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Underground installation of flexible glass-reinforced pipes based on unsaturated polyester resin (GRP-UP) —

Part 3: Installation parameters and application limits

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British Standards

National foreword

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Underground installation of flexible glass-reinforced pipes based on unsaturated polyester resin (GRP-UP) —

Part 3: Installation parameters and application limits

Installation enterrée de canalisations flexibles renforcées de fibres de verre à base de résine polyester insaturée (GRP-UP) —

Partie 3: Paramètres d'installation et limites d'application

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

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The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

In exceptional circumstances, when a technical committee has collected data of a different kind from that which is normally published as an International Standard ("state of the art", for example), it may decide by a simple majority vote of its participating members to publish a Technical Report. A Technical Report is entirely informative in nature and does not have to be reviewed until the data it provides are considered to be no longer valid or useful.

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

ISO 10465-3 was prepared by Technical Committee ISO/TC 138, *Plastics pipes, fittings and valves for the transport of fluids*, Subcommittee SC 6, *Reinforced plastics pipes and fittings for all applications*.

This second edition cancels and replaces the first edition (ISO 10465-3:1999), which has been technically revised to take into account changes made to methods in base documents ATV-A 127 and AWWA M-45 (see Introduction).

ISO 10465 consists of the following parts, under the general title *Underground installation of flexible glass-reinforced pipes based on unsaturated polyester resin (GRP-UP)*:

— *Part 1: Installation procedures* [Technical Specification]

— *Part 2: Comparison of static calculation methods* [Technical Report]

— *Part 3: Installation parameters and application limits* [Technical Report]

Introduction

Work in ISO/TC 5/SC 6 (now ISO/TC 138) on writing International Standards for the use of glass-reinforced plastics (GRP) pipes and fittings was approved at the subcommittee meeting in Oslo in 1979. An ad hoc group was established and the responsibility for drafting various International Standards was later given to a Task Group (now ISO/TC 138/SC 6).

At the SC 6 meeting in London in 1980, Sweden proposed that a working group be formed to develop documents regarding a code of practice for GRP pipes. This was approved by SC 6, and Working Group 4 (WG 4) was formed for this purpose. Since 1982, many WG 4 meetings have been held which have considered the following matters:

— procedures for the underground installation of GRP pipes;

— pipe/soil interaction with pipes having different stiffness values;

— minimum design parameters;

— overview of various static calculation methods.

During the work of WG 4, it became evident that unanimous agreement could not be reached within the working group on the specific methods to be employed to address these issues. It was therefore agreed that all parts of the code of practice should be made into a type 3 Technical Report, and this was the form in which this part of ISO 10465 was first published in 1999. Since then the ISO rules dealing with the classification of document types have been revised and this has resulted in the three parts of ISO 10465 now being published as either a Technical Specification or a Technical Report.

ISO 10465-1, published as Technical Report in 1993 and revised as a Technical Specification in 2007, describes procedures for the underground installation of GRP pipes. It concerns particular stiffness classes for which performance requirements have been specified in at least one product standard, but it can also be used as a guide for the installation of pipes of other stiffness classes.

ISO 10465-2, published as a Technical Report in 1999 and revised in 2007, presents a comparison of the two primary methods used internationally for static calculations on underground GRP pipe installations.

These methods are

a) the ATV method given in ATV-A 127, *Guidelines for static calculations on drainage conduits and pipelines*, and

b) the AWWA method given in AWWA manual M-45, *Fiberglass pipe design*.

This part of ISO 10465, published as a Technical Report in 2007, gives additional information, which is useful for static calculations primarily when using an ATV-A 127 type design system in accordance with ISO 10465-2, on items such as:

parameters for deflection calculations;

soil parameters, strain coefficients and shape factors for flexural-strain calculations;

soil moduli and pipe stiffness for buckling calculations with regard to elastic behaviour;

parameters for rerounding and combined-loading calculations;

the influence of traffic loads;

the influence of sheeting;

safety factors.

This Technical Report is not to be regarded as an International Standard. It is proposed for provisional application so that experience may be gained on its use in practice. Comments should be sent to the secretariat of TC 138/SC 6.

Underground installation of flexible glass-reinforced pipes based on unsaturated polyester resin (GRP-UP) —

Part 3: Installation parameters and application limits

1 Scope

This part of ISO 10465 gives supplementary information on parameters and application limits for the underground installation of flexible glass-reinforced pipes based on unsaturated polyester resin (GRP-UP). It is particularly relevant when using an ATV-A 127 type design system.

Explanations for the long-term safety factors incorporated into the GRP system standards based on simplified probabilistic methods are provided in Annex G.

2 Normative references

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

ATV-A 127, *Guidelines for static calculations on drainage conduits and pipelines,* 3rd edition, August 2000, (German Association for Water Pollution Control)

AWWA M-45, *Fiberglass pipe design manual M-45*, 2005 (American Water Works Association)

3 Symbols and abbreviated terms

For the purposes of this document, the following symbols apply.

NOTE 1 This clause also contains symbols and abbreviations from ISO 10465-1 and ISO 10465-3 for completeness.

NOTE 2 Several identical symbols are used in ATV-A 127 and AWWA M-45 to represent different quantities, and where this occurs, the origin of the symbol is given in the rightmost column.

NOTE 3 The format of the symbols listed here has been aligned as far as practicable with the *ISO/IEC Directives*, part 2, namely they appear in Times New Roman italic font. This format may differ slightly from the format used in ATV-A 127 and AWWA M-45.

Symbol	Unit	Meaning
AQL	—	acceptable quality level
a'	—	effective relative projection
a_{f}	—	ageing factor (ATV)
a_{f}	—	distribution factor (AWWA)

B1, B2, B3, B4	—	embedment conditions
b	m	trench width at spring-line
b'	m	distance from trench wall to pipe (see Figure 1)
C_n	—	buckling scalar calibration factor
c_1, c_2, c_3, c_4	—	coefficients used to determine ζ
c_4	—	reduction factor
c_f	—	creep factor
$c_{h,qv}$, $c_{v,qh}$, c_{v,qh^*}, $c_{h,qh}$, c_{h,qh^*}, $c_{v,qv}$, c_{v^*}, c_{v,qh^*}, c_{h,qh^*}, c_{v^*}	—	deformation coefficients
D	mm	mean pipe diameter
D_f	—	shape factor
D_g	—	shape adjustment factor
D_L	—	deflection lag factor
D_{pr}	%	compaction (based on simple proctor)
d_e	m	external pipe diameter
d_i	m	internal pipe diameter
d_m	m	mean pipe diameter $\left[(d_e \times 1\,000) - e\right]$
d_v	mm	vertical deflection
d_{vA}	mm	maximum permissible long-term deflection
d_{vR}	mm	vertical deflection at rupture
$(d_v/d_m)_{permissible}$	%	maximum permissible relative vertical deflection
$(d_v/d_m)_{initial}$	%	initial vertical deflection
$(d_v/d_m)_{50}$	%	long-term (50 year) vertical deflection
$(d_v/d_m)_{ult}$	%	ultimate long-term vertical deflection
E, E_o, E_p, $E_{t,wet}$	N/m^2	apparent flexural moduli of pipe wall
E', E_1, E_2, E_3, E_4, E'_s, E_s, $E_{s,\sigma}$, E_{20}	N/m^2	soil deformation moduli
E_{TH}	N/m^2	tensile hoop modulus
e	mm	pipe wall thickness
e	—	base of natural logarithms (2,718 281 8)
F	—	compaction factor
F_A, F_E	kN	wheel loads

FS	—	calculated safety factor (ATV)
FS	—	design factor = 2,5 (AWWA)
FS_b	—	bending safety factor
FS_{pr}	—	pressure safety factor
f_1	—	reduction factor for creep
f_2	—	reduction factor for ground water in pipe zone
G1, G2, G3, G4	—	soil groups
HDB	—	extrapolated pressure strain at 50 years
H_{EVD}	m	environmental depth of cover
h	m	depth of cover to top of pipe
h_{int}	m	depth at which load from wheels interact
h_w	m	height of water surface above top of pipe
I	m^4/m	second moment of area in longitudinal direction per unit length (of a pipe)
I_f	—	impact factor (AWWA)
i_f	N/mm^2	installation factor
K^*	—	coefficient for bedding reaction pressure
K'	—	modulus of deformation
K_1, K_2	—	ratio of horizontal to vertical soil pressure in soil zones 1 and 2
K_3	—	ratio of horizontal to vertical soil pressures in pipe-zone backfill, when backfill is at top of pipe (see ISO 10465-3:2007, Annex A)
k_{v2}	—	reduction factor to take into account the elastic-plastic soil mass law and preliminary deflections
k_x	—	bedding coefficient
L_1	m	load width parallel to direction of travel
L_2	m	load width perpendicular to direction of travel
$LLDF$	—	live load as a function of depth factor
M	—	sum of bending moments
M_p	—	multiple presence factor
M_s	N/m^2	composite constrained-soil modulus
M_{s1}	N/m^2	value of composite constrained-soil modulus from ISO 10465-3:2007, Table A.3

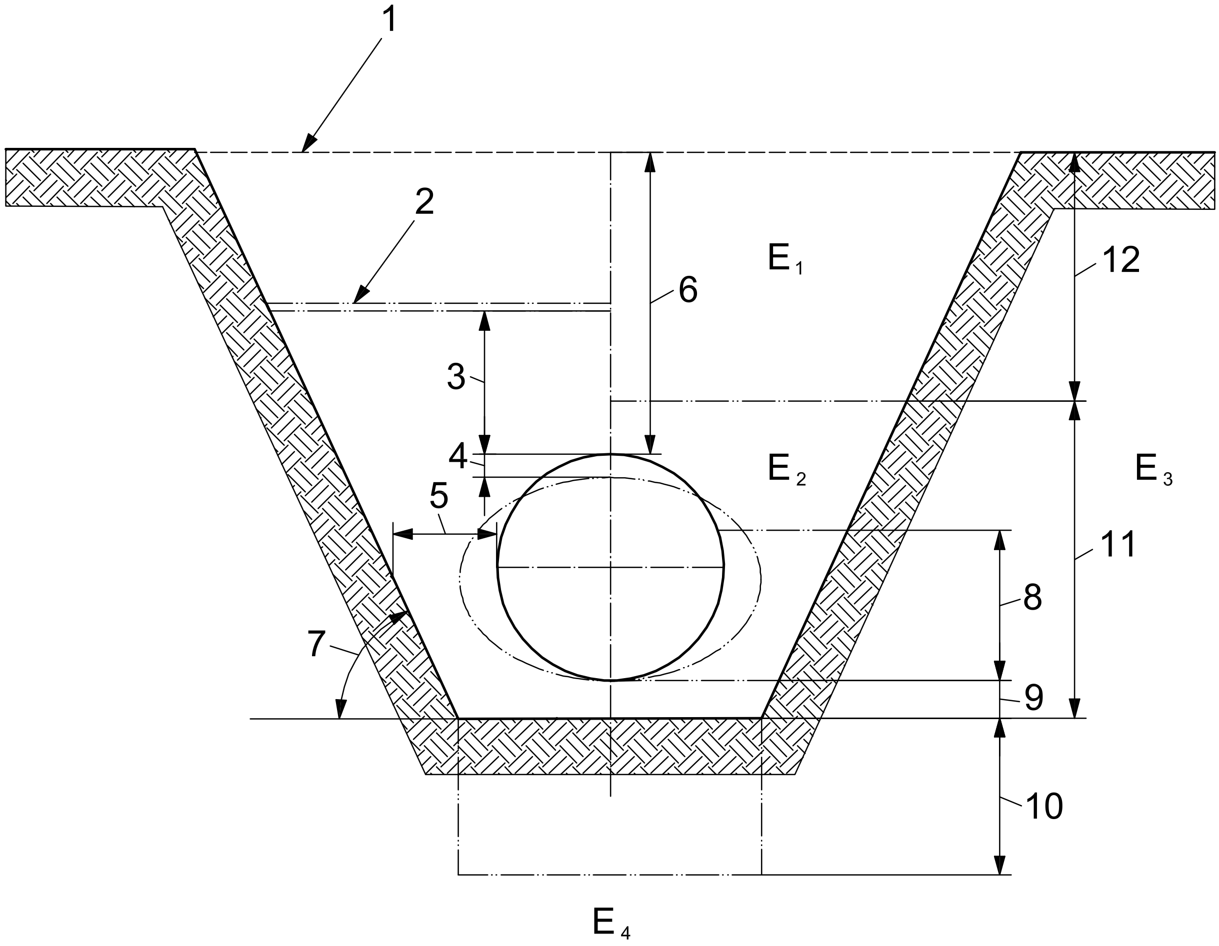
M_{s100}	N/m^2	composite constrained-soil modulus at 100 % SPD
M_{sb}	N/m^2	backfill soil constrained modulus
M_{sn}	N/mm^2	native soil constrained modulus
m_{qv}, m_{qh}, m_{qh*}	—	moment factors
N	—	sum of normal forces
n_{10}	—	number of blows
P	N	magnitude of wheel load
PN	—	nominal pressure (pipe characteristic)
P	bar	internal pressure
P_f	—	probability of failure
P_v	MPa (N/mm^2)	internal under-pressure
P_w	N/m^2	working pressure
$P(X)$	—	probability function
P_{50}	bar	long-term (50 year) failure pressure
p	N/m^2	soil stress resulting from traffic loads
p_E	N/mm^2	pressure due to prismatic soil load
p_e	N/mm^2	external water pressure
p_F	N/m^2	soil stress due to traffic load according to Boussinesq
p_o	N/m^2	soil pressure due to uniformly distributed surface load
p_v	N/mm^2	soil pressure resulting from traffic load
q_a	MPa (N/mm^2)	permissible buckling pressure
q_c	MPa (N/mm^2)	critical buckling pressure
q_{cl}	MPa (N/mm^2)	critical buckling pressure under sustained load
q_{c*w}	N/mm^2	horizontal bedding reaction for pipe and contents
q_h, q_v	N/mm^2	horizontal or vertical soil pressure on pipe
q_{h*}	N/mm^2	horizontal bedding reaction pressure
q_{hLT}	N/mm^2	reduced long-term horizontal soil pressure
$q_{h,50}$	N/mm^2	long-term (50 year) horizontal soil pressure
q_{vLT}	N/mm^2	reduced long-term vertical soil pressure
$q_{v,50}$	N/mm^2	long-term (50 year) vertical soil pressure
q_{vwa}	N/mm^2	vertical load due to pipe and contents

R_h	—	depth-of-fill correction factor
R_w	—	water buoyancy reduction factor
r	—	rerounding factor (AWWA)
r	m	mean pipe radius (AWWA)
r_A, r_E	m	wheel radii (ATV)
r_c	—	rerounding coefficient
r_i	m	pipe internal radius
r_m	m	mean pipe radius
S_{Bh}	N/mm^2	horizontal bedding stiffness
S_{Bv}	N/mm^2	vertical bedding stiffness
S_b	—	long-term ring-bending strain capability of the pipe
S_c	—	soil support combining factor
S_k	N/mm^2	characteristic stress
S_O	N/m^2	long-term pipe stiffness
$S_{O,50}$	N/m^2	long-term pipe stiffness
$\bar{S}_o$	N/m^2	weighted long-term pipe stiffness
S_{OK}	N/m^2	long-term (50 year) pipe stiffness
S_{OL}	N/m^2	long-term (2 year) pipe stiffness
SPD	%	standard proctor density
S_p	N/m^2	initial pipe stiffness
$S_{p,50}$	N/m^2	long-term pipe stiffness
S_R	N/mm^2	$S_p \times 8 \times 10^{-6}$
$S_{R,50}$	N/mm^2	$S_{p,50} \times 8 \times 10^{-6}$
S_{Res}	N/mm^2	standard deviation of strength of pipe
$S_{Res,B}$	N/mm^2	standard deviation of strength of pipe below ground
S_S	N/mm^2	standard deviation of stress in pipe
$S_{S,\,B}$	N/mm^2	standard deviation of stress in pipe below ground
t_l	m	length of tyre footprint
t_w	m	width of tyre footprint
V_{RB}	—	system stiffness
V_S	—	stiffness ratio

W_c	N/m^2	vertical soil load on pipe
W_L	N/m^2	traffic load
X	—	safety index
y	%	coefficient of variation for initial tensile strength
y_R	%	coefficient of variation for tensile strength
z	%	coefficient of variation for initial ultimate deflection
α	°	half the bedding angle (see Figure 2)
α_B	—	reduction factor depending upon trench proportions and embedding conditions
α_{Bi}	—	value from ISO 10465-2:2007, Figure 5
α_D	—	snap-through coefficient
$\alpha\kappa$, $\alpha\kappa_i$, $\alpha\kappa_e$	—	correction factor for extreme curvature of inner or outer edge
β	°	half the horizontal support angle (see Figure 2)
β	°	(ATV) trench wall slope angle (see Figure 1)
γ_b	N/m^3	bulk density of backfill material
γ_w	N/m^3	density of pipe contents
δ	°	trench wall friction angle
δ_h	%	relative horizontal deflection
δ_v	%	relative vertical deflection
δ_{va}	%	negative relative vertical deflection due to traffic and vacuum load
δ_{vc}, δ_{vs}	%	negative relative vertical deflection due to soil load
δ_{v50}	%	long-term relative vertical deflection
δ_{vio}	%	positive relative vertical deflection due to backfilling in pipe zone
δ_{viv}	%	negative relative vertical deflection due to installation irregularities
δ_{vs50}	%	long-term negative relative vertical deflection due to soil load
δ_{vw}	%	negative relative vertical deflection due to weight of pipe
δ_w	%	relative vertical deflection due to traffic load
ε_b	—	bending strain caused by maximum permitted deflection

