REPORT

Design of large diameter buried PE and PP pipes without internal pressure

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Contents

0. Foreword

The scope of this report is to advise a method to calculate the long-term deflection and resistance against buckling for large diameter plastic pipes. The method is also available as a webtool on TEPPFA website as well as being provided by the TEPPFA member companies. This online tool makes the calculations fast and guarantees the correctness of calculation every time.

The calculation of smaller diameter pipes, e.g., below DN1000 is already described in the 1999 TEPPFA publication “The design of Buried Thermoplastic Pipes.” However, during the last few years the development of large diameter plastic pipes, e.g., above DN1000 in diameter has developed rapidly and we have therefore seen the need for a fast and reliable method for those pipes, including structured wall pipes, according to the European standard EN 13476 *“Plastics piping systems for non-pressure underground drainage and sewerage – Structured-wall piping systems.”*

The calculation method was developed by Jan Molin. Jan has long professional experience as senior manager of the Swedish SWECO consulting and JM Geokonsult AB. Jan was contributing to TEPPFA publications at the time of Buried Thermoplastics Pipes studies. In addition, Jan was instrumental in the development of the VAV P92 publication, issued by Svenskt Vatten, in 2005. In the review process of this project, we have had valuable input from Ricky Selle of Selle Consult, in Leipzig.

The TEPPFA Below Ground Application Group is in charge of this work and the input from experts of the TEPPFA member companies has been taken into account, since it has been tremendously beneficial to the method and the report. We owe gratitude especially to Frans Alferink and Erik Guldbæk. Frans took the initiative in this project and kicked it off with passion and profound knowledge of the subject. Erik supported us during the process both with his sense of humour and his clear insight into the end-goal. Unfortunately, both Frans and Erik passed away due to illnesses but the project would not have existed without their enthusiasm, vision and initiative.

Brussels, 4 November 2022
Ludo Debever

1. Introduction

This report is a part of the TEPPFA project “Buried pipes 2” which is a follow-up of the earlier TEPPFA project concerning “Design of buried thermoplastics pipes”, ref./1/, from 1999. In that report, a great deal of the European knowledge at that time was presented.

Some years later, in 2005, a Swedish report from Svenskt Vatten, here called P92, ref./2/, was presented. The report was based on laboratory tests, soil-box tests and full-scale field tests, in the Nordic countries during the period 1965 – 2000. The investigations were made on sewer pipes of different plastic materials, normally with diameters around DN 200 and maximum DN 700.

The technique to produce large diameter plastic pipes has then been further developed and nowadays thermoplastic pipes with diameters up to at least 3000 mm are available in the market.

The present report is complementary to the document P92, ref./2/, for large diameter PE- and PP-pipes. The following requirements are the basis for the report:

- The range of pipe diameters is DN 1000 to DN 3000.
- The range of fill heights above top of pipe is 1,0 m to 6,0 m.
- The minimum cover for pipes SN 2 and SN 4 should be not less than half pipe diameter and always at least 1,0 m.
- The embedment fill around and up to minimum 0,15 m (or the value prescribed by local road authorities) above the pipe shall be a granular soil, placed in shifts and compacted to minimum 90% mod. Proctor. This requirement corresponds to the installation type “well” compaction in the TEPPFA report, ref./1/.

For pipes with structured walls the following document shall be followed:

- Pipes with structured pipe walls shall meet the requirements of European product and system standard EN13476, in particular, the “Ring flexibility 30%” requirement, ref./3/ and ref./4/.

2. Loads

An important part of the loading situation for large diameter pipes is connected to concentrated loads on the soil surface, normally the traffic load. According to the Boussinesq theory, this load is assumed to be spread out with increasing depth below surface. At shallow depth, the effect of traffic loads is concentrated to areas below the individual wheels and bogies but with increasing depth, the load will spread out to larger areas, the pressure intensity will decrease rapidly and the load will be more evenly distributed over a larger area.

