

BS 9295:2020



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Guide to the structural design of buried pipes

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Foreword

Publishing information

This British Standard is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 29 February 2020. It was prepared by Technical Committee B/505, *Wastewater engineering*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

This British Standard supersedes BS 9295:2010, which is withdrawn.

Relationship with other publications

This British Standard is complementary to BS EN 1295-1:2019 and PD CEN/TR 1295-2:2005.

BS EN 1295-1:2019 specifies general requirements for the structural design of buried pipes under various conditions of loading. Guidance is also given on the application of the nationally established methods of design declared by, and used in, CEN member countries at the time it was prepared. The established United Kingdom method is described as BS 9295.

PD CEN/TR 1295-2:2005 summarizes the nationally established methods of design made available to CEN. The United Kingdom method is described in [A.9](#), which is consistent with BS EN 1295-1:1997.

This British Standard gives further information to facilitate in full the structural design of buried pipes under various conditions of loading using the established United Kingdom method; it does not alter any of the provisions of BS EN 1295-1:2019.

Information about this document

This is a full revision of the standard, and introduces the following principal changes:

- incorporation of the content of National Annex A to BS EN 1295:1997; and
- inclusion of the Gumbel method for the design of buried pipes.

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Where websites and webpages have been cited, they are provided for ease of reference and are correct at the time of publication. The location of a webpage or website, or its contents, cannot be guaranteed.

Use of this document

As a guide, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification or a code of practice.

Presentational conventions

The guidance in this standard is presented in roman (i.e. upright) type. Any recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

0 Introduction

This revision of BS 9295 has been prepared to maintain alignment with BS EN 1295-1:2019, which now cites BS 9295 as the primary source of the methods accepted for the structural design of buried pipes in the UK. The opportunity has been taken to review various shortcomings in the earlier UK design methods, many of which have arisen due to changes in the nature of pipes over the period since those methods were originally published.

Changes from the methods previously published as BS EN 1295-1 and BS 9295 include the introduction of a new conceptual model for the design of buried flexible pipes focused on large-diameter structured-wall thermoplastics pipes, explicit consideration of the quality of workmanship envisaged during installation, non-circular shaped concrete pipes, and a wider range of bedding materials. Non-circular shaped pipes in materials other than concrete are not in common use in the UK so their design is not covered by this edition.

1 Scope

This British Standard gives the UK established method for the structural design of buried pipes under various conditions of loading. The procedures are explained, with tables listing the recommended design values for the appropriate variables in the design formulae, figures providing graphical information on vehicle surcharge loadings, and tables of rigid pipe bedding factors.

NOTE The scope of BS EN 1295-1:2019 is restricted to the structural design of water supply pipelines, drains and sewers, and other water industry pipelines, whether operating at atmospheric, greater or lesser pressure.

Some aspects of longitudinal effects are discussed (see [Annex A](#)), but this topic is not fully covered.

The initial selection of options for the pipe material involves consideration of matters beyond structural design and is outside the scope of this standard.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application¹⁾. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Standards publications

BS EN 545, *Ductile iron pipes, fittings, accessories and their joints for water pipelines – Requirements and test methods*

BS EN 598, *Ductile iron pipes, fittings, accessories and their joints for sewerage applications – Requirements and test methods*

BS EN 622-4, *Fibreboards – Specifications – Part 4: Requirements for softboards*

[BS EN 1295-1:2019](#), *Structural design of buried pipelines under various conditions of loading – Part 1: General requirements*

BS EN 1610:2015, *Construction and testing of drains and sewers*

BS EN 1992-1-1:2004+A1:2014, *Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings*

BS EN 13242:2002+A1:2007, *Aggregates for unbound and hydraulically bound materials for use in civil engineering work and road construction*

¹⁾ Documents that are referred to in an informative manner are listed in the Bibliography.

Other publications

[N1] YOUNG, O.C. and M.P. O'REILLY. *A guide to design loadings for buried rigid pipes*. Transport and Road Research Laboratory, Department of Transport. London: HMSO, 1983, Second Impression 1987.

[N2] HIGHWAY AUTHORITIES AND UTILITIES COMMITTEE (HAUC). *New Roads and Street Works Act 1991. Specification for the reinstatement of openings in highways*. HAUC (UK), 2002.

[N3] WASTE AND RESOURCES ACTION PROGRAMME (WRAP). *Quality Protocol: Aggregates from inert waste. End of waste criteria for the production of aggregates from inert waste*. Banbury: WRAP. 2013.

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this British Standard, the terms and definitions given in BS EN 1295-1:2019 and the following apply.

3.1.1 buried pipe system

composite structure comprising the pipe, its joints and the surrounding soil embedment

3.1.2 pipe-soil system

conceptual two-dimensional structural unit comprising the buried pipe and an annulus of the surrounding soil, including pipe embedment and native ground, which interact to resist applied external and internal loads and/or imposed deformations

NOTE The size of soil annulus varies according to the relative stiffness of the pipe and the soil.

3.2 Symbols

For the purposes of this British Standard, the following symbols apply.

NOTE The following table gives the units for the symbols, unless these are given as otherwise in the text.

Symbols	Description	Units ^{A)}
A	cross-sectional area of wall per unit length	mm ²
B_c	outside diameter of pipe	m
B'_c	combined width of multiple pipes laid in parallel	m
B_d	effective width of trench	m
C_c	soil load coefficient in embankment conditions	–
C_d	soil load coefficient in narrow trench conditions	–
C_L	soil modulus adjustment factor (Leonhardt's coefficient)	–
C_w	water load coefficient	–
c_u	undrained compressive strength of cohesive soil	kPa
D	mean diameter of pipe (measured to neutral axis of wall)	m
D_f	strain factor	–
D_i	internal diameter of the pipe	m
D_L	deflection lag factor	–
D_{LSr}	deflection lag factor for semi-rigid pipes	–
D_R	reduction factor on long-term deflection due to internal pressure	–
E	flexural (Young's) modulus of elasticity of pipe material	–

Symbols	Description	Units ^{A)}
E_h	hoop tensile modulus of elasticity of pipe material	–
E_p	modulus of elasticity of pipe material	MPa
E_p^*	plane strain modulus of pipe material	MPa
E_x	creep modulus of elasticity of pipe at time x	MPa
E'	overall modulus of soil reaction	MPa
E'_2	embedment soil (Spangler) modulus	MN/m ²
E'_3	native soil (Spangler) modulus	MN/m ²
F_b	buoyant force (destabilizing force)	kN/m
F_d	downward force (stabilizing force)	kN/m
F_m	bedding factor	–
F_s	factor of safety against buckling	–
F_{se}	factor of safety for rigid pipe material (external load design)	–
F_{si}	factor of safety for rigid pipe (internal pressure design)	–
f_{ctm}	factored tensile stress due to bending	MPa
$f_{ctm:fl}$	factored flexural tensile stress due to bending	MPa
GWL	level of groundwater	–
H	depth of cover to top of pipe	m
H_e	height of plane of equal settlement above top of pipe	m
HN	internal height	m
H_w	height of water table above invert of the pipe	m
I	second moment of area of the pipe wall per unit length	mm ⁴ /mm
I_σ	influence value for determination of soil stresses produced by surface loads	–
K	coefficient of lateral earth pressure	–
K_a	fully active value of K	–
K_o	at-rest value of K	–
k_{sc}	shallow cover reduction factor on soil modulus	–
K_x	deflection coefficient	–
M_H	bending moment, vertical load (soil and external water)	–
M_p	compacted density (modified Proctor density)	%
M_s	constrained soil modulus	MPa
M_{sb}	constrained soil modulus of the pipe zone embedment	MPa
M_{sn}	constrained modulus of native soil of the trench wall	MPa
M_{Ult}	total factored bending moment	kNm/m
M_v	bending moment, horizontal load (soil and external water)	–
m_{qh}	bending moment coefficient for egg-shaped pipe, horizontal	–
m_{qv}	bending moment coefficient for egg-shaped pipe, vertical	–
N	ring thrust in pipe wall	kN/m
N_{SPT}	penetration resistance in blows per 300 mm from a standard penetration test (SPT)	–
N_y	distortional component of ring thrust	kN/m
N_z	uniform component of ring thrust	kN/m
n	pipe-soil stiffness factor	–
n_w	number of waves around circumference of pipe	–
P	vertical pressure due to soil and surcharge	kN/m ²

Symbols	Description	Units ^{A)}
P_{cr}	critical linear buckling pressure of flexible pipes	kN/m ²
P_{crl}	long-term critical buckling pressure	kN/m ²
P_{crs}	short-term critical buckling pressure	kN/m ²
P_e	vertical soil pressure	kN/m ²
P_h	horizontal pressure due to soil load	kN/m ²
P_{Hw}	horizontal pressure at pipe springing level due to external water pressure	kN/m ²
P_i	internal water pressure	kN/m ²
P_{op}	working pressure	kN/m ²
P_r	radial pressure	kN/m ²
P_s	vehicle surcharge pressure	kN/m ²
P_u	short-term vacuum pressure	kN/m ²
P_{ult}	ultimate bursting pressure	kN/m ²
P_v	vertical pressure at mid-height of the pipe	kN/m ²
P'_v	vertical pressure due to soil load	kN/m ²
P_w	hydrostatic pressure due to groundwater at mid-height of the pipe	kN/m ²
P_{Wc}	vertical pressure due to soil load (narrow trench conditions)	kN/m ²
$P_{W'c}$	vertical pressure due to soil load (wide trench conditions)	kN/m ²
P_y	distortional pressure component	kN/m ²
P_z	uniform pressure component	kN/m ²
P'_h	horizontal pressure component of soil effective stress	kN/m ²
P'_v	vertical pressure expressed as a soil effective stress	kN/m ²
P'_y	distortional pressure component of soil effective stress	kN/m ²
P'_z	uniform pressure component of soil effective stress	kN/m ²
ρ	projection ratio	–
q	out-of-roundness factor	–
Q_{vk}	characteristic value of vertical load (concentrated load)	–
q_{vk}	characteristic value of vertical load (uniformly distributed load)	–
r_{i2}	internal radius at crown	m
r_m	mean pipe radius at springing level	m
r_{sd}	settlement deflection ratio	–
S_c	soil support combination factor	–
S_f	settlement of pipe invert	mm
S_g	settlement of the natural ground	mm
SLS	serviceability limit state	–
S_m	settlement of fill	mm
SN	ring stiffness class	–
S_x	ring creep stiffness (flexural) at time x	MPa
S_y	flexural ring stiffness of the pipe	MPa
S_z	compression stiffness of the pipe ring	MPa
t	pipe wall thickness	m
t_1	wall thickness at springings	m
t_2	wall thickness at the crown	m
t_3	wall thickness at the invert	m
ULS	ultimate limit state	–
U_s	uniform surcharge	kN/m ²

Symbols	Description	Units ^{A)}
$W_{e,Ed}$	external load per unit length of pipe	kN/m
W_c	soil load per unit length of pipe in narrow trench conditions	kN/m
W'_c	soil load per unit length of pipe in embankment or wide trench conditions	kN/m
W_{csu}	concentrated surcharge load per unit length of pipe	kN/m
W_d	weight of dry soil above pipe	kN/m
W_e	total external load per unit length of pipe	kN/m
W_f	weight of saturated soil above pipe	kN/m
WN	internal width	m
W_p	proof load for rigid pipes	kN/m
W_{pipe}	pipe weight	kg/m
W_t	crushing strength of rigid pipes (maximum load for concrete pipes)	kN/m
W'_t	crushing strength of rigid pipes with internal pressure per unit length of pipe	kN/m
W_w	weight of liquid content of pipe	kN/m
x	elapsed or extrapolated time under load in creep test	h
Y	flexural stiffness ratio	–
Z	compression stiffness ratio	–
α	bedding angle	°
α_{EC1}	load classification factor from BS EN 1991-2	–
α_z	radial arching factor	–
γ	unit weight of soil	kN/m ³
γ_d	unit weight of dry soil	kN/m ³
γ_G	partial factor for permanent actions	–
$\gamma_{G,intw}$	partial factor for internal water weight	–
γ_H	geostatic load value	–
γ_Q	partial factor for traffic/surcharge	–
γ_w	unit weight of water	kN/m ³
Δ	pipe diameter change	mm
Δ_0/D	component of deflection built in during laying of pipe and construction of embedment	mm
Δ_v	absolute vertical deflection	mm
δ	relative change of pipe diameter	%
δ_v	vertical ring deflection	%
δ_y	total out-of-round deflection of the pipe ring	%
δ_{y0}	initial out-of-round deflection during construction	%
δ_{y1}	out-of-round deflection due to external load	%
δ_{y2}	out-of-round deflection of pipe due to long-term consolidation settlement of embedment	%
δ_z	uniform component of ring deflection	%
ϵ_b	bending strain in pipe wall	%
ϵ_c	combined strain in pipe wall	%
$\epsilon_{c,max}$	maximum compressive fibre strain	%
$\epsilon_{t,max}$	maximum tensile fibre strain	%
ϵ_y	ring bending strain	%
ϵ_z	ring compressive strain	–
μ	coefficient of sliding friction within a mass of fill	–
μ'	coefficient of sliding friction of backfill against trench sides	–
ν	Poisson's ratio of pipe material	–

Symbols	Description	Units ^{A)}
σ	stress in the pipe	MPa
σ_{bs}	bending stress in pipe wall	MPa
σ_c	design hoop stress (previously referred to as combined stress)	MPa
σ_z	mean ring compressive stress at any point in pipe wall	MPa
^{A)} Unless otherwise indicated in the text.		

4 Concepts used in the structural design of buried pipes

4.1 The buried pipe system

The buried pipe system is a composite structure comprising the pipe, its jointing system and the surrounding soil embedment.

All pipes are longitudinal conveyors of fluids, and in the structural design of the buried pipe system it is generally necessary to take account of both the longitudinal and the cross-sectional response to both external and internal loads.

In determining the extent of influence of longitudinal effects on pipe design it is important to distinguish between flexibly jointed and continuously welded systems, and between gravity and pressure systems.

In both longitudinal and cross-sectional response, the soil embedment acts compositely with the pipe. The zone of structural influence of the soil surrounding the pipe depends on the relative properties of both pipe and soil and can extend beyond the trench fill and into the native ground.

4.2 Longitudinal and sectional behaviour

Most of the design formulae in this British Standard relate to the response of the buried pipe system in the plane of cross-section. However, the potential need to design for longitudinal stresses and strains should be taken into account, and include:

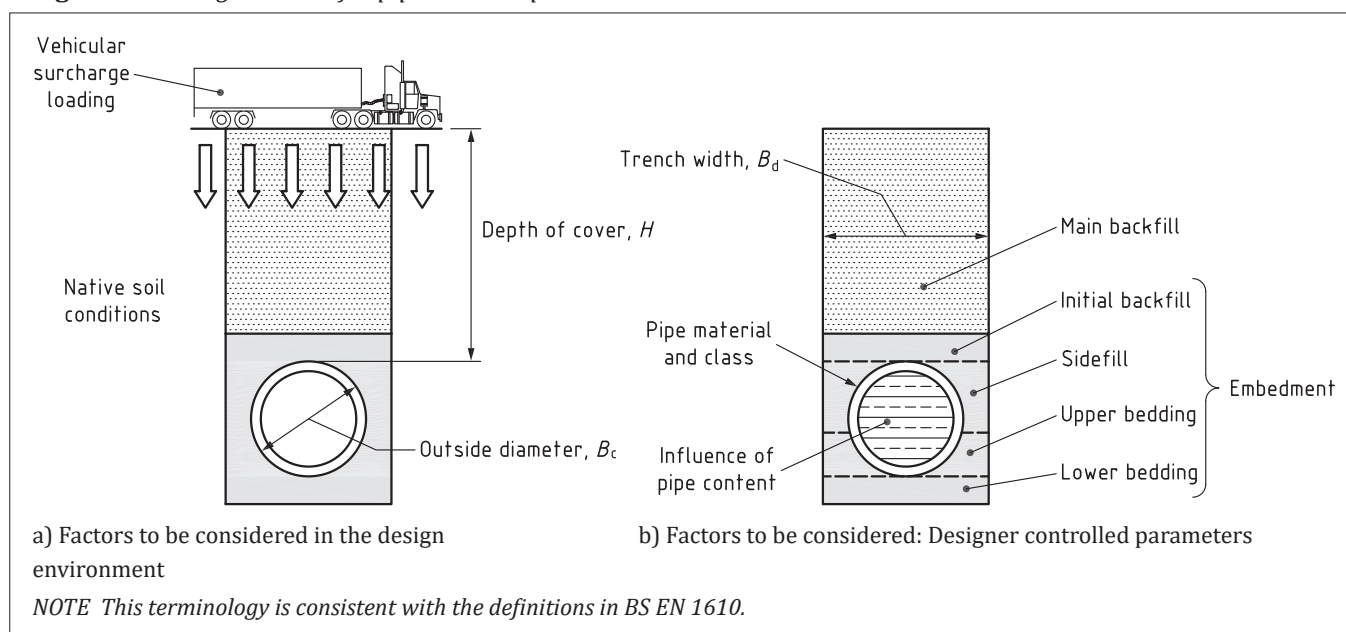
- longitudinal bending and/or joint rotations due to differential settlement or change of embedment (e.g. where the pipe enters other structures);
- for gravity pipes, settlement of the soil embedment which can be critical to control of line and level; and
- for pressure pipes, longitudinal pressurization stresses due, for example, to the Poisson effect and any changes of alignment which can be critical in the selection of joints, and of connections between pipes of different diameters or different materials.

4.3 Classification of buried pipes

Buried pipes are traditionally classified into one of three types: rigid, semi-rigid or flexible. This distinction relates to both the strain capacity of the pipe material and the stiffness of the pipe relative to that of the soil surround, especially in its cross-sectional response.

The purpose of the structural design of the cross-section of buried pipes is to ensure selection of optimum materials and embedments for the given installation, whilst meeting all the necessary design criteria to satisfy the limit states.

NOTE For design criteria see [Figure 1](#).

Figure 1 — Design criteria for pipes laid in open trench

Rigid pipe materials have low strain capacity and high relative stiffness and are designed on the basis of their inherent strength. Their deflection on loading is generally too small to derive significant structural support from the surrounding soil. Load is taken by the pipe and bending moments are developed in the pipe walls. Rigid pipes also attract an amplified backfill load upon burial and obtain a reaction from their bedding in response. The terms “soil load” and “backfill load” are used synonymously to describe the load imposed from the backfill after burial of the pipe.

Flexible pipe materials deflect towards an oval shape in response to loading and are able to develop significant lateral earth pressure from their surrounding embedment.

Semi-rigid pipe materials exhibit a range of behaviour between rigid and flexible.

Pipes of different materials are classified in the UK according to a strength or stiffness criterion to be proven by testing or otherwise established in design. Where the strength of pipes is established in a crushing test, they are classified as rigid. Typical classifications are given in [Table 1](#).

Table 1 — Pipe classification

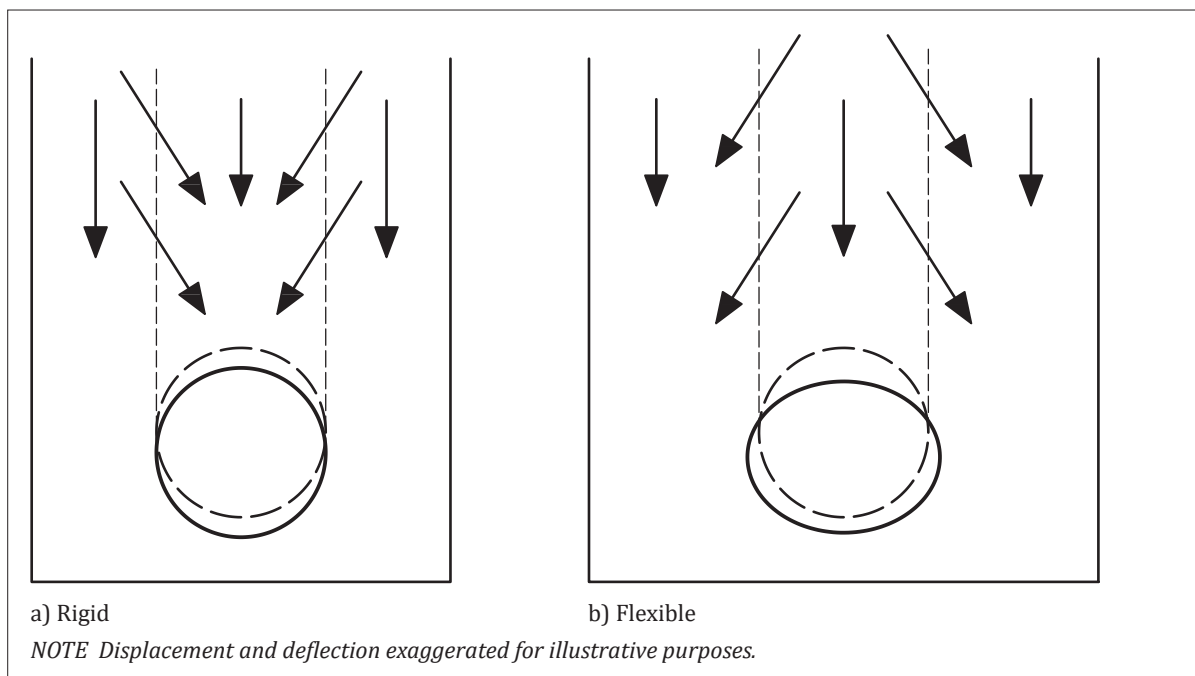
Pipe material	Classification
Clay	Rigid
Concrete	Rigid
Reinforced concrete	Rigid
Ductile iron	Semi-rigid
Polymer concrete	Semi-rigid
Thick-walled steel	Semi-rigid
Steel-reinforced thermoplastics ^{A)}	Flexible
Thermoplastics	Flexible
Glass reinforced plastics ^{A)}	Flexible
Thin-walled steel	Flexible

^{A)} Classed as composite.

When a pipe is installed in an open trench the ground is disturbed. The disturbed soil settles in the pipe trench, even after backfilling is complete. A rigid pipe is stiffer than the surrounding soil and, when consolidation of the backfill takes place in the pipe zone, the soil to the sides of the pipe tends to settle more than the soil over the pipe. By friction, load is transferred to the column of soil over the

pipe so that the pipe attracts more load than the geostatic load alone. In response to this, a rigid pipe might settle marginally into its bedding. A flexible pipe is less stiff than the surrounding soil, so when buried, the column of soil over the flexible pipe settles more than the columns of soil to the sides. In other words, the flexible pipe deflects on loading and tends to shed load away from itself. [Figure 2](#) illustrates this aspect of rigid and flexible pipe behaviour.

Figure 2 — Rigid and flexible pipe behaviour



With time, the load distribution effects diminish as the soil surround settles back to a normally consolidated state. For the wide trench condition illustrated in [Figure 2](#), the most onerous backfill loading condition for a rigid pipe is shortly after installation, before the backfill consolidates. Conversely, the highest predicted backfill load for a flexible pipe is after all the beneficial shedding of load has taken place, so the load used is the geostatic value γH with no coefficients.

[Figure 3](#) shows a schematic diagram of the changes in the soil pressures following the burial of rigid and flexible pipes. For rigid pipes the evolution of soil pressures is shown for both wide and narrow trench conditions. In the narrow trench condition where part of the backfill load is transferred in the short term by friction to the trench walls (see [Clause 6](#)), the breakdown of such friction with time can lead to a long-term increase of vertical load and compensating horizontal load on the pipe.

4.4 Design models

In the traditional view of rigid and flexible pipe behaviour represented in [Figure 2](#), only vertical applied loads are shown. In practice, the buried pipe structure is subject to both vertical and horizontal applied loads in the plane of the cross-section. At the interface of the pipe with the soil both these components of load are modified by relative deformation, as illustrated in [Figure 3](#).

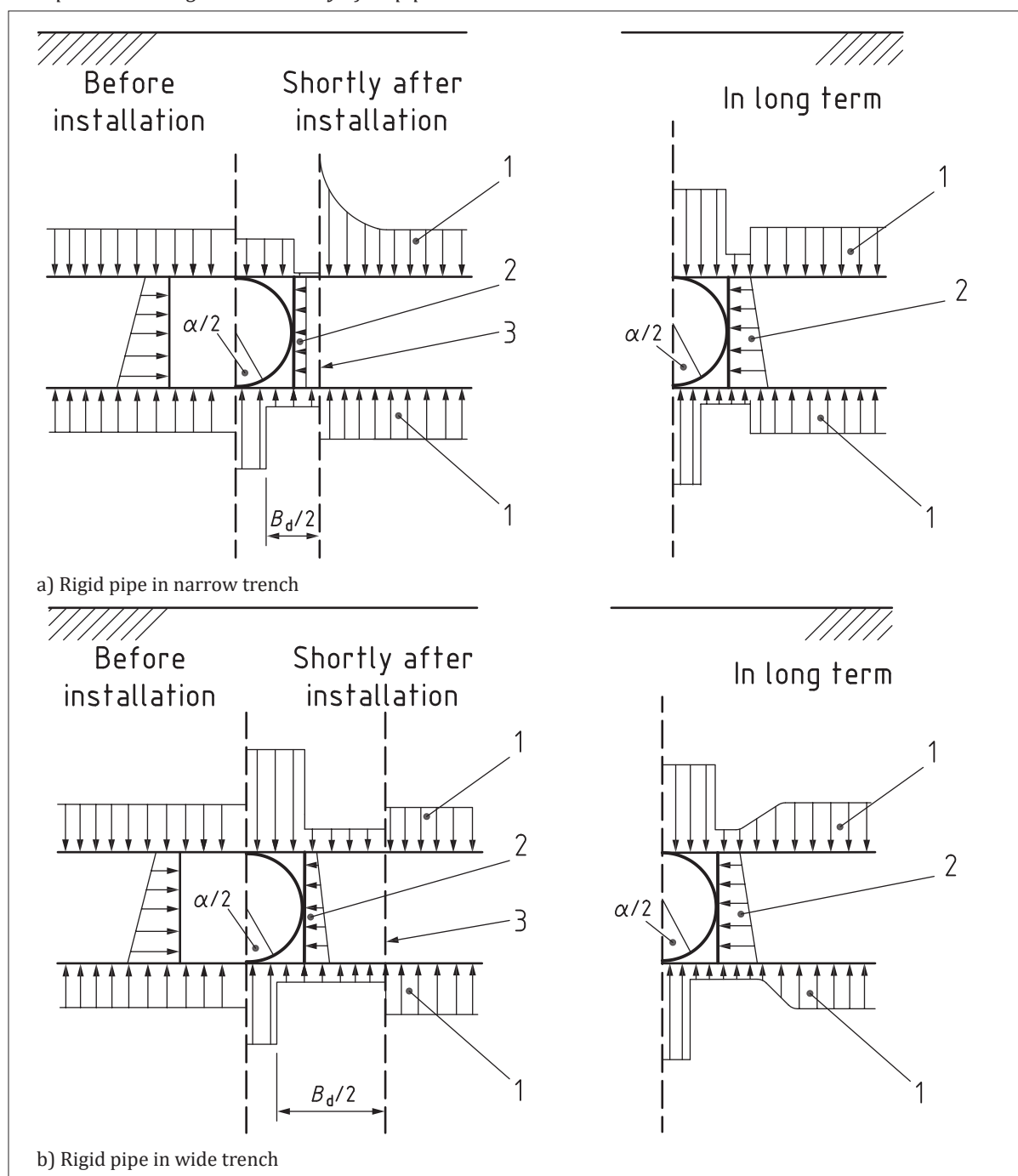
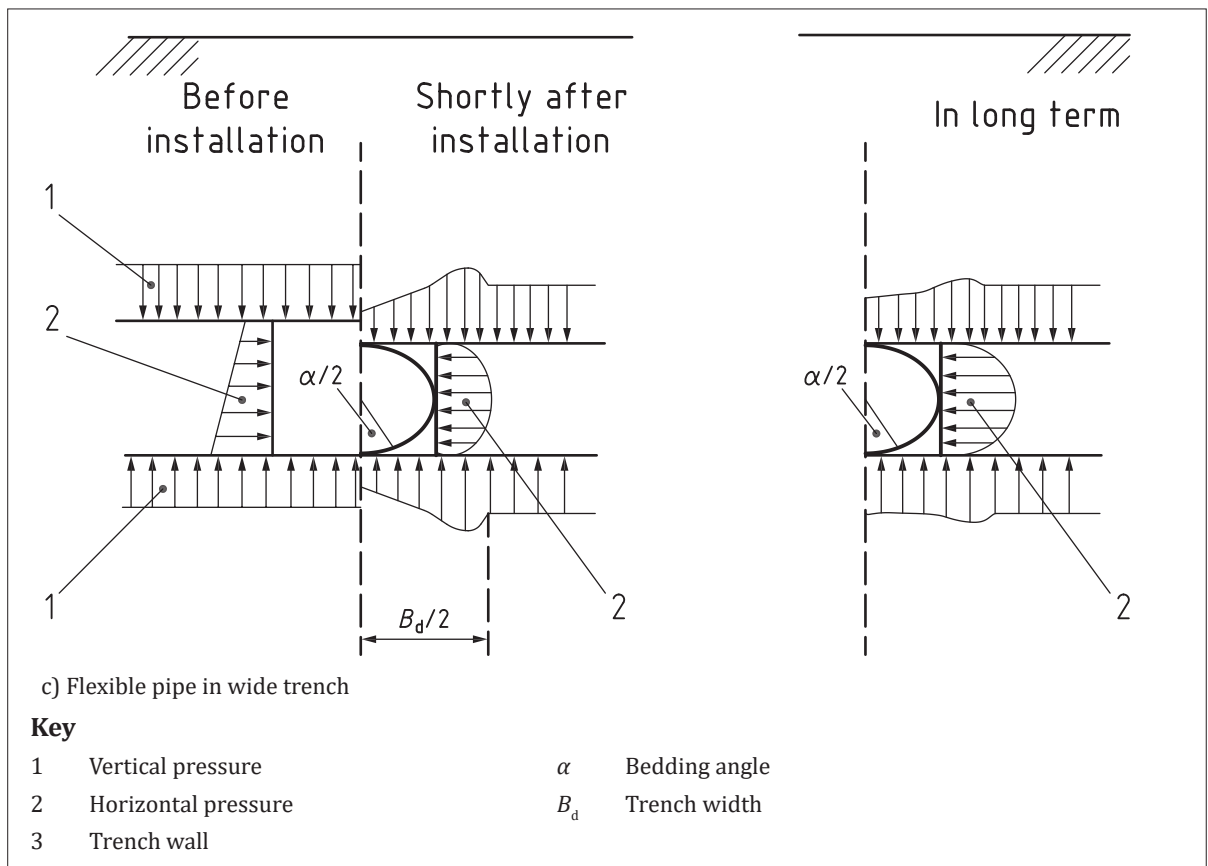
Figure 3 — Soil pressure changes immediately after pipe burial

Figure 3 (continued)



The traditional one-dimensional Marston-Spangler model of buried pipe loading originated with pioneering work at the Iowa Agricultural Engineering Station in the 1940s [1]. Marston examined the effect on vertical soil load reaching the crown of a buried rigid pipe of both the relative settlement of columns of soil either side of the pipe and friction on the walls of a vertically-sided trench. Noting that flexible corrugated metal pipes could support little vertical load on their own, Spangler developed a simple theoretical model in which the vertical load-carrying capacity of such pipes when buried is explained by passive lateral support from the soil surround in response to outward horizontal deflection of the pipe springings. Both models, however, neglect the horizontal component of applied load.

One consequence of neglecting the horizontal component of applied load in the traditional model is that the parameter used to characterize soil stiffness is not a fundamental engineering property of the soil and cannot be directly measured. The parameter concerned, the Spangler modulus, E' (also referred to as the modulus of soil reaction), is a semi-empirical parameter which is a function of the buried pipe system and the applied load distribution, and not the soil alone.

A second limitation of the traditional model is that it examines pipe response only in ring bending, which generates critical bending stresses in rigid pipes and deflection in flexible pipes. This, however, neglects stiffness and deformation of the pipe in a pure ring compression mode, which influences the response and critical design limit state of structured-wall flexible pipes in particular.

To overcome these limitations and to enable inclusion of structured-wall pipes, an additional design model (the Gumbel Method) is introduced in this document (see 7.3) with the following key features:

- replacement of Spangler modulus, E' , with a measurable constrained soil modulus, M_s , of the embedment, including both backfill and native soil; and

- adoption of a two-dimensional model of both loading, i.e. inclusion of the horizontal component of geostatic load, and response, i.e. in ring compression as well as ring bending, of the pipe-soil structure.

4.5 Short-term and long-term stiffness characteristics of polymeric pipe materials

4.5.1 Time-dependent material deformation expressed as apparent stiffness

All polymeric pipe materials, whether thermoplastics or thermosets, exhibit time-dependent creep deformation under load, or conversely relaxation of stresses with time where subject to fixed deformation. In the design of such pipes for limit states governed by stiffness, including deflection and buckling under net external pressure, it is conventional to express the effects of creep deformation or relaxation as an apparent time-dependent elastic modulus, E_x , of the material (also referred to as “creep modulus”) or apparent time-dependent ring stiffness, $S_x (= E_x I / D^3)$, of the pipe (also referred to as “ring creep stiffness”).

These time-dependent values are obtained by dividing the stress or load per unit length applied in a test by the strain or deflection respectively, measured at time x under load. For design load cases which endure beyond standard creep test durations of between 1 000 h and 10 000 h, the stiffness values are extrapolated to the relevant timeframe, up to the specified design life of the buried pipe.

The use of a stiffness parameter in this way to represent the effect of creep deformation under sustained load does not mean that the stiffness characteristics of polymeric pipe materials diminish with time. If a sample that has undergone previous creep deformation is unloaded and subjected to short-term testing, it generally exhibits the same stiffness as at the outset, or even marginally higher stiffness due to ageing effects.

4.5.2 Definitions of short-term stiffness

Conventionally, the value of elastic modulus or ring stiffness designated as “short-term” is that obtained from a standard ring stiffness test, of which the duration is different for thermoplastics and thermosetting pipes.

In the case of thermoplastics pipes, the duration of the standard ring stiffness test to BS EN ISO 9969 is between 36 s and 90 s, dependent on diameter. In the case of thermosetting (GRP) pipes, the total duration of the standard ring stiffness test to BS ISO 7685, including both ramp-up and hold steps, is 3 min. As appreciable creep deformation of thermoplastics occurs between 36 s or 90 s and 3 min under load, this means that the ring stiffness of a thermoplastics pipe of a declared SN (nominal ring stiffness) class is systematically lower on a like-for-like basis than that of a GRP pipe of the same SN class.

4.5.3 Definitions of long-term stiffness, creep ratio and creep factor

Conventionally, the value of elastic modulus or ring stiffness designated as “long-term” is that obtained by extrapolation to a design life of 50 years from a standard ring creep test. Again, however, there are differences in the standard tests for thermoplastics and GRP pipes which require care in the derivation of comparable values of long-term stiffness for design.

In the case of thermoplastics, BS EN ISO 9967 specifies a constant load ring test of 1 000 h duration, from which a “creep ratio” is derived as the ratio of deflection extrapolated to two years to the deflection measured after 6 min under load. Statistical tests are provided to determine the further duration of testing (up to maximum 10 000 h) required to extrapolate ring deflection to fifty years, to enable a fifty-year creep ratio to be determined where relevant for design. However, dividing the short-term ring stiffness determined in accordance with BS EN ISO 9969 by the two-year or fifty-year creep ratio leads to overestimates of the apparent ring stiffness at two years and fifty

years respectively, because it misses the, often substantial, creep deformation that takes place in a thermoplastics pipe between 36 s or 90 s and 6 min under load.

In the case of GRP, BS EN ISO 7864 (test for ring creep stiffness under dry conditions) and BS ISO 10468 (wet conditions) both determine a creep factor defined as the ratio of the deflection after 3 min under load to the deflection at time x , extrapolated from a test of standard 10 000 h duration. Unless otherwise specified, x is taken to be 50 years. It should be noted that:

- the creep factor defined for GRP and other thermosetting pipes is the inverse of the creep ratio defined for thermoplastics pipes; and
- unlike for thermoplastics, the long-term ring stiffness of GRP pipe can be validly obtained by multiplying the short-term ring stiffness determined in accordance with BS ISO 7685 by the dry or wet creep factor, as applicable, because the 3-minute datum point used for the evaluation of creep factor matches the duration of the short-term test.

Finally, one reason for the conventional assumption of a fifty-year design life is that this is the furthest extrapolation that can reliably be made based on test data up to 10 000 h. Where the proven durability of the pipe material based on criteria other than stiffness justifies a design life longer than 50 years, the determination of long-term modulus or ring stiffness requires creep testing for an extended period. For example, to reliably predict creep stiffness at 100 years, at least 20 000 h of data are required.

4.5.4 Relevant timeframes for derivation of stiffness values used in design

In the design of buried flexible pipes, the value of creep stiffness applied in calculations (whether elastic modulus or ring stiffness) should always be matched to the timeframe of the load case under consideration.

The “short-term” modulus or ring stiffness of thermoplastics or GRP pipes declared by manufacturers based on the respective standard tests referred to in [4.5.2](#) and [4.5.3](#) can safely be used where the duration of load is less than the corresponding test duration of between 36 s and 3 min. These short-term values can thus generally be applied, for example, in computing response to internal surge pressures and wheel loads from moving traffic.

For other load cases commonly considered as short term, but which might in fact endure for several hours or days, for example external groundwater pressure due to temporary saturation of trench backfill in storm conditions or vehicle surcharge from stationary traffic, the pipe stiffness applied should be appropriately downrated from the declared short-term value. In the absence of access to the pipe product test data from which the creep stiffness at the precisely relevant time point can be determined, the use of a declared two-year value is recommended as a conservative default for such cases.

For the computation of long-term pipe deflection (ovalization) due to earth and traffic loads, a two-year pipe ring stiffness can also be used safely. This is based on field tests amongst others by TEPPFA [2], as referred to in PD CEN/TS 15223, which have shown that settlement of the embedment around a buried plastics pipe, due to the combined action of backfill load, repeated traffic surcharge and groundwater movements, typically stabilizes within a period of not more than two years. At that point the deformed shape of the pipe ring is effectively fixed, allowing the stresses associated with its deflection to relax out. While this applies in principle to all flexible pipes, not just the thermoplastics tested, in practice the difference between long-term deflection calculated based on a fifty-year rather than two-year ring stiffness is usually very small.

For all other long-term load cases and limit states, including the effects of a sustained groundwater table on pipe strains, and buckling response to combined soil and groundwater loads, the fifty-year

elastic modulus or ring stiffness, appropriately determined from creep tests (see 4.5.3), should generally be used.

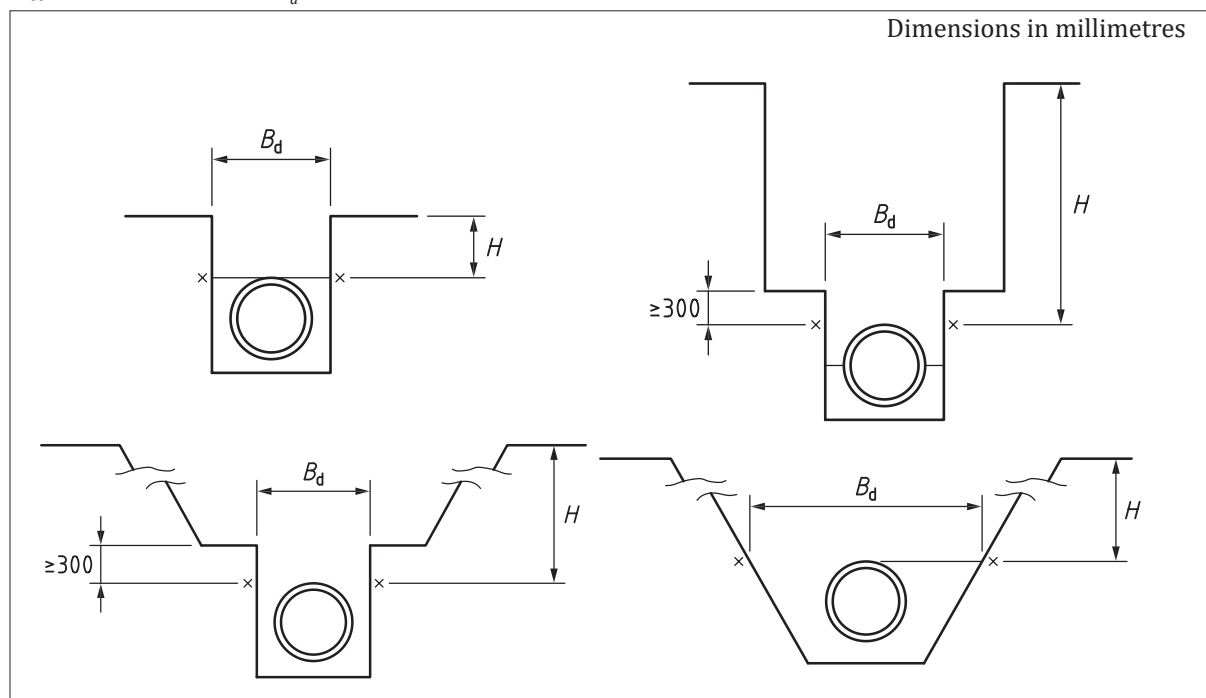
4.6 Classification of native/embedment soils and trench width

The zone of soil which has a structural influence on the buried pipe typically extends between one and two diameters from the pipe wall in all directions.

This zone invariably includes the trench base and, depending on trench width, both pipe embedment material and the native soil of the trench wall.

Trench shape and width are therefore key design parameters. Figure 4 defines the effective trench width, B_d , for different shapes of excavation.

Figure 4 — Effective trench width, B_d



The design of the trench cross-section, including specification of minimum trench width, should be such as to meet the requirements of BS EN 1610.

For rigid pipe design, trench width has an important influence on design loadings.

For flexible pipe design, the effective stiffness of the soil surround is expressed as a combination of the stiffnesses of embedment and native soil, respectively, as a function of trench width and pipe diameter.

The relevant classification and characteristics of both native and embedment soils are detailed in Clause 6, Clause 7 and Clause 8, as applicable.

4.7 Key parameters to be specified by the designer

The design specification should in all cases include as a minimum the following information and be communicated clearly to those constructing the works:

- specified minimum trench width;
- maximum trench width where this is a critical design parameter;
- pipe material class/specification;

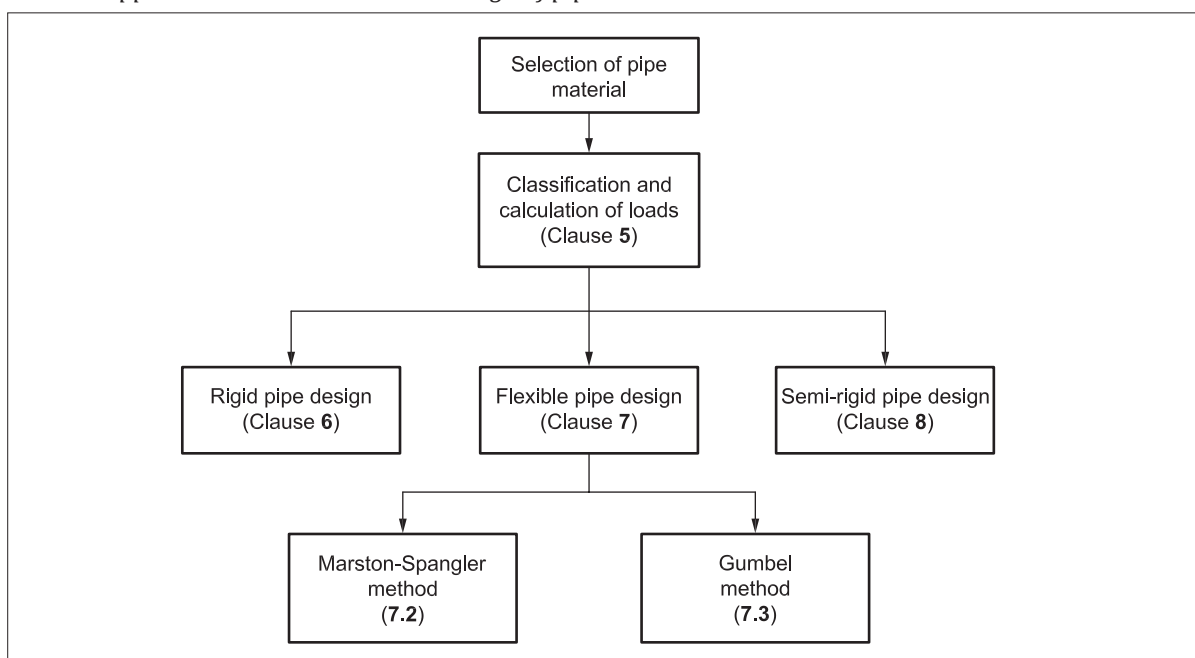
- d) embedment details, including lower bedding, upper bedding and sidefill materials as defined by [Figure 1](#), and levels of compaction required;
- e) assumed range of site conditions, including native soils and groundwater depth;
- f) measures to prevent flotation where applicable; and
- g) requirement for construction to be in accordance with BS EN 1610.

In addition, temporary works scenarios, including minimum cover and construction traffic loading, should be discussed with the installer if possible and incorporated in the design as appropriate. In all cases, the maximum construction traffic loading and associated depth of cover to the pipe assumed by the designer should form part of the construction specification.

4.8 Pipe and design method selection

Rigid, flexible or semi-rigid pipes can be inherently suited to certain applications, but pipe material selection is outside the scope of this British Standard. [Figure 5](#) sets out the clauses of this document that apply in each case.

Figure 5 — *Clauses applicable to the selection and design of pipe*



The choice between rigid pipe design, flexible pipe design or semi-rigid pipe design is determined by the anticipated pipe/soil behaviour (see [4.3](#)).

The choice between the two different flexible pipe design methods given in [7.2](#) and [7.3](#) in sizes below 900 mm is left to the designer, but for sizes above 900 mm and for structured-wall pipes generally, the Gumbel method (see [7.3](#)) is recommended.

5 Classification and calculation of loads

5.1 Introduction

The external loads on buried pipes fall into three principal categories:

- a) earth loads (geostatic);
- b) groundwater loads (hydrostatic); and

- c) surcharge (live) loads, which are further sub-categorized as follows:
- 1) vehicular loads on pipes under roads;
 - 2) agricultural vehicle loads on pipes under fields;
 - 3) construction vehicle loads on pipes on active construction sites; and
 - 4) rail traffic loads on pipes under railways.

