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Design of Buried Thermoplastics Pipes

Results of a European research project

The Design of Buried Thermoplastics Pipes

By

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FOREWORD

In this report the results of a study into the design of buried thermoplastics pipes is reported. The study is a result of co-operative research, sponsored by the European Thermoplastics pipes and fitting association TEPPFA, and the Association of Plastics Manufacturers in Europe APME.

The project has been proceeded by experts from both the industries, as well as from external organisations. Six external leading experts in the field of pipeline design, not necessarily plastics pipes design, have been involved as consultants to the project.

The steering committee advises to bring the result to the public domain, because the information contained herein is valuable for all who are involved in the design of buried pipes systems, as well as to those who are considering improvements of existing methods or for the development of new methods.

The report summarises the experimental work carried out, and analyses the pipe soil interaction process as monitored in this study. Next to this a simple design graph is presented.

In order to make navigation in the report by the reader a little easier, figures, tables and enclosures are numbered using two digits separated by a dot. The first digit refers to the main heading in which the item is used. The second number is just a counter within that heading.

SUMMARY

In this report information is given on the performance of buried pipes, when buried under different conditions. The need for such a study became apparent when at a European level, legislation on how to design buried pipelines became demanding. Next to this, it was realised that still a lot of misunderstanding exists about how flexible pipes, and more specific thermoplastics pipes, behave in real life.

The following five objectives have been defined:

1. Several data sets to become available for validating current and future design methods.
2. One clear design and installation advice to customers.
3. Avoid overkill in installation requirements.
4. Design in balance with feasible installation methods.
5. Increase the confidence in thermoplastics pipes for buried application at the market place.

The result on the first objective is explained in detail in this report, by the presentation of 16 well-documented data sets, and several subsets.

The result on the second one is contained in the presentation of design graphs, in which the type of construction is the main parameter.

The exercise with the different, sometimes extensive, design methods has shown that such designs are only appropriate when at the same time soil properties are well determined, and at the same time, the execution of the installation work is done according to the prescriptions. The Paradox became clear that under such circumstances installations are well carried out and monitored. So limit state conditions, conditions under which design really becomes important, are not likely to occur. The balance between design and installation methods has become clear by this, and also the relevance of the type of construction has become clear.

The study has shown the excellent behaviour of thermoplastics pipes under buried conditions, which success is especially caused by the huge strainability of the material. Although very poor installation condition has been used, no failure or stability problems have occurred during the 2.5 years of monitoring of the buried pipes. The results have shown to line up with earlier published long-term experience of buried plastics pipes.

With this report the end of the work has been announced, still some repetitive measurements have been planned for the next two years. TEPPFA and APME will present these result in a post report.

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1. INTRODUCTION

In this report information is given on the performance of buried pipes, when buried under different conditions. The need for such a study became apparent when at a European level, legislation on how to design buried pipelines became demanding. Although several European experts are working now for over ten years to establish such a method, no method has become available yet. One of their biggest problems is the absence of well-documented experimental data. At the end of the day, amongst many others, the Plastics pipes industry shall have to evaluate the current and future methods on their ability of predicting pipe performance in accordance with actual behaviour. So also for the industry it was felt to be of prime importance that several data sets would become available for that purpose. Another important reason for the study was the fact that the plastics pipes industry has not yet thoroughly discussed the design and installation aspects of their pipes as an industry. It is felt that an unified design and installation advice is another very important objective of this project. Currently a lot of national methods are available and some of them are very detailed whereas others have refrained from such a high level of sophistication. It shall be emphasised that the objective of the industry with this study is to help in obtaining a better understanding of pipe soil interaction in general and for thermoplastics pipes more specific.

It shall be realised that establishing buried pipes systems is not a highly academic exercise. Engineers need tools to predict the performance of the pipeline in order to be able to judge if the circumstances, including the pipe properties, soil properties and workmanship will result in the establishment of the pipeline within a safe application window. This means that the balance between the design method, the input values and the execution of the construction needs to be correct. All the above items will be focussed upon in this study.

It was decided to carry out the work on pipes with a diameter of 315 mm, as this is a rather good average pipe diameter used in many applications. All pipes are of the so-called solid wall types. Solid wall pipes allow easy instrumentation, and next to this the pipes could be studied as pressure as well as non-pressure pipes. The results of the tests on the solid wall pipes can be easily transferred to structured wall pipes as well. Essential is that the structure of the structured wall pipes stay in shape when deflecting. The product standards however utilise a so-called ring flexibility test, to check upon this aspect. Where appropriate, guidance will be given about how to utilise the information for structured wall pipes.

Sand as well as clay was chosen as embedment material, in order to be able to judge the difference between granular versus cohesive soil types.

2. INSTALLATION PRACTICE

Traditionally, pipes are buried by open trench methods. Although nowadays also, so-called NO-DIG methods are used, still the major part of the installation work is carried out using the traditional methods. Most of the installations are performed using granular materials as sidefill, in combination with some kind of compaction in order to avoid huge settlements of the surface after installation.

However, installations are performed in cohesive materials and organic materials as well. Although in general they are in many countries not recommended, good experience has been gained when using flexible pipes made out of strainable materials.

ISO as well as CEN have produced several versions of 'Recommended practices for the design of buried pipes'. Next to this, many national standards as well as manufacturers recommendations exist. These documents all recommend a certain installation procedure.

In practice, non-predictable circumstances affect the true installation of the buried pipes. For instance the compaction level that can be reached for sand depends on its water content. So already depending on weather conditions one obtains a better or worse installation than anticipated. In many cases the soil profile changes over a short distance. Using the as dug material again as embedment of the pipe, does not make an extensive design of any pipe very realistic. In order to obtain more reproducible installation conditions it is sometimes recommended to import special embedment materials, like gravels and selected stones. This however makes the installation more costly. Another negative, but in most cases not considered, effect of imported backfill is that the new soil has another density then the native soil, causing on the long term other effects, as settlement differences along the pipeline. Therefore, using the as dug material as much as possible and providing flexibility and strainability in the pipeline system is the best (robust) solution for obtaining a proper functioning pipeline system. Robust here refers to the fact that the long life performance of the pipe is hardly affected by misfits in design and or installation.

3. FEATURES OF THERMOPLASTICS

For the application of buried flexible pipes of all materials, the following pipe characteristics are of importance:
- Pipe ring stiffness
- Corrosion resistance
- Elongation at break

The pipe ring stiffness is one of the aspects that will be considered in this study and as such further discussed.

Corrosion resistant properties are in general very good for all plastics materials and the pipe will therefore not deteriorate when placed in the soil. It is this corrosion resistant that provides a firm basis for any statement on durability.

With regard to the strainability properties one needs to distinguish between elongation at break as found in a tensile test, the strain developed under creep conditions and the strain under relaxation conditions. For pressure pipes the strain controlled by the stress is the covering factor, although the buried pipe transfers part of the load (internal pressure) to the surrounding soil. For gravity pipes it is known that after installation the pipes are not loaded by a constant load, but they become under a condition of constant strain, giving rise to a stress relaxation process. For the latter case several studies have been performed in Germany and Scandinavia. Tests carried out in Sweden (1) showed that PE and PVC pipes can be subjected to significant strain without any risk of cracking. Constantly deflected pipe samples with strain of the pipe wall exceeding 10 % have been tested for more than 9 years without showing any cracks. In Germany, Hoechst has tested PE pipes, which have been expanded circumferentially by 5 %. After 40 years of testing there is still no sign of cracks in the material. It is concluded that for thermoplastics pipes like PVC and PE no strain limit exists under stress relaxation conditions; however it is proposed to limit the strain to reasonable values anyway. Table 3.1 summarises the values proposed in Reference 1.:

Table 3.1: Allowable strain in buried gravity pipes

Pipe material	Allowable strain [%]
PVC	2.5
PE	5.0

For pressure pipes another test exists to determine the design values. For these pipes so-called Internal Hydrostatic Pressure testing is used to determine the failure curve, which on turn is the basis for the determination of the design values. In this test the pipe is free to expand and gives as such a worse case design condition.

The fact that the design has to be treated differently for gravity than for pressure pipes is because of the visco-elastic behaviour of the material. For linear elastic materials a direct relation between stress and strain is valid. For visco-elastic materials this is not the case. The performance under load is history and time dependent. When the load consists of a pre-scribed displacement than the holding force will decline with time. The issue described here is called "stress relaxation". In order to satisfy the traditional relationship between stress and strain, one needs to apply a lower (apparent) modulus. As stated it is an apparent modulus, since any new loading of the sample utilises the true modulus again.

The same type of process exists when the load is a constant load instead of a prescribed displacement. In such a case the strain will increase. Again in order to satisfy the traditional relationship between stress and strain, one needs to use the apparent modulus. Also in this case the material will respond with the true modulus to any new load. The situation of creep, as the above description is called occurs during Internal Hydrostatic testing of pipes. Essential is that the load is able to follow the creep of the material. As soon as this is hindered, the stress relaxation process becomes active.

It is for the above reasons that the design stress determined in the Internal Hydrostatic pressure test can not be applied straight away to pipes that are not fully loaded to the creep condition. In any other case then free creep, the strain limits as shown in table 3.1 shall be applied.

The Minimum Required tensile Stress values valid for determining the design stress at free creep conditions are shown in table 3.2.

Table 3.2: Minimum Required Stress values (LCL values)

Pipe material	Long term [mPa]
PVC X	X/10
PE 63	6.3
PE 80	8
PE 100	10
PP X	X/10

Note : X refers to ISO 12162

4. DESIGN METHODS

The methods used in this study are partly methods that have been nominated as established methods according to the definition of CEN TC164/165 JWG1, and are fully documented in EN1295 (2). The reader is referred to this document for a formal description of these methods. However next to these established methods also other methods have been used.

For the purpose of the workshop which was held in December 1997, all experts made a short summary explaining the basics of the methods. These descriptions are shown in Enclosure 4.1 In table 4.1 an overview of the methods used is shown.

Table 4.1 : Well-known established design methods

EN1295		**Other methods**	
Method	Used by	Method	Used by
ATV 127	Germany	WG14	(GRP)
Materials Selection Manual	UK	CalVis	-
Fascicule 70	France	Bossen	NL
Önorm B5012	Austria		
VAV P70	Sweden		

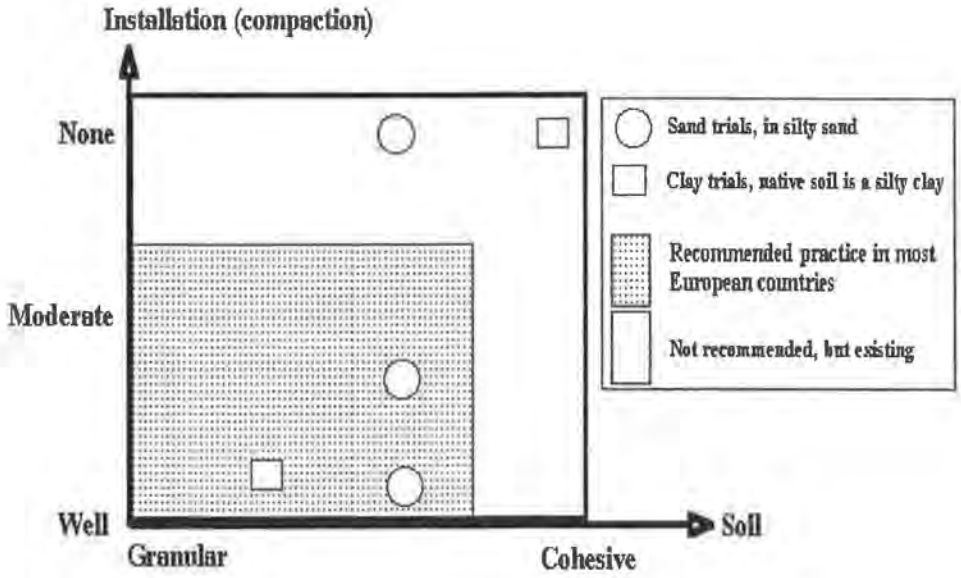
5. EXPERIMENTAL WORK

5.1 Test sites

The project group deliberately carried out the tests at a field instead of in a laboratory in order to fulfil the most important requirement of this study, which is that the result shall reflect real circumstances and installations. Therefore it is needed that a regular contractor is involved to carry out the tests, and that the pipeline length is sufficiently long to allow the proceeding of a normal installation. The European experts as well as the project group emphasised that the study shall at least reflect the less ideal installation circumstances, because especially under these conditions limit state situation are to be expected if any.

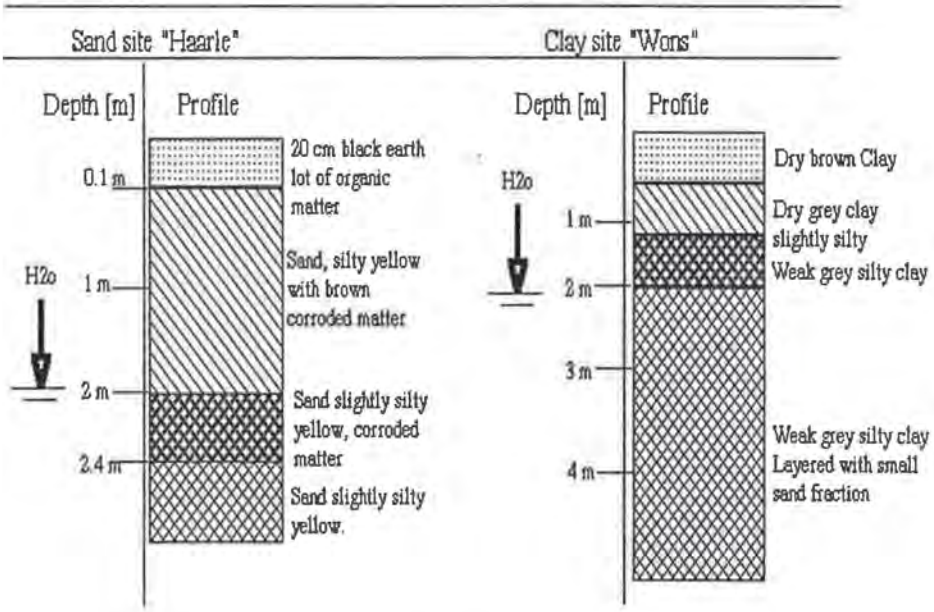
Therefore it was decided to create installations at the border of the normally accepted installation window. A graphical presentation of the relative position of the trials in that window is given in figure 5.1

Figure 5.1 : Location of trials relative to common installations



Two different sites were selected for that purpose. One located at the foot of a hilly area, which area was created during the second ice period some 130.000 years ago, when ice was pushed from Finland /Sweden to the south. About 1 million years ago this area presented the coastline of the European continent. The soil consists of sand with small to moderate amounts of silt. The other site is located near the seaside to current where the soil consists of clay. In figure 5.2 the cross description of the native soil for both test sites is given.

Figure 5.2 :Cross description of soil profile



In table 5.1 the history of the trial is shown. The periods line up with the seasons.

Table 5.1: Trial history, main activities

Activity	4/96	1/97	/97	3/97	4/97	1/98	2/98	3/98	4/98	1/99
Installation Cases 1-12	X									
Measuring Cases 1-12	X	X	X		X		X			X
Traffic loading Cases 1-2/5-6										
Pressurisation Cases 5-6,7-8			X							
Installation Cases 13-16				X						
Measuring Cases 13-16				X	X		X			X
Workshop with Europ. Experts					X					
Final report										X

In table 5.2 the design and experimental cases are listed.

Table 5.2 : Overview of design cases sorted on type of installation

Case	Installation	Embedment	Native soil	Pipe material	Ring stiffness [kN/m2]
1	Well	Sand	Sand	PVC	4
3					2
5				PE	6
9				PVC	2
11				Steel	4
13			Clay	PE	6
15					6
7	Moderate		Sand	PVC	4
2	None				4
4					2
6				PE	6
8				PVC	4
10					2
12				Steel	4
14		Clay	Clay	PE	6
16					6

5.2 Measurement techniques

5.2.1 *Soil properties*

The properties of the soil were measured before and after installation.
The measurements were partly carried out in the field, partly in the laboratory. All laboratory tests were performed at the laboratories of Fugro, certified acc. EN45001, nr. L34.
The following tests were performed:

- Sieve test
- Proctor test
- Grain shape
- Triaxial test
- Cone penetration test
- Oedometer test
- In situ density (Nuclear method and volume ring)
- Hand cone test
- Impact cone test
- Menard pressiometer test
- Cone pressiometer test
- Soil settlement field test

The first 4 tests are performed in the laboratory, whereas the second group of tests is performed in the field during and after installation.

- *Sieve test*
  This test is performed according to NEN 2560.
  In this test the grain size is determined by sieving.

- *Maximum Proctor density*
  This test is performed according to RAW 1995, proef 5.1 STD.
  In this test one start of with dried soil. The weight of the soil sample is measured, after compaction of the soil in a calibrated cylinder. Then a percentage of water is added to the soil and the previous procedure is repeated. By doing this several times with different moisture contents, one obtains a curve from which the maximum density and the related optimum water level is obtained. Later on, the soil samples taken in the field and weighted are related to this maximum density.

- *Oedometer test*
  In this test the consolidation of the soil is tested. It is comparable to the creep ratio test in plastics, but now the same sample is tested at different load levels. The method used is the Keverling Buisman procedure. The sample has a diameter 65 mm and a height varying between 19 and 20 mm.

- *In situ density*
  The in situ density has been determined using two methods, one called the volume ring method and the other called the nuclear density test.
  In the first test, an undisturbed sample is taken from the soil, and weighted. The weight is related to the proctor density found in the laboratory. The second test makes uses of an X-ray source that transmits its radiation through the soil. A receiver is measuring the amount of radiation received. This amount is a measure of the density of the soil.

- *Hand cone penetrometer*
  With this test a small cone is driven by hand into the soil up to depths of 30 – 60 cm. Meanwhile the force is measured.

- *Impact cone*
  This test is very familiar with the static cone test, only now the cone is driven into the ground by using impact energy. The cone, connected to a rod is placed at the soil surface. Then a falling weight is dropped from a fixed length on the rod. The number of blows needed to drive the cone 30 cm into the soil is a measure for the compaction of the soil.

- *Grain shape*
  The shapes of the grains are classified acc. to NEN 5104 and the scale acc. to M. Powers. The shapes, hooked or round, tell something about the geo-technical history of the soil, as well it can help to understand some of the properties.

- *Menard pressiometer test*
  With this test the load-deformation behaviour of the soil is characterised. The test is done in both native soil, as well as shortly after installation. The method used is described in the French standard N NFP94-110.
  First a hole is bored up to the appropriate depth after which a probe is lowered into the borehole. Then by pressurising the probe, and monitoring the volume change, information about the behaviour of the soil is gained. This pressurising is done stepwise, by which a curve of pressure against volume change is obtained. From the test the yield and failure stress of the soil is determined, as well as the soil stiffness value, which for our purpose is probably the most interesting.

- *Cone pressiometer tests*
  This test is in many aspects the same as the Menard test, with the difference that now the hole is not made by boring but by penetration.
  The experience of Fugro is that the latter test is less sensitive for operation, as some comparative tests showed that the reproducibility is better of the cone pressiometer test.
  The Menard test is difficult to perform in loose soil, as the bore hole easily collapses. This was also shown during the trials. With the Menard tests more test failures were shown.
  In light of the objective of the test programme with TEPPFA it was decided to use both test methods, especially since the cone pressiometer test allows to study the effect of unloading and reloading, which provides potential data for further parameter analysis.

