

Structural design of buried pipelines under various conditions of loading —

Part 2: Summary of nationally established methods of design

ICS 23.040.01

National foreword

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Amendments issued since publication

Amd. No.	Date	Comments

This Published Document was published under the authority of the Standards Policy and Strategy Committee on 31 May 2006

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English Version

Structural design of buried pipelines under various conditions of loading - Part 2: Summary of nationally established methods of design

Calcul de résistance mécanique des canalisations
enterrées sous diverses conditions de charge - Partie 2:
Résumé des méthodes nationales de dimensionnement

Statische Berechnung von erdverlegten Rohrleitungen
unter verschiedenen Belastungsbedingungen - Teil 2:
Zusammenstellung national eingeführter
Berechnungsverfahren

This Technical Report was approved by CEN on 28 February 2005. It has been drawn up by the Technical Committee CEN/TC 165.

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Foreword

This Technical Report (CEN/TR 1295-2:2005) has been prepared by Technical Committee CEN/TC 165 “Wastewater engineering”, the secretariat of which is held by DIN.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN [and/or CENELEC] shall not be held responsible for identifying any or all such patent rights.

This Technical Report was prepared by a Joint Working Group (JWG 1) of Technical Committees TC 164, Water supply, the secretariat of which is held by AFNOR, and TC 165, Waste water engineering, the secretariat of which is held by DIN.

This Technical Report is intended for use in conjunction with the series of product standards covering pipes of various materials for the water industry.

This Technical Report includes an Informative Annex A in which are included additional details about the nationally established methods of design declared, submitted by and used in member countries, and collated by the Joint Working Group.

Introduction

The structural design of buried pipelines constitutes a wide ranging and complex field of engineering, which has been the subject of extensive study and research, in many countries over a period of very many years.

Whilst many common features exist between the design methods which have been developed and established in the various member countries of CEN, there are also differences reflecting such matters as geological and climatic variations, as well as different installation and working practices.

In view of these differences, and of the time required to develop a common design method which would fully reflect the various considerations identified in particular national methods, a two stage approach has been adopted for the development of this document.

In accordance with this two stage approach, the Joint Working Group, at its initial meeting, resolved "first to produce a document giving guidance on the application of nationally established methods of structural design of buried pipelines under various conditions of loading, whilst working towards a common method of structural design". This document represents the full implementation of the first part of that resolution.

1 Scope

In addition to EN 1295-1, this Technical Report gives additional guidance when compared with EN 1295-1 on the application of the nationally established methods of design declared by and used in CEN member countries at the time of preparation of this document (see informative Annex A).

This Technical Report is an important source of design expertise, but it cannot include all possible special cases, in which extensions or restrictions to the basic design methods may apply.

Since in practice precise details of types of soil and installation conditions are not always available at the design stage, the choice of design assumptions is left to the judgement of the engineer. In this connection the document can only provide general indications and advice.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1295-1, *Structural design of buried pipelines under various conditions of loading — Part 1: General requirements*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1295-1 apply.

4 Additional details about established methods

Annex A gives for several countries details about the established methods of design declared, submitted by and used in member countries.

Annex A (informative)

Summary of Methods of different countries

A.1 Austria

A.1.1 General remarks

The Austrian Standard Önorm B 5012:2005 is based on the "option 1" contained in CEN/TR 1295-3.

This "option 1" had been prepared by the TG1 of the CEN/TC164-165/JWG1 as a result of thorough study of the subject and long-lasting consultations carried out from 1992 onwards.

Although the design method described in "option 1" was mainly based on two recognised and well-tested "national methods", the German ATV-DVWK-A 127 and the (former) Austrian ÖNORM B 5012-1 and – 2, some new assumptions had to be incorporated in the re-drafted "option 1" in order to consider the most recent experience gained in this domain.

Therafter, it was of course also necessary to prove the correctness of the newly made assumptions by means of comprehensive field testing. This particular kind of testing was done during the period from 2000 to 2004. The tests results obtained were already incorporated in the ÖNORM B 5012 a revision of the former parts 1 and 2 of this standard.

Therefore the ÖNORM B 5012:2005 represents a calculation method which is based on the most recent experience in this field, but it still complies with the calculation principles elaborated by "JWG 1" regarding the "option 1" of CEN/TR 1295-3.

A.1.2 Differences between "option 1" and ÖNORM B 5012

The ÖNORM B 5012 differs in the following points from "option 1":

- 1) Two different applications of ÖNORM B 5012 enable to make
 - i) calculations for pipe design and
 - ii) back-calculations of failure situations

For the pipe design the soil moduli proposed in "option 1" (Table 4) shall be reduced by the factor 0,5, whereas for back-calculations the unchanged soil moduli from "option 1" should be used.

The purpose for this distinction is the following:

The results of calculations for the pipe design should be close to the 95 % fractile of the scattering design values, like deflection, stress or strain. Then the required safety or failure probability is assured by using the safety factors proposed in ÖNORM B 5012.

However, for back-calculations of failure situations the calculation results should be as close as possible to the mean value of the measured values.

- 2) Factor $f_{R,GW}$ is changed in ÖNORM B 5012 in comparison to "option 1" (compare equation 23 in "option 1" and ÖNORM B 5012).
- 3) The soil pressure ratios K_1 and K_2 are partly changed (compare Table 11 in "option 1" and ÖNORM B 5012).

- 4) The recommended support angles α_v are partly changed (compare Table 13 in "option 1" and ÖNORM B 5012).
- 5) In ÖNORM B 5012 the horizontal bedding stiffness S_{Bh} and factor ζ is calculated in a clearer and easier way than in "option 1" (compare 8.3.2 in "option 1" and ÖNORM B 5012).
- 6) The estimated values of relative initial ovalization $\delta_{v,io}$ in "option 1" are reduced in ÖNORM B 5012 by the factor 0,5 (compare Tables 19 and 20 in "option 1" with Table 18 and 19 in ÖNORM B 5012).
- 7) In ÖNORM B 5012 it is proposed to use the theory 2nd order calculation more consequently than in "option 1" (in ÖNORM for flexible pipes with deflections greater than 1 % in comparison to 5 % in "option 1").

A.1.3 Principles

Like "option 1", the ATV-DVWK-A 127 and the ÖNORM model the calculation system of ÖNORM B 5012 is based on the model of the embedded circular or non circular ring. The pipe-soil interaction is taken into account by the following interpretations:

- 1) In vertical direction: Using the shear-stiff beam model above the pipe for the calculation of the vertical loading due to the earth weight and uniformly distributed surcharge;
- 2) In horizontal direction: Using the compatibility condition of the horizontal pipe and soil movements taking into account all loads considered in the calculation (e.g. for the calculation of the horizontal bedding reaction pressures).

