TECHNICAL REPORT

ISO/TR 10465-3

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Underground installation of flexible glass-reinforced thermosetting resin (GRP) pipes —

Part 3:
Installation parameters and application limits

Installation enterrée de canalisations flexibles en plastique renforcé de fibres de verre/résine thermodurcissable (PRV) —

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

Reference number
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Contents

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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.

The main task of ISO technical committees is to prepare International Standards, but in exceptional circumstances a technical committee may propose the publication of a Technical Report of one of the following types:

— type 1, when the required support cannot be obtained for the publication of an International Standard, despite repeated efforts;

— type 2, when the subject is still under technical development or where for any other reason there is the future but not immediate possibility of an agreement on an International Standard;

— type 3, 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).

Technical Reports of types 1 and 2 are subject to review within three years of publication, to decide whether they can be transformed into International Standards. Technical Reports of type 3 do not necessarily have to be reviewed until the data they provide are considered to be no longer valid or useful.

ISO/TR 10465-3, which is a Technical Report of type 2, 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*.

The reasons which led to the decision to publish this document in the form of a type 2 Technical Report are explained in the introduction.

ISO/TR 10465 consists of the following parts, under the general title *Underground installation of flexible glass-reinforced thermosetting resin (GRP) pipes:*

— *Part 1: Installation procedures*

— *Part 2: Comparison of static calculation methods*

— *Part 3: Installation parameters and application limits*

This document 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.

Introduction

Work in ISO/TC 5/SC 6 (now ISO/TC 138) on writing 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 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, twenty-eight WG 4 meetings have been held which have considered the following areas:

— procedures for the underground installation of GRP pipes;

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

— minimum design features;

— an 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. Therefore WG 4 agreed that all documents should be made into a three-part type 2 Technical Report, of which this is part 3.

Part 1 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.

Part 2 presents a comparison of the two primary methods used internationally for static calculations on underground GRP pipe installations (ATV-A 127 and AWWA M-45).

Part 3 gives additional information, which is useful for static calculations when using an ATV-A 127 type design system in accordance with part 2 of this Technical Report, on items such as:

— parameters for deflection calculations;

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

— soil moduli and pipe stiffnesses 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.

Underground installation of flexible glass-reinforced thermosetting resin (GRP) pipes —

Part 3:
Installation parameters and application limits

1 Scope

This part of ISO/TR 10465 gives information on parameters and application limits for the installation of GRP pipes. It is particularly relevant when using an ATV-A 127 type design system.

Explanations of the long-term safety factors incorporated in the GRP system standards, based on simplified probability methods, are given in annex G.

2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this part of ISO/TR 10465. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this part of ISO/TR 10465 are encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references, the latest edition of the normative document referred to applies. Members of ISO and IEC maintain registers of currently valid International Standards.

ISO/TR 10465-1:1993, *Underground installation of flexible glass-reinforced thermosetting resin (GRP) pipes — Part 1: Installation procedures.*

ISO/TR 10465-2:1999, *Underground installation of flexible glass-reinforced thermosetting resin (GRP) pipes — Part 2: Comparison of static calculation methods.*

ASTM D 1586:1984, *Standard test method for penetration test and split-barrel sampling of soils.*

ASTM D 2166:1991, *Standard test method for unconfined compressive strength of cohesive soil.*

ATV-A 127, *Guidelines for static calculations on drainage conduits and pipelines* (December 1988).

AWWA M-45, *Fiberglass pipe design manual M-45* (1997).

BS 1377 (all parts)*, Methods of test for soils for civil engineering purposes.*

DIN 19565-1:1989, *Centrifugally cast and filled polyester resin glass fibre reinforced (UP-GF) pipes and fittings for buried drains and sewers; dimensions and technical delivery conditions.*

OENORM B 4419-1:1985, *Erd- und Grundbau; Untergrunderkundung durch Sondierungen; Rammsondierungen.*

OENORM B 5012-1:1990, *Statische Berechnung erdverlegter Rohrleitungen im Siedlungs- und Industriewasserbau; Grundlagen.*

WRc, Water Research Centre, Swindon, UK: *Pipe materials selection manual — Water supply,* 2nd edition, June 1995.

3 Terminology

Pipeline installation terminology can vary around the world so, where such terms are used in this part of ISO/TR 10465, they will either be described or reference will be made to part 1 or 2, where the relevant descriptions can be found.

4 Symbols and abbreviated terms

For the purposes of this part of ISO/TR 10465, the following symbols apply:

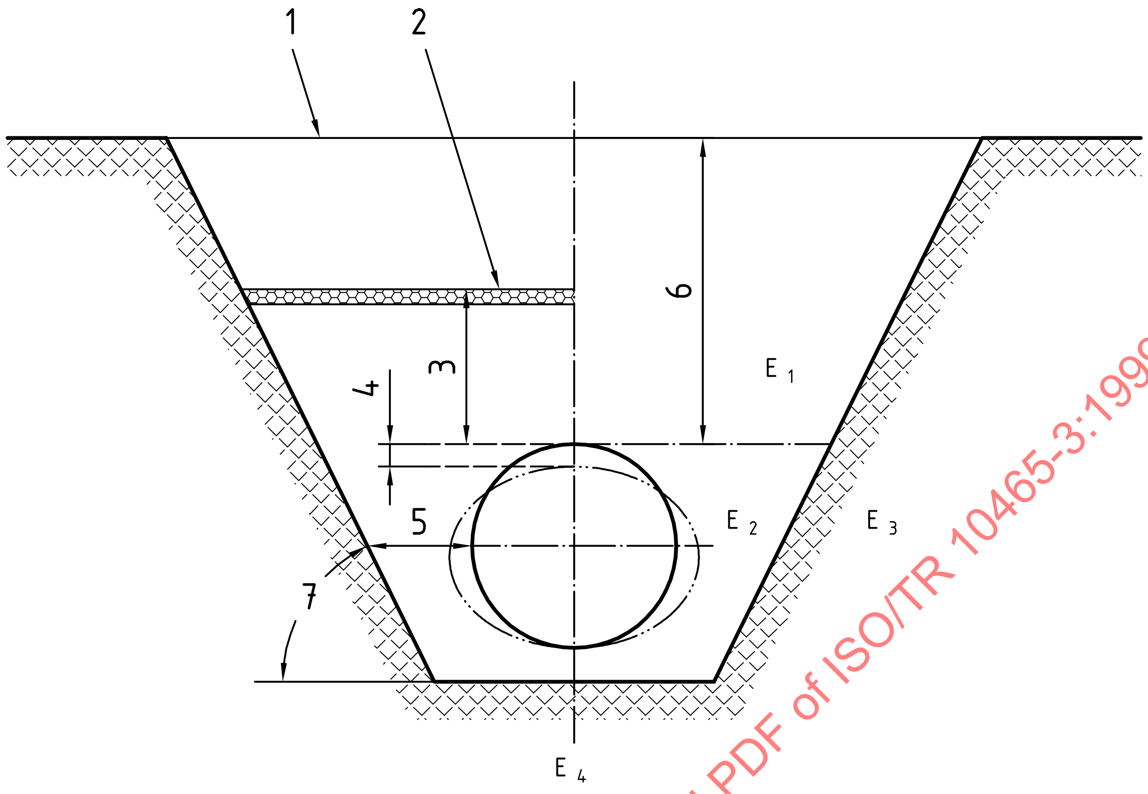
NOTE This clause also contains symbols and abbreviations from parts 1 and 2 for completeness.