- *Cone penetratione test*
  The Cone penetration test used in this programme is the so- called 'Landscout' . It is a light sounding tool, since a relatively small cone is used. This makes the tool more sensitive for local changes in soil stiffness, and it allows penetrating very close to the pipe. Also this test is performed in both native and back filled soil.

- *Tri-axial tests*
  These tests have been performed for the clay used. It describes the clay's behaviour when loaded, and moreover give information about the failure conditions of the clay.

5.2.2 *Pipe properties*

All laboratory tests on the pipes were performed at the laboratories of Wavin M&T, certified acc. EN45001, nr. L054.
The following pipe properties were measured :

- Diameter
- Wall thickness
- Pipe stiffness (ISO 9969)
- Creep ratio (ISO 9967)

5.2.3 *Pipe deflection*

The pipe deflection defined as the relative change of the diameter measured in the vertical direction is the most important measurement to monitor the pipe soil system at different stages of installation. The deflection, expressed in percentage of the nominal outside diameter, is per definition positive when the diameter decreases, and negative when the diameter increases.

The deflection is measured continuously along the pipeline, and results in graphs showing the deflection as a function of the location along the pipeline. It allows the determination of the minimum, average, maximum as well as of the more characteristic 95 % probability level. During the measurement also the horizontal deformation is measured. This value is used as a check up of the vertical. The pattern of the deflection graph of the horizontal versus the vertical will be different in case dirt or other disturbing facts are present. This feature is especially important when measuring operational pipelines. The absolute accuracy of the deflection measurements in pipes with a diameter of 315 mm is +/- 0.25 %.

Deflection measurements have been performed after every significant step during the construction work.

5.2.4 *Pipe strain*

Pipe strain measurements have been performed at every pipe section. The gauges have been glued in the circumferential direction of the pipe. At each cross section 8 gauges are glued, so at every 45°. The strain gauges are intended to give additional information on the pipe performance, especially when certain events take place in the field, like the construction work, the traffic loading and the determination of the pipe response when loaded by internal pressure.
Before using the strain gauges in the field, several validation experiments have been performed in the laboratory, showing that the relative accuracy is within +0 / -10%. Strain gauges tend to underestimate the strain.

6. RESULTS

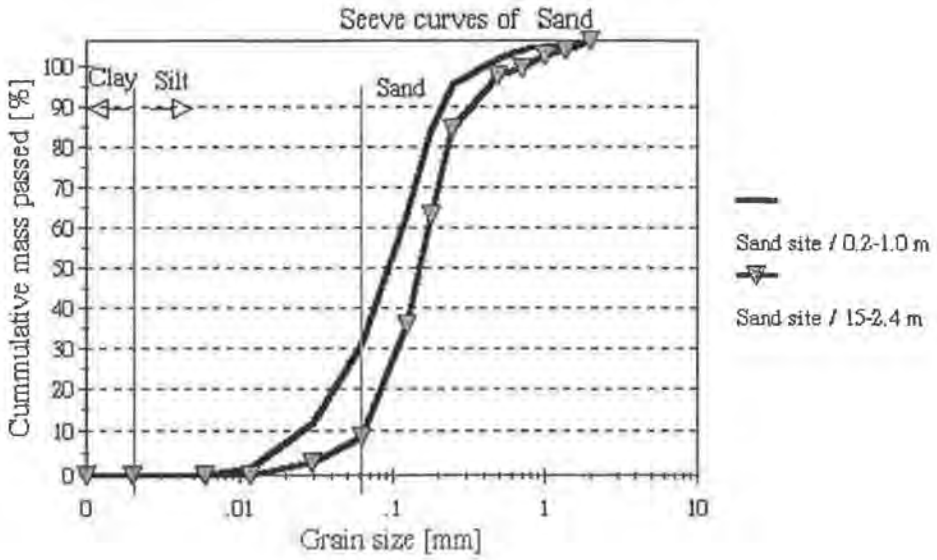
The results of the measurements will be discussed in this section.

6.1 Soil properties

In Enclosure 6.1 one will find more detailed results of the in-situ properties.

In this paragraph part of the results will be discussed.

Figure 6.1 shows the grain size distribution of the soil at the test site Haarle

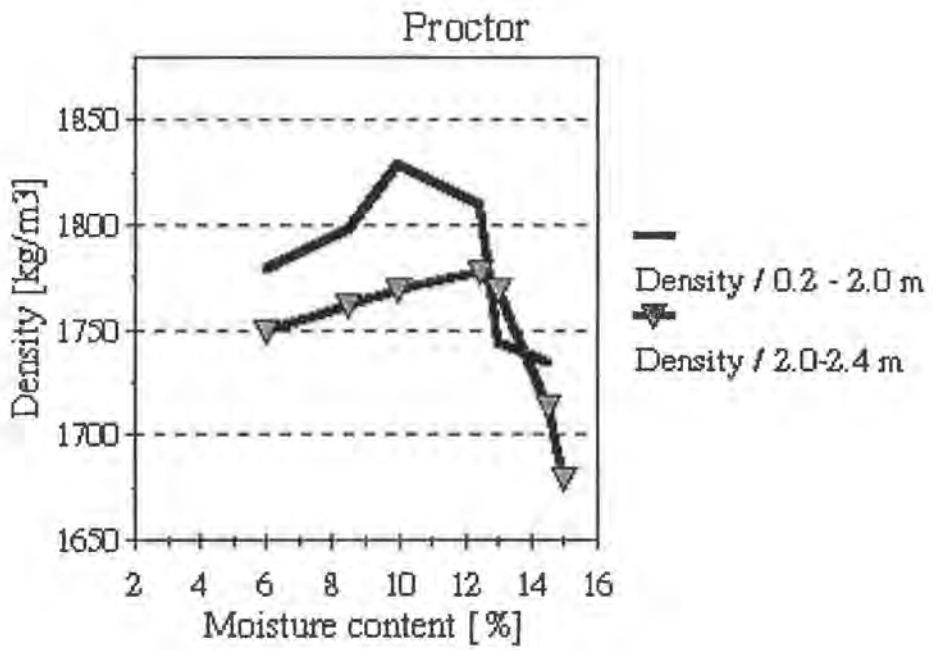


This graph shows the grain size distribution for the sand found at the sand site trial in Haarle. The upper line in the graph shows the result for a sample taken at a depth between 20 and 100 cm. The second line shows the grain size distribution for a sample taken at a depth between 150 and 240 cm. More samples at depths in between were taken at this site, and the distributions are in between the two shown in the graph. An important classifier for the soil is the amount of fines that is present in the soil.

Grain sizes of less than 0.063 are considered as fines, and in several classification systems, the 12% level is chosen as discriminative, between good granular soils and the mixed type slightly cohesive soils.

An example of the proctor test is shown in figure 6.2

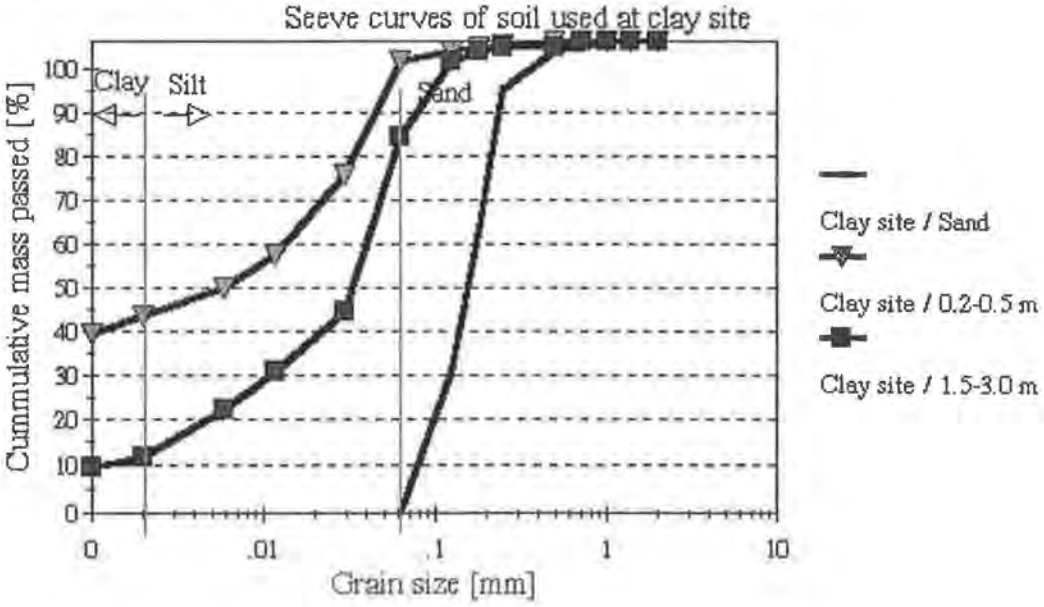
Figure 6.2 : Result of proctor tests at "Haarle" site



What can be seen from this graph is that the maximum density can be achieved when moisture content of about 11 % is present in the soil during compaction. As soon as the moisture content increases, like when the installation is performed in rainy weather, then this optimum density can not be reached.

In figure 6.3 the grain size distribution for the sand used in Wons is shown.
For the installation type designated as "well" sand was used as embedment material, which material was careful compacted. For the installation type designated as "None" the native soil was used. All three curves are shown in figure 6.3

Figure 6.3 : Grain size distributions of soil used in the "Wons" site.



During and after the installation a lot of soil testing was performed. By using the volumetric sampling, one was able to measure the proctor density reached after installation.

In case of the 'none' type of installation, a density of 80-85% standard proctor was found, as when the installation was done utilising a 'well' type of installation, values of 96-100 % proctor were found.

6.1.1 *Soil stiffness*

The cone penetration test, impact cone test, Menard test and cone pressio-meter test are performed to obtain information on the strength / stiffness properties of the soil. These data will be discussed in this section.

The data have been averaged in table 6.1 in order to get a quick impression about the values obtained for the different properties.

Table 6.1 : Soil properties

Property	Well	Moderate	None
Cone resistance [mPa]	6.22	4	0.75
Impact cone [Number of blows]	44	40	8
E cpm [mPa]	8.96 (7.7)	7.7 (7.4)	1.28 (0.93)
Yield cpm [Mpa]	0.73	0.68	0.17
Shear CPM [Mpa]	3.94	2.9	0.48
Failure CPM [Mpa]	1.21	1.09	0.24
E menard [Mpa]	7.45 (5.87)	3.4 (3.4)	(1.3)
Yield Menard [Mpa]	0.54	0.3	
Shear Menard [Mpa]	2.85	1.3	
Failure Menard [Mpa]	0.86	0.53	
Esoil Schmertmann [Mpa]	12.44	8	1.5

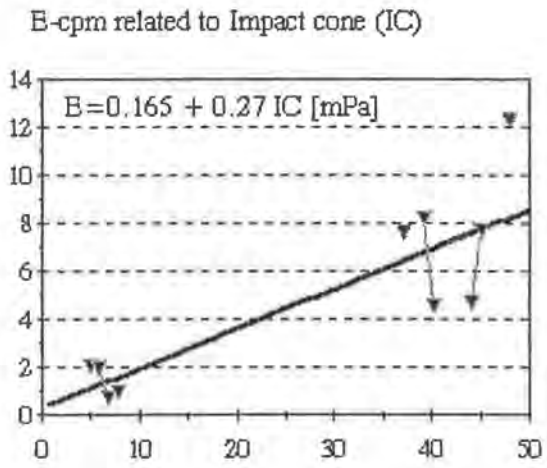
Note : The values between brackets are the average values when also the results from the clay trail are included.

The "Esoil Schmertmann" is obtained by using the information given by John Schmertmann in literature 3. He proposed a relationship between Esoil and qc (the cone bearing capacity) as follows: Es=2*qc for ordinary sand including silty sands. In his paper Schmertmann compared the cone resistance results with the result of the screw plate load test. Following the approach of Schmertmann, an approximate relation can be established between the cone penetration test and the E-Menard as derived from the Cone pressiometer tests.

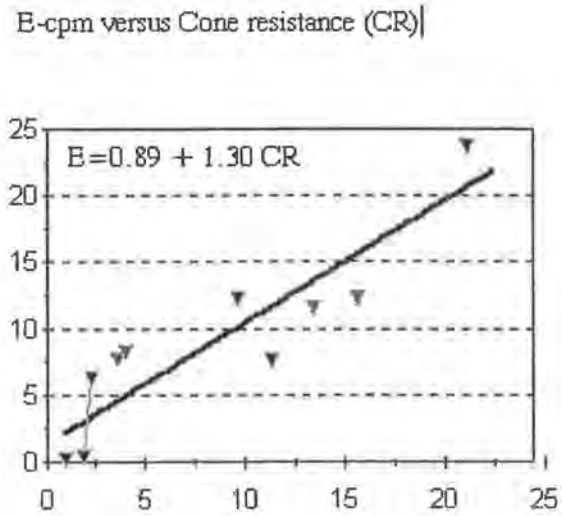
In Figure 6.4 several correlations are shown between the measured soil properties.

Figure 6.4 Correlations between soil properties

The graph below shows the relation between the impact cone, so the number of blows and the Young's modulus as determined with the cone pressiometer test.



The following graph shows the relation between the cone resistance and the Young's modulus as determined by using the cone pressiometer test.



It falls outside the scope of this paper to discuss the soil properties extensively. Using the result of the tests carried out herein; one can draw a band in which the soil modulus is likely to be found. In the first graph, the modulus is plotted relative to the number of blows from the impact cone penetrometer. This last instrument is the easiest useable instrument to check the compaction of the soil in-situ. The instrument can be handled without being specifically skilled. It provides an "a la minute" result, and one can immediately adjust the compaction effort or procedure in order to arrive at the desired compaction level. Further results of soil testing are given in Enclosure 6.1.
In Enclosure 6.2 a table is added with the description of the soils as used in prEN1046, as well as in some draft CEN documents on structural design.

6.1.2 *Pipe properties*

The results of the measurements on the pipe are listed in the table 6.2. For the description of the cases, one is referred to Table 5.2

Table 6.2 : Pipe properties

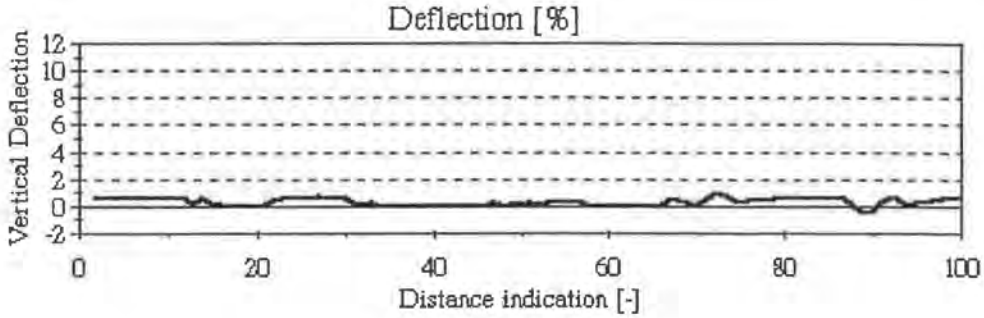
Case	Pipe material	Wall thickness	Measured stiffness / nominal stiffness	Creep ratio
1,2,7,8	PVC	8.3	5.37 / 4	1.62
5,6,13,14,15,16	PE	12.97	7.99 / 6	2.87
3,4,9,10	PVC	7.17	3.52 / 2	1.3
11,12	Steel	2.0	3.7 / 4	1.02

6.2 **Deflection measurements**

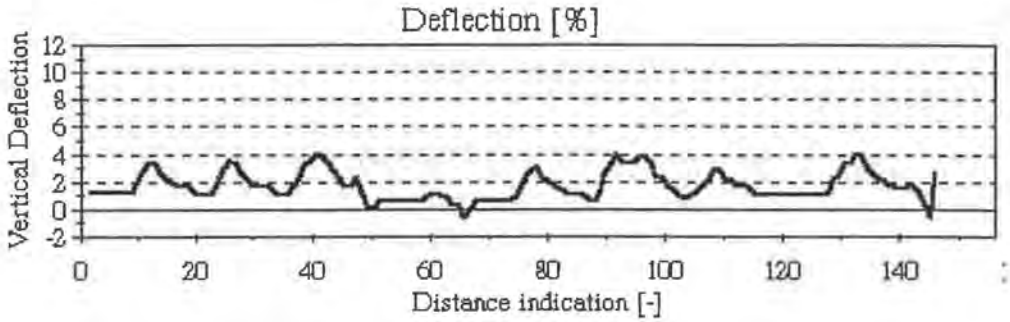
In this section the results of the deflection measurements are discussed. The emphasis will be to discuss what has been learned from the measurements. For a full description of the data sets, including all the details about the results one is referred to section 6.4

First three examples of measured result will be shown. Results of a "well", "moderate" and "none" type of installation or shown.

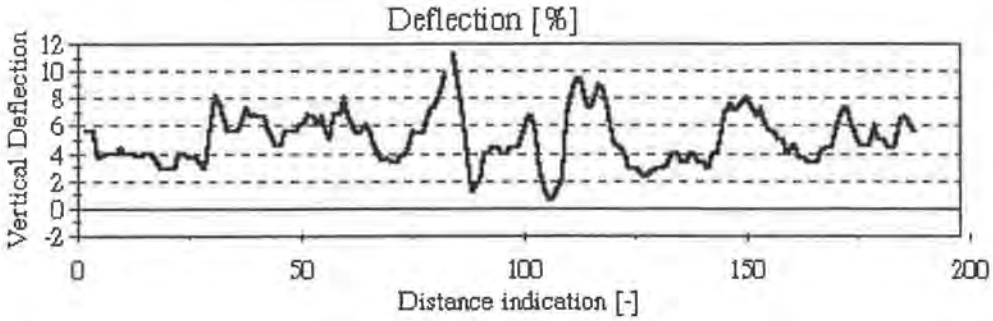
Figure 6.5 : Deflection graph for three types of installation.



- *Installation type "Well"*



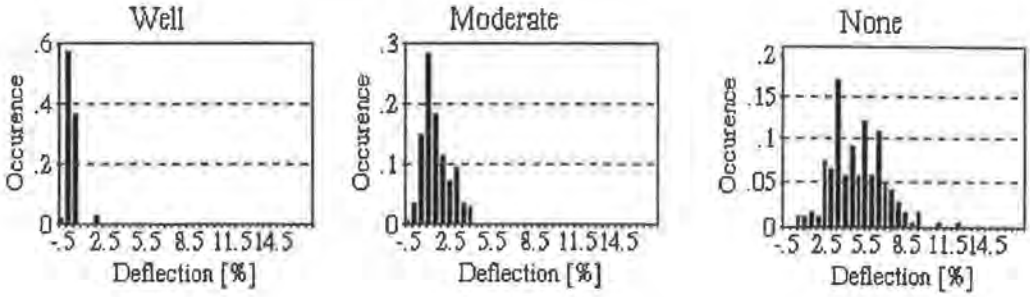
- *Installation type "Moderate"*



- *Installation type "None"*

The distance indicator on the horizontal axis presents a control parameter of the datalogger. The length of each pipe section varies between 10 meter for the steel pipe up to 25 meter for the PVC and PE pipes. It has been explained before, that a sufficient length of pipe needs to be installed in order to allow the contractor to utilise a normal working procedure. In this way also a good impression about the variability to be expected along a pipeline can be obtained. For the three installations as shown above the probability density function is shown in the graphs in figure 6.6.