2.1 Soil load

For the calculation of the soil load on the pipe, the methods in P92, ref./2/, may be used. However, to also take account of the load from the soil at the upper haunches of the pipe, the vertical soil pressure on large diameter pipes shall be calculated as follows, see ref./5/, (BS 9295:2020):

$$qs := \gamma \cdot (H + 0.11 \cdot D) \qquad kN/m^2$$

where γ= density of the fill, kN/m^3

H= fill height, m
D= pipe outside diameter, m

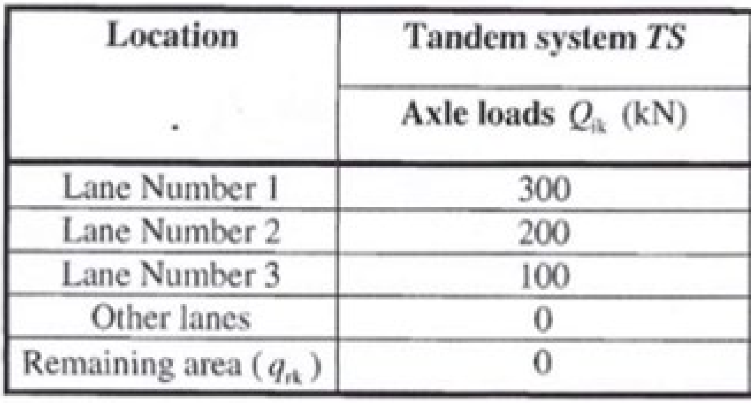
For a gravelly fill the density 19 – 20 kN/m^3 can be used above ground water level and 11 kN/m^3 below ground water level if not other values are prescribed by the designer.

Maximum soil load occurs with the ground water level as low as possible while the maximum total load on the pipe occurs at the highest possible level of the ground water.

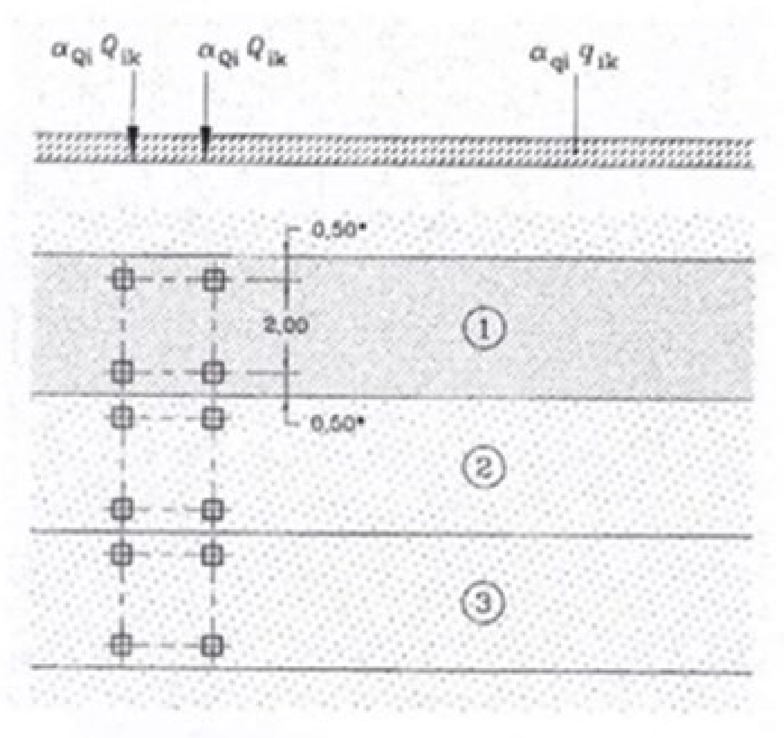
2.2 Traffic loads

2.2.1 Road traffic

In this document, the impact of traffic loads on buried pipes is based on the European Standard EN1991-2:2003, ref./6/. The load case LM1 is valid for road traffic. The dynamic amplification is included in the loads.



Location	Tandem system TS
	Axle loads Q_{ik} (kN)
Lane Number 1	300
Lane Number 2	200
Lane Number 3	100
Other lanes	0
Remaining area (q_{rk})	0



(1) Lane Nr. 1 : Q_{1k} = 300kN ; q_{1k} = 9 kN/m^2

(2) Lane Nr. 2 : Q_{2k} = 200kN ; q_{2k} = 2,5 kN/m^2

(3) Lane Nr. 3 : Q_{3k} = 100kN ; q_{3k} = 2,5 kN/m^2

For wt = 3,00m

Figure 2.1. Load model LM1 for road traffic, excerpt from /6/

In figure 2.2 the mean pressure from the traffic load is shown as a function of installation depth and pipe diameter. The calculation is made in accordance with the Boussinesq theory as described ref./8/.