ε_{comp}	—	compressive strain due to vertical load
ε, ε_t, ε_f	—	calculated flexural strains in pipe wall
ε_{if}	—	flexural strain due to installation irregularities
ε_{max}, ε_R	—	maximum permissible strain due to pressure
ε_{PK}	—	initial bending tensile strain
ε_{PL}	—	long-term bending tensile strain
ε_{pr}	—	calculated strain in pipe wall due to internal pressure
$\bar{\varepsilon}_R$	—	weighted calculated value of outer fibre strain
ε_{tot}	—	total flexural strain
ε_v	—	flexural strain due to total vertical load
ε_{vio}	—	flexural strain due to backfilling in pipe zone
ε_{vw}	—	flexural strain due to weight of pipe
ε_w	—	flexural strain due to pipe contents
ε_{50}	—	long-term maximum bending strain caused by maximum permitted deflection
ζ	—	correction factor for horizontal bedding
η, η_t, η_f, η_{ff}	—	safety factors
η_{haf}	—	combined flexural safety factor
η_{hat}	—	combined tensile safety factor
$\eta_{t,PN}$	—	redefined safety factor for pipe to operate at PN
φ	°	soil internal friction angle
φ'	—	impact factor (ATV)
φ_s	—	variability factor for compacted soil
χ	—	coefficient of safety
χ_P	MN/m^3	unit weight (density) of pipe material
χ_s	N/m^3	unit weight (density) of soil
χ_w	N/m^3	unit weight (density) of water
κ, κ_β	—	reduction factor for distributed load according to silo theory when trench angle, β, is 90°
κ_o, $\kappa_{o\beta}$	—	reduction factor for distributed load according to silo theory when trench angle, β, is not 90°
λ_B, λ_{B50}, λ_P, λ_{PG}, λ_{PG50}, λ_S	—	concentration factors in soil next to pipe

λ_{max}	—	maximum concentration factor
λ_{PLT}	—	long-term value for λ_P
λ_R	—	reduction factor for soil friction with time
μ_{Res}	N/mm^2	mean value of pipe strength (resistance)
$\mu_{Res, A}$	N/mm^2	mean value of strength (resistance) of pipe above ground
$\mu_{Res, B}$	N/mm^2	mean value of strength (resistance) of pipe below ground
$\mu_{S,B}$	N/mm^2	mean value of stress in pipe below ground
ν_s	—	Poisson ratio of soil
ρ	MN/m^3	density of pipe wall material
σ_c	N/mm^2	calculated compressive stress in pipe wall
σ_{PK}	—	initial bending tensile stress
σ_{PL}	—	long-term bending tensile stress
$\bar{\sigma}_R$	—	weighted bending tensile stress
σ_t	N/mm^2	calculated tensile stress in pipe wall



Key

1 ground level
2 water table
3 height of water surface above top of pipe, h_w
4 vertical deflection, d_v
5 distance from trench wall to pipe, b'
6 depth of cover to top of pipe, h
7 trench wall angle, β
8 thickness of primary embedment
9 thickness of bedding
10 thickness of foundation (if required)
11 pipe embedment
12 thickness of backfill

Soil moduli zones

E1 trench backfill above pipe embedment
E2 pipe embedment
E3 undisturbed native soil or *in situ* material to side of trench
E4 undisturbed native soil or *in situ* material below bottom of trench (foundation material)

NOTE 1 The AWWA M-45 design manual uses M_{sb} in zone E_2.

NOTE 2 The AWWA M-45 design manual uses M_{sn} in zones E_3 and E_4.

Figure 1 — Symbols and terminology

4 Parameters for deflection calculations when using an ATV-A 127 type design system

This clause covers the recommended soil parameters and deflection coefficients to use when calculating the initial or long-term deflections in accordance with ATV-A 127.

NOTE In the following calculations deflections having a negative value indicate a reduction in vertical diameter.

4.1 Initial deflection

The measurement of the initial deflection shortly after installation, when the effects of traffic loads are not present, is a very easy way to assess and control the quality of the installation. A calculation of initial deflection should be done for this loading condition.

ATV-A 127 and the AWWA M-45 design manual do not address effects of installation variability, deflection resulting from the pipe's own weight, and the reduction in deflection from the upwards ovalization of the pipe when the pipe zone backfill is compacted. It is recommended that, for deflection calculations, these effects be considered in addition to the effects of soil and superimposed loads. This recommendation is made because these matters have been found significant in practice, especially for pipes having a DN greater than 2000.

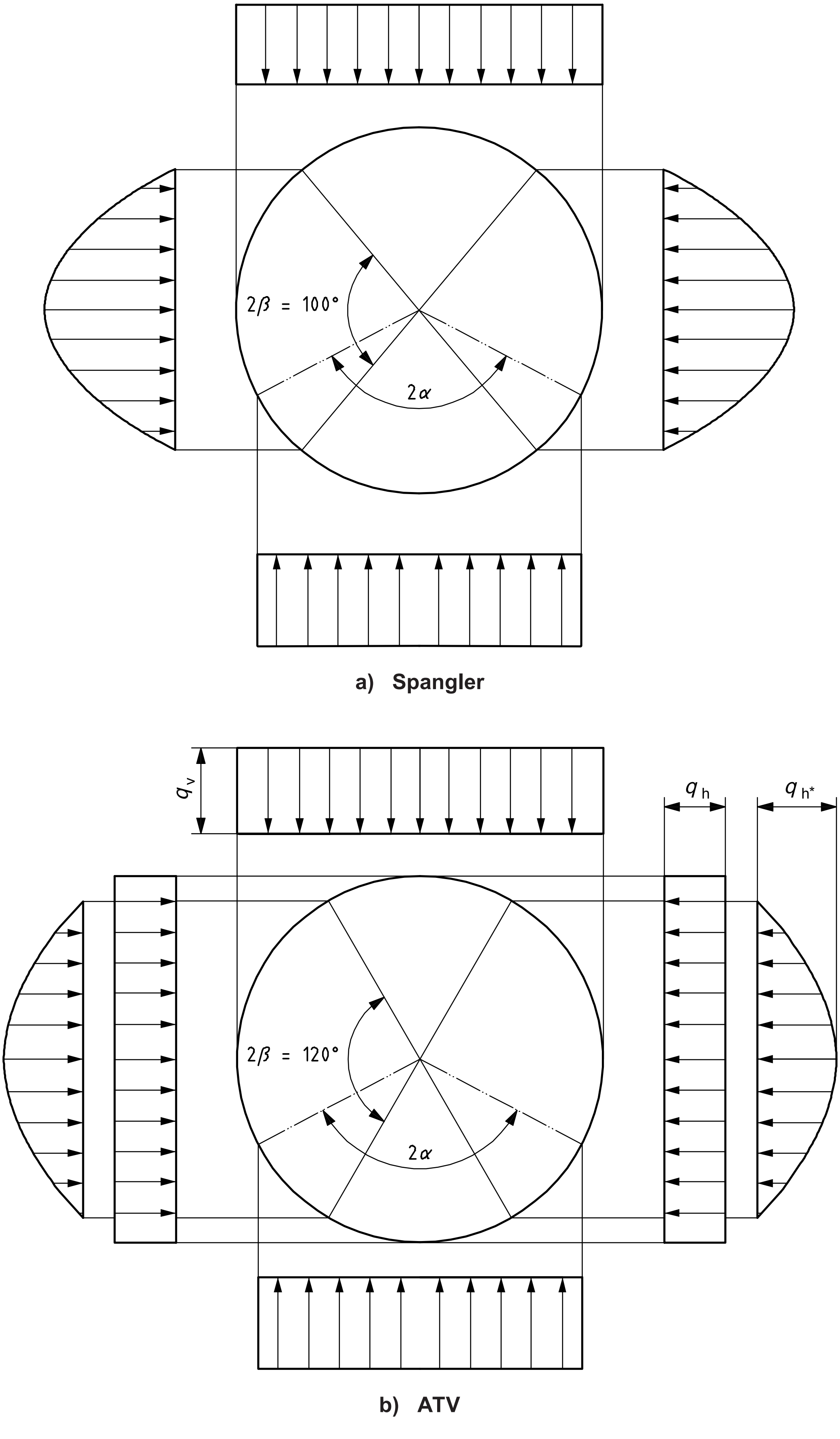


Figure 2 — Soil stress distribution according to Spangler and ATV-A 127

4.1.1 Deflection from vertical soil load and superimposed loads but excluding traffic loads

The change in vertical diameter, δ_v, as a result of external loads is determined using Equation (1).

NOTE 1 This deflection has a negative value which indicates a reduction in vertical diameter.

$$\delta_v = \frac{2 \times r_m}{8} \left[\left(C_{v,qv} \times q_v \right) + \left(C_{v,qh} \times q_h \right) + \left(C_{v,qh^*} \times q_{h^*} \right) \right] \tag{1}$$

This can be converted into relative vertical deflection, in %, δ_{vs}, using

$$\delta_{vs} = \frac{\Delta d_v}{2 \times r_m} \times 100$$

The horizontal change in diameter is determined, if necessary, using Equation (2):

$$\delta_h = \frac{2 \times r_m}{8} \left[\left(C_{h,qv} \times q_v \right) + \left(C_{h,qh} \times q_h \right) + \left(C_{h,qh^*} \times q_{h^*} \right) \right] \tag{2}$$

where

δ_{vs} is the negative relative vertical deflection from soil load;

r_m is the mean radius of the pipe wall;

$C_{v,qv}$ is the deformation coefficient for δ_v as a result of q_v;

C_{v,qh^*} is the deformation coefficient for δ_h as a result of q_h;

$c_{v,qv}$, c_{v,qh^*}, $c_{h,qv}$, c_{h,qh^*} are deformation coefficients (see Tables 1 and 2 and Annex C);

$$q_v = \lambda_{PG} \left[\left(\kappa \times \chi_S \times h \right) + \left(\kappa_o \times p_o \right) \right] \tag{3}$$

$$q_h = K_2 \times \left[\lambda_S \left(\kappa \times \chi_S \times h + \kappa_o \times p_o \right) + \left(\chi_S \times \frac{d_e}{2} \right) \right] \tag{4}$$

$$q_{h^*} = \frac{\left(C_{h,qv} \times q_v \right) + \left(C_{h,qh} \times q_h \right)}{V_{RB} - C_{h,qh}} \tag{5}$$

where

q_v is the vertical soil pressure on pipe, in N/mm^2;

q_h is the horizontal soil pressure on pipe, in N/mm^2;

q_{h^*} is the horizontal bedding reaction pressure, in N/mm^2;

λ_{PG} is the concentration factor for trench widths less than 4 d_e;

$$\lambda_{PG} = \left(\frac{\lambda_P - 1}{3} \times \frac{b}{d_e} \right) + \frac{4 - \lambda_P}{3} \tag{6}$$

NOTE Based on experience, the limits given for λ_{PG} for GRP pipes in ATV-A 127 are not normally reached.

b is the trench width, in metres;

d_e is the external diameter of pipe, in metres;

λ_P is the concentration factor for the soil above the pipe (see Annex B);

κ is the silo theory reduction factor for friction (see ISO/TR 10465-2 and Annex F);

χ_S is the bulk density of the soil (i.e. its weight per unit volume), in N/m^3;

h is the depth of cover to top of pipe, in m;

κ_o is the silo theory reduction factor for a uniformly distributed load (UDL), (see ISO 10465-2 and Annex F);

p_o is the soil pressure applied by a UDL, in N/mm^2;

K_2 is the ratio of the horizontal to the vertical pressure at the pipe spring-line in zone E_2, (see Annex A);

λ_S is the concentration factor in soil adjacent to pipe;

V_{RB} is the system stiffness calculated using Equation (7):

$$V_{RB} = \frac{8 \times S_O}{S_{Bh}} \tag{7}$$

where

S_{Bh} is the horizontal bedding stiffness that is calculated using Equation (8), in N/mm^2:

$$S_{Bh} = 0,6 \times \zeta \times E_2 \tag{8}$$

S_O is the initial pipe stiffness calculated using Equation (9), in N/m^2:

$$S_O = \frac{E_p \times I}{d_m{}^3} \tag{9}$$

E_2 is the pipe zone modulus, N/mm^2 (see Figure 1);

E_3 is the native soil modulus in zone E_3, in N/mm^2 (see Figure 1);

E_p is the apparent flexural modulus of the pipe wall, in N/mm^2;

I is the second moment of area in the longitudinal direction per unit length (of a pipe), in m^4/m;

d_m is mean pipe diameter [$\left(d_e \times 1000\right) - e$];

ζ is the correction factor for horizontal bedding stiffness, given by Equation (10):

$$\zeta = \frac{1,667}{\Delta f + \left(1,667 - \Delta f\right) \times E_2/E_3} \tag{10}$$

where

$$\Delta f = \frac{\left(\frac{b}{d_e} - 1\right)}{0,980 + 0,303\left(\frac{b}{d_e} - 1\right)} \leqslant 1,667 \tag{11}$$

NOTE The correction factor, ζ, takes into account the difference in soil modulus of the pipe embedment material, the native soil and the width of the trench. The above equations are those included in ATV-A 127 for a support angle of 120° but the variable values given in Annex D can be used for other angles. Annex D covers a wider range of support conditions than only the 120° covered by Equation (10).

The relationship between the bedding angle, 2α (see Figure 2) and the coefficients $c_{\text{v,qv}}$ and $c_{\text{h,qv}}$ is shown in Table 1.

The values of $c_{\text{v,qh*}}$, $c_{\text{v,qh}}$, $c_{\text{h,qh*}}$ and $c_{\text{h,qh}}$ for a bedding reaction angle of 120° are given in Table 2.

Table 1 — Values of $c_{\text{v,qv}}$ and $c_{\text{h,qv}}$ in relation to the bedding angle 2α

Bedding angle 2α	$c_{\text{v,qv}}$	$c_{\text{h,qv}}$
60°	–0,105 3	0,102 6
90°	–0,096 6	0,095 6
120°	–0,089 3	0,089 1
180°	–0,083 3	0,083 3

Table 2 — Values of $c_{\text{v,qh}}$, $c_{\text{v,qh*}}$, $c_{\text{h,qh}}$ and $c_{\text{h,qh*}}$ for a bedding reaction angle of 120°

Bedding reaction angle	$C_{\text{v,qh}}$	$C_{\text{v,qh*}}$	$C_{\text{h,qh}}$	$C_{\text{h,qh*}}$
120°	0,083 3	0,064 0	–0,083 3	–0,065 8

4.1.2 Deflection from pipe's own weight

ATV-A 127 does not address this matter; however, when the pipe diameter is DN 2000 or greater and the nominal stiffness of the pipe is less than SN 2000, then it is recommended that account should be taken of the relative deflection resulting from the pipe's own weight, calculated using Equation (12).

$$\delta_{\text{vw}} = -2{,}3 \times e \times \rho \times 10^{-4} \times \frac{1}{S_{\text{O}}} \tag{12}$$

NOTE This deflection has a negative value, which indicates a reduction in vertical diameter.

where

- e is the pipe wall thickness, in millimetres (mm);
- ρ is the specific weight of the pipe wall, in meganewtons per cubic metre (MN/m^3).

4.1.3 Deflection from compaction of pipe zone backfill (initial ovalization):

ATV-A 127 does not address this matter, even though it is known that, when the pipe zone backfill material is being compacted, the horizontal soil pressure generated causes the pipe to ovalize in the vertical direction. The magnitude of this relative vertical deflection can be calculated using Equation (13).

$$\delta_{\text{vio}} = K_3 \times \chi_{\text{S}} \times \frac{d_{\text{e}}}{24 \times S_{\text{O}}} \tag{13}$$

NOTE This deflection has a positive value, which indicates an increase in vertical diameter.

where

K_3 is the ratio of the horizontal to vertical soil pressure in the pipe zone backfill, when the backfill is at the top of the pipe (see Annex A).

4.1.4 Deflection from installation variability

To account for the inevitable variability in installation, there are many different approaches to allow for irregularities with regard to initial deflections. Most of these are based on “adding a few percent”. Several publications (see references [3], [6] and [7]) state that it is not possible to account for the actual measured initial deflections by traditional static calculation methods without incorporating an allowance for the influence of installation irregularities. Such a system, however, shall consider pipe stiffness, pipe diameter and soil conditions. The calculated deflection is then used to estimate the corresponding flexural strain.

Equation (14) allows an estimate of the relative deflection from installation variability to be made.

$$\delta_{\text{viv}} = \left[c_{\text{v,qv}} + \left(c_{\text{v,qh*}} \times K^* \right) \right] \times \frac{i_\text{f}}{S_\text{O}} \tag{14}$$

NOTE This relative deflection has a negative value, which indicates a reduction in vertical diameter.

Values for i_f are obtained from Table 3.

Values for $c_{\text{v,qv}}$ and $c_{\text{v,qh*}}$ are obtained from Annex C.

The coefficient for bedding reaction pressure, K^*, is calculated using Equation (15):

$$K^* = \frac{C_{\text{h,qv}}}{V_{\text{RB}} - C_{\text{h,qh*}}} \tag{15}$$

Table 3 — Values for installation factor, i_f

DN	i_f N/mm^2
⩽ 200	0,012
300	0,011
400	0,010
500	0,009
600	0,008
700	0,007
800	0,006
⩾ 900	0,005

4.1.5 Total initial relative deflection

In accordance with this part of ISO 10465, the estimated initial deflection is determined using Equation (16):

$$\left(\frac{d_\text{v}}{d_\text{m}} \right)_{\text{initial}} = (\delta_{\text{vs}} + \delta_{\text{vw}} + \delta_{\text{vio}} + \delta_{\text{viv}}) \tag{16}$$

where

δ_{vio} is the positive relative vertical deflection from backfilling in the pipe zone;

δ_{viv} is the negative relative vertical deflection from installation variability;

δ_{vs} is the negative relative vertical deflection from soil load;

δ_{vw} is the negative relative vertical deflection from pipe's own weight;

NOTE Relative deflection can be converted to percent deflection by multiplying by 100.

4.2 Long-term deflection calculated using an ATV-A 127 type design system

The calculated long-term deflection will vary according to whether the silo theory or prismatic load is used for the calculation of vertical load.