Symbol	**Unit**	**Meaning**
a_f	—	Ageing factor
a_f	—	Distribution factor
B'	—	Support factor
b	m	Trench width at spring-line
b'	m	Distance from trench wall to pipe (see Figure 1)
c_4	—	Reduction factor
c_f	—	Creep factor
c_h, c_v	—	Deformation coefficients
D_f	—	Shape factor
D_g	—	Shape adjustment factor
D_L	—	Deflection lag factor
d_e	m	External pipe diameter
d_m	mm	Mean pipe diameter $[(d_e \times 1\,000) - e]$
d_v	mm	Vertical deflection
d_{vA}	m	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, $E_{t,wet}$	N/m^2	Apparent flexural moduli of pipe wall
E', E_1, E_2, E_3, E_4, E'_s, E'_t, E_s	N/mm^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	—	Safety factor
FS_b	—	Bending safety factor
FS_{pr}	—	Pressure safety factor
HDB	—	Extrapolated pressure strain at 50 years
H_{EVD}	m	Environmental depth of cover

Symbol	Unit	Meaning
h	m	Depth of cover to top of pipe
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_o	—	Initial ovalization
i_f	N/mm^2	Installation factor
K^*	—	Coefficient for bedding reaction pressure
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 pressure in pipe-zone backfill, when backfill is at top of pipe (see annex A)
k_x	—	Bedding coefficient
M	—	Sum of bending moments
M_s	N/mm^2	Constrained-soil modulus
m_{qv}, m_{qh}, m^*_{qh}	—	Moment factors
N	—	Sum of normal forces
n_{10}	—	Number of blows
P	bar	Internal pressure
PN	—	Nominal pressure
$P(x)$	—	Probability function
P_f	—	Probability of failure
P_v	MPa (N/mm^2)	Internal underpressure
P_W	N/m^2	Working pressure
p_a	N/m^2	External water pressure
p_E	N/mm^2	Pressure due to prismatic soil load
p_F	N/m^2	Pressure due to traffic load according to Boussinesq
p_o	N/mm^2	Soil pressure due to distributed 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_{cs}	MPa (N/mm^2)	Short-term critical buckling pressure
q_{cl}	MPa (N/mm^2)	Critical buckling pressure under sustained load
q_{cw}	MPa (N/mm^2)	Critical buckling pressure due to water
q_h, q_v	N/mm^2	Horizontal and vertical soil pressure on pipe
q_h^*	N/mm^2	Horizontal bedding reaction pressure
$q_{h,50}$	N/mm^2	Long-term (50-year) horizontal soil pressure
q_{hLT}	N/mm^2	Reduced long-term horizontal soil pressure
q_{c^*w}	N/mm^2	Horizontal bedding reaction for pipe and contents
$q_{v,50}$	N/mm^2	Long-term (50-year) vertical soil pressure
q_{vLT}	N/mm^2	Reduced long-term vertical soil pressure
q_{vwa}	N/mm^2	Vertical load due to pipe and contents
R_w	—	Water buoyancy reduction factor
r	—	Rerounding factor

Symbol	Unit	Meaning
r_A, r_E	m	Wheel radii
r_c	—	Rerounding coefficient
S_{Bh}	N/mm^2	Horizontal bedding stiffness
S_{Bv}	N/mm^2	Vertical bedding stiffness
S_b	—	Long-term strain
S_c	—	Soil support combining factor
S_k	N/mm^2	Characteristic stress
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
$s_{Res,A}$	N/mm^2	Standard deviation of strength of pipe above ground
$s_{Res,B}$	N/mm^2	Standard deviation of strength of pipe below ground
s_S	N/mm^2	Standard deviation of stress in pipe
$s_{S,A}$	N/mm^2	Standard deviation of stress in pipe above ground
$s_{S,B}$	N/mm^2	Standard deviation of stress in pipe below ground
SPD	%	Standard Proctor density
V_{RB}	—	System stiffness
V_S	—	Stiffness relation
W_c	N/m^2	Vertical soil load on pipe
W_L	N/m^2	Traffic load
X	—	Safety index
y_R	%	Coefficient of variation for tensile strength
y_{ult}	%	Coefficient of variation for ultimate deflection
α	° (degrees)	Half the bedding angle (see Figure 2)
β	° (degrees)	Half the horizontal support angle (see Figure 2)
χ	—	Reduction factor applied to prismatic soil load to allow for friction
χ_β	—	Reduction factor applied to prismatic soil load to allow for friction and taking into account trench angle (β in ATV and ω in this part of ISO/TR 10465)
χ_o	—	Reduction factor applied to a uniformly distributed load to allow for friction
$\chi_{o\beta}$	—	Reduction factor applied to a uniformly distributed load to allow for friction and taking into account trench angle (β in ATV but ω in this part of ISO/TR 10465)
δ	° (degrees)	Trench wall friction angle
δ_d	mm	Maximum permitted long-term installed deflection
δ_v	%	Relative vertical deflection
δ_{vio}	%	Relative vertical deflection due to backfilling in pipe zone
δ_{viv}	%	Relative vertical deflection due to installation irregularities
δ_{vs}	%	Relative vertical deflection due to soil load
δ_{vw}	%	Relative vertical deflection due to weight of pipe

Symbol	Unit	Meaning
δ_W	%	Relative vertical deflection due to traffic load
ε_{comp}	—	Compressive strain due to vertical load
ε, ε_t, ε_f	—	Calculated flexural strains in pipe wall
ε_{max}	—	Maximum permissible strain due to pressure
ε_{pr}	—	Calculated strain in pipe wall due to internal pressure
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
γ_b	MN/m^3	Bulk density of backfill material
γ_w	MN/m^3	Density of pipe contents
η, η_t, η_f, η_{ff}	—	Safety factors
η_{haf}	—	Combined flexural safety factor
η_{hat}	—	Combined tensile safety factor
φ	° (degrees)	Soil internal friction angle
κ, κ_ω	—	Reduction factor for distributed load according to silo theory when trench angle (ω) is 90°
κ_o, $\kappa_{o\omega}$	—	Reduction factor for distributed load according to silo theory when trench angle (ω) is not 90°
λ_B	—	Concentration factor in soil next to pipe
λ_{max}	—	Maximum concentration factor
λ_R, λ_{RG}, λ_{max}	—	Concentration factors for soil above pipe
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
μ_S	N/mm^2	Mean value of stress in pipe
$\mu_{S,A}$	N/mm^2	Mean value of stress in pipe above ground
$\mu_{S,B}$	N/mm^2	Mean value of stress in pipe below ground
ρ	MN/m^3	Density of pipe-wall material
ρ_D	g/cm^3	Density
σ_c	N/mm^2	Calculated compressive stress in pipe wall
σ_t	N/mm^2	Calculated tensile stress in pipe wall
ν_s	—	Poisson's ratio for soil
ω	° (degrees)	Trench wall angle (see Figure 1) (designated β in ATV-A 127)
ξ	—	Correction factor for horizontal bedding



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, ω

NOTE 1 The AWWA M-45 design manual uses E'_b in zone E_2.

NOTE 2 The AWWA M-45 design manual uses E'_n in zone E_3 and E_4.

NOTE 3 E_1 is the backfill above the pipe zone (E_2) material.

NOTE 4 E_2 is the embedment material to the side of the pipe.

NOTE 5 E_3 is the *in situ* trench wall material.

NOTE 6 E_4 is the *in situ* material underlying the pipe zone material (foundation material).

NOTE 7 In ATV-A 127, β is used for the trench wall angle instead of ω.