A reduction factor of a_{N2} is applied to LM1 according to Note 2, Subclause 4.3.2, in Eurocode 1, EN 1991-2.

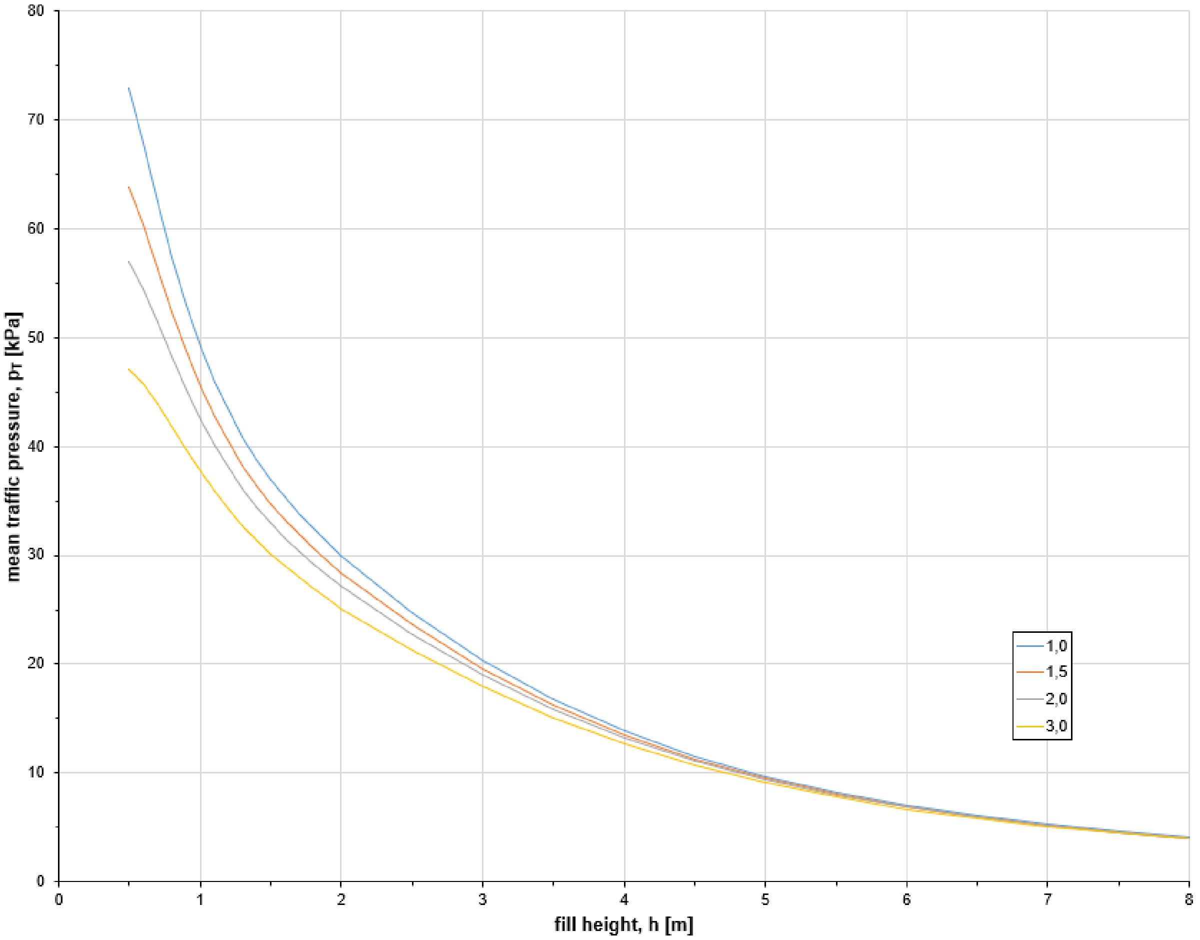


Figure 2.2 Mean value of traffic load on buried PE pipes, load case LM1 (roads) for varying diameters (1.0 to 3.0 m).