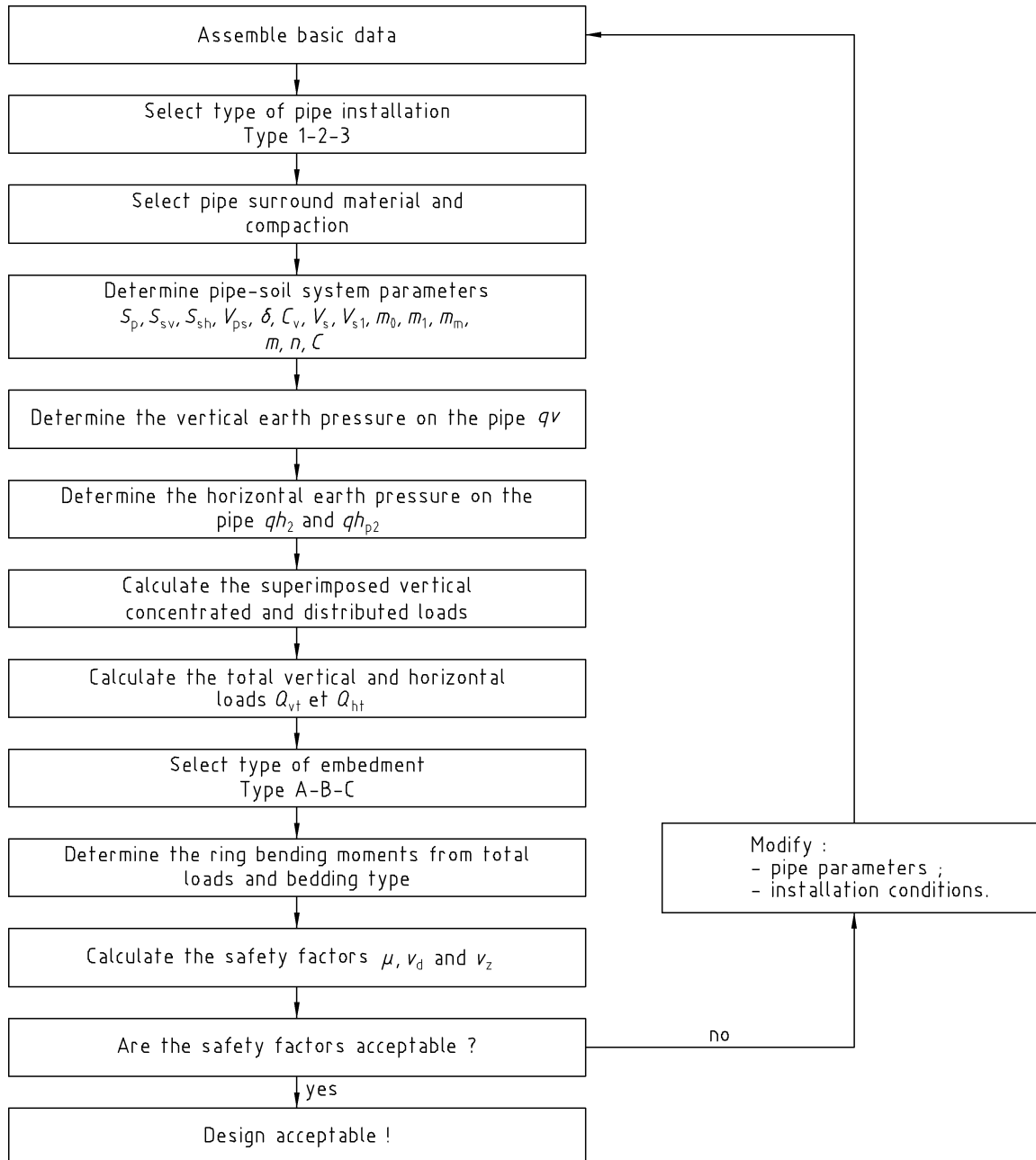
Further descriptions of details about the principles and the calculation method are stated in CEN/TR 1295-3.

A.2 Belgium

A.2.1 General

Calculation procedure of the ISO 2785: Directives for selection of asbestos-cement pipes subject to external loads with or without internal pressure.

A.2.2 Flowchart



A.2.3 Design, equations, tables and charts, symbols and abbreviations

A.2.3.1 Symbols and abbreviations

A width of uniform surcharge of small extent, in metres;

a	distance between two wheels on a single axle of a truck, in metres;
B	width of trench at the crown of the pipe, in metres;
B'	distance of the spring-line of a pipe from the wall of the trench in which it is buried, in metres;
h	distance between two wheels of two different axles of a truck, in metres;
c	diagonal distance between two wheels of two different axles of a truck, in metres;
C, C_{90}	earth-load coefficient for a trench with vertical walls;
C_c	load coefficient for superimposed concentrated moving loads;
C_d	load coefficient for uniform surcharges of small extent;
C_n	load coefficient for uniform surcharges of large extent;
$C_v, C_{d1}, C_{v2}, C_{v3}$	coefficients of vertical deformation of pipe;
C_{h2}, C_{h3}	coefficients of horizontal deformation of pipe;
d	nominal or internal diameter of pipe, in millimetres;
D	external diameter of pipe, in metres;
e	base of natural logarithms;
E	modulus of elasticity, in Newtons per square millimetre;
E_p	modulus of elasticity of pipe, in Newtons per square millimetre;
E_s	modulus of compression of soil, in Newtons per square millimetre;
E_t	modulus of elasticity of road construction material, in Newtons per square millimetre;
E_1, E_2, E_3, E_4	moduli of compression of soil and backfill in various zones of the trench, in Newton per square millimetre;
H, H_1, H_2	heights of earth cover of a pipe, in metres;
H_e	equivalent height of earth cover a pipe laid under a paved road, in metres;
HT	heavy truck;
I	modulus of inertia of the wall of the pipe per unit length, in cubic millimetres;
k	factor of ring-bending moment;
$k_{v1}, k_{h1}, k_{hp}, k_w$	factors of ring-bending moments due to vertical and horizontal loads, horizontal reaction pressure and water-load respectively;
K_1, K_2	coefficients of lateral earth pressure;
L	length of uniform surcharge of small extent, in metres;
LT	light truck;
m, m_0, m_1, m_m	concentration factors of vertical earth pressure over the pipe;

M_e	ultimate ring-bending moment of pipe when tested in accordance with ISO 881 or ISO 160, in Kilonewton metre per metre;
M_m	maximum ring-bending moment in the wall of a buried, Kilonewton metre per metre;
M_1	the ring-bending moment that will fracture the pipe when combined with an internal hydraulic pressure p_1 ;
M_2	the ultimate ring-bending moment when no internal pressure affects the pipe;
n	concentration factor of lateral earth pressure on the sides of the pipe;
P_d	intensity of distributed load, in kilonewtons per square metre;
P_j	pipe projection ratio;
P_w	hydraulic working pressure, in Megapascal;
P_1	the internal hydraulic pressure that will fracture the pipe when combined with a ring-bending moment M_1 ;
P_2	the internal hydraulic pressure that will burst a pipe which is not exposed to any external load;
P_e	crushing load of a pipe when tested in accordance with ISO 881, in kilonewtons per 200 or 300 millimetre lengths of pipe;
P_v	maximum wheel load of a truck, in kilonewtons;
P_{vc}	vertical pressure on a pipe due to moving concentrated surcharge, in kilonewtons per square metre;
P_{vd}	vertical pressure on a pipe due to moving distributed surcharge, in kilonewtons per square metre;
q_v, q_{v1}, q_{v2}	vertical earth pressure on the pipe, in kilonewtons per square metre;
q_{vt}	total vertical pressure due to earth and moving load on the pipe, in kilonewtons per square metre;
q_h, q_{h1}, q_{h2}	horizontal earth pressure on the pipe, in kilonewtons per square metre;
q_{hp}, q_{hp1}, q_{hp2}	horizontal soil reaction pressure on the pipe, in kilonewtons per square metre;
r	mean radius of pipe, in metres;
s	wall thickness of pipe, in metres;
s_p	stiffness of pipe, in Newtons per square metre;
S_{sh}	horizontal stiffness of soil backfill in the zone of the pipe, in Newtons per square millimetre;
S_{sv}	vertical stiffness of pipe bedding, in Newtons per square millimetre;
t_1, t_2	thickness of layers in a road structure, in metres;
V_s, V_{s1}	stiffness ratio;
V_{ps}	pipe-soil system stiffness;

w, w_1, w_2	unit weight of backfill soil, in kilonewtons per cubic metre;
W	crushing load per unit length of pipe when tested in accordance with ISO 160, in kilonewtons per metre;
x_1, x_2, x_3	auxiliary parameter defined in the text;
α	half the bedding angle of pipe;
β	slope of the wall of the trench;
γ	specific weight of water in kilonewtons per cubic metre;
δ	deformation coefficient;
ξ	correction factor;
η_d	reduction factor of the resistance of the pipe to external load due to the action of internal pressure;
η_z	reduction factor of the resistance of the pipe to internal pressure due to the action of external load;
ν_d	safety factor against crushing of a pipe loaded externally without internal pressure;
ν_z	safety factor against bursting of a pipe when a ring-bending moment is applied together with a internal hydraulic pressure;
ρ	angle of internal friction of backfill soil;
ρ'	angle of friction between the backfill soil and the wall of the trench;
$\emptyset$	impact factor.

