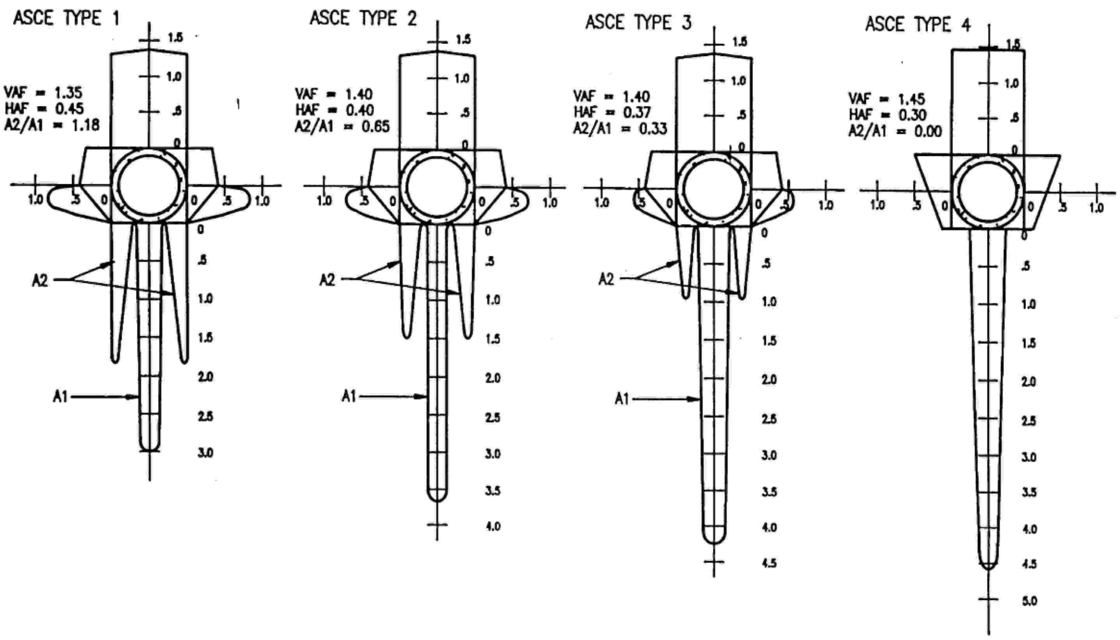
City of Calgary

Standard Practice for the Design and Installation of Rigid Gravity Sewer Pipe in the City of Calgary



[Heger Pressure Distributions for Direct Design of Concrete Pipe]

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January 21, 2008 0082-242-00

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Dear John:

Re: Standard Practice for the Design and Installation of Rigid Gravity Sewer Pipe in the City of Calgary

Please find enclosed the Final Standard Practice for the Design and Installation of Rigid Gravity Sewer Pipe in the City of Calgary, revised in accordance with comments relating to trench bedding factors.

We thank you for the opportunity to work on this very interesting and challenging assignment for the City of Calgary.

Sincerely,

UMA Engineering Ltd.

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1.0 Introduction

This standard practice covers the design and construction of rigid pipe for use in gravity flow applications within the City of Calgary. While the Standard Practice is primarily focused on the use of concrete pipe, it is applicable to other rigid pipe products intended for use in gravity applications.

The standard practice provides an overview of both indirect and direct design methods. As direct design methods are applicable to the standard installations developed for reinforced pre-cast concrete pipe, they are generally not applicable to be applied to other rigid pipe products with the possible exception of the load theory associated with direct design.

The overview provided in the standard practice presents a balance of theoretical and historical context for design practices and recommendations specific to the manner in which indirect and direct design is desired to be carried out in the City of Calgary as well as general guidance as to what situations are most applicable for each design method.

The standard practice is intended to be used as a reference by the owner or owner's engineer in preparing project specifications within the City of Calgary based on the standard design and installation practices specified herein.

The design procedures given in this standard are intended for use by engineers who are familiar with the concept of soil-pipe interaction and of the factors that may impact both the performance of the pipe and of the soil envelope. Before using the design procedures, the engineer should review the guidance and requirements given in the primary design manuals that cover indirect and direct design more fully including a detailed accounting of the theory behind each design method. Both design methods are described fully in the Concrete Pipe Technology Handbook[1] while the Standard Practice of Direct Design is detailed in ASCE Standard Practice 15-98[2].

For ease in use versus other references, the notations utilized are consistent with the Concrete Pipe Technology Handbook and the primary values of dimensions and quantities are expressed in inch-pound (English) units with conversions expressed in SI unit values. For convenience notational standards are re-produced in Appendix A.

1.1 Direct and Indirect Design Process Overview

While the direct and indirect design methods are markedly different they are essentially geared towards reaching the same overall objective, the selection of an appropriate balance of pipe structure and soil supporting structure for a given design condition.

Direct design as a process is well suited to larger diameter pipe both due to its thoroughness of design checks and the ability to achieve a more cost effective design that conventional indirect design with ASTM C76 pipe cannot achieve due to the restrictive nature of Class pipe standard design sections. Due to the most common governing modes of structural failure, it would be prudent to carry out all direct design checks in pipe diameters of 900 mm or larger irrespective of whether the practitioner is utilizing direct or indirect design concepts to ensure that all critical failure modes are reviewed in instances where the capital investment in the product are high as typically are the consequences of failure.

[1] American Concrete Pipe Association, *"Concrete Pipe Technology Handbook – A Presentation of Historical and Current State-of-the-art Design and Installation Methodology",* ACPA, 1993

[2] ASCE, *"ASCE 15-98, Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)"*, American Society of Civil Engineers, 1998

At the highest level each of the design processes involves the following necessary steps:

1. Establish basic design criteria
   - Inside diameter of pipe
   - Height of cover and unit weight of earth
   - Surface design loads
   - Design internal pressure (not possible to use indirect design if required and limited to 15 m of head in direct design applications)
   - Type of Standard Installation
   - Pipe initial design parameters such as wall thickness, concrete strength, thickness of cover over reinforcement, steel arrangement, type and strength of reinforcement (all required for direct design only)
2. Determine design loads and earth pressure distribution
   - In direct design applications earth loads and response is facilitated through the use of the Standard Installations and the Heger pressure distribution model
   - In indirect design this is accomplished through either the Marston-Spangler pressure distribution approach or the Heger pressure distribution assessment for vertical loads and the use of bedding factors
   - Live loads are carried out in identical manners for direct and indirect design.
3. Select design factors
   - In direct design various load and resistance factors and crack control factors are applicable based on a limit states design approach and minimum values permitted by the ASCE Standard Practice
   - In indirect design, a single safety factor is selected based on the recommendations of this Standard Practice and whether the designer is working with reinforced or non-reinforced pipe. Non-reinforced pipe is not permitted in direct design applications.
4. Perform structural analysis
   - In direct design structural analysis involves a comprehensive determination of all moments, thrust, and shears produced by the design loads.
   - In indirect design, structural analysis is limited to applying the appropriate bedding factors to design loads.
5. Design the pipe
   - In direct design the pipe wall is designed selecting the appropriate balance between pipe structure and selected soil structure.

- In indirect design a pipe class strength is specified in terms of an appropriate three edge bearing strength to be supplied in conjunction with a specified installation type.

2.0 External Loads and Pressure Distribution

The designer shall evaluate the various loads that affect the pipe structurally. The effects of loads and the resulting pressures that act on the pipe are complicated by the effects of pipe-soil interaction that occur as a result of subtle deformations of the pipe and the surrounding soil. The significance of pipe-soil interaction and the role it plays in pipe design is discussed more fully in Section 3.0.

While it is necessary to understand different components of loads in different manners dependent of whether the practitioner is utilizing indirect or direct design methods, the same basic range of external loads must be understood in order to assess pipe design requirements.

Typical loads that must be considered when analyzing or designing a buried pipe installation include:

- Weight of the pipe
- Earth loads
- Weight of the fluid and internal pressure, if any
- Live loads
    - Surface concentrated loads
    - Surface surcharge loads

2.1 Pipe Weight

Pipe weight may or not be a significant component of load relative to other loads in buried pipe analysis.

In indirect design, the structural design of the pipe is based upon the strength of the pipe in a three edge bearing test. As the pipe self-weight is already accounted for in a three-edge bearing test it can be ignored in accounting for overall loads in analysis. In direct design, however, pipe weight is a true component of overall loads and should be considered in design, particularly in larger diameter structures.

Approximate weights of pipe may be calculated as follows:

Circular $$W_p = 3.3h(D_i + h) \quad \textbf{(2-1)}$$

The wall thickness for circular pipes is often referred to in standard nomenclature of “A”, “B”, or “C” wall thicknesses. The relationship between wall thickness, wall thickness type and inside diameter is governed by the following expressions (Note: dimensions are in inches):

Wall A $$h = \frac{D_i}{12} \quad \textbf{(2-2)}$$

Wall B $$h = \frac{D_i}{12} + 1 \quad \textbf{(2-3)}$$

Wall C $h = \frac{D_i}{12} + 1.75$ **(2-4)**

2.2 Earth Loads

The earth load that acts on a buried pipe is significantly affected by the relative deformation of the pipe and the adjacent soil. Two common methods are used for estimating earth loads and the resultant pressure distribution around the pipe:

- Heger Pressure Distribution Loads
- Marston-Spangler soil-structure interaction analysis

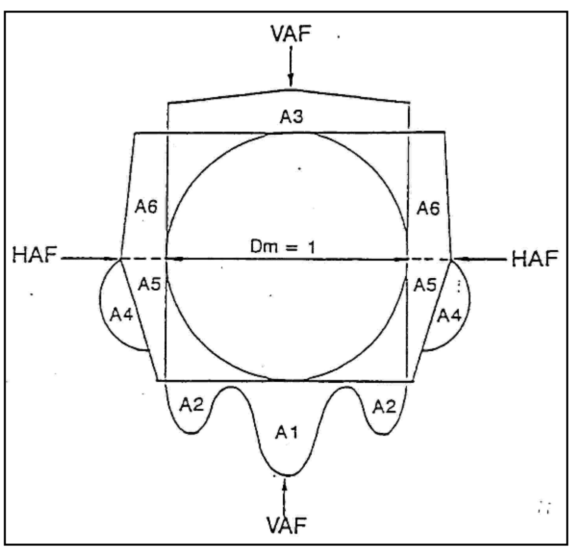
Earth loads and pressure distributions determined via the finite element model (FEM) and model studies used in SPIDA (Soil Pipe Interaction Design and Analysis) are the most current and modern assessment of earth loads and the resultant pressure distributions around rigid pipe. This method of earth load assessment and the soil response is commonly referred to as the Heger Pressure Distributions. This is the method of earth load determination that is used for direct design and is incorporated into the Direct Design Standard Practice ASCE 15-98. In terms of earth load predictions, however, it can be used for both direct and indirect design methods.

Marston-Spangler soil-structure analysis has been utilized for decades to compute earth loads on rigid buried pipes and to form a soil-pipe interaction through the use of bedding factors. In this Standard Practice it is still an acceptable means of determining earth loads for indirect design.

2.2.1 Earth Loads – Heger Pressure Distributions

The major feature of the Heger pressure distributions are the use of nomenclature that relates vertical and horizontal loads to the prism load at the top of the pipe and the use of non-dimensional “Arching Factors” and “Pressure Distribution Ratios” (the pressure bulbs A1, A2, A4, A5, and A6 in Figure 1 below) to define the distribution of loads within the embedment zone in response to the applied vertical and horizontal loads.

Figure 1 - Heger Pressure Distribution



The vertical and horizontal components of earth and horizontal loads on the pipe are defined in terms of arching factors with the following definitions:

$$VAF = \frac{W_e}{PL} \qquad \textbf{(2-5)}$$

Where

VAF = vertical arching factor

W_e = total vertical earth load

PL = prism load

$$HAF = \frac{W_h}{PL} \qquad \textbf{(2-6)}$$

Where

HAF = horizontal arching factor

W_h = total horizontal load on the side of pipe

PL = prism load

The HAF should not be confused with the ratio of lateral to vertical earth load that is used in other design methods. In terms of Heger pressure distributions the ratio of lateral to vertical earth load can be determined by the expression:

$$RatioOfLateralToVeritcalEarthLoad = \frac{HAF}{VAF}$$

The datum for both vertical and horizontal loads on pipes in Heger distributions is the prism load, *PL*, in the form:

$$PL = w\left[H + \frac{D_o(4-\pi)}{8}\right]D_o \qquad \textbf{(2-7)}$$

where,

w = unit weight of soil (lbs/ft^3)

H = height of fill (ft)

D_o = outside pipe diameter (ft)

The prism load, *PL*, is defined as the unit weight of backfill soil over the pipe times the volume of a one foot thick prism over the outside diameter of the pipe.

For any of the Standard pipe-soil installations in the City of Calgary, the VAF and HAF may be established by relating it to soil-structure analysis that has been previously carried out (the SPIDA parametric studies) and, therefore, the resultant earth load and horizontal load on the side of the pipe can be computed through expressions (2-5), (2-6), and (2-7), respectively. The Standard Installation Types for use in the City of Calgary are depicted in Figure 2. While the selection of specific Standard Installation Types is a function of economics (e.g. in terms of the balance invested in pipe structure versus soil structure) and end use considerations (e.g. a Type 4 installation may not be appropriate for use under a pavement due to the amount of consolidation that may be anticipated) each installation Type can be appropriate in the appropriate circumstances.

VAF ratios typically range between 1.2 and 1.5 for positive projecting embankment loads. Higher ratios can develop with soft soils on firm foundations (e.g. without the middle third of the bedding placed loose as noted). VAF ratios for trench installations are generally significantly less than these values and can be significantly less than 1.0 in very narrow trenches with firm natural soil walls.

HAF ratios typically range from 0.5 to 0.3 for positive projecting embankment loads and may drop to less than 0.1 in very narrow trench installations. The optimum balance in pipe design is achieved by ensuring adequate trench widths to facilitate proper placement of embedment material in the haunch area as noted in Figure 2.

Based on the use of the minimum trench widths and the materials noted in the City of Calgary Standard specifications, the VAF and HAF values noted in Table 1 shall be used for design for each installation type.

Table 1 - VAF and HAF for Standard Installations

Standard Installation Type	VAF	HAF
Type 1	1.35	0.45
Type 2	1.40	0.40
Type 3	1.40	0.37
Type 4	1.45	0.30

The principle of the Heger Pressure distributions has been verified in numerous field trials including trials carried out in the City of Calgary[3]. The embedment soil response to applied loads is largely reflected in pressure bulbs A1, A2, A4, and A5 in Figure 1, with pressure bulbs A2 and A4 increasing in value with improved placement of material in the haunch area (i.e. picking up and transferring more of the load) and pressure bulbs A1 and A5 decreasing in value with improved placement of material in the haunch area (i.e. picking up and transferring less of the load).

It is important to understand the principle that increasing the quality of embedment (i.e. higher quality material placed at higher densities) minimizes load transfer directly to the invert pressure bulb and maximizes load transference to the haunch area, which results in a more balanced distribution of pressure around the pipe. This phenomenon is depicted in Figure 3 for each of the ASCE Standard Practice Installations.

[3] Simpson, Gumpertz, & Heger, Inc., *"Instrumented Concrete Pipe test, Cranston Development, Calgary, Alberta"*, February 1999.