The calculation of external geostatic and hydrostatic loads, as well as internal loads due to both dead weight of pipe contents and their pressurization, can be found in the applicable clause for the pipe type designation (see [Figure 5](#)).

5.2 Vehicle surcharge loads

Vehicle loading covers road traffic, field traffic and rail traffic loading. In previous editions of BS 9295 describing the structural design of buried pipes, there was a category for “light road” loading. Vehicle load classification has changed, and any public road is now assumed to be accessible to heavy lorries, with some designated exceptions where weight limits for old bridge structures apply. This edition excludes the “light road” loading category from the main guidance on structural design.

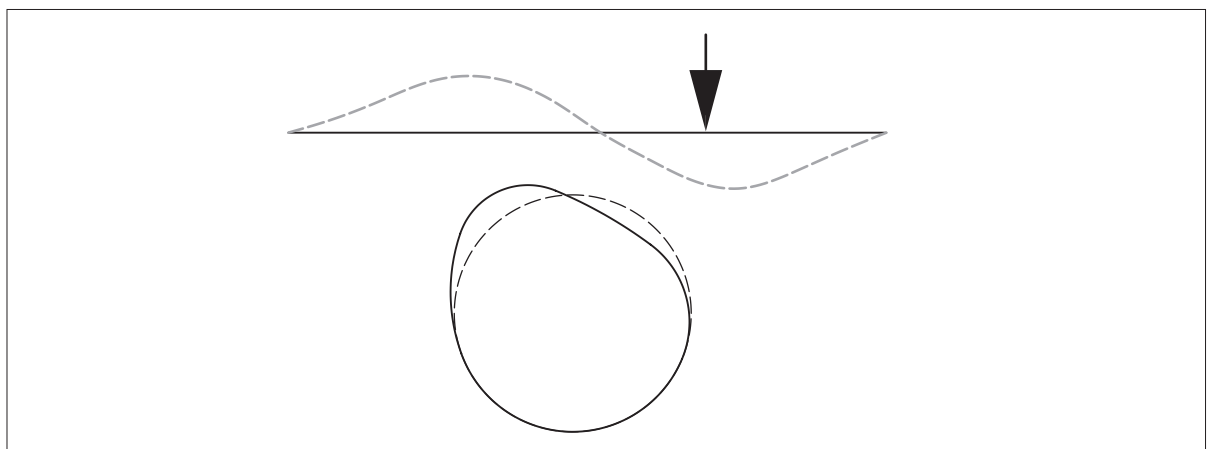
Graphs of surcharge pressure due to road vehicles, other vehicles and railways are given in [5.3](#), [5.4](#) and [5.5](#).

The maximum loading on a pipe depends upon its depth and diameter.

In some circumstances (e.g. shallow-buried, large-diameter, flexible pipes) horizontal loads arising from cornering, braking or eccentric loads (see [Figure 6](#)) should also be accounted for. Such loading can lead to asymmetrical deformation modes resisted primarily by the three-dimensional stiffness of the pipe and shear strength of the ground above the pipe, rather than by horizontal stiffness of the soil at pipe springings level. Shallow-buried flexible pipes subject to surcharge from construction vehicles prior to laying of a road pavement are particularly vulnerable to large asymmetrical deformations associated with shear failure in the relatively unconfined soil surrounding shoulders and crown.

NOTE Analysis of asymmetrical deformations of shallow-buried flexible pipes due to horizontal and/or eccentric vertical surcharge loads is outside the scope of this standard. Guidance on minimum depths of soil cover to reduce the risk that such loading could be more critical for design than vehicle loads applied symmetrically over the pipe crown is given in [5.4](#), [7.3.1](#) and [A.11.4](#).

Figure 6 — Illustration of eccentric surcharge load on pipe



5.3 Road and field surcharge loads

The main road vehicle loadings described in this subclause correspond to the worst case of either Load Model 1 (General Verification) or Load Model 2 as specified in BS EN 1991-2:2003. Field loading

is as specified in the TRRL guide [N1]. As an alternative to these sources, Newmark's integration of Boussinesq's formula may be used to give the resultant load at pipe crown at a certain depth from the wheel loading. Figure 7 and Figure 8 indicate how traffic load decreases with depth of cover, whereas backfill load increases.

Figure 7 — Loading configurations

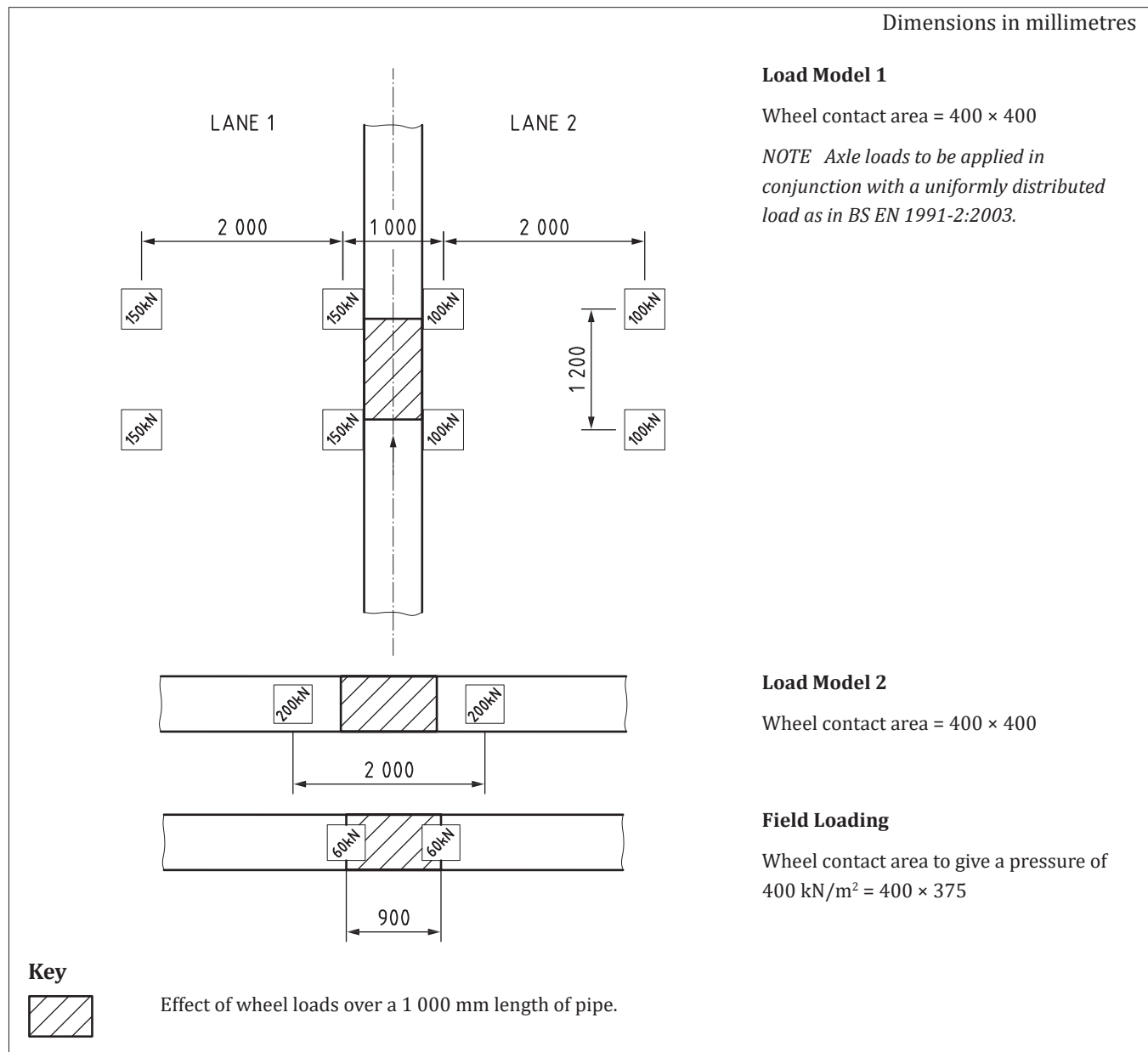
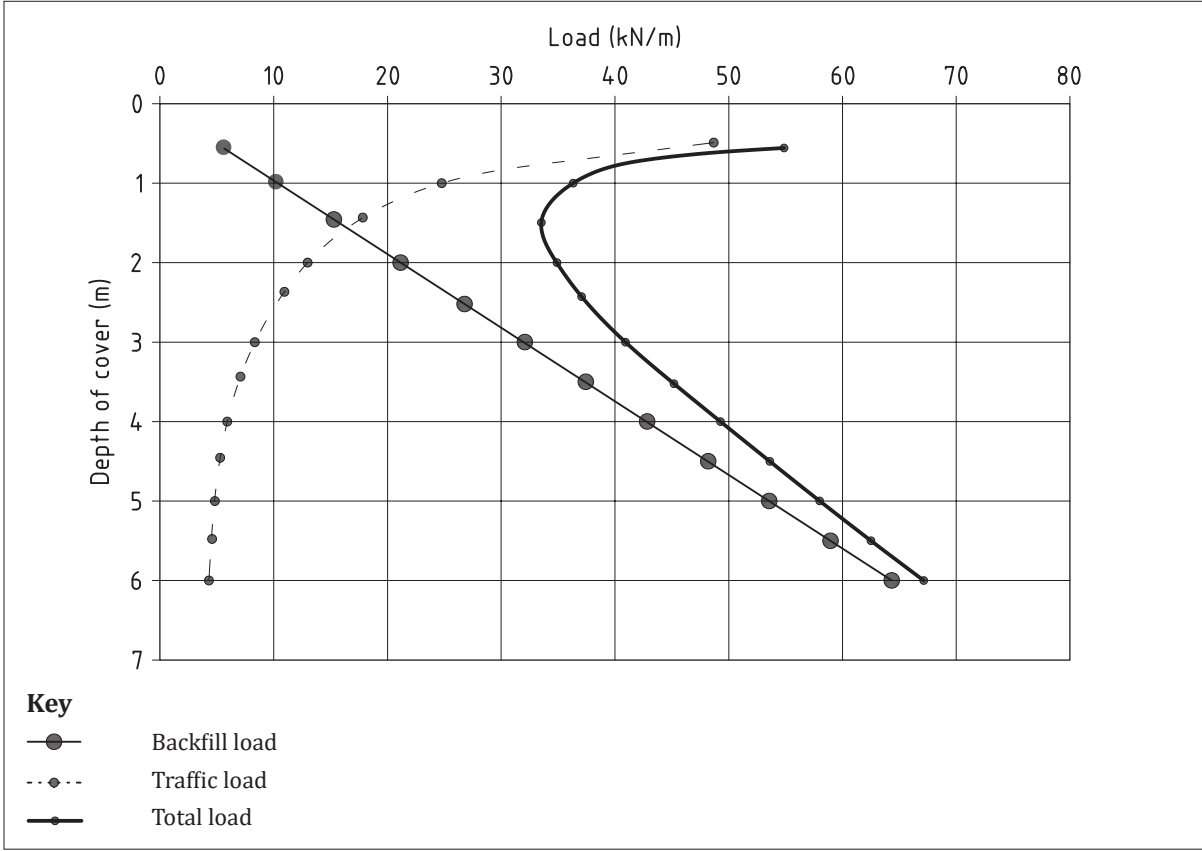


Figure 8 — Example of calculated backfill, road traffic surcharge and combined loadings expressed as a line load on a rigid DN 300 pipe



The values of vehicle surcharge loads tabulated in [Table 2](#) have been determined as the higher of the pressures calculated from Load Model 1 and Load Model 2 of BS EN 1991-2:2003 (see [A.25](#)). Attenuation of wheel loads with depth is in accordance with the dispersion angles given in BS EN 1991-2:2003.

The same data are presented in graphical form in [Figure 9](#).

Table 2 — Surcharge pressures (kN/m^2), P_s , as function of depth of cover to crown, H

H (m)	Main roads	Fields and gardens	H (m)	Main roads	Fields and gardens	H (m)	Main roads	Fields and gardens
0.5	–	85	1.7	45	16	3.8	22	3.8
0.6	144	70	1.8	43	15	4	21	3.4
0.7	120	58	1.9	41	13	4.5	19	2.7
0.8	101	49	2	39	12	5	17	2.2
0.9	86	42	2.2	35	10	5.5	16	1.9
1	74	36	2.4	33	9	6	15	1.6
1.1	67	32	2.6	31	7.7	6.5	14	1.3
1.2	62	28	2.8	29	6.7	7	13	1.2
1.3	58	25	3	27	5.9	8	12	0.9
1.4	54	22	3.2	26	5.2	9	11	0.7
1.5	51	20	3.4	24	4.7	10	10	0.6
1.6	48	18	3.6	23	4.2	–	–	–

The tabulated surcharge pressures can be determined approximately using the following:

$$\text{Roads: } P_s = 76.46 H^{-0.923} \quad (1)$$

$$\text{Fields: } P_s = 37.44 H^{-1.75} \quad (2)$$

Figure 9 — Surcharge pressure (kN/m^2), P_s , due to vehicle wheels



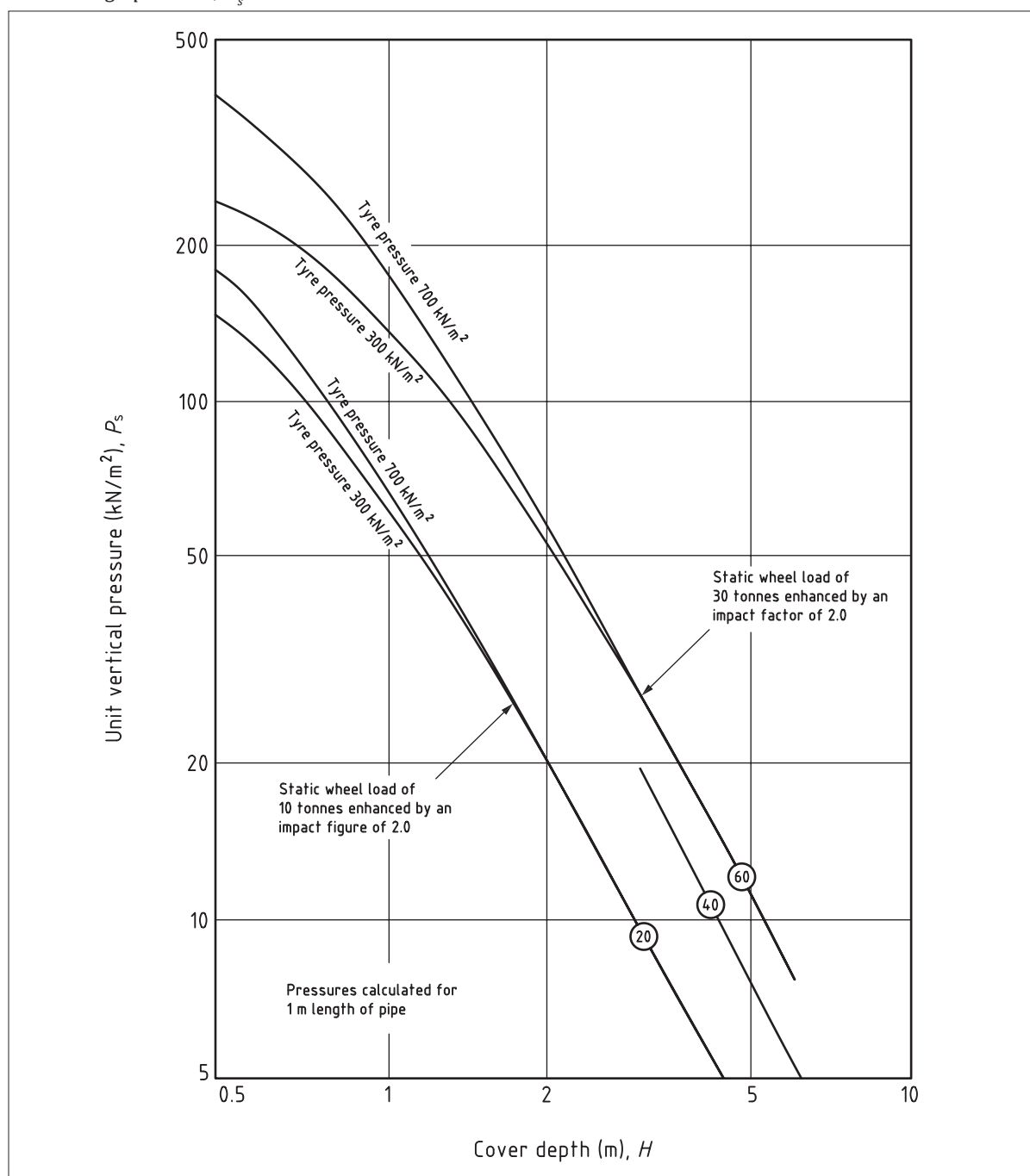
5.4 Surcharge pressure due to construction vehicles

As a guide, [Figure 10](#) presents design surcharge loads due to construction traffic, derived from the TRRL guide [N1], which take account of both wheel loads and tyre pressure of heavy construction plant in use at that time. Construction plant can be particularly damaging to newly constructed pipes, due to a combination of reduced cover to the pipes (lower than the designed final installation) and often poor soil conditions leading to rutting. This puts the construction vehicle wheels much closer to the pipe crown than is usually accounted for in design. These scenarios can be accounted for in the following ways:

- specific construction vehicle design, taking account of the heaviest construction load and the reduced cover; and
- engineer-designated plant crossing points, which either have appropriate levels of cover or are specially engineered to mitigate the loads, e.g. bridging slabs, concrete surrounds, road plates.

These surcharge pressures might not be applicable to other items of construction plant, in which case the TRRL Guide [N1] should be consulted.

The minimum recommended depth of unpaved soil cover over flexible pipes subject to construction vehicle surcharge is the greater of 0.9 m or half the outside diameter of the pipe.

Figure 10 — Surcharge pressure, P_s , due to construction vehicles

5.5 Railway loadings

The main line rail vehicle loading described in this subclause corresponds to BS EN 1991-2:2003, Load Model 71 (load classification factor, $\alpha_{\text{EC1}} = 1.0$), shown in [Figure 11a](#)) and is identical to BS EN 1991-2:2003 and its UK National Annex RU load configuration.

NOTE The α factor is used for load classification of railway traffic loading on bridges and structures beneath the track. The selection of the α factor value is a national choice to be decided by the authority responsible for the infrastructure below the track.

The light rail vehicle loading described in this subclause corresponds to (the now withdrawn) BS 5400-2 RL (Rail London) load configuration shown in [Figure 11b](#)). There is no corresponding Eurocode “light rail load” model in BS EN 1991-2:2003, but a future edition could include this.

An example of combined earth and surcharge pressures, P_s , on a pipe is presented in graphical form in [Figure 8](#). It specifically relates to a road traffic surcharge scenario, but more generally describes the relationship between earth and surcharge loading regardless of the type of traffic.

The dispersal of pressure with depth for the two railway loading models is shown in [Figure 12](#) and [Figure 13](#) for a single-track railway and a pipe crossing it at right angles. The multiplication factor to be used when the crossing is under more than one railway track is shown in [Figure 14](#).

Figure 11 — Loading configurations for LM 71 and RL railway loading models

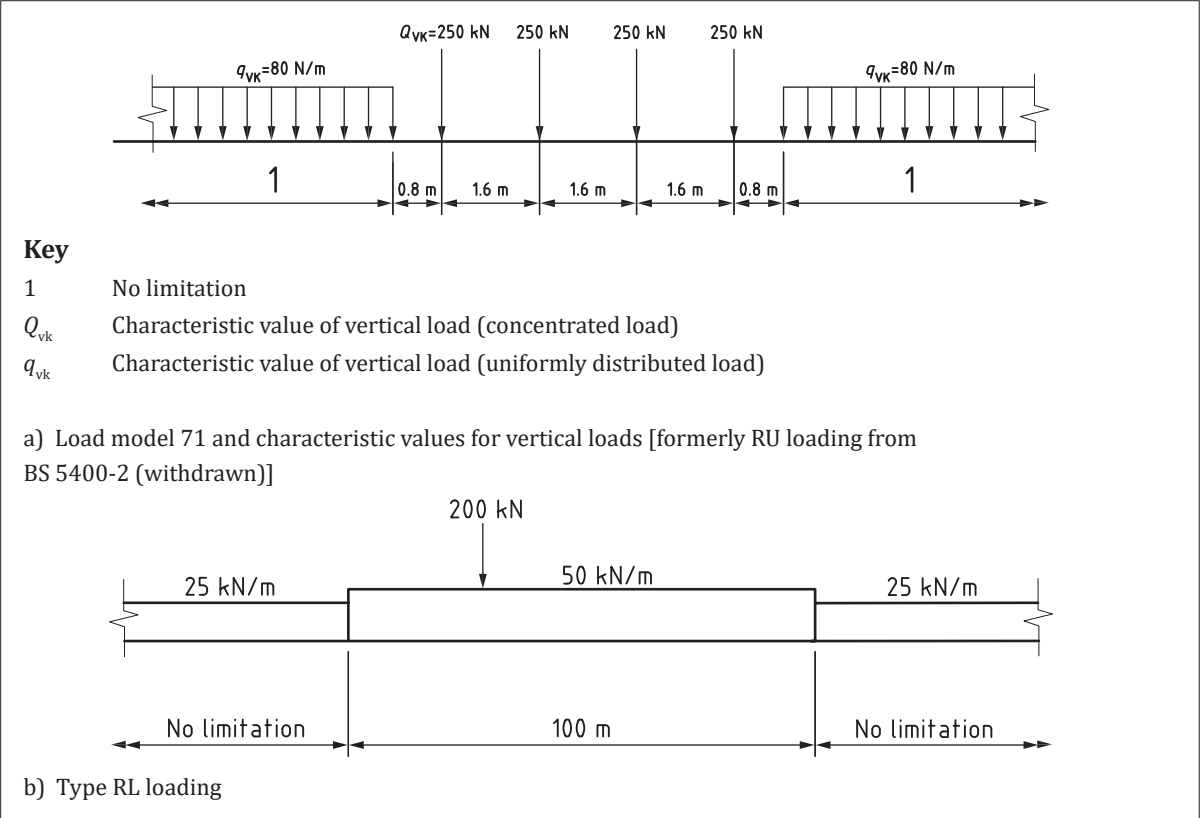
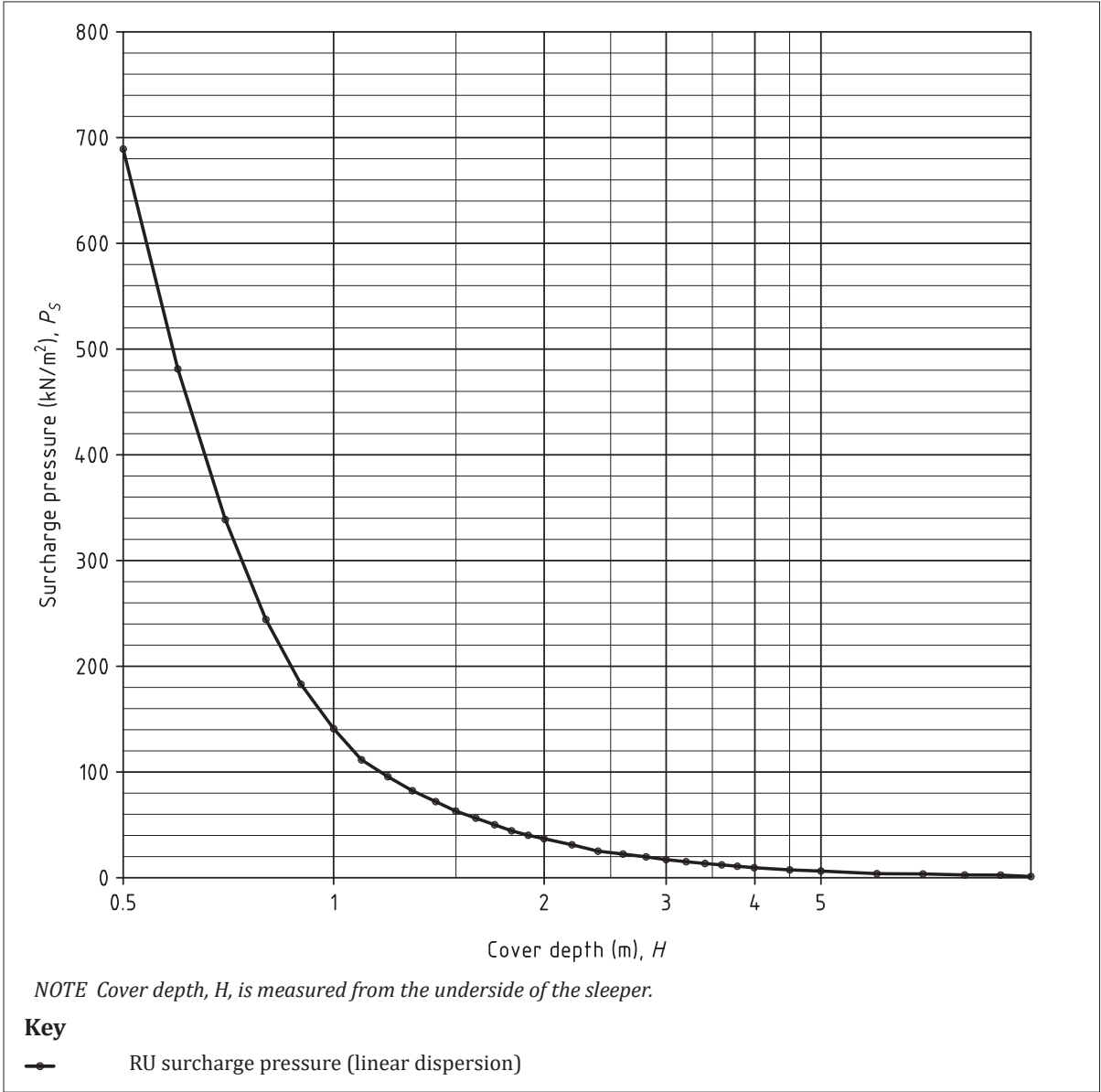


Figure 12 — BS EN 1991-2:2003, load model 71 ($\alpha = 1.0$) (RU to BS 5400-2²⁾ surcharge pressure at depth



²⁾ Withdrawn.

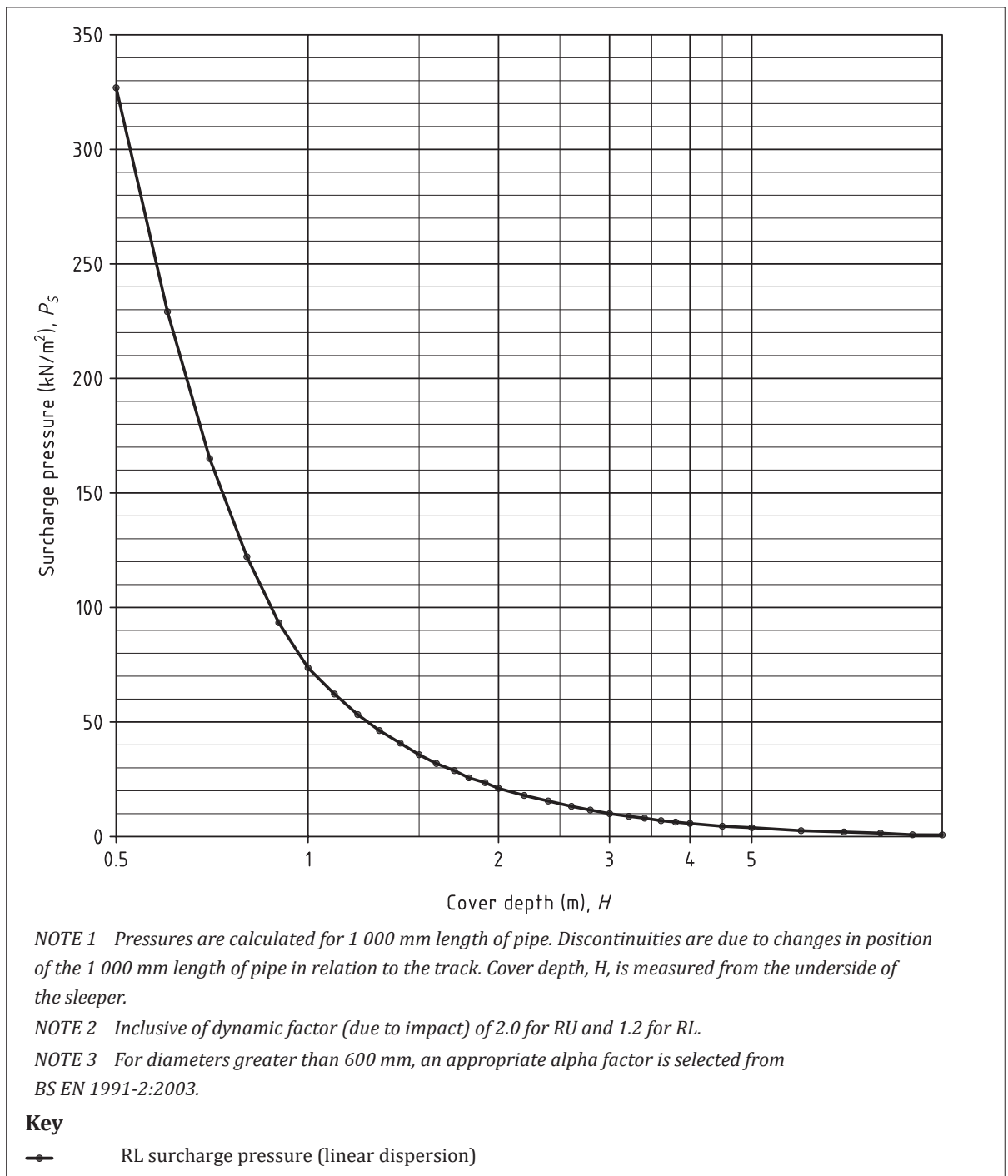
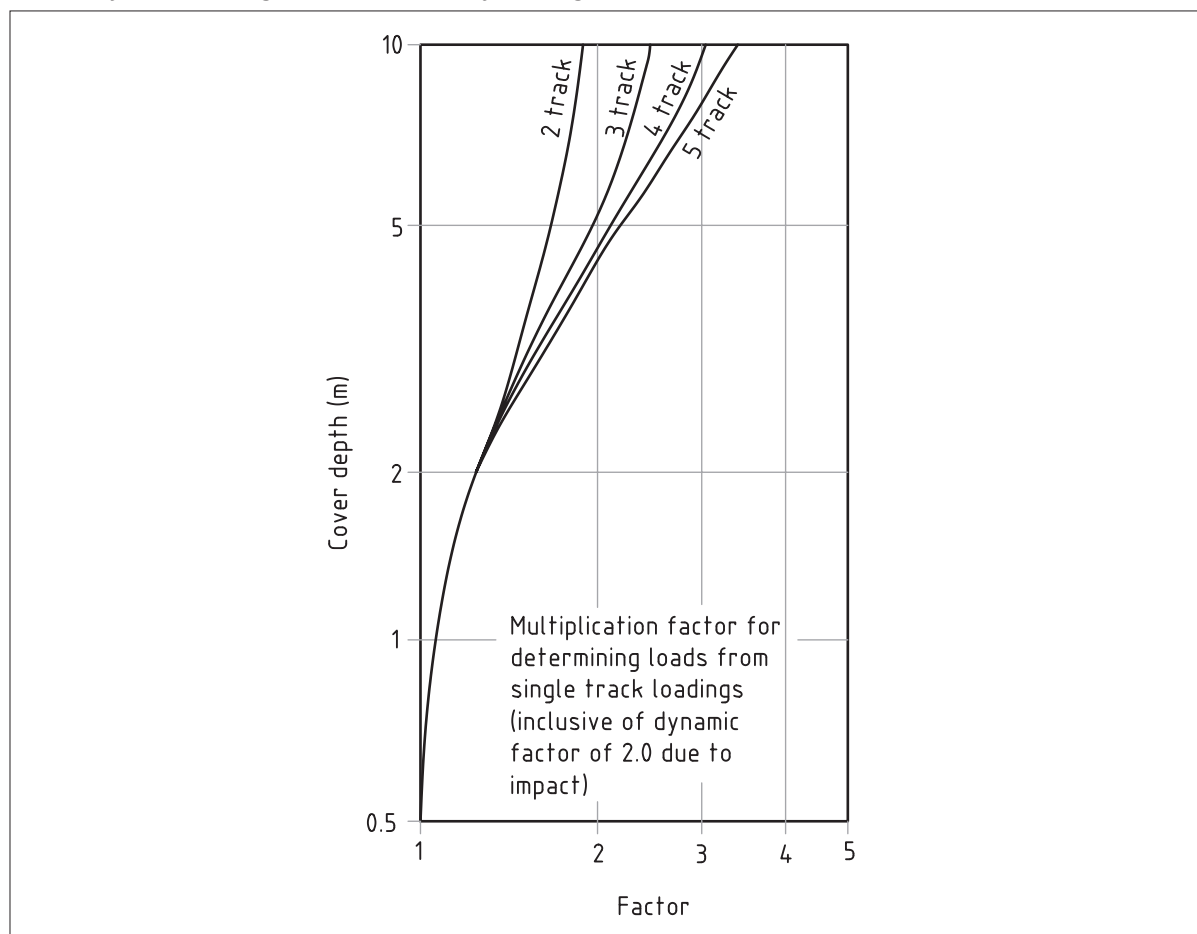
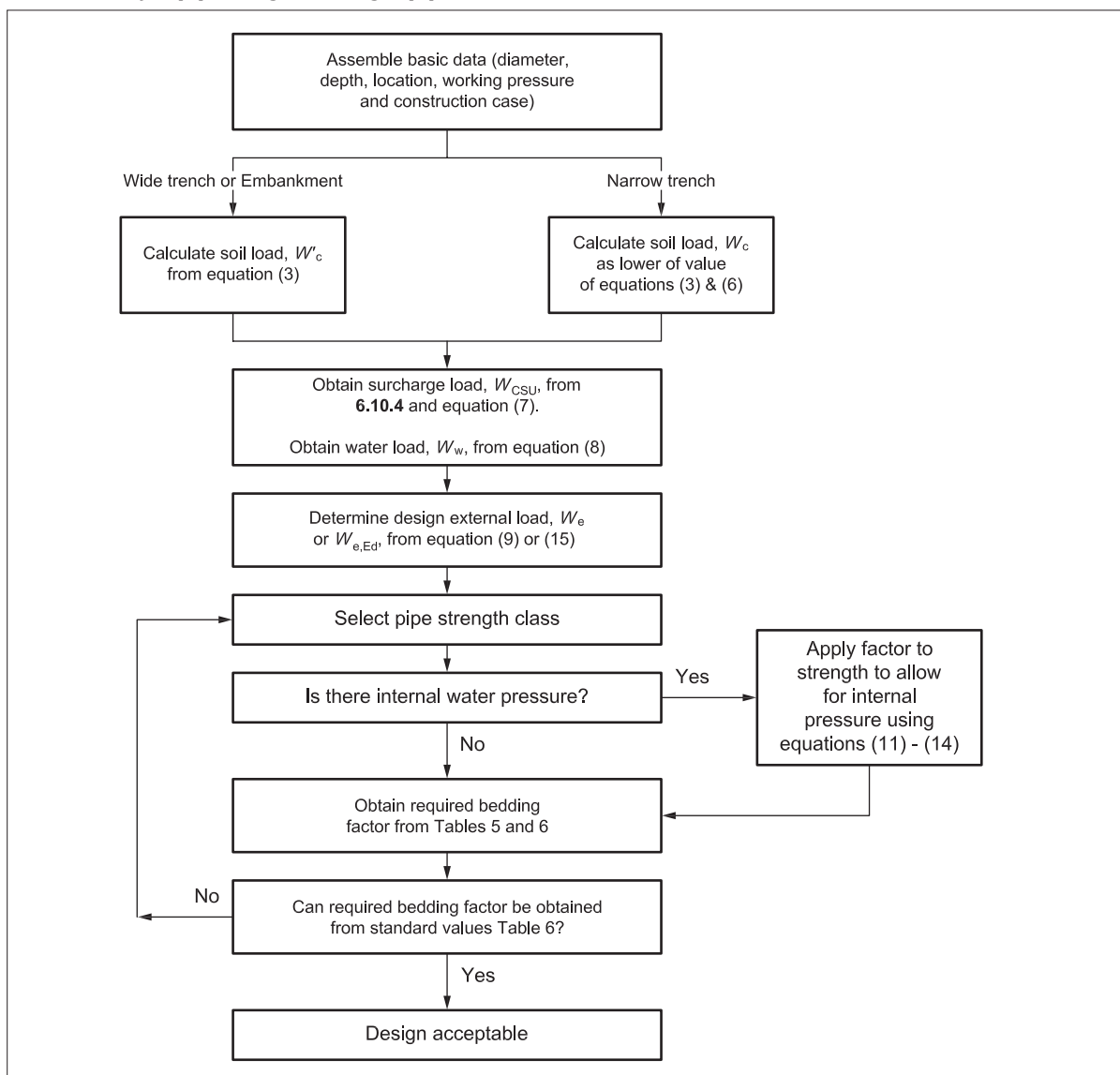
Figure 13 — Rail London (RL to BS 5400-2) surcharge pressures at depth

Figure 14 — Factor for calculating multi-track railway loadings

6 Rigid pipe design

COMMENTARY ON CLAUSE 6

This clause gives the design formulae, identified by equation numbers which are referred to in the flowchart in [Figure 15](#), which sets out the design procedure for rigid pipes.

Figure 15 — Flowchart for pipe design with rigid pipes

6.1 General

Rigid pipes rely on their embedment to distribute the loads imposed by backfill and traffic to reduce the circumferential bending moments in the pipe walls. Their response to vertical loading applied at the crown is to settle marginally into their supporting material until sufficient support reaction, with an associated bedding angle, α (see [Figure 3](#)), is achieved. The larger the bedding angle, the lower the circumferential bending moment induced in the pipe wall.

The basic governing formula for the design of buried rigid pipe is:

$$\text{vertical load capacity} = \text{crushing strength} \times \text{bedding factor}$$

where the bedding factor takes account of bedding angle and soil load distribution generally (see [6.7](#)).

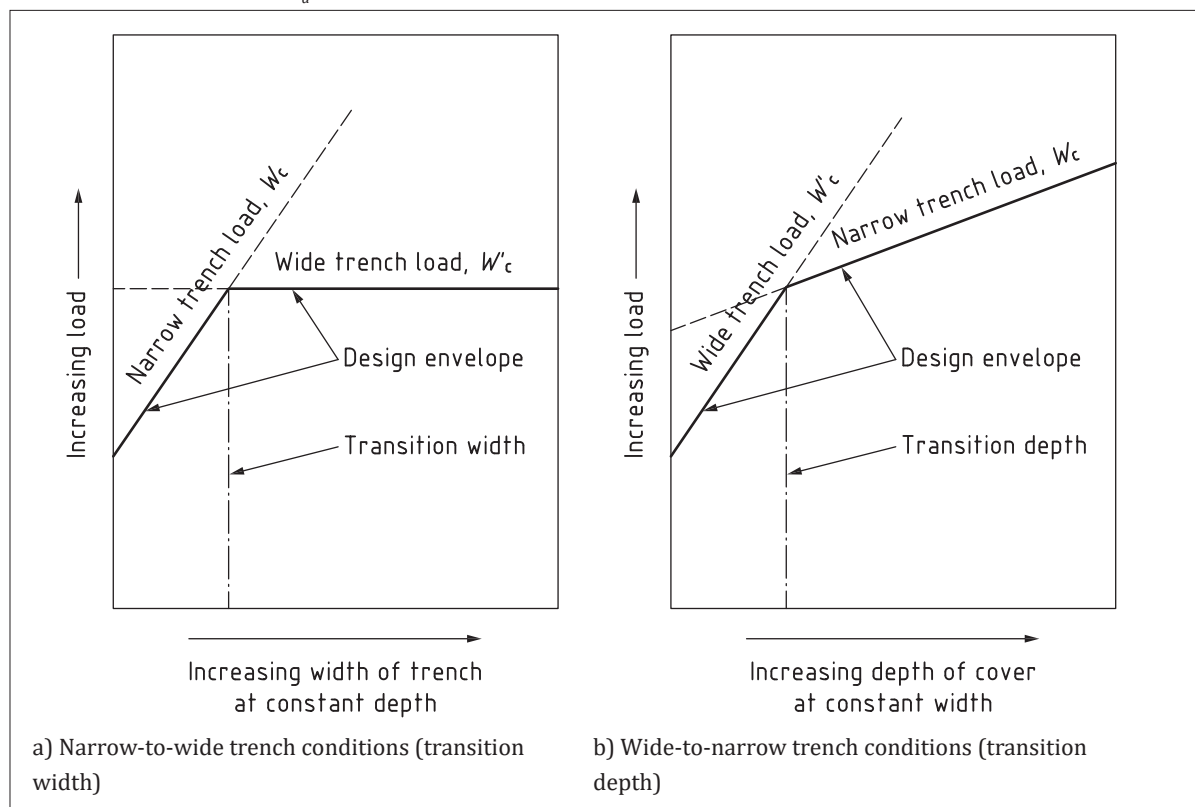
6.2 Wide and narrow trenches

There are two installation conditions: pipes installed in wide trenches or embankments and pipes installed in narrow trenches. For all pipe installations the loads arising from both conditions should be calculated for the selected maximum trench width and the lower of the two loads, W_c or W'_c , used in the design specifying the installation condition. In the absence of specific information, it is safer

to assume the trench is wide. [Figure 16](#) illustrates the effect of trench width on the relative size of these two loads.

A separate calculation of vertical backfill load on the pipe is generally conducted for each condition and the results compared. Depending on the assumed trench geometry, either calculated load might prove to be larger.

Figure 16 — Effect of trench width, B_d , on calculated vertical backfill load on pipe



[Figure 16](#) also illustrates what is known as the “transition width” and the “transition depth”. For a narrow trench at a particular depth the load, W_c , increases as the trench width increases, up to a maximum value given by the wide trench formula.

When the loads calculated by the two methods are equal, the “transition width” is reached. Although the load calculated by the narrow trench formula increases beyond that width, the pipe will not in reality experience a load greater than the wide trench load.

Similarly, in a trench of a particular width but varying depth, it is possible for wide trench conditions to apply in the shallower sections of the trench and for narrow trench conditions to apply in the deeper sections. The depth at which the loads calculated by both formulae are equal is known as the “transition depth”. At depths beyond the transition depth, the narrow load applies.

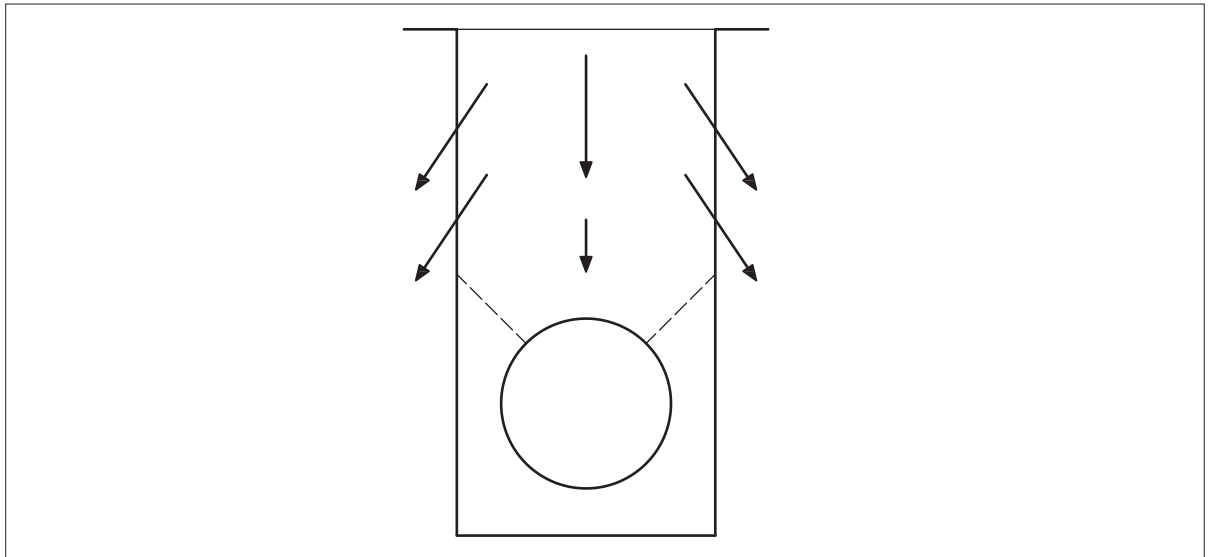
For the wide-to-narrow trench condition [see [Figure 16b](#)] the soil load coefficient, C_d , is used and takes account of what is known as the “silo effect” (see [Figure 17](#)). This is the phenomenon, first observed in grain silos during the 19th century, whereby settlement of backfill transfers load by friction to the walls of a vertically-sided trench. If the trench is sufficiently narrow, this can substantially reduce the vertical load on a rigid pipe compared with that arising in wide trench conditions, at least in the short term [see [Figure 3b](#)].

The theory from which values of C_d are derived assumes that 100% of the friction-reduced backfill load is supported by the pipe: as the trench width increases, the total of this backfill load increases. However, in practice a proportion is supported by the columns of sidefill adjacent to the pipe. This has the effect that the narrow trench formula, which predicts relatively low design loads on the pipe

if the trench walls are sufficiently close, becomes over-conservative as trench width increases. For this reason, it is conventional to adopt the lower of the calculated narrow and wide trench loads for design (see [Figure 16](#)).

If, however, the walls of the trench are not close enough to have a beneficial effect, then the pipe is designed as if in a wide trench. Designers should be mindful of how an installation is likely to be carried out on site and whether this can ensure construction of a trench whose width does not exceed the maximum specified at any point, so that the whole trench depth is available to relieve load.

Figure 17 — *The silo effect*



6.3 Complete and incomplete projection

Within the wide trench design, the concept of complete and incomplete projection is addressed. These refer to a line known as the “plane of equal settlement” and to whether this line falls above ground level (complete projection: see [Figure 18](#)) or below (incomplete projection: see [Figure 19](#)). The plane of equal settlement refers to the point at which the settlements of soil to the sides of the pipe and over the pipe are equal.

