



VOLUME 3
DRAINAGE

VOL. 3-03
DESIGN GUIDELINES

May 2025

TERMS OF USE

The “City of Edmonton Design and Construction Standards Volume 3: Drainage”, henceforth known as “Volume 3”, is made available for use in the City of Edmonton effective as of May, 2025. Volume 3-03: Design Guidelines has been developed to establish standards and guidelines which align with EPCOR’s expectations in the design and construction of drainage infrastructure within the City of Edmonton. Volume 3-03 is presented as accurate and complete as of the effective date and all care has been taken to confirm the accuracy of the information contained herein. The views expressed herein do not necessarily represent those of any individual contributor. No part of these standards absolves any user from the obligation to exercise their professional judgment and follow good practice. Should any user have questions as to the intent or accuracy of any specification or drawing herein, or concern that conflict may exist between the manufacturer’s or suppliers’ recommended installation procedures and Volume 3-03, the user is advised to seek clarification by sending an email to DRENG@epcor.com.

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2025-05-21

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The following is a list of revisions in **Vol. 3-03: Design Guidelines**.

Section	Changes
Section	Changes
	Deleted the tables of Vol 3-01, 3-02, 3-03, 3-04, 3-05, and 3-06 which showed the description of each sub-volume when split Volume 3 in 2021.
Document wide	Replace “manhole(s)” with “maintenance hole(s)”, “MH” is still used for the abbreviation of “maintenance hole”.
1.0	Changed “...for new developments.” to “... for new developments, infill developments and modification of the existing systems.”
1.1	Deleted subsections related to the estimation of sanitary flows and Tables of Sanitary Design Factors from this sub-volume. Added “Refer to “ Design Guidelines – Sanitary Flow Generation for Neighbourhood Design Report, Water Consumption and Fire Flow for Hydraulic Network Analysis ” for the calculation of sanitary flows.” A link was inserted.
1.5.2.iv 2.3.1 the forth bullet	Changed “... Exceptions shall require provision of suitable justification by the designer and shall be subject to the approval of the Engineer.” to “... Exceptions shall require provision of suitable justification by a professional engineer licenced in Alberta and shall be subject to the approval of EPCOR Engineer.”
1.6.1.i	Changed “In accordance with the EPCOR Water Services Bylaw 18100,” to “In accordance with the City of Edmonton Bylaw <u>19627 – EPCOR Drainage Services and Wastewater Treatment Bylaw</u> ,”
3.2.2	Changed “Horizontal Spacing of Sewers” to “Horizontal/Vertical Spacing of Sewer Mains” Added “The minimum vertical clearance when sewer crossing each other shall be 300 mm.” and added formatting bullets.
3.2.3 iii.	Added “if the culverts are determined to be part of the road structure and replaced when the road reaches its expected life time.” Remove the whole bullet as CMP is not generally used in sewer system.
3.2.4	Added “or liner for concrete pipe, etc. The coating or liner may be required for both inside and outside of the pipe or only inside of the pipe depending on the geotechnical condition and sewerage characteristics.” at the end of the paragraph.
3.3.2 Table 3.1	Changed “100” to “2-100” in Column “Minimum Size of Stormwater Service (mm) for “Two-Family House Side by Side”. Added “Detached, Semi-detached, Duplex, and Fourplex” to each class of building.
3.6.1	Divided the paragraph to 8 bullets.
3.6.1.ii	Changed “Manholes of 1200 mm diameter shall be installed at all changes in sewer size, grade or alignment and at all junctions.” To “Maintenance holes shall be installed at all changes in sewer size, grade, or alignment and at all junctions.”
3.6.1.v	Change “Manholes are required at the intended permanent ends of all sewers but are not required at the ends of sewers stubbed off for future extension.” to “Maintenance holes are required at the intended permanent ends of all sewers.”
3.6.2 i.	Changed “For sewers 1200 to 1650 mm in diameter the access manholes may be spaced at a maximum of 500 m. For sewers 1800 mm in diameter or larger the access manholes may be spaced at a maximum of 800 m.” to “For sewers 1200 diameter or larger the access maintenance holes may be spaced at a maximum of 500 m.”
3.6.3 iv	Inserted “iv. In special approved cases of maintenance holes not on the road rights-of-way, if the distance from the center of the maintenance hole to the edge of the roadway curb is more than 4 meter, a minimum 3 meter wide access with proper consideration of vacuum truck turn around shall be built for O/M purpose.”
3.12.3 i	Changed “All sewer outlets shall be ...” to “All sewer outlets/outfalls shall be ...”
3.12.3.ii	Added “if there is a drop greater than 600 mm.” at the end of the paragraph



Section	Changes
3.12.4.i third bullet	Changed “Outfall to other creeks of pipe size greater than or equal to 500 mm” to “Outfall to other creeks of pipe size greater than or equal to 450 mm”
3.13.1	Changed “...the minimum culvert size shall be 400 mm,...” to “...the minimum culvert size shall be 450 mm,...”
4.4.1	Changed “In all cases, designers shall keep a record of structural design calculation associated with each project in accordance with the appropriate Standard Practice. Design calculations for specific projects shall be provided to the City upon request.” to “In all cases, design engineers shall keep a record of structural design calculation associated with each project in accordance with the appropriate Standard Practice. Design calculations for specific projects shall be provided to the City/EPCOR upon request.”



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1.0 SANITARY SEWER DESIGN CRITERIA

This section outlines the methodology and design criteria that apply to the preliminary and detailed design of sanitary sewage conveyance systems for new developments, infill developments, and modification of the existing systems. The emphasis of this section is on those criteria that determine the size and profile of sanitary sewers. Refer to Section **3.0** for other design considerations such as alignments and the detailed design of appurtenances.

1.1 Estimating Sanitary Flows

Refer to “Design Guidelines – Sanitary Flow Generation for Neighbourhood Design Report (NDR), Water Consumption and Fire Flow for Hydraulic Network Analysis (HNA)” for the calculation of sanitary flows.

1.2 Sizing of Sanitary Sewers

1.2.1 Capacity Requirement

All new sanitary sewers shall be designed to have hydraulic capacity such that the sewer is flowing at no more than 80% of the full depth when conveying the estimated total design peak flow rate as determined by methods specified in Section **1.1** above. The design peak flow rate shall be determined for the total planned contributing area based on the ultimate anticipated zoning and density of development.

1.2.2 Methodology for Sizing Sewers

- i. All sanitary sewers in a straight alignment shall be sized using the Manning Equation and an “n” value of 0.013 for all smooth-wall pipe of approved materials.
- ii. Where sanitary sewers are curved, the “n” value used in the calculation should be increased and shall be subject to the approval of the Engineer.
- iii. Sanitary sewers are to be designed to carry the design flow at a flow depth of 80% of the sewer diameter. This results in a flow rate of approximately 86% of the sewers' full flow capacity. Therefore, the required flow capacity for sizing the sewer may be computed using the following relationship:

$$\text{Required full flow sewer capacity} = \frac{\text{estimated total design peak flow rate}}{0.86}$$

1.2.3 Minimum Size for Sanitary Sewers

Excluding service connections, sanitary sewers shall be 200 mm inside diameter or larger. Refer to Section **3.3.2** regarding minimum requirements for sewer service connections.

1.3 Sanitary Sewer Slope Requirements

1.3.1 Velocity Requirements

Sanitary sewers shall be designed to achieve a mean flow velocity when flowing full of not less than 0.6 m/s, to provide for self cleansing.

- For the upstream reaches of the sanitary system, where peak dry weather flow (DWF) is equal to or less than 1.5 L/s, the designer is to follow slopes per **Table 1.1**.
- The designer is to optimize the use of the available elevation differences to provide extra slope in the reaches of the sewer system where design flows are minimal.

1.3.2 The maximum flow velocity shall be limited to 3.0 m/s. This is to prevent undue turbulence, minimize odours, and limit the erosive and momentum effects of the flow.

1.3.3 Minimum Slope Requirements

- i. Minimum slope for the most upstream sanitary sewer(s) that has equal to or less than 1.5 L/s peak DWF shall follow **Table 1.1**.

Table 1.1: The Minimum Slopes Permitted for 200 mm Sewer Based on Peak DWF for the Upstream Reaches of the Sanitary System

Peak DWF (L/s)	Minimum Slope (%) ¹
<0.10	1.70
0.10 – 0.25	1.50
0.25 – 0.40	1.00
0.40 – 0.60	0.83
0.60 – 0.80	0.67
0.80 – 1.00	0.57
1.00 – 1.25	0.50
1.25 – 1.50	0.45
>1.50	0.40

- Consultants shall submit design peak DWF and corresponding velocities for all sewer pipe segments as part of design review and approval.
 - Only 200 mm sanitary sewer size shall be used when peak DWF is less than 5.0 L/s.
- ii. All gravity sewers between maintenance holes shall be laid with uniform slopes equal to or greater than the minimum slopes outlined in **Table 1.2**. Sewers shall not be oversized to justify using flatter slopes.

Table 1.2: The Minimum Slopes Permitted for Various Sanitary Sewer Sizes

Sewer Size (mm)	Minimum Slope (%) for Straight Run	Minimum Slope (%) for Curved Sewer
200	0.40	0.40
250	0.28	0.31
300	0.22	0.25
375	0.15	0.18
450	0.12	0.15
525	0.10	0.13
600 and larger	0.10	0.10

- 1.3.4 Sewer between two maintenance holes shall be installed at the same time. Installation of sewer between two maintenance holes by staging with a stub or stubs is not permitted.

1.4 Required Depth for Sanitary Sewers

Sanitary sewers shall be installed with sufficient depth to meet the following requirements:

- 1.4.1 To permit all buildings to drain by gravity to the sewer main. Special consideration should be given when property elevations may be low with respect to the surface elevation at the road right-of-way. Typically, the obvert of the sewer should be at least 1.0 to 1.5 m lower than proposed basement elevations.
- 1.4.2 To allow sewer services to connect at the crown of the main. Where services tie in to a T-riser maintenance hole, the service shall tie in within the maintenance hole barrel, above the joint with the pipe or a tie-in to a T-riser may be reconsidered.
- 1.4.3 Sewer service pipes will pass beneath or over any adjacent water main while providing acceptable clearances as follows:
- Sewer services below the water main: a minimum of 300 mm separation between the top of the

¹ "Tractive Force Design for Sanitary Self-Cleansing" Journal of Environmental Engineering, Vol. 135, No. 12, Dec 1, 2009.

sewer service pipe and the bottom of the water main.

- Sewer service over a water main: a minimum separation of 500 mm between the bottom of the sewer service pipe and the top of the water main.

1.4.4 To permit sewer services to have a minimum of 2.6 m cover from the proposed ground surface to the top elevation of the service pipe and a minimum of 2.75 m depth from the proposed ground surface elevation to the invert elevation of the service pipe at the property line.

1.4.5 Sufficient depth of cover is to be provided to give complete frost protection.

1.5 Pipe Elevation Considerations at Maintenance Holes, Junctions and Bends

1.5.1 Accounting for Energy Losses

The designer is to ensure that sufficient change in sewer invert elevation is provided across maintenance holes and at junctions and bends to account for energy losses which will occur due to flow transitions, turbulence, and impingement. Refer to Section **3.6.4** for specific requirements and methodology to be applied in this regard.

1.5.2 General Requirements - Sewer Profile at Maintenance Holes

Minimum invert change at maintenance holes and bends:

- i. The invert slope across maintenance holes from inlet to outlet shall not be less than the greater of the slopes of the downstream or upstream sewers.
- ii. The obvert elevation of a sewer entering a maintenance hole shall not be lower than the obvert elevation of the outlet sewer. In the case of a sewer entering a T-riser maintenance hole, the connecting sewer must enter within the T-riser barrel, above the joint with the pipe.
- iii. Where there is a bend (a deflection of the horizontal alignment between incoming and outgoing sewers) a drop in the sewer invert must be provided to account for energy losses. The amount of drop required is relative to the deflection in the sewer alignments and may be determined by the methods described in Section **3.6.4**.
- iv. Bends shall be 90° or less in deflection. Exceptions shall require provision of suitable justification by a professional engineer licenced in Alberta and shall be subject to the approval of EPCOR Engineer.

1.5.3 Junctions

Laterally connecting sewers entering a maintenance hole are to be vertically aligned so that the spring line of the laterally connecting sewer is at or above the 80% flow depth elevation of the outlet sewer. When the laterally connecting sewer is of a similar size to the outlet sewer, the requirements for energy loss provisions of Section **3.6.4** shall apply.

1.5.4 Drops at Sanitary Maintenance Hole

Extreme changes in elevation at sanitary maintenance hole are to be avoided and a smooth transition is to be provided between the inverts of the incoming sewers and the outlet sewer. When this restriction is not feasible and where the elevation difference between incoming and outlet sewers is greater than 1.0 m, a specifically designed drop maintenance hole may be required (refer to Section **3.10**).

1.6 Sanitary Sewer Service Connections

1.6.1 General requirements for sanitary service connections to properties

In accordance with the City of Edmonton Bylaw **19627 – EPCOR Drainage Services and Wastewater Treatment Bylaw**, separate sanitary sewer connections shall be provided for each separately titled lot zoned for residential, commercial, industrial, or institutional land use.

- i. Connections for all proposed separately titled detached and duplex residential lots shall be installed at the time of initial subdivision development.

1.6.2 For lots zoned duplex or multiplex residential land use, where construction of side by side units is anticipated, one sanitary sewer service shall be provided for each unit at the time of initial subdivision

development and located so as to suit potential subdivision of the lot.

- 1.6.3 Where at the time of construction of the improvements the desirable point of service for multiple residential (excluding duplex), commercial, industrial, or institutional zoned lots is unknown, the design and construction of sanitary sewer service connections for these lots may be deferred.
- 1.6.4 Refer to Section 3.3 for detail requirements for sanitary sewer service connections.

2.0 MINOR CONVEYANCE SYSTEM DESIGN

This section outlines the design criteria which apply to the preliminary and detailed design of storm drainage minor conveyance systems for new developments. The emphasis of this section is on those criteria that determine the size and grade profiles of storm sewers and certain elements of the system arrangements, such as inlet requirements.

2.1 Storm Sewers

2.1.1 Sizing of Storm Sewers

i. Capacity requirements

In summary, in accordance with Sections 8.3 and 8.4 - Vol. 3-01 Development Planning Procedure and Framework, the requirements for capacity of storm sewers are as in **Table 2.1**.

Table 2.1: Storm Sewers Capacity Requirements

Capacity Requirement	System Elements
Runoff due to a 1:5 year rainfall	Storm sewers servicing areas of 30 ha or less
1.25 times the runoff due to a 1:5 year rainfall	Storm sewers servicing areas greater than 30 ha

ii. Methodology for sizing storm and foundation drainage sewers

All storm sewers shall be sized using Manning's formula to provide the required capacity when the pipe is flowing full ($\frac{d}{D} = 1$, d = flow depth, D = pipe diameter) conditions.

Table 2.2: Manning's Roughness Coefficient (n)

Pipe Material	Roughness Coefficient (n)
All Smooth-Wall Pipe	0.013
Corrugated Metal Pipe - Unpaved	0.024
Corrugated Metal Pipe - Invert Paved	0.020
Corrugated Metal Pipe - All Paved	0.013

Note: Corrugated metal pipe (CMP) is not approved for use in permanent mainline storm sewers or catch basin leads (refer to Section 3.2.3).

iii. Minimum size for storm and foundation drainage sewers

- Storm sewer shall not be less than 300 mm diameter, with the exception that catch basin (CB) leads may be 250 mm diameter.
- Foundation drain sewers are not to be less than 200 mm diameter.

2.1.2 Storm Sewer Velocity Requirements

- i. All storm sewers shall be designed with mean velocities, when flowing full, of 0.90 to 1.0 m/s based on Manning's formula. Designs based on lower velocities are to be justified on the basis of feasibility or unwarranted cost impacts. Mean velocities below 0.6 m/s will not be allowed.
- ii. Where design velocities in excess of 3.0 m/s are proposed, special provision shall be made to protect against displacement of sewers by erosion or shock. No upper limit to flow velocities in storm sewers is defined. However, the designer shall ensure that supercritical flow does not occur where steep grades are utilized, unless provisions are made in the design to address structural stability and durability concerns. Flow throttling or energy dissipation measures may be required

to control the flow velocity or to accommodate the transition back to sub-critical flow.

- iii. Sewers shall not be designed to operate in super-critical flow conditions during flows less than design capacity conditions. Hydraulic structures are required under super-critical flow regimes and to make the transition from super-critical flow to sub-critical flow. Hydraulic structures are required to minimize life cycle costs and be designed to have a minimum 75 year design life.

2.1.3 Storm Sewer Slope Requirements

- i. It is recommended that all storm sewers be designed with a slope of 0.4% or greater.
- ii. No storm sewer shall have a slope of less than 0.1%.
- iii. The minimum slope shall be 0.4% for the most upstream leg of any storm system i.e. between the terminal maintenance hole and the first maintenance hole downstream.
- iv. All Catch Basin (CB) leads shall have a minimum slope of 1.0%.

The minimum slopes which shall be permitted for various storm and foundation drainage sewer sizes are as in **Table 2.3**.

Table 2.3: The Minimum Slopes Permitted for Storm and Foundation Drainage Sewers.

Sewer Size (mm)	Minimum Slope (%) for Straight Run	Minimum Slope (%) for Curved Sewer
200	0.40 (foundation drain sewer)	
250	0.28 (foundation drain sewer)	
300	0.22	0.25
375	0.15	0.18
450	0.12	0.15
525	0.10	0.13
600 and larger	0.10	0.10

2.2 Depth Requirements for Storm and Foundation Drainage Sewers

All sewers shall be installed with sufficient depth to meet the following requirements:

- 2.2.1 A minimum cover from finished grade to pipe obvert of 2.0 m for storm sewer and foundation drain sewer is to be provided for all pipes smaller than 600 mm diameter. If this cover cannot be achieved then provide adequate pipe insulation to prevent freezing in accordance with insulation manufacturer’s recommended installation procedures and as shown on the drawings. A minimum of 1.5 m of cover to obvert is required for storm sewers equal to or larger than 600 mm in diameter.
- 2.2.2 The depth of the storm service and foundation drain service should match the sanitary service at the property line. When this is not practical, provide a minimum cover from finished grade to pipe obvert of 2.0 m for the storm service and the foundation drain service at the property line. If this cover cannot be achieved, provide adequate pipe insulation to prevent freezing in accordance with insulation manufacturer’s recommended installation procedures and as shown on the drawings.
- 2.2.3 Adequate depth is to be provided to allow for drainage to the sewer main of the interior of sites where interior finished grades may be lower than the finished grade at the property line.
- 2.2.4 Provide adequate depth for CB leads to allow them to extend to proposed CB locations at the required slope of 1.0% or greater.
- 2.2.5 Provide adequate depth to allow sanitary sewer services to cross over the top of the storm sewer and have the required minimum depth at the property line for typical parallel storm and sanitary sewer alignments.
- 2.2.6 The Engineer may determine other depth requirements.

2.3 Maintenance Holes, Junctions and Bends - Pipe Elevation Considerations

- 2.3.1 Minimum Invert Change at Maintenance Holes

- The invert slope across maintenance holes from inlet to outlet shall not be less than the greater of the slopes of the downstream or upstream sewers.
- The obvert elevation of a sewer entering a maintenance hole shall not be lower than the obvert elevation of the outlet sewer.
- Where there is a bend (a deflection of the horizontal alignment between incoming and outgoing sewers) a drop in the sewer invert must be provided to account for energy losses. The amount of the drop required is relative to the deflection in the sewer alignments and may be determined by the methods described in Section **3.6.4**.
- Bends shall be 90° or less. Exceptions shall require provision of suitable justification by a professional engineer licenced in Alberta and shall be subject to the approval of EPCOR Engineer.

2.3.2 Junctions at Maintenance Holes

Where more than one inlet sewer enters a maintenance hole, forming a junction, the laterally connecting sewers entering the maintenance hole are to be vertically aligned so that the spring line of each laterally connecting sewer is at or above the 80% flow depth elevation of the outlet sewer. An exception occurs when the laterally connecting sewer is of a similar size to the outlet sewer, the requirements for energy loss provisions of Section **3.6.4** shall apply.

2.3.3 Drops at Storm Maintenance Holes

- Generally a smooth transition is to be provided between the inverts of incoming sewers and the outlet sewer, and extreme changes in elevation at maintenance holes should be avoided whenever feasible.
- Where drops of 1.0 m or less occur at maintenance hole, the designer is to ensure that free outflow and low backwater conditions will exist in the downstream sewer so that hydraulic jump formation and associated concentrated effects are avoided. For drops of greater than 1.0 m, a specifically designed drop structure may be required to address the hydraulic requirements of the change of elevation, refer to Section **3.10**.

2.4 Drainage of Roadways and Other Public Rights-of-Way - Minor System

2.4.1 General Runoff Interception and Inlet Requirements

- i. Storm sewers and inlets shall be provided to directly drain all streets, alleys, walkways, and other public rights-of-way and to address the level of service requirements of Section 8.3 - Vol. 3-01 Development Planning Procedure and Framework.
- ii. Sufficient inlet capacity is to be provided, in the form of CBs or specifically designed inlets, so that runoff from a 1:5 year rainfall event is conveyed into the minor storm drainage system without inhibiting the use of roadways.
- iii. The following specific considerations and requirements are to be addressed with respect to the 1:5 year rainfall event.

2.4.2 Flow in Gutters

- i. When storm sewers exist within the right-of-way, it is preferable to transport the drainage in the sewer rather than in the roadway gutters. Along sloped roadway sections, sufficient inlet capacity is to be provided to take runoff from the gutter before it reaches the next downstream sag location.
- ii. The depth of flow in gutters should not exceed the top of curb at any point.
- iii. The width of flow along curbsides of roadways should allow for a minimum of one lane width free of ponded water on collector roads and one free lane in each direction on two direction arterial roads.

2.4.3 Flow through Intersections

Drainage should not pass through intersections, but rather sufficient inlet capacity is to be provided to intercept all flow at the uphill side and at the upstream of the curb ramps at sag locations.

2.4.4 Ponding at Sags

The depth of ponding at roadway sag locations and depressions is not to exceed 150 mm and should not reach the rim elevation of any sanitary maintenance hole located within or near the sags. Inlet capacity provisions must consider the entire contributing area that may drain to the design location. At sag locations, the determination of required capacity must account for flow that may bypass inlets at upstream sloped gutter locations.

2.4.5 Maximum Spacing of Inlets

Where closer spacing of inlets is not dictated by the requirements of Section 2.4.2 above, the spacing of storm runoff inlets is to satisfy the following requirements:

- i. Runoff shall not be required to flow a distance greater than 150 m along roadway gutters without reaching a CB or other inlet to the minor storm drainage system.
- ii. In alleys and walkways, runoff shall not be required to flow on the surface a distance greater than 180 m to a point of interception.
- iii. In the design of drainage inlets along walkways, alleys and utility rights-of-way, the designer must consider the total area that may drain to them and space CBs and inlets accordingly. Often residential lots drain onto the walkway, alley, or utility lot.

2.5 Location of Drainage Inlets

2.5.1 Locations for Drainage Inlets on Roadways

- i. Inlets required at sags at intersections should be located at the end of curve (EC) or beginning of curve (BC) of the curb return.
- ii. Where there is a continuous grade through the curb return at an intersection, stormwater CBs and CB maintenance holes shall be located at the uphill side of the curb return (BC). Normal design locations for CBs and CB maintenance holes are at sags at intersections, turning bays, and centre medians as governed by roadway design.
- iii. Location of drainage inlets must adhere to Volume 2: Complete Streets Design and Construction Standards (see Section 3). In situations where the design deviates from i or ii above to comply with Volume 2 Standards, it is the responsibility of the design engineer to prove that the location chosen does not result in conflicts with curb ramps or curb ramp flares, negative impacts to the drainage system, or safety concerns with ponding/icing at intersections.
- iv. Design locations for CBs on residential or other roadways shall be chosen to avoid conflict with driveway crossings wherever possible. Subject to roadway design, drainage inlet locations at the projection of the property lines are preferred in these instances. It is the design engineer's responsibility to ensure the locations of CBs and driveways and other crossings are chosen properly to avoid conflicts. If conflicts are present, the CB and CB leads may have to be relocated at the cost of the Developer.

2.5.2 Location of Drainage Inlets at Sidewalks and Walkways

- i. Minor runoff event flows from swales or other flow channels draining significant areas of residential development, parks, school sites, municipal reserve, public utility lots, or walkway lots shall not cross sidewalks or walkways (walks). To avoid excessive drainage of water across walks, or ponding of water where flow is obstructed by walks, drainage inlets shall be provided at strategic locations on the upstream side of walks to intercept concentrated drainage flows.
- ii. CB inlets are to be installed for this purpose and shall be located a minimum of 600 mm from the edge of the walk.
- iii. Subject to the approval of the Engineer, certain concentrated flows may be permitted to cross a walk through a 1.0 m wide concrete gutter or a monolithic curb and gutter sidewalk section provided with such a flow channel.

2.5.3 Location of Drainage Inlets in Alleys

CBs and CB maintenance holes used to intercept drainage in alleys are to be located generally at the

longitudinal centre line of the alley so as not to be within the typical wheel track area.

2.5.4 Drainage of Private Property

In residential subdivisions, no CBs or leads are to be placed or extended beyond the limits of the public rights-of-way. Low spots at the back or sides of lots must be filled and graded to ensure that all potential collection areas drain directly or indirectly to a public right-of-way. Design requirements for grading of private residential lots are addressed in Section 6.0 - Vol. 3-02 Stormwater Management and Design Manual.

2.5.5 Capacity of Catch Basin Inlets

The designer is responsible for determination of the appropriate design capacity factors for the proposed CB inlets. Available information with regard to the capacity and capture efficiency of the current and non-current CBs is provided in **Appendix A**.

2.6 Storm Sewer Service Connections

2.6.1 General requirements for storm services to properties (detail requirements for storm sewer services, refer to Sections **3.3** and **3.4**.)

i. Residential properties

Foundation drain service connections are to be provided to all new detached, semi-detached and duplex residential units. Storm service connections are also to be provided to the same residential units when roof leaders' discharges from one lot drains to another lot. In addition, storm service connections are to be provided when dictated by geotechnical requirements as identified in the Hydrogeotechnical Impact Assessment or Slope Stability Evaluations for top-of-bank locations.

ii. Commercial/institutional, industrial, and multiple residential properties

Storm sewer service connections for the connection of onsite storm drainage systems and/or roof drains are to be provided to properties zoned or proposed to be zoned for commercial, institutional, industrial, and multiple residential land uses. When required service locations are known, storm service connections should be installed concurrently with the general area servicing. Otherwise, installation of connections may be deferred until the specific property development is proposed.

3.0 SEWERS, APPURTENANCES, AND STRUCTURES

This section contains detailed design information applicable to sewers in general and appurtenances and structures ancillary to sanitary sewer and storm drainage systems. For general design criteria with respect to the various facilities, refer to Section **1.0** (sanitary) and **2.0** (storm and foundation drain).

3.1 Rights-of-Way for Sewer and Drainage Facilities

3.1.1 Use of Public Rights-of-Way

Whenever possible, sewer and drainage system facilities are to be located within road rights-of-way, walkway rights-of-way, or public utility lots. All maintenance holes and drainage inlets and outlets are to be located within and accessible through these rights-of-way.

3.1.2 Easements

Public sewers and associated sewer and drainage facilities required to cross through, or to be located within, privately owned property or lands held or controlled by authorities other than EPCOR, shall be protected by easements naming EPCOR Water Services Inc. as the grantee. Easements shall be suitably located to permit access to the sewer and drainage facilities within them and are to provide to EPCOR rights of surface access for maintenance and for excavation for repair or reconstruction by EPCOR.

Sewer alignments and the easements protecting the sewer are to be on only one side of a property line, i.e. a property line shall not longitudinally bisect an easement. The width of easements for sewers shall be sufficient to provide a minimum clearance of 0.6 m on each side, measured from the limit of the easement to outside edge of the sewer closest to that boundary. The minimum total width of easements for sewers shall be 3.0 m. Wherever feasible, easements are to be located to provide a clearance of 6.0 m between the easement limit and any anticipated or existing structure.

Maintenance holes, drainage inlets, and other sewer system appurtenances should not be located within easements.

Refer to Section 4.4.3 - Vol. 3-02: Stormwater Management and Design Manual in regard to easements and restrictive covenants for wet ponds. Refer to Section 6.4.12 - Vol. 3-02: Stormwater Management and Design Manual in regard to easements for drainage swales.

3.1.3 Restrictions on Service Connections from Sewers in Easements

- i. Service connections to properties zoned for single family or duplex residential land use shall not be permitted from sewer mains located in easements in favour of the City/EPCOR.
- ii. Service connections to public sewers within easements in favour of the City/EPCOR Water Services Inc. may be permitted by the Engineer subject to:
 - The premises being serviced and the easement are located on land zoned for multi-family residential, commercial, institutional, or industrial land use.
 - Physical access to the service connections located in the easement and access across the adjoining property being unobstructed and suitable as determined by the Engineer.

3.2 Sewers

3.2.1 Location of Sewers within Rights-of-Way

Sewer alignment requirements

- i. The alignment of sanitary and storm sewers within public road, walkway, and utility rights-of-way are to conform to the standard drawings for the location of utilities. Refer to Volume 2: Complete Streets Standards Drawings, Cross Sections.
- ii. Sewers are to be laid parallel with the centre line of the roadway or utility right-of-way within which they are located. Sewers should be laid straight wherever possible, however, curving of sewer alignments to parallel curved rights-of-way is acceptable.
- iii. Sewers which are not laid parallel to a right of way centre line should be laid straight, and if crossing a right-of-way, should be aligned as near to perpendicular to the right-of-way as possible.
- iv. The slope of sewers is to be uniform and continuous between maintenance holes.

3.2.2 Horizontal/Vertical Spacing of Sewer Mains

- i. Except where laid in the same trench, sewers running parallel and within the same right-of-way shall be horizontally separated by a minimum of 2.5 m, measured from centre line to centre line. When installed in a common trench, pipes shall be laid with a minimum separation of 250 mm, measured horizontally between vertical lines tangent to the adjacent outside faces of the pipes. For sewers in a common trench, refer to Section 4.3.1 for comments on the design basis and Section 3.3.3 - Vol. 3-05: Drainage Drawing Requirements, Approvals and Asset Acceptance/Transfer for requirements for leakage testing of sanitary sewers.
- ii. The minimum vertical clearance when sewer crossing each other shall be 300 mm.

3.2.3 General Sewer Materials Requirements

i. Concrete pipe

Non-reinforced concrete and reinforced concrete pipe are approved for storm and sanitary sewers, CB leads and permanent culverts. Refer to Section **4.0** for special considerations for using non-reinforced concrete pipe.

ii. PVC pipe

- PVC pipe is approved for use in residential areas for sanitary and storm sewers and services.
- PVC pipe is approved for use as storm sewers or CB leads serving arterial roadways and dangerous goods routes and any sewers conveying flows from these areas.
- PVC pipe for sewers shall not be blue. Refer to Section 2.2.1.1.5 Volume 4: Water.

3.2.4 Corrosion Protection for Sanitary Sewers and Maintenance Holes

Sanitary pipes and maintenance holes with potential H₂S exposure shall be corrosion resistant by either using corrosion resistant material such as plastic or using adequate corrosion resistant coating or liner for concrete pipe, etc. The coating or liner may be required for both inside and outside of the pipe or only inside of the pipe depending on the geotechnical condition and sewerage characteristics.

3.2.5 Prevention of Blockage of Sewers

Where siphons are designed to cross rivers or creeks, provisions must be included for a comminutor/grinder or some other means to prevent blockage of the pipe. Provision of cleanout shall not be accepted. All weather vehicular access must be provided for O&M of the facility.

3.3 Sewer Service Connections

These requirements pertain only to the sewer service connections that are located between the sanitary, storm or foundation drain main, and the edge of the public sewer right-of-way. EPCOR is not responsible for ownership or maintenance of extensions from the sewer service connections into private property.

3.3.1 Extension of Services across Gas Easements

In areas where natural gas distribution facilities are to be installed within an easement across the front or side of the property and the connection to sewer services cross the easement, then install these connections to the private property side of gas easements. This is to reduce the risk of damage to the gas main. Installation shall be as described in the Construction Specifications.

3.3.2 Sizes and Number of Service Connections Required

Table 3.1: Minimum Requirements for Sewer Service Connections

Class of Building	Minimum Size of Sanitary Service (mm)	Minimum Size of Stormwater Service (mm)
Detached - Single-Family Dwelling	150 **	100
Semi-Detached - Two-Family House Side by Side	2 - 150 **	2-100
Duplex - Two-Family House Up and Down	1 - 150 **	100
Fourplex - Four-Family House	150	100
Commercial, Institutional and Industrial	150	150

The minimum size and grade of the foundation service for detached, semi-detached, and duplex residential units shall be 100 mm diameter and 1.0 % grade, respectively.

** Sanitary sewer services for single and duplex residential lots require a 150 mm to 100 mm reducer to be installed at either the property line or public right-of-way boundary on the private side, or at the property side limit of any natural gas system easement which must be crossed. Refer to the Construction Specifications.

3.3.3 Increased Service Connection Requirements

- i. Services of size larger than those indicated in the above table may be required, where, in the opinion of the Engineer, the length of service pipe or other conditions warrant.
- ii. Where more than one sanitary and one storm service is required, the sizes and locations of the services shall be subject to the approval of the Engineer.

3.4 Sewer Service Connection Arrangement

3.4.1 Orientation of Connections

The sanitary and stormwater services shall be oriented so that, when facing the lot being served, the sanitary service connection shall be on the right and the stormwater service connection shall be on the left. The alignment of the service connections shall intersect the property line at an angle as near to 90° as possible.

3.4.2 Location of Connections

- i. Single sewer service connections shall be located towards the middle 4.5 m of the property frontage.
- ii. Dual sewer service connections shall be centred about the projection of the common property line between the properties to be served.
- iii. No more than two service connections can be installed in a single standard sanitary maintenance hole. Service connections must be a minimum of 150 mm from a maintenance hole barrel joint to protect the gasket and structural integrity of the barrel. No new service connections can be connected to existing drill drop maintenance holes (DDMH).