Figure 2 - Standard Installation Types - City of Calgary

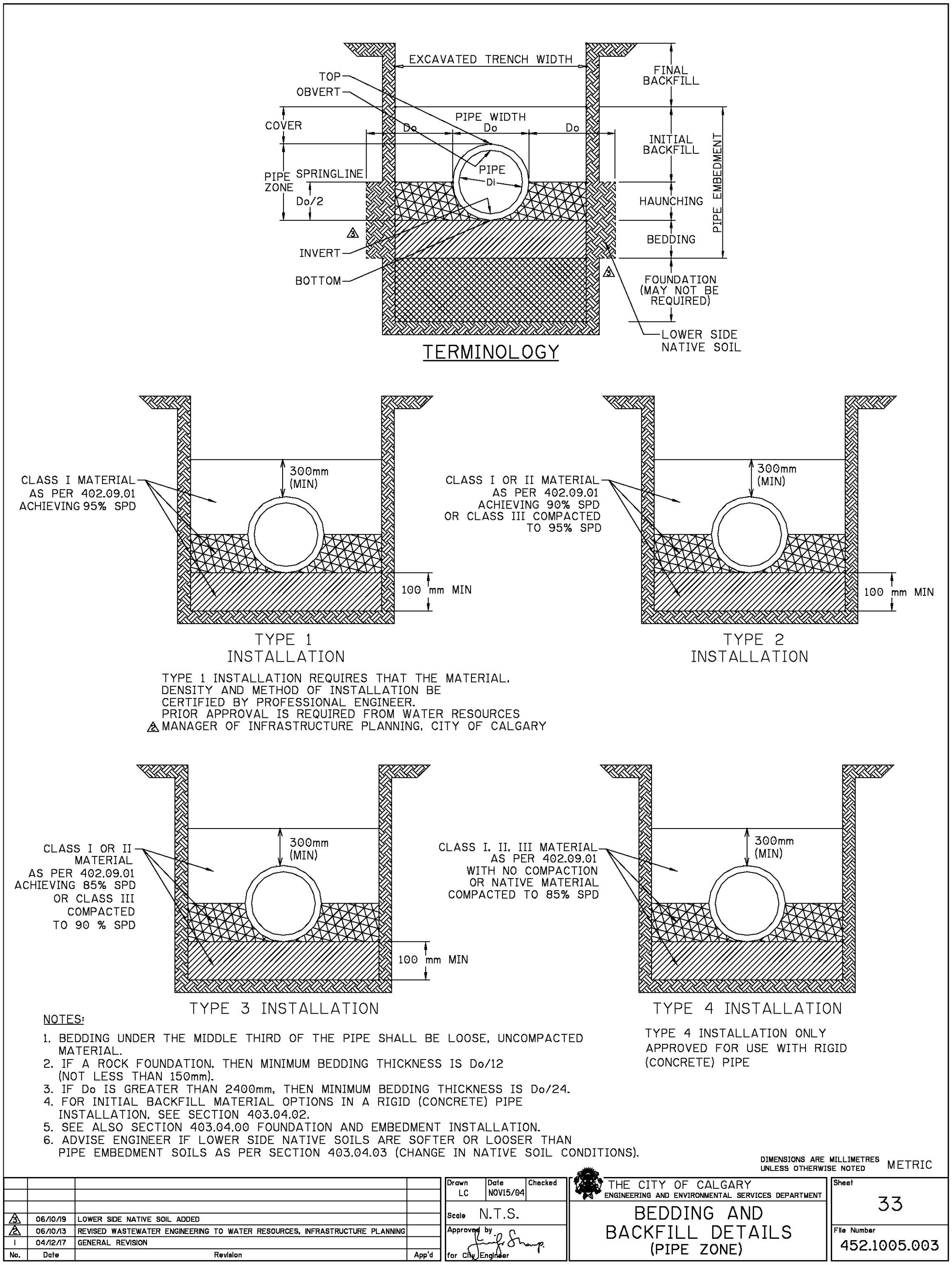
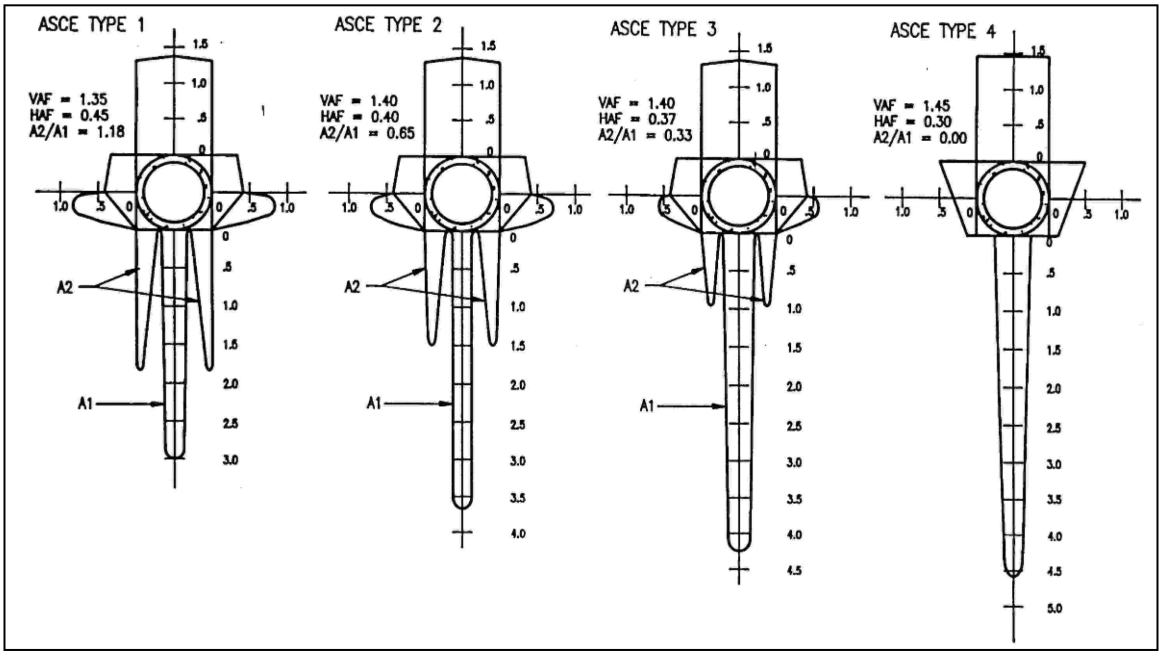


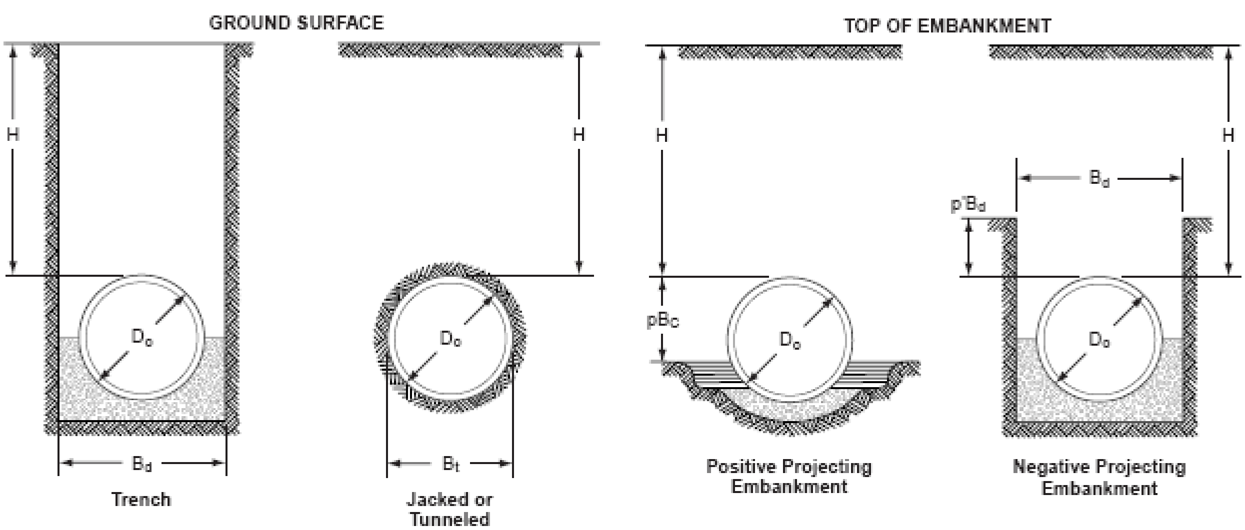
Figure 3 - Pressure Distributions Associated with Standard Installations



2.2.2 Marston-Spangler Soil Structure Analysis

Marston-Spangler soil-structure analysis determined loads on buried pipes for various installation types, the essential features of which are detailed in Figure 4 below.

Figure 4 - Marston-Spangler Installation Types – Essential Features



This Standard Practice will deal with the computational procedure of determining trench and positive projecting embankment loads only. Tunnelled or jacked loads are beyond the scope of this Standard Practice and while usually considerably lower in magnitude than conventional loads, they are influenced by considerably more complex phenomena. From a practical perspective, trench loads and positive projecting embankment loads are the most quantifiable of loading conditions related to open cut installations and typically represent an extreme range of the minimum and maximum earth loads that can occur over buried rigid pipe in conventional construction.

In Marston's research it was determined that earth loads on rigid pipe installed in a trench could be estimated by the following expression:

$$W_e = C_d w B_d^2 \qquad \textbf{(2-8)}$$

where,

C_d = load coefficient as defined below

w = unit weight of soil (lb/ft^3)

B_d = trench width at top of pipe (ft)

And C_d can be determined by the following expression

$$C_d = \frac{1 - e^{-2K\mu'\frac{H}{B_d}}}{2K\mu'} \qquad \textbf{(2-9)}$$

where,

K = Rankine lateral soil pressure coefficient

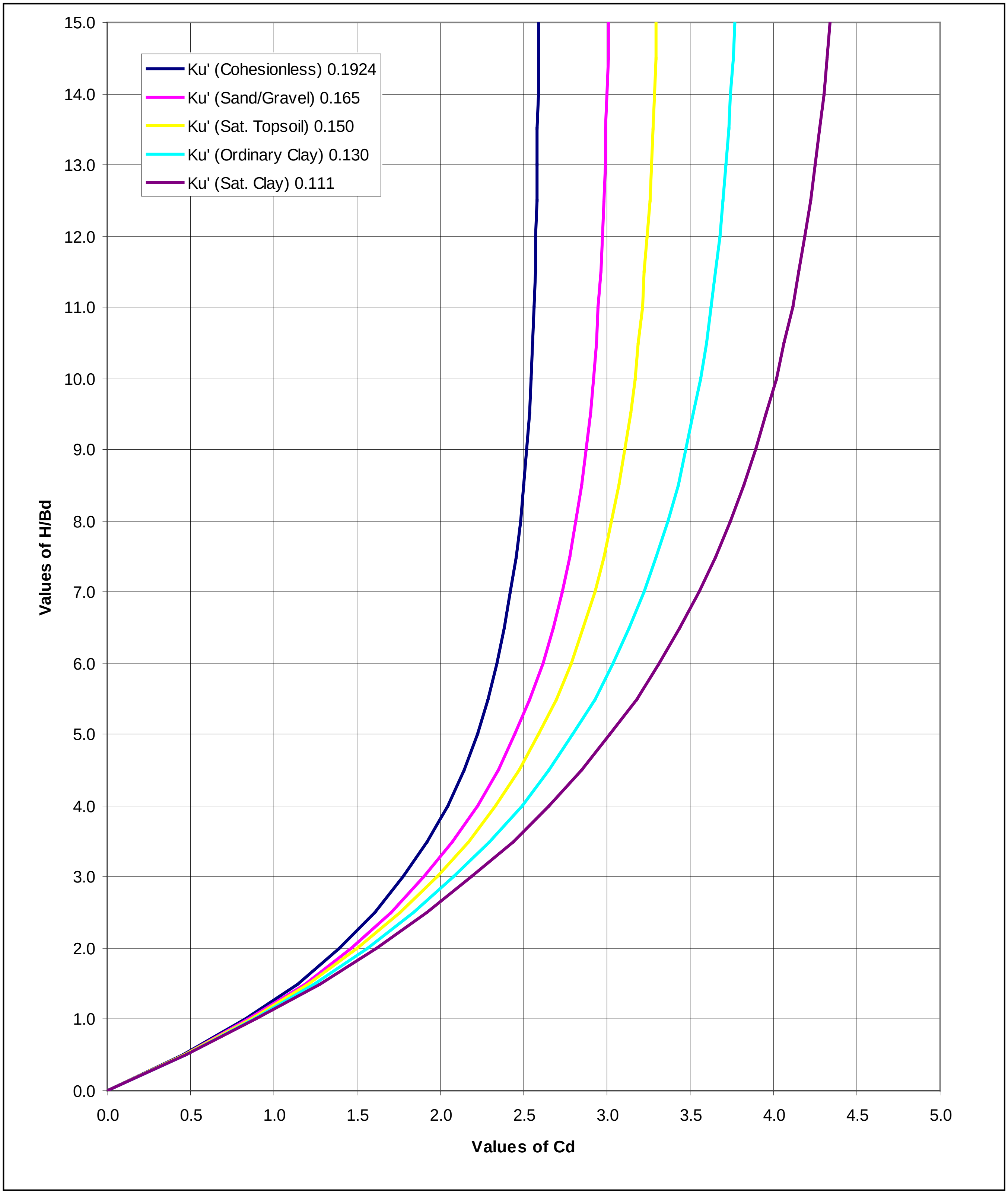
μ' = coefficient of sliding friction between fill material and sides of trench

The product of the Rankine's lateral soil pressure coefficient and the coefficient of sliding friction between fill material and sides of trench angle is summarized for various soil types in Table 2 below.

Table 2 - Product of Rankine Coefficient and Coefficient of Sliding Friction between Fill Material and Sides of Trench

Soil Type	$K\mu'$
Max for Granular materials without cohesion	0.1924
Maximum for Sand and Gravel	0.165
Topsoil	0.150
Maximum for Saturated Clay	0.110

Figure 5 - Trench Load Coefficient, C_d



Earth loads are normally calculated for either the greater of utilizing sand and gravel backfill with a density of 135 lb/ft^3 (2165 kg/m^3) or saturated clay backfill with a density of 120 lb/ft^3 (1920 kg/m^3). Standard Practice in the City of Calgary is to utilize an assumption of sand and gravel backfill for all installations.

Values of C_d may be calculated directly from expression (2-9) above or estimated based on graphical solutions such as Figure 5. Having determined the load coefficient the earth load, W_e, may be computed directly from expression (2-8) above.

Similar to earth loads due to trench conditions, Marston developed the following expression for estimating earth loads on rigid pipe exposed to pure embankment conditions:

$$W_e = C_c w B_c^2 \qquad \textbf{(2-10)}$$

Where,

C_c = positive projecting embankment load coefficient as defined below

B_c = outside diameter of pipe (ft)

The positive projecting embankment load coefficient, C_c, is a function of the ratio of the height of backfill to the outside pipe diameter as well as the following soil and installation parameters:

- Rankine lateral soil pressure coefficient times the internal soil friction angle
- Projection ratio, p, for positive projecting pipe, where p is the ratio of the vertical height of the top of the pipe above the embankment subgrade to the pipe outside diameter.
- Settlement ratio, r_{sd}, where r_{sd} is the ratio of the difference between the settlement of the soil adjacent to the pipe and the top of the pipe.

While considerable work has been undertaken to quantify the parameters impacting positive projection load coefficients, they are complex and do not lend themselves to uniform application by a wide range of practitioners. The most current Concrete Pipe Design Manual and this Standard Practice, therefore, recommend the use of Heger VAF's to determine embankment loading for indirect design applications. As noted in Section 2.2.1, the VAF's for use in Calgary are based on the prism load, PL, and vary according to Standard installation type with:

Prism Load equal to:

$$PL = w\left[H + \frac{D_o(4-\pi)}{8}\right]D_o$$

And the embankment condition earth load determined by:

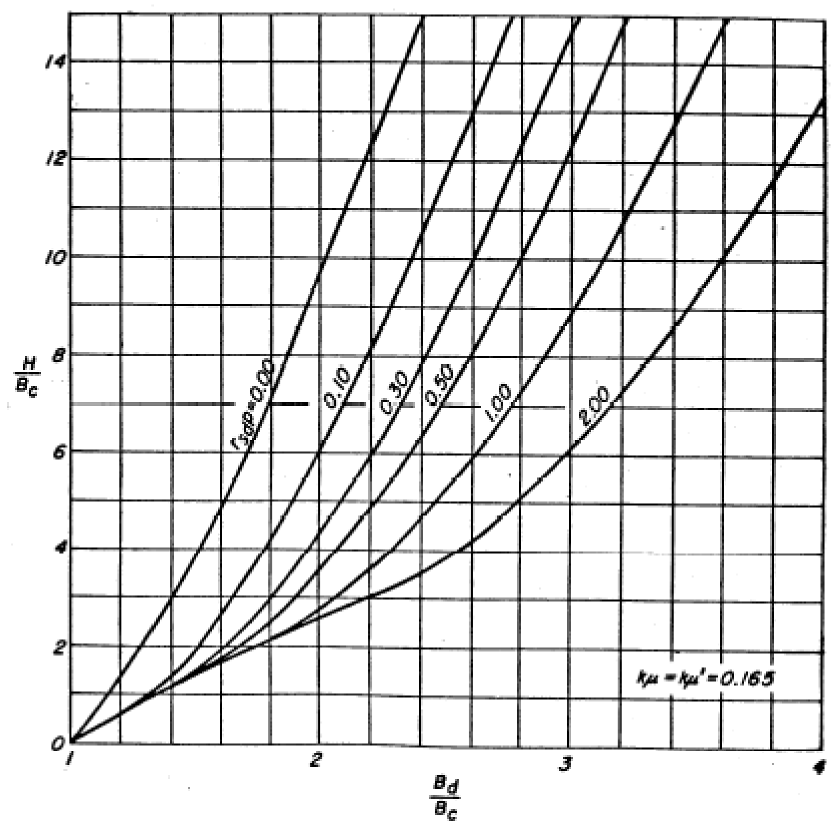
$$W_e = VAF * PL \qquad \textbf{(2-11)}$$

The values for VAF vary in accordance with the Standard Installation Type as detailed in Table 1 in Section 2.2.1.

In embankment loading the earth load is independent of the trench width and, therefore, no contractual controls are necessary to ensure that anticipated earth loading is not in excess of contemplated loading based on a contractor's proposed construction method. In this Standard Practice it is recommended to use embankment loading values to calculate anticipated earth loading unless specific contractual controls are in place to limit trench widths to specific or narrow trench values.

The point at which embankment loading and trench loading are computationally equal is commonly called the transition width. The point at which the transition occurs is complex and is a function of the height of fill, the pipe diameter as well the settlement (r_{sd}) and projection (p) ratios. Figure 6 provides a graphical solution to estimate transition widths for Marston-Spangler analysis for a range of r_{sd}p values in granular backfill. From a practical perspective r_{sd}p values that are less than 0 approach true trench conditions, while r_{sd}p values greater than 2 approach true embankment conditions.

Figure 6 - Transition Width Ratios[4]



NOTE: These values of $\frac{B_d}{B_c}$ are those values at which Trench Condition loads equal Projection Condition loads.