Loads should be calculated using both complete and incomplete projection formulae [equations (4) and (5) in [6.10.2](#)] and the lesser of the two calculated loads used for design.

NOTE Generally this calculation shows that the incomplete projection case is the more common situation, except for pipes installed at a shallow depth of cover.

Figure 18 — Complete projection

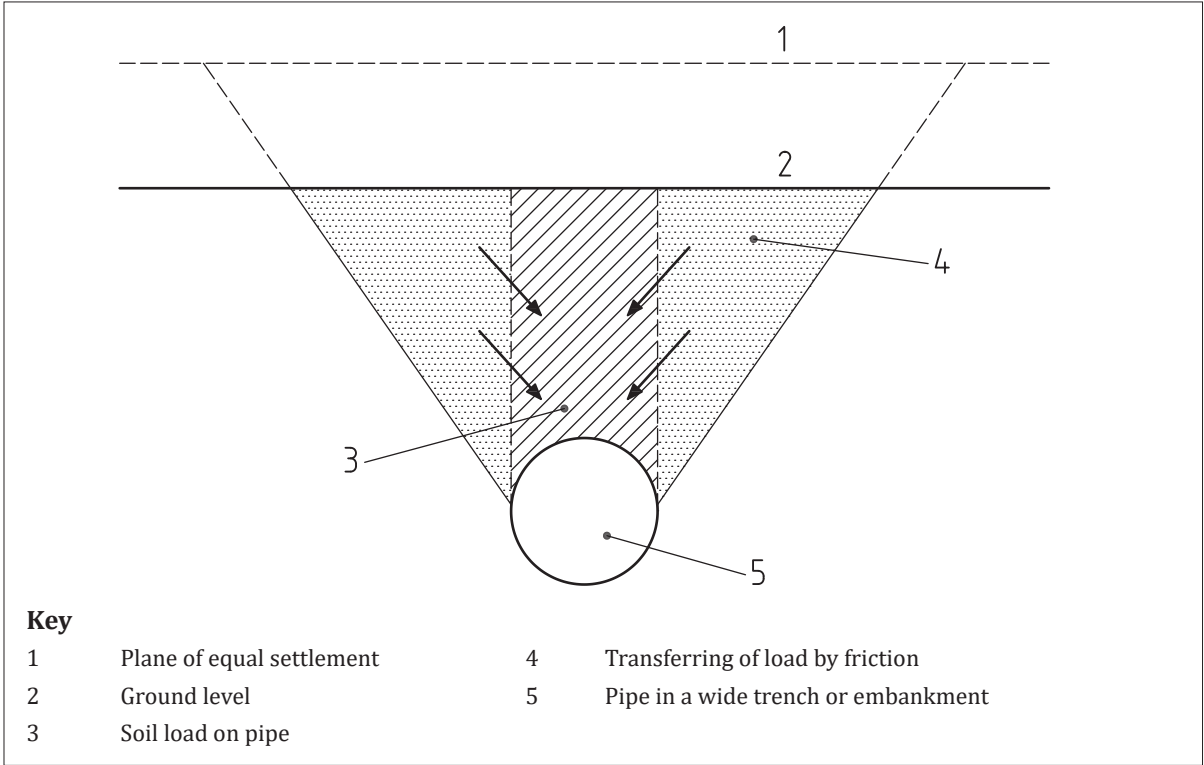
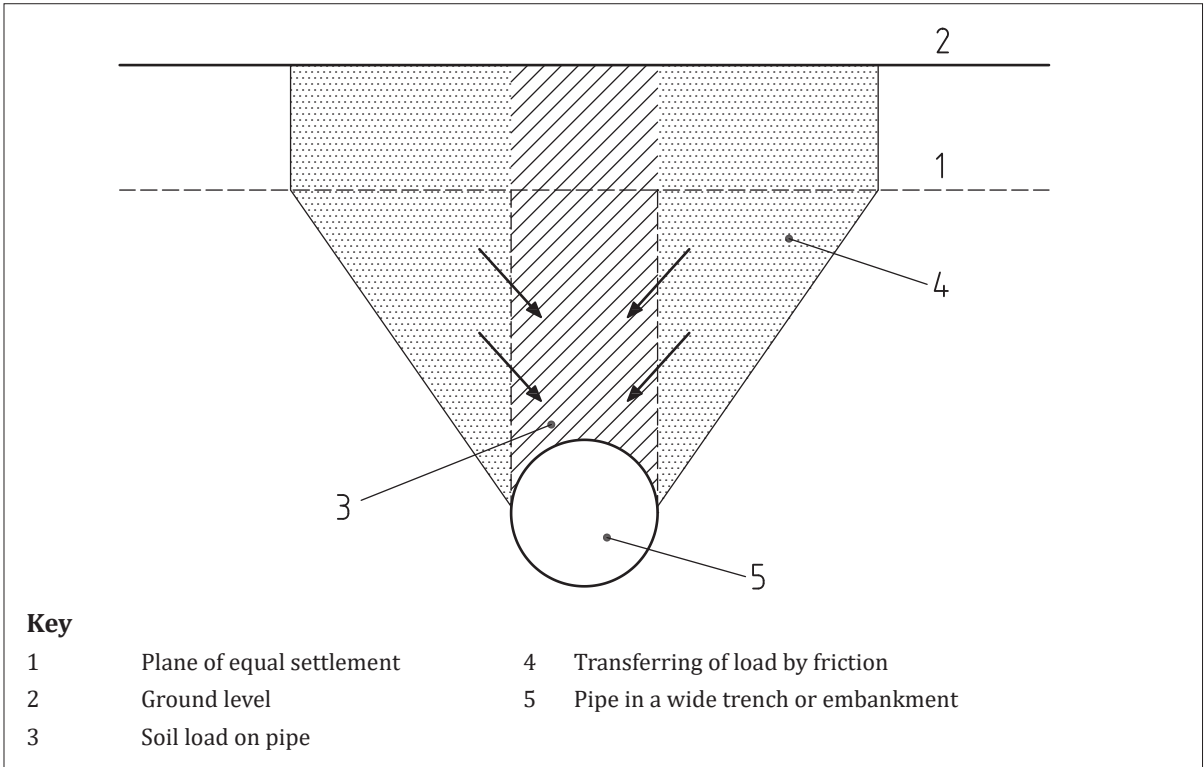


Figure 19 — Incomplete projection



6.4 Settlement deflection ratio, r_{sd}

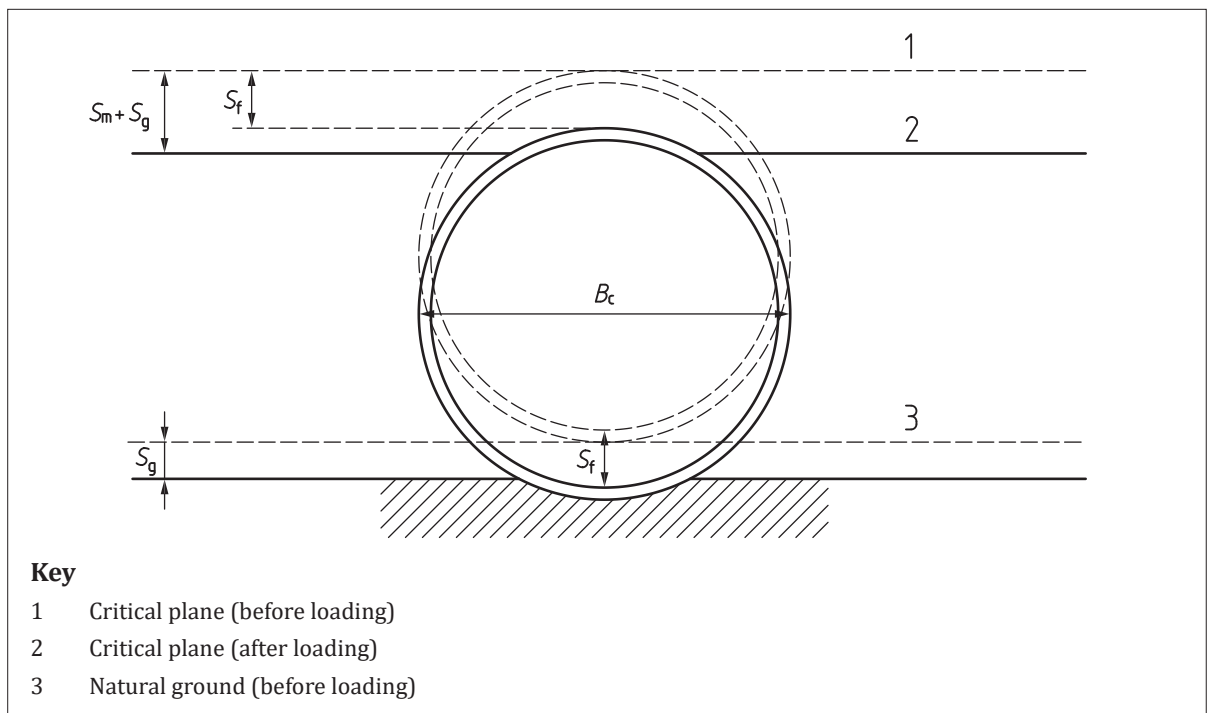
The settlement deflection ratio for a positively projecting pipe, r_{sd} , i.e. in a wide trench or embankment, is given by:

$$r_{sd} = \frac{(S_m + S_g) - S_f}{S_m}$$

where S_m , S_g and S_f are as shown in [Figure 20](#).

NOTE Values of settlement deflection ratio have been determined experimentally and are taken from the TRRL's "Guide to design loadings for buried rigid pipes" [N1] and reproduced in [Table 3](#).

Figure 20 — Settlement deflection ratio



The settlement deflection ratio is evaluated by examining the situation of a pipe being laid on the surface of the ground, with the critical plane being at the pipe crown. Once loading occurs, the natural ground level goes down an amount, S_g , the pipe goes down by the settlement of the pipe invert, S_f , and the critical plane goes down by the settlement of the fill, S_m , plus the settlement of the natural ground, S_g .

The recommended values of the settlement deflection ratio, r_{sd} , and ranges for the native soil modulus, E'_3 , are given in [Table 3](#).

Table 3 — Guide values of Spangler modulus for native soils, E'_3

Soil type	Spangler modulus for soils in various conditions (MN/m ²)				
	Very dense	Dense	Medium dense	Loose	Very loose
Gravel	Over 40	15 to 40	9 to 15	5 to 9	3 to 5
Sand	15 to 20	9 to 15	4 to 9	2 to 4	1 to 2
Clayey, silty sand	10 to 15	6 to 10	2.5 to 6	1.5 to 2.5	0.5 to 1.5
Clay	Very hard	11 to 14			
	Hard	10 to 11			
	Very stiff	6 to 10			
	Stiff	4 to 6			
	Firm	3 to 4			
	Soft	1.5 to 3			
	Very soft	0 to 1.5			

NOTE Further guidance on the use of lightweight aggregates is given in [A.11.4](#).

6.5 Projection ratio, ρ

The projection ratio, ρ , is calculated as the proportion of the pipe external diameter that is above firm bedding level or the natural ground level. For classes D, N and F beddings (see [6.7](#), [Table 5](#)) the value of ρ is unity, and for other granular and concrete beddings ρ is 0.7.

In calculation, the settlement deflection and projection ratios are used in combination, and design values of their product, $r_{sd}\rho$, are often taken as 0.7 for bedding classes D, N and F (see [6.7](#), [Table 5](#)) and 0.5 for bedding classes B and S.

6.6 Soil properties

In the absence of data on the soil type supporting the use of other values it is normal practice to assume the following:

- $K\mu = 0.19$;
- $K\mu' = 0.13$; and
- $\gamma = 19.6 \text{ kN/m}^3$.

The combinations $K\mu$ and $K\mu'$ are generally used as single design parameters, rather than determining values of K , μ and μ' separately.

Where soils are more precisely identified, the values given in [Table 4](#) may be used.

Table 4 — Values of $K\mu$ and $K\mu'$ for specific soil types

Type of soil	$K\mu$ or $K\mu'$
Granular soils without cohesion	0.190
Maximum for sand and gravel	0.165
Saturated top soil	0.150
Maximum for ordinary clay	0.130
Maximum for saturated clay	0.110

The value of $K\mu'$ is taken as the lower of the values for the backfill material and the native soil in the trench sides.

6.7 Bedding factors

The loadbearing capacity of a rigid pipe when buried is the crushing strength of the pipe [determined in accordance with European Standards, e.g. BS EN 295 (all parts) for clay; BS EN 1916 for concrete]

multiplied by the bedding factor, α . The crushing strength is determined by line load testing which produces the most onerous pipe wall circumferential bending moment

The safe crushing strengths specified in European Standards are statistically determined from these tests.

[Table 5](#) shows that the bedding factor increases with the bedding angle. Higher bedding angles result in greater load distribution. Furthermore, horizontal soil loads can have a bending moment-relieving effect that is also reflected in the values of bedding factors.

The purpose of the embedment is to distribute the vertical loads (and the corresponding support reaction) around the pipe. By effectively distributing these loads the pipe can withstand higher loadings than the crushing strength of the pipe alone. For example, a 360° embedment (bedding class S – see [Table 5](#)) has historically been given a bedding factor of 2.2, whereas a non-existent or minimal bedding (bedding class D or N) hardly distributes the loads at all, and results in a bedding factor of only 1.1.

A series of 240 physical tests on clay pipes over a 13-year period at British Ceramic Research Ltd. and the Water Research Centre demonstrated that higher bedding factors were achieved in practice, even using lower than normal levels of workmanship [[3](#)]. This was attributable to the bending moment-relieving effect of lateral soil loading in the wide trench conditions of the tests. The tests also demonstrated that it was the amount of support to the bottom half of the pipe that was the critical determinant of the bedding factor. Hence, the same bedding factor of 2.5 is adopted for wide trench conditions to both class B and class S beddings. The only exception is where trench widths are less than the minima specified in BS EN 1610, in which case the bedding factor for classes B and S revert to 1.5, as for class F, because filling of the upper bedding (under the haunches of the pipe – see [Figure 1](#)) cannot be assured in practice. These different values are reflected in [Table 5](#).

Bedding factors are extensively discussed in *Guide to design loadings for buried rigid pipes* [N1], *Bedding construction and flow capacity of vitrified clay pipelines* [[4](#)] and *Revised bedding factors for vitrified clay pipes* [[3](#)].

6.8 Crushing strength adjustments for pressure pipes

Some rigid pipe materials, such as specially designed concrete, can be used for pressure applications. In this case it is necessary to make an adjustment to the crushing strength of the pipe to account for the fact that, in addition to the backfill and traffic loading, some of the pipe's strength will also be required to contain the internal pressure within the pipe.

6.9 Minimum bedding factor and factor of safety

The process of structural design involves effectively matching the imposed loads to the loadbearing capacity of the pipe and its embedment to achieve an overall and appropriate factor of safety. As rigid pipes are available in few crushing strength classes (one for concrete, up to three for clay), design typically involves establishing that the required minimum bedding factor, when multiplied by the selected pipe strength, at least equals the total imposed design load (see [6.11](#)).

The minimum factors of safety are given in [6.12](#), [Table 8](#).

Recommended design values of the bedding factor required can be obtained from [6.15.6](#), [Table 11](#), and [6.16](#), [Table 12](#). The applicable design values of crushing and proof strengths, W_t and W_p , should be obtained from the applicable product standards.

Where a limit state design is used, the partial safety factors given in [6.13.4](#), [Table 9](#), should be used. The use of a minimum factor of safety is no longer required.

Table 5 — Rigid pipe embedment

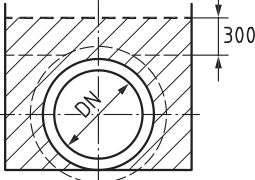
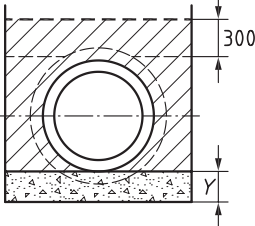
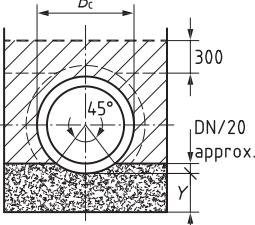
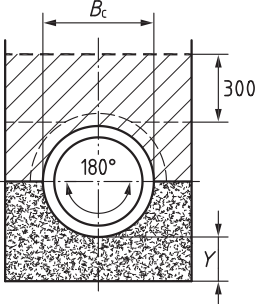
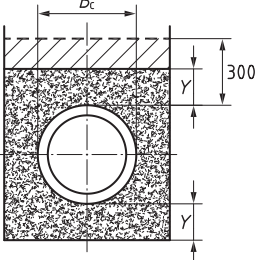
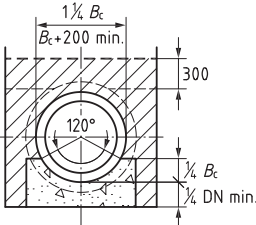
Bedding detail $Y = 1/6 B_c$	Bedding class	Dimensions in millimetres	
		Bedding factors, F_m	
		Narrow trench	Wide trench/embankment ^{A)}
	D	1.1	1.1 to 1.3 (see [N1])
	N	1.1	1.1 to 1.3 (see [N1])
	F	1.5	1.9 (see [4])
	B	1.9 ^{B)}	2.5 (see [4])
	S	2.2 ^{B)}	2.5 (see [4])

Table 5 (continued)

Dimensions in millimetres							
Bedding detail		Bedding class		Bedding factors, F_m			
$Y = 1/6 B_c$				Narrow trench		Wide trench/embankment ^{A)}	
		A Unreinforced		2.6		Wide trench: 2.6 (see [N1]) Embankment: 2.6 to 3.7 (see [N1])	
		Reinforced		3.4		Wide trench: 3.4 (see [N1]) Embankment: 3.4 to 5.7 (see [N1])	
	In situ concrete		Granular bedding material		Selected backfill material		All-in granular material

NOTE 1 The TRRL guide [N1] provides:

- guidance on use of embankment bedding factors; and
- details of class A bedding reinforcement.

NOTE 2 The source of this material is reference [4] and further detail can be found there if required.

^{A)} Applicable to pipes less than 900 mm outside diameter. For outside diameters ≥ 900 mm, see 6.10.1, Table 6.

^{B)} Subject to conformity with minimum trench widths specified by BS EN 1610. For narrower trench widths the bedding factors for classes B and S should be taken as 1.5 for design.

6.10 Design loads for rigid pipes

6.10.1 General

Where designers have sufficient control over the installation stage to ensure conformity with the temporary works specification (trench support, compaction and overall trench width), the load-reducing effect of narrow trenches can be incorporated within the design. In all other cases (typically where the temporary works are likely to be determined by the installation contractor or subject to uncertain site conditions, e.g. proximity of other trenches), the design should be based on the more conservative wide trench model. The bedding factors for the wide trench condition given in Table 5 already take account of relieving lateral pressure, but only for outside diameters up to 900 mm as verified by testing in the UK [4]. For larger diameters, design bedding factors are given in Table 6.

Table 6 — Design bedding factors for pipes laid in wide trench and embankment conditions with bedding classes B and S and outside diameter greater than or equal to 900 mm^{A)}

H/B_c	0.5	1.0	1.5	2.0	3.0	5.0	10	15
$F_m^{B)}$	2.8	2.58	2.44	2.39	2.36	2.35	2.33	2.33

^{A)} Assumes $\rho = 0.7$; $r_{sd} \rho = 0.5$; refer to 6.5.

^{B)} From *Concrete Pipe Design Manual* published by American Concrete Pipe Association [5].

6.10.2 Soil load, W'_c , in wide trench and embankment conditions

Soil load, W'_c , in the wide trench and embankment conditions is given by:

$$W'_c = C_c \gamma B_c^2 \quad (3)$$

The value of C_c is taken as the lower of the values derived from equations (4) and (5). If the lower value is given by equation (4), complete projection conditions exist. If the lower value is given by equation (5), incomplete projection conditions apply.

$$C_c = \frac{e^{2K\mu H/B_c} - 1}{2K\mu} \quad (4)$$

$$C_c = \left[\frac{e^{2K\mu H_e/B_c} - 1}{2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c} \right) e^{2K\mu H_e/B_c} \right] \quad (5)$$

where H_e is the height of the plane of equal settlement above the top of the pipe. Derivation of H_e is complex, but [Table 7](#) provides simplified equations for obtaining C_c .

The full formulae from which the equations in [Table 7](#) are derived are given in the TRRL's *Guide to design loadings for buried rigid pipes* [N1], Clauses 2.4 and 2.5, assuming $K\mu = 0.19$ (see [6.6](#)).

Table 7 — Load coefficients for incomplete projection condition

$r_{sd}\rho$	Equation for C_c
0	$C_c = H/B_c$
+0.1	$C_c = 1.23 (H/B_c) - 0.02$
+0.3	$C_c = 1.39 (H/B_c) - 0.05$
+0.5	$C_c = 1.50 (H/B_c) - 0.07$
+0.7	$C_c = 1.59 (H/B_c) - 0.09$
+1.0	$C_c = 1.69 (H/B_c) - 0.12$
+2.0	$C_c = 1.93 (H/B_c) - 0.17$
+3.0	$C_c = 2.08 (H/B_c) - 0.20$
+4.0	$C_c = 2.19 (H/B_c) - 0.21$
+5.0	$C_c = 2.28 (H/B_c) - 0.22$

6.10.3 Soil load, W_c , in the narrow trench condition

For the narrow trench condition soil load, W_c is given by:

$$W_c = C_d \gamma B_d^2 \quad (6)$$

where:

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

For pipes in trenches, the backfill load is given by equation (3) and equation (6) and the lower value is used for the pipe design.

6.10.4 Concentrated surcharge load, W_{csu}

The concentrated surcharge load, W_{csu} , is given by:

$$W_{\text{csu}} = P_s B_c \quad (7)$$

where the value of the surcharge pressure, P_s , is obtained from [Clause 5](#).

6.10.5 Internal water load, W_w

Equivalent load due to weight of water in pipe, W_w , is given by equation (8):

$$W_w = C_w \gamma_w \pi (D - t)^2 / 4 \quad (8)$$

where the value of C_w is normally taken as 0.75.

6.10.6 Total design external load, W_e

Total design external load, W_e , is given by equation (9):

$$W_e = W_c + W_{\text{csu}} + W_w \quad (9)$$

NOTE Substitute W'_c for W_c for the wide trench or embankment conditions.

6.11 Supporting strength of rigid pipes

The supporting strength of rigid pipes is the product of the pipe strength and the bedding factor and should be at least equal to the total design external load, W_e , given by equation (9).

The minimum recommended bedding factor, F_m , is given by equation (10):

$$F_m \geq W_e F_{\text{se}} / W_t \text{ (or } W'_t) \quad (10)$$

Recommended design values of the bedding factor can be obtained from [Table 5](#), with the design values of crushing strength, W_p , obtained from product standards.

For pressure pipes a reduction factor should be applied to the crushing strength, W'_p , to take account of the effect of the internal water pressure.

a) for reinforced concrete pressure pipes:

$$W'_t = W_t (1 - P_{\text{pop}} / P_{\text{ult}}) \quad (11)$$

in which case, the working pressure in kilonewtons per square metre (kN/m^2), P_w , should not exceed:

$$P_{\text{ult}} (1 - W_c / W_t F_m) / F_{\text{si}} \quad (12)$$

b) for asbestos cement pressure pipes:

$$W'_t = W_t (1 - P_{\text{pop}} / P_{\text{ult}})^{0.5} \quad (13)$$

in which case, the working pressure, P_w , should not exceed:

$$P_{\text{ult}} [1 - (W_c / W_t F_m)^2] / F_{\text{si}} \quad (14)$$

6.12 Factors of safety

The recommended minimum values of safety factors, F_{se} and F_{si} , for the equations in 6.11 are given in Table 8.

Table 8 — Minimum values of F_{se} and F_{si}

Pipe material	F_{se} ^{A)}		F_{si}
	Non-pressure	Pressure	Pressure
Clay	1.25	—	—
Concrete	1.25	1.60	2.0

^{A)} For reinforced concrete pipes, F_{se} should be not less than W_t/W_p .

6.13 Limit state design

6.13.1 Introduction

The traditional approach for rigid pipe design is set out in 6.10. The method provides a minimum bedding factor, based on factor of safety values given in Table 8.

A limit state design approach can be used with verification by the partial factor method as an alternative to global factors of safety. Similar principles are used for the assessment of the minimum bedding factor required.

6.13.2 Limit states

The pipe crushing strength, W_p , is used to determine the ultimate limit state, ULS. Product standards specify this strength as an absolute minimum, which can also be used in place of characteristic strength for the purposes of the ULS design checks.

The serviceability limit state, SLS, is assessed using the proof load, W_p . The proof load is only applicable for reinforced products where a maximum crack width is specified in product standards. The crack width is generally considered to be an SLS for durability issues.

Ultimate limit state (ULS): Pipe crushing strength (W_p)

Serviceability limit state (SLS): Proof load (W_p)

6.13.3 Design loads

Design loads are calculated for the following, using the methodology set out in 6.10:

- soil load: W_c for the narrow trench and W'_c for the wide trench or embankment condition;
- concentrated surcharge load, W_{CSU} ; and
- internal water load, W_w .

6.13.4 Partial safety factors

To calculate the total design load required, partial factors are applied to the respective loads. The partial factors consistent with BS EN 1990:2002+A1:2005 and BS EN 1991-4:2006 are given in Table 9, along with values for the ULS and SLS.

Table 9 — *Design factors of actions*

Action	Type		Limit state		Reference
			ULS	SLS	
Permanent	Soil	γ_G	1.35	1.0	NA to BS EN 1990:2002+A1:2005, Table NA.A2.4(B)
Variable	Traffic/Surcharge	γ_Q	1.35	1.0	NA to BS EN 1990:2002+A1:2005, Table NA.A2.4(B)
Permanent	Internal water	$\gamma_{G,intw}$	1.2	1.0	BS EN 1991-4:2006, NA.A.2.1

6.13.5 Design value of actions

The total design external load in kN/m, $W_{e,Ed}$, is given by equation (15):

$$W_{e,Ed} = \gamma_G W'_c + \gamma_Q W_{csu} + \gamma_{G,intw} W_w \quad (15)$$

NOTE Substitute W_c for W'_c in narrow trench conditions.

6.13.6 Minimum recommended bedding factor, F_m

The minimum bedding factor is calculated for both ULS and SLS as follows.

$$ULS = F_m \geq W_{e,Ed}/W_t \quad (16)$$

$$SLS = F_m \geq W_{e,Ed}/W_p \quad (17)$$

For reinforced pipes the minimum F_m required for both limit states should be calculated and the critical or higher value used to specify the bedding class.

For unreinforced pipes only the ULS is applicable and the resulting minimum F_m is used.

6.14 Design of egg-shaped pipes**6.14.1 Design approach**

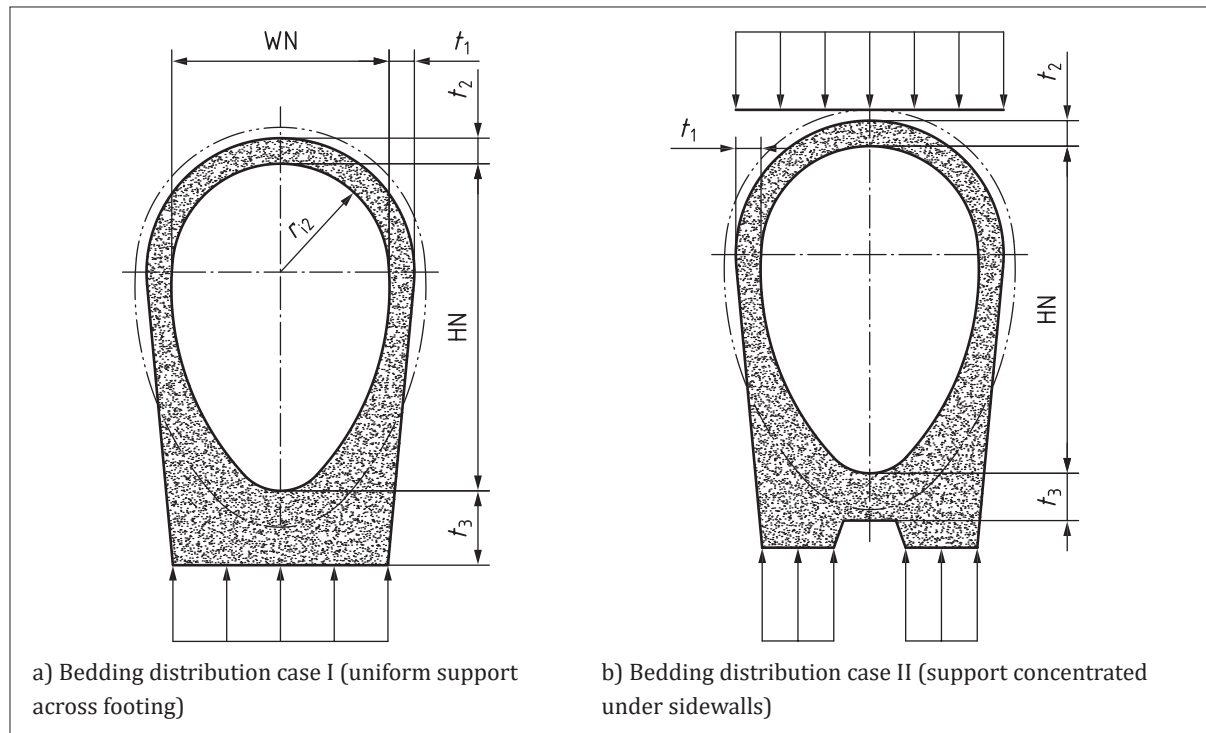
The design method described in this subclause is based on PD CEN/TR 1295-3:2007, as updated in the (forthcoming) German standard DWA-A 127-2 [6], utilizing bending moments at crown, springing and invert level to determine flexural tensile stress and to compare with permissible stress in unreinforced concrete pipes.

6.14.2 Design method**6.14.2.1** Establish values for the following dimensions, in metres (m):

- WN – internal width;
- HN – internal height;
- r_{i2} – internal radius at crown (generally = WN/2);
- t – wall thickness (which can vary with location around cross-section);
- t_1 – wall thickness at springings;
- t_2 – wall thickness at the crown;
- t_3 – wall thickness at the invert; and
- r_m – mean pipe radius at springing level ($= r_{i2} + t_1/2$),

and for the applicable bedding distribution case dependent on the footing design of the pipe cross-section, as defined by [Figure 21](#).

Figure 21 — Dimensions and bedding distribution cases for egg-shaped pipes



6.14.2.2 For embedment of egg-shaped pipes, use only bedding class B or S; the choice between these two does not affect structural design.

6.14.2.3 Determine external loading by calculating soil load (W_c for narrow trench, W'_c for wide trench) and surcharge load, W_{csu} , in accordance with [6.10](#).

6.14.2.4 Determine external pressure components, expressed in kN/m^2 , as follows:

- vertical pressure due to soil load, $P_{wc} = W_c/B_c$ or $P_{wc} = W'_c/B_c$, as applicable;
- vertical pressure due to surcharge (traffic) load, P_s , in accordance with [Clause 5](#);
- horizontal pressure due to soil load, P_h , given by:

$$\text{for narrow trench condition:} \quad P_h = 0 \quad (18)$$

$$\text{for wide trench condition:} \quad P_h = 0.3 \left(\frac{\gamma H B_d - W'_c}{B_d - B_c} \right) \quad (19)$$

- external water pressure at pipe springing level, P_{hw} .

NOTE 1 Horizontal pressure due to variable (traffic) load may be ignored for conservative design.

NOTE 2 Stresses developed from self-weight and internal water in the pipe are typically so small they can be ignored.

6.14.2.5 Calculate the bending moments, M , due to each pressure component determined in accordance with [6.14.2.3](#) by applying the bending moment coefficients given in [Table 10](#) according to location on the pipe cross-section, as follows:

- vertical load (soil and external water):

$$\text{for narrow trench condition:} \quad M_V = m_{qv} \times P_{wc} \times r_m^2 \quad (20)$$

$$\text{for wide trench condition:} \quad M_V = m_{qv} \times P_{wc} \times r_m^2 \quad (21)$$

- vertical traffic load:

$$M = m_{qv} \times P_S \times r_m^2 \quad (22)$$

- horizontal load (soil and external water):

$$\text{for narrow trench condition:} \quad M_H = m_{qh} \times P_{hw} \times r_m^2 \quad (23)$$

$$\text{for wide trench condition:} \quad M_H = m_{qh} \times (P_H + 0.7P_{hw}) \times r_m^2 \quad (24)$$

Table 10 — *Bending moment coefficients for egg-shaped pipes*

Bending distribution class	Location	m_{qv}	m_{qh}
I	Crown	+0.34	-0.54
	Springing	-0.18	+0.31
	Invert	+0.54	-1.05
II	Crown	+0.32	-0.54
	Springing	-0.18	+0.31
	Invert	+0.41	-1.05

Source of coefficients: [\[6\]](#)

6.14.2.6 Calculate total factored net bending moment expressed in kNm/m, M_{Ult} , for each location by applying partial safety factors in accordance with [Table 9](#), i.e.:

$$M_{Ult} = \gamma_G (M_V + M_H) + \gamma_Q M_S \quad (25)$$

6.14.2.7 Calculate factored tensile stress due to bending, $f_{ctm:fl}$, at the intended installation location, expressed in megapascals (MPa) as:

$$f_{ctm:fl} = M_{Ult} / [1\,000 (t^2/6)]. \quad (26)$$

NOTE The reduction in flexural tensile stress due to compressive thrust in the pipe wall is normally small and may be ignored for conservative design.

6.14.2.8 Assess the flexural tensile stress against the manufacturer's declared permissible tensile strength. If this is not available, use the tensile strength for the concrete strength class, f_{ctm} , given in BS EN 1992-1-1:2004+A1:2014, [Table 3.1](#), and modified for flexural tensile stress, $f_{ctm:fl}$, in BS EN 1992-1-1:2004+A1:2014, [3.1.8](#).

The design flexural tensile stress should be less than the permissible stress for a satisfactory design.

6.15 Simplified bedding table for vitrified clay pipes

6.15.1 General

[Table 11](#) has been simplified from the information given in the bedding construction tables available from the Clay Pipe Development Association (CPDA) [\[4\]](#) and is compatible with the method of calculation described in this subclause.

The parameters given in [6.15.2](#) to [6.15.6](#) have been used for vitrified clay pipe design.

6.15.2 Pipes and pipe strengths

[Table 11](#) covers vitrified clay pipes as specified in BS EN 295-1 for classes and crushing strengths normally available in the UK. Other classes and crushing strengths are given in BS EN 295-1.

6.15.3 Pipe embedments and bedding factors

[Table 11](#) shows rigid pipe embedments of classes D, N, F, B and S using the bedding factors given in [Table 5](#).

6.15.4 Loading conditions

Loads have been calculated for pipes laid in trenches either in main roads or in fields and gardens to cover the range of normal applications.

6.15.5 Trench widths

[Table 11](#) assumes the existence of wide trench conditions. Full calculations for narrow trenches could result in more economical use of bedding materials or greater depth of application for the same pipe strength.

6.15.6 Depths of cover

[Table 11](#) sets out the applicable ranges of depths of cover for each pipe size, pipe strength, loading condition and type of embedment, with a maximum depth limited to a practicable value of 6.0 m. Where this depth can be exceeded, the value 6.0+ m is shown in the CPDA tables [\[4\]](#) which also give further values. For pipes buried at depths less than 600 mm, see [A.13](#).

Table 11 — *Simplified table for the embedment of vitrified clay pipes in wide trench conditions*

Nominal diameter mm	Bedding construction class	Bedding factor	Pipe strength class	Crushing strength kN/m	Depth range m	
					Main roads	Fields and gardens
100	D or N	1.1	—	28	0.6 – 5.7	0.4 – 6.0+
				40	0.6 – 6.0+	0.4 – 6.0+
	F	1.9	—	28	0.6 – 6.0+	0.4 – 6.0+
				40	0.6 – 6.0+	0.4 – 6.0+
	B or S	2.5	—	28	0.6 – 6.0+	0.4 – 6.0+
				40	0.6 – 6.0+	0.4 – 6.0+
150	D or N	1.1	—	28	0.7 – 3.4	0.6 – 4.0
				40	0.6 – 5.6	0.6 – 5.9
	F	1.9	—	28	0.6 – 6.0+	0.6 – 6.0+
				40	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	—	28	0.6 – 6.0+	0.6 – 6.0+
				40	0.6 – 6.0+	0.6 – 6.0+

Table 11 (continued)

Nominal diameter mm	Bedding construction class	Bedding factor	Pipe strength class	Crushing strength kN/m	Depth range m	
					Main roads	Fields and gardens
200	D or N	1.1	160	32	0.8 – 2.7	0.6 – 3.5
			200	40	0.6 – 4.0	0.6 – 4.5
			240	48	0.6 – 5.1	0.6 – 6.0+
	F	1.9	160	32	0.6 – 6.0+	0.6 – 6.0+
			200	40	0.6 – 6.0+	0.6 – 6.0+
			240	48	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	160	32	0.6 – 6.0+	0.6 – 6.0+
			200	40	0.6 – 6.0+	0.6 – 6.0+
			240	48	0.6 – 6.0+	0.6 – 6.0+
225	D or N	1.1	160	36	0.9 – 2.7	0.6 – 3.5
			200	45	0.6 – 3.9	0.6 – 4.4
	F	1.9	160	36	0.6 – 5.9	0.6 – 6.0+
			200	45	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	160	36	0.6 – 6.0+	0.6 – 6.0+
			200	45	0.6 – 6.0+	0.6 – 6.0+
300	D or N	1.1	120	36	—	0.6 – 2.5
			160	48	0.8 – 2.7	0.6 – 3.5
			240	72	0.6 – 5.1	0.6 – 5.4
	F	1.9	120	36	0.6 – 4.2	0.6 – 4.6
			160	48	0.6 – 6.0	0.6 – 6.0+
			240	72	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	120	36	0.6 – 5.9	0.6 – 6.0+
			160	48	0.6 – 6.0+	0.6 – 6.0+
			240	72	0.6 – 6.0+	0.6 – 6.0+
375	D or N	1.1	95	36	—	0.9 – 1.9
			120	45	—	0.6 – 2.7
			160	64	0.7 – 3.4	0.6 – 4.0
	F	1.9	95	36	0.7 – 3.2	0.6 – 3.9
			120	45	0.6 – 4.5	0.6 – 5.0
			160	64	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	95	36	0.6 – 4.8	0.6 – 5.2
			120	45	0.6 – 6.0+	0.6 – 6.0+
			160	64	0.6 – 6.0+	0.6 – 6.0+

Table 11 (continued)

Nominal diameter mm	Bedding construction class	Bedding factor	Pipe strength class	Crushing strength kN/m	Depth range m	
					Main roads	Fields and gardens
400	D or N	1.1	95	38	—	0.9 – 1.8
			120	48	—	0.6 – 2.6
			160	64	0.8 – 2.9	0.6 – 3.7
	F	1.9	95	38	0.8 – 3.0	0.6 – 3.8
			120	48	0.6 – 4.4	0.6 – 4.9
			160	64	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	95	38	0.6 – 4.6	0.6 – 5.1
			120	48	0.6 – 6.0+	0.6 – 6.0+
			160	64	0.6 – 6.0+	0.6 – 6.0+
450	D or N	1.1	95	43	—	0.8 – 1.9
			120	54	—	0.6 – 2.7
			160	72	0.8 – 3.0	0.6 – 3.8
	F	1.9	96	43	0.7 – 3.2	0.6 – 3.9
			120	54	0.6 – 4.5	0.6 – 5.0
			160	72	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	96	43	0.6 – 4.8	0.6 – 5.2
			120	54	0.6 – 6.0+	0.6 – 6.0+
			160	72	0.6 – 6.0+	0.6 – 6.0+
500	D or N	1.1	96	48	—	0.8 – 1.9
			120	60	—	0.6 – 2.7
			160	80	0.8 – 3.0	0.6 – 3.8
	F	1.9	96	48	0.7 – 3.2	0.6 – 3.9
			120	60	0.6 – 4.5	0.6 – 5.0
			160	80	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	96	48	0.6 – 4.8	0.6 – 5.2
			120	60	0.6 – 6.0+	0.6 – 6.0+
			160	80	0.6 – 6.0+	0.6 – 6.0+

Table 11 (continued)

Nominal diameter mm	Bedding construction class	Bedding factor	Pipe strength class	Crushing strength kN/m	Depth range m	
					Main roads	Fields and gardens
600	D or N	1.1	80	48	—	—
			95	57	—	0.8 – 1.9
			160	96	0.7 – 3.1	0.6 – 3.8
	F	1.9	80	48	1.0 – 2.1	0.6 – 3.2
			95	57	0.7 – 3.2	0.6 – 3.9
			160	96	0.6 – 6.0+	0.6 – 6.0+
	B or S	2.5	80	48	0.6 – 3.8	0.6 – 4.4
			95	57	0.6 – 4.8	0.6 – 5.2
			160	96	0.6 – 6.0+	0.6 – 6.0+

6.16 Simplified bedding table for concrete pipes

[Table 12](#) sets out the applicable ranges of depths of cover for each pipe size, loading condition and type of embedment for a single strength class to BS EN 1916, with a maximum depth limited to a practicable value of 6.0 m. Where this depth can be exceeded the value 6.0+ m is shown.

Advice on depths greater than 6.0 m should be obtained from the pipe manufacturer (who might have stronger pipes available) and an experienced structural pipe design engineer.

Pipes up to and including DN600 are assumed to be unreinforced, while DN675 and above are assumed to be reinforced. The manufacturer should be consulted to confirm whether pipes are reinforced or unreinforced.

Bedding factors are as given in [Table 12](#).

Table 12 — Simplified bedding table for concrete pipes – Strength class 120

DN	Trench width condition ^{A)}	Bedding class	Bedding factor	Crushing strength kN/m	Depth range ^{B)} m	
					Main roads (Table 2)	Fields and gardens
225	Narrow	F	1.5	27	0.9 – 6.0+	0.6 – 6.0+
		B	1.9			
		S	2.2			
300	Wide	B/S	2.5	27	0.9 – 5.6	—
300	Narrow	F	1.5	36	0.9 – 6.0+	0.6 – 6.0+
		B	1.9			
		S	2.2			
	Wide	B/S	2.5	36	0.9 – 5.4	—
375	Narrow	F	1.5	45	—	0.6 – 3.8
		B	1.9		0.9 – 4.9	0.6 – 6.0+
		S	2.2		0.9 – 6.0+	—

Table 12 (continued)

DN	Trench width condition ^{A)}	Bedding class	Bedding factor	Crushing strength kN/m	Depth range ^{B)} m	
					Main roads (Table 2)	Fields and gardens
375	Wide	B/S	2.5	45	0.9 – 5.6	—
450	Narrow	F	1.5	54	—	0.6 – 4.4
		B	1.9		0.9 – 5.7	0.6 – 6.0+
		S	2.2		0.9 – 6.0+	—
	Wide	B/S	2.5	54	0.9 – 5.9	—
525	Narrow	F	1.5	63	—	0.6 – 5.0
		B	1.9		0.9 – 6.0+	5.0 – 6.0+
		S	2.2		0.9 – 6.0+	—
	Wide	B/S	2.5	63	0.9 – 5.7	—
600	Narrow	F	1.5	72	—	0.6 – 4.6
		B	1.9		0.9 – 6.0	0.6 – 6.0+
		S	2.2		0.9 – 6.0+	—
	Wide	B/S	2.5	72	0.9 – 5.8	—
675	Narrow	F	1.5	81	—	0.6 – 4.6
		B	1.9		0.9 – 4.1	0.6 – 6.0+
		S	2.2		0.9 – 5.9	—
	Wide	B/S	2.5	81	0.9 – 4.8	—
750	Narrow	F	1.5	90	—	0.6 – 3.6
		B	1.9		0.9 – 4.7	0.6 – 5.6
		S	2.2		0.9 – 6.0	0.6 – 6.0+
	Wide	B/S	2.5	90	0.9 – 4.9	—
825	Narrow	F	1.5	99	—	0.6 – 3.8
		B	1.9		0.9 – 4.7	0.6 – 5.8
		S	2.2		0.9 – 6.0+	0.6 – 6.0+
	Wide	B/S	Table 6	99	0.9 – 4.6 ^{C)}	—
900	Narrow	F	1.5	108	—	0.6 – 3.2
		B	1.9		0.9 – 3.6	0.6 – 4.6
		S	2.2		0.9 – 5.2	0.6 – 5.8
	Wide	B/S	Table 6	108	0.9 – 4.6 ^{C)}	—
1 050	Narrow	F	1.5	126	—	0.6 – 3.4
		B	1.9		0.9 – 3.8	0.6 – 5.0
		S	2.2		0.9 – 5.2	0.6 – 6.0+
	Wide	B/S	Table 6	126	0.9 – 4.9 ^{C)}	—
1 200	Narrow	F	1.5	144	—	0.6 – 3.4
		B	1.9		0.9 – 3.9	0.6 – 5.0
		S	2.2		0.9 – 5.2	0.6 – 6.0+
	Wide	B/S	Table 6	144	0.9 – 4.6 ^{C)}	—

Table 12 (continued)

DN	Trench width condition ^{A)}	Bedding class	Bedding factor	Crushing strength kN/m	Depth range ^{B)} m	
					Main roads (Table 2)	Fields and gardens
1 350	Narrow	B	1.9	162	0.9 – 4.0	—
		S	2.2		0.9 – 5.3	—
	Wide	B/S	Table 6	162	0.9 – 4.5 ^{C)}	—
1 500	Narrow	B	1.9	180	0.9 – 3.9	—
		S	2.2		0.9 – 5.3	—
	Wide	B/S	Table 6	180	0.9 – 5.3 ^{C)}	—
1 800	Narrow	B	1.9	216	0.9 – 4.1	—
		S	2.2		0.9 – 5.3	—
	Wide	B/S	Table 6	216	0.9 – 4.6 ^{C)}	—
2 100	Narrow	B	1.9	252	0.9 – 4.1	—
		S	2.2		0.9 – 5.6	—
	Wide	B/S	Table 6	252	0.9 – 4.7 ^{C)}	—
2 400	Narrow	B	1.9	288	0.9 – 4.4	—
		S	2.2		0.9 – 5.8	—
	Wide	B/S	Table 6	288	0.9 – 4.9 ^{C)}	—

^{A)} If the transition is not calculated, wide trench conditions should be applied as the default calculation.

