushing or buckling instability. Bending moments and the development of plastic hinges are usually disregarded, although the current Canadian Highway Bridge Design Code (CHBDC) does account for combined bending moment and axial thrust during construction.

This handbook uses the traditional AISI Method for the design of structures with a diameter or span equal to or less than 3 m. The AISI Method can modify the wall thrust of the conduit in the ring compression method with an arching factor K, when the height of cover is greater than the span, and provides a calculation for elastic and inelastic buckling stress.

For structures with a diameter or span greater than 3 m, the AISI Method or the CHBDC method are used. The AISI method has served designers well for many years, and can continue to be used unless the CHBDC method is specified. The CHBDC method provides a more up to date approach to the determination of thrust and buckling resistance, and is based on ultimate strength principles rather than working stress or service load design.

RESEARCH AND DEVELOPMENT OF BURIED STRUCTURES

Earliest strength tests on corrugated steel pipe were quite crude. The tests included circus elephants balanced on unburied pipe and threshing rigs placed over shallow buried pipe.

Laboratory soil box and hydraulic tests by Talbot, Fowler and others, followed later. Fill loads were measured on buried pipe and on their foundations at Iowa State College (Marston, Spangler and others, 1913) and at the University of North Carolina (Braune, Cain, Janda) in cooperation with the U.S. Bureau of Public Roads.

Large-scale field tests measuring dead loads were run in 1923 on the Illinois Central Railroad by the American Railway Engineering Association (AREA) (Figure 6.1). Measurements with earth pressure cells showed that flexible corrugated pipes carried only 60 percent of the 10.7 m column (or prism) of fill above it, while adjacent soil carried the remaining 40 percent of the load. These tests demonstrated for the first time that a flexible conduit and compacted earth embankment can combine to act as a composite structure.

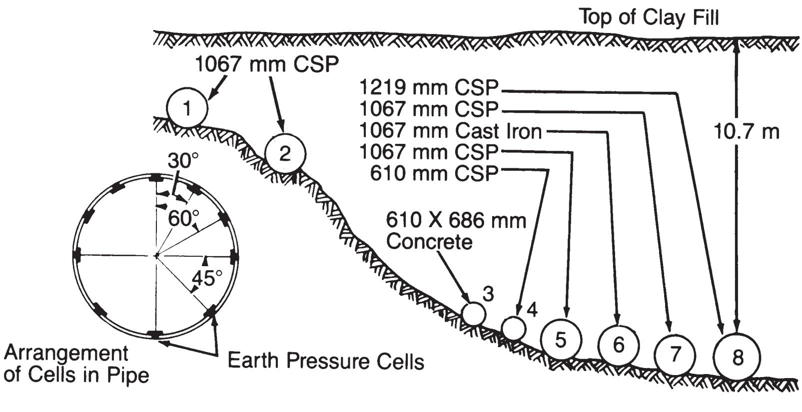


Figure 6.1 American Railway Engineering Association tests on culvert pipe.

Early efforts to rationalize the load-carrying performance of flexible conduits led to the concept of passive side pressures and the Iowa Formula for predicting deflection. Although seldom used for today's design, it has provided insight into the behavior of corrugated steel pipe.

In the 1960's the concept of a thin compression ring supported by soil pressures was introduced. This fundamental concept proved compatible with experience and provided a path to rational design criteria. The national interest in blast effects from nuclear devices supplied a wealth of research and development on buried flexible structures. This work clearly showed the potential for more efficient designs of buried corrugated steel structures. Further extensive research was sponsored by the American Iron and Steel Institute, between 1967 and 1970, at Utah State University under the direction of Dr. Reynold K. Watkins. Procedures, results and conclusions are described in condensed form in Appendix A at the end of this chapter.

The State of California conducted a very significant research project in 1975. Called the D.B. culvert, it was a 3050 mm diameter structural plate pipe, with a 2.8 mm wall thickness, under almost 61 m of fill. It was perhaps the only such pipe drastically underdesigned and expected to fail. The performance data from this structure contributed greatly to the development and verification of new design tools.

Since then, several procedures have been developed using finite element methods: CANDE (Culvert Analysis, Design) is an FHWA (Federal Highway Administration) sponsored computer program by M. Katona, et al. The SCI (Soil/Culvert Interaction) Design Method, by J. M. Duncan, utilizes design graphs and formulas based on finite element analyses.

FHWA Report RD77-131 summarizes the status of Long Span Corrugated Structures. This new family of very large structures has extended the range of corrugated steel to spans over 15 m. Because standard design criteria were not fully applicable to long spans, special or modified design standards were established. This chapter includes the current long span criteria. Research and development on the effects of dead and live loads on the behavior of buried steel structures continues to be a subject of interest.

Stormwater detention tank.

MATERIALS

Steel and Corrugation Properties

Mechanical properties of sheet and plate for structural plate corrugated steel pipe, and deep corrugated structural plate products are provided in Table 6.1.

Section properties for corrugated steel pipe, spiral rib pipe and structural plate corrugated steel pipe products are given in Table 6.2 and Table 6.3.

Ultimate seam strengths for riveted CSP, structural plate CSP, deep corrugated structural plate are provided in Tables 6.4a through 6.4d.

Table 6.1 Mechanical properties of sheet and plate for structural plate products

Steel	Min Tensile Strength, MPa	Min Yield Strength, MPa	Min Elongation in 50 mm	Modulus of Elasticity, MPa
SPCSP	290	195	30%	200×10^3
DCSP	380	275	25%	200×10^3

Note: These mechanical properties are for virgin material prior to corrugating and galvanizing and are conducted in accordance with the requirements of ASTM Standard A370. Corrugated steel with mechanical properties greater than the minimum requirements may be used. The minimum yield strength used for design shall be 230 MPa for structural plate and 300 MPa for deep corrugated structural plate, which shall be achieved through cold working.

Table 6.2 Section properties for corrugated steel pipe, spiral rib pipe and structural plate corrugated steel pipe products

Corru-gation profile, mm	Specified Thickness, mm											
	1.0	1.3	1.6	2.0	2.8	3.0	3.5	4.0	4.2	5.0	6.0	7.0
	Moment of Inertia, I, mm^4/mm											
38x6.5	3.70	5.11	6.46	8.58								
68x13	16.49	22.61	28.37	37.11	54.57		70.16		86.71			
76x25	75.84	103.96	130.40	170.40	249.73		319.77		393.12			
125x25			133.30	173.72	253.24		322.74		394.84			
152x51						1057.25		1457.56		1867.12	2278.31	2675.11
19x19x190*			58.83	77.67	117.17							
	Cross-sectional Wall Area, A, mm^2/mm											
38x6.5	0.896	1.187	1.484	1.929								
68x13	0.885	1.209	1.512	1.966	2.852		3.621		4.411			
76x25	1.016	1.389	1.736	2.259	3.281		4.169		5.084			
125x25			1.549	2.014	2.923		3.711		4.521			
152x51						3.522		4.828		6.149	7.461	8.712
19x19x190*			1.082	1.513	2.523							
	Radius of Gyration, r, mm											
38x65	2.063	2.075	2.087	2.109								
68x13	4.316	4.324	4.332	4.345	4.374		4.402		4.433			
76x25	8.639	8.653	8.666	8.685	8.724		8.758		8.794			
125x25			9.277	9.287	9.308		9.326		9.345			
152x51						17.326		17.375		17.425	17.475	17.523
19x19x190*			7.375	7.164	6.815							

* Ribbed pipe. Properties are effective values.

Table 6.3

Section properties for deep corrugated structural plate products

Type I: **381 x 140 mm** corrugation profile

Specified Thickness mm	Moment of Inertia, I mm^4/mm	Area, A mm^2/mm	Radius of Gyration, r mm	Plastic Section Modulus, Z mm^3/mm
2.81	9096.2	3.720	49.45	165.25
3.53	11710.7	4.783	49.48	212.67
4.27	14333.9	5.846	49.51	260.15
4.79	16039.0	6.536	49.53	291.03
5.54	18743.3	7.628	49.57	339.93
6.23	21445.9	8.716	49.60	388.77
7.11	24164.6	9.808	49.63	437.85

Type II: **400 x 150 mm** corrugation profile

Specified Thickness mm	Moment of Inertia, I mm^4/mm	Area, A mm^2/mm	Radius of Gyration, r mm	Plastic Section Modulus, Z mm^3/mm
4.3	16186	5.792	52.86	273.62
5.0	19060	6.811	52.90	322.05
6.0	23154	8.260	52.95	391.01
7.0	27071	9.640	52.99	456.91
8.0	30759	10.935	53.04	518.88

Table 6.4a

Riveted CSP - Ultimate longitudinal seam strength (kN/m)

Specified Thickness mm	8 mm Rivets		10 mm Rivets			12 mm Rivets
	68 x 13 mm		68 x 13 mm		76 x 25 mm	76 x 25 mm
	Single	Double	Single	Double	Double	Double
1.3	148					
1.6	236	274			387	
2.0	261	401			499	
2.8			341	682		769
3.5			356	712		921
4.2			372	746		1023

Table 6.4b

152 x 51 mm bolted structural plate
Ultimate longitudinal seam strength (kN/m)

Specified Thickness mm	Bolt per Corrugation 2	3	4	Bolt Diameter mm
3.0	745			19
4.0	1120			19
5.0	1470	1650		19
6.0	1840	2135		19
7.0	2100	2660	3200	19

Table 6.4c

Ultimate longitudinal seam strength (kN/m)
Type I: **381 x 140 mm** bolted structural plate

Specified Thickness mm	6 Bolts per Corrugation S_S†	6 Bolts per Corrugation S_M*	Bolt Diameter mm
3.53	905	Consult Manufacturer	19
4.27	1182		19
4.79	1357		19
5.54	1634		19
6.32	1926		19
7.11	2101		19

† per ASTM A796.
* Proprietary design values.

Table 6.4d

Ultimate longitudinal seam strength (kN/m)
Type II: **400 x 150 mm** bolted structural plate

Specified Thickness mm	Bolt Diameter mm	Compressive Seam Strength* S_S kN/m
4.0	19	1191
5.0	19	1735
6.0	19	2063
7.0	19	2238
8.0	19	2238
7.0	22	2688
8.0	22	2688

* Proprietary design values.

Soil Properties

Soils are classified in accordance with Table 6.5. The secant modulus for various soils is in Table 6.6.

Table 6.5

Soil classification for E_s

Soil Group	Grain Size	Soil Types	Unified Soil Classification Symbol*
I	Coarse	Well Graded Gravel or Sandy Gravel	GW
		Poorly Graded Gravel or Sandy Gravel	GP
		Well Graded Sand or Gravelly Sand	SW
		Poorly Graded Sand or Gravelly	SP
II	Medium	Clayey Gravel or Clayey-sandy Gravel	GC
		Clayey Sand or Clayey Gravelly Sand	SC
		Silty Sand or Silty Gravelly Sand	SM

* According to ASTM D2487.92 *Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)*

Table 6.6 Values of E_s for various soils

Soil Group Number*	Standard Proctor Density**	Secant Modulus of Soil, E_s, MPa
I	85%	6
	90%	12
	95%	24
	100%	30
II	85%	3
	90%	6
	95%	12
	100%	15

* According to Table - 6.5

** According to ASTM D698-91 *Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort*

Material Unit Weights

The unit weights of various materials are listed in Table 6.7.

Table 6.7 Unit material weights

Material	Unit Weight kN/m^3
Bituminous Wearing Surface	23.5
Clay and Silt	19.0
Coarse Grained Soil, Rock Fill	21.0
CHBDC - Coarse Grained (Granular) Soil	22.0
Crushed Rock, Glacial Till	22.0
Fine Grained or Sandy Soil	20.0

DESIGN OF BURIED STRUCTURES WITH SPANS UP TO 3 m

A procedure for the structural design of pipe is provided by ASTM A 796 / A 796M, "Standard Practice for Structural Design of Corrugated Steel Pipe, Pipe-Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications." The practice applies to structures installed in accordance with ASTM A 798 / A 798M, "Standard Practice for Installing Factory-Made Corrugated Steel Pipe for Sewers and Other Applications", and ASTM A 807 / A 807M, "Standard Practice for Installing Corrugated Steel Structural Plate Pipe for Sewers and Other Applications." Another similar method is provided by the American Association of State Highway Transportation Officials (AASHTO) Standard Specifications for Highway Bridges. These practices are frequently referenced in project specifications.

In 1967, the American Iron and Steel Institute (AISI) published the first edition of the Handbook of Steel Drainage and Highway Construction Products. The handbook outlined a working stress or service load method for the structural design of corrugated steel pipes and is the basis for most height of cover tables in use today. In this, the second Canadian Edition, the AISI method will be used for the structural design of corrugated steel pipes with a diameter or span that is less than or equal to 3 m.

The design procedures in ASTM A 796 and in AASHTO are similar to the AISI method described below, but they do differ in several respects. For the dead load, ASTM and AASHTO use the weight of the entire prism of soil above a horizontal plane at the top of the pipe and do not recognize the load reduction factor. They also use a different adaptation of the buckling equations. They provide separate flexibility factors for both trench and embankment conditions, some of which are more conservative than those listed here. They also include more specific information on acceptable soil types. In spite of all these differences, the resulting designs will usually not differ greatly from those provided in this chapter.

Minimum Clear Spacing Between Structures

When two or more steel drainage structures are installed in parallel lines, the space between them must be adequate to allow proper backfill placement, particularly in the haunch and compaction area. The minimum spacing requirement depends upon the shape and size of the structure as well as the type of backfill materials,, as indicated in Figure 6.2.

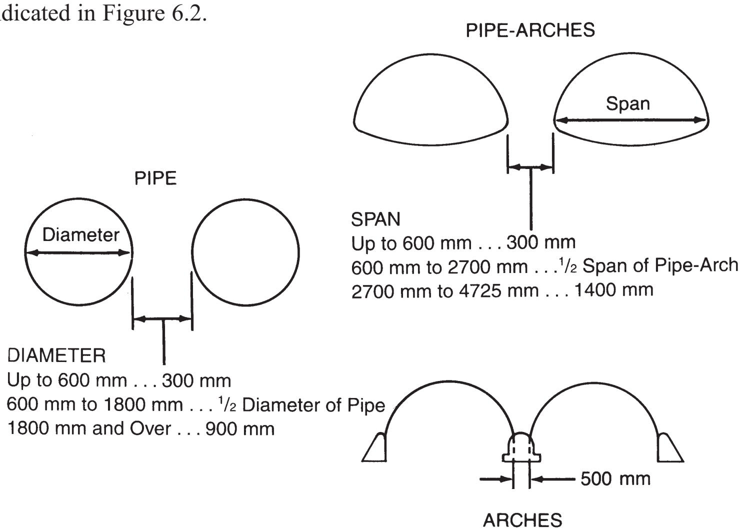


Figure 6.2 Minimum permissible spacings for multiple installations. Spacing can be decreased if CLSM is used as backfill.

Loads

Underground conduits are subject to two principal kinds of loads:

1. Dead loads developed by the trench backfill above, and stationary superimposed uniform or concentrated surface loads; and
2. Live loads caused by temporary moving loads, including impact.

Dead Loads

The dead load is considered to be the soil prism over the pipe:

$$DL = \gamma H$$

where: DL = unit pressure of a soil prism acting on the horizontal plane at the top of the pipe, kPa

γ = unit weight of the soil, kN/m^3

H = height of cover over the pipe, m

Live Loads

The live load (LL) is that portion of the weight of vehicles, trains, or aircraft moving over the pipe, that is distributed through the soil to the pipe. Live loads are greatest when the height of cover over the top of the pipe is small and decrease as the fill height increases.

Quarry underpass under high cover.

Live Loads Under Highways

Live load pressures for AASHTO H-20 and AASHTO H-25 highway loadings, including impact effects, are listed in Table 6.8. Note that these live loads are obtained by positioning the heaviest axle of the design truck centrally above the crown of the pipe at finished grade level. The axle loads are 142.3 kN and 177.9 kN respectively.

Live load pressures for the CAN/CSA-S6-06 design truck, CL-625, are also listed Table 6.8. These live loads are obtained by centering the dual axle of the design truck centrally above the crown of the pipe at finished grade level. Each axle weighs 125 kN. Both single truck and two truck load cases must be considered, and are reflected in the numbers shown.

Table 6.8 Highway and railway live loads (LL)[1]

Highway Loading				Railway Loading	
Depth of Cover,	LL Pressure, KPa			Depth of Cover,	LLPressure, KPa
m	CL-625[2]	H-20[3]	H-25[3]	m	E•80
0.30	61	86	109	0.6	182
0.50	46	56	75	1.0	147
0.75	34	34	46	1.2	133
1.00	26	25	31	1.5	115
1.25	20	17	22	2.0	91
1.50	16	13	16	3.0	53
1.75	14	10	12	4.0	34
2.00	12	8	10	6.0	15
2.25	11	6	8	8.0	7
2.50	10	5	6	9.0	5
2.75	9	-	5		
3.00	8	-	-		
3.50	7	-	-		
4.00	6	-	-		
4.50	5	-	-		

Notes: 1. Neglect live load when less than 5 kPa; use dead load only.
2. Load distribution through soil according to CAN/CSA-S6-06 (unfactored $\sigma_L m_f$, including dynamic load allowance). Note that there is a separate vehicle for Ontario, in which the axles are heavier.
3. Load distribution through soil according to ASTM traditional method (including impact).

Live load pressures are greatest at smaller heights of cover and decrease with increasing cover. Dead load pressures increase with increasing cover. As shown in Figure 6.3, the combined H-20 live load and dead load is lowest at a cover of about 1.5 m.

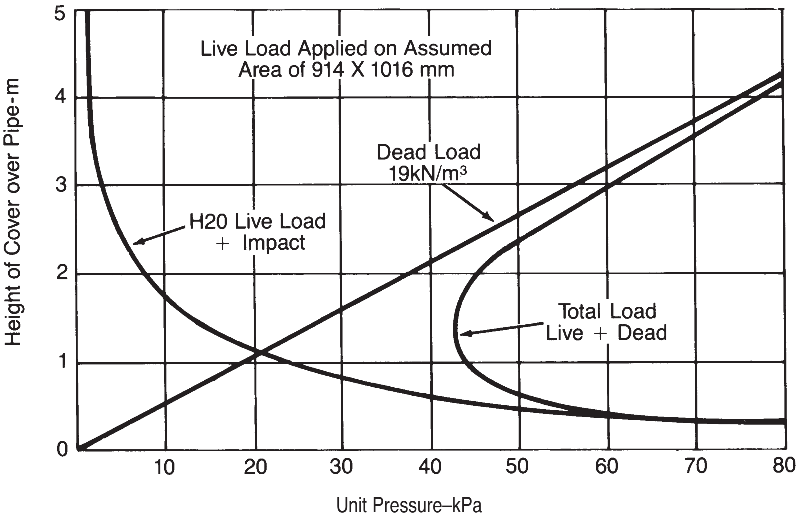


Figure 6.3 Combined H20 highway live load and dead load.

Live Loads Under Railways

Live load pressures for E80 railway loadings, including impact, are also listed in Table 6.8. As shown in Figure 6.4, the combined E 80 live load and dead load is a minimum at about 3.8 m of cover.

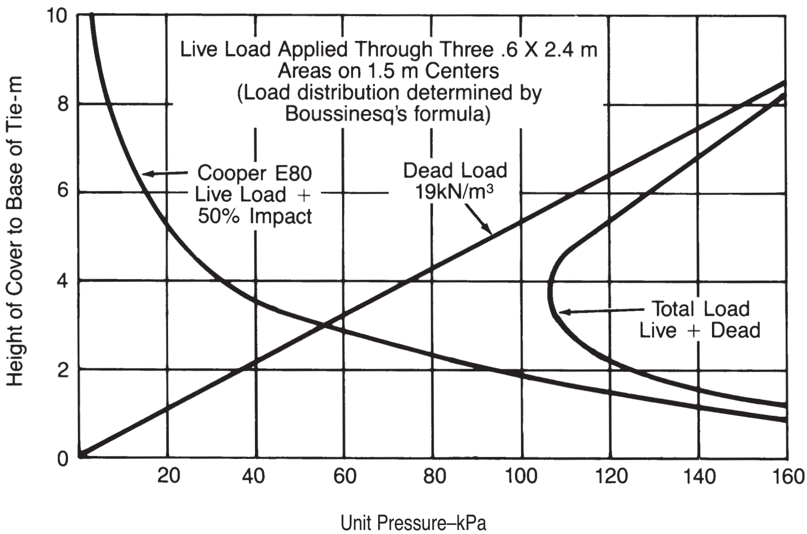


Figure 6.4 Combined E80 railway live load and dead load.

Live Loads Under Airport Runways
Live load pressures for aircraft vary because of the many different wheel configurations and weights. Such pressures must be determined for the specific aircraft for which the installation is designed. The Federal Aviation Administration's publication, "Airport Drainage", provides details.

Impact Loads
Loads caused by the impact or dynamic effects of moving traffic are important only at low heights of cover. Impact load allowances have been included in the live load pressures listed in Table 6.8.

Design Process

The structural design process consists of the following:

1. Check minimum allowable cover.
2. Select the degree of backfill compaction to be required.
3. Calculate the design pressure.
4. Compute the ring compression in the pipe wall.
5. Calculate the allowable compressive stress.
6. Determine the thickness required.
7. Check minimum handling stiffness.
8. Check seam strength requirements (when applicable).
9. Check special considerations for pipe-arches and arches.

1. Minimum Cover
Satisfactory minimum cover requirements have been formulated for corrugated steel pipe and pipe-arches with a diameter or span equal to or less than 3m, designed in accordance with the AISI method. These are based on long-time observations by the corrugated steel pipe industry and regulatory agencies, of structure performance under live loads. From these field observations, the minimum cover requirement was established as a function of shape, loading and corrugation size; values typically used are span divided by either 6 or 8 for highway applications, and span divided by 4 for railway applications. Structure specific minimum cover requirements are outlined in “Height of Cover Tables for Corrugated Steel Conduits”, later in this chapter.

Note that this minimum cover is not always adequate during construction. When construction equipment, frequently heavier than traffic loads for which the pipe has been designed, is to be driven over or close to the buried structure, it is the responsibility of the contractor to provide additional cover to avoid damage to the pipe. The minimum allowable cover for heavy construction loads can be based on structural design calculations or the guidelines presented in Table 6.9.

Table 6.9 General guidelines for minimum cover required for heavy off-road construction equipment

Pipe Span, mm	Minimum Cover (mm) for Indicted Axle Loads (tonnes)*			
	8 - 22	22 - 34	34 - 50	50 - 68
300 - 1050	600	760	900	900
1200 - 1830	900	900	1050	1200
1980 - 3050	900	1050	1200	1200
3200 - 3660	1050	1200	1370	1370

* Minimum cover may vary, depending on local conditions. The contractor must provide the additional cover required to avoid damage to the pipe. Minimum cover is measured from the top of the pipe to the top of the maintained construction roadway surface.

2. Backfill Compaction

Select a percent compaction of pipe backfill for design. The value chosen should reflect the importance and size of the structure, and the quality of backfill material and its installation that can reasonably be expected. The recommended value for routine use is 85% Standard Proctor Density. This assumed value is conservative for ordinary installations in which most specifications call for compaction to 90%. However, for more important structures and higher fill situations, select a higher quality backfill and higher compaction, and require the same in construction. This will increase the allowable fill height and may save on the pipe thickness.

3. Design Pressure

When the height of cover is equal to or greater than the span or diameter of the structure, the load factor chart, Figure 6.5, is used to determine the percentage of the total load acting on the pipe. For routine use, the 85% Standard Proctor Density soil value will result in a factor of 0.86. The load factor, K, is applied to the total load to obtain the design pressure, P_V, acting on the pipe. If the height of cover is less than one pipe diameter, the total load is assumed to act on the pipe (K = 1.0).

The load on the pipe becomes:

$$P_V = K\,(DL + LL), \text{ when } H \geq S$$
$$P_V = (DL + LL), \text{ when } H < S$$

where:
- P_V = design pressure, kPa
- K = load factor
- DL = dead load, kPa
- LL = live load, kPa
- H = height of cover, m
- S = span or diameter, m

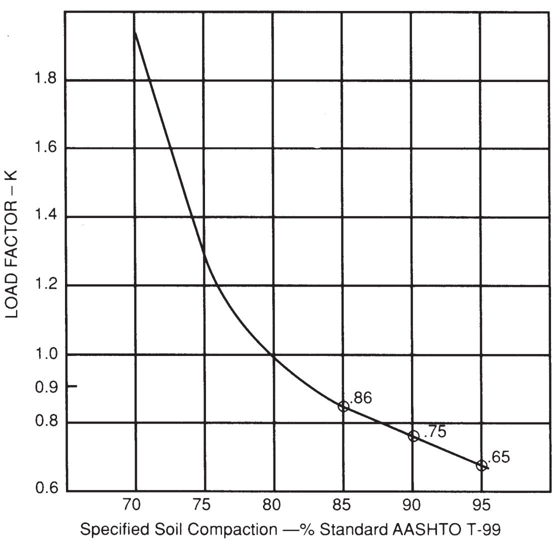


Figure 6.5 Load factors for CSP in backfill compacted to indicated density.

4. Ring Compression

The compressive thrust in the pipe wall is equal to the radial pressure acting on the wall multiplied by the wall radius, or:

$$C = P \bullet R$$

This thrust, called ring compression, is the force carried by the pipe wall. The ring compression force acts tangentially to the pipe wall. For conventional structures in which the top arc approaches a semicircle, it is convenient to substitute half the span for the wall radius.

Then: $$C = P_v \bullet \frac{S}{2}$$

where:
- C = ring compression, kN/m
- P_v = design pressure, kPa
- S = span or diameter, m

5. Allowable Wall Stress

The ultimate compressive stress, f_b, for corrugated steel structures with backfill compacted to 85% Standard Proctor Density and a yield strength of 230 MPa, are shown in Figure 6.6. The ultimate compression in the pipe wall is expressed by the following equations which represent the three behavioural zones that all pipes would be expected to be governed by. The first is the specified yield strength of the steel, which represents the *zone of wall crushing* or *yielding*. The second represents the *interaction zone of yielding and ring buckling*. The third represents the *ring buckling zone.*

$$f_b = f_y = 230, \text{ when } \frac{D}{r} < 294$$

$$f_b = 279.6 - (574.3 \times 10^{-6}) \left(\frac{D}{r}\right)^2, \text{ when } 294 \leq \frac{D}{r} \leq 500$$

$$f_b = \frac{(34 \times 10^6)}{\left(\frac{D}{r}\right)^2}, \text{ when } \frac{D}{r} > 500$$

where:
- f_b = ultimate compressive stress, MPa
- f_y = yield strength, MPa
- D = diameter or span, mm
- r = radius of gyration of the pipe wall (see Tables 6.2 or 6.3), mm

A factor of safety of 2 is applied to the ultimate wall stress to obtain the allowable stress, f_c:

$$f_c = \frac{f_b}{2}$$

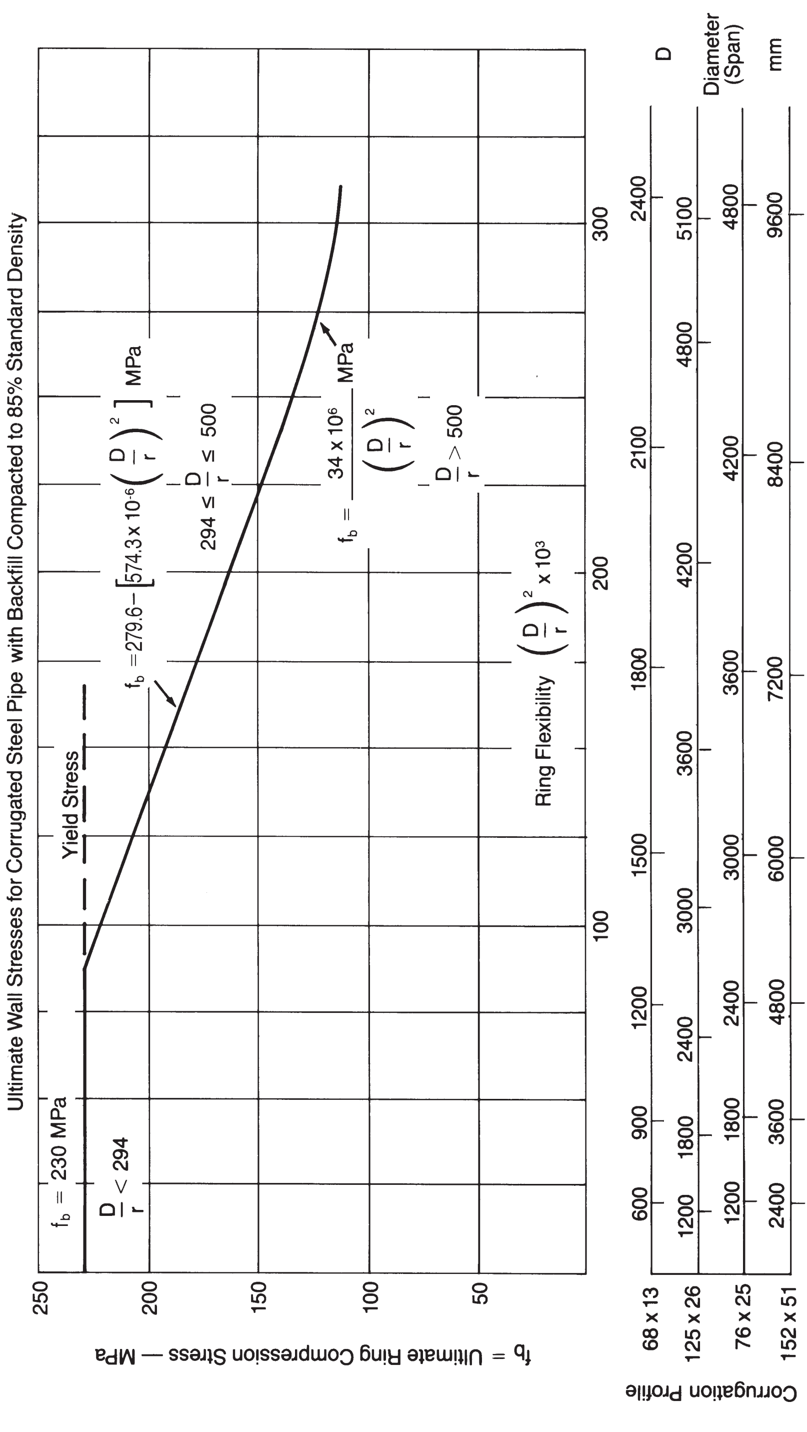


Figure 6.6 Ultimate compressive wall stress for CSP.