Refer to the Construction Specifications for the relative positions and construction details for service connections. Standards Drawings WA-005-007 and WA-005-008 in Volume 4: Water provide service location details.

3.4.3 Sewer Service Connection Depth

- i. Sanitary service connections

The depth of the sanitary sewer service connection at the property line shall be 2.75 m from invert elevation to proposed finished grade. No variation shall be permitted without the written approval of the Engineer.

- ii. Storm service connections

The depth of storm sewer service and/or foundation drainage service connections should match that of the sanitary sewer service connection. However, when this is not practical due to the depth of the storm or foundation drainage sewers available, provide a minimum cover of 2.0 m for the storm service and for the foundation drain service from the proposed finished grade to the pipe above of the service at the property line. If a minimum cover of 2.0 m is not achievable, then provide insulated pipe to prevent freezing.

3.5 Sanitary Control Gates

Control gates should have adequate access for operation and maintenance work. All components and fittings of control gates shall be made out of high corrosion resistant materials such as stainless steel 316.

3.6 Maintenance Holes, Junctions, and Bends

3.6.1 General Maintenance Hole Requirements

- i. All maintenance holes shall be 1200 mm minimum inside diameter and constructed to the Construction Specifications.
- ii. Maintenance holes shall be installed at all changes in sewer size, grade, or alignment and at all junctions.
- iii. Maintenance holes are required to be 1500 mm diameter or larger when connecting sewers 750 mm or larger, as shown on Standard Construction drawings, location and design of maintenance holes connecting 1200 mm or larger sewers shall be designed in accordance with hydraulic considerations contained within this section.
- iv. Maintenance holes for 1200 mm or greater diameter sewer lines shall be installed in accordance with the maximum spacing requirements located herein.
- v. Maintenance holes are required at the intended permanent ends of all sewers.
- vi. All sewers shall have sufficient access maintenance holes for maintenance and to permit air venting. Access maintenance holes shall have unrestrained direct access above sewer invert.
- vii. For access maintenance holes deeper than 12 meters (40 feet) and on sewers 1200 mm in diameter and larger, maintenance hole frames and covers shall be a minimum of 900 mm in diameter to facilitate entry by inspectors' breathing apparatus. For these access maintenance

holes, the top most rung of the maintenance hole shall not be designed to be closer than 750 mm from the lid.

3.6.2 Maximum Spacing of Maintenance Holes for Access

- i. The maximum permitted maintenance hole spacing for all sewers less than 1200 mm in diameter is 150 m. For sewers 1200 mm diameter or larger the access maintenance holes may be spaced at a maximum of 500 m.
- ii. Where sewers are to be used for storage, the maximum spacing of access maintenance holes will be evaluated on a site specific basis.

3.6.3 Location of Maintenance Holes

- i. Maintenance holes for sewers located within roadway rights-of-way shall be located within the travel lanes or centre median as appropriate, between the outside curb lines.
- ii. No standard maintenance hole shall be located such that its centreline is closer than 1.5 m from a roadway curb face. Maintenance holes frames and covers are not to be located within a sidewalk.
- iii. Sanitary maintenance holes are to be located away from roadway sags and low areas where surface runoff might pond. When this is unavoidable or the possibility of inundation by major drainage flows exists then provisions shall be made to waterproof the sanitary maintenance hole.
- iv. In special approved cases of maintenance holes not on the road rights-of-way, if the distance from the center of the maintenance hole to the edge of the roadway curb is more than 4 meters, a minimum 3 meters wide access with proper consideration of vacuum truck turn around shall be built for O/M purpose.

3.6.4 Energy Loss Provisions at Maintenance Holes, Junction, and Bends

- i. There is a loss of energy when flow passes through a bend in a sewer, a maintenance hole on a sewer line, or a point where sewers meet in a maintenance hole or a specially designed junction chamber. These losses can be negligible as in the case of a small diameter sanitary sewer flowing partially full at minimum velocities, or substantial as in the case of a large diameter storm sewer flowing full and turning 90°. It is the designer's responsibility to allow for the losses incurred. In cases where the head available is limited, the designer has to provide a system that is hydraulically smoother.
- ii. Major junctions and bends
 - Analysis requirements

For bends and junctions in large sewers or where high flow velocities are anticipated, or for complex or unusual sewer junctions, detailed analysis may be required. The designer should consult appropriate references; for example:

 - a) Sangster, Wood, Smerdon and Bossy at the University of Missouri, Bulletin No. 41 entitled "Pressure Changes at Storm Drain Junctions";
 - b) ASCE Journal of the Hydraulics Division entitled "Pressure Changes at Open Junctions in Conduits"-HY6-#2057.
 - Guidelines for large bends and junctions

It is recommended that sudden, extreme changes of direction be avoided where large flows and high velocities are involved. Changes of direction in the order of 90° are not recommended. Where necessary, they require the approval of the Engineer and the following guidelines are to be considered:

 - a) The ratio of the radius of the bend R, measured to the pipe centre line, to the pipe's inside diameter D, should be greater than 2.
 - b) When R/D is less than 2, the maximum bend deflection at one point should be 45°, i.e. use two – 45° bends to turn 90°.
 - c) Benching on the outside of bends in maintenance holes should be carried upwards to

provide super-elevation to contain the flow in the channel.

- d) Bends in large sewers should not be at the same location as junctions. Separate structures should be provided to serve each function. Large inflows from opposing directions are not to be combined at one structure.
- e) Maintenance holes and structures where flows change direction must be designed with anchorage to resist thrust and impact forces generated by the flow.
- f) Special consideration must be given to the provisions for safe access to these structures, including appropriate location of maintenance holes.

iii. Minor junctions and bends

Simplified methods for head loss calculation

- To facilitate the rapid determination of head losses in maintenance holes, simplified methods, as outlined below, are adequate for the majority of cases involving pipe sizes, 600 mm and smaller, and low flow velocities.
- The head loss (H_L) is computed by multiplication of the head loss coefficient (K_L) for the particular bend or junction in the maintenance hole by the velocity head of the flow through the outlet sewer.

$$\text{i.e.: } H_L = K_L \frac{V^2}{2g}$$

iv. Bends in small sewers

- Use of standard unit invert drops

For sewers of 600 mm diameter and smaller, with invert slopes less than 1.5 times the minimum slopes permitted by Sections **1.3.3** and **2.1.3**, for sanitary and storm sewers respectively, the following standard drop provisions may be used:

- Deflections less than 45° require a 30 mm invert drop.
- Deflections of 45° to 90° require a 60 mm invert drop.

- Determination of head loss coefficients for bends - design aid

Head loss coefficients (K_L) for bends may be determined in relation to the amount of deflection and channel characteristics as in **Table 3.2**:

Table 3.2: Head Loss Coefficients for Bends

Deflection	Flow Channel Characteristics
90°	No benching or deflector, or provided only to the sewer spring line $K_L = 1.5$
	Benching or deflector provided to the sewer obvert level $K_L = 1.0$
Less than 90°	To determine the head loss coefficient, multiply the head loss coefficient for a 90° bend and the appropriate flow channel type by a head loss ratio factor from Figure 3.1 .

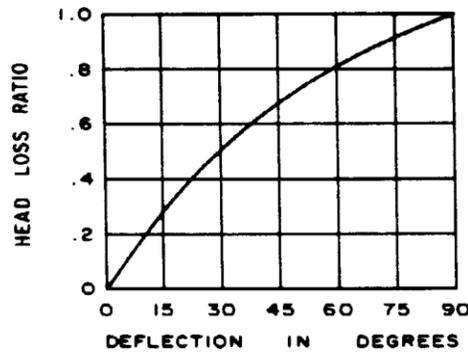


Figure 3.1: Head Loss Ratio Factor for Bends

3.6.5 Junctions with Side Inlets

For junctions with inlets at or near right angles to the outlet, the head loss coefficient applicable will vary depending on whether the incoming flow is deflected toward the inlet, or if incoming flows impinge. When a deflector with the full height and width of the incoming sewers is provided between the inlets the loss is, $K_L = 1.0$, without a deflector between the inlets the loss is, $K_L = 1.5$.

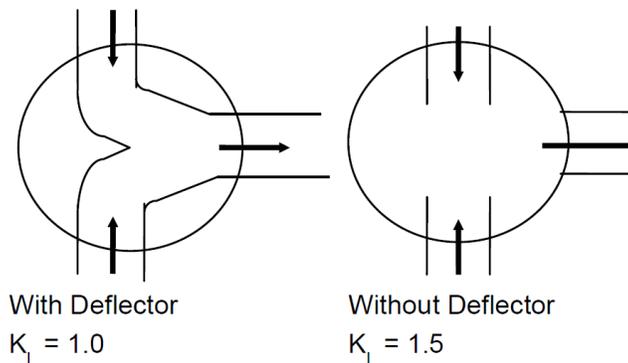


Figure 3.2: Head Loss Coefficient for Junctions with Side Inlets

3.6.6 Cross Junctions

For side and cross junctions, values of K_L are obtained from **Figure 3.3**. Cross junctions where flows from opposite directions impinge at the junction are not recommended. Wherever possible, separate maintenance holes should be used and inlet pipes should be aligned so as to maximize the velocity component to the outlet direction. Measures are to be taken to channel the flow toward the outlet by benching.

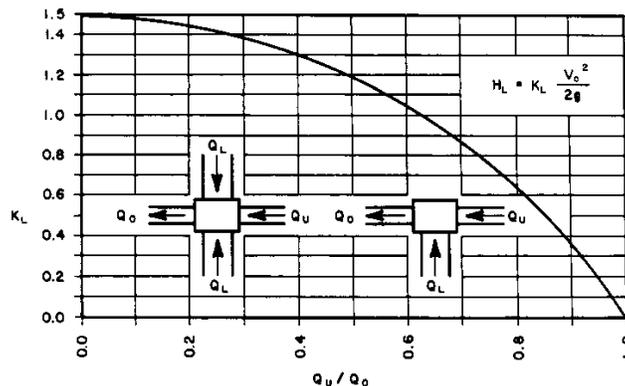


Figure 3.3: Head Loss Coefficients for Junctions

3.7 Catch Basin Inlets

3.7.1 General CB Requirements

Details of approved CB structures and components are provided in the Standard Drawings and Construction Specifications. Staggering of CB and CB maintenance hole grade rings to match with new curb and gutter alignment is not permitted.

3.7.2 Catch Basin Frames and Covers

Table 3.3 lists the current frames and covers and their respective uses. Non-current frames and covers are not to be used for new development and renewal, but only for replacement or modifications associated with previously constructed curb types.

Table 3.3: Catch Basin Frame and Cover Application Summary

Frame and cover type*	Curb type	Minimum barrel size (mm, nominal)	Allowable applications
2A	Straight faced	600	For CBs only
4A	Straight faced	1200	For CB Maintenance Holes only
6	No curb	600	For off roadway locations or temporary inlets on roadways only
K7	80mm rolled faced	600	Current preferred inlet for residential areas
DK7	80mm rolled faced	900	Current preferred inlet for residential areas where increased capacity is needed
8	No curb	600	For alleys, swales, gutters, and curb ramps
F-51 without side inlet	No curb required	900	For situations requiring increased capacity over 2A, or where there is no curb
F-51 with side inlet	Straight faced	900	For situations requiring increased capacity over F-51

* Refer to the Drainage Standards Drawings for information on the various types.

3.8 Catch Basin Leads

3.8.1 CB Lead Size and Grade

The CB lead size and grade shall be based upon hydraulic capacity requirements, except that the minimum inside diameter for any CB lead shall be 250 mm and the minimum grade for CB leads shall be 1.0%.

3.8.2 CB Lead Arrangement

- i. CB leads must enter a maintenance hole or a CB maintenance hole. CB leads must not connect directly to a sewer, or a downstream CB.
- ii. Under no circumstances is the length of lead from a 600 mm CB to exceed 30 m.
- iii. If a CB lead of over 30 m in length is required, a CB maintenance hole must be used as the upstream inlet, rather than a CB.
- iv. CB leads are to be laid straight wherever possible.
- v. CB leads must connect to a maintenance hole on the mainline sewer at the closet location. Long chain of CB to CB maintenance hole to CB maintenance hole to MH is not permitted.

3.9 Catch Basin Maintenance Holes

CB maintenance holes shall be 1200 mm inside diameter and constructed in accordance with the drawings. Installation of new CB maintenance holes directly on an existing storm main is not permitted. If new surface inlet is required, new CB shall be installed and connected to a maintenance hole on storm main with available pipe capacity.

3.10 Drop Maintenance Holes

- i. Drop maintenance holes are to be used to carry flow from sewers at higher elevation to those at a lower elevation. Generally, these shall be vertical drop shafts with inlet (upper) and outlet (lower) connection or chambers.
- ii. Where the invert elevation of a sewer entering a maintenance hole is more than 1.0 m above the invert of the outlet sewer, then that maintenance hole is considered a drop maintenance hole.
- iii. Where inlet pipes entering a sanitary drop maintenance hole are of 300 mm diameter or less, the inlet pipe must protrude into the maintenance hole 150 mm to provide for the attachment of flexible ducting.
- iv. When the size of the inlet pipe is large in comparison to the drop shaft diameter and where it is anticipated that the impingement of flow on the drop shaft wall opposite the inlet may create unstable flow and impede smooth air passage, then the inlet shall provide a smooth transition of flow from the horizontal direction to the vertical (refer to Section 3.11.1).
- v. When the rate of flow and the depth of the drop are of such a magnitude that there is potential for significant entrainment of air, then the drop shaft and lower connection shall be designed to provide for release of the entrained air and ventilation of the drop shaft (refer to Sections 3.11.2 and 3.11.3).

3.11 Design Criteria for Drop Maintenance Holes

3.11.1 Inlet Connection

An inlet connection providing for a smooth transition of flow from horizontal to vertical is required, designed so that at the design flow rate the flow does not back-up in the inlet sewer. There is to be a free outfall of the flow into the drop shaft, with critical depth control at the entrance. To achieve this, the following conditions shall be met:

- The upstream flow in the inlet pipe shall be sub-critical and the pipe shall be of sufficient size so that it does not surcharge.
- The pipe bottom profile, from the spring line down to the invert, shall form a smooth vertical curve between the inlet pipe and the drop shaft, with no sharp breaks in grade, projections, or edges. The radius of the vertical curve shall be such that the nappe of the flow maintains contact with the inlet invert. The actual vertical curve radius used shall incorporate a minimum safety factor of three, i.e. actual invert radius = three times theoretical radius when cavitation begins to occur.

3.11.2 Drop Shaft

- i. A drop shaft of a diameter equal to or larger than that of the largest inlet sewer pipe is recommended. This will ensure that the drop shaft capacity exceeds the inlet sewer capacity with ample provision for air flow and unforeseen conditions. For multiple large inlet pipes connecting to a single drop maintenance hole, an even larger drop shaft should be considered.
- ii. Provision of air vents at intervals along the drop shaft is recommended. These vents should be connected by a vent to a maintenance hole above the inlet and to the outlet. The vent connection may be a pipe placed outside of the drop shaft, or a divider wall within the drop shaft located no more than one-fourth of the shaft diameter from the wall opposite the inlet.
- iii. A standard maintenance hole with a restraining cover mechanism, designed to withstand the pressures resulting from air discharge and surcharging at the maintenance hole, is to be provided directly over the drop shaft, extending from the inlet connection upward to the ground surface.

3.11.3 Outlet Connection

- i. The outlet connection shall provide a hydraulic jump basin to dissipate energy, to convert the flow to sub critical velocity and to allow for the release of entrained air before the flow enters the downstream sewer.
- ii. The hydraulic jump may be of the free or forced type depending on the available length of outlet pipe from the base of the shaft to the connection to the downstream sewer and the anticipated operating tail water conditions.

- iii. An air vent or maintenance hole is to be provided at the crown of the outlet pipe downstream of the drop structure to allow removal of air released at the lower connection. This vent or maintenance hole is to be located upstream of the point where full flow in the outlet pipe is anticipated under design flow conditions. The air vent may be connected to the shaft vent system.

3.12 Storm Sewer Outfall Structures

3.12.1 Requirement for Outfall Structures

At the end of an outfall sewer, energy dissipaters are often necessary to avoid downstream erosion and damage of creeks, ravines, or river banks from high exit flow velocities. Outfall structures are required at locations where it is necessary to convert supercritical flow to subcritical, dissipate flow energy and establish suitably tranquil flow conditions downstream.

3.12.2 Outfall Structure Hydraulic Requirements

- i. When sewers discharge at subcritical flow, then smaller concrete structures with suitable baffles, aprons, and rip-rap are acceptable. For all outfalls, it is required that a rigorous hydraulic analysis be completed, to ensure that the exit velocities do not damage natural watercourses. The final exit velocities, where the flow passes from an apron or erosion control medium to the natural channel, shall not exceed 1.0 m/s and may be further limited depending on site specific soil and flow conditions.
- ii. Appropriate erosion control measures are to be provided at and downstream of the outfall to prevent erosion in the downstream channel.
- iii. Where high outlet tail water conditions or other downstream conditions may result in formation of a forced hydraulic jump within the sewer pipe upstream of the outfall, special consideration shall be given to the bedding and structural requirements of that section of sewer.

3.12.3 Outfall Structure and Stormwater Management Facility (SWMF) Inlet/Outlet Safety Provisions

- i. All sewer outlets/outfalls shall be constructed with provisions to prevent the entrance of children or other unauthorized persons. A grate with vertical bars spaced at no more than 100 mm shall be installed with adequate means for locking in a closed position. Provide for opening or removal of the gate for cleaning or replacing the bars. Grates should be designed to break away under extreme hydraulic loads in the case of blockage. In the case of fully submerged outlets, grates shall not be installed.
- ii. 920 mm high guard rails or fences spaced at no more than 100 mm of corrosion resistant material (galvanized steel, aluminium, or stainless steel) shall be installed along concrete headwalls and wingwalls to provide protection against persons falling if there is a drop greater than 600 mm.
- iii. Outfall Safety and Aesthetics

Outfalls, which are often located in parks, ravines, or on river banks, should be made as safe and attractive as reasonably possible. All outfalls must be constructed with safety provisions to prevent the entrance of unauthorized persons. All outfalls shall be designed in such a way that it can be safely accessed for O&M by foot and by vehicle to the alleys standard that allows safe passage of O&M vehicle. The appearance of these structures is important and cosmetic treatment or concealment is to be considered as part of the design. Concrete surface treatment is recommended to present a pleasing appearance. Bush hammered or exposed aggregate concrete is recommended. Live stakes or bioengineering is encouraged wherever applicable.

3.12.4 Outfall Structure Monitoring

- i. Conditions that would trigger needing an outfall monitoring station:
 - Outfall to North Saskatchewan River of pipe size greater than or equal to 1200 mm.
 - Outfall to Whitemud, Blackmud, or Mill Creeks of pipe size greater than or equal to 900 mm.
 - Outfall to other creeks of pipe size greater than or equal to 450 mm.
 - Equivalent capacity for multiple outfalls servicing the complete development area.

-
- Any outfall upstream of the water treatment plant.
- ii. Subject to EPCOR Water Services review, sewer system design would incorporate:
- Access maintenance hole(s), less than 30 m deep, and connected to an upstream pipe(s) that carries the full flow at the eventual discharge point(s) - i.e. - can use further upstream pipe(s) if the outfall pipe itself is inaccessible under these criteria or outfall pipe is too steep or inappropriate for monitoring (see below).
 - This upstream pipe(s) must have a length of at least ten diameters without slope change, turns, drops, connections or any other flow disturbances, or greater length as necessary to avoid disturbances such as hydraulic jumps near the maintenance hole. If surcharge is expected from the 1:100 year design storm, for this and lesser storms the pipe(s) must always carry flow at less than 6.0 m/s based on Manning's Formula. If surcharge is not expected the velocity may be higher if shown to be necessary.
 - This pipe must not be subject to or hydraulically affected by backwater from the discharge point or downstream structures.
- iii. General monitoring station construction criteria:
- Easy and safe access to site by land.
 - Instrumentation cabinet as per **Figure 3.4**. Area in front of the cabinet must be adequate for safe access while doing work in the cabinet.
 - The cabinet must be located so that it cannot be flooded under any foreseeable circumstances. Cabinets shall be mounted on a concrete base or plinth, or steel support posts founded in concrete bases that ensure stability of the cabinet.
 - Ground surrounding maintenance hole must be open and graded to allow tripod-enabled entry.
 - 100 mm (4") conduit from cabinet underground through wall of maintenance hole near ladder rungs, with pull string. Run must not exceed 140 m from cabinet, including depth down maintenance hole to measuring pipe.
 - Installation of electrical components in cabinet (as specified in cabinet drawing).
 - Arrange access and hookup from a single phase power source (EPCOR Distribution and Transmission) to the cabinet via 100 mm (4") underground conduit or other method as per electrical code.
 - All work to be done to electrical code, including electrical inspection and approval of power source and power conduit before burial.
 - Power and instrumentation conduit locations and sizes to be included in as-built drawings for site.
 - Provision of monitoring instrumentation. EPCOR Water Services will specify and install flow measurement instrumentation. Typically, this will include: Campbell CR800 datalogger, Raven RV50 Sierra Wireless cellular modem, Greyline AVFM 6.1 velocity meter with sensor, and Keller Acculevel pressure sensor; may vary depending on site requirements.
 - Depending on the site and EPCOR Water Services review, provision may be needed for an automatic sampler installation. If surcharge of the monitoring maintenance hole for the 1:100 design storm is less than 2.0 m below the top of maintenance hole, the maintenance hole itself may suffice. If not, a larger slab or equivalent, sampler cabinet, and a conduit from it into the maintenance hole will be needed. The requirement for a sampling site and details will be specified by EPCOR Water Services, System and Industrial Monitoring team.

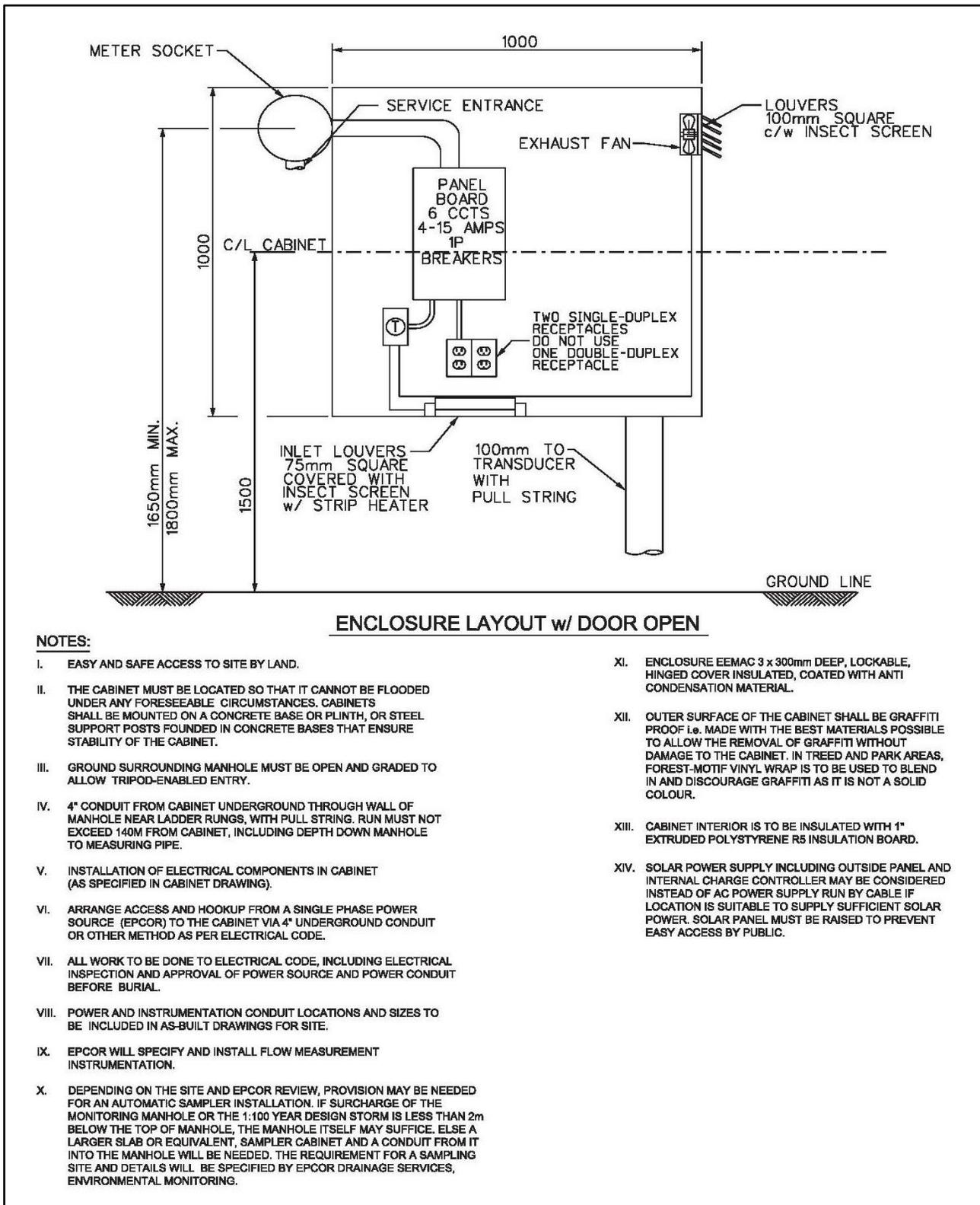


Figure 3.4: Typical AC Powered Instrumentation Cabinet

3.13 Culverts

- 3.13.1 The Engineer may require submission of hydraulic design calculations to identify design flow conditions and inlet head requirements for culverts. The need for energy dissipation and erosion control measures is to be considered for each design. When hydraulic considerations or minimal cover do not govern, the minimum culvert size shall be 450 mm, to allow for reliability and ease of maintenance.
- 3.13.2 Culverts are to be constructed with approved sewer material when they are to be permanent structures. End treatment and traffic protection are to be suitable for the location under consideration. All temporary crossings shall have culverts installed prior to road construction whenever feasible and are to be extended sufficiently to prevent end blockage.
- 3.13.3 Refer to the Construction Specifications for typical end treatment.
- 3.13.4 The discharge flow characteristics of culverts shall be analyzed and appropriate measures taken to avoid erosion. For outlets of large culverts, the requirements of Section **3.12**, Storm Sewer Outfall Structures, shall apply, in respect of erosion control and safety.

3.14 Rural Runoff Inlets

- 3.14.1 The required inlet capacity to accept rural runoff is addressed in the hydraulic portions of the AMP and NDR. Each inlet may be unique and appropriate consideration must be given to provisions for grates, safety, debris interception, sediment catchment and storage, and maintenance. Normally a road right-of-way or a public utility lot is required to permit access to inlets for maintenance purposes. Rural runoff inlets may be located within public lands controlled by authorities other than the City of Edmonton/EPCOR Water Services. However, location of inlets in easements on privately owned property are to be permitted only where warranted by special circumstances.
- 3.14.2 Gratings installed over the ends of rural runoff inlets shall be sized with hydraulic capacity of 200% of the design flow rate to allow for the effects of blockage or fouling of the grate by debris carried by the flow.
- 3.14.3 In general, the considerations of safety and aesthetics identified for sewer outfalls in Section **3.12.3** shall also apply to the design of rural runoff inlets.

3.15 Special Pipe Installation Methods

Where it is proposed to install sewers by special methods, for example tunnelling, jacking, or boring, or where the pipe passes through fill sections or unstable ground, then design loadings and details of the methods to be used for installing and supporting the pipe are to be submitted for EPCOR Engineer's approval.

Refer to the Construction Specifications for details relating to installations by these special methods.

3.16 Crossing Over an Existing Sewer

If the height from underside of the proposed water or sewer pipe to top of sewer pipe is less than 800 mm, excavate around the existing pipe to firm ground; place cement stabilized granular bedding around the existing pipe and up to the underside of the bedding specified for the sewer or water pipe. The top of the stabilized bedding shall extend across the full width of the trench and 0.5 m on either side of the sewer pipe and shall slope downward and outward at 1:1 on both sides of the sewer pipe to firm ground.

4.0 STRUCTURAL DESIGN FOR PIPE

This section outlines responsibilities for materials design, the design basis, parameters, and performance criteria applicable to the selection of sewer pipe.

4.1 Responsibility for Structural Design

The professional engineer responsible for preparation of engineering drawings is also responsible for the structural design of sewer installations.

4.2 General Design Basis

4.2.1 Rigid Pipe

Rigid pipe shall be designed in accordance with “Guidelines for the Design and Installation of Rigid Gravity Sewer Pipe in the City of Edmonton” dated January 2008, located in **Appendix B**.

4.2.2 Flexible Pipe

Flexible thermoplastic pipe shall be designed in accordance with the “Guidelines for the Design and Construction of Flexible Thermoplastic Pipe in the City of Edmonton” dated January 2008, located in **Appendix C**.

4.2.3 Tunnelled Sewer

Tunnel sewers or sewer installed by tunnelling methods shall have their project requirements reviewed with the City/EPCOR Water Services on a project specific basis.

4.3 Methods of Analysis

4.3.1 Common and/or Parallel Trench Pipe Installation

- i. While the City does not prefer installation by common trench methods it acknowledges that common trench installations are required on certain installations.
- ii. Where common and/or parallel trench is contemplated, the Engineer shall carry out an appropriate assessment of any specialized loading cases that may arise. While the Engineer is permitted to utilize any recognized loading theory at their professional discretion to address specialized loading cases a thorough analytical approach that addresses parallel pipes and trenches is presented in “Buried Pipe Design”, 2nd edition, by A.P. Moser, pp 121-136.

4.4 Documentation and Submission of Design Calculations

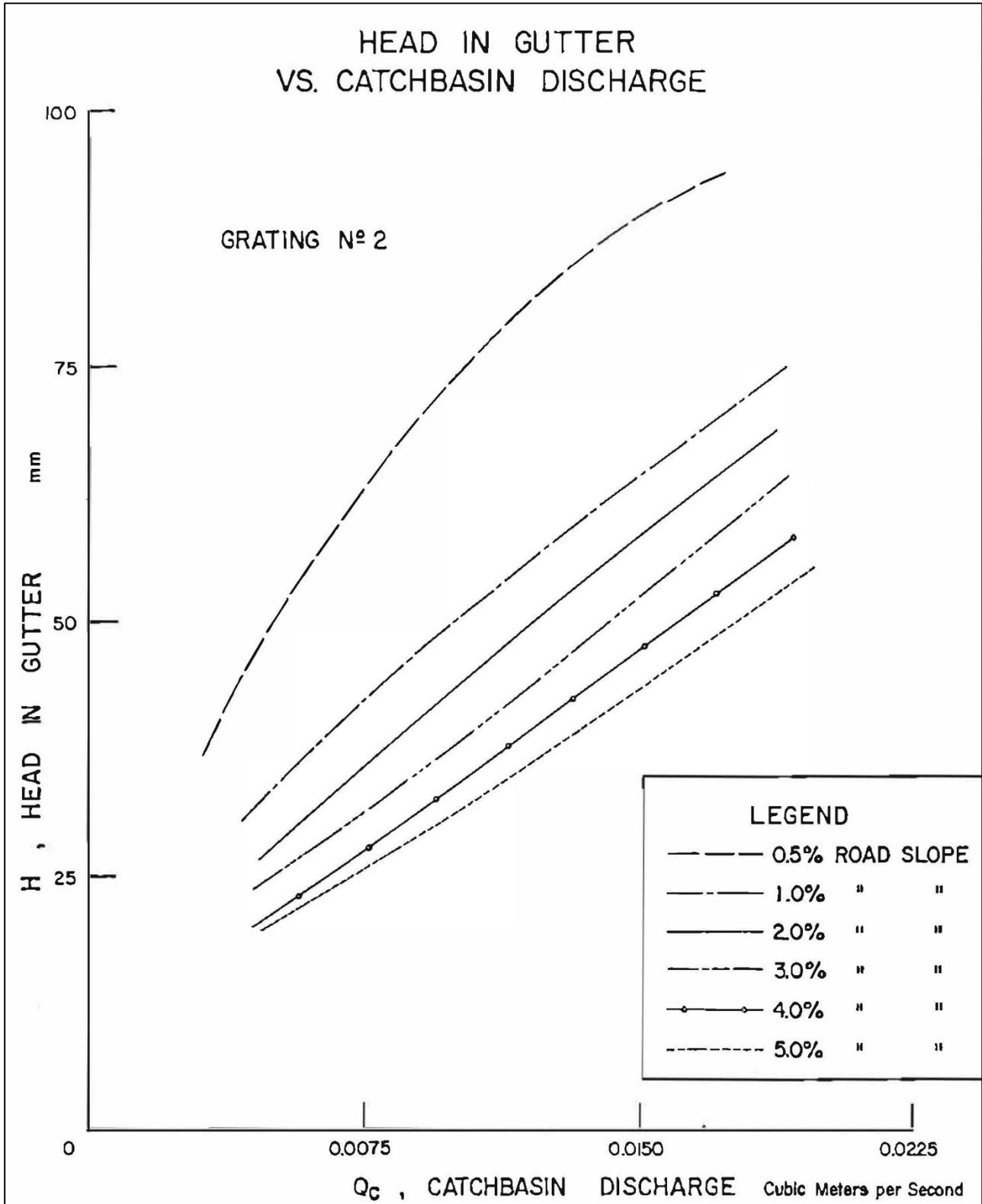
4.4.1 In all cases, design engineers shall keep a record of structural design calculation associated with each project in accordance with the appropriate Standard Practice. Design calculations for specific projects shall be provided to the City/EPCOR upon request.