Figure 6.6 : Probability density function for the three installation types.



It is very clear from the graphs above that the pipe deflection depends very much on the type of installation used. The averages as well as the maximum values are affected by the type of installation. When performing the analysis on all pipes measured, a good estimate for the standard deviation was possible. The graphs above already indicated the importance of the type of installation, but also the pipe stiffness has an affect on the variability parameter, although to a much lesser extend. In table 6.3 the recommended values for the standard deviations are shown

Table 6.3: Summary of standard deviation values related to the type of installation and the pipe stiffness.

Table 6.3 : Summary of variability factors

Nominal Pipe stiffness [kPa]	Well	Moderate	None
2	0.80	1.50	2.40
4	0.50	0.90	2.00
8	0.50	0.80	1.40
16	0.50	0.50	0.50

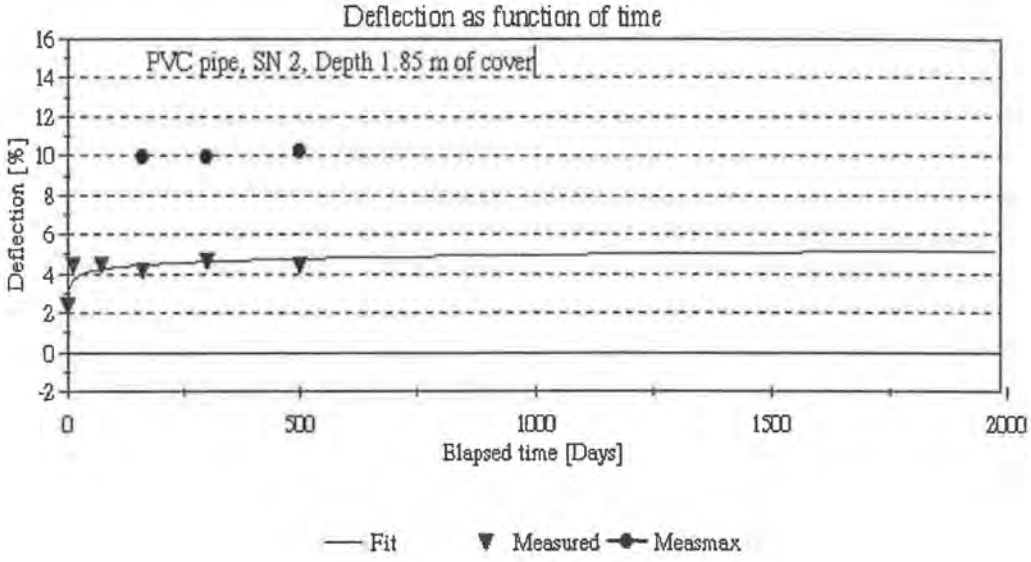
In this way the 95 % probability value can easily be found by adding 2 times the standard deviation to the average value. The standard deviation acts as a kind of installation factor since; as was shown the factor depends largely on the type of installation used.

Another way to obtain a reliable estimate for the maximum deflection level is to utilise a factor of 2. This was shown in literature (4)

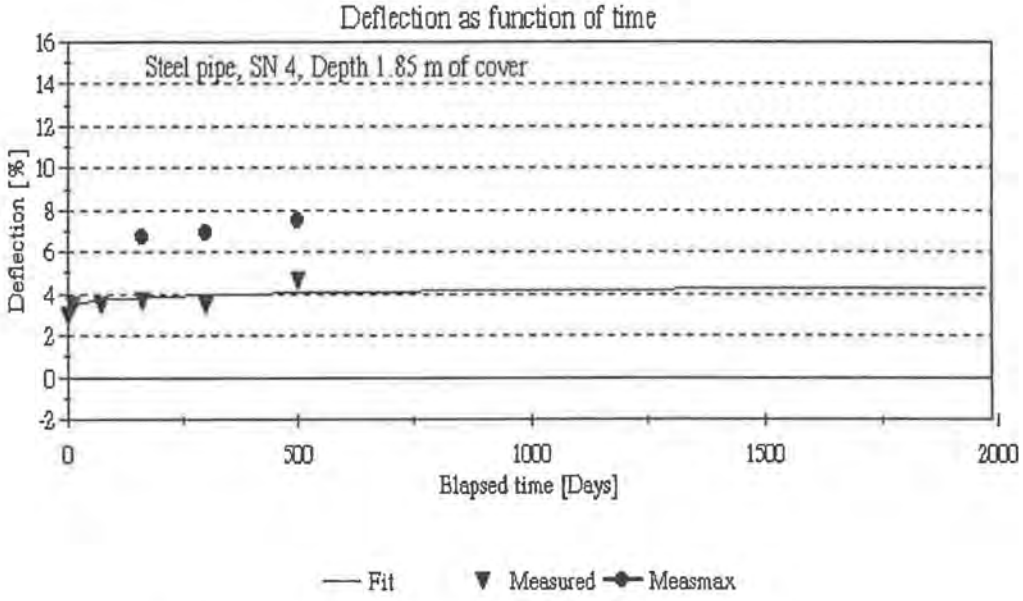
The pipe deflection has been measured several times during and also after installation. These types of measurements are needed to establish the time dependant behaviour of the pipe soil system. For that reason next to plastics pipes, also steel pipes have been installed.

A few examples of these deflection time curves are shown in Figure 6.7.

Figure 6.7: Examples of deflection as a function of time



- *PVC Pipe, installed using a "None" type of installation*

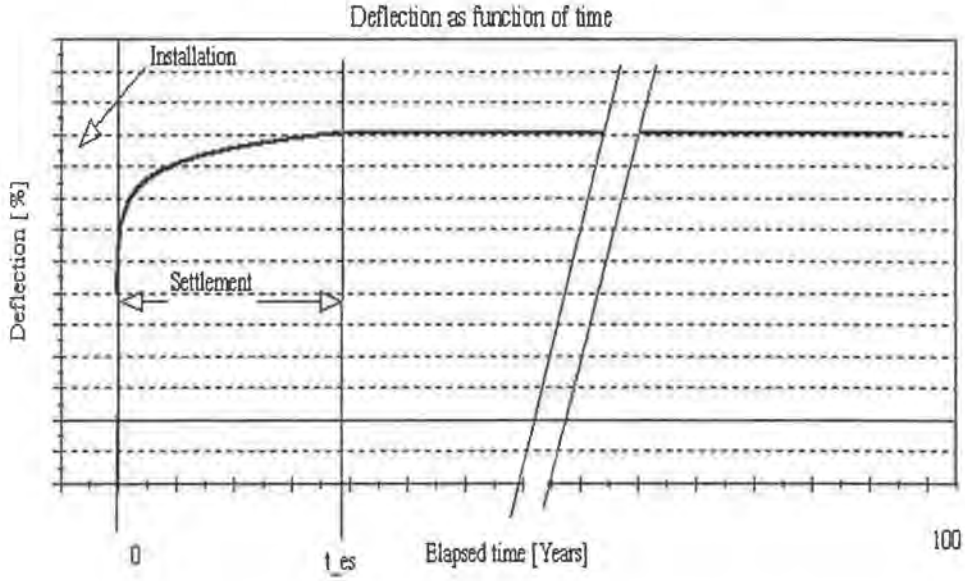


- *Steel Pipe using a "None" type of installation*

The graphs show that the pipe deflection increases slightly after installation, eventually reaching a practical limit. It is also clear that the increase in deflection with time is about the same for the steel and the PVC pipe. This indicates that the increase is primarily caused by the settlement of the soil. The settlement is the ruling factor.

In general, the pipe deflection versus time can be characterised in the following way as presented in figure 6.8

Figure 6.8 : Schematic overview deflection process



The process is divided in three different parts. The first part is the installation phase, and at the end of this phase the installation deflection (δ/D)_inst. is achieved.

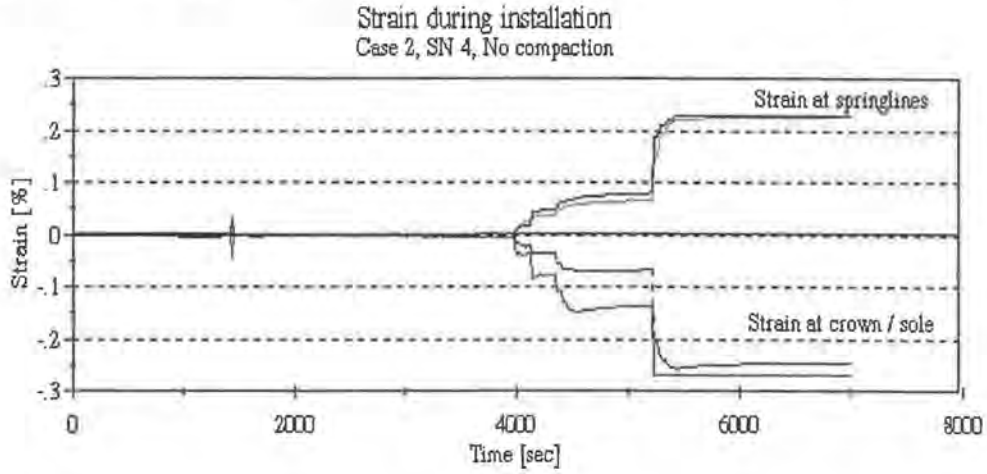
The next phase, the settlement phase, shows an increase of the deflection until a practical limit is reached, (δ/D)_ult. Then the third phase is the phase were no significant increase of deflection takes place. In section 7.4 when discussing the results in more detail, this scheme will be further discussed.

6.3 Strain measurements

The strain at one cross section for each condition has been measured during installation, trafficking, and pressurisation. In this section some typical examples of strain measurements during installation will be shown. It shall be mentioned that measuring strain is a rather delicate process. The strain gauges are rather sensitive and need therefore to be protected against mechanical damage and moisture. On the other hand if they are protected to carefully then the protection might introduce false readings. In this study a feasible midway was chosen. The gauges were protected using aluminium tape and a water-resistant paint.

Nevertheless, several gauges were damaged and turned out not to be able to be calibrated and zeroed. In figure 6.9 the strain development during installation of a PVC pipe, SN4 using a "None" type of installation is shown. The installation start at the point indicated by 4000 seconds on the horizontal axis. What happens is that the deflection increases due to the soil placement, and increases suddenly when a crane start compacting the whole backfill at once.

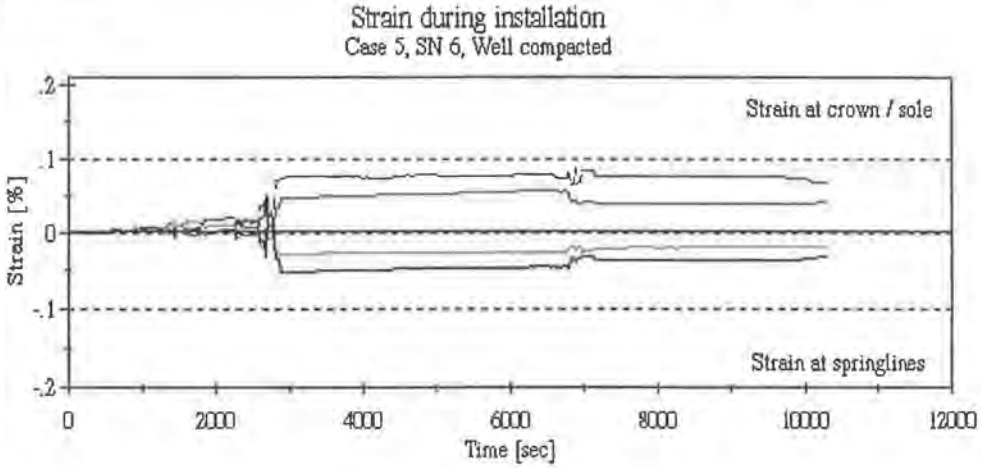
Figure 6.9: Strain development during the installation of a PVC pipe, case 2 using a "none" type of installation.



The strain measured at the springlines shows to be in tension, and the strain values at the crown and sole are in compression. The graph is rather symmetrical, indicating that the deformation pattern is also symmetrical.

Figure 6.10 shows the same type of strain graph, but now for a well type of installation.

Figure 6.10: Strain development during installation for a "Well" type of installation



It shows that the strain values are smaller, and that the positive strain values are now located at the crown and sole of the pipe. At the springline the strains are in compression. The pipe has taken the shape of a standing egg. The strain values are rather small in case of a "well" as well as for a "none" type of installation. Remember that the allowable strain in thermoplastics pipes varies between 2.5 and 5 %. Values that are used as practical limits, but still exhibit a huge degree of safety against failure.

6.4 Formal description of data sets

The results of the measurements are summarised in one sheet per installation case, involving a plot of the deflection along the pipeline, a plot showing some selected soil properties measured after installation and a plot of the deflection as a function of time.

Furthermore two tables are added, one showing the basic characteristics of the installation and the pipe, and one summarising the characteristic results. In this results table, a value called "t_100 [Days]" is added. This is the time after which a further increase of the deflection with 1 % up to 100 years can be expected. The sheets are added in Enclosure 6.3. It shall be mentioned that the groundwater table at the test site "Haarle" has increased to about –50 centimetre to ground level, some 600 days after installations. This means that also the shallow buried pipes are under the groundwater level.

Observations :

- ***Groundwater only effects the pipe deflection in granular soils when the water is raised the first time. It causes the settlement of the soil to speed up.***
- ***"Well" installed pipes show low deflection levels, around 0 %. The increase of deflection is hardly visible.***
- ***When pipes are installed using a "well" type of installation, then the soil properties are significantly higher than in case the pipes are installed using "None" type of installations.***

6.5 Reference studies

In this section an overview of existing experience with the behaviour of buried thermoplastics pipes is given. First it classifies the work in categories and discusses the merits of the different types of work. Following this, the results are compared with each other. A lot of work has been carried out over a period of more than 40 years, by different researchers in different countries. Some of them related the work to each other, whereas others have not made such a link at all. In this section it is not intended to be complete, but it discusses some of the most significant experimental work that have become in reach of the author over a period of 15 years of studies in the behaviour of buried pipes.

The work is classified in the following three categories:

a. Box load tests
b. Numerical simulations
c. Full scale tests

Table 6.5 : Overview of the merits of each approach.

Parameter	Box loading	FEM / FED	Full scale
Cost (Relatively)	low	low	expensive
Estimation of deflection	under	under	-
Involve workmanship	non	non	yes
Natural settlement	non	difficult	yes
Estimate variability	non	difficult	yes, pipe shall be long enough
Allow live loads	non	difficult	yes
Fit with real life	low	low	fit
Parameter effect	tendency	tendency	relative effect
Isolation of effects	good	good	difficult

In general the statement that box loading tests allow performing tests under very well defined conditions is valid, and hence the costs are normally lower than in case of full-scale tests. A typical property of box loading tests is that the pipe soil system responds more rigid than in a full-scale test. Most box loading tests allow only pipe lengths of 3-8 meter to be buried, leaving roughly 1 to 5 meter pipe to be studied without end effects.

As far as the numerical simulations, mostly FEM, are concerned it can be stated that already during modelling a lot of assumptions have to be done in order to make the problem solvable. From the analysis of the field data in the TEPPFA / APME study as well as from the results from others carrying out full scale studies it has become clear that the issue of the behaviour of buried pipes, is for the major part an initial condition problem. The errors in assumption are multiplied during the rest of the design process. Most numerical as well as analytical methods are not very strong in tackling these initial conditions. A typical property of FEM analysis is that it simulates a system stiffer than in accordance with reality.

From the above it is clear that full scale tests shall be preferred, even though they are in most cases more expensive than the other two categories of study. It is also for the above technical reasons that the TEPPFA / APME study utilises mainly the results of these full-scale studies. A short summary of some of the literature is given in Enclosure 6.4.

7. ANALYSIS OF THE RESULTS

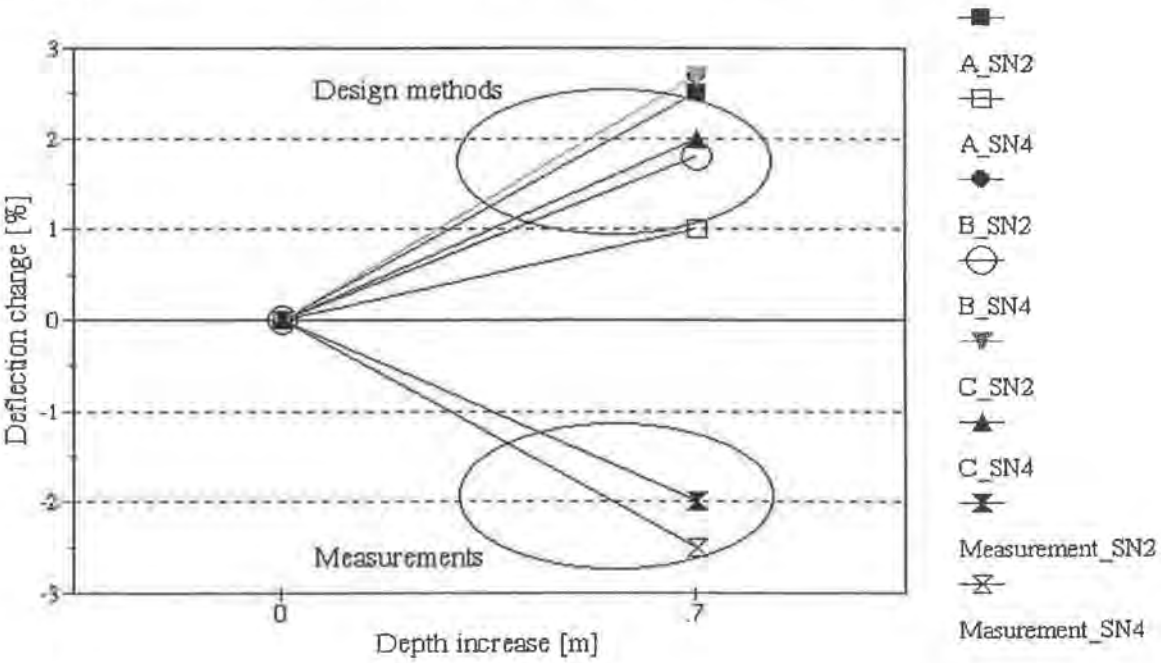
The results of the pipe deflection measurements will be further analysed and discussed in this section.

7.1 The issue of depth of cover

Pipes have been installed at different depths. According to current design methods deflections increase with increasing depth. In most cases the increase in deflection is proportional to the increase in depth, whereas others consider an increase in soil stiffness with depth, and hence the increase in deflection is less than the increase in depth.