^{B)} Table depths were calculated using $W_t/W_p = 0.67$. Some manufacturers test to a higher ratio where $W_t/W_p = 0.8$. This will increase some of the depths in this table. Advice should be obtained from the manufacturer.

^{C)} Advice on greater depths should be obtained from the pipe manufacturer and an experienced structural pipe design engineer.

7 Flexible pipe design

7.1 General

A flexible pipe derives much of its load resistance from the soil surrounding the installed pipe. The stiffness of the native soil, the embedment soil and the pipe are critical input parameters in determining the maximum deflection and buckling resistance of the pipe.

Field studies and practical experience show that installation quality is as critical to the performance of buried flexible pipes as ring stiffness and embedment selection. The need to take explicit account of installation quality is one of the main reasons for the new method for the design of large-diameter structured-wall pipes given in 7.3.

7.2 Marston-Spangler method

COMMENTARY ON 7.2

This subclause gives the design formulae which are referred to in the flowcharts in Figure 22, which summarizes the Marston-Spangler design procedure for three different flexible pipe materials.

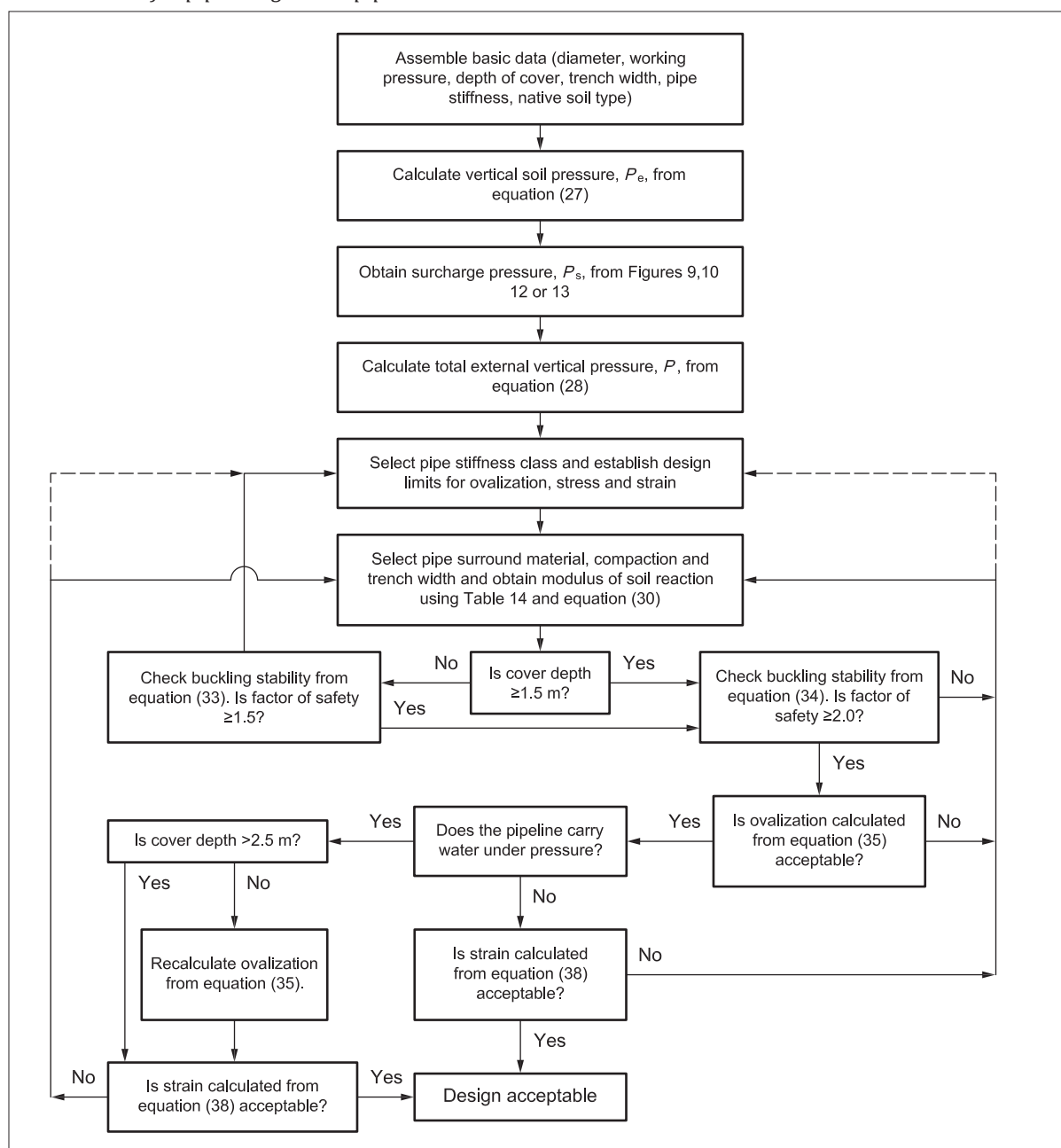
Figure 22A — Flowchart for pipe design: GRP pipes

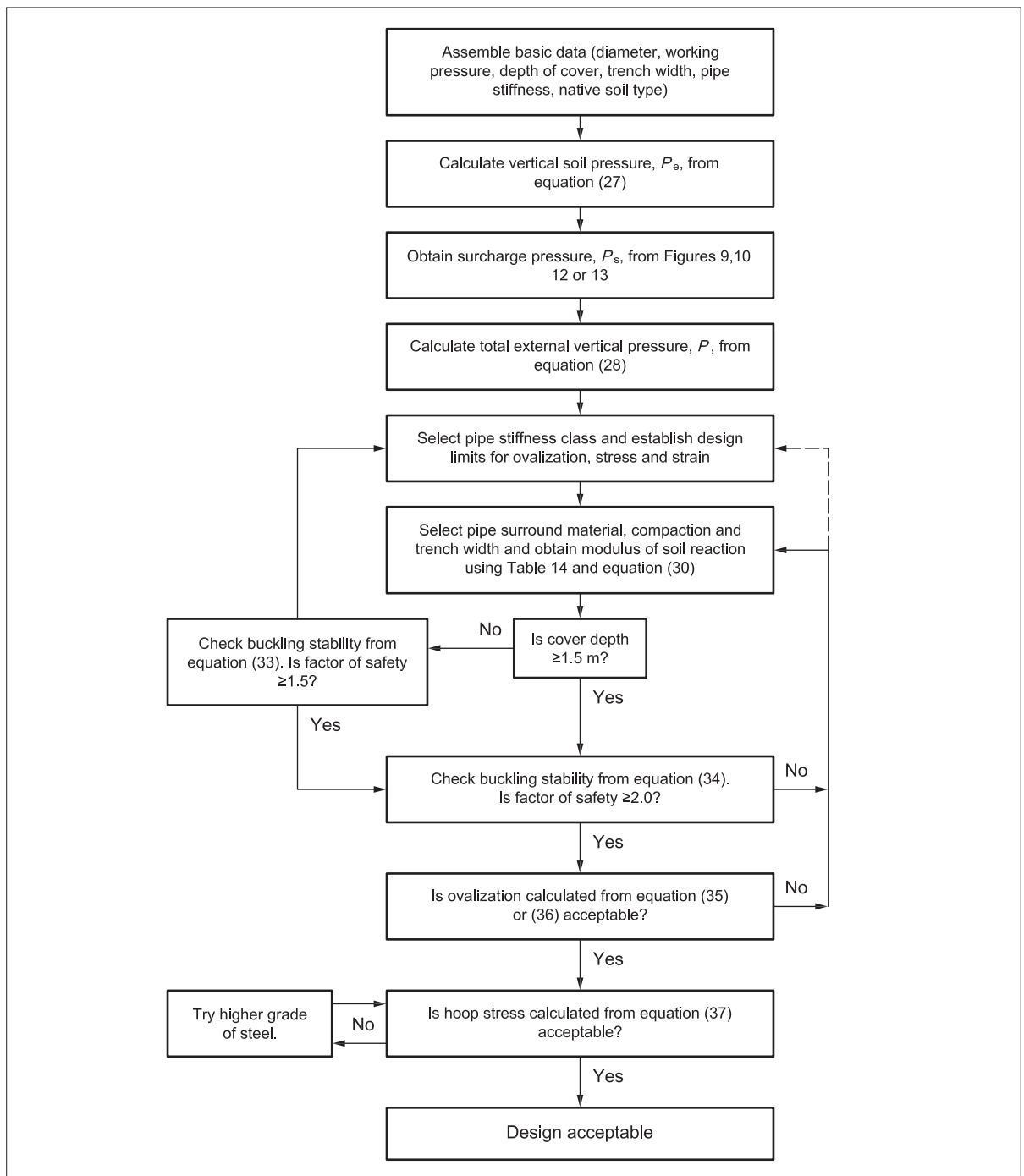
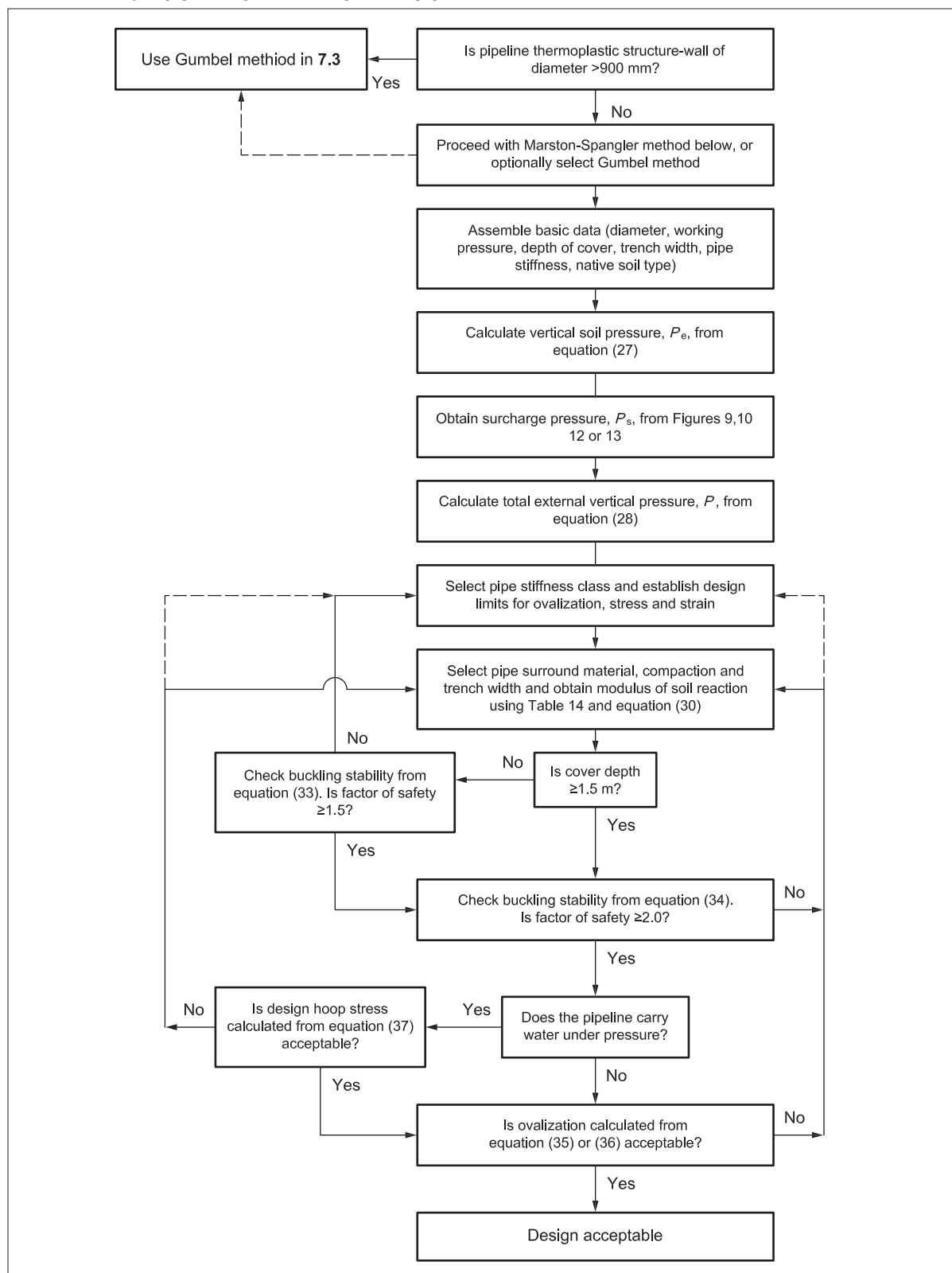
Figure 23B — *Flowchart for pipe design: Steel pipes*

Figure 23C — Flowchart for pipe design: Thermoplastics pipes

7.2.1 Design pressures for flexible pipes

The geostatic backfill load on a flexible pipe is given by γH , with no other coefficients applied. Therefore, vertical soil pressure, P_e :

$$P_e = \gamma H \quad (27)$$

In the absence of specific soils data supporting the use of other values, it is normal practice to assume $\gamma = 19.6 \text{ kN/m}^3$.

The surcharge pressure, P_s , due to vehicle loadings is calculated in accordance with [Clause 5](#).

Total vertical external pressure (kN/m^2), P , is given by:

$$P = P_e + P_s \quad (28)$$

7.2.2 Native soil

The stiffness of the native soil in which a pipeline trench is excavated can be particularly important for the design of flexible pipes. The stiffness of the native soil is measured in terms of the Spangler modulus, E'_3 , which, in combination with the embedment soil modulus, E'_2 , is used to calculate the overall modulus of soil reaction, E' . This Spangler modulus is not a fundamental engineering property of the soil and cannot be directly measured. It is a semi-empirical parameter which is a function of the pipe and soil system (and not the soil alone). [Table 13](#) gives Spangler moduli for various native soils which assume the worst case of shallow depth of burial and the presence of groundwater.

Table 13 — *Guide values of Spangler modulus for native soils, E'_3*

Soil type	Spangler modulus for soils in various conditions (MN/m^2)				
	Very dense	Dense	Medium dense	Loose	Very loose
Gravel	Over 40	15 to 40	9 to 15	5 to 9	3 to 5
Sand	15 to 20	9 to 15	4 to 9	2 to 4	1 to 2
Clayey, silty sand	10 to 15	6 to 10	2.5 to 6	1.5 to 2.5	0.5 to 1.5
Clay	Very hard	11 to 14			
	Hard	10 to 11			
	Very stiff	6 to 10			
	Stiff	4 to 6			
	Firm	3 to 4			
	Soft	1.5 to 3			
	Very soft	0 to 1.5			

NOTE Further guidance on the use of lightweight aggregates is given in [A.11.4](#).

7.2.3 Flexible pipe embedment properties

The embedment soil modulus, E'_2 , for the selected pipe surround material at the chosen level of compaction should be obtained from [Table 14](#).

Low-stiffness flexible pipes, e.g. of ring stiffness of 2 kN/m^2 or less, absorb energy, making it difficult to achieve the higher modified Proctor densities and moduli of soil reaction given in [Table 14](#). Such pipes are also vulnerable to uneven deformations from high compactive effort. For these pipes, therefore, a self-compacting embedment, such as a single-sized gravel, should be used as this achieves a high modulus with little compactive effort.

Table 14 — *Semi-rigid and flexible pipe embedment properties*

Embedment class as in Table 15 and deflection coefficient K_x	Compacted density, M_p (See Note 3) %	Embedment soil modulus, E'_2 MN/m ²	Deflection lag factor, D_L ^{A)}	Strain factor, D_f ^{B)} , for various pipe ring stiffnesses ^{C)} kN/m ²					
				1.25	2.5	5.0	10	15	30 or more
Class S1 $K_x = 0.083$	Self-compacting ^{D)}	10	1.0	4.7	4.5	4.3	4.0	3.75	3.0
Class S2 $K_x = 0.083$	Uncompacted	3	1.5	4.7	4.5	4.3	4.0	3.75	3.0
	80	5	1.25	4.7	4.5	4.3	4.0	3.75	3.0
	85	7	1.0	4.7	4.5	4.3	4.0	3.75	3.25
	90	10	1.0	4.7	4.5	4.3	4.0	3.75	3.5
Class S3 $K_x = 0.100$	85	5	1.5	6.2	5.5	4.75	4.25	4.0	3.25
	90	7	1.25	7.75	6.6	5.5	4.7	4.25	3.5
Class S4 $K_x = 0.100$	85	3	1.5	6.2	5.5	4.75	4.25	4.0	3.5
	90	5	1.25	7.75	6.6	5.5	4.7	4.25	3.5
Class S5 $K_x = 0.100$	85	1	3.0	—	—	—	—	4.0	3.5
	90	3	2.0	—	—	—	—	4.25	3.5
Class B1 $K_x = 0.083$	85	5	1.5	—	—	—	5.0	4.0	3.5
	90	7	1.25	—	—	—	5.5	4.25	3.5
Class B2 $K_x = 0.083$	85	3	2.0	—	—	—	5.5	4.25	3.5
	90	5	1.75	—	—	—	6.0	5.0	3.5

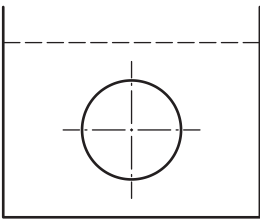
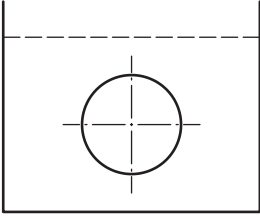
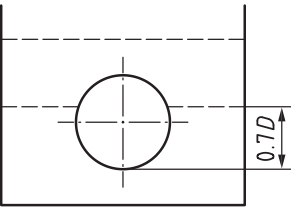
NOTE 1 For construction details of embedment classes see [Table 15](#).

NOTE 2 Quoted values of E'_2 assume pipe is to be installed below groundwater.

NOTE 3 M_p indicates modified Proctor density and corresponds to the heavy compaction test in BS 1377.

- ^{A)} Where the designer can be certain that initial pressurization of at least 3 bar (1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 105 N/m²) will take place within one year of backfilling and the depth of cover does not exceed 2.5 m, a value of 1.0 may be taken for the deflection lag factor.
- ^{B)} These factors were derived for GRP pipes and should be used with caution for other materials. The values in this table have been brought forward from earlier editions of this standard where they were used in conjunction with the Marston-Spangler method.
- ^{C)} Pipe stiffnesses are initial values. Pipe standards for different materials quote nominal stiffnesses at different values. These may be interpolated.
- ^{D)} Single-sized granular material, with compaction fraction value of 0.15 or less, capable of filling space under haunches of pipe with low compactive effort.

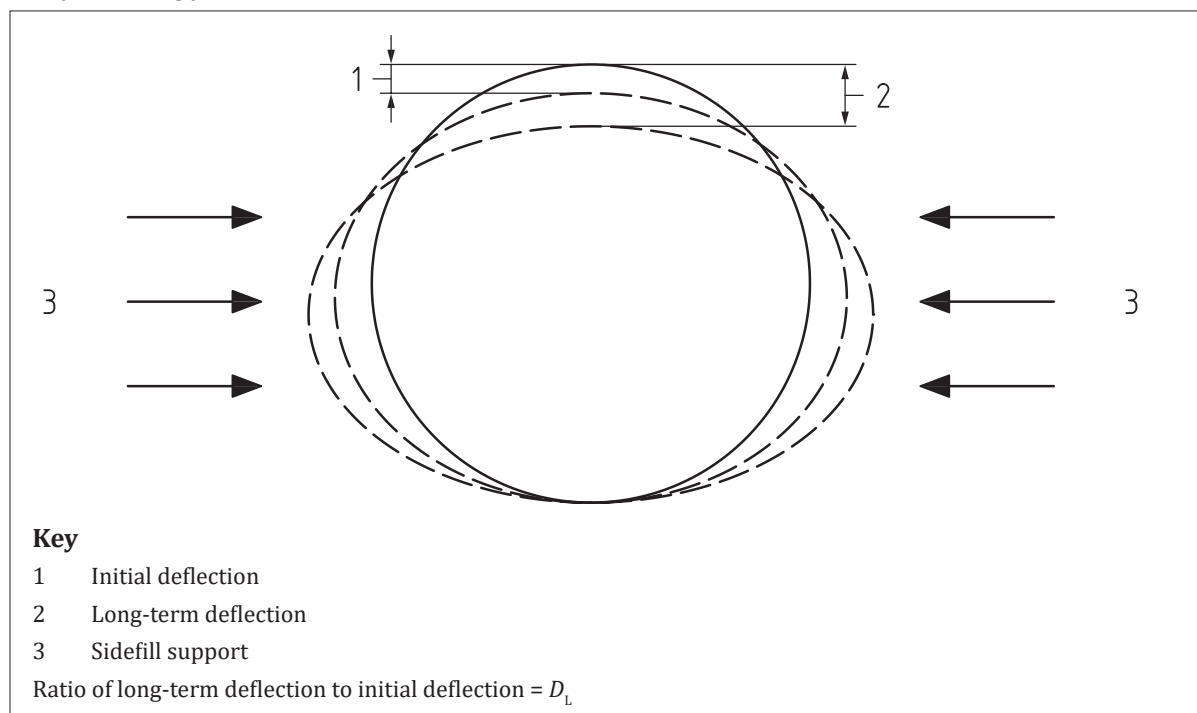
Table 15 — *Flexible pipe embedments*

Embedment class	Embedment configuration	Bed and sidefill materials	Notes
S1 and S2		Class S1: Gravel (single size) Class S2: Gravel (graded)	Normally processed granular materials
S3 to S5		Class S3: Sand and coarse-grained soil with less than or equal to 12% fines Class S4: Coarse-grained soil with more than 12% fines. OR Fine-grained soil, liquid limit less than 50%, medium-to-no plasticity and more than 25% coarse-grained material.	These represent “as-dug” soils and require particularly close control when used with low stiffness pipes.
		Class S5: Fine-grained soil, liquid limit less than 50%, medium-to-no plasticity and less than 25% coarse-grained material.	Class S5 only recommended for use with semi-rigid pipes.
B1 and B2		Class B1 Upper surround as for S3 or S4 Lower surround as for S1 or S2	Class B embedments not recommended for use with pipes of less than 10 kN/m ² stiffness.
		Class B2: Upper surround as for S5 Lower surround as for S1 or S2	

NOTE See [Table 14](#) for design parameters for embedment classes.

7.2.4 Deflection lag factor, D_L

The deflection lag factor, D_L (see [Figure 23](#)), takes account of the settlement of the sidefill over time resulting in further deflection of the pipe.

Figure 23 — Deflection lag factor

D_L gives an indication of the degree of compaction that occurs during the installation for a given embedment. It can be seen from Table 14 that the greater the level of compaction, the lower the deflection lag factor. As the embedment becomes less favourable (for example, classes S5, B1 and B2), the harder it is to achieve a low deflection lag factor.

Where the pipe is to be pressurized to at least 3 bar³⁾ working pressure within one year of burial at a depth of less than 2.5 m, D_L can be taken as 1, since the internal pressure has the beneficial effect of preventing the long-term deflection from taking place.

7.2.5 Overall modulus of soil reaction, E' , and Leonhardt's coefficient, C_L

Leonhardt's coefficient, C_L (soil modulus adjustment factor) [7], is used to derive an overall modulus of soil reaction, E' . The coefficient evaluates the influence the native soil, E'_3 , and trench width have on the embedment modulus, E'_2 .

Leonhardt's coefficient, C_L , is calculated from equation (29).

$$C_L = \frac{0.985 + (0.544B_d / B_c)}{[1.985 - 0.456(B_d / B_c)](E'_2 / E'_3) - [1 - (B_d / B_c)]} \quad (29)$$

The overall modulus of soil reaction, E' , is then calculated from equation (30):

$$E' = E'_2 C_L \quad (30)$$

If the trench width, B_d , is more than 4.3 times the external pipe diameter, B_c , then C_L does not need to be calculated and the overall modulus of soil reaction, E' , can be taken as the embedment soil modulus, E'_2 . This is because where $B_d \geq 4.3B_c$, $C_L = \text{unity}$.

7.2.6 Critical pressure for buckling (long-term and short term), P_{cr1} and P_{crs}

The long-term and short-term critical buckling equations are identical except for the introduction of the creep factor or creep ratio in the long-term equation for plastics pipes, turning the initial pipe ring stiffness into long-term ring stiffness. In equation (31), the long-term and short-term values

³⁾ 1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 10⁵ N/m².

of the modulus, E , are used to calculate P_{crl} and P_{crs} respectively. For metal pipes, the long-term and short-term moduli are identical.

$$P_{\text{cr}} = 0.6(EI / D^3)^{0.33} (E')^{0.67} \quad (31)$$

For solid-wall pipes of homogenous material, $I = t^3/12$. For other pipes the second moment of area, I , should be obtained from the pipe manufacturer.

7.2.7 Factor of safety against buckling, F_s

The equation for the factor of safety against buckling is given in a) or b) as applicable.

a) With soil support:

$$F_s = 1 / \left(\frac{P_e}{P_{\text{crl}}} + \frac{P_s + P_u}{P_{\text{crs}}} \right) \quad (32)$$

In equation (32), the vertical soil pressure, P_e , is divided by the long-term critical buckling pressure, P_{crl} . This is because the earth load is always present. However, the surcharge pressure, P_s , and (if present) the short-term vacuum pressure, P_u , are divided by the short-term critical buckling pressure, P_{crs} . This is because these are transient loads, to which the pipe's response will be with its short-term modulus.

For vacuum sewers and in other cases of sustained negative operational pressure, the factor of safety should be calculated using equation (32a).

$$F_s = 1 / \left(\frac{P_e + P_u}{P_{\text{crl}}} + \frac{P_s}{P_{\text{crs}}} \right) \quad (32a)$$

b) Without soil support:

This case needs only to be considered when $H < 1.5$ m.

$$F_s = P_{\text{cra}} / (P_e + P_u) \quad (33)$$

Where there is deemed to be no soil support, e.g. where a pipe has a shallow burial depth and could lose its side support if trenches for other utilities are dug nearby, equation (34) is used. Here the short-term stiffness value is adopted.

$$P_{\text{cra}} = 24EI / D^3 \quad (34)$$

Unless otherwise specified the factor of safety should be not less than 2, where soil support is taken into account, and not less than 1.5 where soil support is discounted.

7.2.8 Ovalization, Δ/D

Ovalization (ring deflection) of the pipe, Δ/D , is given by equation (35) and consists of two major components; a component built in during construction, which is dependent on installation quality, and an additional component due to backfill and traffic load, which is resisted by the pipe stiffness and soil stiffness.

$$\Delta / D = \frac{K_x \left[(D_L P_c) + P_s \right]}{8 \left(EI / D^3 \right) + 0.061 E'} + (\Delta_0 / D) \quad (35)$$

where:

- Δ_0/D is the component of deflection built in during laying of the pipe and construction of its embedment; design values of Δ_0/D as a function of pipe ring stiffness and installation control are given by [Table 16](#); and
- EI/D^3 is the pipe ring stiffness calculated using the long-term or short-term value of modulus, E , as appropriate.

NOTE The initial deflection is obtained with the value of D_L set at 1.0, and the long-term deflection with the value of D_L from [Table 14](#).

Table 16 — Design values of initial deflection, Δ_0/D

Ring stiffness ^{A)} (kN/m ²)	Initial deflection, Δ_0/D		
	Installation control Type A ^{B)}	Installation control Type B ^{C)}	Installation control Type C ^{D)}
2	1.5%	2.5%	5%
4	1.0%	1.5%	4%
6	0.75%	1.25%	3.5%
8	0.5%	1.0%	3%
9 and above	0.0%	0.5%	1%

^{A)} For intermediate values of ring stiffness up to 10 kN/m², interpolated values may be used. For ring stiffnesses less than 2 kN/m², only type A installation is recommended.

^{B)} Type A:

Installation is monitored and quality-assured by the pipe manufacturer or the asset owner's appointed agent

S1 or S2 embedment class

^{C)} Type B:

Unmonitored installation

S1 embedment class

or

Installation is monitored and quality-assured by the installation contractor

S2 embedment class

or

Installation is monitored and quality-assured by the pipe manufacturer or the asset owner's appointed agent

S3 embedment class

^{D)} Type C

Unmonitored installation of S2 embedment class

All installations are of S3 to S5, B1 and B2 embedment class

The beneficial re-rounding effect that arises from the presence of water pressure within a flexible pipe can also be determined, provided the depth of cover to the pipe is not greater than 2.5 m. If the pipe is buried at a greater depth, then the backfill pressure will be too great to allow re-rounding to occur.

Re-rounding of pressure pipes reduces the initial deflection, (Δ/D) , to a lower value, $(\Delta/D)_R$, given by equation (36):

$$(\Delta/D)_R = (1 - P_i/40)(\Delta/D) \quad (36)$$

where:

P_i is the internal pressure in bars.

Re-rounding should only be addressed in the design where all three of the following conditions are met:

- a) water pressure exceeds 3 bar⁴⁾; and
- b) water pressure is applied within one year of backfilling; and
- c) the depth of cover is less than 2.5 m.

The overall deflection limit is a function of the jointing method used for the pipe system and other serviceability criteria. For push-fit joints, the vertical deflection of the socket is limited to maintain leak tightness. The value of this deflection limit determined by product type testing varies according to the type of pipe and its jointing system, and typically ranges from 3% to 6% (see specific manufacturer information). For systems with welded joints, the limit on vertical deflection is generally 8%.

7.2.9 Design hoop stress in thermoplastics, σ_c

The design hoop stress equation (37) calculates the net hoop stress generated, σ_c , taking account of internal pressure and soil loading in the pipe walls by the presence of the internal pressure. This value is then compared against an allowable value for the pipe material.

$$\sigma_c = \frac{(P_i - P_e)D}{2t} \quad (37)$$

Equation (37) is also used for the calculation of hoop stress in thin-walled steel pipes.

7.2.10 Strain in glass reinforced plastics (GRP) pipes

Since GRP is a composite material, its response to loading is a little more complicated than for a homogeneous material.

For non-pressure pipes the bending strain in the pipe wall, ϵ_b , is given by:

$$\epsilon_b = D_f(\Delta/D)(t/D) \quad (38)$$

For pressure pipes, the combined strain, ϵ_c , is given by adding the bending strain in the pipe wall to the hoop stress due to internal water pressure divided by the hoop modulus of the pipe material:

$$\epsilon_c = D_f(\Delta/D)_R(t/D) + P_i D / 2E_h t \quad (39)$$

The strain factor, D_f , used in equations (38) and (39) is tabulated in [Table 14](#) for different pipe stiffnesses and embedments. This shows that the higher the modulus of soil reaction and the stiffer the pipe, the lower the value of the strain factor. This is because less compactive effort is required to achieve the same embedment modulus. It is very important when choosing a pipe stiffness and embedment to take account of the amount of compaction that will be required in the field to achieve the desired modulus of soil reaction.

⁴⁾ 1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 10⁵ N/m².

Unlike thermoplastics, it is correct to add strains in thermoset pipe materials to ensure that the peak instantaneous, or long-term, strain value does not exceed a prescribed total. This is because failures in the walls of these pipes are typically due to excessive local strains (e.g. external impact can cause high-tensile strains on the inside pipe wall that later become a point of weakness). Unlike thermoplastics pipes, there is no stress relaxation in the walls of GRP pipes under constant strain conditions (i.e. bending strain as the result of an applied ring deflection).

7.3 Gumbel method

7.3.1 General

The design methodology for flexible pipes described in the previous edition of BS 9295, and largely reproduced in 7.2, does not fully reflect the structural behaviour of large-diameter structured-wall thermoplastics or composite (e.g. GRP or steel-reinforced thermoplastics) pipes. The new, alternative method introduced in this edition more accurately describes the structural behaviour of large-diameter circular thermoplastics pipes.

In the simplified form presented here, the Gumbel method [8]:

- applies to circular structured-wall pipes of thermoplastics conforming to BS EN 13476, Types A and B, but has the potential to cover other types of structured-wall pipe in due course;
- can be applied to solid-wall pipes of thermoplastics or other homogeneous flexible materials without restriction;
- is recommended for use to design buried pipes of diameter greater than 900 mm, but can be applied to smaller diameter pipes;
- takes account of non-pressure pipe systems subject to maximum surcharge of 5 m only;
- assumes construction of the pipe trench in accordance with BS EN 1610; and
- applies only where the depth of cover, H , exceeds the greater of 0.6 m under rigidly-paved surfaces or 0.9 m under unpaved or flexibly-paved surfaces, or half the pipe external diameter.

NOTE It is anticipated that future editions of BS 9295 will extend the range of application of this method.

The Gumbel method employs an improved model for the embedment and native soil surround and utilizes true soil parameters that can in principle be derived from standard site investigation tests such as standard penetration tests. The two-dimensional plane strain analysis of a long thin-walled cylinder embedded at depth in an elastic soil continuum provides the basic theoretical model for response of the composite pipe-soil system.

This approach distinguishes external loads applied to the buried pipe system from the interaction forces arising at the pipe-soil interface and, unlike the Marston-Spangler method, uses a variable to explicitly take account of the effect of groundwater. In common with the updated Marston-Spangler method (see 7.2), though, it takes account of the quality of installation, which is one of the most critical factors for the performance of buried pipes in general, and flexible pipes in particular. Like 7.2, this subclause focuses on the sectional response of a single, trench-laid pipe. All the further design criteria raised in Annex A (multiple pipes, embankments, flotation calculations, longitudinal effects, etc.) also apply to this method.

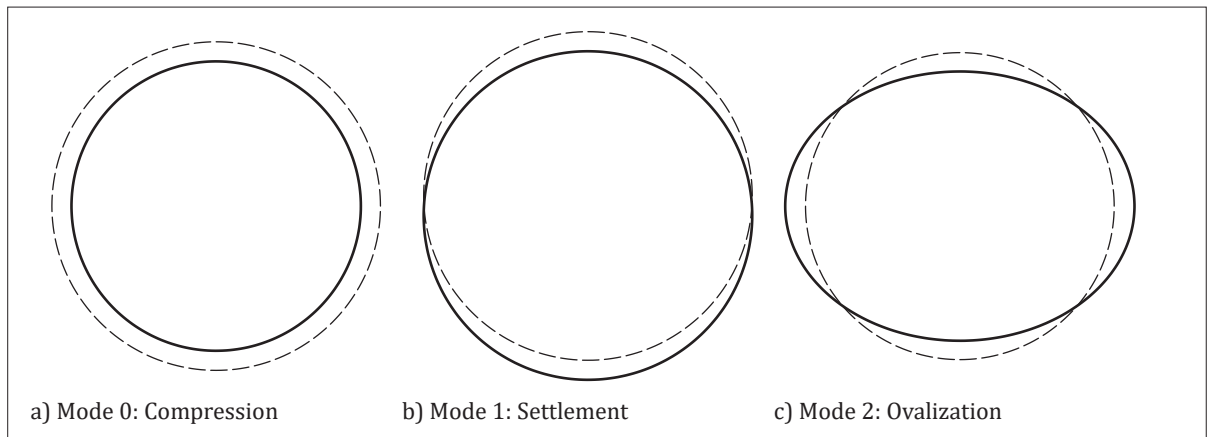
7.3.2 The pipe-soil system

In this method the buried pipe structure is referred to as the “pipe-soil system”, which is defined as a “structural unit comprising the buried pipe and an annulus of the surrounding soil, including pipe embedment and native ground, which interact to resist applied external and internal loads and/or imposed deformations”.

The size of soil annulus forming part of this composite structure varies according to the relative stiffnesses of the pipe and soil and the mode of their relative deformation. It represents the zone of soil in which the state of stress is influenced by structural interaction with the pipe. The important deformation modes of ovalization (mode 2) and compression (mode 0) of the pipe ring are illustrated in [Figure 24](#). This zone typically extends around one diameter from the pipe wall in all directions (see note). The entire soil surround both inside and outside this zone, including all backfill extending to the ground surface, additionally acts by its self-weight and consolidation characteristics as a vertical and horizontal loading medium.

NOTE For cases where another pipe(s) is present or planned within this zone of interaction, see [A.8.2](#) to [A.8.5](#).

Figure 24 — Principal deformation modes of the buried pipe ring



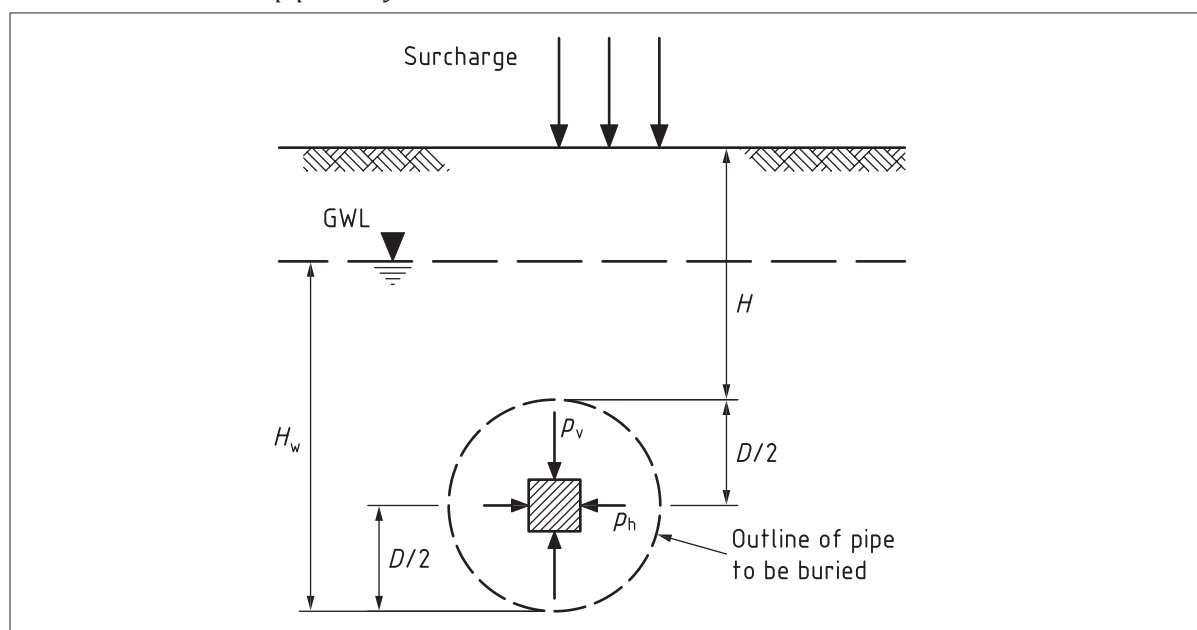
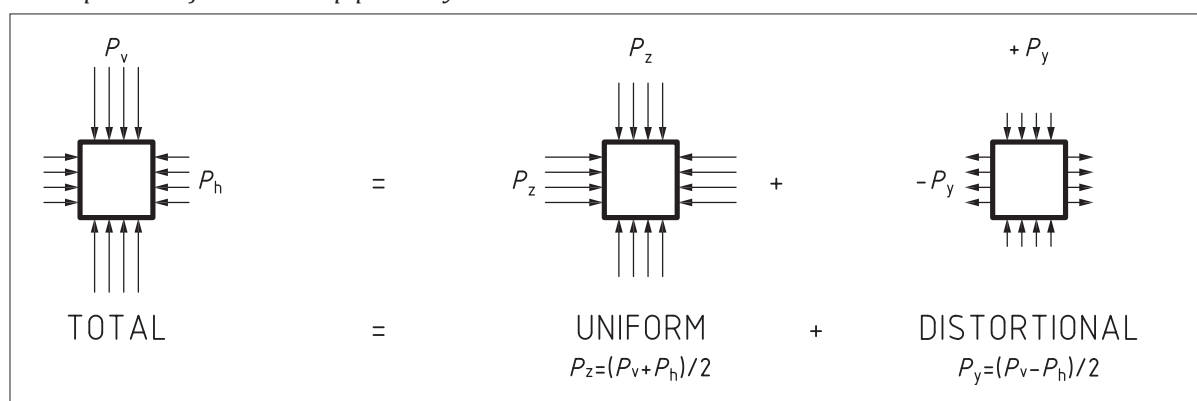
7.3.3 External loads on the pipe-soil system

An important feature of the Gumbel method which distinguishes it from the Marston-Spangler approach is the way in which external loads on the buried pipe structure are calculated. No attempt is made to estimate the soil loads acting directly on the outside surface of the pipe, which are a function of the relative stiffnesses and mode(s) of deformation of pipe and soil. Instead, the loads external to the structure are defined as the vertical and horizontal stresses that would act on an element of soil at the position of the centre line of the pipe if the trench or other method of construction were undertaken but the pipe itself omitted (see [Figure 25](#)). The relatively complex and statically indeterminate interaction forces between pipe and soil are then automatically accounted for by the theoretical model on which the various design formulae are based, and do not need to be explicitly considered.

Two small adjustments are made to this simple loading concept, as follows:

- a) although horizontal load on the pipe-soil system is always appropriately computed at pipe springing level, it is conservative to compute the weight of soil overburden on the pipe centre line; for pipes above a certain size the weight of soil displaced by the upper half pipe is therefore deducted from the vertical system load (see [7.3.5](#)); and
- b) due to its attenuation with depth, the vertical load due to surface surcharge is still conservatively calculated at pipe crown level in accordance with [Clause 5](#).

In the design of buried flexible pipes generally, as explained in [4.3](#), the effect on pipe-soil interaction forces due to settlement of the pipe into its bedding [Mode 1 relative deformation as depicted in [Figure 24b](#)] can safely be neglected. To compute the important pipe responses in the other two principal deformation modes of ring compression [Mode 0 as depicted in [Figure 24a](#)] and ring bending [Mode 2 as depicted in [Figure 24c](#)], the Gumbel method distinguishes between the respective uniform and distortional components of external load that give rise to these deformations. These components of system external load are defined by [Figure 26](#).

Figure 25 — External loads on the pipe-soil system**Figure 26** — Components of load on the pipe-soil system

7.3.4 Pipe properties

The properties that describe the pipe are as follows:

- a) plane strain modulus of pipe material in megapascals (MPa), E_p^*

$$E_p^* = \frac{E_p}{(1 - \nu_p^2)}$$

- b) flexural ring stiffness of the pipe in megapascals (MPa), S_y

$$S_y = \frac{E_p^* I}{(1000D)^3}$$

for solid-wall pipes and structured-wall pipes of BS EN 13476:2018, Type A; or

$$S_y = \frac{E_p I}{(1000D)^3}$$

for structured-wall pipes of BS EN 13476:2018, Type B.

- c) compression stiffness of the pipe ring in megapascals (MPa), S_z

$$S_z = \frac{E_p^* A}{1000D} \quad \text{for solid-wall pipes and structured-wall pipes of BS EN 13476:2018, Type A; or}$$

$$S_z = \frac{E_p A}{1000D} \quad \text{for structured-wall pipes of BS EN 13476:2018, Type B.}$$

where:

- E_p is the modulus of elasticity of the pipe material in megapascals (MPa);
- ν is Poisson's ratio of the pipe material (dimensionless);
- I is the second moment of area of the pipe wall per unit length (mm^4/mm);
- A is the cross-sectional area of wall per unit length (mm^2/mm); and
- D is the mean pipe diameter (measured to neutral axis of wall) in metres (m).

Both short-term and long-term values of pipe properties are required for design and are evaluated using the relevant short and long-term values respectively of the modulus of elasticity, E_p , or of the directly measured flexural ring stiffness S_y , in the formulae in a) to c) above. In accordance with 4.5.4, in the absence of product-specific test data from which the value of creep modulus or ring creep stiffness precisely matching the timeframe of each design load case considered can be derived, the manufacturer's declared values of short-term (0.6 min to 3 min), two-year and fifty-year ring stiffness from standard tests should be used, as indicated in 7.3.8.

NOTE Attention is drawn to expression of S_y in MPa, in contrast to conventional expression of (flexural) ring stiffness in kPa as the basis for nominal stiffness classes SN2, SN4, etc.

7.3.5 Soil properties

7.3.5.1 General

In the Gumbel method, the stiffness of the soil embedment is characterized by the combined constrained soil modulus, M_s , using design values published in North America by AWWA [9] and AASHTO [10], with the following adaptations:

- a) they are mapped to the embedment class values in Table 15;
- b) the degree of compaction is expressed as modified Proctor, M_p , density instead of standard Proctor; and
- c) any increase in M_s values due to confining pressure at larger depths of burial is conservatively neglected.

7.3.5.2 Constrained soil modulus, M_s

The value of M_s used in design is a combination of the constrained modulus of the pipe zone embedment, M_{sb} , with that of the native soil of the trench wall, M_{sn} . M_s is determined by equation (40).

$$M_s = S_c \times M_{sb} \quad (40)$$

where:

- M_s is the combined constrained modulus in megapascals (MPa);
- M_{sb} is the constrained soil modulus of the pipe zone embedment in megapascals (MPa) given in [Table 17](#) as a function of embedment class and degree of compaction (modified Proctor density, M_p) or ability to self-compact (see [A.22.2](#)); and
- S_c is the soil support combination factor (dimensionless) given in [7.3.5.4, Table 19](#) as a function of the ratio M_{sn}/M_{sb} , where M_{sn} is obtained from [7.3.5.3, Table 18](#).

Table 17 — M_{sb} values ^{A)}

Embedment class as in Table 15	Compacted density, M_p %	M_{sb} MPa
S1	Self-compacting ^{B)}	15
S2	80	3
	85	9
	90	15
S3	80	2.5
	85	5
	90	10

^{A)} Values adapted from McGrath “Replacing E' with the Constrained Modulus in Buried Pipe Design” [[11](#)].

^{B)} Single-sized granular material, with compaction fraction value of 0.15 or less, capable of filling space under haunches of pipe with low compactive effort.

7.3.5.3 M_{sn} values

Design values of constrained modulus for native soils, M_{sn} , are given in [Table 18](#) for granular and cohesive native soils and linked to values of SPT and c_u measured in accordance with BS 1377, respectively.

Table 18 — M_{sn} values

Description of soil	Design values	
Granular type soils	N_{SPT} (from SPT) Blows/300 mm	M_{sn} MPa
Very loose	> 0–1	0.34
	1–2	1.4
	2–4	4.8
Loose	4–8	10.3
Medium dense	8–15	20.7
	15–30	34.5
Dense	30–50	69
Very dense	>50	138
Cohesive type soils	c_u (kPa)	M_{sn} MPa
Very soft	0–13	0.34
	13–25	1.4
Soft	25–50	4.8
Firm	50–100	10.3
Stiff	100–200	20.7
Very stiff	200–400	34.5
Hard	400–600	69
	>600	138

NOTE 1 For embankment installation $M_{sb} = M_{sn} = M_s$.