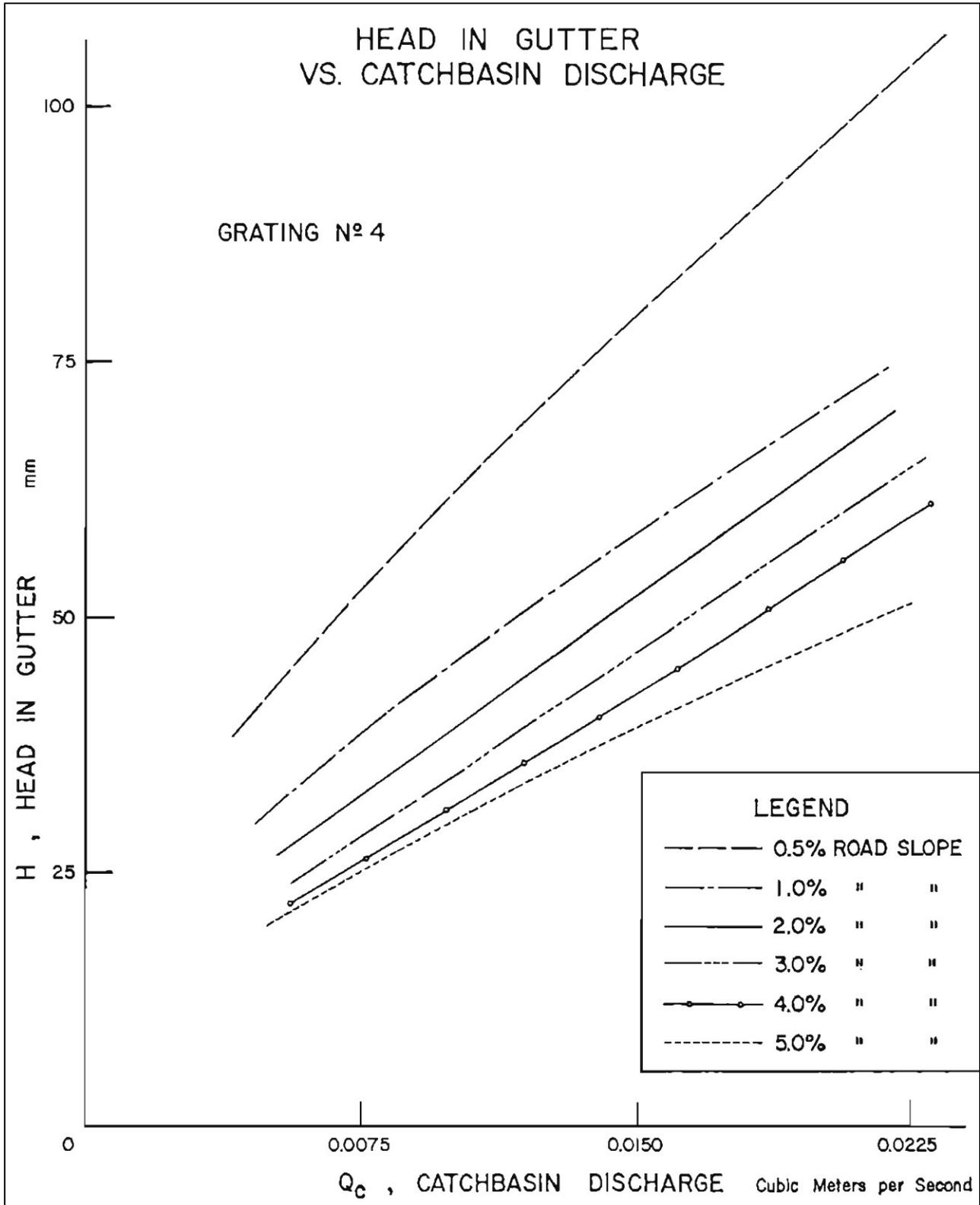
4.4.2 For rigid pipe designs in accordance with the ASCE 15-17, “Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD)”, the submission of pipe design calculations is mandatory.

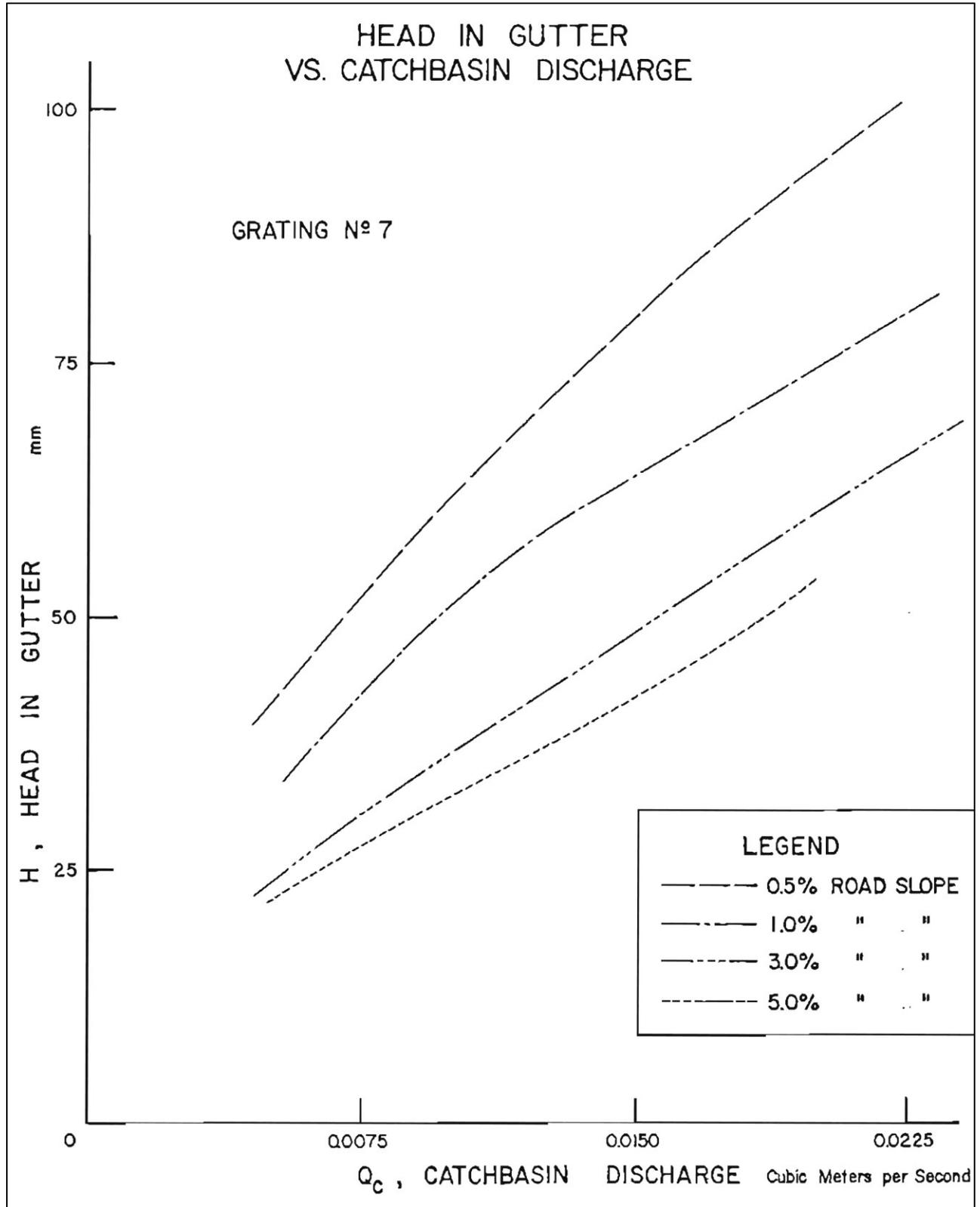


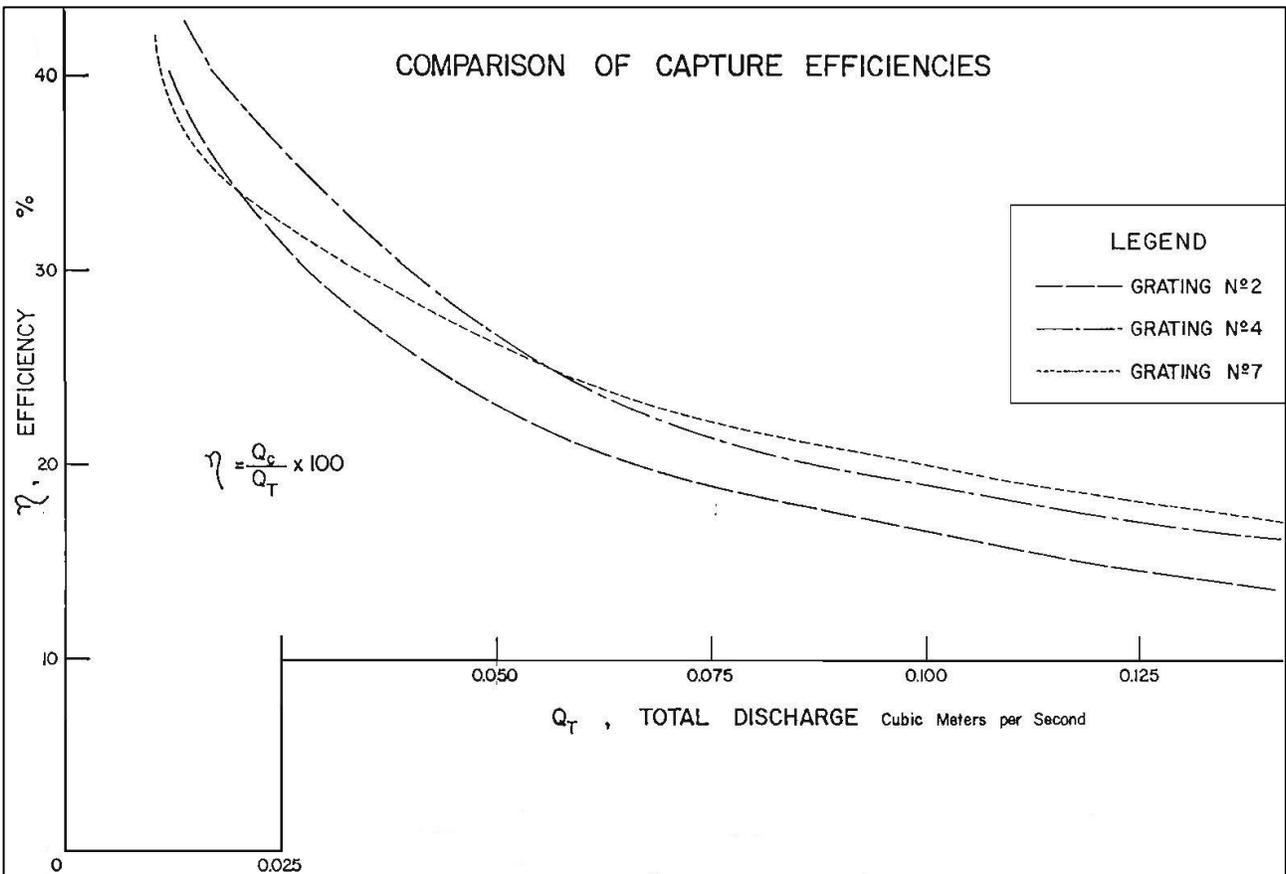
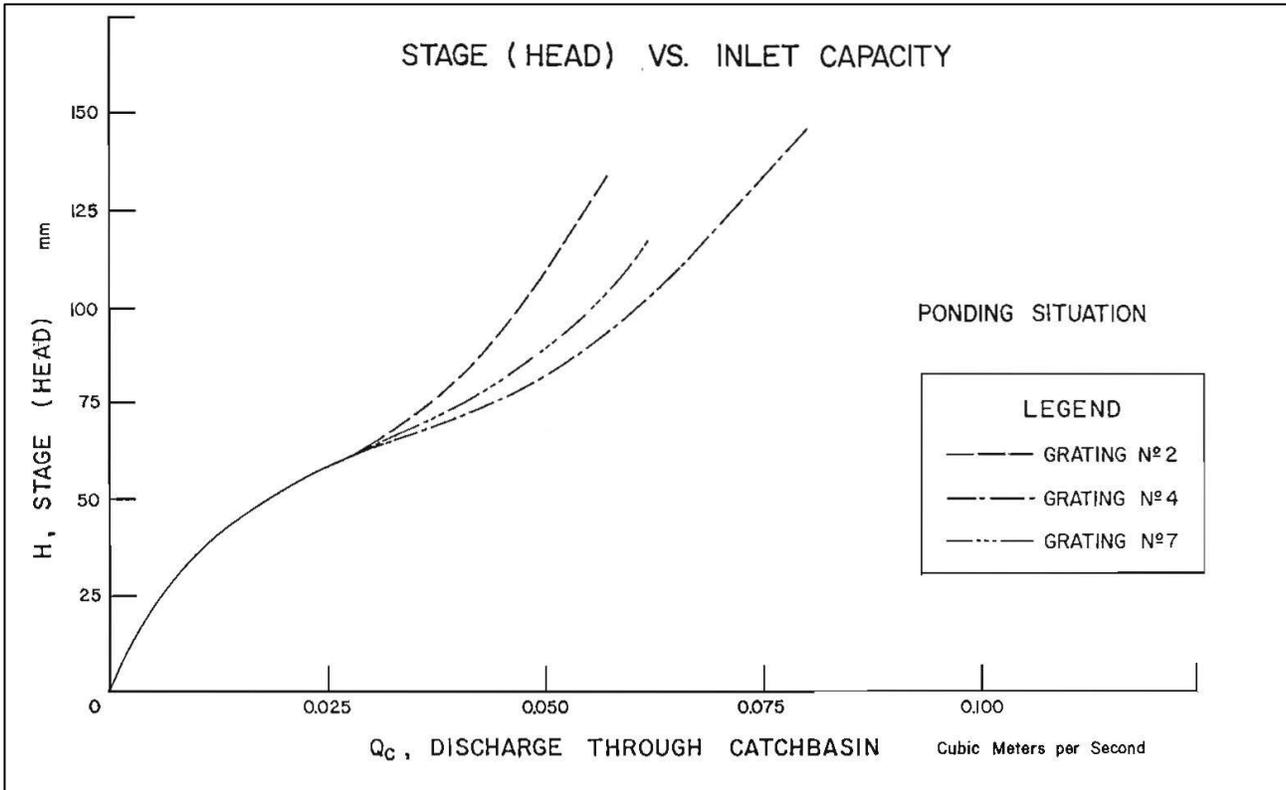
Appendix A

Catch Basin Inlet Capacity Curves

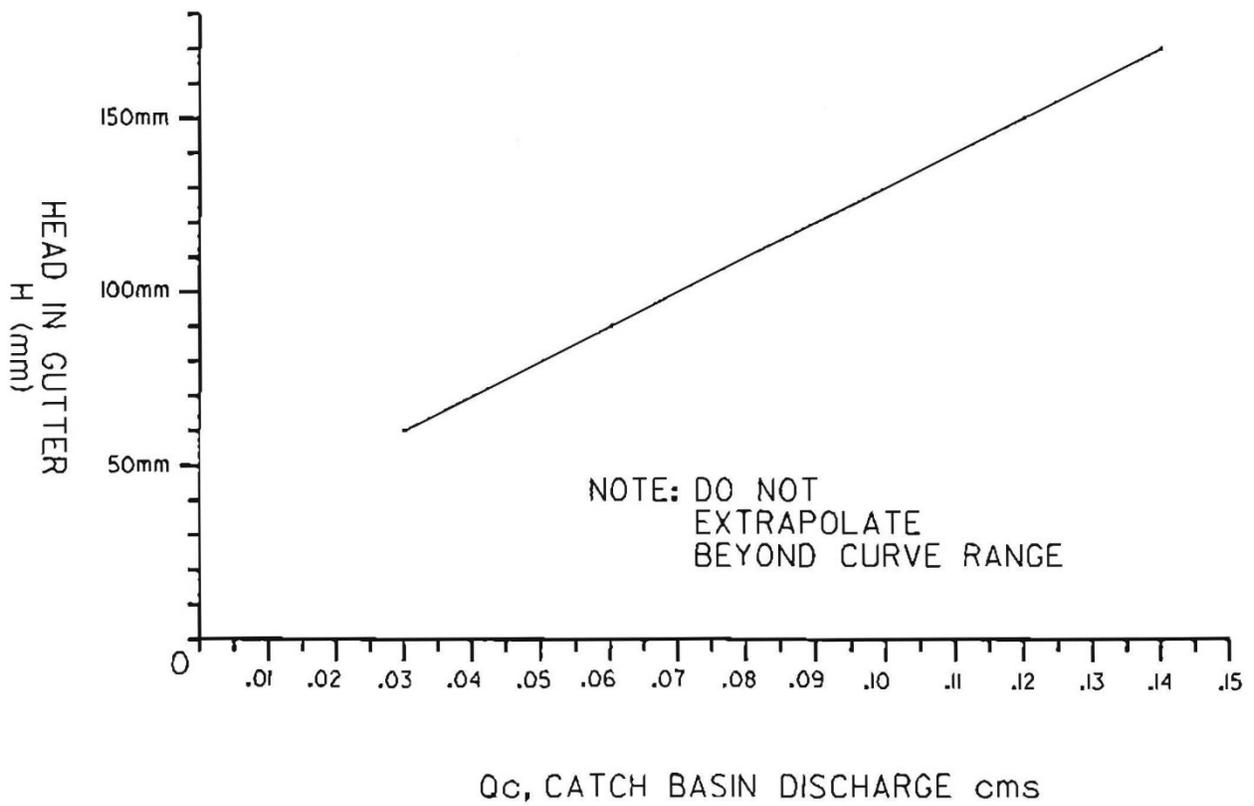


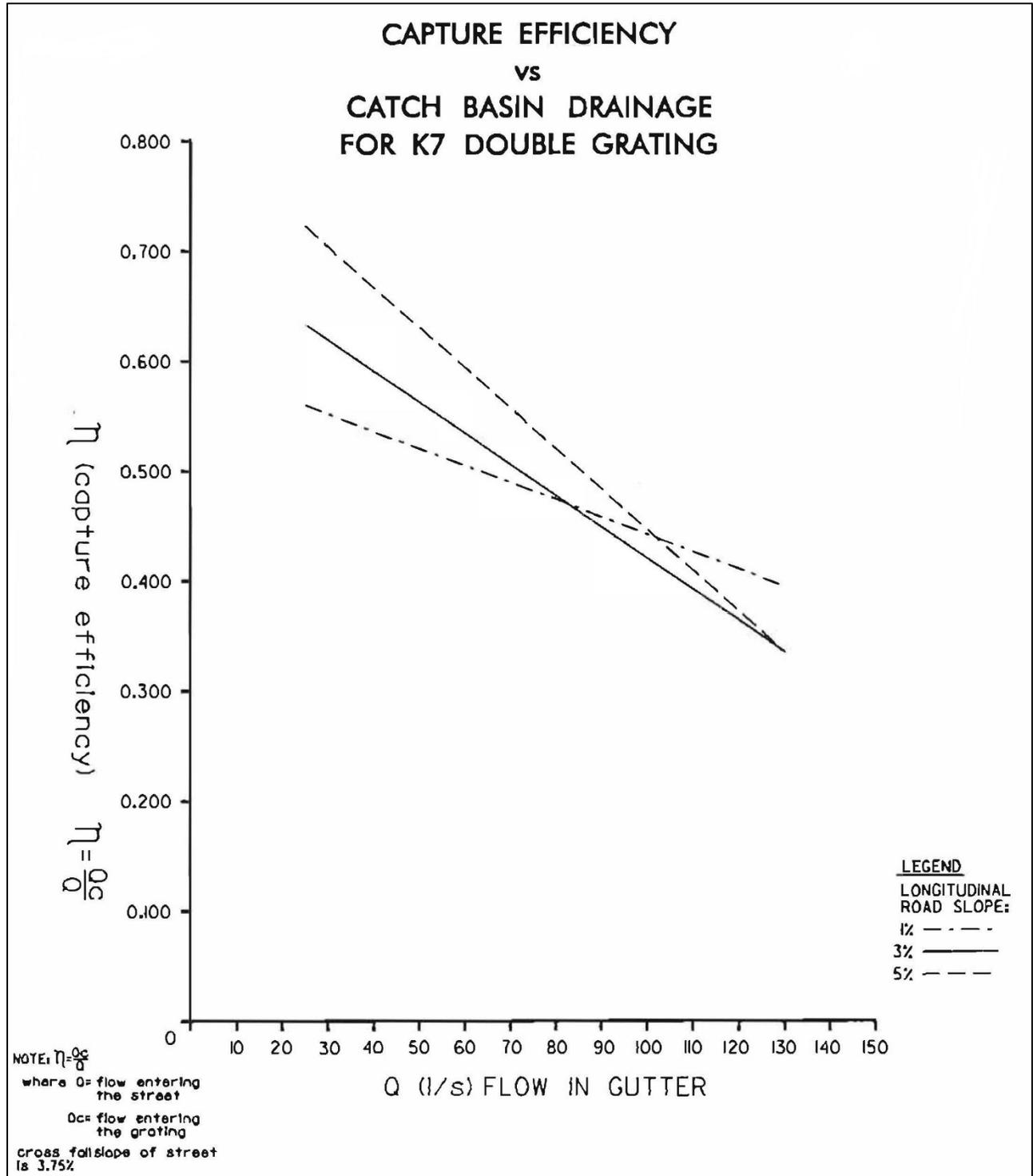


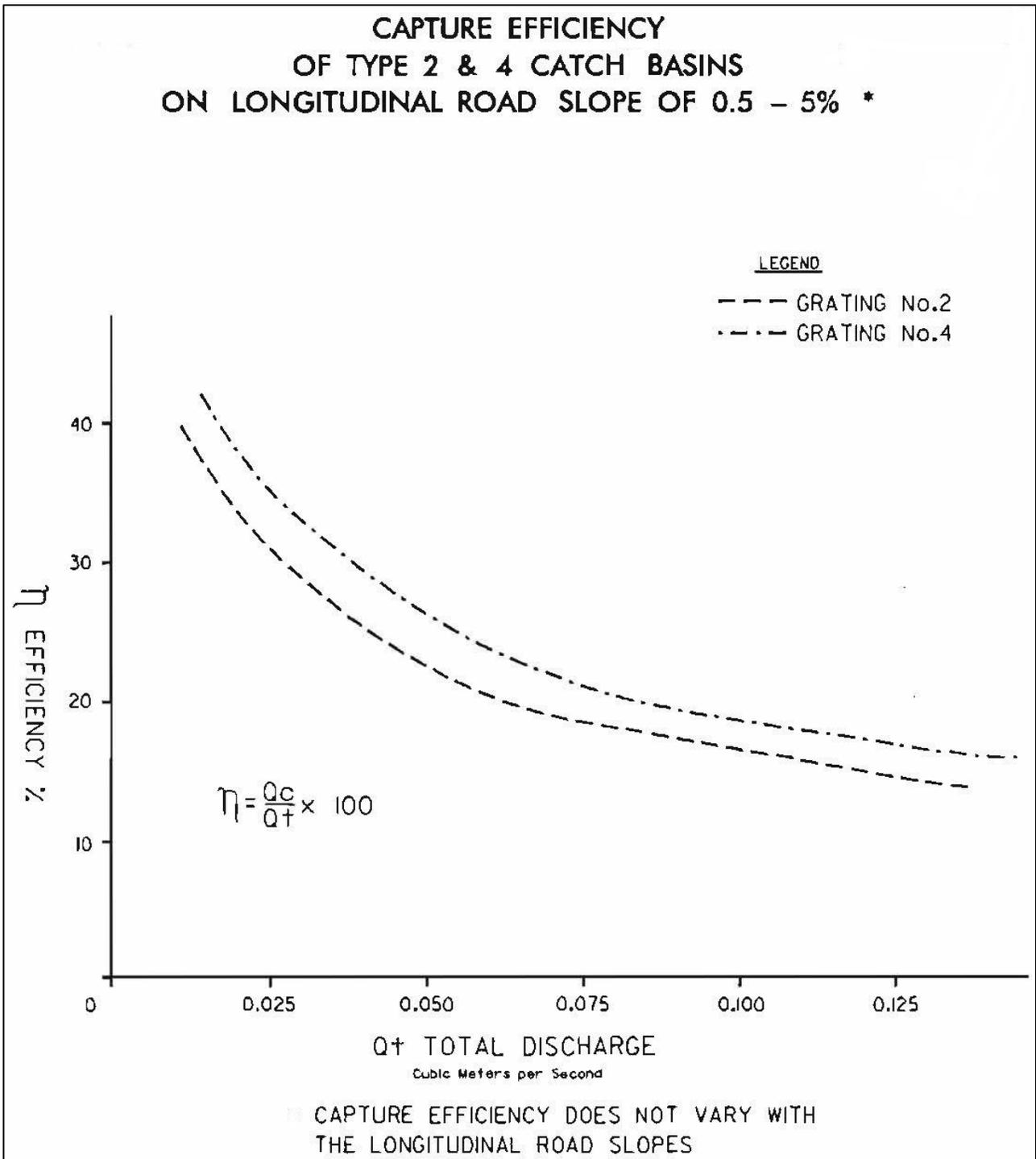




HEAD IN GUTTER
vs
CATCH BASIN DISCHARGE
FOR K7 SINGLE GRATING
IN A SAG LOCATION



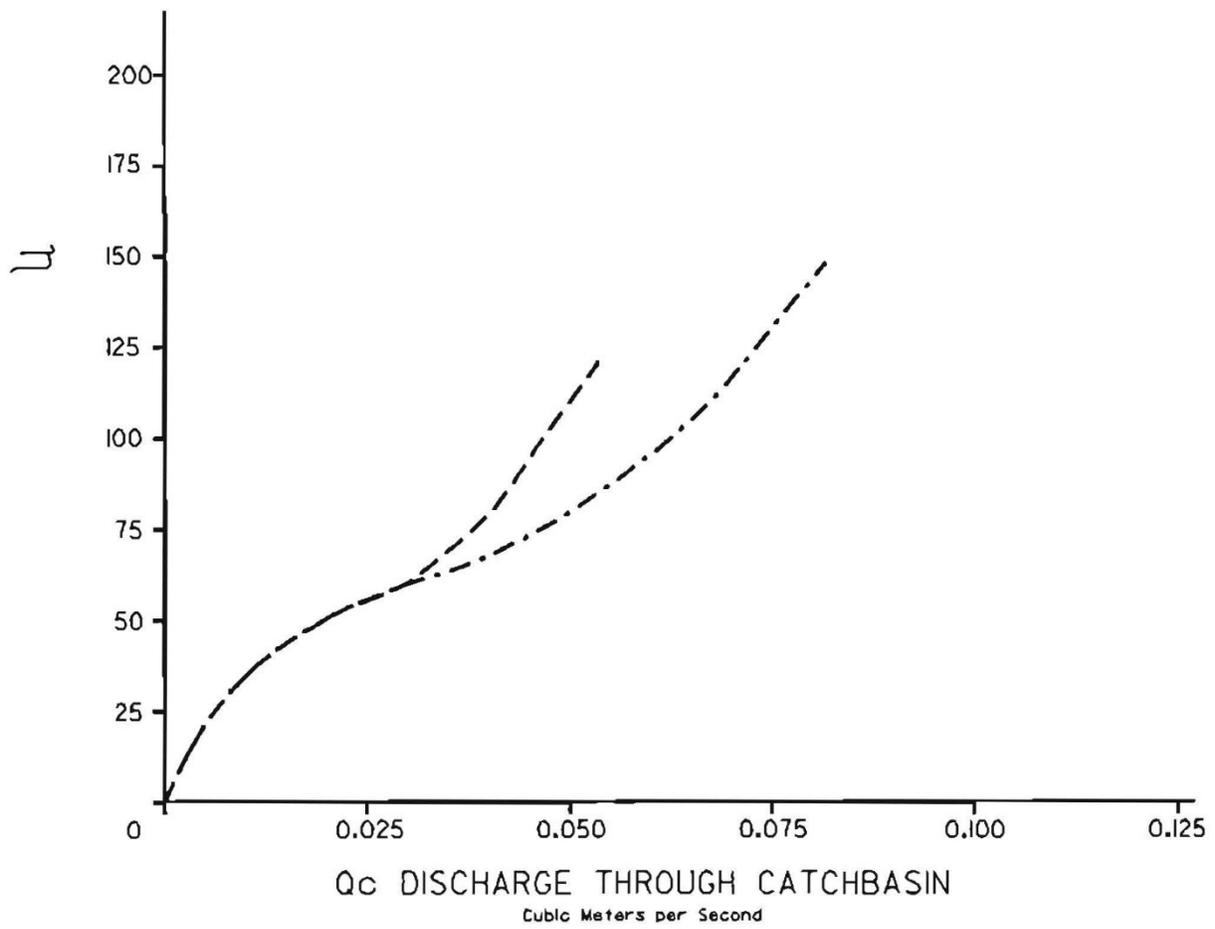




INLET CAPACITY FOR TYPE 2 & 4 CATCH BASINS FOR PONDING SITUATIONS

LEGEND

- - - GRATING No.2
- . . . GRATING No.4





Appendix B

Guidelines for the Design and Installation of Rigid Gravity Sewer Pipe in the City of Edmonton

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1.0 INTRODUCTION

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

The Guidelines 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 Guidelines 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 Edmonton as well as general guidance as to what situations are most applicable for each design method.

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

The design procedures given in this Guidelines 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 Handbook¹ while the Guidelines of Direct Design is detailed in ASCE Standard Practice 15-98².

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 Section **4.0** of this Guidelines.

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 the 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.

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

1.1.1 Establish Basic Design Criteria

- Inside diameter of pipe
- Height of cover and unit weight of earth
- Surface design loads

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

- 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)
- 1.1.2 Determine Design Loads and Earth Pressure Distribution
- In direct design applications earth loads and soil 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 load determination is carried out in an identical manner for direct and indirect design.
- 1.1.3 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 Guidelines and whether the designer is working with reinforced or non-reinforced pipe. Non-reinforced pipe is not permitted in direct design applications.
- 1.1.4 Perform Structural Analysis
- In direct design structural analysis involves a comprehensive determination of all moments, thrust, and shears produced by the applied design loads.
 - In indirect design, structural analysis is limited to applying the appropriate bedding factors to the applied design loads.
- 1.1.5 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 assess pipe design requirements.

- Typical loads that must be considered when analysing 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:

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

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):

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

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

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

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 a form a soil-pipe interaction through the use of bedding factors. In this Guidelines 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 relate 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 2.1** below) to define the distribution of loads within the embedment zone in response to the applied vertical and horizontal loads.

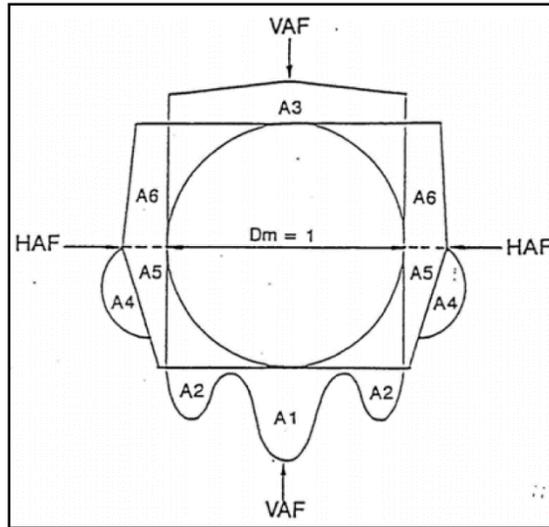


Figure 2.1: Heger Pressure Distribution

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

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

where,

VAF = vertical arching factor

W_e = total vertical earth load

PL = prism load

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

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:

$$\text{Ratio of Lateral to Vertical Load} = \frac{HAF}{VAF}$$

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

$$PL = w \left[H + \frac{D_0(4 - \pi)}{8} \right] D_0 \quad (2-7)$$

where,

w = unit weight of soil (lbs/ft³)

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 Edmonton, 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 Edmonton are depicted in Standard Drawing DR-06-13-01. 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 Standard Drawing DR-06-13-01. While Class A bedding is still permitted for use in Edmonton under special design cases it is not a recognized Standard Installation Type nor is it recommended for widespread use.

Based on the use of the minimum trench widths and the materials noted in the City of Edmonton Standard specifications, the VAF and HAF values noted in **Table 2.1** shall be used for design for each installation type.

Table 2.1: 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 Edmonton³. The embedment soil response to applied loads is largely reflected in pressure bulbs A1, A2, A4, and A5 in **Figure 2.1**, 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 2.2** for each of the ASCE Standard Practice Installations.

³ Simpson, Gumpertz, & Heger, Inc., "Instrumented Concrete Pipe test, Cranston Development, Edmonton, Alberta", February 1999.

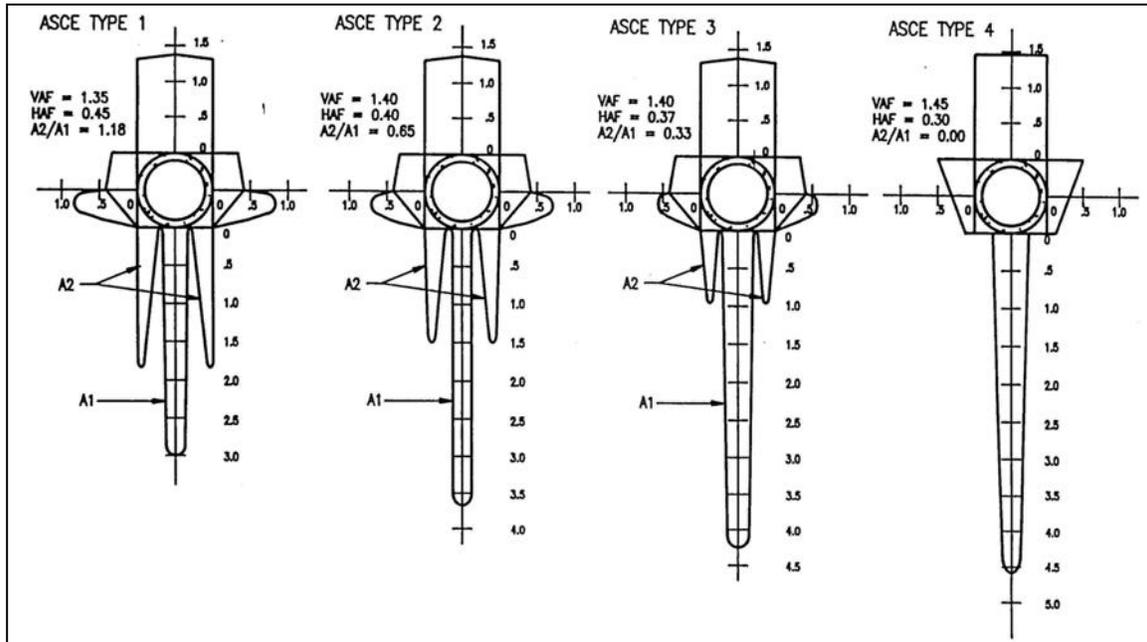


Figure 2.2: Pressure Distributions Associated with Standard Installations

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 2.3** below.