6. Wall Thickness

A required wall area, A, is computed using the calculated compression in the pipe wall, C, and the allowable stress, f_c.

$$A = \frac{C}{f_c}$$

where: A = required area in the pipe wall, mm^2/mm
C = ring compression, kN/m
f_c = allowable stress, MPa

From Table 6.2 or 6.3, select the wall thickness that provides the required area. The properties used in steps 5 and 6 (r, A) must be for the same corrugation.

7. Handling Stiffness

Minimum pipe stiffness requirements, for practical handling and installation without the need for special shape control measures, have been established through experience and have been formulated. The resultant flexibility factor, FF, limits the size of pipe for each combination of corrugation and metal thickness.

$$FF = \frac{D^2}{EI}$$

where: E = modulus of elasticity = 200×10^3 MPa
D = diameter or span, mm
I = moment of inertia of the pipe wall (see Tables 6.2 or 6.3), mm^4/mm

Recommended maximum allowable values of FF for ordinary round and underpass pipe installations are as follows:

68 x 13 mm corrugation,	$FF \leq 0.245$ mm/N
125 x 25 mm corrugation,	$FF \leq 0.188$ mm/N
76 x 25 mm corrugation,	$FF \leq 0.188$ mm/N
152 x 51 mm corrugation,	$FF \leq 0.114$ mm/N

The maximum allowable values of FF for pipe-arch and arch shapes are increased as follows:

Pipe-Arch	$FF \leq 1.5 \times$ FF shown for round pipe
Arch	$FF \leq 1.5 \times$ FF shown for round pipe

Higher values can be used with special care or where experience suggests a higher value is appropriate. Trench conditions, as in the case of storm sewer design, is one example where higher allowable values are appropriate. Aluminum pipe experiences are another. For example, the flexibility factor permitted for aluminum pipe in some specifications is more than twice that recommended above for steel. This has come about because aluminum has only one-third the stiffness of steel, the modulus of elasticity for aluminum being approximately 67×10^3 MPa compared to 200×10^3 MPa for steel. Where this degree of flexibility is acceptable in aluminum, it will be equally acceptable in steel.

For spiral rib pipe, a somewhat different approach is used. To obtain better control, the flexibility factors are varied with corrugation profile, sheet thickness and type of installation, as shown in Table 6.10. The height of cover tables included in this handbook (Table HC-11 and HC-12) are based on a trench-like installation. Since spiral rib pipe is mostly used for storm sewers, in which a trench-like installation is the normal installation method, this covers most applications. A note accompanying the table directs the user to this part of the handbook (Table 6.10) or to the ASTM specification for guidance on embankment-type (Type I) installations. The height of cover table also includes an indication of those larger diameter pipes which require the use of specific backfill materials and special attention to shape control during installation.

Table 6.10

Allowable flexibility factors for spiral rib pipe, **19 x 19 x 190** rib profile

Installation Type	Flexibility Factor, mm/N		
	Thickness, mm		
	1.6	2.0	2.8
I	0.175	0.192	0.219
II	0.212	0.232	0.266
III	0.296	0.324	0.371

Installation types, as shown in Table 6.10, are:

Type I Installations can be an embankment or fill condition. Installations shall meet ASTM A798 requirements. ML and CL materials are typically not recommended. Compaction equipment or methods that cause excessive deflection, distortion, or damage shall not be used.

Type II Installations require trench-like conditions where compaction is obtained by hand, or walk behind equipment, or by saturation and vibration. Backfill materials are the same as for TYPE I installations. Special attention should be paid to proper lift thicknesses. Controlled moisture content and uniform gradation of the backfill may be required to limit the compaction effort while maintaining pipe shape.

Type III Installations have he same requirements as TYPE II installations except that the backfill materials are limited to clean, non-plastic materials that require little or no compaction effort (GP, SP), or to well graded granular materials classified as GW, SW, GM, SM, GC, or SC with a maximum plasticity index (PI) of 10. Maximum loose lift thickness shall be 200 mm. Special attention to moisture content to limit compaction effort may be required. Soil cement or cement slurries may be used in lieu of the selected granular materials.

8. Seam Strength

Most pipe seams develop the full yield strength of the pipe wall. However, there are exceptions. Tables 6.4a and b show those standard riveted and bolted seams (underlined) which do not develop a yield strength equivalent to f_y = 230 MPa. The allowable ring compression accounting for the seam strength considerations, is the ultimate seam strength, shown in Tables 6.4a and 6.4b, divided by the factor of safety of 2.0. Since helical lockseam and continuously-welded-seam pipe have no longitudinal seams, there is no seam strength check for these types of pipe.

9. Special Considerations for Pipe-Arches and Arches

(a) Pipe-Arches

Pipe-arches generate radial corner pressures as illustrated in Figure 6.7. These pressures, which are greater than the applied pressure at the top of the structure, must be limited to the allowable bearing pressure of the soil. This often becomes the limiting design factor, rather than structural strength. Special backfill at the corners, such as crushed stone or controlled low strength material, can extend these limitations. A maximum corner pressure of 300 kPa is suggested for routine use, although the adequacy of the foundation should be confirmed.

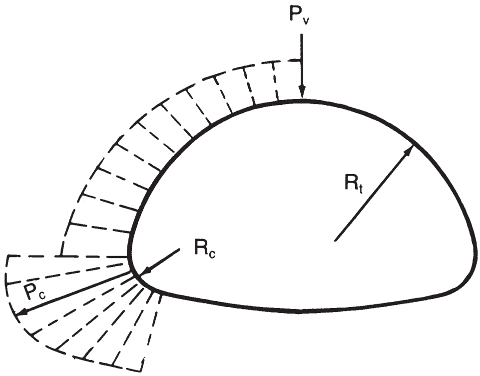


Figure 6.7 Pressure on the pipe-arch varies with radius and is greatest at the corners.

The corner pressure can be calculated as follows. The ring compression force, C, is the same at any point around the structure ignoring the bending strength of the pipe wall and the longitudinal distribution of pressure. From the familiar relationship $C = P_v \times R$, the pressure normal to the wall is inversely proportional to the radius ($P \propto 1/R$). Based on this relationship, the corner pressure, P_c, would be:

$$P_c = \frac{R_t}{R_c} P_v = \frac{R_t}{R_c} (LL + DL)$$

where: P_c = pressure acting on soil at the corners, kPa
R_t = radius of the crown, mm
R_c = radius of the corner, mm
LL = live load pressure, kPa
DL = dead load pressure, kPa

However, this relationship is overly conservative for live loads, such as wheel loads, that are not uniformly distributed over the full pipe length. As the ring compression force generated at the top of the pipe-arch by live loads is transmitted circumferentially down toward the corner region, it is also distributed along the length of the pipe. Thus, the length of the corner region that transmits the live load pressures into the soil is much greater than the length of pipe over which they were initially applied. The corner pressure is therefore calculated as:

$$P_c = \frac{R_t}{R_c} (C_l \bullet LL + DL)$$

where: C_l = longitudinal live load distribution factor

This is the procedure that was used to calculate the height-of-cover limits for pipe-arches in this Handbook. Furthermore, the live load was used without impact because (1) impact loads dissipate between the point on the structure where the load is applied and the corner region, and (2) bearing failures are progressive failures over a significant time period as opposed to the brief time of an impact loading. However, the full live load pressure (including impact and not modified by the C_l factor) should continue to be used to design the pipe wall.

Equations for C_l have been derived for standard highway and railway loadings.

H-20 and H-25 Live Loads

The live load pressures for the H-20 and H-25 live loads have traditionally been based on load application through an assumed 300 mm thick pavement area measuring 914 mm by 1016 mm. The load is distributed at 0.875 to 1 (horizontal to vertical) through the earth fill. In other words, the pressure area at a particular depth has an additional length or width, in addition to the length or width of the loaded area at the surface, of 1.75 times the height of cover.

Figure 6.8 shows how the load is distributed from the wheel loads over a pipe-arch. The pressure, at any height-of-cover, h, below the 1016 mm wide area, is spread over a distribution length L_1 a the top of the structure. The stress in the pipe wall, from this pressure, also spreads longitudinally. the length of the corner which transmits the wheel load is L_2. The reaction length includes an increase of 1.75 times the arc length from the top of the structure to the corner. This arc length can be approximated as one quarter of the circumference of a round pipe having a diameter which is the same as the span of the pipe-arch (add 1.75 x π x span / 4 = 1.37 x span). No overlap of the reaction lengths from the individual wheel loads occurs until L_2 exceeds 1829 mm. When the height of cover exceeds 765 mm, the pressure sones at the top is then L_1 + 1829 and the reaction length is L_2.

The value of C_l is :

$C_l = L_1/L_2$ when $L_2 \leq 1829$ mm

$C_l = 2L_1/L_3$ when $L_2 > 1829$ mm and $h \leq 765$ mm

$C_l = (L_1 + 1829)/L_3$ when $L_2 > 1829$ mm and $h > 765$ mm

where: $L_1 = 1016 + 1.75\ (h - 300)$
$L_2 = L_1 + 1.37s$
$L_3 = L_2 + 1829$
h = height of cover,, mm
s = span, mm

The live load pressures for H-20 and H-25 highway loads, including impact, are as given in Table 6.8. The live load pressures, neglecting impact, are as shown in Table 6.11.

Table 6.11 Highway live loads, neglecting impact[1]

Depth of Cover,	Load, KPa		
m	CL-625[2]	H-20[3]	H-25[3]
0.30	45	77	96
0.50	35	56	75
0.75	27	34	46
1.00	21	25	31
1.25	17	17	22
1.50	15	13	16
1.75	13	10	12
2.00	11	8	10
2.25	10	6	8
2.50	9	5	6
2.75	8	-	5
3.00	7	-	-
3.50	6	-	-
4.00	5	-	-

Notes: 1. Neglect live load when less than 5 kPa; use dead load only.
2. Load distribution through soil according to CAN/CSA-S6-06 (unfactored $\sigma_L m_f$, excluding dynamic load allowance).
3. Load distribution through soil according to ASTM traditional method (excluding impact).

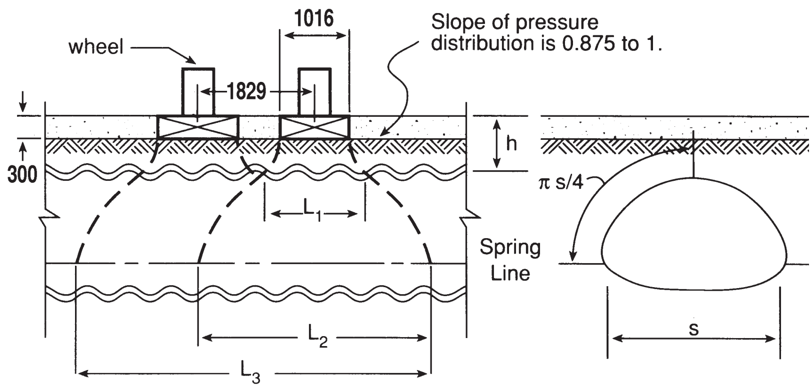


Figure 6.8 Longitudinal distribution of H-20 and H-25 live load corner bearing pressure in pipe-arches.

CL-625 Live Load

The live load pressure for the CL-625 live load are based on a different load distribution model than that used for the H-20 and H-25 live loads. The wheel loads are distributed at 0.5 to 1 (horizontal to vertical) in a longitudinal direction (along the length of the structure) and at a 1 to 1 in a transverse direction (in the direction of the span of the structure). The total load is then determined by distributing the wheel loads over the rectangular area which encloses the individual rectangular areas of the distributed wheel loads.

Figure 6.9 shows how the load is distributed from wheel loads over a pipe-arch. the pressure, at any height-of-cover, h, is spread over a distribution length L_1, at the top of structure. The stress in the pipe wall, from this pressure, also spreads longitudinally. The length of the corner which transmits the wheel load is L_2. The reaction length includes an increase of 1.0 times the arc length from the top of the structure to the corner. The actual distribution of the load through the arc length would be a combination of the longitudinal and transverse distribution slopes, but using a distribution slope of 0.5 to 1 for this calculation provides a conservative result. this arc length can be approximated as one quarter of the circumference of a round pipe having a diameter which is the same as the span of the pipe-arch (add 1.0 x π x span / 4 = 0.785 x span).

The value of C_l is :

$C_l = L_1/L_2$

where: $L_1 = 2400 + h$
$L_2 = L_1 + 0.785s$
h = height of cover,, mm
s = span, mm

The above discussion and Figure 6.8 are based on a single vehicle load. for covers larger than 1350 mm, a two vehicle loading condition governs and the formula for L_1 should be changed to:

$L_1 = 5400 + h$

The live load pressures for the CL-625 highway load including dynamic load allowance, are as given in table 6.8. The live load pressures, neglecting dynamic load allowance are as shown in Table 6.11.

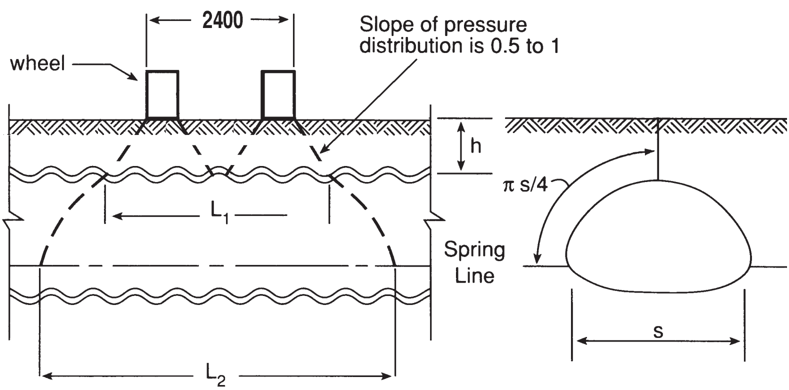


Figure 6.9 Longitudinal distribution of CL-625 live load corner bearing pressure in pipe-arches.

E-80 Railway Live Load
The live load pressures for railway live loads have traditionally been based on load application through a 610 by 2438 mm bearing area. The load is distributed at 0.875

to 1 (horizontal to vertical) through the earth fill. In other words, the pressure area at a particular depth has an additional length or width, in addition to the length or width of the loaded area at the surface, of 1.75 times the height of cover.

Figure 6.10 shows how the load is distributed from tie loads over a pipe-arch. The pressure, at any height-of-cover, h, below the 2438 mm wide tie, is spread over a distribution length L_1 at the top of the structure. The stress in the pipe wall, from this pressure, also spreads longitudinally. The length of the corner which transmits the live load is L_2. This reaction length includes an increase of 1.75 times the arc length from the top of the structure to the corner, using the same approximation for the arc length as described above for the H-20 and H-25 live loads.

The value of C_l is :

$$C_l = L_1/L_2$$

where: $L_1 = 2438 + 1.75h$
$L_2 = L_1 + 1.37s$
h = height of cover,, mm
s = span, mm

The above discussion is based on a single track arrangement, and it may be appropriate to consider overlap of pressure areas for some multiple track arrangements.

The live load pressures for railway live loads, including impact, are as given in Table 6.8. Those live load pressures should be divided by 1.5 to remove the allowance for impact.

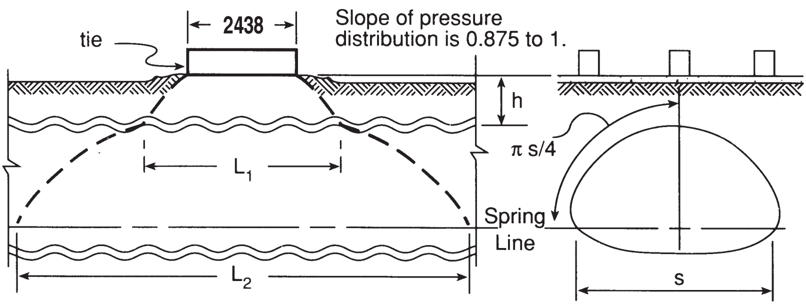


Figure 6.10 Longitudinal distribution of live load corner bearing pressure in pipe-arches under railway loading.

(b) Arches

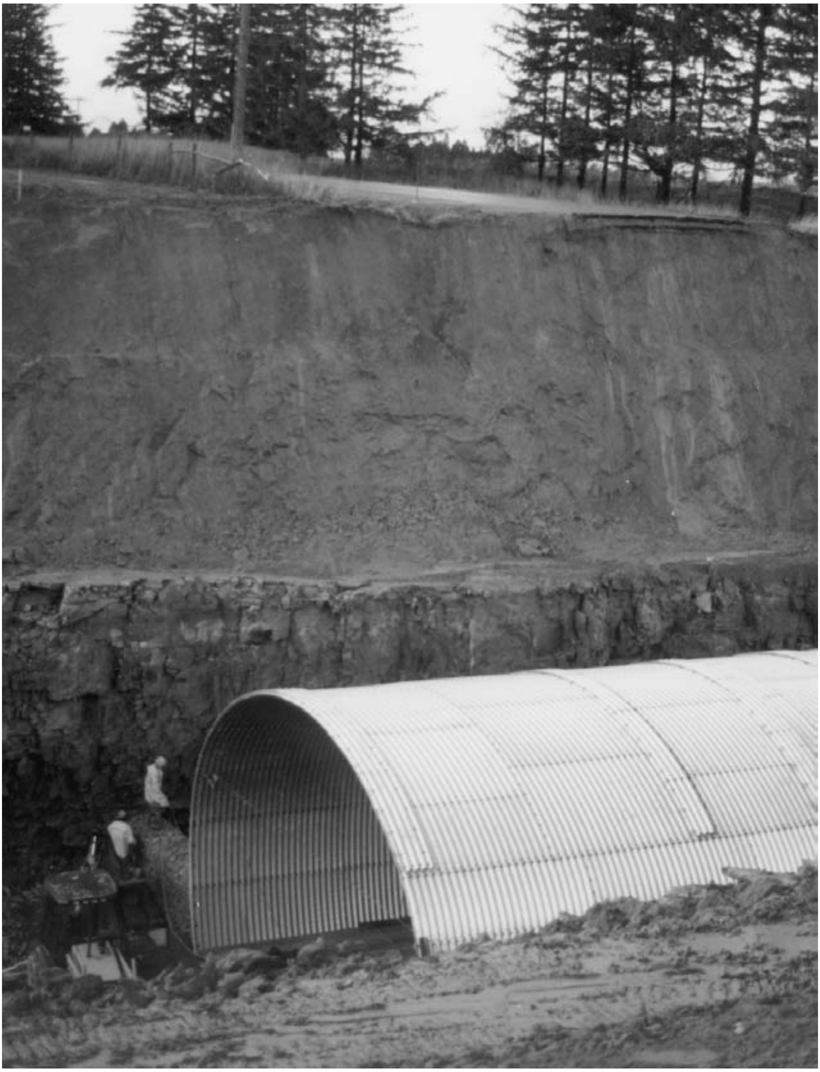
The design of structural plate arches is based on a minimum allowable ratio of rise to span of 0.3 (equivalent to an arch through 124 degrees). The structural design method is the same as for round structural plate pipe.

The design of arches involves two additional important considerations.

The first consideration is the foundation rigidity. It is undesirable to make the steel arch relatively unyielding or fixed compared with the adjacent side fill. The use of massive footings or piles to prevent any settlement of the arch is generally not recommended. When the structure is restrained at the base of the arch or the footings are founded on an unyielding foundation, the influence of column-type buckling must be considered.

The ultimate compressive strength of the arches that are less than semicircular (the rise to span ratio is less than 0.5) has been shown to be less than that of equivalent full round pipe. The standard practice is to use an allowable stress of $0.375f_b$ rather than $0.5f_b$.

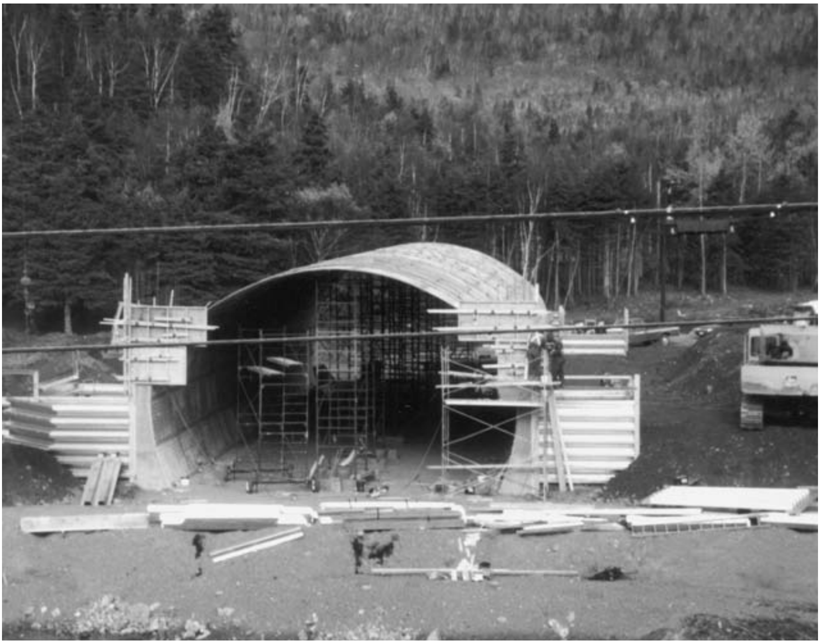
Where poor materials are encountered, consideration must be given to removing some or all of this poor material and replacing it with acceptable material. The footing should be designed to provide uniform longitudinal settlement of acceptable magnitude. Allowing the arch to settle will protect it from potential drag-down



High-profile arch will be under high cover.

forces caused by the consolidation of the adjacent sidefill. An opportunity exists on all arch designs to permit the footing to settle and relieve the load on the arch. Positive soil arching can be assured by such practice, and lower safety factors can be used as a result.

The second consideration is the bearing capacity of soils under footings, the bottom of footing elevation (amount of bury), and the direction of the footing reaction. The value of the reaction is the thrust in the arch. Footing reactions for the arch are considered to act tangential to the plate at its point of connection to the footing. Footings should be set at a depth below maximum predicted scour lines. Alternatively, invert slabs or other appropriate measures can be provided to prevent scour.



Forming for concrete collar and assembly of bin-type retaining wall

DESIGN OF BURIED STRUCTURES WITH SPANS GREATER THAN 3 m

In this publication, the AISI, AASHTO and CHBDC methods are all recognized for the design of soil-metal structures with a diameter or span greater than 3 m. The AISI and AASHTO methods have been used for many years and can continue to be used unless the CHBDC is specifically required.

AISI Method

The AISI design method, described above for structures with spans up to 3 m, is applicable to structures having spans larger than 3 m as well. The standard method described above has been used for structures up to 7.7 m in diameter. Structures with spans exceeding 3 m can be designed using the AISI method as long as the maximum allowable flexibility factor is not exceeded.

AASHTO Method

The structural design of "long span" structures follows the traditional ring compression methods with the exception that the buckling and flexibility factor requirements do not apply. Long span structures are structural plate pipes, pipe-arches and arches that can not be designed by the same method as shorter span structures. They also include all special shapes of any size that involve a relatively large radius of curvature in the crown or side plates, such as; vertical ellipses, horizontal ellipses, underpasses, low profile arches, high profile arches and inverted pears. These structures include special features and must meet a table of minimum requirements (Table 6.12). For ring compression calculations, the span in the formula for thrust is replaced by twice the top arc radius.

Table 6.12 AASHTO minimum requirements for long-span structures with acceptable special features

I. TOP ARC MINIMUM THICKNESS, mm

	Top Radius, m				
	≤ 4.57	4.57-5.18	5.18-6.10	6.10-7.01	7.01-7.62
152 x 51 mm Corrugated Steel Plates	2.77 mm	3.51 mm	4.27 mm	5.54 mm	6.32 mm

II. MINIMUM COVER, mm

	Top Radius, m				
Steel Thickness[a] in mm	≤ 4.57	4.57-5.18	5.18-6.10	6.10-7.01	7.01-7.62
2.77	760				
3.51	760	915			
4.27	760	915	915		
4.78	760	915	915		
5.54	610	760	760	915	
6.32	610	610	760	915	1220
7.11	610	610	760	915	1220

III. GEOMETRIC LIMITS

A. Maximum Plate Radius – 7.62 m
B. Maximum Central Angle of Top Arc = 80°
C. Minimum Ratio, Top Arc Radius to Side Arc Radius = 2
D. Maximum Ratio, Top Arc Radius to Side Arc Radius = 5*

*Note: Sharp radii generate high soil bearing pressures. Avoid high ratios when significant heights of fill are involved.

IV. SPECIAL DESIGNS

Structures not described herein shall be regarded as special designs.

[a]When reinforcing ribs are used the moment of inertia of the composite section shall be equal to or greater than the moment of inertia of the minimum plate thickness shown.

CHBDC Method

The Canadian Highway Bridge Design Code (CHBDC) first introduced in 2001 contains a separate section for the design of buried structures, which includes soil-metal structures and metal box structures. The CHBDC method is based upon the limit states design philosophy and supersedes the Ontario Highway Bridge Design Code (OHBDC) and the CAN/S6-88 Design of Highway Bridges Standard. The CHBDC is available from CSA International as "**CAN/CSA-S6-06 Canadian Highway Bridge Design Code**".

The CHBDC method is similar to the AISI method with the following differences. Limit states design, as used in the CHBDC, is based on ultimate strength principles rather than the traditional working stress or service load design method.

The CHBDC distribution of live load through the fill is accomplished by the use of a thrust calculation, which is a function of the relative axial and flexural rigidity of the structure wall with respect to soil stiffness. The CHBDC uses a strength calculation for combined bending and axial load during construction, based on work by Duncan and Byrne, instead of the AISI flexibility factor check. Additional procedures have been developed for seismic design and fatigue resistance. The CHBDC also includes a revised series of clauses covering the design of metal box structures based on the AASHTO Standard Specifications for Highway Bridges.

Limit States Design

For Limit States Design of soil-metal and metal box structures, the specific limit states that must be investigated under the general categories of Ultimate Limit State (ULS), Serviceability Limit State (SLS), and Fatigue Limit State (FLS) are outlined in Table 6.13

Table 6.13 Specific limit states

Limit State Categories	Structure	Specific Limit State
ULS	Soil-Metal	Compression Failure
		Plastic Hinge During Construction
		Connection Failure
	Metal Box	Plastic Hinge in Top Arc
		Connection Failure
SLS	Soil-Metal	Deformation During Construction
	Metal-Box	Deformation During Construction
FLS	Soil-Metal	Not Applicable
	Metal Box	Stress Range in Conduit Wall

Load Factors

The load factors used to compute factored loads are:

$\alpha_{DL} = 1.25$ Dead Load

$\alpha_{LL} = 1.70$ Live Load

The CHBDC live load factor of 1.70 replaces the value of 1.40 used in the 1991 Edition of the Ontario Highway Bridge Design Code. This increase was done in combination with a decrease in axle load for the design vehicle (CL-W).

The dynamic load allowance (DLA) is dependent on the depth of cover, H. For soil metal structures, it is specified to be 0.4 for zero depth of cover decreasing linearly to 0.1 for a depth of cover of 1.5 m. For depths of cover larger than 1.5 m, the DLA is specified to be 0.1. As a formula, this is expressed as:

$$DLA = 0.4 - 0.2H \geq 0.1$$

For metal box structures, the above formula applies for spans less than 3.6 m. For spans larger than 3.6 m, the value of 0.4 is replaced by 0.3 and the formula becomes:

$$DLA = 0.3 - 0.15H \geq 0.1$$

Material Resistance Factors

The material resistance factors, ϕ, provided in Table 6.14, are used to compute a factored resistance for the walls of soil-metal and metal-box structures.

