GERMAN
ATV-DVWK-STANDARDS

STANDARD
ATV-DVWK-A 127E

Static Calculation of Drains and Sewers

August 2000

Deutsche Vereinigung für Wasserwirtschaft, Abwasser und Abfall e.V.
German Association for Water, Wastewater and Waste

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Contents

Notes for Users

This ATV Standard is the result of honorary, technical-scientific/economic collaboration which has been achieved in accordance with the principles applicable for this activity (statutes, rules of procedure of the ATV and ATV Standard ATV-A 400). For this, according to precedents, there exists an actual presumption that it is textually and technically correct and also generally recognised.

Everyone is at liberty to apply this Standard. However, an obligation for application can arise from legal or administrative regulations, a contract or other legal reason.

This Standard is an important, however, not the sole source of information for correct solutions. With its application no one avoids responsibility for his own action or for the correct application in specific cases; this applies in particular for the correct handling of the margins described in the Standard.

1 Preamble

This standard applies for the static calculation of underground drains and sewers. It can be used analogously for other pipes laid in the ground. With extreme conditions - for example, very large or very small amounts of cover, very large cross-sections, slopes - special consideration is necessary, which can be the basis for deviations from this standard. This also applies for special designs, for example for driven pipes[1)], unstable subsoil and elevated pipelines.

In the standard is presented a calculation method corresponding with today's scientific level, with which pipes of differing rigidity, covering and bedding conditions can be calculated. With this, the stresses compared with older calculation methods can be more accurately acquired. Prerequisites with the validity of the calculation method and for the mathematical security are the standardised material characteristics - ensured through the monitoring of materials - as well as the design in accordance with DIN EN 1610 - ensured by construction supervision.

The standard allows the selection of different parameters by the user. Characteristic values of materials and soil are so matched to the calculation methods that a good agreement with the results of component tests exists. As, in practice, very often no precise details on the types of soil and installation conditions are available, the selection of the assumptions for the calculation lies within the due discretion of the engineer. The standard, in this respect, can only give advice and leads for the normal case. In particular, the assumptions of the soil mechanical characteristic values must take into account the later possibilities for monitoring on the construction site.

The safety coefficients, in accordance with Sect. 9.7, Tables 12 and 13, have been determined under the prerequisites given there and have been matched to the calculation model. If, in the individual case, the measured dispersions of the coefficients of influence are verified, the other safety coefficients can result with the same probability of failure.

For the comparison of deformation in the installed state with the calculation results according to Sect. 8.4 - in particular taking into consideration dispersion - it is recommended that comparable measurements are carried out.

The standard is an important source of knowledge for specialised behaviour in the normal case. It cannot cover all possible special cases, in which advanced or limited measures can be offered.

The calculation examples attached are to simplify the application of the standard.

In the standard presented here numerous new ideas have been elaborated. The ATV-DVWK therefore requests users to make available reports of experience on the practical application of the standard.

Forward to the 2nd Edition

The 1st Edition of the ATV Standard of December 1984 was received positively and met with broad interest.

Experience gathered in the meantime, supplemented by a direct exchange of ideas at seminars on the introduction of this new calculation method as well as the development of the ATV/DVGW Standard ATV-DVWK-A 161 for the static calculation of driven pipes, carried out in the interval, made a new edition appear sensible.

Thus various additions and corrections, serving for better understanding, could be made.

- Load assumptions were expanded for aircraft loads.
- Table 3 was supplemented by the meanwhile standardised glass fibre reinforced plastic pipes (UP-GF).
- The application of the reduction factor α_B (Diag. D5), previously recommended in a note, now becomes mandatory.
- The Equation (6.04) for max. λ was restored from the previous linear to the original form as, in the area of small effective relative projections, too large deviations resulted.
- The calculation of λ_R could be simplified.
- The calculation to take a deformation layer into account could be given more precisely.
- For UP-GF pipes proof of outer fibre strain and a calculation example have been introduced in place of the standardised nominal stiffnesses of the modulus of elasticity.
- Stress and bearing capacity verification is respectively carried out only with pressure distribution in accordance with Bedding Case I or II, the deformation verification according to Bedding Case III.

- The verification for not predominantly static loads was comprehensively formulated identical with ATV Standard ATV-A 161 for driven pipes and a table was supplemented with reduction factors for the respective traffic loads.
- Appendix 2 with the necessary details for static calculation was newly and more clearly designed, so that it can also be used as text in the invitation to tender.

In addition, the contents of the preliminary remarks continues to be recommended for the attention of the user.

Forward to the 3rd Edition

The ATV Standard has proved itself for the static verification of underground drains and sewers. Thus, for example, the determination of the concentration factors for the loading above the pipe has also found international recognition with the concept of the "rigid beam".

Based on new knowledge in pipe statics (trials, comparison with the Finite Element Method, European Standardisation etc) and due to new developments with pipeline systems (e.g. pipes with profiled walls), a requirement for regulation has arisen in various sections of the standard, which are collected together here in a 3rd Edition.

With this, one is concerned with the following new regulations which, in part, also lead to simplifications in the calculation process:

The material characteristic values are adjusted to the current DIN and DIN EN status and are secured through additional regulations.

The deformation module in pipeline zone E_2 can, dependent on the load stress, be increased with built-up embankment sand cover greater than 5 m.

With verification of deformation the reduction of the deformation module in the pipeline zone goes to 2/3.

Verifications of stress and deformation are carried out uniformly using the same support angle 2α (through this only two calculation runs are required with flexible pipes).

The mathematical boundaries between rigid and flexible behaviour is newly determined as $V_{RB} = 1$.

With the determination of the bedding reactions (through compatibility of pipe and soil deformation in the springing) and with verification of deformation, under certain conditions the influence of the normal force and shearing force deformations is taken into consideration.

Deformations can now exceed the previously permitted limiting value of 6 % by up to 50 % if a non-linear stability verification is carried out. An approximation method is given for this.

With verification of stability the deformation of the pipes (structural and elastic) must also now be taken into account.

A new chapter for the peculiarities with the verification of profiled pipes is added; a corresponding Advisory Leaflet ATV-M 127, Part 3 is in preparation.

For the case of additional loading with vertical sheeting (e.g. sheet piling rammed under the pipe invert) attention is drawn to the ATV Working Group 1.5.5 Report "Mathematical formulations for the loading of pipes in trenches using sheet piling revetment" (Korrespondenz Abwasser 12/97) [Not available in English].

In the foreword of the previous editions it has already been established that the validity of ATV Standard ATV-A 127E is limited to standard cases.

In March 1996 ATV Working Group 1.2.3 "Pipe statics" published Advisory Leaflet ATV-M 127-1 "Standard for the static calculation of drains for seepage water from landfills".

It is pointed out that rehabilitation systems (lining measures) may not be verified using Standard ATV-A 127E. For this Advisory Leaflet ATV-M 127-2E "Static calculation for the rehabilitation of drains and sewers using lining and assembly methods" (January 2000) is available.

2 Symbols

Symbol	Unit	Definition
A	m², mm²	Surface area
A_Q	m², mm²	Shear force area (≅ web area with profiled pipes)
a	1	Relative projection
a'	1	Effective relative outreach
a_F	1	Correction factor for road traffic loads
b	m	Trench width at pipe crown level
b_{So}	m	Width of trench bottom
b_D	m	Width of deformation layer
c, c_h, c_v, c_h^*, c_v^*	-	Deformation coefficients
c'	-	Corrected deformation coefficients
c^N, c^S	-	Deformation coefficients to take into account of the normal and/or shear forces
D_{Pr}	%	Compaction (based on simple Proctor density)
d_D	m	Thickness of the deformation layer
d_e	m	External pipe diameter
d_i	m	Internal pipe diameter
d_m	m	Mean pipe diameter
$\Delta d_{frac}/d_m$	%	Characteristic values of relative fracture deformation
E_D	N/mm²	Modulus of elasticity of deformation layer
E_P	N/mm²	Modulus of elasticity of pipe material
E_S	N/mm²	Modulus of deformation of soil
EZ	1	Installation figure
E_1, E_2, E_3, E_4	N/mm²	Modulus of deformation in soil zones 1 - 4
E_{20}	N/mm²	Table value for calculation of E_2
F	kN/m	Force (per unit length of pipe)
F_A, F_E, F_W, F_G	kN/m	Auxiliary loads
F_N	kN/m	Crushing load
tot F	kN/m	Total load
f_1	-	Reduction factor for soil creep

f_2	-	Reduction factor for E_{20} with groundwater
G	-	Soil group
h	m	Height of cover above pipe crown
h_W	m	Height of water level above pipe crown
I	m^4/m, mm^4/mm	Moment of Inertia
K_1, K_2	-	Ground pressure ratio in soil zones 1 and 2
K^*	-	Coefficient for the bedding reaction pressure
K'	-	Modulus for deformation (subgrade modulus)
k_{sr}	-	Bedding modulus with constant radial bedding
M	kNm/m	Bending moment
m	-	Coefficient of moment
N	kN/m	Normal force
n	-	Coefficient of normal force
p, p_V	kN/m^2	Soil stresses due to traffic load
p_E p_F	kN/m^2	Soil stresses due to earth load and surface load
$p_{E,A}$	kN/m^2	Soil stress taking into account buoyancy
p_{ew}	kN/m^2	External water pressure
crit p_{ew}	kN/m^2	Critical external water pressure
p_f	-	Failure probability
p_i	kN/m^2	Internal pressure
p_s	kN/m^2	Area load
Q	kN	Maximum shear force in the pipe wall
q_h	kN/m^2	Horizontal soil stress on pipe
q^*_h	kN/m^2	Horizontal bedding reaction pressure
q_v	kN/m^2	Vertical soil stress on pipe
$q_{v,A}$	kN/m^2	Vertical soil stress taking into account buoyancy
crit q_v	kN/m^2	Critical vertical total load
r_A, r_E	m	Auxiliary radii
r_m	m	Radius of the centroidal axis of the pipe wall
S	-	Area moment 1st grade about the centroidal axis of the cross-section (static moment)
S_{Bh}, S_{Bv}	N/mm^2	Bedding stiffnesses
S_D	N/mm^2	Stiffness of deformation layer
S_P, S_0	N/mm^2, N/m^2, kN/m^2	Pipe stiffness
$\overline{S}_0$	N/mm^2 N/m^2 kN/mm^2	Weighted pipe stiffness
s	mm	Wall thickness
s_{id}	mm	Imaginary wall thickness
V_{RB}	-	System stiffness
V_S	-	Stiffness ratio
W	m^3/m, mm^3/mm	Moment of resistance of the pipe wall
α	°	Half bedding angle
α_{ll}	-	Enlargement ratio of the bending moment for non-linear verifications
α_B	-	Reduction factor
α_D	-	Snap-through coefficient
α_C	-	Correction factor for curvature

β	°	Slope angle
χ	-	Coefficient of safety
χ_{fc}	-	Coefficient of safety for flexural compression
χ_{ft}	-	Coefficient of safety for flexural tension
χ_P	kN/m³	Unit weight of the pipe material
χ_{pew}	-	Safety coefficient for stability under external water pressure
χ_{qv}	-	Safety coefficient for stability verification under earth and traffic loads
χ_S	kN/m³	Unit weight of the soil
χ'_S	kN/m³	Unit weight of the soil under buoyancy
χ_W	kN/m³	Unit weight of water
Δd_v	mm	Change of (pipe) diameter
Δf	-	Valuation constant
δ	°	Angle of wall friction
ε	-	Measured compression set of the deformation layer
ε_R	-	Calculated value of outer fibre strains
$\overline{\varepsilon}_R$	-	Weighted calculated value of outer fibre elongations
ζ	-	Correction factor for horizontal embedment stiffness
κ, κ_β	-	Reduction factor for a trench load i.a.w. the Silo Theory
κ_0, $\kappa_{0\beta}$	-	Reduction factor for a surface load i.a.w. the Silo Theory
κ_{v2}, κ_{a1}, κ_{a2}	-	Reduction factors of the critical buckling load
λ_S	-	Concentration factor in soil adjacent to pipe
λ_{fu}	-	Upper limit of concentration factor above pipe
λ_{fl}	-	Lower limit of concentration factor above pipe
λ_P, λ_{PG}, max λ	-	Concentration factor above pipe
ν	-	Transversal contraction number of the pipe material
σ	N/mm²	Stress
σ_{bT}, σ_{bC}	N/mm²	Limits of tension of the pipe material with bending tension and compression
σ_P	N/mm²	Bending tensile strength, arithmetic value
$\overline{\sigma}_P$	N/mm²	Weighted bending tensile strength
$2\sigma_A$	N/mm²	Stress range (of the pipe material)
τ	N/mm²	Transverse strain
φ	-	Impact coefficient
φ'	°	Angle of internal friction

Indices

e	external
h	horizontal
i	internal
q	as a result of external loads
g	as a result of dead weight
M	momentary values
L	long-term values
v	vertical
w	as a result of water filling

3 Technical Details

3.1 Types of Soil

The following types of soil can be differentiated (symbols in accordance with DIN 18 196 are given in brackets):

Group 1: Non-cohesive soils (GE,GW,GI,SE,SW,SI)

Group 2: Slightly cohesive soils (GU,GT,SU,ST)

Group 3: Cohesive mixed soils, coarse clay (silty sand and gravel, cohesive stony residual soil) ($G\bar{U}$, $G\bar{T}$, $S\bar{U}$, $S\bar{T}$,UL,UM)

Group 4: Cohesive soils (e.g. clay) (TL,TM,TA,OU,OT,OH,UA)

Table 1: Types of soil

Group	Spec. gravity	Spec. Gravity under buoyancy	Internal friction angle	Modulus of elasticity E_S in N/mm² with degrees of compaction D_{Pr} in %						Exponent in Eqn. (3.02)	Reduction factor for creep
	χ_S kN/m³	χ_S' kN/m³	φ' °	85	90	92	95	97	100	Z -	f_1 -
G1	20	11	35	2[2)]	6	9	16	23	40	0.50	1.0
G2	20	11	30	1.2	3	4	8	11	20	0.35	1.0
G3	20	10	25	0.8	2	3	5	8	13	0.20	0.8
G4	20	10	20	0.6	1.5	2	4	6	10	0	0.5

The modulus of elasticity E_S (secant modulus) applies as guidance value for the stress range between 0 and 0.1 N/mm².

For programming, the basic values of the modulus of elasticity E_S with load stresses from 5 m of covering (i.e. p_E = 0.1 N/mm²) can be calculated in accordance with Eqn. (3.01):

$$E_S = \frac{40}{G} \cdot e^{-0.188 \cdot (100 - D_{Pr})} \qquad (3.01)$$

Here, G is the figure of the soil group (e.g. for Soil Group 1, G = 1).

For higher stresses the modulus of elasticity increases. Without special verification, one is to calculate using values in the table in this case also. The higher modulus of elasticity $E_{S,\sigma}$ with built-up cover is calculated as follows, dependent on the load stress p_E, in accordance with Eqns. (3.01) and (3.02):

2) E_S values ≥ 2.0 N/mm² are to be rounded to whole numbers

$$E_{s,\sigma} = E_s \cdot \left(\frac{p_E}{100}\right)^z \tag{3.02}$$

p_E is to be applied in kN/m². For the various soil groups in accordance with Table 1, the exponents z apply as given there.

So far as no precise details exist for the given soil groups, the characteristic values in Table 1 are to be used in individual cases.

For types of soil without substances which cannot be assigned in Table 1 - for example, organic soils, bulk materials, waste - the characteristic values are to be determined for the individual case. With this, particular attention is to be paid to the time-dependent creep behaviour of cohesive and organic soils using the reduction factor f_1. For verification the plate bearing test in accordance with DIN 18 134 can, for example, be applied; the secant module is for the stress range between 0 and the stress under live loads. In the case of verification of the elasticity modulus at least five values are to be determined; the smallest value is decisive.

With appropriate calibration a check can also be carried out using the dynamic plate loading test in accordance with TPBF-StB (German technical test conditions for work in road construction), Part 8.3.

3.2 Traffic Loads

3.2.1 Road Traffic Loads

The standard vehicles (Fig. 1), defined in DIN 1072, are to be used for the determination of loads.

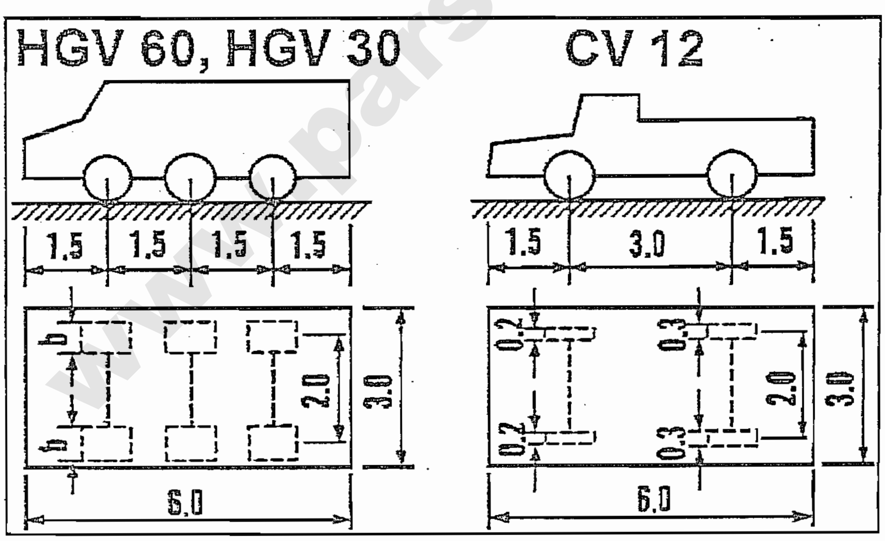


Fig. 1: Standard vehicles

Table 2: Loads and tyre contact areas of standard vehicles

Standard vehicle	Total load	Wheel load	Width of tyre contact area	Length
	KN	KN	m	m
HGV 60	600	100	0.6	0.2
HGV 30	300	50	0.4	0.2
GV 12	120	front 20 rear 40	0.2 0.3	0.2 0.2

Outside traffic areas, CV 12 (commercial vehicle) is to be applied as minimum load., EC vehicles in accordance with Directive 85/3 EGW are also covered with HGV 60 (heavy goods vehicle). These load formulations correspond with DIN 1072 bridge classes 60/30 and 30/30 respectively.

3.2.2 Rail Traffic Loads

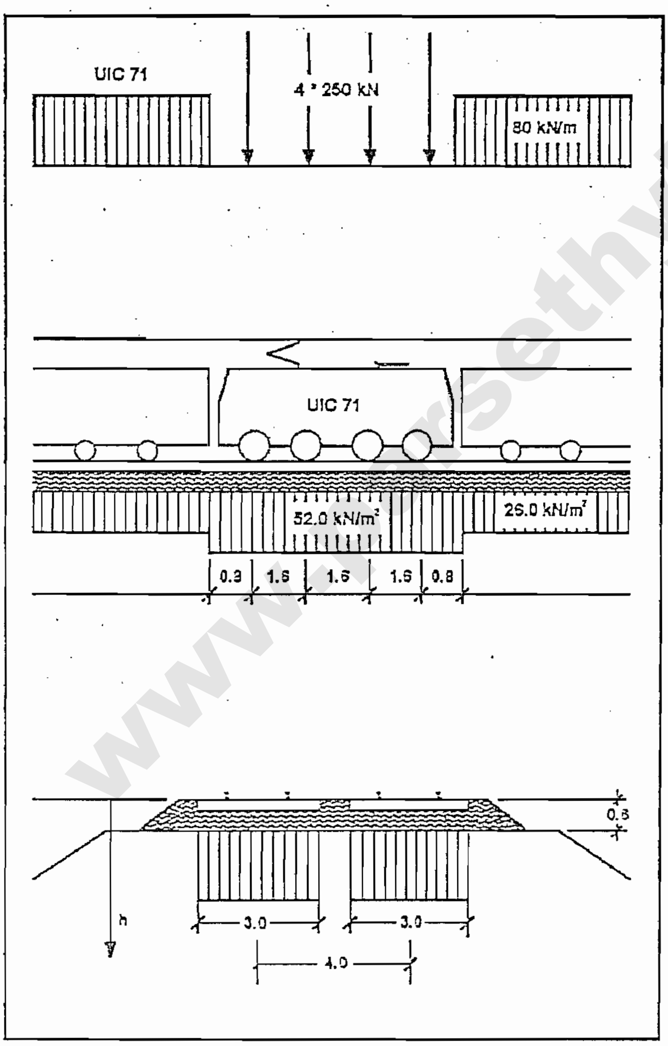


Fig. 2: Loading diagram UIC 71

The loading diagram of the UIC 71 (Fig. 2) given in the DS 804[3] of the Deutsehe Bahn AG is relevant for load determination. Tramlines to be taken into account according to special load details.

3.2.3 *Aircraft Traffic Loads*

Loading diagrams DAC 90 to DAC 750 (Fig. 3) of the Federation of German Commercial Airports (Arbeitsgemeinschaft Deutscher Verkehrsflughafen) or the details of the airport administration concerned are relevant for the determination of load.

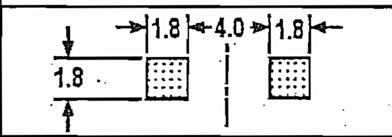
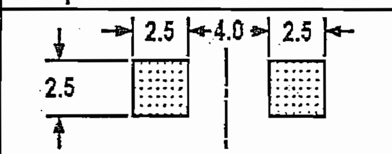
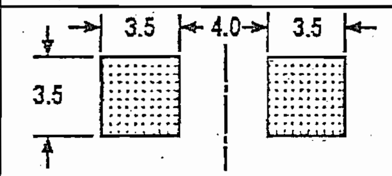
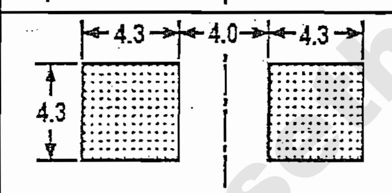
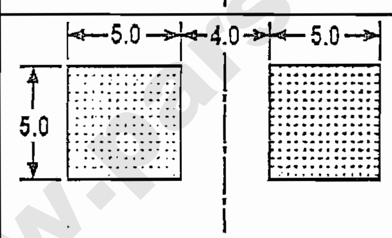
Dimensioning aircraft	Contact area of main undercarriage	Load pressure
t	m	kN/m²
DAC 90		150
DAC 180		150
DAC 350		150
DAC 550		150
DAC 750		150

Fig. 3: Loading diagrams of the dimensioning aircraft (DAC)

3.2.4 *Other Traffic Loads*

Traffic loads under construction site conditions are to be taken into account.

With loads, which are caused by special traffic, for example in heavy industry operations, the total weight, axle loads, dimensions and the size of the wheel contact areas must be given for the individual case.

[3] DS 804: Regulations for Railway Bridges and other Engineering Structures (VEI); Issue 4, valid from 31.07.1996

3.3 Area Loads

Bulk materials, structural foundations, and similar are to be accounted for as area loads (if required also as point loads). Compacted built-up embankments and uncompacted fill do not count as area loads; they are to be dealt with as earth cover.

3.4 Pipe Materials

The characteristic values relevant for dimensioning follow different rules. They are particularly influenced by ageing, behaviour under long-period stressing and temperature. These influences are to be taken into account through the following constraints for the dimensioning period which, however, do not always all have to be relevant:

Ageing	50 years
Behaviour under long-period stressing:	50 years
Pulse loading:	$\geq 2 \cdot 10^6$ load changes
Temperature, longer-term:	20° C
Temperature, short-term (corresponding to a period of two years in 50 years)	DN $\leq$ 400: 45° C
	DN > 400: 35° C

The characteristic values necessary for the determination of shear forces, strains and deformation are given for standard pipes, designed for laying underground, in Table 3. For loading cases, with which high normal strains appear in the pipe wall supplementary determinations are to be made (e.g. load case internal pressure with deep covering). Appropriate pipe standards and material characteristic values are to be applied analogously.

For areas of operation - in particular with chemical stresses - which go beyond the normal case, the relevant characteristic values for this are to be determined in the individual case. The observation of the tabular values in Table 3 or the increased values is to be confirmed by a recognised or accredited test centre and is to be monitored by quality assurance.

With reference to European standard specifications, the requirements of the part of the respective EN, non-harmonised within the scope of the building product directive, always apply.

Table 3: Material characteristic values

Material	Representative value of the modulus of elasticity[4)] E_P		Specific gravity χ_P	Representative value of bending tensile strength[5)] σ_P		Range of stress[6)] $2\sigma_A$
	Short-term $E_{P,K}$ N/mm²	Long-term $E_{P,L}$ N/mm²	KN/m³	Short-term $\sigma_{P,K}$ N/mm²	Long-term $\sigma_{P,L}$ N/mm²	N/mm²
Fibre cement	20,000		20	[7)]		0.4 · $ß_{RBTS}$
Concrete	30,000		24	[8)]		0.4 · $ß_{RBTS}$
Cast iron (CM) (ductile)[9)]	170,000		70.5	[10)]		135
Cast iron (laminar graphite)	100,000		71.5	[11)]		70
Polyvinyl chloride (PVC-U)	3,000 [12) 13)]	1,500 [12) 13) 14)]	14 [15)]	90 [13) 16) 17)]	50 [13) 16) 17)]	[18)]
Polypropylene (PP)[19)] PP-B and PP-H [20)]	1,250 [12) 13)]	312 [13) 21)]	9 [22)]	39 [13) 16) 17)]	17 [13) 16) 17)]	[18)]
PP-R [23)]	800 [12) 13)]	200 [13) 21)]	9 [22)]	27 [13) 16) 17)]	14 [13) 16) 17)]	[18)]
Polyethylene high density (PE-HD) [24)]	800 [12) 13)]	200 [13) 25)]	9.4 [26)]	21 [13) 16) 17)]	14 [13) 16) 17)]	[18)]

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4) Details of figures are representative values which are determined from measurements of deformation on pipes.

5) The compressive strain can also be relevant, in particular with thin-walled pipes. For driven pipes the representative values in ATV Standard ATV-A 125E and ATV-A 161E apply.

6) Observance of the required ring bending tensile strength, the strain in outer fibres or the hoop strength is to be verified after carrying out the fatigue strength test.

7) DIN EN 588; the ring bending strengths are calculated from the lowest values of the load crushing forces (95 % fractile; AQL 4 %).

8) DIN 4032; the ring bending strengths are calculated from the lowest values of the load crushing forces (95 % fractile; AQL 4 %).

9) With verification of deformation and strength (see Sects. 9.4 and 9.5) the cement mortar (CM) cladding can be taken into account in that a sixth of its layer thickness is added to cast iron wall thickness and one seventh of its layer thickness to the steel wall thickness respectively. Using the imaginary wall thickness s_{id} so obtained, the moment of inertia $I = s_{id}^3/12$ is obtained which is to be linked the modulus of elasticity E_R.

10) DIN EN 598; the tensile strength is laid down in the standard specification. The bending tensile strength is given as 550 N/mm² in the DVGW "Study on buried drinking water pipelines made from various materials", issued in June 1971 by the German Minister of the Interior, Appendices 2 and 3.

11) DIN 19522-2; the ring bending tensile strength is to be set the same as the ring compression strength given in the standard specification.

12) Tested in accordance with DIN 54852 (4 point creep bending test), test directive in accordance with DIN 53457, test piece production in accordance with DIN 16776-2.

13) Higher representative values can be applied to the calculation if these are verified for the material used.

14) Determined from the momentary value and the creep ratio (2.0) in accordance with DIN EN 1401-1 and DIN EN ISO 9967 with characteristic values for 2 years for the description of the long-term behaviour. Also permitted for the long-term verification for 50 years.