Figure 1 — Symbols and terminology

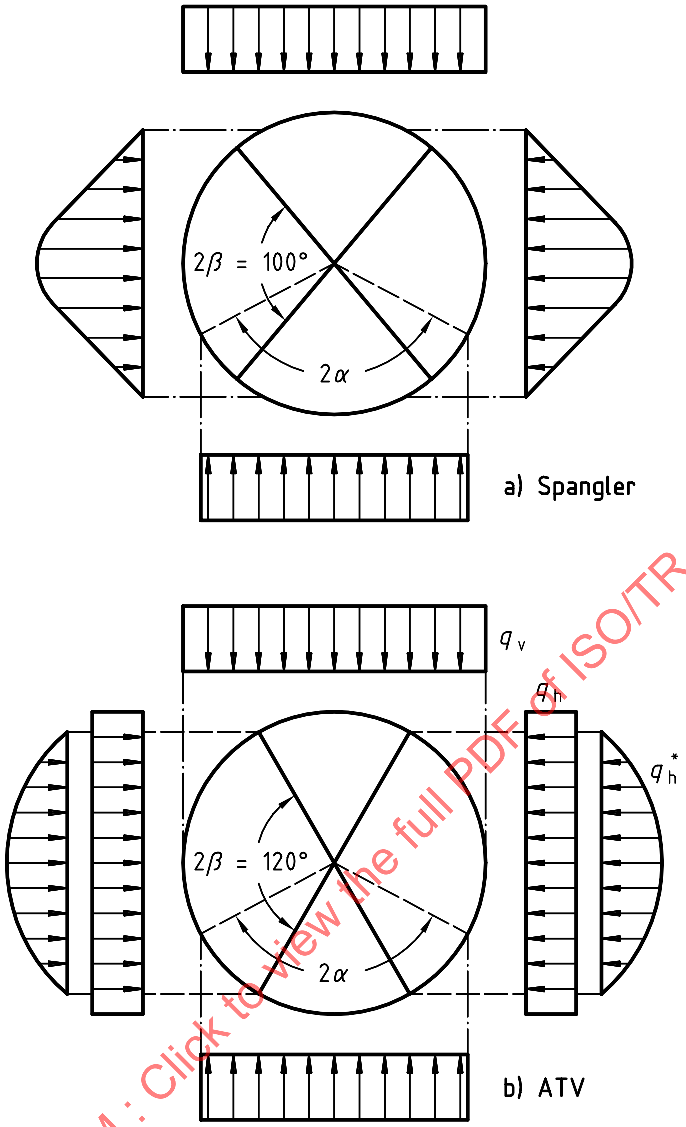


Figure 2 — Soil distribution according to Spangler and ATV-A 127

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

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

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

5.1 Initial deflection

Measurement of the initial deflection shortly after installation, when the effects of traffic loads are not present, is a very easy way of assessing the quality of the installation. The initial deflection should therefore be determined under these loading conditions.

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

5.1.1 Deflection due to vertical soil load and superimposed loads, but excluding traffic loads

The relative vertical deflection δ_v, given by $\delta_{vs} = \frac{d_v}{d_m}$ (% deflection when multiplied by 100), is determined using equation (1):

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

$$\delta_{vs} = \frac{d_v}{d_m} = \left[c_{v1} + \left(c_{v2} \times K^*\right)\right] \times \left(q_v - q_h\right) \times \frac{1}{S_R} \quad (1)$$

where

d_v is the vertical deflection of the pipe, in mm;

d_m is the mean diameter of the pipe, $(d_e \times 1\,000) - e$, in mm;

e is the pipe wall thickness;

$$K^* = \frac{c_{h1}}{V_{RB} - c_{h2}} \quad (2)$$

c_{v1}, c_{v2}, c_{h1}, c_{h2} are deflection coefficients (see annex C);

$$V_{RB} = S_R / S_{Bh} \quad (3)$$

$$S_R = S_p \times 8 \times 10^{-6} \quad \text{(in N/mm}^2\text{)} \quad (4)$$

S_p is the initial pipe stiffness, in N/m^2;

$$S_{Bh} = c_4 \times \xi \times E_2 \quad \text{(in N/mm}^2\text{)} \quad (5)$$

c_4 = 0,6 in ATV-A 127

ξ is a correction factor, given by:

$$\xi = \frac{1{,}44}{f + (1{,}44 - f)\, E_2 / E_3} \quad (6)$$

in which $$f = \frac{\left(\frac{b}{d_e} - 1\right)}{1{,}154 + 0{,}444\left(\frac{b}{d_e} - 1\right)} \leqslant 1{,}44 \quad (7)$$

NOTE The correction factor ξ takes into account the difference in the soil moduli of the pipe embedment material and the native soil, as well as the width of the trench. The above equations are those given in ATV-A 127 for a support angle of 120°, but it is recommended that the equations and values given in annex D be used. Annex D covers a wider range of support conditions than the 120° covered by equation (6). Despite appearances, the equations in annex D for 120° produce a very similar answer to that obtained using equation (6).

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

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

q_v is the vertical pressure due to the soil loads, calculated using equation (8):

$$q_v = (\kappa \times \gamma_b \times h + \kappa_o \times p_o) \times \lambda_{RG} \quad \text{(in N/mm}^2\text{)} \tag{8}$$

NOTE Equation (8) uses values in MN/m^2 and N/mm^2 which are numerically equivalent.

h is the depth of cover, in m;

γ_b is the bulk density of the backfill above the pipe, in MN/m^3;

p_o is the soil pressure due to the distributed load at the surface, in N/mm^2;

κ and κ_o are trench friction coefficients (see ISO/TR 10465-2 or ATV-A 127, as well as annex F of this document);

q_h is the horizontal pressure due to soil loads, calculated using equation (9):

$$q_h = K_2[(\kappa \times \gamma_b \times h + \kappa_o \times p_o) \times \lambda_B + (\gamma_b \times d_e/2)] \quad \text{(in N/mm}^2\text{)} \tag{9}$$

NOTE Equation (9) uses values in MN/m^2 and N/mm^2, which are numerically equivalent.

K_2 is the ratio of the horizontal to the vertical soil pressure in soil zone 2 (see annex A);

λ_B is a concentration factor (see annex B), given by:

$$\lambda_{rg} = \left(\frac{\lambda_r - 1}{3} \times \frac{b}{d_e}\right) + \frac{4 - \lambda_R}{3} \tag{10}$$

NOTE Experience shows that the limits given for λ_{RG} for GRP pipes in ATV-A 127 are not normally reached.

b is the trench width, in m;

d_e is the outside diameter of the pipe, in m;

λ_R is a concentration factor for the the soil above the pipe (see annex B).

5.1.2 Deflection due to weight of pipe

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 relative deflection resulting from the pipe's own weight, calculated using equation (11):

$$\delta_{vw} = -2{,}3 \times e \times \rho \times 10^{-4} \times \frac{1}{S_R} \tag{11}$$

where

e is the pipe wall thickness, in mm;

ρ is the density of the pipe-wall material, in MN/m^3.

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

5.1.3 Deflection due to compaction of pipe zone backfill (initial ovalization)

When the pipe zone backfill material is 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 (12):

$$\delta_{vio} = K_3 \times \gamma_b \times \frac{d_e}{24 \times S_R} \tag{12}$$

where

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

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

5.1.4 Deflection resulting from installation irregularities

There are many different approaches to allow for the effect of the inevitable variation in initial deflection due to irregularities in the installation. Most of these approaches are based on the “add on a few percent“ principle. Several publications (see references [3], [6] and [7]) state that it is not possible to allow 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, must consider pipe stiffness, pipe diameter and soil conditions. The calculated deflection is then used to estimate the corresponding flexural strain.

Equation (13) enables an estimate of the relative deflection due to installation irregularities to be made:

$$\delta_{viv} = [c_{v1} + (c_{v2} \times K^*)] \times i_f / S_R \tag{13}$$

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

Values of i_f are given in Table 1.

Values of c_{v1} and c_{v2} are given in annex C.

Table 1 — Values of 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

5.1.5 Total initial relative deflection

The estimated initial deflection is determined using equation (14):

$$\left(\frac{d_v}{d_m}\right)_{\text{initial}} = (\delta_{vs} + \delta_{vw} + \delta_{vio} + \delta_{viv}) \tag{14}$$

where

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

δ_{viv} is the negative relative vertical deflection caused by installation irregularities;

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

δ_{vw} is the negative relative vertical deflection caused by the weight of the pipe.

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

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

The calculated long-term deflection will vary depending whether silo theory or prismatic load is used to calculate the vertical load.

5.2.1 Soil friction remaining

In ATV-A 127, silo theory is used.

The soil pressure due to the traffic load, p_v, which is transient and not sustained, must be added to q_v but not multiplied by $\lambda_{RG,50}$ to obtain the long-term vertical soil pressure $q_{v,50}$, as shown in equation (15):

$$q_{v,50} = [(\kappa \times \gamma_b \times h + k_o \times p_o) \times \lambda_{RG,50}] + p_v \qquad (15)$$

where

p_o is the soil pressure, in N/mm^2, due to the distributed load;

p_v is the soil pressure, in N/mm^2, due to the traffic load.