In figure 7.1 the pipe deflection is plotted against the depth of cover. The horizontal axis shows the increase in depth. On the vertical axis the increase in deflection is shown. Not only the results of the measurements are shown, but also the results from 3 established design methods are shown. By using the deflection change instead of the absolute value in the graph, the results get less sensitive for wrong judgements of the soil properties. The letters A, B, C refer to the different design methods. The results are shown for pipes with stiffness of 2 and 4 kN/m2. The installation was of the type "None". When considering "Well" type of installations, the deflections tend to be so low, that such a trend analysis gets very sensitive for small errors in the calculated and measured results.

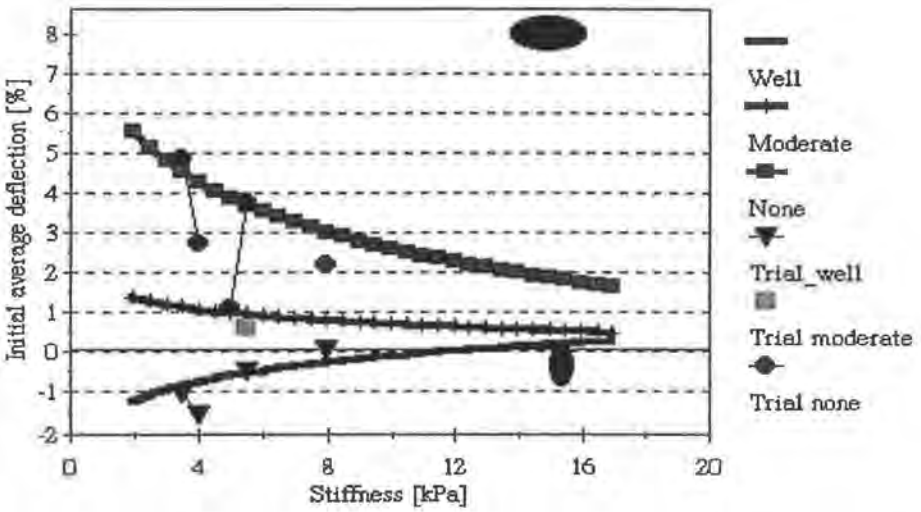
Figure 7.1 : Pipe deflection increase plotted against the increase in depth of cover



The result shows clearly that the design methods predict an increasing deflection with depth. Remarkable is that method 'A' makes quite a distinction between the pipe stiffness, whereas the other methods don't. The results of the deflection measurements show that the deflection becomes lower at greater depth.

Also the effect of pipe stiffness was studied, and in figure 7.2 the result is given. In this graph the average deflection after installation is shown plotted against the pipe stiffness, for the three installation types considered in this study. The results are conservatively fitted.

Figure 7.2 : Pipe deflection plotted against stiffness



What can be concluded from the graph is that the deflection is sensitive for the type of installation, and that the pipe stiffness is only relevant when considering pipes with stiffness less than about 4 kN/m2 in combination with "None" type of installation. It is also clear that pipes with lower stiffness then 4 kN/m2 can be used provided that one is sure that the installation is done in a well to moderate way. In case of a "well" type of installation it is shown that the deflection immediately after installation shows a negative out of roundness. The reason for this is that in order to obtain a "well" type of installation, a lot of compaction energy had to be applied because of the nature of the embedment soil used. If so called free floating soils, like gravel and clean sands would have been used, such negative deflection would not have occurred. Also for the low stiffness pipes around 0 % deflection would have been found.

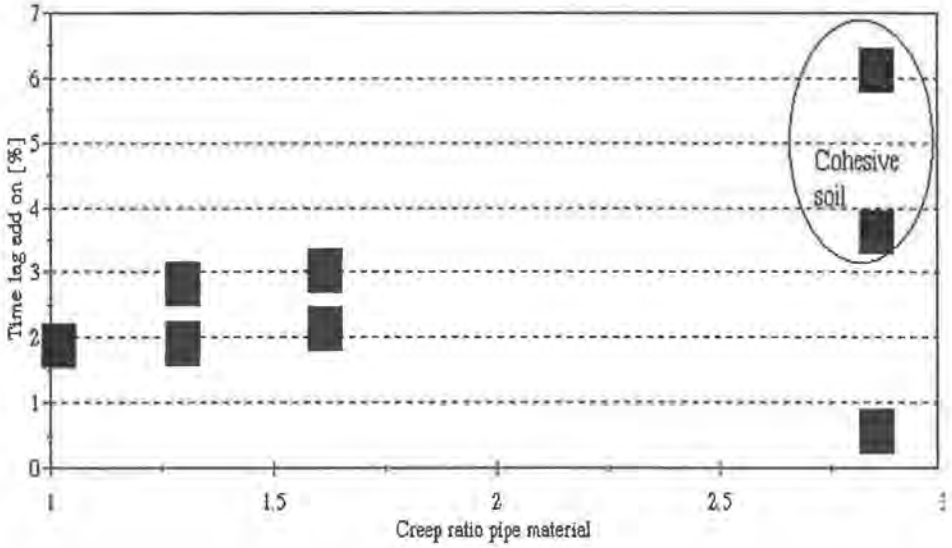
Thermoplastics pipes exhibit visco-elastic behaviour. Debates have been held for many years, whether the creep ratio is a determining factor for the performance of buried plastics pipes.

One theory considers the visco-elastic behaviour fully responsible for the increase in deflection with time, whereas another theory considers the soil settlement as being the cause of the time dependant behaviour. Spangler for instance already noticed that buried steel pipes showed an increase in pipe deflection with time. To accommodate for that increase he proposed a deflection lag factor of 1.5. In this study the effect of the creep ratio is studied by involving next to PVC and PE, also Steel pipes. Figure 7.3 shows the result of the deflection increase over a 100 years period as a function of the creep ratio of the pipes used when utilising a "None" type of compaction. The creep ratio's of the pipes used are measured and listed in table 6.2.

It shall be noticed that the deflections are extrapolated values. Until now measurements have been performed up to 750 days after installation for the pipes installed in sand and up to 500 days after installation for the pipes installed in clay. The measured values have been fitted with a log function, and as such the 100 years estimate has been found.

The fit is considered conservative, as other studies have shown (5) that the increase stops after about 3 months to 2 years depending on the type of installation used. When more future measurements will become available, the curve will show a less progressive behaviour and hence results in a lower extrapolated value. Nevertheless, the current conservative estimate can be utilised with confidence. The graph shows the result for the installations type "None".

Figure 7.3 Effect of creep ratio on deflection increase over an extrapolated 100 years period.

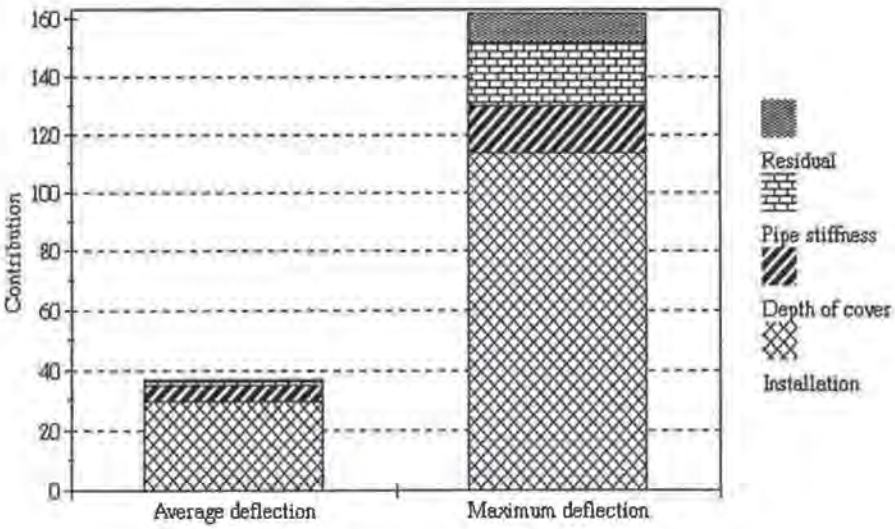


Note: The Time lag factor is an add on factor instead of a multiplier as in case of the traditional Spangler approach

The upright values are those from the trial using cohesive soil. In cohesive soil only PE was installed. The downright value shows the increase for the same pipe, but now installed in silty sand. The cohesive soil needs considerable more time to consolidate than the granular type of soils. It is clear from this graph that the creep ratio of the pipe is not important at all when considering deflection, but that the soil is really the governing factor. Further analysis of the figure shows also that the steel pipe, having a creep ratio of 1.02, shows a comparable deflection increase with that of the PVC and PE pipes.

Another approach that was utilised to discover the relative importance of the different parameters, was the use of the Analysis of Variance. This was done involving the sand trial only. The analysis done on the initial average and maximum deflections is shown in Figure 7.4. The figure shows the contribution of each of the factors (Installation, Depth of cover and Pipe stiffness) to the deflection, expressed as the sum of squares. The parameter pipe material was omitted in first instance. If this would be a wrong decision, then the residual obtained after the analysis would be relatively high. When however the residual becomes low, then the correctness of the decision is confirmed.

Figure 7.4: The effect of the parameters; Installation, Depth of cover and Pipe stiffness on the initial average and maximum deflection.



The result shows clearly that the type of installation is by far the most important parameter. The analysis confirms that pipe stiffness and depth of cover are not important at all; with the remark made in figure 7.2 is has been confirmed that the pipe stiffness becomes more important when pipe stiffness goes under 4 kN/m2 in combination with less good installation types. The assumption that the type of material (creep ratio) is not important either is confirmed, because of the very low residual found in the analysis.

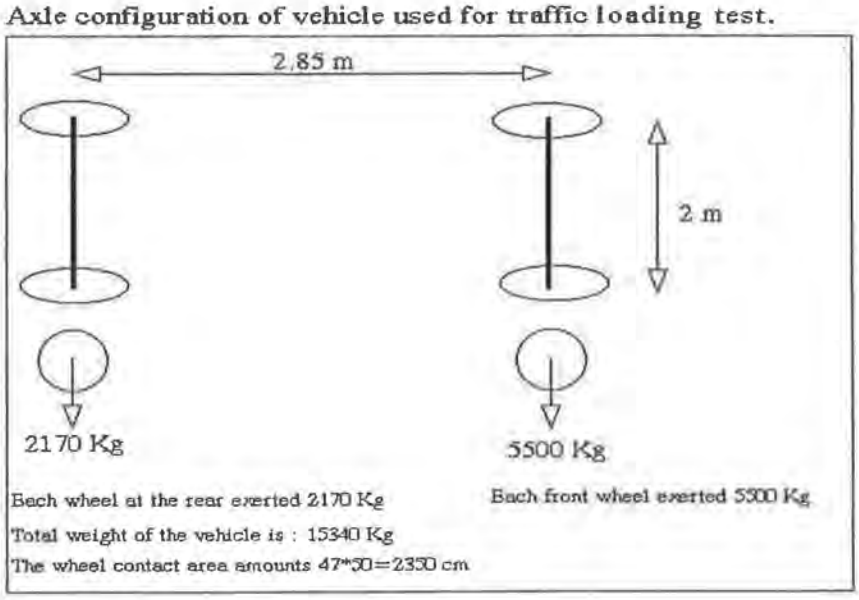
Observations :

- ***Depth of cover is not significant for the pipe deflection of flexible pipes***
- ***The creep ratio of the pipe material used is not significant for determining the pipe deflection.***
- ***Pipe stiffness is becoming an important parameter when considering pipes wit stiffnesses of less than 4 kN/m2 in combination with "None" type of installations***
- ***When pipes are installed according to the standards for installation than deflections will stay low and design using extensive design methods is superflous.***

7.2 The issue of traffic load

In the TEPPFA project full scale traffic loading tests have been performed on flexible pipes buried in silty sand , laying at a depth of 1.15 meter of cover. The axle configuration of the vehicle used is shown in Figure 7.5.

Figure 7.5 : Axle configuration and load during the traffic load test.

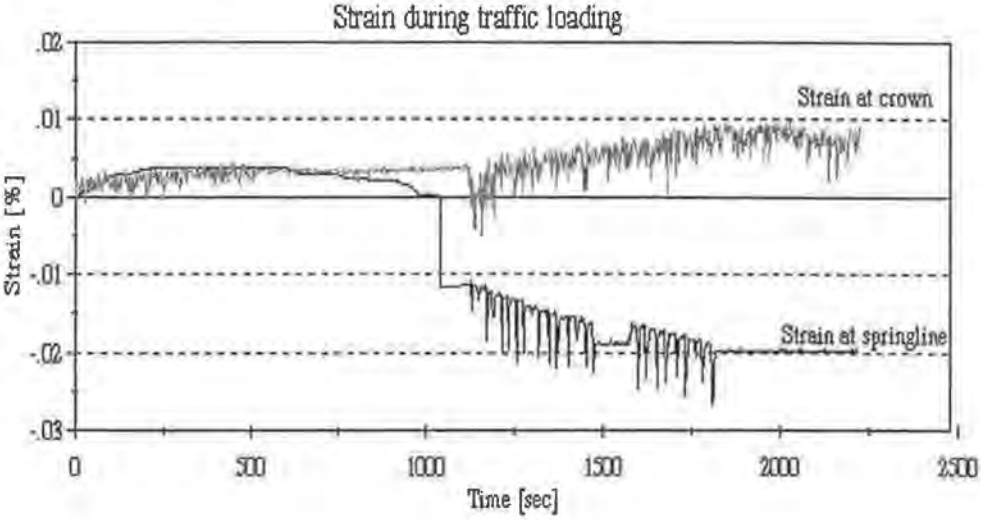


The wheel load passing in the axial direction of the pipe was 5.500 Kg spread over a surface of 0.23 m2 resulting in a surface pressure of approximately 239 KN/m2.

During the loading strain and deflections were measured continuously. The deflection was measured at one fixed point, and didn't show any change during trafficking. After about 60 passes the test was stopped, and the pipe deflection was measured over its whole length. The deflection had changed with 0.25%, which can be considered as non-significant.

The result of the strain measurement during the traffic loading is shown in figure 7.6.

Figure 7.6 : Strain measurement during traffic loading



The first part of the graph, up to 1000 seconds, loading was applied by driving just alongside the trench, so not straight over the top of the pipe, but at the interface native / backfill soil. In the second part, so after 1000 seconds, the truck started to drive straight over the top of the pipe. The strain at the crown shows an increasing compressive strain, and the strain at the side (spring line) increases in tension. The amount of increase is negligible (0.01%!) and not worth mentioning. It was however decided to include the graph in the report for reason that it demonstrates what happens when passing the pipe. It shows that every time the truck passes the increase gets less and less. The soil is settling further under the influence of traffic load, until no further compaction takes place. The deflection change was as mentioned before hardly measurable.

It was decided that the traffic case would be further studied by a literature survey, after which possible additional tests would be carried out on the installed pipes in the TEPPFA project.

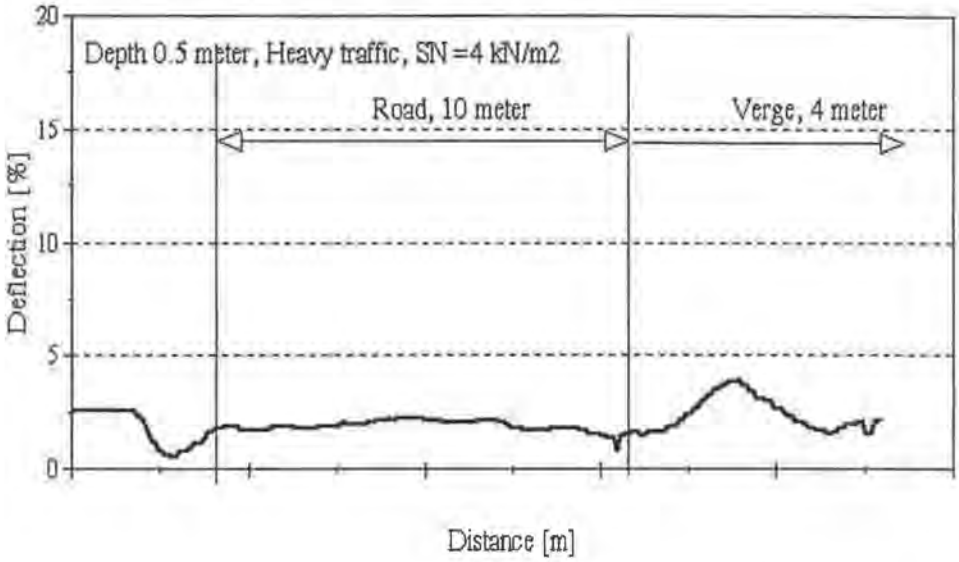
As our main concern is the performance of flexible pipes, attention has been given to these pipes. The results will be discussed as cases.

The Rollepaal case

Wavin M&T buried four dummy pipes under the Rollepaal road at a depth of cover of 50 centimetre when this access road to the industrial estate "De Rollepaal" in Dedemsvaart, was renovated and redone in 1980. The road carries the heaviest vehicles, since along the road quite heavy industry is situated, like machine producing companies, furniture and mailing companies. Of course, as in any serious road structure, a proper type of sand was used. The road cover consists of asphalt.

The length of the pipes was 14 meter from which 10 meter is buried under the road, and 4 meter was buried in the verge. The pipes were for the first time measured some 4 months after installation. A few graphs are shown in the Figure 7.7.

Figure 7.7: Deflection measurements on pipes buried at 50 cm cover in a heavy trafficked road.



In the graph vertical lines are drafted to indicate where the pipe was buried under the road, and where it is buried in the verge. There it can be seen that the deflection pattern is very regular, and the average deflection is rather low. The part buried in the verge shows a more irregular pattern and the deflections are higher. This indicates that in the road area more care has been exercised. It is clear from these measurements that the effect of a careful versus a moderate type of installation is still visible after 18 years.

A summary of the results is shown in table 7.3:

Table 7.3: Summary 18 years reading on PVC pipes buried at a depth of 50 cm, exposed to heavy traffic.

Pipe type	1980	1981	1991	1998
SN 2	0.0 (2.0)	0.0 (2.0)	0.0 (3.5)	0.0 (3.5)
SN 4	1.5 (2.0)	2.0 (2.0)	2.0 (3.0)	2.0 (3.0)
SN 4	0.5 (1.5)	0.5 (1.5)	0.5 (2.0)	0.0 (2.0)
SN 8	1.0 (2.0)	1.5 (2.0)	1.5 (3.0)	1.5 (3.0)

Note : The values between brackets are the average deflections found in the verge
The values without brackets are the values for the pipe under the road.

- It is shown that the pipe deflection of the pipes in the road has not increased significantly over the last 18 years. It turns out that after a first settlement period the deflection has stabilised.
- The pipe deflection of the pipes buried in the road is lower than those buried in the verge. The effect of installation quality and soil type is clearly visible here. Although the soil is the same for both cases, the compaction is less good in the verge, and also the native soil is less good. The variation in deflection existed already from the very start, and hence the higher deflection is caused during installation.
- Pipe deflection of flexible pipes when buried using careful and moderate installation techniques in combination with granular soils, has no relation with long-term pipe material properties.