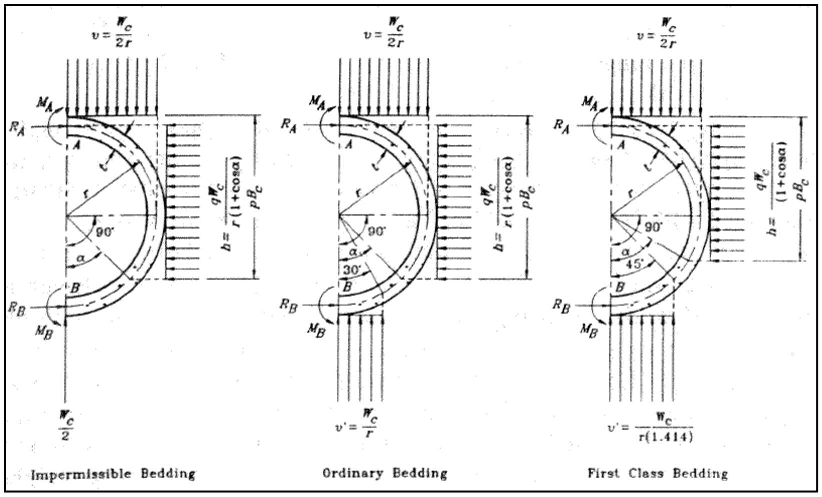
2.2.2.1 Pressure Response – Marston-Spangler Analysis

Marston and Spangler tested different installation configurations and confirmed that the resultant load experienced by the pipe was largely dependent on installation conditions. In their original work bedding

[4] ACPA, *"Concrete Pipe Handbook"* American Concrete Pipe Association, 1998, pp 4-7

classifications included largely qualitative terms ranging from impermissible, ordinary, and first class bedding as depicted in Figure 7 below.

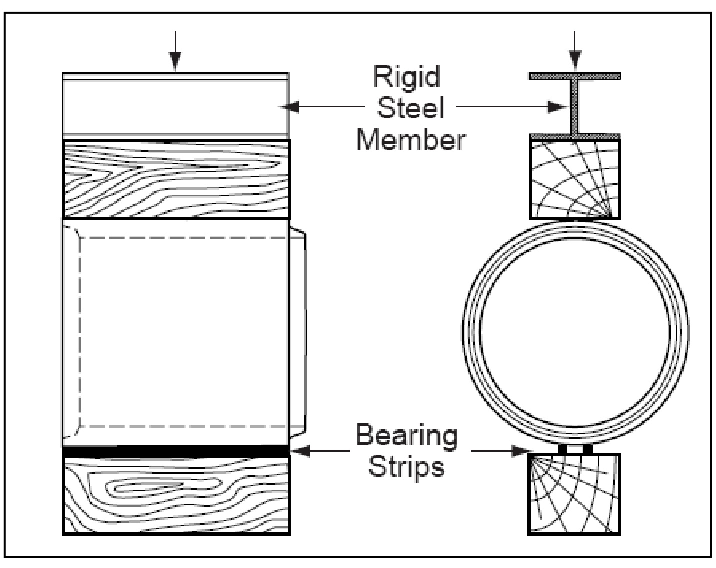
Figure 7 - Marston-Spangler Load Distribution Assumptions for Embankment Conditions



The load response requirements of the pipe in Marston-Spangler analysis is carried out by means of a bedding factor, B_f, which, in theory is the ratio of the strength of the pipe under the installed condition of loading and bedding to the strength of the pipe in a controlled three edge bearing test. This same ratio was originally defined by Spangler as the load factor. This latter term, however, was subsequently defined in the ultimate strength method of reinforced concrete design with an entirely different meaning. To avoid confusion, therefore, Spangler’s term was renamed the bedding factor.

The three-edge bearing test as shown in Figure 8 is the normally accepted plant test that is used as a datum prior to evaluating the in-field strength of an installation. Proper procedures for the test are detailed in Section 4 of CSA Standard A257.0-03 Methods for Determining Physical Properties of Circular Concrete Pipe, Manhole Sections, Catch Basins, and Fittings.

Figure 8 - Three-Edge Bearing Load Test



Spangler's research is documented in a 1933 paper entitled, *The Supporting Strength of Rigid Pipe Culverts*. Spangler presented the three bedding configurations depicted in Figure 7 and the concept of a bedding factor to relate the supporting strength of the buried pipe to the strength obtained in a three-edge bearing test.

Spangler's theory postulated that the bedding factor for a particular pipeline and, consequently, the supporting strength of the buried pipe, was dependent on two installation characteristics:

- Width and quality of contact between the pipe and bedding.
- Magnitude of lateral pressure and the portion of the vertical height of the pipe over which it acts.

For the embankment condition, Spangler developed a general equation for the bedding factor, which partially included the effects of lateral pressure. For the trench condition, he established conservative fixed bedding factors, which neglected the effects of lateral pressure, for each of the three embedment conditions noted.

In theory, Spangler's elastic analysis of the pipe ring resulted in the following equation for bedding factor, B_f.

$$Bf = \frac{1.431}{N - xq}$$

Where:

- N varies with the type of bedding
- x varies with the projection ratio, p
- q varies with the Rankine pressure coefficient K

Parametric studies carried out since Spangler's original work in conjunction with the ASCE Standard Installations have modified the values of recommended bedding factors somewhat, but analytically they remain reasonably true to the original derivation.

The development of bedding factors for Standard Installations follows the same concept utilized in Direct design reinforced concrete design theory. The basic definition of bedding factor is the ratio of maximum moment in the three-edge bearing test to the maximum moment in the buried condition, when the vertical loads under each condition are equal, therefore:

$$B_f = \frac{M_{Test}}{M_{Field}} \qquad \textbf{(2-12)}$$

where:

B_f = bedding factor

M_{Test} = maximum moment in pipe wall under three-edge bearing test load (inch-pounds).

M_{Field} = maximum moment in pipe wall under field loads (inch-pounds).

To evaluate the proper bedding factor relationship, the vertical load on the pipe for each condition must be equal, which occurs when the springline axial thrusts for both conditions are equal. In accordance with the laws of statics and equilibrium, M_{Test} and M_{Field} are:

$$M_{Test} = (0.318 N_{fs}) * (D_i + h) \quad \textbf{(2-13)}$$

$$M_{Field} = (M_{fi}) - (0.38 * h * N_{fi}) - (0.125 * N_{fi} * c) \quad \textbf{(2-14)}$$

where,

N_{fs} = axial thrust at the springline under a three-edge bearing test load (lb/ft)

D_i = internal pipe diameter (inches)

h = pipe wall thickness (inches)

M_{fi} = moment at the invert under field loading (inch-pounds/ft)

N_{fi} = axial thrust at the invert under field loads (lb/ft)

c = thickness of concrete cover over the inner reinforcement, inches

Combining the above equations yields the following expression:

$$B_f = \frac{(0.318 N_{fs}) * (D_i + h)}{(M_{fi}) - (0.38 * h * N_{fi}) - (0.125 * N_{fi} * c)} \quad \textbf{(2-15)}$$

Using the Standard Installations program PIPECAR to calculate moments and thrusts, bedding factors were determined for a range of pipe diameters and depths of burial. These calculations were based on one inch cover over the reinforcement, a moment arm of 0.875d between the resultant tensile and compressive forces, and a reinforcement diameter of 0.075t. Evaluations indicated that for A, B and C pipe wall thicknesses, there was negligible variation in the bedding factor due to pipe wall thickness or the concrete cover, c, over the reinforcement.

Actual bedding factors vary with the size of pipe, the quality of the installation, and the width of the trench, therefore, are truly variable between the minimum values associated with a pure narrow trench installation and the maximum values associated with embankment installations. While a valid analytical approach to determine bedding factors between these two extremes is presented in the Concrete Pipe Technology Handbook[5], it is not very practical to utilize variable bedding factors in day-to-day practice.

This Standard Practice recommends to consider the method used to estimate earth load when determining which bedding factor is appropriate in indirect design. The use of variable bedding factors as indicated above should be restricted to analytical cases in instances where indirect design methods are being utilized to gain a better appreciation of actual pipe-soil interaction under unique circumstances.

In instances where the designer uses traditional Marston-Spangler Trench Loading theory to estimate earth loads, trench bedding factors should be utilized as the actual trench width is very difficult to regulate or control in the field. If Heger VAF's are utlized, however, full embankement bedding factors can be utilized as the design case of full embankment loading with embankment bedding factors will always govern over any proportional reduction in earth loading and horizontal side support. This approach is summarized in Table 3 with the recommended bedding factors for use in indirect design noted in Table 4.

[5] American Concrete Pipe Association, *"Concrete Pipe Technology Handbook – A Presentation of Historical and Current State-of-the-art Design and Installation Methodology",* ACPA, 1993, pp. 3-11

Table 3 - Type of Bedding Factor to Use versus Design Approach

Method Used to Estimate Earth Load	Bedding Factor Selection
Heger VAF's as per Table 1	Use B_{fe} for Embankment Installation and appropriate Installation Type and Diameter from Table 4
Marston-Spangler Trench Loading as per Equation (2-8)	Use B_{ft} for Trench Installation and appropriate Installation Type from Table 4

Table 4 - Bedding Factors (B_f) for Standard Trench and Embankment Installations

B_{ft} - Trench Installation				
Pipe Diameter	**Type 1**	**Type 2**	**Type 3**	**Type 4**
All	2.3	1.9	1.7	1.5
B_{fe} - Embankment Installation				
12 in (300mm)	4.40	3.20	2.50	1.70
15 in (375mm)	4.35	3.15	2.48	1.70
18 in (450mm)	4.30	3.10	2.45	1.70
21 in (525mm)	4.25	3.05	2.43	1.70
24 in (600mm)	4.20	3.00	2.40	1.70
30 in (750mm)	4.10	2.95	2.35	1.70
36 in (900mm)	4.00	2.90	2.30	1.70
42 in (1050mm)	3.97	2.88	2.28	1.70
48 in (1200mm)	3.93	2.87	2.27	1.70
54 in (1350mm)	3.90	2.85	2.25	1.70
60 in (1500mm)	3.87	2.83	2.23	1.70
66 in (1650mm)	3.83	2.82	2.22	1.70
72 in (1800mm)	3.80	2.80	2.20	1.70
144 in (3600mm)	3.60	2.80	2.20	1.70

Where embankment bedding factors are utilized on pipes larger the 1800 mm in diameter, the designer may interpolate between pipe diameters for the correct B_f.

2.2.3 Fluid Loads and Internal Pressure

The weight of fluid in a rigid pipe, W_f, generally produces bending effects that are about the same in magnitude as those caused by pipe weight (except for thrust which is tensile). Unlike pipe weight, however, fluid weight must be considered in both indirect and direct design. While the effects are small in small diameter pipe (~450 mm and smaller), they become increasing significant with increasing diameter and should be considered in design.

Fluid loads can be computed by simply calculating the weight of the fluid per unit length as per the expression:

$$W_f = \frac{\pi D_i^{\,2}}{4} * \gamma_w \qquad \textbf{(2-16)}$$

Where:

γ_w = unit weight of water (lb/ft^3)

D_i = inside diameter of the pipe

If D_i is expressed in inches and W_f is desired in units of lbs/ft, the expression becomes:

$$W_f = (0.5454 \times 10^{-2}) * \gamma_w * D_i^2 \qquad \textbf{(2-17)}$$

Gravity pipes are often designed for full flow conditions with little to no anticipated surcharge conditions. However, under conditions where significant surcharge conditions are anticipated (i.e. the hydraulic grade line is anticipated to rise above the obvert of the pipe), the pipe will be subjected to combined loading and these pressures should be considered in design.

Where internal pressure conditions are anticipated the pipe should only be designed by direct design methods as indirect design methods do not consider internal pressure as a design condition.

2.3 Live Loads

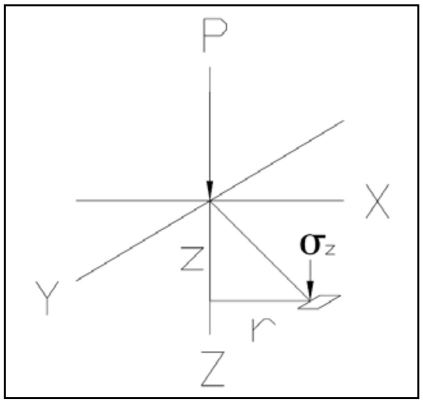
Live loads or surface loads on pipe can introduce significant loads on buried pipe and should be considered in both direct and indirect design. Surface loads can be static loads such as those due to structures or transient loads such as those introduced by concentrated wheel loads (e.g. vehicular or airplanes), the distributed loads due to train traffic, or concentrated or distributed construction traffic loads.

Surface loads are normally classified as either concentrated loads, such as wheel loads, or as uniformly distributed loads, such as those produced by tracked vehicles, rail traffic, and building foundations. While several analytical methods exist for addressing surcharge loading effects, some of which are presented below, the most predominant methods to estimate surface loads are based on a solution by Boussinesq that was developed in 1885.

2.3.1 Boussinesq Load Theory

The Boussinesq equation was developed with the assumption that a point load is applied to a working surface and is transferred through an ideally elastic, isotropic mass of material to act on a small area at depth. The distribution of stress at depth produces a bell-shaped stress distribution for any given depth z. As a rule, the effect of vertical stress will decrease with depth and horizontal distance from the origin. The general expression for the Boussinesq Equation is depicted in Figure 9.

Figure 9 - Boussinesq Equation Stress Distribution with Depth



$$\sigma_z = \beta \frac{P}{z^2}$$

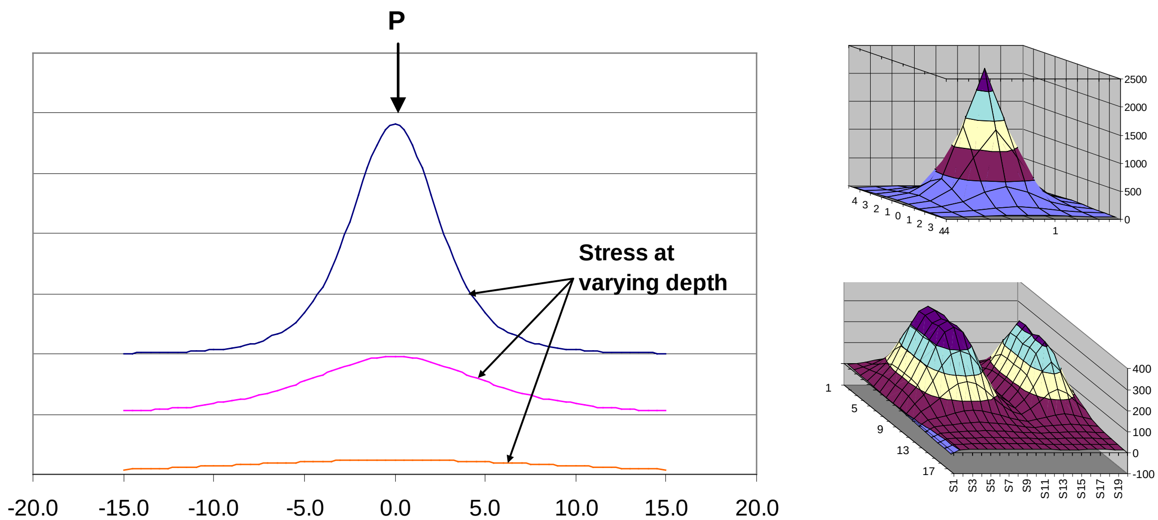
$$\beta = \frac{3}{2\pi} \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{\frac{5}{2}}}$$

Where:

- *σ_z is the vertical stress acting on a plane at depth*
- *P is the concentrated load acting at the surface*
- *r is the radial distance (horizontal) from the point of origin to the plane at depth*
- *z is the vertical distance from the plane of the origin to the plane at depth*

The Boussinesq equation can be used to determine the stresses produced by a concentrated load at the surface acting on a pipe at depth or by a distributed load at the surface acting on a discrete area with depth. In either case it is helpful to examine the effect of changes in depth and distance from the origin to gain an understanding of the influence regions as proposed by Boussinesq theory. Figure 10 is an example of two and three dimensional stress distributions for varying depth and distance from the point of origin.

Figure 10 - Effect of a Point Load Acting at Varying Depth and Distance from Origin



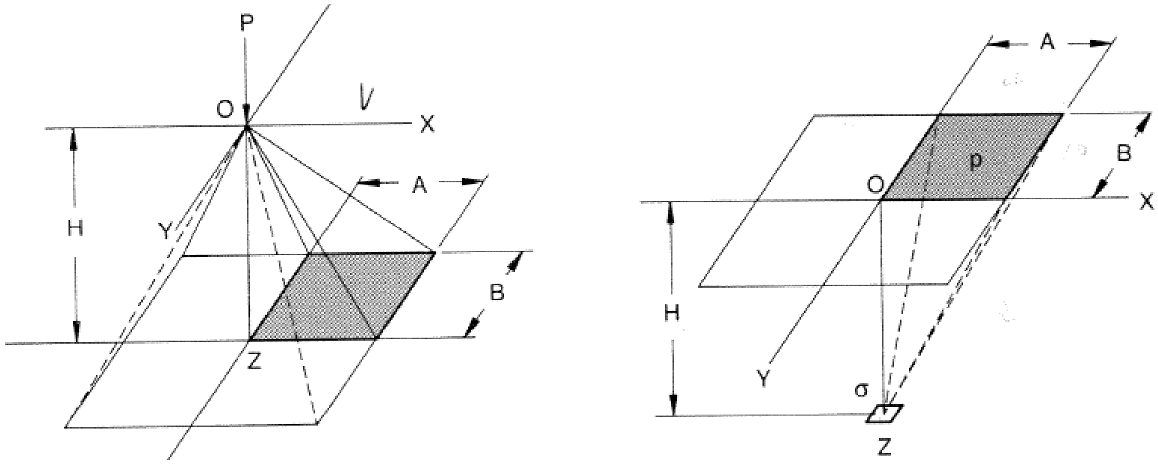
In buried pipe design, it is often necessary to analyze the effects of an external load acting over a point source and being distributing with depth over a larger area or a distributed load at the surface that has a peak value with depth at a specific point. This may take the form of a point load at the surface such as an individual wheel load, or a distributed surface load such as a footing or a tracked piece of construction equipment. Both of these situations can be handled using integrated solutions for the Boussinesq equation.