4.2.1 Residual soil friction

In ATV-A 127, the silo theory is used.

The soil pressure from the traffic load, p_V, which is transient and not sustained, shall be added to q_V but not multiplied by λ_{PG50} to obtain the long-term vertical soil pressure, $q_{v,50}$, calculated using Equation (17).

$$q_{v,50} = \left[\left(\kappa \times \chi_S \times h + \kappa_o \times p_o \right) \times \lambda_{PG50} \right] + p_V \tag{17}$$

where

p_o is the soil pressure, in N/mm^2, from a uniformly distributed load (UDL);

p_V is the soil pressure, in N/mm^2, from the traffic load.

Equations (18) and (19) are used for the calculation of the long-term horizontal soil pressure, $q_{h,50}$, and long-term relative vertical deflection, δ_{vs50}, from soil pressure.

$$q_{h,50} = K_2 \left[\left(\kappa_{50} \times \chi_s \times h \times \lambda_{PG50} \right) + \left(\kappa_{0,50} \times p_o \times \lambda_{B50} \right) + \left(\chi_s \times d_e / 2 \right) \right] \tag{18}$$

$$\delta_{vs50} = \left[c_{v,qv} + \left(c_{v,qh*} \times K^* \right) \right] \times \left(q_{v,50} - q_{h,50} \right) \times 1 / S_{OL} \tag{19}$$

λ_{PG50} Is the long-term concentration factor for trench widths less than $4 \times d_e$.

λ_{PG50} Is calculated using Equation (6) and λ_{B50} using Equation (B.6) in Annex B except that the long-term soil moduli from Annex A and long-term pipe stiffness, S_{OL}, for S_O are used.

To obtain the total long-term relative deflection, $(d_v/d_m)_{50}$, the *initial* deflections from the pipe's own weight, initial ovalization and installation irregularities should be added to the long-term deflection, δ_{v50}, as shown in Equation (20):

$$(d_v/d_m)_{50} = \delta_{v50} + \delta_{vw} + \delta_{vio} + \delta_{viv} \tag{20}$$

NOTE For sign convention, see 4.1.5.

4.2.2 Long-term prismatic soil load (i.e. no soil friction)

In the ATV system, silo theory is used and it is assumed that the reduction of soil load exists for the installed lifetime of the pipe. If, however, the use of prismatic loading is required, which ignores any soil friction, then the long-term deflection is obtained by setting $\lambda_{PG50} = \lambda_{B50} = \kappa = \kappa_o = 1$ in Equations (17) to (20).

4.2.3 Partly residual soil friction

There is a large difference in result depending upon whether silo theory or prismatic soil load is used for long-term deflection calculations. This effect becomes more pronounced as the depth of cover increases. In order to handle this, the so-called “environmental depth of cover”, H_{EDV}, has been introduced in this part of ISO 10465. This depth is defined as the depth down to which the soil friction has been lost due to frost, rain, traffic loads, and can be up to 3 m. Full prismatic load is used for this part of the depth of cover, and silo theory for the rest.

Annex F describes how values for $q_{v,50}$ and $q_{h,50}$ can be calculated using this system. The new values, called q_{vLT} and q_{hLT}, are then used in Equation (19) instead of $q_{v,50}$ and $q_{h,50}$, respectively.

5 Soil parameters, strain coefficients and shape factors for flexural strain calculations

5.1 For equations used in ATV-A 127 type design systems

In ATV-A 127, the absolute value of relative deflection is taken. In this part of ISO 10465, unlike ATV-A 127, the strains are calculated at one position, namely the invert. A positive value indicates tensile strain and a negative value compressive strain.

5.1.1 Flexural strain from the vertical soil load

The flexural strain from the vertical soil load, ε_v, can be calculated using Equation (21):

$$\varepsilon_v = \{(m_{qv} \times q_v) + (m_{qh} \times q_h) + [m_{qh^*} \times K^* \times (q_v - q_h)]\} \frac{e}{d_m \times S_O} \tag{21}$$

NOTE 1 This flexural strain has a positive value, which indicates a tensile strain resulting from a reduction in vertical diameter.

where

values from ATV-A 127 for the bending moment coefficients, m_{qv}; m_{qh}; m_{qh^*}, are given in Annex C;

$$K^* = \frac{c_{h,qv}}{V_{RB} - c_{h,qh^*}} \tag{22}$$

$c_{h,qv}$ and c_{h,qh^*} are horizontal deflection coefficients (see Annex C);

NOTE 2 The values of these coefficients depend upon the choice made for the appropriate vertical soil reaction angle, 2α, or horizontal soil reaction angle, 2β (see Figure 1). The angle selected for 2α and 2β has much more effect on the calculated flexural strain than on the calculated deflection (see 5.2).

q_v and q_h are calculated as described in Clause 4, for initial or long-term deflection.

When calculating the long-term deflections, see Annex E regarding $S_{O,50}$.

5.1.2 Flexural strain from pipe's own weight:

When the pipe diameter is DN 2000 or greater and the nominal stiffness of the pipe is less than SN 2000, then account should be taken of the deflection resulting from the pipe's own weight.

The flexural strain due to the pipe's weight is calculated using Equation (23):

$$\varepsilon_{\mathrm{vw}} = 0{,}000\ 441 \times \chi_{\mathrm{P}} \times \frac{e^2}{d_{\mathrm{m}}} \times \frac{1}{S_{\mathrm{O}}} \tag{23}$$

where

χ_{P} is the specific weight of the pipe wall material, in meganewtons per cubic metre (MN/m^3).

NOTE This flexural strain has a positive value, which indicates a tensile strain resulting from a reduction in vertical diameter.

5.1.3 Flexural strain from initial ovalization

The flexural strain due to the initial ovalization of the pipe during compaction of the backfill in the pipe zone is calculated using Equation (24), which is based upon Equation (13).

$$\varepsilon_{\mathrm{vio}} = -0{,}025 \times K_3 \times \chi_{\mathrm{s}} \times \frac{d_{\mathrm{e}} \times 10^{-3}}{2} \times \frac{e}{d_{\mathrm{m}} \times S_{\mathrm{O}}} \tag{24}$$

NOTE This flexural strain has a negative value, which indicates a compressive strain resulting from an increase in vertical diameter.

5.1.4 Compressive strain from vertical load

The compressive strain, $\varepsilon_{\mathrm{comp}}$, from vertical loads is calculated using Equation (25) which is a simplification of the equations used in ATV-A 127 for estimating the strain at the bottom of the pipe.

$$\varepsilon_{\mathrm{comp}} = -q_{\mathrm{h}} \times \frac{3 \times Y^2}{S_{\mathrm{O}}} - 0{,}577 \times q_{\mathrm{h}^*} \times \frac{3 \times Y^2}{S_{\mathrm{O}}} \tag{25}$$

NOTE This compressive strain has a negative value.

where

$$Y = \frac{e}{d_{\mathrm{m}}} \tag{26}$$

$$q_{\mathrm{h}^*} = K^* \times (q_{\mathrm{v}} - q_{\mathrm{h}}) \tag{27}$$

for K^* see Equation (22).

5.1.5 Flexural strain from pipe contents

In the ATV-A 127 system, it is assumed that the pipe is filled with water before backfilling. This produces a very high calculated flexural strain for GRP pipes, especially for pipes having a low nominal stiffness and large nominal diameter. As it is obvious that the pipe, when full, receives support from the backfill, it is felt that this design approach is unnecessarily pessimistic and unrealistic.

Leonhardt in a private communication has recommended that Equation (28) be used instead of the ATV equation.

$$\delta_W = q_{hw*} \times \frac{e}{d_m \times S_O} \times \left[m_{qv} + \left(m_{qh} \times K^* \right) \right] \tag{28}$$

NOTE This flexural strain has a positive value, which indicates a tensile strain resulting from a reduction in vertical diameter.

where

$$q_{hw*} = q_{vwa} \times K^*; \tag{29}$$

$$q_{vwa} = 0{,}5 \times \chi_w \times \pi \times r_i \tag{30}$$

for K^* see Equation (22);

χ_w is the specific weight of the pipe's contents, in meganewtons per cubic metre (MN/m^3).

5.1.6 Flexural strain from installation irregularities

The flexural strain due to installation irregularities is calculated using Equation (31):

$$\varepsilon_{if} = i_f \times (0{,}25 + m_{qh*} \times K^*) \frac{e}{d_m \times S_O} \tag{31}$$

NOTE This flexural strain has a positive value, which indicates a tensile strain resulting from a reduction in vertical diameter.

For i_f see 4.1.4;

for m_{qh*} see Annex C;

for K^* see Equation (22).

5.1.7 Total flexural strain

The total flexural strain, in percent, is the sum of the strains calculated in 5.1 using Equation (32):

$$\varepsilon_{tot} = (\varepsilon_v + \varepsilon_{vw} + \varepsilon_{vio} + \varepsilon_{comp} + \varepsilon_w + \varepsilon_{if}) \times 100 \tag{32}$$

NOTE Take care to ensure the correct signs are used for the different strains.

5.2 Shape factor, D_f

When buried flexible pipes deflect, they deform into non-elliptical shapes. To allow for this in calculations performed to determine the strain, a shape factor, D_f, is used.

Some design systems, such as AWWA M-45 and the WRC [10], method, specify values for D_f. The ATV-A 127 system does not use D_f values but derives pipe strains based upon values of 2α and 2β. In order to allow the design engineer to compare different systems, it is possible to derive values of D_f using equations from ATV-A 127. This permits examination of the effects of parameters such as pipe stiffness, pipe deflection and bedding angle upon the value of D_f.

5.2.1 Derivation of D_f using ATV-A 127 equations

Although D_f is not used in ATV-A 127, values for D_f can be calculated using Equation (33):

$$\text{Strain} \times \frac{d_m}{e} \times \frac{1}{\text{deflection}} = D_f \tag{33}$$

Using this procedure, D_f has been calculated across a range of bedding angles, 2α, and support angles, 2β, for a series of deflections up to 12 % and covering pipe stiffnesses from SN 1000 to SN 8000.

Analysis of the data from these calculations showed that, for any given pipe stiffness and bedding angle, 2α, and support angle, 2β, the value of D_f could be calculated from an equation of the form:

D_f = (A + B) /% deflection

where

A is a constant between 2,84 and 2,9;

B is a constant related to the pipe stiffness and the bedding and support angles, 2α and 2β.

This equation was used to prepare Table 4.

D_f values can also be derived from measurements made on pipes installed in the ground or in soil boxes. The strain is either measured using strain gauges or calculated using the measured radius of curvature, pipe deflection, wall thickness and diameter. The value for D_f can then be derived using Equation (33) as described above (see also reference [10]).

5.2.1.1 Selection of 2α and 2β values in ATV type systems

From measurements made on pipes in soil boxes, where the pipe zone material has been placed carefully either without compaction or only slightly compacted, the calculated value of D_f indicates that 2α and 2β are equal to 180° (see references [1] and [2]). Similar calculations made using the results from measurements taken on pipes installed in the ground in narrow trenches, subject to typical varying degrees of compaction, gave calculated D_f values indicating that 2α and 2β were between 120° and 150°. From the results of this experimental work, it can be concluded that it is safe to use a value of 150° for 2α and 2β with the expectation that this will cover between 85 % and 90 % of all installation cases.

Table 4 — Calculated D_f values

Pipe nominal stiffness	$2\alpha = 2\beta = 150°$	
	d_v/d_m = 3 %	d_v/d_m = 5 %
SN 1000	7,95	5,99
SN 2000	5,33	4,32
SN 4000	3,98	3,55
SN 8000	3,31	3,18

NOTE Similar calculations can be done for the other stiffness series, namely SN 1250, SN 2500, SN 5000 and SN 10000, and values for D_f very similar to those in Table 4 will be derived. For most purposes, the SN values in Table 4 and in the applicable tables in Annex G can be interchanged, i.e. SN 1250 for SN 1000, SN 2500 for 2000, SN 5000 for SN 4000 and SN 10000 for SN 8000, without significantly affecting the accuracy of the results.

5.2.2 D_f values in AWWA design manual M-45

Table 5 lists the D_f values contained in AWWA design manual M-45, which it states are "valid for deflections of at least 2 % to 3 % assuming inconsistent haunching and stable native soils or where adjustments are made to the width of the trench to offset poor conditions".

A comparison of the M-45 values in Table 5 and the calculated values given in Table 4 shows a similarity in values, especially when pipes are embedded in either dumped or slightly compacted backfill. The differences in nominal stiffness values in Tables 4 and 5 do not, for the purposes of this comparison, significantly affect the D_f values.

Table 5 — D_f values versus pipe zone backfill material and its compaction, from AWWA M-45

Pipe nominal stiffness SN	Gravel		Sand	
	Degree of compaction			
	Dumped to slight	Moderate to slight	Dumped to slight	Moderate to high
1 250	5,5	7,0	6,0	8,0
2 500	4,5	5,5	5,0	6,5
5 000	3,8	4,5	4,0	5,5
10 000	3,3	3,8	3,5	4,5

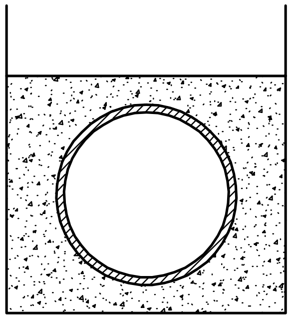
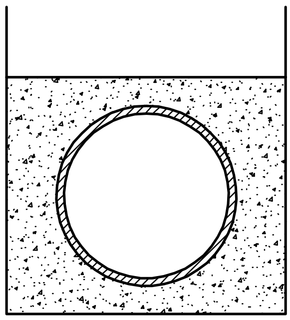
5.2.3 D_f values used in the *Pipe materials selection manual* [10]:

D_f values recommended for use in the WRC design system are shown in Tables 6 and 7.

Table 6 — WRC_f values for various installation conditions and pipe stiffness

Embedment class (see Table 7)	Degree of compaction % MPD	D_f for various nominal pipe stiffnesses 1 250	2 500	5 000	10 000
	Uncompacted	4,7	4,5	4,3	4,0
Class	**80**	4,7	4,5	4,3	4,0
S1	**85**	4,7	4,5	4,3	4,0
	90	4,7	4,5	4,3	4,0
	Uncompacted	4,7	4,5	4,3	4,0
Class	**80**	4,7	4,5	4,3	4,0
S2	**85**	4,7	4,5	4,3	4,0
	90	4,7	4,5	4,3	4,0
Class	**80**	3,50	3,40	3,20	3,10
S3	**85**	6,20	5,50	4,75	4,25
	90	7,75	6,60	5,50	4,70
Class	**85**	6,20	5,50	4,75	4,25
S4	**90**	7,75	6,60	5,50	4,70

Table 7 — WRC embedment class details

Class	Configuration	Bedding and sidefill materials	Notes
S1 and S2		Class S1: Gravel, single-sized Class S2: Gravel, graded	Normally processed granular materials.
S3 and S4		Class S3: Sand and coarse-grained soil with more than 12 % fines Class S4: Coarse-grained soil with more than 12 % fines or Fine-grained soil, with liquid limit less than 50 %, medium to no plasticity and more than 25 % coarse-grained material	These represent “as dug” soils and require particularly close control when used with low-stiffness pipes

6 Influence of soil moduli and pipe stiffness on pipe buckling calculations using ATV-A 127 type design systems

The buckling calculation method used in the ATV-A 127 system does not consider the variation of the soil modulus with depth of cover unlike the buckling calculation method used in the AWWA M-45 design manual. For depths of cover over 1 m, the influence of cover depth can be incorporated into the ATV-A 127 system by the following adaptations.

6.1 Elastic buckling under internal negative pressure for depths of cover over 1 m

Equation (34), taken from ATV-A 127 (Equation 9.06a), calculates the critical pressure from vacuum conditions:

$$\text{critical } q_{\text{v}} = \kappa_{\text{v2}} \times 2\left(8 \times S_{\text{O}} \times S_{\text{Bh}}\right)^{0,5} \qquad (34)$$

where κ_{v2} is a reduction factor to take into account the elastic-plastic soil mass law and preliminary deflections.

In order to use the soil modulus values which are related to the depth of cover (see Annex A) for this type of buckling, it is recommended that the modified form of the equation [Equation (35)] be used.

$$q_{\text{c}} = \kappa_{\text{v2}} \times 2\left(E'_{\text{s}} \times 8 \times S_{\text{O}}\right)^{0,5} \qquad (35)$$

where

$$E'_s = M_s \times \frac{(1+\nu_s)\times(1-2\nu_s)}{(1-\nu_s)} \tag{36}$$

M_s is the constrained-soil modulus (see Annex A) of the pipe zone backfill.

ν_s is the Poisson ratio of the pipe zone backfill (see Annex A).

6.2 Long-term buckling under sustained external load

When calculating for elastic buckling under the effects of sustained external loads it should be remembered that the influence of the native soil will be activated due to the increase of horizontal diameter. This will either reduce the risk of buckling if $E_3 > E_2$ or increase it if $E_3 < E_2$. For either of these conditions, the use of Equation (37) is recommended.

$$q_{cl} = 2(E'_S \times \zeta \times S_R)^{0,5} \tag{37}$$

where ζ is the correction factor (see 4.1.1 and Annex D).

6.3 Value for S_O

For calculations of the long-term buckling resistance to either sustained load or negative pressure, the pipe's long-term elastic stiffness, $S_{O,50}$, should be used (see Annex E).

7 Parameters for rerounding and combined loading calculations

7.1 Rerounding

To allow for the fact that GRP pipes, when subject to internal pressure, will attempt to reround, and hence have a reduction in deflection, Equation (38) has been developed from soil box tests, field measurements and literature studies.

The rerounding factor, r, may be obtained using Equation (38):

$$r = 1 - \frac{P}{30} \tag{38}$$

where P is the internal pressure, in bars.

NOTE 1 ATV-A 127 does not address this matter as it was intended principally for non-pressure systems, so Equation (38), which is used in AWWA M-45, has been included here to correct this omission.