A.2.3.2 Required basic data

— D, s, r, E_p	pipe parameters ;
— B, H	trench/embankment conditions;
— K_1, K_2	coefficients of lateral earth pressure, out of Table A.2;
— ρ	angle of internal friction, out of soil investigation or Table A.1;
— p_j	projection ratio;
— $E_1 - E_2 - E_3 - E_4$	soil conditions, out of soil investigation or Table A.1.

Table A.1 — Properties of soils for calculating earth-load

Group of soil	Types of soil ^a	Unit, weight, ω kN/m ³	ρ degrees	Moduli of compression E_s^b at following Proctor standard densities (%) achieved by self-consolidation compaction N/mm ²					
				85	90	92	95	97	100
1	Non-cohesive	20	35	2,5	6	9	16	23	40
2	Slightly cohesive	20	30	1,2	3	4	8	11	20
3	Mixed cohesive	20	25	0,8	2	3	5	8	14
4	Cohesive	20	20	0,6	1,5	2	4	6	10

^a The four types of soil are:
non-cohesive: gravel, sand;
slightly cohesive: binding non-uniform sand or gravel;
mixed cohesive: rock flour, weathered rock soils, clayey sand;
cohesive: clay, silt, loam.

^b The moduli of compression E_s of the soils are measured by applying the CBR (California Bearing Ratio) method using a round plate of an area of 700 cm².

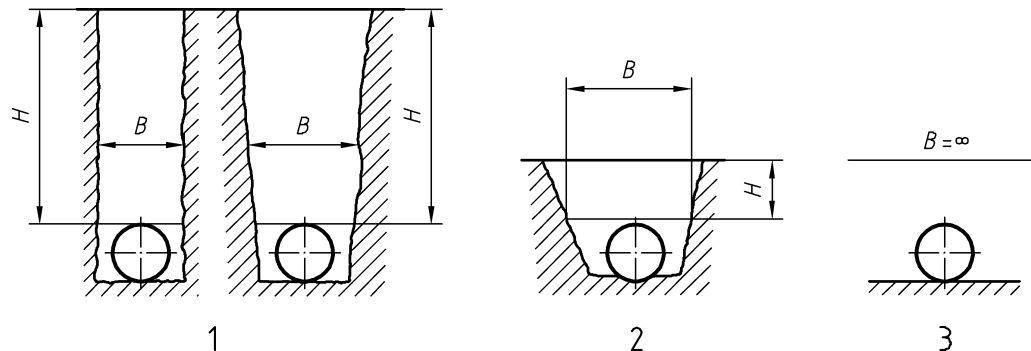
Table A.2 — Coefficients of lateral earth pressures

Group of soil	K_1	K_2
1	0,5	0,4
2	0,5	0,3
3	0,5	0,2
4	0,5	0,1

K_1 and K_2 shall always be considered simultaneously.

A.2.3.3 Selection of type of pipe laying

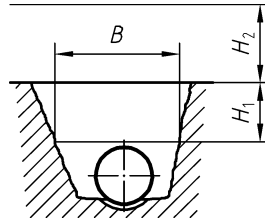
Three types of pipe laying are defined, see Figures A.1, A.2 and A.3.

**key**

- 1 Narrow trenches
- 2 Wide trench
- 3 Embankment conditions: positive projection

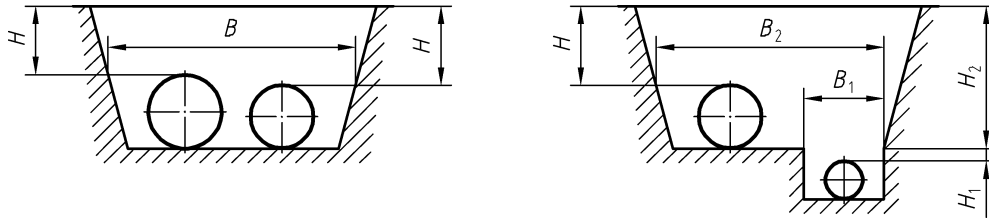
NOTE Type 1 covers trenches, wide trenches and positive projection embankment conditions.

Figure A.1 — Type 1 of laying



NOTE Type 2 covers negative projection conditions.

Figure A.2 — Type 2 of pipe laying



NOTE Type 3, two or more pipelines in a single trench.

Figure A.3 — Type 3 of pipe laying

A.2.3.4 Determination of the pipe soil system parameters

a) Stiffness of the pipe S_p

$$S_p = \frac{Ep}{12} \frac{(s)^3}{r} \quad \text{with} \quad r = \frac{D-s}{2}$$

b) Vertical stiffness of the bedding S_{sv}

$$S_{sv} = \frac{E_2}{P_j}$$

c) Horizontal stiffness of the bedding S_{sh}

$$S_{sh} = 0,6\xi E_2$$

$$\text{with } \xi = \frac{1,662 + 0,639\left(\frac{B}{D} - 1\right)}{\left(\frac{B}{D} - 1\right) + \left[1,662 - 0,361\left(\frac{B}{D} - 1\right)\right] \frac{E_2}{E_3}}$$

d) Pipe-soil fitness V_{ps} (see Figure A.4)

$$V_{ps} = \frac{S_p}{S_{sh}}$$

The distribution of the vertical earth pressure and reaction.

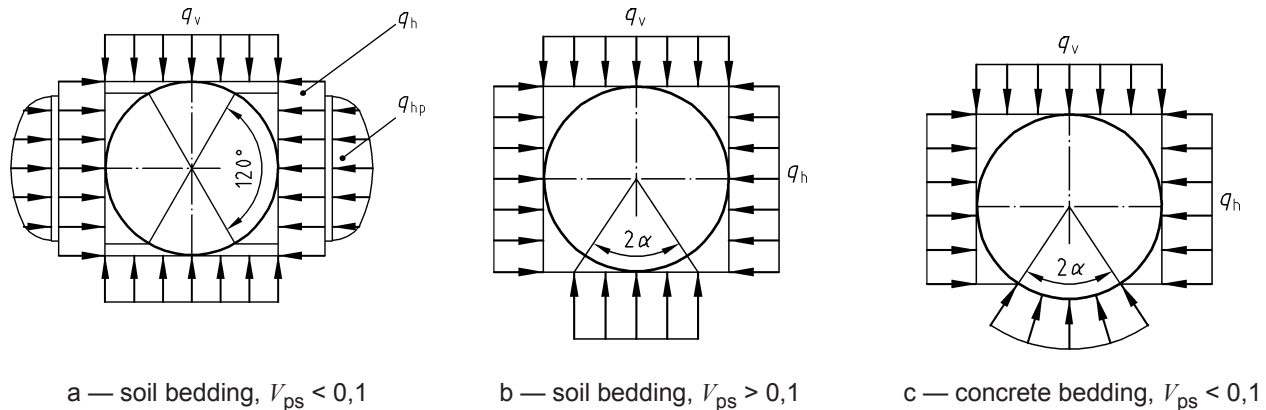


Figure A.4 — Distribution of earth pressure and reactions

For the calculation of ring-bending moments and deflections of the pipe, the vertical earth pressure is always assumed to be rectangularly distributed over its crown, as shown in Figure A.4.