Rewriting equations (9) and (1) gives equations (16) and (17) for the calculation of the the long-term horizontal soil pressure $q_{h,50}$ and the long-term relative vertical deflection $\delta_{vs,50}$ due to the soil pressure:

$$q_{h,50} = K_2[(\kappa_{50} \times \gamma_b \times h\lambda_{B,50}) + (\kappa_{o,50} \times p_o\lambda_{B,50}) + (\gamma_b \times d_e/2)] \qquad (16)$$

$$\delta_{vs,50} = [c_{v1} + (c_{v2} \times K^*)] \times (q_{v,50} - q_{h,50}) \times 1/S_{R,50} \qquad (17)$$

$\lambda_{RG,50}$ is calculated using equation (10) and $\lambda_{B,50}$ using equation (B.5) in annex B, except that the long-term soil moduli from annex A are used and the long-term pipe stiffness $S_{R,50}$ is used instead of S_R.

To obtain the total long-term relative deflection $(d_v/d_m)_{50}$, the **initial** deflections due to the weight of the pipe, the initial ovalization and the installation irregularities should be added to the long-term deflection $\delta_{v,50}$ as shown in equation (18):

$$(d_v/d_m)_{50} = \delta_{v,50} + \delta_{vw} + \delta_{vio} + \delta_{viv} \qquad (18)$$

NOTE For sign convention, see 5.1.5.

5.2.2 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 in soil load remains for the whole installed lifetime of the pipe. If, however, it is necessary to use the prismatic-loading approach, which ignores any soil friction, then the long-term deflection is obtained by putting $\lambda_{RG,50} = \lambda_{B,50} = \kappa = \kappa_o = 1$ in equations (15) to (17).

5.2.3 Soil friction partly remaining

There is a large difference in the result obtained when silo theory, on the one hand, or prismatic soil load, on the other, 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. This depth is defined as the depth down to which the soil friction has been lost due to frost, rain and traffic loads, and can be up to 3 m. Full prismatic loading is used for this part of the depth of cover, and silo theory for the rest.

Annex F describes how values of $q_{v,50}$ and $q_{h,50}$ can be calculated using this system. The new values, called q_{vLT} and q_{hLT}, are then used in equation (17) instead of $q_{v,50}$ and $q_{h,50}$, respectively.

6 Soil parameters, strain coefficients and shape factors for flexural-strain calculations

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

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

6.1.1 Calculation of flexural strain due to vertical soil load

The flexural strain caused by the vertical soil load can be calculated using equation (19):

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

$$\varepsilon_{\mathrm{v}} = \{(m_{\mathrm{qv}} \times q_{\mathrm{v}}) + (m_{\mathrm{qh}} \times q_{\mathrm{h}}) + [m_{\mathrm{qh}}{}^* \times K^* \times (q_{\mathrm{v}} - q_{\mathrm{h}})]\} \times \frac{e}{d_{\mathrm{m}} \times S_{\mathrm{R}}} \qquad (19)$$

where

values for m_{qv}, m_{qh} and $m_{\mathrm{qh}}{}^*$ are given in annex C;

$$K^* = \frac{c_{\mathrm{h1}}}{V_{\mathrm{RB}} - c_{\mathrm{h2}}} \qquad (20)$$

c_{h1} and c_{h2} are horizontal deflection coefficients (see annex C);

NOTE The values of these coefficients depend on the value of the vertical soil-reaction angle 2α and horizontal soil-reaction angle 2β (see Figure 2). **The value selected for 2α and 2β has much more effect on the calculated flexural strain than on the calculated deflection (see 6.2).**

q_{v} and q_{h} are calculated, as described in clause 5, for initial and long-term deflection.

When calculating long-term deflections, see annex E regarding $S_{\mathrm{R,50}}$.

6.1.2 Flexural strain due to weight of pipe

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 weight of the pipe is calculated using equation (21):

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

$$\varepsilon_{\mathrm{vw}} = 0{,}000441 \times \rho \times \frac{e^2}{d_{\mathrm{m}}} \times \frac{1}{S_{\mathrm{R}}} \qquad (21)$$

where ρ is the density of the pipe-wall material, in MN/m^3.

6.1.3 Flexural strain due to 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 (22), which is based on equation (12):

$$\varepsilon_{\mathrm{vio}} = -\,0{,}025 \times K_3 \times \gamma_{\mathrm{b}} \times \frac{d_{\mathrm{e}} \times 10^{-3}}{2} \times \frac{e}{d_{\mathrm{m}} \times S_{\mathrm{R}}} \qquad (22)$$

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

6.1.4 Compressive strain due to vertical load

The compressive strain ε_{comp} caused by vertical loads is calculated using equation (23), which is a simplification of the equations used in ATV-A 127 for estimating the strain at the bottom of the pipe:

NOTE This compressive strain has a negative value.

$$\varepsilon_{comp} = -\left(q_h \times \frac{3 \times Y^2}{S_R}\right) - \left(0{,}577 \times q_h{}^* \times \frac{3 \times Y^2}{S_R}\right) \qquad (23)$$

where

$$Y = \frac{e}{d_m} \qquad (24)$$

$$q_h{}^* = K^* \times (q_v - q_h) \qquad (25)$$

K^* is defined in equation (2).

6.1.5 Flexural strain due to contents of pipe

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 those 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 (26) be used instead of the ATV equation:

NOTE The flexural strain calculated from equation (26) has a positive value, which indicates a tensile strain resulting from a reduction in vertical diameter.

$$\varepsilon_W = q_{hw}{}^* \times \frac{e}{d_m \times S_R} \times \left[m_{qv} + \left(m_{qh} \times K^*\right)\right] \qquad (26)$$

where

$$q_{hw}{}^* = q_{vwa} \times K^* \qquad (27)$$

$$q_{vwa} = 0{,}5 \times \gamma_W \times \pi \times r_i \qquad (28)$$

K^* is defined in equation (2);

γ_W is the density of the pipe contents, in MN/m^3.

6.1.6 Flexural strain due to installation irregularities

The flexural strain due to installation irregularities is calculated using equation (29):

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

$$\varepsilon_{if} = i_f \times (0{,}25 + m_{qh}{}^* \times K^*) \, \frac{e}{d_m \times S_R} \qquad (29)$$

where

i_f is as given in Table 1;

$m_{qh}{}^*$ is as given in annex C;

K^* is defined in equation (2).

6.1.7 Total flexural strain

The total flexural strain, in percent, is the sum of the strains calculated in 6.1.1 to 6.1.6:

$$\varepsilon_{tot} = (\varepsilon_v + \varepsilon_{vw} + \varepsilon_{vio} + \varepsilon_{comp} + \varepsilon_w + \varepsilon_{if}) \times 100 \quad (30)$$

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

6.2 Shape factor D_f

When buried flexible pipes deflect, they deform into non-elliptical shapes, and 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 method, specify values for D_f. The ATV-A 127 system does not use D_f values but derives pipe strains based on 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 on the value of D_f.

6.2.1 Derivation of D_f using ATV-A 127 equations

Although D_f is not used in ATV-A 127, values of D_f can be calculated using equation (31):

$$\text{Strain} \times \frac{d_m}{e} \times \frac{1}{\text{deflection}} = D_f \quad (31)$$

Using this procedure, D_f has been calculated across a range of values of the bedding angle 2α and support angle 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 has shown that, for any given pipe stiffness, bedding angle 2α and support angle 2β, the value of D_f can be calculated from an equation of the form:

$$D_f = A + B/(\%\ \text{deflection})$$

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 2.

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 of D_f can then be derived using equation (31) as described above (see also reference [10]).

From measurements made on pipes in soil boxes, where the pipe zone material had been placed carefully either without compaction or only slightly compacted, the calculated value of D_f indicates 2α and 2β as being equal to 180° (see references [1] and [2]). Similar calculations made using the results from measurements made 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 2 — Calculated D_f values

Nominal pipe 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

6.2.2 D_f values in AWWA design manual M-45

Table 3 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 3 and the calculated values given in Table 2 shows a similarity in values, especially when pipes are embedded in either dumped or slightly compacted backfill. The differences in the nominal stiffness values in Tables 2 and 3 do not, for the purposes of this comparison, significantly affect the D_f values.