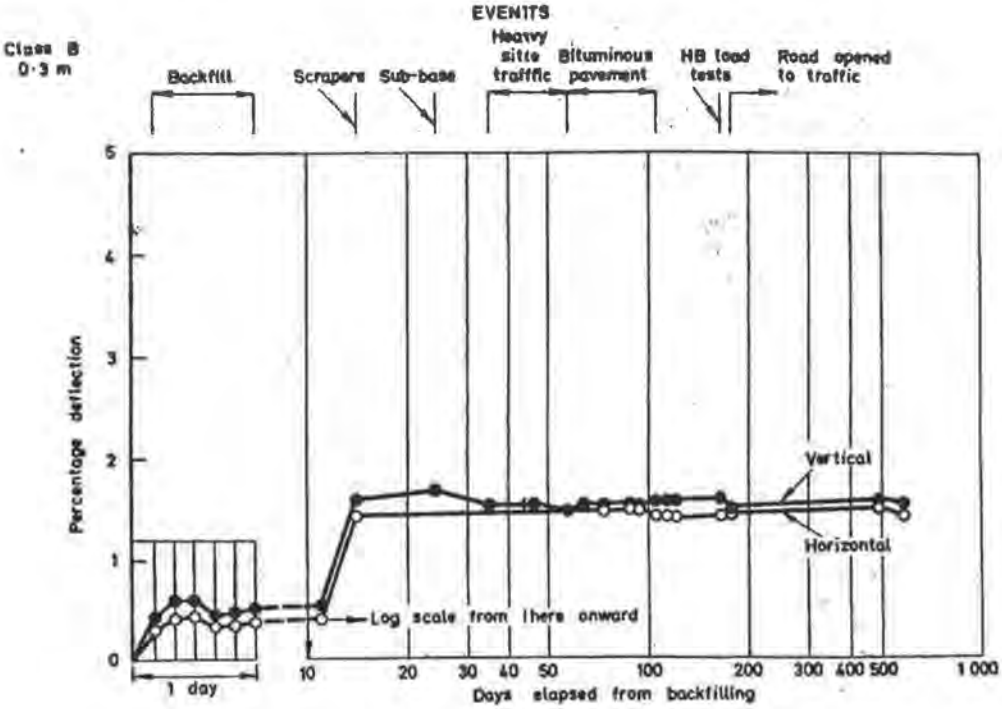
The Weimar case

The companies Pipelife, Omniplast and Uponor Anger sponsored a study into traffic load by the University Weimar. Three pipe types with stiffness of respectively 8 and 32 KPa were buried here at a depth of 50 centimetre. Pipe deflection was calculated by using the ATV127 and the deflections were measured in the field. The pipes have been loaded by a heavy vehicle resulting respectively in a surface pressure of 1324 kPa and 1523 kPa. If these values are compared with the results of an SLW60 load, than in the latter case the surface pressures including the impact factor is 1000 kPa. The results showed that the calculation method results in a overestimation of the deflection by a factor 3 to 4.

The TRRL case

In 1974 J.J Trott and J. Gaunt (6) of the Transport and Road laboratories published the results of an experimental study on a uPVC pipe laid beneath a major road during and after construction. There it was shown that the most significant change took place during the first 5 days out of ten when heavy scrapers passed the pipe on a regular basis. The authors estimated that the scrapers had passed 70 – 100 times during this period, partly unloaded (15 tons), partly loaded (32 tons). In this stage no road structure was established. After the road structure was completed they also carried out a dynamic test using a special vehicle with a static axle load of 38 tons. After this the road was open for traffic. Figure 7.8 shows some of the results.

Figure 7.8 : Results of the TRRL work



It is shown that the deflection increase only took place in the first part of the loading sequence, whereas later on even heavier loading didn't cause a significant increase of the pipe deflection due to traffic.

The York case

In the UK, in York, pipes of a stiffness of 8 kPa have been installed and loaded by traffic.

The results of these trials part of those listed in Lit. 4. In summary it was shown that the pipe deflection increased after the first passes of the truck, but after about 30 times passing the pipe, no further increase was noticed.

The results of the Rollepaal case as well as that of Trott and Gaunt showed that pipe deflection increases the first period after installation. Then the increase stops, which can be explained by the fact that the soil density has reached it's maximum. No further practical compaction and hence deformation of the soil and pipe can be achieved. The pipe deflection is clearly managed by the volume changes, and not by the application of the load itself. The volume of the soil changes under influence of weather conditions and as in case of the traffic loading by the load exerted to the soil.

The Weimar case showed that the deflection increased by artificial loading, however the loading was not repeated so this case does not support this hypothesis yet.

The results of the Weimar case showed that the design method used overestimated the traffic load effect considerably. Stiffness used were 3, 7 and 5 MPa for respectively the embedment, topfill and native soil. Studies performed in the past showed that the real stiffness is much higher, and increase with deformation (compaction) of the granular soil.

Observations:

- ***Traffic and surcharge loads do not effect the flexible pipe deflection after the installation phase. It is likely that the threshold stiffness at which traffic has an effect on the pipe performance depends on the pipe stiffness relative to the soil. In case the pipe becomes the stiffest member it is likely to assume that the load will be born by the pipe.***
- ***As a summary it is recommended to disregard traffic load for flexible pipes.***
- ***The final deflection depends on the consolidation characteristics of the embedment.***

7.3 Effect if internal pressure

A few pipes were pressurised half a year after installation. During pressurisation the deflection at one spot and the strain were measured. The pipes pressurised were all buried in sand. The same kinds of tests are planned to be carried out in clay, however it was decided to postpone these tests to the moment that the pipes will be digged up. In this way the project is allowed to study the pipes under gravity conditions for a longer period of time.

At this stage of the reporting only the measurement results of the sand case will be given, and further discussed when the results of the pressure tests in clay are also available.

For pressure pipes there are actually two extreme cases clear. The first one is the situation that the pipe is buried in such a stiff soil that the pressure will not be able to reround the pipe. At the same time the tensile stresses due to internal pressure can not be developed, because the soil does not allow expansion of the pipe, which is needed to develop this stress. The pipe acts as a membrane between soil and medium. The other extreme condition that can be recognised is the situation of a pipe buried in very soft soil, like peat. In such cases the pipes will completely reround and will become in a same type of condition as during internal hydrostatic testing. In real life both conditions are feasible, and many conditions in between.

To what extend the internal pressure is a relevant design case for thermoplastics pipes, more than only considering the pressure itself is currently under discussion in a separate group, SG1 of CEN TC155. The field trials of this study and it's results will be used to support this group.

In table 7.3 the results so far are summarised.

Table 7.3 : Results of preliminary rerounding tests, using 4 Bar pressure

Change in	PE, SDR 26	PVC, SDR41
Deflection	0.25	0.25
Strain at spring line	0.33	0.19
Strain at crown	0.4	0.15

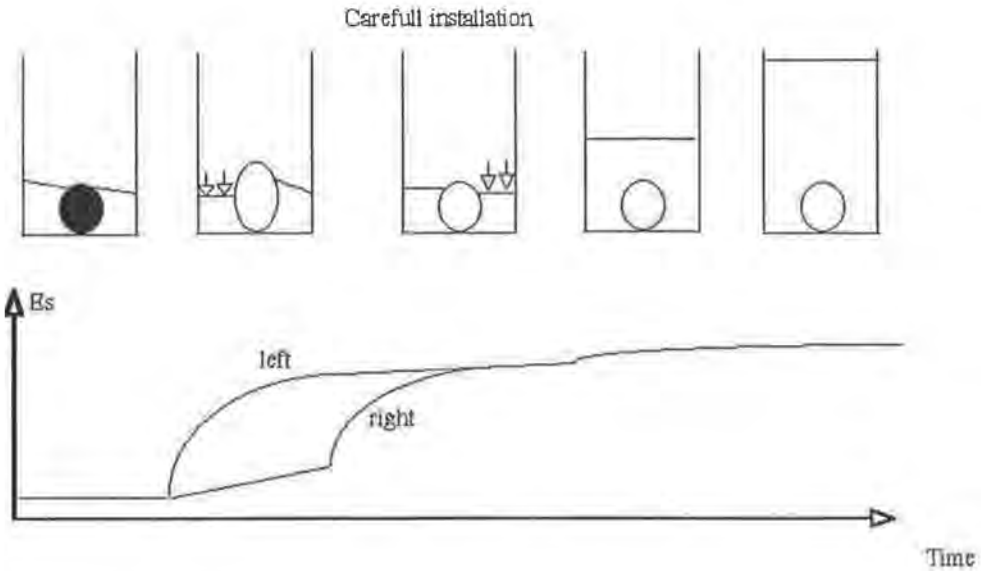
7.4 Discussion

In the former section, the results were shown. Most important observations where:

- The type of installation is the governing factor for the pipe performance.
- Pipe stiffness, depth of cover and traffic load have no significant influence the pipe deflection, other than that it acts as an compaction force.
- Pipe stiffness is only significant when considering low stiffness pipes (2 kPa) in combination with "None" type of installations.

In this section the results will be discussed, with the focus to explain the results found in the trials, supported by the information gained from literature. Before entering a more detailed mathematical analysis and to avoid an early lost link with reality, first the physics will be discussed. This is done by observing the different installation steps, and explaining the effects of each action on the soil and pipe. In figure 7.9 the situation is shown for the installation case "well".

Figure 7.9:The interaction between soil, pipe and man for installation type "Well".



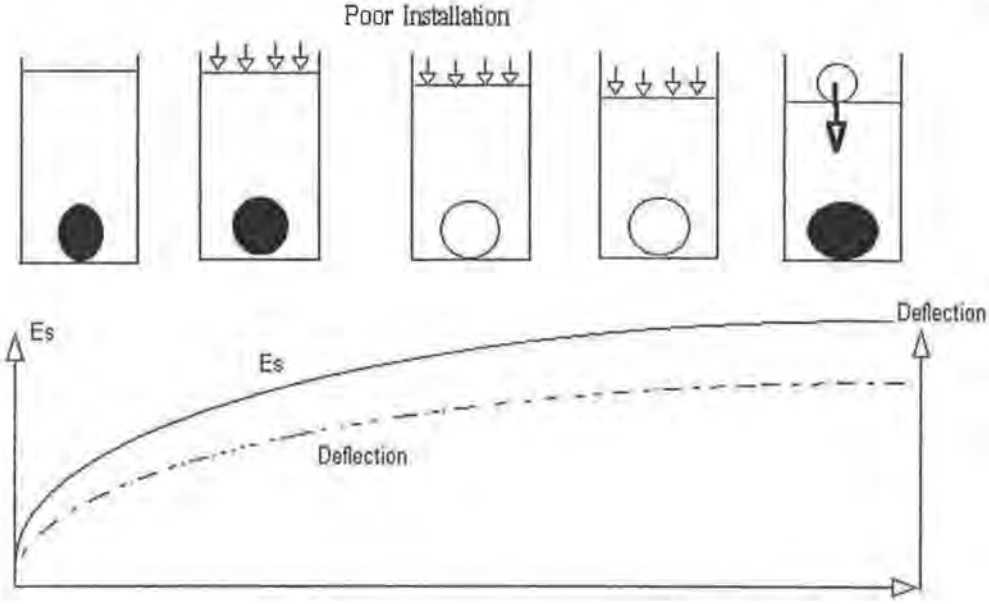
Stage 1 shows the pipe placed at the bedding and soil has been placed loosely around the pipe. This soil can hardly be considered as a consistent material, therefore it needs compaction first. During this compaction the free space between the grains is reduced and the contact stress between the grains is increased. Figure 7.9 shows that the stiffness of the soil is very low. The second picture shows that the left side of the embedment is compacted, and it is shown that the stiffness increases as a result of this action. Also at the right the stiffness increases. The next step in the installation phase is the compaction of the right side of the embedment. The stiffness increases considerable and both left and right side end up with about the same stiffness. The next step in the installation is the application of the next layer of soil, followed by compaction. The stiffness of the soil on both sides of the pipe increases slightly, but not really significant. The installation is completed by further filling and compaction. From this graph it is clear that the compaction over the pipe is not important for the performance of the pipe, but it is obvious that a good compaction of the further trench fill is of importance in order to avoid settlement of the surface. After the last stage it is possible to face some traffic loads.

Design methods consider a load distribution around the pipe, and calculate the response of the ring to this load. An important input parameter is the soil stiffness. The description above has shown that the first part of the installation involves a low soil stiffness.

It also shows that after the third stage the stiffness has increased significantly. Then after completion traffic can load the system, however now the soil stiffness is much larger than in the first phases of the installation.

In the poor installation case, as shown in figure 7.10 the soil is placed loosely around the pipe after which the soil is compacted by means of a heavy truck, or by means of natural processes as by rain and traffic load. The stiffness increases with time together with the reduction of the porosity of the soil. Also the deflection increases at the same time.

Figure 7.10 :The interaction between soil, pipe and man for installation type "None"



Now this condition tends to come closer to what design methods have modelled, however the most significant change in the system is the change of the soil modulus, something that is not anticipated in the models. The models all use one value for the modulus irrespective if the condition of installation is considered or that of surcharge loading like traffic loading. Following the description of the interaction it is clear that the modulus used is to high for reflecting installation effects and to low for the prediction of the effect of traffic load.

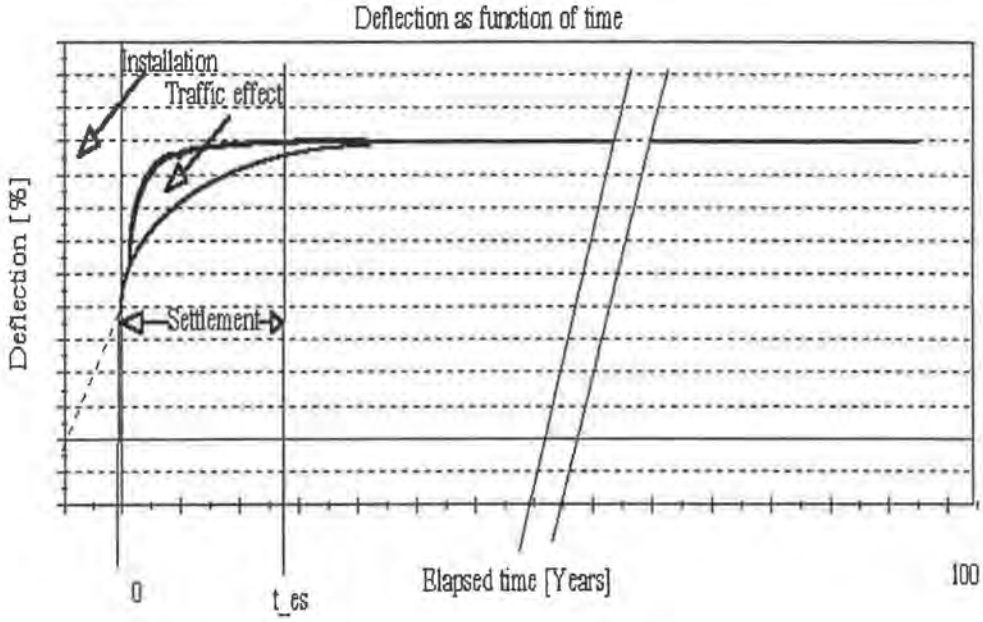
Moreover, the above gives support to consider the pipe soil system as a volume steered process. The flexible pipe follows the soil, and the maximum the soil can do is to compact to it's ultimate value. When the density of the soil has reached it's maximum immediately after installation, as is the case when the installation is done in a careful way, than no significant deflection increase has to be expected.

On the other hand, when the soil is loose after installation, than a more significant increase will be the result, again managed by the potential volume change of the soil.

The validity of the method might become questionable when considering higher stiffness pipes, as pressure pipes, or pipes made out of rigid materials. However also in these cases the amount of load that can be exerted on the pipe is controlled by the volume changes of the soil.

The soil pipe interaction process can be characterised by three stages. Stage 1 is the installation phase, Stage 2 is the settlement stage, and stage three is the stage where the soil is practically fully settled. In this study this stage was defined as the period after which the deflection would not increase more than 1.0 % up to 100 years after installation. This is schematically drawn in figure 7.11.

Figure 7.11 : Deflection as a function of time



An extra line has been added to stage 2. This line represents the settlement process speeded up by the action of traffic load.
The results of the TEPPFA trial were characterised according to this scheme.
The results are summarised in table 7.3.

Table 7.3: Summary of characteristic values

Case	Native / Installation	(δ/D) installation	t_es [Days]	(δ/D) final
1	Sand / Well	-0.5	0	0.0
3		-1.0	0	0
5		0.0	0	0.3
9		-1.5	0	-0.8
11		-1.0	0	0.0
13	Clay / Well	-1.0	388	2.0
15		0.5	5	2.0
7	Sand / Moderate	0.5	12	2.3
2	Sand / None	3.5	67	5.5
4		4.5	35	6.0
6		2.0	0	2.5
8		1.0	422	4.0
10		3.0	419	5.8
12		2.8	26	4.8
14	Clay / None	6.5	897	10.0*
16		3.5	4122	10.0*

** Very conservative estimate, as in clay yielding of the soil will cause significant less increase.*

The difference between "well" and "none" types of installations, become immediately visible in the installation deflections. Another interesting figure shown is the time after which practically no deflection increase is expected anymore. This parameter t_es shows to be zero in those cases where the "Well" type of installation was used. In case the installation is carried out using the "None" type of installation, than this value is higher, although still rather small. So in case of "Well" to "Moderate" types of installations, the pipe / soil system immediately settles after installation, whereas in case of "None" type of installations this takes up to 2 years as a maximum. The difference between the granular type of soil and the cohesive type is more pronounced. This is also according to the expectations, since cohesive soil tends to creep / consolidate slower. In the clay trial the same pipes are used as in the cases 5 and 6 in the sand trial.

A warning is placed at the bottom part of the table, indicating that the expected time, as well as the final pipe deflections, is based on a fit from data up to 500 days. Especially in cohesive soils like the weak clay used in this trial, this period is rather short, especially since clay tends to yield around the pipe. If one has a look at case 16 as shown in Enclosure 6.3, then this situation is already developing. The maximum deflections indicated by the dots in the graphs show that the deflection decreases instead of increases. At the spot of the maximum deflections, also the highest shear stresses will be found causing yielding of the soil, and result in redistribution of the loads.

The difference between the cases 5 and 6 in comparison with the other cases in sand, where PVC and also steel pipes are buried is negligible indicating that the creep ratio of the pipe is not significant in case granular soils are used.

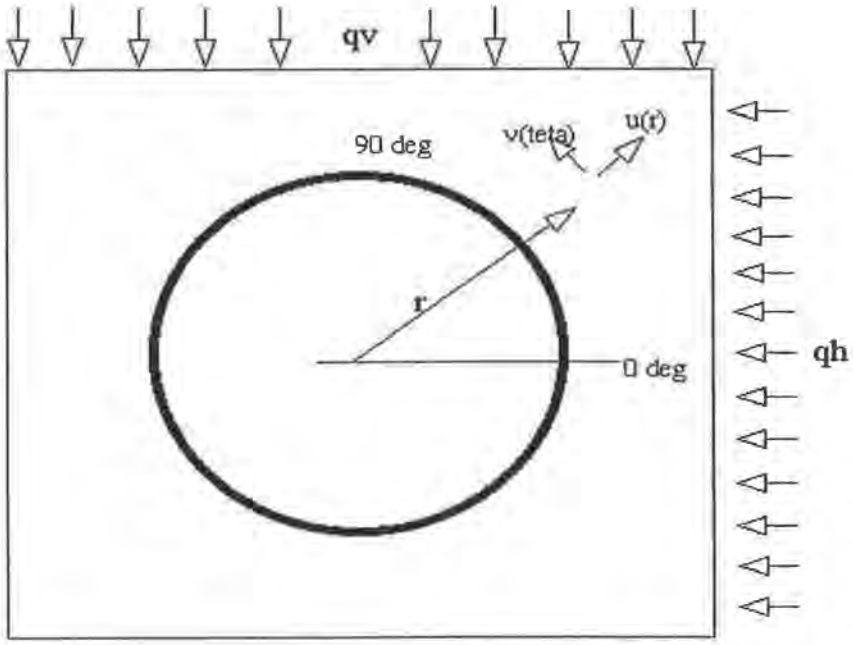
The fact that the time t_es is longer in the clay trial, clearly shows that the settlement of the soil is the governing factor in the time dependant soil pipe interaction process. It could not be checked whether the creep ratio of the pipe material becomes important in case of cohesive soils, because at the clay site only PE is buried.