15) In accordance with DIN EN 1401-1.

16) For plastic materials the bending tensile strength is designated and given as bending strength.

17) Smallest values (lower 95% fractile) corresponding to the Round Robin Test of the raw material producer as well as on the basis of Test Report No. 36893/98-11 of the SKZ (Southern German Plastic Centre) Würzburg.

18) $2 \cdot \sigma_{acperm} \geq 2 \cdot \sigma_{aperm} = (1 - 1R) \cdot \sigma_{max,avail}$; with R = 0.2. The resistance to fatigue with loading which is not mainly static is to be verified for $n = 2 \cdot 10^6$ change of load at 3 Hz.

19) PP-B = block copolymer; PP-H = homopolymer; PR-R = random copolymer.

20) DIN EN 1852-1.

21) Determined from the momentary value and the creep ratio (4.0) in accordance with DIN EN 1852-1 and DIN EN ISO 9967 with characteristic values for 2 years for the description of the long-term behaviour. Also permitted for the long-term verification for 50 years.

22) In accordance with DIN EN 1852-1.

23) DIN EN 1852-1 and DVS 2205-2 (Supplement 1 (Issue 08/97)).

24) PE-HD as PE 63, PE 80 or PE 100 in accordance with DIN EN ISO 12162.

25) Determined from the momentary value and the creep ratio (5.0) in accordance with characteristic values for 2 years for the description of the long-term behaviour. Also permitted for the long-term verification for 50 years.

26) In accordance with prEN 12666-1.

Material	Representative value of the modulus of elasticity[4] E_P		Specific gravity χ_P	Representative value of bending tensile strength[5] σ_P		Range of stress[6] $2\sigma_A$
	Short-term $E_{P,K}$ N/mm²	Long-term $E_{P,L}$ N/mm²	KN/m³	Short-term $\sigma_{P,K}$ N/mm²	Long-term $\sigma_{P,L}$ N/mm²	N/mm²
Steel (CM)[9]	210,000		77	[27]		[28]
Reinforced concrete	30,000		25	[29]		Bst 500 P 80
Prestressed concrete	39,000		25	[30]		[30]
Vitrified clay	50,000		22	[31]		[32]
Unsaturated polyester resin, glass fibre reinforced (UP-GF)	[13) 33)]	[13) 34)]	17.5	[13) 35)]	[13) 35)]	[18]
	Representative values of the hoop stiffness S_0 [N/m²]			Representative values of the relative failure strain $\Delta d_{frac}/d_m$ [%]		
- SN 1250	1,250	625		30	18	
- SN 2500	2,500	1,250		25	15	
- SN 5000	5,000	2,500		20	12	
- SN 10000	10,000	5,000		15	9	

27) DIN 1629: in this standard specification minimum apparent limits of elasticity (0.2 %) and tensile strengths. As representative value σ_P the apparent bending limit of elasticity is decisive which is 1.43 times the minimum apparent limit of elasticity with mainly dead load and with steels St 35 and St 37-2. Valid for steel pipes with wall thickness up to 16 mm. Under loading, which is not mainly static, the minimum apparent limit of elasticity given in the standard specification is to be applied.

28) With loading which is not mainly static the values in accordance with ATV-DVWK Standard ATV-DVWK-A 161E: 1990, Sect 7.2, are relevant.

29) DIN 4035.

30) DIN 4227.

31) DIN EN 295: the ring tensile strengths are calculated from the minimum of the load crushing forces (95 % fractile; AQL 4 %)..

32) DIN EN 295-1.

33) $S_{0,min}$ in accordance with prEN 1636.

34) Determined from the momentary value and the creep ratio (2.0) with characteristic values for 2 years for the description of the long-term behaviour. Also permitted for the long-term verification for 50 years. Tests take place in accordance with DIN EN 1228 (momentary) and DIN EN 1225 (long-term).

35) The following applies: $\varepsilon_R = \pm 4.28 \cdot s/d_m \cdot \Delta d_{frac}/d_m$ with Δd_{frac} in accordance with prEN 1636 (momentary and long-term) with the respective values for s and d_m.

4 Construction Work

Basis for the application of the calculation method is the placement of pipelines, in particular the support, embedding and compaction, laid down in DIN EN 1610 (see also ATV Standard ATV A-139E [36]).

The type of shuttering is to be take into account with the calculation of the earth load (see also DIN 4124 and DIN EN 1610 respectively).

Under traffic areas the [German] ZTVA-StB (Zusätzliche Technische Vertragsbedingungen und Richtlinien für Aufgrabungen in Verkehrsflächen [Supplementary Technical Contractual Conditions for Excavations in Traffic Areas]) apply. Outside traffic areas these apply analogously.

4.1 Suitable Types of Soil

In the area of the pipeline zone (pipe support and bedding up to 0.15 m above the pipe crown, with embanked covering at least 1.5 d_e laterally from the pipe). Only compactible soils or permitted types of soil in accordance with DIN EN 1610 may be used; for pipes, for whose dimensioning a verification of deformation is necessary, only soil from Groups G1, G2 and G3 (see Sect. 3.1) is to be placed. In the area above the pipeline zone all types of soil given in Sect. 3.1. may be employed.

The laying of pipelines in organic soils (HN, HZ, F, see DIN 18 196)requires special measures with regard to the support and embedding of the pipes as well as the filling of the open cut.

4.2 Notes for Installation

The embedding of the pipeline is part of the task of forming the pipe support and essentially determines the distribution of load and the pressure distribution on the circumference of the pipe as well as the possibility of forming a lateral ground pressure with a load relieving effect on the pipeline. Attention is to be paid to agreement between the effective bedding angle with that of the static calculation.

Unfavourable effects with incorrect employment of shuttering plates and equipment cannot be included within the scope of the calculation model applied here. Both gradual drawing following compaction as well as drawing following backfilling and compaction lead to considerable disturbance in the soil. An approach to rough consideration is given in Sect. 6.2.2.

With groundwater effects there is a danger in the area of the pipeline zone. So far as they are not countered by design measures, the possible increase of the load in the calculation in accordance with Table 8 is to be considered (reduction of the deformation module E_{20}).

[36] ATV Standard ATV-139E " Laying and Testing of Drains and Sewers, published by ATV-DVWK e.V. German Association for Water, Waste and water, Theodor-Heuss-Allee 17, D-53773 Hennef

Determination of loading according to surcharge condition A4 (Sect. 5.2.1.2) or embedding condition B4 (Sect. 6.2.1) respectively, assumes that, in the trench backfill, the degree of compaction in accordance with ZTVE-StB 9[37] is verified. During the embedding of flexible pipes the increase of the vertical diameter (deformation) may not exceed the amount of the calculated short-term value of the deformation (reduction of the vertical diameter).

In order to prevent the impairment of the pipelines through dynamic strains as a result of soil compaction, only certain compactors are permitted with pipe laying. Which light equipment may be used in the pipeline zone and which medium and heavy equipment may be employed above the pipeline zone are given in ATV-DVWK Standard ATV-DVWK-A 139E and in the "Advisory Leaflet for the Backfilling of Service Trenches".

Medium and heavy rammers and vibrators may first be employed upwards from 1 m above the pipe crown - measured in the compacted state. Dependent on the type of equipment (service weight), type of soil and covering height even larger effect depths and thus higher minimum covering result.

The static calculation in general assumes that loading and reaction over the pipe length are evenly distributed. The pipe bedding is therefore to be so formed that axial bending and point loads - for example at sleeve joints - are avoided.

5 Loading

5.1 Load Cases

Pipelines can be stressed through the load cases earth load, traffic load and other area loads, which are covered in the following sections. The load cases dead weight, water filling and water pressure are dealt with in Section 8. Axial bending, temperature difference and buoyancy are also to be taken into account.

5.2 Mean Vertical Soil Stresses at the Level of the Pipe Crown

First, the stress in the soil is determined independent of the pipe material. The load distribution on pipe and soil, which can lead to an increase of stress above the pipe, is part of Sect. 6.

5.2.1 Earth Load and Evenly Distributed Area Loads (Bulk Materials)

5.2.1.1 Silo Theory

Friction forces on existing trench walls can lead to a reduction of the ground stress and justify the application of the Silo Theory under the assumption that the trench walls (friction surfaces) remain over a long period. According to the Silo Theory, one obtains the average vertical stress as a result of the earth load in a horizontal section of the trench backfill at a separation of h from the surface

$$p_E = \kappa \cdot \chi_B \cdot h \qquad (5.01)$$

[37] Additional Technical Regulations and Standards for Earthworks in Road Construction, published by The Federal German Ministry for Traffic. Obtainable through the Research Corporation for Road and Traffic Affairs, Alfred-Schütte-Allee 10, D-50679 Köln

For an evenly distributed area load p_O the average vertical stress is

$$p_E = \kappa_O \cdot p_O \tag{5.02}$$

Further prerequisite for the application of the reduction factors is $E_1 \leq E_3$ for κ and $E_1 < E_3$ for κ_O and, for all cases, a degree of compaction of the trench backfill of $D_{Pr} > 90$ %.

With increasing trench widths b, κ and κ_O steadily approach the value 1. In the case of embanked cover therefore

$$p_E = \chi_B \cdot h + p \tag{5.03}$$

A steady increase is also assumed for the dependence of the stress above the pipe on the trench width (see Sect. 6.4). With this, the previous customary differentiation into trench conditions and embankment conditions is dispensed with. Decisive for the reduction of the earth load are the side pressure on the trench walls, emphasised by the ratio k1 of horizontal to vertical earth load, and the effective wall friction angle δ.

Thus, according to the Silo Theory

$$\kappa = \frac{1 - e^{-2\frac{h}{b}K_1 \cdot \tan\delta}}{2\frac{h}{b}K_1 \cdot \tan\delta} \tag{5.04}$$

$$\kappa_O = e^{-2\frac{h}{b}K_1 \cdot \tan\delta} \tag{5.05}$$

Tables T1 and T2 are given in Appendix 1 for κ and κ_O.

For $\delta = 0$, $\kappa = \kappa_0 = 1$.

5.2.1.2 Covering Conditions for the Backfilling of Trenches

With the trench backfill above the pipeline zone, four covering conditions A1 to A4 are differentiated:

A1: Trench backfill compacted against the natural soil by layers (without verification of the degree of compaction); applies also for beam pile walls (Berlin shuttering).

A2: Vertical shuttering of the pipe trench using trench sheeting, which is first removed after backfilling.
Shuttering plates or equipment which, with the backfilling of the trench, are removed step-by-step.
Uncompacted trench backfill.
Washing-in of the backfill (suitable only with soil of Group 1).

A3: Vertical shuttering of the pipe trench using sheet piling, lightweight piling profiles, wooden beams, shuttering plates or equipment which are first removed following compaction.

A4: Trench backfill compacted against the natural soil by layers, with verification of the degree of compaction required according to ZTVE-StB (see Sect. 4.2); applies also for beam pile walls (Berlin shuttering). Covering condition A4 is not applicable with soil of Group 4.

The allocated representative values K_1 and δ are to be taken from Table 4. The corresponding deformation modules EB are given in Sect. 6.2.2, Table 8.

Table 4: Earth pressure ratio K_1 and wall friction angle δ

Covering conditions	K1	δ
A1	0.5	$\frac{2}{3}\varphi'$
A2	0.5	$\frac{1}{3}\varphi'$
A3	0.5	0
A4	0.5	φ'

If soil other than that excavated is used for the backfill of the open cut then the respectively smaller friction angle is relevant.

5.2.1.3 Trench Shapes

5.2.1.3.1 Trench with Parallel Walls

The relevant geometrical values are to be taken from Fig. 4 For this case Eqns. (5.04) and (5.05) apply.

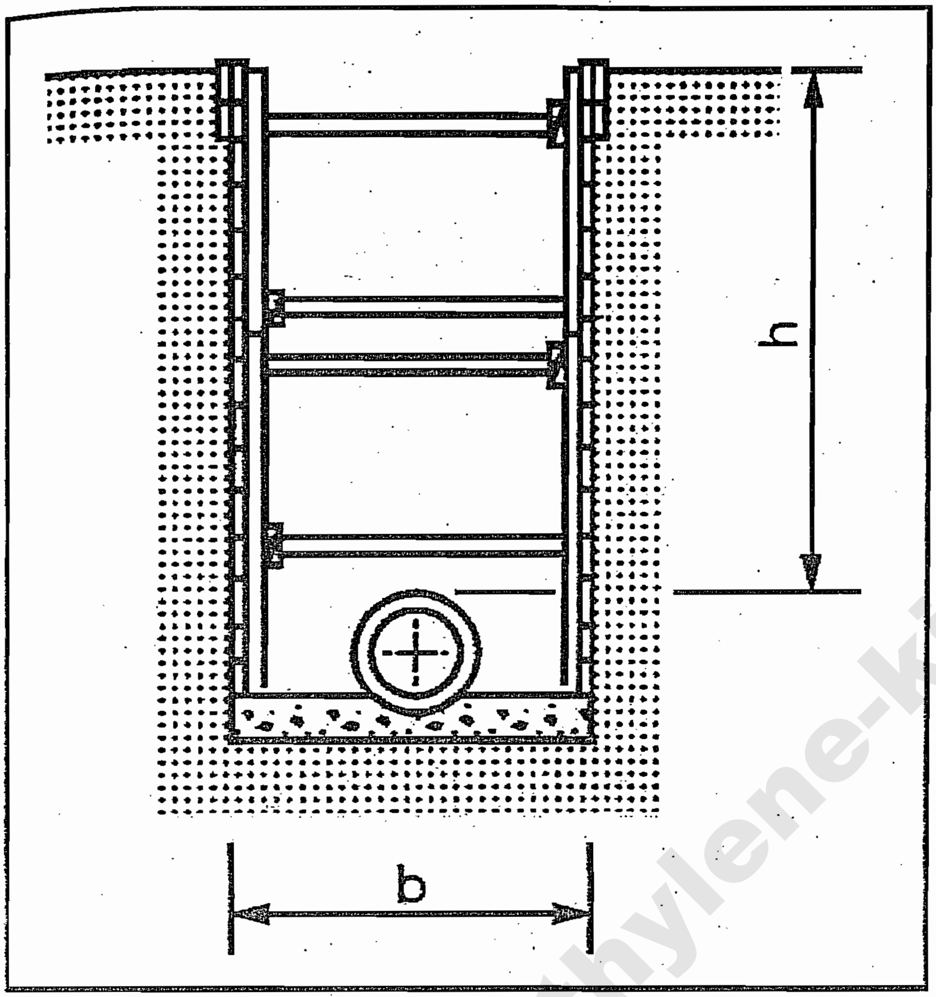


Fig.4: Trench with parallel walls

5.2.1.3.2 Trench with Sloped Walls

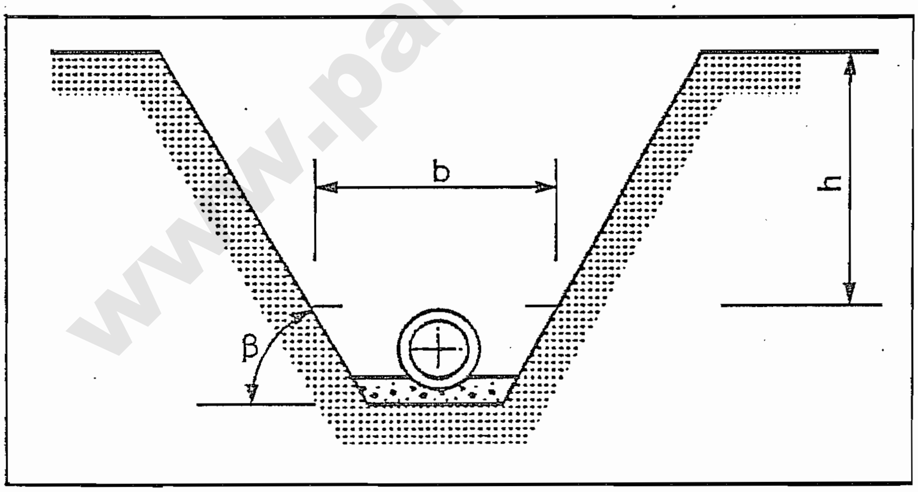
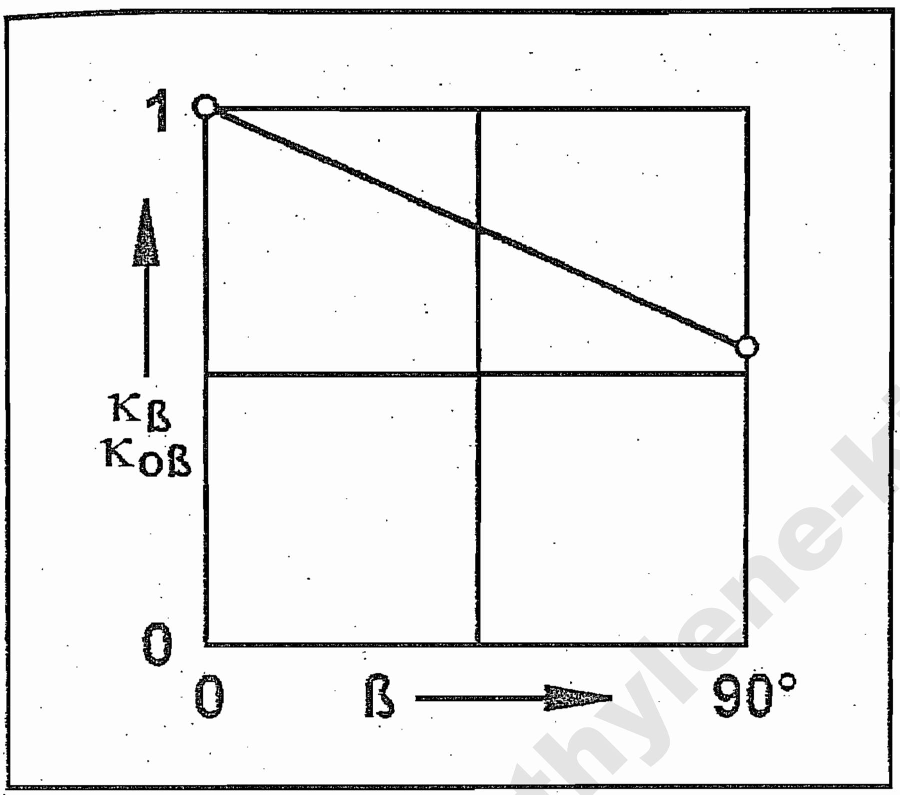


Fig. 5: Trench with sloped walls

For an arbitrary slope angle ß one obtains κ_β through linear interpolation according to the slope angle between $\kappa_\beta = 1$ for $\beta = 0°$ and κ for $\beta = 90°$ (parallel walled trench, Eqn. 5.04), see also Diag. D1.

$$\kappa_{ß} = 1 - \frac{ß}{90} + \kappa \frac{ß}{90} \qquad ß \text{ in degrees} \qquad (5.06)$$

Applies also for $\kappa_{Oß}$



Diag. D1: $\kappa_{ß}$ **and** $\kappa_{Oß}$ **for trenches with sloped walls**

5.2.1.3.3 Stepped Trench

The loading is made up as mean value of two load cases which correspond with the shape of the trench right and left of the trench axis. The calculated trench shapes for these load cases result through mirroring, in each case of a part of the trench on the symmetry axis of the pipe (see Fig. 6). For the upper pipe there also results a simple trench as well as a partially formed trench which can be calculated as further trench.

With other trench shapes one proceeds analogously.

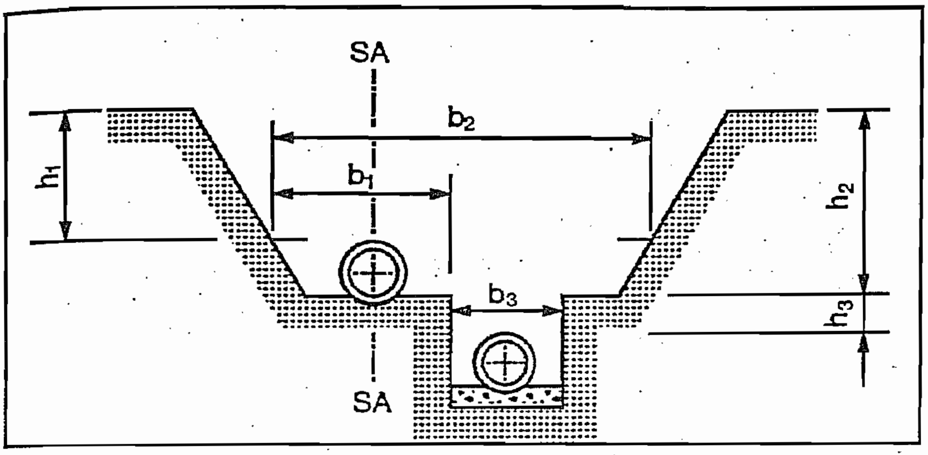


Fig. 6: Stepped trench

5.2.2 Traffic Loads and Limited Area Loads

5.2.2.1 Road Traffic Loads

The soil stress p as a result of road traffic loads in dependence on the cover height and on the pipe diameter are calculated according to the following approximate equation:

$$p = a_F \cdot p_F \tag{5.07}$$

with

$$p_F = \frac{F_A}{r_A^{\,2} \cdot \pi}\left\{1 - \left[\frac{1}{1+\left(\frac{r_A}{h}\right)^2}\right]^{\frac{3}{2}}\right\} + \frac{3 \cdot F_E}{2 \cdot \pi \cdot h^2}\left[\frac{1}{1+\left(\frac{r_E}{h}\right)^2}\right]^{\frac{5}{2}} \tag{5.08}$$

$$a_F = 1 - \frac{0.9}{0.9 + \frac{4h^2 + h^6}{1.1 d_m^{2/3}}} \tag{5.09}$$

$$d_m = \frac{d_a + d_i}{2} \qquad (5.10$$

p_F is an approximation for the maximum stress according to Boussinesq under wheel loads and wheel contact areas in accordance with DIN 1072 [38].

[38] The loading arrangements HGV 60/30 and 30/30 scheduled in DIN 1072 are not used for buried pipelines as the calculated increase of the total load (earth and traffic load) is slight and is balanced by arithmetic non-application of the increase of the side pressure. The existing earth cover additionally dampens momentary loads, also the impact coefficient covers minor increases of load.

a_F is a correction factor to take into account the pressure spread over the pipe cross-section and the length of pipe also carrying loads with small cover heights. This approximation is based on a pressure spread with the slope 2:1.

The auxiliary loads F_A and F_E as well as the auxiliary radii r_A and r_E are given in Table 5.

Table 5: Auxiliary loads and auxiliary radii

Standard vehicle	F_A kN	F_E kN	r_A m	r_E m
HGV 60	100	500	0.25	1.82
HGV 30	50	250	0.18	1.82
CV 12	40	80	0.15	2.26

The dimensions h and d_m are to be applied in m for the dimensionless factor a_F to be applied in Eqn. (5.09). Eqn. (5.09) applies within the limits

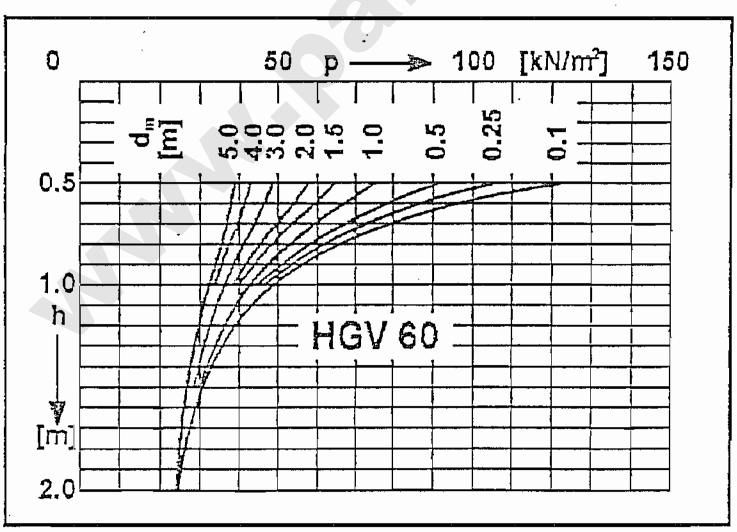
h ≥ 0.5 m
dm ≤ 5.0 m

The result of Eqn. (5.07) is shown in Diag. D2.

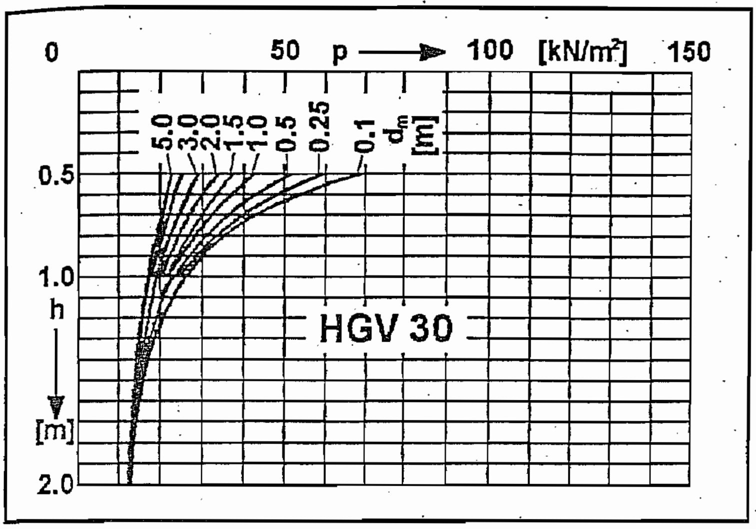
Special consideration is to be given for cover heights h < 0.5 m.

Horizontal stresses in the soil as a result of traffic loads are not taken into account.

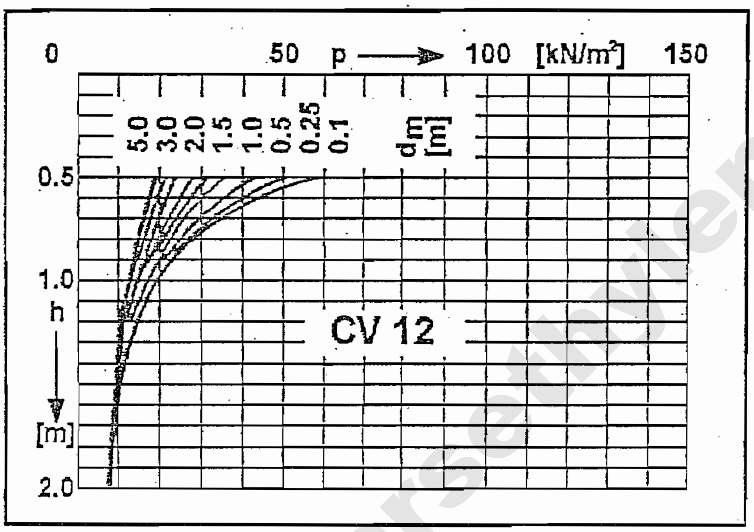
The vertical stresses in the soil as a result of traffic loads may be calculated for verification of security against fatigue loading behaviour (see Sect. 9.7.4), without special proof, with a cover height increased by 0.3 m. Through this it is taken into account that, for frequent load change, a road surface with favourable load distribution is always available.