NOTE 2 The *Pipe materials selection manual* [10] uses 40 in place of 30 in Equation (38).

7.2 Combined effects of internal pressure and external bending loads

In the AWWA design manual M-45, the combined effect of internal pressure and external load is calculated by using either strain alone or stress alone.

In ATV-A 127, rerounding is not considered and the calculation is made on the assumption that flexural and tensile strengths are equal and the stresses are “additive”.

8 Traffic loads

8.1 General

The design requirements relating to traffic wheel-loading vary from one country to another, but the basic principles used result in the assumption that there is a pressure load applied to the pipe at its crown. The value of this load is related to the applied wheel load and the depth of burial. Basically, all design systems used for normal installations can be related to the Boussinesq work, which considers this dynamic load as a vertically applied force without a related horizontal component (such as is considered with static loading).

In addition to the different traffic load design systems that exist, there are different requirements specified by road construction authorities for the allowable pavement deflections permitted above buried pipeline installations (such deflections being not only dependent on the pipe movement, but also on the characteristics of the road base and surfacing material). For calculations relating to pipelines under highways, the specifier should consider using higher values for E_2 because high levels of compaction are likely to be employed.

8.2 Influence on allowable initial deflection

The allowable initial deflection only considers soil and superimposed loads, not traffic loads. Hence traffic loads have no effect on allowable initial deflection.

8.3 Soil pressure from traffic loads

The design requirements relating to traffic wheel-loading vary from one country to another; the relevant national codes should be used for design.

9 Influence of sheeting

Installation of pipes in deep trenches or in weak soils frequently necessitates the use of shields, sheeting or trench wall supports to prevent the collapse of the trench walls and to protect the workers. The types of method used vary according to the soil conditions, the equipment used and the experience of the workers carrying out the installation.

In some instances, shields are moved laterally as the installation progresses, and, in other situations, sections of the shield are extracted vertically. Also, in some installations the shields are left in place.

It is essential, when designing the installation, that the engineer be aware of the intended method of shield extraction so that the correct assumption for the values of soil compaction and hence modulus can be made in the calculation. ATV-A 127 takes this into account in its embedment classes.

There are four embedment conditions, labelled B1 to B4.

B1 The embedment material is compacted in layers against the native soil or in the embanked covering (without verification of the degree of compaction). This condition also applies to beam pile walls.

B2 Vertical shuttering, using trench sheeting in the pipe zone, which reaches the bottom of the trench and is withdrawn after backfilling.

Shuttering plates or equipment assumes that the compaction of the soil takes place after the trench sheeting is withdrawn.

B3 Vertical shuttering using sheet piling or light-weight piling profiles in the pipe zone. The shuttering reaches down below the bottom of the trench, and the embedment is compacted against the sheeting.

B4 Embedment material compacted in layers against the native soil or in the embanked covering (with verification of the degree of compaction). Covering condition A4 (see ATV-A 127) is not applicable when using soil of Group 4.

Control of the placing of the backfill and its compaction around the pipe can best be achieved by progressive vertical extraction of the shields while placing the backfill. The shields should be raised in increments of approximately 300 mm, and the backfill should be placed and compacted progressively, ensuring that it is firmly placed below the bottom edge of the shield and compacted to the required value. This installation operation should be continued until the lower edges of the shields are above the specified compaction zone.

If shields are being moved laterally, the extent of the incremental movement shall be such that there is no danger of the trench wall collapsing, and care shall be taken to check that disturbance of the placed and compacted backfill does not occur when the next incremental move is made. Where thick shields are used, there is a risk that the compacted backfill is supported by the rear edge of the shield, and there may be a loss of support as it is moved forward.

Added complications can arise if a geotextile fabric is specified. It is recommended that the designer specify the installation technique to be followed in order to ensure correct placing of the fabric at the bottom and sides of the trench.

When designing the pipe installation and selecting the pipe stiffness for installations requiring shields, the designer should assume that a lower figure for the compaction of the backfill will be obtained.

10 Safety factors for gravity pipes and pressure pipes

The AWWA M-45 design manual addresses the subject of design safety factors for both pressure and non-pressure applications as well as combined stress or strain conditions resulting from buried pressurized situations. The ATV-A 127 system, being primarily concerned with gravity conditions, only considers flexural safety.

The AWWA M-45 system is based upon long-term mean values of stress and strain, whilst the ISO system standards are based upon minimum long-term lower confidence limit (LCL) values combined with probabilistic failure considerations. These differences of concept lead to differences in the safety factor values used in the different documents, particularly with respect to combined loading conditions.

10.1 Gravity pipes

10.1.1 Safety factors for GRP gravity pipes given in ATV-A 127

ATV-A 127 recommends the following minimum safety factors for GRP pipes:

a) Against failure due to fracture:

- 2,0 for a probability of failure of 10^{-5}, i.e. 1 in 100 000 (normal case);
- 1,75 for a probability of failure of 10^{-3}, i.e. 1 in 1 000 (special case).

b) Against failure due to instability:

- 2,0 for a probability of failure of 10^{-5}, i.e. 1 in 100 000 (normal case);
- 1,6 for a probability of failure of 10^{-3}, i.e. 1 in 1 000 (special case)

These factors are based on the assumption that the initial deformation has been taken into account. If it has not, then the factors should be increased to 2,5 and 2,0 respectively.

The normal case covers the following situations:

- there is a risk of groundwater;
- there is a risk of loss of the pipeline;
- failure has significant economic consequences.

The special case covers the following situations:

- there is a no risk of groundwater;
- there is a small risk of loss of the pipeline;
- failure has slight economic consequences.

The flexural safety factors are not based upon the manufacturers declared long-term ultimate ring deflection, but are related to the specified long-term ultimate ring deflection in DIN 19565-1, which assumes that these values are *mean values*. If instead the safety factor were based upon an LCL value, such as is used in the ISO product system standard, then a lower safety factor could be used and still give the same level of risk against failure.

There are also other reasons for the safety factors in ATV being high, namely:

the semi-probabilistic calculation which was made to arrive at these values used a bedding angle $2\alpha = 90°$, which is extremely conservative (see 5.2.1);

the ATV equations were modified to include an “uncertainty factor”, which increased the safety factor value from for instance 1,75 to 2,0.

statistical data used came from *three* different manufacturing units which were also in different countries.

10.1.2 AWWA M-45 design manual safety factors for gravity pipes

The AWWA design manual M-45 specifies a 1,5 safety factor on the manufacturer's stated 50-year long-term ring-bending strain (S_b) value. This applies to the bending condition for pressure pipes and would also be considered for non-pressure or gravity pipes used for water supply. For drains and sewers, the long-term consideration is acid strain corrosion, and the application of the 1,5 factor should then be to the 50-year strain corrosion value. The 50 year values are mean values derived from tests performed according to ASTM standard test methods. In AWWA design manual M-45, the specified *initial* value for the safety factor against instability is 2,5.

10.1.3 Recommendations

The minimum requirements specified in the ISO system standards should be used for the safety calculations unless the pipe manufacturer can demonstrate, using long-term testing, that the product's long-term ultimate ring deflection can be expected to be greater than those minimum requirements.

The long-term ultimate ring deflection values are converted into strain, as a percent, using Equation (39):

$$\varepsilon_{50} = D_g \times \frac{e}{d_m} \times \left(\frac{d_v}{d_m}\right)_{ult} \tag{39}$$

where

$\left(\frac{d_v}{d_m}\right)_{ult}$ is the ultimate relative vertical deflection, expressed as a percent:

$$D_g = \frac{4{,}28 \times 100}{\left[1 + \left(\frac{d_v}{2 \times d_m}\right)_{ult}\right]^2} \tag{40}$$

The safety factor in flexure, η_f, is then calculated using Equation (41):

$$\eta_f = \frac{\varepsilon_{50}}{\varepsilon_{tot}} \tag{41}$$

where

ε_{50} is the long-term ultimate ring-bending strain or the strain from long-term strain corrosion tests;

ε_{tot} is the total long-term ring-bending strain from pipe deflection;

η_f should be at least 1,5.

10.2 Pressure pipes

The principle of designing buried pressure pipes to withstand the combined effects of tensile hoop stresses from internal pressure and the bending stresses from earth loading, superimposed loads and traffic loads is well established and covered in AWWA M-45. As ATV-A 127 is not concerned with pressure pipe, consideration is not given to this subject in that document.

ISO system standards are for both pressure and gravity pipe, but as the installation design principles relating to combined loading are not contained within that document and the pipe design assumptions are based upon minimum long-term values and not mean values (as used in AWWA M-45), it is appropriate that the combined loading design principles should be considered. The system standards specify a minimum long-term failure pressure safety factor value of 1,3 × PN.

Consideration has been given to the relationships between probability of failure, the variability of the product as manufactured, the long-term tensile and flexural safety factors used in the design, and the combined tensile and flexural safety factors, η_{hat} and η_{haf}, derived from consideration of the effects of combined stress.

The following subclauses define the combined safety factors and give guidance to the manufacturer and designer concerning the minimum safety factors which should be used in design and achieved in production, having due regard to the manufacturing process variability.

Details concerning the derivation of formulae and safety factor recommendations are contained in Annex G.

10.2.1 Combined loading in AWWA M-45

In AWWA M-45 there are two combined factors of safety, FS_{pr} and FS_b, where FS_{pr} corresponds to the global factor of safety in tension, η_{hat}, (see 10.2.2), and they have minimum values of 1,8 and 1,5 respectively. For buried pipes, they are used in Equations (42) and (43), and in Equation (44) for non-buried pipes.

$$\frac{\varepsilon_{pr}}{HDB} \leqslant \frac{1 - \left(\dfrac{\varepsilon_b \, r_c}{S_b} \right)}{FS_{pr}} \tag{42}$$

$$\frac{\varepsilon_b \, r_c}{S_b} \leqslant \frac{1 - \left(\dfrac{\varepsilon_{pr}}{HDB} \right)}{FS_b} \tag{43}$$

$$\frac{\varepsilon_{pr}}{HDB} \leqslant \frac{1}{FS_{pr}} \tag{44}$$

where

ε_{pr} is the working strain due to internal pressure;

ε_b is the bending strain due to maximum permitted deflection;

r_c is the rerounding coefficient;

S_b is the long-term ring-bending strain capability of the pipe;

HDB is the hydrostatic design basis based on strain.

In Annex G there is no factor equivalent to FS_b for the following two reasons:

this factor has little influence on pipes used today for buried pipes;

to obtain values for η_{hat}, probabilistic principles are employed which, for the reasons given in Clause G.2, cannot be used for the flexural property.

10.2.2 Recommended combined safety factors for use in ISO product standards

Combined safety factors for the tensile and flexural properties which consider the interaction of these two properties on each other have been developed from concepts used in AWWA M-45. This combined interaction is calculated using Equation (45).

$$\eta_{hat} = \eta_t \times \left(1 - \frac{1}{\eta_{ff}}\right) \tag{45}$$

If the factor of safety, $\eta_{t,PN}$ is defined relative to the nominal pressure class of the pipe, PN, then Equation (46) should be used.

$$\eta_{hat} = \eta_{t,PN} \times \left(1 - \frac{1}{\eta_{ff}}\right) \times \frac{PN}{p} \tag{46}$$

where

η_{hat} is the global factor of safety for tension;

$$\eta_{ff} = \frac{\eta_f}{r} \tag{47}$$

η_f is the factor of safety in flexure [see Equation (41)];

η_t is the factor of safety in hoop tension;

r is the rerounding factor (see 7.1);

PN and p are expressed in bars.

The required global minimum long-term safety factor in tension, η_{hat}, can be derived using Equation (48) from knowledge of the product's variability.

$$\eta_{hat} = \frac{1}{1 - X \times y \times 0{,}01} \geqslant 1{,}5 \tag{48}$$

where

X is the safety index (see Table 8);

y is the coefficient of variation, as a percentage of the tensile strength (derived from factory production records);

$$y = \frac{\text{standard deviation initial tensile strength}}{\text{mean initial tensile strength}} \times 100.$$

This assumes that the long-term coefficient of variation is the same as the initial.

Table 8 — Safety index, X

P_f	10^{-2}	10^{-3}	10^{-4}	10^{-5}
X	2,32	3,09	3,72	4,26

If the probability of failure, P_f, is assumed to be 10^{-4}, then, from Table 8, $X = 3,72$. It can be seen from Equation (48) that if the minimum global factor of safety η_{hat} is 1,5, then y cannot be greater than 9 %. If y is greater than 9 %, then the factor of safety, η_{hat}, will have to be greater than 1,5.

Safety factors in probabilistic calculations are applied to *mean* values. If it is required to apply the factor of safety to a lower confidence limit rather than a mean value, then the notation should be changed, for instance from $\eta_{t,PN}$ to $\eta_{t,PN,97,5}$, to denote a lower confidence limit of 97,5 %.

The following recommendations are made.

a) The value to use for P_f may be found in national regulations or, if this is not the case, it is recommended that a value of 10^{-4} (i.e. a probability of failure of 1 in 10 000) be used.

b) The minimum value for the global factor of safety, η_{hat}, should not be less than 1,5.

10.3 Safety factors in buckling calculations

10.3.1 Specified safety factors in AWWA M-45

In AWWA design manual M-45, the *initial* safety factor is used and a value of 2,5 is specified.

10.3.2 Specified safety factors in ATV-A 127

In ATV-A 127, the following excessive *long-term* safety factors are specified, possibly for the same reasons as given in 10.1.1:

— For $P_f = 10^{-3}$: 2,0

— For $P_f = 10^{-5}$: 2,5

It is recommended, when designing for buckling using equations according to 6.1, to use a single long-term safety factor of 2, which will give a result similar to the AWWA document, assuming an ageing factor of 0,7.

Annex A
(informative)

Soil parameters

A.1 General

ATV-A 127 type design systems calculate not only the vertical but also the horizontal pipe deflections (see reference [8]). AWWA M-45, which is based on the Watson-Spangler Iowa formula, only addresses vertical loads and deflections.

Various investigations have shown that, with adjustments to both design methods, it would be possible to use the same soil moduli in both and obtain calculated vertical deflections that are very similar. To avoid the current inevitable confusion arising from the different values used in the two methods, it would be very desirable for these values to be standardized and based on laboratory-derived values. At the moment this is not likely to occur due to the basic differences in the design methods.

The following clauses detail soil information used in ATV-A 127 and AWWA type design systems. The apparent similarities in some areas may form the basis of agreements leading to a common soil modulus specification in the future (see references [1], [2], [6], [7] and [8]).

A.2 Value for Poisson ratio, ν_s

The Poisson ratio for a soil can be determined using Equation (A.1):

$$\nu_s = \frac{1 - \sin\varphi}{2 - \sin\varphi} \qquad \text{(A.1)}$$

where φ is the internal friction angle of the soil.

A.3 Factors for use in ATV-A 127 type calculation systems

A.3.1 Values for K_1, K_2 and K_3

a) K_1

In ATV-A 127, irrespective of soil type, K_1 is always equal to 0,5. It is recommended that this value and concept remain unchanged.

b) K_2

Soil box tests and site measurements have shown that a fixed value of 0,4 could be used for all cases. It is recommended that a value of 0,4 be used for K_2.

c) K_3

The value to use for K_3, which is the relationship between the horizontal and vertical soil pressures in the pipe zone backfill when the backfill is at the top of the pipe, is obtained from Equation (A.2). This equation, which is used in initial ovalization calculations, has been derived from field measurements and soil box testing.

$$K_3 = \frac{K_2}{8} + \frac{20}{d_e \times 1000} \times F \tag{A.2}$$

where

d_e is between 0,1 m and 0,9 m inclusive;

F is the applicable compaction factor according to Table A.1.

NOTE For pipe sizes greater than DN 900, use 0,9 m for d_e in Equation (A.2).

Table A.1 — Values for compaction factor, F

Compaction	F
Heavy	3,0
Moderate	2,0
Slight	1,5
Dumped	1,0

A.3.2 Values for δ and φ

In ATV-A 127, the value for the trench wall friction angle, δ, depends on factors such as the soil's internal friction angle, and can be simplified to be taken normally as 0,66 × φ. The values for φ can be taken from Table A.2.

Table A.2 — Values for soil internal friction angle, φ

Soil group as in ATV-A 127	φ degrees
G1	35
G2	30
G3	25
G4	20

A.4 Soil moduli

A.4.1 General

From an inspection of the literature, there is a diversity of opinion on the question of soil moduli values. This can be seen from the following summary.

According to Watkins, E' is a semi-empirical constant and cannot be obtained from laboratory testing.

Molin (see Reference [19]) has stated that the soil modulus for deflection calculations can be obtained from laboratory testing and introduces the term secant modulus, E'_s [see Equation (36)]. When deflection calculations are made using the Iowa equation with E' equal to 2 × E'_s, the same answer is achieved as when using Molin's deflection equation with secant modulus, E'_s.

$$E'_S = M_S \times \frac{(1+\nu_S)\times(1-2\nu_S)}{(1-\nu_S)} \tag{36}$$

Hartley-Duncan (see Reference [15]) and others have suggested that E' should be equal to the constrained-soil modulus, M_S, which can be obtained from laboratory testing.

A.4.2 Values for use in ATV-A 127 type design systems

Tests (see reference [2]) were made using a sand backfill, and measurements were taken of the pipe deflection and strain at various soil pressures corresponding to depths of cover between 1 m and 10 m. The constrained-soil modulus of the sand was determined in a laboratory for the same depths of cover. When these values were used together with K_2 = 0,4 and $2\alpha = 2\beta$ = 180°, very good agreement was obtained between the measured and calculated deflections. When, however, clay was used in the tests, it was found that K_2 had to be set at 0,5 to obtain a correlation. It is concluded that:

it is very important to use a soil modulus which can be measured *in situ* or on samples in the laboratory;

it is also very important to use a soil modulus which has been related to soil pressure or depth of cover.