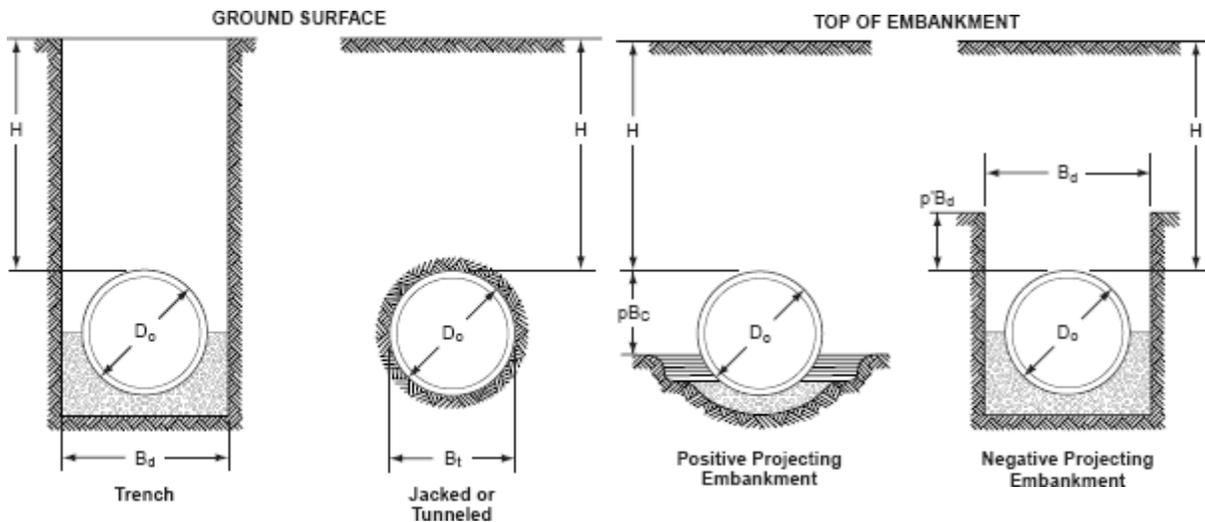


Figure 2.3: Marston-Spangler Installation Types – Essential Features

This Guidelines 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 Guidelines 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:

$$W_e = C_d w B_d^2 \quad (2-8)$$

where,

C_d = load coefficient as defined below

w = unit weight of soil (lb/ft³)

B_d = trench width at top of pipe (ft)

And, C_d can be determined by the following expression

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

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 2.2** below.

Table 2.2: 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

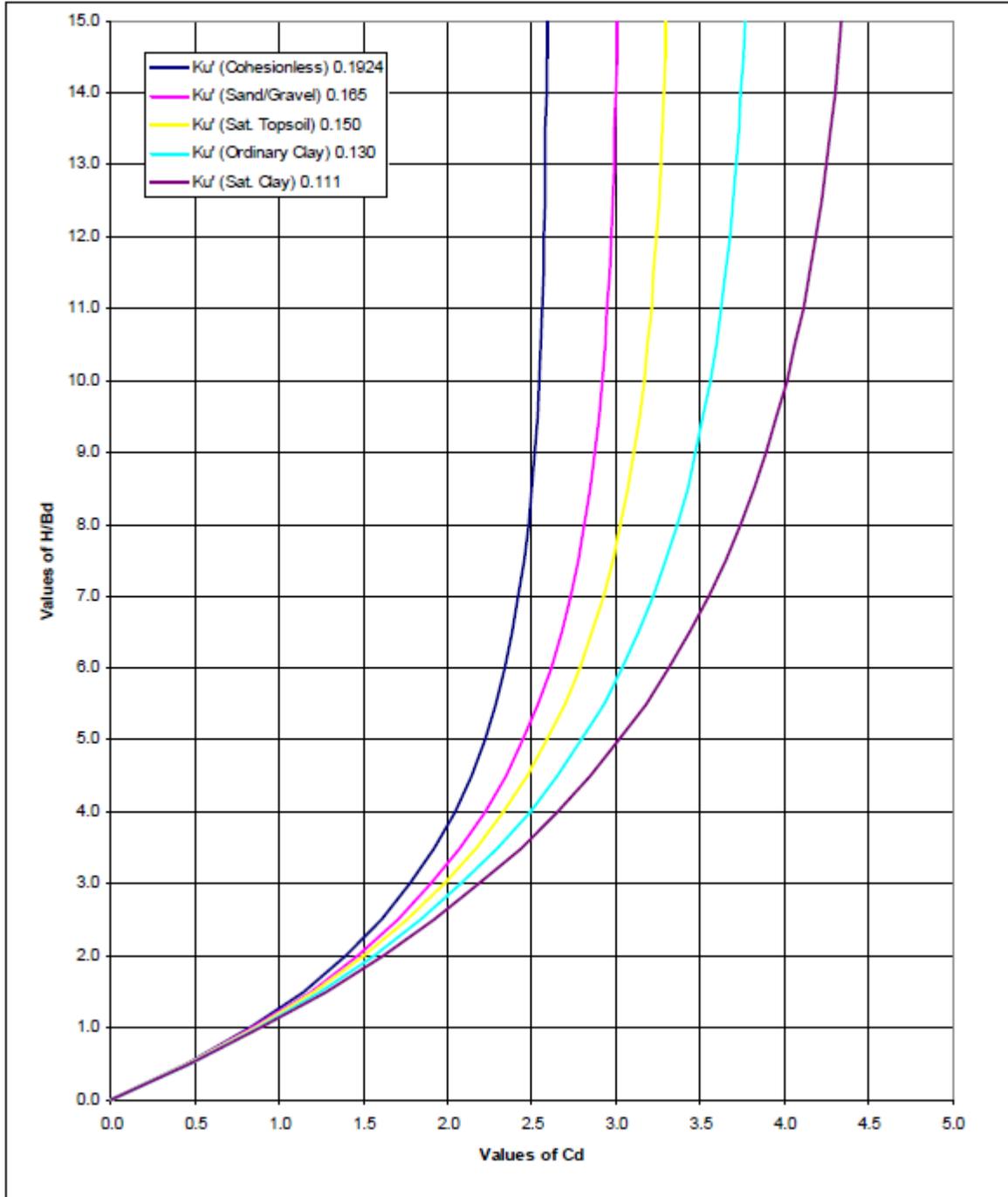


Figure 2.4: Trench Load Coefficient, C_d

Earth loads are normally calculated for either the greater of utilizing sand and gravel backfill with a density of 135 lb/ft³ (2165 kg/m³) or saturated clay backfill with a density of 120 lb/ft³ (1920 kg/m³). Guidelines in the City of Edmonton 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 2.4**. 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:

$$W_e = C_c w B_c^2 \quad (2-10)$$

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 Guidelines, 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 Edmonton are based on the prism load, PL , and vary according to Standard installation type with:

Prism Load equal to:
$$PL = w \left[H + \frac{D_0(4-\pi)}{8} \right] D_0$$

And the embankment condition earth load determined by:

$$W_e = VAF \times PL \quad (2-11)$$

The values for *VAF* vary in accordance with the Standard Installation Type as detailed in **Table 2.1** 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 Guidelines 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 2.5** 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.

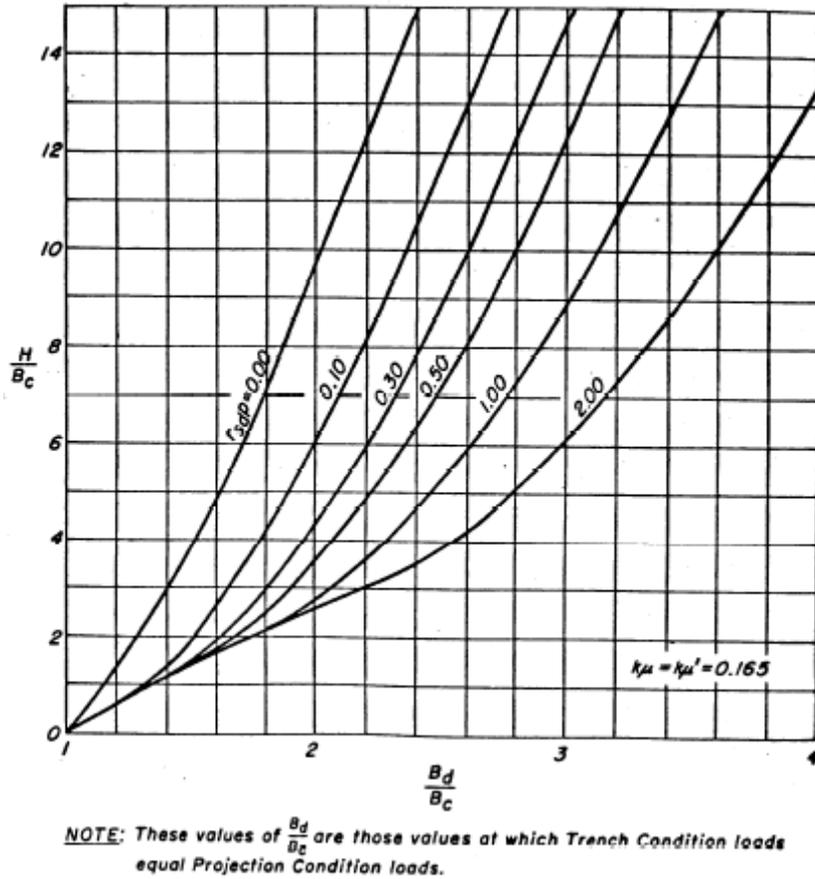


Figure 2.5: Transition Width Ratios⁴

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 classifications included largely qualitative terms ranging from impermissible, ordinary, and first class bedding as depicted in **Figure 2.6** below.

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

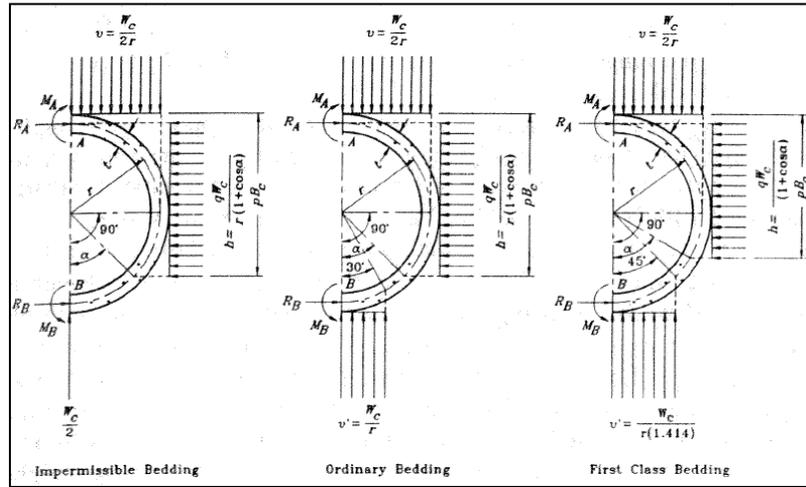


Figure 2.6: Marston-Spangler Load Distribution Assumptions for Embankment Conditions

The load response requirements of the pipe in Marston-Spangler analysis is carried out by means of a bedding factor, Bf, 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 2.7** 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, Maintenance hole Sections, Catch Basins, and Fittings.

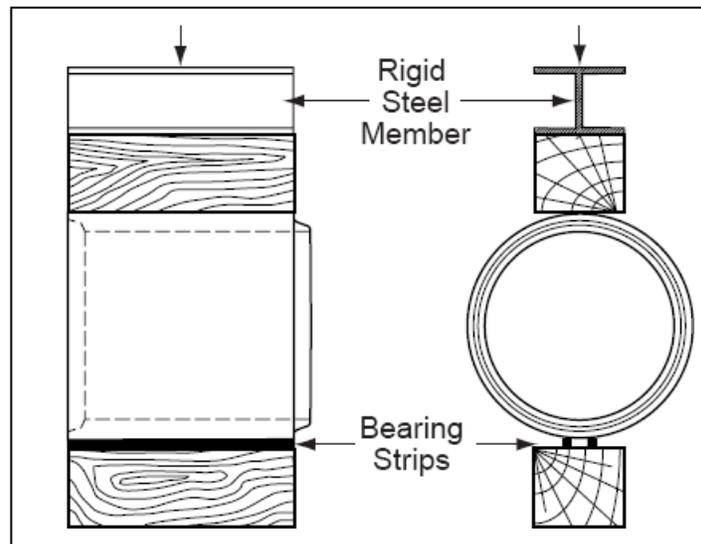


Figure 2.7: Three-Edge Bearing Load Test

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 2.6** 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 .

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

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:

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

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:

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

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

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:

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

Using the Standard Installations software design package provided by American Concrete Pipe Association 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 Handbook⁵, it is not very practical to utilize variable bedding factors in day-to-day practice.

This Guidelines 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 VAFs are utilized, however, full embankment 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 2.3** with the recommended bedding factors for use in indirect design noted in Table 2.4: Bedding Factors (B_f) for Standard Trench and Embankment Installations.

Table 2.3: Type of Bedding Factor to Use versus Design Approach

Method Used to Estimate Earth Load	Bedding Factor Selection
Heger VAFs as per Table 2.1	Use B_{fe} for Embankment Installation and appropriate Installation Type and Diameter from Table 2.4
Marston-Spangler Trench Loading as per Equation (2-8)	Use B_{ft} for Trench Installation and appropriate Installation Type from Table 2.4

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 2.4: Bedding Factors (B_f) for Standard Trench and Embankment Installations

Pipe Diameter	Type 1	Type 2	Type 3	Type 4
B_{ft} - Trench Installation				
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 than 1800 mm in diameter, the designer may interpolate between pipe diameters for the correct B_f . In instances where Class A bedding has been provided the designer should consult the Concrete Pipe Technology Handbook for guidance on bedding factor selection.

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 increasingly 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:

$$W_f = \frac{\pi D_i^2}{4} \times \gamma_w \quad (2-16)$$

where,

γ_w = unit weight of water (lb/ft³)

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:

$$W_f = 0.5454 \times 10^2 \times \gamma_w \times D_i^2 \quad (2-17)$$

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 crown 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 2.8**.

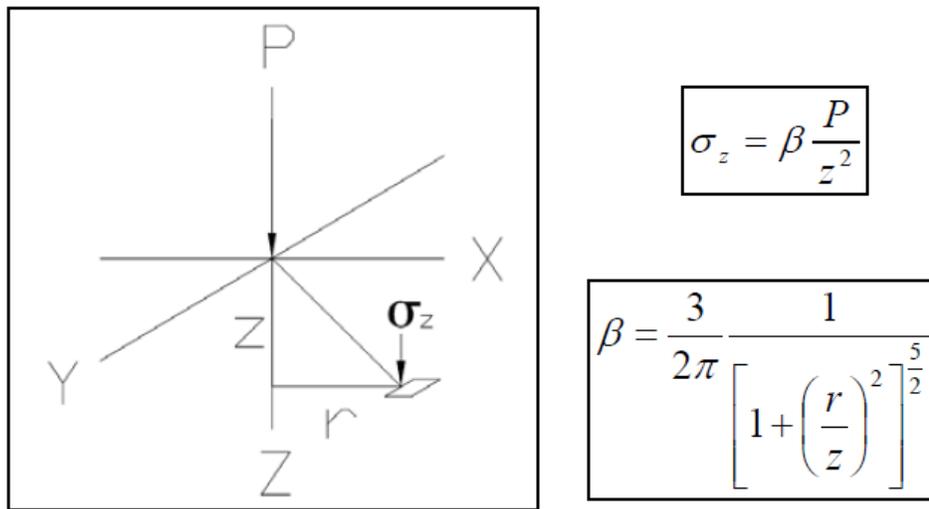


Figure 2.8: Boussinesq Equation Stress Distribution with Depth

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 2.9** is an example of two and three dimensional stress distributions for varying depth and distance from the point of origin.

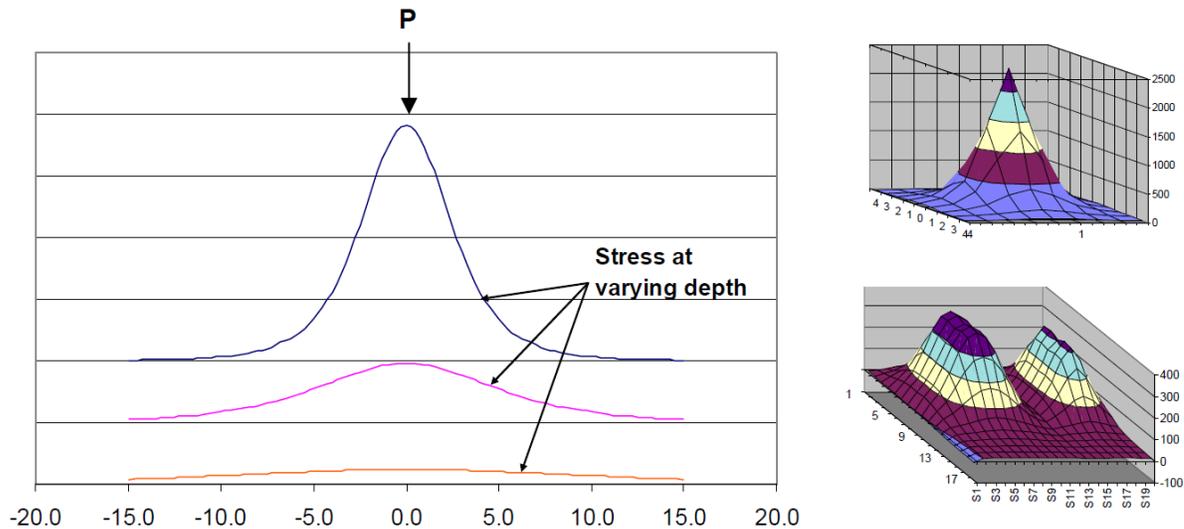


Figure 2.9: Effect of a Point Load Acting at Varying Depth and Distance from Origin

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 2.10 (a) shows the basic configuration for a concentrated point load acting over a rectangular area at depth. **Figure 2.10(b)** shows the basic configuration for a rectangular distributed load acting over a point at depth.

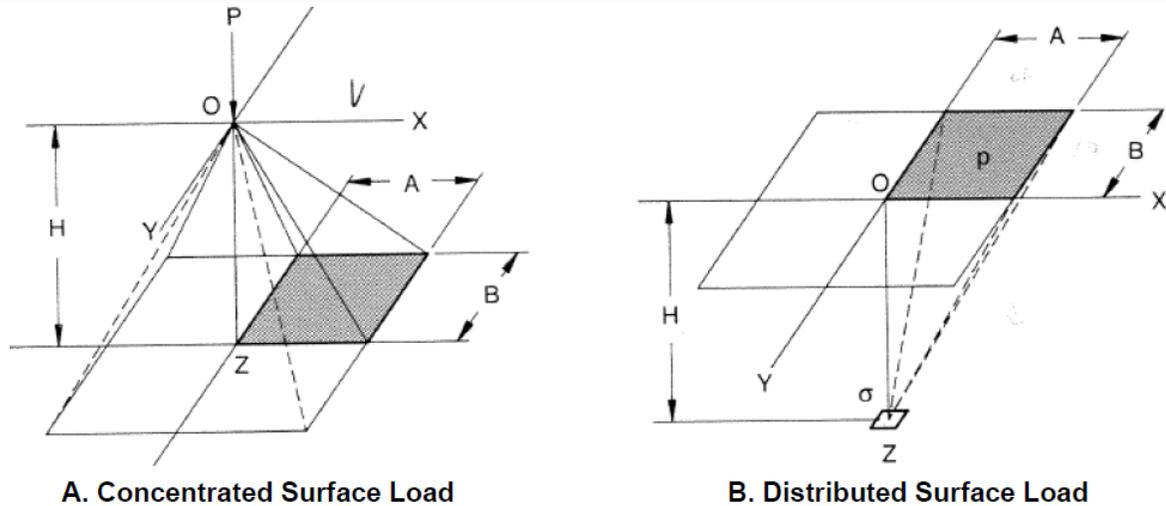


Figure 2.10: Basic Geometry and Theory for Boussinesq Integrations

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

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

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

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

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 2.10**.

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:

$$W_{AB} = C_t p \tag{2-18}$$

$$\sigma_{AB} = C_t p \tag{2-19}$$

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, C_t , are presented in **Table 2.5**.

Table 2.5: Values of Load Coefficient C_f for use in Holl's and Newmark's Integrations

n = B/H	m = A/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 applied 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 2.11** and **Figure 2.12**, for three typical loading cases for concentrated and distributed loads, respectively.

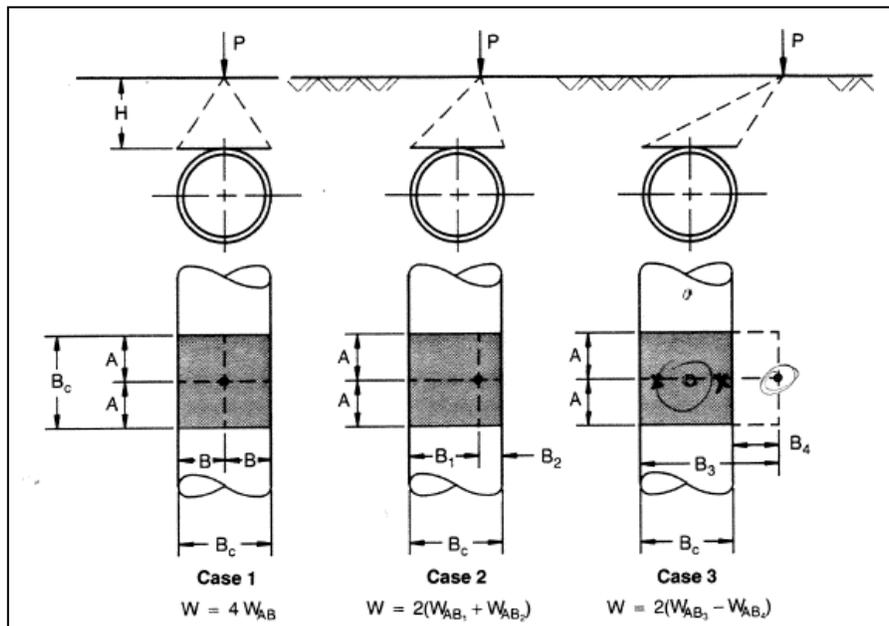


Figure 2.11: Procedure for Calculating Offset Concentrated Surface Loads

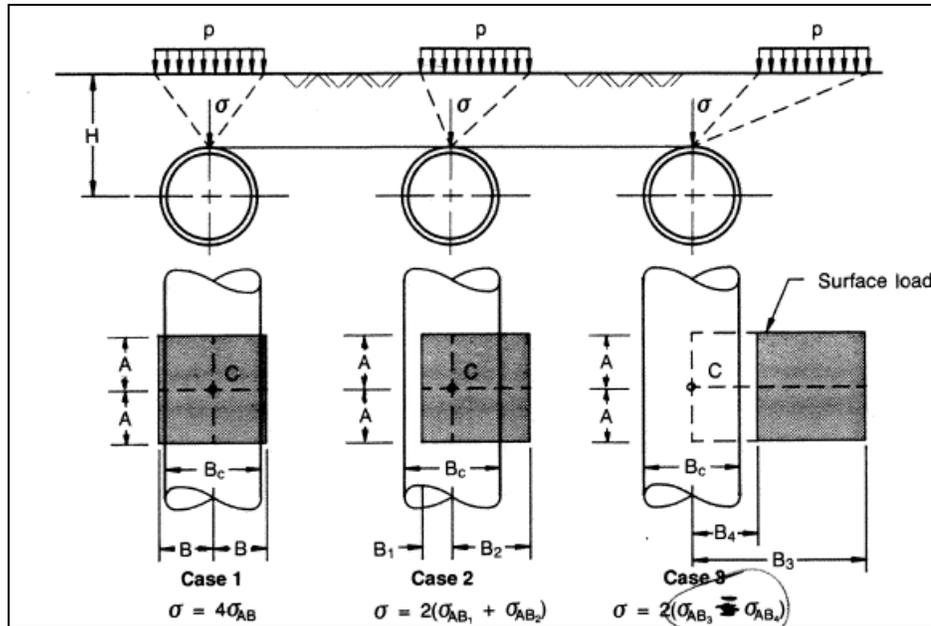


Figure 2.12: Procedure for Calculating Offset Distributed Surface Loads

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:

$$W_s = \frac{W_{AB}}{B_c} \tag{2-20}$$

And the following form for distributed loads:

$$W_s = \sigma \times B_c \tag{2-21}$$

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:

$$W_L = W_s(1 + I_f) \tag{2-22}$$

This Guidelines 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 Guidelines. Table 2.6 outlines recommended impact factors at varying depths of cover.

Table 2.6: Recommended Impact Factors for Vehicular Loads

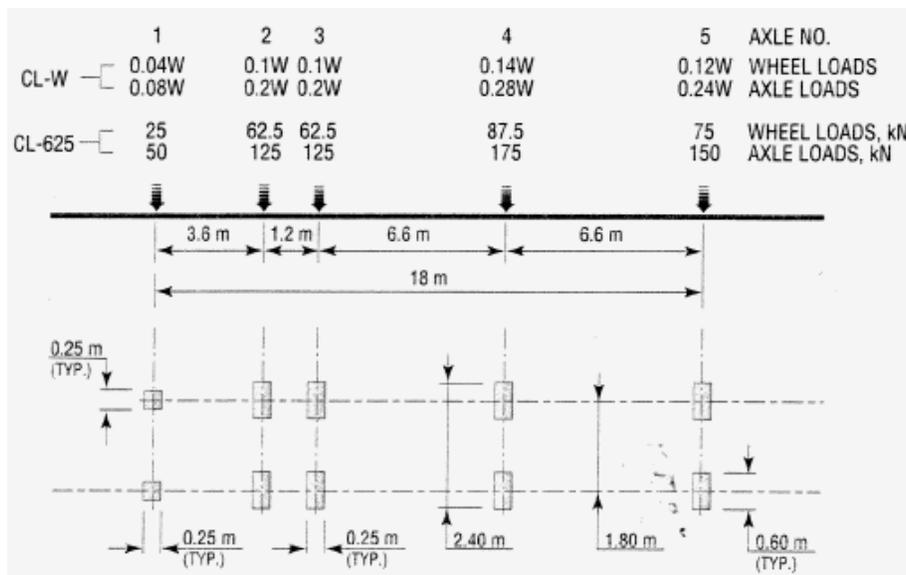
Impact Factor		
Cover (ft)	Cover (m)	I_f
1'-0"	0.30	0.50
2'-0"	0.61	0.50
2'-6"	0.76	0.43
3'-0"	0.91	0.38
3'-6"	1.07	0.30
4'-0"	1.22	0.23
4'-6"	1.37	0.17
5'-0"	1.52	0.10
5'-6"	1.68	0.04
5'-9"+	1.75	0.00

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 625 kN. The load is distributed over both sets of dual tires (each 0.60 m x 0.25 m), at approximately 1.80 m centre on centre. The per-axle load distribution for CL-W series trucks is shown in **Figure 2.13** from the CHBDC.



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

Figure 2.13: CL-W Truck load distribution⁶

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 2.7: 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 0.25 m x 0.51 m (10"x20") spaced at approximately 1.83 m (6.0 ft). The load is assumed to increase with depth in a pyramidal fashion as depicted in **Figure 2.14**.

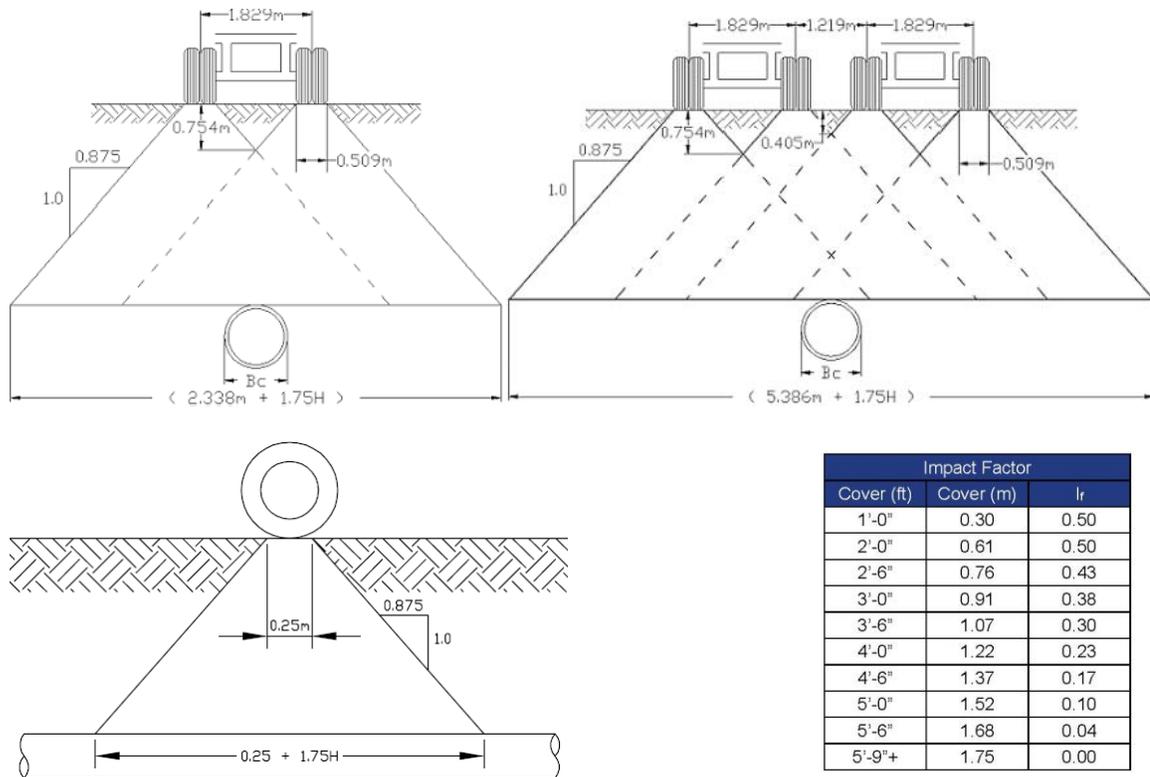
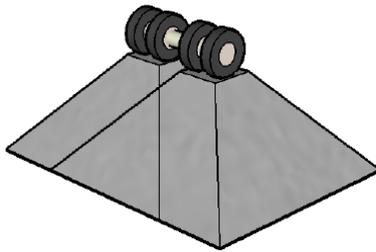


Figure 2.14: Zones of Influence and Impact Factors at Depth⁷

At a depth of 0.75 m (2.5 ft) 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 0.75 m, the single axle load can be divided by two. For depths greater than 0.75 m, the pressure can be calculated as noted in **Figure 2.15**.

⁷ Figure 2.7: Ameron Concrete Cylinder Pipe Design Manual 1988



$$w_L = \frac{\text{Single Axle Load (SAL)}}{(2.34 + 1.75H)(0.25 + 1.75H)}$$

SI Units

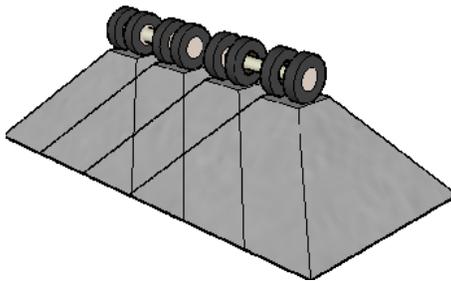
$$w_L = \frac{SAL}{(7.67 + 1.75H)(0.83 + 1.75H)}$$

Imperial Units

Figure 2.15: AASHTO Method for Single Vehicle Loads

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 2.16**.



$$w_L = \frac{\text{Double Axle Load (DAL)}}{(5.39 + 1.75H)(0.25 + 1.75H)}$$

SI Units

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

Imperial Units

Figure 2.16: AASHTO Method for Dual Passing Vehicles

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:

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

Minimum live loads to be covered by this Guidelines 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 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.

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

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 2.17** shows the suggested axle configuration and corresponding load.

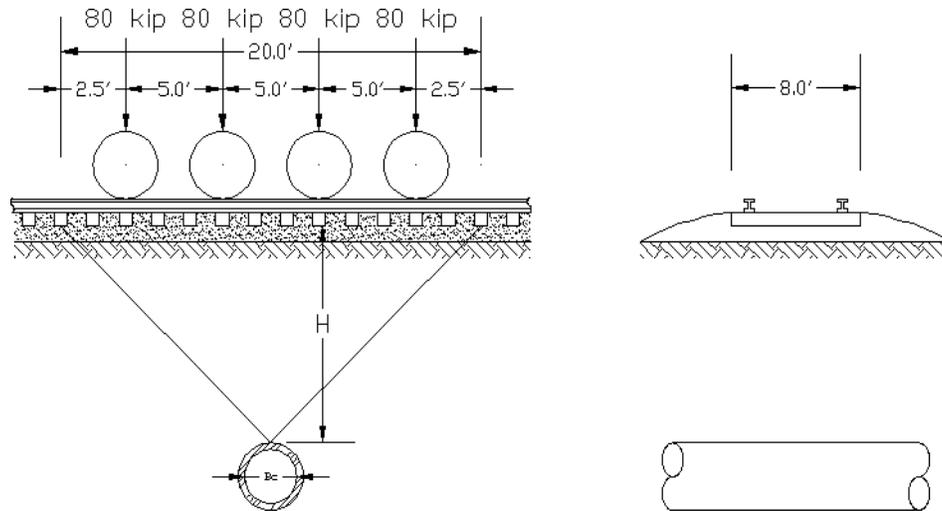


Figure 2.17: Cooper E-Series Axle Spacing and Load Configuration

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:

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

Where

p = distributed surface load in lb/ft²

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 for railway loading 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 2.18**.