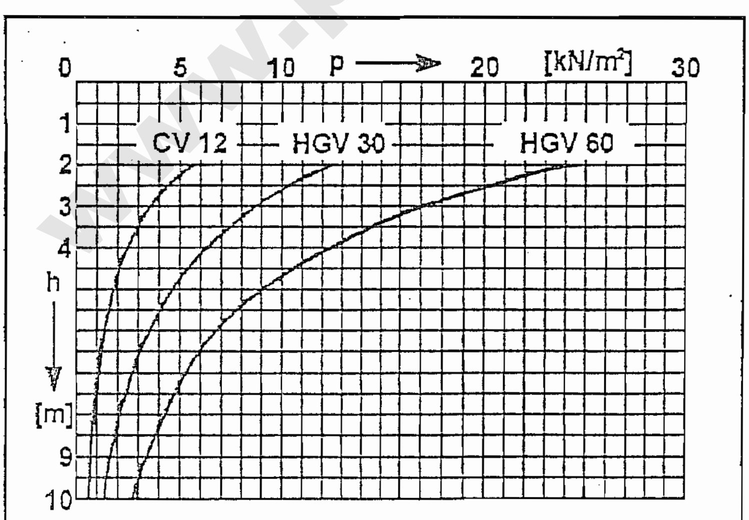
Diag. D2a: Soil stress p as a result of HGV 60 h = 0.5 m to h = 2 m



Diag. D2b: Soil stress p as a result of HGV 30 h = 0.5 m to h = 2 m



Diag. D2c: Soil stress p as a result of CV 12 h = 0.5 m to h = 2 m



Diag. D2d: Soil stress p as a result of HGV 60, HGV 30, CV 12 h = 2 m to h = 10 m

The stresses resulting from traffic loads are to be multiplied by the impact allowance φ:

$$p_V = \varphi \cdot p \tag{5.11}$$

Coefficients of restitution are to be taken from Table 6.

Table 6: Coefficients of restitution for road traffic loads (in accordance with DIN 4033)

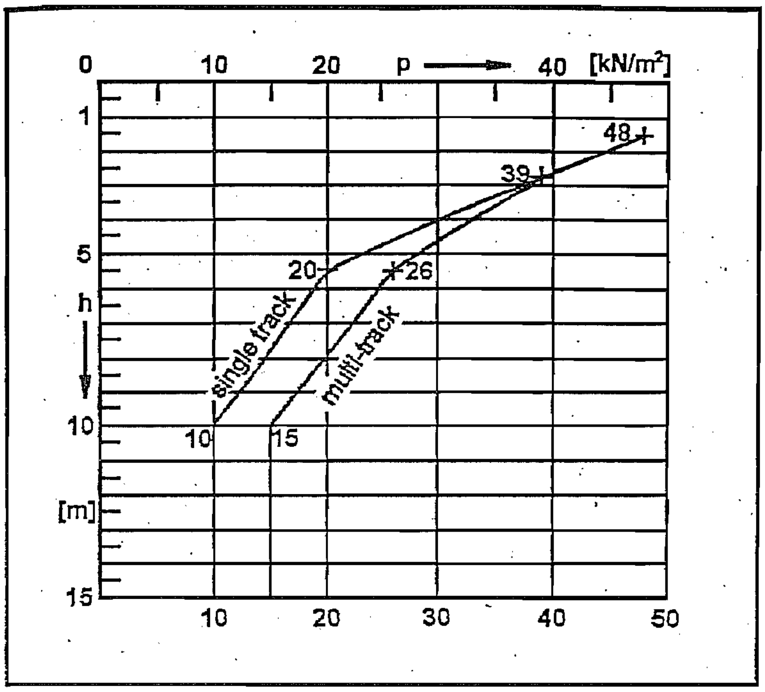
Standard vehicle	φ
HGV 60	1.2
HGV 30	1.4
CV 12	1.5

5.2.2.2 Rail Traffic Loads

The load-distributive effect on rails and sleepers is taken into account with the determination of the vertical stresses in the soil as a result of rail traffic loads. Calculation is in accordance with DS 804 with a vertical soil stress p in the level of the pipe crown in dependence on the cover height h up to the upper edge of the sleepers (see Table 7 and Diag. D3).

Table 7: Soil stresses p as a result of rail traffic loads

h	p in kN/m²	
m	1 track	2 and more tracks
1.50	48	48
2.75	39	39
5.50	20	26
≥ 10.00	10	15
Interpolation may be carried out linearly between the values given		



Diag. D3: Soil stresses as a result of rail traffic loads

The minimum cover is the greater of the two values

$$h = 1.50\ m$$

or

$$h = d_1$$
$$p_V = \varphi \cdot P$$

For pipes under tracks the impact coefficient is

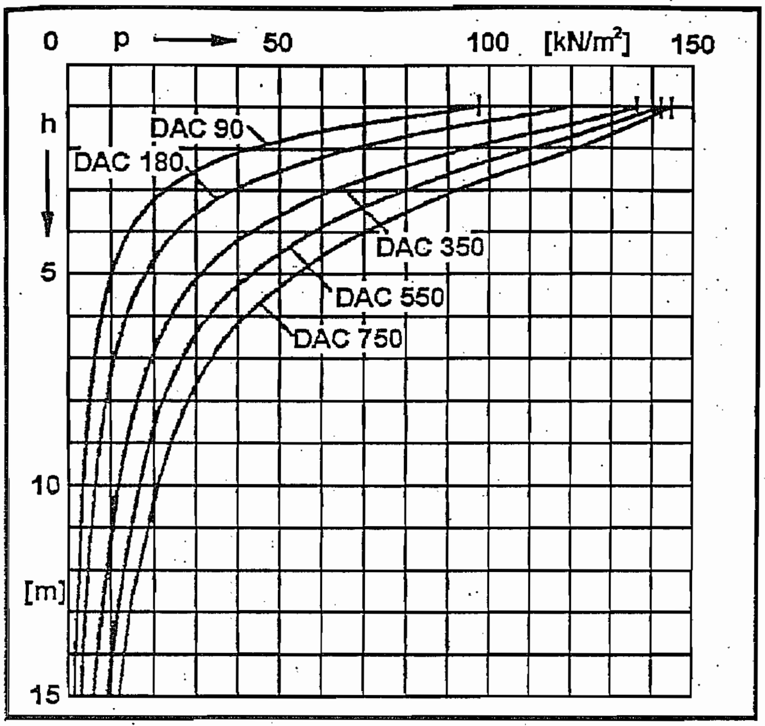
$$\varphi = 1.40 - 0.10(h - 0.60) \geq 1.0 \tag{5.12}$$

whereby h is to be applied in m.

5.2.2.3 Aircraft Traffic Loads

The soil stresses p_V as a result of dimensioning aircraft can be taken from Diag. D4 for h ≥ 1 m.

For aircraft loads the maximum impact factor of the relevant main undercarriage $\varphi = 1.5$. With the formulation of the soil stress p_V in accordance with Diag. D4, the impact coefficient and the load-distributing effect of the aircraft operating surfaces are already taken into account.



Diag. D4: Soil stress p_V as a result of aircraft traffic loads

5.2.2.4 Limited Area Loads

The influence of limited area loads may be approximated as follows:

Area within the pressure propagation 2:1

> Calculation of the stresses in the soil as isohedric load within the area limited by the pressure propagation 2:1.

Area outside of partly outside the pressure propagation 2:1, but within pressure propagation 1:1

> Calculation of the stresses in the soil as isohedric load within the area limited by the pressure propagation 1:1.

Area outside pressure propagation 1:1

> No influence from the concentrated area loads.

5.2.2.5 Loading Due to Construction Site Traffic

Possible higher loads in individual cases are to be taken into account.

5.3 Internal Pressure

Stresses from internal pressure are superposed linearly on those from external loads. With higher operational pressures (above the level of backwater) dimensioning can be carried out using non-linear superposition[39].

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39) Netzer, W.; Pattis, O.: Overlapping of internal and external pressure loading of buried pipelines (mathematical investigation with the application of second order theory). 3R international (1989) p. 96 - 105.

The introduction of internal pressure not only leads to additional stresses and elongatons in the annular direction but can also change the deformation of flexible and semi-rigid pipes. In addition, with pressure pipelines with changes of direction or other discontinuities, the tensile loads in the axial direction or the shear forces are to be calculated [40].

With different values for stability under tensile load and under bending tensile load, these are, if necessary, to be taken into account in a suitable manner, with verification of safety, in a common, weighted coefficient of safety.

6 Load Distribution

6.1 Redistribution of Soil Stresses

As a result of the different deformation capability of the pipe and of the surrounding soil the average soil stresses calculated in accordance with Sect. 5.2.1 rearrange themselves.

The scale of this redistribution is given by the concentration factors λ_P for the stresses the pipe and above λ_S for the stress in the soil alongside the pipe. The idealised form of the redistribution, with $b/d_e = \infty$, can be seen in Fig. 7.

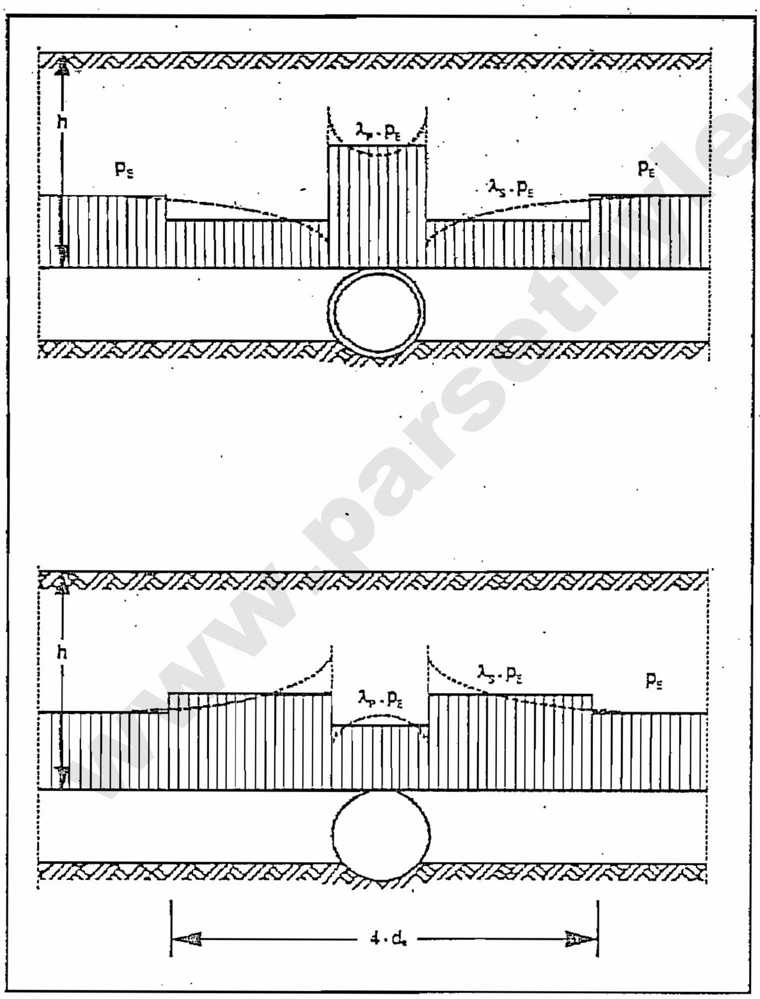


Fig.7: Redistribution of soil stresses (above: rigid pipe, below: flexible pipe)

[40] See DIN EN 1295-1, September 1997, Chap. 6.1, Para. 2.

6.2 Relevant Parameters

6.2.1 Embedding Conditions for the Pipeline

For the embedding in the pipeline zone four embedding conditions B1 to B4 are differentiated:

B1: Compacted embedding against the natural soil by layers or in the embanked covering (without verification of the degree of compaction); applies also for beam pile walls (Berlin shuttering).

B2: Vertical shuttering in the pipeline zone using trench sheeting, which reaches the bottom of the trench and which is first removed after backfilling.
Shuttering plates or equipment under the assumption that the compaction of the soil takes place after withdrawal of the trench sheeting.

B3: Vertical shuttering within the pipeline zone using sheet piling or lightweight piling profiles and compaction against the trench sheet [41)], which reaches down below the trench bottom.

B4: Compacted embedment against the natural soil by layers or in the embanked covering, with verification of the degree of compaction required according to ZTVE-StB (see Sect. 4.2). Embedding condition A4 is not applicable with soil of Group 4.

The representative values E_S and K_2 are to be taken from the following sections.

6.2.2 Deformation Modulus E_S

The deformation modules of the soil employed with the calculation are differentiated according to the following zones (see Fig. 8):

[41)] Vertical trench sheeting using wooden planks, revetting plates or devices, which are first removed following backfilling and compaction of the pipeline zone, cannot be calculated with certainty using a computer model. For the mathematical estimation of the increase of the load as a result of ramming, attention is drawn to the ATV Working Group 1.5.5 "Methods of revetting" ATV Report "Approaches for the calculation of pipe loading in trenches with sheet revetting" in *Korrespondenz Abwasser* 12/97 [Not translated into English]

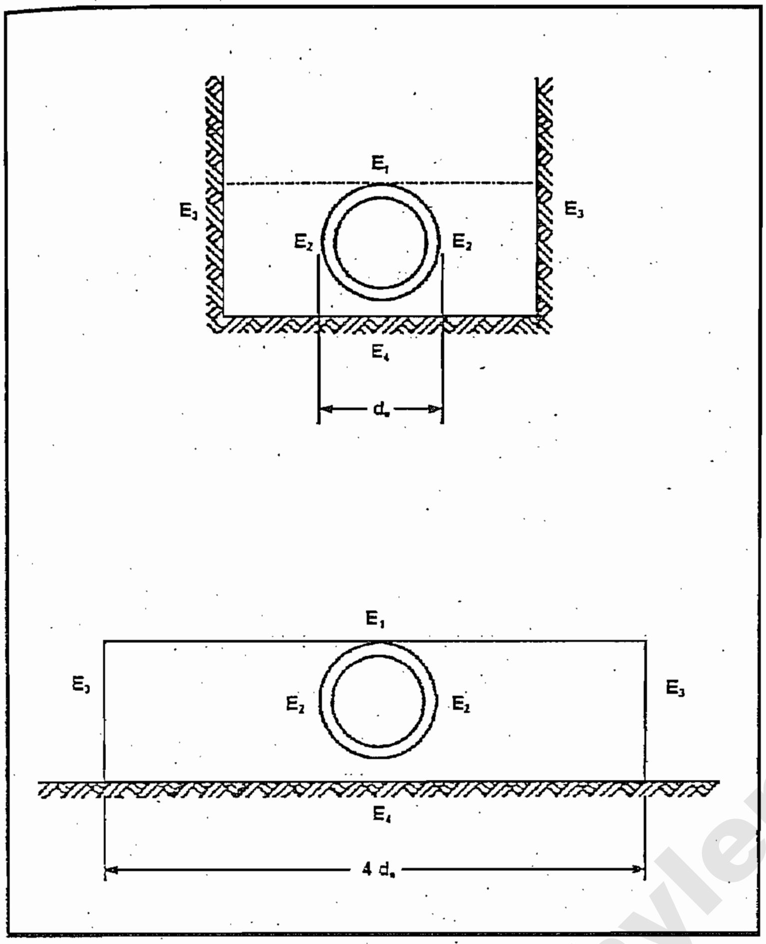


Fig. 8: Designation of the deformation modules for the various soil zones.

Covering above the pipe crown	E_1
Pipeline zone to the side of the pipe	E_2
Existing soil alongside the trench or emplaced soil alongside the pipeline zone	E_3
Soil below the pipe	E_4

The degrees of compaction corresponding with the covering conditions in accordance with Sect 5.2.1.2 and with the embedding conditions in accordance with Sect. 6.2.1 as well as the deformation modules E_1 and E_{20} (corresponding to E_8 from Table 1) allocated as standard values are to be taken from Table 8. With this covering conditions A1 to A4 appear randomly combined with embedding conditions B1 to B4.

As a rule, natural soils have a degree of compaction of D_{Pr} = 90 to 97 %; the associated values E_3 can be taken from Table 3. With placement of the excavated soil in the pipeline zone in accordance with DIN EN 1610, $E_3 = E_{20}$ is to be applied. If the excavated soil is to be placed as cover only, $E_3 = E_1$ is to be applied so far as, in the individual case, a higher degree of compaction for E_3 is verified.

Table 8: Representative values of the deformation modules E_1 and E_{20} independent of the initial compaction

Covering condition		A1		A2 and A3 **)		A4	
Embedding condition		B1		B2 and B3		B4	
Degree of compaction D_{Pr} in % Deformation module E_1 and E_{20} in N/mm²		D_{Pr} *)	E_1, E_{20}	D_{PR} *)	E_1, E_{20}	D_{PR} *)	E_1, E_{20}
	G1	95	16	90	6	97	23
Soil	G2	95	8	90	3	97	11
Group	G3	92	3	90	2	95	5
	G4	92	2	90	1.5	-	-

With equal compaction of the soil alongside and above the pipe, $E_{20} = E_1$ can be achieved. E_{20} may not be assumed to be greater than E_1, with the exception of exchange of soil in the pipeline zone or embedding condition B4.

Lesser compaction alongside the pipe in narrow trenches is taken into account in Diag. D5 (Note minimum trench width in accordance with DIN EN 1610!).

Subsidence as a result of the influence of groundwater in the pipeline zone is taken into account by a reduction of the E_{20} value using the factor f_2 in accordance with Eqn. (6.01):

$$f_2 = \frac{D_{Pr} - 75}{20} \leq 1 \qquad (6.01)$$

*) D_{Pr} is to be applied for the calculation in line with the table value for the respective embedding condition.

**) Compaction and deformation modules according to A2 and A3 may be used only if the initial compaction A1 is maintained

As a rule, with laying in the embankment, E1 = E20 = E3 can be set.

The notes with regard to E_2 and E_3 in Sect. 8.4 are to be additionally observed for the calculation of deformation.

With soils (loose rock) $E_4 = 10 \cdot E_1$ is assumed, so far as, in individual cases, no more accurate details are available. With foundation on rock (bed rock), E_1 can be significantly greater.

In order to take into account these influences as well as the difficulties with compaction in narrow trenches in the earth load calculations, the following formulation applies:

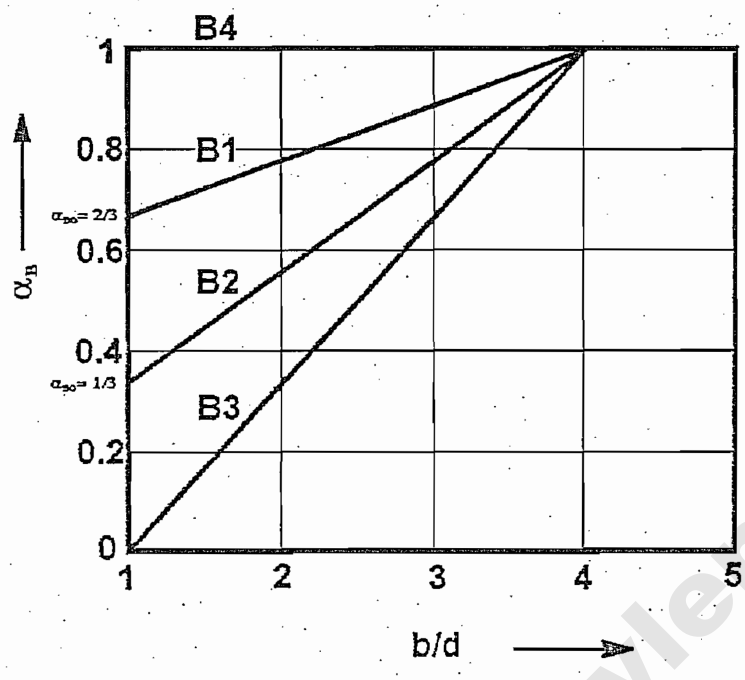
The effective deformation modulus E_2 is calculated using f_1 from Table 1, f_2 from Table 8 and α_B from Diag. D5 as

$$E_2 = f_1 \cdot f_2 \cdot \alpha_B \cdot E_{20} \qquad (E_{20} \text{ from Table 8}) \quad (6.02)$$

with

$$\alpha_B = 1 - \left(4 - \frac{b}{d_e}\right) \cdot \frac{1 - \alpha_{Bi}}{3} \leq 1 \qquad (6.03)$$

With trenches with additional embanked cover and a bottom width of $b_{So} < 3d_e$, E_2 may not be applied greater than E_3.



Diag. D5: Reduction factors α_B for E_2

6.2.3 Earth Pressure Ratio K_2

The earth pressure ratio in the soil alongside the pipe is determined a follows, taking into account the system stiffness V_{RB} (see Sect. 6.3.3:

For pipes with $V_{RB} > 1$, K_2 is calculated in accordance with Table 9, Column 2. The horizontal bedding pressure q_h* (see Sect. 7.3) is then to made equal to zero. Alternatively one can also carry out the calculation using values according to Column 3.

For pipes with $V_{RB} \leq 1$ is calculated using K_2 in accordance with Table 9, Column 3, whereby q_h* is estimated in accordance with Sect. 7.3.

K_2 values are not clear soil-mechanically defined characteristic values; they cover various types of influence with the aim of linearisation and are adjusted to measured values.

Table 9: Ground pressure ratio K_2

1	2	3
Soil Group	K_2	
	$V_{RB} > 1$	$V_{RB} \leq 1$
G1	0.5	0.4
G2	0.5	0.3
G3	0.5	0.2
G4	0.5	0.1
Bedding reaction pressure	$q^*_h = 0$	$q^*_h > 0$

6.2.4 Relative Projection a

The relative projection a can be seen from the examples in Fig. 9.

The projection $a \cdot d_e$ is the height of the soil layer alongside and, if necessary, laterally under the pipe which, compared with the vertical deformation, can experience deviating subsidence.

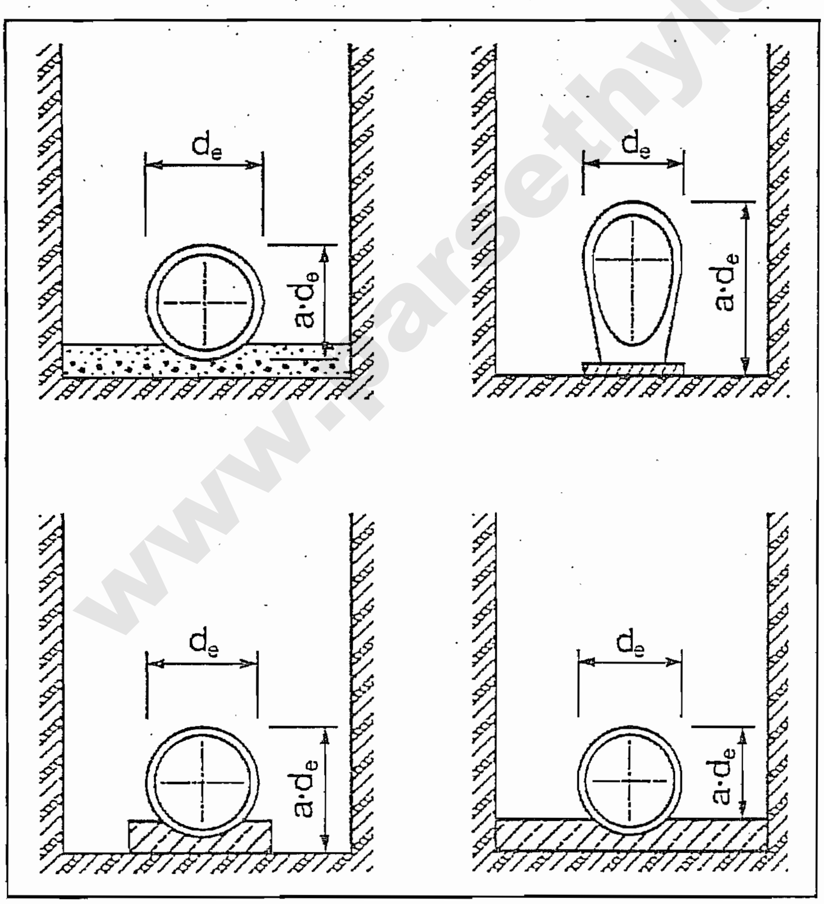


Fig. 9: Relative projection

6.3 Concentration Factors and Stiffness Ratio

The calculation of the concentration factor assumes infinitely rigid pipes on soft soils in wide cover. This gives a maximum concentration factor of max λ.

The deformation of the pipe is taken into account in the next step. From this results the concentration factor λ_R which is determined through the stiffness ratio V_S in accordance with Sect. 6.3.

6.3.1 Maximum Concentration Factor Max λ

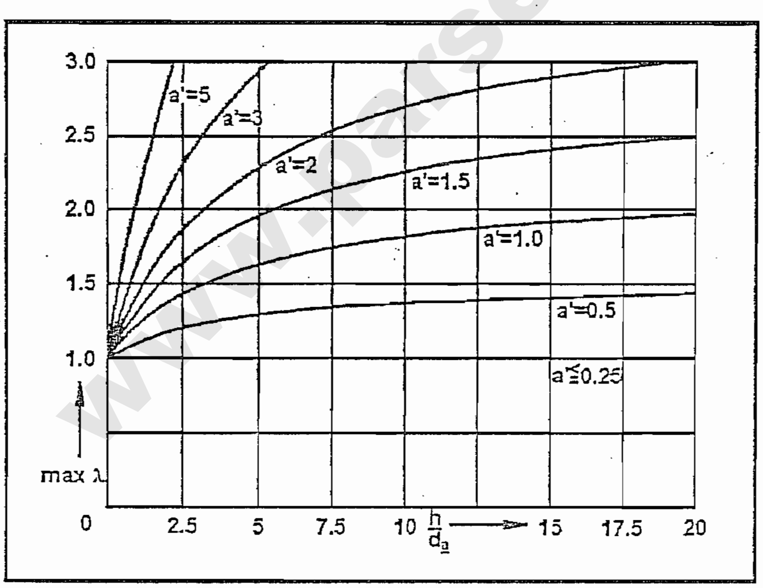
The maximum concentration factor max. λ is:

$$\max \lambda = 1 + \frac{\frac{h}{d_e}}{\frac{3.5}{a'} + \frac{2.2}{\frac{E_4}{E_1} \cdot (a' - 0.25)} + \left[\frac{0.62}{a'} + \frac{1.6}{\frac{E_4}{E_1} \cdot (a' - 0.25)}\right] \cdot \frac{h}{d_e}} \tag{6.04}$$

with the effective relative projection

$$a' = a \cdot \frac{E_1}{E_2} \geq 0.26 \tag{6.05}$$

The function max. λ is represented in Diag. D6 for various values a' and for $E_4 = 10 \cdot E_1$.