The distribution of the reaction depends on the pipe-soil system stiffness V_{ps} :

- case 1: pipe on soil-bedding and $V_{ps} < 0,1$, according to Figure A.4.a, i.e. vertically directed reaction, rectangularly distributed over the full width D of pipe, regardless of the actual bedding angle;
- case 2: pipe on soil-bedding and $V_{ps} > 0,1$, according to Figure A.4.b, i.e. vertically directed reaction, rectangularly distributed along the bedding angle 2α ;
- case 3: in the case of rigid bedding (for example a concrete cradle) and when $V_{ps} > 0,1$, according to Figure A.4.c, i.e. radially directed and evenly distributed reaction along the bedding angle 2α .

e) Deformation

$$\delta = \frac{C_{h1}}{V_{ps} - C_{h3}}$$

f) Deformation factors C_v

$$C_v = C_{v1} + C_{vs} \cdot \delta$$

C_v , C_{h1} , C_{v3} , C_{h3} out of table A.3.

Table A.3 — Deformation factors

Factors corresponding to vertical soil pressure q_v

Bedding angle 2α degrees	Case 1, Figure A.4.a		Case 2, Figure A.4.b		Case 3, Figure A.4.c	
	C_{v1}	C_{h1}	C_{v1}	C_{h1}	C_{v1}	C_{h1}
60	- 0,0833	+ 0,0833	- 0,1053	+ 0,1026	- 0,1041	+ 0,1017
90	- 0,0833	+ 0,0833	- 0,0966	+ 0,0956	- 0,0916	+ 0,0916
120	- 0,0833	+ 0,0833	- 0,0893	+ 0,0891	- 0,0763	+ 0,0777
180	- 0,0833	+ 0,0833	- 0,0833	+ 0,0833	- 0,0417	+ 0,0417

Factors corresponding to horizontal soil pressures q_h and q_{hp}

Bedding angle 2α degrees	Factors for q_h				Factors for q_{hp}	
	Cases 1 and 2 Figures A.4.a and b		Case 3 Figure A.4.c		Case 1 Figure A.4.a	
	C_{v2}	C_{h2}	C_{v2}	C_{h2}	C_{v3}	C_{h3}
60	+ 0,0833	- 0,0833	+ 0,0827	- 0,0829	+ 0,0640	- 0,0658
90	+ 0,0833	- 0,0833	+ 0,0798	- 0,0805	+ 0,0640	- 0,0658
120	+ 0,0833	- 0,0833	+ 0,0721	- 0,0735	+ 0,0640	- 0,0658
180	+ 0,0833	- 0,0833	+ 0,0417	- 0,0417	+ 0,0640	- 0,0658

NOTE Factors C_v and C_h correspond to the following equations for calculating the deflection of the pipe:

$$\Delta D_v = 2 C_v q r^4 / EI$$

and

$$\Delta D_h = 2 C_h q r^4 / EI$$

where

$$C_v = C_{v1} \text{ or } C_{v2} \text{ or } C_{v3};$$

$$C_h = C_{h1} \text{ or } C_{h2} \text{ or } C_{h3};$$

$$q = q_{hv} \text{ or } q_h \text{ or } q_{hp}$$

g) Stiffness ratio V_s

$$V_s = \frac{S_p}{|C_v| S_{sv}} \text{ with lateral earth pressure } q_{hp};$$

$$V_s = \frac{S}{|C_{v1}| S_{vs}} \text{ without lateral earth pressure } q_{hp};$$

h) Concentration factor of vertical earth pressure over the pipe m_0

$$m_0 = \frac{4K_2}{3 + K_2}$$

i) Concentration factor of vertical earth pressure over the pipe m_1

$$m_1 = \left(\frac{m_m V_s + \frac{(m_m - 1)m_0 \cdot V_{s1}}{1 - m_0}}{V_s + \frac{(m_1 - 1)V_{s1}}{1 - m_0}} \right)$$

j) Concentration factor of vertical earth pressure over the pipe m_m

$$m_m = 1 + \frac{H}{D} \frac{1}{\frac{3,5}{P_j} + 2,2 \frac{E_1}{E_4} \left(\frac{1}{p_j - 0,25} \right) + \frac{H}{D} \frac{0,62}{p_j} + 1,6 \frac{E_1}{E_4} \left(\frac{1}{p_j - 0,25} \right)}$$

k) Concentration factor of vertical earth pressure over the pipe m

$$m = \frac{m_1 - 1}{3} \cdot \frac{B}{1} + \frac{4 - m_1}{3} \quad \text{for } 1 < \frac{B}{D} < 4$$

$$m = m_1 = C^{tc} \quad \text{for } 4 < \frac{B}{D}$$

$$m_{1m} = 1 + 4K_1 \tan \rho$$

l) Concentration factor of lateral earth pressure on the side of the pipe n

$$n = \frac{4 - m_1}{3}$$

m) Earth load coefficient C

$$C = \frac{B^2}{2K_1 H \tan \rho'} \left(1 - e^{-2K_1 \tan \rho' \frac{H}{B}} \right)$$

A.2.3.5 Determination of the vertical earth pressure on the pipe q_v

$$q_v = m \cdot C \cdot w \cdot H$$

A.2.3.6 Determination of the horizontal earth pressure on the pipe Q_{h2} and Q_{hp2}

$$q_v = n \cdot K_2 \cdot C \cdot w \cdot H$$

$$q_{hp2} = \delta \cdot (Q_v - Q_h)$$

A.2.3.7 Calculation of the superimposed vertical concentrated and distributed loads

— concentrated load:

$$P_{vc} = P_v \cdot C_c \cdot \mu$$

where

P_v wheel load.

$$C_c = \frac{1}{D} - \frac{2}{HD} \left[\arcsin \left(2H \sqrt{\frac{X_1}{X_2 X_3}} \right) - \frac{2HD}{\sqrt{X_1}} \left(\frac{1}{X_2} + \frac{1}{X_3} \right) + \frac{1}{H_2} \Sigma I \right]$$

$$x_1 = 4H^2 + D^2 + 1$$

$$x_2 = 4H^2 + 1$$

$$x_3 = 4H^2 + D^3$$

$\Sigma I = f(\text{wheel distance, axle numbers, traffic load})$

$$\varnothing = 1,5/1,4/1,2$$

— distributed load:

$$P_{vd} = P_d \cdot C_d \cdot \emptyset$$

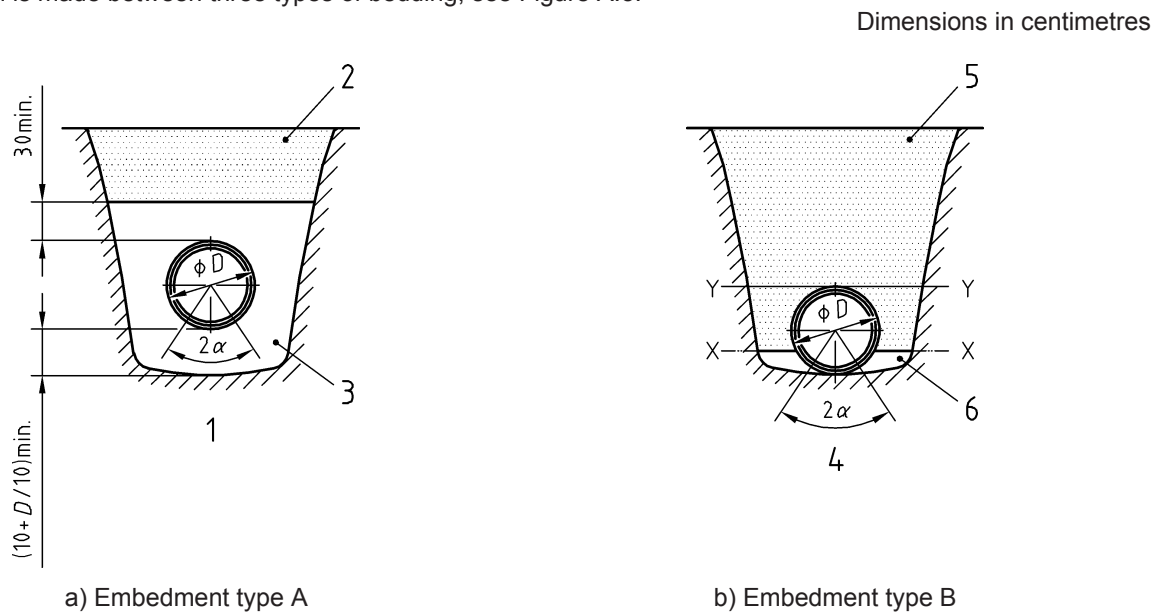
with

$$C_d = f\left(\frac{A}{2H}, \frac{L}{2H}\right) \text{ out of graphic}$$

$$\emptyset = 1,2$$

A.2.3.8 Selection of the type of bedding

A distinction is made between three types of bedding, see Figure A.5.



NOTE Bedding types A and B correspond to case 1 or 2 of load distribution in Figures A.4.a or A.4.b, depending of the value of the pipe system stiffness V_{ps} .

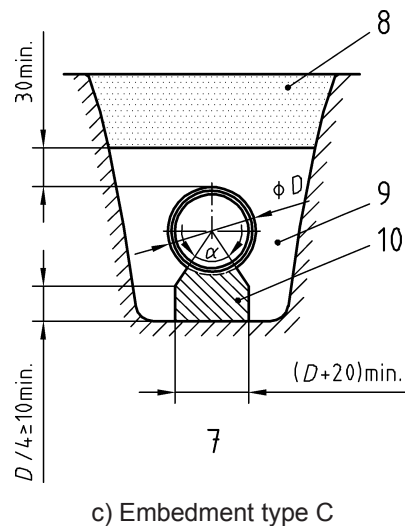


Figure A.5 — Beddings

A.2.3.9 Determination of the ring bending moments

The maximum ring bending moment M_m in the wall of the buried pipe is calculated as follows:

$$M_m = (k_v \cdot q_{vt} + k_h \cdot q_h + k_{hp} \cdot q_{hp}) r^2 + k_w \cdot \gamma \cdot r^3$$

with:

k_v, k_h, k_{hp}, k_w the ring bending moment factors given in table A.4.

Table A.4 — Factor for the calculation of ring-bending moments in buried pipes

Bedding angle 2 α	Cross-section of pipe	Ring-bending moment factors			
		k_v	k_h	k_{hp}	k_w
Case 1 of load distribution (bedding types A or B, $V_{ps} < 0,1$)					
180	Crown	+ 0,250	- 0,250	- 0,181	+ 0,172
	Spring line	- 0,250	+ 0,250	+ 0,208	- 0,196
	Bottom	+ 0,250	- 0,250	- 0,181	+ 0,220
Case 2 of load distribution (bedding types A or B, $V_{ps} > 0,1$)					
60	Crown	+ 0,286	- 0,250	-	+ 0,229
	Spring line	- 0,293	+ 0,250	-	- 0,264
	Bottom	+ 0,377	- 0,250	-	+ 0,420
90	Crown	+ 0,273	- 0,250	-	+ 0,210
	Spring line	- 0,279	+ 0,250	-	- 0,243
	Bottom	+ 0,313	- 0,250	-	+ 0,321
120	Crown	+ 0,261	- 0,250	-	+ 1,190
	Spring line	- 0,265	+ 0,250	-	- 0,220
	Bottom	+ 0,275	- 0,250	-	+ 0,260
Case 3 of load distribution (bedding types C, $V_{ps} > 0,1$)					
90	Crown	+ 0,266	- 0,245	-	+ 0,189
	Spring line	- 0,271	+ 0,244	-	- 0,230
	Bottom	+ 0,277	- 0,224	-	+ 0,262
120	Crown	+ 0,240	- 0,232	-	+ 0,157
	Spring line	- 0,240	+ 0,228	-	- 0,181
	Bottom	+ 0,202	- 0,187	-	+ 0,145
180	Crown	+ 0,163	- 0,163	-	+ 0,035
	Spring line	- 0,125	+ 0,125	-	0,000
	Bottom	+ 0,087	- 0,087	-	+ 0,035

A.2.3.10 Calculation of the safety factor

$$\mu = \frac{M_e}{M_m}$$

$$\nu_d = \frac{M_e}{M_m} \sqrt{1 - \frac{p_w}{P_2}}$$

$$\nu_z = \frac{P_2}{P_w} \left[1 - \frac{(M_m)^2}{M_e} \right]$$

where:

- μ is the safety factor against crushing of a pipe loaded externally without any internal pressure;
- ν_d is the safety factor against crushing when an internal hydraulic pressure P_w is applied together with a ring bending moment M_m ;
- ν_z is the safety factor against bursting when a ring bending moment M_m is applied together with an internal hydraulic pressure P_w .

A.2.3.11 Are the safety factors acceptable?

The recommended minimum safety factor against crushing of non-pressure pipes is:

$$\mu = 1,5$$

The recommended minimum safety factors for pressure pipes under combined loads according to the different diameters are:

- from 175 mm to 200 mm: $\nu_d = 2,5$ and $\nu_z = 3,5$;
- from 250 mm to 500 mm: $\nu_d = 2,5$ and $\nu_z = 3,0$;
- from 600 mm to 2500 mm: $\nu_d = 2,5$ and $\nu_z = 2,5$.

NOTE When the maximum working pressure does not exceed 0,3 MPa, the two safety factors may be reduced to 2,0 each.

A.3 Denmark

A.3.1 General

The design method described below is a simplified method which can be used generally for rigid pipes. However, it will always be acceptable to use more accurate methods if their quality is proved.

The course of the calculations in accordance with the principle using the procedures and equations described below is illustrated on the flowchart below. In practice, routine design is usually carried out with the aid of tables of charts based on these principles.

List of symbols

d_i	internal diameter of pipe, m;
d_y	external diameter of pipe, m;
F	crushing test load, kN/m;
f_t	bending tensile strength, MN/m^2 ;
h_d	depth of cover, m;
k	factor;
l	length of pipe, m;
P	axle load, kN;
p	railway line load, kN/m;
Q	wheel load, kN;
q	uniformly distributed surface load, kN/m^2 ;
r	bearing capacity, kN/m^2 ;
t	pipe wall thickness, m;
v	loading per unit of area, kN/m^2 ;
v_g	equivalent addition of load for self-weight of pipe, kN/m^2 ;
v_j	vertical earth load, kN/m^2 ;
v_q	vertical pipe loading from uniformly distributed surface load, kN/m^2 ;
v_t	vertical pipe loading from traffic load, kN/m^2 ;
v_w	equivalent addition for water load, kN/m^2 ;
γ	(without index) unit weight of backfill above water table, kN/m^3 ;
γ	(with index) partial safety factor;

λ earth load coefficient (Marston coefficient);

General indices:

c concrete;

d design;

f load dependant;

k characteristic;

m material dependant;

s steel.