NOTE 2 When permanent solid sheeting designed to last the life of the pipe is used in the pipe zone, M_s is based solely on M_{sb} .

7.3.5.4 Values for the soil support combination factor, S_c

The ratio M_{sn}/M_{sb} , as well as B_d/B_c , needs to be calculated so that the value of S_c can be derived.

In Table 19, B_d is the trench width and B_c is the outside diameter of the pipe. In-between values of S_c may be determined by linear interpolation from adjacent values.

Using the information in Table 19, the value of M_s can be obtained for the calculations.

Table 19 — Values of soil support combination factor, S_c

M_{sn}/M_{sb}	B_d/B_c							
	1.25	1.5	1.75	2	2.5	3	4	5
0.005	0.02	0.05	0.08	0.12	0.23	0.43	0.72	1.00
0.01	0.03	0.07	0.11	0.15	0.27	0.47	0.74	1.00
0.02	0.05	0.10	0.15	0.20	0.32	0.52	0.77	1.00
0.05	0.10	0.15	0.20	0.27	0.38	0.58	0.80	1.00
0.1	0.15	0.20	0.27	0.35	0.46	0.65	0.84	1.00
0.2	0.25	0.30	0.38	0.47	0.58	0.75	0.88	1.00
0.4	0.45	0.50	0.56	0.64	0.75	0.85	0.93	1.00
0.6	0.65	0.70	0.75	0.81	0.87	0.94	0.98	1.00
0.8	0.84	0.87	0.90	0.93	0.96	0.98	1.00	1.00
≥1	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00

SOURCE: Adapted from AWWA M45, *Fiberglass Pipe Design*, Third Edition [9].

7.3.6 Relative stiffness parameters

Following the definition and evaluation of pipe and soil properties, some important ratios for the evaluation of the structural design of the pipe are defined. The distribution of loads within the composite pipe-soil system is governed by two relative stiffness parameters:

- a) Flexural stiffness ratio, Y

$$Y = \frac{M_s}{S_y}$$

- b) Compression stiffness ratio, Z

$$Z = \frac{M_s}{S_z}$$

Both the short-term and long-term values of flexural and compression stiffness ratios are required for design and are evaluated using the short and long-term values respectively of S_y and S_z in the formulae in a) and b).

NOTE The behaviour of the pipe-soil system can be usefully described in terms of flexural stiffness ratio.

7.3.7 Computation of external system loads

The uniform and distortional components of external system load shown in [Figure 26](#) are determined by first computing the vertical and horizontal load components in accordance with 1) to 5) below. These load components are determined either as soil total stresses or, to take full account of any groundwater table above the pipe, as the combination of groundwater pressure and soil effective stresses.

The various components of external system load are then computed as follows:

- 1) Vertical pressure (soil total stress) at mid-height of the pipe in kilonewtons per square metre (kN/m^2), P_v , is given by:

$$P_v = P_e + P_s$$

where:

P_e is the vertical soil pressure in kilonewtons per square metre (kN/m^2), computed to take account of the weight of soil displaced by the upper half the pipe as:

$$P_e = \gamma(H + 0.11B_c)$$

where:

γ is the unit weight of soil in kilonewtons per cubic metre (kN/m^3); and

NOTE In the absence of specific data supporting the use of other values, it is normal practice to assume $\gamma = 19.6 \text{ kN/m}^3$.

P_s is the vehicle surcharge pressure in kilonewtons per square metre (kN/m^2), conservatively computed at pipe crown level in accordance with [Clause 5](#).

2) Effect of groundwater at mid-height of the pipe

Where applicable, the hydrostatic pressure due to groundwater at mid-height of the pipe in kilonewtons per square metre (kN/m^2), P_w , is given by:

$$P_w = \gamma_w (H_w - B_c / 2)$$

where:

γ_w is the unit weight of water in kilonewtons per cubic metre (kN/m^3); and

H_w is the height of water table above invert of the pipe in metres (m) (see [Figure 25](#)).

Except where the topography and other conditions make this unrealistic, the groundwater level should be assumed to be at ground level for this calculation.

3) The vertical pressure expressed as a soil effective stress, P'_v , is then given by:

$$P'_v = P_v - P_w$$

4) The horizontal pressure expressed as a soil effective stress, P'_h , at mid-height of the pipe is given by:

$$P'_h = K [\gamma(H + 0.5B_c) - P_w]$$

from which the horizontal pressure expressed as soil total stress, P_h , at mid-height of the pipe is obtained as:

$$P_h = P'_h + P_w$$

where:

K is the coefficient of lateral earth pressure applicable to soil effective stresses.

Values of K for design are presented in [Table 20](#). For uncompacted (loose) backfill, K values could range from a fully active value, K_a , to an at-rest value, K_o . For uncompacted soils from embedment class S3, the lowest probable values have been selected to allow for the possible presence of voids. For compacted (medium/dense) backfill, the lowest probable values correspond to the at-rest K_o value. Low values of K imply more distortional loading conditions, which in turn result in increased load-dependent deflection and flexural stress in the pipe.

Table 20 — K values for design

Embedment class	Assumed state of backfill	
	Uncompacted	Compacted
S1	0.4	0.4 ^{A)}
S2	0.3	0.5
S3	0.2	0.5

^{A)} Self-compacted.

Source: Gumbel et al. [\[12\]](#).

- 5) Finally, the uniform and distortional components of external load, expressed as both total and effective stresses acting externally on the pipe-soil system, are obtained as follows.

- i) Total stresses:

$$P_z = (P_v + P_h)/2$$

$$P_y = (P_v - P_h)/2$$

where:

P_z is the uniform pressure component of soil total stress in kilonewtons per square metre (kN/m²); and

P_y is the distortional pressure component of soil total stress in kilonewtons per square metre (kN/m²).

- ii) Effective stresses:

$$P'_z = P_z - P_w; \text{ and}$$

$$P'_y = P_y.$$

where:

P'_z is the uniform pressure component of soil effective stress in kilonewtons per square metre (kN/m²); and

P'_y is the distortional pressure component of soil effective stress in kilonewtons per square metre (kN/m²).

Groundwater can have either adverse or beneficial effects under different temporary or permanent load conditions. In the absence of detailed knowledge of the groundwater regime over the life of the pipeline, the following is recommended:

- for computation of the distortional (out-of-round) component of ring deflection, δ_y , and the associated maximum bending tensile strain, $\epsilon_{t,max}$, assume loading with groundwater table below pipe invert;
- for computation of the uniform (compressive) component of ring deflection, δ_z , and maximum compressive ring thrust N , assume groundwater table at ground level; and
- for computation of flotation forces, assume groundwater table at ground level.

7.3.8 Design limit states of the pipe ring

7.3.8.1 General

The principal performance criteria for design of the buried flexible pipe system are as follows.

- Ring deflection (see 7.3.8.2):** defined as the relative change, δ , of pipe diameter. It is limited to an allowable value in any direction, determined chiefly from serviceability considerations.
- Linear buckling (see 7.3.8.3):** defined as out-of-round bending deformation of the pipe ring in mode 2 or higher, due to the action of local compressive thrust, independent of any ring compressive strain, leading to reversal of curvature or snap-through at any point on the pipe circumference. Linear buckling is treated as an ultimate limit state (ULS) irrespective of whether collapse of the pipe ensues.

- c) **Ring strength (see 7.3.8.4):** ULS of material failure of the pipe wall due to combined hoop compression and bending stress or strain.

7.3.8.2 Ring deflection

The vertical deflection, δ_v , of the pipe ring relative to the pipe diameter, commonly expressed as a percentage and equivalent to the “ovalization” defined in the Marston-Spangler method [see equation (35)], is given by:

$$\delta_v = \frac{\Delta_v}{D} \quad (41)$$

where:

Δ_v is the absolute vertical deflection (in millimetres), taking account of both uniform and distortional effects.

Relative vertical deflection comprises uniform and distortional components:

$$\delta_v = \delta_z + \delta_y$$

of which the uniform component, δ_z , is given by:

$$\delta_z = \frac{\alpha_z \times P'_z + P_w}{2 S_z 1000}$$

where the radial arching factor, α_z , is given by:

$$\alpha_z = \frac{4}{3 + Z}$$

where:

Z is the long-term compression stiffness ratio defined in 7.3.6.

The long-term (two-year, or by default fifty-year) value of the compressive ring stiffness, S_z , of the pipe, and hence of stiffness ratio Z and arching factor α_z , should be used for the determination of δ_z .

δ_z can be appreciable and exceed the limit for leak tightness of spigot and socket joints.

The distortional or total out-of-round component of deflection, δ_y , is determined as follows:

$$\delta_y = \delta_{y0} + \delta_{y1} + \delta_{y2}$$

where:

δ_{y0} is the initial out-of-round deflection of the pipe during construction of the embedment (in the absence of specific data, default values of δ_{y0} for design are given in terms of percentage of the mean diameter of the pipe in Table 21 as a function of stiffness class and installation quality);

δ_{y1} is the out-of-round deflection due to weight of main backfill and any surcharges calculated as follows:

$$\delta_{y1} = \frac{4P_y}{(108 S_y + M_s) \times 1000}$$

in which the long-term (two-year, or by default fifty-year) value of the flexural ring stiffness, S_y , of the pipe should be used; and

δ_{y2} is the out-of-round deflection of the pipe due to long-term consolidation settlement of the embedment given in terms of percentage of the mean diameter of the pipe in [Table 22](#).

Table 21 — *Design values of initial deflection, δ_{y0}*

Ring ^{A)} stiffness (kN/m ²)	Initial deflection, δ_{y0}		
	Installation control Type A ^{B)}	Installation control Type B ^{C)}	Installation control Type C ^{D)}
2	1.5%	2.5%	5%
4	1.0%	1.5%	4%
6	0.75%	1.25%	3.5%
8	0.5%	1.0%	3%
9 and above	0.0%	0.5%	1%

^{A)} For intermediate values of ring stiffness up to 10 kN/m², interpolated values may be used. For ring stiffnesses less than 2 kN/m², only type A installation is recommended.

^{B)} Type A

Installation is monitored and quality-assured by the pipe manufacturer or the asset owner's appointed agent

S1 or S2 embedment class

^{C)} Type B

Unmonitored installation

S1 embedment class

or

Installation is monitored and quality-assured by the installation contractor

S2 embedment class

or

Installation is monitored and quality-assured by the pipe manufacturer or the asset owner's appointed agent

S3 embedment class

^{D)} Type C

Unmonitored installation of S2 embedment class

All installations are of S3 to S5, B1 and B2 embedment class

Table 22 — Values of δ_{y2} for design

Embedment class	Compacted density, M_p	δ_{y2}
	%	%
S1	–	0
S2	80	3
	85	2
	90	1
S3	80	4
	85	3
	90	2

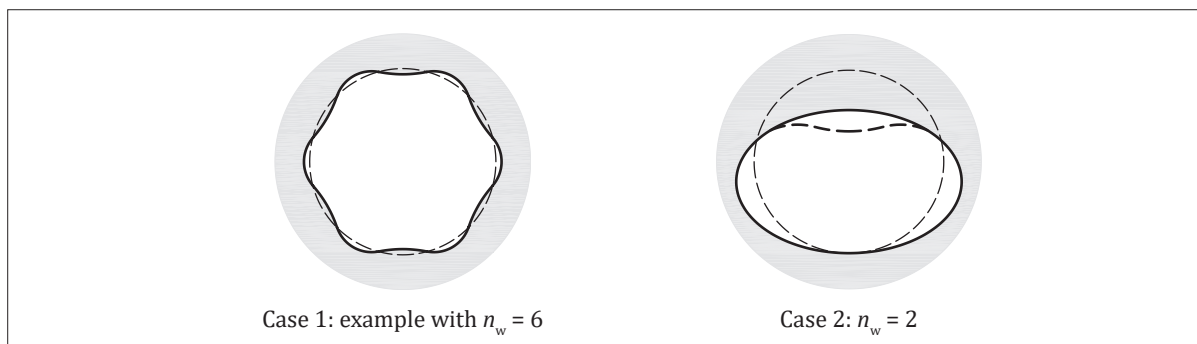
The overall ring deflection limit is a function of the jointing method used for the pipe system and other serviceability criteria. For push-fit joints the vertical ring deflection of the socket is limited to maintain leak tightness. The value of this deflection limit determined by product type testing varies according to the type of pipe and its jointing system, and typically ranges from 3% to 6%. For systems with welded joints, the limit on vertical deflection is generally 8%.

7.3.8.3 Linear buckling

Two modes of linear buckling are illustrated in [Figure 27](#) and are distinguished as follows:

- Case 1: Multi-wave mode, with $n_w > 2$, which is not sensitive to out-of-roundness; and
- Case 2: Elliptical mode of buckling, with $n_w = 2$, which is sensitive to out-of-roundness of the pipe, where:

n_w is the number of waves around circumference of pipe.

Figure 27 — Linear buckling modes

For each of the two cases in [Figure 27](#), the critical linear buckling pressure in kilonewtons per square metre (kN/m^2), P_{cr} , can be obtained from:

$$P_{cr_case1} = 0.945(M_s)^{2/3}(S_y)^{1/3} \times 1\,000$$

$$P_{cr_case2} = 24qS_y + \frac{k_{sc}M_s}{18} \times 1\,000$$

where:

q is the out-of-roundness factor:

$$q = \left[\frac{1 - \delta_y}{(1 + \delta_y)^2} \right]^3$$

k_{sc} is the shallow cover reduction factor on soil modulus (applicable in the Case 2 buckling formula only), for which design values are given as a function of cover to outside diameter ratio H/B_c in [Table 23](#).

Table 23 — Values of k_{sc} for design

H/B_c	0.5	0.6	0.7	0.8	0.9	1.0	≥ 1.4
k_{sc}	0.80	0.85	0.89	0.92	0.94	0.96	1

NOTE 1 For intermediate values of H/B_c , values of k_{sc} may be interpolated.

NOTE 2 These design values of k_{sc} apply for $H \geq 0.9$ m only.

For the determination of P_{cr} both the short-term and long-term value of the flexural ring stiffness of the pipe should be used in conjunction with the relevant factors of safety.

The factor of safety, F_s , against linear buckling is defined for the following two scenarios, taking account of the maximum external pressure in the pipe-soil system. The factor of safety against buckling should be not less than two.

a) Scenario 1: long-term stability

$$F_s = \frac{P_{cr \text{ case1}}}{P_e}$$

$$F_s = \frac{P_{cr \text{ case2}}}{P_e}$$

NOTE 1 For scenario 1, the long-term value of the flexural ring stiffness of the pipe is used.

b) Scenario 2: short-term stability

$$F_s = \frac{P_{cr \text{ case1}}}{P_e + P_s + P_u}$$

$$F_s = \frac{P_{cr \text{ case2}}}{P_e + P_s + P_u}$$

where:

P_u is the value of any short-term vacuum pressure present (see [7.2.7](#)).

NOTE 2 For scenario 2, the short-term value of the flexural ring stiffness of the pipe is used.

7.3.8.4 Ring strength

7.3.8.4.1 Ring thrust in pipe wall, N

The ring thrust in the pipe wall, N , has uniform and distortional components. The uniform component of ring thrust, N_z , is given by:

$$N_z = (\alpha_z \times P'_z + P_w) 7 \times B_c / 2$$

The value of the distortional component of ring thrust, N_y , is given in kilonewtons per metre (kN/m) by:

$$N_y = \frac{4 \times (144 + Y) \times P'_y}{360 + \left[(7 + Z) \times \left(\frac{Y}{2} \right) \right]} \times \frac{B_c}{2}$$

The maximum and minimum wall thrusts around the pipe perimeter, which generally occur at the springings and crown or invert respectively (at least where $P_v > P_h$), are then given by:

$$N_{\max} = N_z + N_y; \text{ and}$$

$$N_{\min} = N_z - N_y$$

7.3.8.4.2 Ring compressive stresses in pipe wall, σ_z

The mean ring compressive stress in the pipe wall at any point on its perimeter, expressed in megapascals (MPa), σ_z , is given by:

$$\sigma_z = \frac{N}{A}$$

where N is the local value of ring thrust.

The maximum value of σ_z generally occurring at the pipe springings is given by:

$$\sigma_{z,\max} = N_{\max} / A$$

As the ring thrust in a buried flexible pipe is greatest in the short term, before any stress-relieving creep of the pipe material takes place, the value of $N_{\max}$ in the above formula for $\sigma_{z,\max}$ should be determined using the short-term values of Y and Z in the underlying formula for N_y .

NOTE For polyethylene pipes of grade PE80 or PE100, a design limit state value of 8.0 MPa may be assumed for compressive strength. For other materials, specific data need to be sought.

7.3.8.4.3 Ring compressive strain, ε_z

For solid-wall pipes, and structured-wall pipes of BS EN 13476:2018, Type A, the mean ring compressive strain at any point in the pipe wall, ε_z , is given by:

$$\varepsilon_{z,\max} = \frac{N}{E_p^* A}$$

For the determination of strain, the long-term values of both ring thrust, N , and pipe plane strain modulus, E_p^* , should be used.

The maximum value of ε_z generally occurring at the pipe springings is given by:

$$\varepsilon_z = \frac{N_{\max}}{E_p^* A}$$

The minimum value of ε_z generally occurring at the pipe crown or invert is given by:

$$\varepsilon_{z,\min} = \frac{N_{\min}}{E_p^* A}$$

For structured-wall pipes of BS EN 13476:2018, Type B, substitute Young's modulus, E_p , for plane strain modulus, E_p^* , in the above formulae for compressive strain, ε_z .

7.3.8.4.4 Ring bending strain, ε_y

The maximum ring bending strain in the pipe wall, ε_y , is given by:

$$\varepsilon_y = D_f \delta_y (t/D)$$

where:

- D_f is the strain factor obtained as a function of short-term flexural ring stiffness, S_y , from [Table 14](#);
- δ_y is the long-term ring deflection; and
- t is the wall thickness of solid-wall pipe or, in the case of structured-wall pipe with an asymmetric profile, twice the largest distance from the neutral axis of the wall section to the inside or outside surface of the pipe, as applicable.

7.3.8.4.5 Combined strain

The maximum compressive and tensile fibre strains in the pipe wall due to combined thrust and bending are given by:

- a) maximum compressive fibre strain $\varepsilon_{c,\max} = \varepsilon_y + \varepsilon_{z,\max}$; and
- b) maximum tensile fibre strain $\varepsilon_{t,\max} = \varepsilon_y - \varepsilon_{z,\min}$.

In structured-wall pipes with asymmetric wall profile, the ε_y used in the expression for $\varepsilon_{c,\max}$ should be calculated on the outside surface of the pipe, whereas in the expression for $\varepsilon_{c,\min}$ it should be calculated on the inside surface of the pipe.

In the absence of detailed material information, the long-term strain limits that should be assumed for design purposes for polyethylene are 5% in compression and 2.5% in tension. Appreciably higher values may be used where supported by appropriate testing. For other materials, specific data need to be obtained.

8 Semi-rigid pipe design

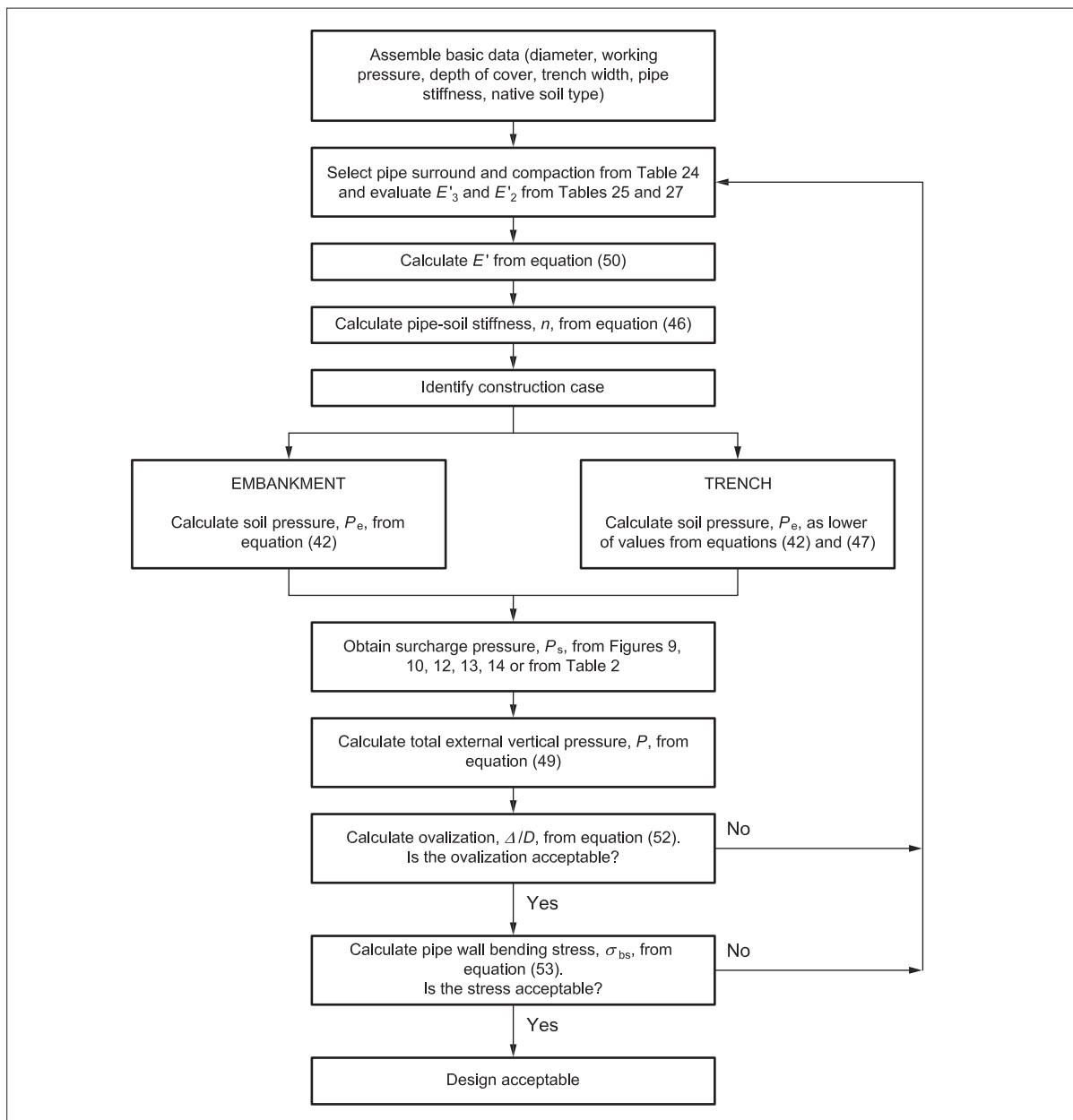
COMMENTARY ON **CLAUSE 8**

This clause gives the design formulae referred to in [Figure 28](#) which sets out design procedures for semi-rigid pipes of thick-walled steel and ductile iron by the Marston-Spangler method only. The design of polymer concrete pipes taking account of long-term creep effects is outside the current scope. The Gumbel method is also potentially applicable to semi-rigid pipe design, but this again is not considered in the present edition of this standard.

8.1 General

Semi-rigid pipes are those that display elements of both rigid and flexible pipe behaviour, i.e. they both deflect and settle in response to loading. Typically, the pipe materials that fall into this category are ductile iron and thicker walled steel.

Figure 28 — Flowchart for design of semi-rigid pipes of ductile iron

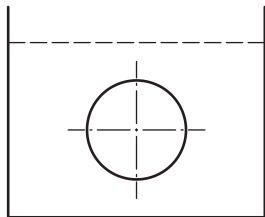
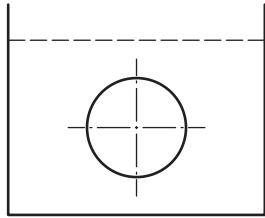
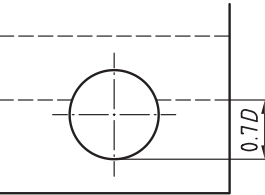
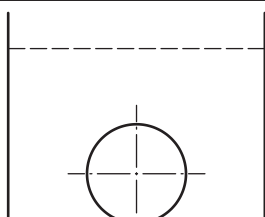


8.2 Installation conditions

There are two installation conditions for pipes: in wide trenches or embankments and in narrow trenches. For all pipe installations the loads arising from both conditions should be calculated for the selected maximum trench width and the lower of the two vertical soil pressures, $P_{e,wide}$ and $P_{e,narrow}$, used in the design. In the absence of specific information, a wide trench should be assumed.

Semi-rigid pipe embedment classifications are given in [Table 24](#).

Table 24 — *Semi-rigid pipe embedments*

Embedment class	Embedment configuration	Bed and sidefill materials	Notes
S1 and S2		Class S1: Gravel (single size) Class S2: Gravel (graded)	Normally processed granular materials
S3 to S5		Class S3: Sand and coarse-grained soil with less than or equal to 12% fines. Class S4: Coarse-grained soil with more than 12% fines. OR Fine-grained soil, liquid limit less than 50%, medium-to-no plasticity and more than 25% coarse-grained material.	These represent “as-dug” soils and require particularly close control when used with low stiffness pipes.
		Class S5: Fine-grained soil, liquid limit less than 50%, medium-to-no plasticity and less than 25% coarse-grained material.	Class S5 only recommended for use with semi-rigid pipes.
B1 and B2		Class B1: Upper surround as for S3 or S4 Lower surround as for S1 or S2 Class B2: Upper surround as for S5 Lower surround as for S1 or S2	Class B embedments not recommended for use with pipes of less than 10 kN/m ² stiffness.
D		Classes S1 to S5	Only suitable for semi-rigid pipes with high beam strength. Soil properties from Table 25 , but with $K_x = 0.110$ for all embedment classes.

NOTE See [Table 25](#) for design parameters for embedment classes.

Table 25 — *Semi-rigid and flexible pipe embedment properties*

Embedment class as in Table 24 and deflection coefficient K_x	Compacted density, M_p (See Note 3) %	Embedment soil modulus, E'_2 MN/m ²	Deflection lag factor, D_L ^{a)}	Strain factor, D_p for various pipe ring stiffnesses ^{b)} kN/m ²					
				1.25	2.5	5.0	10	15	30 or more
Class S1 $K_x = 0.083$	Uncompacted	5	1.5	4.7	4.5	4.3	4.0	3.75	3.0
	Self-compacted ^{c)}	10	1.0	4.7	4.5	4.3	4.0	3.75	3.0
Class S2 $K_x = 0.083$	Uncompacted	3	1.5	4.7	4.5	4.3	4.0	3.75	3.0
	80	5	1.25	4.7	4.5	4.3	4.0	3.75	3.0
	85	7	1.0	4.7	4.5	4.3	4.0	3.75	3.25
	90	10	1.0	4.7	4.5	4.3	4.0	3.75	3.5
Class S3 $K_x = 0.100$	85	5	1.5	6.2	5.5	4.75	4.25	4.0	3.25
	90	7	1.25	7.75	6.6	5.5	4.7	4.25	3.5
Class S4 $K_x = 0.100$	85	3	1.5	6.2	5.5	4.75	4.25	4.0	3.5
	90	5	1.25	7.75	6.6	5.5	4.7	4.25	3.5
Class S5 $K_x = 0.100$	85	1	3.0	—	—	—	—	4.0	3.5
	90	3	2.0	—	—	—	—	4.25	3.5
Class B1 $K_x = 0.083$	85	5	1.5	—	—	—	5.0	4.0	3.5
	90	7	1.25	—	—	—	5.5	4.25	3.5
Class B2 $K_x = 0.083$	85	3	2.0	—	—	—	5.5	4.25	3.5
	90	5	1.75	—	—	—	6.0	5.0	3.5

NOTE 1 For construction details of embedment classes see [Table 24](#).

NOTE 2 Quoted values of E'_2 assume pipe is to be installed below groundwater.

NOTE 3 M_p indicates modified Proctor density and corresponds to the heavy compaction test in BS 1377.

^{a)} Where the designer can be certain that initial pressurization of at least 3 bar (1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 10⁵ N/m²) will take place within one year of backfilling and the depth of cover does not exceed 2.5 m, a value of 1.0 may be taken for the deflection lag factor.

^{b)} Pipe stiffnesses are initial values.

^{c)} Single-sized granular material capable of filling space under haunches of pipe with low compactive effort.

8.3 Design pressures for semi-rigid pipes

8.3.1 Vertical soil pressure, $P_{e,wide}$ in embankment or wide trench conditions

Vertical soil pressure, $P_{e,wide}$ in embankment or wide trench conditions is obtained from:

$$P_{e,wide} = C_c \gamma B_c \quad (42)$$

where C_c is taken as the lower of the values derived from equations (43) and (44).

$$C_c = \frac{e^{2K\mu H/B_c} - 1}{2K\mu} \quad (43)$$

$$C_c = \left[\frac{e^{2K\mu H_e/B_c} - 1}{2K\mu} + \left(\frac{H}{B_c} - \frac{H_e}{B_c} \right) e^{2K\mu H_e/B_c} \right] \quad (44)$$

where H_e is the height of the plane of equal settlement above the top of the pipe. Derivation of H_e is complex, but [Table 26](#) provides simplified equations for obtaining C_c .

Equation (44) is solved using [Table 26](#), with the settlement deflection ratio, r_{sd} , determined as follows:

$$r_{sd} = 0.7 (1 - n) \quad (45)$$

where the value of n is derived from equation (46).

$$n = \frac{E' / D_L}{(105EI / D^3) + (0.8E' / D_L)} \quad (46)$$

Table 26 — Load coefficients for incomplete projection condition

$r_{sd}\rho$	Equation for C_c
0	$C_c = H/B_c$
+0.1	$C_c = 1.23 (H/B_c) - 0.02$
+0.3	$C_c = 1.39 (H/B_c) - 0.05$
+0.5	$C_c = 1.50 (H/B_c) - 0.07$
+0.7	$C_c = 1.59 (H/B_c) - 0.09$
+1.0	$C_c = 1.69 (H/B_c) - 0.12$
+2.0	$C_c = 1.93 (H/B_c) - 0.17$
+3.0	$C_c = 2.08 (H/B_c) - 0.20$
+4.0	$C_c = 2.19 (H/B_c) - 0.21$
+5.0	$C_c = 2.28 (H/B_c) - 0.22$

When the pipe behaves as a rigid pipe (i.e. $n = 0$) then $r_{sd} = 0.7$, a value for a rigid pipe on a normal foundation. When the pipe is fully flexible ($n = 1$) then $r_{sd} = 0$ and the vertical pressure is the geostatic pressure, as used in flexible pipe design.

If the lower value is given by equation (43), complete projection conditions exist, whilst if the lower value is given by equation (44), incomplete projection conditions exist. Values for the projection ratio, ρ , are given in [6.5](#). An alternative method to using equation (44) for incomplete projection conditions is to use equation (A.10) in [Annex A: A.24](#).

The full formulae from which the equations in [Table 26](#) are derived are given in the TRRL's *Guide to design loadings for buried rigid pipes* [N1], Clauses 2.4 and 2.5, assuming $K\mu = 0.19$ (see [6.6](#)).

8.3.2 Vertical soil pressure, P_e , in the narrow trench condition

The design value of vertical soil pressure, $P_{e,narrow}$, is the lower of the values obtained from equation (42) and equation (47):

$$P_{e,narrow} = \frac{C_d \gamma B_d^2}{nB_d + (1-n)B_c} \quad (47)$$

with the value of C_d obtained from equation (48).

$$C_d = \frac{1 - e^{-2K\mu'H/B_d}}{2K\mu'} \quad (48)$$

8.3.3 Total vertical external pressure, P

Total vertical external pressure, P , is obtained from:

$$P = P_e + P_s \quad (49)$$

where P_s is the surcharge pressure due to vehicle loadings calculated in accordance with [Clause 5](#).

8.3.4 Deflection lag factor

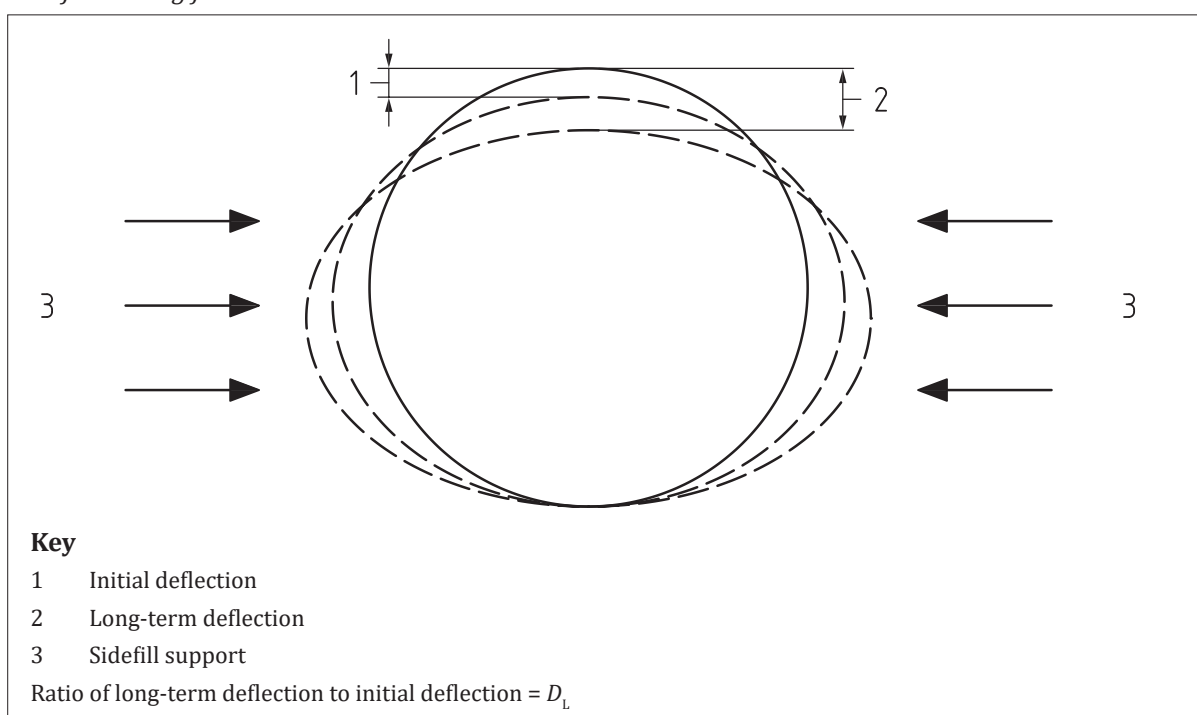
The deflection lag factor, D_L , takes account of the relaxation of the pressure of the sidefill over time resulting in further deflection of the pipe (see Figure 29).

D_L gives an indication of the degree of compaction that occurs during the installation for a given embedment. It can be seen from Table 25 that the greater the level of compaction, the lower the deflection lag factor. As the embedment becomes less favourable (for example, classes S5, B1 and B2), the harder it is to achieve a low deflection lag factor.

The value of D_L is modified to D_{LSR} when used in the semi-rigid ovalization analysis and leads to a ratio of initial to long-term deflection that is much lower than that for a flexible pipeline.

Where the pipeline is to be pressurized to at least 3 bar⁵⁾ working pressure within one year of burial, at a depth of less than 2.5 m, D_L can be taken as 1, since the internal pressure has the beneficial effect of preventing the long-term deflection from taking place.

Figure 29 — Deflection lag factor



8.3.5 Relative pipe-soil stiffness factor

The relative pipe-soil stiffness factor, n , is obtained from equation (46) by taking account of the pipe surround material and its degree of compaction. To calculate this value, the deflection lag factor, D_L , needs to be selected from Table 25 for the intended embedment and its compaction.

Generally, n has a value between 0 and 1. If the minimum pipe stiffness (EI/D^3) is much greater than the soil stiffness (E'/D_L) then n tends to 0. Conversely, if the pipe stiffness is much lower than the soil stiffness then n tends to 1.25. For design purposes the maximum value of n is usually limited to 1. The minimum pipe stiffness (EI/D^3) values for ductile iron pipes are given in BS EN 545 and BS EN 598.

Design values of E' are obtained using Table 25, Table 26 and Table 27, and equation (50) and equation (51). Design values of D_L are obtained using Table 25.

⁵⁾ 1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 10⁵ N/m²

Table 27 — *Guide values of Spangler modulus for native soils, E'_3*

Soil type	Spangler modulus for soils in various conditions (MN/m ²)				
	Very dense	Dense	Medium dense	Loose	Very loose
Gravel	Over 40	15 to 40	9 to 15	5 to 9	3 to 5
Sand	15 to 20	9 to 15	4 to 9	2 to 4	1 to 2
Clayey, silty sand	10 to 15	6 to 10	2.5 to 6	1.5 to 2.5	0.5 to 1.5
Clay	Very hard	11 to 14			
	Hard	10 to 11			
	Very stiff	6 to 10			
	Stiff	4 to 6			
	Firm	3 to 4			
	Soft	1.5 to 3			
	Very soft	0 to 1.5			

NOTE Further guidance on the use of lightweight aggregates is given in [A.11.4](#).

8.4 Semi-rigid pipe embedment properties

8.4.1 Embedment modulus of soil reaction, E'_2

The embedment modulus of soil reaction, E'_2 , for the selected pipe surround material at the chosen level of compaction is obtained from [Table 25](#). Guidance on evaluation of the corresponding modulus for the native soil, E'_3 , is given in [Table 27](#), taking account of site investigation data. The effective overall modulus, E' , can then be obtained using equation (50) for a particular trench width.

$$E' = E'_2 C_L \quad (50)$$

where Leonhardt's coefficient, C_L , is calculated from equation (51) to obtain the effective pipe-soil stiffness factor:

$$C_L = \frac{0.985 + (0.544 B_d / B_c)}{[1.985 - 0.456 (B_d / B_c)] (E'_2 / E'_3) - [1 - (B_d / B_c)]} \quad (51)$$

NOTE If the trench width is more than 4.3 times the external pipe diameter, the value of E' is equal to the value of E'_2 . C_L has an upper limiting value of 1.

8.4.2 Ovalization or ring deflection, Δ/D

The ovalization or ring deflection of the pipeline, Δ/D , takes account of the backfill and traffic load placed upon it, which is resisted by the pipe stiffness and soil stiffness. Semi-rigid pipe materials exhibit different ovalization characteristics according to their stiffness. For metallic pipes, no creep of the material occurs, so the Young's Modulus value, E , used in the stiffness calculation is constant. However, for plastics pipes, a short-term value is used for the initial deflection, but when calculating long-term deflection, the creep factor of the material needs to be applied to the Young's Modulus value.

Ovalization also takes account of the beneficial re-rounding effect that comes from the presence of water pressure within the pipeline. This effect is assumed to be present when the depth of cover to the pipeline does not exceed 2.5 m. If the pipeline is buried at a greater depth, the backfill pressure will be too great to allow re-rounding to occur.

The semi-rigid version includes the modified deflection lag factor, D_{LSR} , which is used in equation (52) for ovalization.

$$\frac{\Delta}{D} = \frac{K_x (D_{\text{LSR}} P_e + P_s)}{8EI / D^3 + 0.061E'} \quad (52)$$

NOTE The initial deflection is obtained with the value of D_{LSR} set at 1, and the long-term deflection with $D_{\text{LSR}} = 1 + 0.8n(D_L - 1)$, where D_L is the value obtained from [Table 25](#). Where the working pressure exceeds 3 bar⁶⁾ and the depth of cover is less than 2.5 m, the long-term deflection can be reduced by the factor $D_R = 1 - (P_i/40)$, where P_i is the internal pressure in bars.

8.4.3 Strain factor, D_f

The strain factor, D_f , is used in [8.4.4](#) to determine bending stress, and values are shown in [Table 25](#) for different pipe stiffness and embedments. These demonstrate that the higher the modulus of soil reaction and the stiffer the pipe, the lower the value of the strain factor. This is because less compactive energy is required to achieve the same embedment modulus. It is very important when choosing a pipe stiffness and embedment to take account of the amount of compaction that will be required in the field to achieve the desired modulus of soil reaction.

8.4.4 Bending stress, σ_{bs}

The bending stress in the pipe walls due to the external load on the pipeline, σ_{bs} , is calculated using equation (53) to ensure that the allowable bending stress is not exceeded. In the case of ductile iron pipes, the allowable ovalization values are given in BS EN 545 and BS EN 598, and these limit the bending stresses.

$$\sigma_{\text{bs}} = ED_f(\Delta/D)(t/D) \quad (53)$$

8.5 Design data for semi-rigid pipes

It is normal practice to assume the following:

- a) $K\mu = 0.19$; and
- b) $K\mu' = 0.13$.

Values of the deflection coefficient, K_x , deflection lag factor [modified to D_{LSR} using the information in the note to [8.4.2](#)], D_L , and strain factor, D_f , for use in equation (52) and equation (53), can be obtained from [Table 25](#).

8.6 Bedding for ductile iron pipes

8.6.1 Pipe details

Water pipes with flexible joints should conform to BS EN 545. DN80 to DN300 pipes should be class 40, and DN350 to DN600 pipes should be class 30. Pressure sewer pipes with flexible joints should conform to BS EN 598.

8.6.2 Embedment classes

Embedment classes are defined in [Table 24](#), and those relevant to the embedment tables for ductile iron pipe are reproduced in [Table 28](#).

⁶⁾ 1 bar = 100 kPa = 0.1 MPa = 0.1 N/mm² = 10⁵ N/m².

Table 28 — *Embedment classes*

Class	Component	Materials
S1	Bed and sidefill	Gravel (single size)
S2		Gravel (graded)
S3		Sand and coarse-grained soil with <12% fines
S4		Coarse-grained soil with >12% fines
		OR Fine-grained soil, liquid limit <50%, medium-to-no plasticity and >25% coarse-grained material
S5		Fine-grained soil, liquid limit <50%, medium-to-no plasticity and <25% coarse-grained material
D	Bed	Trench bottom with joint holes to ensure that the barrel is fully supported
	Sidefill	S3 to S5

8.7 Simplified bedding table for ductile iron pipes

8.7.1 Native soil embedment

The simplified embedment tables ([Table 29](#) to [Table 32](#)) have been derived using class D with S3, S4 and S5, but they can also be used for classes S3, S4 and S5. An additional embedment, “Poor soils”, has been included as these are not covered in [Table 24](#). When installing pipes in poor soils, the guidance in [A.10](#) should be followed. For native gravels, embedment class D/S3 should be used.

8.7.2 Imported embedments

The imported embedments (classes S1 and S2) have not been evaluated individually in the embedment tables but have been combined using a modulus of soil reaction, E'_2 , that is the lower of classes S1 and S2 at a particular value of compaction.

8.7.3 Native modulus of soil reaction

The modulus of soil reaction for native soils, E'_3 , is given in [Table 27](#). For native embedments the modulus is taken as that for the embedment class at 85% modified Proctor, M_p , density. For “Poor soils” the overall modulus for the embedment, E' , is taken as zero, i.e. the pipe can withstand the soil and vehicle loads without any lateral support from the embedment. For imported embedments it is assumed that $E'_3 = 1 \text{ MN/m}^2$. This value compares with, for example, the range 0 MN/m² to 1.5 MN/m² for very soft clay.

8.7.4 Trench conditions

The trench widths, B_d , are expressed in [Table 29](#) to [Table 32](#) relative to the external diameter, B_c , of the pipe, i.e. in terms of the difference $B_d - B_c$. In selecting a trench width, account should be taken of the type of compaction equipment, any trench sheeting and site practice. Compaction should be carried out in layers and any trench sheeting should be pulled progressively to enable the specified modified Proctor density, M_p , to be achieved.

8.7.5 Embankment installations

The embankment installation is indicated in the trench width column (Embank.) of each table ([Table 29](#) to [Table 32](#)) and the minimum width assumed for any compaction each side of the pipe was 1.65 times the external pipe diameter.

8.7.6 Safe depths of cover

The safe depths of cover in [Table 29](#) to [Table 32](#) were calculated in accordance with [8.1](#) to [8.5](#). In the determination of the safe depths the beneficial influence of internal pressure has been omitted and the maximum was terminated at 6.0 m. For larger sizes of pipe, greater depths or parameters outside those used in the tables, the pipe manufacturer and an experienced structural pipeline design engineer should be consulted.

Table 29 — Safe depths of cover (*m*) for ductile iron water pipe with field surcharge

DN	80	100	150	200	250	300	350	400	450	500	600
Class	M_p (%)	$B_d - B_c$ (m)									
D/S3	85	0.3	0.5 – 6.0+								
		0.6									
		0.9									
	90	Embank.	0.5 – 6.0+								
		0.3									
		0.6									
		0.9									
		Embank.									
	85	0.3	0.5 – 6.0+								
		0.6									
		0.9									
D/S4	85	Embank.	0.5 – 6.0+								
		0.3									
		0.6									
	90	0.9	0.5 – 6.0+								
		Embank.									
		0.3									
		0.6									
		0.9									
	85	Embank.	0.5 – 6.0+								
		0.3									
		0.6									
D/S5	85	0.9	0.5 – 6.0+								
		Embank.									
		0.3									
	90	0.6	0.5 – 6.0+								
		0.9									
		Embank.									
		0.3									
	85	0.6	0.5 – 6.0+								
		0.9									
		Embank.									
	90	0.3	0.5 – 6.0+								
		0.6									
		0.9									
		Embank.									
Poor soils	85	0.3	0.5 – 6.0+								
		0.6									
		0.9									
	90	Embank.	0.5 – 6.0+								
		0.3									
		0.6									
		0.9									
	85	Embank.	0.5 – 6.0+								
		0.3									
		0.6									
	90	0.9	0.5 – 6.0+								
		Embank.									
		0.3									
		0.6									

Table 29 (continued)

DN		80	100	150	200	250	300	350	400	450	500	600
Class	M_p (%)	$B_d - B_c$ (m)										
S1 or S2	Uncompacted	0.3	0.5 – 6.0+									
		0.6										
		0.9										
	Embank.	0.5 – 6.0+					0.5 – 5.2		0.5 – 4.8	0.5 – 4.8	0.5 – 5.4	0.5 – 5.5
		80	0.3	0.5 – 6.0+								
0.6												
0.9												
	Embank.	0.5 – 6.0+										

NOTE See explanations in subclauses 8.7.1 to 8.7.6.