In order to further discuss the results, use will be made of the model presented first by Höeg (11)

The model used is a ring embedded in a soil envelope, on which envelope a uniform load distribution is placed. The feature of this approach is that it doesn't involve arbitrary discussions about load distributions around the pipeline, but uses the free field stresses as a starting point. These stresses have been proved by Höeg to exist at a distance of about 1-2 the diameter away from the pipe. Furthermore the model was used by Chua et all (12) who made further significant contributions to the design issue of buried pipes in general.

Another feature of the Höeg model is that for certain fixed factors the method delivers the well-known Spangler type of methods. In this study the method is used to see if what has been learned from the field trials can be implemented in design methods, and to check if analytical methods have it already partly incorporated. It also provides a link to the more rigid pipes.
The model is shown in figure 7.12

Figure 7.12 : The basis for the HÖEG model



Höeg developed the equations for the displacements and the stresses in the soil. For r=R, the displacements are the same for the pipes outside boundary. Höeg developed two solutions, one for the case of non-slip and one for the case of slip at the interface soil pipe. In this chapter the situation for the non-slip case will be considered. Below the formulas are given in a general form. For the detailed description one is referred literature 11.

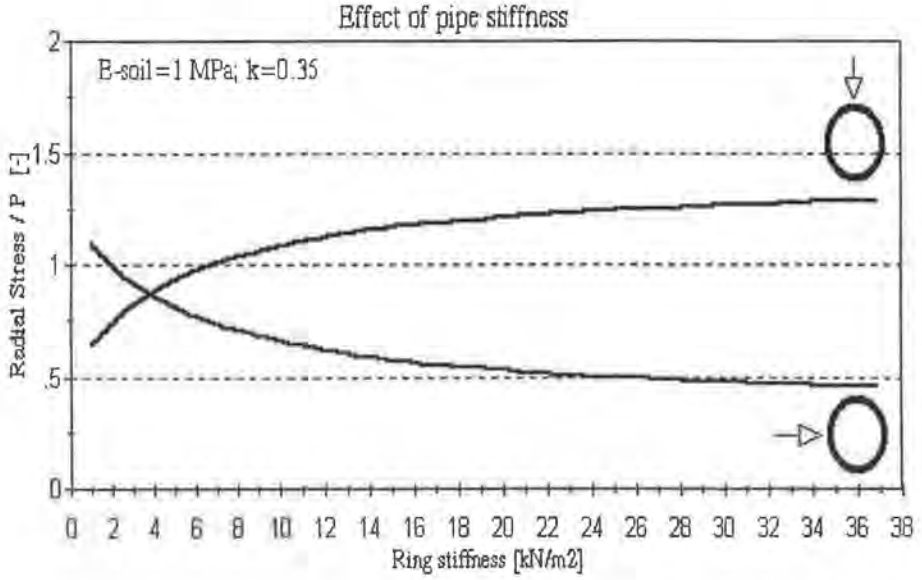
Displacement in radial direction u(r) :

- $u = qv * (f1 + f2 \cos(2*\theta))$
- $\sigma_\theta = qv * (f3 + f4 \cos(2*\theta))$
- $\sigma_r = qv * (f5 + f6 \cos(2*\theta))$
- $\tau_{r\theta} = qv * (f6 + f7 \sin(2*\theta))$

The formulas can be divided in two separate terms, one term independent of the rotation θ, and one term, which depends on this rotation. The f1- f7 are functions of the radius, the k-value and the stiffness properties of the soil and the pipe. The "k" is the k0-value, which is the ratio between vertical and horizontal stress in the soil in case the soil is in rest.

This model is applied to check what the effect of pipe stiffness and soil stiffness is on load concentration and deflection.
In figure 7.13 the effect of pipe stiffness on the radial stress at the soil pipe interface is shown at the springline and at the crown.

Figure 7.13: Effect of pipe stiffness on radial stress on the pipe soil interface

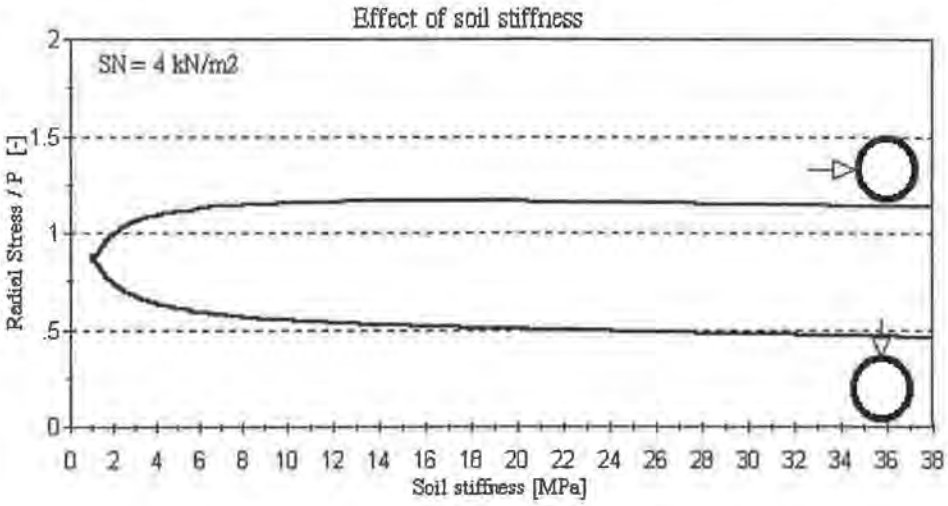


At the vertical axes the radial stress divided by the applied load is plotted, as a dimensionless value. When the load is known, one can multiply the value at the vertical axis with the load to get the actual radial stress value at the pipes outside skin. The graph is plotted for a soil stiffness value of 1 Mpa, and by using a k-value (ratio between horizontal and vertical stress) of 0.35

The graph shows clearly the effect of increasing pipe stiffness. The stress at the springline declines, and the stress at the crown of the pipe increases significantly.

Figure 7.14 shows the effect of the soil stiffness on the radial stresses.

Figure 7.14 : Effect of soil stiffness on radial pipe soil interface stress

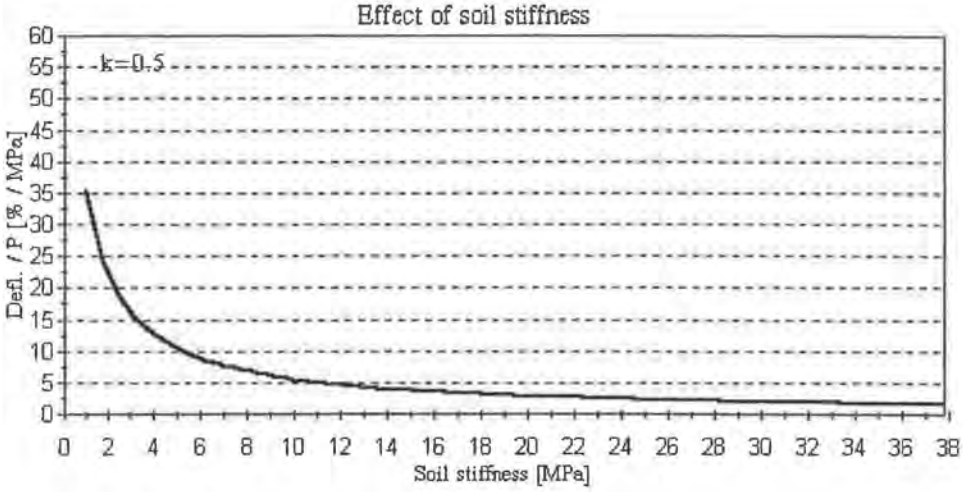


The graph is plotted for a pipe ring stiffness of 4 kN/m2. When the soil stiffness increases than the horizontal contact pressure increases, and the vertical one decreases.

After the above exercises, which show how the load is attracted when the element gets stiffer, another exercise will be shown in which the effect of the soil stiffness on the deflection will be discussed. Also in this case the vertical axes will be normalised for the load.

In figure 7.15 on the vertical axis the % deflection is shown per MPa load, where at the horizontal axis the soil stiffness is plotted.

Figure 7.15: The effect of soil stiffness on pipe deflection



Now let's consider the effect of depth. Suppose the depth increases from 1 to 6 meter. What is the effect on deflection for a "well" type of installation and what for a "None" type of installation?

Table 7.4: Effect of load on pipe deflection acc. to Höeg's model

Value	Well	None
E-soil [MN/m2]	11	1.3
Density [kg/m3]	1811	1530
P (1 m) [kN/m2]	18	15.3
P (6 m) [kN/m2]	108	91.8
Deflection Increase [%]	0.45	2.0

It shall be emphasised that especially in the case of a poor compaction, the soil stiffness will increase during the further deflection of the pipe and compaction of the soil. This non-linear effect has not been considered in the above exercise. Another important effect that is disregarded here is the effect of the in situ soil stress, which has an effect on the soil stiffness. Also the effects of load shedding has not been taken into account in this exercise. Therefore the results shown in the table are extremely conservative.

It shall be noted that the condition for 6 meter of cover results in about the same soil load as a heavy traffic load at a depth of 80-100 cm. As such it also explains why traffic load does not have a significant effect on pipe deflection when installed in a "well" type of installation. This situation is normal when heavy traffic loads are to be considered. In case the installation is done in a less good way, then the deflection increases, and as known from literature (4) the increase is directly linked to the further compaction of the soil, because after several passages of the traffic the deflection increase stops. The ultimate level is obvious related to the level of compaction. After some time (long when there is no traffic, short when there is traffic) the ultimate density has been reached, equal to the well-compacted condition.
It shows that the density level reached is on itself not important to determine the deflection, but the way it is reached is most significant.

Observations:

- ***The type of installation determines the effect of load on the pipe soil system, like depth of cover (soil load) and traffic load.***
- ***The traffic as well as the soil load acts as a compaction effort of the soil beneath. Eventually the maximum density is the controlling factor for the deflection, and not the load !***
- ***When analysing the physics of the pipe-soil interaction, it becomes clear that the performance of flexible pipes is volume controlled and not load controlled.***
- ***Utilising load distributions around a ring, can lead to wrong interpretations like in case the second order theory is used, something that is obvious when one concentrates on the loaded ring. It shall anyway be realised that using second order theories, wil lead to an amplification of the error in the assumptions.***
- ***There is no such thing as one steady state soil property valid over the whole process of installation and of the pipe soil system. When design methods want to have a good chance to estimate the correct deflection, bending momenets and strains in any pipe, than they have to consider this aspect.***

8 DESIGN APPROACHES

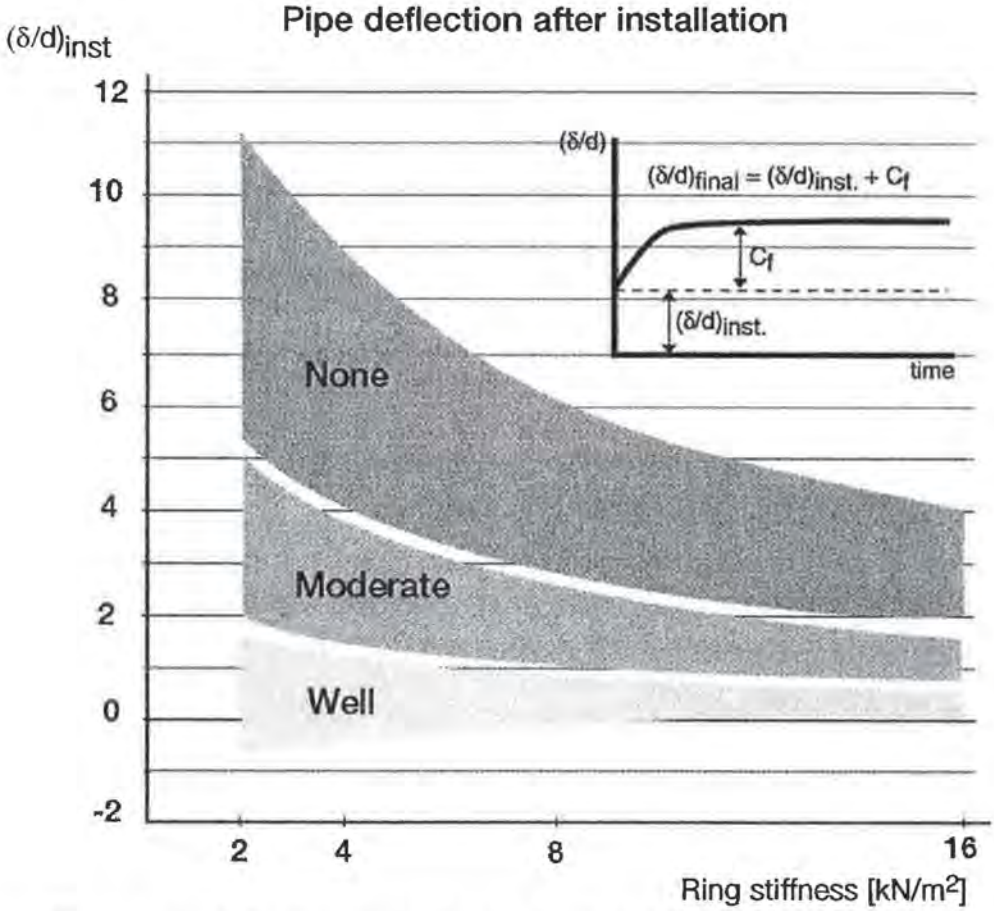
Based on the results discussed in this report several design approaches can be proposed. It has become clear that for thermoplastics pipes utilising a huge strainability, design can be kept rather simple. It was also shown that if one is concerned about the deflection level than it is wise to put some more effort in the installation of the pipe. Another important observation is that flexible pipes follow the soil settlement, and the behaviour is managed by this. Load is therefore not an issue for flexible pipes. Therefore based on the results of this research work, and after cross check with other experimental work done, the design approach using simple graphs is strongly recommended.

8.1 Design graph

The design graph contains three installation groups. Also the add on factors for the deflection are given. The add on factors or consolidation factors and have to be added to the value for the initial deflection which can be obtained from the graph. The consolidation factors are fully linked to the type of installation.

In the design graphs-areas are given for each installation group. So for each group a range of deflections are given. The lower boundary of each group represents the average deflection expected and the upper boundary the maximum, or more precise, the 95 % probability value. At the backside of the graph the conditions, for which the graph is valid, are listed.

Figure 8.1 : Design Graph



Note: ***The average deflections immediately after installation are represented by the lower boundary of each area, and the maximum values by the upper boundaries.***

Installation types and related consolidation deformation

"Well" compaction, C_f = 1.0

The embedment soil of a granular type is placed carefully in the haunching zone and compacted, followed by placing the soil in shifts of maximum 30 cm, after which each layer is compacted carefully. The pipe shall at least be covered by a layer of 15 cm. The trench is further filled with soil of any type and compacted. Typical values of standard proctor are above 94%.

"Moderate" compaction, C_f = 2.0

The embedment soil of a granular type is placed in shifts of maximum 50 cm, after which each layer is compacted carefully. The pipe shall at least be covered by a layer of 15 cm. The trench is further filled with soil of any type and compacted. Typical values for the proctor density are in the range of 87-94%.

"None" compaction in granular soil, C_f = 3.0

The embedment soil of a granular type is added without compaction.

"None" compaction in clay, C_f = 4.0

The embedment soil of a cohesive type is added without compaction.

The design graph is valid under the following conditions:

- Depths between 0.80 meter up to and including 6 meter.
- Designers first need to establish allowable deflections, average and maximum.
  (National requirements, product standards etc.)
- Pipes fulfil the requirements as listed in prEN 13476, prEN 12666, EN 1852 and EN 1401.
- Installation categories "well", "moderate" and "none" should reflect the level of workmanship on which the designer can rely upon.
- Sheet piles shall be removed before compaction, in accordance with the recommendations in EN 1610.
  If however the sheet piles are removed after compaction, one shall realize that the well or moderate compaction level will be reduced to the "None" compaction level.
- Pipes with diameters up to 1100 mm.
- Depth / diameter ratio at least above 2.0.
- Deflections are unlikely to be exceeded in practice for the circumstances described.
- For the deflection mentioned in the graph, the strain will be far below the design limit, and does not need to be given attention to in the design.

Other design considerations

It has been indicated above that strain is not an issue when the above conditions are fulfilled, nevertheless when one would like to determine the strain values as well, the following relationship can be used:

$$\varepsilon = df * (\delta/D) * s/D \qquad (8.1)$$

in which :

ε	=	Bending strain [%]
df	=	Strain factor, conservatively chosen to be 6.
(δ/D)	=	Pipe deflection [%]
s	=	Wallthickness [mm]
D	=	Pipe diameter [mm]

In case of structured wall pipes "s" shall be replaced by 2*a.

In which "a" is the distance between outer fibre and neutral axis.

All the results are valid for structured wall pipes as long as they fulfil the requirements in prEN 13476. Essential is that the pipes do not loose their structure when deflected. In prEN 13476 the so-called ring flexibility tests is testing upon this aspect.
The strain value calculated by formula 8.1 is the so-called outer fibre bending strain, valid for linear elastic materials. In case of visco-elastic materials the outer fibre strain is lower in reality due to redistribution of bending stresses.

No buckling or collapse of pipes has taken place during and after the installation in the field trials, even though the pipes have been installed at a depth of 3 meter with a high groundwater table and by using soft soils.
Nevertheless, buckling is a design issue, and for the check upon buckling the set of formulas as proposed in (1) is recommended. In summary :

- Weak soils, like soft silt or clay and SN at least 2 kN/m2:

  $$qperm = (24*SN + 2/3*Et) / F \qquad (8.2)$$

- Firm soils, like gravel, sand and silty sand and SN at least 2 kN/m2:

  $$qperm = 5.63 / F * \sqrt{(SN * Et)} \qquad (8.3)$$

In the above formulas the :

F	= Safety factor against buckling	[-]
SN	= Pipe ring stiffness	[kPa]
qperm	= Permissible Load	[kPa]
Et	= Tangent modulus of the soil	[kPa]

For the material and soil parameters, the short-term values shall be used. Only when pipes are buried at greater depths (> 6 meter) in soft soils and continuously exposed to high groundwater tables, the long-term values are recommended.