Diag. D6: Concentration max. λ for b/de = ∞ and $E_4 = 10 \cdot E_1$

6.3.2 Concentration Factors λ_P and λ_S

The concentration factor λ_P is dependent on the largest value λ_{max} (see Sect. 6.3.1), on the stiffness ratio V_S (see Sect. 6.3.3 as well as the effective relative outreach a' and on the ground pressure ratio K_2:

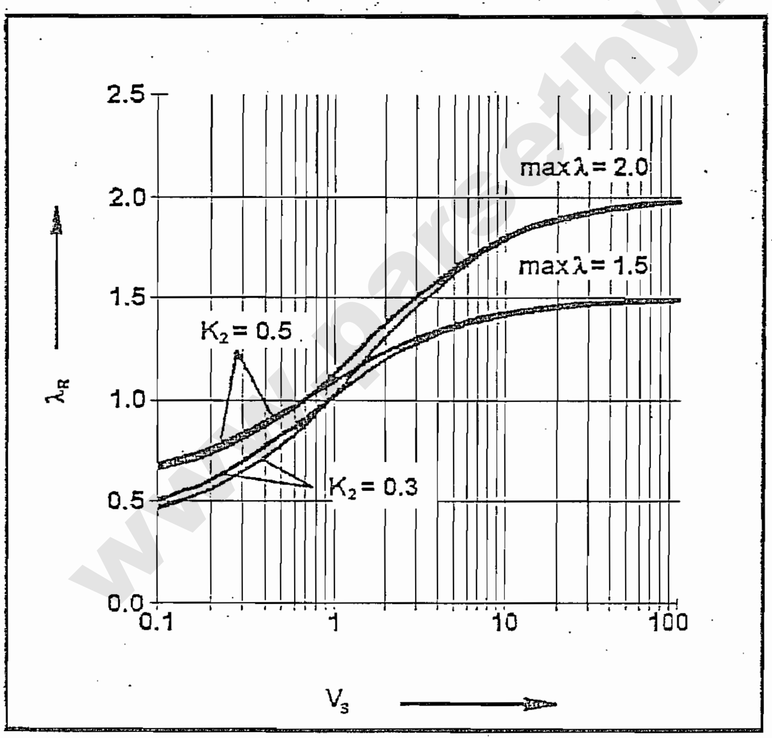
$$\lambda_P = \frac{\max\lambda \cdot V_S + a' \frac{4 \cdot K_2 \, K'}{3} \cdot \frac{\max\lambda - 1}{a' - 0.25}}{V_S + a' \frac{3 + K_2 \cdot K'}{3} \cdot a' - 0.25 \frac{\max\lambda - 1}{a' - 0.25}} \tag{6.06a}$$

For pipes of high stiffness ($V_{RP} > 1$) the calculation is continued using λ_P in accordance with Sect. 6.3.1.

The following applies for K'

$$K' = \frac{c_{v,qh} + \frac{c_{h,qh}}{c_{h,qv}} \cdot c_{v,qh^*} \cdot K^*}{c_{v,qv} + c_{v,qh^*} \cdot K^*} \tag{6.06b}$$

The deformation coefficient $c_{v,vq} = c_{v,qh}$ or $c_{h,qv} = -c_{h,qh}$ respectively (also with $2\alpha = 180°$ and with negligible N and Q deformation) $K' = 1$.



Diag. D7: Concentration factor λ_P for a' = 1, K_2 = 0.3 and 0.5

The concentration factor λ_S results from the imaginary form of the stress transposition (see Fig. 7) and from reasons of equilibrium as

$$\lambda_S = \frac{4 - \lambda_S}{3} \qquad (6.07)$$

As λ_S cannot be negative it results from Eqn. (6.07) that λ_S does not exceed the value 4.

The influence of the relative trench width on the stress transposition is dealt with in Sect. 6.4.

6.3.3 Stiffness Ratio

The stiffness ratio is dependent on the pipe stiffness S_P, on the coefficient of the vertical change of diameter c_v or $c_{v,qv}$, possibly from the stiffness of a deformation layer S_D as well as on the vertical bedding stiffness of the soil to the side of the pipe S_{Sv}.

Due to more idealised assumptions the calculation model applies only from a minimum pipe stiffness with long-term loading of min S_P 0 3 · 10^{-3} N/mm^2 and min $S_O = 3.75 \cdot 10^{-3}$ N/mm^2 respectively.

The stiffness ratio is calculated as follows:

a) taking into account the horizontal bedding reaction pressure

$$V_S = \frac{8 \cdot S_O}{|c_v{}^*| \cdot S_{Bv}} \qquad (6.08a)$$

b) without taking into account the horizontal bedding reaction pressure (see Sect. 6.2.3)

$$V_S = \frac{8 \cdot S_O}{|c_{v,qv}| \cdot S_{Bv}} \qquad (6.08b)$$

c) taking into account a deformation layer above rigid pipes for simplified verification

$$V_S = \frac{S_D}{S_{Bv}} \qquad (6.08c)$$

The relative effective outreach must be calculated taking into account the width and height of the deformation layer.

$$a' = \frac{a \cdot d_e + d_D}{b_D} \cdot \frac{E_1}{E_2} \geq 0.26 \qquad (6.09)$$

$K_2 = 0$ is to be applied in Eqn. (6.06a).

In addition, $b \geq 2b_D$ and $h \geq 2b_D$.

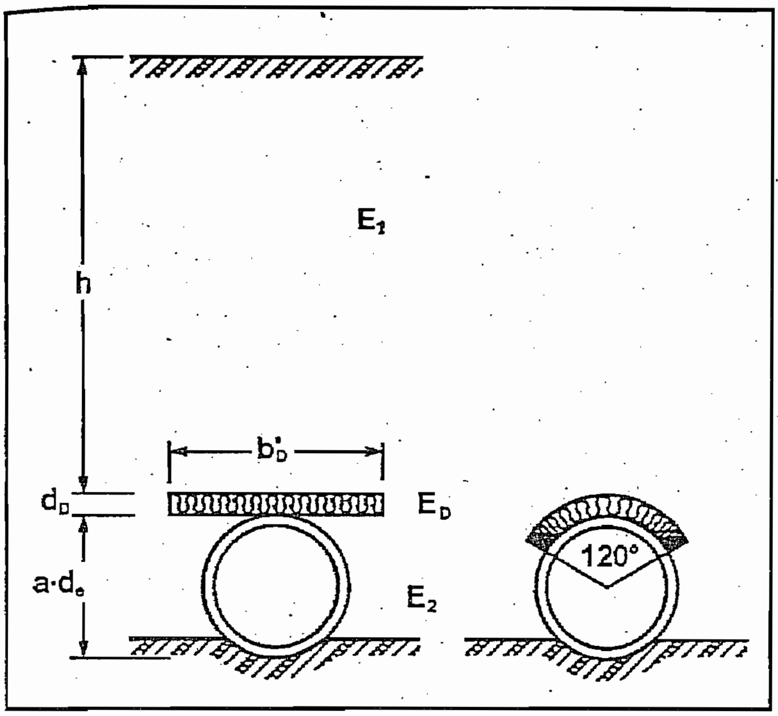


Fig. 10: Deformation layer

For Eqns. (6.08a) to (6.08c) the following additional relationships apply:

pipe stiffness

$$S_P = \frac{E_P \cdot I}{r_m^3} \tag{6.10a}$$

$$= \frac{E_P}{12} \cdot \left(\frac{s}{r_m}\right)^3 \text{ with smooth-walled pipes}$$

or

$$S_O = \frac{E_P \cdot I}{d_m^3} \tag{6.10b}$$

$$\text{i.e } S_P = 8 \cdot S_O$$

Pipe stiffness $\bar{S}_0$ with long-term verification for pipes with nominal E-modules and nominal sizes

$$\bar{S}_O = \frac{I}{d_m^3} \cdot \frac{p_E \cdot E_{PL} + p_V \cdot E_{PL}}{p_E + p_V} \tag{6.10c}$$

Pipe stiffness $\bar{S}_0$ with long-term verification for pipes with nominal stiffness

$$\bar{S}_0 = \frac{p_E \cdot S_{0L} + p_V \cdot S_{0K}}{p_E + p_V} \tag{6.10d}$$

Stiffness of the deformation layer

$$S_D = E_D \cdot \frac{b_D}{d_D} \tag{6.11}$$

With non-linear deformation behaviour of the filling material, the deformation module

$$E_D = \frac{\lambda_{PG} \cdot p_E}{\varepsilon} \tag{6.11a}$$

is to be determined using the measured compression set ε. The stress/compression set curve is to be determined on a sample $d_D/b_D \leq 1$,

with E_D - deformation module of the deformation layer42)
b_D - width of the deformation layer
d_D - thickness of the deformation layer
ε - measured compression set of the deformation layer

Vertical bedding stiffness

$$S_{Bv} = \frac{E_2}{a} \tag{6.12}$$

Coefficient for the vertical deformation Δd_v

$$c_v = c_{v,qv} + c_{v,qh^*} \cdot K^* \tag{6.13}$$

with:

$c_{v,qv}$ = deformation coefficient for Δd_v as a result of q_v
c_{v,qh^*} = deformation coefficient for Δd_h as a result of q_h
K^* = bedding reaction pressure

$$K^* = \frac{c_{h,qv}}{V_{RB} - c_{h,qh^*}} \tag{6.14}$$

with:

$c_{h,qv}$ = deformation coefficient for Δd_v as a result of q_v
c_{h,qh^*} = deformation coefficient for Δd_h as a result of q_h

System stiffness

$$V_{RB} = \frac{8 \cdot S_0}{S_{Bh}} \tag{6.15}$$

The degree of the utilisation of horizontal bedding reaction pressures is covered with the system stiffness V_{RB}.

Horizontal bedding stiffness

$$S_{Bh} = 0.6 \cdot \zeta \cdot E_2 \tag{6.16}$$

The factor 0.6 takes into account the propagation of stress in the soil under the horizontal bedding reaction pressure q_h^* (see Sect. 7.3).

The correction factor ζ for the horizontal bedding stiffness, assuming a parallel shaped distribution of the bedding reaction stresses (Diag. 8), is

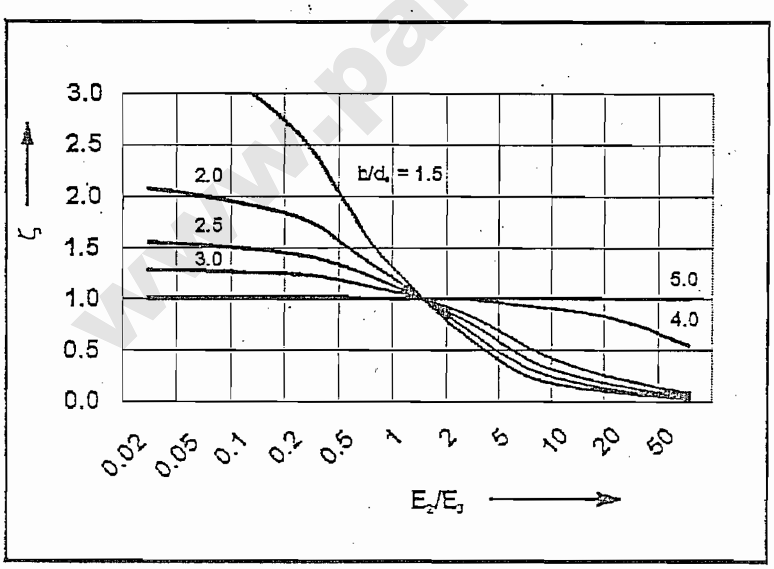
$$\zeta = \frac{1.667}{\Delta f + (1.667 - \Delta f)\frac{E_2}{E_3}} \tag{6.17}$$

with

$$\zeta = \frac{\frac{b}{d_e} - 1}{0.980 + 0.303\left(\frac{b}{d_e} - 1\right)} \leq 1.667 \tag{6.18}$$

It takes into account the different deformation moduli of the soil alongside the pipe (E_2) and the surrounding soil alongside the trench or alongside the pipeline zone (E_3).

With sloped trenches, the trench width at abutment height is to be applied in the place of the trench width b. For $E_2 = E_3$, $\zeta = 1$.



Diag. D8: Correction factor ζ

The deformation coefficients for Bedding Cases I and III described in Sect. 7, are given in Tables 10a to 10c.

Table 10a applies with negligible normal force deformations in the case

$$\frac{I}{A \cdot r_m^2} < 0.001 \tag{6.19a}$$

and with negligible lateral force deformation in the case

$$\frac{I}{A \cdot r_m^2} \cdot \kappa_Q < 0.001 \tag{6.19b}$$

with $\kappa_Q \cong A/A_Q$ (= 1.2 with rectangular cross-sections)
and $A_Q = \cong$ web area with profiled pipes.

If the condition (6.19a) or (6.19b) is not met, then the deformation coefficients c from Table 10a are to be corrected with the aid of c^N and c^S:

$$c' = c + \frac{I}{A \cdot r_m^2} \cdot \left[c^N + 2(1+\nu) \cdot \kappa_Q \cdot c^S\right] \tag{6.20}$$

In this c^N and c^S, the deformation coefficients according to Tables 10b and 10c to take into account the normal and lateral force deformations with the transversal contraction number $\upsilon = 0.3$.

Corrections according to Eqn. (6.20) are, as a rule, not necessary if smooth pipes with homogenous wall structure and material ($\upsilon \approx 0.3$) are calculated.

The deformation coefficients are to be applied in Eqns. (7.02) and (8.16a,b) with the correct sign.

Table 10a: Deformation coefficients for bending moments

Bedding angle	Vertical				Horizontal			
2α	$c_{v,qv}$	$c_{v,qh}$	$c_{v,w}$	c_{v,qh^*}	$c_{h,qv}$	$c_{h,qh}$	$c_{v,w}$	c_{h,qh^*}
60°	-0.1053		-0.0637		+0.1026		+0.0611	
90°	-0.0966	+0.0833	-0.0550	+0.0640	+0.0956	-0.0833	+0.0541	-0.0658
120°	-0.0893		-0.0477		+0.0891		+0.0476	
180°	-0.0833		-0.0417		+0.0833		+0.0418	

Table 10b: Deformation coefficients for normal forces [43)]

Bedding angle	Vertical			Horizontal		
2α	$c^N_{v,qv}$	$c^N_{v,qh}$	c^N_{v,qh^*}	$c^N_{h,qv}$	$c^N_{h,qh}$	c^N_{h,qh^*}
60°	-0,704	-0.681	-0.247	-0.380	-0.684	-0.437
90°	-0.697			-0.366		
120°	-0.683			-0.352		
180°	-0.648			-0.338		

Table 10c: Deformation coefficients for lateral forces [43)]

Bedding angle	Vertical			Horizontal		
2α	$c^L_{v,qv}$	$c^L_{v,qh}$	c^L_{v,qh^*}	$c^L_{h,qv}$	$c^L_{h,qh}$	c^L_{h,qh^*}
60°	-0,439	+0.335	+0.243	+0.396	-0.335	-0.274
90°	-0.389			+0.374		
120°	-0.359			+0.354		
180°	-0.335			+0.335		

6.4 Influence of the Relative Trench Width

According to Fig 7, the stress transposition spreads ranges over a width of 4de. The following assumption is made for the concentration factor λ_{PG} in trenches with smaller widths:

$$1 \le b/d_e \le 4: \lambda_{PG} = \frac{\lambda_P - 1}{3} \cdot \frac{b}{d_e} + \frac{4 - \lambda_P}{3} \qquad (6.21a)$$

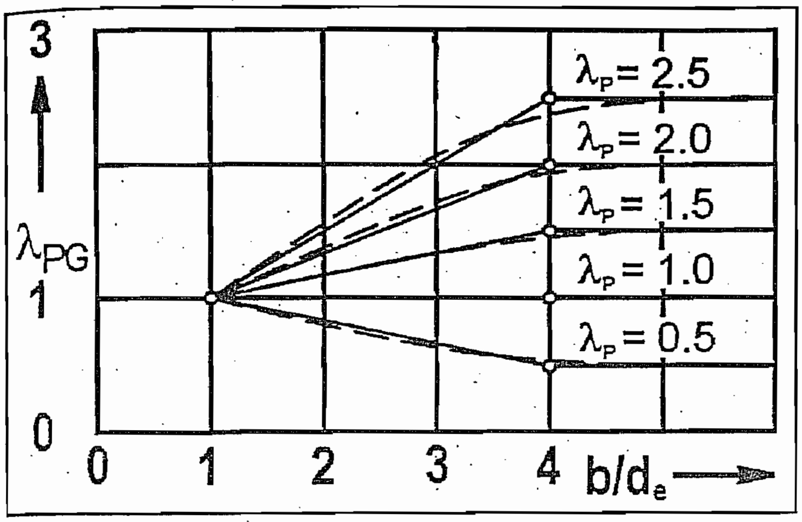
For larger trench widths the concentration factor remains unchanged:

$$4 \le b/d_e \le \infty: \qquad \lambda_{PG} = \lambda_P = \text{const.} \qquad (6.21b)$$

The function λ_{PG} is shown in Diag. 9 for various values λ_P. The concentration factor λ_B is, according to Eqn. (6.07), independent of trench width.

If, however, λ_{PG} is limited by λ_{fu} or λ_{fl} for reasons of equilibrium within the trench, there results

$$\lambda_B = \frac{\frac{b}{d_e} - \lambda_{fu,u}}{\frac{d}{d_e} - 1} \qquad (6.22)$$



Dig. 9: Concentration factor λ_{PG}

6.5 Limiting Value of the Concentration Factor

The concentration factor is limited, both upwards and downwards, by the shear resistance of the soil.

$$\lambda_{fl} \leq \lambda_{PG} \leq \lambda_{fu}$$

The upper limiting value for λ_{PG} is dependent on the parameters distortion pressure, friction, cohesion and structural resistance as well as vertical stress in the soil[44)]. These dependencies are approximated through a linear tension dependent on the covering height h:

$$h \leq 10 m \ \lambda_{fu} = 4.0 - 0.15 \cdot h \qquad (6.23a)$$

whereby h is to be applied in m.

$$h \leq 10 m \ \lambda_{fu} = 2.5 = const \qquad (6.23b)$$

The lower limiting value, which can be relevant for flexible pipes or pipes with deformation layer, are calculated according to the silo theory. With this, in Eqn. (5.04) κ is replaced by λ_{fl} and b by d_e or b_D. K_1 and δ are applied in accordance with Table 4, Covering Condition A4.

---

44) derivation for λ_{fu}:
$\lambda_{fu} = 1 + 2\eta \ K1 \tan \varphi'$
η is the height above the pipe crown related to d_e up to which the shear forces are assumed to be effective; the figure 2 can be applied as constant for η.
K1 = 1 with h = 0, reducing linearly to K1 = 0.5 with h ≥ 10 m
$\varphi' = 37.5°$

6.6 Vertical Total Load

The vertical total load of the pipe is

$$q_v = \lambda_{PG} \cdot p_E + p_V \tag{6.24}$$

The right-hand side of the equation contains the results from Eqns. (5.01), (5.02) and (5.11).

7 Pressure Distribution at the Pipe Circumference

The pressure distribution at the pipe circumference is dependent on the design of the support, on the backfill in the pipeline zone and on the deformation behaviour of the pipe. The simplified pressure distribution given in Sects. 7.2 and 7.3 are typical for the placement conditions normal in sewer construction.

7.1 Distribution of the Imposed Load

The imposed load, for all types of pipe, is assumed to be vertically aligned and rectangularly distributed.

7.2 Bearing Pressures (Bedding Cases)

7.2.1 Bedding Case I

Support in the soil. Vertically aligned and rectangularly distributed reactions. This bedding case applies for the stress verification and elongation verification (short-term and long-term) of rigid and flexible pipes (see Sect. 9.1). For the deformation verification the same bedding angle as for the stress and elongation verification is relevant.

With flexible pipes the bedding angle, as a rule, is assumed as $2\alpha = 120°$ with bedding conditions B2 or B3; with B1 and B4 $2\alpha = 180°$ may be applied.

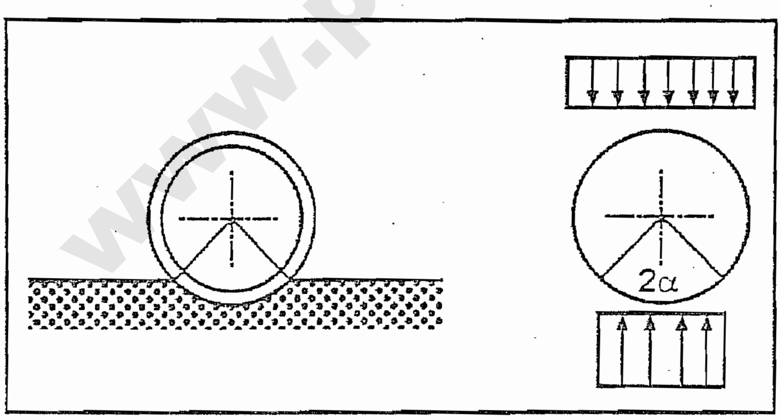


Fig. 11: Bedding case I

7.2.2 *Bedding Case II*

Solid concrete support only for rigid pipes (see Sect. 9.1). Radially aligned and rectangularly distributed reactions.

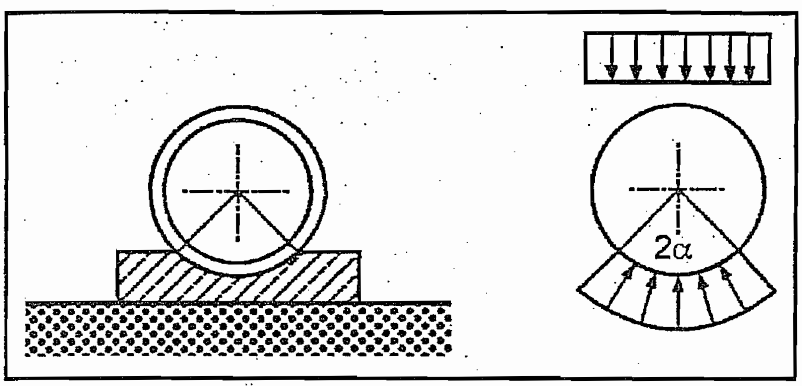


Fig. 12: Bedding Case II

7.2.3 *Bedding Case III*

Support and bedding in soil for flexible pipes. Vertically aligned and rectangularly distributed reactions.

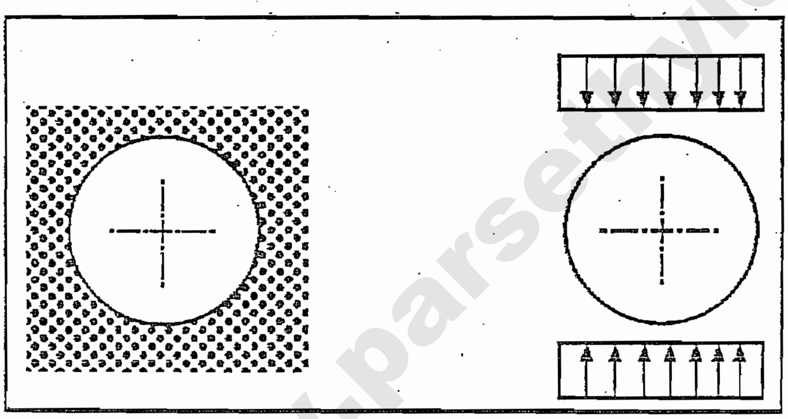


Fig. 13: Bedding case III

7.3 Lateral Pressure

The lateral pressure on the pipeline is made up from the component q' as a result of vertical earth load and, if necessary, from the bedding reaction pressure q_h^* as a result of pipe deformation (see Fig. 14).

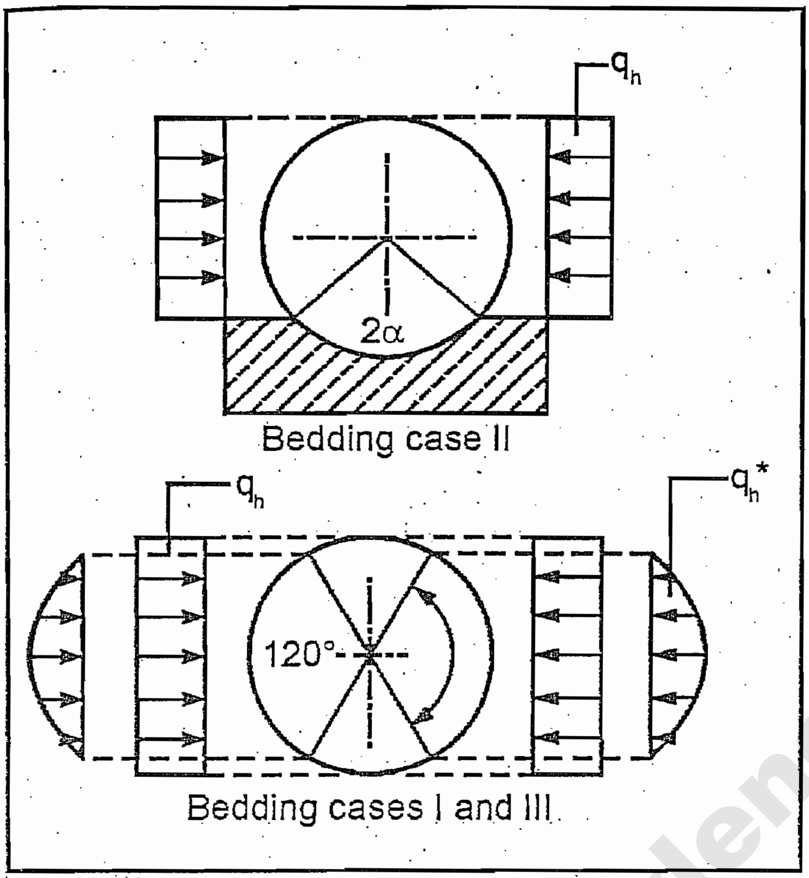


Fig. 14: Lateral pressure

The lateral pressure q_h - with Bedding Case II only above the support - is dependent on the vertical pressure in the soil alongside the pipeline:

$$q_h = K_2 \cdot \left(\lambda_S \cdot p_E + \chi_S \cdot \frac{d_e}{2} \right) \tag{7.01}$$

The bedding reaction pressure q_h* resulting from pipe deformation is applied as a parabola with an aperture angle of 120°:

$$q_h^* = \frac{c_{h,qv} \cdot q_v + c_{h,qh} \cdot q_h}{V_{RB} - c_{h,qh^*}} \tag{7.02a}$$

with q_v from Eqn. (7.01), V_{RB} from Eqn. (6.15) and $c_{h,qv}$, $c_{h,qh}$ and c_{h,qh^*} from Table 10a and, if required, Eqn. (6.20).

For the loading case with water filling the following applies:

$$q_{hw}^* = \frac{c_{h,w} \cdot q_w}{V_{RB} - c_{h,qh^*}} \tag{7.02b}$$

with $c_{h,w}$ from Table 10a as well as

$$q_w = \frac{F_w}{d_m} \tag{7.03}$$

and

$$F_w = 1.0 r_i^2 \pi \chi_w \tag{7.04}$$

8 Sectional Forces, Stresses, Elongations, Deformations

Below only sectional forces, stresses and deformation in the annular direction are investigated, i.e. the pressure distribution in the longitudinal direction (loading and reaction) is assumed to be constant.

8.1 Sectional Forces

Corresponding with the pressure distributions at the pipe circumference (see Sect. 7), the bending moments M and normal forces N are determined for external loads as well as for dead weight and water filling and, if necessary, water pressure. The shear forces in the annular direction can be ignored, however, not with profiled pipes (see Sect. 9.6). The coefficients of moment m and the coefficients of normal force n are given for various bedding cases, in Appx. 1, Table T3. The coefficients apply only for circular shapes with constant wall thickness over the circumference.

With non-constant wall thickness over the circumference or with non-constant moments of inertia as well as with other than circular shapes, e.g. oval profile, mouth-shaped profile, rectangular cross-section, other sectional force coefficients apply. They are to be determined according to static rules.

Using the coefficients m and n, the sectional forces are determined according to the following equations.