Holl's integration for instance, allows us to analyze the effect of a point load acting on a rectangular area at depth, having one corner directly below the origin.

Newmark's solution on the other hand, is an integration of the Boussinesq equation for a rectangular, uniformly distributed load resulting in a unit pressure at a point below the surface.

Figure 11 (a) shows the basic configuration for a concentrated point load acting over a rectangular area at depth. Figure 11(b) shows the basic configuration for a rectangular distributed load acting over a point at depth.

Figure 11 - Basic Geometry and Theory for Boussinesq Integrations



A. Concentrated Surface Load **B. Distributed Surface Load**

The result for Holl's Integration for a concentrated point load at the surface is:

$$\frac{\sigma}{p}=\frac{1}{4}-\frac{1}{2\pi}\left[\left(\sin^{-1}H\sqrt{\frac{A^2+B^2+H^2}{\left(A^2+H^2\right)\left(B^2+H^2\right)}}\right)-\frac{ABH}{\sqrt{A^2+B^2+H^2}}\left(\frac{1}{A^2+H^2}+\frac{1}{B^2+H^2}\right)\right]$$

The result for Newmark's Integration for a rectangular distributed surface load is:

$$\frac{\sigma}{p}=\frac{1}{4\pi}\left[\frac{2ABH\sqrt{\left(A^2+B^2+H^2\right)}}{H^2(A^2+B^2+H^2)+A^2B^2}\,\frac{A^2+B^2+2H^2}{A^2+B^2+H^2}+\left(\sin^{-1}\frac{2ABH\sqrt{\left(A^2+B^2+H^2\right)}}{H^2(A^2+B^2+H^2)+A^2B^2}\right)\right]$$

Where in each case:

- H is the vertical distance from surface to pipe crown
- A and B are dimensions of the rectangle as seen in Figure 11.

As the equations are considered cumbersome by most to use, the solutions are often reduced to the form of W_{AB} for concentrated loads and σ_{AB} for distributed loads as follows:

$$W_{AB}=C_t p \qquad \textbf{(2-18)}$$

$$\sigma_{AB}=C_t p \qquad \textbf{(2-19)}$$

where,

C_t = load coefficient dependent on the magnitude of A, B, and H

p = unit surface load, either in the form of a concentrated load for Holl's solution or in terms of and average load per unit area in the case of Newmark's solution.

Values of the load coefficient, Ct, are presented in Table 5

Table 5 - Values of Load Coefficient C_t for use in Holl's and Newmark's Integrations

	m = A/H																
n = B/H	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.5**	**2.0**	**2.5**	**3.0**	**5.0**	**10.0**
0.1	0.005	0.009	0.013	0.017	0.020	0.022	0.024	0.026	0.027	0.028	0.029	0.030	0.031	0.031	0.031	0.032	0.032
0.2	0.009	0.018	0.026	0.033	0.039	0.043	0.047	0.050	0.053	0.055	0.057	0.059	0.061	0.062	0.062	0.062	0.062
0.3	0.013	0.026	0.037	0.047	0.056	0.063	0.069	0.073	0.077	0.079	0.083	0.086	0.089	0.089	0.090	0.090	0.090
0.4	0.017	0.033	0.047	0.060	0.071	0.080	0.087	0.093	0.098	0.101	0.106	0.110	0.113	0.114	0.115	0.115	0.115
0.5	0.020	0.039	0.056	0.071	0.084	0.095	0.103	0.110	0.116	0.120	0.126	0.131	0.135	0.136	0.137	0.137	0.137
0.6	0.022	0.043	0.063	0.080	0.095	0.107	0.117	0.125	0.131	0.136	0.143	0.149	0.153	0.155	0.155	0.156	0.156
0.7	0.024	0.047	0.069	0.087	0.103	0.117	0.128	0.137	0.144	0.149	0.157	0.164	0.169	0.170	0.171	0.172	0.172
0.8	0.026	0.050	0.073	0.093	0.110	0.125	0.137	0.146	0.154	0.160	0.168	0.176	0.181	0.183	0.184	0.185	0.185
0.9	0.027	0.053	0.077	0.098	0.116	0.131	0.144	0.154	0.162	0.168	0.178	0.186	0.192	0.194	0.195	0.196	0.196
1.0	0.028	0.055	0.079	0.101	0.120	0.136	0.149	0.160	0.168	0.175	0.185	0.194	0.200	0.202	0.203	0.204	0.205
1.2	0.029	0.057	0.083	0.106	0.126	0.143	0.157	0.168	0.178	0.185	0.196	0.205	0.212	0.215	0.216	0.217	0.218
1.5	0.030	0.059	0.086	0.110	0.131	0.149	0.164	0.176	0.186	0.194	0.205	0.216	0.224	0.227	0.228	0.230	0.230
2.0	0.031	0.061	0.089	0.113	0.135	0.153	0.169	0.181	0.192	0.200	0.212	0.224	0.232	0.236	0.238	0.240	0.240
2.5	0.031	0.062	0.089	0.114	0.136	0.155	0.170	0.183	0.194	0.202	0.215	0.227	0.236	0.240	0.242	0.244	0.244
3.0	0.031	0.062	0.090	0.115	0.137	0.155	0.171	0.184	0.195	0.203	0.216	0.228	0.238	0.242	0.244	0.246	0.247
5.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.204	0.217	0.230	0.240	0.244	0.246	0.249	0.249
10.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.205	0.218	0.230	0.240	0.244	0.247	0.249	0.250

In practice loads are not always apply directly above the point of interest, but rather at some offset point or eccentricity. In cases such as these, the load can be calculated by a simple algebraic difference of applied stresses. This methodology is depicted in Figure 12 and Figure 13, for three typical loading cases for concentrated and distributed loads, respectfully.

Figure 12 - Procedure for Calculating Offset Concentrated Surface Loads

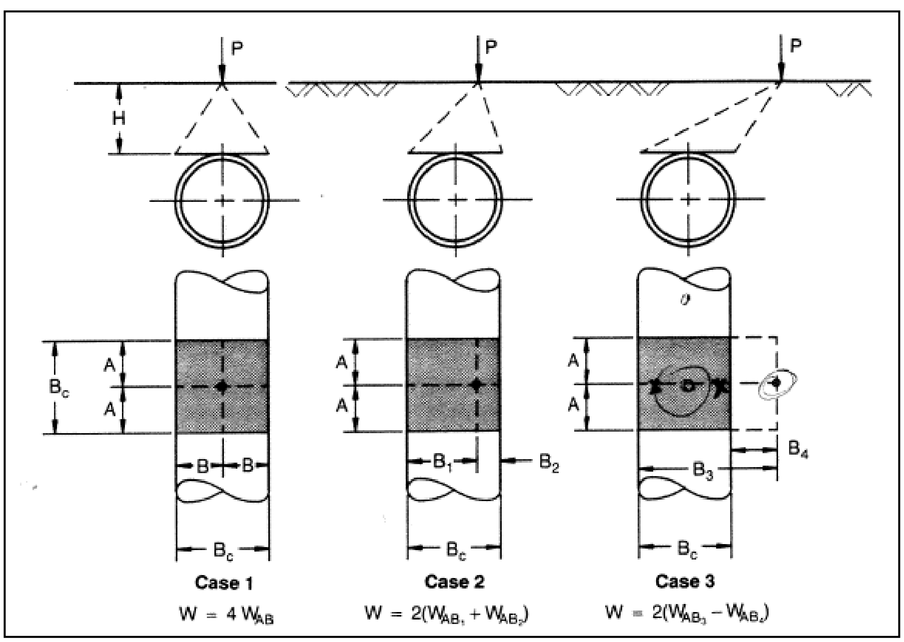
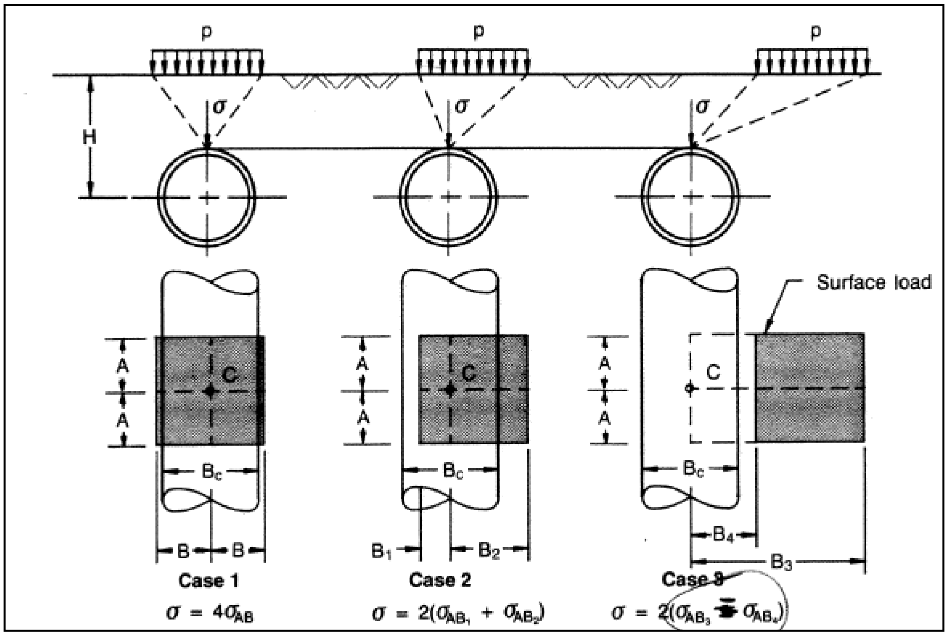


Figure 13 - Procedure for Calculating Offset Distributed Surface Loads



To express live loads in the same units as those calculated in the preceding sections for earth and fluid loads, they must be expressed in the form of load/linear length along the pipe. For concentrated live loads this would take the form of:

$$W_S = \frac{W_{AB}}{B_C} \qquad \textbf{(2-20)}$$

And the following form for distributed loads:

$$W_S = \sigma * B_C \qquad \textbf{(2-21)}$$

2.3.2 Impact Factors

Transient surface loads at shallow covers produce dynamic effects which amplify the magnitude of live loads. Shallow transient loads, therefore, should be modified by an Impact Factor, I_f, such that live loads are calculated as follows:

$$W_L = W_S(1 + I_f) \qquad \textbf{(2-22)}$$

This Standard Practice recommends ignoring the impacts of pavement bridging for standard vehicular loads and to decrease impact factors with increasing depth. AASHTO has prepared guidelines for impact factors for unpaved surfaces and these are recommended for use in this Standard Practice. Table 6 outlines recommended impact factors at varying depths of cover.

Table 6 - Recommended Impact Factors for Vehicular Loads

Impact Factor		
Cover (ft)	Cover (m)	I_f
1'-0"	0.30	0.50
2'-0"	0.61	0.50
2'-6"	0.76	0.43
3'-0"	0.91	0.38
3'-6"	1.07	0.30
4'-0"	1.22	0.23
4'-6"	1.37	0.17
5'-0"	1.52	0.10
5'-6"	1.68	0.04
5'-9" +	1.75	0.00

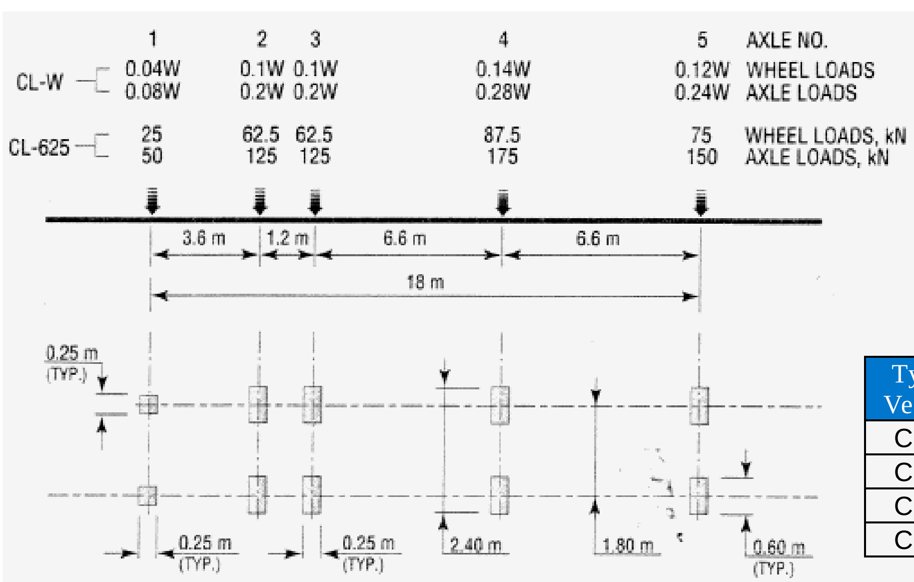
For railway loading, the American Railway Engineering and Maintenance-of-Way Association (AREMA) recommend the use of an impact factor of 40% at minimum covers of 300 mm decreasing to zero at 3 m of cover.

2.3.3 Truck and Traffic Loads – AASHTO Method

The simplified AASHTO Method can be used to estimate concentrated wheel loads for either AASHTO series vehicles or standard vehicle configurations conforming to the CL series trucks as set out in the CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC).

The CL-W series truck, for example, is a simplified five-axle vehicle for which the W indicates the total gross vehicle load in kN as set out in the CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC). A CL-625 design vehicle would therefore have a gross vehicle weight of 625kN. The load is distributed over both sets of dual tires (each 0.60m x 0.25m), at approximately 1.80m centre on centre. The per-axle load distribution for CL-W series trucks is shown in Figure 14 from the CHBDC.

Figure 14 - CL-W Truck load distribution[6]



Typical Vehicles
CL-625
CL-750
CL-800
CL-850

[6] Figure 2.5: CAN/CSA-S6-00 Canadian Highway Bridge Design Code

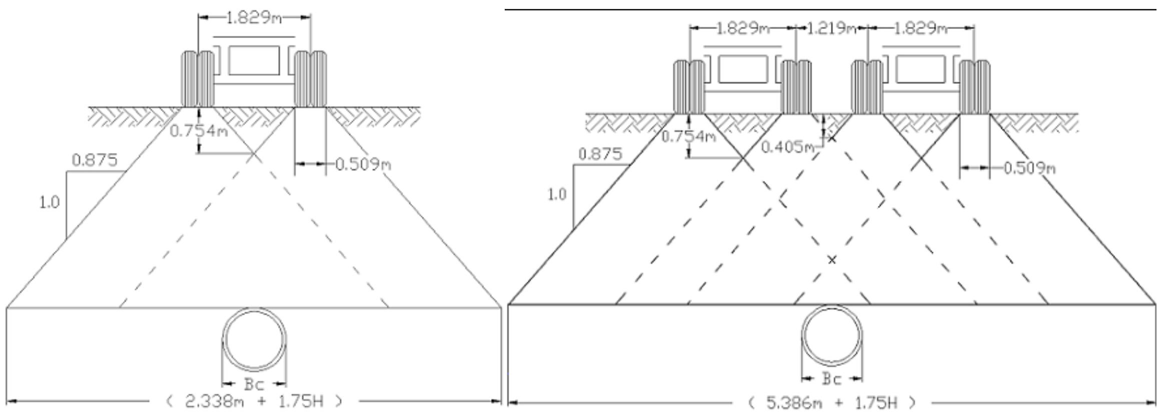
The AASHTO HS series design vehicle also represents a simplified or idealized five-axle truck. In this case however, the associated load is given for the single axle carrying the largest load. The following table lists some typical AASHTO design vehicles and their associated loads.

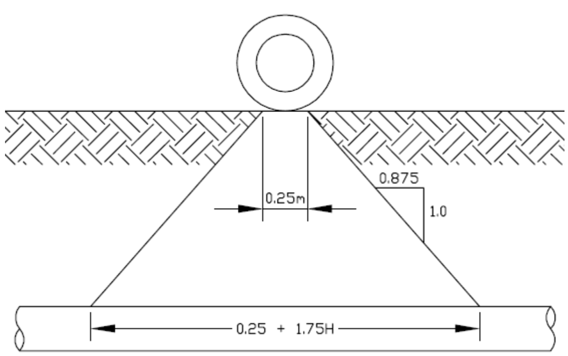
Table 7 - Typical AASHTO Design Vehicles

Design Vehicle	Single Axle (lb)	Single Axle (kg)	Single Axle Load (kN)
HS 20 (MS 18.15)	32,000	14,520	142
HS 25 (MS 22.69)	40,000	18,150	178
HS 30 (MS 27.23)	48,000	21,780	214
HSS 25 (MSS 22.95)	40,500	18,360	180
HS 20 (LRFD)	32,600	14,790	145

Under the AASHTO simplified live load method the load for a single axle is considered to be distributed over dual tires with a total contact area of 0.25m x 0.51m (10"x20") spaced at approximately 1.83m (6.0ft). The load is assumed to increase with depth in a pyramidal fashion as depicted in Figure 15.