Table 3 — D_f values for different pipe-zone backfill materials and degrees of compaction (from AWWA M-45)

Nominal pipe stiffness	Gravel		Sand	
	Degree of compaction			
	Dumped to slight	Moderate to slight	Dumped to slight	Moderate to high
SN 1250	5,5	7,0	6,0	8,0
SN 2500	4,5	5,5	5,0	6,5
SN 5000	3,8	4,5	4,0	5,5
SN 10000	3,3	3,8	3,5	4,5

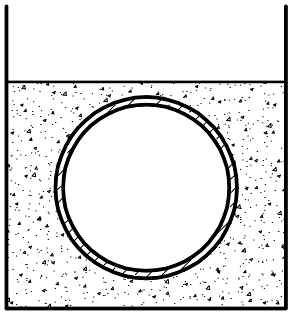
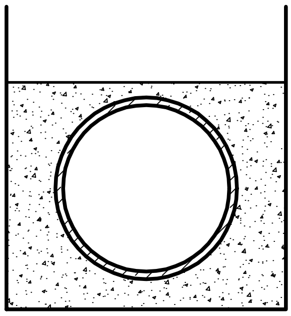
6.2.3 D_f values used in WRc Pipe materials selection manual

D_f values recommended for use in the WRc design system are shown in Tables 4 and 5.

Table 4 — WRc D_f values for various installation conditions and pipe stiffnesses

Embedment class (see Table 5)	Degree of compaction % MPD	D_f for various nominal pipe stiffnesses			
		SN 1250	SN 2500	SN 5000	SN 10000
Class S1	Uncompacted	4,7	4,5	4,3	4,0
	80	4,7	4,5	4,3	4,0
	85	4,7	4,5	4,3	4,0
	90	4,7	4,5	4,3	4,0
Class S2	Uncompacted	4,7	4,5	4,3	4,0
	80	4,7	4,5	4,3	4,0
	85	4,7	4,5	4,3	4,0
	90	4,7	4,5	4,3	4,0
Class S3	80	3,50	3,40	3,20	3,10
	85	6,20	5,50	4,75	4,25
	90	7,75	6,60	5,50	4,70
Class S4	85	6,20	5,50	4,75	4,25
	90	7,75	6,60	5,50	4,70

Table 5 — 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

7 Influence of soil modulus 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:

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

ATV-A 127 has the following equation for calculating the critical pressure under vacuum conditions (see ATV-A 127, equation 9.05):

$$\text{crit. } q_\text{v} = 2\left(S_\text{R} \times S_\text{Bh}\right)^{0,5} \qquad (32)$$

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 (33)] be used:

$$q_\text{c} = 2(E'_\text{s} \times S_\text{R})^{0,5} \qquad (33)$$

where

$$E'_\text{s} = M_\text{s} \times \frac{\left(1 + \nu_\text{s}\right) \times \left(1 - 2\nu_\text{s}\right)}{\left(1 - \nu_\text{s}\right)} \qquad (34)$$

M_s is the constrained-soil modulus (see annex A) of the pipe-zone backfill;

ν_s is Poisson’s ratio for the pipe-zone backfill (see annex A).

7.2 Long-term buckling under sustained external loads

When calculating elastic buckling under the effects of sustained external loads, it should be remembered that the influence of the native soil will be greater due to the increase in 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 (35) is recommended:

$$q_{cl} = 2(E'_s \times \xi \times S_R)^{0,5} \tag{35}$$

where ξ is a correction factor (see 5.1.1 and annex D).

7.3 Value of S_R

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

8 Parameters for rerounding and combined-loading calculations

8.1 Rerounding

To allow for the fact that GRP pipes, when subject to internal pressure, will attempt to reround and hence give a reduction in deflection, equation (36) has been developed from soil box tests, field measurements and literature studies.

The rerounding factor r may be obtained using equation (36):

$$r = 1 - \frac{P}{30} \tag{36}$$

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 (35), which is used in AWWA M-45, has been included here to correct this omission.

NOTE 2 The WRc *Pipe materials selection manual* uses 40 in place of the 30 in equation (36).

8.2 Combined effects of internal pressure and external bending loads

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

9 Traffic loads

9.1 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 Boussinesq's 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 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 of E_2 because high levels of compaction are likely to be employed.

9.2 Influence on permissible initial deflection

The permissible initial deflection only takes into account soil and superimposed loads, not traffic loads. Hence traffic loads have no effect on permissible initial deflection.

9.3 Soil pressure due to traffic loads

The design requirements relating to traffic-wheel loads vary from one country to another, and the relevant national codes should be used for design purposes.

10 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 methods used vary depending on the soil conditions, the equipment available and the experience of the workers.

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

It is essential, when designing an installation, that the engineer be aware of the intended method of shield extraction, so that the correct assumptions for the values of soil compaction and hence modulus can be made in the calculation.

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 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 should be such that there is no danger of the trench wall collapsing, and care should 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 will be 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 will be obtained for the degree of compaction of the backfill.

11 Safety factors for gravity pipes and pressure pipes

11.1 General

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

11.2 Gravity pipes

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

In ATV-A 127, the following flexural safety factors are specified for centrifugally cast GRP pipes:

— 1,75 for a failure probability P_f of 10^{-3}, i.e. 1 in 1 000;

— 2,00 for a failure probability P_f of 10^{-5}, i.e. 1 in 100 000.

These safety factors are not based on the manufacturer's 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 on an LCL value, such as is used in the draft product standard for polyester pipes, then a lower safety factor could be used and still give the same level of risk of failure.

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

— the semi-probability calculation which was made to arrive at these values uses a bedding angle 2α of 90°, which is extremely conservative (see 6.2.1);

— the ATV equations include an “uncertainty factor“, which increases the safety factor value from for instance 1,75 to 2,0;

— statistical data used came from three different manufacturing units, which were located in different countries.

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

The AWWA M-45 design manual specifies a safety factor of 1,5 for 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 factor 1,5 should then be to the 50-year strain corrosion value. The 50-year values are mean values derived from tests performed in accordance with ASTM test methods.

11.2.3 Recommendations

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

The long-term ultimate ring deflection values are converted into strain, in percent, using equation (37):

$$\varepsilon_{50} = D_g \times \frac{e}{d_m} \times \left(\frac{d_v}{d_m}\right)_{ult} \tag{37}$$

where

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

$$D_g = \frac{4{,}28 \times 100}{\left[1 + \left(\frac{d_v}{2 \times d_m}\right)_{ult}\right]^2} \tag{38}$$

The flexural safety factor η_f is then calculated using equation (39):

$$\eta_f = \frac{\varepsilon_{50}}{\varepsilon_{tot}} \tag{39}$$

where

ε_{50} is 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 due to pipe deflection;

η_f should be at least 1,5.

11.3 Pressure pipes

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

The ISO system standard is for both pressure and gravity pipes, but, as the installation design principles relating to combined loading are not contained within that document and the pipe design assumptions are based on minimum long-term values and not mean values (as used in M-45), it is appropriate that the combined-loading design principles should be considered. The draft system standard specifies a minimum long-term failure pressure safety factor value of 1,3 × PN which in some buried installations will not satisfy the recommended minimum combined-loading safety factor requirements for buried GRP pipe products.

Consideration has been given to the relationships between probability of failure, variations in 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 variations inherent in the manufacturing process.

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

11.3.1 Combined loading in AWWA M-45

In AWWA M-45, there are two combined safety factors, FS_{pr} and FS_b, where FS_{pr} corresponds to the global safety factor in tension η_{hat} (see 11.3.2), and they have minimum values of 1,8 and 1,5, respectively. They are used in inequalities (40) and (41) for buried pipes, and in inequality (42) for non-buried pipes:

$$\frac{\varepsilon_{pr}}{HDB} \leqslant \frac{1 - \left(\dfrac{\varepsilon_b r_c}{S_b}\right)}{FS_{pr}} \tag{40}$$

$$\frac{\varepsilon_b r_c}{S_b} \leqslant \frac{1 - \left(\dfrac{\varepsilon_{pr}}{HDB}\right)}{FS_b} \tag{41}$$

$$\frac{\varepsilon_{pr}}{HDB} \leqslant \frac{1}{FS_{pr}} \tag{42}$$

where

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

ε_{b} is the bending strain corresponding to the 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 hydraulic 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 pipelines;

— to obtain values for η_{hat}, probability principles are employed which, for the reasons given in subclause G.1.1 of annex G, cannot be used for flexural properties.