Vertical total loading q_v:

$$M_{qv} = m_{qv} \cdot q_v \cdot r_m^2 \tag{8.01}$$

$$N_{qv} = n_{qv} \cdot q_v \cdot r_m \tag{8.02}$$

Side pressure q_h:

$$M_{qh} = m_{qh} \cdot q_h \cdot r_m^2 \tag{8.03}$$

$$N_{qh} = n_{qh} \cdot q_h \cdot r_m \tag{8.04}$$

Horizontal bedding reaction pressure q_h^* as a result of earth loads:

$$M_{qh^*} = m_{qh^*} \cdot q_h^* \cdot r_m^2 \tag{8.05a}$$

$$N_{qh^*} = n_{qh^*} \cdot r_m \tag{8.06a}$$

Horizontal bedding reaction pressure q_h^* as a result of water filling:

$$M_{qw}^* = m_{qh}^* \cdot q_{hw}^* \cdot r_m^2 \tag{8.05b}$$

$$N_{qh} = n_{qh} \cdot q_h \cdot r_m \tag{8.06b}$$

Dead weight:

$$M_g = m_g \cdot \chi_R \cdot s \cdot r_m^2 \tag{8.07}$$

$$N_g = n_g \cdot \chi_R \cdot s \cdot r_m \tag{8.08}$$

or

$$M_g = m_g' \cdot F_g \cdot r_m \tag{8.07a}$$

$$N_g = n_g' \cdot F_g \tag{8.08a}$$

with $F_g = 2 \cdot r_m \cdot \pi \cdot s \cdot \chi_R$

Water filling:

$$M_w = m_w \cdot \chi_w \, r_m^3 \tag{8.09}$$

$$N_w = n_w \cdot \chi_w \cdot r_m^2 \tag{8.10}$$

or

$$M_w = m_w' \cdot F_w \cdot r_m \tag{8.09a}$$

$$N_w = n_w' \cdot F_w \tag{8.10a}$$

with F' according to Eqn. (7.04)

Water pressure:

$$M_{pw} = (p_i - p_e) r_i \cdot r_e \left(\frac{1}{2} - \frac{r_i \cdot r_e}{r_e^2 - r_i^2} \cdot \ln \frac{r_e}{r_i} \right) \tag{8.11}$$

$$N_{pw} = p_i \cdot r_i - p_e \cdot r_e \tag{8.12}$$

8.2 Stresses

Using the sectional forces determined in Sect. 8.1 the stresses are calculated as

$$\sigma = \frac{N}{A} \pm \frac{M}{W} \alpha_k \tag{8.13}$$

using the correction factor α_k to take into account the curvature of the inner and outer edge fibre

$$\alpha_{ki} = 1 + \frac{1}{3}\frac{s}{r_m} = \frac{3 \cdot d_i + 5 \cdot s}{3 \cdot d_i + 3 \cdot s} \tag{8.14a}$$

$$\alpha_{ke} = 1 + \frac{1}{3}\frac{s}{r_m} = \frac{3 \cdot d_i + s}{3 \cdot d_i + 3 \cdot s} \tag{8.14b}$$

8.3 Elongations

Using the sectional forces determined in Sect. 8.1 the edge fibre elongations are calculated as

$$\varepsilon = \frac{\sigma}{E} = \frac{s}{2r_m^{\,3} \cdot 8s_0} \cdot \left(\frac{s \cdot N}{6} + M \cdot \alpha_k\right) \tag{8.15}$$

8.4 Deformations

Depending on the pressure distribution at the circumference of the pipe, the change of vertical diameter Δdv as a result of external loads can be calculated in accordance with Eqn. (8.16a), whereby the slight influence of the water filling on the deformation can be neglected:

$$\Delta d_v = \frac{2 \cdot r_m}{8_0} \cdot \left(c_{v,qv} \cdot q_v + c_{v,qh} \cdot q_h + c_{v,qh^*} \cdot q_h^{\,*}\right) \tag{8.16a}$$

with q_v from Eqn. (6.24), q_h from Eqn. (7.01), q_h^* from Eqn. (7.02a), S_0 from Eqn. (6.10b) and $c_{v,qv}$, $c_{v,qh}$ and c_{v,qh^*} from Table 10a and, if necessary, from Eqn. (6.20).

For measurements of pipe deformation the horizontal change of diameter is to be determined if necessary:

$$\Delta d_h = \frac{2 \cdot r_m}{8_0} \cdot \left(c_{h,qv} \cdot q_v + c_{h,qh} \cdot q_h + c_{h,qh^*} \cdot q_h^{\,*}\right) \tag{8.16b}$$

The relative vertical deformation derives from Eqn. (8.16a).

$$\delta_v = \frac{\Delta d_v}{2 \cdot r_m} \cdot 100 \quad \text{in \%} \tag{8.17}$$

9 Dimensioning

9.1 Relevant Verifications

Pipes are designated as rigid or flexible depending on the combined effect of pipe stiffness and soil deformation.

Pipes are rigid where the loading causes no significant deformation and thus has no effects on the pressure distribution($V_{RB} > 1$). The verification of stress/elongation or the simplified verification of carrying capacity is to be carried out in accordance with (9.02) for its dimensioning.

Pipes are flexible whose deformation significantly influence the loading and pressure distribution as the soil is a component part of the carrying system($V_{RB} \leq 1$). The verification of stress/elongation, verification of deformation and the verification of stability are to be carried out for its dimensioning.

Pipes whose behaviour is not clearly rigid or flexible are to be verified as rigid and flexible.

9.2 Verification of Stress/Elongation

The stresses determined in accordance with Sect. 8.2 or the edge fibre elongation determined in accordance with Sect. 8.3 for service conditions are to be compared with the characteristic values σ_R and ε_R from Table 3. The existing safety values result from the relationship of both stresses and elongations:

$$\chi = \frac{\sigma_P}{\sigma} = \frac{\varepsilon_P}{\varepsilon} \qquad (9.01a)$$

and

$$\chi = \frac{\overline{\sigma_P}}{\sigma} = \frac{\overline{\varepsilon_P}}{\varepsilon} \qquad (9.01b)$$

With long-term verification for pipes with nominal moduli of elasticity and nominal size or nominal stiffness the following calculated values are to be employed:

$$\bar{\sigma}_R = \frac{p_E \cdot \sigma_{PL} + p_V \cdot \sigma_{PK}}{p_E + p_V} \qquad (9.01c)$$

and

$$\bar{\varepsilon}_R = \frac{p_E \cdot \varepsilon_{PL} + p_V \cdot \varepsilon_{PK}}{p_E + p_V} \qquad (9.01d)$$

For the dimensioning of reinforced concrete pipes, DIN 4025 (future DIN EN 1916 together with Din 1201) applies; for the dimensioning of reinforced concrete and prestressed concrete pipes, DIN EN 639, DIN EN 640 and DIN EN 642.

With predominantly tensile stresses (high internal pressure) the tensile strength is to be applied for σ_P and the ultimate elongation for ring tension applied for ε_P.

9.3 Verification of Carrying Capacity

For pipes with defined crown compressive force F_N [45]), the existing safety coefficient χ is determined, taking into account the installation figure EZ, in accordance with Eqn. (9.02) as,

$$\chi = \frac{F_N}{\text{tot}\,F} \cdot EZ \qquad (9.02)$$

If one calculates simplified without dead weight of the pipe, without water filling and without side pressure, installation figures in accordance with Table 11 apply as well as

$$\text{tot}\,F = q_v \cdot d_e \qquad (9.03)$$

For concrete pipes with a foot in accordance with DIN 4032 Form KFW with the wall thickness at the crown s_2 and the wall thickness at the invert s_3, the following applies:

$$EZ = 1.07 \cdot \left(\frac{s_3}{s_2}\right)^2 \qquad (9.04)$$

For pipes with oval cross-section in accordance with DIN 4032 Form EF, EZ can be applied = 2.1 = const.

Table 11: Installation figures [46])

Bedding case	Bedding angle 2α	Installation figure (Bedding figure) EZ
I	60° 90° 120°	1.59 1.91 2.18
II	90° 120° 180°	2.17 2.50 3.68

9.4 Verification of Deformation

For flexible pipes the vertical diameter change in accordance with Sect. 8.4 is to be compared with the permitted value perm. δ_v.

For long-term verification perm. δ_v = 6 %.

45) DIN EN 295, DIN EN 588, DIN 4032 (future DIN EN 1916 together with DIN 1201)

46) The installation figures are calculated from the ratio of the bending moments in the load crushing test and the bending moments in the emplaced condition, caused solely through vertical loading with the associated bedding reactions

For deformations, which in justified individual cases exceed the limit of 6 %, verification is to be carried out that the existing $\chi \geq 5$ x nec. (necessary) χ with the existing χ according to Eqn. (9.12) and nec. χ according to Table 13. With the existing $\chi < 5$ x nec. χ the deformation δ_v (maximum 9 %) has to be calculated using a non-linear procedure e.g. approximation by multiplication of the deformation determined according to Eqn. (8.17) using the enlargement ratio factor

$$\alpha_{II} = \frac{1}{1 - \frac{1}{\chi}} \tag{9.05}$$

For pipes under railway lines perm. (permitted) δ_v = 2 % and perm. δ_v = 10 mm.

The associated short-term value which serves for verification immediately following installation is, in any case, determined without application of traffic and surface loads.

The planner can apply perm. δ_v < 6 % if traffic area or type of traffic make this necessary.

9.5 Verification of Stability

9.5.1 General

The verification of stability serves for the determination of the safety separation between critical load and actual existing loading. This takes place in the following taking into account the influence of vertical total load (earth and traffic load), external water pressure (groundwater) as well as of superimposition of vertical total load and external water pressure.

The verification of stability can additionally be carried out for verification with buckling and percussion loads in accordance with Sect. 9.5.3 ("classical verification") also as non-linear stability verification (non-linear determination of sectional sizes and subsequent verification of stress) in accordance with Sect. 9.5.4. In the case of relative vertical total deformations δ_v > 6 % an additional non-linear verification of stability must be carried out.

The imperfections to be applied with all verification of stability are to approximated as closely as possible to the relevant buckling shape.

With earth/traffic loads the relevant buckling shape for ca. $V_{RB} > 10^{-3}$ is double-wave and for lower values multiple-wave. However, calculations have shown only a slight dependence of the size of deformation on the buckling load so that here a global reduction suffices.

With external water pressure the relevant buckling shape with ca. V_{RB} > 0.03 is double-wave (the limiting case is the unbedded pipe with an oval buckling shape) and for V_{RB} > 0.03 single-wave (breakdown taking place, for example, at the invert in heart shape). The types of imperfection associated with this must be differentiated as a sole occurrence of the more unfavourable single-wave figure is less probable in practice. A combination of both imperfections can be selected.

9.5.2 *Imperfections*

With all verification of stability the effects of imperfections on the critical loads are to be taken into account: Imperfections are deformations as result of

- manufacturing and processing inaccuracies, effects of transport and storage as well as internal stresses (Type A);
- placement, i.e. elastic deformations, according to Sect. 9.4 (Type B).

So far as no detailed verification takes place in individual cases, the elastic deformations (Type B) in accordance with Eqn. (8.17) are to be increased globally by 1 % to cover deformations of Type A: these are then taken into account mathematically as double-wave imperfection (ovalisation).

If locally limited (local) imperfections occur then their type and size are to be determined or accurately estimated. This applies generally for

- laying in groundwater with $\chi \leq 5.0$ with χ in according to Eqn. (9.11);
- pronounced single loads;
- covering $h \leq d_e$ or $h \leq 0.80$ m with metalled roadways;
- welding joints in the longitudinal direction (also with spiral welding).

The designation "preliminary deformation" is used below in place of "imperfection".

The reduction factors resulting from the shape (Type A; Type B) and size of the preliminary deformation are to be taken from the following diagrams in Appendix 1:

- D11 for κ_{v2} with double-wave preliminary deformations and earth/traffic load;
- D12 for κ_{a2} with double-wave preliminary deformations and external pressure (water pressure or vacuum;
- D13 for κ_{a1} with local preliminary deformations and external pressure (water pressure) or vacuum.

9.5.3 *Verification of Stability with Buckling and Snap-Through Loads*

The following relationships apply under the condition of constant radial bedding. A tangential bedding is ignored.

9.5.3.1 *Vertical Total Load*

First, the buckling load crit q_v is to be determined according to the following relationship:

$$\text{crit } q_v = 2 \cdot \kappa_{v2} \cdot \sqrt{8S_0 \cdot S_{Bh}} \quad \text{for } V_{RB} \leq 0.1 \qquad (9.06a)$$

$$\text{crit } q_v = \kappa_{v2} \cdot \left(3 + \frac{1}{3 \cdot V_{RB}}\right) \cdot 8S_0 \quad \text{for } V_{RB} > 0.1 \qquad (9.06b)$$

κ_{v2} = reduction factor to take into account the elastic-plastic soil mass law and of preliminary deformations in accordance with Diag. D11.

The values for S_0 and S_{Bh} result from the Eqns. (6.10a) and (6.16). with plastic pipes S_0 in Eqn. (9.06) is to be replaced by $\overline{S}_0$ according to Eqns. (6.10c,d).

If, in deviation from the normal case, the soil types OU, OT or OH of Group G4 are present in the pipeline zone, the critical buckling load crit q_v is to be determined in accordance with Eqn. (9.08).

The safety coefficient against buckling is

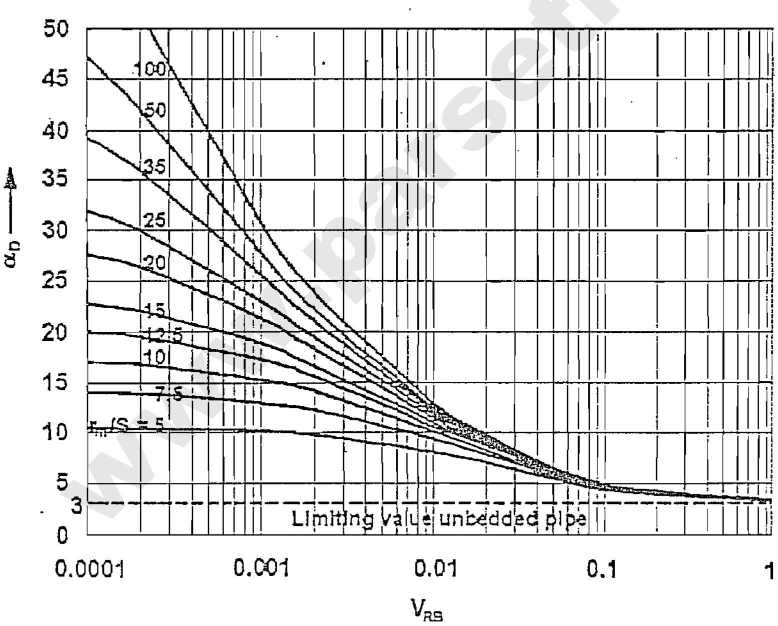
$$\chi = \frac{\text{crit } q_v}{q_v} \qquad (9.07)$$

with the vertical total load q_v according to Eqn. (6.24).

9.5.3.2 External Water Pressure

If, in comparison with the external water pressure, the earth pressure can be ignored, the following breakdown load applies:

$$\text{crit } p_e = \kappa_e \cdot \alpha_D \cdot 8S_0 \qquad (9.08)$$



Diag. D10: Snap-Through coefficient α_D for the critical external water pressure with the snap-through coefficient α_D as function of the system stiffness V_{RB}, the radius/wall thickness ratio r_m/s according to Diag. D10 and the reduction factor κ_{a2} for preliminary deformation according to Diag. D12 or κ_{a1} according to Diag. D13.

With plastic pipes S_0 is the long-term pipe stiffness in accordance with Eqn. (6.10b) with

$$E_P = E_{PL}$$

If there is a simultaneous taking into consideration of double wave and local preliminary deformation necessary in accordance with Sect. 9.5.2, then the common reduction factor is to be determined by multiplication:

$$\kappa_a = \kappa_{a1} \cdot \kappa_{a2} \tag{9.09}$$

The external water pressure relevant for the calculation is the hydrostatic pressure on the invert of the pipe, so far as a reduced formulation is not justified through permanently assured operating conditions (partial filling).

$$p_e = \chi_w \cdot h_w \tag{9.10}$$

In areas in which one has to reckon with increase of groundwater level, corresponding loads are to be taken into account.

The safety factor against buckling is

$$\chi = \frac{\text{crit } p_e}{p_e} \tag{9.11}$$

9.5.3.3 Simultaneously Effective Vertical Total Load and External Water Pressure

The superposition of both loading cases, using Eqns. (9.06a,b), (9.08) and (9.10) as well as taking into buoyancy with the vertical total load, leads to security against buckling

$$\chi = \frac{1}{\dfrac{q_{v,A}}{\text{crit } q_v} + \dfrac{p_e}{\text{crit } p_e}} \tag{9.12}$$

Verification of safety against buoyancy of the pipeline is to be carried out with pipes with small covering heights.

9.5.4 Non-linear Stability Verification

9.5.4.1 Calculation of a Rigid and Movable Bearings Model

In deviation from the verification in accordance with Sects. 9.5.1 and 9.5.2 the stability verification can also be carried out using a non-linear procedure [47].

With this

- the load formulations in accordance with Sect. 7 without q_h^* are to be employed,
- times χ loads are to be applied (χ according to Table 12),
- the elastic bedded circular ring with the exclusion of tension springs is to be employed as calculation model (partially bedded ring),
- the springiness value from S_{Bh} as discrete springs in close proximity ($\leq 10°$) or as constant radial bedding using the bedding modulus $k_{sr} = S_{Bh}/r_m$ are to be applied,
- pertinent imperfections of Type A are to be applied (approximated as geometric alternative deformations),
- justified assumptions are to be made on possible non-linear material behaviour of the pipe material.

With snap-through, such as in the external water pressure load case, the limited validity of theories of small deformations, that is also the Second Order Theory, is to be observed. Normal rigid and moving bearings programmes in accordance with the Second Order Theory may therefore not be employed without estimation of the limits of their validity.

9.5.4.2 Method of Approximation Using Enlargement Factors α_{II}

In place of the non-linear stability verification in accordance with Sect. 9.5.4.1 as an approximation one can proceed as follows:

1. The sectional parameters from earth and traffic loads N_{qv} and M_{qv} are determined linearly, comp. Sect. 8.1.

2. The normal force from external water pressure N_{pe} is calculated in accordance with Eqn. (8.12) and the bending moment M_{pe} in accordance with Eqn. (9.13) with the aid of the coefficient m_{pe} from Diag. D14 in Appendix 1:

   $$M_{pe} = m_{pe} \cdot p_e \cdot r_m^2 \qquad (9.13)$$

   The bending moments (not the normal forces) are to be increased by the factors $\alpha_{II,qv}$ and $\alpha_{II,pew}$ for the non-linear influences in accordance with Eqns. 89.14a,b):

$$\alpha_{II,pe} = \frac{1}{1 - \dfrac{\chi_{qv} \cdot p_e}{\text{crit } q_v}} \qquad (9.14a)$$

[47] When taking into account the limiting conditions in the soil, if required also using the Finite Model Method

In Eqn. (9.14a), for the safety coefficients with earth and traffic loads, χ_{qv} = 2.0 applies for $h/d_e < 3$ and $\chi_{qv} = 1.5$ for $h/d_e \geq 3$

$$\alpha_{II,pe} = \frac{1}{1 - \frac{\chi_{pe} \cdot p_e}{\text{crit } p_e}} \quad \text{with } \chi_{pe} = 2 \tag{9.14b}$$

In the critical loads crit q_v and crit p_{ew}, the preliminary deformations of the pipe are to be taken into account using the reduction factors κ.

3. Finally, the maximum tensions and/or elongations in the invert of the pipe are to be calculated separately for q_v and p_{ew}.

$$\sigma_{So} = \frac{N_{So}}{A} \pm \alpha_c \cdot \frac{M_o}{W} \cdot \alpha_{II} \tag{9.15a}$$

$$\varepsilon_{So} = \frac{\sigma_{So}}{E_R} \tag{9.15b}$$

4. From this result the individual safety factors

$$\chi_{qv} = \frac{\sigma_P}{\sigma_{qv}} \text{ and } \chi_{qv} = \frac{\varepsilon_P}{\varepsilon_{qv}} \text{ respectively} \tag{9.16a}$$

$$\chi_{pe} = \frac{\sigma_P}{\sigma_{pew}} \text{ and } \chi_{pe} = \frac{\varepsilon_P}{\varepsilon_{pew}} \text{ respectively} \tag{9.16b}$$

With plastic pipes the verification (9.16a) is carried out using the short-term bending tensile strengths, and (9.16b) carried out using the long-term bending tensile strengths (or limiting elongations). With the simultaneous occurrence of q_v and p_{ew} the interaction equation (9.12) is to be applied.

9.6 Supplementary Notes for Profiled Pipes

The statements below reflect the fundamentals according to which the suitability of the selected profile dimensions is to be verified in individual appraisals: The appraisal is to be produced by an expert recognised by the ATV-DVWK or another member association of the EWA. Further statements are part of the ATV Advisory Leaflet ATV-M 127E, Part 3, upon which work has started [Translator's note: This Advisory Leaflet is now available in both German and English).

9.6.1 General

Profiled pipes have wall cross-sections in accordance with DIN 19 566-1, Draft 2.96, Fig. 1 or, after publication, in DIN EN 13476-1. with profiled pipes the verification required in Sects. 9.2, 9.4 and 9.5 are to be modified and additional verification is to be carried out:

- verification of a sufficient profile rigidity (buckling of thin-walled profile components, comp DIN 19 566-2, Draft 2.96 and mathematical verification).

- if required, verification of multi-axial stress conditions.

The application of the equivalent wall thickness

$$s' = \sqrt[3]{12 \cdot I}$$

with profiled pipes is permitted for the determination of the pipe stiffness and the bending deformations only, not, however, for stress and stability verification.

9.6.2 Additions for Stress/Deformation Verification

In Eqn. (8.13) there is to be differentiation in the denominator between the resistance moments of both outer fibres W_e and W_i. Eqns. (8.14a,b) apply only as approximations - if necessary α_{ke} and α_{ki} are to be derived separately. Tensile and compressive stresses are to verified separately.

With direct pressure loading of the pipe surface thin-walled components of the cross-section are to be verified for these strains. the determination of the sectional parameters takes place in accordance with the rules of the plate or, if necessary, shell theories.

The maximum transverse stresses of the cross-section are to be determined:

$$\tau = \frac{Q \cdot S}{I \cdot s_i} \qquad (9.17)$$

with

Q = maximum shear force in the pipe wall
S = area moment, 1st degree, about the axis through the c of g (static moment)
s_i = wall thickness at the point of the cross-section considered (e.g. the web).

The shear force can be calculated from the determined stress distribution around the pipe circumference using equilibrium considerations. The verification for the transverse stress determined from the shear force is to be carried out for $\tau_P = 0.5\sigma_P$ (short-term and long-term) using safety coefficient χ in accordance with Table 12.

At points on the pipe circumference with simultaneous large bending moments and shear forces (e.g. between haunch and invert), verification is to be carried out with associated values for N, M and Q.

If larger tension, compression and transverse stresses meet then, at this point, then the reduced stresses are to be verified using a failure criterion appropriate to the material (e.g. conical rupture criterion for HDPE).

9.6.3 Additions for Deformation Verification

As a rule, with the determination of the deformation of pipes with profiled walls, the criterion $I/(A \cdot r_m^2)$ in accordance with Eqns. (6.19a,b) is not met. The deformation coefficient in accordance with Tables 10a-c are also valid for profiled pipes ($\upsilon \approx 0.3$).

9.6.4 Additions for Stability Verification

To take account of the normal force deformation of profiled pipes, the parameter k^* in place of r_m/s with the determination of the snap-through coefficients α_D in accordance with Diag. D10.

$$k^* = \sqrt{\frac{A}{12 \cdot I}}\, r_m \tag{9.18}$$

The snap-through coefficients α_D in accordance with Diag. D10 apply only with $\kappa_Q \leq 5$.

In addition, a sufficient profile stiffness is to be verified. This verification against buckling of thin-walled profile components can be carried out experimentally or as mathematical verification in accordance with the classic buckling theory taking into account material creep.

9.7 Safety

9.7.1 Basis

The safety coefficients are determined on the basis of probabilistic reliability theory. With this the spread of the loading capacity of the pipes (e.g. stability, dimensions) and the loading (e.g. soil characteristics, traffic loads, placement conditions) are taken into account.

Due to the different spreads of stability, dimensions, stiffness and test methods as well as the different demands on the support function of the soil, there are different safety factors for the various pipe materials with the same probability of failure.

For the loading case of internal pressure varying sub-safety coefficients can be applied.

9.7.2 Safety Coefficient against Failure of Load Carrying

Under this fall the failure types fracture and instability. The necessary safety coefficients are given, dependent on the safety classes, in Tables 12 and 13. Probabilities of failure p_f are allocated to the safety classes.

The safety coefficients refer to the materials

- fibre cement cast iron (ZM), high density polyethylene (HDPE), polypropylene, polyvinyl chloride (PVC-U) and vitrified clay on the 5 % fractile of the ring bending tensile strength;
- reinforced concrete on the calculated values in accordance with DIN 1045;
- steel (ZM) on the 5 % fractile of the minimum yield point (or 0.2 % limit);
- unsaturated polyester, glass fibre reinforced (UP-GF) on the minimum ductility in accordance with DIN 19 565 - 1, Tables 23/24 together with prEN 1636.

Safety Class A - normal case

- hazarding of the groundwater
- prejudicing of usage
- failure has significant economic consequences.

Safety Class B: special case

- no hazarding of groundwater
- small prejudicing of usage
- failure has slight economic consequences.

If preliminary deformation is taken into account with stability verification then, for Safety Class B nec. χ = 2.0 can be applied and, for safety Class B nec. χ = 1.6.

Table 12: Safety coefficients, failure due to fracture

Pipe material	Nec. χ	
	Safety Class A (normal case) $p_f = 10^{-5}$	Safety Class b (special case) $p_f = 10^{-3}$
Fibre cement Concrete Vitrified clay	2.2	1.8
Reinforced concrete	1.75	1.4
High density polyethylene (HDPE) Polyvinyl chloride (PVC-U)	2.5	2.0
Polypropylene (PP-B, PP-H, PP-R)	2.5	2.0
Steel -(ZM) Cast iron -(ZM)	1.5	1.3
Unsaturated polyester glass-fibre reinforced (UP-GF)	2.0	1.75

Table 13: Safety coefficients, failure through instability

Pipe material	Nec. χ	
	Safety Class A (normal case) $p_f = 10^{-5}$	Safety Class b (special case) $p_f = 10^{-3}$
Steel -(ZM) Cast iron -(ZM) High density polyethylene (HDPE) Polypropylene (PP) Polyvinyl chloride (PVC-U) Unsaturated polyester glass-fibre reinforced (UP-GF)	2.5 (2.0 with the taking into account of pipe preliminary deformation)	2.0 (1.6 with the taking into account of pipe preliminary deformation)

9.7.3 Safety against Non-Permitted Large Deformations

With the permitted short- and long-term deformation according to Sect. 9.4 it is assumed that this limiting value applies as 90 % fractile.

9.7.4 Safety against Failure with Loading which is not Predominantly Permanent

The verification of safety with loading which is not predominantly static is necessary, if pipes are laid under the railway tracks and under aircraft movement areas. Verification can be necessary under roads; as a rule with a cover of more than 1.5 m this is not necessary.

Appropriate information on pipe driving under railway areas can be taken from ATV Standard ATV-A 125E[48)] and DS 804.

Vertical stresses in the soil as a result of road traffic loads may, without special verification, be calculated using a covering height increased by 0.30 m. Through this account is taken that, for frequent changes of load, a road surface with favourable load distribution is always present.

When safety against failure with loading which is not predominantly static has to be verified, only those pipes may be employed whose stress range is standardised or has been determined by an officially recognised test institute and is ensured through quality assurance.