Figure 15 - Zones of Influence and Impact Factors at Depth[7]



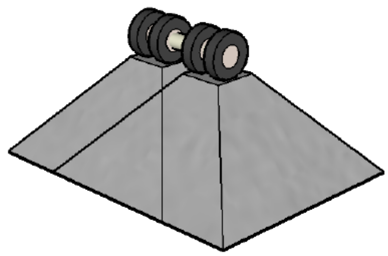


Impact Factor		
Cover (ft)	Cover (m)	I_f
1'-0"	0.30	0.50
2'-0"	0.61	0.50
2'-6"	0.76	0.43
3'-0"	0.91	0.38
3'-6"	1.07	0.30
4'-0"	1.22	0.23
4'-6"	1.37	0.17
5'-0"	1.52	0.10
5'-6"	1.68	0.04
5'-9" +	1.75	0.00

[7] Figure 2.7: Ameron Concrete Cylinder Pipe Design Manual 1988

At a depth of 0.75m (2.5ft) the influence areas overlap and the total load from both sets of tires is assumed to be evenly distributed over the entire area. Thus, for depths less than 0.75m, the single axle load can be divided by two. For depths greater than 0.75m, the pressure can be calculated as noted in Figure 16.

Figure 16 - AASHTO Method for Single Vehicle Loads



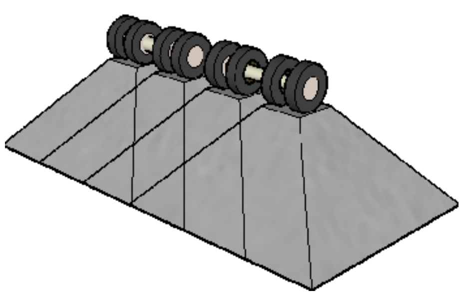
$$w_L = \frac{\text{single axle load}}{(2.34 + 1.75H)(0.25 + 1.75H)}$$ *SI units*

$$w_L = \frac{SAL}{(7.67 + 1.75H)(0.83 + 1.75H)}$$ *Imperial units*

Where H is the depth below the surface at which the load is to be estimated.

In some situations, it may be prudent to consider the effect of more than one vehicle. For calculating the live load effect of two passing trucks, refer to Figure 17

Figure 17 - AASHTO Method for Dual Passing Vehicles



$$w_L = \frac{\text{double axle load}}{(5.39 + 1.75H)(0.25 + 1.75H)}$$ *SI units*

$$w_L = \frac{DAL}{(17.67 + 1.75H)(0.83 + 1.75H)}$$ *Imperial units*

Once the pressure per unit length w_L has been determined, the total live load W_L must again be converted to pipe load units consistent with the load per unit length format identified for earth loads and include the effects of impact loads. The expression is then in the form of:

$$W_L = w_L B_C (1 + I_f) \qquad \textbf{(2-23)}$$

Minimum live loads to be covered by this Standard Practice would be based on the AASHTO method using calculated vehicular load due to a CL 800 design vehicle.

2.3.4 Cooper Series Railway Loads

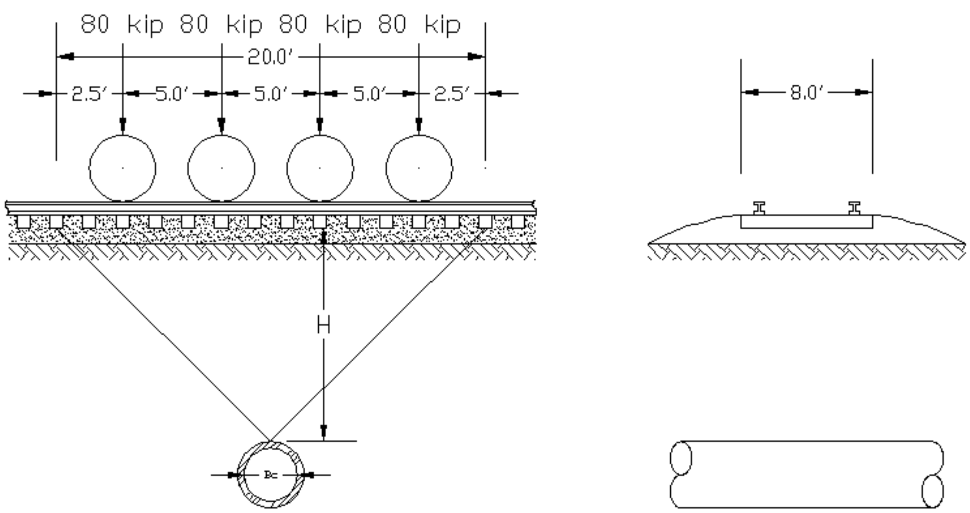
A live load due to a passing train can be calculated using a design vehicle concept set out by the American Railway Engineering and Maintenance-of-Way Association (AREMA)[8], known as Cooper Series loading. The magnitude of the loading will vary dependent on the nature of the crossing; however, a minimum Cooper E-80 loading is normally used for mainline railway crossings in Canada. The designer

[8] Chapter 8, Part 10, AREMA Manual of Railway Engineering 1999

is cautioned to check with local railway authorities, however, as more recent trends have been utilizing increasing Cooper loads with some crossings design for traffic Cooper loads up to the E-100 level.

With design vehicles or locomotives designated as Cooper E-Series vehicles, the E designation corresponds to the axle weight of the train in kips. A Cooper E-80 load, for example, would have a design axle weight of 80 kips, with 4 axles in total. The axle load is assumed to be uniformly distributed by the railway ties over an area of 20 ft long by 8 ft wide (6 m long by 2.4 m wide). Figure 18 shows the suggested axle configuration and corresponding load.

Figure 18 - Cooper E-Series Axle Spacing and Load Configuration



In addition to the axle load the tracks are assumed an applied load of 200 lb/lin ft. Total Cooper series loading, therefore, in terms of a distributed load at ground surface would be:

$$p = \frac{E*1000}{20*8} + \frac{200*20}{20*8} = 25(E+1)$$

Where

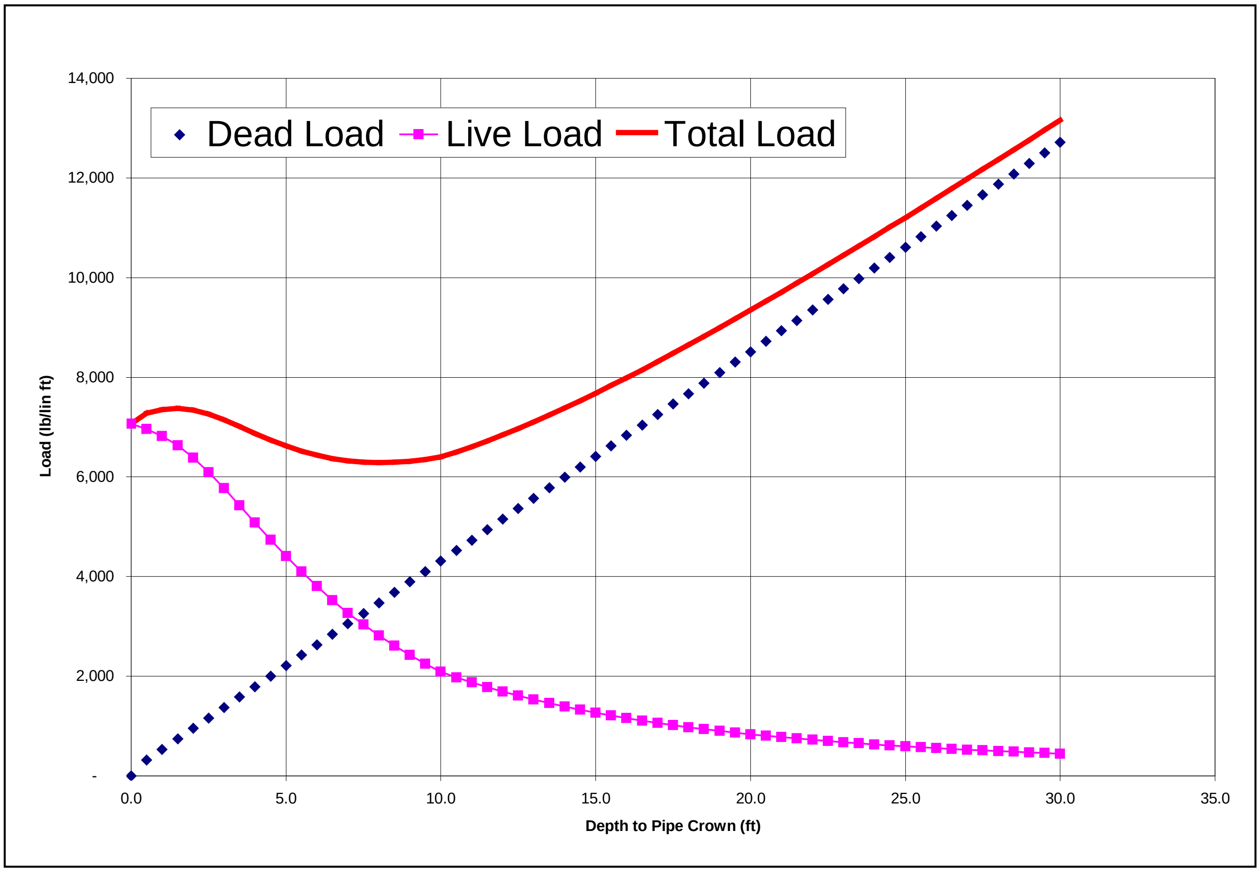
p = distributed surface load in lb/ft^2

E = Cooper series load

The load W_S acting on the pipe at depth H can then be calculated using Newmark's integration of the Boussinesq solution as described in Section 2.3.1 of this report and the Impact Factors described in Section 2.3.2.

The total contribution of the locomotive and the dead load can be seen graphically for an E80 Cooper load in the example shown in Figure 19.

Figure 19 - Typical Live and Dead Load Components with a Cooper E80 Live Load



3.0 Pipe Design

After determining the basic design criteria and the design loads and resultant pressure distribution, the remainder of the design process in terms of pipe selection can be carried out.

As indicated in Section 1.1, structural design of the pipe is completed in the following final three steps in the overall design process:

1. Select design factors

2. Perform structural analysis

3. Design the pipe

While there are numerous similarities in terms of determining relevant basic design requirements and assessing design loads and pressure distributions, the structural design procedures employed using direct and indirect methods are markedly different.

Even from a process perspective, indirect design usually has a designer ultimately selecting an appropriate pipe strength based on a specified installation condition, while in direct design the designer of record typically specifies a range of design criteria to be utilized and a range of acceptable installation types, and reviews the Shop Drawing design submission of a contractor or subcontractor (usually a pipe manufacturer) to check for conformance to the specified requirements and the requirements of a prescriptive Standard Practice.

The primary purpose of the conventional designer in becoming well versed in direct design is typically to facilitate an educated review in the Shop Drawing process as well as increasing one's understanding of the true economies that can be achieved in design by gaining a more thorough understanding of all of the factors that impact structural requirements for reinforced concrete pipe design.

3.1 Direct Design – Overview of Limit States Design Factors and Structural Design Process

Direct design was developed as a Standard Practice under ASCE Standard Practice 15. The most current version of the Standard Practice at the time of this Standard Practice development was ASCE 15-98[9].

The ASCE Standard Installation Direct Design (SIDD) Standard Practice was developed to ensure that all possible modes of failure were evaluated for concrete pipe and to assure that appropriate factors of safety were attached to each aspect of the design process in proportion to the level of uncertainty associated with that aspect of the design process. This is known as the limit states design method. SIDD designs use limit states design methods to evaluate reinforcing steel requirements for:

1. Service cracking based on the degree of crack control desired,

2. Ultimate flexural load

3. Limiting conditions for concrete radial tension strength

[9] ASCE, *"ASCE 15-98, Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)"*, American Society of Civil Engineers, 1998

4. Limiting conditions for shear (diagonal tension)

The latter two checks are not carried out in indirect design yet are common governing conditions in the intermediate to larger diameter range when direct design is carried out. Further, as bedding and load distribution around the pipe is better distributed to minimize overall steel requirements they also become more critical limiting conditions to assess.

The overall SIDD design procedure involves structural design to provide:

- Minimum ultimate strength equal to the strength required for expected service loading multiplied by a load factor.

- Control of crack width at the expected service load to maintain suitable protection of reinforcement from corrosion, and to limit infiltration or exfiltration of fluids.

In addition, provisions are incorporated to account for the potential reduction of nominal strength and crack control because of variations from nominal design dimensions and strength properties.

As opposed to the single factor of safety utilized in indirect design, direct design uses individual load factors for strength design that are multipliers of the governing moments, thrusts, and shears to account for variations in load and their effects in actual installation from those calculated using the design assumptions and to provide a margin of safety against structural failure. The following load factors are required to be used based on the ASCE Standard Manual of Practice and minimum required load factors recommended for use in the City of Calgary:

- Dead and earth load - shear and moment 1.3

- Dead and earth load - compressive thrust
  - Tension reinforcement 1.0
  - Concrete compression 1.3

- Live load - shear and moment - single truck 2.17
  - thrust - single truck 1.3
  - shear and moment - multiple trucks 1.3
  - thrusts - multiple trucks 1.0

- Internal pressure - tensile thrust 1.5

Strength reduction factors are applied to account for variations in material properties that occur as a result of their manufacture or due to the fabrication of the pipe. These are applied as multipliers of the parameters that define the strength of the pipe. The ASCE Standard Manual of Practice recommends the following strength reduction factors:

- Reinforcement: tensile yield strength 0.95
- Concrete: shear and radial tension 0.90

Crack control factors can be applied if specific application requirements are more stringent than 0.01". For normal gravity applications, a service crack width factor of F_{cr} = 1.0 is adequate.

Where non-circular steel arrangements are selected, a minimum cage misorientation factor of $\theta = 10^{o}$ should be utilized. Similarly there are provisions to increase or decrease process factors based on a manufacturer's substantiated ability to deliver increased performance in radial or diagonal tension. Under this Standard Practice, process factors for both radial and diagonal tension shall be 1.0.

Structural design of the pipe using the ASCE Standard Practice is then carried out in the following manner:

1. The amount of reinforcement required near the inner and outer pipe faces of the pipe wall is determined, based on the tensile yield strength limit state. For most circular pipe the inner reinforcement area is usually governed by the combined factored moment and thrust that act at the invert. The outer reinforcement is usually governed by the combined factored moment and thrust near the springline.

2. A check is carried out to determine if the maximum factored moments that cause tension at the inside face (at the invert and crown), combined with the associated thrusts at those locations, cause radial tension stresses that exceed the radial tension strength limit.

3. A check is carried out to determine if the maximum factored moments at the crown, invert, or springline, combined with the associated thrust at those locations, cause compressive strains that exceed the appropriate limits.

4. A check is carried out at critical wall sections to determine if the critical shear force exceeds the shear (diagonal tension) strength limit. This is a critical check in larger diameter pipelines.

5. If any of the strength limits are exceeded the design is modified accordingly.

6. A check is then carried out to determine if the service load moments at the crown, invert, or springline, combined with the associated thrusts, cause reinforcement stresses that exceed the service load limit for crack width control. The reinforcement area that is required for flexural tension strength (or the increased area when required for shear) must be sufficient to provide the desired degree of crack control.

While the designer can use hand computations based on the formulae developed and prepared for the ASCE Standard Practice, it is assumed that direct design is typically carried out using the software design package developed by Simpson, Gumpertz, & Heger to evaluate Standard Installations known as Pipecar[10].

3.1.1 Direct Design – Reasonable Assumptions for Initial Design Parameters

The direct design process requires the designer to make a series of assumptions relative to initial pipe design parameters such as wall thickness, concrete strength, thickness of cover over reinforcement, steel arrangement, type and strength of reinforcement. While all of these parameters can have significant variance dependent on the manufacturer of the pipe, there are both practical and reasonable considerations that should be accepted and understood by the local design community. A brief discussion follows for each of the initial pipe design parameters.

3.1.1.1 Wall Thickness

As noted in Section 2.1 reinforced concrete pipe is typically manufactured in one of three standard wall thickness configurations, Wall A, B, or C. Of the manufacturers that most commonly supply the Calgary market most diameter ranges are normally supplied with only a single standard Wall thickness configuration in each diameter range and typically in either a “B” Wall or “C” Wall configuration. The exact configuration carried can be ascertained by applying the standard dimensional formulae noted in Section 2.1 and reviewing each manufacturer’s catalogue.

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[10] Simpson, Gumpertz, & Heger, Inc., *“Pipecar, A Computer Program for the Analysis and Design of Circular and Reinforced Concrete Pipe”,* Version 3.07, October 2001

The designer is encouraged to examine the impact of varying wall thickness configurations on design (not to actually modify them but to understand the sensitivity of design to the different manufacturers standard wall thickness sections), particularly for designs based on “A” or “B” Wall thicknesses, as these design’s more commonly encounter limiting conditions where the wall thickness is inadequate to overcome compression and the use of a thicker wall will be required to meet some design conditions in lower classes of Standard Installations (i.e. higher Installation Type numbers).

3.1.1.2 Concrete Strength

Concrete strengths is usually specified as the standard 28-day compressive strength as defined in ASTM C39/C39M-05e1 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.

Typical design practice locally is to use strengths between f_c' = 4,000 psi (28 MPa) and f_c' = 5,000 psi (35 MPa). Higher strengths can be readily be obtained but the designer is cautioned to pursue evidence of the manufacturer to consistently deliver the required design strength in accordance with Appendix A, Clause A.7.2.3 of the ASCE Standard Practice and the time period that the pipes are actually being installed in. While modern precast manufacturing processed can readily achieve much higher 28 day strengths than the above typical design values, larger diameter pipe often has a much tighter time frame between manufacture and installation and the designer should be cognizant of this in their selection of an appropriate design value.