Table 30 — Safe depths of cover (*m*) for ductile iron water pipe with main road surcharge

DN		80	100	150	200	250	300	350	400	450	500	600	
Class	M_p (%)	$B_d - B_c$ (m)											
D/S3	85	0.6 – 6.0+											
		0.3											
		0.6											
		0.9											
	90	0.6 – 6.0+											
		Embank.											
		0.3											
		0.6											
		0.6 – 6.0+											
		Embank.											
		0.9											
		0.6											
D/S4	85	0.6 – 6.0+											
		Embank.											
		0.3											
		0.6											
	90	0.6 – 6.0+											
		Embank.											
		0.9											
		0.6											
D/S5	85	0.6 – 6.0+											
		Embank.											
		0.3											
		0.6											
	90	0.6 – 6.0+											
		Embank.											
		0.9											
		0.6											
Poor soils		0.6 – 6.0+											
		Embank.											
		0.3											
		0.6											

Table 30 (continued)

DN	80	100	150	200	250	300	350	400	450	500	600
Class	M_p (%)	$B_d - B_c$ (m)									
S1 or S2	Uncompacted	0.3	0.6 – 6.0+								
		0.6	0.6 – 6.0+								
		0.9	0.6 – 6.0+								
		Embank.									
	80	0.3	0.6 – 6.0+								
		0.6									
		0.9	0.6 – 6.0+								
		Embank.									
	85	0.3	0.6 – 6.0+								
		0.6	0.6 – 6.0+								
		0.9	0.6 – 6.0+								
		Embank.									
90	Uncompacted	0.3	0.6 – 6.0+								
		0.6	0.6 – 6.0								
		0.9	0.6 – 6.0								
		Embank.									
	80	0.3	0.6 – 6.0								
		0.6	0.6 – 6.0								

NOTE See explanations in subclauses 8.7.1 to 8.7.6.

Table 31 — Safe depths of cover (*m*) for ductile iron sewer pipe with field surcharge

DN	Class	M_p (%)	$B_d - B_c$ (m)	80	100	150	200	250	300	350	400	450	500	600
D/S3	85		0.3						0.5 – 6.0+					
			0.6											
			0.9											
			Embank.											
	90		0.3											
			0.6											
			0.9											
D/S4	85		Embank.											
			0.3											
			0.6											
			0.9											
	90		Embank.											
			0.3											
			0.6											
D/S5	85		Embank.											
			0.3											
			0.6											
			0.9											
	90		Embank.											
			0.3											
			0.6											
Poor soils	85		Embank.											
			0.3											
			0.6											
			0.9											
	90		Embank.											
			0.3											
			0.6											

Table 31 (continued)

DN		80	100	150	200	250	300	350	400	450	500	600	
Class	M_p (%)	$B_d - B_c$ (m)											
S1 or S2	Uncompacted	0.3	0.5 – 6.0+										
		0.6	0.5 – 6.0+										
		0.9	0.5 – 6.0+										
		Embank.	0.5 – 6.0+										
	80	0.3	0.5 – 6.0+										
		0.6	0.5 – 6.0+										
		0.9	0.5 – 6.0+										
		Embank.	0.5 – 6.0+										
	85	0.3	0.5 – 6.0+										
		0.6	0.5 – 6.0+										
		0.9	0.5 – 6.0+										
		Embank.	0.5 – 6.0+										
0.5 – 6.0+													

Table 32 — Safe depths of cover (*m*) for ductile iron sewer pipe with main road surcharge

DN	80	100	150	200	250	300	350	400	450	500	600
Class	M_p (%)	$B_d - B_c$ (m)									
D/S3	85	0.3	0.6 – 6.0								
		0.6									
		0.9									
		Embank.									
	90	0.3	0.6 – 6.0								
		0.6									
		0.9									
		Embank.									
D/S4	85	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									
	90	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									
D/S5	85	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									
	90	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									
Poor soils	85	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									
	90	0.3	0.6 – 6.0+								
		0.6									
		0.9									
		Embank.									

Table 32 (continued)

DN																						
Class	M_p (%)	B_d - B_c (m)	80	100	150	200	250	300	350	400	450	500	600									
S1 or S2	Uncompacted	0.3				0.6 - 6.0+			0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+	1.3 - 5.0									
		0.6												0.7 - 6.0	0.8 - 6.0+	0.9 - 5.0	1.0 - 4.1	1.2 - 3.1				
		0.9																	0.6 - 6.0	0.7 - 4.7	0.8 - 4.0	0.9 - 3.4
	Embank.	0.6 - 6.0+								0.6 - 4.8	0.6 - 4.3	0.7 - 3.8	0.7 - 3.5	0.8 - 3.4								
	80														0.3	0.6 - 6.0+	0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+	1.2 - 6.0+	
	0.6	0.6 - 6.0+								0.7 - 6.0	0.8 - 6.0+	0.9 - 5.9+	1.0 - 4.5									
	0.9													0.6 - 6.0+	0.7 - 6.0							0.8 - 6.0+
	Embank.	0.6 - 6.0+								0.6 - 6.0+	0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+			1.0 - 6.0+	1.1 - 6.0+					
	85													0.3	0.6 - 6.0+			0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+	1.1 - 6.0+
	0.6													0.6 - 6.0+								
0.9	0.6 - 6.0+	0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+																	
Embank.						0.6 - 6.0+	0.6 - 6.0+	0.7 - 6.0	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+	1.1 - 6.0+										
90	0.3	0.6 - 6.0+	0.7 - 0.6	0.8 - 6.0+	0.9 - 6.0+								1.0 - 6.0+	1.1 - 6.0+								
0.6	0.6 - 6.0+														0.7 - 0.6	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+				
0.9						0.6 - 6.0+	0.7 - 0.6	0.8 - 6.0+	0.9 - 6.0+	1.0 - 6.0+												
Embank.	0.6 - 6.0+	0.6 - 6.0+	0.7 - 0.6	0.8 - 6.0+	0.9 - 6.0+						1.0 - 6.0+	1.1 - 6.0+										

NOTE See explanations in subclauses 8.7.1 to 8.7.6.

NOTE See explanations in subclauses 8.7.1 to 8.7.6.

Annex A (informative)

Other design considerations

A.1 Removal of trench support systems

The narrow trench load, as determined by equation (6), assumes that there will be relief of load due to friction at the fill/trench wall interface. However, if a trench is backfilled and compacted prior to the complete withdrawal of the trench support, the relieving friction at the interface can be substantially lost and the pipe load increased. In such cases the narrow trench load should be taken as equal to the total weight of the trench backfill.

Soil load for rigid pipes are obtained from equation (3):

$$W'_c = C_c \gamma B_c^2$$

Soil pressure for semi-rigid pipes are obtained from equation (A.1):

$$P_c = H \gamma \frac{B_d}{B_c} \quad (\text{A.1})$$

The narrow trench loads are then compared with the wide trench load [equation (3) for rigid, equation (6) for semi-rigid] and the lower value is used for the pipe design.

For flexible pipes, the void formed by removing the trench support system between the trench wall and the backfill will reduce support from the native soil significantly and increase pipe deflection.

Where the pipe installation relies on the sidefill compaction then it should be carried out in layers and the trench support pulled progressively to enable the specified compaction to be achieved.

A.2 Uniformly distributed surcharge loads of large extent

Uniformly distributed surcharge loads of large extent, often of a temporary nature, can occur through the raising of the ground surface level by filling with soil or other material. They can be allowed for in design by assuming a notional increase in the cover depth, H . If the uniform surcharge in kilonewtons per square metre (kN/m^2) is U_s and the unit weight of the soil in kilonewtons per cubic metre (kN/m^3) is γ then H is notionally assumed to be increased by U_s/γ m.

The effect of vehicles operating on the raised surface will be diminished because of the increased cover depth. In calculating the total load on the pipes, the true cover depth should be taken, plus the increased fill load, unless the original fill load plus vehicle load was the greater.

A.3 Uniformly distributed surcharge loads of limited extent

The vertical soil stress, σ , at a point below the ground surface [point O in [Figure A.1b](#)] is given by $U_s I_\sigma$, where the influence value, I_σ , is shown in [Figure A.1](#). The surcharge may be in a fixed position in relation to the pipe or it may be moveable. In the latter case it should be taken to be in the position giving greatest load on the pipe, generally with the centre of gravity of the surcharge vertically above the pipe centre line. In general, it is necessary to divide the load into several rectangles, as illustrated in [Figure A.2](#), and obtain the answer by algebraic summation. The point O [see [Figure A.1b](#)] should

always be vertically below one corner of the rectangle. Non-rectangular surcharges need to be split into approximate rectangles.

Figure A.1 — Soil loads and stresses produced by surface loads (after Fadum [13])

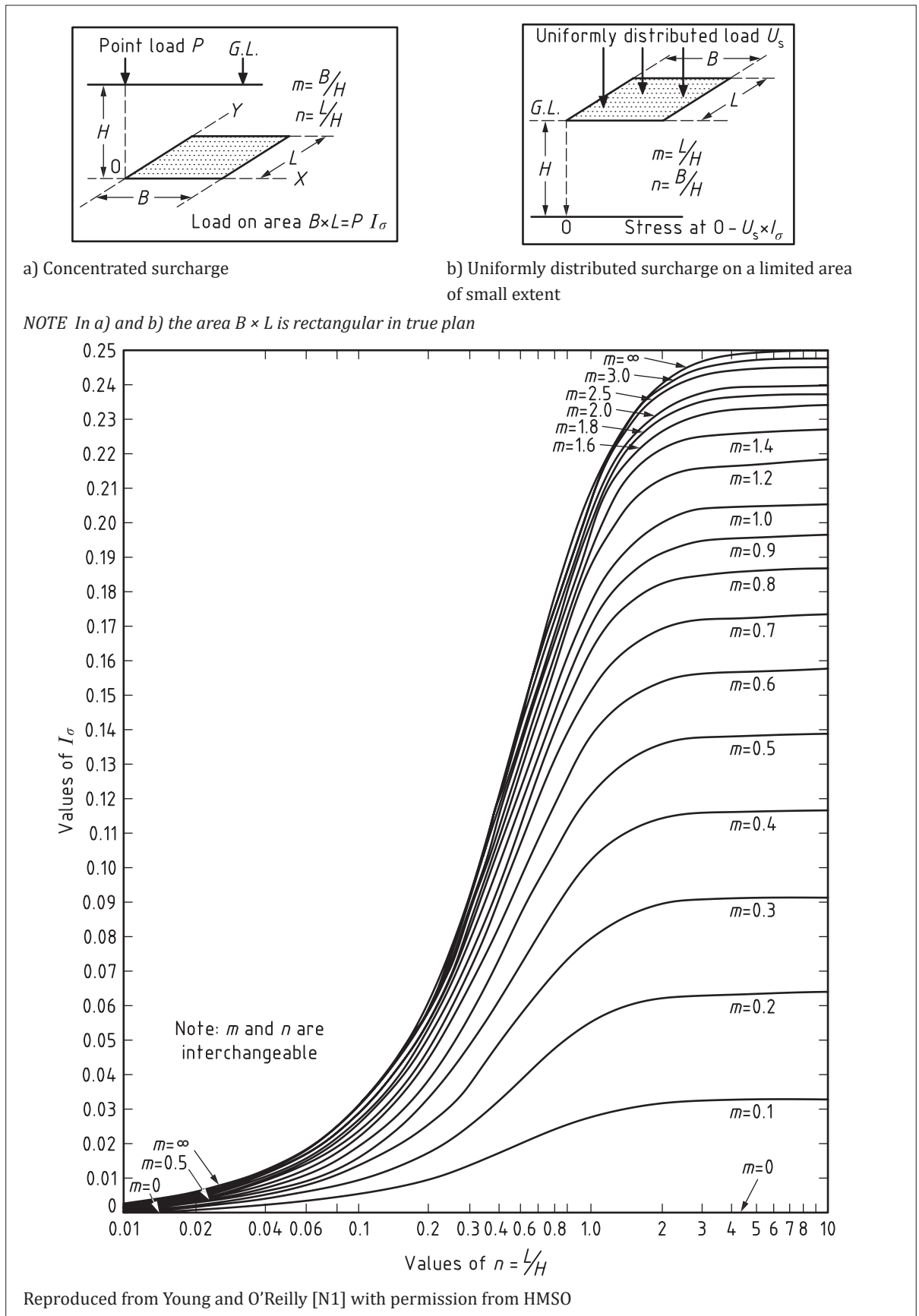
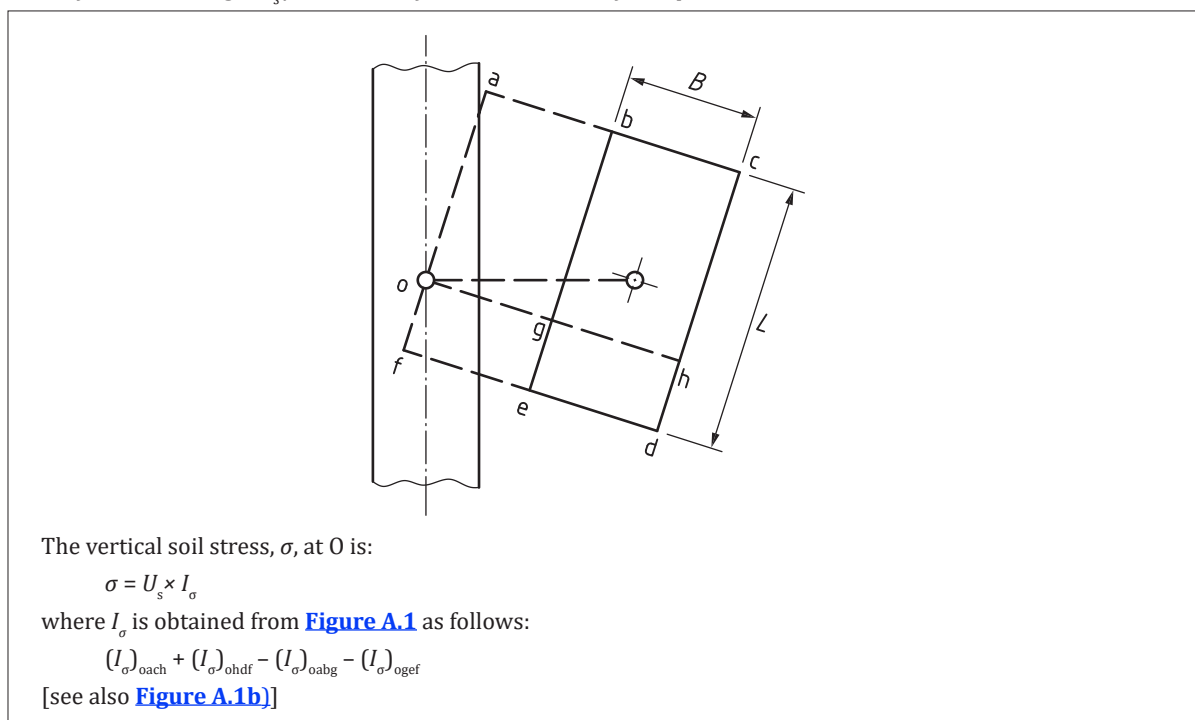


Figure A.2 — Uniform surcharge U_s /unit area of limited extent in fixed position

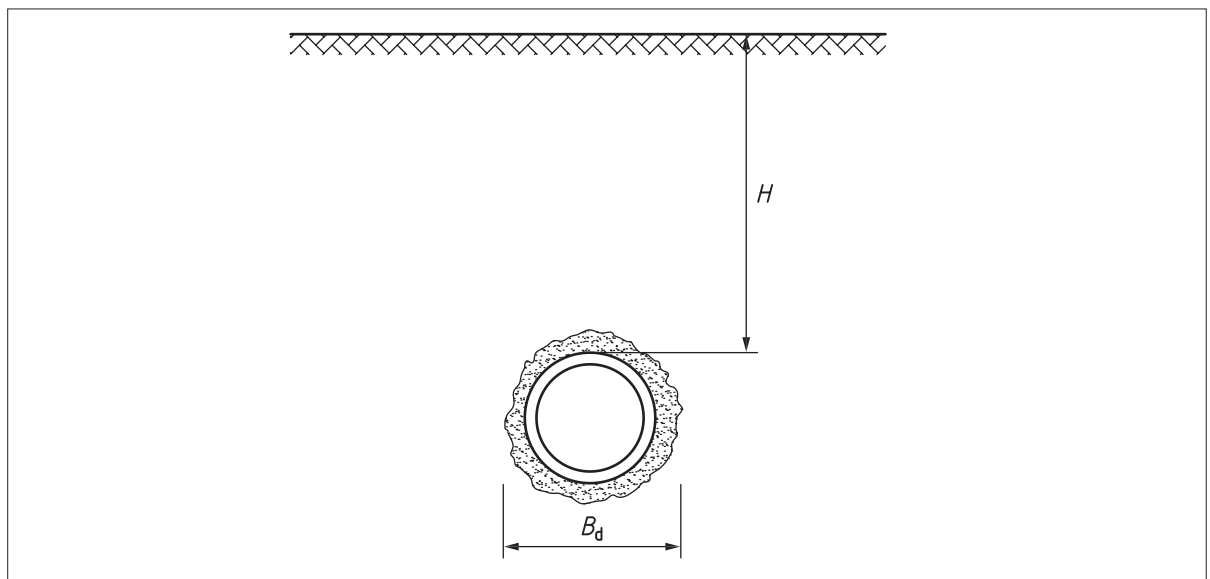
Having determined the vertical soil stress at the pipe crown the same stress is assumed to exist across the full breadth of the pipe and for unit run of pipeline, i.e. pipe load = stress $\times B_c$ per unit length. The procedure is approximate but errs on the conservative side.

The soil stress at the pipe crown produced by such a surcharge can be calculated in the manner illustrated in [Figure A.2](#).

Heavy vehicles with broad caterpillar tracks may be treated in a similar manner.

A.4 Pipes installed by trenchless installation techniques

When a pipe is installed by jacking, heading or another trenchless method, the overlying soil can bridge over the pipe and relieve it of a substantial part of the load that would otherwise be imposed if it were installed in a trench. There is also the possibility of a reduction of bending moment in the pipe walls, particularly when the pipe is back-grouted after installation as a result of the development of horizontal soil resistance. Such pipes should be designed as if laid in a trench having the same width as the hole cut through the soil (see [Figure A.3](#)) using the $K\mu$ value of the poorest soil over, or adjacent to, the pipe. This procedure is likely to produce a conservative design but, given the indeterminate nature of the problem, the uncertainty in the amount of bridging and in the development of side pressure and the considerable difficulty of replacing damaged pipes, this is probably beneficial. In any event, a jacked pipe needs to be sufficiently strong to resist the jacking forces applied during installation.

Figure A.3 — *Pipe in tunnel, heading or jacked into place*

A.5 Pipes installed in tunnels and headings

Where the hydraulic design requires pipes of less than 1 000 mm diameter at depth, it might be appropriate to install such pipes within a tunnel or a heading or a larger jacked pipe. For conduits larger than 1 000 mm diameter, one of the conventional types of tunnel or pipe jacking (see [A.4](#)) is the normal solution when trenching is inappropriate. In most cases a tunnel or heading is a permanent structure, but on occasions it might only be needed during the construction of the pipeline.

A.6 Bedding factor for rigid pipes under an embankment

Because it is difficult to ensure good compaction of the sidefills for pipes in trenches, the effect of any lateral pressure on rigid pipes in trenches is generally neglected and this practice should be adopted where pipes are laid in a trench dug into an embankment. However, for pipes laid during construction of an embankment and where succeeding layers of fill are then placed and compacted against and over the pipes, the active pressure generated within the fill will act horizontally on the part of the pipe exposed above the natural ground and produce moments in the pipe opposite to those produced by the vertical loads. The lateral pressure effectively reduces the effect of the vertical loads on the pipe but these are generally neglected because of uncertainty that the beneficial active pressure will be present.

A.7 Longitudinal flexibility

The design procedure in [A.1](#) to [A.6](#) assumes that the support is uniformly distributed along the pipe.

Local concentration of support:

- a) causes the pipes to be loaded as beams (applies mainly to the smaller diameters);
- b) results in increased ring bending at the support points;
- c) produces increased shear stresses in the pipe walls; and
- d) causes vertical shearing forces at joints.

Each of these can lower the supporting strength of a pipe. Although the use of flexible joints considerably reduces the effects of a) to c) they can still act within the individual pipe length if care is not taken to ensure that there are no hard spots within the prepared bed or backfill.

When concrete bedding is used, unless the bedding is designed as a rigid beam spanning between supports, it is important to retain the flexibility of the joints. Gaps should be left in the in-situ concrete at each joint formed by vertical sheets of compressible material such as expanded polystyrene. It is essential that concrete is not allowed to enter the pipe joint.

At structures, e.g. buildings or manholes, the projecting piece of pipe should be kept as short as possible, just sufficient to make the joint, so as to reduce the length of the cantilever. Unless differential settlement is expected to be minimal the next adjoining pipe should also be kept short, so that the joints at each end act as a pair of hinges. If considerable settlement is expected, a second short pipe should be used before commencing with full-length pipes (see [A.10.3](#)).

A.8 Multiple pipes in trench

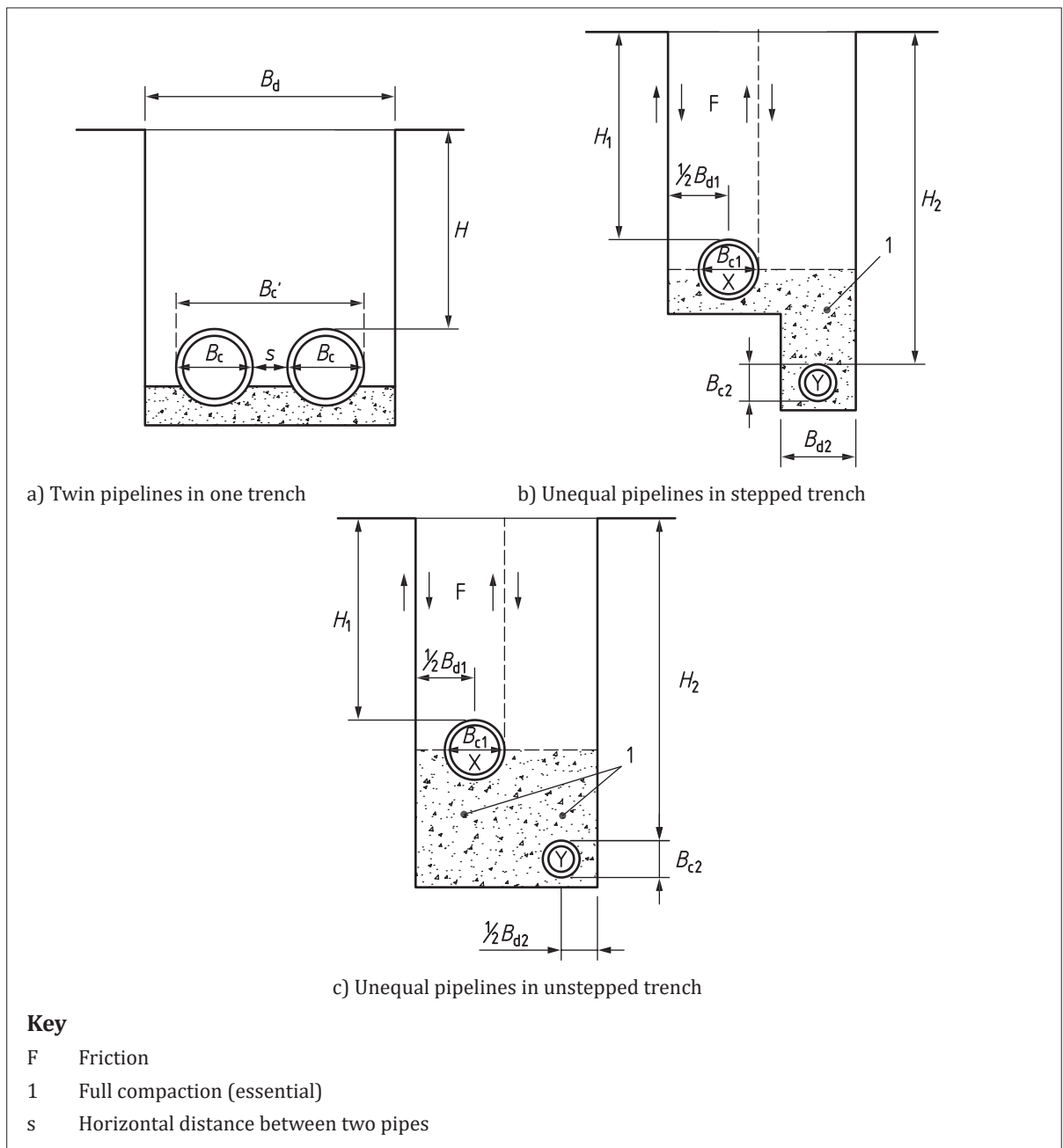
A.8.1 General

For convenience, two or more pipes are often laid in a single trench, the pipes being either at the same or at differing levels [see [Figure A.4a](#)]. In the latter case, the upper pipe is sometimes installed on a step in the trench [see [Figure A.4b](#)]. There are risks attached to this practice since, even in firm ground, there is always the likelihood that the step will break or crack away intermittently so that support for the upper pipe will be non-uniform. As a result, the upper pipe might be overstressed due to inadequate arc of support from the bedding. Also, longitudinal bending stresses might be excessive, especially if the pipes are relatively long, or there might be leakage/failure at the joints due to shear. Only adequate compaction of the fill around the lower pipe and up to the springing of the upper pipe will eliminate this risk. The alternative is to omit the step, as shown in [Figure A.4c](#). Again, full compaction of the bedding materials around the pipes is essential.

For calculation of the soil load:

- a) for rigid pipes, the soil loads can be estimated in accordance with [A.8.2](#) to [A.8.4](#), though no special calculation is needed if the pipe is $\leq$ DN375 and is designed for wide trench loading;
- b) for semi-rigid pipes, the principles applied to rigid pipes can be used with the soil pressures determined by equations (37) and (42); and
- c) for flexible pipes, the geostatic pressure should be used.

The loads from surcharges are determined in accordance with [Clause 5](#).

Figure A.4 — Multiple pipe trenches**A.8.2 Twin equal diameter pipelines (rigid pipes) (see [Figure A.4](#))**

Twin equal diameter pipelines may be treated as equivalent to a single pipe of outside width $B'_c = 2B_c + s$. The load on this equivalent pipe is the lesser of the two values, $C_d \gamma B_d^2$ or $C_c \gamma B'^c^2$, where the value of C_c is based on the ratio H / B'_c . The fill load on each pipe is one half of the load on the equivalent pipe. If s exceeds approximately $B_c/2$ the method errs on the conservative side. The method may be adapted to multiple pipes in a trench.

A.8.3 Unequal diameter pipelines in stepped trench (rigid pipes) (see [Figure A.4](#))

The load on the left-hand half of pipe X is given by $\frac{1}{2} C_{d1} \gamma B_{d1}^2$ or by $\frac{1}{2} C_{c1} \gamma B_{c1}^2$, whichever is the lesser. The load on the right-hand half of pipe X is given by $\frac{1}{2} C_{c1} \gamma B_{c1}^2$. The values of C_{d1} and C_{c1} are those appropriate to depth H_1 . Provided the top of pipe Y is appreciably below the step it may be treated as being laid in a trench of width B_{d2} and the total weight on it is the lesser of either

$C_{d2}\gamma B_{d2}^2$ or $C_{c2}\gamma B_{c2}^2$. The values of C_{d2} and C_{c2} are those appropriate to depth H_2 . If, however, the top of pipe Y is not below the level of the step it should be treated in two halves as for pipe X.

In calculating the load on the right-hand half of pipe X, the value of $r_{sd}\rho$ used to determine C_{c1} should be taken into account. If the fill below the step is poorly compacted this has the effect of increasing the effective value of $r_{sd}\rho$, and due allowances should be made for this. However, because of the risk of the step breaking away (see [A.8.1](#)), care should be exercised to obtain a high degree of compaction and to ensure that settlement of this lower fill is reduced to an absolute minimum, by employing aggregate that is easy to compact and working it thoroughly into position.

A.8.4 Unequal diameter pipelines in unstepped trench (rigid pipes) (see [Figure A.4](#))

The halves of both pipes are treated separately as for pipe X in [A.8.3](#).

Because of the greater thickness of bedding for the upper pipe, particularly if it is considerably higher than the lower, the compaction of the bedding prior to laying the upper pipe and of the bedding up to the springing should be such as to minimize settlement and to ensure that such settlement is substantially uniform along the length of the pipe.

The method of treating each pipe in separate halves is also applicable to multiple pipes in a trench.

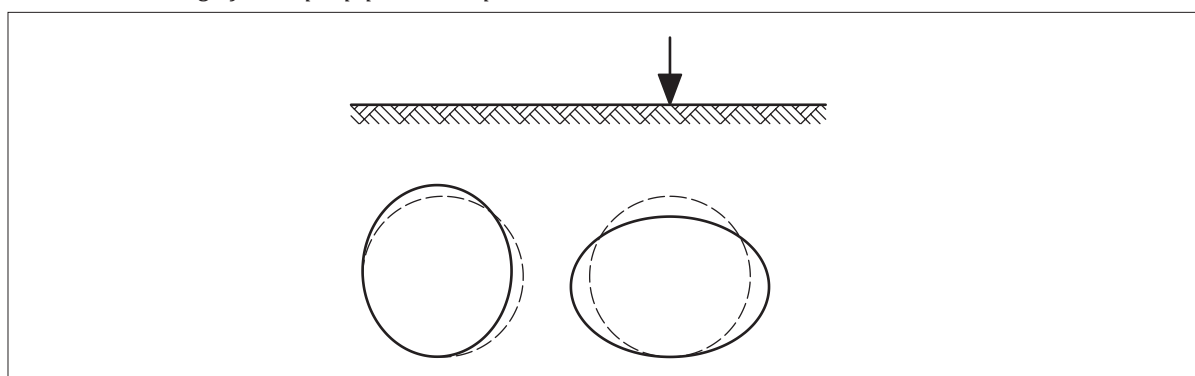
A.8.5 Multiple pipes (rigid or flexible) in a trench or embankment

For installations of multiple pipes laid parallel and at the same level within a common trench or embankment, a self-compacted type of backfill is recommended to ensure that there are no voids in the upper bedding or sidefill.

Where material requiring compaction is used, the pipes should be spaced sufficiently far apart to allow space for compaction equipment. The upper bedding and sidefill material between the pipes should be compacted to the same compaction class as the material between the pipe and the trench wall.

Where the pipes are installed under a trafficked area, there should be enough soil cover to avoid overloading the pipes. In such cases, the ring compression strength of the pipe wall and asymmetric deformation mode should be taken into account (see [Figure A.5](#)).

Figure A.5 — *Uneven loading of multiple pipes laid in parallel*



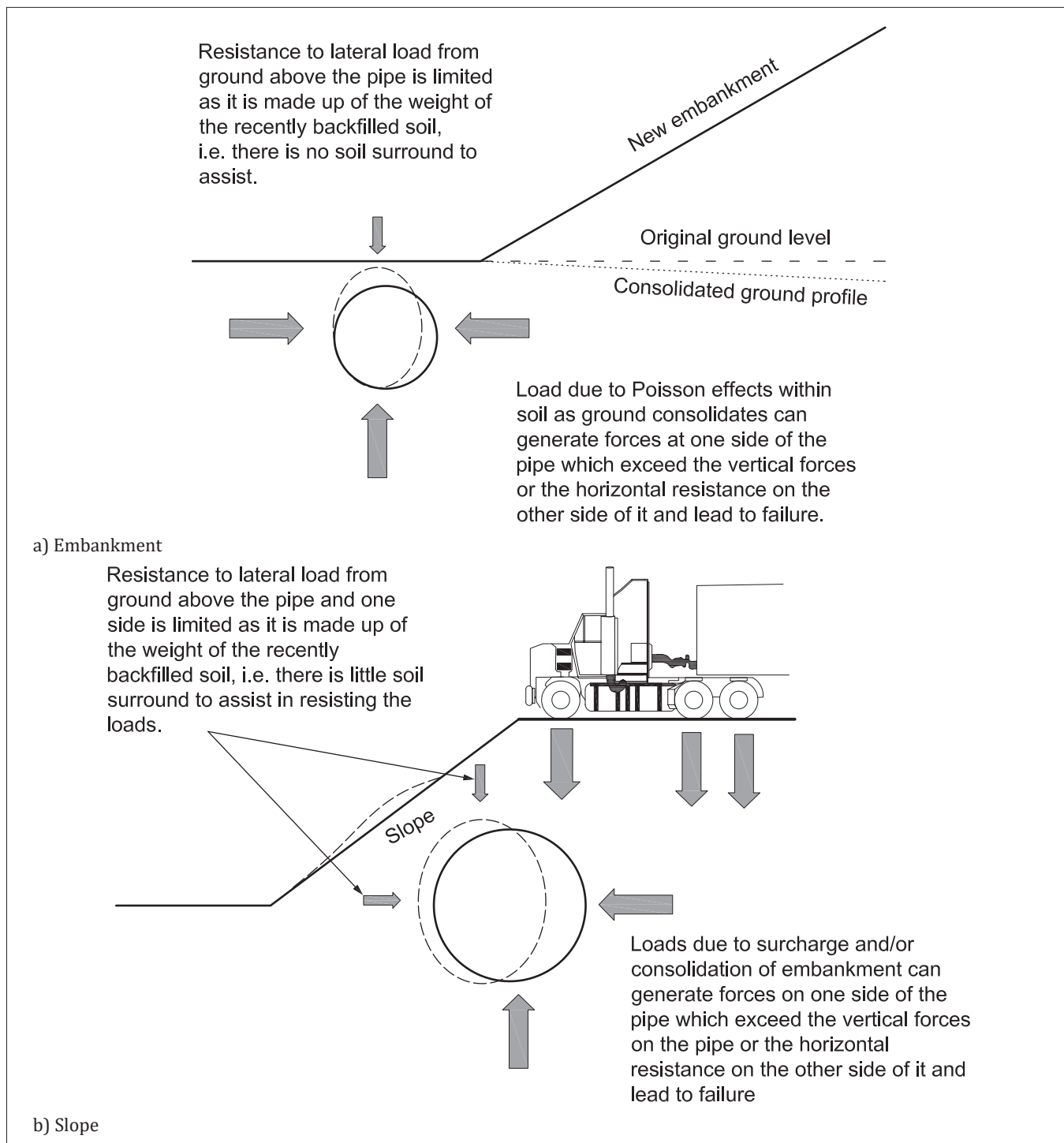
In all cases, the backfilling should proceed from one end of the set of pipes, filling all gaps uniformly from a platform of cover over the filled section which can safely bear the weight of the construction plant.

A.8.6 Installation of pipes adjacent to steep slopes or embankments

Special attention should be given when single or multiple pipes are installed adjacent to or within steep slopes or embankments (see [Figure A.6](#)).

The designer should determine and take account of the various loading scenarios that could exist within the ground at different stages in the construction and the years following construction, as circumstances can occur which result in forces being generated on one side of the pipe which are greater than the pipe and soil surround can resist. Care should be taken with larger diameter, relatively shallow flexible pipes where eccentric loadings and limited resistance from the recently backfilled soil above or to one side of the pipe can result in failures.

Figure A.6 — *Circumstances which can result in large lateral loads: Embankment*



A.9 Buoyancy effects

A.9.1 Submerged pipeline

The effect of submergence is to reduce the “effective” vertical pressure exerted on the pipe by the backfill, owing to the buoyancy of the solid particles, while simultaneously exerting a hydrostatic pressure on the pipe periphery of a magnitude dependent on the depth of submergence. Such hydrostatic pressure produces negligible bending moments in the walls of a circular pipe, apart from the effect of the differing pressures at the top and bottom of the pipe, and this effect will be largely cancelled when the pipe runs full. The net effect on a circular pipe of submergence is therefore to reduce the bending moment. This is not the case for a non-circular conduit where the effect of the hydrostatic pressure should be allowed for.

In practice most pipelines, owing to de-watering during construction and possibly during repair, will be subjected to the full overburden loads for some period during their life. Therefore, no reduction should be applied for the effects of submergence.

A.9.2 Buried pipelines below groundwater table

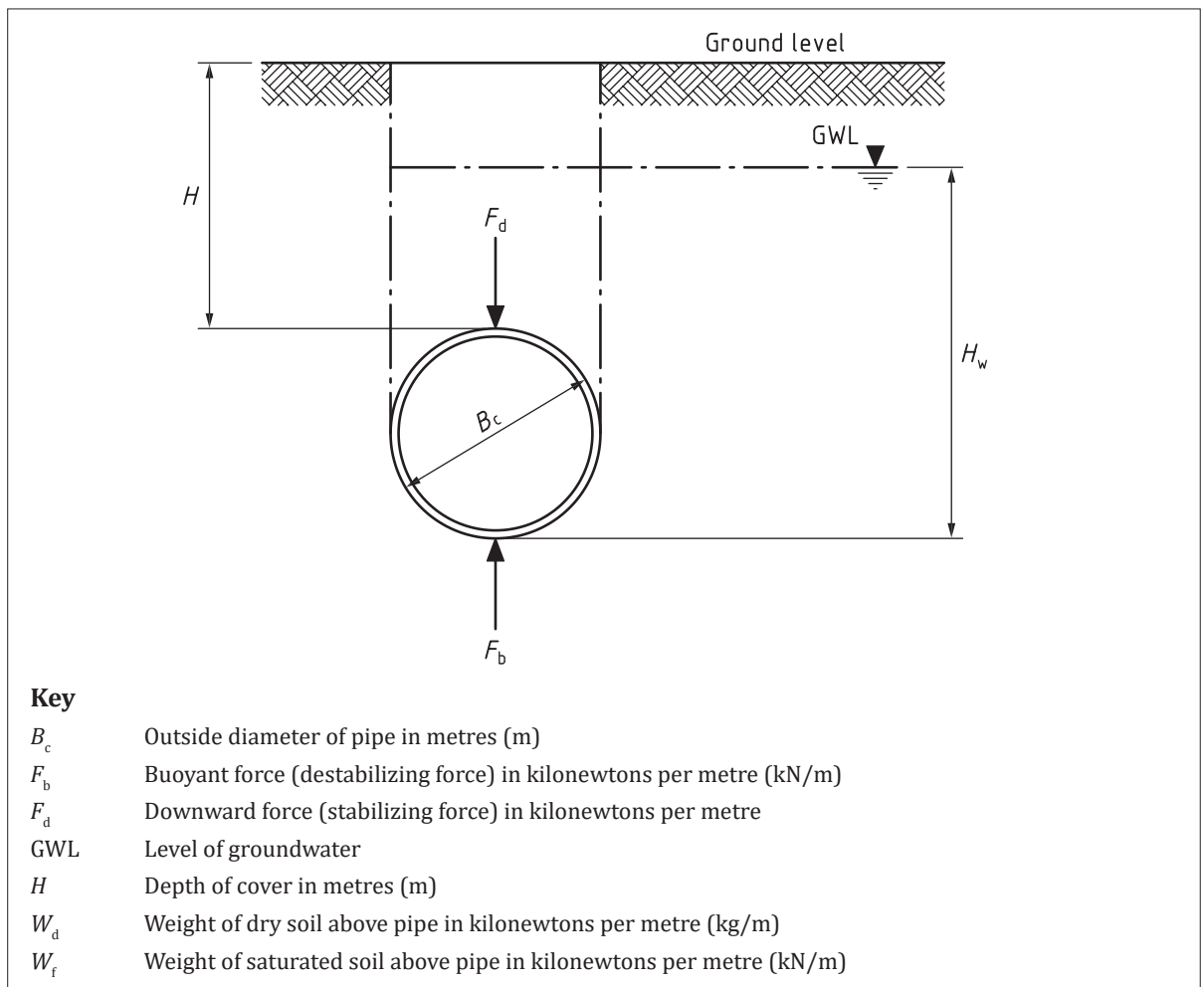
Buried pipelines located below the groundwater table are subject to a buoyant uplift force equal to the weight of water displaced. Usually this force is balanced by the weight of the backfill above the pipeline, but care should be exercised when designing large-diameter pipelines at shallow depths of cover and pipelines which are expected to stand empty for long periods of time.

High groundwater can float buried pipe, causing upward movement off-grade as well as catastrophic upheaval. This is not an issue for plastic pipes alone. Flotation of metal or concrete pipes can occur at shallow cover when the pipes are empty.

Flotation occurs when the groundwater surrounds the pipe, producing a buoyant force greater than the sum of the downward forces provided by the soil weight on top and the weight of the pipe and its contents.

Flotation is not generally a design consideration for buried pipe where the pipeline runs full or nearly full of liquid or where groundwater is always below the pipe invert. A rule of thumb is that when a pipe is buried in common saturated soil with at least one pipe diameter of cover, pipe is generally not subject to groundwater flotation. However, if burial is in lighter weight soils or with less cover, groundwater flotation should be allowed for.

A schematic of groundwater flotation forces is given in [Figure A.7](#).

Figure A.7 — Schematic of groundwater flotation forces

The following are basic equations for flotation calculations. In general, only the soil prism load above the pipe as indicated in [Figure A.7](#) should be taken into account during the flotation calculations.

- a) Downward force (stabilizing force) in kilonewtons per metre (kN/m), F_d

$$F_d = W_{\text{pipe}} + W_f + W_d + W_w$$

where:

W_{pipe} is the pipe weight in kilogrammes per metre (kg/m); and

W_w is the weight of liquid content of pipe in kilonewtons per metre (kN/m).

- b) Buoyant force (destabilizing force) in kilonewtons per metre (kN/m), F_b

$$F_b = \gamma_w \frac{\pi}{4} B_c^2$$

where:

γ_w is the unit weight of water in kilonewtons per cubic metre (kN/m³) (9.8 kN/m³).

- c) Weight of saturated soil above pipe in kilonewtons per metre (kg/m), W_f

$$W_f = (\gamma_d - \gamma_w) \left[\frac{B_c^2 (4 - \pi)}{8} + B_c (H_w - B_c) \right]$$

where:

γ_d is the unit weight of dry soil in kilonewtons per cubic metre (kN/m³).

NOTE Where $(\gamma_d - \gamma_w) < \gamma_w$, γ_w may be used in place of $(\gamma_d - \gamma_w)$.

d) Weight of dry soil above pipe in kilonewtons per metre (kN/m), W_d

$$W_d = \gamma_d (H - H_w + B_c) B_c$$

e) Weight of liquid content of pipe in kilonewtons per metre (kN/m), W_w

When the pipe is 100% full of liquid (assumed here to be water), the liquid weight is:

$$W_{w,100\%} = \gamma_w \frac{\pi D_i^2}{4}$$

When the pipe is 50% full, the liquid weight is:

$$W_{w,50\%} = \gamma_w \frac{\pi D_i^2}{8}$$

where:

D_i is the internal diameter of the pipe in metres (m).

The factor of safety against flotation should be greater than 1.1, i.e.:

$$\frac{F_d}{F_b} > 1.1$$

Unless verified to be lower, the highest short-term water table should be assumed to be at ground surface level.

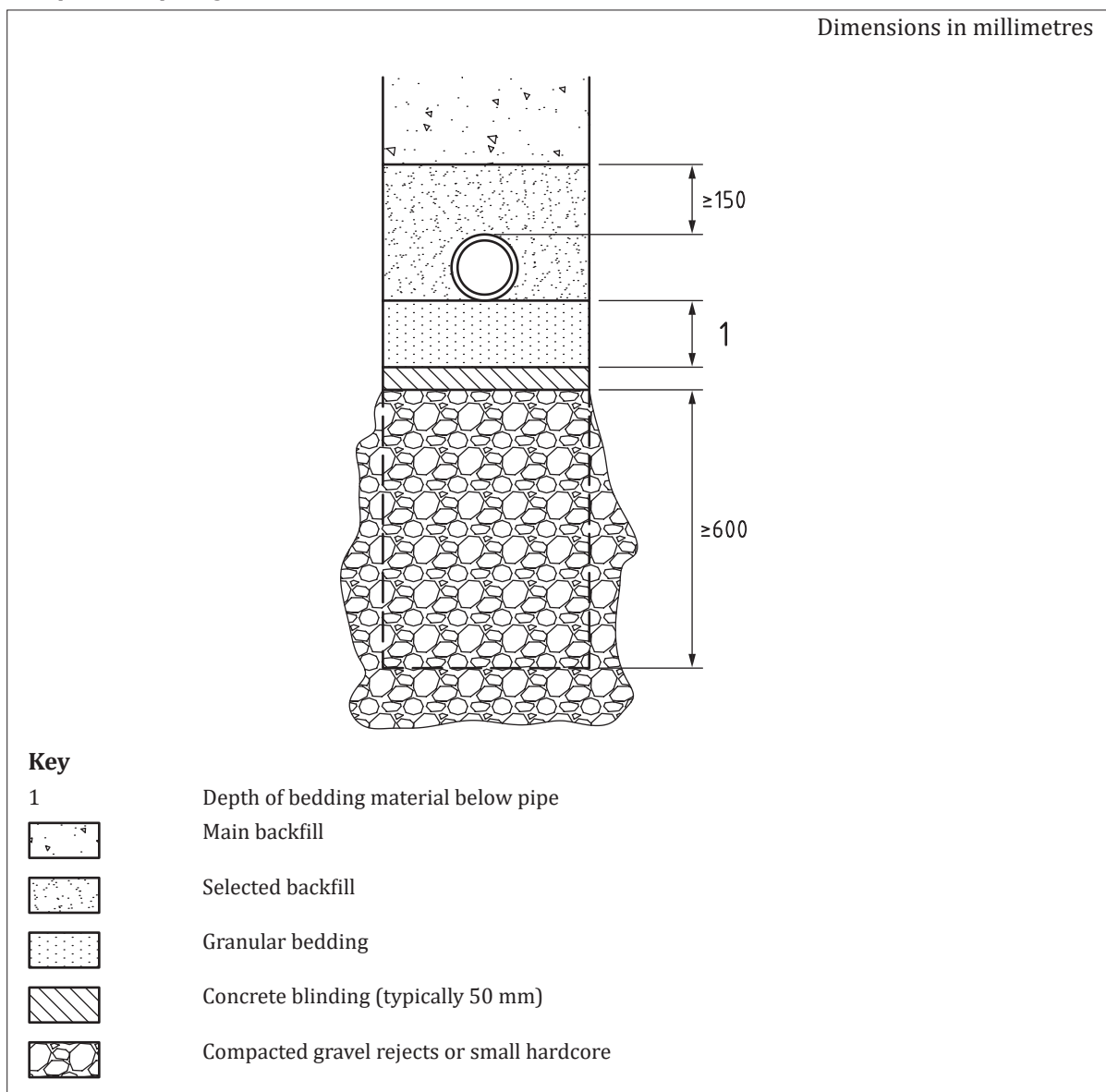
A.10 Pipelines in poor ground

A.10.1 Bedding construction in poor ground

Where the trench formation has little bearing strength and will not support pipe bedding material effectively, it might be necessary to provide a stable formation before pipe-laying, particularly for gravity drainage or sewerage systems. Such conditions most commonly occur in peat, silty ground, soft-to-very-soft alluvial clays, running sand and artificially filled ground.