The pressure curves shown in the figure above are including the load distribution of a stiff road pavement with a thickness of 200mm and a load-spreading twice as large as in the native soil below.

2.2.2 Railway traffic

The impact of railway loads on a buried pipe is not included in this report. The loading requirements for some of the different load models for railway systems can be found in ref./5/ and ref./6/.

2.3 Ground water load

The ground water will give the following average ring pressure on the pipe:

$$q_w = 10 * (H - Hw + \frac{D}{2}) \qquad kN/m^2 \qquad (2.3)$$

where H= fill height, m

H_w= distance between top of the fill and the ground water level, m

D= outside diameter of pipe, m

If the ground water surface is below the ground level, the validity condition for equ. (2.3) could be written as:

$$0 \leq H_w \leq (H + \frac{D}{2})$$

If the ground water surface is above the ground level, H_w in equ. (2.3) shall be given a negative value.

3. Ring deflection

3.1 Load distribution model

On a buried flexible pipe there are acting vertical forces, caused by the weight of overburden fill and traffic loads, and reacting horizontal forces including soil reactions partly caused by movements of the pipe wall. The load model can be divided into two parts: one dealing with the soil load and the other with the traffic load as illustrated in *Fig. 3.1*.

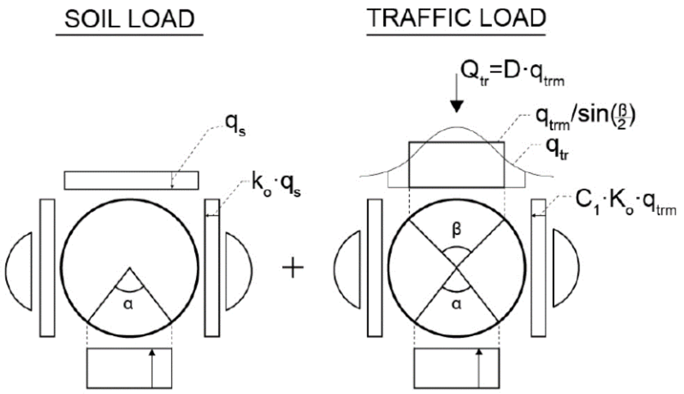


Figure 3.1 Assumed load distribution model

The result from the two calculation models shall be added together for the total solution.

3.2 Soil modulus

In P92, ref./2/, values are presented for the soil modulus based on tests on sand in a large diameter (0,5 m) oedometer with a wall of floating rings. The values chosen for the tangent modulus include an extra margin for the risk of insufficient quality of the compaction in ordinary sewer pipe construction work.

For large diameter pipes, the normal execution of the installation work is expected to be more qualified than for the small diameter pipes. In ref./5/ (BS9295:2020) there are given values for the constrained soil modulus M_s*, Table 3.1,* and these values are expected to be representative for the fill around the large diameter pipes dealt with in this report.

Table 3.1 *Constrained soil modulus M_s*

Bed and side fill materials	Compaction Mod.Proctor	Ms MN/m^2
Gravel (normally processed material)	85% 90%	9 15
Sand and coarse-grained soil with less than 12% fines	85% 90%	5 10

The following equation is suggested as design value for calculation of the deflection and buckling of large diameter plastic pipes at fill heights 1 to 6 m including the effect of a highground water level:

$$E'_{sd} = 0{,}7 * M_s \qquad MN/m^2 \qquad (3.1)$$

The values given in *Table 3.1* are valid for pipes in a trench where the surrounding soil has the same or higher soil modulus than the fill in the trench. Otherwise, the values shall be corrected as advised in ref./5/.

Example: *The pipe ditch has a width of 2*D and the native soil has a modulus of half the modulus for the fill around the pipe. The combined modulus for E'_{sd} for calculation of the deflection will, according to ref./5/, then be 0,5*M_s.*

The materials for the bed and the side-fill given in *Table 1* are in accordance with the installation category “well” in the TEPPFA document, ref./1/. According to this document, the soil surrounding the pipe shall be a granular soil placed carefully in the haunching zone and compacted to 90% mod. Proctor.

More detailed information of the soil modulus can also be found for instance, in ref./7/ (KTHreport 112).