For the verification of fatigue strength (stress range) the shear forces from traffic loading may be multiplied by a reduction factor αV in accordance with Table 14.

$$\text{dyn } p_V = \alpha_V \cdot \varphi \cdot p \qquad (9.19)$$

$$N_{qv} = n_{qv} \cdot \text{dyn } p_V \cdot r_m$$

$$M_{qv} = m_{qv} \cdot \text{dyn } p_V \cdot r_m^2 \qquad V \qquad (9.20)$$

With flexible pipes the following additionally applies:

$$N_{qh^*} = n_{qh^*} \cdot \text{dyn } p_{Vh^*} \cdot r_m$$

$$M_{qh^*} = m_{qh^*} \cdot \text{dyn } p_{vh^*} \cdot r_m^2 \qquad (9.21)$$

with

$$\text{dyn } p_{Vh^*} = \text{dyn } p_V \cdot K^* \qquad (9.22)$$

The supporting effect of the bedding reaction pressure dyn p_{Vh}^* may only be applied with Soil Groups G1 and G2 and Proctor densities $D_{Pr} \geq 97$ % ($S_{Sh} \geq 6$ N/mm²). The dynamic stress component follows with

48) ATV-A 125E, Issue 1996, p.

$$\text{dyn } \sigma_{pV} = \frac{N_{qv} + N_{qh}}{A} + \frac{M_{qv} + M_{qh}}{W} \cdot \alpha_c \qquad (9.23)$$

and thus the fatigue verification

$$\text{dyn } \sigma_{pV} \leq \frac{2\,\sigma_A}{\chi} \,^{49)} \qquad (9.24)$$

with $2\sigma_A$ = stress range of the pipe employed.

Table 14: Reduction factor α_v

Traffic load	Reduction factor α_v
HGV 60	0.5
HGV 30	0.8
UIC 71	1.0
DAC	0.6

A verification under loading which is not predominantly static is unnecessary for GV 12 as equivalent load for placement in open spaces.

49) With UIC required safety χ = 2, otherwise χ = 1

Appendix 1: Tables

T1,2: Reduction factor κ for a trench load and κ_o for a surface load according to the Silo theory

Table T1 a: Table of the earth load coefficient κ for trench loads (Silo theory)

δ = φ'				
φ'	20.00	25.00	30.00	35.00
δ	20.00	25.00	30.00	35.00
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.98	0.98	0.97	0.97
0.2	0.96	0.95	0.94	0.93
0.3	0.95	0.93	0.92	0.90
0.4	0.93	0.91	0.89	0.87
0.5	0.91	0.89	0.87	0.84
0.6	0.90	0.87	0.85	0.82
0.7	0.88	0.85	0.82	0.79
0.8	0.87	0.83	0.80	0.77
0.9	0.85	0.82	0.78	0.74
1.0	0.84	0.80	0.76	0.72
1.2	0.81	0.77	0.72	0.68
1.4	0.78	0.73	0.69	0.64
1.6	0.76	0.70	0.65	0.60
1.8	0.73	0.68	0.62	0.57
2.0	0.71	0.65	0.59	0.54
2.2	0.69	0.63	0.57	0.51
2.4	0.67	0.60	0.54	0.48
2.6	0.65	0.58	0.52	0.46
2.8	0.63	0.56	0.50	0.44
3.0	0.61	0.54	0.48	0.42
3.2	0.59	0.52	0.46	0.40
3.4	0.57	0.50	0.44	0.38
3.6	0.56	0.48	0.42	0.36
3.8	0.56	0.47	0.40	0.35
4.0	0.53	0.45	0.39	0.34
4.2	0.51	0.44	0.38	0.32
4.4	0.50	0.42	0.36	0.31
4.6	0.49	0.41	0.35	0.30
4.8	0.47	0.40	0.34	0.29
5.0	0.46	0.39	0.33	0.28
5.5	0.43	0.36	0.30	0.25
6.0	0.41	0.34	0.28	0.23
6.5	0.38	0.31	0.26	0.22
7.0	0.36	0.29	0.24	0.20
7.5	0.34	0.28	0.23	0.19
8.0	0.32	0.26	0.21	0.18
8.5	0.31	0.25	0.20	0.17
9.0	0.29	0.23	0.19	0.16
9.5	0.28	0.22	0.18	0.15
10.0	0.27	0.21	0.17	0.14

Table T1 b: Table of the earth load coefficient κ_0 for trench loads (Silo theory)

φ'	20.00	25.00	30.00	35.00
δ	13.33	16.67	20.00	23.33
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.99	0.99	0.98	0.98
0.2	0.98	0.97	0.96	0.96
0.3	0.97	0.96	0.95	0.94
0.4	0.95	0.94	0.93	0.92
0.5	0.94	0.93	0.91	0.90
0.6	0.93	0.92	0.90	0.88
0.7	0.92	0.90	0.88	0.86
0.8	0.91	0.89	0.87	0.85
0.9	0.90	0.88	0.85	0.83
1.0	0.89	0.86	0.84	0.81
1.2	0.87	0.84	0.81	0.78
1.4	0.85	0.82	0.78	0.75
1.6	0.83	0.79	0.76	0.72
1.8	0.81	0.77	0.73	0.70
2.0	0.80	0.75	0.71	0.67
2.2	0.78	0.73	0.69	0.65
2.4	0.76	0.71	0.67	0.62
2.6	0.75	0.69	0.65	0.60
2.8	0.73	0.68	0.63	0.58
3.0	0.72	0.66	0.61	0.56
3.2	0.70	0.64	0.59	0.54
3.4	0.69	0.63	0.57	0.52
3.6	0.67	0.61	0.56	0.51
3.8	0.66	0.60	0.54	0.49
4.0	0.65	0.58	0.53	0.48
4.2	0.63	0.57	0.51	0.46
4.4	0.62	0.56	0.50	0.45
4.6	0.61	0.54	0.49	0.43
4.8	0.60	0.53	0.47	0.42
5.0	0.59	0.52	0.46	0.41
5.5	0.56	0.49	0.43	0.38
6.0	0.53	0.46	0.41	0.36
6.5	0.51	0.44	0.38	0.34
7.0	0.49	0.42	0.36	0.32
7.5	0.47	0.40	0.34	0.30
8.0	0.45	0.38	0.32	0.28
8.5	0.43	0.36	0.31	0.27
9.0	0.41	0.35	0.29	0.25
9.5	0.40	0.33	0.28	0.24
10.0	0.38	0.32	0.27	0.23

Table T1 c: Table of the earth load coefficient κ for trench loads (Silo theory)

$\delta = \frac{1}{3}\varphi'$				
φ'	20.00	25.00	30.00	35.00
δ	6.67	8.33	10.00	11.67
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.99	0.99	0.99	0.99
0.2	0.99	0.99	0.98	0.98
0.3	0.98	0.98	0.97	0.97
0.4	0.98	0.97	0.97	0.96
0.5	0.97	0.96	0.96	0.95
0.6	0.97	0.96	0.95	0.94
0.7	0.96	0.95	0.94	0.93
0.8	0.95	0.94	0.93	0.92
0.9	0.95	0.94	0.92	0.91
1.0	0.94	0.93	0.92	0.90
1.2	0.93	0.92	0.90	0.89
1.4	0.92	0.90	0.89	0.87
1.6	0.91	0.89	0.87	0.85
1.8	0.90	0.88	0.86	0.84
2.0	0.89	0.87	0.84	0.82
2.2	0.88	0.85	0.83	0.80
2.4	0.87	0.84	0.82	0.79
2.6	0.86	0.83	0.80	0.77
2.8	0.85	0.82	0.79	0.76
3.0	0.84	0.81	0.78	0.75
3.2	0.83	0.80	0.76	0.73
3.4	0.83	0.79	0.75	0.72
3.6	0.82	0.78	0.74	0.71
3.8	0.81	0.77	0.73	0.69
4.0	0.80	0.76	0.72	0.68
4.2	0.79	0.75	0.71	0.67
4.4	0.78	0.74	0.70	0.66
4.6	0.77	0.73	0.69	0.65
4.8	0.77	0.72	0.67	0.63
5.0	0.76	0.71	0.66	0.62
5.5	0.74	0.69	0.64	0.60
6.0	0.72	0.67	0.62	0.57
6.5	0.70	0.64	0.60	0.55
7.0	0.68	0.63	0.57	0.53
7.5	0.67	0.61	0.55	0.51
8.0	0.65	0.59	0.54	0.49
8.5	0.63	0.57	0.52	0.47
9.0	0.62	0.56	0.50	0.45
9.5	0.60	0.54	0.49	0.44
10.0	0.59	0.52	0.47	0.42

Table T2 a: Table of the earth load coefficient κ_0 for trench loads (Silo theory)

$\delta = \varphi'$				
φ'	20.00	25.00	30.00	35.00
δ	20.00	25.00	30.00	35.00
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.96	0.95	0.94	0.93
0.2	0.93	0.91	0.89	0.87
0.3	0.90	0.87	0.84	0.81
0.4	0.86	0.83	0.79	0.76
0.5	0.83	0.79	0.75	0.70
0.6	0.80	0.76	0.71	0.66
0.7	0.78	0.72	0.67	0.61
0.8	0.75	0.69	0.63	0.57
0.9	0.72	0.66	0.59	0.53
1.0	0.69	0.63	0.56	0.50
1.2	0.65	0.57	0.50	0.43
1.4	0.60	0.52	0.45	0.38
1.6	0.56	0.47	0.40	0.33
1.8	0.52	0.43	0.35	0.28
2.0	0.48	0.39	0.32	0.25
2.2	0.45	0.36	0.28	0.21
2.4	0.42	0.33	0.25	0.19
2.6	0.39	0.30	0.22	0.16
2.8	0.36	0.27	0.20	0.14
3.0	0.34	0.25	0.18	0.12
3.2	0.31	0.22	0.16	0.11
3.4	0.29	0.20	0.14	0.09
3.6	0.27	0.19	0.13	0.08
3.8	0.25	0.17	0.11	0.07
4.0	0.23	0.15	0.10	0.06
4.2	0.22	0.14	0.09	0.05
4.4	0.20	0.13	0.08	0.05
4.6	0.19	0.12	0.07	0.04
4.8	0.17	0.11	0.06	0.03
5.0	0.16	0.10	0.05	0.03
5.5	0.14	0.08	0.04	0.02
6.0	0.11	0.06	0.03	0.01
6.5	0.09	0.05	0.02	0.01
7.0	0.08	0.04	0.02	0.01
7.5	0.07	0.03	0.01	0.01
8.0	0.05	0.02	0.01	0.00
8.5	0.05	0.02	0.01	0.00
9.0	0.04	0.02	0.01	0.00
9.5	0.03	0.01	0.00	0.00
10.0	0.03	0.01	0.00	0.00

Table T2 b: Table of the earth load coefficient κ for trench loads (Silo theory)

φ'	20.00	25.00	30.00	35.00
δ	13.33	16.67	20.00	23.33
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.98	0.97	0.96	0.96
0.2	0.95	0.94	0.93	0.92
0.3	0.93	0.91	0.90	0.88
0.4	0.91	0.89	0.86	0.84
0.5	0.89	0.86	0.83	0.81
0.6	0.87	0.84	0.80	0.77
0.7	0.85	0.81	0.78	0.74
0.8	0.83	0.79	0.75	0.71
0.9	0.81	0.76	0.72	0.68
1.0	0.79	0.74	0.69	0.65
1.2	0.75	0.70	0.65	0.60
1.4	0.72	0.66	0.60	0.55
1.6	0.68	0.62	0.56	0.50
1.8	0.65	0.58	0.52	0.46
2.0	0.62	0.55	0.48	0.42
2.2	0.59	0.52	0.45	0.39
2.4	0.57	0.49	0.42	0.36
2.6	0.54	0.46	0.39	0.33
2.8	0.51	0.43	0.36	0.30
3.0	0.49	0.41	0.34	0.27
3.2	0.47	0.38	0.31	0.25
3.4	0.45	0.36	0.29	0.23
3.6	0.43	0.34	0.27	0.21
3.8	0.41	0.32	0.25	0.19
4.0	0.39	0.30	0.23	0.18
4.2	0.37	0.28	0.22	0.16
4.4	0.35	0.27	0.20	0.15
4.6	0.34	0.25	0.19	0.14
4.8	0.32	0.24	0.17	0.13
5.0	0.31	0.22	0.16	0.12
5.5	0.273	0.19	0.14	0.09
6.0	0.24	0.17	0.11	0.08
6.5	0.21	0.14	0.09	0.06
7.0	0.19	0.12	0.08	0.05
7.5	0.17	0.11	0.07	0.04
8.0	0.15	0.09	0.05	0.03
8.5	0.13	0.08	0.05	0.03
9.0	0.12	0.07	0.04	0.02
9.5	0.11	0.06	0.03	0.02
10.0	0.09	0.05	0.03	0.01

Table T2 c: Table of the earth load coefficient κ_0 for trench loads (Silo theory)

$$\delta = \frac{1}{3}\varphi'$$

φ'	20.00	25.00	30.00	35.00
δ	6.67	8.33	10.00	11.67
K_1	0.50	0.50	0.50	0.50
h/b				
0.0	1.00	1.00	1.00	1.00
0.1	0.99	0.99	0.98	0.98
0.2	0.98	0.97	0.97	0.96
0.3	0.97	0.96	0.95	0.94
0.4	0.95	0.94	0.93	0.92
0.5	0.94	0.93	0.92	0.90
0.6	0.93	0.92	0.90	0.88
0.7	0.92	0.90	0.88	0.87
0.8	0.91	0.89	0.87	0.85
0.9	0.90	0.88	0.85	0.83
1.0	0.89	0.86	0.84	0.81
1.2	0.87	0.84	0.81	0.78
1.4	0.85	0.81	0.78	0.75
1.6	0.83	0.79	0.75	0.72
1.8	0.81	0.77	0.73	0.69
2.0	0.79	0.75	0.70	0.66
2.2	0.77	0.72	0.68	0.63
2.4	0.76	0.70	0.65	0.61
2.6	0.74	0.68	0.63	0.58
2.8	0.72	0.66	0.61	0.56
3.0	0.70	0.64	0.59	0.54
3.2	0.69	0.63	0.57	0.52
3.4	0.67	0.61	0.55	0.50
3.6	0.66	0.59	0.53	0.48
3.8	0.64	0.57	0.51	0.46
4.0	0.63	0.56	0.49	0.44
4.2	0.61	0.54	0.48	0.42
4.4	0.60	0.52	0.46	0.40
4.6	0.58	0.51	0.44	0.39
4.8	0.57	0.50	0.43	0.37
5.0	0.56	0.48	0.41	0.36
5.5	0.53	0.45	0.38	0.32
6.0	0.50	0.42	0.35	0.29
6.5	0.47	0.39	0.32	0.26
7.0	0.44	0.36	0.29	0.24
7.5	0.42	0.33	0.27	0.21
8.0	0.39	0.31	0.24	0.19
8.5	0.37	0.29	0.22	0.17
9.0	0.35	0.27	0.20	0.16
9.5	0.33	0.25	0.19	0.14
10.0	0.31	0.23	0.17	0.13

T3: Bending moments and normal force coefficients [50)]

Table T3 I: Bending moment and normal force coefficients

Bedding case l/2α	Interface	Moment coefficients m_{qv}	m_{qh}	m^*_{qh}	m_g / m'_g	m_w / m'_w	Normal force coefficients n_{qv}	n_{qh}	n^*_{qh}	n_g / n'_g	n_w / n'_w
l/60°	Crown	+0.286	-0.250	-1.181	+0.459	+0.029	+0.080	-1.000	-0577	+0.417	+0.708
					+0.073	+0.073				+0.066	+0.225
	Haunch	-0.293	+0.250	+0.208	-0.529	-0.264	-1.000	0	0	-1.571	+0.215
					-0.084	-0.084				-0.250	+0.068
	Invert	+0.377	-0.250	-0.181	+0.840	+0.420	-0.080	-1.000	-0.577	-0.417	+1.292
					+0.134	+0.134				-0.066	+0.411
l/90°	Crown	+0.274	-0.250	-1.181	+0.419	+0.210	+0.053	-1.000	-0577	+0.333	+0.667
					+0.067	+0.067				+0.053	+0.212
	Haunch	-0.279	+0.250	+0.208	-0.485	-0.243	-1.000	0	0	-1.571	+0.215
					-0.077	-0.077				-0.250	+0.068
	Invert	+0.314	-0.250	-0.181	+0.642	+0.321	-0.053	-1.000	-0.577	-0.333	+1.333
					+0.102	+0.102				-0.053	+0.424
l/120°	Crown	+0.261	-0.250	-1.181	+0.381	+0.190	+0.027	-1.000	-0577	+0.250	+0.625
					+0.061	+0.061				+0.040	+0.199
	Haunch	-0.265	+0.250	+0.208	-0.440	-0.220	-1.000	0	0	-1.571	+0.215
					-0.070	-0.070				-0.250	+0.068
	Invert	+0.275	-0.250	-0.181	+0.520	+0.260	-0.027	-1.000	-0.577	-0.250	+1.375
					+0.083	+0.083				-0.040	+0.438
l/180°	Crown	+0.250	-0.250	-1.181	+0.345	+0.172	0	-1.000	-0577	+0.167	+0.583
					+0.055	+0.055				+0.027	+0.186
	Haunch	-0.250	+0.250	+0.208	-0.393	-0.196	-1.000	0	0	-1.571	+0.215
					-0.063	-0.063				-0.250	+0.068
	Invert	+0.250	-0.250	-0.181	+0.441	+0.220	0	-1.000	-0.577	-0.167	+1.417
					+0.070	+0.070				-0.027	+0.451

Mathematical sign: Moment + tension on inside of pipe, - tension on outside of pipe; Normal force + tension, - compression

50) The table values apply only for circular pipes with the same moment of inertia all round. The associated m- and n-values are to be determined for other cross-sections (comp. Hornung/Kittel)

Table T3 II: Bending moment and normal force coefficients

Bedding case II/2α	Interface	Moment coefficients m_{qv}	m_{qh}	m_g / m'_g	m_w / m'_w	Normal force coefficients n_{qv}	n_{qh}	n_g / n'_g	n_w / n'_w
90°	Crown	+0.266	-0.245	+0.396	+0.198	+0.038	-0.989	+0.285	+0.643
				+0.063	+0.063			+0.045	+0.205
	Haunch	-0.271	+0.244	-0.460	-0.230	-1.000	0	-1.571	+0.215
				-0.073	-0.073			-0.250	+0.068
	Invert	+0.277	-0.224	+0.524	+0.262	-0.452	-0.718	-1.587	+0.707
				+0.083	+0.083			-0.253	+0.225
120°	Crown	+0.240	-0.232	+0.314	+0.157	-0.020	-0.960	+0.105	+0.552
				+0.050	+0.050			+0.016	+0.176
	Haunch	-0.240	+0.228	-0.362	-0.181	-1.000	0	-1.571	+0.215
				-0.058	-0.058			-0.250	+0.068
	Invert	+0.202	-0.187	+0.291	+0.145	-0.058	-0.540	-1.918	+0.541
				+0.046	+0.046			-0.305	+0.172
180°	Crown	+0.163	-0.163	+0.071	+0.035	+0.212	-0.788	-0.500	+0.250
				+0.011	+0.011			-0.080	+0.080
	Haunch	-0.125	+0.125	0	0	-1.000	0	-1.571	+0.215
				0	0			-0.250	+0.068
	Invert	+0.087	-0.087	-0.071	-0.035	-0.788	-0.212	-2.642	+0.179
				-0.011	-0.011			-0.420	+0.057

Mathematical sign: Moment + tension on inside of pipe; - tension on outside of pipe. Normal force + tension; - compression

T4: Buckling of buried pipes

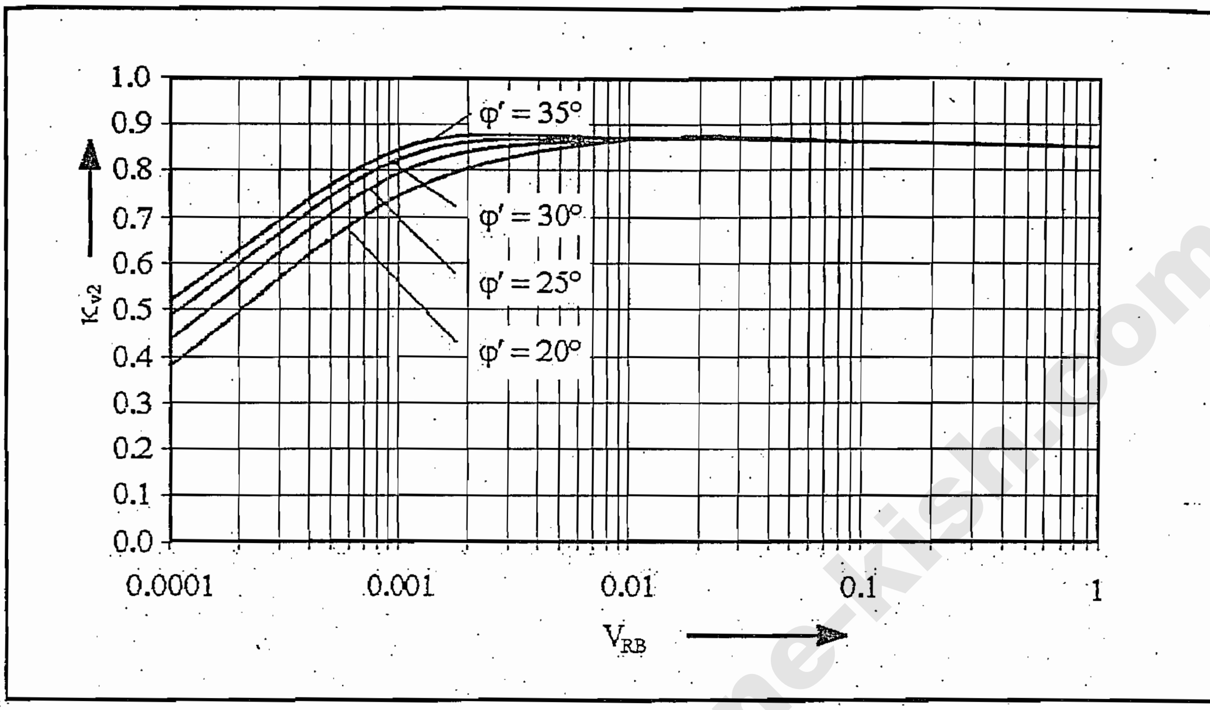


Diagram D 11: Reduction factor κ_{v2} of buckling loads with earth/traffic loads q_v

Approximation formula for κ_{v2} in accordance with Diagram D 11:

$$\kappa_{v2} = x + 0.36 \cdot (\log V_{RB} + 4) \leq 0.9$$

with
- $x = 0.52$ for $\varphi' = 35°$
- $x = 0.5$ for $\varphi' = 30°$
- $x = 0.46$ for $\varphi' = 25°$
- $x = 0.4$ for $\varphi' = 20°$

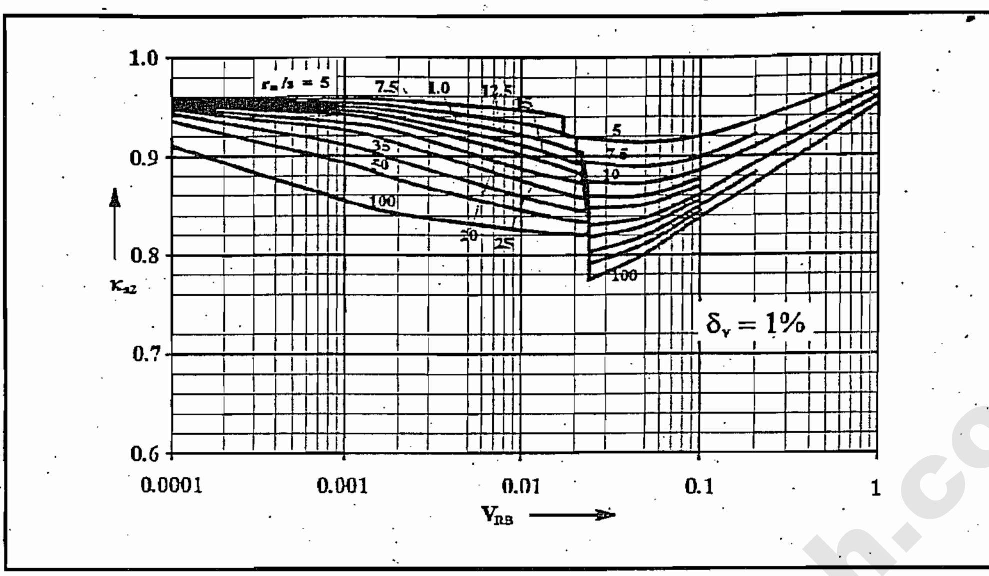


Diagram D 12a: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); δ_v = 1 %

Approximation formulas for κ_{a2} and κ_{a1} in accordance with diagrams D 12a-f and D 13a-f:

$$x = \log V_{RB}$$

$$a = \frac{1}{8}(k_0 - 2 \cdot k_2 + k_4)$$

$$b = \frac{1}{4}(3 \cdot k_0 - 4 \cdot k_2 + k_4)$$

$$\kappa_{a2} = a \cdot x^2 + b \cdot x + k_0$$

The supporting values k_0, k_2 and k_4 are presented in an accompanying publication in the *Korrespondenz Abwasser*.