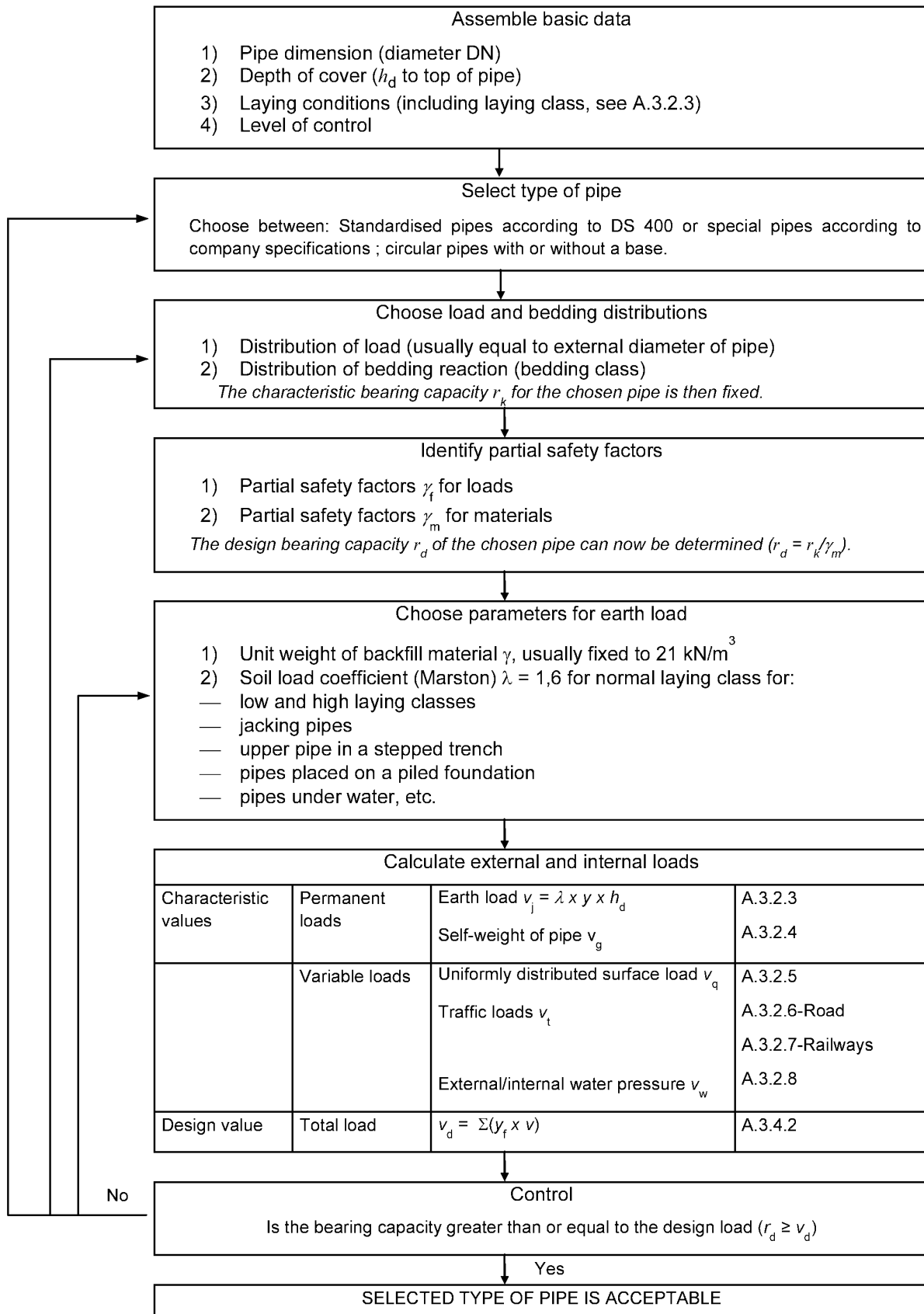


Figure A.6 — Flowchart for pipeline design

A.3.2 Charges

A.3.2.1 Types of loads

When designing a buried pipe the following types of loads shall be considered:

- Permanent loads
 - earth load;
 - self-weight of pipe.
- Variable loads
 - distributed surface load;
 - traffic loads;
 - loads from external and internal water pressure.

A.3.2.2 Distributed of load and bedding reaction

The vertical load is assumed uniformly distributed over a width equal to the external diameter of the pipe.

The horizontal load is assumed uniformly distributed over a width equal to:

- the external diameter of the pipe when using circular pipes;
- the height to the pipe from its top to its axis when using pipes with base.

The bedding reaction is assumed to be a vertical action uniformly distributed over a width equal to:

- 0,5 times the external diameter circular pipes laid with normal bedding;
- 0,7 times the external diameter of circular pipes laid with an improved bedding;
- the external width of circular jacking pipes;
- the width of the base when using pipes with a base.

In the longitudinal direction of the pipe the bedding reaction is assumed to be uniformly distributed. If the length of the pipe is large in relation to its diameter due consideration shall be given as to the validity of this assumption.

NOTE If no exact evaluation is carried out the bedding reaction can be assumed uniformly distributed if the useful length is smaller than or equal to:

1,0 m	for	$d_i \leq 0,15 \text{ m}$;
$d_i / 0,15 \text{ m}$	for	$0,15 \text{ m} < d_i \leq 0,40 \text{ m}$;
2,7 m	for	$d_i > 0,40 \text{ m}$.

A.3.2.3 Earth load

The characteristic vertical load on a pipe from the backfill is determined by:

$$v_j = \lambda \gamma h_d^2 \text{ kN/m}^2,$$

with the relieving effect of the lateral pressure being included in λ .

If a closer examination is not carried out and the laying conditions are not extreme, the following values for λ can be used:

- for low laying class (with no requirements for the stiffness of the sidefill): $1,62 + 0,50 h_d/d_y + 0,54 d_y/l h_d$;
- for normal laying class (with the stiffness of the sidefill at least the same as the stiffness of the backfill): 1,6;
- for high laying class (with the stiffness of the sidefill at least five times the stiffness of the backfill): 1,4;
- for jacking pipes: 1,0;
- for the upper pipe in a stepped trench: $1,62 + 0,25 h_d/d_y + 0,27 d_y/l h_d$;
- for pipes supported on piles: $1,62 + 0,50 h_d/d_y + 0,54 d_y/l h_d$;
- for pipes laid under water: $1,62 + 0,50 h_d/d_y + 0,54 d_y/l h_d$.

The unit weight γ of the backfill may be fixed at 21 kN/m^3 . For underwater backfill, γ may however be fixed at 11 kN/m^3 .

A.3.2.4 Self-weight of pipes

The loading effect of the self-weight of the pipe shall be included, either as a deduction in the load bearing capacity of the pipe or as an equivalent addition to the vertical load.

A.3.2.5 Uniformly distributed surface load

The action on a pipe from an uniformly distributed characteristic surface load q is given in kN/m^2 and determined by:

$$v_q = \lambda q.$$

A.3.2.6 Traffic load from roads

A three-axle group is assumed in which each axle load consists of two wheel loads Q with a centre-to-centre distance of 2,0 m and an axle-to-axle distance of 1,5 m. The contact surface of the wheel load is assumed to be a rectangle with sides of 0,2 m in the direction of travel and 0,6 m across the same.

The characteristic value of Q is assumed to be 65 kN for normal road traffic and 100 kN for heavy road traffic. These loads include an impact factor which is independent of the earth cover.

The load v_t from a wheel load Q is determined - in case of depth of cover of more than some 1,3 m - in accordance with Boussinesq's theory.