Minimum Clear Spacing Between Conduits

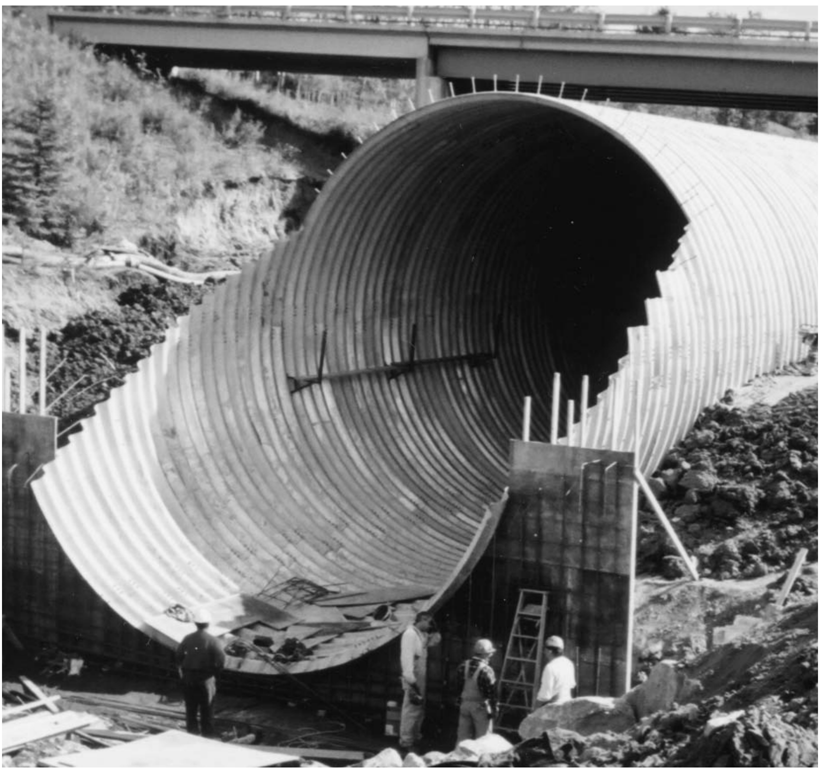
For multiple structure installations with shallow corrugations, the smallest clear spacing between adjacent structures should be not less than 1000 mm, nor less than

Table 6.14 Material resistance factors

Type of Structure	Component of Resistance	Material Resistance Factor
Soil-metal with shallow corrugations	Compressive strength	ϕ_t = 0.80
	Plastic hinge during construction	ϕ_{hc} = 0.90
	Connections	ϕ_j = 0.70
Soil-metal with deep corrugations	Compressive strength	ϕ_t = 0.80
	Plastic hinge	ϕ_h = 0.85
	Plastic hinge during construction	ϕ_{hc} = 0.90
	Connections	ϕ_j = 0.70
Metal box	Compressive strength	ϕ_t = 0.90
	Plastic hinge	ϕ_h = 0.90
	Connections	ϕ_j = 0.70

one tenth of the largest span. The minimum clear spacing between two or more structures should also be sufficient for practicality of construction, especially for the placement and compaction of soil. Where space is restricted, a controlled low strength material (CLSM), with a minimum 28 day compressive strength of 7-10 MPa, may be used in lieu of compacted soil. Cast-in-place concrete or grout may also be used. If CLSM or other cementitious material is used, the design must consider uplift of the structure while the material is still wet.

For soil-metal structures with deep corrugations, the minimum clear span spacing between adjacent conduits shall be 1000 mm.



Installation of large diameter pipe with step-beveled end.

Spiral rib pipe inlet into large cast-in-place box.

Design Process: Soil-Metal Structures

The structural design process consists of the following:

1. Check minimum allowable cover.
2. Calculate dead load thrust.
3. Calculate live load thrust.
4. Calculate earthquake thrust (if required).
5. Calculate total factored thrust.
6. Calculate the compressive stress.
7. Calculate the wall strength in compression.
8. Check wall strength requirements during construction.
9. Check wall strength of completed structures with deep corrugations.
10. Check seam strength.
11. Check difference in plate thicknesses of adjacent plates.
12. Check plate radius of curvature.

1. **Minimum Cover**

For soil-metal structures designed in accordance with the CHBDC method, the minimum allowable depth of cover is the largest of

a) 0.6 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{1/2}$ m

and

c) $0.4\left(\frac{D_h}{D_v}\right)^2$ m

where: D_h = horizontal dimension (effective span) of the structure as defined in Figure 6.11, m

D_v = vertical dimension (effective rise) of the structure as defined in Figure 6.11, m

For soil-metal structures with deep corrugations, the minimum depth of cover shall be the smaller of 1.5 m and the minimum depth of cover for structures with shallow corrugations but the same conduit size.

The minimum depth of cover requirement as illustrated in Figure 6.12, is meant to ensure that bending moments in the wall due to live loads are limited to a level which can be safely neglected in the design. It is also intended to prevent upheaval of a soil wedge above and to one side of a soil-metal structure due to the application of a large surface load. Shallower depths of cover may be used with the use of special features such as ribs on the structure crown, relieving slabs, earth reinforcing, or with the use of deep corrugated structural plate.

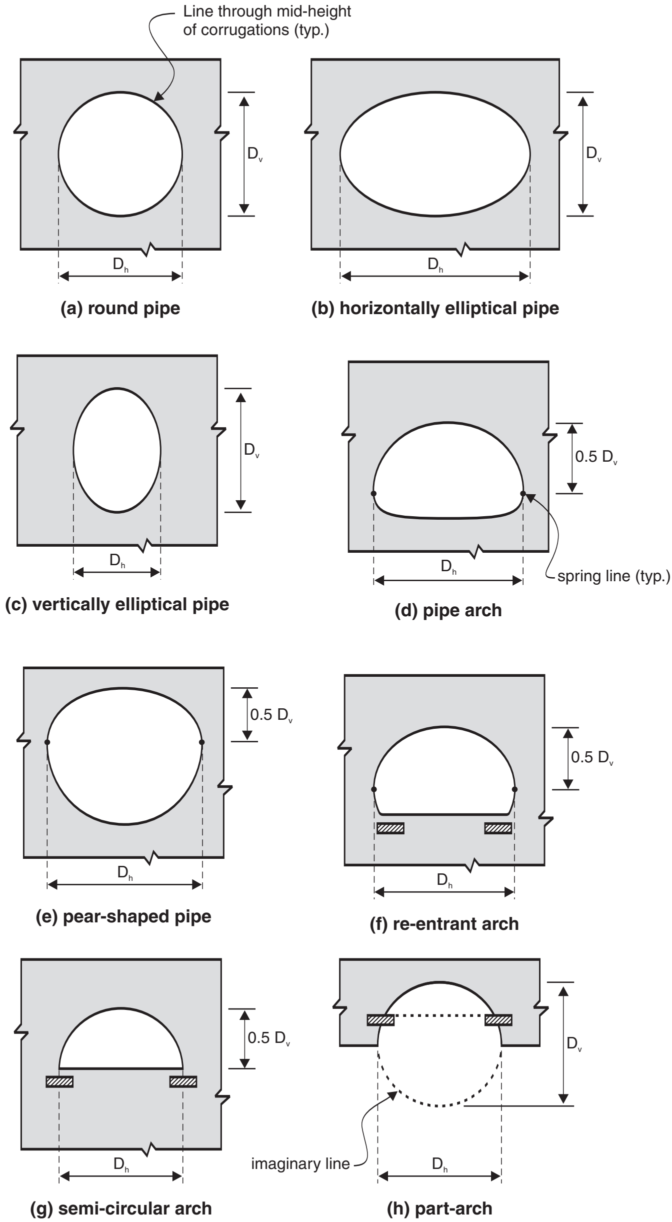


Figure 6.11 Definitions for D_h and D_v for various shapes.

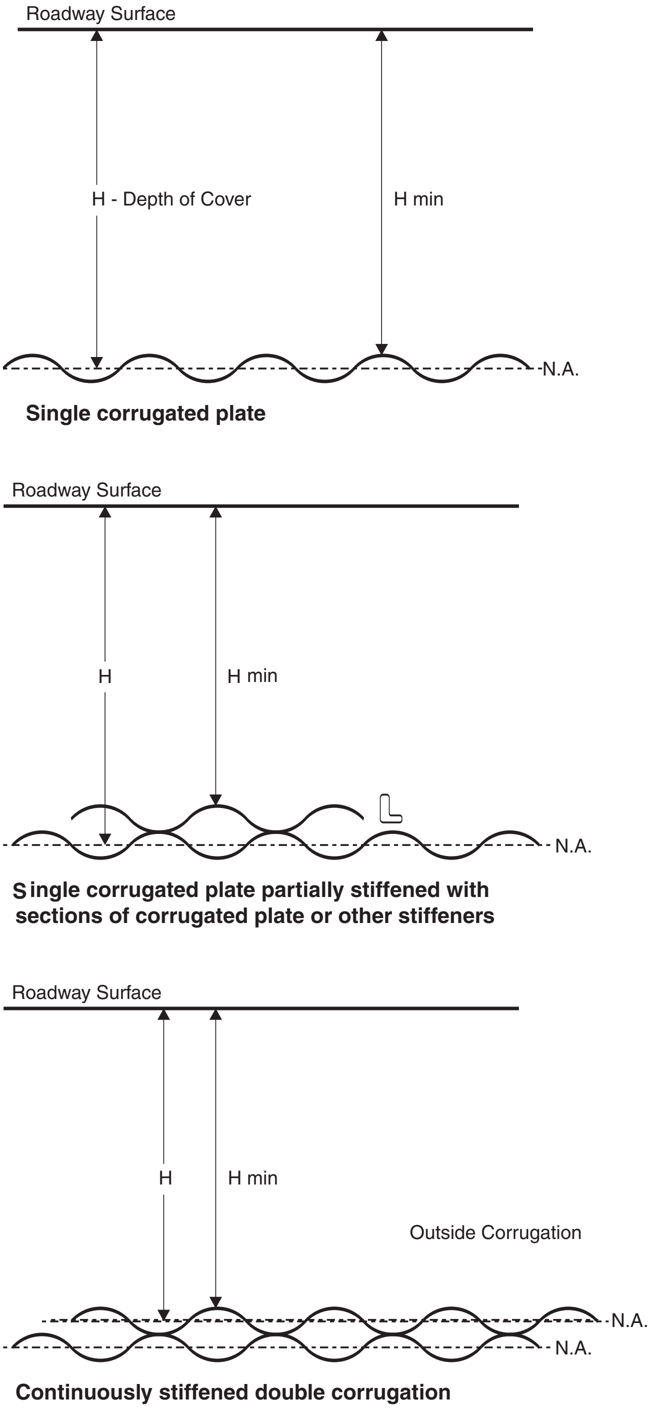


Figure 6.12 Depth of cover to soil-metal structures and metal-box structures.

2. **Dead Load Thrust**

The dead load thrust in the walls due to the overburden (dead) loads shall be calculated from:

$$T_D = 0.5\,(1.0 - 0.1\,C_S)\,A_f\,W$$

where:
- T_D = dead load thrust, kN/m
- A_f = arching factor used to calculate the thrust due to dead load in the wall, as defined in Figure 6.13
- C_S = axial stiffness parameter
- W = dead weight of the column of material above the structure, as defined in Figure 6.14, kN/m

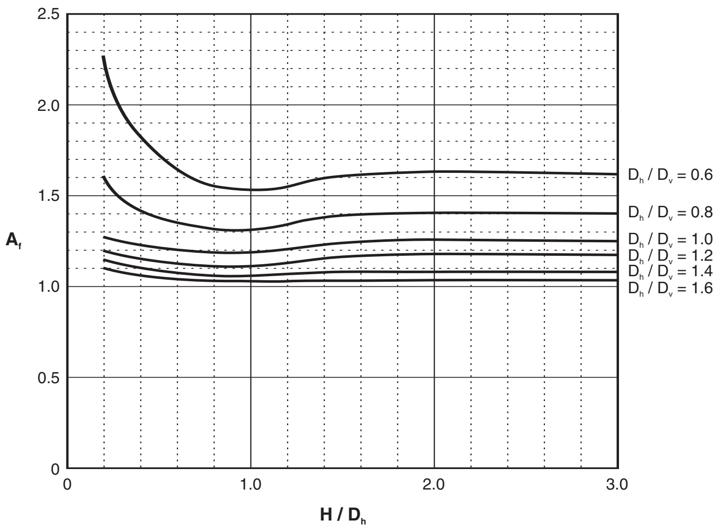


Figure 6.13 Arching factor, A_f.

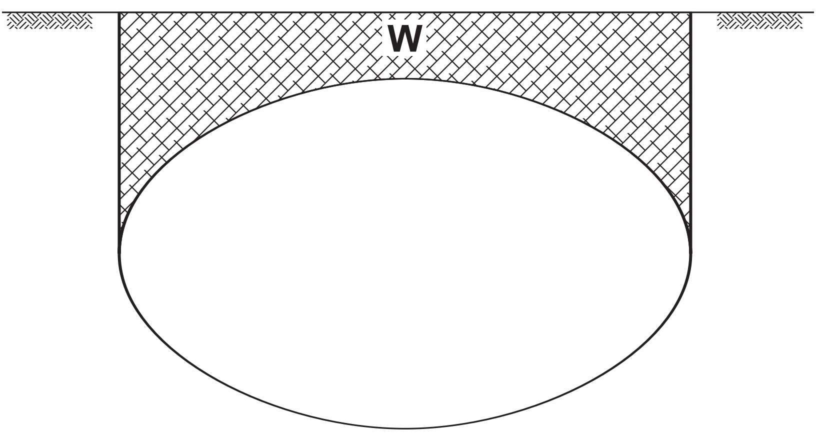


Figure 6.14 Area used in the calculation of W.

The axial stiffness parameter, C_S, is calculated from:

$$C_S = 1000\, E_S\, D_v / EA$$

where:
E_S = secant modulus of soil stiffness, as defined in Table 6.6, MPa
D_v = vertical dimension (effective rise) of the structure, as defined in Figure 6.11, m
E = modulus of elasticity of the structure metal, MPa
A = cross-sectional area of the corrugation profile, mm^2/mm

3. **Live Load Thrust**
The live load thrust is assumed to be constant around the structure, and is given by the lesser of:

$$T_L = 0.5\, D_h\, \sigma_L\, m_f$$

or

$$T_L = 0.5\, l_t\, \sigma_L\, m_f$$

where:
T_L = live load thrust due to unfactored live load, kN/m

D_h = horizontal dimension (effective span) of the structure, as defined in Figure 6.11, m

l_t = distance between the outermost axles including the tire footprints, placed in accordance with item (iii) plus 2H

H = height of cover, m

σ_L = uniformly-distributed pressure at the crown (top) of the structure resulting from the load distribution of the unfactored live load through the fill, kPa

m_f = modification factor for multi-lane loading; its value is specified to be 1.0 or 0.9 for one or two loaded lanes respectively (loading of more lanes does not govern)

The design live load vehicle is as shown in Figure 6.15. Note that there is a separate vehicle for Ontario (CL-625-ONT). The axles of interest (the second and third axles) are 140 kN each rather than 125 kN. The positioning of adjacent design vehicles, in order to obtain the maximum effect, is as shown in Figure 6.16.

The tire footprint for the design vehicle wheel loads measure 250 mm long by 600 mm wide.

The load case yielding the maximum value of $\sigma_L \bullet m_f$ is obtained as follows:

i) within the span length, position as many axles of the CL-W Truck or Trucks at the road surface above the conduit as would give the maximum total load;

ii) distribute the rectangular wheel loads through the fill down to the crown level at a slope of one vertically to one horizontally in the transverse

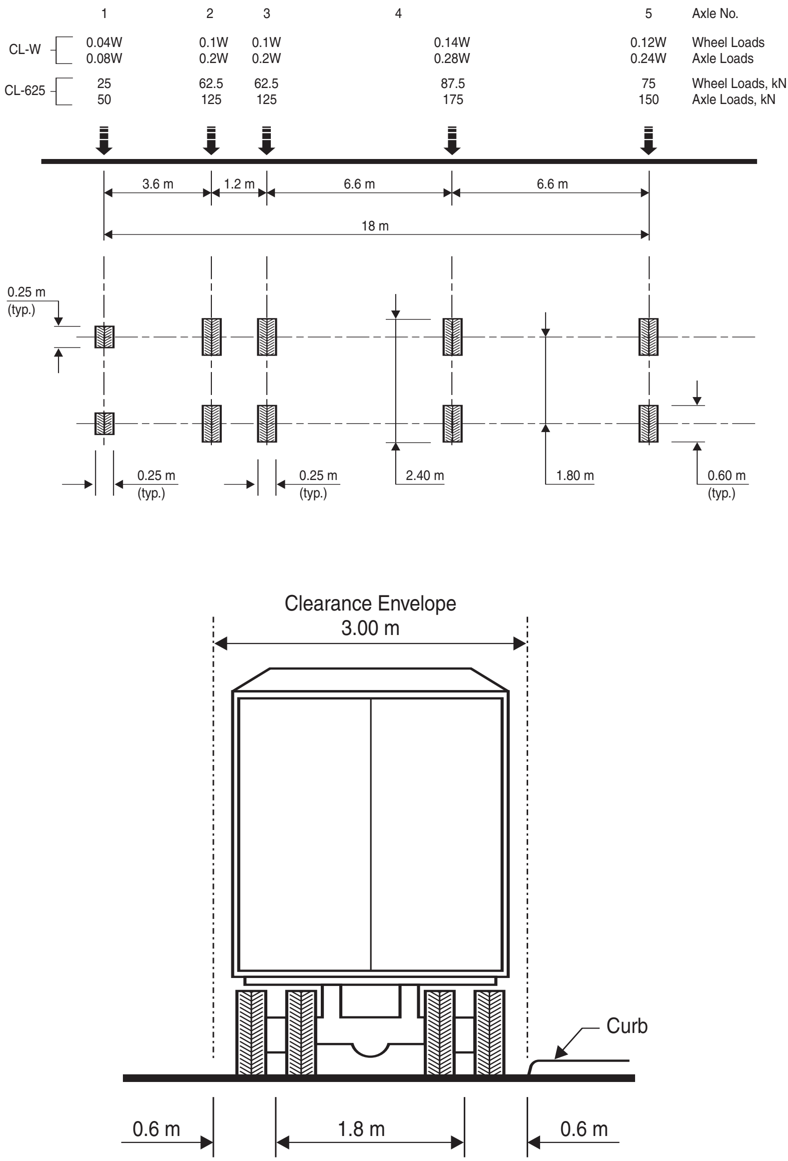


Figure 6.15 CHBDC CL-625 design truck.

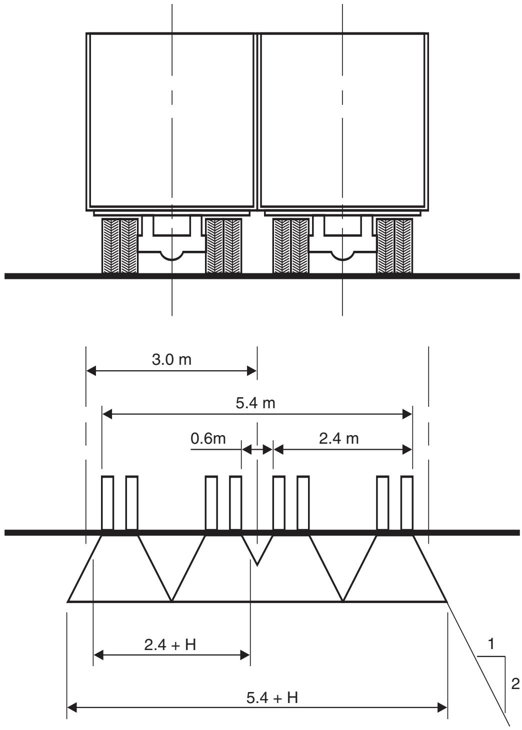


Figure 6.16 Positioning of adjacent design vehicles.

direction of the conduit and two vertically to one horizontally in the longitudinal direction;

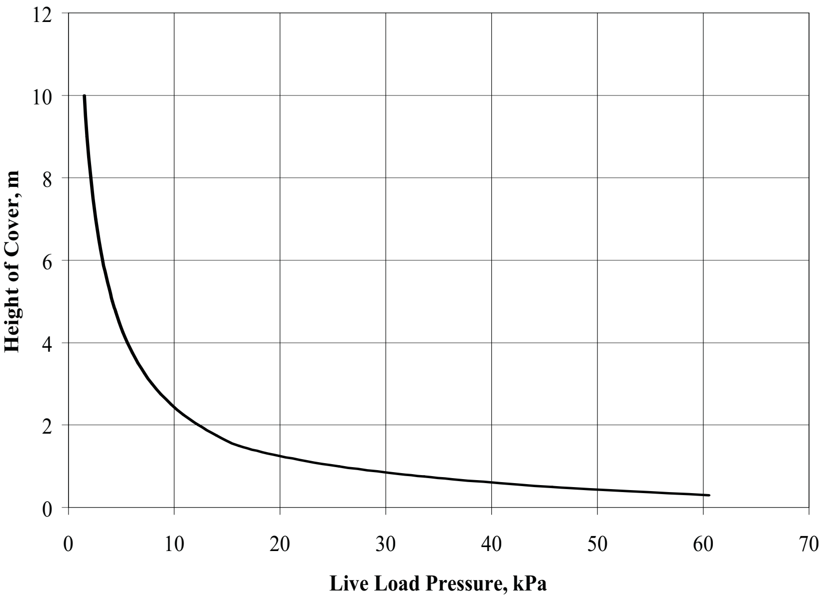
iii) obtain the equivalent unifromly distributed presure σ_L by assuming that the total wheel loads considered in item (i) are uniformly distributed over the rectangular area that encloses the individual rectangular areas obtained in item (ii); and

iv) Multiply the resulting pressure by the multi-lane loading modification factor.

The term $\sigma_L \bullet m_f$ can be regarded as the effective static live load pressure at the crown level. Since the value of this pressure varies only with the height of cover, it can be plotted as shown in Figure 6.17 (includes DLA). Tables 6.8 and 6.11 give the values of the effective pressure for specific heights of cover including the dynamic load allowances and neglecting it, respectively. It can be seen from Figure 6.17 that, for depths of cover larger than 4.5 m, the effective live-load pressure at the crown level is less than 5 kPa. This is less than 5% of the unfactored live plus dead load, and is therefore often ignored.

4. **Earthquake Thrust**

Buried structures should be designed to resist inertial forces associated with a seismic event having a 10% chance of being exceeded in 50 years. The vertical component of the earthquake acceleration ratio, A_V, is 2/3 of the horizontal ground acceleration ratio, A_H. A_H is the zonal acceleration ratio as specified in Clause 4.4.3 of the Canadian Highway Bridge Design Code. Amplification of these accelerations



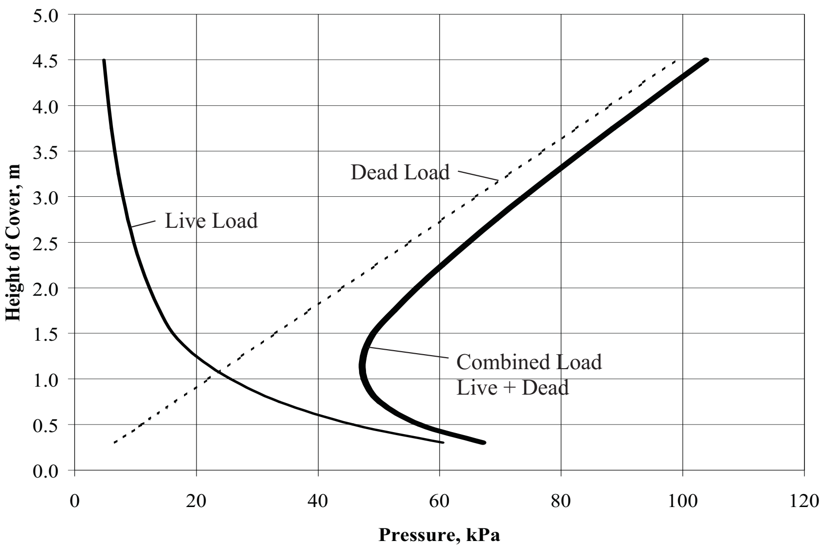


Figure 6.17 Variation of pressure with cover.

should be considered where a significant thickness of less competent soil overlies rock or firm ground.

The additional thrust due to earthquake loading, T_E, is obtained from:

$$T_E = T_D A_V$$

where: T_E = thrust in the wall of a soil-metal structure due to earthquake loading, kN/m

T_D = thrust in the structure wall due to unfactored dead load, kN/m

A_V = vertical acceleration ratio due to earthquake loading = 2/3 the horizontal acceleration ratio, A_H, dimensionless

The total factored thrust including the earthquake effects, T_f, is obtained from:

$$T_f = \alpha_D T_D + T_E = (\alpha_D + A_V)\, T_D$$

where: T_f = thrust in the structure wall due to factored loads, including earthquake loading, kN/m

α_D = dead load factor, dimensionless

5. **Total Thrust**

The thrust in the wall due to factored live loads and dead loads, T_f, is calculated according to the following equation:

$$T_f = \alpha_D T_D + \alpha_L T_L\,(1 + DLA)$$

where: T_f = thrust in the structure wall due to factored loads, kN/m

α_D = dead load factor, dimensionless

T_D = thrust in the wall due to unfactored dead load, kN/m

α_L = live load factor, dimensionless

T_L = thrust in the wall due to unfactored live load, kN/m

DLA = dynamic load allowance expressed as a fraction of live load

6. **Compressive Stress at the Ultimate Limit State (ULS)**

At the ULS, the compressive stress should not exceed the factored failure compressive stress.

$$\sigma \leq f_b$$

where: $\sigma = T_f / A$

σ = compressive stress at the ULS, MPa

f_b = factored wall failure stress in compression, MPa

T_f = factored thrust in the wall (maximum of the values in steps 4 and 5, kN/m

A = cross-sectional area of the corrugation profile, mm^2/mm

7. **Wall Strength in Compression**

a) for $R \leq R_e$

$$f_b = \phi_t F_m \left(F_y - \frac{(F_y KR)^2}{12\,E\,r^2 \rho} \right)$$

b) for $R > R_e$

$$f_b = 3\phi_t \rho F_m E \Big/ \left(\frac{KR}{r} \right)^2$$

where: i) E_m for the side and bottom portions of the structure wall should be the same as E_s, but for the top portion of the wall it is obtained from:

$$E_m = E_s \left[1 - \left(\frac{R_c}{R_c + 1000[H + H']} \right)^2 \right]$$

ii) λ for the top portion of the wall of all structures, except circular arches with rise-to-span ratios of less than 0.4, is obtained from:

$$\lambda = 1.22 \left[1.0 + 1.6 \left(\frac{EI}{E_m R_c^3} \right)^{1/4} \right]$$

For all other cases λ is 1.22.

iii) $$K = \lambda \left(\frac{EI}{E_m R^3} \right)^{1/4}$$

iv) $$\rho = \left(1000 \frac{[H + H']}{R_c} \right)^{1/2} \leq 1.0$$

v) $$R_e = \frac{r}{K} \left(\frac{6E\rho}{F_y} \right)^{1/2}$$

vi) F_m = 1.0 for single structure installations, and for multiple structures,

$$F_m = \left(0.85 + \frac{0.3S}{D_h} \right) \leq 1.0$$

and where:

D_h = horizontal dimension (effective span) of the largest structure, in the case of multiple structures, as defined in Figure 6.11, m

E = modulus of elasticity of the structure metal, MPa

E_m = modified modulus of soil stiffness, MPa

E_s = secant modulus of soil stiffness, as defined in Table 6.6, MPa

F_m = reduction factor for modifying wall strength in multi-structure installations

F_y = cold-formed yield strength of the structure wall, MPa

H = depth of cover, m

H' = half the vertical distance between crown and springline, m

I = moment of inertia of the corrugation profile, mm^4/mm

K = factor representing the relative stiffness of the structure wall with respect to the adjacent soil

R = radius of curvature of the wall, measured at the neutral axis of the corrugation, at a transverse section, mm

R_c = R at the crown or top of the structure, mm

R_e = equivalent radius, mm

r = radius of gyration of the corrugation profile, mm

S = the least transverse clear spacing between adjacent structures, m

λ = factor used in calculating K

ρ = reduction factor for buckling stress in the structure wall

ϕ_t = resistance factor for compressive strength of soil-metal structures = 0.8

Assembly of structural plate pipe.

8. **Strength Requirements During Construction**

The combined effects of the bending moment and axial thrust, arising from the unfactored dead load and the unfactored live load resulting from specified construction equipment, should not exceed the factored plastic moment capacity of the section at all stages of construction. The combined bending moment and axial thrust are calculated as follows:

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| \leq 1.0$$

where:

$P = T_D + T_C$ (for $H_c <$ minimum cover, P is assumed to be zero)

$P_{Pf} = \phi_{hc} A F_y$

$\left|\frac{M}{M_{Pf}}\right| =$ the absolute value of the ratio M/M_{Pf}

$M = M_l + M_B + M_C$

$M_{Pf} = \phi_{hc} M_P$

$M_l = k_{Ml} R_B \gamma D_h^3$

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$

$M_C = k_{M3} R_L D_h L_c$

$k_{M1} = 0.0046 - 0.0010 \text{ Log}_{10}(N_F)$ for $N_F \leq 5{,}000$
$k_{M1} = 0.0009$ for $N_F > 5{,}000$

$k_{M2} = 0.018 - 0.004 \text{ Log}_{10}(N_F)$ for $N_F \leq 5{,}000$
$k_{M2} = 0.0032$ for $N_F > 5{,}000$

$k_{M3} = 0.120 - 0.018 \text{ Log}_{10}(N_F)$ for $N_F \leq 100{,}000$
$k_{M3} = 0.030$ for $N_F > 100{,}000$

$R_B = 0.67 + 0.87[(D_v/2D_h) - 0.2]$ for $0.2 \leq D_v/2D_h \leq 0.35$
$R_B = 0.80 + 1.33[(D_v/2D_h) - 0.35]$ for $0.35 < D_v/2D_h \leq 0.5$
$R_B = D_v/D_h$ for $D_v/2D_h > 0.5$

$R_L = [0.265 - 0.053 \text{ Log}_{10}(N_F)]/(H_c/D_h)^{0.75} \leq 1.0$

$N_F = E_s(1000D_h)^3/EI$

$L_c = A_c/k_4$

and where:

A = cross-sectional area of the corrugation profile, mm^2/mm

A_c = axle load of construction equipment to be used above the structure during construction, kN

D_h, D_v = span and effective rise dimensions relating to the cross-sectional shape of the structure as defined in Figure 6.11

E = modulus of elasticity of the steel, MPa

E_s = secant modulus of soil stiffness, MPa (see Table 6.6)

F_y = cold-formed yield strength of the structural wall, MPa

H_c = depth of cover at intermediate stages of construction, m

I = moment of inertia about the neutral axis of the corrugated section, mm^4/mm

k_{M1}, k_{M2}, k_{M3} = factors used in calculating moments during construction

k_4 = factor used in calculating live load moments resulting from construction load, m (see Table 6.15)

L_c = line load equivalent to the construction load, kN/m

M = unfactored moment, kN.m/m

M_1 = moment resulting from fill to the crown level, kN.m/m

M_B = moment due to a height of fill, H_c, above the crown, kN.m/m

M_C = moment due to construction live loads, kN.m/m

M_P = unfactored plastic moment capacity, kN.m/m

M_{Pf} = factored plastic moment capacity, kN.m/m

N_F = flexibility number used in calculating moments during construction

P = unfactored thrust, kN/m

P_{Pf} = factored compressive strength, kN/m

R_B, R_L = parameters used in calculating moments during construction

T_C = additional thrust due to construction live loads, kN/m

T_D = thrust due to unfactored dead load, kN/m

γ = unit weight of soil, kN/m^3

ϕ_{hc} = resistance factor for formation of a plastic hinge = 0.90

Table 6.15 Values of the factor k_4 for calculating equivalent line loads

	k_4, m		
Depth of Cover, m	2 Wheels per Axle	4 Wheels per Axle	8 Wheels per Axle
0.3	1.3	1.5	2.6
0.6	1.6	2.0	2.8
0.9	2.1	2.7	3.2
1.5	3.7	3.8	4.1
2.1	4.4	4.4	4.5
3.0	4.9	4.9	4.9

9. **Wall Strength of Completed Structure With Deep Corrugations**

An additional check for soil-metal structures with deep corrugations requires the combined effects of the bending moment and axial thrust at the ultimate limit state shall not exceed the factored plastic moment capacity of the section in the completed (design) grade. The combined bending moment and axial thrust are calculated as follows:

$$\left[\frac{T_f}{P_{Pf}}\right]^2 + \left|\frac{M_f}{M_{Pf}}\right| \leq 1.0$$

where: Factored Comprehensive Strength of Section (P_{Pf}) = $\phi_h A F_y$
Factored Plastic Moment Capacity of Section (M_{Pf}) = $\phi_h M_p$
Maximum Thrust due to Factored Loads (T_f) as per step 2.
Maximum Moment due to Factored Loads -
$M_f = | \alpha_D M_1 + \alpha_D M_D | + \alpha_L M_L (1 + DLA)$

where: $M_1 = k_{M1} R_B \gamma D_h^3$
$M_D = - k_{M2} R_B \gamma D_h^2 H_e$ where H_e = smaller of H and $D_h/2$
$M_L = k_{M3} R_U\ D_h A_L / k_4$

$$R_U = \frac{[0.265\text{-}0.053\log_{10}(N_F)]}{\left(\frac{H}{D_h}\right)^{0.75}} \leq 1.0$$

Where k_{m1}, k_{m2}, k_{m3} and R_B are obtained from step 8., A_L is the weight of the second axle of the CL-W Truck and k_4 is obtained from Table 6.15. For H greater than 3.0 m, k_4 shall be assumed to be 4.9 m.