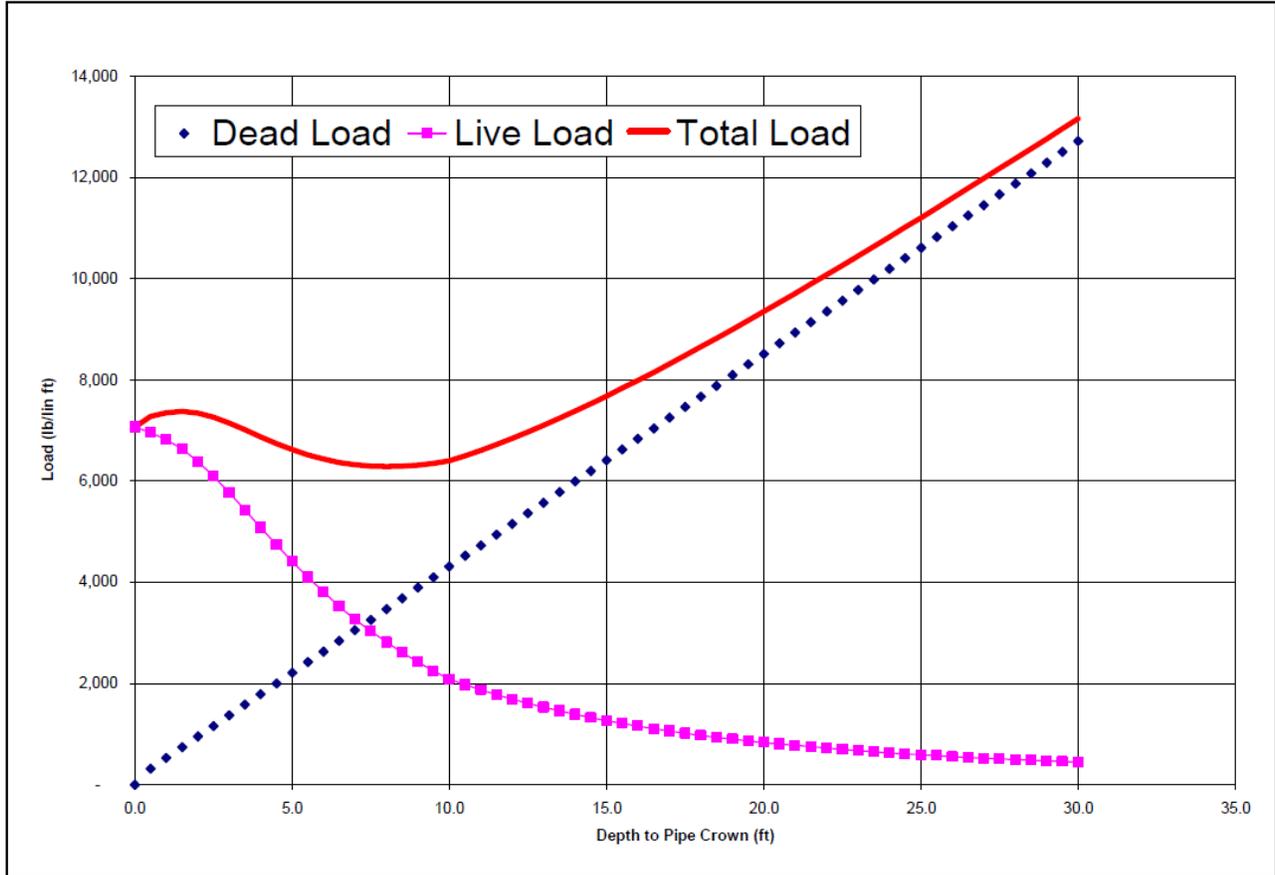


Figure 2.18: Typical Live and Dead Load Components with a Cooper E80 Live Load

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:

- i. Select design factors
- ii. Perform structural analysis
- iii. 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 Guidelines development was ASCE 15- 98⁹.

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:

- i. Service cracking based on the degree of crack control desired;
- ii. Ultimate flexural load;
- iii. Limiting conditions for concrete radial tension strength;
- iv. 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.

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

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 Edmonton:

- 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 $\theta = 10^\circ$ 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 Guidelines, 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 to evaluate Standard Installations.

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 Edmonton 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.

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 designs 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 Edmonton 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 software design package, 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

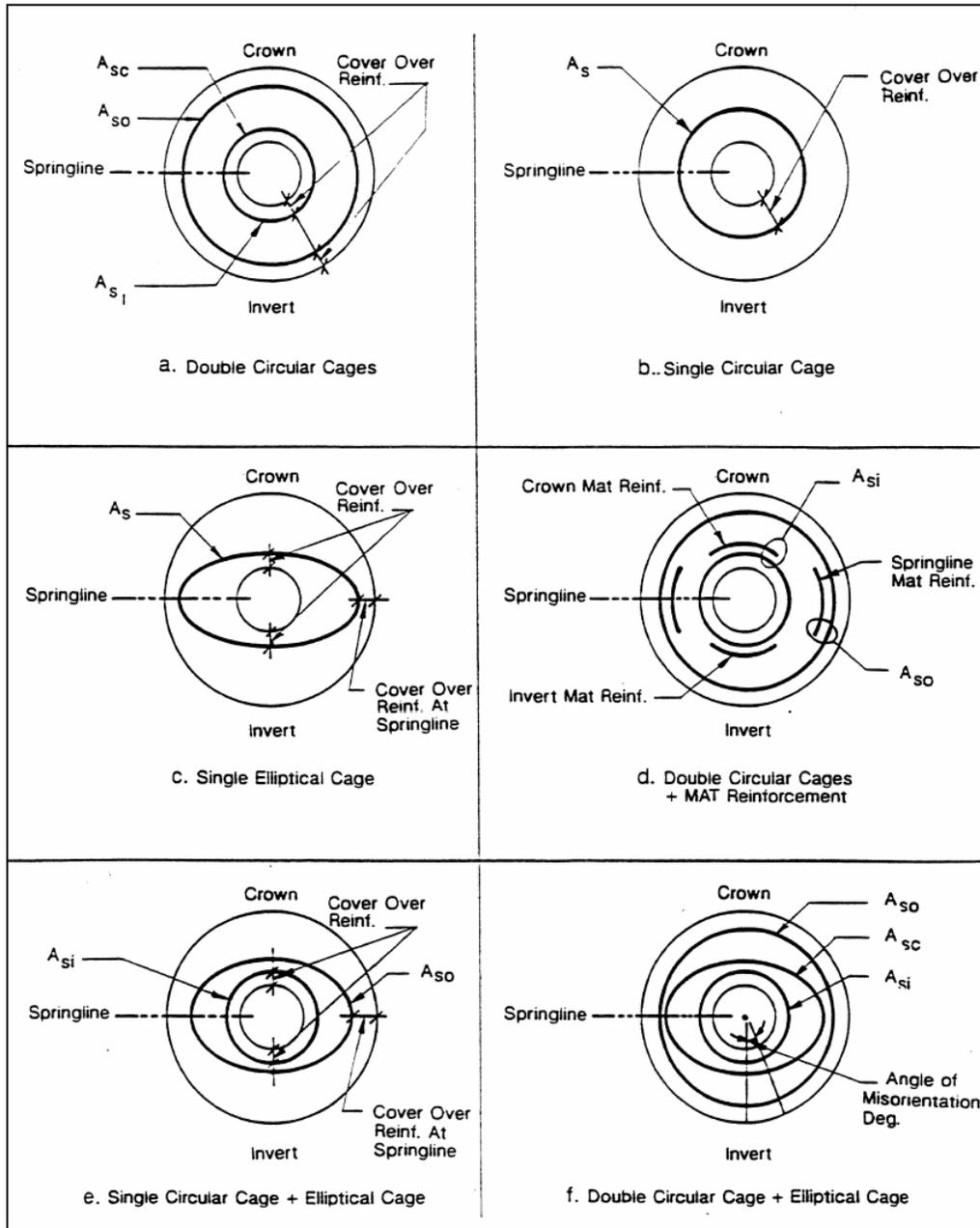


Figure 3.1: Typical Reinforcing Steel Arrangements

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 3.1** while **Figure 3.2** depicts a unique reinforcing scheme required to overcome excessive radial or diagonal tension.

The vast majority of designs can be accomplished with the use of steel arrangements a.) or b.) from **Figure 3.1** (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, have specific

handling considerations in the field that should be brought to the contractor's and field inspection personnel's attention.

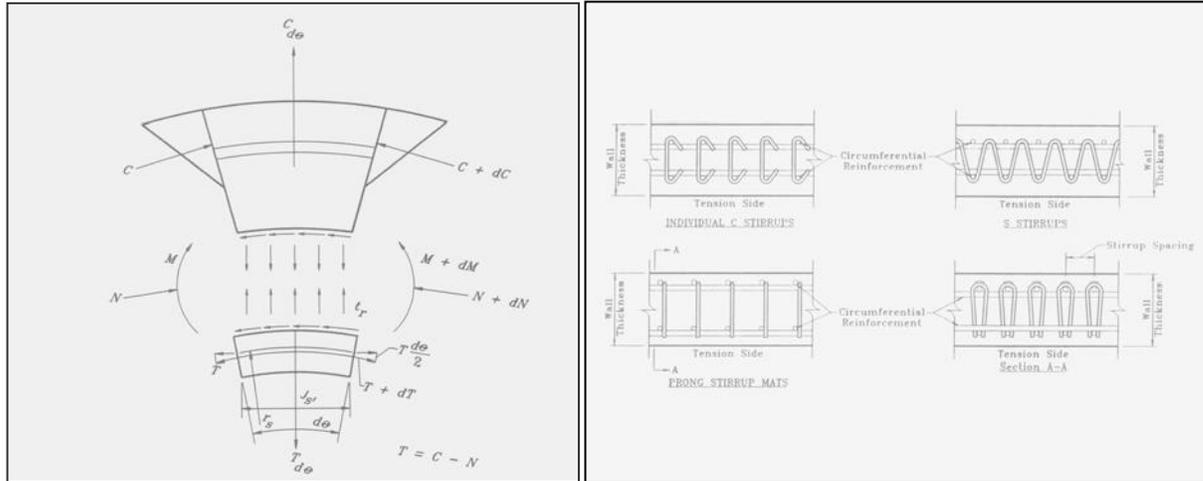


Figure 3.2: Stirrup Requirements and Arrangements

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 software design package 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) unless the manufacturer can produce a reliable rationale for higher design values.

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.

3.2 Indirect Design

In Section 2.0 of this Guidelines, 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:

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

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:

$$TEB = \frac{(W_e + W_L + W_f)}{B_f} \times FS \quad (3-1)$$

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.

Sample problems applying the indirect design method are contained in Section **5.0** of this Guidelines for the practitioner's convenience.

4.0 NOTATIONS FOR INDIRECT AND DIRECT DESIGN

Symbol	Definition	Units used in this Guidelines
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	
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

Symbol	Definition	Units used in this Guidelines
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

5.0 INDIRECT DESIGN – SAMPLE PIPE SELECTION PROBLEMS

EXAMPLE 1

Trench Installation and the Use of Marston-Spangler Trench Loading Theory

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/ft³]. 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 Marston-Spangler methods. The settlement ratio/projection ratio product (r_{sdp}), to ascertain an estimate of transition width, is approximately 0.7 for ordinary soil.

Determine the required pipe class for this situation.

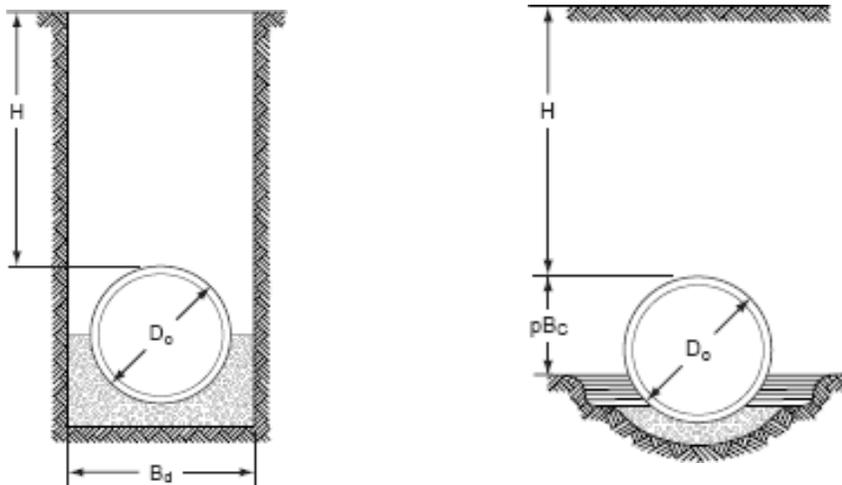


Figure: Example 1

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.

While the installation is in a trench, the trench width needs to be compared to the transition width to ascertain which of the two installation conditions noted in **Figure: Example 1** above, applies. The designer can reference **Figure 2.5: Transition Width Ratios**. Based on $r_{sdp} = 0.7$, and known values for H/B_c ($20/3 = 5.27$), it can be interpolated from **Figure 2.5** that:

$$\frac{B_{d-transition}}{B_c} \approx 2.4$$

Therefore, the transition width = $2.4 \times B_c = 2.4 \times 3.79 = 9.09$ feet

As the proposed trench width is less than the transition width trench loading theory applies and the earth load can be estimated by **Equation (2-8)** as follows:

$$W_e = C_d W B_d^2$$

where,

C_d = load coefficient as defined below

w = unit weight of soil (lb/ft³) = 135 lb/ft³

B_d = trench width at top of pipe (ft) = 3 + 2 × 2 = 7 feet

And C_d can be determined by **Equation (2-9)**:

$$C_d = \frac{1 - e^{-2K\mu' \frac{H}{B_d}}}{2K\mu'}$$

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 2.2**, where it can be seen that the maximum $K\mu'$ value for a sand and gravel backfill material is 0.165.

Based on this C_d can be determined to be 1.73 and the earth load, W_e is determined to be:

$$W_e = 1.73 \times 135 \times 7.79^2 = 14,173 \text{ [lb/ft]}$$

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 2.16**:

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

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

$$w_L = 53 \text{ [lb/ft}^2\text{]}$$

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 2.6** outlines recommended impact factors at varying depths of cover).

Therefore:

$$W_L = 53 \times 3.79 = 201 \text{ [lb/ft]}$$

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/ft³]. Thus from **Equation (2-16)**:

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

4. Selection of Bedding Factor

As the designer has chosen to use Marston Loading theory for trenches and we have determined that trench loading as opposed to embankment loading conditions exist, we should use trench bedding factors from **Table 2.4**: Bedding Factors (B_f) for Standard Trench and Embankment Installations. While this is a very conservative assumption it is the only valid assumption that is permitted by the Guidelines when using Marston Trench Load Theory and the practical considerations of an unregulated trench width during the construction phase.

Based on a 36" diameter pipe and a Type 2 Installation, the $B_{fi} = 1.9$.

5. Pipe Strength Requirement

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

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

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:

$$TEB = \frac{(14,173+201+441)}{1.9} \times 1 = 7797 \text{ [lb/ft]}$$

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

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

Therefore:

$$D_{0.01} = \frac{7797}{3} = 2599 \text{ [lb/ft/ft]}$$

As per ASTM C76 and Section **3.2.1**, $D_{0.01} = 2599$ [lb/ft/ft] correlates to a CL-V pipe. The completed design has actual FOS against service cracking and ultimate failure as follows:

$$\text{Service cracking} \quad FOS = \frac{D_{0.01 \text{ ClassV}}}{D_{\text{Applied TEB}}} = \frac{3000}{2599} = 1.15$$

$$\text{Ultimate} \quad FOS = \frac{D_u \text{ ClassV}}{D_{\text{Applied TEB}}} = \frac{3750}{2599} = 1.44$$

As these are both greater than our design objectives (FOS of 1.0 for service cracking and 1.25 for ultimate for TEB capacity greater than or equal to 3000 lb/ft/ft diameter) the design is adequate.

EXAMPLE 2

Use of Heger VAFs 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/ft³]. 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 VAFs and, therefore, it is not required to determine transition width and accordingly no estimate settlement ratio/projection ratio product (r_{sdp}) is required to be made.

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

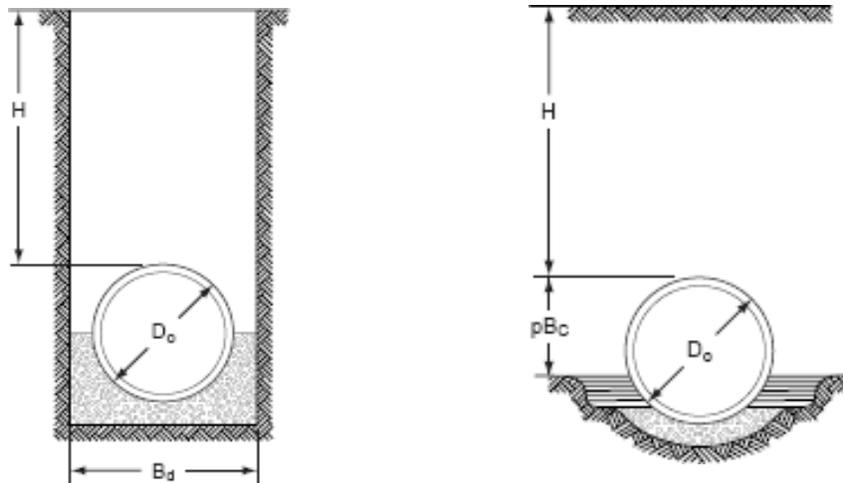


Figure: Example 2

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 VAFs to estimate earth loading, earth loads are already based on their most conservative values, embankment conditions (as depicted to the right of the **Figure: Example 2**). 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:

$$W_e = VAF \times PL \text{ [lb/ft]}$$

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/ft³]

H = height of fill above pipe [ft]

D_o = outside diameter of pipe [ft]

Based on **Table 2.1** - VAF and HAF for Standard Installations, VAF for a Type 2 Installation would be 1.40. Therefore:

$$PL = 135 \left[20 + \frac{3.79(4-\pi)}{8} \right] \times 3.79 = 10,441 \text{ [lb/ft]}$$

For a Type 2 installation, VAF = 1.40, therefore $W_e = 1.40 \times 10,104 = 14,617$ [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 2.16**:

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

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

$$w_L = 53 \text{ [lb/ft}^2\text{]}$$

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 2.6**).

Therefore:

$$W_L = 53 \times 3.79 = 201 \text{ [lb/ft]}$$

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/ft³]. Thus from **Equation (2-16)**:

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

4. Selection of Bedding Factor

As we are using Heger VAFs 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 2.4** - 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 2.4** as $B_{fe} = 2.9$.

5. Pipe Strength Requirement

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

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

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:

$$TEB = \frac{(14,617+201+441)}{2.9} \times 1 = 5262 \text{ [lb/ft]}$$

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

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

Therefore:

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

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:

$$\text{Service cracking} \quad FOS = \frac{D_{0.01 \text{ Class IV}}}{D_{\text{Applied TEB}}} = \frac{2000}{1754} = 1.14$$

$$\text{Ultimate} \quad FOS = \frac{D_u \text{ Class IV}}{D_{\text{Applied TEB}}} = \frac{3000}{1754} = 1.71$$

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.



Appendix C

Guidelines for the Design and Construction of Flexible Thermoplastic Pipe in the City of Edmonton



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PART I: GENERAL

1.0 Scope

- 1.1 This Guidelines covers the design and construction of flexible thermoplastic pipe for use in installations within the City of Edmonton. While the Guidelines is primarily focused on the use of PVC pipe, it does address HDPE pipe products to illustrate some of the subtle differences between the two thermoplastics that must be addressed in design.
- 1.2 When buried, it must be recognized that thermoplastic pipes are a composite structure made up of the plastic ring of the pipe and the soil envelope around them, and that both materials play a vital part in the structural design requirements for the pipe. It also essential that the designer and installer recognize that the soil envelope in typical trench installations is composed of two components – the embedment zone soil and the native soil and that the interaction of these materials can play a significant role in pipe performance.
- 1.3 Part II of this Guidelines presents the proposed design method for flexible pipe design using the standard installation configurations that are specified herein. This design method is predicated on the principle that controlling deflection to within acceptable limits will be sufficient to meet both structural requirements of the pipe based on the materials specifically covered in this standard and the standard installations detailed herein, and the functional requirements of pipe performance such as joint integrity, connections to other structures, etc. in the majority of design situations. This does not preclude the fact that the designer should carry out the appropriate structural design checks as detailed in Part II of the Guidelines to ensure that performance limiting factors other than deflection do not control in any site specific design.
- 1.4 Part III of this Guidelines presents the construction requirements for thermoplastic pipe designed and installed in accordance with this Guidelines.
- 1.5 This Guidelines shall be used as a reference by the owner or owner's engineer in preparing project specifications within the City of Edmonton based on the standard design and installation practices specified herein.
- 1.6 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 given in Part II, the engineer should review the guidance and requirements given in other sections of this Guidelines and its accompanying commentary.
- 1.7 The values of dimensions and quantities are expressed in SI unit values with conversions expressed in inch-pound (English) units for convenience.

2.0 Applicable Documents

2.1 ASTM (American Society for Testing and Materials)

- 2.1.1 D420-98 Guide to Site Characterization for Engineering, Design, and Construction Purposes
- 2.1.2 D2321-00 Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications
- 2.1.3 D2487-00 Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)
- 2.1.4 D2488-00 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
- 2.1.5 D3034-00 Standard Specification for Type PSM Poly(Vinyl Chloride) (PVC) Sewer Pipe and Fittings
- 2.1.6 D3212-96a Standard Specification for Joints for Drain and Sewer Plastic Pipes Using Flexible Elastomeric Seals
- 2.1.7 D3350-02a Standard Specification for Polyethylene Plastics Pipe and Fittings Materials

- 2.1.8 F679-01 Standard Specification for Poly(Vinyl Chloride) (PVC) Large-Diameter Plastic Gravity Sewer Pipe and Fittings
- 2.1.9 F794-99 Standard Specification for Poly(Vinyl Chloride) (PVC) Profile Gravity Sewer Pipe and Fittings Based on Controlled Inside Diameter
- 2.1.10 F894-98a Standard Specification for Polyethylene (PE) Large Diameter Profile Wall Sewer and Drain Pipe

2.2 CSA (Canadian Standards Association)

- 2.2.1 B182.2 PVC Sewer Pipe and Fittings (PSM Type)
- 2.2.2 B182.4 Profile PVC Sewer Pipe and Fittings
- 2.2.3 B182.6 Profile Polyethylene Sewer Pipe and Fittings for Leak-Proof Sewer Applications
- 2.2.4 B182.8 Profile Polyethylene Storm Sewer and Drainage Pipe and Fittings
- 2.2.5 B182.11 Standard Practice for the Installation of Thermoplastic Drain, Storm, and Sewer Pipe and Fittings

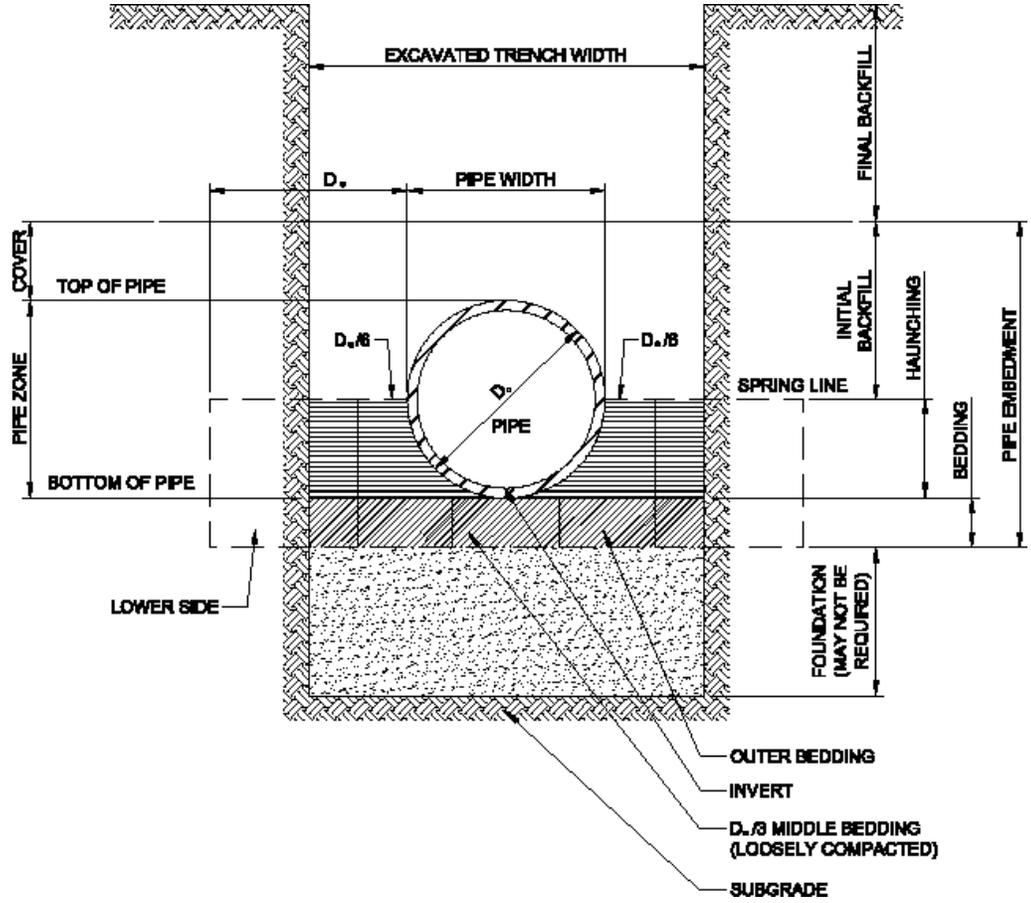
2.3 AWWA (American Water Works Association)

- 2.3.1 AWWA M45, Fiberglass Pipe Design Manual

3.0 Definitions

3.1 Figure 1 illustrates the definitions and limits of the terms, foundation, subgrade, bedding, haunch, lower side, initial backfill, pipe zone, embedment zone, backfill or overfill, invert, crown, springline, top of pipe, and bottom of pipe as used in this Guidelines.

Figure 1: Standard Terminology



4.0 Notations

A = cross sectional area (m²)

B_c = width of pipe (m)

B_d = width of trench (m)

C_c = coefficient of curvature (unitless)

C_u = coefficient of uniformity (unitless)

D, d = pipe diameter (m)

Δx = horizontal deflection (m)

Δy = vertical deflection (m)

D_L = deflection lag factor (unitless)

E = flexural modulus of elasticity (kPa)

E' = modulus of soil reaction (kPa)

E'_b = modulus of soil reaction - embedment soils (kPa)

E'_{design} = modulus of soil reaction - composite design value (kPa)

E'_{native} = modulus of soil reaction - native soils (kPa)

ϵ = bending strain (mm/mm)

Γ = soil density

H = height of cover (m)

I = moment of inertia

I_f = impact factor (unitless)

J = Masada's bedding angle/bedding factor constant

K = bedding factor (unitless)

LL = liquid limit

M = bending moment

ν = Poisson's Ratio

P = external load expressed as a pressure (kPa)

P_{cr} = critical buckling pressure (kPa)

PI = plasticity index

PS = pipe stiffness (kPa)

q_u = unconfined compressive strength

R = radius

S_c = composite soil support factor (unitless)

σ_y = yield point stress

SPD = standard Proctor dry density

SPT = standard penetration test blow count

t = wall thickness

W = total load (kN/m)

W_D = earth load (kN/m)

W_L = live load (kN/m)

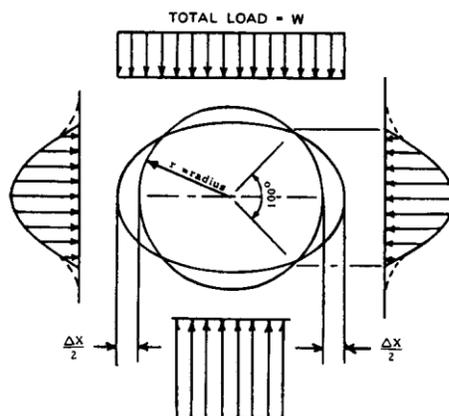
w_L = live load pressure (kN/m²)

5.0 Summary of Guidelines Approach

- 5.1 The design approach of this Guidelines is based upon the assumptions inherent in the original Spangler load distribution¹ for flexible pipe. In this approach, the vertical reaction on the bottom of the pipe is equal to the vertical load on the top of the pipe and is equally distributed over the bedding. Passive horizontal pressures on the sides of the pipe have a parabolic distribution over the middle 100° of the pipe (see Figure 2).

Figure 2: Load Distribution Based on Spangler²

BASIS OF SPANGLER'S DERIVATION OF THE IOWA FORMULA FOR DEFLECTION OF BURIED PIPES



- 5.2 Earth load effects are computed based upon the pressure distributions presented herein. While both embankment loading and trench loading nomenclature are presented for clarity, all design is based upon developing full prism loads as opposed to Marston load theory.
- 5.3 Soil stiffness values (modulus of soil reaction, E') for material in the embedment zone are based upon the research of Duncan and Hartley³ and McGrath⁴. The soil stiffness values to be utilized in design are based upon a direct substitution of the one-dimensional constrained modulus, M_s , for E' . In the absence of direct measurement of constrained modulus values, the design values determined by McGrath's research are recommended for use herein.
- 5.4 The soil stiffness values should be further modified, if required, based on the trench width and the nature and properties of native soils encountered in accordance with the procedure articulated in AWWA Manual of Practice M45⁵.

¹ Watkins, R.K. and M.G. Spangler, "Some Characteristics of the Modulus of Passive Resistance of Soil – A Study in Similitude", Highway Research Board Proceedings, 1958.

² Uni-Bell PVC Pipe Association, "Handbook of PVC Pipe – Design and Construction", 3rd Edition September 1991, pp. 204

³ Hartley, J.D. and J.M. Duncan, "E' and Its Variation with Depth", Journal of Transportation Engineering, September 1987.

⁴ McGrath, T.J., "Replacing E' with the Constrained Modulus in Flexible Pipe Design", Proceedings of the Pipeline Division Conference, San Diego, ASCE, 1998.

⁵ American Water Works Association, "Manual of Water Supply Practices – M45; Fiberglass Pipe Design", 1st Edition, 1996.

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- 5.5** Lastly, the Modified Iowa formula, as developed by Spangler-Watkins, should be corrected to solve for vertical as opposed to horizontal deflection in accordance with the procedure proposed by Masada⁶ and reproduced herein and the recommendations presented in Part II of the Guidelines.

⁶ Masada, T., "Modified Iowa Formula for Vertical Deflection of Buried Pipe", Journal of Transportation Engineering, September/October 2000.

PART II: DESIGN METHOD FOR FLEXIBLE PIPE DESIGN USING CITY OF EDMONTON SPECIFIED INSTALLATION CONDITIONS**6.0 General**

6.1 Design criteria and methodology shall conform to the applicable sections of this Guidelines.

6.2 The designer shall carry out design checks in accordance with this Guidelines to ensure that the maximum localized distortion and net tension strain of the installed thermoplastic pipe shall not exceed the specified limits based upon the pipe selected for use, the embedment material properties specified, the native soil conditions that are anticipated to be encountered, and the installation configuration specified.

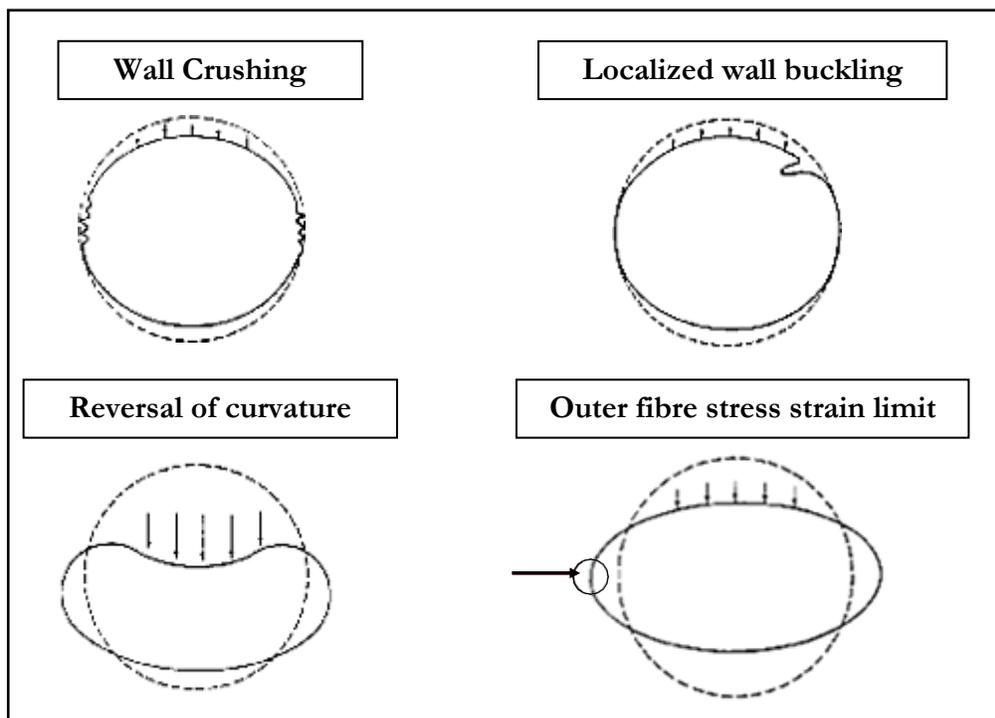
6.3 As the native soil component can significantly impact both short and long-term pipe performance, and its impact may vary with both trench configuration and embedment material selection, the designer shall clearly indicate the combination of native soils, embedment soils, and installation configuration assumed in design and articulate this information to the installer in the manner prescribed by Section 7.2.

7.0 Design Requirements**7.1 General Design Approach**

The performance limits for thermoplastic pipe can include wall crushing (stress), localized wall buckling, reversal of curvature (over-deflection), excessive deflection (i.e. deflection that compromises functional performance), strain limits, longitudinal stresses, shear loadings, and fatigue (see typical examples of most common modes in Figure 3).