Although soft trench formations are sometimes simply stabilized with concrete, this is unlikely to assure long-term stability in all cases, and a form of flexible bedding construction is the preferred method of dealing with this situation. For example, the trench formation can be over-excavated to allow for the placing and compaction of hardcore or aggregate in layers to form a firm trench bottom and achieve some local ground compaction. Where necessary, this should be blinded before the pipes are laid on their specified bedding.

The pipe bedding construction requirements are calculated in the normal way, but it is important that wide trench design criteria are used because narrow trench conditions cannot be guaranteed in this situation (see [Figure A.8](#)).

Figure A.8 — Pipelines in poor ground

Where groundwater is present at a level above trench formation in fine-grained soils, the strength of pipe beddings can be reduced. Granular bedding material encourages water movement, washing fines out of the surrounding ground and causing a loss of support to the bedding and pipeline.

The traditional method of dealing with this problem was to include a proportion of coarse sand in granular bedding material to fill the interstices that might otherwise take up the fine material from around the trench. This limits the movement of fines, but the bedding material requires much more compaction energy than if it were single-sized or graded.

A more effective method is to wrap the whole of the bedding construction in a geotextile as shown in [Figure A.9](#), including any additional compacted material in the trench bottom for soft ground conditions.

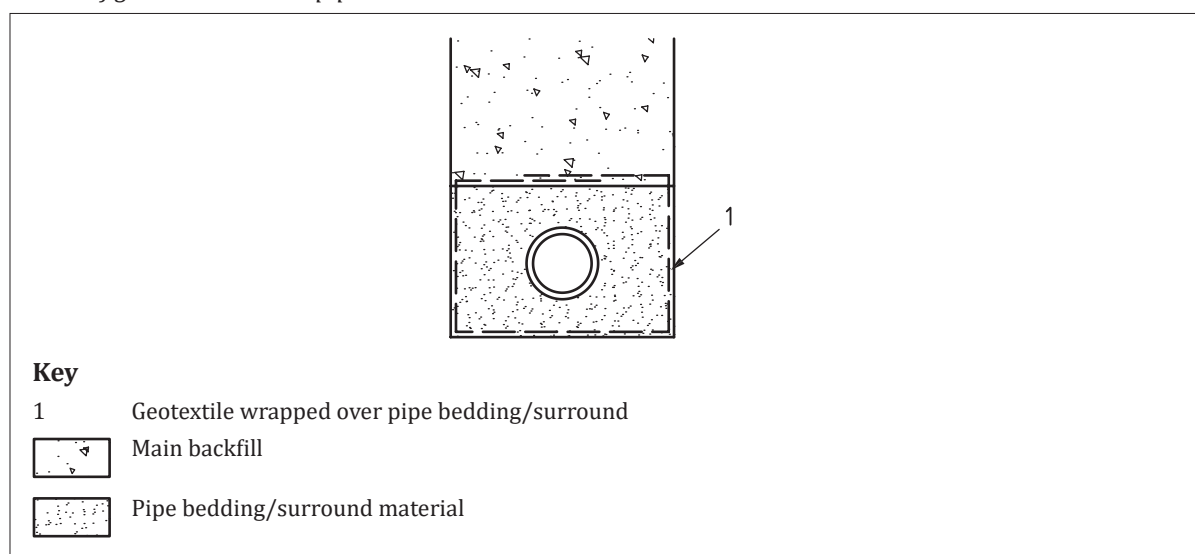
This method allows the movement of water through the bedding material but tends to prevent the movement of fine material and retain it in the ground around the trench. The specification for the geotextile, particularly the pore size, should be related to the nature of the fines in the ground, and specialist advice might be required.

Prior to commencing pipe-laying it is essential to satisfactorily de-water the trench formation. Any well point de-watering should also be suitably filtered to prevent continuous removal of fine sands and silts. Sump pumping from the end of the trench is not recommended, even when filtered, as instability of the formation can arise.

Care should be exercised when using “drag-box” or similar trench support systems in waterlogged, fine-grained soils because the pipe and bedding are likely to be disturbed when the support is removed. Steel or timber sheet trench support systems are recommended. It is important that backfilling proceeds progressively as the support system is removed.

Where pipes enter a rigid structure (manhole or building) care is needed to provide adequate flexibility to deal with the differential settlement (see [A.10.3](#)).

Figure A.9 — *Use of geotextile around pipe embedment*

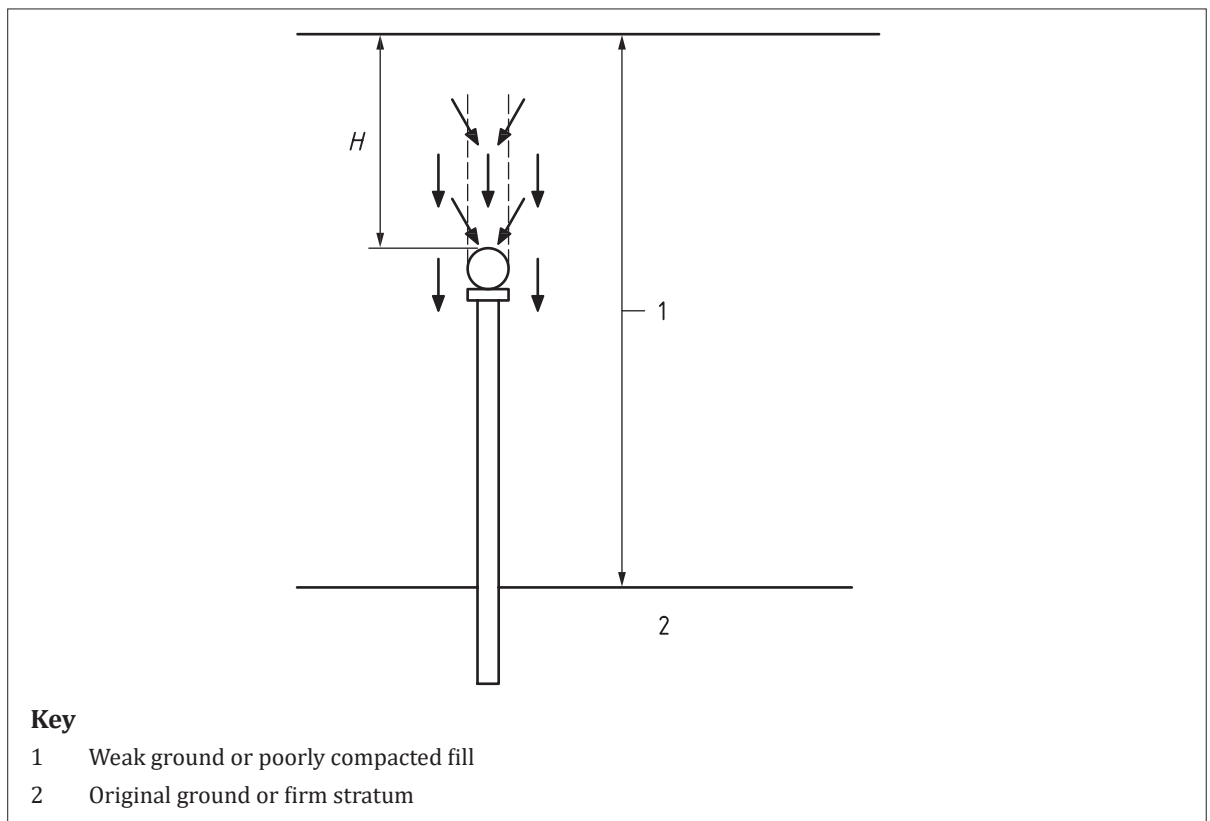


A.10.2 Pipelines supported on piles

Where a pipeline is to be laid in very poor ground such as bogs, weak soils in low-lying coastal areas, or poorly compacted fill, the pipeline can be supported on piles driven down into firmer ground, as shown in [Figure A.10](#). In such cases the pipes can be subjected to a much-enhanced loading when, as is almost inevitable, the adjoining ground settles relative to the pipeline.

Rigid pipes are usually laid with concrete haunching on a reinforced concrete beam of sufficient strength to span the piles. Flexibility at joints is generally unnecessary, although the effect of deflection of the beam on the pipes needs to be taken into account.

Semi-rigid pipes usually have sufficient beam strength to span piles and are located in concrete saddles to minimize the local support stresses. A competent structural pipeline engineer or the pipe manufacturer should be consulted for design details.

Figure A.10 — Pipeline on piles

A study of soil loads for piled installations [14] demonstrated that the load coefficient can be taken as:

$$C_c = \frac{H}{B_c} \left(0.433 \frac{H}{B_c} + 1 \right) \quad (\text{A.2})$$

C_c from equation (A.2) is then substituted into equation (3) to obtain the soil load, i.e.:

$$W'_c = C_c \gamma B_c^2$$

A.10.3 Rocker pipes

Where a pipeline is built into a manhole or inspection chamber, or other structure such as a ground beam or concrete surround, some differential settlement between the pipeline and structure is to be expected.

To avoid the consequent high shear loads on the pipe built into the structure, the first flexible joint on the pipeline should be provided as close as practicable to the face of the structure. This should be within 150 mm for pipe diameters less than 300 mm.

A short-length “rocker” pipe should be laid next before any full-length pipes are used, to isolate the pipeline from small relative movements caused by differential settlement. The maximum length of the rocker pipe should be as shown in Table A.1. Short-length pipes with pre-formed joints are available from manufacturers or plain-ended pipes may be cut to size.

Table A.1 — *Recommended maximum length of rocker pipes*

Nominal pipe diameter	Maximum length
mm	m
<300	0.6
301 – 450	0.75
451 – 750	1.00
>750	1.25

NOTE For ductile iron pipes $\geq$ DN1100 the maximum length is given by $DN + 100$ mm.

The angular performance of the joints of short rocker pipes subject to significant settlement should be taken into account.

Where large differential settlements are anticipated, for example in waterlogged soils, peat, silt or made ground, the number of short-length pipes may be increased, but care is necessary to ensure that allowable angular deflections at individual pipe joints are not exceeded. Shallow gradients should be avoided in soils where large settlements are likely, to minimize the possibility of deposition of solids.

Pipe materials with a high degree of longitudinal continuity and flexibility, such as welded polyethylene and welded steel, do not typically require rocker pipes at structures, but other engineering solutions might be required where settlements are large (e.g. not limited to bedding consolidation) or where the shear strength of the pipe is low (e.g. most structured-walled plastic pipe types).

A.10.4 Non-circular conduits

Examples of non-circular concrete pipes available are “eggs” and ellipses. They are available with flexible joints and may be designed in the same way as a circular pipe of the same plan width. Such non-circular sections are particularly useful for drainage where enhanced flow is needed.

Rectangular section culverts are also available in concrete. Their design is the same as for other structures in reinforced concrete, with the dimensions and reinforcement determined by analysis. For example, culverts under highways and railways need to carry the loads set out in BS EN 1991-2 and BS 5400-2⁷⁾ (for main and light rail, respectively) and the structural members are required to be designed to sustain the bending moments produced by the most adverse combination of loads and pressure. They are particularly useful for service ducts and subways under roads and railways where headroom is restricted, and they can be installed much more rapidly than in-situ reinforced concrete structures.

A.11 Trench backfilled with concrete

A.11.1 Lean concrete

In some cases, the best solution in the circumstances might be to backfill a trench with lean concrete. Such a decision should not be taken lightly or to compensate for poor workmanship as this will only lead to later difficulties. Where lean concrete is used its compressive strength should not exceed 5 MN/m² at 28 days. Other than expense there are drawbacks to the use of lean concrete backfill over pipes. Alternating stiff and weak, or hard and soft, conditions in nature often presents difficulties and it is not generally good engineering practice to achieve this by putting a rigid backfill in pipe trenches. Just as hard spots cause problems below a pipe they can be a problem above. It is difficult in practice to ensure that some of the lean concrete backfill has not pierced or displaced the cushioning layer over the pipe and so is resting in places on the pipe. Similarly, piped services straddling the trench could be encased in the rigid backfill. Excavation through lean concrete backfill for repairs or future

⁷⁾ Withdrawn.

connections to the pipe is difficult and it is likely that the excavation will be made by attacking the backfill from the sides, thus enlarging the opening in the road and increasing the danger of damage to nearby services.

With a vertical-sided trench, due to shrinkage of either the concrete or the soil, contact between the backfill and surrounding ground can be lost or the soil/concrete interface can become lubricated, possibly diminishing significantly any relieving friction at the interfaces. As in the case of delayed removal of trench sheeting, it is advisable to assume that the whole weight of the backfill will be carried by a rigid pipe and apply equation (3); for semi-rigid pipe equation (47) should be used with $C_d = H/B_d$, and for flexible pipes equation (27) should be used. For a V-shaped or stepped trench the argument would not apply provided there is a cushion of at least 150 mm, but preferably 300 mm, of soil between the top of the pipe and the bottom of the concrete filling. In such a case, much of the weight can be borne directly by the soil at the sides provided the shear strength of the soil in the steps or V-slopes is adequate to carry it.

Lean concrete is a very useful material for reinstating the layers of the road itself and nothing in the foregoing should be construed as implying the contrary.

A.11.2 Foamed concrete

Foamed concrete is often used for reinstating trenches, particularly in highways, but it can also be used in new construction as an alternative to lean concrete.

Advantages of foamed concrete include the following:

- a) very easy to place;
- b) self-levelling and self-compacting;
- c) similar density and strength to original soil;
- d) lighter than lean or structural concretes;
- e) can be excavated;
- f) local materials can be reused; and
- g) can form part of the pavement construction (but not the surface course).

As it is self-compacting, there is likely to be good contact between the concrete and the wall of the trench, providing some reduction in loading. However, for small pipes (e.g. DN600) it is prudent to ignore this effect, assume that the full weight of the backfill will be carried by a rigid pipe, and apply equation (3); for a semi-rigid pipe equation (47) should be used with $C_d = H/B_d$; and for a flexible pipe equation (27) should be used.

Foamed concrete should conform to TRL Application Guide AG39 [N2]. Within highways, the material should be approved by the Highway Authority. A C2 mix (a compressive strength of 2 N/mm² to 10 N/mm² at 90 days) may be used for all layers up to sub-base in all road types and for sub-base in Road Types 3 and 4⁸⁾. A C4 mix (4 N/mm² to 10 N/mm² at 90 days) should be used for road base and for sub-base in Road Types 1 and 2⁸⁾.

Because of its fluidity, foamed concrete presents a drowning hazard for the first three to six hours after pouring.

A.11.3 Concrete surround to flexible pipes

Concrete surround is not normally needed for flexible pipes.

Where plastics pipes under internal pressure are to be surrounded in concrete, some, if not all, of the hoop stress will be transferred to the concrete, which is typically much stiffer than the pipe.

⁸⁾ As defined by the Highway Authorities and Utilities Committee [HAUC (UK)] [N3].

The designer should check specific cases but, except for small pipe sizes, should include steel reinforcement to carry the hoop load. Additionally, the pipe might need some soft wrapping material at entry and exit from the concrete surround to provide a transition from the rigid surround. For GRP pipe fittings surrounded by concrete it might be more cost and structurally effective to reinforce the concrete rather than to attempt to allow expansion and wrap the full length of the pipe or fitting in soft material. This is because compression of the wrap will induce hoop load on the concrete and the fittings themselves might need reinforcement to withstand the distortion caused by internal pressure.

A.11.4 Use of lightweight aggregates as bedding, sidefill and backfill

The weight of the backfill can become too great for the pipe laid at depth. One option for addressing this is for the designer to specify the use of lightweight backfill to reduce the load on the pipe. In such applications, it is normal to use the standard materials for the pipe embedments in order to ensure that the pipe receives the necessary support from its surround.

While, under certain circumstances, there can be a case for making use of lightweight aggregates for pipe embedments, the designer should proceed with care and the approach outlined in this document might not apply.

One reason for this is that the stiffness of a soil embedment that provides support to a flexible pipe is particularly sensitive to confining pressure, which reduces where lightweight aggregates are used.

While any beneficial increase in soil stiffness with increasing depth is conservatively neglected in both Marston-Spangler and Gumbel methods, the design values of E' and M_s given in [Table 14](#) and [Table 17](#), respectively, nevertheless assume use of pipe embedment and backfill of unit weight not less than 16 kN/m³.

Due to their relatively low shear strength associated with low confining pressure, lightweight aggregates also provide reduced resistance to potentially large asymmetrical deformations of shallow-buried flexible pipe of the type described in [5.2](#) and illustrated in [Figure 6](#).

The designer should in any event ensure that sufficient horizontal constraint is always provided by the sidefill and any trench walls present for all load conditions for the pipe in situations featuring lightweight aggregates.

Furthermore, where groundwater is present or could rise even in the short term above pipe invert level, design checks in accordance with [A.9.2](#) are essential because lightweight aggregates provide less resistance to flotation and in extremes could themselves be buoyant.

A.12 Thrust blocks

Thrust is generated in pressure pipelines where there are changes in direction, changes in diameter, closed valves, etc. Generally, such thrust is resisted either by concrete thrust blocks cast against undisturbed soil or by some form of continuity in the pipeline which enables the soil friction to be mobilized, such as welded joints in the case of steel or polyethylene or anchored joints in the case of ductile iron. It is essential that the anchorage length is checked to ensure its sufficiency, and the effect of any joints which will not transmit longitudinal tension accounted for. These joints might need to be tied or anchored via an in-line thrust block. Where anchored joints are used, for example using toothed gaskets for ductile iron pipe, allowance should be made for the movement which is required for the anchorage to take effect.

A.13 Pipes at shallow depths

A.13.1 General

Where pipes are to be laid at shallow depths, special precautions are required to reduce the risk of damage.

A.13.2 Protection to shallow pipelines

Shallow pipelines might need to be protected by more than normal bedding and backfill materials, especially when laid at an early stage of a contract where the cover is less than that specified.

Two clear examples of this are:

- when a sewer or drain is laid in a road which has only been brought up to formation level, where the pipe bedding has been designed assuming full depth of cover to finished road level; and
- where building works are taking place close to a drain run previously laid to a specification suitable for “fields and gardens” and the pipeline is subjected to unexpected loading due to delivery lorries, dumpers, fork lift trucks, etc.

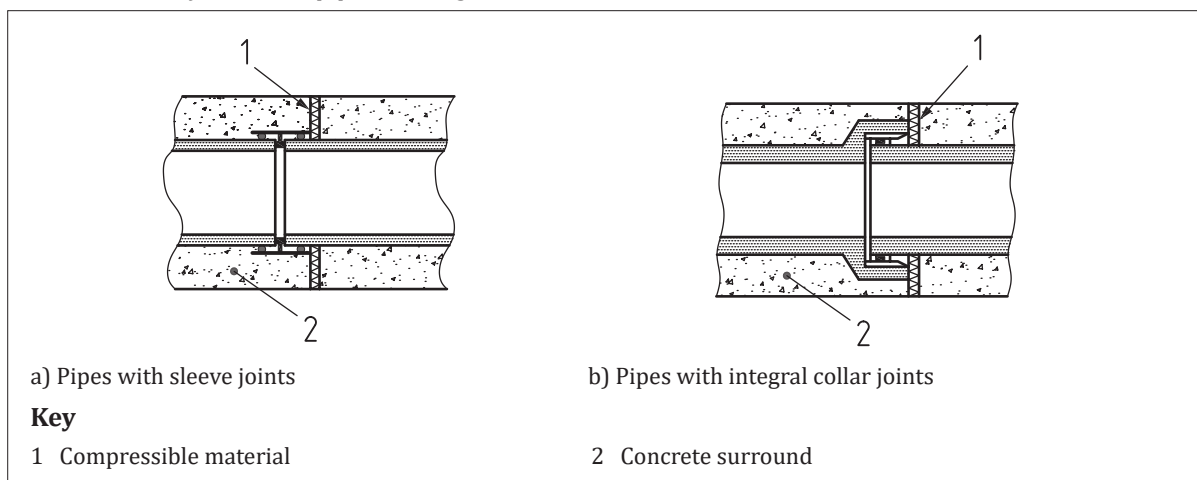
Where pipelines are likely to be exposed to construction activity and cannot be isolated by directing site traffic away from them, they can be temporarily bridged or protected by a stronger bedding combination.

In general, pipes can be safely laid using granular bedding without the need for a concrete bed or surround, provided that the effective depth of cover is at least 0.6 m, the required bedding factors are achieved and there are no additional imposed loads.

The flexibility of a pipeline bedded on, or surrounded with, concrete should normally be maintained by the provision of flexible construction joints through the concrete at pipe joints. These should be made from either bitumen-impregnated insulating board conforming to BS EN 622-4 or other equally compressible material such as expanded polystyrene. The board should be cut to fit the pipes and placed at the face of the pipe sockets or at one end of sleeve joints. The joint material should be at least 18 mm thick.

This procedure allows for flexible movement of the pipe joints while retaining the strength given by the concrete surround and should normally be carried out at every joint, as shown in [Figure A.11](#), particularly in building drainage applications.

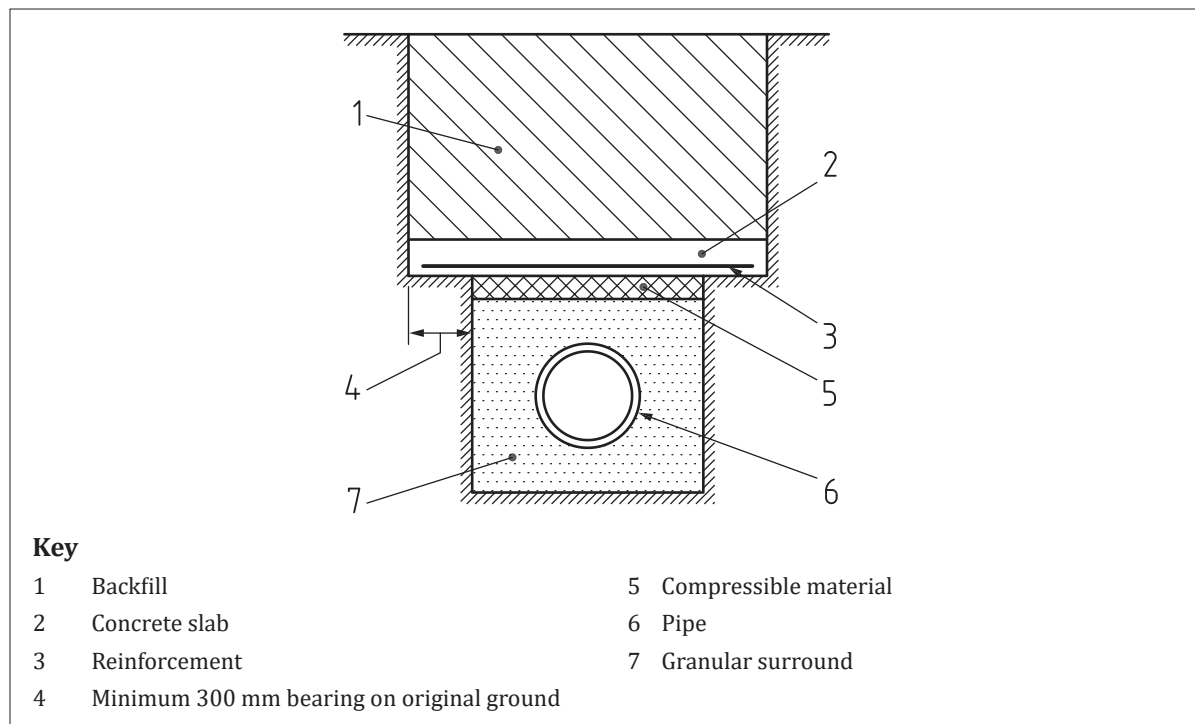
Figure A.11 — Protection of a shallow pipeline using concrete surround



Where more uniform support of the pipeline is found, the construction joints at pipe joints can be less frequent. However, they should be not more than 5 m apart.

An alternative method of protection is to use concrete slabs of sufficient strength to span the trench, as shown in [Figure A.12](#).

Figure A.12 — Protection of shallow pipeline with a slab



This method of protection is intended to isolate the pipeline from imposed loading, particularly traffic loading, which is critical at shallow depths. To achieve this, the slab and its bearing need to be structurally capable of carrying the imposed load. In roads with a reinforced concrete slab construction, this is easily accomplished by continuing the slab over the trench. Separate slabs might also need to be reinforced except, for example in gardens, where no wheel load is anticipated.

It is important that in all cases the slab completely spans the trench, bearing on the original ground on both sides, and does not simply rest within the trench. The width of bearing required will vary with the pipe diameter, trench width and ground conditions, but should be not less than 300 mm.

It is advisable to make sure that any movement or deflection of the slabs does not load the pipeline by introducing a layer of compressible material, such as expanded polystyrene, immediately below the slab. The pipe should be bedded and surrounded in appropriate material in the normal way, as shown in [Figure A.12](#).

In all cases, backfilling should be carried out in accordance with BS EN 1610:2015, Clause **11**.

Where concrete backfill to trenches is demanded for early permanent reinstatement, either using lean mix or foamed concrete, care should be taken that this is not allowed to generate a high concentrated load on the pipes. It is therefore necessary to ensure that the concrete backfill is well supported by the trench sides. This can be achieved by a stepped or battered trench. Concrete should not be placed between trench sheets which are subsequently removed, which would eliminate the friction between the concrete and the trench walls.

A.14 Laying rigid pipes with concrete beds or surrounds

Where a pipe is to be bedded in or surrounded by concrete, it is important to ensure that the trench formation will provide a firm foundation for the concrete bed, for example by using a blinding layer of weak concrete or granular material, or its value in strengthening the pipeline will be lost. It might also be necessary to excavate soft spots and compact in some more suitable material, such as granular bedding material or small hardcore.

It is important that the dimensions for concrete bedding or surround are sufficient to ensure that the specified bedding factors are achieved. Any concrete bed or surround should extend at least 150 mm either side of the pipe. The depth of concrete below the pipe, and above the pipe for a surround, should be at least 150 mm or one quarter of the outside diameter, whichever is the greater.

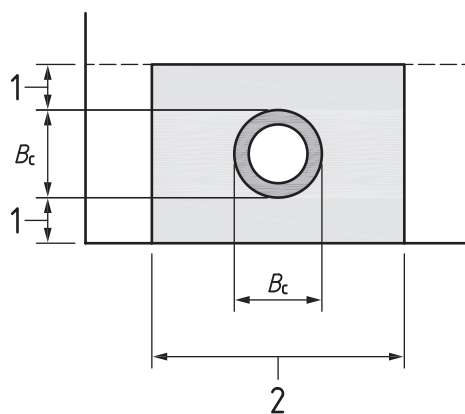
All concrete for pipe bedding should be of structural quality, C20/25 as a minimum, and should be thoroughly compacted into place. Care should be taken in placing concrete so as not to move pipes or construction joints.

Plain and reinforced concrete beddings and surrounds are illustrated in [Figure A.13](#). The use of concrete arches is not recommended because it is difficult to ensure adequate support of the base of the arch at the sides of the pipes. Additionally, since the width of the top of the concrete, rather than the outside diameter of the pipe, is used to calculate the load on the pipe/bed construction, this higher load can counter or even overtake the higher bedding strength of an arch or surround.

For reinforced concrete beds, the minimum transverse steel area should be not less than 0.4% of the area of the concrete in longitudinal section.

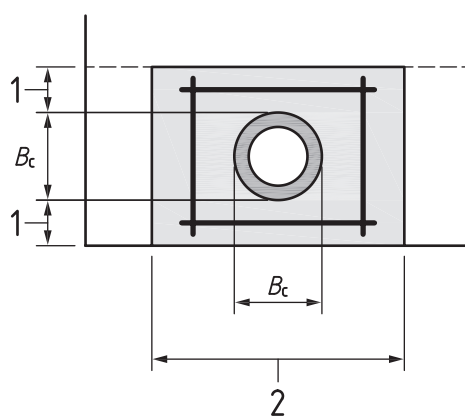
If the area of transverse steel is increased to 1.0% of the concrete area in longitudinal section in a concrete bed or surround both above and below the pipe, the bedding factor may be increased up to 4.8. This bedding factor has been derived from the 4.8 for a 1.0% reinforced concrete arch.

The area of vertical steel within the reinforced surround and longitudinal steel in bedding or surround is nominal for construction purposes, where flexibility at joints is maintained.

Figure A.13 — Plain and reinforced beddings and surrounds

a) Unreinforced concrete surround: bedding factor 4.5 (maximum diameter 600 mm)

SOURCE: ASCE *Gravity sanitary sewer design and construction* [15]



b) Reinforced concrete surround: bedding factor 4.8 (4 × 1% steel)

Reference: TRL Simplified tables [16]

Key

- 1 The larger of $\frac{1}{4}B_c$ or 150 mm
- 2 The larger of $1\frac{1}{2}B_c$ or $B_c + 300$ mm

A.15 Laying pipework under buildings

Pipework under buildings should be adequately supported and measures taken to reduce the risk of pipes being adversely affected by ground movement.

If ground movement is anticipated, additional precautions should be taken, such as increasing the pipeline gradient, to protect against backfall, and using shorter pipe lengths to add extra joint, increasing the pipeline's ability to accommodate any ground movement.

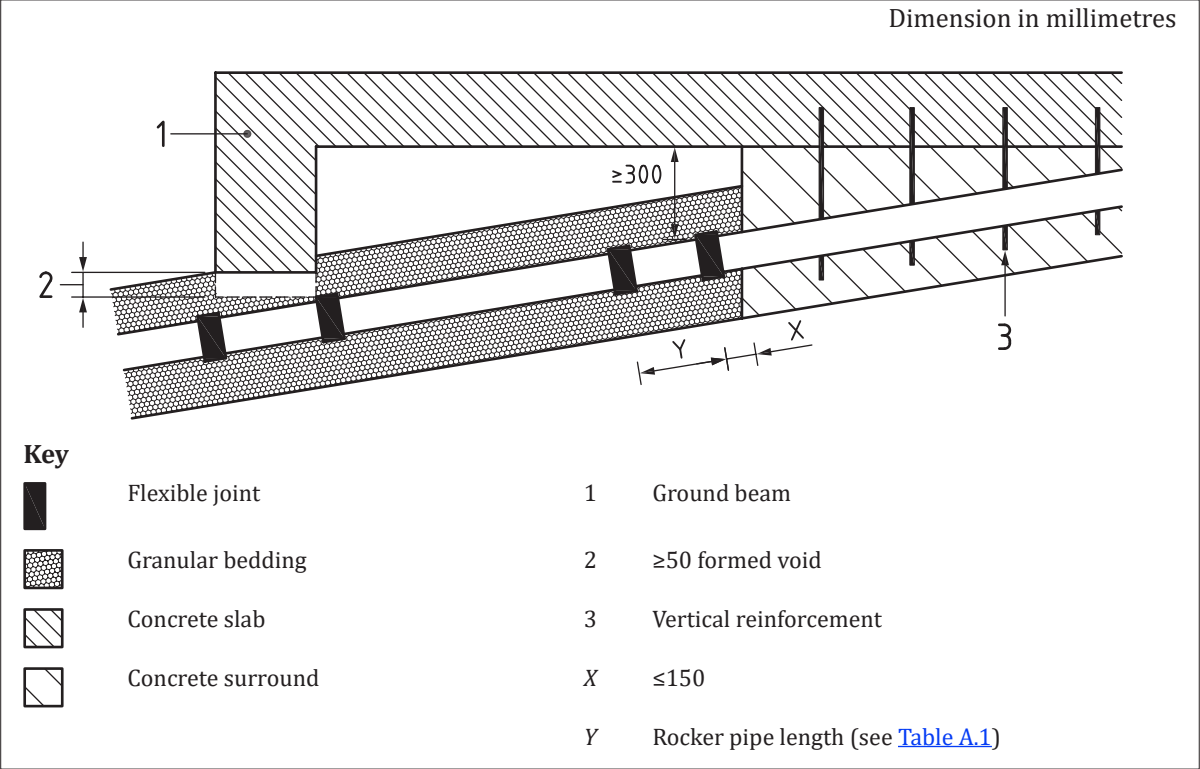
Bricks or blocks should not be used as temporary pipeline supports to achieve the required gradients. If they are not removed they can cause structural failure in rigid pipes and intrusions into the invert of flexible pipes, adversely affecting the drain run in operation, both hydraulically and structurally.

Providing a bed and haunch of granular material adds mass and stability to the pipework to resist the effects of in-service use or any maintenance operations, such as rodding and jetting, and prevents pipe joints from becoming dislocated and allowing the pipeline contents to leak into the subsoil or underfloor void.

A.16 Pipework under floor slabs

Where a pipeline has less than 300 mm cover under a loadbearing floor slab (see [Figure A.14](#)) it should be surrounded with concrete integral with the slab and preferably in the same pour. Additional pipeline flexibility should be provided upon leaving the concrete surround by the addition of rocker pipes (see [A.10.3](#) and [Figure A.14](#)). Further joints should be provided where the pipeline passes under the edge beam (see [Figure A.14](#)).

Figure A.14 — Pipework under a floor slab



A.17 Pipework laid in a void under a suspended floor

Plastics and ductile iron pipe systems can be suspended with height-adjustable bracketing systems, which can mitigate some of the issues associated with laying pipework under buildings. Access for the installation of suspended pipework might not be possible due to the sequence of construction.

Where pipework is laid in subsoils under a suspended floor slab, pipeline loads are anticipated to be minimal. Depending upon local circumstances the pipeline depth can vary from being installed in a shallow trench to being above the original ground level. Granular bed, sidefill and, in some cases, backfill will be required to maintain the pipeline’s vertical and horizontal alignment and stability.

Any soft ground should be excavated and replaced with a suitable compactible material to create a stable trench base upon which a granular bed can be formed (see [Figure A.14](#)).

A.18 Laying pipework through substructure walls

Where pipework is installed under a building, measures should be taken to prevent damage or misalignment as the pipework passes through substructure walls by adding rocker pipes (see [A.10.3](#)). This can be achieved by either:

- building in a length of pipe with its joints as close as possible to the wall faces with a maximum length of 150 mm (dimension X) which is connected to a rocker pipe with a maximum length of 600 mm (dimension Y) (see [Figure A.15](#)); or
- forming an opening to give at least a 50 mm clearance around the pipe and masking the opening with rigid sheet materials to prevent ingress of fill or vermin, ensuring that the void is also filled with a compressible sealant to prevent ingress of gas, though this might be difficult to achieve in practice (see [Figure A.16](#)).

Figure A.15 — *Pipework built in to the wall*

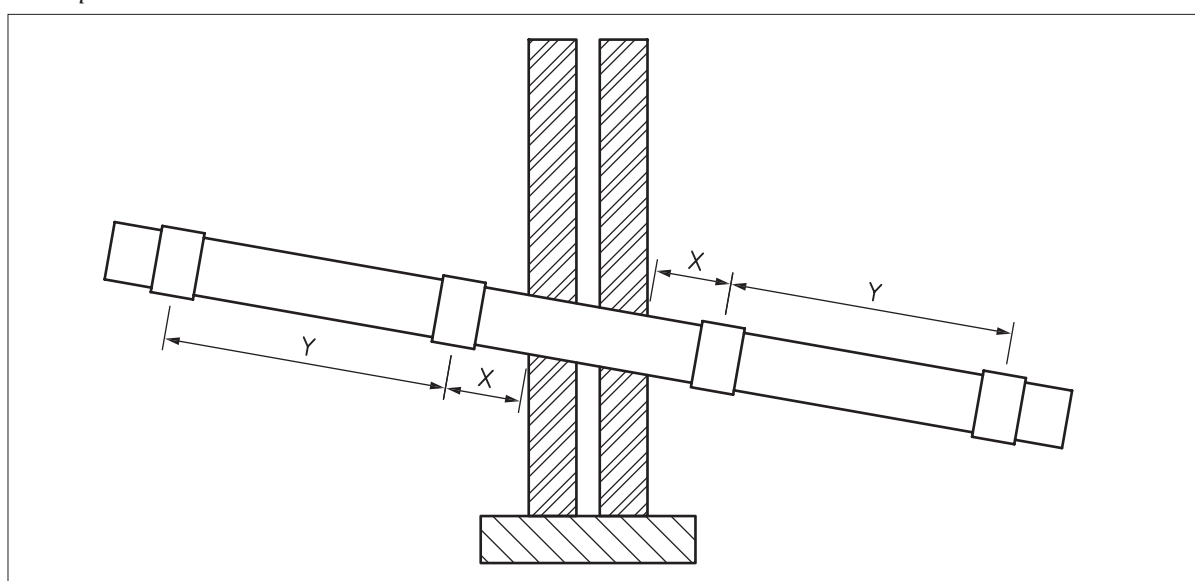
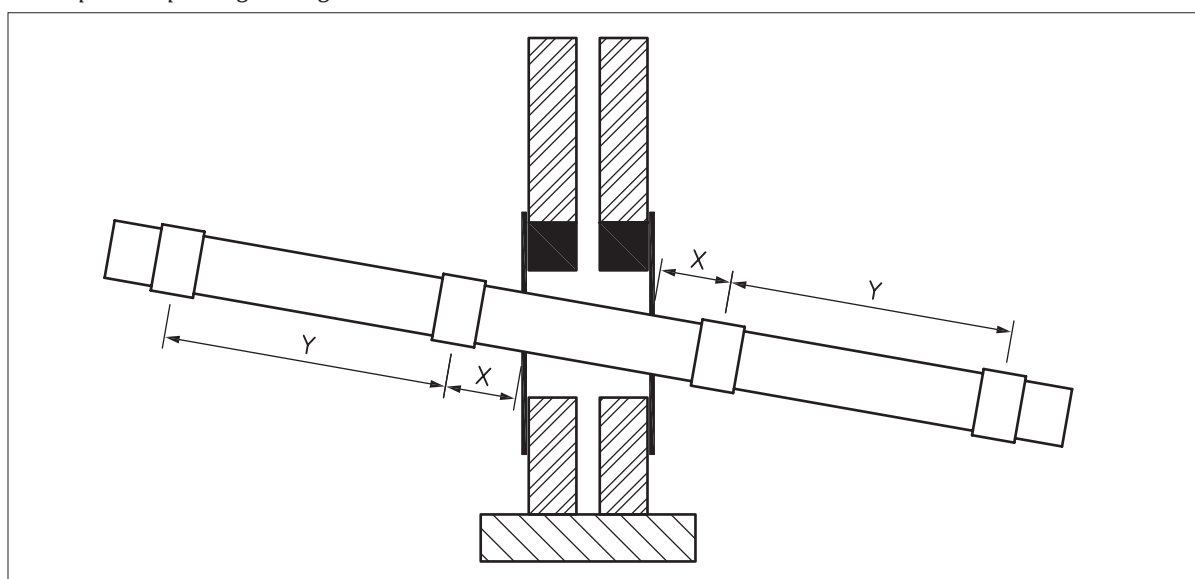


Figure A.16 — *Pipework passing through the wall*



A.19 Laying pipework in trenches adjacent to foundations

Where a building foundation is already constructed, the installation of a new pipeline should not reduce the support to the foundation or the building.

It should be determined at the design stage if the pipeline is to be a drain or a sewer, as the future pipeline ownership influences the design guidance.

NOTE For a drain, see the Building Regulations Approved Document H [17], Section 2 Foul Drainage, Diagram 8 and Protection to shallow pipelines 2.25 d; for a sewer or lateral drain, see Sewers for Adoption [18].

The Building Regulations Approved Document H [17], NHBC standards guidance [19] and *Sewers for Adoption* [18] contain cautionary guidance when proposing to install a pipeline in the 45° load zone originating from building foundations and in adjacent zones.

Account should be taken of the pipeline and all influencing zones of the proposed excavation for the pipeline. The trench bottom might be in the load zone, whereas the pipeline might not.

The provision of trench fill with mass concrete to support the adjacent foundation might be appropriate to stabilize the trench to support the foundation, whilst not being an appropriate solution for the pipeline. This can concentrate load on the pipe and/or reduce pipeline flexibility and prevent future external pipeline connections and is therefore not advised.

There are many specific and often conflicting issues to be taken into account for both the pipeline and the foundation that make general guidance unsuitable. It is recommended that a pipeline is not installed in these zones and is instead redesigned and/or rerouted to remove it from these zones.

A.20 Laying foundations adjacent to an existing drain or sewer

When designing a foundation to be constructed near an existing pipeline, or one that has been discovered during excavation for the foundation, the ownership of the pipeline should be established first as this impacts upon the design guidance to be followed (see A.21).

The relative influences of both the foundation and pipeline should be established from each to the other. The outcome of this could require a change to the foundation design or a rerouting of the pipeline or potentially prevent the foundation from being installed.

A.21 Longitudinal effects

A.21.1 General

Buried pipes experience different types of loading and hence exhibit different types of effects. Pipelines exhibit longitudinal effects as soon as successive cross-sections differ in cross-sectional properties or loading situation. Although the strength and stability of the cross-section is of primary interest for pressure containment, in many cases the longitudinal effects are responsible for pipeline failure.

The limit state conditions to be checked for longitudinal effects are set out in Table A.2.

Table A.2 — *Overview of limit state conditions to be checked for longitudinal effects*

Longitudinal effect	Pipe failure	Joint failure	Gradient loss
Uneven bedding	✓	✓	✓
Directional changes	✓	✓	–
Pressure loading	✓	✓	–
Temperature difference	–	✓	–
Soil settlements and subsidence	✓	✓	✓
Transition zones	✓	✓	✓

A.21.2 Failure modes

A.21.2.1 General

Failure of a water or sewage pipeline can result from longitudinal effects in various ways, causing leakage of the pipeline and contamination of the environment or, through local scour, endangering neighbouring structures or other pipelines.

Pipeline failures can result from rupture of pipe material or joints or appendages installed in the pipeline system. The latter are not discussed in this subclause, but such appendages should not be decisive in the safety evaluation of the pipeline system as a whole. Most pipeline failures, however, result from rupture failure or leakage of joints.

A.21.2.2 Pipeline failure

A pipeline can fail due to excessive stressing or straining of the pipe material. Overstressing of the material can arise from excessive overall or local loadings, while overstraining can result from instability or near instability of part of the pipeline structure. Special attention should be given to stresses and strains at bends, especially small radius and mitre bends, T-pieces and other discontinuities in the pipeline due to stress intensification. New pipeline material might be appropriate for the application, but ageing and other decay phenomena can affect the material such that over time the pipeline could cease to be fit for purpose and failures occur.

Pipeline failure can also be caused by third party damage. Cover, additional wall thickness, other pipe or coating material, rerouting or other possible measures should be used to prevent third-party damage and enhance the durability of the pipeline structure.

A.21.2.3 Joint failure

For a continuous pipeline formed from pipes that are joined together by a process that anticipates continuity of the pipeline properties over the joints, the joints should meet the same minimum strength requirements as the pipe. Special attention should be given to local discontinuity in stiffness of the pipe.

For a mechanically-jointed or articulated pipeline, the joints can have some freedom of movement. Due to longitudinal effects, the joints remain tight, and local stressing remains within the strength of the joint material shaped to constitute the joint. Axial displacements and angular rotations should remain within the limits of the joint construction, and the joints should be strong enough to withstand internal forces (axial forces, shear forces and bending and twisting moments) and local reaction forces from the soil or supporting structures. Special attention should be given to the tightness of the joints in a pipeline near bends and T-pieces, or at transitions from one material to the other.

A.21.2.4 Gradient loss

Larger displacements of a pipeline, especially by movement of the surrounding soil due to external effects (mining subsidence or consolidation settlements or shrinkage of disturbed soil), can lead to a

change in gradient of the pipeline. Where the pipeline operation or discharging depends on a gradient being maintained, such potential change in gradient should be allowed for in design.

A.21.3 Sources of longitudinal effects

A.21.3.1 General

In addition to the stress/strain imposed on a local cross-section of pipeline under pressure and local loadings (ring effect), longitudinal effects cause additional stressing and deformation of the pipeline (beam effect). Such longitudinal effects can influence the integrity of the pipeline as a whole and could lead to failure of the pipeline.

Shear, as opposed to beam failure, can occur due to differential settlement or lack of longitudinal support.

In addition to the effects on the pipe barrel, joints are vulnerable to longitudinal effects, including pull-out, shear and rotation, which can exceed their design capacity. These aspects should be checked, in particular, for pressure pipes.

A.21.3.2 Discontinuous support

Trench bottoms are never perfectly even and the soil properties beneath the pipeline can vary due to variations in the soil formation or the degree of disturbance of the soil. At transitions from one construction method to another or local rigid supports the pipeline can be sensitive to such variations in support. Ground anchors to prevent buoyancy effects or local piled supports can also constitute local rigid supports. Therefore, depending on the level of workmanship, a certain amount of construction subsidence should be assumed to account for these effects.

A.21.3.3 Directional changes

Directional changes can be designed by means of manufactured bends, field bends or T-pieces, or be shaped during construction by means of the rotational freedom in joints. Cold spring bending of pipes (elastic bending) can be applied to accommodate local space requirements. Elastic bending also occurs at towing siphons or horizontal directional drilling crossings. However, unintended directional changes can result from (larger) soil movements. In all cases the operational loadings of the pipe (pressure and temperature differences) should be accounted for to ensure sufficient strength and local stability of the pipeline under the pipe/soil interaction.

If the freedom of movement of the joints is constrained, internal forces in the pipeline and in joints should be analysed to ensure stressing and straining of the pipe material and joints remain within the limits specified.