10. **Seam Strength**

The factored strength of longitudinal seams, $\phi_j S_s$, should not be less than T_f. The strength, S_s, may be evaluated experimentally or be from approved test data or published standards. In equation form:

$$\phi_j S_s \geq T_f$$

where: ϕ_j = resistance factor for connections = 0.70
S_s = axial strength of a longitudinal connection (see Tables 6.4b, 6.4c and 6.4d), kN/m
T_f = maximum thrust due to factored loads, kN/m

11. **Maximum Difference in Plate Thickness**
The difference in the thicknesses of lapping plates shall not exceed 1 mm if the thinner plate has a thickness of less than 3.1 mm, nor shall it exceed 1.5 mm if the thinner plate has a thickness between 3.1 and 3.5 mm. There is no limitation on the lapping plate thickness difference for connections where the thinner plate has a thickness exceeding 3.5 mm.

12. **Radius of Curvature**
The radius of curvature of the conduit wall, R, at any location, shall not be less than $0.2R_c$ (where R_c is the radius of the crown plate). The ratio of the radii of mating plates at a longitudinal connection should not be greater than 8.

Design Process: Metal Box Structures

The structural design process consists of the following:

1. Check dimensional requirements.
2. Check minimum cover.
3. Calculate dead load moments.
4. Calculate live load moments.
5. Calculate factored crown and haunch moments.
6. Calculate earthquake moments (if required).
7. Calculate flexural capacity at the ultimate limit state.
8. Check fatigue resistance.
9. Check seam strength.
10. Calculate the footing reaction.

1. **Dimensional Requirements**
The following provisions, accounting for soil-structure interaction, apply to the design of metal box structures (with element terminology as shown in Figure 6.18), having the dimensional limitations shown in Table 6.16 and a depth of cover up to 1.5 m. For metal box structures beyond these limits, the structure can be analysed using either the Non Linear Soil Structure Interaction Program (NLSSIP) or an acceptable alternate method.

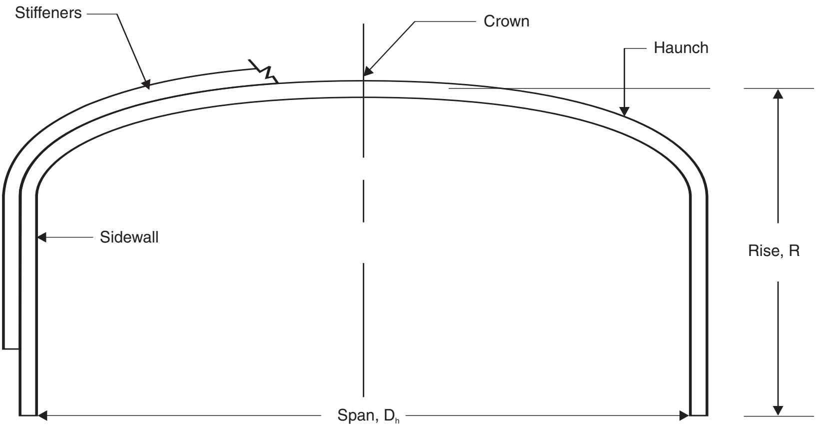


Figure 6.18 Metal-box culvert - terminology.

Table 6.16
Metal box structure dimensional limitations

Element	Minimum	Maximum
Rise (R)	0.8 m	3.2 m
Span (D_h)	2.7 m	8.0 m

2. **Minimum Cover**
For metal box structures designed in accordance with the CHBDC method, the minimum height of cover, H_{min}, illustrated in Figure 6.12 is 300 mm. Metal box structures differ greatly from soil-metal structures. They must be designed to resist live load moments.

3. **Dead Load Bending Moments**
The intensities of bending moments in the crown and the haunch due to dead loads, M_{cD} and M_{hD}, are fractions of M_D. They are given by:

$$M_D = k_1\gamma D_h^3 + k_2\gamma \left[H - \left(0.3 + \frac{d_c}{2000}\right)\right] D_h^2$$

$$M_{cD} = \kappa M_D$$

$$M_{hD} = (1-\kappa)\, M_D$$

where:
$k_1 = 0.0053 - 0.00024\,(3.28D_h - 12)$
$k_2 = 0.053$
$\kappa = 0.70 - 0.0328\, D_h$

and where: M_D = sum of the intensities of bending moments in the crown and haunch due to dead load, kN.m/m

k_1, k_2 = factors used in calculating dead load moment

γ = unit weight of soil, kN/m^3

D_h = span dimension of the structure cross-section as defined in Figure 6.18, m

H = depth of cover, m

M_{cD} = crown bending moment due to dead load, kN.m/m

κ = crown moment coefficient used to calculate the crown and haunch bending moments

M_{hD} = haunch bending moment due to dead load, kN.m/m

d_c = corrugation depth, mm

4. **Live Load Bending Moments**
The intensities of bending moments in the crown and the haunch due to live loads, M_{cL} and M_{hL}, are fractions of M_L. They are given by:

$$M_L = C_1 k_3 L_L D_h$$

$$M_{cL} = \kappa M_L$$

$$M_{hL} = (1-\kappa)\, k_R M_L$$

where: $k_3 = 0.08 / \left(\frac{H}{D_h}\right)^{0.2}$ for $D_h \leq 6.0$ m

$k_3 = [0.08 - 0.002(3.28D_h - 20)] / \left(\frac{H}{D_h}\right)^{0.2}$ for $6 \text{ m} < D_h < 8$ m

$L_L = A_L/k_4$

$k_R = 0.425H + 0.48 \leq 1.0$

$C_1 = 1.0$ for single axles, $0.5 + \frac{D_h}{15.24} \leq 1.0$ for multiple axles

and where: A_L = the weight of a single axle of the CHBDC truck for $D_h < 3.6$m or the combined weight of the two closely-spaced axles of the CHBDC truck for $D_h \geq 3.6$ m, kN

D_h = span dimension of the structure cross-section as defined in Figure 6.18, m

H = depth of cover, m

k_R = haunch moment reduction factor for metal box structure

k_3, k_4 = factors used in calculating live load moments (for k_4, see Table 6.14)

L_L = line load equivalent to the live load, kN/m

M_L = sum of the crown and haunch bending moments due to live load, kN.m/m

M_{cL} = crown bending moment due to live load, kN.m/m

M_{hL} = haunch bending moment due to live load, kN.m/m

κ = crown moment coefficient used to calculate the crown and haunch bending moments (see step 3. Dead Load Bending Moments)

5. Factored Crown and Haunch Bending Moments

The factored crown and haunch bending moments, M_{cf} and M_{hf}, induced by factored dead and live loads, shall be calculated according to the following equations:

$$M_{cf} = \alpha_D M_{cD} + \alpha_L M_{cL}\ (1 + DLA)$$
$$M_{hf} = \alpha_D M_{hD} + \alpha_L M_{hL}\ (1 + DLA)$$

where: M_{cf} = total factored crown bending moment, kN.m/m

M_{cD} = crown bending moment due to dead load, kN.m/m

M_{cL} = crown bending moment due to live load, kN.m/m

M_{hf} = total factored haunch bending moment, kN.m/m

M_{hD} = haunch bending moment due to dead load, kN.m/m

M_{hL} = haunch bending moment due to live load, kN.m/m

DLA = dynamic load allowance expressed as a fraction of the live load (see “Load Factors”)

α_D = dead load factor

α_L = live load factor

6. **Earthquake Bending Moments**

For metal box structures, the additional moment due to the effect of earthquake, M_E, is:

$$M_E = M_D \bullet A_V$$

where: $A_v = \frac{2}{3} A_H$

M_E = additional moment due to earthquake loading, kN.m/m

M_D = sum of the intensities of bending moments at the crown and haunch due to dead load, kN.m/m

A_V = vertical acceleration ratio due to earthquake loading, dimensionless

A_H = horizontal acceleration due to earthquake loading from the table provided in the CHBDC, dimensionless

The total factored moments, M_{cf} and M_{hf}, including the earthquake effects, are obtained as follows:

$$M_{cf} = \kappa\ (\alpha_D M_D + M_E)$$

$$M_{hf} = (1\text{-}\kappa)\ (\alpha_D M_D + M_E)$$

where: M_{cf} = total factored crown bending moment, kN.m/m

κ = crown moment coefficient used to calculate the crown and haunch bending moments

α_D = dead load factor

M_{hf} = total factored haunch bending moment, kN.m/m

7. **Flexural Capacity at Ultimate Limit State**
At the ULS, neither the factored crown moment, $M_{cf,}$ nor the factored haunch moment, M_{hf}, can exceed the factored plastic moment capacity M_{Pf}.

$$M_{cf} \leq M_{pf}$$

$$M_{hf} \leq M_{pf}$$

The values of M_{cf} and M_{hf} are the maximum values from steps 5 and 6.

The factored plastic moment capacity is calculated as:

$$M_{Pf} = \phi_h M_P$$

where: M_{Pf} = factored plastic moment capacity, kN.m/m

ϕ_h = resistance factor for plastic hinge = 0.9

M_P = unfactored plastic moment capacity of the section (see table 6.3), kN.m/m

Note: where plates are cross-corrugated to facilitate curving in the haunch areas, the haunch moment resistance shall be reduced accordingly.

8. **Fatigue Resistance**
Bolted seams should not be located in the vicinity of the crown nor in areas of maximum moments at the haunches. The computed stress range due to live load only should not exceed the stress range F_{sr}. The stress range must be determined by considering the stresses resulting from the live load moment extremes. The value for F_{sr}, for both corrugated plates and connections, is determined from section 10.17 of CAN/CSA-S6. The plate is considered as a category A stress range and the connection is considered as a category E stress range.

where F_{sr} = fatigue stress range for fatigue resistance

9. **Seam Strength**
For metal box structure walls designed only for bending moments, the factored moment resistance of longitudinal seams, $\phi_j S_M$, shall not be less than M_{cf} or M_{hf} (at the seam locations). For side walls designed for both axial thrust and bending moments, the factored axial strength of longitudinal seams, $\phi_j S_s$, shall not be less

than T_f. The strengths, S_M and S_s, may be evaluated experimentally or obtained from published standards. In equation form:

$$\phi_j S_M \geq M_{cf}, M_{hf}$$

$$\phi_j S_s \geq T_f$$

where:

S_M = flexural strength of a longitudinal connection (see Table 6.4c), kN.m/m

M_{cf}, M_{hf} = see Step 5, factored crown and haunch bending moments

S_s = axial strength of a longitudinal connection (see Table 6.4c), kN/m

T_f = maximum thrust due to factored loads, kN/m

ϕ_j = resistance factor for connections = 0.70

Connections shall be designed at the ultimate limit state for the larger of:

a) The calculated moment due to factored loads at the connection, and
b) 75% of the factored resistance of the member, $\phi_h M_p$.

10. **Footing Reactions**

The footing reaction for a box culvert may be determined using the following equation:

$$V = \gamma\,(HD_h/2 + D_h^2/40) + A_L/[2.4 + 2(H + R)]$$

where:

V = reaction acting in the direction of the box culvert straight side, kN/m

γ = unit weight of soil, kN/m^3

H = depth of cover, m

D_h = maximum span, m

A_L = axle load, kN

R = rise of box culvert, m

The vertical and horizontal footing reaction components are given by the equations:

$$V_V = V \cos\theta$$

$$V_H = V \sin\theta$$

where:

V_V = vertical footing reaction, kN/m

θ = angle between the sidewall and a vertical plane

V_H = horizontal footing reaction, kN/m

HEIGHT OF COVER TABLES

The following tables are presented for the designer's convenience for use in routine applications. They present the structural load-carrying capacity of CSP products, in terms of recommended minimum and maximum depths-of-cover. It is recommended that SPCSP structures be individually designed.

1. The values are based on the design methods which are outlined in this chapter. Tables are provided for structures designed using the AISI design method.

2. Live load includes impact.

3. The tables are based on the following values for soil and steel parameters:

Soil Group 1	- Well graded gravel or sandy gravel - Design compaction = 85% Standard Proctor Density - Compaction = 90% Standard Proctor Density (recommended for installation) - Unit weight - 19 kN/m^3 - Secant Modulus - 6 MPa - K = 0.86 (for $H \geq 0$)
Steel Yield Strength	- 230 MPa

4. Minimum cover is from neutral axis of corrugation profile at top of pipe to the bottom of flexible pavement. A footprint of 914 mm x 1016 mm at the bottom of the flexible pavement is used. Minimum cover may need to be increased for construction overloads.

5. Foundation investigation is recommended practice, to ensure adequate foundation support, particularly on high fills.

6. The steel wall thicknesses are industry recommended allowable minimums for structural design strength only, and do not consider other design factors such as unusual site conditions or abnormal environmental conditions affecting service life.

7. These minimum wall thicknesses assume that bedding and backfill material meet accepted engineering standards, compaction density is at least 85% Standard Proctor and that recognized workmanlike construction installation procedures are practiced. (Refer to Chapter 7).

8. The CSP industry recommends that drainage design warrants engineering consideration of all relevant factors towards metal thickness selection. The larger the pipe size, the greater attention that should be paid to all aspects of design and construction.

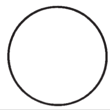
9. Where large or important projects can justify individual structure design, or when the quality of regular installations is known to be above that used here, the design procedure illustrated in the examples, included in this chapter, should be used with the appropriate values of soil and steel parameters.

List of Tables

Table No.	Shapes: Rnd Pipe	Shapes: Pipe-Arch	Shapes: Arch	Loading: CL-625	Loading: E 80	Size of Corrugations, mm: 68 × 13	76 × 25	125 × 25	152 × 51	Spiral Rib
HC- 1	X			X	X	X				
HC- 2	X			X	X		X			
HC- 3	X			X	X			X		
HC- 4		X		X		X				
HC- 5		X		X			X	X		
HC- 6		X			X	X				
HC- 7	X			X	X				X	
HC- 8		X		X					X	
HC- 9			X	X					X	
HC-10			X		X				X	
HC-11	X			X						X
HC-12		X		X						X

Table HC-1

Corrugated Steel Pipe (CSP)
Corrugation Profile **68 x 13 mm**



Helical or Annular CSP

Inside Diameter mm	Minimum Cover CL-625	Minimum Cover E-80	Maximum Cover, m — Specified Wall Thickness, mm: 1.3	1.6	2.0	2.8	3.5	4.2
	mm	mm						
300	300	300	54	67	88			
400	300	300	41	51	67			
500	300	300	33	41	53	78		
600	300	300	27	34	45	65		
700	300	300		29	38	56		
800	300	300		26	34	49		
900	300	300		23	30	43	55	
1000	300	300		21	27	39	50	61
1200	300	300			22	33	42	51
1400	300	500				26	34	41
1600	300	500				21	27	33
1800	300	500					22	27
2000	300	500						21

Notes: (1) Important - please refer to foreword on these tables.
(2) Pipe sizes above the heavy line have flexibility factors (FF) not exceeding 0.245 mm/N.

Table HC-2

Corrugated Steel Pipe (CSP)
Corrugation Profile **76 x 25 mm**

Helical or Annular CSP

Inside Diameter mm	Minimum Cover		Maximum Cover, m				
	CL-625	E-80	Specified Wall Thickness, mm				
	mm		1.6	2.0	2.8	3.5	4.2
1200	300	500	19	25	37		
1400	300	500	17	22	32	41	
1600	300	500	15	19	28	36	44
1800	300	500	13	17	25	32	39
2000	300	500	12	15	22	28	35
2200	300	700	10	14	20	26	32
2400	300	700		13	19	24	29
2700	500	700		11	16	20	25
3000	500	1000			13	17	21
3300	500	1000			11	14	18
3600	500	1000				12	15

Notes: (1) Important - please refer to foreword on these tables.
(2) Pipe sizes above the heavy line have flexibility factors (FF) not exceeding 0.188 mm/N.

Table HC-3

Corrugated Steel Pipe (CSP)
Corrugation Profile **125 x 25 mm**

Helical CSP

Inside Diameter mm	Minimum Cover		Maximum Cover, m				
	CL-625	E-80	Specified Wall Thickness, mm				
	mm		1.6	2.0	2.8	3.5	4.2
1200	300	500	17	23	33		
1400	300	500	15	19	28	36	44
1600	300	500	13	17	25	32	39
1800	300	500	11	15	22	28	34
2000	300	500	10	13	20	25	31
2200	300	700	9	12	18	23	28
2400	300	700	8	11	16	21	26
2700	500	700		10	15	19	23
3000	500	1000			12	16	19
3300	500	1000			10	13	16
3600	500	1000				11	14

Notes: (1) Important - please refer to foreword on these tables.
(2) Pipe sizes above the heavy line have flexibility factors (FF) not exceeding 0.188 mm/N.

Table HC-4

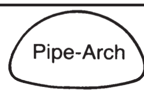
CL-625 Highway Loading
Corrugation Profile: **68 x 13 mm**

Span, mm	Rise, mm	Minimum Cover, mm	Minimum Specified Wall Thickness, mm	Maximum Depth of Cover, m to restrict Corner Pressure to the following:		
				200 kPa	300 kPa	400 kPa
560	420	300	1.6	4.8	7.4	9.9
680	500	300	1.6	5.0	7.6	10.1
800	580	300	1.6	4.9	7.4	9.9
910	660	300	1.6	4.9	7.5	10.0
1030	740	300	1.6	4.8	7.3	9.7
1150	820	300	1.6	4.7	7.2	9.6
1390	970	300	1.6	4.6	7.1	9.5
1630	1120	300	2.0	4.6	7.0	9.4
1880	1260	300	2.8	4.5	6.8	9.2
2130	1400	300	3.5	4.4	6.8	9.1

Notes: (1) Important - please refer to foreword on these tables.

Table HC-5

CL-625 Highway Loading
Corrugation Profiles
76 x 25 mm and 125 x 25 mm

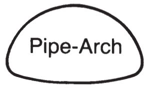


Span, mm	Rise, mm	Minimum Cover, mm	Minimum Specified Wall Thickness, mm		Maximum Depth of Cover, m to restrict Corner Pressure to the following:		
			76 x 25	125 x 25	200 kPa	300 kPa	400 kPa
1330	1030	300	2.0	2.0	5.2	7.9	10.5
1550	1200	300	2.0	2.0	5.2	7.9	10.5
1780	1360	300	2.0	2.0	5.2	7.9	10.6
2010	1530	300	2.0	2.0	5.1	7.8	10.4
2230	1700	300	2.0	2.0	5.4	8.1	10.9
2500	1830	350	2.0	2.0	5.3	8.0	10.7
2800	1950	350	2.0	2.0	5.2	7.9	10.6

Notes: (1) Important - please refer to foreword on these tables.

Table HC-6

E-80 Railway Loading
Corrugation Profiles **68 x 13 mm**



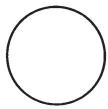
Span, mm	Rise, mm	Minimum Specified Wall Thickness, mm — Depth-of-Cover Range*, m: 0.6 to 0.9 m	0.9 to 1.5	1.5 to 2.4	2.4 to 4.6
560	420	2.8	2.0	2.0	2.0
680	500	2.8	2.8	2.0	2.0
800	580	3.5	2.8	2.8	2.8
910	660	—	3.5	2.8	2.8
1030	740	—	3.5	3.5	2.8
1150	820	—	—	3.5	3.5
1390	970	—	—	—	3.5

* Values interpolated from empirical results gathered over 25 years.

NOTES: (1) Please refer to Foreword.
(2) Live load includes impact, and dead load is based on a unit weight of backfill material of 19 k N/m³.

Table HC-7

Depth-of-Cover Limits for SPCSP Round Pipe
Highway H-20, or Railway E-80 Loadings
Corrugation Profile **152 x 51 mm**



Inside Diameter mm	Periphery (Hole Spaces) N	Minimum Cover, mm: CL-625	E-80	Maximum Cover, m — Specified Wall Thickness, mm: 3.0	4.0	5.0	6.0	7.0
1500	20N	300	500	31	43	55	67	79
1660	22N	300	500	28	39	50	61	71
1810	24N	300	500	26	36	46	56	65
1970	26N	300	500	24	33	42	51	60
2120	28N	300	500	22	31	39	48	56
2280	30N	300	500	21	29	37	45	52
2430	32N	500	500	19	27	34	42	49
2590	34N	500	700	18	25	32	39	46
2740	36N	500	700	17	24	31	37	43
3050	40N	500	700	15	21	27	33	39
3360	44N	500	700	14.5	19	25	30	35
3670	48N	500	1000	13	18	23	28	32
3990	52N	700	1000	12	16.5	21	25	30
4300	56N	700	1000	11	15.5	19.5	24	28
4610	60N	700	1000	10.5	14.5	18.5	22	26
4920	64N	700	1000	9.5	13.5	17	21	24.5
5230	68N	700	1250	9	12	15.5	19	22.5
5540	72N	700	1250		11	14.5	17.5	20.5
5850	76N	1000	1250		10.5	13	16	19
6160	80N	1000	1250			12	15	17.5
6470	84N	1000	1500			11	13.5	16
6780	88N	1000	1500			10	12.5	14.5
7090	92N	1000	1500				11.5	13.5
7400	96N	1000	1500				10.5	12
7710	100N	1000	1500					11
8020	104N	1000	1500					10

Notes: (1) Important - please refer to foreword on these tables.
(2) Pipe sizes above the heavy line have flexibility factors (FF) not exceeding 0.114 mm/N.

Table HC-8

Depth-of-Cover Limits for SPCSP Pipe-Arch
CL-625 Highway Loadings
Corrugation Profile **152 x 51 mm**

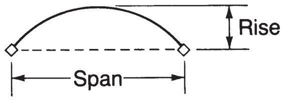
Span, mm	Rise, mm	Minimum Cover, mm	Minimum Thickness, mm	Maximum Depth of Cover, m to restrict Corner Pressure to the following:			
				100 kPa	200 kPa	300 kPa	400 kPa
2050	1520	300	3.0	2.9	6.0	9.1	12.2
2240	1630	300	3.0	2.7	5.6	8.6	11.5
2440	1750	350	3.0	2.5	5.4	8.2	11.0
2590	1880	350	3.0	2.5	5.4	8.2	11.0
2690	2080	350	3.0	2.8	5.9	8.9	11.9
3100	1980	400	3.0	1.9	4.1	6.3	8.4
3400	2010	450	3.0	1.4	3.3	5.1	6.9
3730	2290	500	3.0	1.5	3.5	5.3	7.2
3890	2690	500	3.0	1.9	4.2	6.4	8.6
4370	2870	550	3.0	1.6	3.6	5.6	7.5
4720	3070	600	3.0	1.5	3.4	5.2	7.0
5050	3330	650	3.0	1.5	3.3	5.0	6.8
5490	3530	700	3.0	1.3	3.0	4.6	6.2
5890	3710	750	3.0	1.2	2.8	4.3	5.8
6250	3910	800	4.0	1.1	2.6	4.1	5.5

Notes: (1) Important - please refer to foreword on these tables.

Table HC-9

$.30 \leq \frac{\text{Rise}}{\text{Span}} \leq .50$

Depth-of-Cover Limits for SPCSP Arches
CL-625 Highway Loadings
Corrugation Profile **152 x 51 mm**



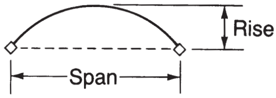
Inside Dimensions		Minimum Cover	Maximum Cover, m				
			Specified Wall Thickness, mm				
Span mm	Radius mm	mm	3.0	4.0	5.0	6.0	7.0
1520	760	300	23	32	41	50	58
1830	930	300	19	27	34	41	48
	910	300	19	27	34	41	48
2130	1090	300	17	23	29	36	42
	1070	300	17	23	29	36	42
2440	1230	500	14	20	26	31	36
	1220	500	14	20	26	31	36
2740	1400	500	13	18	23	28	32
	1370	500	13	18	23	28	32
3050	1540	500	11	16	20	25	29
	1520	500	11	16	20	25	29
3350	1710	500	10.5	14	18	23	26
	1680	500	10.5	14	19	23	27
3660	1850	500	9.5	13	17	21	24
	1830	500	9.5	13	17	21	24
3960	2010	500	9	12	16	19	22
	1980	500	9	12	16	19	22
4270	2160	700	8.5	11	15	18	21
	2130	700	8.5	11	15	18	21
4570	2340	700	7.5	10	13	16	19
	2290	700	7.5	10	14	17	19
4880	2480	700		10	13	15	18
	2440	700		10	13	15	18
5180	2620	700		9	12	14	17
	2590	700		9	12	14	17
5490	2820	700		8	10	13	15
	2740	700		8.5	11	13	15
5790	2950	1000			10	12	14
	2900	1000			10	12	14
6100	3100	1000			9	11	13
	3050	1000			9	11	13

Notes: (1) Important - please refer to foreword on these tables.
(2) For structural plate arches R/S > .50, use round pip tables or 1.39 x these values.

Table HC-10

Depth-of-Cover Limits for SPCSP Arches
E-80 Railway Loadings
Corrugation Profile **152 x 51 mm**

$.30 \lessapprox \frac{Rise}{Span} \lessapprox .50$



Inside Dimensions*		Minimum Cover, mm	Maximum Depth-of-Cover, m				
			Specified Wall Thickness, mm				
Span, mm	Radius, mm		3.0	4.0	5.0	6.0	7.0
1520	760	500	24	34	43	52	61
1830	930		20	27	35	42	49
	910		20	28	36	43	51
2130	1090		17	23	30	36	42
	1070		17	24	30	37	43
2440	1230		15	21	26	32	37
	1220	500	15	21	27	32	38
2740	1400	700	13	18	23	28	33
	1370		14	19	24	29	34
3050	1540		12	17	21	26	30
	1520		12	17	21	26	30
3350	1710		11	15	19	23	27
	1680	700	11	15	19	23	27
3660	1850	1000	10	14	18	21	25
	1830		10	14	18	22	25
3960	2010		9	13	16	20	23
	1980		9.5	13	16	20	23
4270	2160		8	12	15	18	21
	2130		8.5	12	15	18	22
4570	2340		7	11	14	17	20
	2290		7.5	11	14	17	20
4880	2480			10	13	16	19
	2440	1000		10	13	16	19
5180	2620	1250		9.5	12	15	17
	2590			9.5	12	15	17
5490	2820			8.0	11	13	15
	2740			8.5	11	14	16
5790	2950				10	12	14
	2900				10	13	15
6100	3100				9.5	11	13
	3050	1250			9.5	12	14

For structural plate arches R/S ⩾ .50 use round pipe tables or 1.39 × these values.

Table HC-11

Depth-of-Cover Limits for round Spiral Rib Pipes
19 x 19 x 190 mm rib profile
CL-625 live load

Diameter, mm	Minimum Cover, mm	Maximum Cover, m Specified Thickness, mm		
		1.6	2.0	2.8
450	300	22.7	33.6	
525	300	19.4	28.8	50.6
600	300	17.0	25.2	44.3
750	300	13.6	20.2	35.4
900	300	11.3	16.8	29.5
1050	300	9.7	14.4	25.3
1200	300	8.5*	12.6	22.1
1350	350	7.5*	11.2	19.7
1500	400	6.8*	10.1*	17.7
1650	450		9.1*	16.1
1800	450		8.4*	14.7
2100	550			12.6*
2400	600			11.0*

*These installations require attention to backfill material and compaction methods used. Refer to the "Design of Buried Structures With Spans Up To 3 m - Handling Stiffness" discussion for more details on the various installation types.