It is recommended that, when performing ATV-A 127 type calculation systems in accordance with Clauses 4 and 6 of this part of ISO 10465, the soil modulus values given in Austrian Standard ON B 5012 (see Reference [9]) are used because they have been obtained in a laboratory from tests for the constrained-soil modulus, M_S, at 1 m depth of cover. This satisfies the two conclusions given earlier.

A.4.2.1 Backfill soil modulus

A.4.2.1.1 Austrian standard ON B 5012

A.4.2.1.1.1 Soil group definitions

The four soil groups in Austrian Standard ON B 5012, which may be considered for backfill, are defined as follows:

a) Soil group 1

— Gravel, gravel-sand mixtures (GW, GP);

— Sand, sand-gravel mixtures (SW, SP) having at least 40 % of particles larger than 2,0 mm and a maximum of 5 % silt.

b) Soil group 2

— Gravel-silt mixtures (GM) having maximum 15 % silt;

— Gravel-clay mixtures (GC) having maximum 15 % clay;

— Sand-silt mixtures (SM) having maximum 15 % silt.

— Sand-clay mixtures (SC) having maximum 15 % clay and less than 40 % of particles larger than 2,0 mm.

c) Soil group 3

— As group 2 (GM, GC, SM, SC), but the amount of silt or clay can be up to 40 %.

d) Soil group 4

— Silty or clayey soils (ML, CL, MH, CH) from low to high plasticity and a content of fine-grain materials above 40 %.

NOTE The letters in brackets are the group symbols used in the unified soil classification system.

A.4.2.1.1.2 Soil group moduli

Based on the soil groups defined in A.4.2.1.1.1, the Austrian standard gives values for constrained-soil moduli that depend upon the degree of compaction, as shown in Table A.3, and are considered valid when using a vertical soil pressure equivalent to 1 m depth of cover.

Table A.3 — Backfill constrained-soil moduli, M_s, for the various soil groups at 1 m depth

Soil group	Compaction (based on simple proctor) D_{pr} %						
	85	90	92	95	97	100	102
	Soil modulus M_s N/m^2						
1	3,8	5,3	6,0	7,2	8,2	10,0	11,4
2	2,1	2,9	3,3	4,0	4,5	5,0	6,3
3	1,3	1,8	2,1	2,5	2,9	3,5	4,0
4	0,9	1,2	1,4	1,7	1,9	2,3	2,6

For depths of cover other than 1 m (without ground water for soil groups 1 and 2), the constrained-soil modulus can be estimated using Equation (A.3):

$$M_s = M_{s1} \times (\kappa \times h)^f \qquad \text{(A.3)}$$

where

M_{s1} is the value from Table A.3;

f is 0,4.

When ground water is present, the constrained-soil modulus for groups 1 and 2 is estimated using Equation (A.4):

$$M_s = M_{s1} \times \left[h \times \kappa \times \left(1 - 0{,}39 \times \frac{h_w}{h} \right) \right]^f \qquad \text{(A.4)}$$

where

f is 0,4;

κ is the reduction factor (silo theory);

h is the depth of cover, in metres;

h_w is the height of ground water above top of pipe, in metres.

For compactions (based on standard proctor density) other than those shown in Table A.3, use Equation (A.5):

$$M_s = M_{s,100} \times 10^{\left[2{,}8 \times \left(\frac{SPD}{100} - 1 \right) \right] f} \qquad \text{(A.5)}$$

where M_{s100} is the modulus at 100 % SPD.

NOTE 1 In ON B 5012, the exponent f is equal to 0,5. The value 0,4 has been chosen with regard to other investigations that show values lower than 0,5.

NOTE 2 In ON B 5012, soil pressure is used instead of $\kappa \times$ h.

For distributed surface load, use Equation (A.6):

$$M_s = M_{s1} \times \left[h \times \kappa \times \left(1 - 0{,}39 \times \frac{h_w}{h} \right) + \frac{p_o \times \kappa_o}{\gamma_b} \right]^f \tag{A.6}$$

where

p_o is the pressure from uniformly distributed surface load, in newtons per square millimetre (N/mm^2);

κ_o is the reduction factor for distributed load according to silo theory when trench angle, β, is not 90°;

γ_b is the bulk specific weight of the backfill material, in meganewtons per cubic metre (MN/m^3).

A.4.2.1.2 Reduction factors for long-term soil moduli E_1 and E_2

The values given in Table A.4 are the reduction factors to be applied to the backfill soil moduli in zones E_1 and E_2 to account for long-term changes in this property.

Table A.4 — Long-term reduction factors for ATV-A 127 soil groups

ATV-A 127 soil group	Reduction factor
G1	0,90
G2	0,85
G3	0,80
G4	0,75
NOTE Native soil moduli do not normally need to be reduced.	

A.4.2.2 Native soil modulus derived from impact measurements

In this part of ISO 10465, several methods of static design are described and of these the following make reference to impact tests as a means of measuring the soil's elastic modulus. AWWA M-45 refers to ASTM D1586 [4], whilst the WRC method refers to BS 1377 [6]. The Austrian ATV type design uses ON B 4419-1 [8].

There are significant differences in these test methods, which result in different values for the soil's elastic modulus being equated to the same number of blows, depending upon the test method. The differences between these methods is given in the following summary of key features.

A.4.2.2.1 ASTM D1586 and BS 1377

A 50 mm diameter standard split-barrel sampler is driven into the ground at the bottom of the hole by repeated blows from a drop hammer of mass 63,5 kg falling a distance of 0,76 m. The sampler is driven a total of 450 mm into the soil and the number of blows taken for the last 300 mm is recorded. For sands and cohesive soils a standard cutting shoe is used, but for coarse-grained soils a solid conical shoe is preferred.

A.4.2.2.2 ON B 4419-1 SRS 15 method

A 43,7 mm diameter standard solid conical shoe with a mass of 18 kg is driven into the ground at the bottom of the hole by repeated blows from a drop hammer of mass 50 kg falling a distance of 0,5 m. The shoe is driven a total of 100 mm into the soil and the number of blows required to achieve this penetration is recorded as the n_{10} value. This conical-shaped shoe is used for all types of soil.

The ON B 4419-1 method is preferred for obtaining the soil modulus for use in ATV-type calculation methods, and in this case it is recommended that test method SRS 15 be used for site investigations.

Table A.5 shows the relationship between the numbers of blows, n_{10}, based on test method SRS 15, and SPD.

Table A.5 — Relationship between n_{10} using SRS 15 and SPD

n_{10}	SPD %
$\leqslant$ 2	$\leqslant$ 90
> 2	> 90
> 7	> 95
> 12	> 97
> 15	> 98
> 30	> 100

Table A.6 shows the relationship between SPD and soil modulus for the various soil groups used in ATV. The modulus values in this table are an average of values from ON B 5012 for 1 m depth of cover and ATV-A 127 values for up to 5 m depth of cover. From this combination it has been found possible to obtain typical native soil moduli for the normal range of depths of cover between 1 m and 5 m.

It should be noted that native soil moduli are usually independent of depth of cover at the depths of cover normally encountered in pipeline design.

Table A.6 — Relationship between SPD and native soil modulus, E_3

Soil groups as ATV	SPD %				
	85	**90**	**95**	**97**	**100**
	Native soil modulus E_3 N/mm^2				
G1	3,8	5,3	10,0	13,0	20,0
G2	2,1	2,9	5,3	6,7	10,3
G3	1,3	1,8	3,3	4,6	6,7
G4	0,9	1,3	2,5	3,3	4,9

By combining Tables A.5 and A.6, Table A.7 is obtained. Note that for the same number of blows the soil modulus varies with soil group.

Table A.7 — Native soil modulus, E_3

Number of blows [a] (SRS 15) n_{10}	Non-cohesive soil descriptor Groups 1 and 2	Soil groups as ATV-A 127				Cohesive soil descriptor Groups 3 and 4
		G1	G2	G3	G4	
		Native soil modulus E_3 N/mm^2				
> 30	very dense	20	10	7	5	hard
> 12	dense	13	7	5	3	firm
> 7	medium	10	5	3	2	semi-soft
> 2	loose	5	3	2	1	soft
⩽ 2	very loose	4	2	1	0,5	soft and/or plastic

[a] Number of blows per 10 cm movement using test method SRS 15 in accordance with ON B 4419-1.

A.4.3 Soil moduli used in AWWA-type design systems

A.4.3.1 Backfill soil moduli

A.4.3.1.1 Values from AWWA M-45

AWWA has adopted M_s as the soil design parameter. This is a change from the historic practice of using E'. At moderate depths of fill, values for M_s are similar to values for E', and M_s may be substituted for E' in the Iowa formula. Table A.8 gives values for M_s as a function of soil-confining pressure (i.e. depth of cover). Table A.9 defines the terms for soil stiffness categories (SC1, etc.), used in Table A.8, and Table A.10 gives the AWWA recommendations for the use of the various soil stiffness categories.

Table A.8 — Backfill soil modulus, M_s, values, expressed in N/mm^2, from AWWA M-45, based on soil type and compaction condition

Vertical stress level [c] kPa	Depth for soil density of 18,8 kN/m^3 m	Stiffness categories 1 and 2 (SC1 and SC2) Sands and gravels [a, b]			
		Sn-100 MPa	**Sn-95** MPa	**Sn-90** MPa	**Sn-85** MPa
6,9	0,4	16,2	13,8	8,8	3,2
34,5	1,8	23,8	17,9	10,3	3,6
69,0	3,7	29,0	20,7	11,2	3,9
138,0	7,3	37,9	23,8	12,4	4,5
276,0	14,6	51,7	29,3	14,5	5,7
414,0	22,0	64,1	34,5	17,2	6,9
		Stiffness category 3 (SC3) Silts			
			Si-95 MPa	**Si-90** MPa	**Si-85** MPa
6,9	0,4		9,8	4,6	2,5
34,5	1,8		11,5	5,1	2,7
69,0	3,7		12,2	5,2	2,8
138,0	7,3		13,0	5,4	3,0
276,0	14,6		14,4	6,2	3,5
414,0	22,0		15,9	7,1	4,1
		Stiffness category 4 (SC4) Clays			
			Cl-95 MPa	**Cl-90** MPa	**Cl-85** MPa
6,9	0,4		3,7	1,8	0,9
34,5	1,8		4,3	2,2	1,2
69,0	3,7		4,8	2,5	1,4
138,0	7,3		5,1	2,7	1,6
276,0	14,6		5,6	3,2	2,0
414,0	22,0		6,2	3,6	2,4

NOTE 1 SC1 soils have the highest stiffness and require the least amount of compactive energy to achieve a given density. SC5 soils, which are not recommended for use as backfill, have the lowest stiffness and require substantial effort to achieve a given density.

NOTE 2 The numerical suffix to the SPD (standard proctor density) indicates the compaction level of the soil as a percentage of maximum dry density determined in accordance with ASTM D698 or AASHTO T99.

NOTE 3 Designers may interpolate intermediate values of M_s for vertical stress levels not shown in the table.

NOTE 4 For pipe installed below the water table, the modulus should be corrected for reduced vertical stress due to buoyancy, and by an additional factor of 1,00 for SC1 and SC2 soils with $SPD > 95$; 0,85 for SC2 soils with SPD of 90; 0,70 for SC2 soils with SPD of 85; 0,50 for SC3 soils and 0,30 for SC4 soils.

NOTE 5 It is recommended to embed pipe with a stiffness of 1 250 or less only in SC1 or SC2 soil.

[a] SC1 soils have higher stiffness than SC2 soils, but data on specific soil stiffness values are not available at the current time. Until such data are available, the soil stiffness of dumped SC1 soils can be taken as equivalent to SC2 soils compacted to 90 % of maximum standard proctor density SPD90, and the soil stiffness of compacted SC1 soils can be taken as equivalent to SC2 soils compacted to 100 % of maximum standard proctor density, SPD100. Even if dumped, SC1 materials should always be worked into the haunch zone.

[b] The soil types are defined by a two-letter designation used by AASHTO to indicate general soil classification: Sn for sands and gravels, Si for silts and Cl for clays. Specific soil groups that fall into these categories, based on ASTM D2487 and AASHTO M145, are listed in Table A.9.

[c] Vertical stress level is the vertical effective soil stress at the spring-line elevation of the pipe. It is normally computed as the design soil unit weight times the depth of fill. Buoyant unit weight should be used below the groundwater level.

Table A.9 — Soil stiffness categories used in AWWA M-45

Soil stiffness category	Unified soil classification system (USCS) Soil groups [a]	AASHTO soil groups [b]
SC1	Crushed rock: = 15 % sand, maximum 25 % passing the 3/8 in. sieve and maximum 5 % passing No. 200 sieve (Note 3)	
SC2	Clean, coarse-grained soils: SW, SP, GW, GP or any soil beginning with one of these symbols with 12 % or less passing No. 200 sieve (Note 4)	A1, A3
SC3	Coarse-grained soils with fines: GM, GC, SM, SC, or any soil beginning with one of these symbols, containing 12 % or more fines Sandy or gravelly fine-grained soils: CL, ML (or CL-ML, CL/ML, ML/CL) with 30 % or more retained on a No. 200 sieve	A-2-4, A-2-5, A-2-6 or A-4 or A-6 soils with more than 30 % retained on a No. 200 sieve
SC4	Fine-grained soils: CL, ML (or CL-ML, CL/ML, ML/CL) with 30 % or less retained on a No. 200 sieve	A-2-7 or A-4 or A-6 soils with 30 % or less retained on a No. 200 sieve
SC5	Highly plastic and organic soils: MH, CH, OL, OH, PT	A5, A7

NOTE 1 SC1 soils have higher stiffness than SC2 soils but data on specific soil stiffness values are not available at the current time. Until such data are available, the soil stiffness of dumped SC1 soils can be taken as equivalent to SC2 soils compacted to 90 % of maximum standard proctor density (SC2-90), and the soil stiffness of compacted SC1 soils can be taken as equivalent to SC2 soils compacted to 100 % of maximum standard proctor density (SC2-100). Even if dumped, SC1 materials should always be worked into the haunch zone.

NOTE 2 Uniform fine sands (SP) with more than 50 % passing a No. 100 sieve (0,006 in., 0,15 mm) are very sensitive to moisture and should not be used as backfill for fiberglass pipe unless specifically allowed in the contract documents. If use of these materials is allowed, then compaction and handling procedures should follow the guidelines for SC3 materials.

NOTE 3 SC5 materials are not considered to be suitable as an embedment material. They may be used as final backfill as permitted by the engineer.

NOTE 4 SPD the is standard proctor density as determined by ASTM D698 (AASHTO T99).

[a] ASTM D2487:2000, *Standard classification of soils for engineering purposes* (Unified Soil Classification System).

[b] AASHTO M145, *Classification of soils and soil aggregate mixtures*.

Table A.10 — Recommendations for installation and use of soils and aggregates for foundation and pipe-zone embedment in AWWA M-45

Field of application	Soil stiffness category			
	SC1	SC2	SC3	SC4
General recommendations and restrictions	Acceptable and common where no migration is probable or when combined with a geotextile filter media. Suitable for use as a drainage blanket and under drain where adjacent material is suitably graded, or when used with a geotextile filter fabric.	Where hydraulic gradient exists, check gradation to minimize migration. Clean groups are suitable for use as a drainage blanket and underdrain. Uniform fine sands (SP) with more than 50 % passing a #100 sieve [0,006 in., (0,15 mm)] behave like silts and should be treated as SC3 soils.	Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes having a stiffness of 1250 or less.	Difficult to achieve high-soil stiffness. Do not use where water conditions in trench prevent proper placement and compaction. Not recommended for use with pipes having a stiffness of 1250 or less.
Foundation	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above.	Suitable as foundation and for replacing over-excavated and unstable trench bottom as restricted above. Install and compact in 300 mm maximum layers.	Suitable for replacing over-excavated trench bottom as restricted above. Install and compact in 150 mm maximum layers.	Not suitable.
Pipe zone embedment	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Work material under pipe to provide uniform haunch support.	Suitable as restricted above. Difficult to place and compact in the haunch zone.	Suitable as restricted above. Difficult to place and compact in the haunch zone.
Embedment compaction				
Minimum, recommended density, *SPD*	minimum density typically achieved by dumped placement	85 %	90 %	95 %
Relative compactive effort required to achieve minimum density	low	moderate	high	very high
Compaction methods	vibration or impact	vibration or impact	impact	impact
Required moisture control	none	none	maintain near optimum to minimize compactive effort	maintain near optimum to minimize compactive effort

A.4.3.1.2 Values from WRC

Table A.11 is taken from the *Pipe materials selection manual* [10].

Table A.11 — WRC embedment soil modulus, E'_2, values for various installation conditions

Embedment class (see Table 7)	Compaction MPD [a] %	Soil modulus E'_2 MN/m^2
Class S1	Uncompacted	5
	80	7
	85	7
	90	10
Class S2	Uncompacted	3
	80	5
	85	7
	90	10
Class S3	80	3
	85	5
	90	7
Class S4	85	3
	90	5

a Modified proctor density.

A.4.3.2 Native soil moduli

A.4.3.2.1 Values from AWWA M-45

The values given in Table A.12 are the native soil moduli at pipe zone level.

Table A.12 — Native *in situ* soil moduli, M_{sn}, from AWWA M-45

Granular		Cohesive		M_{sn}
Blows/300 mm [a]	Description	Unconfined compressive strength, q_u [b] N/mm^2	Description	N/mm^2
0 to 1	Very very loose	0 to 0,012	Very very soft	0,34
1 to 2	Very loose	0,012 to 0,024	Very soft	1,4
2 to 4	Very loose	0,024 to 0,048	Soft	5
4 to 8	Loose	0,048 to 0,096	Firm	10
8 to 15	Slightly compact	0,096 to 0,192	Stiff	21
15 to 30	Compact	0,192 to 0,383	Very stiff	35
30 to 50	Dense	0,383 to 0,575	Hard	70
$>$ 50	Very dense	$>$ 0,575	Very hard	138
ROCK				345

a Standard penetration test as per ASTM D1586 [4].

b Unconfined compressive strength of cohesive soils as per ASTM D2166 [5].