3.3 Deflection caused by soil fill

$$\frac{\delta s}{D} := qs \cdot \left(\frac{C \cdot b1 - 0.083 \cdot K0}{8 \cdot SN + 0.061 \cdot E'sd} \right) \qquad (3.2)$$

where
δ_s= vertical diameter change due to soil load,m
D= mean diameter of pipe, m
q_s= soil load, kN/m^2
C= vertical load factor, chosen value 1,0
b_1= coefficient, see *Table 3.2*
K_0= horizontal soil pressure coefficient, chosen value 0,5
SN= ring stiffness, kN/m^2
$E`_{sd}$= soil modulus, kN/m^2

The ring stiffness SN for calculation of ring deflection and buckling is defined as follows:

$$SN = \frac{EI}{(1-\nu^2) * D^3} \qquad kN/m^2 \qquad (3.2a)$$

I= moment of inertia of the pipe per metre length of pipe, m^4/m
E= modulus of the pipe material, *kPa*
ν= Poisson`s ratio for PE, chosen value 0,45
D= mean diameter of pipe, *m*

The value of SN can be calculated with equ. (3.2a). The short-term value can also be directly established by ring testing according to ref./9/.

Values for the coefficient *b1* are given in *Table 3.2* for different support angles α and load angles β, see ref. /10/.

Table 3.2 Coefficient b_1 as function of the support angle at the bottom of the pipe, α, and the load distribution angle at the top of the pipe, β, see Fig. 3.1.

Angles	Coefficient b_1					
α \ β	0°	30°	60°	90°	120°	180°
0°	0,148	0,145	0,138	0,130	0,122	0,116
30°	0,145	0,143	0,134	0,126	0,118	0,113
60°	0,138	0,134	0,128	0,117	0,113	0,106
90°	0,130	0,126	0,117	0,109	0,102	0,096
120°	0,122	0,118	0,113	0,102	0,097	0,089
180°	0,116	0,113	0,106	0,096	0,089	0,083

The bottom angle α normally can be set to minimum 90°. A higher value, e.g. 120°, can be chosen if that can be justified by the specifications for the compaction of the fill under the pipe including also a thorough inspection during the execution of that work.

For soil load the top angle β can be chosen to 180° for all cases.

For estimation of the local maximum deflection due to effects of unforeseen installation measures and other irregularities along the pipeline, additions shall be made to the calculated mean value caused by soil and traffic loads, see clause 3.5.

3.4 Deflection caused by traffic load

The deflection caused by traffic loads is assumed to follow almost the same pattern as the deflection caused by the soil load. However, at small fill heights the traffic load is more concentrated over the top part of the pipe than the soil load. This can be considered by assuming the total traffic load to be distributed over only a part of the pipe diameter as indicated in *Fig. 3.1.*
The deflection caused by traffic load can be estimated as follows:

$$\frac{\delta tr}{D} := qtrm \cdot \left(\frac{C \cdot b1 - 0.083 \cdot K0 \cdot C1}{8 \cdot SN + 0.061 \cdot E'sd} \right) \qquad (3.3)$$

where δ_{tr}= change of vertical diameter length due to traffic load, m
D= mean diameter of pipe, m
q_{trm}= traffic load, mean value over 1 m length of pipe, kN/m^2
C_1= factor for correction of the horizontal soil pressure at rest
$E`_{sd}$= soil modulus, kN/m^2

Also, for traffic loads the bottom angle α normally can be set to minimum 90°. A higher value might be possible as indicated for the deflection caused by the overburden fill, see clause 3.3.

The value of the top angle β depends for traffic loads on the pipe diameter D and the overburden fill height *H*. A rough estimation based on the pressure curves caused by the load LM1 in figure 2.2 indicates top angles β between 45° and almost 180°. This means that the value of the coefficient b_1 varies between roughly 0,12 and 0,10 at a support angle of 90°. The value $b_1 = 0,12$ can normally be used as design value for the calculation of pipe deflection caused by traffic loads for all pipe diameters and fill heights treated in this report.