The maximum strength that can be used in the ASCE Standard practice is limited to f_c' = 7000 psi (48 MPa). This is because the experimental basis for some of the semi-empirical design procedures has never been verified on pipes with strengths in excess of this value.

3.1.1.3 Thickness of Cover over Reinforcing Steel

Most designs are based on a minimum of 25 mm of cover over the reinforcing steel for corrosion protection and are not that sensitive to reinforcement cover beyond that.

The designer should be cognizant of steel placement in designs where service cracking governs in design, as the baseline for service cracking control, F_{cr} = 1.0, is 0.01 inch cracking measured at a point 1 inch (25 mm) beyond the inner or outer reinforcement. In pipe designed to have 1 inch (25 mm) of cover, this corresponds to the inner or outer surface, however, if the pipe is designed (or built) with greater cover, the crack at the surface would be greater than the 0.01 inch criterion.

3.1.1.4 Steel Arrangement and Reinforcing Type

Most precast reinforced concrete pipe products are manufactured using closely spaced wire reinforcement in the form of welded wire fabric (either supplied as a product or wrapped on a cage making machine in the pipe fabricating plant). Local manufacturers in Calgary have cage making machines and currently use closely spaced welded wire fabric either smooth or in a deformed form (Type 2 or 3 below).

As a designer previewing designs with *Pipecar*, consult your local manufacturer to determine what standard practice is for them, in terms of steel selection for inventory and what practical limitations they have in their manufacturing processes.

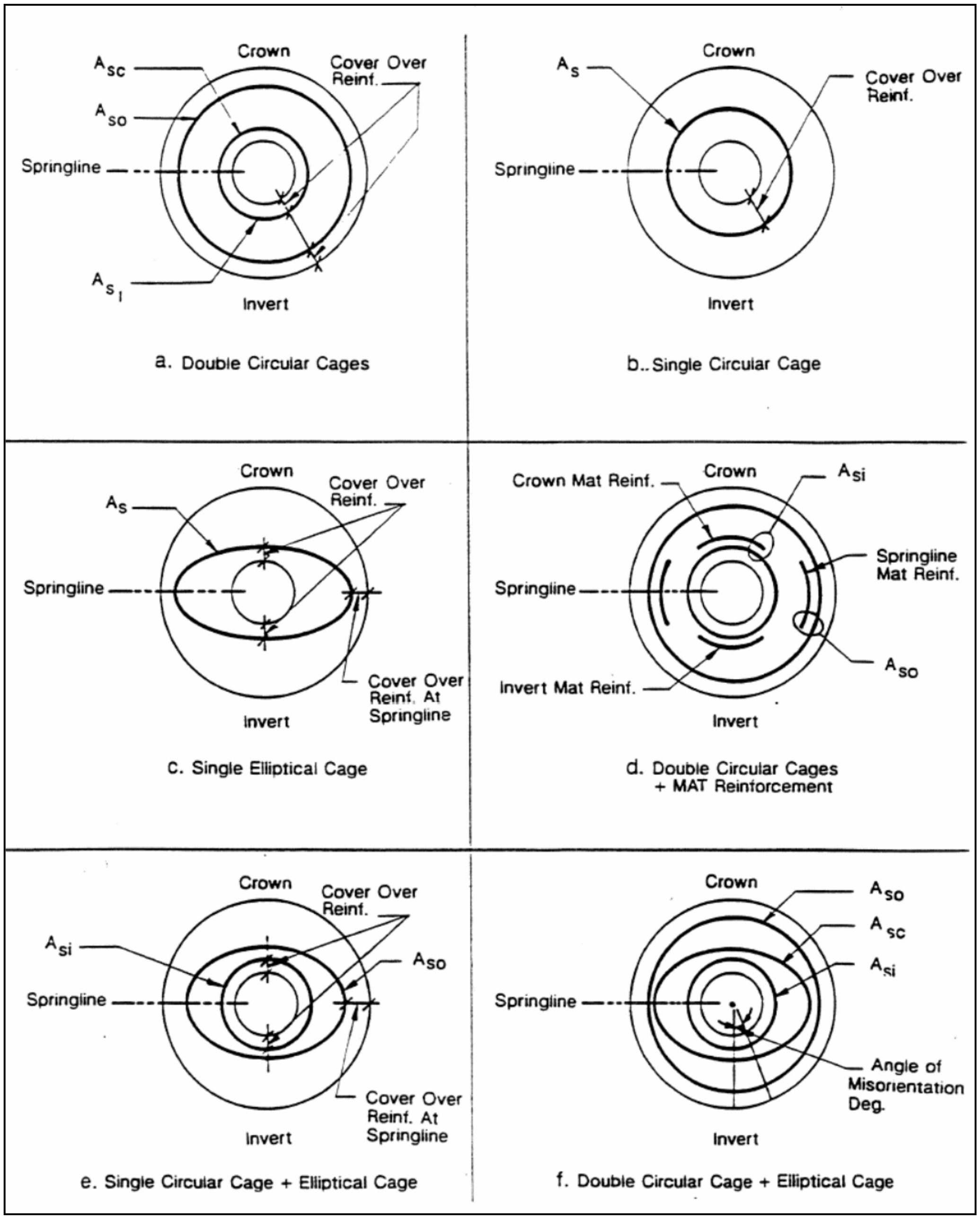
Reinforcement types are classified in the design procedure for crack width control in ascending order in terms of their bonding qualities as follows:

Type 1 – smooth wire or bars, or smooth welded wire fabric with cross wire spacing in excess of 8 inches (200 mm).

Type 2 – welded smooth wire fabric with cross wire spacing of 8 inches (200 mm) or less.

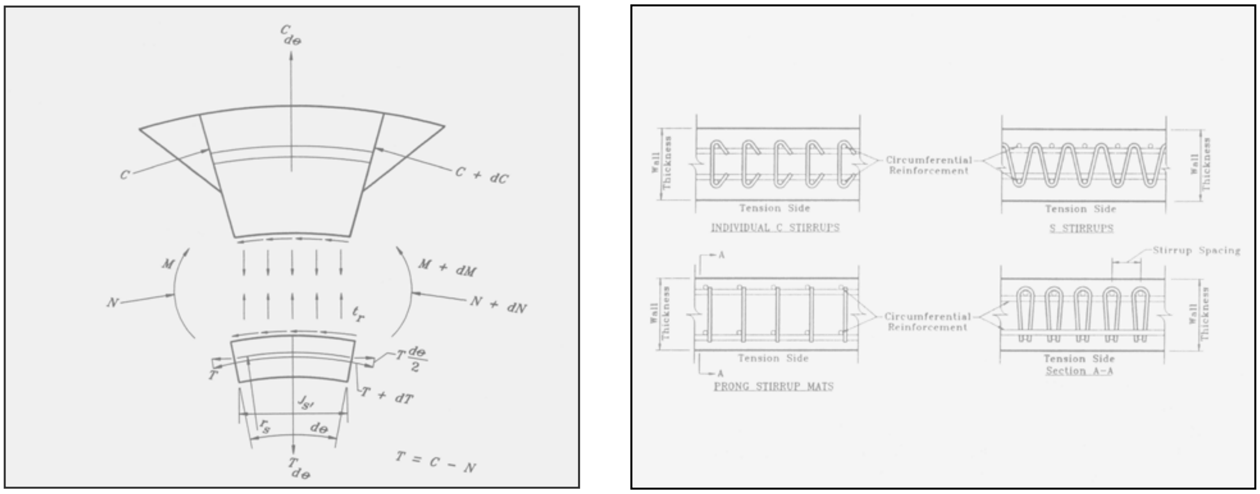
Type 3 - cold drawn deformed wire, or welded deformed wire fabric, or deformed steel mild steel bars

Figure 20 - Typical Reinforcing Steel Arrangements



One of the primary reasons to carry out a preliminary screening of design checks is to examine whether any unusual reinforcing arrangements are required that may require special considerations in handling or in manufacture. A variety of reinforcing schemes are depicted in Figure 20 while Figure 21 depicts a unique reinforcing scheme required to overcome excessive radial or diagonal tension.

Figure 21 - Stirrup Requirements and Arrangements



The vast majority of designs can be accomplished with the use of steel arrangements a.) or b.) from Figure 20 (double or single circular cages). If so, no special precautions are required to be undertaken to transfer the design to construction. All other reinforcing schemes including all reinforcing schemes involving stirrups require that the pipe be installed in a specific orientation and, therefore, would pose specific handling concerns in the field that should be brought to the contractor's and field inspection personnel's attention.

3.1.1.5 Strength of Steel Reinforcement

The strength of steel reinforcement typically has a marked impact on overall design and design values should be based on demonstrated long term performance and consistency in supply.

Based on current steel supply to the local market place it is reasonable to be utilizing a design value of steel yield strength of 65 ksi (448 MPa).

Higher values may be utilized when using *Pipecar* for analytical purposes (e.g. when trying to assess a definitive limit state, for example or to better quantify risk) based on more detailed assessment of strength, however, the current maximum limit recommended for design purposes is 65 ksi (448 MPa).

3.1.2 Direct Design - Designing the Pipe

As noted earlier, the primary role of the conventional designer in the direct design process is more of a screening role and a higher level review of economics by carrying out reviews to examine the overall benefits of upgrading embedment support on reducing structural requirements for the pipe, especially in instances where it eliminates the need for unusual or more complex reinforcing requirements.

Many screening reviews will highlight the subtleties and limitations of different manufacturer's use of fixed wall thickness configurations, particularly thinner wall configurations, when trying to meet extreme loading cases.

Appendix B of this Standard Practice provides an overview of the *Pipecar* input screens with guidance on user input requirements and the fixed range of design assumptions that are either limited by the ASCE Standard Practice or recommended for use in the City of Calgary, based on this Standard Practice.

3.2 Indirect Design

In Section 2.0 of this Manual, guidance was provided on the first two steps in the design process, the selection of basic design criteria and the determination of design loads and resulting pressure distribution around the pipe. This section will focus on the last three aspects of the overall design process; the selection of design factors, structural analysis, and the design of the pipe.

3.2.1 Indirect Design – Design Factors

Unlike the limit states approach of direct design, indirect design utilizes a single factor of safety approach to account for all uncertainty that exists in the design/installation process.

Standard practice in the application of indirect design in North America has been to design to allow service cracking to occur and to define the factor of safety as the relationship between ultimate strength in a D_{LOAD} three-edge bearing strength test and the 0.01 inch crack D_{LOAD}. Specifically, the following factors of safety are required by both ASTM C76-05b Standard Specification for Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe and ASTM C655-04e1 Standard Specification for Reinforced Concrete D-Load Culvert, Storm Drain, and Sewer Pipe (note only ASTM C76 indirect design is permitted in the City of Calgary):

- For $D_{0.01}$ loads of 2000 lb/ft/ft of diameter or less FS = 1.5
- For $D_{0.01}$ loads > 2000 lb/ft/ft of diameter and < 3000 lb/ft/ft of diameter FS = a linear reduction from 1.5 to 1.25
- For $D_{0.01}$ loads of 3000 lb/ft/ft of diameter or more FS = 1.25

For ASTM C76 pipe, this reasonably assures the designer of the following relationships:

1. Class I Pipe
    - $D_{0.01}$= 800 lbf/lin ft/ft diameter
    - D_U= 1200 lbf/lin ft/ft diameter
2. Class II Pipe
    - $D_{0.01}$= 1000 lbf/lin ft/ft diameter
    - D_U= 1500 lbf/lin ft/ft diameter
3. Class III Pipe
    - $D_{0.01}$= 1350 lbf/lin ft/ft diameter
    - D_U= 2000 lbf/lin ft/ft diameter
4. Class IV Pipe
    - $D_{0.01}$= 2000 lbf/lin ft/ft diameter
    - D_U= 3000 lbf/lin ft/ft diameter

5. Class V Pipe

   - $D_{0.01}$= 3000 lbf/lin ft/ft diameter
   - D_U= 3750 lbf/lin ft/ft diameter

The designer is cautioned to understand these relationships, evaluate them on a case by case basis dependent on the degree of contractual controls in place to ensure that loading and pipe support objectives will be met, the consequences of failure, and acceptability of the service cracking criterion for the intended application (e.g. some higher risk wastewater applications, may warrant more stringent crack control) and adjust factors of safety accordingly. The above factors of safety are the minimum permitted under this Standard Practice.

Where non-reinforced concrete pipe conforming to ASTM C14-05a Standard Specification for Non-reinforced Concrete Sewer, Storm Drain, and Culvert Pipe is utilized there is obviously no protection between service cracking and ultimate load even though the pipe will continue to function in typical pipe soil interaction applications. In using non-reinforced concrete pipe a minimum FS of 1.5 is recommended on the load required to produce 0.01 cracking.

3.2.2 Indirect Design – Structural Analysis and Design of the Pipe

In indirect design the process of structural analysis and design of the pipe is a seamless and simple one. Design is based on:

1. Acquisition of basic design criteria (in terms of pipe size, etc.)

2. Calculation of design loads and pressure response in terms of W_e, W_L, W_f, and B_f.

3. Rationalizing an appropriate Factor of Safety

Structural analysis and pipe selection then consist of determining the required strength of the pipe in a three-edge bearing test (TEB) as per the following expression:

$$TEB = \frac{(W_e + W_L + W_f)}{B_f} * FS \qquad \textbf{(3-1)}$$

If service cracking can be tolerated (and 0.01 inch cracking is acceptable performance in most applications) then the FS = 1.0 in the above formula for reinforced pipe and 1.5 for non-reinforced pipe applies. Where more stringent criteria need to be applied to the service cracking criterion based on the designer's assessment of risk, uncertainty or the intended application; an increased FS should be applied.

Applied in the above manner the above pipe selection method yields factors between service cracking and ultimate failure varying from 1.5 to 1.25 dependent on the strength class selected as noted in Section 3.2.1. Again based on the designer's assessment of risk, uncertainty or the intended application; an increased FS could be applied.

In any event designers would be encouraged to evaluate pipe performance utilizing direct design methods to ascertain the governing modes of failure under the intended application. A limiting feature of indirect design as previously noted is its focus entirely on service cracking and its relationship to ultimate flexural load. While these are typically valid governing failure modes for smaller diameter pipe (typically 450 mm and smaller), they are often not the governing failure mode on intermediate to larger diameter pipe. In these instances the designer would be well advised to utilize direct design methods to reasonably ensure that design life objectives are truly achieved.

A sample problem applying the indirect design method is contained in Appendix C for the practitioner's convenience.

Appendix A
Notations for Indirect and Direct Design

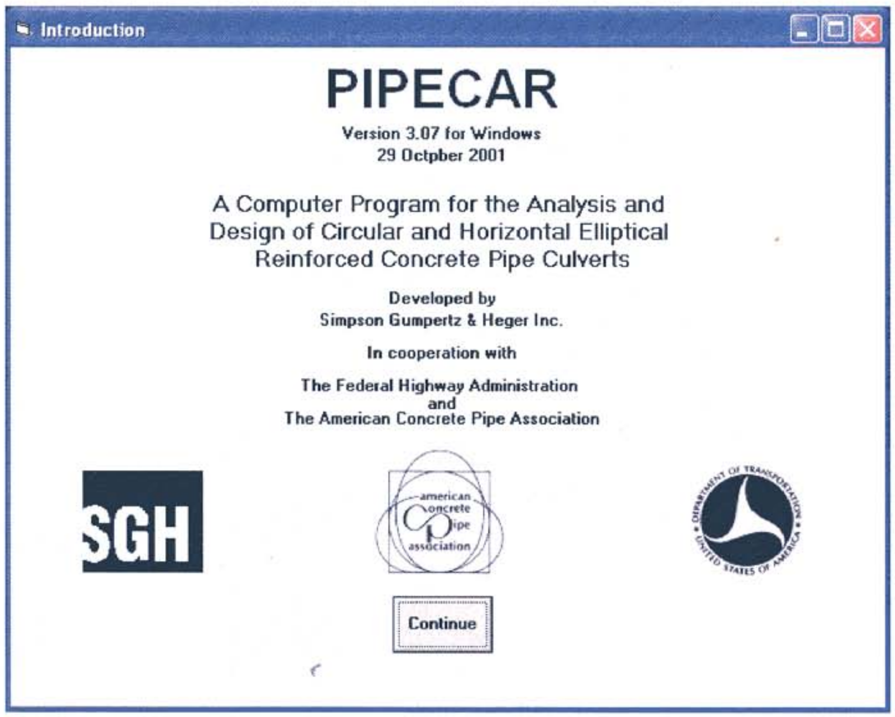
Symbol	Definition	Units used in this Standard Practice
μ'	coefficient of friction for trench backfill against sides of trench)	
B_c	outside diameter of pipe	feet
B_d	width of trench at top of pipe	feet
B_f	bedding factor	
B_{fe}	bedding factor – true embankment conditions	
B_{ft}	bedding factor – narrow trench condition	
B_t	diameter of tunneled hole	feet
C_c	coefficient for calculating Marston earth load in positive projecting embankments	
C_d	coefficient for calculating Marston earth load in trenches	lbs/foot
$D_{0.01}$	0.01 inch crack load (D-load)	lbs/ft/ft of diameter
D_i	inside diameter of pipe	inches
D_o	outside diameter of pipe	inches
D_u	ultimate D-load	lbs/ft/ft of diameter
F_{cr}	crack width control factor for adjusting crack control relative to average maximum crack width of 0.01 inch at 1 inch from the tension reinforcement when F_{cr} = 1.0	
FS,FOS	factor of safety	
h	wall thickness	inches
H	design height of earth above top of pipe	feet
HAF	horizontal arching factor	defined by Equation 2-6
I_f	impact factor	
K	ratio of lateral to vertical pressure (Rankine earth pressure coefficient)	
M_{Field}	maximum moment in pipe wall under field loads	inch-lbs
M_{Test}	maximum moment in pipe wall under three-edge bearing test load	inch-lbs
N	coefficient to determine bedding factor that varies with bedding type	
p	projection ratio (ratio of distance between natural ground and top of pipe to outside diameter of pipe	
p'	negative projection ratio (ratio of height of natural ground above top of pipe to outside diameter of pipe	
PL	prism load (weight of the column of earth over the outside diameter of the pipe)	lbs/foot
q	coefficient to determine bedding factor that varies with Rankine pressure coefficient	