A.4.3.2.2 Values from WRC

Table A.13 is taken from the *Pipe materials selection manual* [10].

Table A.13 — Guide values for soil modulus in various conditions

<table>
<tr><th colspan="6">Native soil modulus
E'_s
N/mm^2</th></tr>
<tr><th>Soil type</th><th>Very dense</th><th>Dense</th><th>Medium dense</th><th>Loose</th><th>Very loose</th></tr>
<tr><td>Gravel</td><td>over 40</td><td>15 to 40</td><td>9,0 to 15</td><td>5,0 to 9,0</td><td>3,0 to 5,0</td></tr>
<tr><td>Sand</td><td>15 to 20</td><td>9 to 15</td><td>4,0 to 9</td><td>2,0 to 4,0</td><td>1,0 to 2,0</td></tr>
<tr><td>Clayey, silty sand</td><td>10 to 15</td><td>6 to 10</td><td>2,5 to 6</td><td>1,5 to 2,5</td><td>0,5 to 1,5</td></tr>
<tr><td rowspan="7">Clay</td><td>Very hard</td><td colspan="4">11,0 to 14,0</td></tr>
<tr><td>Hard</td><td colspan="4">10,0 to 11,0</td></tr>
<tr><td>Very stiff</td><td colspan="4">6,0 to 10,0</td></tr>
<tr><td>Stiff</td><td colspan="4">4,0 to 6,0</td></tr>
<tr><td>Firm</td><td colspan="4">3,0 to 4,0</td></tr>
<tr><td>Soft</td><td colspan="4">1,5 to 3,0</td></tr>
<tr><td>Very soft</td><td colspan="4">0,0 to 1,5</td></tr>
<tr><td colspan="6">NOTE 1 The quoted values for soil modulus indicate likely values at shallow depths and with groundwater present. They will thus be conservative for pipelines above groundwater level or at depths greater than 1 m.
NOTE 2 For trenches cut in rock, a modulus value of 40 MN/m^2 can be taken as conservative.
NOTE 3 For installations in peat, the structural behaviour of pipelines can be significantly different from other soils, and other design methods may be appropriate (see for example WRC Sea outfall design guide).</td></tr>
</table>

Annex B
(informative)

Determination of concentration factors used in ATV-A 127

B.1 Maximum concentration factor, λ_{max}

$$\lambda_{max} = 1 + \frac{\dfrac{h}{d_e}}{\dfrac{3,5}{a'} + \dfrac{2,2}{\dfrac{E_4}{E_1} \times (a' - 0,25)} + \left[\dfrac{0,62}{a'} + \dfrac{1,6}{\dfrac{E_4}{E_1} \times (a' - 0,25)}\right] \times \dfrac{h}{d_e}} \quad \text{(B.1)}$$

B.2 Concentration factor, λ_P

$$\lambda_P = \frac{\lambda_{max} \times V_s + a' \dfrac{4 \times K_2 \times K'}{3} \times \dfrac{\lambda_{max} - 1}{a' - 0,25}}{V_s + a' \times \dfrac{3 + K_2 \times K'}{3} \times a' - 0,25 \times \dfrac{\lambda_{max} - 1}{a' - 0,25}} \quad \text{(B.2)}$$

where the effective relative projection

$$a' = a \times \frac{E_1}{E_2} \geqslant 0,26 \quad \text{(B.3)}$$

Under normal bedding and installation conditions, it is recommended to use a = 1 for flexible pipes.

$$V_s = \frac{8 \times S_O}{|c_{v*}| \times S_{Bv}} \quad \text{(B.4)}$$

$$c_{v*} = c_{v,qv} + c_{v,qh*} \times K^* \quad \text{(B.5)}$$

B.3 Modified concentration factor, λ_B

$$\lambda_B = \frac{4 - \lambda_P}{3} \quad \text{(B.6)}$$

Annex C
(informative)

Loading coefficients used in ATV-A 127

Table C.1 — Values for $c_{v,qv}$, $c_{h,qv}$, m_{qv} and m_{qh} in relation to 2α

2α degrees	$c_{v,qv}$	$c_{h,qv}$	m_{qv}	m_{qh}
60	–0,105 3	0,102 6	0,377	–0,25
70	–0,102 4	0,100 3	0,353	–0,25
80	–0,099 4	0,098 0	0,332	–0,25
90	–0,096 6	0,095 6	0,314	–0,25
100	–0,093 9	0,093 3	0,299	–0,25
110	–0,091 4	0,091 1	0,286	–0,25
120	–0,089 3	0,089 1	0,275	–0,25
130	–0,087 4	0,087 4	0,267	–0,25
140	–0,085 9	0,085 9	0,261	–0,25
150	–0,084 8	0,084 8	0,256	–0,25
160	–0,084 0	0,084 0	0,253	–0,25
170	–0,083 5	0,083 5	0,251	–0,25
180	–0,083 3	0,083 3	0,250	–0,25

NOTE Only a few of the coefficients given in Table C.1 are in ATV-A 127. The coefficients in Tables C.1, C.2 and D.1 have been derived using equations supplied by Leonhardt.

Table C.2 — Values for $c_{v,qh}$, $c_{h,qh}$ and m_{qh}^* in relation to 2β

2β degrees	$c_{v,qh}^*$	$c_{h,qh}^*$	m_{qh}^*
90	0,056 1	–0,058 5	–0,156
100	0,059 3	–0,061 5	–0,166
110	0,061 9	–0,063 9	–0,174
120	0,064 0	–0,065 8	–0,181
130	0,065 6	–0,067 3	–0,187
140	0,066 8	–0,068 4	–0,191
150	0,067 7	–0,069 2	–0,194
160	0,068 3	–0,069 7	–0,196
170	0,068 7	–0,070 1	–0,198
180	0,068 7 [a]	–0,070 1 [a]	–0,198

[a] Soil box test results for $2\beta = 180°$ (see references [1] and [2]) show better agreement between calculated and measured values when $c_{v,qh} = 0{,}069\,4$ and $c_{h,qh} = 0{,}070\,0$. It is therefore recommended that these experimentally derived values be used in place of the applicable values in Table C.2. In ATV-A 127, m_{qh^*} is = –0,181.

Annex D
(informative)

Horizontal bedding correction factors

D.1 General

The correction factor, ζ, takes into account the difference in soil modulus of the pipe embedment material and the native soil as well as the width of the trench. Equations (6) and (7) from the main text are those included in ATV-A 127 for a support angle 2β of 120°. However Leonhardt, the originator of this factor, recommends the use of the equations and variable values given in this annex, which cover a wider range of support conditions. Despite appearances, for 2β equal to 120°, this annex produces an answer very similar to that obtained using Equation (6).

D.2 Correction factor for horizontal bedding stiffness, ζ, in ATV-A 127

The stiffness of the soil at the side of the pipe (horizontal bedding stiffness) in ATV-A 127 is defined as shown in Equation (8). This equation assumes a support angle, 2β, of 120°:

$$S_{\text{Bh}} = 0{,}6 \times \zeta \times E_2 \tag{8}$$

In Table D.1, a coefficient c_4 is included which in Equation (D.3) is the constant 0,6. This allows for stress propagation in the soil under the horizontal reaction pressure, $q_{\text{h*}}$ [see Figure 2 b)].

The equations used in ATV-A 127 to determine the correction factor, ζ, for the assumed 120° support angle are Equations (10) and (11).

$$\zeta = \frac{1{,}667}{\Delta f + \left(1{,}667 - \Delta f\right) \times E_2 / E_3} \tag{10}$$

$$\text{where } \Delta f = \frac{\left(\dfrac{b}{d_{\text{e}}} - 1\right)}{0{,}980 + 0{,}303\left(\dfrac{b}{d_{\text{e}}} - 1\right)} \leqslant 1{,}667 \tag{11}$$

NOTE The correction factor, ζ, takes into account the difference in soil modulus of the pipe embedment material, the native soil and the width of the trench. The above equations are those included in ATV-A 127 for a support angle of 120°, but the variable values given in Annex D can be used for other angles. Annex D covers a wider range of support conditions than only the 120° covered by Equation (10).

Leonhardt's equations for the general case are Equations (D.1), (D.2) and (D.3). The variables for insertion in these equations are given in Table D.1 and depend upon the assumed support angle 2β.

$$\zeta = \frac{c_1}{c + (c_1 - c) \times E_2 / E_3} \tag{D.1}$$

$$c = \frac{\left(\dfrac{b}{d_e} - 1\right)}{c_2 + c_3 \times \left(\dfrac{b}{d_e} - 1\right)} \tag{D.2}$$

$$S_{\text{Bh}} = c_4 \times \zeta \times E_2 \tag{D.3}$$

Table D.1 — Variables to determine correction factor

2β =	90°	100°	110°	120°	130°	140°	150°	160°	170°	180°
c_1	1,480	1,563	1,613	1,667	1,695	1,754	1,754	1,786	1,786	1,786
c_2	0,976	0,978	0,979	0,980	0,981	0,981	0,982	0,982	0,982	0,982
c_3	0,380	0,348	0,323	0,303	0,288	0,277	0,268	0,262	0,259	0,258
c_4	0,680	0,640	0,620	0,600	0,590	0,570	0,570	0,560	0,560	0,560

The ranges of validity of Equations (D.1) and (D.2) and their variables are as follows:

$0 \leqslant b/d_e \leqslant 4,3$

$b/d_e > 4,3 \quad \zeta = 1,0$

D.3 AWWA M-45 soil support combining factor, S_c

In the AWWA M-45 design manual, there is a similar correction factor but it is termed "combining factor S_c" and has values different from those given by Equations (D.1) and (D.2). The values for S_c shown in Table D.2 should not be substituted for ζ in ATV-A 127 type systems.

Table D.2 — Values for the soil support combining factor, S_c

$\frac{M_{sn}}{M_{sb}}$	$\frac{b}{d}$							
	1,25	**1,50**	**1,75**	**2,00**	**2,5**	**3,00**	**4,00**	**5,00**
0,005	0,02	0,05	0,08	0,12	0,23	0,43	0,72	1,00
0,01	0,03	0,07	0,11	0,15	0,27	0,47	0,74	1,00
0,02	0,05	0,10	0,15	0,20	0,32	0,52	0,77	1,00
0,05	0,10	0,15	0,20	0,27	0,38	0,58	0,80	1,00
0,1	0,15	0,20	0,27	0,35	0,46	0,65	0,84	1,00
0,2	0,25	0,30	0,38	0,47	0,58	0,75	0,88	1,00
0,4	0,45	0,50	0,56	0,64	0,75	0,85	0,93	1,00
0,6	0,65	0,70	0,75	0,81	0,87	0,94	0,98	1,00
0,8	0,84	0,87	0,90	0,93	0,96	0,98	1,00	1,00
1,0	1,00	1,00	1,00	1,00	1,00	1,00	1,00	1,00
1,5	1,40	1,30	1,20	1,12	1,06	1,03	1,00	1,00
2	1,70	1,50	1,40	1,30	1,20	1,10	1,05	1,00
3	2,20	1,80	1,65	1,50	1,35	1,20	1,10	1,00
⩾ 5	3,00	2,20	1,90	1,70	1,50	1,30	1,15	1,00

b is the trench width at the spring-line, in m;

d is the pipe diameter, in m;

M_{sn} is the native soil constrained modulus, in N/m^2;

M_{sb} is the embedment soil constrained modulus, in N/m^2.

NOTE Intermediate values for S_c can be determined by linear interpolation between adjacent values.

Annex E
(informative)

Selection of long-term stiffness

E.1 Long-term stiffness and buckling

E.1.1 Theory

If a ring of GRP is kept at a constant deflection, the load required to maintain that deflection decreases with time; hence the apparent flexural modulus decreases with time. In this case the modulus is called the relaxation modulus.

If at that constant deflection the load is incrementally increased, the relationship between the incremental increase in stress and the incremental increase in strain will not correspond to the relaxation modulus, but to a higher value. The modulus applying to that period of incremental loading is called the instantaneous elastic modulus.

If the constant deflection test ring is maintained in a dry condition, then the instantaneous elastic modulus of the ring will be equal to the original elastic modulus of the untested specimen. If, however, the test ring is in water and the elapsed time has been long enough for the water to influence the properties of the material, then the instantaneous elastic modulus, $E_{\mathrm{t,wet}}$, (although higher than the relaxation modulus) will be lower than the initial modulus. There has been an ageing of the material. Hence:

$$E_{\mathrm{t,wet}} = a_{\mathrm{f}} \times E_{\mathrm{o}} \tag{E.1}$$

where

a_{f} is the ageing factor;

E_{o} is the initial elastic modulus.

The same thing happens if there is a constant load applied to the pipe ring. The apparent modulus of the pipe is decreased because of creep deflection under load. If this test is conducted on specimens exposed to water, the apparent modulus will be lower because of the ageing factor. Similarly, a short-term incremental increase in load will show the relationship to the initial modulus value given in Equation (E.1).

For static calculation of underground GRP pipes, the above visco-elastic characteristics should be considered. For water-hammer and traffic load when considering buckling, the short-term instantaneous load conditions apply. Hence the initial stiffness (or modulus) and the ageing factor apply to the design condition, not the creep factor.

Where pipe is installed in weak soil there will be continuous creeping of the pipe under load; this often increases with time. Here, there can be a risk for a long-term sustained buckling which is frequently called creep buckling.

E.1.2 Recommendation

For all buckling cases, the initial pipe stiffness should be multiplied by the long-term ageing factor to obtain the value of the pipe stiffness to be used for long-term buckling resistance calculations.

E.2 Long-term stiffness and deflection

In buried conditions, the deflection tends to remain unchanged with time, thus the pipe's installation condition is similar to a load relaxation condition. However, with time there is normally an increase in the load due to loss of friction between the soil layers. This increase in soil load is normally incremental and rapid.

E.2.1 Recommendation

For long-term deflection calculations, it is recommended that the long-term pipe stiffness be used, i.e. the initial pipe stiffness multiplied by the creep factor or relaxation factor, both of which include the ageing effect.

E.2.2 Soil and traffic loads

For a correct calculation of long-term deflection and flexural strain, two separate calculations for the vertical load, q_v, should be made; one for soil load, δ_{vc}, using the long-term pipe creep or relaxation stiffness, and one for traffic load and vacuum loads, δ_{va}, using the initial pipe stiffness times the ageing factor, a_f.

E.3 Ageing factors

The ageing factor can be obtained from a test such as that outlined as follows.

A ring of pipe complying with the requirements of ISO 7685 is subjected to the initial stiffness test in accordance with ISO 7685. One of the three pairs of lines marked on the test piece is selected as the reference pair, and the stiffness obtained for this pair of lines is noted. The test piece is then immersed in water and laid horizontal so that it is free of load. At predefined moments in time, the test piece is removed from the water and the initial stiffness determined using the selected reference lines. The test continues for at least 10 000 h, and the results are analysed in accordance with ISO 10928 to obtain an extrapolated 50-year stiffness. The ageing factor, a_f, is obtained from Equation (E.2):

$$a_f = \frac{\text{extrapolated 50 year stiffness}}{\text{initial stiffness}} \tag{E.2}$$

Annex F
(informative)

Partly residual soil friction used in ATV-A 127 type calculation systems

F.1 General

As stated in 4.2.1, the silo soil load is used in ATV-A 127 for both the long-term and short-term conditions. However, it is recommended that the prismatic soil load be used for long-term conditions at depths of cover up to 3 m. This is to take into account the loss of soil friction due to factors such as frost, rain, traffic, etc. In order to incorporate this concept into the calculations, the term *environmental depth of cover*, H_{EVD}, has been introduced. For depths of cover greater than 3 m, it is assumed that the soil friction remains not only at the trench wall, but also within the backfill above the top of the pipe for the lifetime of the pipes, thereby reducing the vertical soil load. In the calculations this is achieved through the factor λ_{R}. For additional details concerning wide trenches, see ISO 10465-2:2007, 4.2.2.

F.2 Long-term vertical soil pressure, $q_{\text{v,50}}$

The long-term vertical pressure, $q_{\text{v,50}}$, is calculated using Equation (F.1):

$$q_{\text{v,50}} = \left\{ \left(\chi_{\text{S}} \times H_{\text{EVD}} \right) + \left[\chi_{\text{S}} \times \left(h - H_{\text{EVD}} \right) \times \kappa \times \lambda_{\text{PG50}} \right] + \left(p_{\text{o}} \times \kappa_{\text{o}} \right) + p_{\text{v}} \right\} \quad \text{(F.1)}$$

where

H_{EVD} is the depth down to which the friction has been lost due to frost, rain, traffic loads, etc., expressed in metres (up to a maximum of 3 m);

h is the depth of cover, in metres;

κ and κ_{o} are in accordance with ISO 10465-2, but using ($h - H_{\text{EVD}}$) instead of h;

λ_{PG50} is in accordance with Equation (6), except λ_{P} is replaced by λ_{PLT},

where

$$\lambda_{\text{PLT}} = 1 - \frac{h - H_{\text{EVD}}}{h} \times \left(1 - \lambda_{\text{P}} \right) \quad \text{(F.2)}$$

λ_{PLT} is the long-term value for the concentration factor λ_{P} (see Annex B), using long-term values for soil modulus, S_{p}, S_{Bv}, etc.