11.3.2 Recommended combined safety factors for use in ISO product standards

Combined safety factors for the tensile and flexural properties which take into account the interaction of these two properties with each other have been developed from concepts used in AWWA M-45. This combined interaction is calculated using equation (43):

$$\eta_{hat} = \eta_{t} \times \left(1 - \frac{1}{\eta_{ff}}\right) \quad (43)$$

If the safety factor $\eta_{t,PN}$ is defined relative to the nominal pressure class of the pipe, PN, then equation (44) should be used:

$$\eta_{hat} = \eta_{t,PN} \times \left(1 - \frac{1}{\eta_{ff}}\right) \times \frac{PN}{P} \quad (44)$$

where

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

$$\eta_{ff} = \frac{\eta_{f}}{r} \quad (45)$$

η_{f} is the flexural safety factor [see equation (39)];

η_{t} is the hoop-tension safety factor;

r is the rerounding factor (see 8.1);

PN and P are in bars.

The required global minimum long-term safety factor in tension, η_{hat}, can be derived from equation (46) from knowledge of variations in the product.

$$\eta_{hat} = \frac{1}{1 - X \times y \times 0{,}01} \geqslant 1{,}5 \quad (46)$$

where

X is the safety index (see Table 6);

y is the coefficient of variation in percent of the tensile strength (derived from factory production records), given by:

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

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

Table 6 — Safety index X

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

If P_f is assumed to be 10^{-4}, then, from Table 6, X is 3,72. It can be seen from equation (46) that, if the minimum global safety factor η_{hat} is 1,5, then y cannot be greater than 9 %. If y is greater than 9 %, then the safety factor η_{hat} will have to be greater than 1,5.

Safety factors in probability calculations are applied to **mean** values. When it is required that the safety factor be applied 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, then 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 safety factor η_{hat} should not be less than 1,5.

11.4 Safety factors in buckling calculations

11.4.1 Safety factors specified in AWWA M-45

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

11.4.2 Safety factors specified 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 11.2.1:

— For $P_f = 10^{-3}$: 2,0;

— For $P_f = 10^{-5}$: 2,5.

It is recommended when designing for buckling using the equations given in 7.1 that a single long-term safety factor of 2 be used, which will give a similar result 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 deflection (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 of Poisson's ratio v_s

Poisson's ratio for a soil can be determined using equation (A.1):

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

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 of K_1, K_2 and K_3

A.3.1.1 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.

A.3.1.2 K_2

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

A.3.1.3 K_3

The value to use for K_3, which is the ratio of the horizontal and vertical soil pressure 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:

$$K_3 = \frac{K_2}{8} + \frac{20}{d_e \times 1000} \times F \qquad \text{(A.2)}$$

where

$0,1 \text{ m} \leqslant d_e \leqslant 0,9 \text{ m}$;

F is the appropriate compaction factor taken from Table A.1.

NOTE For pipe sizes greater than DN 900, use 0,9 m for d_e in this equation.

Table A.1 — Values of compaction factor F

Degree of compaction	F
Heavy	3,0
Moderate	2,0
Slight	1,5
Dumped	1,0

A.3.2 Values of δ and φ

In ATV, the value of the trench-wall friction angle δ depends on factors such as the internal soil friction angle which can normally be simplified to $0,66 \times \varphi$. The values of φ can be taken from Table A.2.

Table A.2 — Values of internal soil friction angle φ

ATV-A 127 soil group	φ degrees
1	35
2	30
3	25
4	20

A.4 Soil moduli

A.4.1 General

Inspection of the literature shows that there is a diversity of opinion on the question of soil-modulus values, as can be seen from the following summary:

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

Molin (see reference [9]) 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 (34)]. When deflection calculations are made using the Iowa equation with E' equal to $2 \times E'_s$, the same answer is obtained as when using Molin's deflection equation with the secant modulus E'_s.

Duncan (see reference [5]) 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]) have been carried out using sand backfill and measurements 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, when performing ATV-A 127 type calculations in accordance with clauses 5 and 7 of this part of ISO/TR 10465, that the soil-modulus values given in Austrian Standard ON B-5012-1 are used because they have been obtained from laboratory determinations of the constrained soil modulus M_s at 1 m depth of cover. This satisfies the two conditions given above.

A.4.2.1 Backfill-soil modulus

A.4.2.1.1 Austrian standard OENORM B 5012-1

A.4.2.1.1.1 Soil-group definitions

The four soil groups in OENORM B 5012-1 which may be considered for backfill are defined as follows:

Soil group 1

Gravel, gravel-sand mixtures (GW, GP)

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

Soil group 2

Gravel-silt mixtures (GM)	maximum 15 % silt
Gravel-clay mixtures ((GC)	maximum 15 % clay
Sand-silt mixtures (SM)	maximum 15 % silt
Sand-clay mixtures (SC)	maximum 15 % clay and less than 40 % of particles larger than 2,0 mm

Soil group 3

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

Soil group 4

Silty or clayey soils (ML, CL, MH, CH) from low to high plasticity and containing more than 40 % of fine-grain materials

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 standard gives values for constrained soil moduli which depend on the degree of compaction as shown in Table A.3 and which are considered valid when using a vertical soil pressure equivalent to 1 m depth of cover.

Table A.3 — Constrained backfill-soil moduli M_s for various soil groups at 1 m

Soil group	Standard Proctor density (SPD)						
	85	90	92	95	97	100	102
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 calculated using equation (A.3):

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

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

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

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

where

f is 0,4;

κ is the reduction factor (from silo theory);

h is the depth of cover, in m;

h_w is the ground-water level above the top of the pipe, in m.

For % SPD values other than those shown in Table A.3, use equation (A.5):

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

where $M_{s,100}$ is the modulus at 100 % SPD.

NOTE 1 In OENORM B 5012-1, the power f is equal to 0,5. The value 0,4 has been chosen taking into account other investigations which give values lower than 0,5.

NOTE 2 In the Austrian Standard, soil pressure is used instead of $\kappa \times h$.

For the 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 due to the distributed surface load, in N/mm^2;

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

γ_b is the bulk density of the backfill material, in MN/m^3.

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

Table A.4 gives the values of the reduction factors to be applied to the backfill-soil moduli in zones E_1 and E_2 to allow for long-term changes in these moduli.

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

ATV soil group	Reduction factor
1	0,90
2	0,85
3	0,80
4	0,75

NOTE Native-soil moduli do not normally need to be reduced.

A.4.2.2 Native-soil modulus derived from impact measurements

Of the static design methods described in this part of ISO/TR 10465, the following make reference to impact tests as a means of measuring the modulus of elasticity of the soil. AWWA M-45 refers to ASTM D 1586 while the WRc method refers to BS 1377. The Austrian ATV type design uses OENORM B 4419-1.

There are significant differences in these test methods, which results in the same number of blows giving different values of the modulus of elasticity, depending on the method. The key differences between these methods are given in the following summary:

A.4.2.2.1 ASTM D 1586 and BS 1377

A standard 50-mm-diameter 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 required 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 OENORM B 4419-1 SRS 15 method

A standard 43,7-mm-diameter 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. The conical shoe is used for all types of soil.

The Austrian OENORM B 4419-1 method is preferred for determining 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 on-site investigations.