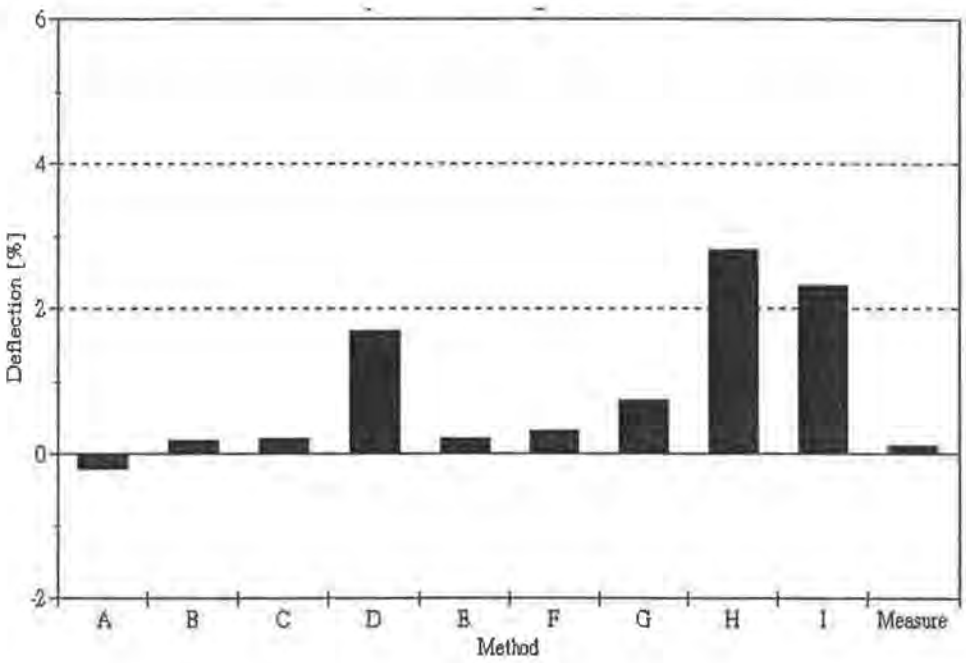
9. COMPARISON WITH DESIGN METHODS

The installation cases were also considered in a normal design exercise. Several European experts were contracted to carry out the design based on the type of information normally available in most cases. To this information belongs the descriptions of the site, the soil type used, a grain size distribution, dimensions of the trench, the nominal pipe sizes and properties and the anticipated construction procedure.

The experts first made the design calculations, after which they received the results of both soil and pipe measurements, including the deflection values measured during and after installation. The results were discussed during a workshop that was held in December 1997, together with the experts. In this section the general trend of the outcome will be given. The methods are coded with A, B etc.

The first result shown in figure 9.1 is the prediction of the deflection for a pipe which was installed in using a "well" type.

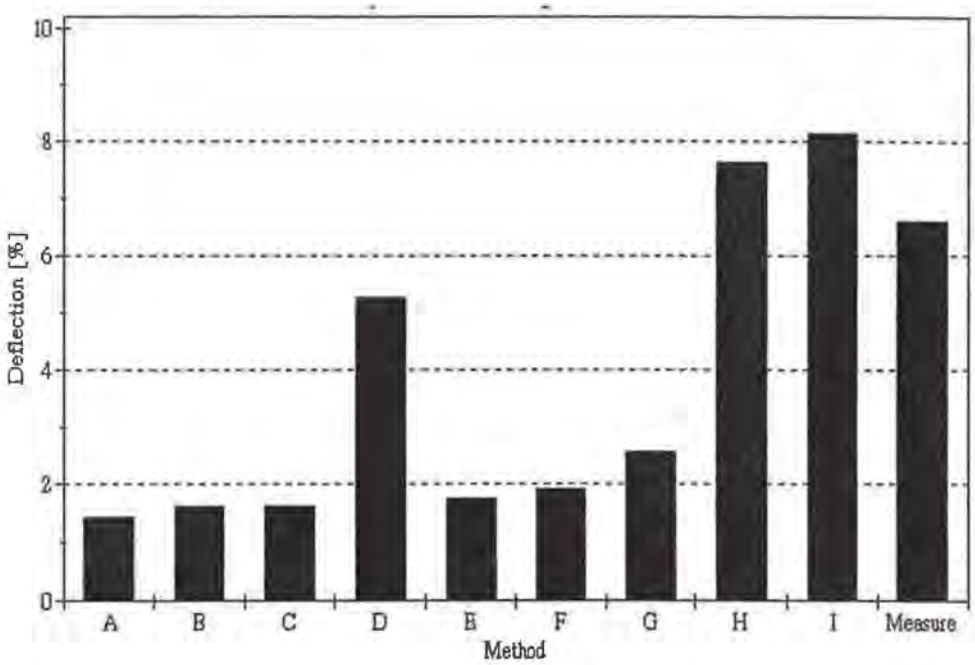
Figure 9.1 : Comparison with design methods for a "well" type of installation.



It is shown that several methods predict the measured deflection quite well, as where others show deflection levels, which are 2 % higher than the measured value. The values however are all so low, that in such cases design is not an issue at all.

The same type of result is shown in figure 9.2 for the situation that a pipe is installed in a "None" type of installation.

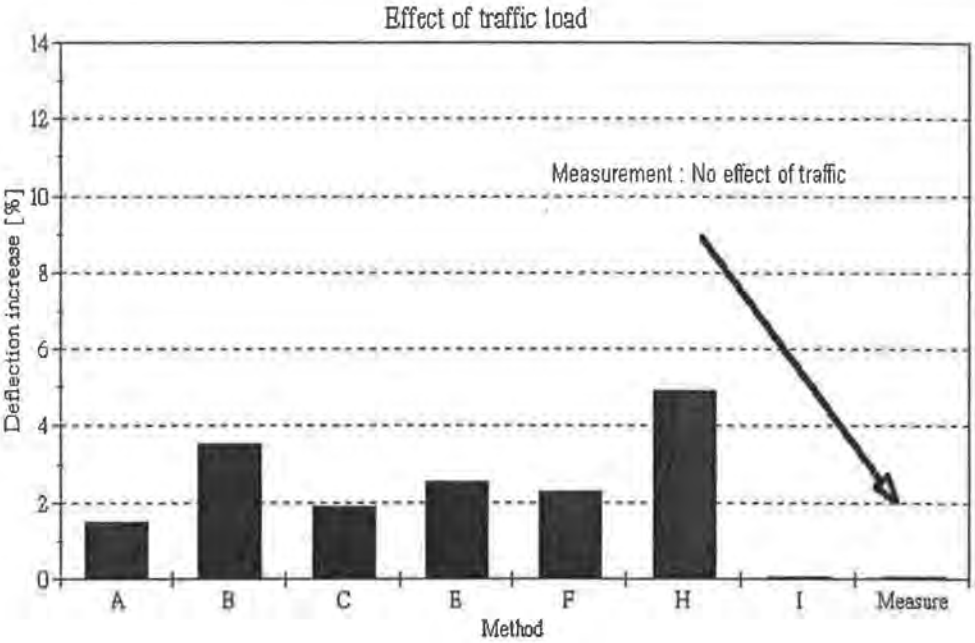
Figure 9.2 : Comparison with design methods for a "None" type of installation



The result shows that now several methods underestimate the deflection considerable, where as some only slightly overestimate the deflection. This was further discussed in a workshop with the European experts and most of the experts explained that when more detailed soil information is available, they also could predict the deflection rather accurate. When they utilised the after-installation information about the soil and the construction procedure, they indeed showed to be able to predict the measured values rather well. Here a paradox became clear that the deflection prediction can be done using detailed / sophisticated methods when the above conditions are fulfilled. However in such cases installations become so well done that design is not an issue at all.

In figure 9.3 the result is shown for the case traffic is present. Here it shall be mentioned that the traffic load used in the field could not be simulated by some of the methods, as they utilise fixed traffic classes. What they did was to use the traffic class which came closest to the exerted load in the test.

Figure 9.3 : Effect of traffic load.



Method I, as well as the measurements show that the deflection is not affected by the traffic load, in case of using "None" type of installation in combination with a pipe stiffness of 2 kN/m2. The other methods indicate that there is a significant effect of traffic load. Bear in mind, that the combination of heavy traffic and "None" type of installations is a rather artificial one. See also for the explanation the section 7.

Observations :

- ***In "Wel" type of installations the methods all predict the deflection quite accurate.***
- ***In "None" type of installations most methods showed to have difficulties to define the correct input values, resulting sometimes in underestimation as well as in overestimation of the actual deflection***
- ***Almost all methods overestimate the effect of traffic considerably. The traffic load tests showed that the deflection is not affected by traffic.***
- ***The exercise illustrated the sensitivity of those methods. One needs to be rather confident about the soil data and the workmanship, in order to produce an accurate prediction of the deflection.***

10. RECCOMENDED PRACTICE

Based on the results and the experience gained in this study the following is recommended:

- Thermoplastics pipes can be installed in cohesive as well as in granular soil types, using "well" to "None" type of installation conditions, without obtaining structural problems. The key property for this is the huge strainability of the materials used.
- However, in order to obtain reasonable deflections in the pipes it is recommended to use a "well" to "moderate" type of installation.
- It is recommended to promote the use of simple control tools like the impact cone. Contractors can use them themselves.
- It has become crystal clear that deflection and strain, as well as bending moments, are ruled by the type of installation. When deflection shall be limited then it is advised to utilise good installation conditions.
- If nevertheless an estimate of pipe deflection has to be made, than it is recommended to use the design graphs as presented in section 8.1. They are easy to use, based on actual field experience and incorporate the feature that they immediately sign what the effect on deflection is, when the anticipated installation conditions can not be reached in real life.

11. CONCLUSIONS

- No pipe failures or instability has been encountered during he trials, although some of the pipes have been installed in an extremely poor way.
- The results have shown the following:
  - Depth of cover is not important for the pipe deflection.
  - Traffic load has no significant effect on pipe deflection.
  - Pipe stiffness is not important when considering pipes installed using a "Well" to "Moderate" type of installation.
  - The creep ratio of the pipe material is not important for the determination of the deflection.
  - Pipe/soil performance is a volume steered process.
  - The analysis of the installation has indicated that the current approached design does not adequately reflect the real physics of pipe soil interaction. It is for this reason valid to state that it is useless to create a high level of mathematical sophistication in such design methods.
  - The importance of the balance between the determination of the soil properties, the design and the construction is clearly shown to be of utmost importance.
  - Design is a whole process involving all decisions regarding what is to be constructed, and how it is to be constructed. Mathematical formulae can only help to support some of the decisions.
  - The results of the tests showed to be consistent with many earlier studies.
  - It was shown that carrying out full-scale field trials allowed developing a normal construction work and variation.
  - It was shown that a good pipe deflection prediction could be obtained by applying a simple design graph. If however, still the need is felt to calculate with the traditional formulas, then it is advised to split the design in an installation phase and a life phase, in which different soil moduli shall be used.
  - The big problem encountered in all the design methods is that they are fully load driven. Using loads on systems is only sensible when the stiffness properties are accurately known, not to mention that the correct type of stiffness needs to be known. In case of soil pipe interaction the load is put on a system comprised by the pipe and the soil. The stiffness properties of the pipe are rather well known, but those of the touched soil only rather gross. The initial load might be determined quite well, but the redistribution of the loads over the system components is a result of the stiffness properties of both components. The assumptions done regarding the soil properties are multiplied over and over again before arriving at the deflection, bending moment etc. An old phrase (at about 1350) of Mr. Ocwin Razur, which phrase seems to fit quite well to the issue of designing pipes sophisticated using analytical methods: "***Hypothesis should never be multiplied without necessity***".

12. ACKNOWLEDGEMENT

The project was steered by experts from the industry. A small project group consisting of the project manager and two external experts in the field of design of buried pipes supervised the project.

Next to these consultants, European design experts, where contracted, to provide the design using the different established methods, as well as to assist in the results evaluation.

The persons working in the different groups are listed below:

Steering Committee

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Mr. M. Giay	Rehau	
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Project group

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L.E Janson	SWECO	(Supervisor)
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European design experts

M. Gerbault	Consultant
G. Leonhardt	(D+L partners)
W. Netzer	University of Innsbruck
L.E Janson	SWECO
J. Olliff	Montgomery Watson
H. Schneider	Comtec

Enclosures

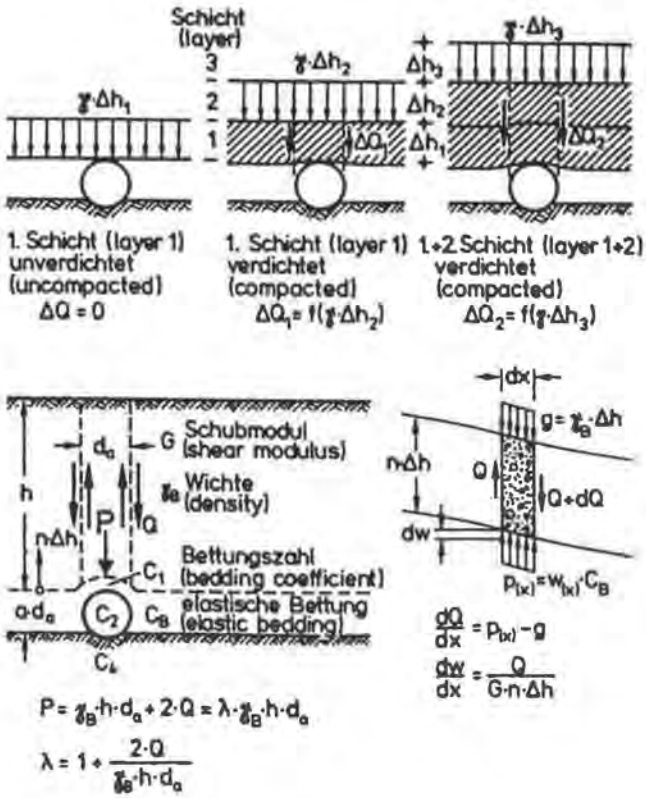
Enclosure 4.1:

Short description of the design methods used

1. DESIGN METHOD
by G. Leonhardt

The analysis of earth loads according to the ATV-method is taking into account the deformation behaviour of pipe and soil and their influence on each other. First the soil pressure on a level of the pipe crown is calculated. Subsequently the redistribution of the soil stresses resulting from the different deformation behaviour of pipe and soil is determined. With this method it is possible to take into account:

- different trench widths with a continuous transition from trenches to embankments;
- different pipe stiffness with a continuous transition from rigid to flexible pipes;
- different moduli of deformation in the surrounding soil beside and above the pipe.



2. Description of the *METHOD OF DESIGN* as drafted by *CEN/TC 155 / WG 14* "Plastics piping systems - glass-reinforced thermosetting plastics (GRP) pipes - structural analysis of buried GRP pipelines" (intended to be published as an env)

The design approach developed by CEN/TC 155/WG 14 is mainly based on the regulation given in ATV A 127 "Guideline for the structural design of sewerage and drainage pipelines". This guideline is briefly described in EN 1295-1, annex B.2.6 and, more detailed, in prEN 1295-2. A general description is expected in the workshop to be given by G. Leonhardt (for ATV) and W. Netzer (for ÖNorm, which is similar to ATV).

Therefore in the following only the main differences relative to ATV A 127 are being high-lighted.

Road Traffic Load

Four levels are identified covering the maximum loads given in Europe as of to date; it is intended as soon as possible to replace the existing systems with those provided by EUROCODE 7. The impact factor is depth dependent; a distribution factor relative to the type of pavement is introduced; the response of the pipe to traffic load is based on initial pipe properties because traffic loading never is a sustained load.

Construction Phase Traffic Load

Very heavy vehicles acting during the construction phase of the pipeline can be accounted for if necessary.

Soil Parameters

The approach identifies six instead of four soil types to cover the needs within Europe; the classification system is based on prEN 1046. The properties are similar to those used in the draft prepared by CEN/TC 164-165 / JWG / TG 1. The depth dependency of the soil modulus is taken into account.

Reaction Angles

Two vertical bedding angles (120°, 180°) are given which are to be used depending on two different support cases. Depending on the trench/pipe proportions and on the ratio between backfill material and native soil moduli four horizontal reaction angles are given (100°, 110°, 130°, 140°).

Internal Pressure

The pipe response on internal short-term and/or long-term working pressure can be accounted for. The analysis of the combined loading (tension due to internal pressure and bending due to external soil, traffic etc. loads) relative to factors of safety accounts for the special material behaviour of GRP.

Initial Ovalization

Flexible pipes have an induced positive vertical deflection, i.e. increase in vertical diameter, caused by the active soil pressure stemming from compaction efforts. This is taken into account depending on soil group, stiffness and compaction class.

Axial Effects/Response

Guidance is given on how to deal with e.g. prevented Poisson effects, biaxial loadings and other effects.

Buckling

Two types of buckling are identified, namely "creep buckling" and "elastic buckling" and the pipe response to both types is analysed accordingly. In addition a special type of buckling is accounted for, namely "local buckling of ribbed pipes".

QUESTIONS

Level

According to the intentions of the working group the current approach covers the highest level in accordance with EUROCODE 7. It is intended to, based on the existing document, develop possibly two, but at least one, more level(s), namely one approach where quite a few variables are fixed to cover a range of approximately 80% of projects. A further step has been discussed, also based on the current approach, to be "reduced" to tables and/or diagrams. In both latter cases possibly the minimum required factors of safety have to be increased.

Input parameters

Basically as for ATV and/or ÖNorm.

Installation standard

The document solely refers to prEN 1046 "Plastics piping and dusting systems - Systems outside building structures for the conveyance of water or sewage - Practices for installation above and below ground".

Calculated values

Based on the choice of the input parameters as recommended in the document the calculated values aim to be average results.

Field survey

According to the document it is allowed to use measured data for soil and pipe properties. If none are available recommendations on soils properties are given and pipe properties have to be taken from product standards.

Soil properties

Basically as for ATV and/or ÖNorm.

3. *SWEDISH CALCULATION METHOD* (Plastics Pipes)

Extract of the Swedish Publication VAV P70 "Buried gravity sewer plastics pipes", Stockholm 1992 (In Swedish).

3.1. Short-term Deflection

According to the Swedish method, the maximum vertical deflection is determined in the following way. First, the theoretical deflection is calculated. To this value are added deformation effects caused by the installation method used and to the effect of uneven pipe bed conditions:

$$(\delta/D)_M = (\delta/D)_q + I_f + B_f \qquad (1)$$

where:

$(\delta/D)_M$ = maximum deflection
$(\delta/D)_q$ = theoretically calculated deflection caused by soil and traffic load
I_f = installation factor
B_f = bedding factor

Hence eq. (1) gives the short-term maximum deflection. According to experience the short-term average deflection is in most cases estimated by just excluding the bedding factor B_f.

The theoretical deflection caused by loads can be calculated as follows:

$$(\delta/D)_q = q \cdot \frac{C \cdot b_1 - 0{,}083\, K_o}{8\, S_R + 0.061\, E'_s} \qquad (2)$$

where:

C = load factor
b_1 = load distribution factor, which is 0.083 at a bedding angle $\alpha = 180\,°$ and 0.096 at a bedding angle $\alpha = 90\,°$.
K_o = soil pressure coefficient at rest (= 0.5)
S_R = EI/D^3 = ring stiffness of the pipe (kN/m^2)
D = diameter of the neutral line of the pipe wall (m)
E'_s = secant modulus of the side fill (kN/m^2)

For flexible pipes with firm soil as surrounding fill the Swedish calculation method involves a simplification of eq. (2) by using $C = 1$, $b_1 = 0.083$ and $K_o = 0.5$ giving:

$$(\delta/D)_q = \frac{0.083\, q}{16\, S_R + 0.122\, E'_s} \qquad (3)$$

The values of the secant modulus E'_s for granular material have been determined by laboratory tests in a hollow cylinder apparatus. The minimum values found are given in **Fig. 1**.