F.3 Long-term horizontal soil pressure, $q_{\text{h,50}}$

The long-term horizontal soil pressure, $q_{\text{h,50}}$, is calculated using Equation (F.3):

$$q_{\text{h,50}} = (K_2 \times \chi_{\text{S}} \times H_{\text{EVD}}) + \left[\lambda_{\text{BLT}} \times \chi_{\text{S}} \times (h - H_{\text{EVD}}) + (\kappa_{\text{o}} \times K_2 \times p_{\text{o}}) + (K_2 \times \chi_{\text{S}} \times \frac{d_{\text{e}}}{2}) \right] \quad \text{(F.3)}$$

where

$$\lambda_{\mathrm{BLT}} = \left[1 + \frac{(h - H_{\mathrm{EVD}})}{h} \times (\lambda_{\mathrm{B}} - 1) \right] \tag{F.4}$$

For the case where K_1 = 0,5, Equations (F.5) and (F.6) can be used for the calculation of κ and κ_{o}:

$$\kappa = \frac{1 - e^{-\left(\frac{h}{b} \times \tan \delta\right)}}{\frac{h}{b} \times \tan \delta} \tag{F.5}$$

$$\kappa_{\mathrm{o}} = e^{-\left(\frac{h}{b} \times \tan \delta\right)} \tag{F6}$$

If the trench angle, β (see Figure 2), is not 90°, then

$$\kappa_{\beta} = 1 - \frac{\beta}{90} + \kappa \times \frac{\beta}{90} \tag{F7}$$

$$\kappa_{\mathrm{o}\beta} = 1 - \frac{\beta}{90} + \kappa_{\mathrm{o}} \times \frac{\beta}{90} \tag{F8}$$

Annex G
(informative)

Application limits for GRP pressure pipes installed underground

G.1 General

In 5.2.1.1 the following Note appears which is also applicable to this annex:

NOTE Similar calculations can be done for the other stiffness series, namely SN 1250, SN 2500, SN 5000 and SN 10000, and values for D_f very similar to those in Table 4 will be derived. For most purposes, the SN values in Table 4 and in the applicable tables in Annex G can be interchanged, i.e. SN 1250 for SN 1000, SN 2500 for 2000, SN 5000 for SN 4000 and SN 10000 for SN 8000, without significantly affecting the accuracy of the results.

G.1.1 Minimum or mean

This annex presents the basic concepts which have been used to establish the safety design method presented in Clause 10 of this part of ISO 10465 for GRP pressure pipes complying with the relevant ISO GRP pipe system standards.

The system standard gives the required minimum long-term factor of safety against pressure failure from which the manufacturer, using the pressure regression ratio derived from long-term tests, can determine the required minimum initial six-minute failure pressure for the pipe. From this value the manufacturer determines the design strength, taking into account the mean burst strength and the standard deviation of the test results. It should be noted that in the system standard the minimum long-term factor of safety against pressure failure is applied to the *minimum* long-term strength.

The pipe produced to comply with the requirements of the appropriate system standard is required to have a *minimum long-term tensile* strength. To ensure that this is achieved, the *mean initial* strength of the factory-tested product must be sufficiently high, taking into account the product's variability, to have a 97,5 % LCL (lower confidence limit) at 50 years that is equal to or greater than the minimum requirement. The design procedures to be followed to achieve this are outlined in the system standard.

The ISO system standard for GRP pipe specifies the minimum long-term ultimate ring deflection which the pipe test piece must sustain without failure, but there is no requirement to determine the actual deflection at failure, hence neither a mean value nor a standard deviation for that characteristic can be obtained.

In this annex it is assumed that the coefficient of variation of the initial ultimate ring deflection, z, will have a value of 9 %. Using this assumption, and the specified minimum initial ultimate deflection stated in the system standard, the minimum long-term ultimate ring deflection value can be determined.

EXAMPLE Consider a pipe of nominal stiffness SN 8000 which has a specified minimum initial ultimate ring deflection of 10,3 % which is manufactured in a factory having an AQL of 6,5 %. The long-term mean value will have to be equal or exceed the value derived from Equation (G.1).

$$\left(\frac{d_v}{d_m}\right)_{ult} = \frac{10,3}{1 - z \times 0,01 \times 1,52} = 11,93\ \% \qquad \text{(G.1)}$$

where

1,52 is the multiplier for an AQL of 6,5 %;

z is the coefficient of variation, in percent;

$$z = \frac{\text{standard deviation of initial ultimate ring deflection}}{\text{mean initial ultimate ring deflection}} \times 100$$

G.1.2 Safety factors

G.1.2.1 General

In the descriptions which follow, there are two distinct classes of safety factors, namely material and combined.

G.1.2.2 Safety factors η_t and η_f

The safety factors, η_t and η_f, are each related to a property of the material, namely circumferential tensile strength, or flexural strength ignoring rerounding. If the symbol η_{tPN} is used instead of η_t, it represents the safety factor required to allow the pipe to operate at its nominal pressure.

G.1.2.3 Combined safety factors

The combined safety factors η_{hat} and η_{haf} are applied directly to one material property, but take into account the influence of one on the other, i.e. tensile influenced by flexural strength, and flexural influenced by tensile strength.

G.2 Semi-probabilistic calculation system

Ideally, assessing the probability of failure of a pipe for underground pipe installations requires the use of semi-probabilistic calculations. These are very involved and require extensive data covering not only the pipe, but also the installation properties and loading variability, which makes it impractical to use them as the basis for establishing safety factors in International Standards.

For pipes made from composite materials which have multiple values for some material properties, this is even more complicated.

In principle, the semi-probabilistic calculations are based upon Equation (G.2):

$$X = \frac{\mu_{\text{Res}} - \mu_{\text{S}}}{\left(S_{\text{Res}}^{2} + S_{\text{S}}^{2}\right)^{0,5}} \qquad \text{(G.2)}$$

where

μ_{Res} is the mean value for the strength (resistance) of the pipe;

$\mu_{\text{S,B}}$ is the mean value for the stress in the pipe;

S_{Res} is the standard deviation of the strength;

S_{S} is the standard deviation of the stress in the pipe;

X is the safety index (see Tables G.2 and G.3).

Table G.1 lists some of the parameters which have to be determined for a semi-probabilistic calculation.

Use of this procedure to determine the safety factors for inclusion in International Standards would require that agreed values for the mean and the standard deviation be established for the parameters to be used for all pipe materials under consideration. Such an international agreement would also have to cover matters such as soil parameters, depths of cover, trench width, traffic loads and internal pressure for each of the cases which are used in the application. It is evident that such agreements are difficult to reach, and thus far it has been found impossible to use this approach.

An alternative procedure, termed "simplified probabilistic calculation system", is outlined in the following subclauses for the calculation of failure probabilities and the establishment of long-term safety factors for underground GRP pressure pipes.

Table G.1 —Typical parameters used for the calculation of failure probabilities

Parameter
Long-term tensile strength
Flexural ultimate strain (long-term = 50 years)
Working pressure (internal)
Wall thickness
Pipe stiffness
Traffic load
Trench width
Depth of cover
Soil pressure ratio in soil zone 2
Soil pressure ratio in soil zone 3
Vertical bedding reaction angle
Horizontal soil reaction angle
Soil modulus in soil zone 2
Soil modulus in soil zone 3
Soil density in backfill zone

G.3 Simplified probabilistic calculation system

This clause gives guidance on how to determine the long-term safety factor based upon a required probability of failure.

The basic equation for the simplified probabilistic calculation for buried pipes is Equation (G.3), which has been derived from a modified Equation (G.2):

$$X = \frac{\mu_{\text{Res,B}} - \mu_{\text{S,B}}}{\left(s_{\text{Res,B}}{}^{2} + s_{\text{S,B}}{}^{2}\right)^{0,5}} \tag{G.2}$$

$$X = \frac{\eta_{\text{hat}} - 1}{y \times 0{,}01 \times \eta_{\text{hat}}} \tag{G.3}$$

Equation (G.3) was obtained from Equation (G.2) using the following values:

$$s_{\text{S,B}} = 0$$

$$s_{\text{Res,B}} = y \times 0{,}01 \times \mu_{\text{Res,B}} \tag{G.4}$$

$$\mu_{\text{Res,B}} = \mu_{\text{S,B}} \times \eta_{\text{hat}} \tag{G.5}$$

where

y is the coefficient of variation of the tensile strength, in %;

$\mu_{Res,B}$ is the mean value for the strength (resistance) of the pipe underground;

$\mu_{S,B}$ is the mean value for the load applied (stress) in the pipe underground;

$S_{Res,B}$ is the standard deviation of the strength of the pipe underground;

$S_{S,B}$ is the standard deviation of the stress in the pipe underground;

X is the safety index;

η_{hat} is the combined safety factor for tension which takes into account the effect of flexure on tension.

In order to be able to make the simplified probability calculation, the combined safety factor, η_{hat}, is required. The value of the safety index, X, is obtained from statistical tables, such as Tables G.2 and G.3, using the calculated value for the probability function $P(X)$ derived from Equation (G.6):

$$P(X) = 1 - P_f \tag{G.6}$$

Using this value for X and the known value for the coefficient of variation of the tensile strength, y, (in %) in Equation (G.7), the minimum value for the combined factor of safety for tension, η_{hat}, can be obtained. If this calculated minimum value for η_{hat} is less than 1,5, use 1,5; otherwise use the calculated value.

$$\eta_{hat} = \frac{1}{1 - X \times y \times 0{,}01} \tag{G.7}$$

As an example, let us calculate the minimum value for η_{hat} to ensure a minimum probability of failure of 1 in 10 000 when the coefficient of variation of the tensile strength, y, is 10 %.

P_f = 1/10 000, which is 10^{-4}, so from Equation (G.6) $P(X)$ is 0,999 9.

Using Table G.3 for a $P(X)$ value of 0,999 9, a value of 3,72 for X is obtained.

Substituting in Equation (G.7) gives

$$\eta_{hat} = \frac{1}{1 - X \times y \times 0{,}01} = \frac{1}{1 - 3{,}72 \times 10 \times 0{,}01} = 1{,}59$$

For this case, the calculated minimum value is greater than 1,5, therefore the minimum value to use for η_{hat} is 1,59.

This safety factor is defined in Equation (G.8) and illustrated in Figure G.1:

$$\eta_{hat} = \eta_t \times \left(1 - \frac{1}{\eta_{ff}}\right) \tag{G.8}$$

where

$$\eta_{ff} = \frac{\eta_f}{r} \tag{G.9}$$

η_t is the tensile safety factor;

η_f is the flexural safety factor;

r is the rerounding factor.

The tensile safety factor, η_t, is defined by Equation (G.10) obtained from, σ_{50} and σ_t, or, alternatively, long-term burst pressure, P_{50}, and the working pressure, P_w, as shown in Equation (G.10):

$$\eta_t = \frac{\sigma_{50}}{\sigma_t} = \frac{P_{50}}{P_w} \tag{G.10}$$

where

σ_{50} is the long-term tensile strength;

σ_t is the calculated stress in the pipe wall;

P_{50} is the long-term failure pressure;

P_w is the working pressure.

If η_t is for the condition that the working pressure, P_w, is equal to the nominal pressure PN, then η_t is written as $\eta_{t,PN}$ and Equation (G.8) is written as Equation (G.11):

$$\eta_{t,PN} = \frac{\eta_{hat}}{1 - \frac{1}{\eta_{ff}}} \tag{G.11}$$

where

η_{hat} is obtained as described earlier;

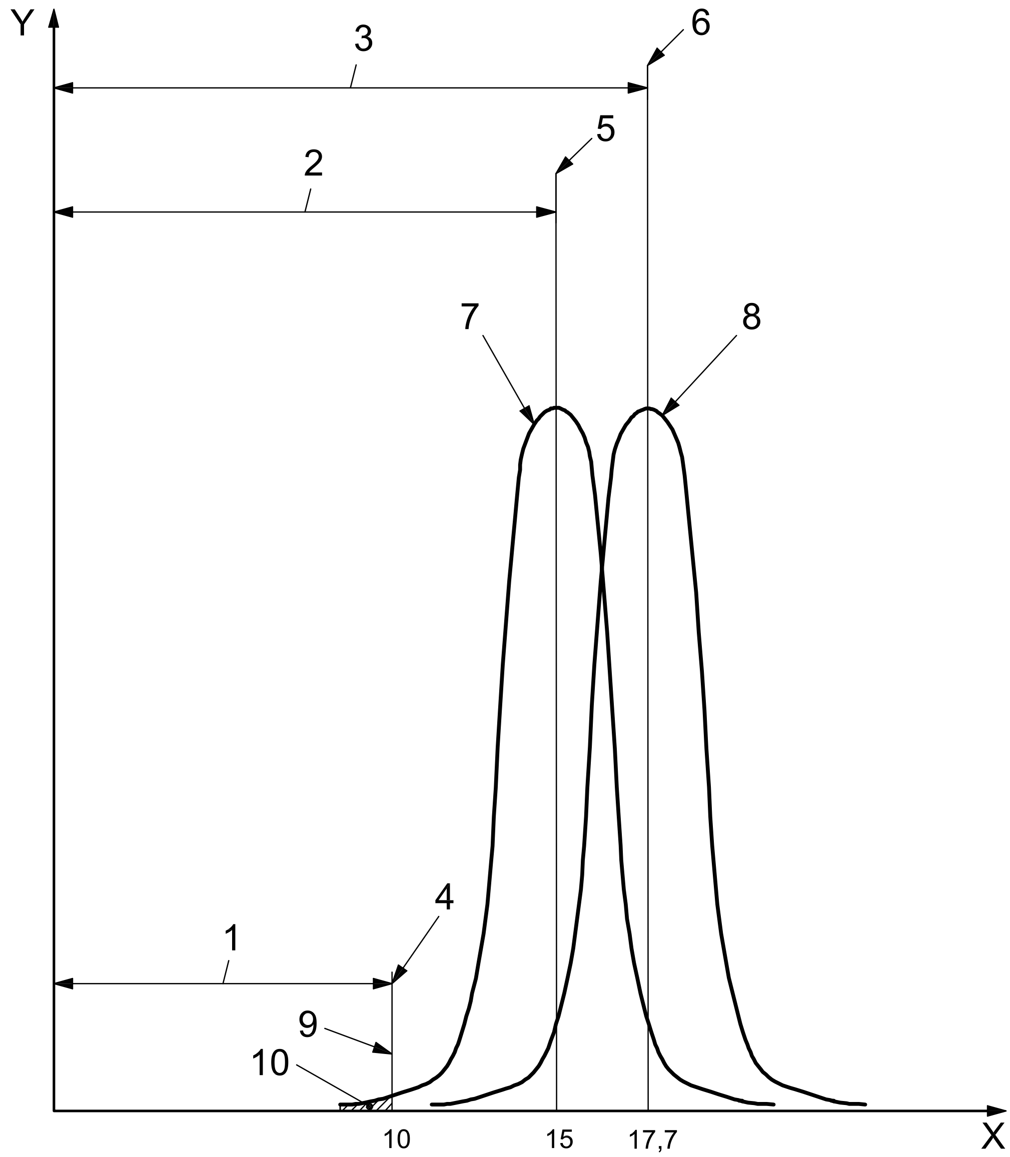
η_{ff} is obtained from tables such as Table G.5 or Table G.9;

$\eta_{t,PN}$ is the factor of safety applicable to the mean value of the minimum long-term failure pressure, and takes into account combined loading.

On combining Equations (G.7) and (G.11), $\eta_{t,PN}$ is seen to be Equation (G.12):

$$\eta_{t,PN(mean)} = \frac{1}{\left(1 - X \times y \times 0{,}01\right) \times \left[1 - \frac{1}{\eta_{ff}}\right]} \tag{G.12}$$

This simplified probabilistic calculation is illustrated in Figure G.1.



Key

X pressure (bar)