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

Table A.5 — Relationship between n_{10}, used in method 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 the values in OENORM B 5012-1 for 1 m depth of cover and those in ATV-A 127 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 (in %) and native-soil modulus (in N/mm^2)

ATV soil groups	SPD				
	85 %	90 %	95 %	97 %	100 %
1	3,8	5,3	10,0	13,0	20,0
2	2,1	2,9	5,3	6,7	10,3
3	1,3	1,8	3,3	4,6	6,7
4	0,9	1,3	2,5	3,3	4,9

Combining Tables A.5 and A.6 gives Table A.7:

Table A.7 — Native-soil modulus E_3 (in N/mm^2)

Number of blows n_{10} (SRS 15)	Non-cohesive soils (groups 1 and 2)	ATV-A 127 soil groups				Cohesive soils (groups 3 and 4)
		1	2	3	4	
> 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
$\leqslant 2$	Very loose	4	2	1	0,5	Soft and/or plastic

NOTE 1 For the same number of blows, the soil modulus varies with the soil group.

NOTE 2 n_{10} is the number of blows per 10 cm movement using test method SRS 15 in accordance with OENORM 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

The values given in Table A.8 are the backfill-soil moduli at the pipe-zone level.

Table A.8 — Backfill-soil modulus values expressed in N/mm² from AWWA M-45

Soil stiffness category	**Soil type** (unified classification system)	**Degree of compaction**			
		Dumped	**Slight** (Proctor density < 85 %, relative density < 40 %)	**Moderate** (Proctor density 85 % to 95 %, relative density 40 % to 70 %)	**High** (Proctor density > 95 %, relative density > 70 %)
SC 1	Crushed rock with < 15 % sand, maximum 25 % passing 10 mm sieve and maximum 5 % fines	7	21	21	21
SC 2	Coarse-grained soils with little or no fines (GW, GP, SW, SP, GW-GC, SP-SM), or any dual-symbol or borderline soil beginning with one of these symbols and containing 12 % fines or less	1,4	7	14	21
SC 3	Coarse-grained soils with fines (GM, GC, SM, SC, GC-GM, GC/SC), or any dual-symbol or borderline soil beginning with one of these symbols and containing more than 12 % fines	0,7	2,8	7	14
SC 3	Fine-grained soils with medium to no plasticity (CL, ML, ML-CL), or a borderline soil (ML/CL), or any dual-symbol or borderline soil beginning with one of these symbols and containing < 30 % coarse-grained particles	0,7	2,8	7	14
SC 4	Fine-grained soils with medium to no plasticity (CL, ML, ML-CL), or a borderline soil (ML/CL), or any dual-symbol or borderline soil beginning with one of these symbols and containing < 30 % coarse grained particles	0,34	1,4	2,8	7
SC 5	Highly compressible fine-grained soils (CH, MH, OL, OH, PT), or a borderline soil (CH/MH), or any dual-symbol or borderline soil beginning with one of these symbols	Soils in this category require special engineering analysis to determine the required density, moisture content and compactive effort.			

A.4.3.1.2 Values from WRc

Table A.9 is taken from the WRc *Pipe materials selection manual — Water supply*, 2nd edition, June 1995.

Table A.9 — Values of WRc embedment-soil modulus E'_2 for various installation conditions

Embedment class (see Table 5)	**Compaction** % MPD	**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.4.3.2 Native-soil moduli

A.4.3.2.1 Values from AWWA M-45

The values given in Table A.10 are the for native-soil modulus at pipe-zone level.

Table A.10 — Values of *in situ* native-soil modulus E'_n from AWWA M-45

Granular		**Cohesive**		E'_n N/mm^2
Blows/300 mm [a]	**Description**	**Unconfined compressive strength, q_u** (N/mm^2) [b]	**Description**	
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 D 1586.

[b] Unconfined compressive strength of cohesive soils as per ASTM D 2166.

A.4.3.2.2 Values from WRc

Table A.11 is taken from the WRc *Pipe materials selection manual — Water supply*, 2nd edition, June 1995.

Table A.11 — Guide values of native-soil modulus in various conditions

Soil type	Soil modulus E'_s (N/mm^2) Very dense	Dense	Medium dense	Loose	Very loose
Gravel	over 40	15 to 40	9,0 to 15	5,0 to 9,0	3,0 to 5,0
Sand	15 to 20	9 to 15	4,0 to 9	2,0 to 4,0	1,0 to 2,0
Clayey, silty sand	10 to 15	6 to 10	2,5 to 6	1,5 to 2,5	0,5 to 1,5
Clay	Very hard: 11,0 to 14,0 Hard: 10,0 to 11,0 Very stiff: 6,0 to 10,0 Stiff: 4,0 to 6,0 Firm: 3,0 to 4,0 Soft: 1,5 to 3,0 Very soft: 0,0 to 1,5				

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 may be significantly different from that in other soils, and other design methods may be appropriate (see for example WRc's sea outfall design guide).

Annex B
(informative)

Determination of concentration factors used in ATV-A 127

B.1 Maximum concentration factor λ_{max}

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

B.2 Concentration factor λ_R

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

where

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

NOTE Under normal bedding and installation conditions, it is recommended that a = 1 be used for flexible pipes.

$$V_S = \frac{S_R}{|c_v{}^*| \times S_{Bv}} \quad \text{(B.4)}$$

$$c_v{}^* = c_{v1} + c_{v2} \times K^* \quad \text{(B.5)}$$

B.3 Concentration factor λ_B

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

Annex C
(informative)

Loading coefficients used in ATV-A 127

Table C.1 — Values of c_{v1}, c_{h1}, m_{qv} and m_{qh} as a function of 2α

2α	c_{v1}	c_{h1}	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 of c_{v2}, c_{h2} and m_{qh}^* as a function of 2β

2β	c_{v2}	c_{h2}	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] in the bibliography) show better agreement between calculated and measured values when c_{v2} = 0,069 4 and c_{h2} = 0,070 0. It is therefore recommended that these experimentally derived values be used in place of the applicable values in Table C.2.

Annex D
(informative)

Horizontal bedding correction factors

D.1 General

The correction factor ξ takes into account the difference in soil modulus between the pipe embedment material and the native soil, as well as the width of the trench. Equations (6) and (7) 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 a very similar answer to that obtained using equation (6).

D.2 Correction factor ξ for horizontal bedding stiffness 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 (5). This equation assumes a support angle 2β of 120°:

$$S_{\mathrm{Bh}} = 0{,}6 \times \xi \times E_2 \tag{5}$$

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

The equations used in ATV-A 127 to determine the correction factor ξ for the assumed 120° support angle are equations (6) and (7).

$$\xi = \frac{1{,}44}{f + (1{,}44 - f) \times E_2 / E_3} \tag{6}$$

$$f = \frac{\left(\dfrac{b}{d_{\mathrm{e}}} - 1\right)}{1{,}154 + 0{,}444\left(\dfrac{b}{d_{\mathrm{e}}} - 1\right)} \leqslant 1{,}44 \tag{7}$$

Leonhardt's equations for the general case are equations (D.1), (D.2) and (D.3). The values of the coefficients in these equations are given in Table D.1 as a function of the value assumed for the support angle 2β.

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

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

$$S_{\mathrm{Bh}} = c_4 \times \xi \times E_2 \tag{D.3}$$

Table D.1 — Values of coefficients used to determine correction factor ξ

Coefficient	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 range of validity of equations (D.1) and (D.2), and of the coefficients in them, is:

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

For $b/d_e > 4{,}3$, $\xi = 1{,}0$

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 the "combining factor" S_c and has values different from those given by equations (D.1) and (D.2). **The values of S_c given in Table D.2 should not be substituted for ξ in ATV-A 127 type systems.**

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

E'_n/E'_b	b/d					
	1,5	**2**	**2,5**	**3**	**4**	**5**
0,1	0,15	0,30	0,60	0,80	0,90	1,00
0,2	0,30	0,45	0,70	0,85	0,92	1,00
0,4	0,50	0,60	0,80	0,90	0,95	1,00
0,6	0,70	0,80	0,90	0,95	1,00	1,00
0,8	0,85	0,90	0,95	0,98	1,00	1,00
1,0	1,00	1,00	1,00	1,00	1,00	1,00
1,5	1,30	1,15	1,10	1,05	1,00	1,00
2,0	1,50	1,30	1,15	1,10	1,05	1,00
3,0	1,75	1,45	1,30	1,20	1,08	1,00
⩾ 5,0	2,00	1,60	1,40	1,25	1,10	1,00

NOTE 1 Intermediate values of S_c may be determined by linear interpolation between adjacent values.