Table HC-12

Depth-of-Cover Limits for Spiral Rib Pipe-arches
19 x 19 x 190 mm rib profile
CL-625 live load

Span, mm	Rise, mm	Equivalent Diameter, mm	Minimum Cover, mm	Maximum Height of Fill (m) to Limit Corner Bearing Pressures to a Maximum of 200 kPa — Metal Thickness (mm) 1.6	2.0	2.8
500	410	450	300	4.0	4.0	
580	490	525	300	5.2	5.2	5.2
680	540	600	300	5.2	5.2	5.2
830	660	750	300	5.2	5.2	5.2
1010	790	900	300	4.4	4.4	4.4
1160	920	1050	300	5.1	5.1	5.1
1340	1050	1200	300		4.4	4.4
1520	1200	1350	340		5.3*	5.3
1670	1300	1500	380		5.1*	5.1
1850	1400	1650	410		4.7*	4.7

*These installations require attention to backfill material and compaction methods used. Refer to the "Design of Buried Structures With Spans Up To 3 m - Handling Stiffness" discussion for more details on the various installation types.

DESIGN EXAMPLES

The following examples illustrate the application of design procedures using the CHBDC design method summarized in the preceding pages. They include:

1. 8955 x 6070 mm horizontal ellipse under 3.0 m of cover
2. 6260 x 3910 mm pipe-arch under 2.0 m of cover
3. 6165 x 1900 mm metal box under 0.90 m of cover
4. 3600 mm diameter pipe under 8.0 m of cover, located in Vancouver, B.C., (seismic consideration)
5. 6100 x 2530 mm arch under 1.1 m of cover
6. 11000 x 6385 mm arch under 2.5 m of cover
7. 13000 x 4512 mm arch under 3.0 m of cover

Example 1

Given: Horizontal Ellipse, 8955 mm span x 6070 mm rise
Height of Cover, H =3.0 m
CL 625 Live Load
Soil Group 1, 90%-95% Standard Proctor Density
Unit Weight of Soil $\gamma = 22\,kN/m^3$
Secant Modulus Es = 12 MPa

Required: Determine wall thickness for a 152 x 51 corrugation

Geometric Data: D_h = 9.006 m, D_v = 6.121 m (neutral axis)
Crown radius (Rc) = 5763 mm (N.A.), $\theta_{top} = 80^0$
Side radius (R_2) = 2235 mm (N.A.)
Bottom radius (R_3) = 5763 mm (N.A.)

1. Minimum Cover (H_{min}) is the largest of:

a) 0.6 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{0.5}$ = 1.82 m <u>Governs</u>

c) $0.4\left(\frac{D_h}{D_v}\right)^2 = 0.87m$

H_{min} = 1.82 m, H_{min}<H (3.0 m)

2. Dead Load Thrust (T_D)

$T_D = 0.5\,(1.0 - 0.1\,C_S)\,A_f\,W$
A_f = 1.094 (Figure 6.13)

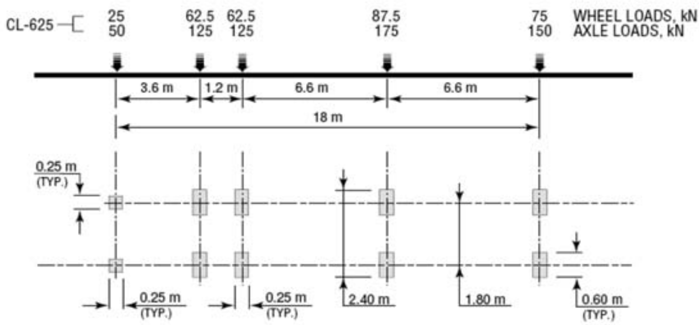
$$C_s = \frac{1000 E_S D_v}{EA}$$

Where: E = 200 000 MPa, E_s = 12 MPa
6.0mm Plate Area = 7.461 mm²/mm
C_s = 0.0492

W= $\gamma\left[(HD_h) + Area\ Between\ Springline\ \&\ Crown\right]$
TopRise = 3.06m, N.A. Inside End Area of structure above springline = 21.26 m²
$W = 22[3.0(9.006) + 6.3] = 733.1$ kN/m
$T_D = 0.5[1.0\text{-}0.1(0.0492)]1.094(733.1)$
$\boldsymbol{T_D = 399.1}$ **kN/m**

3. Live Load Thrust (T_L)

$T_L = 0.5$ *(lesser of D_h and l_t)* $\sigma_L m_f$



Position as many axles of the CL-625 truck within the span length, (D_h = 9.006 m), at the road surface as would give the maximum total load. In this example, placing axles 2, 3, and 4 above the structure produces the highest total load (P = 425 kN)

1 lane, (m_f =1.0)

$$\sigma_L = \frac{AxleLoad}{l_t w} \quad \text{where:}$$

l_t = a + 2(H) = 8.05 + 2(3) = 14.05 m

w = b + H = 1.8 + 0.6 + 3 = 5.4m

$$\sigma_L = \frac{425}{14.05(5.4)} = 5.6\ kPa$$

$$\sigma_L m_f = 5.6(1.0) = 5.6\ kPa$$

2 lanes, (m_f =0.9)

l_t (14.05 m) > D_h (9.006 m)

$w = 5.4 + H = 8.4m$

$$\sigma_L = \frac{2(425)}{14.05(8.4)} = 7.2\,kPa$$

$\sigma_L m_f = 7.2(0.9) = 6.48\ kPa$ <u>Governs</u>

T_L = 0.5 (lesser of D_h and l_t) $\sigma_L m_f$

= 0.5 (9.008) 6.48

$\boldsymbol{T_L = 29.19\ kN/m}$

4. Earthquake Thrust – Not Applicable

5. Total Factored Thrust *(T_f)*

$T_f = \alpha_D T_D + \alpha_L T_L\ (1+ DLA)$

$\alpha_D = 1.25$

$\alpha_L = 1.70$

$DLA = 0.40(1\text{-}0.5D_e) \geq 0.10$

Where: D_e = H = 3.0 m

$DLA = 0.10$

T_f = 1.25(399.1) + 1.70(29.19)(1.1)

$\boldsymbol{T_f = 543.4\ kN/m}$

6. Compressive Stress at ULS (σ)

$\sigma = \dfrac{T_f}{Area}$ where Area = 7.461 mm^2/mm for 6.0 mm plate thickness

σ **= 74.2 *MPa***

7.0 Wall Strength in Compression

Definition of upper zone

i) $\theta_0 = 1.6 + 0.2\log\left[\dfrac{EI}{E_m R^3}\right]$ radians where:

$$E_m = E_S\left[1 - \left[\frac{R_C}{R_C + 1000[H + H']}\right]^2\right]$$

E_s = 12.0 MPa, R = R_c = 5763 mm, H = 3.0 m, H' = $\dfrac{D_v}{4}$ = 1.53 m

$I_{6.0mm} = 2278.3\dfrac{mm^4}{mm}$ then

E_m = 8.237 MPa, $\theta_0 = 51.1^0$

ii) $\lambda = 1.22\left[1.0 + 1.6\left[\dfrac{EI}{E_m R_c^3}\right]^{0.25}\right]$

$\lambda = 1.4745$

iii) $K = \lambda\left[\dfrac{EI}{E_m R^3}\right]^{0.25}$

$K = 0.1922$

iv) $\rho = \left[1000\dfrac{(H + H')}{R_c}\right]^{0.5} \leq 1.0$

$\rho = 0.8866$

v) $R_e = \dfrac{r}{K}\left[\dfrac{6E\rho}{F_y}\right]^{0.5}$

for 6.0 mm plate thickness, r = 17.48 mm

R_e = 6185 mm

vi) F_m = 1.0 for single conduit

vii) $R < R_e$

$$f_b = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\rho}\right]$$

***f_b* = 104.1 MPa**

As $\sigma\,(74.2) < f_b(104.1)$, 6.0 mm plate thickness satisfies compressive stress criteria where R =5763 mm in the upper zone.

viii) Check Lower Zone Arcs for Wall Strength in Compression

$\lambda = 1.22$, $E_m = E_s = 12$ MPa, $\rho = 0.8866$

$R_2 = 5763$ mm (top/bottom) $R_3 = 2235$ mm (side)

	R_2 (Top/Bottom)	R_3 (side)
K	0.1448	0.2946
R_e	8210 mm	4035 mm
f_b	$R_2<R_e$ $f_b = 138.7\ MPa$	$R_3<R_e$ $f_b = 155.8\ MPa$

Where:

$$K = \lambda\left[\frac{EI}{E_m R^3}\right]^{.25}$$

$$R_e = \frac{r}{K}\left[\frac{6E\rho}{F_y}\right]^{0.5} mm$$

$$f_b = \phi_t F_m\left[F_y - \frac{\left(F_y KR\right)^2}{12Er^2\rho}\right] MPa$$

when R $\le R_e$

As $\sigma(74.2) < f_b$, 6.0 mm plate thickness satisfies compressive stress criteria for all radii within the lower zone.

8.0 Strength Requirements During Construction

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \le 1$$

Where:

$P = T_D + T_C$ and for $H_c < H_{min}$ $P = 0$

$P_{pf} = \phi_{hc} A F_y$

$M = M_1 + M_B + M_C$

$M_{Pf} = \phi_{hc}\ M_p$

Where:

$M_1 = k_{M1} R_B\, \gamma\, D_h^3$

$M_B = -\,k_{M2}\, R_B\, \gamma\, D_h^2\, H_c$

$M_C = k_{M3}\, R_L\, D_h\, L_C$

Where:

$k_{M1} = 0.0046 - 0.0010 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0009$ for $N_F > 5000$

$k_{M2} = 0.018 - 0.004 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0032$ for $N_F > 5000$

$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$

$= 0.030$ for $N_F > 100\,000$

$R_B = 0.67 + 0.87\left[\left(D_v / 2D_h\right) - 0.2\right] for\ 0.2 \le D_v / 2D_h \le 0.35$

$= 0.80 + 1.33\left[\left(D_v / 2D_h\right) - 0.35\right] for\ 0.35 < D_v / 2D_h \le 0.50$

$= D_v / D_h\ for\ D_v / 2D_h > 0.5$

$$R_L = \frac{\left[0.265 - 0.053\log_{10}\left(N_F\right)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$

$$L_c = \frac{A_C}{k_4}$$

$$\left|\frac{M}{M_{Pf}}\right| = \textit{absolute value of the ratio } \frac{M}{M_{Pf}}$$

$$N_F = E_s(1000D_h)^3 / EI$$

Select $H_c < H_{min}$, $H_c = 0.6$m, therefore P=0 as $H_c < H_{min}$

a) Assume Construction Axle (A_c) = 250 kN, 4 wheels per axle
$M = M_1 + M_B + M_C$

Calculate Flexibility Number N_F
$N_F = E_s(1000D_h)^3 / EI$
E_s =12, D_h = 9.006, I =2278.3 mm^4/mm
N_f = 19237

k_{M1} = 0.0009 (for N_f > 5000)
k_{M2} = 0.0032 (for N_f > 5000)
$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ when $N_F \le 100\,000$
= 0.0429

$D_v/2D_h$ = 0.339
$R_B = 0.67 + 0.87\left[(D_v/2D_h) - 0.2\right]$ *for* $0.2 \le D_v/2D_h \le 0.35$
R_B = 0.7917

$M_1 = k_{M1} R_B \gamma D_h^3$
M_1 = 11.45 kN-m/m

$M_B = -k_{M2} R_B \lambda D_h^2 H_c$
M_B = -2.71 kN-m/m

$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$

R_L = 0.2893
$L_c = \frac{A_C}{k_4}$ where k_4 = 2.0m, (Table 6.15, 4 wheels per axle & H_c = 0.6m)
Lc = 250/2 = 125.0 kN/m

$M_C = k_{M3} R_L D_h L_C$
M_C = 13.97 kN-m/m

$M = M_1 + M_B + M_C$
M = 11.45 + (-2.72) +13.97 = 22.71 kN-m/m

$M_{Pf} = \phi_{hc} M_p$
$\phi_{hc} = 0.9$, $M_p = 26.69\ kN-m/m$
M_{Pf} = 24.02 kN-m/m

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \leq 1, \text{P} = 0$$

$$\left|\frac{M}{M_{pf}}\right| = \left|\frac{22.71}{24.02}\right| = 0.94 < 1.0$$

Therefore 6.0 mm plate thickness satisfies wall strength requirements during construction for 0.6 m of cover. It is recommended to check other applicable construction axle loads and cover heights in the same manner.

9.0 Factored Longitudinal Seam Strength *(S_f)*

$T_f < S_f$

Where $S_f = \phi_j S_s$ and: $\phi_j = 0.70$, $S_s = 1840$ kN/m for 6.0 mm plate thickness

S_f = 1288 kN/m

$T_f = 553.4 < S_f$ OK

10.0 Plate thickness difference – Not Applicable

11.0 Radius of Curvature

$\frac{R_{crown}}{R} \leq 5.0$, where: $R_{crown} = 5763$ mm, $R_{side} = 2235$ mm

$\frac{5763}{2235} = 2.6$ OK

Example 2

Given: Pipe-arch, 6250 mm span x 3910 mm rise
Height of Cover, H =2.0 m
CL 625 Live Load
Soil Group 1, 90%-95% Standard Proctor Density
Unit Weight of Soil $\gamma = 22\ kN/m^3$
Secant Modulus Es = 12 MPa

Required: Determine wall thickness for a 152 x 51 corrugation

Geometric Data: $D_h = 6.297$ m, $D_v = 5.652$ m (neutral axis)
Crown radius (Rc) = 3175 mm (N.A.), $\theta_{top} = 162.8^0$
Corner or haunch radius (R_2) = 840 mm (N.A.)
Bottom radius (R_3) = 9625 mm (N.A.)

1. Minimum Cover (H_{min}) is the largest of:

a) 0.6 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{0.5} = 1.159$ m <u>Governs</u>

c) $0.4\left(\frac{D_h}{D_v}\right)^2 = 0.497\ m$

$H_{min} = 1.159\ m$, $H_{min} < H\ (2.0\ m)$

2. Dead Load Thrust (T_D)

$T_D = 0.5\ (1.0 - 0.1\ C_S)\ A_f\ W$
$A_f = 1.20$ (Figure 6.13)
$$C_s = \frac{1000 E_S D_v}{EA}$$
Where: E = 200 000 MPa, E_s = 12 MPa
4.0 mm Plate Area = 4.828 mm^2/mm
$C_s = 0.0702$

$W = \gamma\left[(H\ D_h) + Area\ Between\ Springline \,\&\, Crown\right]$
TopRise = 2.80m, N.A. Inside End Area of structure above springline = 13.596m^2
$W = 22[2.0(6.297) + 4.03]$
$W = 365.85$ kN/m
$T_D = 0.5[1.0\text{-}0.1(.0702)]1.2(365.85)$
$\boldsymbol{T_D = 217.97}$ **kN/m**

3. Live Load Thrust (T_L)

$T_L = 0.5$ (lesser of D_h and l_t) $\sigma_L m_f$

Position as many axles of the CL-625 truck within the span length, (D_h = 6.297 m), at the road surface as would give the maximum total load. In this example, placing axles 2 and 3 above the structure produces the highest total load (P = 250 kN)

1 lane, (m_f = 1.0)
$$\sigma_L = \frac{AxleLoad}{l_t w} \quad \text{where:}$$
$l_t = a + 2(H) = 1.45 + 2(2) = 5.45$ m
$w = b + H = 2.4 + 2 = 4.4$ m
$$\sigma_L = \frac{250}{5.45(4.4)} = 10.42\ kPa$$
$$\sigma_L\, m_f = 10.42\ kPa$$

2 lanes, (m_f = 0.9)
l_t (5.45 m) < D_h (6.297 m)
$w = b + H = 5.4 + 2 = 7.4$ m
$$\sigma_L = \frac{2(250)}{5.45(7.4)} = 12.40\,kPa$$
$\sigma_L\, m_f = 11.16\ kPa$ <u>Governs</u>

$T_L = 0.5$ (lesser of D_h and l_t) $\sigma_L m_f$
$= 0.5\ (5.45)\ 11.16$
$\boldsymbol{T_L = 30.41}$ **kN/m**

4. Earthquake Thrust – Not Applicable

5. Total Factored Thrust (T_f)

$T_f = \alpha_D T_D + \alpha_L T_L\ (1 + DLA)$
$\alpha_D = 1.25$
$\alpha_L = 1.70$
$DLA = 0.40(1\text{-}0.5D_e) \geq 0.10$

Where: $D_e = H = 2.0$m
$DLA = 0.10$
$T_f = 1.25(217.97) + 1.70(30.41)(1.10)$
$\boldsymbol{T_f = 329.33}$ **kN/m**

6. Compressive Stress at ULS (σ)

$\sigma = \dfrac{T_f}{Area}$ where Area = 4.828 mm^2/mm for 4.0 mm plate thickness

σ = 68.2 MPa

7.0 Wall Strength in Compression (f_b)

Definition of upper zone

i) $$\theta_0 = 1.6 + 0.2\log\left[\frac{EI}{E_m R^3}\right]$$ radians where:

$$E_m = E_S\left[1 - \left[\frac{R_C}{R_C + 1000[H + H']}\right]^2\right]$$

E_s = 12.0 MPa, R = R_c = 3175mm, H = 2.0m, H' = $\dfrac{D_v}{4}$ = 1.413 m

$I_{4.0mm} = 1457.6\dfrac{mm^4}{mm}$ then

E_m = 9.213 MPa, $\theta_0 = 57.3^0$

ii) $$\lambda = 1.22\left[1.0 + 1.6\left[\frac{EI}{E_m R_c^3}\right]^{0.25}\right]$$

$\lambda = 1.5664$

iii) $$K = \lambda\left[\frac{EI}{E_m R^3}\right]^{0.25}$$

$K = 0.2777$

iv) $$\rho = \left[1000\frac{(H + H')}{R_c}\right]^{0.5} \leq 1.0$$

$\rho = 1.0$

v) $$R_e = \frac{r}{K}\left[\frac{6E\rho}{F_y}\right]^{0.5}$$

for 4.0 mm plate thickness, r = 17.38 mm

R_e = 4520 mm

vi) F_m = 1.0 for single conduit

vii) $R < R_e$

$$f_b = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\rho}\right]$$

f_b = *138.6 MPa*

As $\sigma < f_b$, 4.0 mm plate thickness satisfies compressive stress criteria where R = 3175 mm in the upper zone.

viii) Check Lower Zone Arcs for Wall Strength In Compression

$\lambda = 1.22$, $E_m = E_s = 12$ MPa, $\rho = 1.0$

R_1 = 3175 mm (top) R_2 = 840 mm (corner), R_3 = 9625 mm (bottom)

	R_1 (Top)	R_2 (Corner)	R_3 (Bottom)
K	0.2025	0.5489	0.0881
R_e	6198 mm	2286 mm	14239 mm
f_b	$R_1<R_e$ $f_b = 159.9\ MPa$	$R_2<R_e$ $f_b = 171.6\ MPa$	$R_3<R_e$ $f_b = 142.0\ MPa$

Where:

$$K = \lambda\left[\frac{EI}{E_m R^3}\right]^{.25}$$

$$R_e = \frac{r}{K}\left[\frac{6E\rho}{F_y}\right]^{0.5} mm$$

$$f_b = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\rho}\right] MPa \text{ when } R \le R_e$$

As $\sigma(68.2) < f_b$, 4.0 mm plate thickness satisfies compressive stress criteria for all radii within the lower zone.

8. Strength Requirements During Construction

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \le 1$$

Where:

$P = T_D + T_C$ and for $H_c < H_{min}$ $P = 0$

$P_{pf} = \phi_{hc} A F_y$

$M = M_1 + M_B + M_C$

$M_{Pf} = \phi_{hc} M_p$

Where:

$M_1 = k_{M1} R_B \gamma D_h^3$

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$

$M_C = k_{M3} R_L D_h L_C$

Where:

$k_{M1} = 0.0046 - 0.0010 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0009$ for $N_F > 5000$

$k_{M2} = 0.018 - 0.004 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0032$ for $N_F > 5000$

$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$

$= 0.030$ for $N_F > 100\,000$

$R_B = 0.67 + 0.87\left[(D_v/2D_h) - 0.2\right]$ *for* $0.2 \le D_v/2D_h \le 0.35$

$= 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right]$ *for* $0.35 < D_v/2D_h \le 0.50$

$= D_v/D_h$ *for* $D_v/2D_h > 0.5$

$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$

$$L_c = \frac{A_C}{k_4}$$

$$\left|\frac{M}{M_{Pf}}\right| = \textit{absolute value of the ratio } \frac{M}{M_{Pf}}$$

$$N_F = E_s(1000D_h)^3 / EI$$

Select $H_c < H_{min}$, $H_c = 0.6$ m, therefore $P=0$ as $H_c < H_{min}$

b) Assume Construction Axle (A_c) = 250 kN, 4 wheels per axle
$M = M_1 + M_B + M_C$

Calculate Flexibility Number N_F

$$N_F = E_s(1000D_h)^3 / EI$$

$E_s = 12$, $D_h = 6.297$, $I = 1457.6$ mm^4/mm
$N_f = 10278$

$k_{M1} = 0.0009$ (for $N_f > 5000$)
$k_{M2} = 0.0032$ (for $N_f > 5000$)
$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \leq 100\,000$
$= 0.0478$

$D_v/2D_h = 0.4488$
$R_B = 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right]$ *for* $0.35 < D_v/2D_h \leq 0.50$
$R_B = 0.9314$

$M_1 = k_{M1} R_B \gamma D_h^3$
$M_1 = 4.60$ kN-m/m

$M_B = -k_{M2} R_B \lambda D_h^2 H_c$
$M_B = -1.56$ kN-m/m

$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \leq 1.0$$

$R_L = 0.3054$

$L_c = \frac{A_C}{k_4}$ where $k_4 = 2.0$m, (Table 6.15, 4 wheels per axle & $H_c = 0.6$m)

$L_c = 250/2 = 125.0$ kN/m
$M_C = k_{M3} R_L D_h L_C$
$M_C = 11.49$ kN-m/m

$M = M_1 + M_B + M_C$
$M = 4.60 + (-1.56) + 11.49 = 14.53$ kN-m/m

$M_{Pf} = \phi_{hc} M_p$
$\phi_{hc} = 0.9, M_p = 16.98\ kN - m/m$
$M_{Pf} = 15.28$ kN-m/m

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \leq 1,\ P = 0$$

$$\left|\frac{M}{M_{pf}}\right| = \left|\frac{14.53}{15.28}\right| = 0.95 < 1.0$$

Therefore 4.0 mm plate thickness satisfies wall strength requirements during construction for 0.6 m of cover. It is recommended to check other applicable construction axle loads and cover heights in the same manner.

9.0 Factored Longitudinal Seam Strength *(S_f)*

$T_f < S_f$

Where $S_f = \phi_j S_s$ and: $\phi_j = 0.70, S_s = 1120$ kN/m for 4.0 mm plate thickness

S_f = 784 kN/m

$T_f = 329.3 < S_f$ OK

10.0 Plate thickness difference – Not Applicable

11.0 Radius of Curvature

$\frac{R_{crown}}{R} \leq 5.0$, where: $R_{crown} = 3175$ mm, $R_{corner} = 840$ mm

$\frac{3175}{840} = 3.8$ OK

Example 3

Given: Metal Box Span 6165 mm Rise 1900 mm (inside crest)
Height of cover, H = 0.9 m (900 mm) to neutral axis of the structural plate
Live Load, LL = CL 625
Soil Group I, 90% 95% Standard Proctor Density
Unit weight of soil $\gamma = 22$ kN/m^3

Find: Crown and haunch plate thickness, reinforcing requirements.

Solution: Corrugations, 381 x 140 mm, $D_h = 6.305$ m, R = 1.970 m (neutral axis)

1. Dimension Requirements

Rise = 1.97 m	> 0.8 m	∴ OK
	< 3.2 m	∴ OK
Span = 6.305 m	> 2.7 m	∴ OK
	< 8.0 m	∴ OK

2. Minimum Cover

$H_{MIN} = 900 - 70 = 830$ mm > 300 mm ∴ OK

3. Dead Load Bending Moments

$$M_D = k_1\gamma D_h^3 + k_2\gamma\left[H - \left[0.3 + \frac{d_c}{2000}\right]\right]D_h^2$$

k_1 = 0.0053 - 0.00024 (3.28 D_h - 12)
= 0.0053 - 0.00024 [3.28 (6.3) - 12] = 0.00322

$k_2 = 0.053$

$$M_D = (.00322)(22.0)(6.3)^3 + (0.053)(22.0)\left[0.9 - \left[0.3 + \frac{140}{2000}\right]\right](6.3)^2$$

$$= 17.72 + 24.53 = 42.25 \text{ KN-m/m}$$

$M_{cD} = \kappa M_D$ $\qquad \kappa = 0.70 - 0.0328\ D_h = 0.70 - .0328\ (6.3) = 0.493$

$M_{hD} = (1 - \kappa)\ MD$

$M_{cD} = (0.493)\ (42.25) = 20.83$ kN-m/m

$M_{hD} = (1 - 0.493)\ (42.25) = 21.42$ kN-m/m

4. Live Load Bending Moments

$$M_L = C_1 k_3 L_L D_h$$

$$C_1 = 0.5 + \frac{D_h}{15.24} = 0.5 + \frac{6.3}{15.24} = 0.913 < 1.0 \text{ (multiple axle)}$$

$$k_3 = [0.08 - 0.002(3.28D_h - 20)] / \left(\frac{H}{D_h}\right)^{0.2} \text{ for } 6\text{m} < D_h < 8\text{m}$$

$$k_3 = [0.08 - 0.002(3.28(6.3) - 20)] / \left(\frac{0.9}{6.3}\right)^{0.2} = 0.1161$$

$L_L = A_L / k_4$ $\qquad A_L = 250$ kN, 4 wheels/axle, $k_4 = 2.7$

$= 250/2.7 = 92.6$ kN/m

$M_L = (0.913)\ (0.116)\ (92.6)\ (6.3) = 61.78$ kN-m/m

$M_{cL} = \kappa M_L = (0.493)\ (61.78) = 30.45$ kN-m/m

$M_{hL} = (1 - \kappa)\ k_R\ M_L$ $\qquad k_R = 0.425H + 0.48$

$= (1 - .493)\ (0.862)\ (61.78)$ $\qquad = 0\ .425\ (0.9) + .48 = 0.862 < 1.0$

$= 26.99$ kN-m/m

5. Earthquake Bending Moments - Not applicable

6. Factored Crown and Haunch Bending Moments

$M_{cf} = \alpha_D\ M_{cD} + \alpha_L\ M_{cL}\ (1 + DLA)$

$M_{hf} = \alpha_D\ M_{hD} + \alpha_L\ M_{hL}\ (1 + DLA)$

$\alpha_D = 1.25 \qquad \alpha_L = 1.70$

$DLA = 0.30\ (1.0 - 0.5\ (H)) = 0.30\ (1.0 - 0.5\ (0.9)) = 0.165 > 0.1$ OK

$Mcf = 1.25\ (20.83) + 1.70\ (30.45)\ (1 + 0.165) = 86.34$ kNm/m

$M_{hf} = 1.25\ (21.42) + 1.70\ (26.99)\ (1 + 0.165) = 80.22$ kNm/m

7. Flexural Capacity at Ultimate Limit State

$M_{Pf} = \varphi_h\ M_P$ $\qquad \varphi_h = 0.9$

Try 381 x 140 x 7.01 mm plate

$M_pf = Z \times Fy$

$M_{Pf} = 0.90\ (437.85)\ (300 \text{ MPa})\ (10^{-3})$

$= 118.2 \text{ kNm/m} > M_{cf}$

∴ **OK** for crown. Reinforcement is not required.

M_{Pf} = 0.90 (437.85) (300 MPa) (10^{-3})
= 118.2 kNm/m > M_{hf}

∴ **OK** for haunch. Reinforcement is not required.

8. Fatigue Resistance

Longitudinal bolted seams shall not be located in the vicinity of the crown nor in areas of maximum live load moments at haunches.

9. Bending Moment across a Connection
Bending moments across connections shall be designed at the ultimate limit state for the larger of:

a) the calculated moment due to factored loads at the connections;
b) 75% of the factored resistance of the member, $\varphi_h M_p$.

M_{Pf} = $\varphi_h M_P$ $\varphi_h = 0.9$
= 0.75 (M_{Pf})
= 0.75 (118.2kNm/m)
= 88.65 kNm/m > M_{hf}, M_{cf}

∴ **OK** for haunch and crown. Reinforcement is not required.

Example 4

Given: Round CSP – 3600 mm inside diameter
Height of Cover, H =8.0 m
CL 625 Live Load
Soil Group 1, 90%-95% Standard Proctor Density
Unit Weight of Soil $\gamma = 22 kN/m^3$
Secant Modulus Es = 12 MPa
Project Location – Vancouver, B.C.

Required: Determine wall thickness for a 125 x 25 corrugation profile

Geometric Data: D_h = 3.625 m, D_v = 3.625 m (neutral axis)
Crown radius (Rc) = 1812 mm (N.A.)