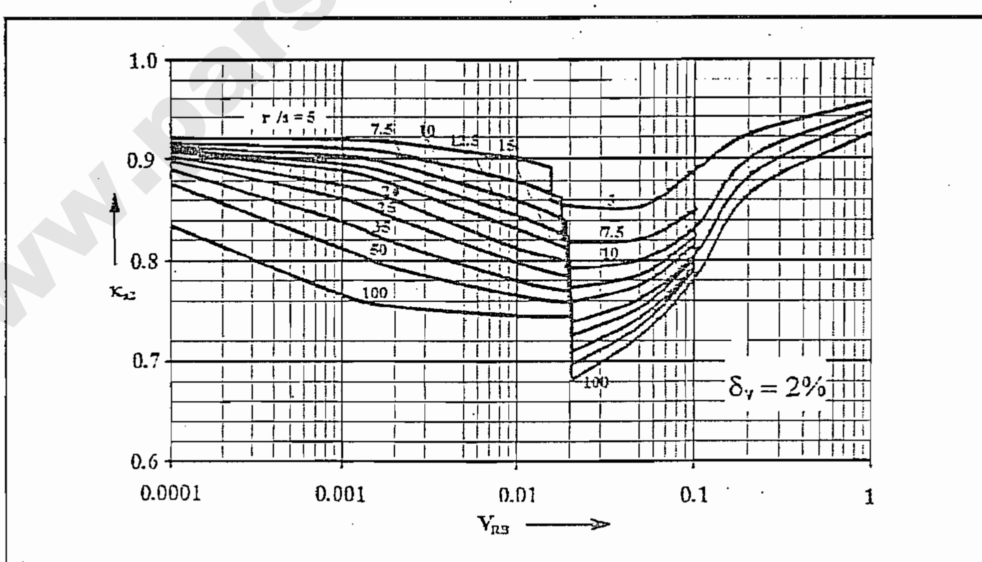


Diagram D 12b: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); δ_v = 2 %

Approximation formulas for κ_{a2} in accordance with Diags. D 12a and D 12b:

$$\kappa_{a2} = a \cdot x^2 + b \cdot x + c$$

with $x = \log V_{RB}$, $a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4)$, $b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$

	r_m/s	k_0	k_2	k_4
$\delta_v = 1\,\%$	5	0.98	0.92	0.96
	10	0.97	0.87	0.956
(D 12a)	15	0.96	0.85	0.952
	25	0.957	0.83	0.95
	50	0.953	0.8	0.94
	100	0.95	0.79	0.92
$\delta_v = 2\%$	5	0.97	0.86	0.93
	10	0.965	0.81	0.92
(D 12b)	15	0.96	0.78	0.91
	25	0.955	0.76	0.905
	50	0.95	0.73	0.85
	100	0.94	0.71	0.85

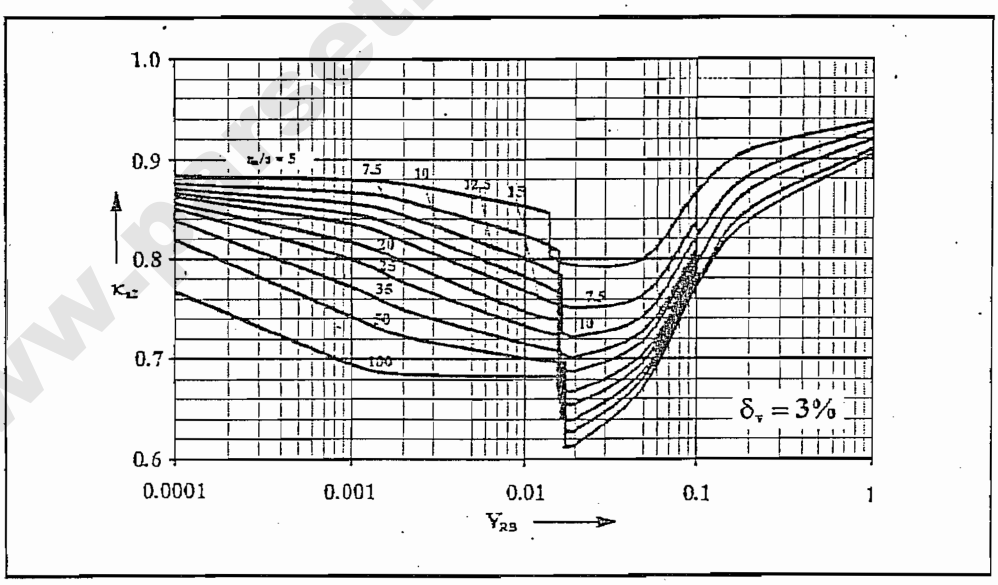


Diagram D 12c: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); $\delta_v = 3\%$

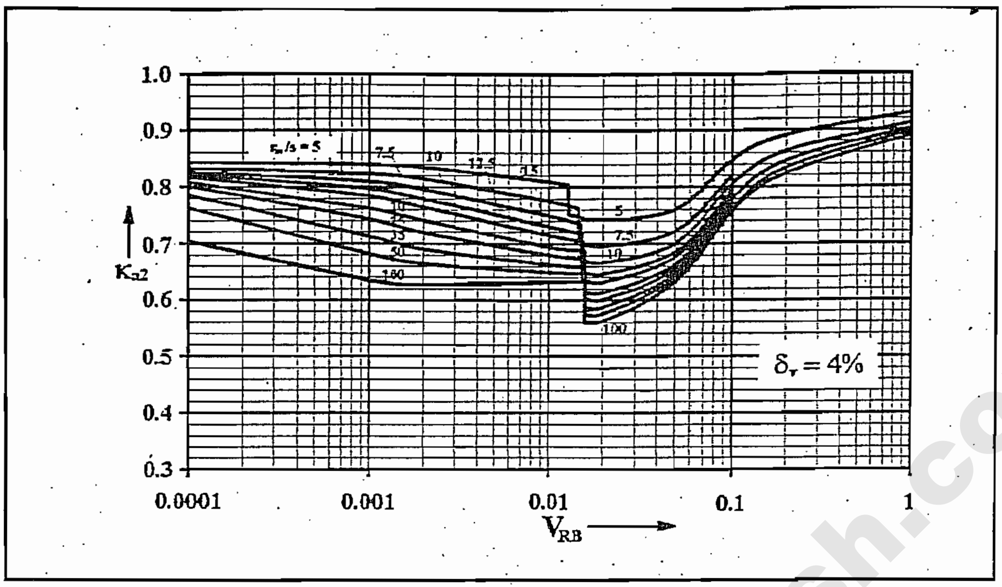


Diagram D 12d: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); δ_v = 4%

Approximation formulas for κ_{a2} in accordance with Diags. D 12c and D 12d

$$\kappa_{a2} = a \cdot x^2 + b \cdot x + c$$

$$\text{with} \quad x = \log V_{RB}, \quad a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4), \quad b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$$

	r_m/s	k_0	k_2	k_4
	5	0.96	0.82	0.89
δ_v = 3%	10	0.95	0.74	0.88
	15	0.94	0.72	0.87
(D 12c)	25	0.935	0.68	0.86
	50	0.93	0.65	0.83
	100	0.92	0.63	0.78
	5	0.94	0.77	0.86
δ_v = 4	10	0.93	0.69	0.84
	15	0.92	0.66	0.83
(D 12d)	25	0.915	0.63	0.82
	50	0.91	0.6	0.78
	100	0.9	0.59	0.73

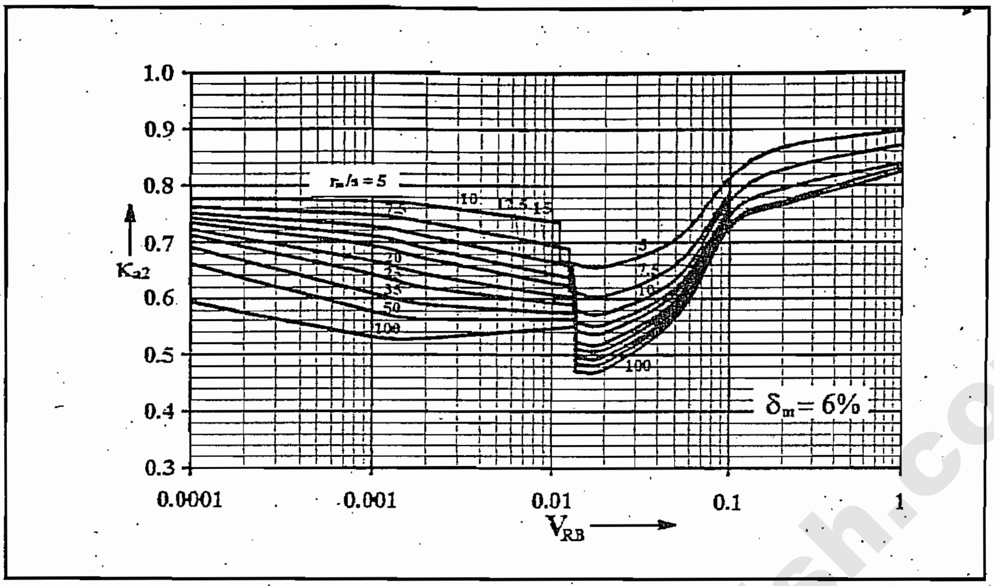


Diagram D 12e: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); $\delta_v = 6\%$

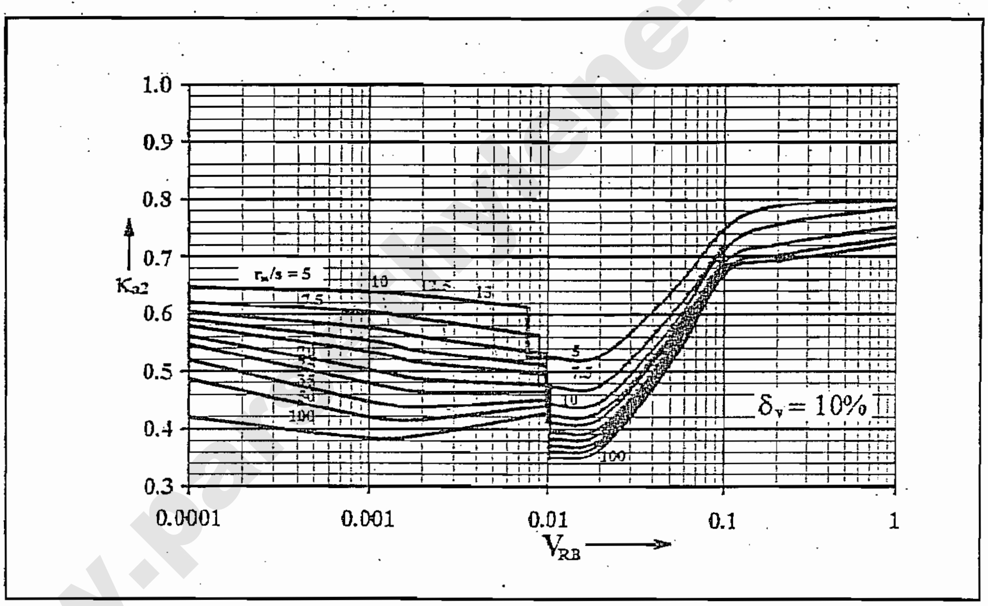


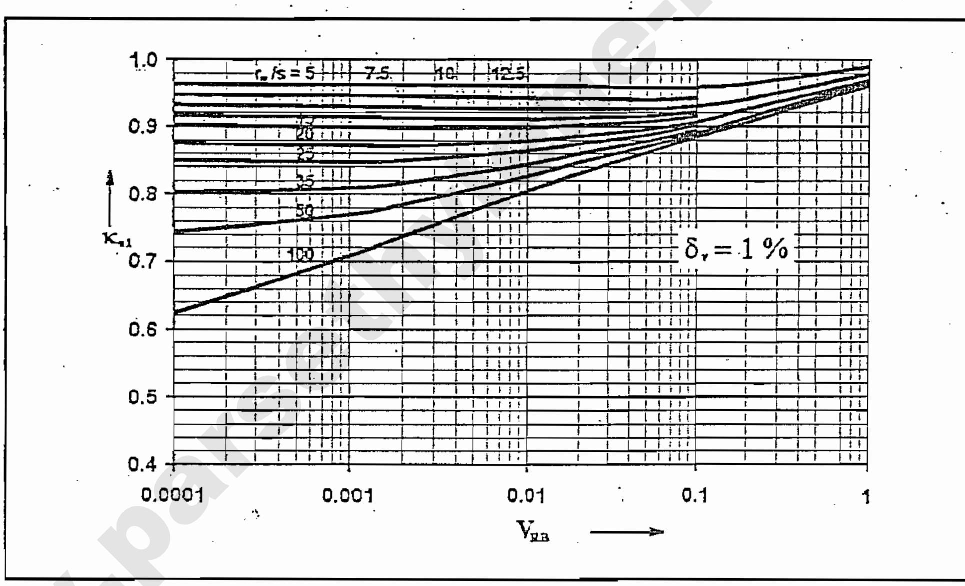
Diagram D 12f: Reduction factor κ_{a2} of the snap-through load with external water pressure (double wave preliminary deformation); $\delta_v = 10\%$

Approximation formulas for κ_{a2} in accordance with Diags. D 12e and D 12f:

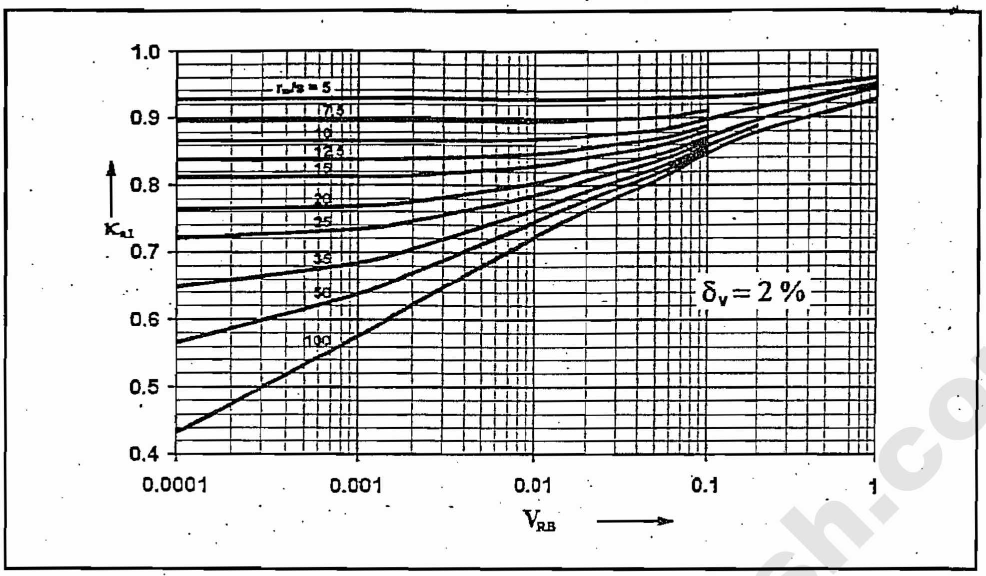
$$\kappa_{a2} = a \cdot x^2 + b \cdot x + c$$

with $x = \log V_{RB}$, $a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4)$, $b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$

	r_m/s	k_0	k_2	k_4
	5	0.91	0.68	0.79
$\delta_v = 6\ \%$	10	0.88	0.6	0.77
	15	0.865	0.56	0.75
(D 12e)	25	0.85	0.53	0.73
	50	0.845	0.51	0.68
	100	0.84	0.5	0.62
	5	0.83	0.55	0.67
$\delta_v = 10\%$	10	0.8	0.46	0.62
	15	0.79	0.44	0.6
(D 12f)	25	0.77	0.41	0.57
	50	0.76	0.39	0.51
	100	0.75	0.38	0.45



Diag. D13a: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); $\delta_v = 1\%$



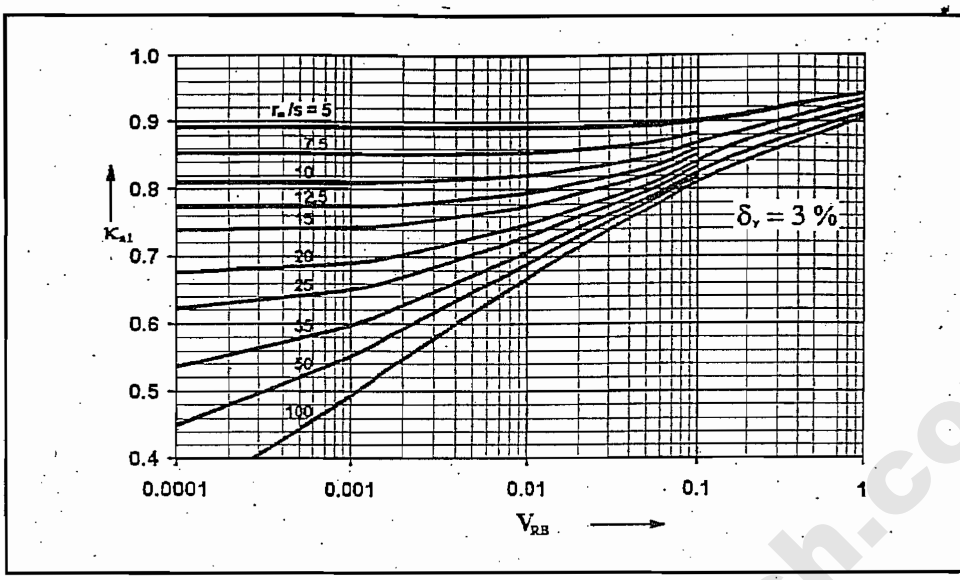
Diag. D 13b: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); $\delta_v = 2\ \%$

Approximation formulas for κ_{a1} in accordance with Diags. D 13a and D 13b:

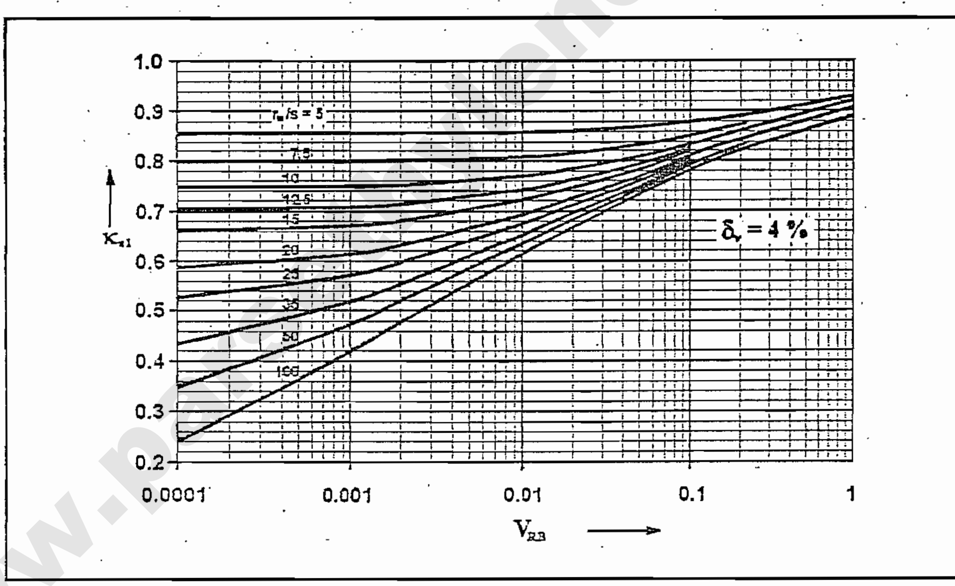
$$\kappa_{a1} = a \cdot x^2 + b \cdot x + c$$

with $\quad x = \log V_{RB},\ \ a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4),\ \ b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$

	r_m/s	k_0	k_2	k_4
	5	0.99	0.96	0.963
$\delta_v = 1\ \%$	10	0.98	0.925	0.931
	15	0.975	0.898	0.901
(D 13a)	25	0.97	0.863	0.849
	50	0.968	0.826	0.743
	100	0.963	0.804	0.622
	5	0.953	0.924	0.927
$\delta_v = 2\ \%$	10	0.945	0.865	0.866
	15	0.942	0.827	0.813
(D 13b)	25	0.935	0.784	0.723
	50	0.93	0.744	0.568
	100	0.923	0.722	0.436



Diag. D13c: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); δ_v = 3 %



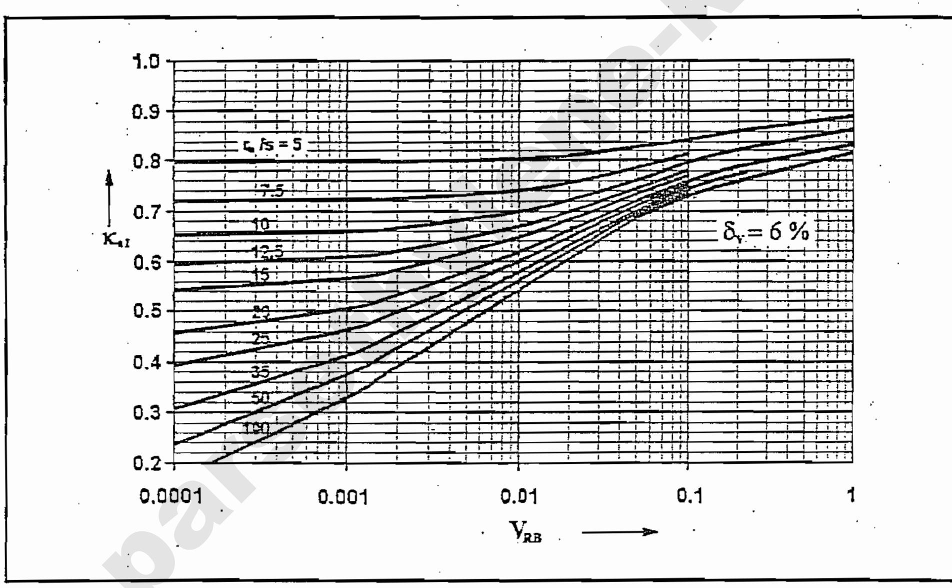
Diag. D13d: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); δ_v = 4 %

Approximation formulas for κ_{a1} in accordance with Diags. D 13c and D 13d:

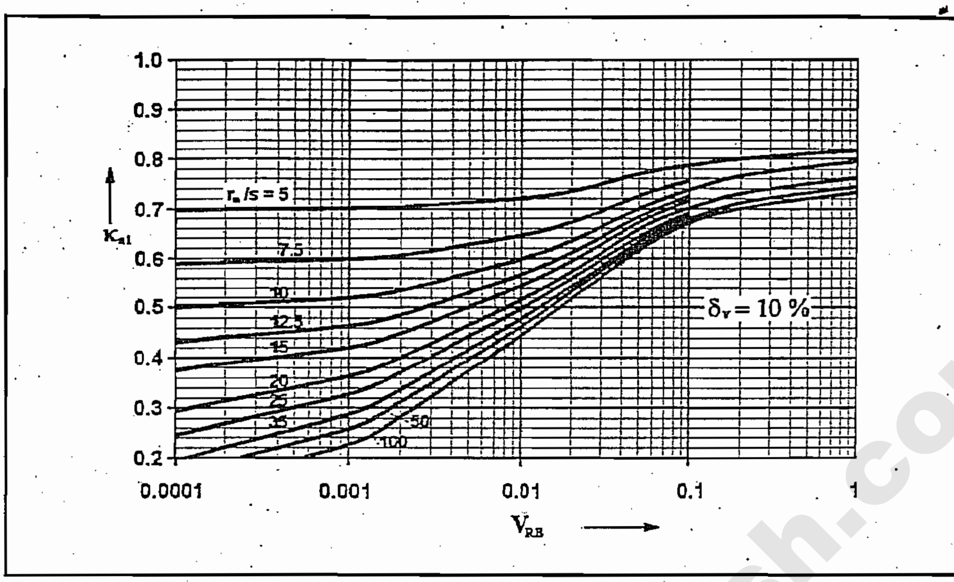
$$\kappa_{a1} = a \cdot x^2 + b \cdot x + c$$

with $x = \log V_{RB}$, $a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4)$, $b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$

	r_m/s	k_0	k_2	k_4
$\delta_v = 3$ %	5	0.937	0.89	0.892
	10	0.93	0.815	0.806
(D 13c)	15	0.925	0.77	0.735
	25	0.915	0.725	0.618
	50	0.91	0.684	0.444
	100	0.905	0.663	0.321
$\delta_v = 4$ %	5	0.929	0.859	0.859
	10	0.921	0.771	0.751
(D 13d)	15	0.92	0.723	0.664
	25	0.902	0.676	0.53
	50	0.897	0.637	0.353
	100	0.89	0.616	0.245



Diag. D13e: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); $\delta_v = 6$ %



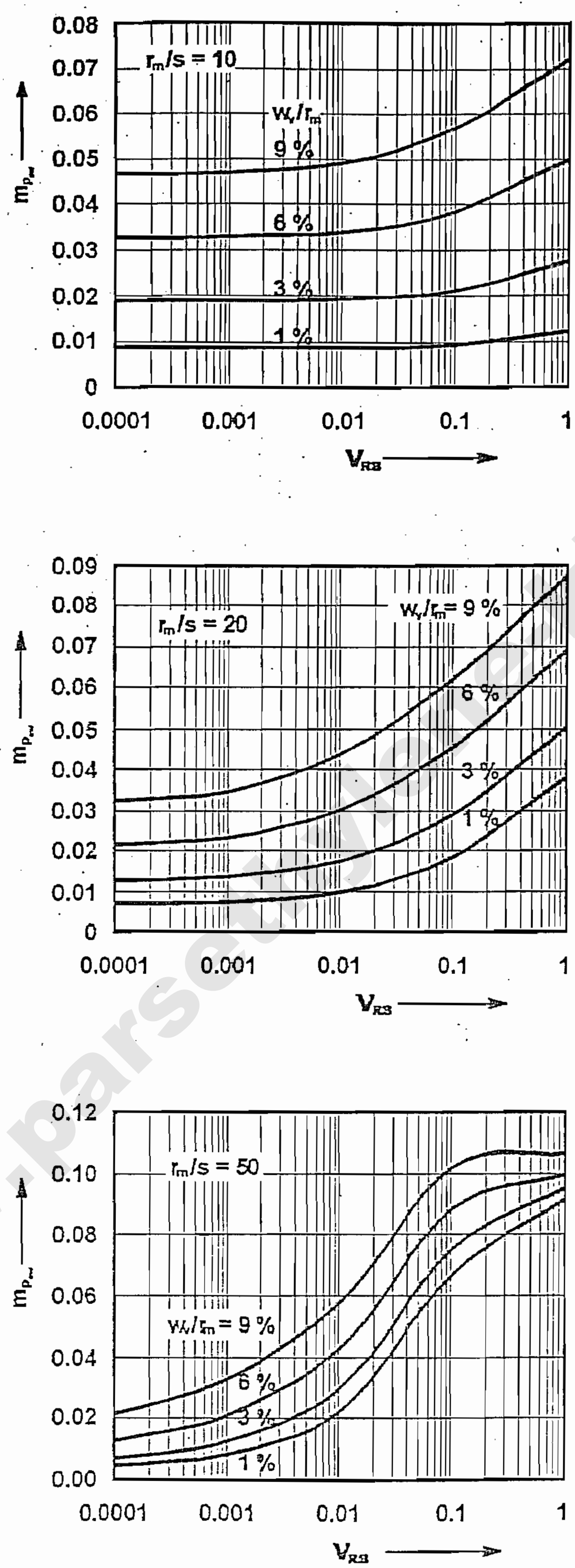
Diag. D13f: Reduction factor κ_{a1} of the snap-through load with external water pressure (local preliminary deformation); δ_v = 10 %

Approximation formulas for κ_{a1} in accordance with Diags. D 13e and D 13f:

$$\kappa_{a1} = a \cdot x^2 + b \cdot x + c$$

with $\quad x = \log V_{RB}, \quad a = \frac{1}{8} \cdot (k_0 - 2k_2 + k_4), \quad b = \frac{1}{4} \cdot (3k_0 - 4k_2 + k_4)$

	r_m/s	k_0	k_2	k_4
	5	0.888	0.802	0.797
δ_v = 6 %	10	0.863	0.698	0.652
	15	0.85	0.647	0.544
(D 13e)	25	0.845	0.57	0.4
	50	0.84	0.53	0.245
	100	0.83	0.51	0.175
	5	0.8	0.707	0.688
δ_v = 10 %	10	0.78	0.59	0.496
	15	0.77	0.52	0.38
(D 13f)	25	0.75	0.46	0.255
	50	0.74	0.41	0.175
	100	0.73	0.37	0.143



Diag. 14a - c: Coefficients of bending moment m_{pew} (invert) for the loading case external water pressure (necessary for the nonlinear stability verification, approximation method)

Appendix 2: Details on static calculation

(Task questionnaire, recommended as performance description)

TELEFAX/Address

☐ We request a no-charge offer
☐ We commission a static calculation
☐ We commission a static examination
in accordance with the given loading and installation conditions for the project in:

Postal/Zip code | Town/Country

LOADING AND EMPLACEMENT CONDITIONS - OPEN CUT

Nominal width	DN	DN	DN	
Pipeline length				m
Pipe made from: (s. A127, Table 3)				

Details on the loading

Covering height above pipe crown

min. h				m
max. h				m

Traffic load
☐ HGV 60
☐ HGV 30
☐ CV 12
☐ UIC 71 multitrack
☐ UIC 71 single-track
☐ No live load
☐ ....

Area Load p_o .... kN/m² on terrain surface
Internal pres. p_i .... bar from backpressure
Other loads ....
....