Symbol	Definition	Units used in this Standard Practice
r_{sd}	settlement ratio – ratio of the difference between the settlement of the soil adjacent to the pipe and the top of the pipe	
VAF	vertical arching factor	defined by Equation 2-5
w	unit weight of soil	lbs/ft^3
W_{AB}	live load due to a concentrated surface load per unit area (no impact)	lbs/ft^2
W_e	vertical earth load on pipe	lbs/foot
W_f	weight of fluid in the pipe	lbs/foot
W_h	horizontal (lateral load on pipe)	lbs/foot
W_L	live load with impact	lbs/foot
w_L	live load per unit area due to a concentrated surface load - AASHTO method	lbs/ft^2
W_p	weight of the pipe	lbs/foot
W_S	live load without impact	lbs/foot
x	coefficient to determine bedding factor that varies with the projection ratio	
σ	live load due to a distributed surface load per unit area (no impact)	lbs/ft^2

Appendix B
Direct Design Sample Application of Pipecar and Recommended Ranges of Input Values

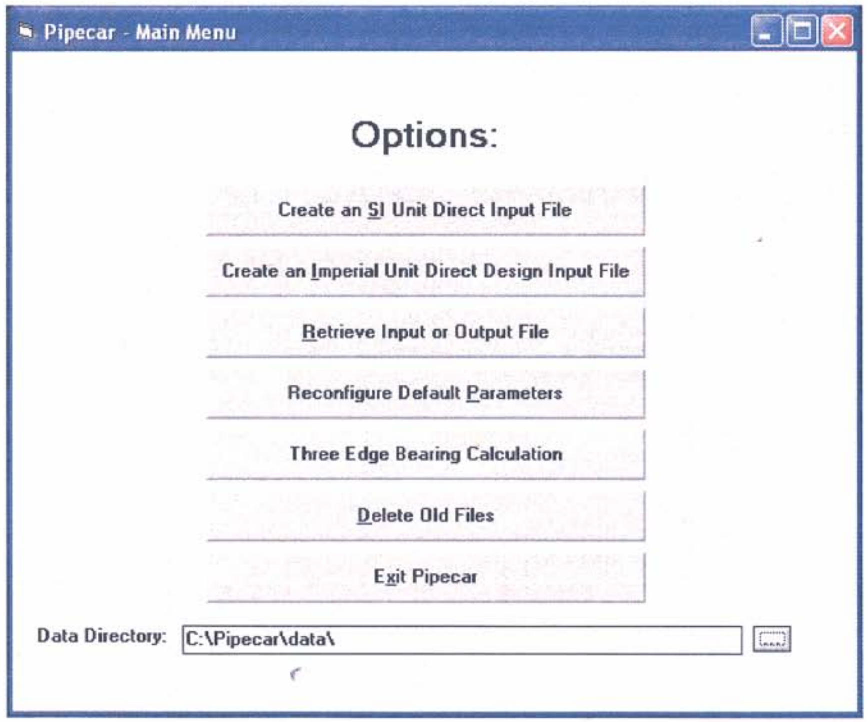
Design Aids – PIPECAR™

Intro screen



PIPECAR™

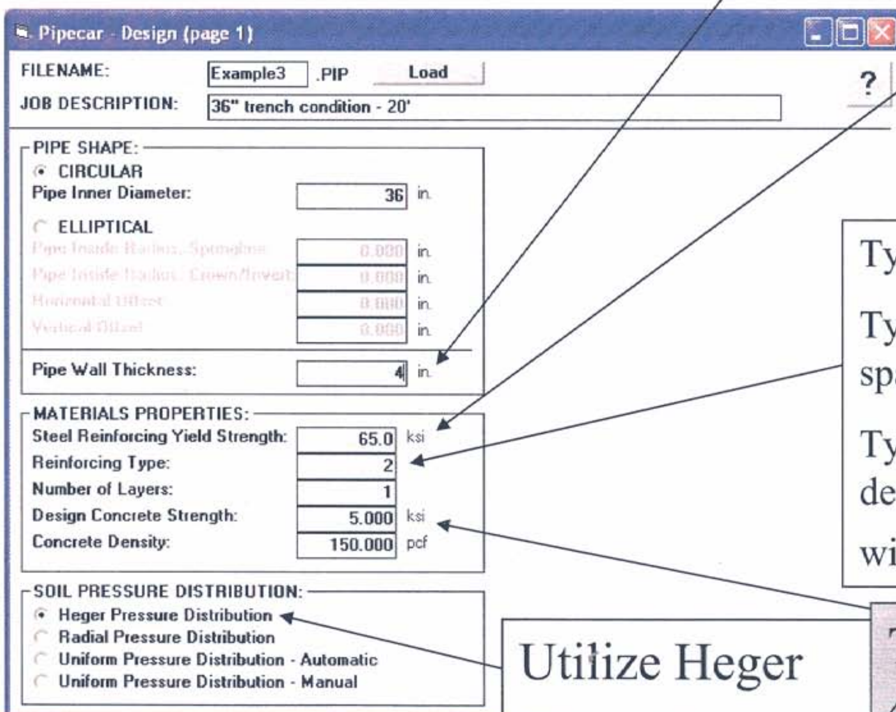
Main Menu



On following Input pages,

- Note:
  - shaded areas require designer's judgment or design specifics
  - Remaining values dictated by Calgary and ASCE Standard Practice

PIPECAR™
Page 1 Input



"A" Wall = D_i/12

"B" Wall = D_i/12 +1

"C" Wall = D_i/12 + 1.75

Max Reinforcing Yield Strength for

Design = 65.0 ksi

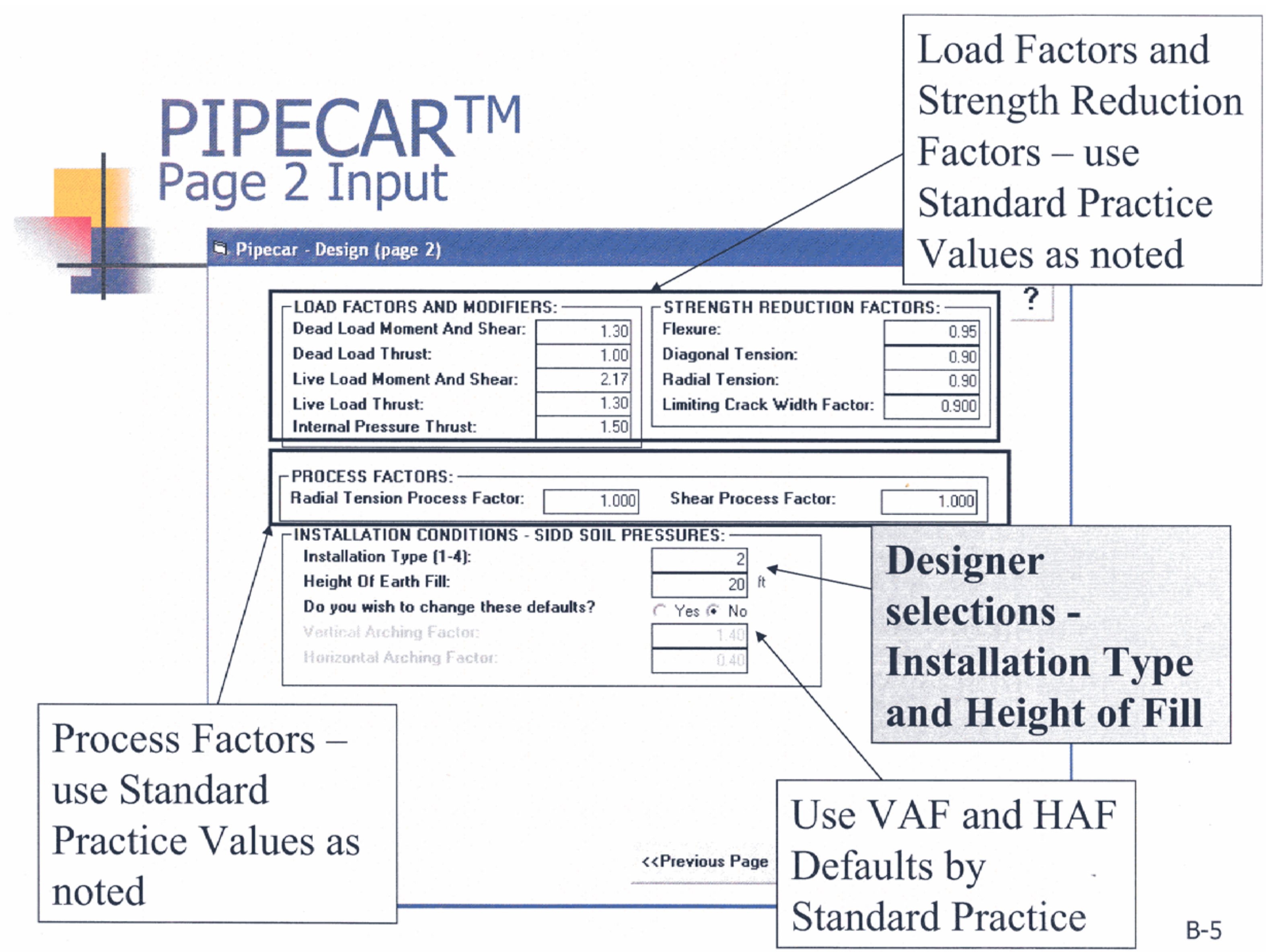
Type 1 – Smooth wire > 8" spacing

Type 2 – Smooth WWF < 8" spacing

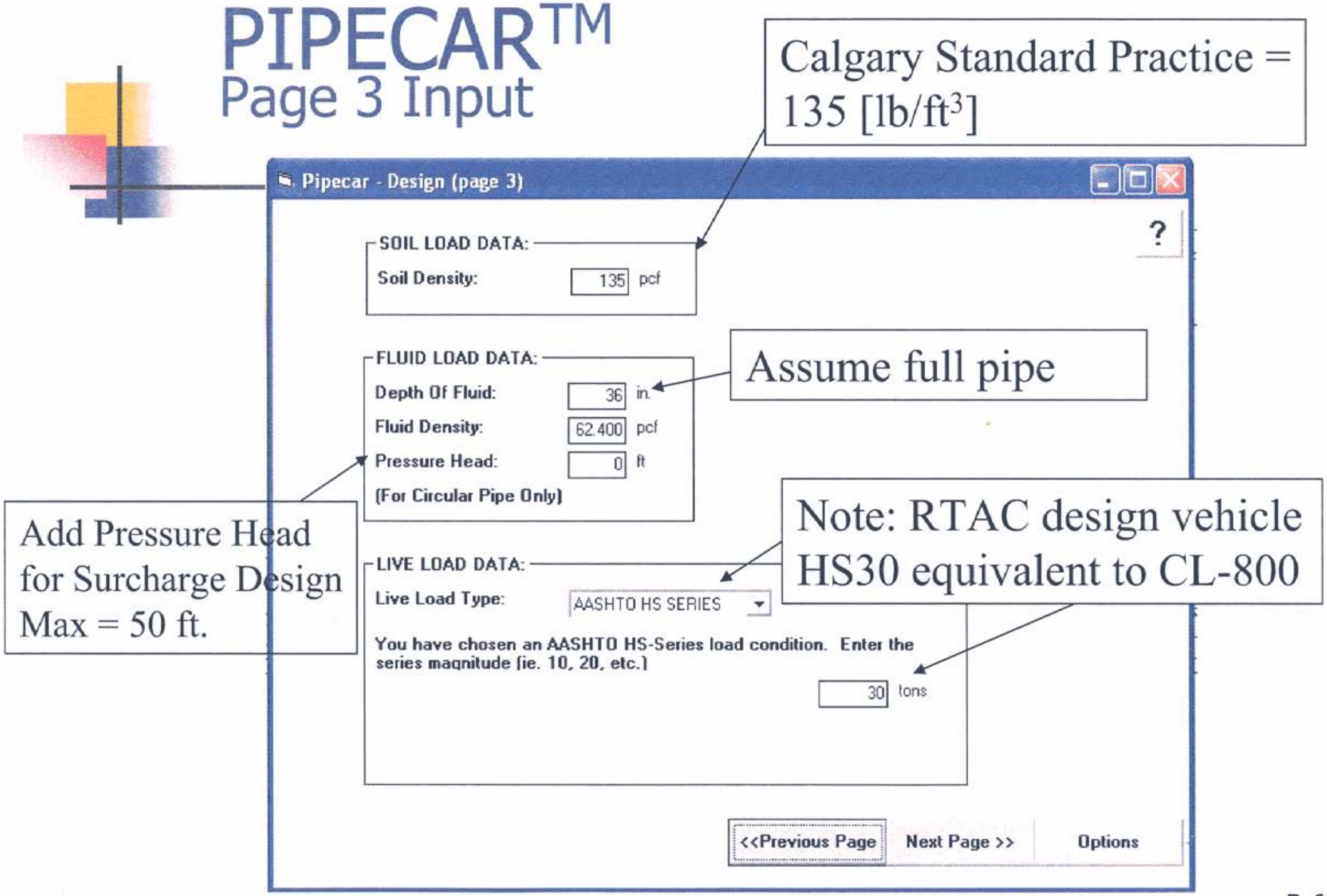
Type 3 – Deformed WWF, deformed bars or any reinforcing with stirrups

Typical Concrete Strength 4-5 ksi, max 7 ksi

Utilize Heger pressure distribution


PIPECAR™
Page 2 Input
Pipecar - Design (page 2)
LOAD FACTORS AND MODIFIERS:
Dead Load Moment And Shear: 1.30
Dead Load Thrust: 1.00
Live Load Moment And Shear: 2.17
Live Load Thrust: 1.30
Internal Pressure Thrust: 1.50
STRENGTH REDUCTION FACTORS:
Flexure: 0.95
Diagonal Tension: 0.90
Radial Tension: 0.90
Limiting Crack Width Factor: 0.900
PROCESS FACTORS:
Radial Tension Process Factor: 1.000
Shear Process Factor: 1.000
INSTALLATION CONDITIONS - SIDD SOIL PRESSURES:
Installation Type (1-4): 2
Height Of Earth Fill: 20 ft
Do you wish to change these defaults? Yes No
Vertical Arching Factor: 1.40
Horizontal Arching Factor: 0.40
<<Previous Page
Load Factors and Strength Reduction Factors – use Standard Practice Values as noted
Process Factors – use Standard Practice Values as noted
Designer selections - Installation Type and Height of Fill
Use VAF and HAF Defaults by Standard Practice