Y frequency

1 PN

2 PN × η_{hat}

3 PN × $\eta_{t,PN}$, mean

4 $\mu_{S,B}$ mean value of stress in pipe below ground

5 $\mu_{Res, B}$ mean value of strength of pipe below ground

6 $\mu_{Res, A}$ mean value of strength of pipe above ground

7 pipe installed below ground

8 pipe installed above ground

9 99,99 % LCL

10 0,01 % of population

Figure G.1 — Typical simplified probability calculation

Table G.2 — Probability of failure

X	$P(X)$	X	$P(X)$	X	$P(X)$	X	$P(X)$	X	$P(X)$
0,00	0,500 000	1,00	0,841 345	2,00	0,977 250	3,00	0,998 650	4,00	0,999 968 314
0,02	0,507 978	1,02	0,846 136	2,02	0,978 308	3,02	0,998 736	4,02	0,999 970 887
0,04	0,515 953	1,04	0,850 830	2,04	0,979 325	3,04	0,998 817	4,04	0,999 973 261
0,06	0,523 922	1,06	0,855 428	2,06	0,980 301	3,06	0,998 893	4,06	0,999 975 451
0,08	0,531 881	1,08	0,859 929	2,08	0,981 237	3,08	0,998 965	4,08	0,999 977 470
0,10	0,539 828	1,10	0,864 334	2,10	0,982 136	3,10	0,999 032	4,10	0,999 979 331
0,12	0,547 758	1,12	0,868 643	2,12	0,982 997	3,12	0,999 096	4,12	0,999 981 046
0,14	0,555 670	1,14	0,872 857	2,14	0,983 823	3,14	0,999 155	4,14	0,999 982 625
0,16	0,563 559	1,16	0,876 976	2,16	0,984 614	3,16	0,999 211	4,16	0,999 984 078
0,18	0,571 424	1,18	0,881 000	2,18	0,985 371	3,18	0,999 264	4,18	0,999 985 416
0,20	0,579 260	1,20	0,884 930	2,20	0,986 097	3,20	0,999 313	4,20	0,999 986 646
0,22	0,587 064	1,22	0,888 767	2,22	0,986 791	3,22	0,999 359	4,22	0,999 987 777
0,24	0,594 835	1,24	0,892 512	2,24	0,987 455	3,24	0,999 402	4,24	0,999 988 817
0,26	0,602 568	1,26	0,896 165	2,26	0,988 089	3,26	0,999 443	4,26	0,999 989 772
0,28	0,610 261	1,28	0,899 727	2,28	0,988 696	3,28	0,999 481	4,28	0,999 990 649
0,30	0,617 911	1,30	0,903 199	2,30	0,989 276	3,30	0,999 517	4,30	0,999 991 454
0,32	0,625 516	1,32	0,906 582	2,32	0,989 830	3,32	0,999 550	4,32	0,999 992 193
0,34	0,633 072	1,34	0,909 877	2,34	0,990 358	3,34	0,999 581	4,34	0,999 992 870
0,36	0,640 576	1,36	0,913 085	2,36	0,990 863	3,36	0,999 610	4,36	0,999 993 492
0,38	0,648 027	1,38	0,916 207	2,38	0,991 344	3,38	0,999 638	4,38	0,999 994 061
0,40	0,655 422	1,40	0,919 243	2,40	0,991 802	3,40	0,999 663	4,40	0,999 994 583
0,42	0,662 757	1,42	0,922 196	2,42	0,992 240	3,42	0,999 687	4,42	0,999 995 061
0,44	0,670 031	1,44	0,925 066	2,44	0,992 656	3,44	0,999 709	4,44	0,999 995 498
0,46	0,677 242	1,46	0,927 855	2,46	0,993 053	3,46	0,999 730	4,46	0,999 995 898
0,48	0,684 386	1,48	0,930 563	2,48	0,993 431	3,48	0,999 749	4,48	0,999 996 264
0,50	0,691 462	1,50	0,933 193	2,50	0,993 790	3,50	0,999 767	4,50	0,999 996 599
0,52	0,698 468	1,52	0,935 744	2,52	0,994 132	3,52	0,999 784	4,52	0,999 996 905
0,54	0,705 402	1,54	0,938 220	2,54	0,994 457	3,54	0,999 800	4,54	0,999 997 185
0,56	0,712 260	1,56	0,940 620	2,56	0,994 766	3,56	0,999 815	4,56	0,999 997 440
0,58	0,719 043	1,58	0,942 947	2,58	0,995 060	3,58	0,999 828	4,58	0,999 997 673
0,60	0,725 747	1,60	0,945 201	2,60	0,995 339	3,60	0,999 841	4,60	0,999 997 885
0,62	0,732 371	1,62	0,947 384	2,62	0,995 603	3,62	0,999 853	4,62	0,999 998 079
0,64	0,738 914	1,64	0,949 497	2,64	0,995 855	3,64	0,999 864	4,64	0,999 998 256
0,66	0,745 373	1,66	0,951 543	2,66	0,996 093	3,66	0,999 874	4,66	0,999 998 417
0,68	0,751 748	1,68	0,953 521	2,68	0,996 319	3,68	0,999 883	4,68	0,999 998 564
0,70	0,758 036	1,70	0,955 435	2,70	0,996 533	3,70	0,999 892	4,70	0,999 998 698
0,72	0,764 238	1,72	0,957 284	2,72	0,996 736	3,72	0,999 900	4,72	0,999 998 819
0,74	0,770 350	1,74	0,959 071	2,74	0,996 928	3,74	0,999 908	4,74	0,999 998 930
0,76	0,776 373	1,76	0,960 796	2,76	0,997 110	3,76	0,999 915	4,76	0,999 999 031
0,78	0,782 305	1,78	0,962 462	2,78	0,997 282	3,78	0,999 922	4,78	0,999 999 122
0,80	0,788 145	1,80	0,964 070	2,80	0,997 445	3,80	0,999 928	4,80	0,999 999 206
0,82	0,793 892	1,82	0,965 621	2,82	0,997 599	3,82	0,999 933	4,82	0,999 999 281
0,84	0,799 546	1,84	0,967 116	2,84	0,997 744	3,84	0,999 938	4,84	0,999 999 350
0,86	0,805 106	1,86	0,968 557	2,86	0,997 882	3,86	0,999 943	4,86	0,999 999 412
0,88	0,810 570	1,88	0,969 946	2,88	0,998 012	3,88	0,999 948	4,88	0,999 999 469
0,90	0,815 940	1,90	0,971 284	2,90	0,998 134	3,90	0,999 952	4,90	0,999 999 520
0,92	0,821 214	1,92	0,972 571	2,92	0,998 250	3,92	0,999 956	4,92	0,999 999 567
0,94	0,826 391	1,94	0,973 810	2,94	0,998 359	3,94	0,999 959	4,94	0,999 999 609
0,96	0,831 472	1,96	0,975 002	2,96	0,998 462	3,96	0,999 963	4,96	0,999 999 647
0,98	0,836 457	1,98	0,976 148	2,98	0,998 559	3,98	0,999 966	4,98	0,999 999 682
1,00	0,841 345	2,00	0,977 250	3,00	0,998 650	4,00	0,999 968	5,00	0,999 999 713

G.4 Calculation of $\eta_{t,PN}$

This clause describes the calculation of $\eta_{t,PN}$ when P_w = PN and considering the installation conditions, ultimate deflection properties and product variability.

A commonly used value for failure probability, P_f, is 10^{-4}, which from Table G.2 means that X = 3,72.

Probabilistic calculations use mean values, and hence η_{ff} relates to a mean value; thus the η_{tPN} obtained relates also to a mean value, which, if required, can be converted to a LCL.

The following calculations have been made assuming a coefficient of variation of 9 % for both tensile strength and flexural ultimate strain. This means that $y = z$ = 9 %.

A value of 3 % for initial pipe deflection *before pressurizing* has been chosen for these calculations because this is a common requirement for pressure pipes. Due to rerounding, the deflection is reduced and the long-term calculation is based upon this reduced deflection. Field investigations have shown that a further reduction occurs which is neglected in these calculations.

All probabilistic calculations start by using mean values. In the following calculations therefore, the assumed 3 % initial deflection before pressurization is a mean value, and the 150° used for 2α and 2β when calculating D_f also represents mean values.

G.4.1 Determination of η_{ff}

$$\eta_{ff} = \frac{\varepsilon_{50}\,(\text{mean})}{\varepsilon_f\,(\text{mean})} \times \frac{1}{r} \tag{G.13}$$

where

ε_{50} is the ultimate strain in flexure at 50 years;

ε_f is the strain due to deflection;

r is the rerounding factor.

$$r = 1 - \frac{P}{30} \tag{G.14}$$

The value for ε_{50} is either determined using Equation (G.15) or declared by the manufacturer.

$$\varepsilon_{50} = D_g \times \left(\frac{d_v}{d_m}\right)_{ult} \times \frac{e}{d_m} \tag{G.15}$$

where

$$D_g = \frac{4{,}28}{\left(1 + 0{,}5\left(\frac{d_v}{d_m}\right)_{ult}\right)^2} \tag{G.16}$$

$\left(\frac{d_v}{d_m}\right)_{ult}$ is the extrapolated ultimate relative ring deflection at 50 years

e is the pipe wall thickness, in millimetres

d_m is the mean diameter, in millimetres

The value for ε_f is calculated from the allowable deflection in the ground using Equation (G.17).

$$\varepsilon_f = D_f \times \left(\frac{d_v}{d_m}\right)_{\text{allowable}} \times \frac{e}{d_m} \tag{G.17}$$

Hence, by substitution in Equation (G.13) assuming a typical value for pressure pipes of 3 % for $\left(\frac{d_v}{d_m}\right)_{\text{allowable}}$

η_{ff} is found to be $$\eta_{ff} = \frac{D_g \times \left(\frac{d_v}{d_m}\right)_{\text{ult}}}{r \times D_f \times 3} \tag{G.18}$$

The values for the parameters to be used in these calculations are summarized in Table G.3.

Table G.3 — Parameters

SN	Standards values $\frac{y_{u,wet,x}}{d_m}$	Mean value $\left(\frac{d_v}{d_m}\right)_{\text{ult}}$	Calculated D_g	D_f (see Table 4)
8000	9,7 %	11,24 %	3,84	3,31
4000	12,2 %	14,13 %	3,73	3,98
2000	15,4 %	17,84 %	3,61	5,33
1000	19,4 %	22,47 %	3,46	7,95
NOTE The D_f values are for 3 % initial deflection and $2\alpha = 2\beta = 150°$ according to 5.2.1.1 and Table 4.				

Using the values given in Table G.3 for $\left(\frac{d_v}{d_m}\right)_{\text{ult}}$, D_g and D_f in Equation (G.18), the mean values for η_{ff} shown in Table G.4 for a range of pressure and stiffness classes are obtained. The mean values in Table G.4 have been obtained using Equation (1), a value of 9 % for y and, z and an AQL of 6,5 %.

Table G.4 — Values for η_{ff} (mean)

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	26,05	9,30	6,51	5,43	5,01	4,74
4000	26,52	9,47	6,63	5,52	5,10	4,82
2000	24,15	8,63	6,04	5,03	4,64	4,39
1000	19,56	6,98	4,89	4,07	3,76	3,56

G.4.2 Determination of $\eta_{t,\,PN}$

Assuming $P_f = 10^{-4}$, then $X = 3{,}72$ and if $y = 9$ %, then from Equation (G.7) a value of 1,503 3 for η_{hat} is obtained. In the following calculations, a value of 1,5 has been used for η_{hat}.

Using the applicable value from Table G.5 and $\eta_{hat} = 1{,}5$, the values obtained for $\eta_{t,PN}$ using Equation (G.11) are shown in Table G.5.

Table G.5 — Mean values for $\eta_{t,PN}$

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	1,56	1,68	1,77	1,84	1,87	1,90
4000	1,56	1,68	1,77	1,83	1,87	1,89
2000	1,56	1,70	1,80	1,87	1,91	1,94
1000	1,58	1,75	1,89	1,99	2,04	2,09

When designing polyester GRP pipes, the value for $\eta_{t,PN}$ is related to the 97,5 % LCL and $\eta_{t,PN(97,5\ \%LCL)}$ can be obtained from Equation (G.19):

$$\eta_{t,PN(97,5\%LCL)} = \eta_{t,PN(mean)} \times \left(1 - y \times 0{,}01 \times 1{,}96\right) \tag{G.19}$$

Using the values given in Table G.5 and y = 9 % in Equation (G.19), the resulting calculated values for $\eta_{t,PN(97,5\ \%LCL)}$ are shown in Table G.6

Table G.6 — Values for $\eta_{t,\ PN\ (97,5\ \%LCL)}$

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	1,28	1,38	1,46	1,51	1,54	1,57
4000	1,28	1,38	1,45	1,51	1,54	1,56
2000	1,29	1,40	1,48	1,54	1,57	1,60
1000	1,30	1,44	1,55	1,64	1,68	1,72

G.5 Example of simplified probabilistic calculation

In Figure G.1, the curve $f_{R,A}(x)$ is the distribution for an undeflected pipe above ground whilst the curve $f_{R,B}(x)$ is the distribution for the same pipe in a deflected condition underground, using in both cases the following parameters:

Safety index, for a probability of failure at 50 years of 1 in 10 000 X = 3,72

Coefficient of variation of the pipe's tensile strength, in % y = 9 %

Nominal stiffness SN = 8 000

Nominal pressure, in bars PN = 10 bar

Working pressure, in bars p = 10 bar

From Table G.5, the factor of safety in hoop tension for a buried SN 8000, PN 10 pipe operating at 10 bars $\eta_{t,PN}$ = 1,77 PN

From Table G.4, the factor of safety in flexure after rerounding for a buried SN 8000, PN 10 pipe operating at 10 bars η_{ff} = 6,51

Mean value for the strength of the pipe underground $\mu_{Res,B} = 1{,}77 \times (1 - \frac{1}{6{,}51}) \times PN \approx 1{,}5 \times PN$

Combined factor of safety $\eta_{hat} = \eta_{t,PN} \times \left(1 - \frac{1}{\eta_{ff}}\right) = 1{,}77 \times \left(1 - \frac{1}{6{,}51}\right)$

$\eta_{hat} \approx 1{,}5$

Mean value of the pressure applied to the pipe underground $\mu_{S,B} = (1 - 3{,}72 \times 9 \times 0{,}01) \times 1{,}5 \times PN \approx PN$

The pressure which can be applied to the pipe underground $\mu_{Res,B}/\eta_{hat} = 1{,}5 \times PN / 1{,}5 = PN$

G.6 Examples using some application limits from AWWA M-45

G.6.1 Assumptions

The procedure described in Clause G.4 is followed, using the following assumptions:

a failure probability of 10^{-4} is required, which means that X is 3,72;

values for D_f are taken from 5.2, Table 4, for 3 % initial deflection, but adjusted for the preferred ISO stiffness classes and based on the pipe being installed in gravel with dumped to slight compaction;

the coefficient of variation, y, is 9 %; and

the maximum deflection is 5 %.

Assuming that the 5 % permitted maximum deflection is a 97,5 % UCL value, the mean long-term ultimate ring deflection has to be calculated.

Assuming a standard deviation of 1 % deflection, the "theoretical" mean deflection is 5 % – 1,96 × 1 % = approx 3 %. This is equivalent to the requirement that the long-term deflection must not exceed 5 % for 97,5 % of the time.

G.6.2 Determination of η_{ff}

The values for the parameters to be used in these calculations are summarized in Table G.7.

Table G.7 — Parameters

SN	Standards values $\frac{y_{u,wet,x}}{d_m}$	Mean value $\left(\frac{d_v}{d_m}\right)_{ult}$	Calculated D_g	D_f (see Table 4)
8000	9,7 %	11,24 %	3,84	3,44
4000	12,2 %	14,13 %	3,73	4,00
2000	15,4 %	17,84 %	3,61	4,79
1000	19,4 %	22,47 %	3,46	5,90

Using the values given in Table G.7 in Equation (G.18) for $\left(\frac{d_v}{d_m}\right)_{ult}$, D_g and D_f, the mean values for η_{ff} shown in Table G.8 for a range of pressure and stiffness classes are obtained.

Table G.8 — Values for η_{ff} (mean)

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	25,07	8,95	6,27	5,22	4,82	4,56
4000	26,38	9,42	6,60	5,50	5,07	4,80
2000	26,87	9,60	6,72	5,60	5,17	4,89
1000	26,35	9,41	6,59	5,49	5,07	4,79

G.6.3 Determination of $\eta_{t,PN}$

As a probability of failure of $P_f = 10^{-4}$ is required, then $X = 3{,}72$ and, as $y = 9$ %, then from Equation (G.7) a value of 1,503 3 for η_{hat} is obtained. In the following calculations, a value of 1,5 has been used for η_{hat}.

Using the applicable value from Table G.8 and $\eta_{hat} = 1{,}5$, the values obtained for $\eta_{t,PN}$ using Equation (G.11) are shown in Table G.9.

Table G.9 — Mean values for $\eta_{t,PN}$

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	1,56	1,69	1,78	1,86	1,89	1,92
4000	1,56	1,68	1,77	1,83	1,87	1,90
2000	1,56	1,67	1,76	1,83	1,86	1,89
1000	1,56	1,68	1,77	1,83	1,87	1,90

Using the values given in Table G.9 and $y = 9$ % in Equation (G.19), the resulting calculated values for $\eta_{t,PN(97,5\ \%LCL)}$ are shown in Table G.10.

Table G.10 — Values for $\eta_{t,PN(97,5\ \%LCL)}$

SN	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
8000	1,29	1,39	1,47	1,53	1,56	1,58
4000	1,28	1,38	1,46	1,51	1,54	1,56
2000	1,28	1,38	1,45	1,50	1,53	1,55
1000	1,28	1,38	1,46	1,51	1,54	1,56

G.7 Conclusion

Comparing the values for $\eta_{t,PN(97,5\ \%LCL)}$ shown in Tables G.6 and G.10 gives the following highest values which satisfy the requirements for the applicable pressure class.

Table G.11 — Highest values for $\eta_{t,PN(97,5\ \%LCL)}$ from Tables G.6 and G.10

PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
1,30	1,44	1,55	1,64	1,68	1,72

The highest values for $\eta_{t,PN,\ mean}$ obtained from Tables G.5 and G.9 are shown in Table G.12.

Table G.12 — Highest values for $\eta_{t,PN,\ mean}$ from Tables G.5 and G.9

PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
1,58	1,75	1,89	1,99	2,04	2,09

Tables G.11 and G.12 both show that η_{hat} is not less than 1,5 for a pipe installed in the ground with a mean initial deflection of 3 % and a probability of failure not greater than 10^{-4} when the pipe is operating at an internal pressure equal to its nominal pressure rating. It is therefore recommended that the applicable values given in Table G.13 be included in GRP product standards as the minimum factors of safety to be applied to the long-term 97,5 % LCL and mean value, respectively, when the pipes are required to operate at their nominal pressure rating when installed in the ground with a maximum average initial deflection of 3 %.

Table G.13 — Recommended minimum factors of safety for inclusion in GRP product standards

Property to which factor of safety is to be applied	PN 32 [a]	PN 25	PN 16	PN 10	PN 6	PN 4	PN 2,5
Minimum factor of safety to be applied to long-term 97,5 % LCL ($\eta_{t,PN,97,5\ \%\ LCL}$)	1,3	1,3	1,45	1,55	1,6	1,65	1,7
Minimum factor of safety to be applied to long-term mean value ($\eta_{t,PN,mean}$)	1,6	1,6	1,8	1,9	2,0	2,05	2,1

[a] The pressure class PN 32 has been included to cover the minimum factor of safety that can be applied to pressure classes of PN 32 or greater.

This recommendation is based on the following:

the design satisfies the requirement that the long-term probability of failure will be at least 1:10 000 at 50 years;

the coefficient of variation in flexure is assumed to be 9 %;

the coefficient of variation in tension is assumed to be 9 %. If the actual value is higher than this then the factors of safety $\eta_{t,PN,\ mean}$ and $\eta_{t,PN,(97,5\ \%\ LCL)}$ must be increased;

the factor of safety in flexure is 1,5 based upon a coefficient of variation of 9 % and a required minimum probability of failure of 1:10 000;

the minimum value for the tensile factor of safety $\eta_{t,PN(97,5\ \%\ LCL)}$ is 1,3;

the minimum value for the tensile factor of safety $\eta_{t,PN,\ mean}$ is 1,5;

these factors of safety take into account the rerounding of the pipe from an assumed average 3 % initial deflection.

As can be seen from this part of ISO 10465, a comparison between the ATV type design and AWWA design shows that the safety incorporated into both of these static calculation systems is almost identical.

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