Soil type: i.a.w. ATV-A 127

	In-situ soil (Trench excavation)	Cover-ing	Pipe-line zone
G 1: Noncohesive sand and gravel	☐	☐	☐
G 2: Slightly cohesive sand and gravel	☐	☐	☐
G 3: Cohesive mixed soils and course clay	☐	☐	☐
G 4: Cohesive soils (e.g. clay)	☐	☐	☐
Other soil:	☐	☐	☐

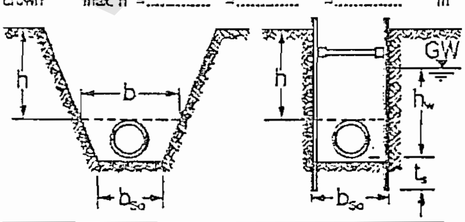
Degree of compaction of in-situ soil: D_{Pr} = ...%
Degree of compaction of other soils: D_{Pr} = ...%

From ATV-A 127, Table 1, deviating soil characteristic values:
- Unit weight .... kN/m³
- Friction angle .... °
- Deformation module .... N/mm²
- in relevant stress range 0 to .... N/mm²

Subsoil: (under the pipe)
☐ Like surrounding soil
☐ Very hard, stony or rocky
☐ Non load bearing soil: ....
....
Founding the pipeline on:
....
Depth of this founding below bottom of pipe .... m

Groundwater

Not present	☐	☐	☐	
Present	☐	☐	☐	
Max. height above crown max. h =	....	=	=	m



Bedding

Type
☐ on surrounding soil
☐ sand or gravel-sand bedding
☐ concrete bedding
☐ ....

Thickness of the upper bedding
☐ 0.07·da (60° support)
☐ 0.15·da (90° support)
☐ 0.25·da (120° support)
☐ 0 Laying on flat trench bottom and packing up of pendentive

Trench shape

Type
☐ Broad trench, backfill or built-up embankment
☐ Single trench*
☐ Multiple trench* } Attach longitudinal
☐ Stepped trench* } and cross-sections

* Load reducing affect applicable only if both trench walls remain over a long period
☐ Yes
☐ No

Details on construction work

Trench width (incl. sheeting thickness) at pipe

Crown g_o				m
Bottom g_u				m

Slope angle β
☐ 45°
☐ 60°
☐ 90°
☐ ....°

Trench sheeting

Type
☐ No sheeting
☐ Thin sheeting
☐ Horizontal (also Berlin) sheeting
☐ Vertical trench sheeting
☐ Vertical light bulkheads
☐ Vertical planks (area of cover only)
☐ Vertical bulkheads*
☐ ....

* Fixing depth in soil below bottom of trench t_s = ... m

Dismantling of sheeting
☐ by steps with backfilling
☐ in one go after backfilling
☐ by steps in the pipeline zone only with effective later compaction
☐ ....

Soil compaction

	Bed-ding	Cov-ering
Compact by layers, without verification of the degree of compaction	☐	☐
Compact by layers, with verification of the degree of compaction i.a.w. ZTVE-StB* (D_{Pr} = 97 %)	☐	☐
Uncompacted	☐	☐

* German additional technical regulations

Date:
Stamp:
(Address)

Signature:

Annexes:
☐ Specification and schedule of prices - performance description
☐ Site plan
☐ Longitudinal section
☐ Cross-section
☐ Soil expertise
☐ Additional technical regulations
☐ Traffic load scheme
☐ Sketches for ....

Appendix 3: Calculation examples

In the following examples are shown the calculation procedure for a rigid pipe, a flexible pipe using the modulus of elasticity E and a flexible pipe using nominal stiffness of nominal width DN 500.

The input data are selected for a normal case of placement, whereby peculiarities such as soil exchange, and the effects of groundwater and trench lining are also recorded.

The complete formulas are not given in the calculation procedure they can, however, be compared at any time with the text of the standard using the given formula, table, diagram and section numbers.

Input data	Qty	Unit	Rigid pipe[1]	Flexible pipe using E[2]	Flexible pipe using nominal stiffness[3]
Pipe					
Nominal width	DN	[1]	500	500	500
Internal diameter	d_i	[mm]	500	475.6	510
External diameter	d_e	[mm]	670	500	530
Wall thickness (pipe material)	s	[mm]	85	12.2	10
Cement mortar wall thickness	s_z	[mm]	-	-	-
Crushing load	F_N	[kN/m	80	-	-
Nominal stiffness	SN	[N/m²]	-	-	5000
Unit wt. gravity pipe material	χ_P	[kN/m³]	24	14	17.5
Unit wt. cement mortaring	χ_Z	[kN/m³]	-	-	-
Modulus of elasticity of pipe material (short-term characteristic)	E_{PK}	[N/mm²]	-	3000	-
Modulus of elasticity of pipe material (long-term characteristic)	E_{PL}	[N/mm²]	30000	1500	-
Vertical fracture deformation (short-term)	δ_{vFS}	[%]	-	-	20
Vertical fracture deformation (long-term)	δ_{vFL}	[%]	-	-	12
Safety class	-	[-]	A	A	A
Safety coefficients:					
failure due to fracture	χ	[1]	2.2	2.5	2.0
failure due to instability	χ	[1]	-	2.0	2.0
Permitted vertical deformation of diameter (long-term)	δ_v	[%]	-	6 (9)	6 (9)
Preliminary deformation (Type A)		[%]	-	1	1
Soil					
Surrounding soil:					
Soil group	G	[-]	G3	G3	G3
Compactness	D_{Pr}	[%]	90	90	90
Internal angle of friction	φ'	[°]	25	25	25
Soil characteristics from soil expertises			Yes ☐ No ☒	Yes ☐ No ☒	Yes ☐ No ☒
Groundwater present			Yes ☒ No ☐	Yes ☒ No ☐	Yes ☒ No ☐
Max. groundwater level above pipe invert	max h_w	[m]	1.5	1.5	1.5
Min. groundwater level below pipe invert	min h_w	[m]	-0.5	-0.5	-0.5

Input data	Qty	Unit	Rigid pipe[1]	Flexible pipe using E[2]	Flexible pipe using nominal stiffness[3]
Backfilling of pipeline zone:					35
Soil group	G	[-]	G1	G1	G1
Unit wt.	γ_S	[kN/m³]	20	20	20
Internal friction angle	φ'	[°]	35	35	35
Covering:					
Soil group	G	[-]	G3	G3	G3
Unit wt.	γ_S	[kN/m³]	20	20	20
Unit wt. under buoyancy	γ_S'	[kN/m³]	10	10	10
Internal friction angle	φ'	[°]	25	25	25
Placement conditions					
Covering height	h	[m]	3.0	3.0	3.0
Trench width	b	[m]	1.6	1.6	1.6
Embankment angle	ß	[°]	90	90	90
Covering condition	A		A2	A2	A2
Embedment condition	B		B2	B2	B2
Bedding case			I	I	I
Bedding angle	2α	[°]	90	120	120
Relative projection	a	[1]	1.0	1.0	1.0
Trench walls retained long-term			Yes ☒ No ☐	Yes ☒ No ☐	Yes ☒ No ☐
Deformation moduli E_S:					
above the pipe	E_1 [4]	[N/mm²]	2.0	2.0	2.0
next to the pipe	$E_{2,0}$ [5)6)]	[N/mm²]	6.0	6.0	6.0
surrounding soil next to the pipe	E_3 [7]	[N/mm²]	2.0	2.0	2.0
surrounding soil under the pipe	E_4 [8]	[N/mm²]	20.0	20.0	20.0
Earth pressure ratio	K_1 [9]	[1]	0.5	0.5	0.5
Wall friction angle	δ [9]	[°]	$\frac{1}{3}\varphi' = 8.3$	$\frac{1}{3}\varphi' = 8.3$	$\frac{1}{3}\varphi' = 8.3$
Loading					
Traffic load:					
Standard vehicle			HGV 60	HGV 60	HGV 60
Number of tracks			-	-	-
Dimensioning aircraft			-	-	-
Area load	p_o	[kN/m²]	-	-	-
Water filling	γ_w	[kN/m³]	10	10	10
Internal pressure	p_i	[kN/m²]	-	-	-

Footnote: 1) Concrete pipe DIN 4032
2) PVC-U pipe DIN V 19534
3) UP-GF pipe DIN 19565
4) From soil group and covering condition in accordance with Table 8
5) From soil group and embedment condition in accordance with Table 8
6) $E_2 > E_1$ is permitted due to soil exchange in the pipeline zone
7) In accordance with Table 1
8) 6.2.2: $E_4 = 10 \cdot E1$
9)From covering condition in accordance with Table4

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe Stress verification	Flexible pipe using E-modulus Stress verification/ deformation	Flexible pipe using E-modulus Deformation/ stability verification	Flexible pipe using nominal stiffness Elongation verification/ deformation	Flexible pipe using nominal stiffness Elongation/ deformation/ stability verification
		Short-term	Short-term	Long-term	Short-term	Long-term
5.2.1	Earth load					
(5.04)	κ [1]	0.874	0.874	0.874	0.874	0.874
(5.06)	κ_β [1]	-	-	-	-	-
(5.01)	p_E [kN/m^2]	52.5	52.5	52.5	52.5	52.5
5.2.2	Traffic load					
D2d, (5.07)	p [kN/m^2]	17.36	17.36	17.36	17.36	17.36
Table 6	φ [1]	1.2	1.2	1.2	1.2	1.2
(5.11)	p_V [kN/m^2]	20.8	20.8	20.8	20.8	20.8
6.	Load distribut^n					
D5	α_B [1]	0.642	0.822	0.822	0.782	0.782
Table 1	f_1 [1]	1	1	1	1	1
(6.01)	f_2 [1]	0.75	0.75	0.75	0.75	0.75
Table 8, (6.02)	E_2 [1)] [N/mm^2]	2.90	3.70	3.70	3.52	3.52
	r_m [mm]	292.5	243.9	243.9	260	260
Table 3	E_P [N/mm^2]	30,000	3,000	1,500	-	-
(6.10e)	I [mm^4/mm]	51,180	151.3	151.3	83.3	83.3
(6.10b)	S_o [N/mm^2]	7.67	0.00391	0.00195		
(6.10c)	$\overline{S}_0$ [N/mm^2]		-	0.00251		
DIN 19565-1	S_o [N/mm^2]				0.005	0.0025
(6.10d)	$\overline{S}_0$ [N/mm^2]					0.00321
(6.10)	E_P [N/mm^2]	-	-	-	8436	5416
(6.18)	Δf [1]	1.010	1.371	1.371	1.300	1.300
(6.17)	ζ [1]	0.851	0.869	0.869	0.857	0.857
(6.16)	S_{Bh} [N/mm^2]	1.47	1.929	1.929	1.81	1.81
(6.15)	V_{RB} [1]	41.60	0.0162	0.0104	0.0221	0.0142
6.2.3, Table 9	K_2 [1]	0.5	0.4	0.4	0.4	0.4
(6.05)	a' [1]	0.693	0.541	0.541	0.568	0.568
6.3.1, (6.04)	max λ [1]	1.40	1.344	1.344	1.357	1.357
(6.12)	S_{Bv} [N/mm^2]	-	3.70	3.70	3.52	3.52
(6.19a)	I/Ar_m^2 [1]	-	0.00021	0.00021	0.00012	0.00012
(6.19b)	$I.\kappa_Q/Ar_m^2$ [1]	-	0.00025	0.00025	0.00015	0.00015
			< 0.001	< 0.001	< 0.001	< 0.001
Table 10a	$c_{n,qv}$ [1]	-	+ 00891	+ 0.0891	+ 0.0891	+ 0.0891
Table 10a	$c_{n,qh}$ [1]	-	- 0.0833	- 0.0833	- 0.0833	- 0.0833
Table 10a	c_{n,qh^*} [1]	-	- 0.0658	- 0.0658	- 0.0658	- 0.0658
(6.14)	K^* [1]	-	1.086	1.169	1.014	1.014
Table 10a	$c_{v,qv}$ [1]	-	- 0.0893	- 0.0893	- 0.0893	- 0.0893
Table 10a	$c_{n,qh}$ [1]	-	+ 0.0833	+ 0.0833	+ 0.0833	+ 0.0833
Table 10a	c_{n,qh^*} [1]	-	+ 0.0640	+ 0.0640	+ 0.0640	+ 0.0640
(6.13)	c_v^* [1]	-	- 0.0198	- 0.0145	- 0.0244	- 0.0180
(6.08a)	V_S [1]		0.428	0.375	0.466	0.405
(6.08b)	V_S [1]	219.9	-	-	-	-
(6.06b)	K' [1]	-	0.927	0.921	0.927	0.925
(6.06a)	λ_P [1]	1.40 [2)]	0.777	0.748	0.802	0.771
(6.21a)	λ_{RG} [1]	1.19	0.836	0.815	0.867	0.846
	$(1 \leq b/d_a \leq 4)$					
(6.23a)	λ_{fu} [1]	3.55	3.55	3.55	3.55	3.55
6.5	λ_{fl} [1]	0.42	0.34	0.34	0.35	0.35

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe	Flexible pipe using E-modulus		Flexible pipe using nominal stiffness	
		Stress verification	Stress verification/ deformation	Deformation/ stability verification	Elongation verification/ deformation	Elongation/ deformation/ stability verification
		Short time	Short-term	Long-term	Short-term	Long-term
(6.22)	λ_S [1]	0.866	1.074	1.084	1.066	1.076
6.6	Vertical total load					
(6.24)	q_v [3)] [kN/m²]	83.0	64.7	63.6	66.4	65.2
7.3	Side pressure					
(7.01)	q_h [kN/m²]	26.1	24.5	24.7	24.5	24.7
(7.02a)	q_{h^*} [kN/m²]	-	45.4	47.3	44.1	46.9
(7.02b)	q_{h^*w} [kN/m²]	-	2.1	2.4	2.1	2.3
8.1	Cutting forces (M_{qh^*} and N_{qh^*} calculated using qh* and qh*w)					
	Crown					
(8.01)	M_{qv} [kNm/m]	1.946	1.005	0.988	1.172	1.150
(8.03)	M_{qh} [kNm/m]	- 0.558	- 0.365	- 0.368	- 0.414	- 0.417
(8.05)	M_{qh}* [kNm/m]	-	- 0.511	- 0.536	- 0.566	- 0.581
(8.07)	M_g [kNm/m]	0.073	0.004	0.004	0.005	0.005
(8.09)	M_w [kNm/m]	0.053	0.028	0.028	0.033	0.033
	ΣM [kNm/m]	1.514	0.160	0.116	0.230	0.190
(8.02)	N_{qv} [kN/m]	1.287	0.426	0.419	0.466	0.457
(8.04)	N_{qh} [kN/m]	- 7.629	- 5.987	- 6.036	- 6.370	- 6.422
(8.06)	N_{qh}* [kN/m]	-	- 6.685	- 7.000	- 6.937	- 7.382
(8.08))	N_g [kN/m]	0.199	0.010	0.010	0.011	0.011
(8.10)	N_w [kN/m]	0.571	0.372	0.372	0.423	0.423
	ΣM [kN/m]	- 5.572	-11.8640	-12.235	-12.397	-12.913
	Haunch					
(8.01)	M_{qv} [kNm/m]	- 1.982	- 1.020	- 1.003	- 1.189	- 1.167
(8.03)	M_{qh} [kNm/m]	0.588	0.365	0.368	0.414	0.417
(8.05)	M_{qh}* [kNm/m]	-	0.588	0.616	0.648	0.692
(8.07)	M_g [kNm/m]	- 0.085	- 0.004	- 0.004	- 0.005	- 0.005
(8.09)	M_w [kNm/m]	- 0.061	- 0.032	- 0.032	- 0.039	- 0.039
	ΣM [kNm/m]	- 1.569	- 0.103	- 0.056	- 0.171	- 0.102
(8.02)	N_{qv} [kN/m]	-24.284	-15.782	-15.518	-17.264	-16.951
(8.04)	N_{qh} [kN/m]	0	0	0	0	0
(8.06)	N_{qh}* [kN/m]	-	0	0	0	0
(8.08))	N_g [kN/m]	- 0.937	- 0.065	- 0.065	- 0.071	- 0.071
(8.10)	N_w [kN/m]	0.184	0.128	0.128	0.145	0.145
	ΣM [kN/m]	-25.038	-15.720	-15.455	-17.190	-16.877
	Invert					
(8.01)	M_{qv} [kNm/m]	2.230	1.059	1.041	1.234	1.211
(8.03)	M_{qh} [kNm/m]	- 0.558	- 0.365	- 0.368	- 0.414	- 0.417
(8.05)	M_{qh}* [kNm/m]	-	- 0.511	- 0.536	- 0.566	- 0.602
(8.07)	M_g [kNm/m]	0.112	0.005	0.005	0.006	0.006
(8.09)	M_w [kNm/m]	0.080	0.038	0.038	0.046	0.046
	ΣM [kNm/m]	1.865	0.225	0.180	0.306	0.244

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe Stress verification Short time	Flexible pipe using E-modulus Stress verification/ deformation Short-term	 Deformation/ stability verification Long-term	Flexible pipe using nominal stiffness Elongation verification/ deformation Short-term	 Elongation/ deformation/ stability verification Long-term
(8.02)	N_{qv} [kN/m]	- 1.287	- 0.426	- 0.419	- 0.466	- 0.457
(8.04)	N_{qh} [kN/m]	- 7.629	- 5.987	- 6.036	- 6.370	- 6.422
(8.06)	$N_{qh}*$ [kN/m]	-	- 6.685	- 7.000	- 6.934	- 7.382
(8.08))	N_g [kN/m]	- 0.199	0.010	- 0.010	- 0.011	- 0.011
(8.10)	N_w [kN/m]	1.140	0.818	0.818	0.929	0.929
	ΣM [kN/m]	- 7.974	- 12.290	- 12.647	- 12 852	- 13.343
8.2	**Stresses** Cross-section values of the pipe wall					
	Area A [mm²/mm]	85.0	12.2	12.2	10.0	10.0
	Resistance moment $W = s^2/6$ [mm³/mm]	1204.2	24.8	24.8	16.7	16.7
(8.14a)	α_{ki} [1]	1.097	1.017	1.017	1.013	1.013
(8.14b)	α_{ke} [1]	0.903	0.983	0.983	0.987	0.987
Table 3	σ_P [N/mm²]	6.4	90	50	-	-
(9.01c)	$\overline{\sigma}_P$ [N/mm²]			61.4		
(8.13)	σ_{crown} [N/mm²]	+ 1.31	+ 5.59	+ 3.76	-	-
(8.13)	$\sigma_{haun.}$ [N/mm²]	+ 0.88	+ 2.79	+ 0.95	-	-
(8.13)	σ_{invert} [N/mm²]	+ 1.60	+ 8.22	+ 6.46	-	-
9.2	Stress verif. (safety features)					
(9.01a,b)	χ_{crown} [1]	4.9	16.1	16.3	-	-
(9.01a,B)	$\chi_{haun.}$ [1]	7.3	32.2	64.6	-	-
(9.01a,b)	χ_{invert} [1]	4.0	10.9	9.5	-	-
Table 12	nec χ [1]	2.2	2.5	2.5	-	-
8.3	Elongation					
Table 3	ε_P [%]	-	-	-	±1.646	±0.988
(9.01d)	$\overline{\varepsilon}_P$ [%]	-	-	-	-	±1.175
(8.15)	ε_{crown} [%]	-	-	-	- 0.176	- 0.231
(8.15)	ε_{haunch} [%]	-	-	-	- 0.138	- 0.145
(8.15)	ε_{invert} [%]	-	-	-	- 0.222	- 0.291
(9.2)	Elong. verif. (safety features)					
(9.01a,b)	χ_{crown} [1]	-	-	-	9.4	5.1
(9.01a,B)	χ_{haunch} [1]	-	-	-	11.9	8.1
(9.01a,b)	χ_{invert} [1]	-	-	-	7.4	4.0
Table 12	nec χ [1]	-	-	-	2.0	2.0
8.3	Deformation					

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe	Flexible pipe using E-modulus		Flexible pipe using nominal stiffness	
		Stress verification	Stress verification/ deformation	Deformation/ stability verification	Elongation verification/ deformation	Elongation/ deformation/ stability verification
		Short time	Short-term	Long-term	Short-term	Long-term
(6.24)	$q_{v,E}$ [kN/m^2]	-	43.9	-	45.5	-
(7.01)	$q_{h,E}$ [kN/m^2]	-	24.5	-	24.5	-
(7.02)	$q_{h^*,E}$ [kN/m^2]	-	22.8	-	22.9	-
(8.16a)	Δd_v [mm]	-	- 6.5	-14.3	- 5.1	-15.5
(8.17)	δ_v [%]	-	1.3	3.0	1.0	3.0
9.4	Deformation verification					
9.4	Perm. δ_v [%] $\leq$ 6%: linear calculation > 6%: non-linear calculation	-	-	6	-	6
9.5	**Stability verification** (safety feature)					
9.5.3.1	Vertical total load					
Table 1	φ' [°]	-	-	35	-	35
Appx. 1: D11, Approx. formula	κ_{v2} [1]	-	-	0.9	-	0.9
(9.06)	crit q_v [N/mm^2]	-	-	0.354	-	0.388
Table 1	χ_B' [kN/m^3]	-	-	10	-	10
(6.24)	$q_{v,A}$ [N/mm^2]	-	-	0.0565	-	0.0579
(9.07)	χ [1]	-	-	6.3	-	6.7
Table 13	nec χ [1]	-	-	2.0	-	2.0
9.5.3.2	Ext. water pressure 4)					
Calculated using S_o	V_{RB} [1]			0.0081		0.0110
9.5.2	δ_v+1 [1]	-	-	4.0	-	4.0
	r_m/s [%]	-	-	20	-	26
Appx. 1: D12	κ_{a2} [1]	-	-	12.5	-	11.5
D10	α_D [1]	-	-	16.3	-	-
(9.08)	crit p_{ew} [N/mm^2]	-	-	0.137	-	0.154
(9.10)	p_{ew} [N/mm^2]	-	-	0.015	-	0.015
(9.11)	χ [N/mm^2]	-	-	9.1	-	10.3
Table 13	nec χ [1]	-	-	2.0	-	2.0
9.5.3.3	Simult. acting vertical. total load and ext. water pres.					
(9.12)	χ [1]	-	-	3.7	-	4.1
Table 13	nec χ [1]	-	-	2.0	-	2.0

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe	Flexible pipe using E-modulus		Flexible pipe using nominal stiffness	
		Stress verification	Stress verification/ deformation	Deformation/ stability verification	Elongation verification/ deformation	Elongation/ deformation/ stability verification
		Short time	Short-term	Long-term	Short-term	Long-term
9.5.4	**Non-linear stability verification** (only necessary with deformations δv > 6% and available χ < 5 · nec. χ					
Given 5)	max h_w [m]	-	-	2.5	-	2.5
Given 5)	δ_v [%]	-	-	6.5	-	6.5
9.5.4.2	Vertical total load					
(6.24)	$q_{v,A}$ [N/mm²]	-	-	0.0493	-	0.0504
(7.01)	$q_{h,A}$ [N/mm²]	-	-	0.0162	-	0.0161
(7.02a)	$q_{h^*,A}$ [N/mm²]	-	-	0.0399	-	0.0405
(9.14a)	$\alpha_{II,qv}$ [1]	-	-	1.26	-	1.27
	Invert:					
(8.01)	M_{qv} [kNm/m]	-	-	0.806	-	0.937
(8.03)	M_{qh} [kNm/m]	-	-	- 0.241	-	- 0.272
(8.05)	M_{qh^*} [kNm/m]	-	-	- 0.430	-	- 0.482
(8.07)	M_g [kNm/m]	-	-	0.005	-	0.006
(8.09)	M_w [kNm/m]	-	-	- 6)	-	- 6)
	ΣM [kNm/m]	-	-	0.140	-	0.189
(8.02)	N_{qv} [kN/m]	-	-	- 0.325	-	- 0.354
(8.04)	N_{qh} [kN/m]	-	-	- 3.951	-	- 4.188
(8.06)	N_{qh^*} [kN/m]	-	-	- 5.615	-	- 5.911
(8.08))	N_g [kN/m]	-	-	- 0.010	-	- 0.011
(8.10)	N_w [kN/m]	-	-	-	-	-
	ΣM [kN/m]	-	-	- 9.901	-	-10.462
(9.15a)	σ_{So} [N/mm²]	-	-	+6.42	-	-
(9.15b)	εSo [%]	-	-	-	-	- 0.275
(5.01)	$p_{E,A}$ [kN/m²]	-	-	35.0	-	35.0
9.5.4.2	$\overline{\sigma}_R$ [N/mm²]	-	-	64.9	-	-
9.5.4.2	ε_R [%]	-	-	-	-	±1.233
(9.16)		-	-	10.1	-	4.48
9.5.4.2	Ext. water pressure 4)					
9.5.2	δv + 1 [%]	-	-	7.5	-	7.5
Appx 1: D12e,f	κa2 [19	-	-	0.57	-	0.52
(9.08)	crit p_{ew} [N/mm²]	-	-	0.111	-	0.120
(9.10)	p_{ew} [N/mm²]	-	-	- 0.025	-	- 0.025
(9.14b)	$\alpha_{II,pw}$ [1]	-	-	1.82	-	1.71
	Invert:					
D14b	m_{pew} [1]	-	-	0.034	-	0.038
(9.13)	$M_{so,pew}$ [mm/mm]	-	-	50.6	-	64.2
(8.12)	$N_{so,pew}$ [N/mm]	-	-	- 6.25	-	- 6.62
(9.15a)	$\sigma_{So,pew}$ [N/mm²]	-	-	+3.27	-	-

Section (Formula No.) Table No. Diag No.	Calculation step	Rigid pipe	Flexible pipe using E-modulus		Flexible pipe using nominal stiffness	
		Stress verification	Stress verification/ deformation	Deformation/ stability verification	Elongation verification/ deformation	Elongation/ deformation/ stability verification
		Short time	Short-term	Long-term	Short-term	Long-term
9.5.4.2	σ_P [N/mm²]	-	-	50	-	-
(6.10)	E_P [N/mm²]	-	-	-	-	4218 [7)]
(9.15b)	$\varepsilon_{So,pew}$ [%]	-	-	-	-	0.170
9.5.4.2	ε_P [%]	-	-	-	-	±0.988
(9.16)	χ_{pew} [1]	-	-	15.3	-	5.81
9.5.3.3	Simult. acting vertical. total load and ext. water pres.					
(9.12)	χ [1]	-	-	6.08	-	2.53
Table 13	nec χ [1]	-	-	2.0	-	2.0
9.4	Non-linear verification of deformat^n					
(9.12)	Avail χ [1]	-	-	6.08	-	2.53
9.4	< 5 nec χ	-	-	<10	-	<10
(9.16)	χ_{qv} [1]	-	-	10.1	-	4.48
(9.05)	α_{II} [1]	...-	-	1.11	-	1.29
9.4	α_{II} . δv [%]	-	-	7.22	-	8.39
9.4	perm δ_v [%]	-	-	9	-	9

Footnotes:

1) $E_2 > E_1$ is permitted due to soil exchange in the pipeline zone.
2) For pipes of greater stiffness ($V_{RB} > 1$) $\lambda_R = \max \lambda$ (6.3.2).
3) As the groundwater can sink below the invert of the pipe q_v can be calculated without buoyancy.
4) Verification is necessary as groundwater exists above the invert of the pipe.
5) The values max h_w and δ_v are increased compared with the initial example in order to demonstrate the calculation step for non-linear verification.
6) For stability verification the load case of pipe without water fill is relevant.
7) The calculation of the elongation in the pipe wall as a result of external water pressure takes place using the long-term E modulus of the pipe material.

Appendix 4: Literature [where there is no known translation into English a courtesy translation is provided in square brackets]

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