The horizontal correction factor C_1 for the soil pressure at rest is related to the part of the vertical traffic pressure acting on the fill at the sides of the pipe. An estimation of the factor C_1 has been based on the load case LM1. A conservative value of the factor C_1 has then been obtained by dividing the vertical pressure at the distance 0,5*D beside the pipe wall with the maximum vertical pressure at the top of the pipe.

Table 3.3 Estimated values for the coefficient C_1.

Diameter m	Fill height m	Coefficient C1
1,0	1,0	0,8
	2,0	0,9
	3,0	0,9
	4,0	0,9
	6,0	1,0
2,0	1,0	0,3
	2,0	0,8
	3,0	0,8
	4,0	0,9
	6,0	0,9
3,0	1,0	0,2
	2,0	0,6
	3,0	0,7
	4,0	0,8
	6,0	0,8

For calculation of the pipe deflection caused by traffic loads the short-term value of the stiffness SN normally shall also be used for the long-term deflection of the pipe as the traffic load has a short duration. The effect of many load repetitions is taken care of by the time-lag factor, see clause 3.5.

3.5 Long term deflection

After installation of the pipeline many field observations have shown that the pipe deflection varies along the pipeline. Some of these variations occur during the installation work and are not related to soil or traffic loads. They can be handled by two additional terms, Installation factor, I_f and Bedding factor, B_f, see ref./2/.
The long-term effect on the deflection related to loads from soil fill and traffic is represented by a time lag factor, *DL*.

The total value for the long-term deflection can then be expressed as follows:

$$\frac{\delta}{D} = DL\left(\frac{\delta_s}{D}+\frac{\delta_{tr}}{D}\right) + I_f + B_f \qquad (3.4)$$

where I_f = Installation factor, %
B_f = Bedding factor, %
DL= time-lag factor

$\frac{\delta_s}{D}$ = short-term deflection caused by soil load
$\frac{\delta_{tr}}{D}$ = short-term deflection caused by traffic loads

Suggested values for the terms I_f and B_f valid for large diameter pipes are given in *Table 3.4*.

Table 3.4 Suggested values for I_f, B_f and DL

Pipe class, SN kPa	I_f %	B_f %	Time lag factor DL
2	0,5	2,0	2,0
4	0,5	1,0	2,0
8	0,5	0,75	2,0

The values for I_f and B_f are chosen as half the values given for small diameter pipes in ref./2/. For pipes installed on the shelf in a multi-pipe trench, additional load and deflection of the pipe may occur due to settlements in the trench below the level of the pipe foundation.

For PE pipes the deflection is normally not critical for the stress or strain in the material as long as the pipe is surrounded by well compacted soil fill. However, the deflection might be critical due to operational reasons or jointing difficulties. The maximum allowable deflection shall be given by the design engineer or the producer of the pipe. The deflection is suggested to not exceed 15 % of the diameter, but in any case, local regulations shall be observed.

4. Ring forces

4.1 Bending moment

The following estimation of the long-term bending strain in the pipe-wall is given in ref./2/:

$$\varepsilon_M = 6 * \left(\frac{\delta}{D}\right) * \left(\frac{s}{D}\right) \qquad (4.1)$$

where s= thickness of the pipe wall for a solid wall pipe. In case of a pipe with an asymmetric structured wall s is twice the largest distance between the outer wall and the neutral layer of the profile, m
$\frac{\delta}{D}$ = deflection acc. to eq. (3.4) without the terms I_f and B_f

The reason for ignoring the installation and bedding factors, I_f and B_f, is that they represent a part of the deflection that will remain more or less constant by time and the stresses caused by them are expected to decrease substantially over time due to relaxation.

This expression (4.1) can also be rewritten to a bending moment as follows:

$$M = 12 * \delta * SN * D \quad \text{kNm/m} \qquad (4.2)$$

The above formulae are valid for pipes with stiffness SN≤ 8 kN/m^2.

4.2 Ring normal force

The ring normal force in the pipe wall can be calculated as follows:

$$N_r = 0{,}5 * D * q_d \quad kN/m \qquad (4.3)$$

where q_d= designing ring load according to equ. (5.2), kN/m^2
D= outside diameter of the pipe, *m*

The dimensioning situation occurs with the ground water level as high as possible. Under the ground water level, the effective density of the soil shall be used, often chosen 11 kN/m^3.