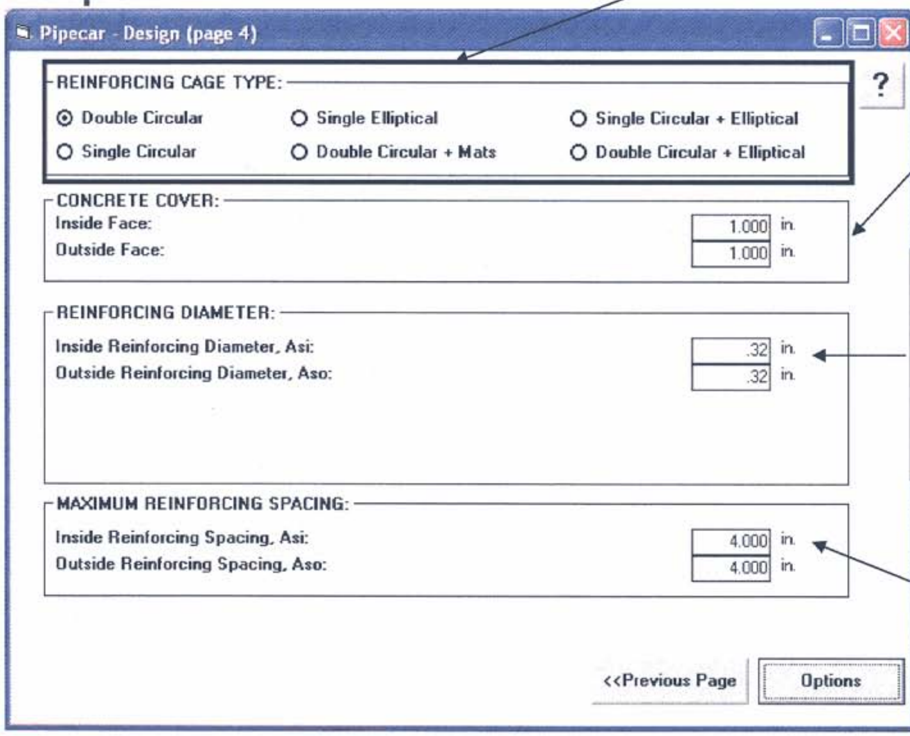
PIPECAR™
Page 3 Input



PIPECAR™
Page 4 Input

Designer Selection

Minimum 1 inch cover by Standard Practice

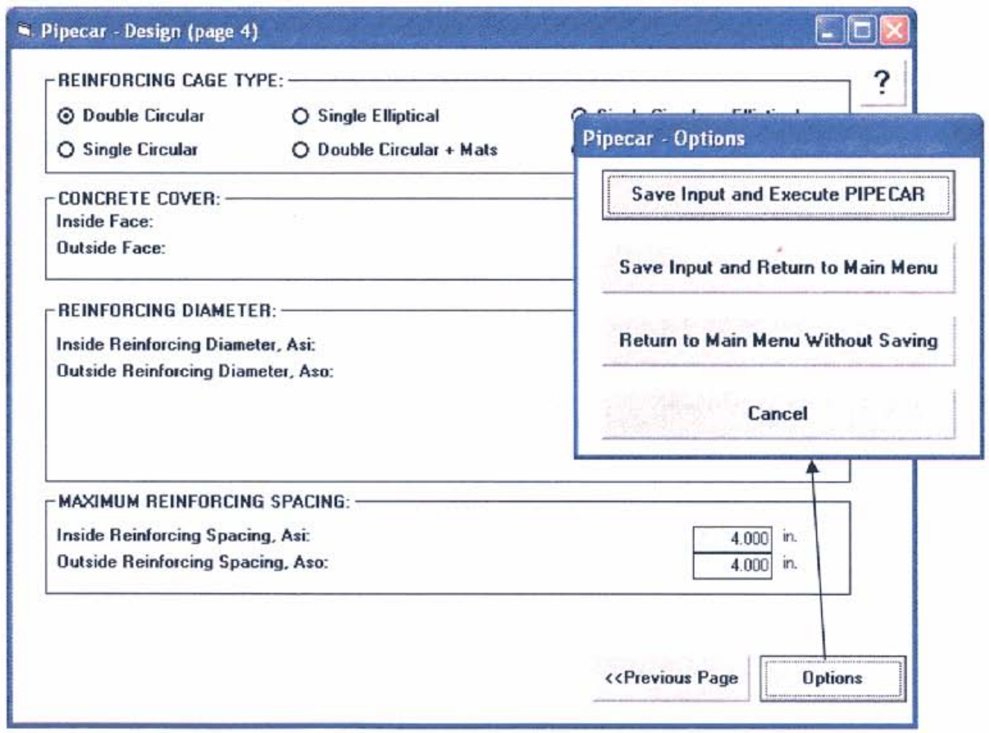


Only used to calculate moment angles, not sensitive in most design, use default values and check final design

Use default values and check sensitivity, designs not generally governed by max spacing

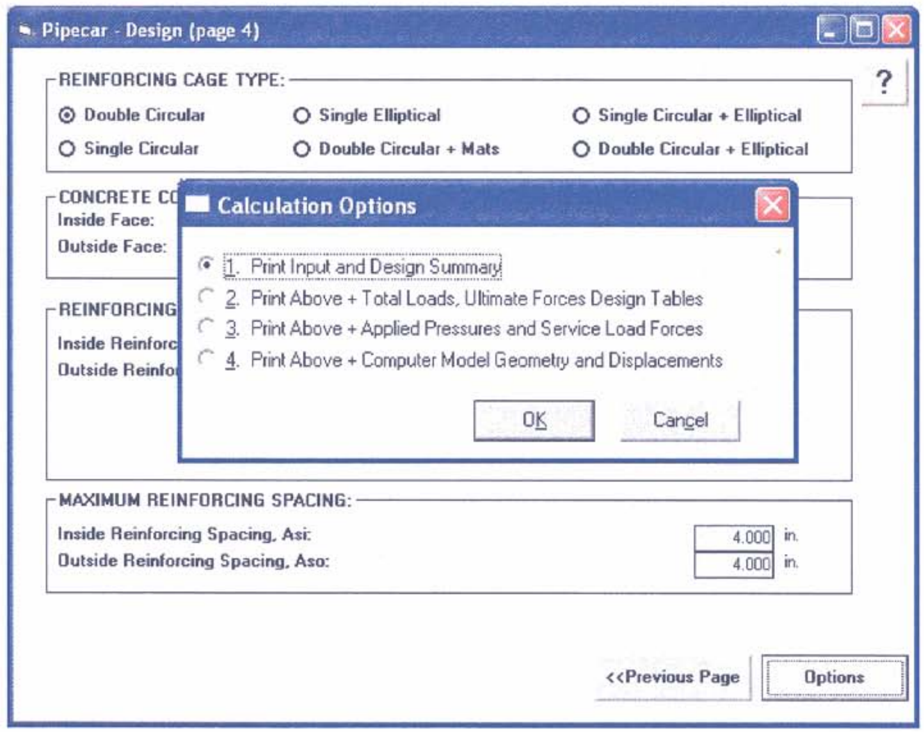
PIPECAR™

Hit Options to save and Execute

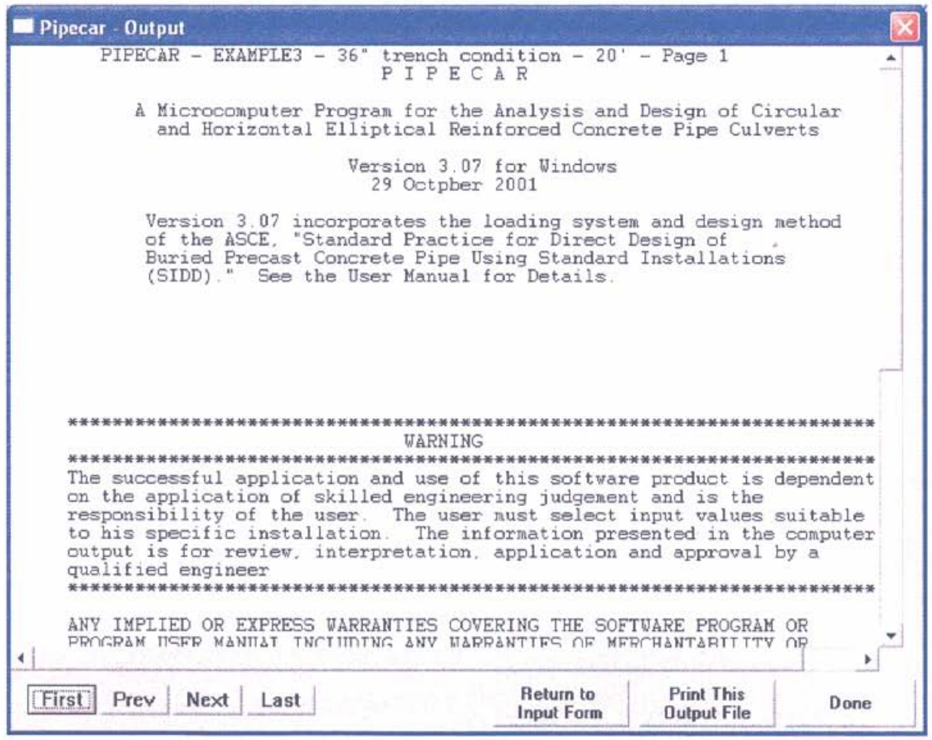


PIPECAR™

Select Desired Level of Output



PIPECAR™
Output Screens – Page 1



PIPECAR™
Ultimate Force summary Table

Pipecar - Output

1 PIPECAR - EXAMPLE3 - 36" trench condition - 20' - Page 4

TABLE OF ULTIMATE FORCES

LOCATION DESIGN	MOMENT	THRUST	SHEAR
DEG FROM INVERT	IN.KIPS/FT	KIPS/FT	KIPS/FT
.00	44.880	2.016	.000
12.25	35.557	2.690	3.656
90.00	-31.930	6.474	.002
157.75	23.369	2.230	-2.320
180.00	32.889	1.485	.000

First | Prev | Next | Last | Return to Input Form | Print This Output File | Done

PIPECAR™
Governing Design Table

Pipecar - Output

90.00	ASO	2.84	.120	.000	.000	
157.75	ASC	2.84				.599
180.00	ASC	2.84	.180	.471	.482	

NOTES:

1. REINFORCING REQUIRED FOR 0.01 INCH CRACK IS DETERMINED BY MULTIPLYING THE DESIGN INDEX BY THE FLEXURAL REINFORCEMENT.
2. IF THE DIAGONAL TENSION INDEX IS GREATER THAN 1.0, THE REINFORCEMENT REQUIRED FOR DIAGONAL TENSION IS GREATER THAN THE FLEXURAL AND CRACKING REINFORCEMENT. IF THE DIAGONAL TENSION INDEX IS GREATER THAN 1.0 AND THE REQUIRED REINFORCING RATIO IS GREATER THAN 0.02 THEN STIRRUPS MUST BE USED. THIS IS INDICATED BY A "*" AFTER THE DIAGONAL TENSION INDEX.
3. IF THE RADIAL TENSION INDEX IS GREATER THAN 1, STIRRUPS MUST USED.

REINF.	DEPTH	GOVERNING DESIGN			
	IN.	REQUIRED AREA (SQ.IN./FT)	REINF. RATIO	STIRRUPS REQUIRED?	GOVERNING MODE
ASI	2.84	.252	.0074	NO	FLEXURE
ASO	2.84	.120	.0035	NO	FLEXURE
ASC	2.84	.180	.0053	NO	FLEXURE

PIPECAR™
Required Reinforcing Scheme Design Table

Pipecar - Output

PIPECAR - EXAMPLEPRB - 36 inch design - Page 7

R E I N F O R C I N G   D A T A

REINFORCING CAGE TYPE: DOUBLE CIRCULAR

	d * (IN.)	MAXIMUM REINFORCING SPACING(IN.)	REINFORCING AREA (SQ.IN./FT)
INVERT- INSIDE REINFORCING	2.800	4.000	.317
SPRINGLINE- OUTSIDE REINFORCING	2.800	4.000	.138
CROWN- INSIDE REINFORCING	2.800	4.000	.207

>>> IF THIS PIPE IS MANUFACTURED WITH ANY REINFORCING SCHEME OTHER THAN A SINGLE OR DOUBLE CIRCULAR CAGE, THE MANUFACTURER MUST CLEARLY MARK THE PIPE TO INDICATE THE PROPER ORIENTATION IN THE GROUND. REINFORCING DESIGNS SHOULD ALLOW FOR SOME TOLERANCE IN LOCATION OF PIPE INVERT IN THE FIELD INSTALLATION.

* d IS THE DISTANCE FROM THE COMPRESSION FACE TO THE CENTROID OF THE TENSION REINFORCEMENT AND IS USED TO DETERMINE THE REQUIRED REINFORCEMENT. IF SELECTION OF REINFORCEMENT RESULTS IN A SMALLER d THAN THIS VALUE, THE DESIGN SHOULD BE REEVALUATED.

First | Prev | Next | Last | Return to Input Form | Print This Output File | Done

UMA | AECOM

Appendix C
Indirect Design – Sample Pipe Selection Problem

EXAMPLE PROBLEMS
EXAMPLE C.1
Use of Heger VAF's in Indirect Design

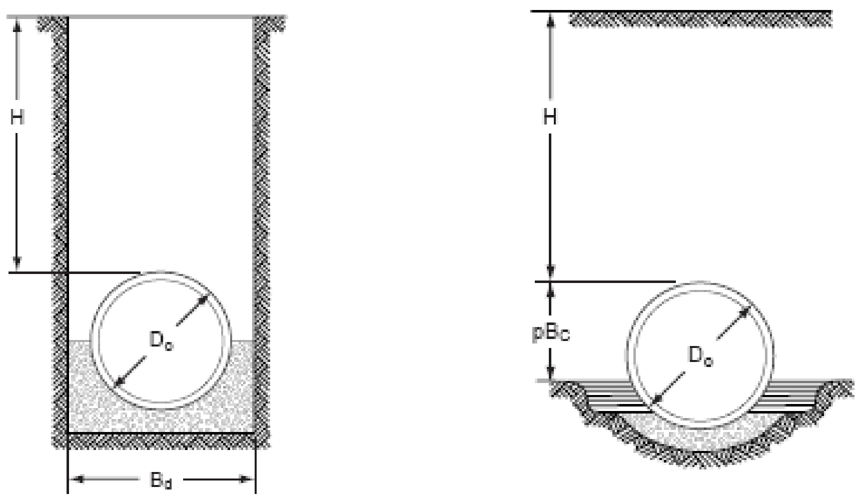
A 36" circular pipe is to be installed in a trench with 20' of cover over the top of the pipe. The intended width of the trench is 2' wider than the pipe on each side of the pipe and there are no contractual controls in place to ensure that trench width is rigidly controlled to this value. The local supplier of concrete pipe indicates that their 36" pipe is manufactured with a C-wall, wall thickness configuration.

The pipe will be installed in a Type 2 installation condition, and will be backfilled with sand and gravel material having a unit weight of 135 [lb/ft^3]. The pipe alignment is a major arterial with a high probability of exposing the pipe to dual passing vehicles.

The designer has chosen to estimate earth loads using Heger VAF's and, therefore, it is not required to determine transition width and accordingly no estimate settlement ratio/projection ratio product ($r_{sd}p$)is required to be made.

Determine the required pipe class for this situation and the revised analytical approach.

Example C.1



1. Determine the Earth Load

The C-wall configuration means that the wall thickness of the pipe is 4.75 inches (Equation 2-4) and the outside diameter of the pipe, B_c becomes 3.79 feet. The intended trench width, B_d, is then 7.79 feet. However, as the designer is utilizing Heger VAF's to estimate earth loading, earth loads are already based on their most conservative values, embankment conditions (as depicted to the right of the Figure above). It is not required, therefore, to estimate transition width.

To determine the Earth Load, we can use simplified Heger distribution based the weight of the prism of soil above the pipe multiplied by a vertical arching factor (VAF) selected by installation type (Modified form of Equation (2-5)). In this case, we will be using a Type 2 standard installation therefore:

W_e = VAF*PL [lb/ft]
Based on Equation (2-7):

$$PL = w * \left(H + \frac{D_o(4-\pi)}{8}\right) * D_o \text{ [lb/ft]}$$

Where VAF = vertical arching factor based on installation type

w = unit weight of soil [lb/ft^3]
H = height of fill above pipe [ft]
D_o = outside diameter of pipe [ft]

Based on Table 1 - VAF and HAF for Standard Installations, VAF for a Type 2 Installation would be 1.40. Therefore:

PL = 135 (20 + (3.79 (4– π)/8)) 3.79 = 10,441 [lb/ft]

For a Type 2 installation, VAF = 1.40, therefore $\mathbf{W_e}$ = 1.40 * 10,104 = **14,617** [lb/ft]

2. Determine the Live Load

Based on the design condition of a major arterial, we shall select two passing CL-800 vehicles for the live load. As depicted in the Equations in Figure 17:

$$w_L = \frac{100,600}{(17.67 + 1.75H)(0.83+1.75H)}$$

$$w_L = \frac{100,600}{(17.67 + 1.75(20))(0.83+1.75(20))}$$

w_L = 53 [lb/ft^2]

These are converted to a live load using Equation (2-23); $W_L = w_L B_c(1+I_f)$ where I_f is the impact factor which is zero for depths greater than 6 feet (see Table 6).

Therefore:

W_L = 53 *3.79 = **201** [lb/ft]

3. Determine the Fluid Load

Fluid load will be based on the inside area of the pipe and a fluid density of 62.4 [lb/ft^3]. Thus from Equation (2-16):

$$W_f = \frac{\pi * D_i^2}{4} * 62.4 = \mathbf{441} \text{ [lb/ft]}$$

4. Selection of Bedding Factor

As we are using Heger VAF's to estimate earth loads which are based on embankment loading conditions (the most conservative earth loading condition), we can safely use embankment bedding factors from Table 4 - Bedding Factors (B_f) for Standard Trench and Embankment Installations. This is because any reduction in horizontal support that may result from a narrower trench in the construction phase will also be accompanied by a proportional reduction in real earth loading.

Based on a 36" diameter pipe and a Type 2 Installation an embankment loading factor can be determined from Table 4 as $\mathbf{B_{fe} = 2.9}$.

5. Pipe Strength Requirement

The required 3-Edge Bearing Strength is given by Equation (3-1):

$$TEB = \frac{(W_e + W_L + W_f)}{B_f} * FS$$

Based on the use of reinforced concrete pipe, conservative loading and bedding support assumptions, and the acceptability of 0.01" service cracking as a design condition, a TEB factor of safety of 1.0 is appropriate:

$$TEB = \frac{(14{,}617+201+441)}{2.9} * 1.0 = 5262 \text{ [lb/ft]}$$

The required D-Load in units of lbs/ft/ft of diameter is given by:

$$D_{0.01} = \frac{TEB}{D_i}$$

Therefore:

$$D_{0.01} = \frac{5262}{3} = \mathbf{1754} \text{ [lb/ft/ft]}$$

As per ASTM C76 and Section 3.2.1, $D_{0.01}$ = 1754 [lb/ft/ft] correlates to a CL-IV pipe. The completed design has actual FOS against service cracking and ultimate failure as follows:

Service cracking
$$FOS = \frac{D_{0.01 ClassIV}}{D_{AppliedTEB}} = \frac{2000}{1754} = 1.14$$

Ultimate
$$FOS = \frac{D_{uClassIV}}{D_{AppliedTEB}} = \frac{3000}{1754} = 1.71$$

As these are both greater than our design objectives (FOS of 1.0 for service cracking and 1.5 for ultimate for TEB capacity greater less 2000 lb/ft/ft diameter) the design is adequate.