NOTE 2 E'_b is the modulus of the pipe-zone embedment, in N/mm^2.

NOTE 3 E'_n is the modulus of the native soil, in N/mm^2.

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 deflected 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 increased incrementally, 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 which applies to that period of incremental loading is called the instantaneous elastic modulus.

If the constant-deflection test is carried out in dry conditions, 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 immersed in water, and the elapsed time is long enough for the water to influence the properties of the material, then the instantaneous elastic modulus $E_{t,wet}$, although higher than the relaxation modulus, will be lower than the initial modulus. Ageing of the material has taken place. Hence:

$$E_{t,wet} = a_f \times E_o \tag{E.1}$$

where

a_f is the ageing factor;

E_o is the initial elastic modulus.

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

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

Where a 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 of 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

E.2.1 Theory

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

E.2.2 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 factor a_f.

E.2.3 Soil and traffic loads

To calculate the long-term deflection and flexural strain correctly, two separate calculations of the vertical load q_v should be made: one for the soil load δ_{vc}, using the long-term pipe creep or relaxation stiffness, and one for the traffic load and underpressure load δ_{va}, using the initial pipe stiffness times the ageing factor a_f.

E.3 Ageing factor

The ageing factor can be determined by a method such as that outlined below:

A ring of pipe complying with the requirements of ISO 7685 is subjected to the initial stiffness test in accordance with that International Standard. One of the three pairs of lines marked on the test piece is selected as the reference pair of lines 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 not subjected to any 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 is continued for at least 10 000 h and the results are analysed in accordance with ISO 10928 to obtain the 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 remaining soil friction used in ATV-A 127 type calculation systems

F.1 General

As stated in 5.2.1, the silo soil load is used in ATV-A 127 for both 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_{EDV} 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 pipe, thereby reducing the vertical soil load. In the calculations, this assumption is represented by the factor λ_R. For additional details concerning wide trenches, see 6.2.2 of part 2 of this Technical Report.

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

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

$$q_{v50} = (\gamma_b \times H_{EDV}) + [\gamma_b \times (h - H_{EDV}) \times \kappa \times \lambda_{RG50}] + (p_o \times \kappa_o) + p_v \tag{F.1}$$

where

H_{EDV} is the depth, in m, down to which friction has been lost due to frost, rain, traffic loads, etc, and may be up to 3 m;

h is the depth of cover, in m;

κ and κ_o are as defined in part 2 of this Technical Report, but using $h - H_{EDV}$ instead of h;

$\lambda_{RG,50}$ is as given by equation (9), but using λ_{RLT} instead of λ_R;

where $$\lambda_{RLT} = 1 - \frac{h - H_{EDV}}{h} \times (1 - \lambda_R) \tag{F.2}$$

λ_R is the long-term value of the concentration factor λ_R (see annex B) using long-term values of the soil modulus S_R, S_{Bv}, etc.

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

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

$$q_{h,50} = (K_2 \times \gamma_b \times H_{EDV}) + [\lambda_{BLT} \times \gamma_b \times (h - H_{EDV})] + (\kappa_o \times K_2 \times p_o) + \left(K_2 \times \gamma_b \times \frac{d_e}{2}\right) \tag{F.3}$$

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

NOTE In the case where K_1 = 0,5, equations (F.5) and (F.6) can be used for the calculation of κ and κ_o:

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

$$\kappa_o = e^{-\left(\frac{h}{b} \times \tan\delta\right)} \quad \text{(F.6)}$$

If the trench angle ω (see Figure 1) is not 90°, then

$$\kappa_\omega = 1 - \frac{\omega}{90} + \kappa \times \frac{\omega}{90} \quad \text{(F.7)}$$

$$\kappa_{o\omega} = 1 - \frac{\omega}{90} + \kappa_o \times \frac{\omega}{90} \quad \text{(F.8)}$$

NOTE ATV-A 127 uses the symbol β for the trench angle, not ω as in the equations above.

Annex G
(informative)

Application limits for underground GRP pressure pipes

G.1 General

G.1.1 Minimum or mean

This annex presents the basic concepts which have been used to establish the safe-design method given in subclause 11.2 of this part of ISO/TR 10465 for GRP pressure pipes complying with the relevant ISO GRP-pipe system standards.

The system standard gives the required minimum long-term safety factor 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 safety factor against pressure failure is applied to the **minimum** long-term strength.

Pipes produced in compliance with the requirements of the appropriate system standard are required to have a **minimum long-term tensile** strength. To ensure this, the **mean initial** strength of the factory-tested product must be sufficiently high, taking into account variations in the product, 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 pipes 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 property can be determined.

In this annex, it is assumed that the coefficient of variation z of the initial ultimate ring deflection 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 can be determined.

EXAMPLE

Consider a pipe of nominal stiffness SN 8000 which has a specified minimum initial ultimate ring deflection of 10,3 % and which is manufactured in a factory having an AQL of 6,5 %. The long-term mean value will have to 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 %, given by:

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

G.1.2 Safety factors

G.1.2.1 General

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

G.1.2.2 Material safety factors η_t and η_f

The safety factors η_t and η_f are each related to a property of the material, the circumferential tensile strength, i.e. the flexural strength ignoring rerounding. If the symbol η_{tPN} is used instead of η_t, this means that it is 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 and flexural influenced by tensile.

G.2 Semi-probability calculation system

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

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

In principle, the semi-probability calculations are based on equation (G.2):

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

where

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

μ_S is the mean value of 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 Table G.2).

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

To use this procedure to determine safety factors for inclusion in International Standards would require that agreed values of 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 aspects such as soil parameters, depth of cover, trench width, traffic load and internal pressure for each of the cases which are used in the application. It is evident that such agreements would be difficult to reach, and this approach has therefore been discarded.

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

Table G.1 — Typical parameters

Long-term tensile strength
Ultimate flexural strain (long-term, i.e. 50-year)
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 probability-calculation system

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

The basic equation for the simplified probability calculation for buried pipes is equation (G.3) which is derived from a modified version of equation (G.2):

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

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

Equation (G.3) is obtained from equation (G.2') by making the following substitutions:

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

$$s_{\mathrm{Res,B}} = y \times 0{,}01 \times \mu_{\mathrm{Res,B}} \qquad \text{(G.4)}$$

$$\mu_{\mathrm{Res,B}} = \mu_{\mathrm{S,B}} \times \eta_{\mathrm{hat}} \qquad \text{(G.5)}$$

where

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

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

$\mu_{\mathrm{S,B}}$ is the mean value of the load (stress) on the buried pipe;

$s_{\mathrm{Res,B}}$ is the standard deviation of the strength of the buried pipe;

$s_{S,B}$ is the standard deviation of the stress in the buried pipe;

X is the safety index;

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

In order to be able to carry out 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 Table G.2, using the calculated value of the probability function $P(x)$ derived from equation (G.6):

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

Inserting this value of X and the known value of the coefficient of variation y of the tensile strength, in %, in equation (G.7), the minimum value of the combined tensile safety factor η_{hat} is obtained. If this calculated minimum value of η_{hat} is less than 1,5, then use 1,5. Otherwise, use the calculated value.

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

EXAMPLE

What is the minimum value of η_{hat} necessary to ensure a minimum probability of failure of 1 in 10 000 when the coefficient of variation y of the tensile strength is 10 %?

P_f is 1 in 10 000, which is 10^{-4}; so from equation (G.6) $P(x)$ is 0,999 9.

Using Table G.2 for a $P(x)$ value of 0,999 9, a value of 3,72 is obtained for X.

Substituting in equation (G.7) gives:

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

In this case, the calculated minimum value is greater than 1,5. Thus the calculated value of 1,59 is used.

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, from the long-term burst pressure p_{50} and the working pressure p_w as shown in equation (G.10):

$$\eta_t = \frac{\sigma_{50}}{\sigma_t} = \frac{p_{50}}{p_w} \tag{G.10}$$