NOTE With a good approximation, the traffic loading can be determined on the basis of an assumption that the distribution of stress through the ground is 1:2. This leads to the following expression for the loading from the specific load group in kN/m^2 :

$$v_t = 6 \cdot Q / (h_d + 2,6)(h_d + 3,2)$$

At depths of cover $h_d < 1,34 \text{ m}$ the loading is changed into the action from one wheel load:

$$v_t = Q / (h_d + 0,2)(h_d + 0,6)$$

which value may be reduced as a function of the pipe diameter by using the following factor:

$$k = 1 - (1 - 0,75 \cdot h_d)(10 + 3 \cdot d_y - 1/d_y)/18$$

A.3.2.7 Traffic load from railway tracks

The standard railway loading of the Danish State Railways shall be used as a basis for the calculation of the loads from railways, i.e.:

- characteristic axle load inclusive impact factor: $P = 410 \text{ kN}$;
- evenly distributed line $p = 80 \text{ kN/m}$.

NOTE If a stress distribution of 1:2 in the ground is assumed across the line road, the load can be determined by the following expression:

The loading for one track in kN/m^2 :

$$v_t = (P/h_d + p)(h_d + 2,2)$$

The loading from two or more tracks:

for $h_d < 2,3 \text{ m}$: as for one track

for $h_d \geq 2,3 \text{ m}$: $v_t = (8 \cdot P/(h_d + 6,4) + 2 \cdot p)/(h_d + 6,7)$

A.3.2.8 Load from external and internal water pressure

The effect on a pipe due to its water-filled state shall usually be included, either as a deduction in the bearing capacity of the pipe or as an equivalent addition to the vertical load.

The effect on a pipe due to an internal positive or negative pressure shall be included. A dimensioning positive pressure shall include both the hydrostatic and the hydrodynamic pressure, as a possible surge in addition to the working pressure.

In principle, the effect on a pipe due to an external water pressure shall be included.

A.3.3 Safety

A.3.3.1 Safety analysis

The safety shall be evaluated in accordance with the partial coefficient method. The load bearing capacity of a pipe can either be determined arithmetically alone or by a combination of calculation and testing.

Safety analysis by calculation:

When safety is evaluated arithmetically requirements are made on the materials; this includes their strength, the execution of calculations, execution of the work and control of pipe materials and execution of workmanship.

Safety analysis by combining calculation and testing:

When safety is evaluated through a combination of calculation and testing the same requirements shall be made as for arithmetical evaluation alone; requirements on pipe materials shall be replaced by requirements on bearing capacity of the pipes as established in a given test method.

The control of the bearing capacity of the pipes can be carried out in a destructive or a non-destructive test. The test method shall be worked out so as to achieve a test result governed by the same properties as those that determine the bearing capacity under the existing load exerted upon the buried pipe.

A.3.3.2 Limit states

Serviceability limit state:

The serviceability limit state is considered as having been reached when the first crack forms in an unreinforced

pipe and when the cracks formed will affect the working life of a reinforced pipe.

Ultimate limit state:

For unreinforced pipes the ultimate limit state coincides with the serviceability state. For reinforced pipes this state has been reached when a yield mechanism has been formed or when the bearing capacity of the pipe may otherwise be considered exhausted.

NOTE For reinforced pipes the splitting off of the cover on the inside of the pipe may create a possible ultimate limit state. Shear rupture in the pipe wall due to displacement may appear if the pipe is exerted to a line load.

A.3.4 Partial safety factors

A.3.4.1 Action combinations

Two action combinations shall be examined:

- action combination 1 for the serviceability state;
- action combination 2 for the ultimate limit state.

A.3.4.2 Design loads

The design load ν_d is determined as the sum of the characteristic permanent load and the characteristic variable load, both multiplied by the appropriated partial safety factor γ_f :

$$\nu_d = \Sigma(\gamma_f \cdot \nu)$$

For action combination 1 $\gamma_f = 1,0$ for both types of loads and for action combination 2 $\gamma_f = 1,0$ for the permanent load and 1,3 for the variable load.

A.3.4.3 Design material parameters

If the load bearing capacity of the pipe is estimated on the basis of full scale tests the partial safety factor $\gamma_m = 1,5$ in the case of moderate control (factory production control), 1,4 in the case of normal control and 1,3 in the case of extended control.

For reinforced pipes which are structural analysed solely on the basis of calculations, partial safety factors for the reinforcement respectively the concrete shall be fixed as follows:

- γ_s : 1,4 for normal control and 1,3 for extended control;
- γ_c : 1,8 for normal control and 1,7 for extended control.

A.3.5 Calculations

It shall be proved that the design load bearing capacity of a pipe is greater than the design effect of actions considered.

A.3.5.1 Determination of effects of action

Serviceability limit state:

When determining the internal forces with a view to evaluating the serviceability limit state the elasticity theory shall be used with the commonly accepted approximations.

Ultimate limit state:

When determining the internal forces with a view to evaluating the ultimate limit state the elasticity theory shall be applied in the case of unreinforced pipes, and either the elasticity theory or the plasticity theory in the case of reinforced pipes. If the plasticity theory is applied, it is an assumption that the required ductility is present in the structure.

A.3.5.2 Determination of the load bearing capacity of unreinforced pipes

The load bearing capacity of unreinforced pipes is determined by a calculation of their strength on the basis of the actual laying conditions as compared to the design strength determined on the basis of the test load declared for the pipe.

NOTE The characteristic load bearing capacity for a pipe in kN/m^2 may be determined by:

$$r_k = k F_k / d_y$$

where

k is the ratio between the maximum stress from a load 1 during the crushing test and in buried conditions;

F_k is the declared crushing test load of the pipe, kN/m .

The safest determination of k is achieved when the maximum tensile stress during the test is present in the same point, or at any rate in sections of the same wall thickness and curve of the gravity line as in the underground conditions. The test arrangement should be selected so that the crack load and the rupture load are identical.

The crushing test load and the load bearing capacity can be fixed on the basis of a bending tensile stress determined in the usual calculation on the basis of the elasticity theory with plane stress distribution in the cross-section of the pipe wall compared to a bending tensile strength f_t by:

$$f_t = 4,6(1 + 0,2d_i - t) / (d_i^{0,25})$$

If the effective length of the pipe l_n is larger than the critical length l_c for which the bedding reaction in the longitudinal direction of the pipe can be taken as evenly distributed the bearing capacity in kN/m^2 shall be calculated from:

$$r_k = k(l_c / l_n)^2 F_k d_y$$

A.3.5.3 Determination of the load bearing capacity of reinforced pipes

The load bearing capacity may be determined on the basis of tests or calculations. If tests are used the test load shall be arranged so as to come as close as possible to the distribution of the moment and shear force to be found in the relevant soil load. If calculations are applied the rules of the standard for Design of Concrete Structures shall be applied.

A.3.5.4 Determination of laying depths

The maximum and possibly minimum acceptable laying depths for a pipe shall be determined by a load estimation on the basis of the actual design loads compared to the design load bearing capacity in kN/m^2 :

$$v_d \leq r_d = r_k / \gamma_m$$

A.4 France

A.4.1 Scope

The pipes concerned are those:

- laid in traditional way on continuous bedding on the ground in trenches or beneath embankments;
- with depth of cover greater than 0,8 m, when laid under road;
- forming part of a gravity system, internal water head being not greater than 4 metres (0,04 MPa) (for pressurised pipelines - see Fascicule 71 of CCTG).

A.4.2 Original features of the method

Original features of the method are:

- this method applies to all types pipes without discontinuity for the full range of ring stiffness;
- it takes into account the specific laying conditions: soil types, compaction conditions, water table, conditions under which trench sheeting is removed;
- the calculation of bending moments and diametrical deflection uses the second-order theory which makes it possible to take buckling into account;
- the model takes into account soil/pipe interaction;
- the safety is verified at different limits-states;
- flow charts indicate the routing of calculation;
- give the values of the design parameters;
- a large number of tables and charts make the calculation easier.