A.21.3.4 Pressure loading

The pressure should be contained by the circumferential strength of the pipe cross-section. However, the resulting hoop strain has a contraction (Poisson) component in the longitudinal direction that causes the pipeline to contract axially, while the axial thrust force resulting from the pressure loading acting at bends, closed valves or T-pieces can cause an elongation of the pipeline. In an articulated pipeline the thrust force can be opposed by tensile resistant joints. Over a longer distance the thrust force is transferred to the soil by friction between pipe and soil. In a straight, articulated pipeline only the contraction remains. The thrust force and contraction strain work out as loadings on the pipeline system as a whole and should be accounted for to ensure tightness and safe operation of the pipeline.

A.21.3.5 Temperature differences

Operational temperature can differ from the temperature at which the pipeline was installed, the “construction temperature”.

Cooling down of the buried pipeline tends to contract the pipe material and pull-out joints. The soil counteracts this contraction and a tensile axial force results, reinforcing the tensile force from pressure loading.

A temperature increase expands the pipeline and the counteraction by the soil results in a decrease in tensile axial force or even a compressive force. Compressive forces can lead to axial instability if initial imperfections in the pipeline alignment are available and the compressive force becomes high enough to cause instability. Free spans can also initiate such instability. This phenomenon, however, only plays a role in relatively hot lines or in material with a high temperature expansion coefficient.

At short radius bends, due to the shape and reduced bending stiffness, expansion of the pipeline locally concentrates bending moments and soil pressure. In buried pipelines short radius bends should be avoided. Bends less than $R = 5D$ should be given special attention.

Where the pipeline is attached to rigid structures or is T-connected to another pipeline or branch or is in any way locally restrained, axial displacements can highly load that connection.

In stiff soils, expansion thrust forces at bends can cause concentrated lateral soil reactions that laterally and axially stress the cross-section.

Temperature influence on strength and stiffness properties of the pipe material should be taken into account.

A.21.3.6 Settlements and subsidence

The soil continuum in which the pipeline is buried can be subject to internal deformation. Such deformation can, for instance, result from displacements deep in the subsoil due to mining or past mining activities. Subsidence often occurs only in the vertical direction, but for larger deformations in the deep subsoil, horizontal deformations will also result at grade. Sudden shear deformations in the subsoil can extend to grade depending on the rock/soil profile beneath. In earthquake-prone areas such shear deformations often take place and destroy any pipeline that crosses a fault. Landslides can result from earthquakes as well, but slow soil flow can also occur at slopes. Such slopes should be avoided, unless pipelines are designed to accommodate the deformation loadings.

Apart from subsidence, consolidation settlements can result from additional loading from soil fills on a water-containing, compressible soil layer with low permeability. If the pipeline with content has a much greater dead-weight than the replaced soil, the difference in weight acts as an additional load to the subsoil and, as such, the pipeline causes consolidation settlement from its dead-weight. Heavy soil backfill in a trench relative to the specific weight of the surrounding native soil can also lead to consolidation of the subsoil.

In general, consolidation settlements are a one-time soil body deformation. However, shrinkage and swelling can also be seasonal.

Differences in deformations of the surrounding soil body along the pipeline are of importance. Care, for instance, should be taken if rigid supports are applied to pipelines in areas that are subject to subsidence or settlements. Due to downward movement of the soil, the overburden topsoil can load the pipeline to a higher extent than the weight of the soil on top of the pipeline only. At the transition to a non-supported section of the pipeline, heavy bending can occur.

In addition to bending the pipeline, local lateral soil reactions act on the cross-section of the pipe. In particular, low-pressure, thin-walled, large-diameter pipes can become highly stressed as a result.

Settlements or subsidence can also cause axial tensile forces in the pipeline, due to the extra length required to follow the displacement profile. For articulated pipelines such extra length is obtained by pulling out of the joints, with possible leakage as a result. Joints should be capable of absorbing such axial displacements and angular rotations.

In locations where frost and thawing cycles occur during winter, ice lenses can locally cause frost heave in soils sensitive to this phenomenon. Care should be taken to avoid backfill of trenches with such sensitive soils. Heavy frost in the top layer of soil can stick to surrounding rigid structures and apply extra loading to a pipeline. Extra frost influence can occur at locations where a pipeline is near a wind-blown, soil-retaining structure or in crossings of unpaved roads regularly cleared of snow.

Special attention should be given to pipeline sections at crossings with roads, railways, waterways, rivers or dykes because, due to soil fills, settlement profiles can show considerable settlement differences over short distances. At these crossings the alignment of the pipeline can show a narrow profile with several bends to follow the soil profile. The flexibility behaviour of the pipeline under the combination of operational and settlement loads can become complex, requiring advanced computation to obtain a realistic result.

In a river, local scour can expose the pipeline, forming a free span. Current loadings are then applied to the pipeline, and floating debris or ship anchors can hook behind or be dropped down upon the exposed pipeline. Such exposure should be avoided.

Settlements and subsidence of the soil continuum, axial forces, bending moments and shear forces can impose strain on the buried pipe both in axial and circumferential directions. The soil body forces the pipeline into the deformation profile of the soil, whereas the stiffness of the pipeline counteracts this deformation loading. The difference in displacement profile produces an equilibrium system of soil loadings and reactions acting on the pipeline, generating internal forces.

The pipeline should be capable of withstanding such internal forces and local lateral loading within the context of allowable stresses, strains, deformations and displacements, rotations and translations in joints and within the limits of gradient losses.

For the majority of pipelines it is sufficient to prove the impossibility of soil settlement or subsidence or show that the pipeline can follow the deformation profile of the soil with sufficiently low impact on the pipeline.

However, in more complex situations, the pipeline should be carefully analysed.

A.21.3.7 Transition zones

Transition zones are pipeline sections where the conditions of the pipeline change considerably due, for example, to a change of construction method (for instance, a boring connected to a trench-laid pipeline) or connection to a pipeline of different diameter or to a rigid structure (for instance, a pumping station). Sections of a pipeline supported by piles or other rigid supports show a transition zone at the connection to a non-pipe supported section.

If no soil settlements or mining subsidence are expected, it might still be necessary to take account of the potential for a jump in construction subsidence, especially if a deeper excavation has been made locally, for instance at a construction pit, where greater subsidence of the disturbed soil can be expected due to shrinkage. In general, at each location where there is a transition in circumstances for the pipeline the possibility of a transition effect on pipeline behaviour should be taken into account.

Care should be taken in areas where former water courses are filled with sediments due to geological changes. Such irregularities in the subsoil can be missed by regular soil mechanical investigation with borings/soundings at intervals of 50 m to 100 m along the pipeline routing.

A.22 Bedding materials for pipes

A.22.1 Types of bedding

Bedding materials suitable for the embedment of pipes can come from a variety of sources, extending the range beyond primary aggregates [see a) to e)]. Care is necessary in selection to ensure that these materials will provide adequate support and protection to the pipe and not cause chemical degradation of the pipe material.

a) Natural aggregate

Aggregate from primary mineral sources which has been subjected to nothing more than mechanical processing (quarried or dredged).

b) Recycled aggregate

Aggregate resulting from the processing of inert and inorganic material previously used in construction, which can be used with all bedding classes subject to a satisfactory compaction fraction value.

c) Manufactured or secondary aggregate

Aggregate of mineral origin resulting from an industrial process involving thermal or other modification or a by-product from another industry or process.

d) Lightweight aggregate (manufactured or secondary)

Aggregate of mineral origin having a particle density not greater than 2 000 kg/m³ (see [A.11.4](#)).

e) As-dug native subsoils

Excavated material which is suitable for pipe embedment without processing, but which should not be contaminated with domestic, building or industrial waste and should be free from organic and combustible material. Subsoils should not be used in a frozen state.

A.22.2 Suitable materials

A.22.2.1 Compaction fraction (CF)

Materials with the compaction fractions in [Table A.3](#), as demonstrated by testing in accordance with [A.23](#), are suitable for particular applications.

The simple test in [A.23](#) can be carried out on site to determine the level to which an aggregate can be compacted with minimal mechanical effort. This compares a loosely filled volume to a compacted volume of the same mass, with the result expressed as a fraction. The lower the fraction, the greater the self-compacting properties and therefore the structural support afforded to the pipe.

Table A.3 — *Compaction fractions for bedding materials*

Compaction fraction	Materials
CF 0.15 or less	Material suitable for use with rigid pipes with bedding classes (F - 1.9), (B - 2.5) and (S - 2.5), or for non-pressure flexible pipes
CF between 0.15 and 0.3	Material suitable but requires extra care in compaction Not suitable if the material is subject to waterlogged conditions after laying Material suitable for use with rigid pipes with bedding classes (F - 1.5), (B - 1.9) and (S - 2.2), and flexible pressure pipes
CF greater than 0.3	Material only suitable for use with rigid pipe bedding classes (D and N - 1.1)

A.22.2.2 Gradings

Gradings for bedding material for various pipe diameters are given in [Table A.4](#) and can be obtained from manufacturers' instructions.

Table A.4 — *Gradings*

Pipe diameter DN	Single sized mm	Graded mm
100 – 125	20	14 – 5
150 – 200	20	14 – 5
225 – 300	20	20 – 5
375 – 500	20	14 – 5
Exceeding 500	20 or 40	14 – 5
		20 – 5
		40 – 5

A.22.2.3 Geometrical criteria

Bedding materials should conform to the geometrical requirements of BS EN 13242:2002+A1:2007, Clause 4, for:

- a) aggregate size;
- b) grading;
- c) flakiness index; and
- d) percentage of crushed or broken particles.

A.22.2.4 Physical criteria

Bedding materials should conform to the physical requirements of BS EN 13242:2002+A1:2007, Clause 5, for:

- a) particle density; and
- b) water absorption.

A.22.2.5 Water soluble sulfate

Bedding materials should conform to the water-soluble sulfate content requirements of BS EN 13242:2002+A1:2007, 6.4.

A.22.2.6 Maximum recommended level of impurity (by mass)

[Table A.5](#) sets out the maximum recommended level of impurity (by mass) for bedding material.

Table A.5 — *Maximum recommended level of impurity (by mass)*

Impurity	Maximum percentage by mass
Asphalt and tar	5
Wood	2
Glass	5
Sulfates. Acid soluble SO ₃	1
Other foreign material	1 by volume if ultra-lightweight
Maximum recommended level of fines	
Fines	5
Masonry content (plaster/mortar)	5

A.22.2.7 Certification of aggregates for embedment

A.22.2.7.1 Natural aggregates

Each supply of aggregate to site should be provided with such documentation as is necessary to support evidence of conformity with BS EN 13242:2002+A1:2007.

A.22.2.7.2 Recycled aggregate and end of waste criteria

In addition to the documentation recommended in [A.22.2.7.1](#), recycled aggregates should also conform to the WRAP Quality Protocol, *Aggregates from inert waste* [N3].

Aggregates from recovered inert waste produced in accordance with the WRAP Quality Protocol are not likely to be waste and might therefore be exempt from the requirements of the EU Waste Framework Directive [\[20\]](#).

A.23 Compaction fraction test

A.23.1 Apparatus

A.23.1.1 *Open-ended cylinder*, approximately 250 mm long and 150^{+10}_{-5} internal diameter (DN150 pipe is suitable).

A.23.1.2 *Metal rammer*, with striking face approximately 40 mm diameter and weighing 0.8 kg to 1.3 kg.

A.23.1.3 *Rule*.

A.23.2 Method

Obtain a representative sample which is more than sufficient to fill the cylinder ([A.23.1.1](#)) (approximately 10 kg). Ensure that the moisture content of the sample does not differ from the bulk of material at the time of its use in the trench.

To obtain this sample, heap approximately 50 kg of the material to be tested into a cone shape on a clean flat surface after turning over the material three times. Flatten the top of the cone then divide with a spade vertically down the centre into four quarters. Discard one pair of the opposite quarters and remix the other pair of opposite quarters into a cone shape. Repeat this procedure until the required mass of sample remains.

Place the cylinder on a firm flat surface and gently pour the sample material into it, loosely and without tamping. Strike off the top surface level with the top of the cylinder and remove all surplus spilled material. Lift the cylinder up clear of its contents and place on a fresh area of flat surface. Place approximately one quarter of this material back in the cylinder and tamp vigorously with the metal rammer ([A.23.1.2](#)) until no further compaction can be obtained. Repeat with the second quarter, tamping as before, and so on for the third and fourth quarters, tamping the final surface as level as possible. Do not tamp so vigorously as to break the material being compacted.

A.23.3 Determination of compaction fraction

Measure from the top of the cylinder to the surface of the compacted material using the rule ([A.23.1.3](#)) and divide this measurement by the height of the cylinder to obtain the compaction fraction of the material under test.

A.24 Load coefficients for the incomplete projection condition

Determine the load coefficient, C_p , using equation (5) with the height of the plane of equal settlement, H_e , as an input. The determination of this parameter is not given in BS EN 1295-1:2019,

but the TRRL report *A guide to design loadings for buried rigid pipes* [N1], which is based on original work by Spangler [21], showed that H_e can be found from:

$$\frac{e^{2K\mu H_e/B_c} - 1}{2K\mu} \left(\frac{1}{2K\mu} + \frac{H - H_e}{B_c} + \frac{r_{sd}\rho}{3} \right) + \frac{1}{2} \left(\frac{H_e}{B_c} \right)^2 + \frac{r_{sd}\rho}{3} \left(\frac{H - H_e}{B_c} \right) e^{2K\mu H_e/B_c} - \frac{H_e}{2K\mu B_c} - \frac{HH_e}{B_c^2} = r_{sd}\rho \frac{H}{B_c} \quad (\text{A.9})$$

Solutions to this equation for specific values of $r_{sd}\rho$ are given in [Table 7](#); an alternative is to use the following approximation:

$$C_c = \left[0.68(r_{sd}\rho)^{0.41} + 1 \right] \frac{H}{B_c} - 0.1(r_{sd}\rho)^{0.57} \quad (\text{A.10})$$

The error, when compared with [Table 7](#), is within $\pm 4.4\%$ in the range: $r_{sd}\rho$ of 0 to 5 and $H/B_c \geq 0.3$. When the load is given by the incomplete projection condition then the error is reduced and is within -1.4% to $+2.8\%$. For $H/B_c < 0.3$ the complete projection condition can be assumed.

The errors in using the approximation instead of the equations in [Table 7](#) are detailed in [Table A.6](#).

Table A.6 — Equation errors (percent)

$r_{sd}\rho$	H/B_c						
	0.3	0.5	0.7	1	10	100	500
0	0	0	0	0	0	0	0
0.1	+1	+1.7	+2.1	+2.3	+2.8	+2.8	+2.8
0.3	+2	+1.9	+1.9	+1.9	+1.8	+1.8	+1.8
0.5	+1.6	+1.3	+1.1	+1	+0.8	+0.8	+0.8
0.7	+2	+1	+0.7	+0.4	-0.1	-0.2	-0.2
1	+4.4	+2.1	+1.2	+0.6	-0.5	-0.6	-0.6
2	+3.3	+1	+0.3	-0.3	-1.3	-1.4	-1.4
3	+2.1	+0.8	+0.3	0	-0.6	-0.6	-0.6
4	-1.6	-0.6	-0.2	0	+0.4	+0.5	+0.5
5	-4.2	-1.4	-0.4	+0.3	+1.4	+1.5	+1.6

A.25 Calculation of surcharge pressures in [Table 2](#)

The values in [Table 2](#) were determined by considering both Load Model 1 (General Verification) and Load Model 2 as defined in BS EN 1991-2:2003. Load Model 3 was not considered as pipes are short span structures and Load Model 3 is not generally critical.

A wheel contact area of (400 × 400) mm was assumed, with the appropriate axle spacing dispersed through the pavement and soil based on dispersion angles defined in BS EN 1991-2. The angle for dispersion through the pavement or concrete slab is 45° [see BS EN 1991-2:2003, **4.3.6** (2)] and 30° for soil (see BS EN 1991-2:2003, **4.9.1**, Note 2). An assumption of 150 mm for the pavement depth was used.

To enable production of [Table 2](#), where dispersal zones of more than 2 wheels overlap, the total load was spread over the combined dispersal zone to calculate the pressure. For a rigorous analysis, see PD 6694-1.

The critical value of Load Model 1 and Load Model 2 determined the values in [Table 2](#).

A.26 Pipe calculation examples

A.26.1 Design of a concrete pipeline in accordance with [Clause 6](#) Rigid pipe design

A.26.1.1 Example 1

Design criteria:

Unreinforced 450 mm diameter concrete pipe to BS 5911-1:2002+A2:2010, Class 120

Cover depth from road level to top of pipe = 4.5 m

Subject to main road loading

Control over installation

Design data:

Narrow trench design

Internal diameter DN = 0.45 m

Wall thickness t = 0.06 m

External diameter B_c = 0.57 m

Trench width B_d = 1.2 m

Unit weight of soil γ = 19.6 kN/m³

No site investigation information so consider conservative properties in [6.6](#).

Wide trench load

[6.10.2](#)

$$W'_c = C_c \gamma B_c^2 \quad \text{Eq(3)}$$

$$\text{From } \textcolor{blue}{6.5} \ r_{sd} \rho = 0.5 \text{ for bedding class B or S}$$

$$C_c = 1.5 \times (4.5/0.57) - 0.07 = 11.77 \quad \text{Table 7}$$

$$W'_c = 11.77 \times 19.6 \times 0.57^2 = 74.95 \text{ kN/m}$$

Narrow trench load

$$W_c = C_d \gamma B_d^2 \quad \text{Eq(6)}$$

where

$$C_d = \frac{1 - e^{-2K\mu'H/B_d}}{2K\mu'} \quad \textcolor{blue}{6.6}$$

$$C_d = \frac{1 - e^{(-2 \times 0.13 \times 4.5/1.2)}}{2 \times 0.13} = 2.39$$

NOTE C_d may also be obtained using charts in reference [\[3\]](#).

$$W_c = 2.39 \times 19.6 \times 1.2^2 = 67.4 \text{ kN/m}$$

$$\text{Considering the lower value of Eq (3) and (6)} \quad W'_c = 67.4 \text{ kN/m}$$

Concentrated surcharge load[6.10.4](#)

$$W_{\text{csu}} = P_s B_c \quad \text{Eq(7)}$$

From [Table 2](#) $H = 4.5$, $P_s = 19 \text{ kN/m}^2$

$$W_{\text{csu}} = 19 \times 0.57 = 10.83 \text{ kN/m}$$

Water load[6.10.5](#)

$$W_w = C_w \gamma_w \pi (D - t)^2 / 4 \quad \text{where } C_w = 0.75 \quad \text{Eq(8)}$$

$$D = 0.45 + 0.06 = 0.51$$

$$W_w = 0.75 \times 9.81 \times 3.142 \times (0.51 - 0.06)^2 / 4 = 1.2 \text{ kN/m}$$

Total external load[6.10.6](#)

$$\begin{aligned} W_e &= W_c + W_{\text{csu}} + W_w \\ &= 67.4 + 10.83 + 1.2 = 79.43 \text{ kN/m} \end{aligned} \quad \text{Eq(9)}$$

Bedding factor required[6.11](#)

$$F_m \geq W_e F_{\text{se}} / W_t \text{ (or } W'_t) \quad \text{Eq(10)}$$

$$\text{Unreinforced pipe (DN < 0.675)} \quad F_{\text{se}} = 1.25 \quad \text{Table 8}$$

$$\text{Class 120 pipe} \quad W_t = 120 \times 0.45 = 54 \text{ kN/m}$$

$$F_m \text{ required} \geq 79.43 \times 1.25 / 54 = 1.83 \quad \text{From Table 5}$$

Select Class B or S bedding where for narrow trench F_m provided = 1.9 or 2.2 (exceeding required minimum of 1.83)

A.26.1.2 Example 2**Design criteria:**

Unreinforced 450 mm diameter concrete pipe to BS 5911-1:2002+A2:2010, Class 120

Cover depth from road level to top of pipe = 4.5 m

Subject to main road loading

Limited control over installation and trench width

Design data:**Wide trench design**

$$\text{Internal diameter} \quad \text{DN} = 0.45 \text{ m}$$

$$\text{Wall thickness} \quad t = 0.06 \text{ m}$$

$$\text{External diameter} \quad B_c = 0.57 \text{ m}$$

$$\text{Trench width} \quad B_d = 1.2 \text{ m}$$

Unit weight of soil $\gamma = 19.6 \text{ kN/m}^3$

No site investigation information so consider conservative properties in [6.6](#).

Wide trench load, concentrated surcharge load, water load – as for [A.26.1.1](#)

Total external load [6.10.6](#)

$$W_e = W'_c + W_{csu} + W_w \quad \text{Eq(9)}$$

$$= 74.95 + 10.83 + 1.2 = 86.98 \text{ kN/m}$$

Bedding factor required [6.11](#)

$$F_m \geq W_e F_{se} / W_t \text{ (or } W'_t) \quad \text{Eq(10)}$$

Unreinforced pipe (DN < 0.675) $F_{se} = 1.25$ [Table 8](#)

Class 120 pipe $W_t = 120 \times 0.45 = 54 \text{ kN/m}$

F_m required $\geq 86.98 \times 1.25 / 54 = 2.01$ From [Table 5](#)

Select Class B or S bedding where for wide trench F_m provided = 2.5 (exceeding required minimum of 2.0)

A.26.1.3 Example 3

Design criteria:

Reinforced 1 200 mm diameter concrete pipe to BS 5911-1:2002+A2:2010, Class 120

Cover depth from road level to top of pipe = 4.2 m

Subject to main road loading

Control over installation and trench width

Design data:

Narrow trench design

Internal diameter DN = 1.2 m

Wall thickness $t = 0.12 \text{ m}$

External diameter $B_c = 1.44 \text{ m}$

Trench width $B_d = 2.4 \text{ m}$

Unit weight of soil $\gamma = 19.6 \text{ kN/m}^3$

No site investigation information so consider conservative properties in [6.6](#).

Wide trench load**6.10.2**

$$W'_c = C_c \gamma B_c^2 \quad \text{Eq(3)}$$

From 6.5 $r_{sd}\rho$ = 0.5 for bedding class B or S

$$C_c = 1.5 \times (4.2/1.44) - 0.07 = 4.30 \quad \text{Table 7}$$

$$W'_c = 4.30 \times 19.6 \times 1.44^2 = 174.76 \text{ kN/m}$$

Narrow trench load

$$W_c = C_d \gamma B_d^2 \quad \text{Eq(6)}$$

where

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

$$K\mu' = 0.13 \quad \text{6.6}$$

$$C_d = \frac{1 - e^{(-2 \times 0.13 \times 4.2/2.4)}}{2 \times 0.13} = 1.41$$

NOTE C_d may also be obtained using charts in reference [3].

$$W_c = 1.41 \times 19.6 \times 2.4^2 = 159.18 \text{ kN/m}$$

Considering the lower value of Eq (3) and (6) $W'_c = 159.18 \text{ kN/m}$

Concentrated surcharge load**6.10.4**

$$W_{csu} = P_s B_c \quad \text{Eq(7)}$$

From Table 2 $H = 4.5$, $P_s = 19 \text{ kN/m}^2$

$$W_{csu} = 19 \times 1.44 = 27.36 \text{ kN/m}$$

Water load**6.10.5**

$$W_w = C_w \gamma_w \pi (D - t)^2 / 4 \quad \text{where } C_w = 0.75 \quad \text{Eq(8)}$$

$$D = 1.2 + 0.12 = 1.32$$

$$W_w = 0.75 \times 9.81 \times 3.142 \times (1.32 - 0.12)^2 / 4 = 8.32 \text{ kN/m}$$

Total external load**6.10.6**

$$W_e = W_c + W_{csu} + W_w \quad \text{Eq(9)}$$

$$= 159.18 + 27.36 + 8.32 = 194.86 \text{ kN/m}$$

Bedding factor required**6.11**

$$F_m \geq W_e F_{se} / W_t \text{ (or } W'_t) \quad \text{Eq(10)}$$

Reinforced pipe (DN > 0.675)	F_{se}	= 1.5	Table 8
Class 120 pipe	$W_t = 120 \times 1.2$	= 144 kN/m	
F_m required	$\geq 194.86 \times 1.5/144$	= 2.03	From Table 5

Select Class S bedding where for narrow trench F_m provided = 2.2 (exceeding required minimum of 2.03)

A.26.1.4 Example 4

Design criteria:

Reinforced 1200 mm diameter concrete pipe to BS 5911-1:2002+A2:2010, Class 120

Cover depth from road level to top of pipe = 4.2 m

Subject to main road loading

No control over installation and trench width

Design data:

Wide trench design

Internal diameter	DN	= 1.2 m
Wall thickness	t	= 0.12 m
External diameter	B_c	= 1.44 m
Trench width	B_d	= 2.4 m
Unit weight of soil	γ	= 19.6 kN/m ³

No site investigation information so consider conservative properties in [6.6](#).

Wide trench load, concentrated surcharge load, water load – as for [A.26.1.3](#)

Total external load [6.10.6](#)

$$W_e = W'_c + W_{csu} + W_w \quad \text{Eq(9)}$$

$$= 174.76 + 27.36 + 8.32 = 210.44 \text{ kN/m}$$

Bedding factor required [6.11](#)

$$F_m \geq W_e F_{se} / W_t \text{ (or } W'_t) \quad \text{Eq(10)}$$

Reinforced pipe (DN > 0.675)	F_{se}	= 1.5	Table 8
Class 120 pipe	$W_t = 120 \times 1.2$	= 144 kN/m	
F_m required	$\geq 210.44 \times 1.5/144$	= 2.19	From Table 5
DN > 0.9 m	$H/B_c = 4.2/1.44$	= 2.91	
F_m provided		= 2.36	Table 6

Select Class B or S bedding for wide trench where F_m provided = 2.36 (exceeding required minimum of 2.19)

A.26.1.5 Example 5: Limit state design (see 6.13)**Design criteria:**

Reinforced 1 200 mm diameter concrete pipe to BS 5911-1:2002+A2:2010, Class 120

Cover depth from road level to top pipe = 4.2 m

Subject to main road loading

No control over installation and trench width

Design data:**Narrow trench design**

Internal diameter	DN	= 1.2 m
Wall thickness	t	= 0.12 m
External diameter	B_c	= 1.44 m
Trench width	B_d	= 2.4 m
Unit weight of soil	γ	= 19.6 kN/m ³

No site investigation information so consider conservative properties in 6.6.

Partial safety factors

Table 9	SLS	ULS
γ_G	1.0	1.35
γ_Q	1.0	1.35
$\gamma_{G;intw}$	1.0	1.20

Trench loads, Concentrated surcharge load, Water load – as for A.26.1.4**Design value of actions**[6.13.5](#)

Serviceability	W_e	$= W_c + W_{csu} + W_w$	
		$= 174.76 + 27.36 + 8.32$	$= 210.44 \text{ kN/m}$
Ultimate	$W_{e,Ed}$	$= \gamma_G W_c + \gamma_Q W_{csu} + \gamma_{G;intw} W_w$	Eq(15)
		$= (1.35 \times 174.76) + (1.35 \times 27.36) + (1.2 \times 8.32) = 282.84 \text{ kN/m}$	

Bedding factor required[6.13.6](#)

ULS	$F_m \geq W_{e,Ed} / W_t$		Eq(16)
-----	---------------------------	--	--------

F_m	$\geq 282.84 / 144$	$= 1.96$
-------	---------------------	----------

SLS	$F_m \geq W_{e,Ed} / W_p$		Eq(17)
-----	---------------------------	--	--------

$$\text{where } W_p = W_t / F_{se} = 144 / 1.5 = 96 \text{ kN/m}$$

$$F_m \geq 210.44 / 96 = 2.19$$

Reinforced pipe, consider critical F_m required of SLS and ULS, F_m required = 2.19

$$DN > 0.9 \text{ m} \quad H/B_c = 4.2/1.44 = 2.92$$

$$F_m \text{ provided} = 2.36 \quad \text{From Table 6}$$

Select Class B or S bed for wide trench where F_m provided = 2.36 (exceeding required minimum of 2.19)

A.26.2 Design of a vitrified clay pipeline in accordance with [Clause 6](#) Rigid pipe design

Design criteria:

300 mm diameter clay pipe to BS EN 295-1, Class 240

Cover depth from road level to top of pipe = 6.0 m

Subject to main road loading

Limited control over installation and trench width

Design data:

Wide trench design

$$\text{Internal diameter} \quad DN = 0.30 \text{ m}$$

$$\text{Wall thickness} \quad t = 0.029 \text{ m}$$

$$\text{External diameter} \quad B_c = 0.358 \text{ m}$$

$$\text{Trench width} \quad B_d = \text{Wide}$$

$$\text{Unit weight of soil} \quad \gamma = 19.6 \text{ kN/m}^3$$

No site investigation information so consider conservative properties in [6.6](#).

Wide trench load

[6.10.2](#)

$$W'_c = C_c \gamma B_c^2 \quad \text{Eq(3)}$$

$$\text{From } \text{6.5 } r_{sd} \rho = 0.5 \text{ for bedding class B or S}$$

$$C_c = 1.5 \times (6.0/0.358) - 0.07 = 25.07 \quad \text{Table 7}$$

$$W'_c = 25.07 \times 19.6 \times 0.358^2 = 62.98 \text{ kN/m}$$

Concentrated surcharge load

[6.10.4](#)

$$W_{csu} = P_s B_c \quad \text{Eq(7)}$$

$$H = 6.0, P_s = 15 \text{ kN/m}^2 \quad \text{From Table 2}$$

$$W_{csu} = 15 \times 0.358 = 5.37 \text{ kN/m}$$

Water load[6.10.5](#)

$$W_w = C_w \gamma_w \pi (D - t)^2 / 4 \quad \text{where } C_w = 0.75$$

Eq(8)

$$D = 0.30 + 0.029$$

$$= 0.329$$

$$W_w = 0.75 \times 9.81 \times 3.142 \times (0.329 - 0.029)^2 / 4$$

$$= 0.52 \text{ kN/m}$$

Total external load[6.10.6](#)

$$W_e = W'_c + W_{csu} + W_w$$

Eq(9)

$$= 62.98 + 5.37 + 0.52$$

$$= 68.87 \text{ kN/m}$$

Bedding factor required[6.11](#)

$$F_m \geq W_e F_{se} / W_t \text{ (or } W'_t)$$

Eq(10)

$$\text{Clay pipe} \quad F_{se} = 1.25$$

[Table 8](#)

$$\text{Class 240 pipe} \quad W_t = 240 \times 0.30$$

$$= 72 \text{ kN/m}$$

$$F_m \text{ required} \geq 68.87 \times 1.25 / 72$$

$$= 1.20$$

From [Table 5](#)**Select Class F bedding in a wide trench condition where F_m provides = 1.9****or****Select Class B or S bedding in a wide trench condition where F_m provides = 2.5**

From [Table 11](#), a DN 300 clay pipe embedded within a class F, B or S bedding can be laid between the depths of 0.6 m and 6.0 m (>6.0 to be confirmed by calculation).

A.26.3 Design of a polyethylene pipeline in accordance with [Clause 7](#) Flexible pipe design**Design criteria:**

1 200 mm diameter structured-wall polyethylene pipe with Type A symmetrical wall profile

Ring stiffness 2 kN/m²

Cover depth from road level to top of pipe = 4.2 m

Subject to main road loading

Control over installation and trench width

Pipe properties:

	Symbol	Unit	Value
Pipe outside diameter	B_c	m	1.288
Mean pipe diameter	D	m	1.244
Pipe internal diameter	D_i	m	1.200
Pipe short-term modulus of elasticity	E_{p_short}	MPa	1 000
Pipe long-term (two-year) modulus of elasticity	E_{p_long2}	MPa	250
Pipe long term (fifty-year) modulus of elasticity	E_{p_long50}	MPa	150

	Symbol	Unit	Value
Poisson's ratio of pipe	ν	–	0.45
Second moment of area of pipe wall per unit length	I	mm ⁴ /mm	4 548
Cross-sectional area of pipe wall per unit length	A	mm ² /mm	16.23
Wall thickness (i.e. full height of profile)	t	m	0.044
Pipe weight	–	kg/m	58

Soil properties:

	Symbol	Unit	Value
Native soil is a medium dense granular soil with an SPT value of 20	M_{sn} (from Table 18)	MPa	34.5
Coefficient of lateral earth pressure (from table below)	K	–	0.4
Unit weight of soil	γ	kN/m ³	19.6
Unit weight of water	γ_w	kN/m ³	10
Height of water table above invert of pipe	H_w	m	5.488
Initial out-of-round deflection during construction	δ_{y0}	–	0.015
Out-of-round deflection due to long-term consolidation	δ_{y2}	–	0.02

Embedment properties:

	Symbol	Unit	Value
S2 embedment			
Compacted density (from Table 17)	M_p	%	80
Constrained modulus in the pipe zone (from Table 17)	M_{sb}	MPa	3
Width of trench	B_d	m	2.2

Calculate the plane strain modulus, flexural ring stiffness and compression stiffness of the pipe ring for the short-term and long-term scenarios (see [7.3.4](#)).

Plane strain modulus of pipe material in megapascals (MPa), E_p^* :

$$E_{p(\text{short})}^* = \frac{E_{p(\text{short})}}{(1 - \nu_p^2)}$$

$$E_{p(\text{short})}^* = \frac{1000}{(1 - 0.45^2)}$$

$$E_{p(\text{short})}^* = 1254 \text{ MPa}$$

For all long-term load cases, conservatively apply fifty-year modulus (see [4.5.3](#)):

$$E_{p(\text{long})}^* = \frac{E_{p(\text{long})}}{(1 - \nu_p^2)}$$

$$E_{p(\text{long}50)}^* = \frac{150}{(1 - 0.45^2)}$$

$$E_{p(\text{long}50)}^* = 188 \text{ MPa}$$

Flexural ring stiffness of the pipe in megapascals (MPa), S_y :

$$S_{y(\text{short})} = \frac{E_{p(\text{short})}^* I}{(1000D)^3}$$

$$S_{y(\text{short})} = \frac{1254 \times 4548}{(1000 \times 1.244)^3}$$

$$S_{y(\text{short})} = 0.0030 \text{ MPa}$$

$$S_{y(\text{long})} = \frac{E_{p(\text{long})}^* I}{(1000D)^3}$$

$$S_{y(\text{long}50)} = \frac{188 \times 4548}{(1000 \times 1.244)^3}$$

$$S_{y(\text{long}50)} = 0.00044 \text{ MPa}$$

Compression stiffness of the pipe ring in megapascals (MPa), S_z :

$$S_{z(\text{short})} = \frac{E_{p(\text{short})}^* A}{1000D}$$

$$S_{z(\text{short})} = \frac{1254 \times 16.23}{1000 \times 1.244}$$

$$S_{z(\text{short})} = 16.36 \text{ MPa}$$

$$S_{z(\text{long})} = \frac{E_{p(\text{long})}^* A}{1000D}$$

$$S_{z(\text{long}50)} = \frac{188 \times 16.23}{1000 \times 1.244}$$

$$S_{z(\text{long}50)} = 2.45 \text{ MPa}$$

Calculate the constrained soil modulus, M_s , using Equation (40):

$$M_s = S_c \times M_{sb}$$

$$\frac{M_{SN}}{M_{SB}} = \frac{34.5}{3} = 11.5$$

$$\frac{B_d}{B_c} = \frac{2.2}{1.288} = 1.7$$

Reading from [Table 19](#), $\frac{M_{SN}}{M_{SB}} > 5$ and $\frac{B_d}{B_c} = 1.7$, so interpolating between the values of S_c of 2.2 and 1.9 gives $S_c = 1.96$:

$$M_s = S_c \times M_{sb}$$

$$M_s = 1.96 \times 3 = 5.88$$

Calculate the flexural stiffness ratio and the compression stiffness ratio (see [7.3.6](#)):

Flexural stiffness ratio, Y :

$$Y_{\text{short}} = \frac{M_s}{S_{y(\text{short})}}$$

$$Y_{\text{short}} = \frac{5.88}{0.0030}$$

$$Y_{\text{short}} = 1960$$

$$Y_{\text{long50}} = \frac{M_s}{S_{y(\text{long})}}$$

$$Y_{\text{long50}} = \frac{5.88}{0.00044}$$

$$Y_{\text{long50}} = 13363$$

Compression stiffness ratio, Z :

$$Z_{\text{short}} = \frac{M_s}{S_{z(\text{short})}}$$

$$Z_{\text{short}} = \frac{5.88}{16.36} = 0.359$$

$$Z_{\text{long}} = \frac{M_s}{S_{z(\text{long})}}$$

$$Z_{\text{long50}} = \frac{5.88}{2.45} = 2.396$$

Calculate the vertical pressure at the mid-height of the pipe (see [7.3.7](#)). The vertical pressure is given by:

$$P_v = P_e + P_s$$

$$P_e = \gamma(H + 0.11B_c)$$

$$P_e = 19.6(4.2 + 0.11 \times 1.288) = 85.1 \text{ kN/m}^2$$

Vehicle surcharge pressure, P_s , can be computed from Equation (1) in [subclause 5.3](#):

$$P_s = 76.46 H^{0.923}$$

$$P_s = 76.46 \times 4.2^{0.923} = 20.33 \text{ kN/m}^2$$

$$P_v = 85 + 20.33 = 105.3 \text{ kN/m}^2$$

Effect of groundwater at mid-height of the pipe:

$$P_w = \gamma_w (H_w - B_c / 2)$$

$$P_w = 10 \left(5.488 - \frac{1.288}{2} \right) = 48.44 \text{ kN/m}^2$$

Calculate the soil effective stress P'_v

$$P'_v = P_v - P_w$$

$$P'_v = 105.33 - 48.44 = 56.9 \text{ kN/m}^2$$

Calculate the horizontal pressure (soil total stress), P_h :

$$P_h = P'_h + P_w$$

$$P'_h = K \left[\gamma (H + 0.5B_c) - P_w \right]$$

The value of K is read from [Table 20](#). In this example, K for an S2 compacted embedment = 0.5:

$$P'_h = 0.5[19.6(4.2 + 0.5 \times 1.288) - 48.44]$$

$$P'_h = 23.25$$

$$P_h = 23.25 + 48.44 = 71.7$$

Calculate the total stresses:

$$P_z = (P_v + P_h)/2$$

$$P_y = (P_v - P_h)/2$$

$$P_z = (105.3 + 71.7)/2 = 88.5 \text{ kN/m}^2$$

$$P_y = (105.3 - 71.7)/2 = 16.8 \text{ kN/m}^2$$

Calculate the effective stresses:

$$P'_z = P_z - P_w$$

$$P'_z = 88.5 - 48.44 = 40.1 \text{ kN/m}^2$$

Ring deflection calculations (see 7.3.8.2)

Long-term vertical deflection, $\delta_v = \delta_z + \delta_y$

$$\delta_z = \frac{\alpha_z \times P'_z + P_w}{2S_z 1000} \text{ where the radial arching factor is given by } \alpha_z = \frac{4}{3 + z}$$

To determine long-term deflection, the long-term value of Z is used to compute α_z :

$$\alpha_{z, \text{long-term}} = \frac{4}{3 + 2.396} = 0.741$$

$$\delta_z = \frac{\alpha_z \times P'_z + P_w}{2S_z 1000} = \frac{0.741 \times 39.97 + 48.44}{2 \times 4.09 \times 1000} = 0.0095 \text{ or } 0.95\%$$

$$\delta_y = \delta_{y0} + \delta_{y1} + \delta_{y2}$$

δ_{y0} is obtained from Table 21, and in this design example, for a pipe ring stiffness of 2 kN/m² and an S2 embedment (meaning installation control Type A) = 1.5%.

δ_{y2} is obtained from Table 22, and in this design example, an S2 embedment compacted to 80% = 3%.

The long-term value of δ_{y1} is calculated using the long-term value of S_y as follows:

$$\delta_{y1, \text{long-term}} = \frac{4Py}{(108S_y + M_s) \times 1000} = \frac{4 \times 16.8}{(108 \times 0.00044 + 5.88) \times 1000} = 0.0113$$

$$\delta_y = \delta_{y0} + \delta_{y1} + \delta_{y2}$$

$$\delta_y = 0.015 + 0.0113 + 0.03 = 0.056 \text{ or } 5.6\%$$

Linear buckling calculations (see 7.3.8.3)

Case 1: Multi-wave mode:

$$P_{\text{cr_case1}} = 0.945 (M_s)^{2/3} (S_y)^{1/3} \times 1\,000$$

$$P_{\text{cr_case1 (short term)}} = 0.945 (5.88)^{2/3} (0.003)^{1/3} \times 1\,000 = 452.4 \text{ kN/m}^2$$

$$P_{\text{cr_case1 (long term)}} = 0.945 (5.88)^{2/3} (0.00044)^{1/3} \times 1\,000 = 238.6 \text{ kN/m}^2$$

Case 2: Elliptical mode of buckling:

$$P_{\text{cr_case2}} = 24qS_y + \frac{k_{\text{sc}}M_s}{18} \times 1\,000$$

where:

 q is the out-of-roundness factor:

$$q = \left[\frac{1 - \delta_y}{(1 + \delta_y)^2} \right]^3 = \left[\frac{1 - 0.056}{(1 + 0.046)^2} \right]^3 = 0.607; \text{ and}$$

 k_{sc} is the shallow cover reduction factor on soil modulus from Table 23 as follows:

$$H/B_c = 4.2/1.288 = 3.26$$

By interpolation, $k_{\text{sc}} = 0.925$.

$$P_{\text{cr_case2short term}} = 24 \times 0.607 \times 0.003 + \frac{0.925 \times 5.88}{18} \times 1\,000 = 302.2 \text{ kN/m}^2$$

$$P_{\text{cr_case2long term}} = 24 \times 0.607 \times 0.00044 + \frac{0.925 \times 5.88}{18} \times 1\,000 = 302.2 \text{ kN/m}^2$$

Check the long-term stability:

$$F_s = \frac{P_{\text{cr_case1}}}{P_e} = \frac{238.6}{85} = 2.81$$

$$F_s = \frac{P_{\text{cr_case2}}}{P_e} = \frac{326.7}{85} = 3.55$$

Check the short-term stability:

$$F_s = \frac{P_{\text{cr_case1}}}{P_e + P_s} = \frac{452.43}{85 + 20.33} = 4.30$$

$$F_s = \frac{P_{\text{cr_case2}}}{P_e + P_s} = \frac{326.71}{85 + 20.33} = 2.87$$

Ring strength (see 7.3.8.4)

Components of ring thrust in the pipe wall, in both short term and long term (see 7.3.8.4.1):

$$N_z = (\alpha_z \times P'_z + P_w) \times B_c / 2$$

$$\text{Short term: } N_z = (1.191 \times 40.1 + 48.44) \times 1.288 / 2 = 61.94 \text{ kN/m}$$

$$\text{Long term: } N_z = (0.741 \times 40.1 + 48.44) \times 1.288 / 2 = 50.34 \text{ kN/m}$$

$$N_y = \frac{4 \times (144 + Y) \times P'_y}{360 + \left[(7 + Z) \times \left(\frac{Y}{2} \right) \right]} \times \frac{B_c}{2}$$

$$\text{Short term: } N_y = 12.02 \text{ kN/m}$$

$$\text{Long term: } N_y = 9.26 \text{ kN/m}$$

Maximum ring compressive stress in the pipe wall at springings in short term (see 7.3.8.4.2):

$$\sigma_{z,\max} = (N_z + N_y) / A = (61.94 + 12.02) / 16.23 = 4.56 \text{ MPa}$$

(Less than the design limit of 8.0 MPa, so acceptable)

Long-term ring compressive strain, ε_z (see 7.3.8.4.3):

$$\varepsilon_{z,\max} = \frac{N_z + N_y}{E_p^* A} = \frac{50.34 + 9.26}{1488 \times 16.23} = 0.0195 \text{ or } 1.95\%$$

$$\varepsilon_{z,\min} = \frac{N_z - N_y}{E_p^* A} = \frac{50.34 - 9.26}{1488 \times 16.23} = 0.0135 \text{ or } 1.35\%$$

Long-term ring bending strain, ε_y (see 7.3.8.4.4):

$$\varepsilon_y = D_f \delta_y (t/D)$$

Read the value for D_f from Table 14. In this design example, a class S2 embedment compacted to 80%. Pipe stiffness is 2 kN/m², closest value for that of a 2.5 kN/m² stiffness. D_f taken as 4.5.

t is the wall thickness of solid-wall pipe, or twice the largest distance from neutral axis of the wall section to the inside or outside surface of the pipe. In this design case, the profile has height 0.044 m.

$$\varepsilon_y = 4.5 \times 0.056 \times (0.044/1.244) = 0.0089$$

Calculate the long-term combined strain (see 7.3.8.4.5).

$$\text{Maximum compressive fibre strain, } \varepsilon_{c,\max} = \varepsilon_y + \varepsilon_z = 0.0089 + 0.0195 = 0.0284 \text{ or } 2.84\%$$

(Less than the default design limit of 5%, so acceptable)

$$\text{Maximum tensile fibre strain, } \varepsilon_{t,\max} = \varepsilon_y - \varepsilon_{z,\min} = 0.0089 - 0.0135 = -0.0046 \text{ or } -0.46\%$$

Negative value means no tension in wall section long term but check for tensile strain in the short term.

Short-term ring compressive strain, ε_z (see 7.3.8.4.3):

$$\varepsilon_{z,\max} = \frac{N_z + N_y}{E_p^* A} = \frac{61.94 + 12.02}{1254 \times 16.23} = 0.003\,63 \text{ or } 0.36\%$$

$$\varepsilon_{z,\min} = \frac{N_z - N_y}{E_p^* A} = \frac{61.94 - 12.02}{1254 \times 16.23} = 0.002\,45 \text{ or } 0.25\%$$

Short-term ring bending strain, ε_y (see 7.3.8.4.4):

$$\varepsilon_y = D_f \delta_y (t/D)$$

Where the short-term ring deflection δ_y is given by:

$$\delta_y = \delta_{y0} + \delta_{y1,\text{short-term}} = 0.015 + 0.0108 = 0.025\,8 \text{ or } 2.58\%$$

and so $\varepsilon_y = 4.5 \times 0.025\,8 \times (0.044/1.244) = 0.004\,11$

Thus, in short term:

maximum tensile fibre strain, $\varepsilon_{t,\max} = \varepsilon_y - \varepsilon_{z,\min} = 0.004\,11 - 0.002\,45 = 0.001\,66 \text{ or } 0.17\%$

(Negative value means no tension in wall section – tensile strain less than the default design limit of 2.5%, so acceptable)

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