In practice, limiting deflection to within tolerable limits is satisfactory to meet all performance requirements for PVC thermoplastic pipe products in the vast majority of non-pressure applications. The designer is encouraged to determine the conditions under which other performance limits will govern in design to facilitate streamlining the design process. However, the designer should understand that he alone is responsible for carrying out all necessary performance limit checks for each specific design situation.

Both low DR and solid-wall and HDPE thermoplastic pipe products should be reviewed by the designer for the full range of design checks before applying the design principles articulated in this Guidelines so that the designer is fully cognizant of the performance limiting factors that will govern in design.

Figure 3: Typical Performance Limiting Modes for Gravity Thermoplastic Pipe

The three parameters that are most essential to consider in all flexible pipe design include load (primarily driven by depth of bury), soil stiffness in the pipe zone (both embedment and native soil), and pipe stiffness.

Soil is obviously a major component of the soil-pipe interaction system and is actually the component that supports the load. While the designer must take this into account in developing his design assumptions, the installer ultimately must be aware of those design assumptions, such that soil conditions in the field that are at variance with the design assumptions can be readily identified and the design, if necessary, modified to account for actual field conditions.

The design process, therefore, consists of:

- Determining external loading conditions;
- Assessing whether any special design conditions other than conventional trench loading will govern in design;
- Determining or estimating in-situ soil conditions based on either site specific geotechnical investigations or experience;
- Selection of the desired balance of soil and pipe stiffness to meet the anticipated loading conditions for the duration of the design period; and
- Articulating the assumptions utilized in design to the installer to ensure that any conditions that arise or become apparent during construction that are at variance with the design assumptions can be reviewed to confirm whether the design is still valid or requires some modification to meet the design objective.

7.2 Minimum Information Transfer to Contractor and Contract Administrator

The minimum level of information transfer to the installer for each design where the use of flexible thermoplastic pipe is contemplated includes:

- 7.2.1 Pipe material and minimum pipe stiffness
- 7.2.2 Assumed installation configuration

- 7.2.3 Embedment material and required placement density
- 7.2.4 Assumed trench width and assumed native soil characteristics (qualitative description and E'_{native} value)

8.0 Pipe Material Requirements

Pipe material requirements are general pipe material requirements to conform to this Guidelines. They are not to be construed as general approval for the use of these products within the City of Edmonton. Specific products approvals are addressed by EPCOR on a product-by-product basis outside of this Guidelines.

8.1 Smooth-wall PVC Products

- 8.1.1 Smooth wall PVC pipe products and fittings shall conform to Sections 4 and 5 of CSA Standard B182.2 for all basic material requirements and manufactured quality and dimensional tolerance.
- 8.1.2 Materials used for pipe shall come from a single compound manufacturer and shall have a cell classification of 12454-B, 12454-C, or 12364-C as defined in ASTM Standard D1784. Materials used for moulded fittings shall come from a single compound manufacturer and shall have a cell classification of 12454-B, 12454-C, or 13343-C as defined in ASTM Standard D1784.
- 8.1.3 Notwithstanding the requirements of Section 4 of CSA Standard B182.2, compounds with different cell classifications than that noted above shall not be used without the prior approval of the City of Edmonton.

8.2 Profile PVC Products

- 8.2.1 Closed profile, dual-wall corrugated, and open profile PVC pipe products and fittings shall conform to Sections 4 and 5 of CSA Standard B182.4 for all basic material requirements and manufactured quality and dimensional tolerance.
- 8.2.2 Materials used for pipe and fittings shall come from a single compound manufacturer and shall have a cell classification of 12454-B, 12454-C, or 12364-C as defined in ASTM Standard D1784.
- 8.2.3 Notwithstanding the requirements of Section 4 of CSA Standard B182.4, compounds with different cell classifications than that noted above shall not be used without the prior approval of the City of Edmonton.

8.3 Polyethylene (PE) Profile Wall Products

- 8.3.1 Closed profile and open profile PE pipe products and fittings shall conform to Sections 4 and 5 of CSA Standards B182.6 and B182.8 for all basic material requirements and manufactured quality and dimensional tolerance for sanitary and storm sewer applications, respectively.
- 8.3.2 Materials used for pipe and fabricated fittings shall come from a single compound manufacturer and shall be made from virgin polyethylene compounds having the following minimum cell classifications as defined in ASTM Standard D3350:

Product	Outside Profile, Corrugations	Inside Lining, Waterway Wall
Storm Sewer and Fabricated Fittings	324420 C or 324420 E	321120C or 321120E
Sanitary Sewer and Fabricated Fittings	324430 C or 324430 E	324430 C or 324430 E

- 8.3.3 Resin compounds shall be tested for slow crack growth resistance in accordance with Appendix SP-NCTL in ASTM Standard D5397 as modified in Clause 8.8 of CSA B182.8.

9.0 Bedding and Foundation Material Requirements

9.1 Classification of Materials

Materials for use as foundation, embedment, and backfill are classified in Table 1. They include natural, manufactured, and processed aggregates and the soil types classified according to ASTM Test Method D2487.

9.2 Installation and Intended Use of Materials

Table 2 provides recommendations on installation and use based on class of soil or aggregates and their location in the trench.

Class I, Class II, and Class III materials are suitable for use as foundation material and in the embedment zone subject to the limitations noted herein and in Table 2.

Class IV-A materials should only be used in the embedment zone in special design cases, as they would not normally be construed as a desirable embedment material for flexible pipe.

Class IV-B, Class V Soils, and Frozen Materials are not recommended for embedment, and should be excluded from the final backfill except where specifically allowed by project specifications.

9.3 Description of Embedment Material

Sections 9.3.1 through 9.3.7 describe characteristics of materials recommended for use in the embedment zone.

Table 1: Classes of Embedment and Backfill Materials⁷

Class	Type	Soil Group Symbol D2487	Description	Percentage Passing Sieve Sizes			Atterberg Limits		Coefficients	
				20 mm (3/4 in)	4.75 mm (No. 4)	0.075 mm (No. 200)	LL	PI	Uniformity C _u	Curvature C _c
IA	Manufactured Aggregates: open-graded, clean.	None	Angular, crushed stone or rock, crushed gravel, broken coral, crushed slag, cinders or shells; large void content, contain little or no fines.	100%	≤10%	<5%	Non Plastic			
IB	Manufactured, Processed Aggregates; dense- graded, clean.	None	Angular, crushed stone (or other Class 1A materials) and stone/sand mixtures with gradations selected to minimize migration of adjacent soils; contain little or no fines (see commentary in Part V).	100%	≤50%	<5%	Non Plastic			
II	Coarse-Grained Soils, clean	GW	Well-graded gravels and gravel-sand mixtures; little or no fines.	100%	<50% of "Coarse Fraction"	<5%	Non Plastic	>4	1 to 3	
		GP	Poorly-graded gravels and gravel-sand mixtures; little or no fines.					<4	<1 or >3	
		SW	Well-graded sands and gravelly sands; little or no fines.		>6			1 to 3		
		SP	Poorly-graded sands and gravelly sands; little or no fines.		<6			<1 or >3		
	Coarse-Grained Soils, borderline clean to w/fines	e.g. GW-GC, SP-SM	Sands and gravels which are borderline between clean and with fines.	100%	Varies	5% to 12%	Non Plastic		Same as for GW, GP, SW and SP	
III	Coarse-Grained Soils With Fines	GM	Silty gravels, gravel-sand-silt mixtures.	100%	<50% of "Coarse Fraction"	12% to 50%		<4 or <"A" Line		
		GC	Clayey gravels, gravel-sand-clay mixtures.					<7 and >"A" Line		
		SM	Silty sands, sand-silt mixtures.		>4 or <"A" Line					
		SC	Clayey sands, sand-clay mixtures.		>7 and >"A" Line					

⁷ Table excerpt from D2321-00 Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications. Maximum aggregate size modified.

Class	Type	Soil Group Symbol D2487	Description	Percentage Passing Sieve Sizes			Atterberg Limits		Coefficients	
				20 mm (3/4 in)	4.75 mm (No. 4)	0.075 mm (No. 200)	LL	PI	Uniformity C _u	Curvature C _c
IVA ^A	Fine-Grained Soils (inorganic)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, silts with slight plasticity.	100%	100%	>50%	<50	<4 or <"A" Line		
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.					>7 and >"A" Line		
IVB	Fine-Grained Soils (inorganic)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.	100%	100%	>50%	>50	<"A" Line		
		CH	Inorganic clays of high plasticity, fat clays					>"A" Line		
V	Organic Soils	OL	Organic silts and organic silty clays of low plasticity.	100%	100%	>50%	<50	<4 or <"A" Line		
		OH	Organic clays of medium to high plasticity, organic silts.					<"A" Line		
	Highly Organic	PT	Peat and other high organic soils.				>50	<"A" Line		

^A Includes Test Method D2487 borderline classifications and dual symbols depending on plasticity index and liquid limits.

NOTE – "Coarse Fraction" as used in this table is defined as material retained on a 0.075 mm (No. 200) sieve.

Table 2: Recommendations for Installation and Use of Soils and Aggregates for Foundation, Embedment and Backfill⁸

Soil Classes (see Table 1) ^A					
	Class IA	Class IB	Class II	Class III	Class IV-A
General Recommendations and Restrictions	Do not use where Conditions may cause migration of fines from adjacent soil and loss of pipe support. Suitable for use as a drainage blanket and underdrain in rock cuts where adjacent material is suitably graded (see Commentary in Part V)	Process materials As required to obtain gradation which will minimize migration of adjacent materials (see Commentary in Part V). Suitable for use as drainage blanket and underdrain.	Where hydraulic gradient exists, check gradation to minimize migration. "Clean" groups suitable for use as drainage blanket and underdrain	Do not use where water conditions in trench may cause instability.	Obtain geotechnical evaluation of proposed material. May not be suitable under high earth fills, surface applied wheel loads, and under heavy vibratory compactors and tampers. Do not use where water conditions in trench may cause instability.
Foundation	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 150 mm maximum layers.	Suitable as foundation and for replacing over-excavated and unstable trench bottom. Install and compact in 150 mm maximum layers.	Suitable as a foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 150 mm maximum layers.	Suitable as foundation and for replacing over-excavated trench bottom as restricted above. Do not use in thicknesses greater than 300 mm total. Install and compact in 150 mm maximum layers.	Suitable only in undisturbed condition and where trench is dry. Remove all loose material and provide firm, uniform trench bottom before bedding is placed.
Bedding	Suitable as restricted above. Install in 150 mm maximum layers. Level final grade by hand. Minimum depth 100 mm (150 mm in rock cuts).	Install and compact in 150 mm maximum layers. Level final grade by hand. Minimum depth 100 mm (150 mm in rock cuts).	Suitable as restricted above. Install and compact in 150 mm maximum layers. Level final grade by hand. Minimum depth 100 mm (150 mm in rock cuts).	Suitable only in dry trench conditions. Install and compact in 150 mm maximum layers. Level final grade by hand. Minimum depth 100 mm (150 mm in rock cuts).	Suitable only in dry trench conditions and when optimum placement and compaction control is maintained. Install and compact in 150 mm maximum layers. Level final grade by hand. Minimum depth 100 mm (150 mm in rock cuts).

⁸ Table excerpt from D2321-00 Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity- Flow Applications. Minimum initial backfill and embedment compaction values modified.

Soil Classes (see Table 1) ^A					
	Class IA	Class IB	Class II	Class III	Class IV-A
Haunching	Suitable as restricted above. Install in 150 mm maximum layers. Work in around pipe by hand to provide uniform support.	Install and compact in 150 mm maximum layers. Work in around pipe by hand to provide uniform support.	Suitable as restricted above. Install and compact in 150 mm maximum layers. Work in around pipe by hand to provide uniform support.	Suitable as restricted above. Install and compact in 150 mm maximum layers. Work in around pipe by hand to provide uniform support.	Suitable only in dry trench conditions and when optimum placement and compaction control is maintained. Install and compact in 150 mm maximum layers. Work in around pipe by hand to provide uniform support.
Initial Backfill	Suitable as restricted above. Install to a minimum of 150 mm above pipe crown.	Install and compact to a minimum of 150 mm above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 300 mm above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 300 mm above pipe crown.	Suitable as restricted above. Install and compact to a minimum of 300 mm above pipe crown.
Embedment Compaction	Place and work by hand to insure all excavated voids and haunch areas are filled. For high densities use vibratory compactors.	Minimum density 90% Std. Proctor Use hand tampers or vibratory compactors.	Minimum density 90% Std. Proctor Use hand tampers or vibratory compactors.	Minimum density 95% Std. Proctor Use hand tampers or vibratory compactors. Maintain moisture content near optimum to minimize compactive effort.	Minimum density 95% Std. Proctor Use hand tampers or impact tampers. Maintain moisture content near optimum to minimize compactive effort.
Final Backfill	Compact as required by the Engineer.	Compact as required by the Engineer.	Compact as required by the Engineer.	Compact as required by the Engineer.	Suitable as restricted above. Compact as required by the Engineer.

^A Class IV-B (MH-CH) and Class V (OL, OH, PT) Materials are unsuitable as embedment. They may be used as final backfill as permitted by the Engineer.

^B When using mechanical compactors avoid contact with pipe. When compacting over pipe crown maintain a minimum of 150 mm cover when using small mechanical compactors. When using larger compactors maintain minimum clearances as required by the Engineer (see Commentary in Part V).

^C The minimum densities given in the table are intended as the compaction requirements for obtaining satisfactory embedment stiffness in most installation conditions (see Section 13).

9.3.1 Class IA Materials

Class IA materials provide maximum stability and pipe support for a given density due to angular interlock of particles. With minimum effort these materials can be installed at relatively high densities over a wide range of moisture contents. In addition, the high permeability of Class IA materials may aid in the control of water, and these materials are often desirable for embedment in rock cuts where water is frequently encountered. However, when ground water flow is anticipated, consideration should be given to the potential for migration of fines from adjacent materials into the open-graded Class IA materials (see commentary in Part V).

9.3.2 Class IB Materials

Class IB materials are processed by mixing Class IA and natural or processed sands to produce a particle size distribution that minimizes migration from adjacent materials that contain fines (see

commentary in Part V). They are more densely graded than Class IA materials and thus require more compactive effort to achieve the minimum density specified. When properly compacted, Class IB materials offer high stiffness and strength and, depending on the amount of fines, may be relatively free draining.

9.3.3 Class II Materials

Class II materials, when compacted, provide a relatively high level of pipe support. In most respects, they have all the desirable characteristics of Class IB materials when densely graded. However, open graded groups may allow migration and the sizes should be checked for compatibility with adjacent material (see commentary in Part V). Typically, Class II materials consist of rounded particles and are less stable than angular materials unless they are confined and compacted.

9.3.4 Class III Materials

Class III materials provide less support for a given density than Class I or Class II materials. High levels of compactive effort may be required unless moisture content is controlled. These materials provide reasonable levels of pipe support once proper density is achieved.

9.3.5 Class IV-A Materials

Class IV-A materials require a geotechnical evaluation prior to use and are only permitted to be used in special design applications such as in cut-off walls or in areas where a short section of low permeability soil is required by design.

Moisture content must be near optimum to minimize compactive effort and achieve the required density. Properly placed and compacted, Class IV-A materials can provide reasonable levels of pipe support; however, these materials may not be suitable under high fills, surface applied wheel loads, or under heavy vibratory compactors and tampers. Do not use where water conditions in the trench may cause instability and result in uncontrolled water content.

9.3.6 Moisture Content of Embedment Material

The moisture content of embedment materials must be within suitable limits to permit placement and compaction to required levels with reasonable effort. For non-free draining soils (that is, Class III, Class IVA, and some borderline Class II soils), moisture content is normally required to be held to +3% of optimum (see ASTM Test Methods D698). The practicality of obtaining and maintaining the required limits on moisture content is an important criterion for selecting materials, since failure to achieve required density, especially in the pipe zone, may result in excessive deflection. Where a chance for water in the trench exists, embedment materials should be selected for their ability to be readily densified while saturated (that is, free-draining, cohesionless granular materials).

9.3.7 Maximum Particle Size

Maximum particle size for embedment is limited to material passing a 20 mm (3/4 in.) sieve (see Table 1). To enhance placement around small diameter pipe and to prevent damage to the pipe wall, a smaller maximum size may be required (see commentary in Part V). When final backfill contains rocks, cobbles, etc., the Engineer may require greater initial backfill cover levels (see Figure 1).

10.0 Characterization of Native Soil Conditions

10.1 Characterization of Native Soils

Native soils must be characterized to determine their potential impact on both short and long-term pipe performance.

Soil characterization to evaluate short-term implications shall be geared towards assessing the impact of native soils on the modulus of soil reaction, E' .

Soil characterization to evaluate potential long-term implications shall be geared towards assessing the potential for migration of native soils into the embedment material or other conditions that may cause degradation of the embedment material's performance with time.

10.2 Implication of Native Soils versus Embedment Material Selection

Short-term performance shall be evaluated to determine whether the modulus of soil reaction in design, E'_{design} , needs to be adjusted based on native soil conditions in accordance with Section 13.2.1.6.

Potential native soil impact on long-term pipe performance shall be assessed in accordance with the recommendations for matching various embedment classes to native soil conditions in Table 2.

11.0 Standard Installation Configurations

Standard installation configurations are presented on Figure 10, Figure 11, and Figure 12 in Part III of this Guidelines for narrow, sub-ditch, and wide trenches.

12.0 External Loads

The designer shall evaluate external loads in response to both dead and live loads. Based upon the specifics of the installation, the designer may be required to assess specialized loading conditions such as those noted in Section 12.3.

12.1 Dead Load Design Requirements

The earth load from fill over the pipe shall be calculated based on the prism load as determined by:

$$W_D = \gamma \times H \times B_C \quad (1)$$

where, $\gamma = \rho \times g$

The minimum density (ρ) used in design shall be 2165 kg/m³ (135 lb/ft³), and the acceleration of gravity (g) used shall be 9.8064 m/s². Should an engineered backfill be utilized with densities markedly higher or lower than this value, the designer shall review the specifics of the material's long-term performance characteristics with the Approving Authority to seek approval for use of alternate design values.

12.2 Minimum Live Load Requirements

12.2.1 Minimum live load requirements shall be the live load generated by a CL-800 truck as defined by Canadian Highway Bridge Design Code (CAN/CSA-S6-00). Where warranted based on traffic volumes, sewer alignment, and the nature of the traffic route, the designer shall review the possible impact of dual or passing CL-800 trucks.

12.2.2 Where pipes cross or could be impacted by railway loads, live loads shall be estimated based on the AREA designated Cooper E-series loads. The minimum live load for consideration in design shall be a Cooper E-80 live load unless the Approving Authority indicates that a greater live load needs to be accommodated.

12.2.3 Requirements for aircraft or other live loads shall be as required by Approving Authority in each specific design.

12.3 Special Design Considerations

The designer shall note that the primary design checks articulated in this Guidelines relate to dead and live loads acting on a single conduit in a variety of conventional trench configurations. There can exist, in design, a number of conditions that warrant special consideration as unique design conditions that are beyond the scope of the design checks suggested by Section 13.0. This could include:

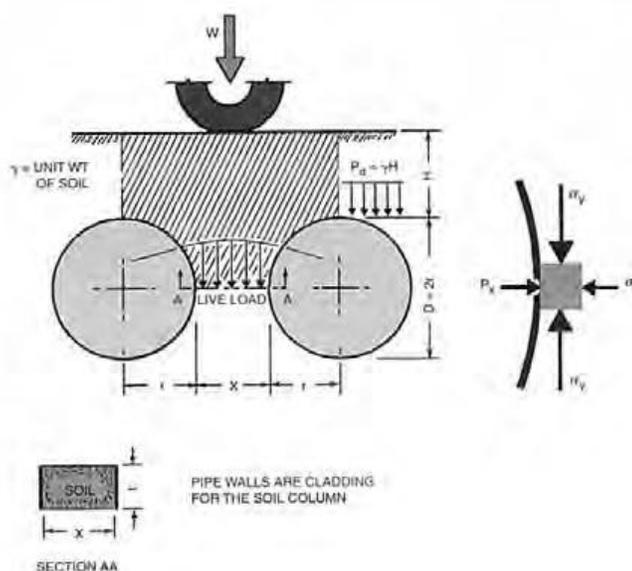
- i. Shallow Parallel pipes subjected to heavy surface loads
- ii. Parallel trenches
- iii. Sloped trench walls
- iv. Situations involving longitudinal bending, support spacing, and thermal contraction and expansion.

A brief discussion on each of these situations follows complete with references to additional resources to evaluate these unique design situations.

12.3.1 Shallow Parallel Pipes subjected to Heavy Surface Loads

Where buried pipes are installed in parallel as illustrated in Figure 4 below, the principles of analysis for single pipes still apply. The design of parallel pipes, however, subjected to heavy surface loads requires additional analysis to determine minimum cover requirements. The designer should consult a suitable reference to conduct this analysis such as the analytical technique proposed by Moser⁹.

Figure 4: Shallow Parallel Pipes, Heavy Surface Loads



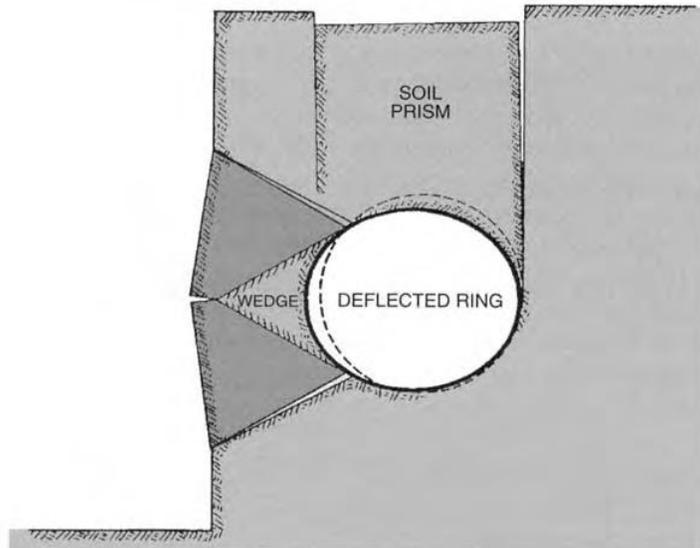
12.3.2 Parallel trenches to Existing Flexible Pipes

Where a parallel trench is cut adjacent an in-place flexible pipe, the width of sidefill soil beside the flexible pipe should be reviewed to ensure that it is sufficiently thick to maintain adequate side support for the pipe (see Figure 5). A suitable analytical technique for this analysis is presented in Moser¹⁰.

⁹ A.P. Moser, "Buried Pipe Design – 2nd Edition", published by McGraw-Hill, 2001, pp. 121.

¹⁰ A.P. Moser, "Buried Pipe Design – 2nd Edition", published by McGraw-Hill, 2001, pp. 130.

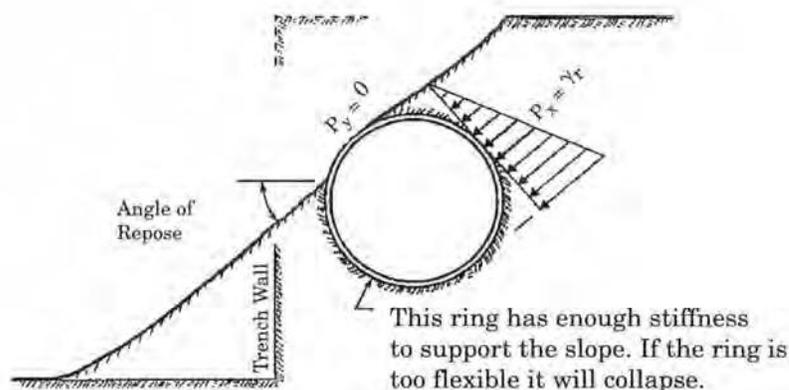
Figure 5: Vertical Trench Parallel to Flexible Pipe Initiating Active Soil Wedge



12.3.3 Sloped trench walls

Where sloped trench walls are cut adjacent to flexible pipes at deeper heights of cover (see Figure 6), the pipe ring stiffness should be reviewed to determine that it is sufficient to withstand the resulting pressure distribution that is imposed upon the pipe. A suitable analytical technique is presented in Moser¹¹.

Figure 6: Slope Adjacent Trench Wall – Pressure Distribution



12.3.4 Longitudinal Bending, Support Spacing, and Thermal Contraction and Expansion

12.3.4.1 Longitudinal Bending

Where flexible pipe is required by design to be subjected to horizontal alignment modifications without the use of bends, deflection typically occurs as a result of longitudinal pipe bending as opposed to individual joint offsets. Where the designer or installer intends to accomplish horizontal offsets in this manner they should review the analytical method and performance limitations of the specific products in use. Analytical procedures and performance limitations for PVC pipe are presented in the PVC pipe Handbook¹².

¹¹ A.P. Moser, "Buried Pipe Design – 2nd Edition", published by McGraw-Hill, 2001, pp. 132.

¹² Uni-Bell PVC Pipe Association, "Handbook of PVC Pipe – Design & Construction", 4th edition, August 2001.

12.3.4.2 Support Spacing

In buried applications, a flexible pipe's strength in longitudinal bending is rarely, if ever, a performance limiting design feature. Where flexible pipe is required to be supported either temporarily or in permanent free span installations such as pipe installed within encasement pipes, its strength in longitudinal bending must be reviewed in greater detail. This is particularly true for some profile wall configurations that provide equivalent strength in terms of equivalent ring stiffness to solid wall products but markedly lower strength in longitudinal bending. Support spacing requirements for both solid wall and profile wall PVC products are presented in the PVC pipe Handbook¹³.

12.3.4.3 Thermal Contraction and Expansion

Flexible thermoplastic materials have markedly higher coefficients of thermal contraction and expansion than most rigid pipe materials. This is particularly true for thermoplastics such as HDPE. Where flexible thermoplastic pipes, however, are installed in buried applications, even with shallow cover, there is typically enough skin friction to overcome axial contraction and expansion (e.g. about 600 mm of cover is generally sufficient to overcome axial movement in smooth wall HDPE pipe). Where thermoplastics are installed in special design situations without the benefit of skin friction, such as in encasement pipes, the effects of thermal contraction and expansion should be reviewed closely.

13.0 Specific Design Approach

13.1 Design Objective

While deflection is required in flexible pipe installations to transfer overburden load to the adjacent soils, deflection must be controlled within tolerable limits to meet both structural and functional requirements for the pipe installation. Controlling deflection to acceptable levels will:

- Avoid reversal of curvature
- Limit bending and strain
- Avoid pipe flattening
- Maintain hydraulics
- Maintain hydrostatic integrity at joints

Controlling deflection will be a function of the load, pipe stiffness, and soil stiffness. In practice, deflection can readily be controlled to within acceptable limits with:

- Proper material selection (both pipe and embedment material)
- Proper construction techniques

While the designer has limited control over the use of proper construction techniques, he can have a greater assurance that his design will be successfully implemented in practice by ensuring that the design is practical and achievable with adherence to normal good pipe installation practices. Any design that requires the use of specialized materials or an unusual level of installer effort to assure success should have those additional requirements clearly articulated to the installer as an output of the design process, to ensure that the installer can make the appropriate adjustments to their normal construction method(s).

13.2 Deflection and Deflection Limits

For PVC pipe materials specified in Sections 8.1 and 8.2, short and long-term deflection shall meet the requirements of Table 3. HDPE deflection limits will vary with DR and will be identified at a later date.

¹³ Uni-Bell PVC Pipe Association, "Handbook of PVC Pipe – Design & Construction", 4th edition, August 2001.

Table 3: Short and Long-Term Deflection Requirements

Maximum Allowable Deflection	
Short-term	Long-term
5.00%	7.50%

Short-term deflection shall be deemed to be any deflection measured not sooner than 30 days after backfilling an installation up to 1 year after backfilling an installation.

Long-term deflection shall be deemed to be any deflection measured after 1 year of backfilling.

Allowable deflection limits for specific pipe materials shall be measured as indicated in Part IV, which incorporates the appropriate allowances for out-of-roundness and other manufacturing tolerances permitted by this Guidelines.

13.2.1 Modified Iowa Formula

The modified Iowa formula in the following form shall be used to estimate horizontal deflection (expressed as a percent change in original diameter):

$$\frac{\Delta x}{d} (\%) = \frac{100D_L K P}{0.149(PS) + 0.061E'_{design}} \tag{2}$$

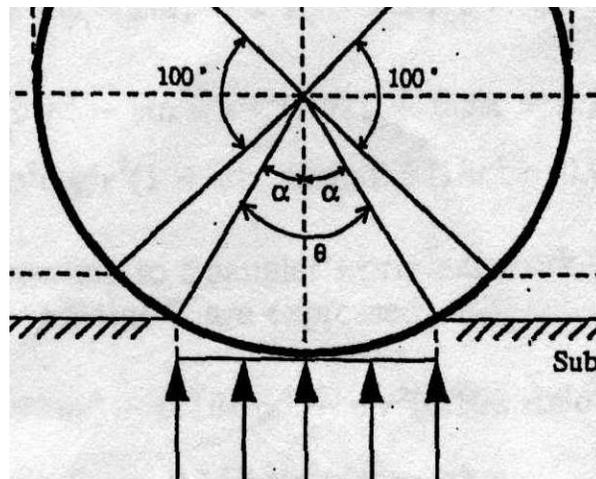
13.2.1.1 Deflection Lag Factor, D_L

A deflection lag factor, D_L , of 1.0 shall be used in all analysis where long-term loading has been estimated based on prism load theory.

13.2.1.2 Bedding Factor, K

A bedding factor, K , of 0.10 shall be utilized in design, for all standard installation configurations specified herein. This is based on the assumption that bedding angles of 60-75 degrees are readily achievable in practice with adherence to good pipe installation practices (see Figure 7 for an illustration of bedding angle).

Figure 7: Bedding Angle θ , a Measure of the Pipe's Interface with the Bedding Material



13.2.1.3 External Load, P

External loads shall be estimated as detailed in Section 12.0 for the appropriate dead and live loading condition. For use in the modified Iowa formula, dead and live loads shall be converted to the equivalent overburden pressure acting over the pipe as follows:

$$P = \frac{W_D + W_L}{B_C} \tag{3}$$

13.2.1.4 Pipe Stiffness, *PS*

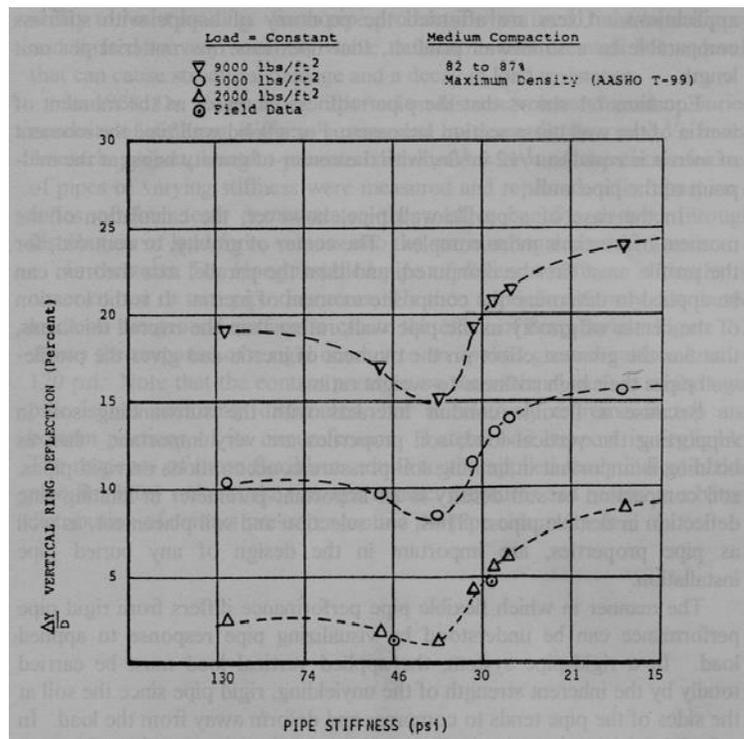
Pipe stiffness, *PS*, shall be the load required to deflect the pipe to 5% deflection as measured in an ASTM D2412 parallel-plate loading test. The pipe stiffness value is calculated by dividing the force per unit length by the deflection. While these values are commonly reported in units of kilopascals (kPa) in SI and pounds per inch² (psi) in the inch-pound system, the values do not represent an equivalent resisting force and should not be construed as such.

The minimum *PS* recommended by this Guidelines is 320 kPa (46 psi).

If lower pipe stiffness materials are used the designer should exercise considerable caution, carry out all necessary design checks, and carefully consider all contributing factors that may impact pipe-soil interaction. It would be prudent if using pipe materials with less than 320 kPa (46 psi) *PS*, to employ only Class I embedment material.

In carrying out analytical checks for pipes with *PS* values less than 320 kPa (46 psi), the designer should note that the analytical model proposed herein may no longer be valid as experimental load cell tests have shown markedly greater observed vertical deflection for pipe products with *PS* values less than 260 kPa (37 psi). This fact is illustrated in Figure 8 based on research carried out at the Utah State.