A.4.3 Description

A.4.3.1 Parameters used

The pipe parameters are the following:

D_m	mid-thickness diameter;
e	thickness of the pipe wall;
e_0	out of roundness before application of loads;
E_{Ti}	short-term modulus of elasticity;
E_{Tv}	long-term modulus of elasticity;
CR	stiffness class for flexible pipes;
ras_i	short-term ring stiffness;
ras_v	long-term ring stiffness;
ν_T	Poisson's ratio.

The soil and laying parameters, which depend on the soil type (5 groups typified) and the degree of compaction (3 cases) are the following:

E_s	modulus of elasticity;
γ	density;
k_1	shear coefficient;
ν_s	Poisson's ratio;
k_2	horizontal earth pressure coefficient;
2α	conventional bedding reaction angle.

The values of parameters E_s , k_2 and 2α are given in tables.

The presence of ground water is taken into account by reducing the value of E_s .

The conditions under which sheeting is removed (3 cases) are taken into account by reducing the values of E_s , 2α , k_1 and k_2 .

Stiffness criterion RIG:

The stiffness criterion characterises the cross sectional behaviour of the pipe in the soil:



Figure A.7 — Cross-sectional behaviour of pipes in the soil

A.4.3.2 Calculation of actions

Vertical pressure due to backfill, p_r :

Vertical pressure due to backfill is calculated with the following equation:

$$p_r = C\gamma H$$

with

p_r	vertical pressure;
γ	density of soil;
H	depth of cover;
C	concentration factor.

— in the case of flexible pipe: $C = 1$

— in the case of rigid pipe:

— laid under embankment: $C = C_2$

— laid in trench:	narrow trench:	$C = C_1$
	wide trench:	$C = C_2$

$C1$ and $C2$ are calculated using Marston's theory ($C1$ and $C2$ can be taken from charts).

Vertical pressure due to live loads, p_{er} :

Live loads are calculated using Frolich's theory, considering the standard convoy defined in the Figure A.8.

The load due to the wheels passing directly above the centreline of the pipe is increased by an impact factor of 1,6, and for the other wheels by a impact factor of 1.

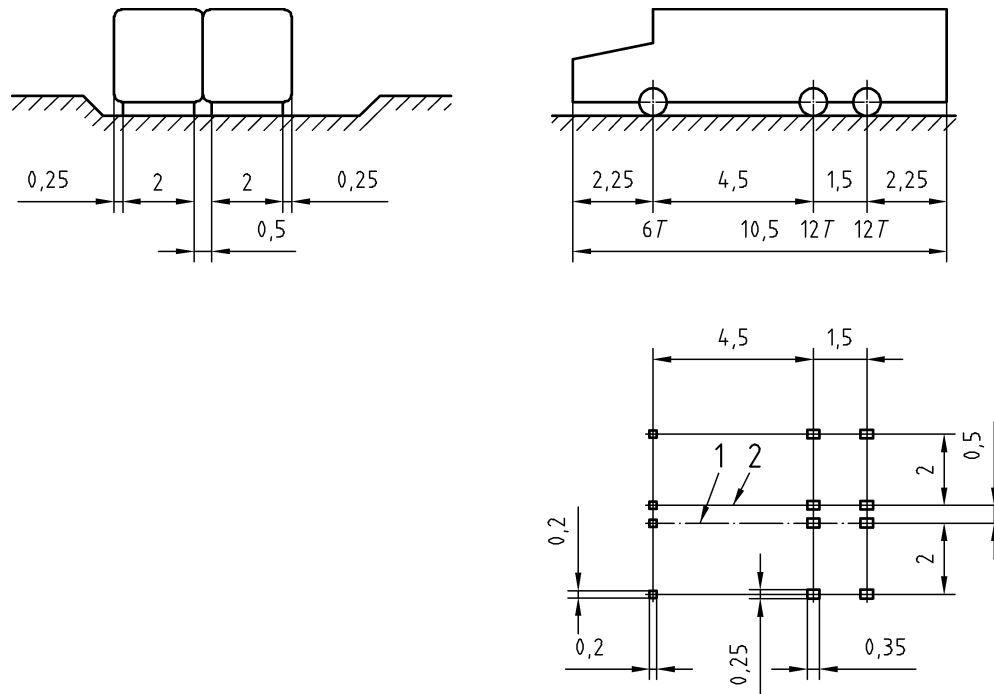


Figure A.8 — Standard convoy

NOTE A table gives the values of p_{er} for different DN and H for $D_e \leq 1,2 D_i$.

Vertical pressure due to permanent surface loads, p_{ep} :

For pipes laid in narrow trench, it is calculated with the following equation:

$$p_{ep} = p_o \cdot e^{-2k_1 H/B}$$

where

p_o permanent surface pressure;

H depth of cover;

B trench width.

Vertical pressure due to construction, p_{ec} :

The vertical pressure due to construction shall be taken into account if it is greater than that due to traffic and permanent surface loads.

Total vertical pressure, p_v :

Total vertical pressure is given by the following equation:

$$p_v = p_r + \max(p_{er} + p_{ep} + p_{ec})$$

Horizontal pressure, p_h :

Horizontal pressure is given by the following equation:

$$p_h = k_2 \cdot p_v$$

External hydrostatic pressure, p_{we} :

External hydrostatic pressure (due to the water table) is considered as uniform, and calculated at the springline of the pipe.

Average external confining pressure, $\bar{p}$

The average external confining pressure is the hydrostatic pressure equivalent to the vertical and horizontal pressures. It is used in particular to verify buckling.

It is given by the following equation:

$$\bar{p} = p_{we} + p_v \left(\frac{1+k_2}{2} \right)$$

A.4.3.3 Calculation of efforts and deflection

In the model used, the pipe is considered an infinitely long cylindrical shell with elastic behaviour. The pipe is taken to have a geometrical imperfection prior to the application of any loads, i.e. it does not have an ideal circular cross section (production tolerances or deflection due to self weight).

The soil is considered as elastic. It is modelled by an infinite number of elastic springs acting perpendicularly to the pipe wall (Winkler assumption).

The model also assumes that there is no slippage between the soil and the pipe, and that radial displacement of the pipe wall results in reception pressure of the soil proportional to the displacement.

Mathematical analysis results in determination of:

- the critical buckling pressure (to be compared to the average confining pressure when verifying buckling);
- the maximum bending moment and deflection, taking account of second-order effects and soil/pipe interaction.

A.4.3.4 Verification of safety

Ultimate limit state verification:

In principle the following verifications are performed for rigid and flexible pipes:

- pipe with rigid behaviour: short-term strength (bearing capacity or stress or resisting moment);
- pipe with flexible behaviour: short- and long-term strength (bearing capacity or stress or resisting moment) and buckling stability.

The following safety factors shall be applied:

- on actions: 1,25;
- on materials: depending on the type of pipe and its man-entry capability.

Table A.5 — Safety factors

	Non man-entry	Man-entry
Reinforced concrete	1,4	1,4
Non reinforced concrete	1,6	1,76
Fibre cement	1,5	1,65
Compact PVC	1,2	1,32
Cast iron	1,2	1,2
Vitrified clay	1,6	1,76

The safety factor for buckling is 2,5.

Serviceability limit state verification:

In principle the following verification:

- pipe with rigid behaviour: for reinforced-concrete pipes, it is verified that under service loading any cracks are stable and of limited opening.
- pipe with flexible behaviour: it is verified that under service loads short-term and long-term deflection remain lower than allowable values.

Fatigue limit state verification:

This verification is generally only applicable to pipes with flexible behaviour laid directly beneath heavy-trafficked pavements.

Choice of parameters used in the design method