1. Minimum Cover (H_{min}) is the largest of:

a) 0.60 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{0.5} = 0.60$ m

c) $0.4\left(\frac{D_h}{D_v}\right)^2 = 0.40\,m$

H_{min} = 0.60 m, $H_{min} < H$ (8.0 m)

2. Dead Load Thrust (T_D)

$T_D = 0.5\,(1.0 - 0.1\,C_S)\,A_f\,W$
A_f = 1.25 (Figure 6.13)

$$C_s = \frac{1000 E_S D_v}{EA}$$

Where: E = 200 000 MPa, E_s = 12 MPa
4.2 mm wall thickness, Area = 4.521 mm^2/mm
$C_s = 0.0481$

$W = \gamma\left[\left(H\ D_h\right) + Area\ Between\ \ Springline \,\&\, Crown\right]$
TopRise = 1.812 m, N.A. Inside End Area of structure above springline = 5.161m^2
$W = 22[8.0(3.625) + 1.41]$
$W = 669.0$ kN/m
$T_D = 0.5[1.0\text{-}0.1(.0481)]1.25(669.0)$
$\boldsymbol{T_D = 416.1}$ **kN/m**

3. **Live Load Thrust (T_L)**

$T_L = 0.5$ *(lesser of* D_h *and* l_t*)* $\sigma_L m_f$

Position as many axles of the CL-625 truck within the span length, (D_h = 3.625 m), at the road surface as would give the maximum total load. In this example, placing axles 2 and 3 above the structure produces the highest total load (P = 250 kN)

1 lane, (m_f =1.0)

$$\sigma_L = \frac{AxleLoad}{l_t w} \quad \text{where:}$$

l_t = a + 2(H) = 1.45 + 2(8) = 17.45 m
w = b + H = 2.4 + 8 = 10.4 m

$$\sigma_L = \frac{250}{17.45(10.4)} = 1.37\,kPa$$

$$\sigma_L m_f = 1.37\,kPa$$

2 lanes, (m_f =0.9)
l_t (17.45 m) > D_h (3.625 m)
w = b + H = 5.4 + 8 = 13.4 m

$$\sigma_L = \frac{2(250)}{17.45(13.4)} = 2.14\ kPa$$

$\sigma_L m_f = 1.92\ kPa$ Governs

T_L = 0.5 (lesser of D_h and l_t) $\sigma_L m_f$
= 0.5 (3.625) 1.92
$\boldsymbol{T_L = 3.48}$ **kN/m**

4. **Earthquake Thrust**
$T_E = T_D A_V$
$T_f = \alpha_D T_D + T_E$
$A_V = 2/3 A_H$
Zonal acceleration ratio (A) = 0.2 for Vancouver, $A_H = A$
$A_V = 0.133$
$T_D = 416.2$ kN/m
$T_E = (416.2)(0.133)$
$\boldsymbol{T_E = 55.5}$ **kN/m**

$T_f = 1.25(416.2) + 55.5$
$\boldsymbol{T_f = 575.7}$ ***kN/m (ULS Combination 5)***

5. **Total Factored Thrust (T_f)**

$T_f = \alpha_D T_D + \alpha_L T_L (1 + DLA)$ *(ULS Combination 1)*
$\alpha_D = 1.25$

$\alpha_L = 1.70$

$DLA = 0.40(1-0.5D_e) \geq 0.10$

Where: $D_e = H = 8.0$ m

$DLA = 0.10$

$T_f = 1.25(416.2) + 1.70(3.48)(1.10)$

T_f = 526.8 kN/m (ULS Combination 1)

Therefore ULS Combination 5 governs

T_f = 575.7 kN/m

6. Compressive Stress at ULS *(σ)*

$$\sigma = \frac{T_f}{Area}$$ where Area = 4.521 mm²/mm for 4.2 mm wall thickness

σ = 127.3 MPa

7.0 Wall Strength in Compression *(f_b)*

Definition of upper zone

i) $\theta_0 = 1.6 + 0.2\log\left[\frac{EI}{E_m R^3}\right]$ radians where:

$$E_m = E_S\left[1 - \left[\frac{R_C}{R_C + 1000[H + H']}\right]^2\right]$$

E_s = 12.0 MPa, R = R_c = 1812 mm, H = 8.0 m, H' = $\frac{D_v}{4}$ = 0.906 m

$I_{4.2mm} = 394.84 \frac{mm^4}{mm}$ then

E_m = 11.65 MPa, $\theta_0 = 57.9^0$

ii) $\lambda = 1.22\left[1.0 + 1.6\left[\frac{EI}{E_m R_c^3}\right]^{0.25}\right]$

$\lambda = 1.5786$

iii) $K = \lambda\left[\frac{EI}{E_m R^3}\right]^{0.25}$

$K = 0.2899$

iv) $\rho = \left[1000\frac{(H + H')}{R_c}\right]^{0.5} \leq 1.0$

$\rho = 1.0$

v) $R_e = \frac{r}{K}\left[\frac{6E\rho}{F_y}\right]^{0.5}$

for 4.2 mm wall thickness, r = 9.345 mm

R_e = 2328 mm

vi) F_m = 1.0 for single conduit

vii) $R < R_e$

$$f_b = \phi_t F_m \left[F_y - \frac{(F_y KR)^2}{12Er^2\rho} \right] \text{ where: } \phi_t = 0.8$$

$f_b = 128.2\ MPa$

As $\sigma < f_b$, 4.2 mm wall thickness satisfies compressive stress criteria where R = 1812 mm in the upper zone.

viii) Check Lower Zone Arcs for Wall Strength In Compression

$$\lambda = 1.22,\ E_m = E_s = 12\ \text{MPa},\ \rho = 1.0$$

R_1 = 1812 mm

	R_1 (Top)
K	0.2224
R_e	3035 mm
f_b	$R_1 < R_e$ $f_b = 151.1\ MPa$

Where:

$$K = \lambda \left[\frac{EI}{E_m R^3} \right]^{.25}$$

$$R_e = \frac{r}{K} \left[\frac{6E\rho}{F_y} \right]^{0.5} mm$$

$$f_b = \phi_t F_m \left[F_y - \frac{(F_y KR)^2}{12Er^2\rho} \right] MPa \text{ when } R \le R_e$$

As $\sigma(127.3) < f_b$, 4.2 mm wall thickness satisfies compressive stress criteria within the lower zone.

8. Strength Requirements During Construction

$$\left[\frac{P}{P_{pf}} \right]^2 + \left| \frac{M}{M_{pf}} \right| \le 1$$

Where:

$P = T_D + T_C$ and for $H_c < H_{min}$ $P = 0$

$P_{pf} = \phi_{hc} A F_y$

$M = M_1 + M_B + M_C$

$M_{Pf} = \phi_{hc} M_p$

Where:

$M_1 = k_{M1} R_B \gamma D_h^3$

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$

$M_C = k_{M3} R_L D_h L_C$

Where:

$k_{M1} = 0.0046 - 0.0010 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0009$ for $N_F > 5000$

$k_{M2} = 0.018 - 0.004 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0032$ for $N_F > 5000$

$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$

$= 0.030$ for $N_F > 100\,000$

$$R_B = 0.67 + 0.87\left[(D_v/2D_h) - 0.2\right] for\ 0.2 \le D_v/2D_h \le 0.35$$
$$= 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right] for\ 0.35 < D_v/2D_h \le 0.50$$
$$= D_v/D_h\ for\ D_v/2D_h > 0.5$$
$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$
$$L_c = \frac{A_C}{k_4}$$
$$\left|\frac{M}{M_{Pf}}\right| = absolute\ value\ of\ the\ ratio\ \frac{M}{M_{Pf}}$$
$$N_F = E_s(1000D_h)^3/EI$$

Select $H_c < H_{min}$, $H_c = 0.6$ m, therefore P=0 as $H_c < H_{min}$

c) Assume Construction Axle (A_c) = 250 kN, 4 wheels per axle
$M = M_l + M_B + M_C$

Calculate Flexibility Number N_F
$$N_F = E_s(1000D_h)^3/EI$$

$E_s = 12$, $D_h = 3.625$, $I = 394.84$ mm^4/mm
$N_f = 7238$

$k_{M1} = 0.0009$ (for $N_f > 5000$)
$k_{M2} = 0.0032$ (for $N_f > 5000$)
$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$
$= 0.0505$

$D_v/2D_h = 0.5$
$$R_B = 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right] for\ 0.35 < D_v/2D_h \le 0.50$$
$R_B = 0.9995$

$M_l = k_{M1} R_B \gamma D_h^3$
$M_l = 0.94$ kN-m/m

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$
$M_B = -0.55$ kN-m/m

$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$
$R_L = 0.2329$

$L_c = \frac{A_C}{k_4}$ where $k_4 = 2.0$m, (Table 6.15, 4 wheels per axle & $H_c = 0.6$m)
$L_c = 250/2 = 125.0$ kN/m
$M_C = k_{M3} R_L D_h L_C$
$M_C = 5.33$ kN-m/m

$M = M_l + M_B + M_C$
$M = 0.94 + (-0.55) + 5.33 = 5.72$ kN-m/m

$$M_{Pf} = \phi_{hc} \, M_p$$
$$\phi_{hc} = 0.9, M_p = 8.78 \, kN - m/m$$
M_{Pf} = 7.90 kN-m/m
$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \leq 1, \text{ P } = 0$$
$$\left|\frac{M}{M_{pf}}\right| = \left|\frac{5.72}{7.90}\right| = 0.72 < 1.0$$

Therefore 4.2 mm wall thickness satisfies wall strength requirements during construction for 0.6m of cover. It is recommended to check other applicable construction axle loads and cover heights in the same manner.

9.0 Factored Longitudinal Seam Strength *(S_f)* - Not applicable to CSP

10.0 Plate thickness difference – Not Applicable to CSP

11.0 Radius of Curvature – Not Applicable to CSP

Example 5

Given: Arch Span 6100 mm Rise 3050 mm (inside radius = 3050 mm)
Height of cover, H = 1.10 m
Live Load, LL = CL - 625
Soil - Group I, 90% - 95% Standard Proctor Density
Unit weight of soil γ = 22 kN/m^3
Secant Modulus Es = 12 MPa
Find: Wall Thickness, try 4 mm.
Solution: Corrugations, 152 x 51 mm, Dh = Dv = 6.151 m, R_c = 3076 (neutral axis)/2.

1. Minimum Cover is the largest of:

a) 0.6 m

b) $\dfrac{D_h}{6}\left(\dfrac{D_h}{D_v}\right)^{1/2} = \dfrac{6.15}{6}\left(\dfrac{6.15}{6.15}\right)^{1/2} = 1.025m$

c) $0.4\left(\dfrac{D_h}{D_v}\right)^2 = 0.4\left(\dfrac{6.15}{6.15}\right)^2 = 0.4m$

$\therefore$ Height of cover = 1.10 m > minimum height of cover = 1.03 m, **OK**

2. Dead Load Thrust

T_D = 0.5 (1.0 - 0.1 C_s) A_f W

A_f = 1.276 (From Figure 6.13)

$$C_s = \frac{1000 E_s D_v}{EA} \quad \text{where } D_v = 6.15 \text{ m}$$
E = 200 x10^3 MPa
E_s = 12 MPa

Try 152 x 51 x 4.0 mm thick plate, A = 4.828 mm^2/mm

$$C_s = \frac{1000E_s D_v}{EA} = \frac{1000(12)(6.15)}{(200 \times 10^3)(4.828)} = 0.076$$

W = γ[(H D_h) + Area above springline & below crown]

W = 22 [(1.1)(6.15) + 4.015] = 237.2 kN/m

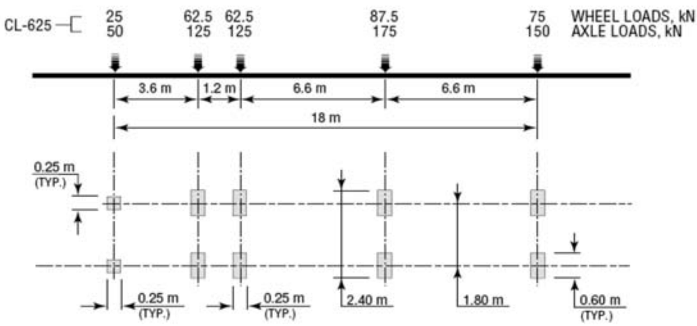
T_D = 0.5 [1.0 - 0.1(.071)] (1.276) (237.2) = 150.2 kN/m

3. Live Load Thrust

T_L = 0.5 (lesser of D_h and l_t) σ_L m_f

D_h = 6.15 m

Position as many axles of the CL-625 truck as would give the maximum total load. For CL - 625, try axles and lane combinations to find highest pressure. In this example, placing axles 1 through 3 above the structure produces the highest total load (P=300kN).



<u>1 Lane</u> m_f = 1.0

l_t = a + 2(H) = 5.05 + 2(1.1) = 7.25 m > Dh

w = b + H = 1.8 +0.6 + 1.1 = 3.5 m

$$\sigma_L = \frac{AL}{l_t \times w} = \frac{300}{7.25 \times 3.5} = 11.82\,\text{kN/m}^2$$

σ_L m_f = (11.82)(1.0) = 11.82 kN/m^2 <u>Governs</u>

<u>2 Lane</u> m_f = 0.9

l_t = 5.05 + 2(H) = 5.05 + 2(1.1) = 7.25 m > Dh

w = 5.4 + H = 5.4 + 1.1 = 6.5 m

$$\sigma_L = \frac{2AL}{l_t \times w} = \frac{2(300)}{7.25 \times 6.5} = 12.73\,\text{kN/m}^2$$

σ_L mf = (12.73)(0.9) = 11.46 kN/m^2

T_L = 0.5 (6.15) (11.82) (1.0) = 36.35 kN/m

4. Earthquake Thrust - Not applicable

5. Total Thrust

$T_f = \alpha_D T_D + \alpha_L T_L (1 + DLA)$

$\alpha_D = 1.25$
$\alpha_L = 1.70$
$H = D_E = 1.10$ m $\therefore DLA = 0.4(1.0 - 0.5 D_E), \geq 0.10$
$= 0.4(1.0 - 0.5 (1.1)) = 0.18$

$T_f = 1.25 (150.2) + 1.70 (36.35) (1 + 0.18) = 260.7$ kN/m

6. Compressive Stress at the ULS

$\sigma = T_f/A = 260.7/4.828 = 53.99$MPa

7. Wall Strength in Compression

Upper portion:

$$\text{i) } E_m = E_s\left[1-\left(\frac{R_c}{R_c+1000(H+H')}\right)^2\right]$$

H' = Min (D_v /4 and Rise / 2) = Min (6.15/4 and 3.05/2) = 1.525

$$E_m = E_s\left[1-\left(\frac{R_c}{R_c+1000(H+H')}\right)^2\right] = 12.0\left[1-\left(\frac{3076}{3076+1000(1.100+1.525)}\right)^2\right] = 8.51$$

$$\text{ii) } \lambda = 1.22\left[1.0+1.6\left(\frac{EI}{E_m R_c^{\,3}}\right)^{1/4}\right] = 1.22\left[1.0+1.6\left(\frac{(200\times10^3)\times(1457.56)}{(8.51)(3076)^3}\right)^{1/4}\right] = 1.58$$

$$\text{iii) } K = \lambda\left[\frac{EI}{E_m R^3}\right]^{1/4} = 1.58\left[\frac{(200\times10^3)(1457.56)}{(8.51)(3076)^3}\right]^{1/4} = 0.29$$

$$\text{iv) } \rho = \left(1000\frac{(H+H')}{R_c}\right)^{1/2} = \left(1000\frac{(1.100+1.525)}{3076}\right)^{1/2} = 0.924 \leq 1.0$$

$$\text{v) } R_e = \frac{r}{K}\left(\frac{6E\tilde{n}}{F_y}\right)^{1/2} = \frac{17.38}{0.29}\left(\frac{6(200\times10^3)(0.924)}{230}\right)^{1/2} = 4117 > R = 3076$$

vi) $F_m = 1.0$ for single conduit

$R < R_e$

$$f_b = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\tilde{n}}\right] = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\tilde{n}}\right]$$

$$= 0.8(1.0)\left[230 - \frac{\{(230)(0.29)(3076)\}^2}{12(200\times10^3)(17.38)^2(0.924)}\right] = 132.7 \text{ MPa} > \sigma = 53.99 \text{ MPa}$$

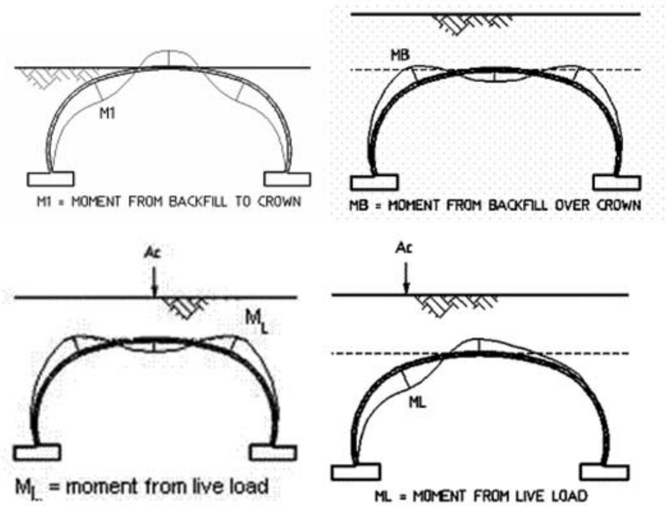
$\therefore$4 mm plate thickness satisfies the strength requirements in compression.

8. Strength Requirements during Construction

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| \le 1.0$$

$P = T_D + T_C$
$P_{Pf} = \phi_{hc} A F_y$
$P_{pf} = \phi_{hc}$ A Fy = (0.90) (4.828 mm2/mm) (230 MPa) = 999.4 kN/m
$M_{pf} = \phi_{hc}\ M_p = \phi_{hc}\ Z\ F_y$
$M_{pf} = 0.90\ (73.826)\ (230)\ (10^{-3}) = 15.3$ kN-m/m

$M = M_l + M_B + M_C$



$M_l = k_{M1} R_B \gamma D_h^3$

$$N_f = \frac{E_s(1000D_h)^3}{EI} = \frac{12[(1000)(6.15)]^3}{(200 \times 10^3 \times 1457.56)} = 9579.9$$

$k_{M1} = 0.0009$; $(N_f > 5000)$
$k_{M2} = 0.0032$; $(N_f > 5000)$
$k_{M3} = 0.120 - 0.018 \log 10\ (9579.9) = 0.048$; $(N_f < 100{,}000)$
$R_B = 0.80 + 1.33\ [D_v/2D_h - 0.35]$ for $0.35 < D_v/2D_h = 0.50$ 0.50
$R_B = 0.80 + 1.33\ [0.50 - 0.35] = 1.0$

$M_l = 0.0009\ (1.0)\ (22.0)\ (6.15)^3 = 4.61$ kN-m/m

$M_B = -\ k_{M2}\ R_B\ \gamma\ D_h^2\ H_c = -\ 0.0032\ (1.0)\ (22.0)\ (6.15)^2\ (1.1) = -2.93$ kN-m/m

$Mc = k_{M3}\ R_L\ D_h\ L_c$
$R_L = [0.265 - 0.053 \log 10\ (N_f)]/(H_c/D_h)^{0.75} = 0.196 < 1.0$
$L_c = A_c/k_4$
k_4 (Table6.15, 4 wheels/axle & H = 1.10 m); $k_4 = 3.07$
$L_c = 250/3.07 = 81.5$ kN/m

$M_c = 0.048\ (0.196)\ (6.15)\ (81.5) = 4.76$ kN-m/m
$M = M_l + M_B + M_c$
$M = 4.61 + (-2.93) + 4.76 = 6.44$ kN-m/m
Recalculate ratio using P, and other new value.

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| = \left(\frac{186.6}{999.4}\right)^2 + \left|\frac{6.44}{15.3}\right| = 0.456 \leq 1.0$$

∴4 plate thickness satisfies the strength requirements during construction, **OK**

Check any other applicable construction axle loads and cover heights in same manner.

9. Connection Strength

$S_r = \phi_j S_s = 0.70(1120$ kN/m$) = 784$ kN/m

$T_f = 260.7$ kN/m $< S_r$ ∴ OK

10. Difference in Plate Thickness – All plates are the same thickness. Not applicable

11. Radius of Curvature – Not applicable, single radius

__**Example 6**__

Given: Arch Span 11000 mm x Rise 6385 mm (inside crest)
Height of cover, H = 2.5 m
Live Load, LL = CL 625
Unit weight of soil $\gamma = 22$ kN/m^3
Soil Group I, 90% ~ 95% Standard Proctor Density
Secant Modulus Es = 12 MPa
Find: Wall Thickness. Try 8 gauge (4.18 mm) plates.
Solution: Corrugations, 381 x 140 mm, $D_h = 11.14$ m, $D_v = 9.8$ m, $R_c = 6700$ (neutral axis), $R_{side} = 4700$ (neutral axis)

1. Minimum Cover:

Smaller of 1.5 m and the largest of

a) 0.6 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{1/2} = \frac{11.14}{6}\left(\frac{11.14}{9.8}\right)^{1/2} = 1.98$m or

c) $0.4\left(\frac{D_h}{D_v}\right)^2 = 0.4\left(\frac{11.14}{9.8}\right)^2 = 0.52$m

∴ Height of cover = 2.5 m > minimum height of cover = 1.5 m, **OK**

2. Dead Load Thrust

$T_D = 0.5\ (1.0 - 0.1\ C_s)\ A_f\ W$

$A_f = 1.218$ (From Figure 6.13)

$C_s = \dfrac{1000E_sD_v}{EA}$ where D_v = 9.8 m
E = 200 x 10^3 MPa
E_s = 12 MPa

Try 381 x 140 x 4.18 mm thick plate, A = 5.846 mm^2/mm

$$C_s = \frac{1000E_sD_v}{EA} = \frac{1000(12)(9.8)}{(200 \times 10^3)(5.846)} = 0.101$$

W = γ[(H D_h) + Area above springline & below crown]

W = 22 [(2.5)(11.14) + 9.648] = 824.96 kN/m

T_D = 0.5 [1.0 - 0.1(.101)] (1.218) (824.96) = 497.33 kN/m

3. Live Load Thrust

T_L = 0.5 (lesser of D_h and l_t) σ_L m_f

D_h = 11.14 m

For CL - 625 LL, AL = 0.4 (625) = 250 kN Tandem

Position as many axles of the CL-625 truck as would give the maximum total load. For CL - 625, try axles and lane combinations to find highest pressure. In this example, placing axles 4 and 5 above the structure produces the highest total load (P=325kN).

1 Lane m_f = 1.0
l_t = 6.85 + 2(H) = 6.85 + 2(2.5) = 11.85 m > Dh
w = 2.4 + H = 2.4 + 2.5 = 4.9 m

$$\sigma_L = \frac{AL}{l_t \times w} = \frac{325}{11.85 \times 4.9} = 5.60\,\text{kN/m}^2$$

σ_L m_f = (5.60)(1.0) = 5.60 kN/m^2

2 Lane m_f = 0.9
l_t = 6.85 + 2(H) 6.85 + 2(2.5) = 11.85 m Dh
w = 5.4 + H = 5.4 + 2.5 = 7.9 m

$$\sigma_L = \frac{2AL}{l_t \times w} = \frac{2(325)}{11.85 \times 7.9} = 6.94\,\text{kN/m}^2$$

σ_L m_f = (6.94)(0.9) = 6.24 kN/m^2 Governs

T_L = 0.5 (11.14) (6.24) = 34.7kN/m

4. Earthquake Thrust - Not applicable

5. Total Thrust

$T_f = \alpha_D T_D + \alpha_L T_L (1 + DLA)$

α_D = 1.25
α_L = 1.70

DLA = maximum $\left(0.1,\ 0.4\left(1 - \frac{H}{2}\right)\right.$ $\therefore$ DLA = 0.1

$T_f = 1.25\ (497.33) + 1.70\ (34.7)\ (1 + 0.1) = 686.6\text{kN/m}$

6. Compressive Stress at the ULS

$\sigma = T_f/A = 686.6/5.846 = 117.4\text{MPa}$

7. Wall Strength in Compression

Upper portion with the largest radius is the worst case scenario. Check upper portion only:

i) $E_m = E_s\left[1-\left(\dfrac{R_c}{R_c + 1000(H+H')}\right)^2\right]$

H' = Min (D_v /4 and Rise / 2) = Min (9.80/4 and 6.385/2) = 2.45

$$E_m = E_s\left[1-\left(\frac{R_c}{R_c + 1000(H+H')}\right)^2\right] = 12.0\left[1-\left(\frac{6700}{6700+1000(2.5+2.45)}\right)^2\right] = 8.03$$

$$\text{ii) } \lambda = 1.22\left[1.0+1.6\left(\frac{EI}{E_m R_c^{\ 3}}\right)^{1/4}\right] = 1.22\left[1.0+1.6\left(\frac{(200\times10^3)\times(14333.9)}{(8.03)(6700)^3}\right)^{1/4}\right] = 1.58$$

$$\text{iii) } K = \lambda\left[\frac{EI}{E_m R^3}\right]^{1/4} = 1.58\left[\frac{(200\times10^3)(14333.9)}{(8.03)(6700)^3}\right]^{1/4} = 0.29$$

$$\text{iv) } \rho = \left(1000\frac{(H+H')}{R_c}\right)^{1/2} = \left(1000\frac{(2.5+2.45)}{6700}\right)^{1/2} = 0.86 \le 1.0$$

$$\text{v) } R_e = \frac{r}{K}\left(\frac{6E\tilde{n}}{F_y}\right)^{1/2} = \frac{49.52}{0.29}\left(\frac{6(200\times10^3)(0.86)}{300}\right)^{1/2} = 9887 > R = 6700$$

vi) $F_m = 1.0$ for single conduit

$R < R_e$

$$f_b = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\tilde{n}}\right] = \phi_t F_m\left[F_y - \frac{(F_y KR)^2}{12Er^2\tilde{n}}\right]$$

$$= 0.8(1.0)\left[300 - \frac{\{(300)(0.29)(6700)\}^2}{12(200\times10^3)(49.52)^2(0.86)}\right] = 184.9\,\text{MPa} > \sigma = 117.4\text{MPa}$$

$\therefore$4.18 plate thickness satisfies the strength requirements in compression, OK

8. Strength Requirements during Construction

Check a CAT D4 Dozer applied to 1.0 m of cover.

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| \le 1.0$$

$P = T_D + T_C$

$P_{Pf} = \phi_{hc} A F_y$

$M = M_I + M_B + M_C$

$M_{pf} = \phi_{hc} M_p$
For $H_c/D_h < 0.2$, $P = 0.0$
Check $H_c = 1.00$ m & $A_c = 38$ kN (construction axle load)
$H_c/D_h = 1.0/11.14 = 0.09 < 0.2$
$P = 0.0$ kN/m
$P_{pf} = \phi_{hc}\, A\, Fy = (0.90)\ (5.846\ mm^2/mm)\ (300\ MPa) = 1578.4$ kN/m
$M = M_I + M_B + M_C$

$$M_1 = k_{M1} R_B \gamma D_h^3$$

$$N_f = \frac{E_s (1000 D_h)^3}{EI} = \frac{12[(1000)(11.14)]^3}{(200 \times 10^3 \times 14333.9)} = 5787$$

$k_{M1} = 0.0009$; ($N_f > 5000$)
$k_{M2} = 0.0032$; ($N_f > 5000$)
$k_{M3} = 0.120 - 0.018 \log10\ (5787) = 0.0523$; ($N_f < 100{,}000$)
$R_B = 0.80 + 1.33\ [D_v/2D_h - 0.35]$ for $0.35 < D_v/2D_h = 0.44\ \ 0.5$
$R_B = 0.80 + 1.33\ [0.44 - 0.35] = 0.92$

$M_1 = 0.0009\ (0.92)\ (22.0)\ (11.14)^3 = 25.17$ kN-m/m

$M_B = - k_{M2}\, R_B\, \gamma\, D_h^2\, H_c = - 0.0032\ (0.92)\ (22.0)\ (11.14)^2\ (1.0) = -8.03$ kN-m/m

$Mc = k_{M3}\, R_L\, D_h\, L_c$
$R_L = [0.265 - 0.053 \log10\ (N_f)]/(H_c/D_h)^{0.75} = 0.40 < 1.0$
$L_c = A_c/k_4$
k_4 (Table6.15, 2 wheels/axle & H = 1.0 m); $k_4 = 2.367$
$L_c = 38/2.367 = 16$ kN/m
$M_c = 0.0523\ (0.4)\ (11.14)\ (16) = 3.7$ kN-m/m
$M = M_1 + M_B + M_c$
$M = 25.17 + (-8.03) + 3.7 = 20.9$ kN-m/m
$M_{pf} = \phi_{hc}\, M_p = \phi_{hc}\, Z\, F_y$
$M_{pf} = 0.90\ (260.15)\ (300)\ (10^{-3}) = 70.2$ kN-m/m

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| = \left(\frac{0}{1578.4}\right)^2 + \left|\frac{20.9}{70.2}\right| = 0.30 \quad \leq 1.0$$

$\therefore$ 4.18 mm plate thickness satisfies the strength requirements during construction, OK

Check any other applicable construction axle loads and cover heights in same manner.