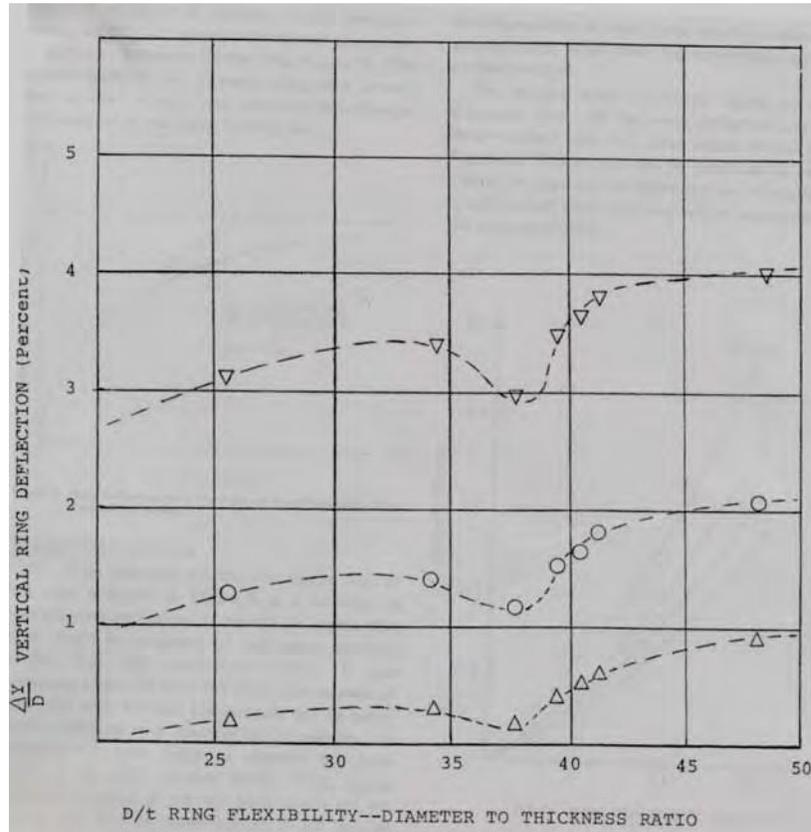
Figure 8: Observed Vertical Ring Deflection in Buried Plastic Pipe as a Function of Pipe Stiffness¹⁴



Note that Figure 8 represents medium embedment compaction conditions (approximately 85% Standard Proctor Density). Under similar loading conditions, denser embedment conditions have a significant impact on the observed vertical deflection for pipe with *PS* values less than 260 kPa (37 psi). This is evident from Figure 9, representing compaction density of approximately 90-94% Standard Proctor Density.

¹⁴ Uni-Bell PVC Pipe Association, "Handbook of PVC Pipe – Design & Construction", 4th edition, August 2001.

Figure 9: Observed Vertical Ring Deflection in Buried Plastic Pipe as a Function of Pipe Stiffness, Denser Embedment Compaction (Note Loading and Embedment Conditions as per Figure 8)¹⁵



13.2.1.5 Modulus of Soil Reaction, E'_b – Embedment Soils

The values for modulus of soil reaction for embedment soils may be estimated based upon a direct substitution of the one-dimensional constrained modulus, M_s , for E' . The values published by McGrath have been related to embedment materials permitted for use in the City of Edmonton by this Guidelines and are reproduced in Table 4 below. These values may be utilized in design subject to the cautionary notes below.

Table 4: E'_b Values for Embedment Soil based on McGrath

Height of Cover	Class I, II Embedment			Class III Embedment			Class IVA Embedment		
	95% SPD	90% SPD	85% SPD	95% SPD	90% SPD	85% SPD	95% SPD	90% SPD	85% SPD
0-2 m (3-6 ft)	13.8 (2000)	8.8 (1300)	3.2 (500)	9.8 (1400)	4.6 (700)	2.5 (400)	3.7 (500)	1.8 (300)	0.9 (100)
2-4 m (6-13 ft)	17.9 (2600)	10.3 (1500)	3.6 (500)	11.5 (1700)	5.1 (700)	2.7 (400)	4.3 (600)	2.2 (300)	1.2 (200)
4-8 m (13-26 ft)	20.7 (3000)	11.2 (1600)	3.9 (600)	12.2 (1800)	5.2 (800)	2.8 (400)	4.8 (700)	2.4 (300)	1.4 (200)

Note 1: E' in MPa (psi rounded to nearest 100 in brackets)

Note 2: Use E' values for 4-8 m of cover and for all heights of cover greater than 8 m.

¹⁵ A.P. Moser, R.K. Watkins, and R.R. Bishop, "The Structural Response of Buried PVC Pipe", Utah State University, 1972

The following commentary is provided to the designer in terms of selection of appropriate design values from the above table:

- Class IV-A materials (fine grained soils, CL and ML) are only permitted as embedment materials in specialized design situations (such as cut-off walls, for example). In practice, obtaining uniform densities greater than 85% with fine- grained materials is very difficult to attain unless considerable quality control efforts are exercised and moisture is tightly controlled during construction.
- In practice, consistently obtaining densities higher than 90% is very difficult to achieve with the use of Class III materials (standard bedding sand with greater than 12% fines). Where greater values are required to facilitate design, the designer is encouraged to review the feasibility of utilizing a higher standard of embedment material to achieve a more practical, readily achievable design for the installer.
- In practice, densities of 90% or more are readily achieved with moderate compactive effort with Class II materials. The practitioner is encouraged to review the Commentary in Part V, Section B7 to determine the appropriate methods of compaction for each embedment class.
- In practice, it is requires considerable compactive effort to consistently achieve densities of 95% or higher in the embedment zone unless Class I materials are utilized. In situations where site conditions and design requirements truly require the consistent development of densities as high 95% SPD, the designer would be wise to require the use of Class I embedment materials.
- The designer is encouraged not to arbitrarily specify an unreasonably high level of compactive effort unless that level of effort is required by design. As illustrated in Part V and the design examples of Part VI, consistently achieving composite E'_{design} values in excess of 1000 MPa is what is truly required for adequate long-term performance in the vast majority of design situations.
- The designer is further advised to exercise caution for any construction to be carried out under winter conditions, as the use of frozen embedment materials can preclude achieving any of the density values noted irrespective of the level of compactive effort exercised due to the difficulties in generating free moisture in the embedment material under winter construction conditions.

13.2.1.6 Influence of Native Soils (Determining Composite E' Values)

The E' value to be utilized in design shall be a composite E'_{design} value, based upon the E'_b , of the embedment material as indicated in Section 13.2.1.5 and the designer's understanding of both native soil conditions, E'_{native} and specified trench width.

E'_{native} values can be estimated based upon Table 5 below.

Table 5: E'_{native} for Various Native Soil Conditions

In-situ Soils				
Granular		Cohesive		E'_{native}
SPT (Blows/0.3 m)	Description	Unconfined Compressive Strength q_u (kPa)	Description	kPa (psi)
>0-1	very, very loose	>0-12	very, very soft	345 (50)
1-2	very loose	12-24	very soft	1380 (200)
2-4		24-48	soft	4825 (700)
4-8	loose	48-96	medium	10,340 (1,500)
8-15	slightly loose	96-192	stiff	20,680 (3,000)
15-30	compact	192-383	very stiff	34,470 (5,000)
30-50	dense	383-575	hard	68,940 (10,000)
>50	very dense	>575	very hard	137,880 (20,000)

The designer shall determine an E'_{design} based upon combined interaction of the embedment soils specified, the native soils anticipated, and the specified trench width. The value for E'_{design} shall be determined from the expression:

$$E'_{design} = S_c \times E_b \tag{4}$$

where, S_c is determined interpolation of the values provided in Table 6 below.

Table 6: Values of S_c , Versus E'_b and E'_{native}

E'_{native}/E'_b	B_d/B_c	B_d/B_c	B_d/B_c	B_d/B_c	B_d/B_c	B_d/B_c
	1.5	2	2.5	3	4	5
0.1	0.15	0.30	0.60	0.80	0.90	1.00
0.2	0.30	0.45	0.70	0.85	0.92	1.00
0.4	0.50	0.60	0.80	0.90	0.95	1.00
0.6	0.70	0.80	0.90	0.95	1.00	1.00
0.8	0.85	0.90	0.95	0.98	1.00	1.00
1.0	1.00	1.00	1.00	1.00	1.00	1.00

13.2.1.7 Calculation of Vertical Deflection

Computed values for horizontal deflection shall be converted to vertical deflection based on Masada's¹⁶ simplified integration of the modified Iowa formula:

$$\left| \frac{\Delta y}{\Delta x} \right| = 1 + \frac{0.0094 E'_{design}}{(PS)} \tag{5}$$

13.3 Strain Limits

Strain is more commonly a performance limiting factor in thermosetting (e.g. fiberglass, CIPP, GRP) as opposed to thermoplastic (e.g. PVC, HDPE) materials. Strain as described herein is total circumferential strain, which is comprised of bending strain, ring compression strain, hoop strain due to internal pressure, and strain due to Poisson's effect. In gravity sewer applications, bending strain is by far the largest and other components are typical small in comparison. Therefore, if bending strains approach the materials strain limit, a more comprehensive review would be warranted.

13.3.1 Bending Strain

Bending strain in the hoop direction may be reasonably approximated by the following expression:

$$\epsilon \approx \frac{t}{D} \times \frac{3 \frac{\Delta y}{D}}{1 - 2 \frac{\Delta y}{D}} \tag{6}$$

13.3.2 Wall Crushing

Wall crushing describes the condition of localized yielding for a ductile material or cracking failure for brittle materials. The performance limit is reached when the in-wall stress reaches the yield stress or ultimate stress of the pipe material. Ring compression stress is the primary contributor to this performance limit, where:

$$Ring\ Compression = \frac{PD}{2A} \tag{7}$$

¹⁶ Masada, T., "Modified Iowa Formula for Vertical Deflection of Buried Pipe", Journal of Transportation Engineering, September/October 2000.

However, wall crushing can also be influenced by circumferential bending stresses, where:

$$\text{Bending Stress} = \frac{Mt}{I} \quad (8)$$

Wall crushing is typically performance limiting in only rigid or brittle pipe products. In flexible thermoplastic pipes, it is not usually performance limiting unless stiffer pipes are subjected to very deep cover, in highly compacted backfill.

13.3.3 Localized Wall Buckling

Localized wall buckling is not normally performance limiting in conventionally buried gravity sewer pipes. Localized buckling may govern in the design of flexible pipes subjected to internal vacuum, high external hydrostatic pressure, or in instances where pipe is subjected to high soil pressures in very highly compacted soil. Localized buckling typically governs in flexible pipes installed as close-fitting liners and should be reviewed more closely in profile wall applications, dependent on the design of the profile section, and particularly in instances when HDPE profile pipe is utilized to its lower flexural modulus.

For long circular tubes subjected to plain strain, the critical buckling pressure is determined by:

$$P_{cr} = \frac{Et^3}{4(1-\nu^2)R^3} \quad (9)$$

For buckling in the inelastic range (materials with a pronounced yield point), the critical buckling point in terms of the materials yield point is:

$$P_{cr} = \frac{t}{R} \times \frac{\sigma_y}{1 + \frac{\sigma_y R^2}{Et^2}} \quad (10)$$

However, critical buckling pressures can be significantly impacted by the geometry of the deflected conduit and the nature of the medium that the pipe is buried in. Calculated critical buckling pressure should be modified to account for geometric effects and should be reviewed to assess the impact, if any, of the surrounding medium¹⁷.

In restrained buckling situations, such as in close-fitting liner pipe installations, the flexural modulus is also impacted by phenomenon of creep and the use of the apparent long-term flexural modulus as determined by ASTM D2990¹⁸ is more appropriate than use of the short-term modulus.

For a thorough review of localized buckling in soil situations the designer should review Moser¹⁹ and for the use of thermoplastics as close-fitting liners the designer should review the recommended design procedure in Appendix X1 of ASTM Standard F1216²⁰.

¹⁷ F1216-07b Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin- Impregnated Tube, pp 5

¹⁸ D2990-01 Standard Test Methods for Tensile, Compressive, and Flexural Creep and Creep-Rupture of Plastics

¹⁹ A.P. Moser, *Buried Pipe Design – 2nd Edition*, published by McGraw-Hill, 2001, pp. 110, pp 470

²⁰ F1216-07b Standard Practice for Rehabilitation of Existing Pipelines and Conduits by the Inversion and Curing of a Resin- Impregnated Tube

PART III: CONSTRUCTION OF SOIL/FLEXIBLE PIPE SYSTEMS**14.0 General**

14.1 The soil-flexible thermoplastic pipe system shall be in configurations that conform to the requirements of Figure 10, Figure 11, and Figure 12, the criteria and design concepts presented in Parts I and II, and to the line and grade designated on the plans and the City of Edmonton Standard Specifications. Owners are advised to provide for or require adequate inspection of the pipe installation at the construction site.

15.0 Safety

15.1 Safety requirements for construction shall be in accordance with the applicable federal, provincial, and local standard regulations.

16.0 Excavation

16.1 The maximum earth load on flexible pipes results from the consolidated prism of soil directly over the pipe, which has been considered in design by this Guidelines. The load on the pipe will not increase beyond these values with increasing trench width. The installer, therefore, shall construct the trench as wide as is dictated by practical and economic considerations but in all cases wide enough to permit proper placement of the material in the embedment zone.

17.0 Trench Construction**17.1 General**

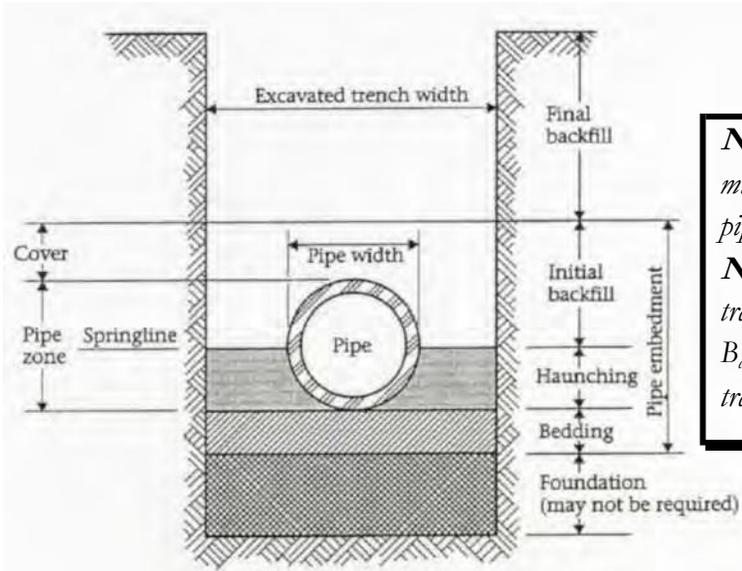
Standard construction practices may necessitate the construction of supported or unsupported trenches in variations of narrow or wide trench configurations.

17.1.1 Unsupported trenches include

- Narrow, unsupported vertical-walled trenches;
- Sub-ditch trenches; and
- Wide trenches

17.1.2 Supported trenches may involve the construction of either narrow vertical-walled trenches or sub-ditch trenches but as supported trenches with the appropriate movable sheeting, trench boxes, shields, or other protective apparatus in place to facilitate construction.

Figure 10: Narrow Unsupported Trench – Typical



Note 1: Do not over densify middle-third of bedding under pipe.

Note 2: Technical transition to wide trench at $B_d/D_o > 5$. Practical transition at $B_d/D_o > 3$.

Figure 11: Sub-ditch Trench Configurations - Typical

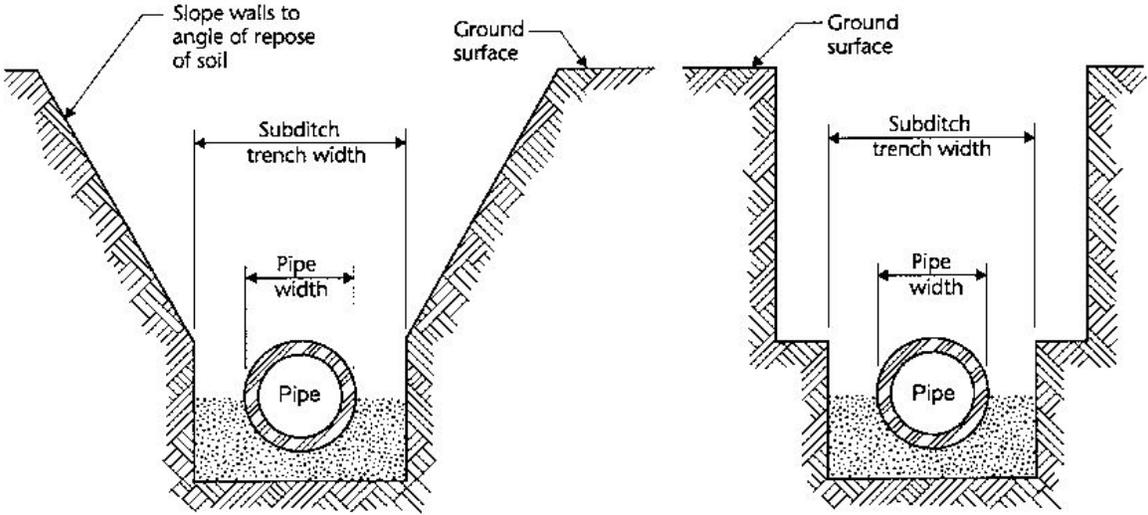
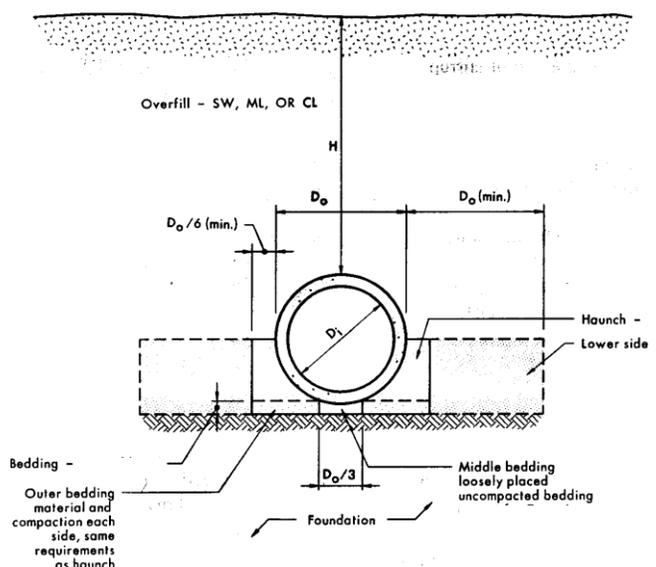


Figure 12: Wide Trench –Typical



Note 1: Do not over densify middle-third of bedding under pipe.

Note 2: Technical transition to wide trench at $B_d/D_o > 5$. Practical transition at $B_d/D_o > 3$.

- 17.1.3** A wide trench is defined as any trench whose width at the top of the pipe measures wider than 5 pipe diameters. By inference, all trenches less than 5 pipe diameters are narrow trenches.

From a practical perspective, the influence of native soils on embedment soils diminishes rapidly at trench widths beyond 3 pipe diameters. Installers should review the values reprinted in Table 6 of Part II of this Guidelines to gain an appreciation for conditions under which native soils may impact embedment soils in a deleterious manner.

17.2 Narrow, Unsupported Vertical-Walled Trenches

- 17.2.1** Where site conditions and safety regulations permit, the trench may be constructed as a narrow, unsupported vertical-walled trench. The width of trench under these conditions shall be the minimum required for a worker to safely place and compact material within the embedment zone in accordance with the specified installation requirements and the compaction equipment and methods required to achieve the specified embedment densities.

- 17.2.2** The installer should note that the embedment soil support in all narrow trench installations is impacted by native soil characteristics. At trench widths less than 3 pipe diameters, native soil characteristics have an increasingly significant impact on embedment soil support (see Table 6 of Part II). The installer, therefore, should pay particular attention to the designer of record's design assumption for native soils in all narrow trench installations and report soils at variance with the design assumptions to the Engineer in a prompt manner to determine what design modifications, if any, are required to be implemented.

17.3 Unsupported Sub-ditch Trenches

- 17.3.1** Sub-ditch trenches are variations of the narrow vertical wall trenches, where the vertical-walled portion above the pipe has been backcut or sloped. The minimum width of the lower trench for sub-ditch trenches shall conform to the requirements of 17.2.1.

- 17.3.2** The installer should note that sub-ditch trenches, by design, have the narrowest of trench widths within the embedment zone and, therefore, pipe performance will be significantly impacted by native soil characteristics in all sub-ditch trench applications. As noted in 17.2.2, the installer shall promptly notify the Engineer in all cases where the conditions encountered are at variance with the stated design assumptions.

17.4 Wide Trenches – See Figure 6

- 17.4.1** Where design or field conditions dictate that a wide trench configuration be utilized the minimum

width of embedment zone densification shall extend for a distance of 2.5 pipe diameters on either side of the pipe. The designer may permit a narrower width of embedment zone densification if it can be demonstrated that the composite embedment zone structure will produce acceptable pipe functional and structural behavior. In these cases the requirements for material type and density outside the embedment zone shall be clearly articulated to the installer.

- 17.4.2** In instances where wide trench construction is employed, the installer is not required to inform the Engineer of native soil condition characteristics that are at variance with the design assumptions.

17.5 Supported trenches

17.5.1 Support of Trench Walls

Where required based on safety regulations, field conditions, or design, the pipe shall be installed in a supported trench.

Where unstable or flowing soil conditions are encountered in the trench wall, such as may be encountered in excavations below the water table and/or in weak non-cohesive soils, the unstable soils shall be stabilized prior to proceeding with pipe installation.

When supports such as trench sheeting, trench jacks, trench shields, or boxes are used, ensure that support of the pipe and its embedment is maintained throughout installation. Ensure that sheeting, where required, is sufficiently tight to prevent washing out of the trench wall from behind the sheeting. Provide tight support of trench walls below existing utilities or other obstructions that restrict driving of sheeting.

17.5.2 Supports Left in Place

Unless otherwise directed by the Engineer, sheeting driven into or below the pipe zone should be left in place to preclude loss of support of foundation or embedment zone material. When top of sheeting is to be cut off, make cut 500 mm or more above the crown of the pipe. Leave rangers, walers, and braces in place as required to support cutoff sheeting and the trench wall in the vicinity of the pipe zone. Timber sheeting to be left in place is considered a permanent structural member and shall be treated against biological degradation as necessary, and against decay if above the groundwater table. Certain preservative and protective compounds react adversely with thermoplastics, and their use should be avoided in proximity to the pipe material.

17.5.3 Movable Trench Wall Support

Do not disturb the installed pipe and its embedment when using movable trench boxes and shields. Movable supports shall not be used below the top of the pipe zone unless an approved method is used to maintain the integrity of the embedment material. Before moving supports, place and compact embedment to sufficient depths to ensure protection of the pipe. As supports are moved, finish placing and compaction of embedment material.

17.5.4 Removable Trench Wall Support

Where sheeting or other trench wall supports are used within or below the pipe zone, ensure the foundation and embedment materials are not disturbed by support removal. Fill any voids left on removal of supports and compact all material to required densities.

18.0 Foundation

- 18.1** The foundation soil shall be moderately firm to hard in situ soil, stabilized soil, or compacted fill material.

- 18.2** When unsuitable or unstable material is encountered, the foundation shall be stabilized.

- 18.3** Where groundwater and soil characteristics may contribute to the migration of soil fines into or out of the foundation, embedment soils, sidefill, and/or backfill materials, methods to prevent migration shall be provided. Commentary on the potential and means to preclude migration of soil fines are presented in Part V of this Guidelines.

19.0 Bedding and Initial Backfill Requirements

19.1 Verification that Proposed Construction Method is Consistent with Design Intent

Project specific design requirements for the in-place density of outside bedding material, haunch material, and initial backfill shall be noted on the plans or in the project specifications. As the precise measurement of these densities in-place during construction is often not technically feasible, the installer shall demonstrate to the Engineer for the project that their proposed method of placement of these materials is sufficient to achieve the specified results, through a trial compaction demonstration.

Should the materials proposed for use in the embedment zone change during the course of the works the installer shall notify the Engineer and carry out additional compaction trials, sufficient to demonstrate that their proposed method of placement is consistent with achieving the specified requirements.

The trial compaction demonstration shall in no way relieve the installer from their contractual requirement of meeting the minimum performance criteria for completed installations as specified herein.

19.2 Placement of Bedding Materials

19.2.1 The bedding shall be constructed as required by the project specifications and in accordance with the installer's proposed construction method as verified in the compaction trial demonstration. Bedding shall be placed in such a manner to maximize the bedding angle achieved, to provide uniform load-bearing reaction, and to maintain the specified pipe grade.

19.2.2 The bedding layer shall be placed as uniformly as possible to the required density, except that loose, un-compacted material shall be placed under the middle third of the pipe, prior to placement of the pipe.

19.2.3 Bell holes shall be excavated in the bedding when installing pipe with expanded bells such that the barrel and not the pipe bells support the pipe.

19.3 Placement of Haunch and Initial Backfill Materials

19.3.1 Placement of haunching and initial backfill embedment materials shall be carried out by methods that will not disturb or damage the pipe.

19.3.2 Work in and tamp the haunching material in the area between the bedding and the underside of the pipe before placement and compaction of the remainder of the material in the embedment zone.

19.3.3 Use compaction equipment and methods that are compatible with the materials used, the location in the trench, and the in-place densities required. In addition to the requirements of the compaction trial demonstration, review commentary in Part V of this Guidelines.

19.3.4 The primary purpose of initial backfill is to protect the pipe from any impact damage that may arise from the placement of overfill materials. Minimum thickness of the initial backfill layer shall be as indicated on the standard installation drawings. In instances where overfill material contains large objects or is required to be deposited from very high heights, initial backfill shall be extended to such additional height above the pipe as is necessary to prevent damage from occurring to the pipe during backfilling operations.

19.3.5 Before using heavy compaction or construction equipment directly over the pipe, ensure that sufficient backfill has been placed over the pipe to prevent damaging either the pipe or the embedment zone materials as indicated in Section 22.0.

20.0 Change in Native Soil Conditions

20.1 The designer will apprise the installer of the assumed in-situ soil conditions that the design was based on. As noted in Part II of this Guidelines, in-situ soil properties can significantly impact both short and long-term pipe performance in narrow trench and sub-ditch type trench configurations. Should a change in site conditions be observed that would result in impacting either short or long-

term pipe and/or embedment soil performance, the installer shall notify the Engineer, such that the validity of the original design concept can be reviewed by the designer of record. If necessary, the design will be modified to suit the actual conditions encountered in the field.

20.2 Where such modifications are required, they shall be addressed as a change in site conditions and valued for payment in accordance with the requirements of the specific contract provisions for changed site conditions. Where no adjustments are required, there shall be no adjustments in contract price.

20.3 In all instances where the designer of record's input is sought, it shall be provided in as expeditious a manner as possible so as to minimize the impact on construction progress.

21.0 Backfill (Overfill) Materials

21.1 Construction of the backfill zone shall be as specified in the specific project requirements.

21.2 The soil shall be approved material containing no debris, organic matter, frozen material, or large stones or other object that may be detrimental to the pipe or the embedment materials. The presence of such material in the embedment may preclude uniform compaction and result in excessive localized deflections.

21.3 The installer shall ensure that there is sufficient cover over the pipe and embedment zone materials to facilitate all construction operations associated with the placement and compaction of overfill material.

22.0 Minimum Cover Requirements for Construction Loads

22.1 To preclude damage to the pipe and disturbance to the embedment zone, a minimum depth of backfill should be maintained before allowing vehicles or heavy construction equipment traverse the pipe trench.

22.2 The minimum depth of cover should be established by the project engineer based on the specific project requirements.

22.3 In the absence of such a detailed investigation, the installer shall meet the following minimum cover requirements before allowing vehicles or construction equipment to traffic the trench surface, assuming that the minimum embedment zone densities as noted in Table 2:

- Provide minimum cover of at least 600 mm or one pipe diameter (whichever is larger) where Class I embedment materials have been utilized, or
- Provide minimum cover of at least 900 mm or one pipe diameter (whichever is larger) where Class II or lower embedment materials have been utilized, and
- Allow at least 1200 mm of cover before using a hydrohammer for compaction directly over the pipe, and
- Where construction loads may be excessive (e.g. cranes, earth moving equipment, etc.) consult with the project engineer to determine minimum operating cover requirements.

23.0 Connection of Flexible Pipe to Maintenance Holes

23.1 The installer shall use flexible water stops, resilient connectors, or other flexible systems approved by the project engineer to make watertight connections to maintenance holes and other structures.

23.2 The designer should review the structural requirements associated with installing flexible pipes within maintenance holes and should ensure that sufficient maintenance hole structure is provided to accommodate the installation of a flexible pipe.

24.0 Completion of Construction Criteria and Acceptance Testing

24.1 Vertical and Horizontal Alignment Tolerances

The pipe shall be installed to the line and grade noted on the construction drawings. Acceptance variance shall be:

- 6 mm plus 20 mm per m of diameter for vertical grade, and

- within 150 mm of the designated alignment for horizontal grade of pipes up to 900 mm in diameter or 50 mm per 300 mm of diameter of the designated alignment for pipes greater than 900 mm in diameter, and

No variance from grade shall be permitted that results in individual joint deflections in excess of the manufacturer's recommended value to maintain hydrostatic integrity to the limits specified herein.

24.2 Infiltration/Exfiltration Limits

Elastomeric gasket joints for pipe and fittings shall meet the requirements of ASTM D3212, except that the internal hydrostatic pressure shall be 100 kPa (15 psi).

24.3 CCTV Inspection

All pipe up to and including 1200 mm NPS shall be inspected by CCTV Inspection methods as per Section 23 – Inspection of Sewers of the City of Edmonton Vol. 3-06: Construction Specifications. Pipes larger than 1200 mm NPS shall be inspected by man-entry methods as per Section 23 – Inspection of Sewers of the City of Edmonton Vol. 3-06: Construction Specifications.

24.4 Deflection Testing

Where closed circuit television (CCTV) or visual walk-through inspections show evidence of excessive or non-symmetrical deflection (e.g. a non-elliptical deformation pattern), formal deflection tests shall be conducted. Where formal deflection testing is required it shall be carried out in accordance with the procedures of Part IV of this Guidelines to confirm that the installed pipe meets the requirements for either short or long-term deflection limits as per Section 13.2 and Part IV. Deflection tests shall not be carried out sooner than 30 days after installation and backfilling complete to assess short-term deflection and not sooner than 1 year to assess long-term deflection.

PART IV: MANDREL REQUIREMENTS FOR DEFLECTION TESTING

A1.0 Scope

Part IV covers the technical requirements for deflection testing of flexible thermoplastic pipe installations within the City of Edmonton designed and constructed in accordance with this Guidelines.

A2.0 Inspection Method

Where closed circuit television (CCTV) or visual walk-through inspections show evidence of excessive or non-symmetrical deflection (e.g. a non-elliptical deformation pattern), formal deflection tests shall be conducted.

Where formal deflection tests are required:

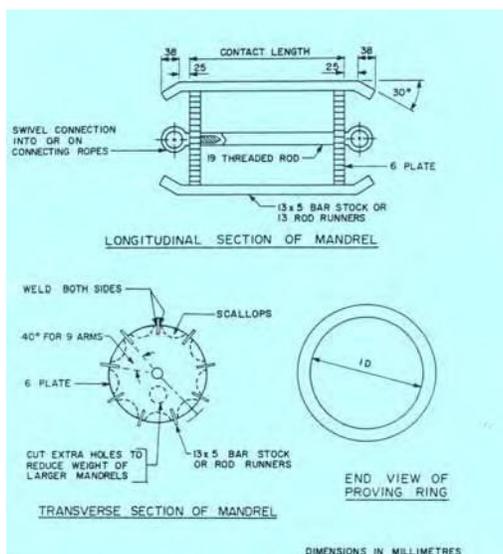
- Pipe up to and including 900 mm NPS diameter shall be inspected with “go/no-go” mandrel device as described herein.
- Pipe larger than 900 mm NPS diameter shall be inspected with a suitable proving device to confirm that vertical deflection does not exceed either the maximum allowable short or long-term deflection limits stipulated by Section 13.2.

The mandrel or proving device shall be pulled through the pipe in such a manner so as to ensure that excessive force is not used to advance the device through any deflected portion of the pipe.

Deflection testing shall be performed in conjunction with a closed circuit television inspection. The mandrel shall be located in front of, and in clear view of, the television camera. An appropriate distance is typically from 1.5 to 2.5 pipe diameters in front of the television camera.

The mandrel shall be cylindrical in shape, constructed with 9 evenly spaced arms and shall generally conform to Figure A1.

Figure A1: General Mandrel Configuration



Mandrels larger than 450 mm in diameter shall be constructed of special breakdown devices to facilitate entry through access maintenance holes.

A2.0 Mandrel Dimensional Requirements

The minimum diameter of the circle scribed around the outside of the mandrel arms shall be equal to the values indicated in Section A3 for each specific pipe material, within a tolerance of ± 0.25 millimetres. The contact length of the mandrel shall be measured between the points of contact on the mandrel arm as indicated in Figure A1. The outside radius of the mandrel arms shall be

checked for conformance with these specifications with a proving ring.

An oversized proving ring may be used, which shall be manufactured to a diameter equal to the outside diameter of the mandrel plus 1 millimetre, to facilitate undertaking measurements to confirm that the size of the mandrel conforms the dimensions and dimensional tolerances specified herein. The proving ring shall be manufactured to within 0.25 millimetres of the specified size. The proving ring shall be fabricated from 6 millimetre minimum thick steel.

As an alternative, a “go/no-go” proving ring device shall be permitted in which case the proving ring shall be sized up to 0.30 millimetres less than the circle that would be scribed by the specified mandrel size. If a “go/no-go” proving ring is utilized, an acceptable mandrel will not be able to pass through the proving ring. “Go/no-go” proving rings shall not be less than 0.1 millimetres of the specified dimension.

The radius of mandrel arm required to assess short and long-term deflection limits is noted in Section A3 for all pipe materials contemplated by this Guidelines.

The barrel section of the mandrel shall have a contact length of at least 75% of the base inside diameter of the pipe.

A3.0 Acceptance Test Limits

Mandrel or visual walk-through proving devices shall be sized to confirm that either short or long-term vertical deflection limits are not in excess of the appropriate allowance as dictated by Section 13.2. Deflection shall be measured versus the appropriate base inside diameter for each specific pipe material as indicated in the following sections.

Base inside diameter for the purposes of this Guidelines is the base inside diameter as defined by the appropriate CSA Standard that governs manufacture of the specific pipe being tested.

The base inside diameter is the minimum inside pipe diameter prior to calculating allowable deflection and is derived by subtracting a statistical tolerance package from the pipe’s average inside diameter. The tolerance package includes allowances for variation in outside diameter, over-thickened walls, and initial out-of-roundness.