4.3 Design requirement

The combined long-term strain ε_C caused by the ring forces, normal force and moment, can be calculated as follows:

Max compressive strain: $\varepsilon_C = \frac{Nr}{E*A} + \varepsilon_M$ (4.4a)

Max tensile strain: $\varepsilon_C = \frac{Nr}{E*A} - \varepsilon_M$ (4.4b)

where N_r = normal ring force, *kN/m*
E= long term value for the E-modulus, *kPa*
A= area of the pipe wall, m^2/m (equal to the wall thickness *s* for a solid wall)

In ref./5/ is suggested that the compressive strain for PE should not exceed 5 % and the tensile strain not 2,5 %. However, other and higher values may be used where supported by appropriate testing. For PP, a limit value for the specific material type has to be defined by the design engineer/producer.

For pipes with structured walls the risk of local instability of the different elements of the wall also needs to be checked by calculation or by appropriate testing.

5. Buckling

Exposed to high compressive ring pressure, the pipe wall can collapse through buckling. For a buried pipe, the surrounding soil will have a stiffening effect which gives a more complex buckling pattern of the pipe wall than the pipe being surrounded by air or water. The soil around the pipe will increase the buckling pressure.
All pipes treated in this report are assumed to be surrounded by well compacted fill. The buckling pressure for the pipes can then be calculated as follows:

$$q_b = 5{,}65 \cdot \sqrt{E'sd \cdot SN} \quad kN/m^2 \qquad (5.1)$$

where: E'_{sd} = modulus for the surrounding soil, kN/m^2.

Values for the modulus E'_{sd} given in equation (3.1), which include the effect of a high ground water table, can be used as design values in the buckling calculation.

The buckling safety shall be calculated with the following design load q_d:

$$q_d = q_{sd} + q_{trd} + q_{wd} \quad kN/m^2 \qquad (5.2)$$

where:

$q_{sd} = \gamma_f \cdot q_s$ design soil pressure at high ground water level, γ_f = 1,0
$q_{trd} = \gamma_f \cdot q_{trm}$ design pressure from traffic loads, kN/m^2, γ_f= 1,3
$q_{wd} = \gamma_f \cdot q_w$ ground water pressure at the centre of pipe, γ_f= 1,0

γ_f = partial coefficient

Ovality or deformations in the pipe will influence the buckling pressure negatively. For a pipe in firm soil this can be taken into account by multiplying the theoretical buckling pressure with a reduction factor β:

$$\beta = \left(1 - 3 \cdot \frac{\delta}{D}\right) \tag{5.3}$$

Satisfactory buckling safety is secured if the following condition is fulfilled:

$$q_d \leq \frac{\beta \cdot q_b}{\gamma_{Rd}} \quad kN/m^2 \tag{5.4}$$

where:

q_b= buckling pressure, *kN/m*^{2}
γ_{Rd}= safety coefficient for buckling

The values prescribed in national standards shall be used for the safety coefficient γ_{Rd}. However, as required in ref./5/, a value not less than 2,0 is suggested for large diameter pipes installed in firm soil.

The buckling calculation has to be made in two steps as the design value of the pipe stiffness SN is different in the two load cases, ref./5/:

Case a. Combined load of soil, traffic and ground water. The short-term value for the pipe stiffness SN should be used.

Case b. Combined load of soil and ground water. The long-term value for the pipe stiffness SN should be used. For PE this value is often set to 25% of the short-term value. For PP a value for the specific material type has to be defined by the design engineer/producer.

Both cases have to comply with the condition (5.4).

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About TEPPFA

TEPPFA is the European Plastic Pipes and Fittings Association founded in 1991 with headquarters in Brussels. TEPPFA’s 14 multinational company members and 15 national associations across Europe represent 350 companies that manufacture plastic pipes and fittings. TEPPFA’s members have an annual production volume of 3 million tonnes directly employing 40,000 people with €12 billion combined annual sales. TEPPFA positions itself as polymer neutral. TEPPFA’s final products are subdivided into two application groups: above ground systems for hot and cold water, surface heating and cooling, waste water discharge and rainwater drainage, and below ground systems for sewers, stormwater and drainage, drinking water and gas supply and, cable ducts.