9. Strength Requirements for Completed Structure

$$\left(\frac{T_f}{P_{Pf}}\right)^2 + \left|\frac{M_f}{M_{Pf}}\right| \leq 1.0$$

$T_f = \alpha_D\, T_D + \alpha_L\, T_L\ (1 + DLA)$
$P_{Pf} = \phi_h A F_y$

$$M_f = |\alpha_D M_1 + \alpha_D M_D| + \alpha_L M_L (1 + DLA)$$

$M_{pf} = \phi_h M_p$

$T_f = 1.25\ (497.3) + 1.70\ (334.7)\ (1 + 0.1) = 686.6 kN/m$

$P_{pf} = \phi_h\ A\ Fy = (0.85)\ (5.846\ mm^2/mm)\ (300\ MPa) = 1490.7\ kN/m$

$$M_1 = k_{M1} R_B \gamma D_h^3$$

$$N_f = \frac{E_s(1000D_h)^3}{EI} = \frac{12[(1000)(11.14)]^3}{(200 \times 10^3 \times 14333.9)} = 5787$$

$k_{M1} = 0.0009;\ (N_f > 5000)$

$R_B = 0.80 + 1.33\ [D_v/2D_h - 0.35]$ for $0.35 < D_v/2D_h = 0.44\ \ 0.5$

$R_B = 0.80 + 1.33\ [0.44 - 0.35] = 0.92$

$M_1 = 0.0009\ (0.92)\ (22.0)\ (11.14)^3 = 25.17\ kN\text{-}m/m$

$$M_D = -k_{M2} R_B \gamma D_h^2 H$$

$k_{M2} = 0.0032;\ (N_f > 5000)$

$M_D = -.0032(0.92)(22.0)(11.14)^2(2.5) = -20.09\ kN\text{-}m/m$

$$M_L = \frac{k_{M3} R_U D_h A_L}{k_4}$$

$k_{M3} = 0.120 - 0.018\ \log10\ (5787) = 0.0523;\ (N_f < 100{,}000)$

$$R_U = \frac{0.265 - 0.053 \mathrm{Log}_{10} N_F}{(H/D_h)^{0.75}} = \frac{0.265 - 0.053 Log_{10}(5787)}{(2.5/11.14)^{0.75}} = 0.201 \leq 1.0$$

k_4 (Table 6.18, 4 wheels/axle & H = 2.5 m); $k_4 = 4.622$

$$M_L = \frac{k_{M3} R_U D_h A_L}{k_4} = \frac{.0523(.201)(11.14)(250)}{4.622} = 6.33\ kN\text{-}m/m$$

$$M_f = |(1.25)(25.17) + (1.25)(-20.09)| + 1.7(6.33)(1 + 0.1) = 18.19\ kN\text{-}m/m$$

$M_{pf} = \phi_h\ M_p = \phi_h\ Z\ F_y$

$M_{pf} = 0.85\ (260.15)\ (300)\ (10^{-3}) = 66.33\ kN\text{-}m/m$

$$\left(\frac{P}{P_{Pf}}\right)^2 + \left|\frac{M}{M_{Pf}}\right| = \left(\frac{686.6}{1490.7}\right)^2 + \left|\frac{18.19}{66.33}\right| = 0.482 \quad \leq 1.0$$

$\therefore$4.18 mm plate thickness satisfies the strength requirements for completed structure, **OK**

10. Connection Strength

$S_r = \phi_j\, S_s = 0.7\ (1270\ kN/m) = 889\ kN/m$

$T_f = 686.6 kN/m < S_r\ \therefore$ OK

11. Difference in Plate Thickness: Not applicable since plates are the same thickness.

12. Radius of Curvature

$R \geq 0.2\ R_c$ $\qquad$ $R_c = 6700$ mm, $R_{side} = 4700$ mm

$R = 4700\ \geq 0.2\ (6700) = 1340$ mm $\therefore$ **OK**

Example 7

Given: Deep Corrugated Structural Plate Arch
13000 mm span x 6512 mm rise
Height of Cover, H =3.0 m
CL 625 Live Load
Soil Group 1, 90%-95% Standard Proctor Density
Unit Weight of Soil $\gamma = 22\ kN/m^3$
Secant Modulus Es = 12 MPa
Construction Vehicle Axle Load =250 kN (axle has 4 wheels)
Overall width of wheels = 2400 mm

Required: Determine wall thickness for a 400 x 150 corrugation

Geometric Data: D_h = 13.15 m, D_v = 13.15 m (neutral axis)
Crown radius (Rc) = 6575 mm (N.A.), $\theta_{top} = 180.2^0$

1. Minimum Cover (H_{min})is the largest of:

a) 0.6 m

b) $\frac{D_h}{6}\left(\frac{D_h}{D_v}\right)^{0.5} = 2.19$ m

c) $0.4\left(\frac{D_h}{D_v}\right)^2 = 0.4\ m$

For deep corrugated structures H_{min} shall be the smaller of 1.5m and the minimum depth of cover for structures with shallow corrugations but the same conduit size. Therefore H_{min} = 1.50 m governs. H_{min}<H (3. 0m)

2. Dead Load Thrust (T_D)

$T_D = 0.5\ (1.0 - 0.1\ C_S)\ A_f\ W$
A_f = 1.264 (Figure 6.13)
$C_s = \frac{1000 E_S D_v}{EA}$
Where: E = 200 000 MPa, E_s = 12 MPa, 5.0 mm Plate Area = 6.811 mm^2/mm
$C_s = 0.1158$

W= $\gamma\left[(HD_h) + Area\ Between\ Springline \& Crown\right]$
TopRise = 6.575 m
N.A. Inside End Area of structure above springline = 67.906 m^2
$W = 22[3.0(13.15)+18.56] = 1276.1\ kN/m$
$T_D = 0.5[1.0\text{-}0.1(0.1158)]1.264(1276.1)$
$\boldsymbol{T_D = 797.1\ kN/m}$

3. Live Load Thrust (T_L)

$T_L = 0.5$ *(lesser of* D_h *and* l_t*)* $\sigma_L m_f$

Position as many axles of the CL-625 truck within the span length, (D_h = 13.15 m), at the road surface as would give the maximum total load. In this example, placing axles 2, 3 and 4 above the structure produces the highest total load (P = 425 kN)

1 lane, (m_f =1.0)

$$\sigma_L = \frac{AxleLoad}{l_t w}$$

where: l_t = a + 2(H) = 1.2 + 6.6 + 0.25 + 2(3) = 14.05 m
w = b + H = 2.4 + 3.0 = 5.4 m

$$\sigma_L = \frac{425}{14.05(5.4)} = 5.6\ kPa$$

$$\sigma_L m_f = 5.6\ kPa$$

2 lanes, (m_f =0.9)
l_t (14.05 m) > D_h (13.15m) D_h governs
w = b+ H = 5.4 + 3.0 = 8.4 m

$$\sigma_L = \frac{2(425)}{14.05(8.4)} = 7.2\,kPa$$

$\sigma_L m_f = 6.48\ kPa$ Governs

T_L = 0.5 (lesser of D_h and l_t) $\sigma_L m_f$
= 0.5 (13.15) 6.48

$\boldsymbol{T_L = 42.6\ kN/m}$

4. Earthquake Thrust – Not Applicable

5. Total Factored Thrust *(T_f)*

$T_f = \alpha_D T_D + \alpha_L T_L\ (1+ DLA)$
α_D = 1.25
α_L = 1.70
$DLA = 0.40(1\text{-}0.5D_e) \geq 0.10$
Where: D_e = H = 3.0 m
$DLA = 0.10$
T_f = 1.25(797.1) + 1.70(42.6)(1.1)
$\boldsymbol{T_f = 1076.0\ kN/m}$

6. Compressive Stress at ULS *(σ)*

$$\sigma = \frac{T_f}{Area}$$ where Area = 6.811 mm^2/mm for 5.0 mm plate thickness

$\boldsymbol{\sigma = 158.0\ MPa}$

7.0 Wall Strength in Compression
Definition of upper zone

i) $\theta_0 = 1.6 + 0.2\log\left[\frac{EI}{E_m R^3}\right]$ radians where:

$$E_m = E_S\left[1 - \left[\frac{R_C}{R_C + 1000[H + H']}\right]^2\right]$$

E_s = 12.0 MPa, R = R_c = 6575mm, H = 3.0m, H' = $\frac{D_v}{4}$ = 3.287 m

$I_{5.0mm} = 19060.0\,\frac{mm^4}{mm}$ then

E_m = 8.864 MPa, $\theta_0 = 59.4^0$

ii) $$\lambda = 1.22\left[1.0 + 1.6\left[\frac{EI}{E_m R_c^3}\right]^{0.25}\right]$$

$\lambda = 1.605$

iii) $$K = \lambda \left[\frac{EI}{E_m R^3} \right]^{0.25}$$

$K = 0.3165$

iv) $$\rho = \left[1000 \frac{(H + H')}{R_c} \right]^{0.5} \leq 1.0$$

$\rho = 0.9779$

v) $$R_e = \frac{r}{K} \left[\frac{6E\rho}{F_y} \right]^{0.5}, F_y = 300\ MPa$$

for 5.0 mm plate thickness, r = 52.90 mm

$R_e = 10452$ mm

vi) $F_m = 1.0$ for single conduit

vii) $R < R_e$

$$f_b = \phi_t F_m \left[F_y - \frac{(F_y KR)^2}{12Er^2\rho} \right],\ \phi_t = 0.8,\ F_y = 300\ MPa$$

f_b = 192.5 MPa

As $\sigma < f_b$, 5.0 mm plate thickness satisfies compressive stress criteria where R =6575 mm in the upper zone.

viii) Check Lower Zone Arc for Wall Strength in Compression

$\lambda = 1.22,\ E_m = E_s = 12$ MPa, $\rho = 0.9779$

$R_2 = 6575$ mm

	R_2 (6575 mm)
K	0.2231
R_e	14832 mm
f_b	$R_2 < R_e$ $f_b = 216.4\ MPa$

Where:

$$K = \lambda \left[\frac{EI}{E_m R^3} \right]^{.25}$$

$$R_e = \frac{r}{K} \left[\frac{6E\rho}{F_y} \right]^{0.5} mm$$

$$f_b = \phi_t F_m \left[F_y - \frac{(F_y KR)^2}{12Er^2\rho} \right] MPa$$

when R $\leq R_e$

As $\sigma(158.0) < f_b(192.5)$, 5.0 mm plate thickness satisfies compressive stress criteria for the radius within the lower zone.

8.0 Strength Requirements During Construction

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \le 1$$

Where:

$P = T_D + T_C$ and for $H_c < H_{min}$ $P = 0$

$P_{pf} = \phi_{hc} A F_y$

$M = M_1 + M_B + M_C$

$M_{Pf} = \phi_{hc} M_p$

Where:

$M_1 = k_{M1} R_B \gamma D_h^3$

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$

$M_C = k_{M3} R_L D_h L_C$

Where:

$k_{M1} = 0.0046 - 0.0010 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0009$ for $N_F > 5000$

$k_{M2} = 0.018 - 0.004 \log_{10}(N_F)$ for $N_F \le 5000$

$= 0.0032$ for $N_F > 5000$

$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ for $N_F \le 100\,000$

$= 0.030$ for $N_F > 100\,000$

$R_B = 0.67 + 0.87\left[(D_v/2D_h) - 0.2\right]$ *for* $0.2 \le D_v/2D_h \le 0.35$

$= 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right]$ *for* $0.35 < D_v/2D_h \le 0.50$

$= D_v/D_h$ *for* $D_v/2D_h > 0.5$

$$R_L = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \le 1.0$$

$$L_c = \frac{A_C}{k_4}$$

$\left|\frac{M}{M_{Pf}}\right|$ = *absolute value of the ratio* $\frac{M}{M_{Pf}}$

$N_F = E_s(1000D_h)^3/EI$

Select $H_c = H_{min}$, $H_c = 1.5$ m, therefore $P = 0$

d) Construction Axle (A_c) = 250 kN, 4 wheels per axle

$M = M_1 + M_B + M_C$

Calculate Flexibility Number N_F

$N_F = E_s(1000D_h)^3/EI$

$E_s = 12$, $D_h = 13.15$, $I = 19060.0$ mm^4/mm

$N_f = 7158$

$k_{M1} = 0.0009$ (for $N_f > 5000$)

$k_{M2} = 0.0032$ (for $N_f > 5000$)

$k_{M3} = 0.120 - 0.018 \log_{10}(N_F)$ when $N_F \le 100\,000$

$= 0.0506$

$D_v/2D_h = 0.5$

$R_B = 0.80 + 1.33\left[(D_v/2D_h) - 0.35\right]$ *for* $0.35 < D_v/2D_h \le 0.50$

$R_B = 0.9995$

$M_1 = k_{M1} R_B \gamma D_h^3$
$M_1 = 45.0$ kN-m/m

$M_B = -k_{M2} R_B \gamma D_h^2 H_c$
$M_B = -18.25$ kN-m/m

$$R_L = \frac{[0.265 - 0.053\log_{10}(N_F)]}{\left(\frac{H_c}{D_h}\right)^{0.75}} \leq 1.0$$

$R_L = 0.3092$

$L_c = \frac{A_C}{k_4}$ where $k_4 = 3.8$ m, (Table 6.15, 4 wheels per axle & $H_c = 1.5$ m)

Lc = 250/3.8 = 65.8 kN/m

$M_C = k_{M3} R_L D_h L_C$
$M_C = 13.54$ kN-m/m

$M = M_1 + M_B + M_C$
$M = 45.0 + (-18.25) + 13.54 = 40.28$ kN-m/m

$M_{Pf} = \phi_{hc} M_p$
$\phi_{hc} = 0.9,\ M_p = 96.5\ kN - m/m$
$M_{Pf} = 86.85$ kN-m/m

$$\left[\frac{P}{P_{pf}}\right]^2 + \left|\frac{M}{M_{pf}}\right| \leq 1,\ P = 0$$

$$\left|\frac{M}{M_{pf}}\right| = \left|\frac{40.28}{86.85}\right| = 0.463 < 1.0$$

Therefore 5.0 mm plate thickness satisfies wall strength requirements during construction for 1.5 m of cover. It is recommended to check other applicable construction axle loads and cover heights in the same manner.

9.0 Factored Longitudinal Seam Strength *(S_f)*

$T_f < S_f$

Where $S_f = \phi_j S_s$ and: $\phi_j = 0.70, S_s = 1735$ kN/m for 5.0 mm plate thickness

S_f = 1215 kN/m

$T_f = 1076.0 < S_f$ OK

10.0 Plate thickness difference – Not Applicable

11.0 Radius of Curvature – Not Applicable for single radius arch

12.0 Wall Strength of Completed Structure
(Applicable for deep corrugated structures only)

$$\left[\frac{T_f}{P_{pf}}\right]^2 + \left|\frac{M_f}{M_{pf}}\right| \leq 1.0$$ where:

Factored Compressive Strength of Section $(P_{pf}) = \phi_h A F_y$

Factored Plastic Moment Capacity of Section $(M_{pf}) = \phi_h M_p$

Maximum Thrust due to factored loads $(T_f) = 1076.0\ kN/m$ $\quad (step\ 5)$

Maximum Moment due to factored loads -

$$M_f = \left|\alpha_D M_1 + \alpha_D M_D\right| + \alpha_L M_L (1 + DLA)$$

Where: $\quad M_1 = k_{M1} R_B \gamma D_h^3$

$M_D = -k_{M2} R_B \gamma D_h^2 H_e$ $\qquad$ where H_e = smaller of H and $D_h/2$

$M_L = k_{M3} R_U D_h A_L / k_4$

$$R_u = \frac{\left[0.265 - 0.053\log_{10}(N_F)\right]}{\left(\frac{H}{D_h}\right)^{0.75}} \leq 1.0$$

Previously calculated (step 8b):
$k_{m1} = 0.0009$, $k_{m2} = 0.0032$, $k_{m3} = 0.0506$, $R_B = 0.9995$, $N_F = 7158$
M_1 = 45.0 kN-m/m

H = 3.0m, $D_h/2 = 13.15/2 = 6.575$m, therefore $H_e = 3.0$m
$M_D = -0.0032*0.9995*22.0*13.15^2*3.0$
M_D = -36.5 kN-m/m

$R_u = 0.1839$
$A_L = 250$ kN
$k_4 = 4.9$m (Table 6.15, 4 wheels per axle & H =3.0m)
$M_L = 0.0506*0.1839*13.15*250/4.9$
M_L = 6.24 kN-m/m

$M_f = |1.25(45.0 + (-36.5))| + 1.70(6.24)(1.0 + 0.10)$
M_f = 22.3 kN-m/m

$M_{pf} = \phi_h M_p$
Where: $\phi_h = 0.85$, $M_p = 96.5$ kN-m/m for 5.0mm plate thickness
***M_{pf}* = 82.0 kN-m/m**

$$\left|\frac{M_f}{M_{pf}}\right| = \frac{22.3}{82.0} = 0.27$$

$P_{pf} = \phi_h A F_y$
Where: $\phi_h = 0.85$, $A = 6.811$ mm^2/mm, F_y =300 MPa
P_{pf} = 1736.8 kN/m

$$\left[\frac{T_f}{P_{pf}}\right]^2 = 0.38$$

$$\left[\frac{T_f}{P_{pf}}\right]^2 + \left|\frac{M_f}{M_{pf}}\right| = 0.65, < 1.0$$

Therefore 5.0 mm plate thickness satisfies wall strength of completed structure.

DESIGN OF FITTINGS REINFORCEMENT

In storm drain projects, branch lines are commonly connected to the main line. Because of the hole cut in the main line for the branch connection, reinforcement may be required. Industry practices have, however, varied. The National Corrugated Steel Pipe Association therefore commissioned a study to develop a standard basis for determining reinforcement requirements.

In Phase 1 of this project, a methodology for addressing the problem was developed based on the finite element method. In Phase 2 of this project, specific reinforcement requirements were established for corrugated steel pipe with main line diameters up to 1200mm, and with branch line diameters from 600 mm up to the diameter of the pipe. Specific wall thicknesses from 1.6 to 4.2 mm were considered, depending on the main line diameter, as well as a depth of cover of 3, 6 and 9 m.

The study showed that the need for reinforcement increases with increasing branch diameter, with increasing depth of cover, and with decreasing thickness. In general, there are three categories (listed in order of increasing reinforcement requirements): (1) cases where no reinforcing is required, (2) cases where longitudinal reinforcement (a tension strap) is required above and below the branch, and (3) cases where both longitudinal and circumferential reinforcement is required as shown in Figure 6.19.

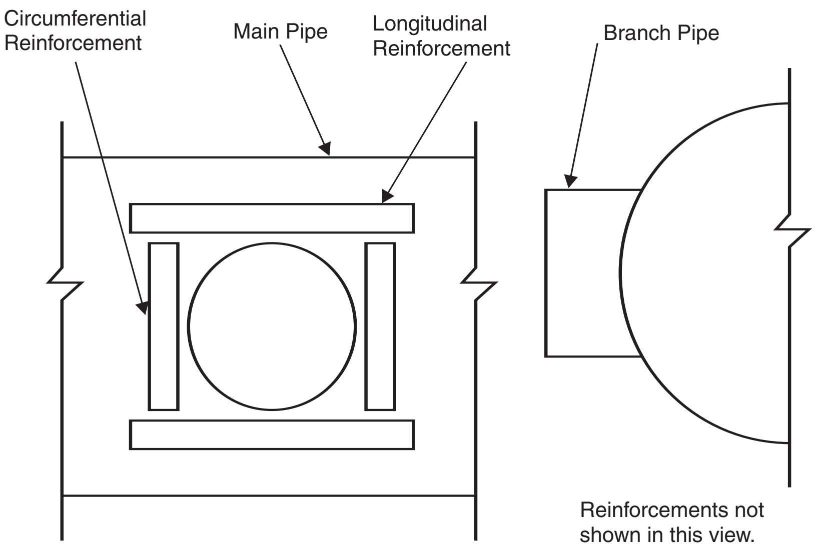


Figure 6.19 Schematic of reinforcements.

To facilitate the design of reinforcement, tables were developed to indicate when each type of reinforcement is required. Also, tables were developed that can be used to calculate the minimum cross section area that must be provided by longitudinal reinforcement when required. For circumferential reinforcement, a simple equation was developed to determine the minimum required area. Information was also presented on fastening methods for connecting the reinforcement.

As an alternative to providing longitudinal and/or circumferential reinforcement, the wall thickness of the main pipe can be increased. It is also permissible to provide a saddle plate as illustrated in Figure 6.20. Saddle plates must be of the same material and wall profile as the main pipe and must extend onto the main pipe on all sides from the branch pipe. The saddle plate must be continuously welded to a stub having the same diameter as the branch pipe. The stub must be at least 300mm long. The saddle plate must be connected to the main pipe with sufficient fasteners (welds, bolts or screws) so that there are no large gaps and so that it will act structurally with the main pipe.

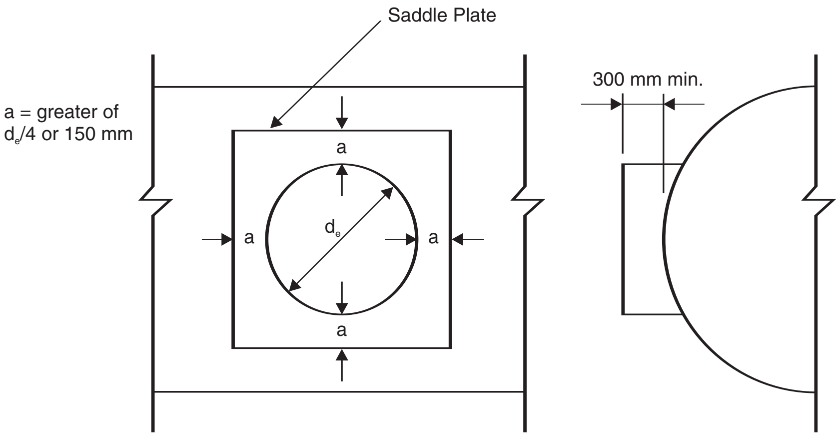


Figure 6.20 Schematic of saddle plates.

As an alternative to providing the required longitudinal reinforcement resulting from the study, it is also permissible to provide beam type reinforcement, as illustrated in Figure 6.21, designed using recognized engineering principles. The beam can be one or more channel or angle sections attached to the main pipe above and below the opening for the branch.

A report titled "NCSPA Design Data Sheet No. 18", was updated and made available in September, 1999. The NCSPA report consists of two parts. Part 1, Design of Reinforcement, includes a seven step design procedure, a design example, and the design tables. Part 2, Background and Discussion, reviews the analysis made, design assumptions, and further details.

ASTM standard A998, titled "Standard Practice for Structural Design of Reinforcements of Fittings in Factory-Made Corrugated Steel Pipe for Sewers and Other Applications", was based on the work done for the NCSPA and provides additional details and information.

Although the NCSPA report covers the most common branch connections, it does not cover all possible geometries or allow for all possible installation conditions.

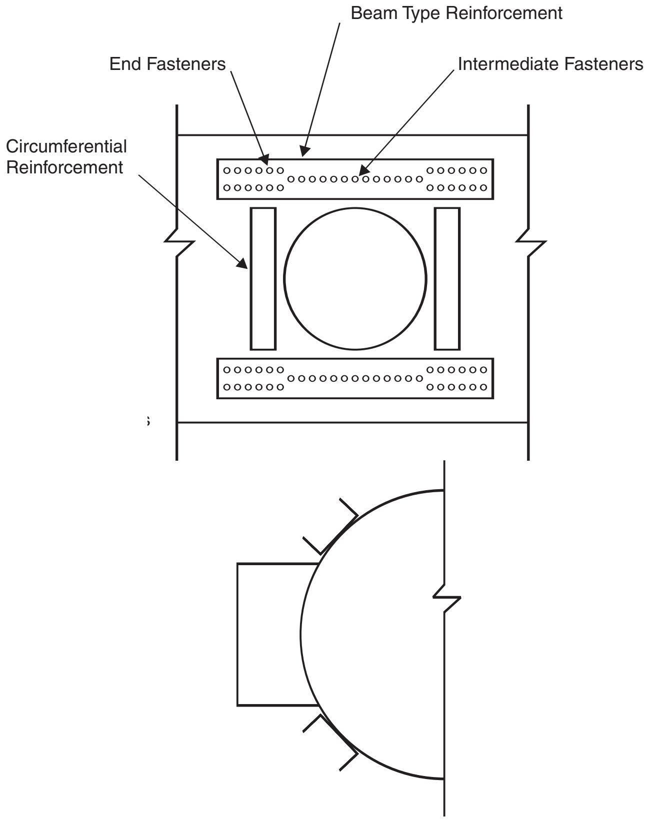


Figure 6.21

COLUMN OR END LOADS

Tests were conducted on riveted corrugated steel pipe at the University of North Carolina in 1927 and the following was determined:

1. Proper size and spacing of circumferential rivets in corrugated steel pipe used as columns.
2. Supporting strength of corrugated steel pipe used for bridge piers and caissons and for columns in general construction.
3. Maximum pressure that can be safely exerted on the end of a corrugated pipe in jacking it through an embankment, without buckling the corrugations.

Further tests were made at the University of Illinois in 1936. The results of these and the earlier tests are shown in Figure 6.22. Subsequent tests at Ohio State University in 1965 confirmed that these short column results are conservative for both annular and helically corrugated pipe. However, the results are only applicable to pipe with 13 mm deep corrugations.

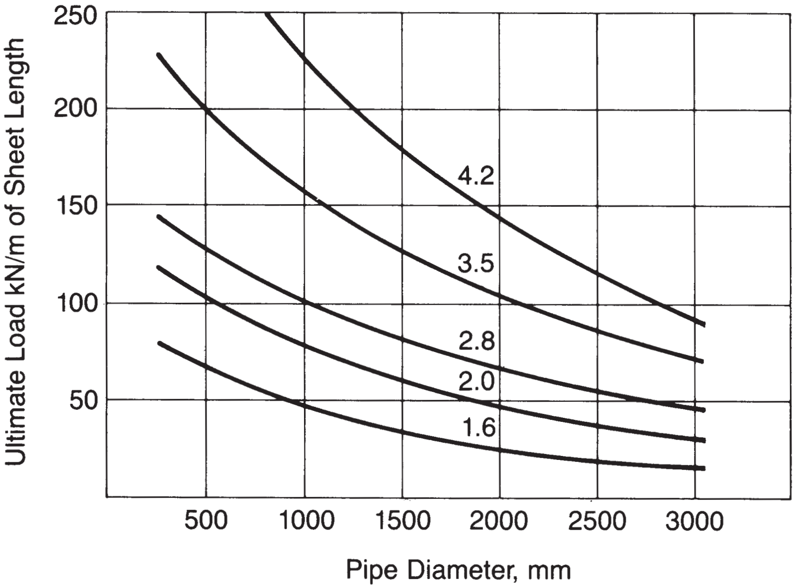


Figure 6.22 Ultimate unit compressive strength of short standard corrugated pipe columns as determined at the University of Illinois.

EXTERNAL HYDROSTATIC PRESSURE

Pipes not buried in compacted soil and which are subjected to external hydrostatic pressure must be designed for buckling as circular tubes under uniform external pressure. The variable passive soil pressure upon which the ring compression design is based, is not available in this load condition and the pipe ring itself must resist the bending moments resulting from out-of-roundness.

The "Theory of Elastic Stability" by Timoshenko and Gere details methods of analysis for thin tubes. Correlation has not been made with these buckling equations and corrugated pipe. However, a few tests have been conducted which suggest that a modified form of the equations will provide the approximate collapse pressure of corrugated steel pipe.

The Timoshenko buckling equation is:

$$P_{cr} = 3EI/[(1-\upsilon^2)R^3]$$

where: P_{cr} = critical pressure, MPa
E = modulus of elasticity of pipe wall = 200 x 10^3 MPa
I = moment of inertia of pipe wall, mm^4/mm
υ = Poisson's ratio = 0.3 for steel
R = radius of pipe, mm

To provide for slight out-of-roundness and other variations, the estimated collapse pressure, P_E, is calculated as:

$$P_E = P_{cr}/2 \text{ MPa}$$

or $P_E = 3EI/[2(1-\upsilon^2)R^3]$

For corrugated steel pipe, $P_E = 330 \times 10^3 \times I/R^3$ MPa

This equation, with suitable safety factors, is useful as a design guide. Pipe for critical applications should be tested to collapse to verify its collapse pressure.

AERIAL SPANS

Should the need arise to run sewers above ground to cross ravines or streams, CSP aerial sewers supported on bents afford an economical solution. The supports are located at pipe joints. Table 6.17 provides allowable spans for this purpose. The table provides for pipes flowing full of water, including the weight of an asphalt-coated pipe. The bending moments were calculated on the basis of a simple span and limited to a factored ultimate bending moment. Ultimate moments were determined theoretically and verified by limited testing.

Aerial span.

Table 6.17
Allowable span, in metres, for CSP flowing full

Pipe Diameter, mm	Specified Steel Thickness, mm				
	1.6	2.0	2.8	3.5	4.2
	68 x 13 mm Corrugation				
600	4.0	4.6	6.1	—	—
800	3.7	4.6	6.1	7.6	—
1000	3.7	4.6	6.1	7.6	—
1200	3.4	4.3	5.8	7.6	9.1
1400	—	4.3	5.8	7.3	8.8
1600	—	4.3	5.8	7.3	8.8
1800	—	—	5.5	7.3	8.8
2000	—	—	—	7.0	8.5
	125 x 26 & 76 x 25 mm Corrugation				
1200	2.7	3.4	4.66	—	—
1400	2.4	3.0	4.3	5.5	—
1600	2.4	3.0	4.3	5.5	—
1800	2.4	3.0	4.3	5.5	6.7
2000	2.4	3.0	4.3	5.5	6.7
2200	—	3.0	4.3	5.5	6.7
2400	—	3.0	4.3	5.5	6.7
2700	—	—	4.3	5.5	6.4
3000	—	—	—	5.2	6.4
	152 x 51 mm Corrugation				
1810	4.0	5.0	6.1	—	—
2120	3.6	5.0	6.1	6.9	8.0
3050	3.6	4.7	5.8	6.9	8.0
3670	3.6	4.7	5.8	6.9	8.0
4300	3.2	4.7	5.8	6.6	7.8
4920	3.2	4.7	5.5	6.6	7.8
5540	—	4.4	5.5	6.6	7.8
6160	—	4.4	5.5	6.4	7.5

END TREATMENT

Designing the ends of a flexible culvert requires additional considerations beyond those addressed in the ring compression design of the culvert barrel. End treatment design must also consider unbalanced soil loadings due to skews or excessive cross slopes, the residual strength of skew cut or bevel cut ends, and possible hydraulic action due to flow forces, uplift and scour.

Pipes skewed to an embankment (crossing at an angle other than 90˚) are subjected to unbalanced soil loads through and beyond the area of the fill slope. The unbalance is easily seen by imagining a section through the pipe and backfill perpendicular to the pipe's longitudinal axis. The amount of imbalance depends on the degree of skew (angle), diameter or span of the pipe, and the slope of the embankment. Unbalanced soil loads typically are not a serious consideration when skews are maintained within the limits of Figure 6.23. Where multiple runs of pipe are used, the total width of the entire run (including the space between the pipes) should be considered in lieu of the span or diameter of a single pipe.