A3.1 Solid Wall DR 35 and DR 41 PVC Pipe

<i>Spec CSA B182.2, Solid Wall PVC, DR 35</i>						
NPS	Average Inside Diameter	Base Inside Diameter (BID)	Allowable Vertical Deflection (mm)		Radius of Mandrel Arm (r) (mm)	
			Short Term	Long Term	Short Term	Long Term
	(mm)	(mm)	5.00%	7.50%	Short Term	Long Term
100	100.57	98.40	93.5	91.0	47.0	45.9
150	149.73	146.50	139.2	135.5	70.0	68.3
200	200.42	196.11	186.3	181.4	93.7	91.4
250	250.55	245.16	232.9	226.8	117.1	114.3
300	298.27	291.86	277.3	270.0	139.4	136.0
375	365.09	357.25	339.4	330.5	170.6	166.5
450	446.23	436.64	414.8	403.9	208.5	203.5
525	526.08	514.77	489.0	476.2	245.8	239.9
600	591.84	579.11	550.2	535.7	276.6	269.9
675	666.99	652.64	620.0	603.7	311.7	304.2
750	763.57	747.68	710.3	691.6	357.1	348.5
900	913.89	894.77	850.0	827.7	427.3	417.0
1050	1061.84	1039.51	987.5	961.5	496.4	484.5
1200	1212.14	1186.60	1127.3	1097.6	566.7	553.0

<i>Spec CSA B182.2, Solid Wall PVC, DR 41</i>						
NPS	Average Inside Diameter	Base Inside Diameter (BID)	Allowable Vertical Deflection (mm)		Radius of Mandrel Arm (r) (mm)	
	(mm)	(mm)	Short Term	Long Term	Short Term	Long Term
			5.00%	7.50%		
200	202.33	198.02	188.1	183.2	94.6	92.3
250	252.92	247.55	235.2	229.0	118.2	115.4
300	301.09	294.69	280.0	272.6	140.7	137.3
375	368.53	360.69	342.7	333.6	172.3	168.1
450	450.43	440.85	418.8	407.8	210.5	205.5
525	531.04	519.75	493.8	480.8	248.2	242.2
600	597.42	584.71	555.5	540.9	279.2	272.5
675	673.28	658.97	626.0	609.5	314.7	307.1
750	770.78	754.91	717.2	698.3	360.5	351.8
900	922.51	903.42	858.2	835.7	431.4	421.0
1050	1071.86	1049.55	997.1	970.8	501.2	489.1
1200	1223.58	1198.07	1138.2	1108.2	572.1	558.4

A3.2 Profile Wall PVC Pipe

<i>Spec CSA B182.4, Profile PVC Pipe, PS 320 kPa</i>						
NPS	Average Inside Diameter	Base Inside Diameter (BID)	Allowable Vertical Deflection (mm)		Radius of Mandrel Arm (r) (mm)	
	(mm)	(mm)	Short Term	Long Term	Short Term	Long Term
			5.00%	7.50%		
100	100.71	98.54	93.6	91.1	47.1	45.9
150	149.71	146.51	139.2	135.5	70.0	68.3
200	200.41	196.11	186.3	181.4	93.7	91.4
250	250.55	245.19	232.9	226.8	117.1	114.3
300	298.13	291.75	277.2	269.9	139.3	136.0
375	365.09	357.28	339.4	330.5	170.6	166.5
450	448.31	438.70	416.8	405.8	209.5	204.5
525	527.05	515.75	490.0	477.1	246.3	240.4
600	596.90	584.17	555.0	540.4	279.0	272.3
675	673.10	659.49	626.5	610.0	314.9	307.4
750	749.30	734.14	697.4	679.1	350.6	342.1
900	901.70	883.46	839.3	817.2	421.9	411.7
1050	1054.10	1032.79	981.2	955.3	493.2	481.3
1200	1206.50	1182.12	1123.0	1093.5	564.5	550.9

PART V: COMMENTARY²¹

- B.1 Those concerned with the service performance of a buried flexible pipe should understand factors that can affect this performance. Accordingly, key considerations in the design and execution of a satisfactory installation of buried flexible thermoplastic pipe that provided a basis for the development of this practice are given in this Part.
- B.2 *General* – Sub-surface conditions should be adequately investigated prior to construction, in accordance with Practice D 420, as a basis for establishing requirements for foundation, embedment and backfill materials and construction methods. The type of pipe selected should be suited for the job conditions.
- B.3 *Load/Deflection Performance* – The thermoplastic pipes considered in this practice are classified as flexible conduits since in carrying load they deform (deflect) to develop support from the surrounding embedment. This interaction of pipe and soil provides a pipe-soil structure capable of supporting earth fills and surface live loads of considerable magnitude. The design, specification and construction of the buried flexible pipe system should recognize that embedment materials must be selected, placed and compacted so that pipe and soil act in concert to carry the applied loads without excessive strains from deflections or localized pipe wall distortions.
- B.4 *Pipe Deflection* – Pipe deflection is the diametral change in the pipe-soil system resulting from the process of installing the pipe (construction deflection), static and live loads applied to the pipe (load-induced deflection), and time dependent soil response (deflection lag). Construction and load induced deflections together constitute initial pipe deflection. Additional time dependent deflections are attributed primarily to changes in embedment and in-situ soils, and trench settlement. The sum of initial and time dependent deflections constitutes total deflection. The analytical methods proposed in this Guidelines are intended to limit total deflection to within acceptable limits.
- B.4.1 *Construction Deflection* – Construction deflections are induced during the process of installing and embedding flexible pipe, even before significant earth and surface loads are applied. The magnitude of construction deflections depends on such factors as the method and extent of compaction of the embedment materials, type of embedment, water conditions in the trench, pipe stiffness, uniformity of embedment support, pipe out-of-roundness, and installation workmanship in general. These deflections may exceed the subsequent load-induced deflections. Compaction of the side fill may result in negative vertical deflections (that is, increases in pipe vertical diameter and decreases in horizontal diameter).
- B.4.2 *Load-Inducted Deflection* – Load-induced deflections result from backfill loads and other superimposed loads that are applied after the pipe is embedded.
- B.4.3 *Short-term Deflection* – Short-term deflection is the deflection in the installed and backfilled pipe. It is the total of construction deflections and load-induced deflections determined after a sufficient portion of the long-term load has developed on the pipe. For the purposes of this Guidelines the short-term deflection shall be total deflection as measured after a time period not shorter than 30 days after backfilling.
- B.4.4 *Time Dependent Factors* – Time dependent factors include changes in soil stiffness in the pipe embedment zone and native trench soils, as well as loading changes due to trench settlement over time. These changes typically add to the short-term deflection; the time involved varies from a few days to several years depending on soil types, their placement, and initial compaction. Time dependent factors are accounted for in this Guidelines by adjusting acceptable short-term deflection limits by a factor of 1.5.
- B.4.5 *Long-term Deflection* – Long-term deflection is the total long-term deflection of the pipe. It consists of initial deflection adjusted for time dependent factors as noted. While acknowledged the time-dependent deflection can occur for many years, the experience has shown that the vast majority of long-term deflection (typically 90% or more) has occurred after the first year of installation. For the purposes of this Guidelines, therefore, the long-term deflection shall be considered to be any

²¹ Modified from ASTM 2321-00, Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity Applications

- deflection measured one year or later after backfilling.
- B.5 *Deflection Criteria* – Deflection criteria are the limits set for the design and acceptance of buried flexible pipe installation. Deflection limits for specific pipe systems may be derived from both structural and practical considerations. Structural considerations include pipe cracking, yielding, strength, strain, and local distortion. Practical considerations include such factors as flow requirements, clearance for inspection and cleaning, and maintenance of joint seals. Acceptable short and long-term deflection limits are presented for all pipes addressed by this Guidelines in Part IV.
- B.6 *Deflection Control* – Embedment materials should be selected, placed, and compacted so as to minimize total deflections and, in any event, to maintain installed deflections within specific limits. Methods of placement, compaction, and moisture control should be selected based on soil types given in Table 1 of Part II of this Guidelines and on recommendations given in Table 2 of Part II of this Guidelines. The amount of load-induced deflection is primarily a function of the stiffness of the pipe and soil embedment system. Other factors that are important in obtaining deflection control are outlined below.
- B.6.1 *Embedment at Pipe Haunches* – Lack of adequate compaction of embedment material in the haunch zone can result in excessive deflection, since it is this material that supports the vertical loads applied to the pipe. A key objective during installation of flexible thermoplastic pipe (or any pipe) is to work in and compact embedment material under pipe haunches, to ensure complete contact with the pipe bottom, and to fill voids below the pipe.
- B.6.2 *Embedment Density* – Embedment density requirements should be determined by the engineer based on deflection limits established for the pipe, pipe stiffness, and installation quality control, as well as the characteristics of the in-situ soil and compatibility characteristics of the embedment materials used. The minimum densities given in Table 2 are based on attaining an average modulus of soil reaction (E') of greater than 6.9 MPa (1000 psi) except under special circumstances where Class IVA embedment material is used. Where higher modulus of soil reaction values are required the designer should refer to Table 4 as well as making the appropriate adjustments if necessary to account for the impact of native soils that may have modulus values lower than the proposed embedment soils.
- B.7 *Compaction Methods* – Achieving desired densities for specific types of materials depends on the methods used to impart compactive energy. Coarse-grained, clean materials such as crushed stone, gravels, and sand are more readily compacted using vibratory equipment, whereas fine materials with high plasticity require kneading and impact force along with controlled water content to achieve acceptable densities. In pipe trenches, small, hand-held or walk-behind compactors are required, not only to preclude damage to the pipe, but to ensure thorough compaction in the confined areas around the pipe and along the trench wall. As examples, vibratory plate tampers work well for coarse grained materials of Class I and Class II, whereas hand tampers or air driven hand-held impact rammers are suitable for the fine-grained, plastic groups of Class III and IV A. Gas or diesel powered jumping jacks or small, walk-behind vibratory rollers impart both vibratory and kneading or impact force, and hence are suitable for most classes of embedment and backfill material.
- B.8 *Migration* – When coarse and open-graded material is placed adjacent to a finer material, fines may migrate into the coarser material under the action of hydraulic gradient from ground water flow. Significant hydraulic gradients may arise in the pipeline trench during construction when water levels are being controlled by various pumping or well-pointing methods, or after construction when permeable under drain or embedment materials act as a “French” drain under high ground water levels. Field experience shows that migration can result in significant loss of pipe support and continuing deflections that may exceed design limits. The gradation and relative size of the embedment and adjacent materials must be compatible in order to minimize migration (see B.8.1 below). In general, where significant ground water flow is anticipated, avoid placing coarse, open-graded materials, such as Class IA, above, below, or adjacent to finer materials, unless methods are employed to impede migration such as the use of an appropriate stone filter or filter fabric along the boundary of the incompatible materials. To guard against loss of pipe support from lateral

migration of fines from the trench wall into open-graded embedment materials, it is sufficient to follow the minimum embedment width guidelines in B.10.

- B.8.1 The following filter gradation criteria may be used to restrict migration of fines into the voids of coarser material under a hydraulic gradient:
- B.8.1.1 $D_{15}/d_{85} < 5$ where D_{15} is the sieve opening size passing 15% by weight of the coarser material and d_{85} is the sieve opening size passing 85% by weight of the finer material.
- B.8.1.2 $D_{50}/d_{50} < 25$ where D_{50} is the sieve opening size passing 50% by weight of the coarser material and d_{50} is the sieve opening size passing 50% by weight of the finer material. This criterion need not apply if the coarser material is well-graded (see Test Method D 2487).
- B.8.1.3 If the finer material is a medium to highly plastic clay without sand or silt partings (CL or CH), then the following criterion may be used in lieu of B.8.1.1: $D_{15} < 15\%$ by weight of the coarser material.

Note – Materials selected for use based on filter gradation criteria, such as in B.8.1, should be handled and placed in a manner that will minimize segregation.

- B.9 *Maximum Particle Size* – Limiting particle size to 20 mm (¾ in.) or less enhances placement of embedment material for nominal pipe sizes 200 mm (8 in.) through 375 mm (15 in.). For smaller pipe, a particle size of about 10% of the nominal pipe diameter is recommended.
- B.10 *Embedment Width for Adequate Support* – In certain conditions, a minimum width of embedment material is required to ensure that adequate embedment stiffness is developed to support the pipe. These conditions arise where in-situ lateral soil resistance is negligible, such as in very poor native soils (for example, peat, muck, or highly expansive soils) or along highway embankments. Under these conditions, for small diameter pipe (12 in (300mm) or less), embedment should be placed and compacted to a point at least 2.5 pipe diameters on either side of the pipe. For pipe larger than 12 in. (300mm), the engineer should establish the minimum embedment width based on an evaluation of parameters such as pipe stiffness, embedment stiffness, nature of in-situ soil, and magnitude of construction and service loads.
- B.11 *Other Design and Construction Criteria* – The design and construction of the pipe system should recognize conditions that may induce excessive shear, longitudinal bending, or compression loading in the pipe. Live loads applied by construction and service traffic may result in large, cumulative pipe deflections if the pipe is installed with a low density embedment and shallow cover. Other sources of loads on buried pipes are: freezing and thawing of the ground in the vicinity of the pipe, rising and falling of the ground water table, hydrostatic pressure due to ground water, and localized differential settlement loads occurring next to structures such as maintenance holes and foundations. Where external loads are deemed to be excessive, the pipe should be installed in casing pipe or other load limiting structures.

PART VI: DESIGN EXAMPLES

This Guidelines provides a couple of design examples to compute deflection based on the application of the analytical model recommended by this Guidelines.

The design method recommended by this Guidelines is comprised of the following basic steps:

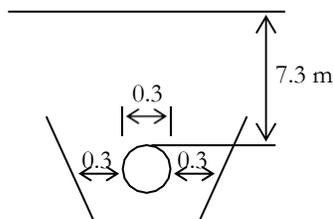
1. Determine external loading (both Dead and Live Loading). Dead loading is directly related to the height of cover, while live loading will be a function of the height of cover and the anticipated live loading vehicle (e.g. standard truck loads, railway loads, and/or airport loading).
2. Evaluate whether any special design conditions need to be evaluated and evaluate them independently.
3. Determine a representative Modulus of Soil Reaction, E' . The effective or composite Modulus of Soil Reaction will be a function of the embedment soil we select, native soil conditions, and trench width.
4. Select the remaining Modified Iowa Formula parameters including the deflection lag factor (D_L), bedding factor (K), and pipe stiffness (PS).
5. Calculate horizontal deflection utilizing the Modified Iowa Formula.
6. Calculate the vertical deflection using Masada's simplified integration of the Modified Iowa Formula. Review the answer versus our performance limits for deflection to determine whether we need to carry out additional iterations with modified bedding conditions, increased trench width, etc.

This is intended to be a relatively simple set of examples, and purposely has omitted reviews of any evaluation of specialized design conditions.

EXAMPLE NO. 1

A 300 mm PVC pipe is to be installed with a maximum of 7.3 m of cover. Proposed material for use as bedding and initial backfill is standard City of Edmonton bedding sand. This material has been confirmed to have a fines content in the 5-12% range.

The trench configuration is anticipated to be a wyed sub-ditch type of trench with a trench width of O.D. plus 0.6 m at pipe depth (0.9 m).



The installation location is within the right-of-way of a typical residential subdivision within the City of Edmonton.

Based on geotechnical investigations carried out in the area, native soils in the pipe zone are predominately comprised of cohesive soils with visual descriptions varying from soft to very soft. Grain size approaches silt or varved silts in clay. Based on the borehole investigations these soils are reported to have unconfined compressive strengths on the order of 15-20 kPa at anticipated pipe depth.

Design computations would include:

1. **Dead and Live Load**

- a. **Dead Load** (as per Clause 12.1 of the Guidelines)

$$W_D = \rho \times g \times H \times B_C$$

$$W_D = (2100\text{kg/m}^3) \times (9.8064 \text{ m/s}^2) \times 7.3 \text{ m} \times 0.3 \text{ m} \times (1 \text{ kN}/1000 \text{ N})$$

$$= 45.10 \text{ kN/m}$$

b. Live Load

We will use the AASHTO Live Load calculation method for simplicity. As this is a residential street we will assume only 1 large truck as opposed multiple passing trucks. Using this method average pressure is calculated (in SI units) by:

$$w_L = \frac{\text{Axle Load}}{(2.34 \text{ m} + 1.75H)(0.25 \text{ m} + 1.75H)}$$

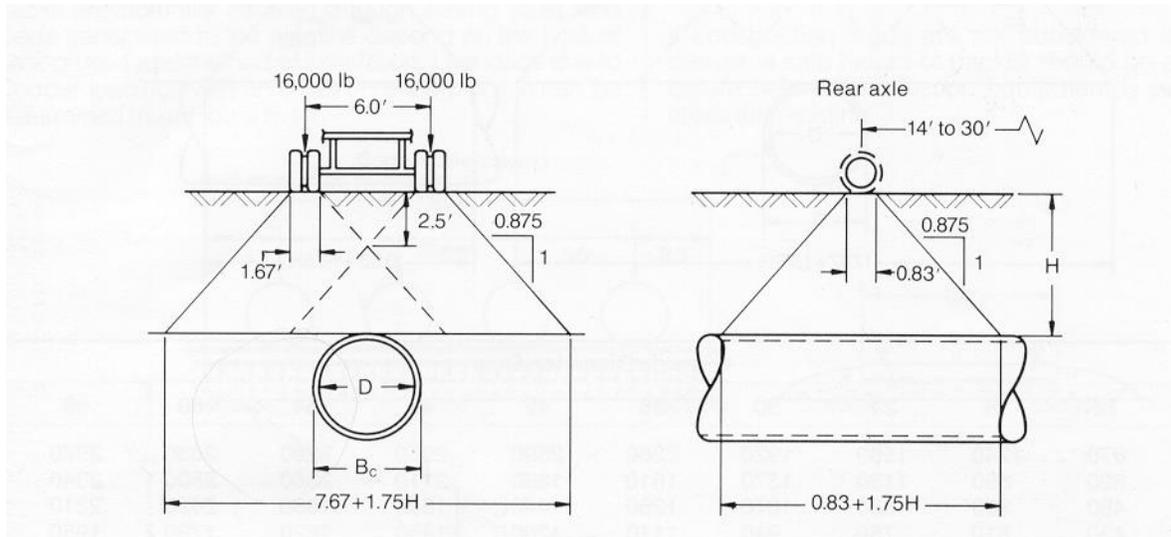
Total live load per unit length of pipe is then calculated by:

$$W_L = w_L B_C (1 + I_f),$$

where, I_f = Impact factor

Typical Impact factors (I_f) range from 0.5 at 0.3 m of cover to 0 at 1.8 m of cover or greater.

The AASHTO method is calculating an average stress at pipe depth based on the load distribution assumptions noted in the Figure below.



We will use an AASHTO HS 20 vehicle (depicted above in Imperial units), which has a total axle load of 142.34 kN (32,000 lbf).

Using SI units, live load pressure is then:

$$w_L = \frac{142.34 \text{ kN}}{(2.34\text{m} + 1.75 \times 7.3\text{m})(0.25\text{m} + 1.75 \times 7.3\text{m})} = 0.72 \frac{\text{kN}}{\text{m}^2} = 0.72 \text{ kPA}$$

Total live load is therefore,

$$W_L = 0.72 \text{ kN/m}^2 \times 0.3 \text{ m} (1 + 0) = 0.22 \text{ kN/m}$$

c. Total Dead + Live Load as a Pressure

As per Clause 13.2.1.3 of our Guidelines:

$$P = \frac{W_D + W_L}{B_C}$$

Therefore, our total live + dead load is:

$$P = \frac{45.0 \frac{kN}{m} + 0.22 \frac{kN}{m}}{0.30 \text{ m}} = 151.06 \frac{kN}{m^2} = 151.06 \text{ kPa}$$

We now move on to the Iowa Formula.

2. Evaluate Special Design Conditions

Based on a review of Section 12.3 of the Guidelines, none present.

3. Determine Modulus of Soil Reaction E'_{design}

E'_{design} will be a function of our embedment material, E'_b , native soil conditions, E'_{native} , and our selected trench width, B_D .

Standard the City of Edmonton bedding sand with less than 12% fines, is a Class II embedment material (Table 1 of the Guidelines). We will assume a minimum of 90% of the maximum standard Proctor dry density (SPD) will be achieved. Based on Table 4 of the Guidelines, E'_b at all heights of cover greater than 4 m is 11.2 MPa or **11,200 kPa**.

The native soil conditions, E'_{native} , can be estimated based on the geotechnical data. Based on Table 5 of the Guidelines, unconfined compressive strengths of 15-20 kPa for the native soils do indeed correspond to the visual descriptor very soft. The native E'_{native} , can read from Table 5 as **1380 kPa**.

The composite value for E'_{design} can be estimated by determining the modifying factor, S_c , from Table 6 of the Guidelines by knowing:

$$\frac{E'_{native}}{E'_b} = \frac{1380 \text{ kPa}}{11,200 \text{ kPa}} = 0.12, \text{ and}$$

$$\frac{B_D}{B_C} = \frac{0.9 \text{ m}}{0.3 \text{ m}} = 3$$

Interpolating from the table, $S_c = 0.81$, E'_{design} can be calculated by:

$$E'_{design} = S_c \times E'_b = 0.81 \times 11,200 \text{ kPa} = 9090 \text{ kPa}$$

4. Select remaining parameters for the Modified Iowa Formula

We will need values for:

$$\text{Deflection Lag} = D_L = 1.0,$$

(Clause 13.2.1.1 of the Guidelines) where a Prism Load is used in design, and

$$\text{Bedding Factor} = K = 0.10$$

as per Clause 13.2.1.2 of the Guidelines for 60°-75° bedding angle, and

$$\text{Pipe Stiffness} = PS = 320 \text{ kPa}$$

for a DR 35 PVC pipe as per Clause 13.2.1.4 of the Guidelines.

5. Calculate Horizontal Deflection

Using the Modified Iowa formula calculate maximum anticipated long-term horizontal deflection (Clause 13.2.1 of the Guidelines):

$$\frac{\Delta x}{d} (\%) = \frac{100 D_L K P}{0.149(PS) + 0.061 E'_{design}} = \frac{100 \times 1.0 \times 0.10 \times 151.06 \text{ kPa}}{0.149 \times 320 \text{ kPa} + 0.061 \times 9090 \text{ kPa}} = 2.51\%$$

6. Calculate Vertical Deflection

Calculate the deflection ratio with Masada's simplified formula as follows (Clause 13.2.1.7 of the Guidelines):

$$\frac{|\Delta y|}{\Delta x} = 1 + \frac{0.0094 E'_{design}}{(PS)}$$

$$\frac{|\Delta y|}{\Delta x} = 1 + \frac{0.0094 \times 9090 \text{ kPa}}{320 \text{ kPa}} = 1.27$$

Therefore, anticipated long-term vertical deflection equals:

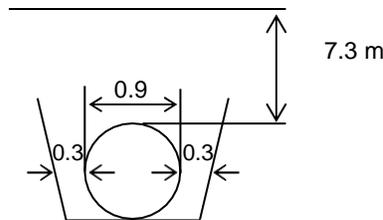
$$\frac{\Delta y}{d} = \frac{\Delta x}{d} \times 1.27 = 2.51\% \times 1.27 = 3.18\%$$

Which is less than our long-term acceptable limit of 7.50% as per Table 3 of the Guidelines, and is O.K.

EXAMPLE NO. 2

A 900 mm PVC pipe is to be installed with a maximum of 7.3 m of cover. Proposed material for use as bedding and initial backfill is standard the City of Edmonton bedding sand. This material has been confirmed to have a fines content in the 5-12% range.

The trench configuration is anticipated to be a wyed sub-ditch type of trench with a trench width of O.D. plus 0.6 m at pipe depth (1.5 m).



The installation location is within the busier right-of-way that may encounter multiple passing trucks.

Based on geotechnical investigations carried out in the area, native soils in the pipe zone are predominately comprised of cohesive soils as per Example No. 1 with visual descriptions varying from soft to very soft. Grain size approaches silt or varved silts in clay. Based on the borehole investigations these soils are reported to have unconfined compressive strengths on the order of 15-20 kPa at anticipated pipe depth.

Design should determine:

1. Dead and Live Load

a. Dead Load (as per Clause 12.1 of the Guidelines)

$$W_D = \rho \times g \times H \times B_c$$

$$W_D = (2100 \text{ kg/m}^3) \times (9.8064 \text{ m/s}^2) \times 7.3 \text{ m} \times 0.9 \text{ m} \times (1 \text{ kN/1000 N})$$

$$= 135.30 \text{ kN/m}$$

b. Live Load

We will use the AASHTO Live Load calculation method for multiple passing trucks. Using this method average pressure is calculated (in SI units) by:

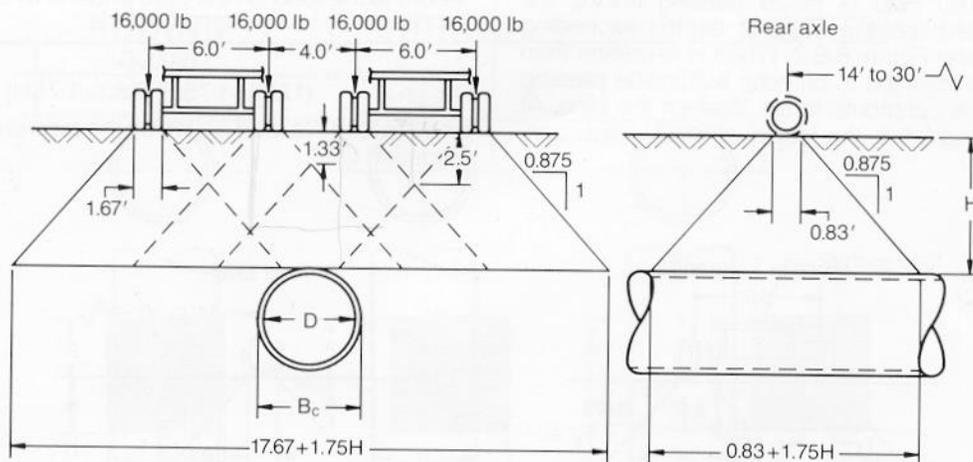
$$w_L = \frac{\text{Axle Load}}{(5.39 \text{ m} + 1.75H)(0.25 \text{ m} + 1.75H)}$$

Total live load per unit length of pipe is then calculated by:

$$W_L = w_L B_c (1 + I_f),$$

where, I_f = Impact factor

Typical Impact factors (*I*) range from 0.5 at 0.3 m of cover to 0 at 1.8 m of cover or greater. The AASHTO method for multiple trucks is calculating an average stress at pipe depth based on the load distribution assumptions noted in the Figure below.



We will continue to use an AASHTO HS 20 vehicle (depicted above in Imperial units), which has a total axle load of 142.34 kN (32,000 lbf) per truck for a total load of 284.69 kN (64,000 lbf).

Using SI units, live load pressure is then:

$$w_L = \frac{284.69 \text{ kN}}{(5.39 \text{ m} + 1.75 \times 7.3 \text{ m})(0.25 \text{ m} + 1.75 \times 7.3 \text{ m})} = 1.20 \frac{\text{kN}}{\text{m}^2} = 1.2 \text{ kPa}$$

$$W_L = 1.20 \frac{\text{kN}}{\text{m}^2} \times 0.9 \text{ m} \times (1 + 0) = 1.08 \frac{\text{kN}}{\text{m}}$$

Total live load is therefore,

c. Total Dead + Live Load as a Pressure

As per Clause 13.2.1.3 of our Guidelines:

$$P = \frac{W_D + W_L}{B_c}$$

Therefore, our total live + dead load is:

$$P = \frac{135.3 \frac{\text{kN}}{\text{m}} + 1.08 \frac{\text{kN}}{\text{m}}}{0.9 \text{ m}} = 151.54 \frac{\text{kN}}{\text{m}^2} = 151.54 \text{ kPa}$$

We now move on to the Iowa Formula.

2. Evaluate Special Design Conditions

Based on a review of Section 12.3 of the Guidelines, none present.

3. Determine Modulus of Soil Reaction E'_{design}

E'_{design} will be a function of our embedment material, E'_b , native soil conditions, E'_{native} , and our selected trench width, B_D .

Standard the City of Edmonton bedding sand with less than 12% fines, is a Class II

embedment material (Table 1 of the Guidelines). We will assume a minimum of 90% of the maximum standard Proctor dry density (SPD) will be achieved. Based on Table 4 of the Guidelines, E'_b at all heights of cover greater than 4 m is 11.2 MPa or **11,200 kPa**.

The native soil conditions, E'_{native} , can be estimated based on the geotechnical data. Based on Table 5 of the Guidelines, unconfined compressive strengths of 15-20 kPa for the native soils do indeed correspond to the visual descriptor very soft. The native E'_{native} , can read from Table 5 as **1380 kPa**.

The composite value for E'_{design} can be estimated by determining the modifying factor, S_c , from Table 6 of the Guidelines by knowing:

$$\frac{E'_{native}}{E'_b} = \frac{1380 \text{ kPa}}{11,200 \text{ kPa}} = 0.12, \text{ and}$$

$$\frac{B_D}{B_C} = \frac{1.5 \text{ m}}{0.9 \text{ m}} = 1.67$$

Interpolating from the table, $S_c = 0.23$, E'_{design} can be calculated by:

$$E'_{design} = S_c \times E'_b = 0.23 \times 11,200 \text{ kPa} = 2630 \text{ kPa}$$

4. Select remaining parameters for the Modified Iowa Formula

We will need values for:

$$\text{Deflection Lag} = D_L = 1.0 ,$$

(Clause 13.2.1.1 of the Guidelines) where a Prism Load is used in design, and

$$\text{Bedding Factor} = K = 0.10$$

as per Clause 13.2.1.2 of the Guidelines for 60°-75° bedding angle, and

$$\text{Pipe Stiffness} = PS = 320 \text{ kPa}$$

for a DR 35 PVC pipe as per Clause 13.2.1.4 of the Guidelines.

5. Calculate Horizontal Deflection

Using the Modified Iowa formula, calculate the maximum anticipated long-term horizontal deflection (Clause 13.2.1 of the Guidelines):

$$\frac{\Delta x}{d} (\%) = \frac{100 D_L K P}{0.149(PS) + 0.061 E'_{design}} = \frac{100 \times 1.0 \times 0.10 \times 151.54 \text{ kPa}}{0.149 \times 320 \text{ kPa} + 0.061 \times 2630 \text{ kPa}} = 7.28\% < 7.5\%$$

6. Calculate Vertical Deflection

Calculate the deflection ratio with Masada's simplified formula as follows (Clause 13.2.1.7 of the Guidelines):

$$\left| \frac{\Delta y}{\Delta x} \right| = 1 + \frac{0.0094 E'_{design}}{(PS)}$$

$$\left| \frac{\Delta y}{\Delta x} \right| = 1 + \frac{0.0094 \times 2630 \text{ kPa}}{320 \text{ kPa}} = 1.08$$

Therefore, anticipated long-term vertical deflection equals:

$$\frac{\Delta y}{d} = \frac{\Delta x}{d} \times 1.08 = 7.28\% \times 1.08 = 7.84\% > 7.5\% \quad \blacktriangleleft \text{Exceeds long-term deflection limit}$$

This is greater than our long-term acceptable limit of 7.50% (see Table 3 of the Guidelines) and is not O.K. We could either increase minimum trench width or upgrade the density

requirements for the backfill material. We are going to try increasing minimum density requirements to 95% SPD. If this is truly required as a minimum density our Guidelines recommends we utilize Class I embedment materials, which in this application would well be advised to be a crushed, well-graded aggregate material to prevent long-term soil migration.

This changes things as follows:

$E'_b = 20.7 \text{ MPa} = 20,700 \text{ kPa}$ based on Table 4.

$$\frac{E'_{native}}{E'_b} = \frac{1380 \text{ kPa}}{20,700 \text{ kPa}} = 0.07, \text{ and}$$

$$\frac{B_D}{B_C} = \frac{1.5 \text{ m}}{0.9 \text{ m}} = 1.67$$

By interpolating in Table 6 using a $\frac{E'_{native}}{E'_b}$ value of 0.1, we get:

$$E'_{design} = S_c \times E'_b = 0.20 \times 20,700 \text{ kPa} = 4140 \text{ kPa}$$

Horizontal deflection becomes:

$$\frac{\Delta x}{d} (\%) = \frac{100D_L KP}{0.149(PS) + 0.061E'_{design}} = \frac{100 \times 1.0 \times 0.10 \times 151.54 \text{ kPa}}{0.149 \times 320 \text{ kPa} + 0.061 \times 4140 \text{ kPa}} = 5.05\%$$

Our deflection ratio becomes:

$$\left| \frac{\Delta y}{\Delta x} \right| = 1 + \frac{0.0094 \times 4140 \text{ kPa}}{320 \text{ kPa}} = 1.12$$

Anticipated long-term vertical deflection then becomes

$$\frac{\Delta y}{d} = \frac{\Delta x}{d} \times 1.12 = 5.05\% \times 1.12 = 5.66\% < 7.5\%$$

An alternative to upgrading embedment material would be to increase trench width. Upgrading the trench width to $2.5 \times B_C$ may not only be more effective at reducing deflection but more practical than changing embedment materials. Let us see how it would fair. Our design E' becomes:

$$\frac{E'_{native}}{E'_b} = \frac{1380 \text{ kPa}}{11,200 \text{ kPa}} = 0.12, \text{ and}$$

$$\frac{B_D}{B_C} = 2.5$$

$$E'_{design} = S_c \times E'_b = 0.62 \times 11,200 = 6980 \text{ kPa}$$

Horizontal deflection becomes:

$$\frac{\Delta x}{d} (\%) = \frac{100D_L KP}{0.149(PS) + 0.061E'_{design}} = \frac{100 \times 1.0 \times 0.10 \times 151.54 \text{ kPa}}{0.149 \times 320 \text{ kPa} + 0.061 \times 6980 \text{ kPa}} = 3.2\%$$

Our deflection ratio becomes:

$$\left| \frac{\Delta y}{\Delta x} \right| = 1 + \frac{0.0094 \times 6980 \text{ kPa}}{320 \text{ kPa}} = 1.21$$

Anticipated long-term vertical deflection now becomes:

$$\frac{\Delta y}{d} = \frac{\Delta x}{d} \times 1.21 = 3.2\% \times 1.21 = 3.86\%$$

This level of anticipated long-term deflection is O.K.

From this analysis, it is evident that increasing trench width is more effective at reducing deflection than increasing densities in the embedment zone. It also doesn't require acquiring a brand new bedding material or force a contractor to achieve densities that are, from a practical perspective, much harder to achieve.