Where skews must exceed these limits, the embankment may be shaped or warped to balance the loads and ensure side support. Figure 6.24 provides typical examples of both properly and improperly balanced embankments. Alternatively, full headwalls can be used. A headwall, designed to carry the thrust forces of the cut end of the pipe, can provide for nearly any degree of skew required.

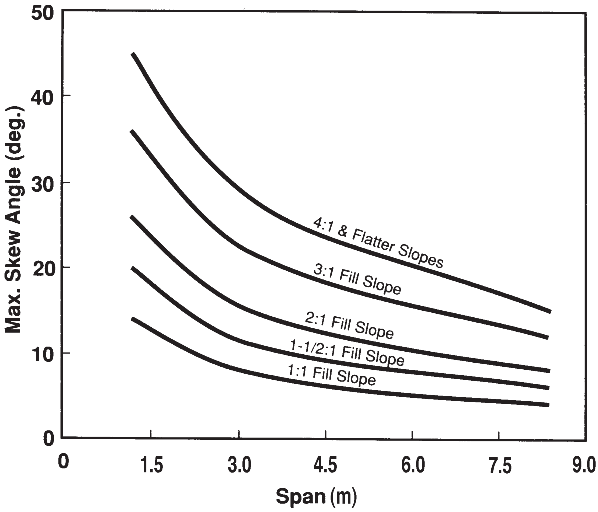


Figure 6.23 Suggested limits for skews to embankments unless the embankment is warped for support or full head walls are provided.

Square end pipes are recommended for most applications . In multiple runs, the ends must be extended so they are aligned perpendicularly as shown for "Proper Balance" in Figure 6.24. Adequate side support at the ends of multiple runs cannot be achieved if they are staggered as shown for "Improper Balance" in Figure 6.24.

Skew cut, bevel cut or skew-bevel cut ends are sometimes used for hydraulic or aesthetic reasons. When the pipe ends are cut, the compression ring is interrupted and pipe strength in the cut area is limited to the bending strength of the corrugation.

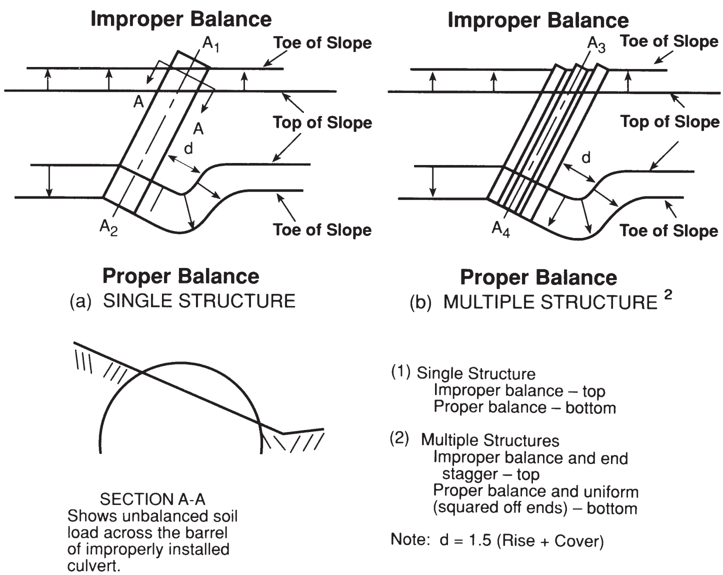


Figure 6.24 Properly and improperly balanced (warped) embankment fills for single and multiple culvert conditions.

Simple skew cut ends can accommodate soil and installation loads if they are limited to the skew angle limits of Figure 6.23. However, hydraulic flow forces must also be considered. Headwalls, concrete collars and other reinforcement can be provided as necessary.

Compacting backfill over high-profile arch.

Bevel cuts, as shown in Figure 6.25, can be done in several ways. Step bevels are recommended for all sizes of pipes. Step bevels are typically limited on long span and larger structural plate pipes, depending on the rise (height) of the structure. Full and partial bevels are typically applicable only to smaller pipes as suggested by Table 6.18. Full bevels are not recommended for multiple radius shapes such as pipe-arches and underpasses, or with bevel slopes flatter than 3:1.

All types of bevel cut ends typically require protection, especially when hydraulic flow forces are anticipated. The cut portion should be anchored to slope pavement, slope collars or headwalls at approximately 450 mm intervals. Cutoff walls or other types of toe anchorage are recommended to avoid scour or hydraulic uplift.

Skew bevel cut ends may be used where they meet the criteria for both skew and bevel cut ends.

Hydraulic forces, on inlet or outlet ends, are difficult to quantify. When structures are designed to flow full under pressure, where flow velocities are high or where flows are expected to increase abruptly, significant hydraulic forces should be anticipated. Equalizer pipes and slow flowing canal crossings for instance, do not provide the same level of concern.

Where significant hydraulic forces are anticipated, important design considerations include: support and protection of the pipe end (especially the inlet), erosion of the fill embankment, undercutting or piping of the backfill or bedding, and hydraulic uplift. Slope collars, or slope pavements with proper pipe end anchorage, can provide support for the pipe end and reduce erosion concerns. A compacted 300 mm thick clay cap over the fill slope, with proper erosion protection such as riprap, helps keep water from infiltrating and eroding the backfill. Toe or cutoff walls, placed to an adequate depth, keep flow from undermining the invert and provide anchorage for the pipe end.

Half headwalls with cut-off walls and more elaborate full headwalls not only stiffen the pipe end against damage from water energy, but also improve the efficiency of the inlet. See Figures 6.26 and 6.27.

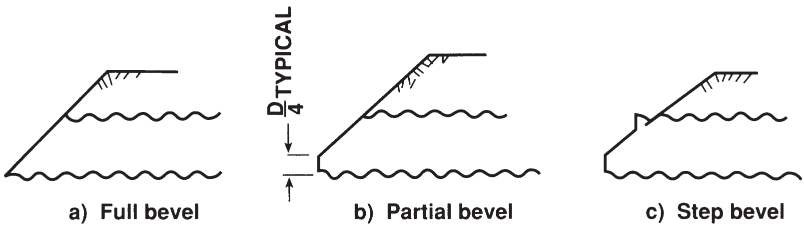


Figure 6.25 Types of beveled ends.

Table 6.18

Recommended diameter (or span) limits for full or partial bevel cut ends

Specified Thickness, mm	Corrugation Type		
	68 x 13	76 x 25 & 125 x 25	152 x 51
1.60	1220	1980	—
2.00	1370	2130	—
2.77/2.80	1520	2440	3960
3.50	1680	2740	4270
4.27/4.78	1830	2900	4570
5.54	—	—	5030
6.32	—	—	5330
7.11	—	—	5490

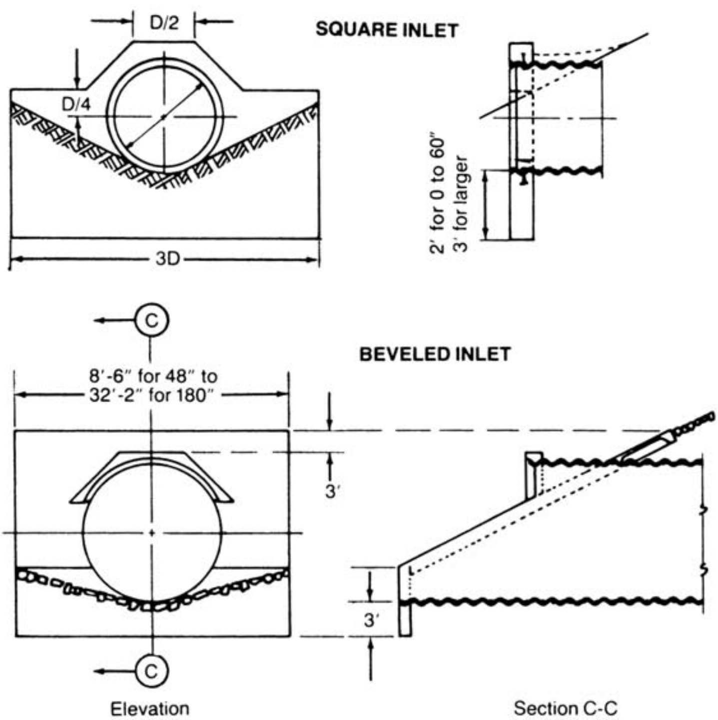


Figure 6.26 Treatment of inlet end of large corrugated steel structures as recommended by the Federal Highway Administration.

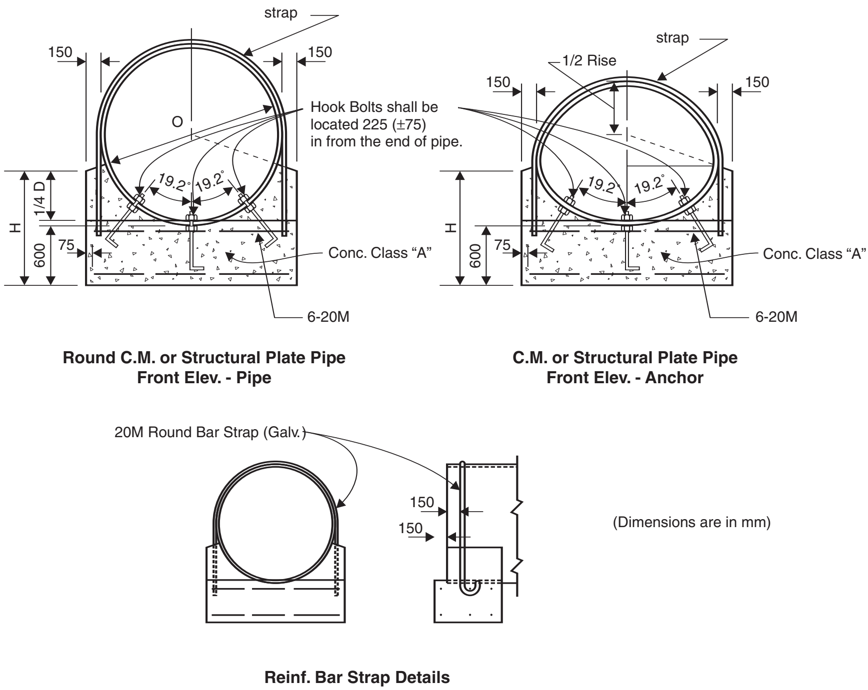


Figure 6.27 Treatment of inlet end of corrugated or structural plate corrugated steel pipe.

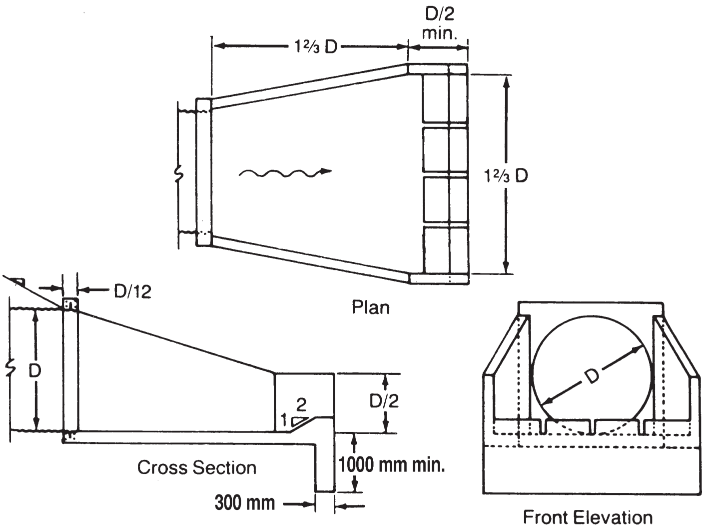


Figure 6.28 Treatment of outlet end of large corrugated steel structures.

The examples of end treatments shown in Figure 6.26, 6.27 and 6.28 perform both hydraulic and structural functions. Besides improving hydraulic flow and supporting skew or bevel cut ends, these treatments provide cut-off walls below and beside the pipe to protect the backfill and embankment slope from piping and erosion.

Temporary Bracing

The ends of structures may require temporary horizontal bracing to prevent distortion during backfill and the construction of headwalls. The end of a structure, cut on an extreme skew and/or bevel, may require support by shoring until the slope pavement or reinforcing collar is completed.

Standard Designs

Most highway and railway design offices have adequate design standards suitable for their applications and jurisdictions. Reference to these is valuable for design of headwalls, riprap protection and slope pavements. For typical end treatments recommended by the U.S. Federal Highway Administration, see Figures 6.26 and 6.28.

APPENDIX A

Utah Test Program 1967 - 1970

Extensive research on buried corrugated steel structures was sponsored by the American Iron and Steel Institute and carried out at Utah State University in Logan, Utah, under the direction of Dr. Reynold K. Watkins. The methods, results and conclusions are summarized here.

Scope

Approximately 130 pipes, 6 m long, in size ranging from 600 to 1500 mm diameter, were loaded to performance limit in low grade soil backfills compacted to between 70% and 99% standard Proctor density. Riveted, spot welded and helical pipe fabrications were included in both 68 x 13 mm and 75 x 25 mm corrugations. Confined compression tests were conducted on six different soils to correlate results to commonly used backfill materials.

Procedure

The test cell was constructed of 16 mm steel plate of elliptic cross-section (see Figure 6.29). The cell was 7.3 m long, 4.6 m wide and 5.5 m high. Steel trusses, pinned to the top of the cell walls, supported hydraulic cylinders which applied a uniform pressure of up to 960 kPa on the top of the soil.

The backfill material used was a silty sand, installed in lifts and compacted with manually operated mechanical compactors. Pipes were instrumented with several pressure gauges around the circumference to measure soil pressures on the pipe. The compactive effort and soil moisture contents were varied to obtain 70% to 99% standard Proctor densities.

After backfilling, steel plates were placed on top of the soil to improve the bearing of the hydraulic rams. Load was applied in planned increments with the following readings taken: loading force, soil pressure on the pipe, vertical deflection, and ring profile. Testing was terminated when the limit of the hydraulic rams was

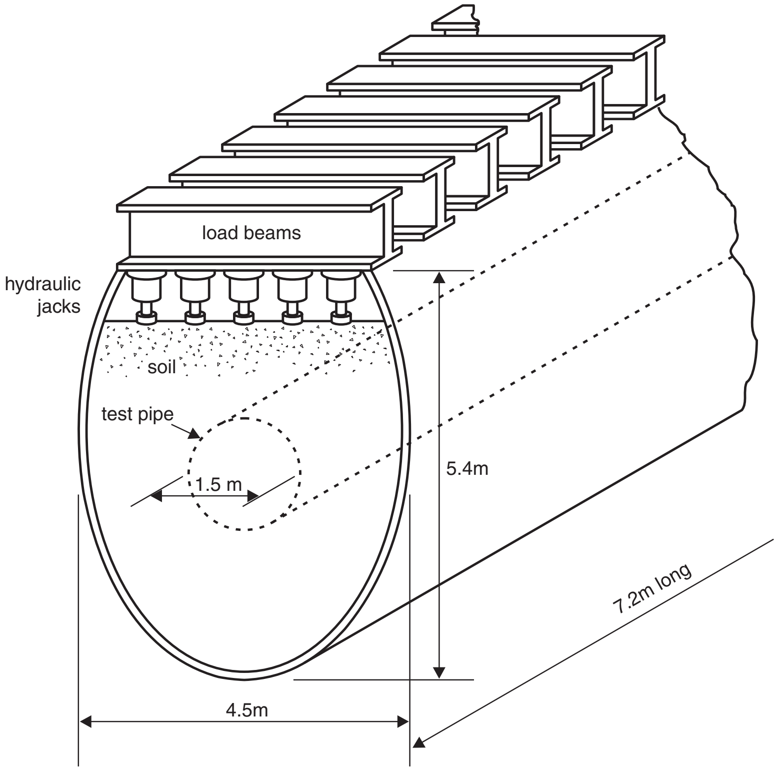


Figure 6.29 Diagramatic sketch of test cell showing method of applying load with hydraulic jacks.

reached. The pipe did not reach their full structural capacity in the test cell. Therefore, the pipes will perform under field loads much higher than those recorded in the test.

Results

Results of the test, plotted for five levels of standard Proctor backfill density, are shown in Figure 6.30. Assuming the load applied by the hydraulic rams equals the pressure acting on the pipe, the ultimate steel stresses are plotted on the previously used buckling chart (see Figure 6.6). It is immediately apparent that most of the steel stresses, calculated by this criteria, are fictitious because they greatly exceed the yield strength. This is explained by Figure 6.31, which illustrates how the applied load is actually carried. The load is carried in part by the soil arch formed in the compacted backfill. The soil arch forms as load is applied and pipe and soil strains occur. Because the stresses on the ordinate are calculated from the total load, with no reduction for the load carried by the soil in arching action, they are designated "apparent stress".

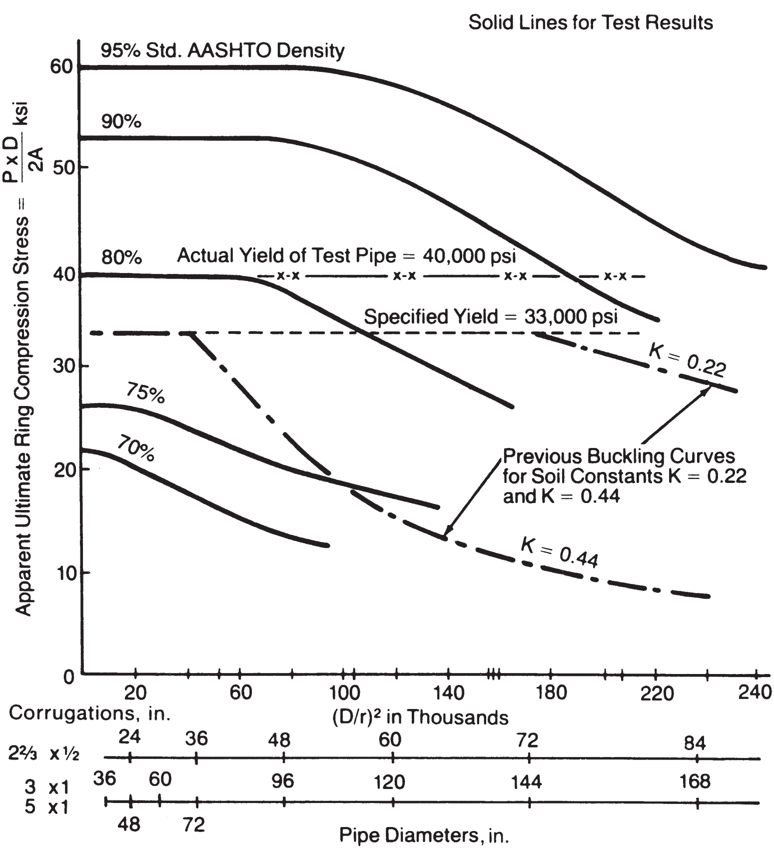


Figure 6.30 Results of Utah loading tests on corrugated steel pipe, showing apparent ultimate ring compression stress as a function of diameter and corrugations for various values of soil density determined by AASHTO.

Twin high-profile arch mine underpass, with sheeting and bin-type retaining wall end treatment.

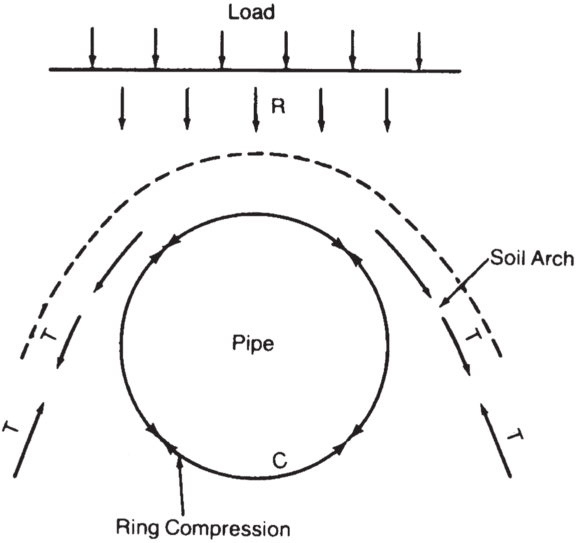


Figure 6.31 Diagram showing how load P_V is partly carried by means of soil arch over the pipe.

Discussion and Conclusions

A prime objective of the Utah program was to establish a practical correlation between backfill density and pipe-behavior.

The then-current design criteria of AISI, AASHTO, and FHWA set ultimate buckling stresses below yield for common combinations of standard corrugations, diameters, and backfill density. These criteria, however, were based on hydrostatic theory modified by model studies.

The Utah program provided, for the first time, ultimate performance data on full-scale soil-steel installations, utilizing a low-grade backfill soil and normal field methods and equipment.

The Utah research confirmed what has been observed in field installations for decades: the quality and density of backfill required to permit the pipe to carry high stress levels to or near the yield strength, is of ordinary magnitude, and is comparable to current common practices for most highway embankments. The soil moduli previously used for wall buckling and deflection criteria were correlated to an unrealistically high level of soil compaction. The test results (Figure 6.30) are plotted on the old buckling stress graph. The wide disparity between the K = 0.44 curve for 85% compaction and the actual performance results at 85% compaction is readily apparent.

Critical Density

The existence of a critical density for flexible pipes had been observed before the research established the zone of "critical density" between 70% and 80% Standard Proctor Density. Critical density is a narrow zone separating the levels of backfill compaction which will and will not prevent deflection failure of the pipe. At 70% Standard Proctor Density, the pipe will not carry stresses anywhere near the yield strength and the ultimate failure mode is a collapse or excessive deflection beyond

20 to 25%. At 80% Standard Proctor Density, there is enough soil support to preclude the deflection collapse and the pipe carries stress near the yield strength.

The test soil used in the Utah research was classified as "low grade" for pipe backfill. Specifically, it was a silty sand which bulked very easily and could be placed to a wide range of Standard Proctor Densities (something very necessary to a good test program). This soil was, quite purposely, far from an ideal material and not representative of what would be obtained in a normal installation.

A number of laboratory tests and field observations were made on a full range of construction soils. These soils showed the same relationships of soil modulus to Standard Proctor Density exhibited by the test soil.

The relationship between pipe performance and backfill properties is simplified to Standard Proctor Density. It has been shown that various other moduli, such as confined compression modulus and secant modulus, can be used for more accurate results. However, these criteria are not currently in a state of practical usefulness for the pipe designer. The backfill can be designed, specified or evaluated on the basis of percent Standard Proctor Density, regardless of soil type. The only exceptions are unstable soils, such as those which turn plastic with moisture even though they have been well compacted to 85% or more Standard Proctor Density and confined in the fill. Such soils would, of course, not be suitable for the base of a high embankment, much less for pipe backfill.

Seam Strength

The Utah research included various types of seams. Pipe seam construction had no apparent effect on the strength of the pipe. Pipe seams failed only after the pipe reached ultimate load and other modes of failure were developing.

The magnitude of compressive load on the pipe wall was 25% to over 100% greater than the ultimate strength values previously used. However, these values were from tests on uncurved and unsupported columns of corrugated sheets with a seam in the middle (a convenient method of testing, but not representative of installed characteristics). When the Ohio State University tests (made on full-scale pipe rings confined in plattens) are examined, the results are compatible with the Utah results (See Figure 6.32). The standard pipe seams tested in this program are satisfactory for ultimate performance of the pipe itself.

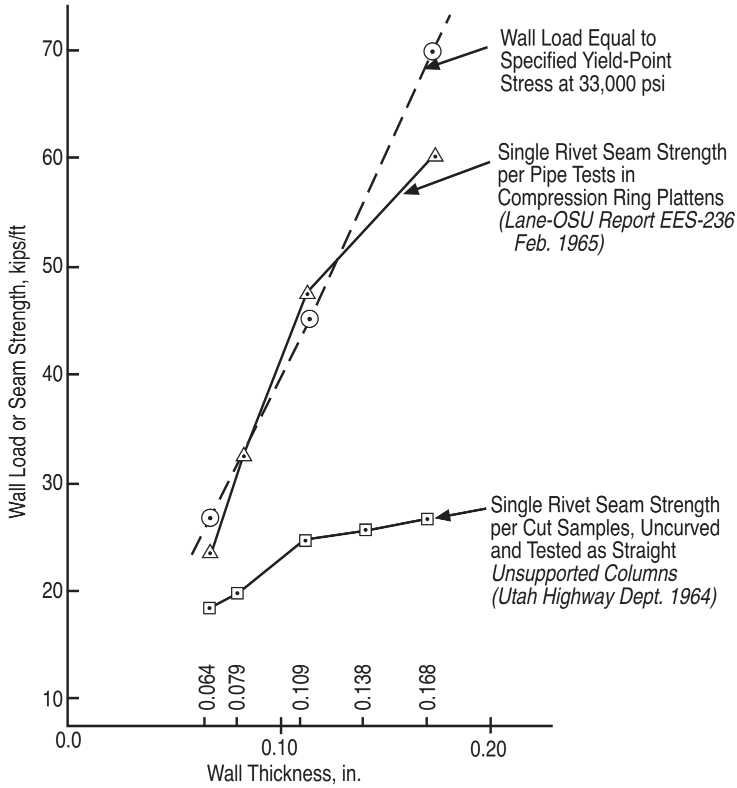


Figure 6.32 Comparison of single-riveted seams tested as straight, unsupported columns by Utah and those tested in compression ring patterns (comparable to actual service) by Ohio. Pipe seams have no apparent effect on the strength of the pipe.

BIBLIOGRAPHY

AASHTO, LRFD *Bridge Design Specifications*, American Association of State Highway and Transportation Officials, 444 N. Capitol St., N.W., Ste. 249, Washington, D.C. 20001.

AASHTO, *Standard Specifications for Highway Bridges*, American Association of State Highway and Transportation Officials, 444 N. Capitol St., N.W., Ste. 249, Washington, D.C. 20001.

Abel, J. F., Falby, W. E., Kulhawy, F. H., Selig, E. T., "Review of the Design and Construction of Long-Span, Corrugated Culverts," August 1977, Federal Highway Administration, Office of Research and Development, Washington, DC 20590.

ASTM, "Standard Practice for Structural design of Corrugated Steel Pipe, Pipe-Arches, and Arches for Storm and Sanitary Sewers and Other Buried Applications," A796/A796M, *Annual Book of Standards*, Vol. 01.06, American Society for Testing and Materials, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959.

ASTM, "Standard Practice for Structural Design of Reinforcements for Fittings in Factory-Made Corrugated Steel Pipe for Sewers and Other Applications." A998,IA998M, *Annual Book of Standards,* Vol. 01.06,American Society for Testing and Materials, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959.

AREMA, *Engineering Manual,* American Railway Engineering and Maintenance-of-Way Association," 8201 Corporate Drive, Ste. 1125, Landover. MD, 20785-2230.

Bacher, A. E., "Proof Testing of a Structural Plate Pipe with Varying Bedding and Backfill Parameters." Federal Highway Administration Reports in Progress, California Department of Transportation, Sacramento, CA 95805.

Bakht, Baider, "Live Load Testing of Soil-Steel Structures, SDR-80-4," August 1980. Ministry of Transportation & Communications, 1200 Wilson Avenue, Central Building Downsview, Ontario, Canada, M3M 1J8.

Bums, J.A., and Richard R.H., " Attenuation of Stresses for Buried Cylinders," *Proceedings*, Symposium on Soil-Structure Interaction, University of Arizona, Sept. 1964.

CSA, *Canadian Highway Bridge Design Code*, Canadian Standards Association - International, 178 Rexdale Boulevard, Toronto, Ontario, Canada M9W 1R3.

Demmin, J., "Field Verification of Ring Compression Design," *Highway Research Record No. 116*, Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, Washington, DC 20418, 1966.

Duncan, J. M., "Soil-Culvert Interaction Method for Design of Culverts," *Transportation Research Record 678*, Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, Washington, DC 20418, 1978.

FHWA, "CANDE-89 Culvert Analysis and Design Program," FHWA-RD-89-168, Federal Highway Administration, U.S. Department of Transportation, 400 7th Street SW, Washington, DC 20590,1989.

Katona, M. G., et al., "CANDE - A Modern Approach for Structural Design and Analysis of Buried Culverts," FHWA-RD-77-5, Federal Highway Administration, U.S. Department of Transportation, 400 7th Street SW, Washington, DC 20590, 1976.

Lane, W. W., "Comparative Studies on Corrugated Metal Culvert Pipes," Building Research Laboratory Report No. EES-236, Ohio State University, Columbus, OH, 1965.

Loutzenheiser, D. W.. "Pipe Culvert Inlet and Outlet Protection," FHWA Notice N 5040.3, Federal Highway Administration, U.S. Department of Transportation, 400 7th Street SW, Washington, DC 20590, 1974.

Marston, Anson, "The Theory of External Loads on Closed Conduits," Bulletin No.96, Iowa Engineering Experimental Station, Ames, IA, 1930.

Meyerhoff, G. G., and Baikie, L. D., "Strength of Steel Culverts Bearing Against Compacted Sand Backfill," *Highway Research Record No. 30,* Transportation Research Board, National Academy of Sciences, 2101 Constitution Avenue, Washington, DC 20418, 1963.

Meyerhoff, G. G., and Fisher, C.L., "Composite Design of Underground Steel Structures," *Engineering Journal*, Engineering Institute of Canada, September 1963.

Timoshenko. S. P., and Gere, J. M., *Theory of Elastic Stability,* 2nd ed., McGraw-Hill, New York., 1964.

Watkins, R. K., Ghavami, and Longhurst, G., "Minimum Cover of Buried Flexible Conduits," *Journal of the Pipeline Division*, ASCE, Vol. 94, No. PL 1, Proc. Paper 6195, October, 1968.

Watkins, R. K., and Moser, R. P., "The Structural Performance of Buried Corrugated Steel Pipes," Utah State University, Logan, Utah, 1969.

White, H.L. and Layer, J P., "The Corrugated Metal Conduit as a Compression Ring," *Proceedings,* Transportation Research Board, Vol. 39, 1960.