For compacted firm clay as surrounding fill E'_s may vary between 200 kN/m^2 and 2,000 kN/m^2. In other cases with clay as fill E'_s is recommended to be put at zero.

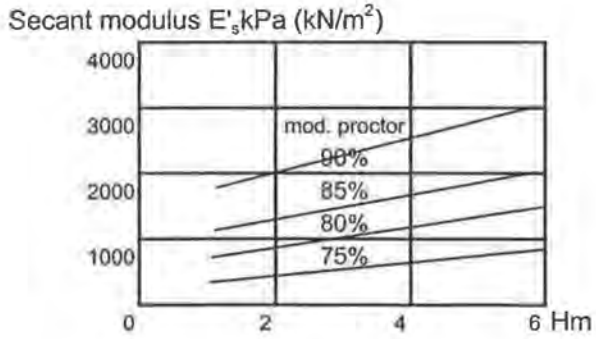


Fig. 1 *Secant modulus for granular soil versus depth of cover. Minimum values.)*

Based on a number of deflection measurements on plastics sewer pipes Sweden, the values of I_f and B_f given in **Table 1** are suggested for guidance provided sand or gravel for the surrounding fill.

Table 1. *Values for guidance of installation factor I_f and bedding factor B_f*

	Installation factor I_f %
Pipe in a stepped trench	
- No supervision	1-2
- With supervision	0
Heavy construction traffic and H < 1,5 m	1-2
Compaction of fill above the pipe with heavy equipment, > 0,6 kN	0-1

	Bedding factor B_f %	
	Careful execution	Normal execution
No supervision		
- No stones [1])	2	4
- Soil with stones or rock	3	5
With supervision		
- No stones [1])	1	2
- With stones	2	3

[1]) *Refers to trench bottom conditions.*

Negative values of I_f can be obtained if the side fill is heavily compact.

3.2 Long term Deflection

The calculated pipe deflection according to eq. (2) or eq. (3) gives minimum short-term value immediately after completed installation and backfilling.

4. TEPPFA PROJECT
Description of the *FASCICULE 70 METHOD*
Marcel Gerbault / 29/09/1997

The Fascicule 70 is the technical regulation applicable to the design and the installation of pipelines for sewers. For water distribution purposes, it is the Fascicule 71.

The method of calculation of Fasc. 70 has two main steps: the 1st one is the evaluation of loads on pipe; the 2nd is to determine the pipe 'response' to the loads.

For loads distribution, the model of MARSTON is used for rigid pipe-soil systems, when determining the vertical load pressures, and the FROELICH method (an alternate to Boussinesq theory) is used for pressures due to surface loads.

For the determination of the pipe response (deflection, strains, stresses, etc...) a soil-structure interactive model is used, based on Winkler assumptions: distributed springs all around the pipe. In order to get a model valid in a continuous way from rigid to flexible behaviour, the equilibrium equations are written for the displaced configuration (2nd order theory). The results are not linear versus the loads; an initial out-of-roundness may have great influence, depending on the flexibility of the pipe. This type of model was used by Timoshenko and Luscher to get the critical buckling pressure. It was used also by other authors, like Mr. J. GUMBEL or Prof. FALTER.

The parameters having influence on the system behaviour are, of course, the pipe characteristics (geometry, deformability and strength or serviceability limits) and the soil parameters.

In order to make easy the choice of soil parameters for design purposes, soils were divided into 6 groups (2 of them are not usable for pipe embedment), more or less comparable to ATV' groups.

3 classes of compaction are defined. 'Non compacted' means that there is no control for compaction. 'Compacted and controlled' means that the process of compaction for the soil used has been defined and studied. 'Compacted, controlled and verified (or inspected)' means that, in addition to the last one an inspection in situ is made, after pipe installation.

The choice of the soil groups and the compaction classe leads to recommended figures for the main soil parameters. Esoil modulus (it is the soil Young elasticity modulus, and more precisely the modulus related to the 'deviatoric' component), k_1 and k_2 factors for horizontal soil pressures above and on the sides of the pipe, and bedding angle 2α.

These figures may be corrected depending on several conditions like:
- the presence of ground water, which may alter soils characteristics when they are sensitive to water;
- the influence of installation conditions which may disturb the soil compacity, like type of withdrawal of trench wall protection, depending on the trench width and the wall protection thickness.

Correcting factors are given in the Fascicule 70, trying to simulate the pipe installation method. As the Fascicule 70 is also the installation standard, the connection between design and installation is easily made. In case of no structural design, the best type of installation (i.e. the most careful for the pipe itself) is assumed by the pipe provider.

When a structural design is made, the designer makes the choice of the pipe and gives the installation specifications to the contractor. Of course, in such case, a minimum of information on soil characteristics are needed, particularly a geotechnical survey, not only for the pipe structural design, but also for avoiding problems of longitudinal settlements.

The approach used is a semi-probabilistic one, using 'characteristic figures' of forces and strength, based on the fractile of 5%. These figures are used for the calculation of the Serviceability Limit States, like, for instance, cracks or strains limitations for corrosion purpose, ovalization limitation for hydraulic purposes, etc... For resistance verifications, the characteristic figures are changed into design figures, related to the fractile of 5‰, by use of γ factors, splitted into a load factor γ_A and a material factor γ_M. A specific factor for other inaccuracies, like assumptions for the model of calculation, is included in γ_A. The value of γ_A is 1,25. Pipe manufacturers are asked to give their guaranteed material factor γ_M. This splitting is necessary due to the fact that the pipe response is not linear.

This semi-probabilistic approach corresponds to the TC 250 philosophy and the figures to a probability of structural failure of 1.10^{-6} during a 'design life' of 50 years.
So, the figures recommended for soil parameters are intended to be related to the 'characteristic' ones (fractile of 5%).

For the 'non compacted' classe, no special compaction studies are done or written specifications given; but that does not mean absence of compaction. For the 'compacted and controlled' classe, the adequation of the material used for compaction, the number of passes, the water content and the thickness of the soil to be compacted are studied, in the light of previous similar installations, and may be controlled by measurement made on the site, for instance on the surface, near the trench, like for road construction. For the 'compacted, controlled and verified' classe, in situ tests are performed on the back fill, upon the pipe, and, on some circumstances, in the embedment on the side of the pipe. These in situ tests are mainly dynamic penetrometers tests, calibrated by laboratory tests; it might be density measurements (gamma-densitometry), or also Ménard's pressiometer tests, but only one point of measurement, per meter of depth, may be made. In case of frictional soils (unstability of bore holes in sand for instance) a steel sleeve is used, with a special 'window' allowing the contact of the pressurometer cell and the soil. The frequency of such inspections (number of in situ controls) is not standardized for the time being.

The method used in Fascicule 70 is on the level 2 basis, according to Eurocode 7. This means that simple assumptions made for the model, in one hand, and on the mathematical treatment, on the other hand, leads to formulae usable with 'hand' calculation.

The same model, but with more refined assumptions on the loads, on the material constitutive law, which need computer programs, has already used. This is the case of the program ' TUBE'. It corresponds to the level 3 of Eurocode 7.

Several publications have been made on this Subject. Refer also DOC N 7; 41; 104; 106 of TC 164/165/JWG 1/TG 1.

5. *BOSSEN METHOD* for Calculation of Pine Deflection

Loads on the pipe

Bossen considers:

+ Primary loadings

+ Secundary loadings

+ Reaction Forces from Elastic Bedding

+ Composed Buckling

Calculates the Bending moments and Normal Forces and arrives at a simple formulae for Flexibility Calculation:

$$\frac{0{,}15 + 0{,}081.n}{0{,}2\,C - 0{,}7 - 0{,}3.n + 24.\,S/Pv}$$

Where:

F = Deflection

n = Compaction factor (1 = good to 4 = bad)

C = Buisman factor

S = Pipe Stiffness

With internal pressure:

24. S/Pv becomes : I/Pv.(24.S + Pi)

Traffic Load and Groundwater level:

Equivalent Height

Together: Heq = H - 10^3. Hw/p + ϕ P / π g ρ H^2

Buisman Factor estimation:

Peat/Moor/Water rich soil	C = 3 - 10
Clay	C = 10 - 20
Sandy clay	C = 20 - 50
Sand	C>50

Bossen method is fundamentally well described and evaluated

Advantages:
Limited input parameters
Simple formulae
Usable for all types of soil

Disadvantage:
Estimation of factors should be perfectioned

Experience:

Fits quite well with practise in most of the cases.

Case	Condition	Deflection Min – Max	Deflection Average	C factor	n-value	With Pi Avg defl.
1	A	1,2 – 1,5%	1,30%	60-80	1	
2	C	3,8 – 5,2%	4,50%	30-50	4	
3	A	1,3 – 1,7%	1,50%	60-80	1	
4	C	4,6 – 7,1%	5,60%	30-50	4	
5	A	1,1 – 1,3%	1,20%	60-80	1	0,70%
6	C	3,2 – 4,1%	3,70%	30-50	4	1,70%
7	B	2,3 – 2,7%	2,50%	50-60	2	1,40%
8	C	4,3 – 6,4%	5,20%	30-50	4	2,50%
9	A	1,4 – 1,8%	1,60%	60-80	1	
10	C	4,9 – 8,2%	6,20%	30-50	4	
11						
12						
13	A	1,1 – 1,3%	1,20%	60-80	1	0,70%
14	C	4,4 – 4,5%	4,50%	10-20	4	1,50%
15	A	1,3 – 1,6%	1,40%	60-80	1	1,00%
16	C	7,8 – 9,3%	8,50%	10-20	4	3,00%

Remarks:

1. Calculated values are all initial deflections, short term
2. The spread in calculated deformations are due to the choice of an estimated range of soil properties. The compaction factor 'n' is chosen at one fixed level.

6. TEPPFA - Research 1997
PRINCIPLES OF ÖNORM B 5012

The Austrian pipe design standard ÖNORM B 5012 consists on 2 parts.
Part 1 deals with the input parameter for the static calculation. It contains basic informations regarding the definition of laying and installation conditions, soil mechanic parameters and traffic loads.
Part 2 deals with the mechanical model and the rules for calculation including safety factors based on the probabilistic theory of reliability.

1. Level according to Eurocode 7:
   The Austrian standard NORM B 5012 fulfills all requirements for a level 3 calculation according to Eurocode 7. It provides the calculation of a buried pipe as an interactive system on the basis of all available knowledge about the behaviour of pipe and soil.

   The most important principles are:
   1. Only measured or measurable and controllable soil parameter are used. The soil stiffness modulus can be measured by the Oedometer test or load plate test. The influence of compaction (DPr) and stress is taken into account.
   2. The side pressure coefficient K is used in accordance with generally used soil mechanical rules.
   3. Pipe soil interaction is considered with the following assumptions:
      - in vertical direction with the shear stiff beam (vertical load)
      - in horizontal direction with the Kany method for an infinite load strip (horizontal bedding reaction pressure)
   4. Theory 2nd order effects for flexible pipes are considered. Thus effects like buckling or superposition of internal and external loading can be dealed with.
   5. Many decisive influence factors can be taken into account like influence of installation procedure (compaction in backfilling and embedding zone, initial ovalisation, existence of a gap in the embedding zone), influence of different soil behaviour in backfilling and embedding zone and in the natural soil beside and below the trench, influence of groundwater on loads and soil properties, short and longterm behaviour of pipe and soil material.
   6. The safety factors used are calculated on the basis of the probabilistic theory of reliability for two safety classes.
      Calculations with the factors for safety class 1 provide a failure probability of 10.e-3, calculations for safety class 2 (normal case) a failure probability of 10.e-5, provided that the rules and input data of ÖNORM B 5012 are used.

2. Most significant input parameter:
   The most significant input parameter are pipe properties like diameter, wall thickness, E modulus, permissible or ultimate stress, strain or deflection, self weight soil properties like stiffness modulus, self weight, angle of internal friction for different zones installation characteristics like depth of cover, trench or embankment condition, trench geometrie, embedment and laying type, vertical and horizontal bedding angle, loading like traffic load, other surface loads, internal and external water pressure, safety class or safety factor.

3. Reference to installation standards:
   In absence of any detailed installation standard four installation cases are defined in part 1 of ÖNORM B 5012.

4. Probability of design results (definition of calculated values):
Provided that the rules, input data and safety factors of ÖNORM B 5012 are used, the allowable results have a failure probability of 10.e-3 for safety class 1 and 10.e-5 for safety class 2. If other input values (e.g. mean values) are used, the results are orientated at the probability of the input values.

5. Field survey:
Field survey is requested in ÖNORM B 5012. The soil properties to be checked, the methods for the checks, place and number of tests are standardized in ÖNORM B 5016.
The following Austrian standards are recommended for the most important tests

ÖNORM B 4400:	soil classification
ÖNORM B 4410:	water content
ÖNORM B 4412:	grain distribution, sievetest
ÖNORM B 4417:	load plate test
ÖNORM B 4418:	Proctor density
ÖNORM B 4419-1:	cone penetration test

A special "Hand Penetrometer" is described in ÖNORM B 5016.

Innsbruck, 21.11.1997 W. Netzer

7. TEPPFA/EPMA
RESEARCH PROJECT
Workshop Meeting 2-3 December 1997
Presentation by Jonathan Olliff
Established Methods of Pipeline Design in the UK

Until 1962, the structural design of sewers, in the UK, was based entirely on empirical methods. Furthermore, although crushing strength tests for clay pipes were first employed in the 1850s, they were not formally incorporated into a British Standard until 1966.

The design code introduced in 1962 covered only rigid pipes, and closely followed Marston theory, though incorporating the results of UK research into bedding factors, and vehicle surcharge loads.

With increasing use of plastics pipes in the 1960's, there was a need for flexible pipe design to be put on a comparable basis, and Spangler's method was adopted, with the modification that trench backfill design loads were not reduced according to "Silo theory".

Although the resulting rigid and flexible pipe design procedures were both based on Iowa work, there was little apparent connection between the two. The introduction of ductile iron pipes around 1970, prompted extensive research into their structural behaviour by the then Stanton and Staveley company, led by Mr Barrie Greatorex.

Although Greatorex's work became established as the UK method for semi-rigid pipes (initially by way of Stanton & Staveley's design manuals, and since 1988, with the writer's assistance, via WRc's "Pipe Materials Selection Manual"), its details and significance are unfortunately little known. In fact, far more than a design procedure for semi-rigid pipes was developed. What resulted was nothing less than the generalisation of the Iowa rigid and flexible pipe theories, via the use of a pipe: soil stiffness ratio function which provided a smooth and continuous link between them, whilst giving the same results at the ends of the stiffness ratio spectrum. This improvement to pipeline design theory was accompanied by extensive laboratory and field work which, amongst other things, established the soil modulus values used for both flexible and semi-rigid pipeline design.

The British approach to pipeline design has sought to be pragmatic, and also conservative. Thus the recommended numerical values for crucial design parameters such as bedding factors for rigid pipes, and soil moduli for flexible pipes, have always been set at what were believed to be "safe" levels. The strongly-held British view that the pursuit of precision in the design theory is of little value unless it can be matched with similar precision in the control, and mathematical representation, of the work of often unskilled and unsupervised men in the bottom of trenches, has resulted in a reluctance to attempt to quantify the statistical reliability of the pipeline design procedures. It is generally believed, however, that the probability that the reality will not be less favourable than the design is of the order of 90-95 percent.

Prompted by similar motives, British practice has taken account of experience and research in other countries, and has incorporated corresponding modifications to design practice. The Leonhardt procedure for modifying embedment soil moduli to reflect the influence of native soils was recommended in 1981, and Molin's strain factor, used particularly in the design of GRP pipelines, was attributed varying design values in 1988. The UK was, in fact, the first country to use the strain factor to reflect the influence of embedment compaction procedures on flexible pipe deformation.

J.L. Olliff
November 1997

8. Summary of UK Design Procedure
Non-Pressure Pipelines using Rigid, Semi-Rigid & Flexible Pipes

1. Calculation of Vertical Soil Pressure (Pvs)

$P_{vs} = C_1 C_2 wH$

Coefficients for embankments and wide trench installations:

$C_1 = 1 + 0.585 (1 - n)^{048} - 0.875 (1 - n) D/H$

$C_2 = 1.0$

Coefficients for narrow trench installations:

$C_1 = (B/D) / (1 - n + n B/D)$

$C_2 = (1 - e^{-x}) / x$

$n = E_{ss}/(105S + 0.8 E_{ss})$

$x = 2KmH/B$

2. Vertical Pressures due to Traffic (P_{vt})

Major roads	$P_{vt} = (87 - 5H) / H^{0.8}$
Normal roads	$P_{vt} = (70 - 5H) / H^{1.38}$
Off-road	$P_{vt} = 38 / H^{1.55}$
Railway	$P_{vt} = 150 / H^{1.9}$
Construction site	$P_{vt} = 265 / (0.45 + H^2)$

3. Flexible and Semi-Rigid Pipe Response

Deflection, $Y = \dfrac{0.083 (P_{vs} + P_{vt})}{8S + 0.061 E_{ss}}$

Bending strain = $Df.Y.t/D$

With Df from 3.0 to 7.75

4. Rigid Pipe Response

Minimum required bedding factor:

$$Fm = 1.25D (P_{vs} + P_{vt})/Wt$$

Bedding angle	Bedding factor Fm
0	1.1
45	1.5
180	1.9 - 2.5
360	2.2 - 2.5

Notation:

B	Trench width
D	Pipe diameter
E_{ss}	Spangler soil modulus
H	Depth of cover
K	Coefficient of lateral earth pressure
m	Soil friction coefficient
S	Pipe stiffness
t	Pipe wall thickness
w	Unit weight of soil

J.L. Olliff
November 1997

9. *CalVis*

Basics
The model is based on the response of a ring embedded in a soil envelope. Formulas for stress strain and deformation in both soil and pipe are available from this model.
The time dependent behaviour of the pipe soil system is obtained by making use of the 'correspondence' principle in combination with a direct inversion method. To this end relaxation data from both soil and pipe are needed.

The model is used to predict the average behaviour of the pipe soil system, so calculating average initial and final average deflections etc. Then 95 % probability levels are obtained by applying a multiplication factor to this average deflection etc. This multiplication factor depends on pipe stiffness and installation factor, and is given in a table. Then values are based on the information obtained from 'PiPer'.
The model makes use of a fixed load (3*D soil load).

Input values
Primary (To be supplied by user)
+ Soil group
+ Installation type
+ Depth of cover
+ Pipe diameter
+ Pipe material

Secondary (These parameters are taken from a predefined table)
- Soil stiffness (Es kPa)
- Relaxation factor of soil (ms -)
- Relaxation factor of pipe (mp -)
- Angle of internal shear soil (phie deg)

Application window
- All soils, including peat
- All pipes
For thick walled pipes (D/s<10) slight adaptation is necessary for strain and stress.

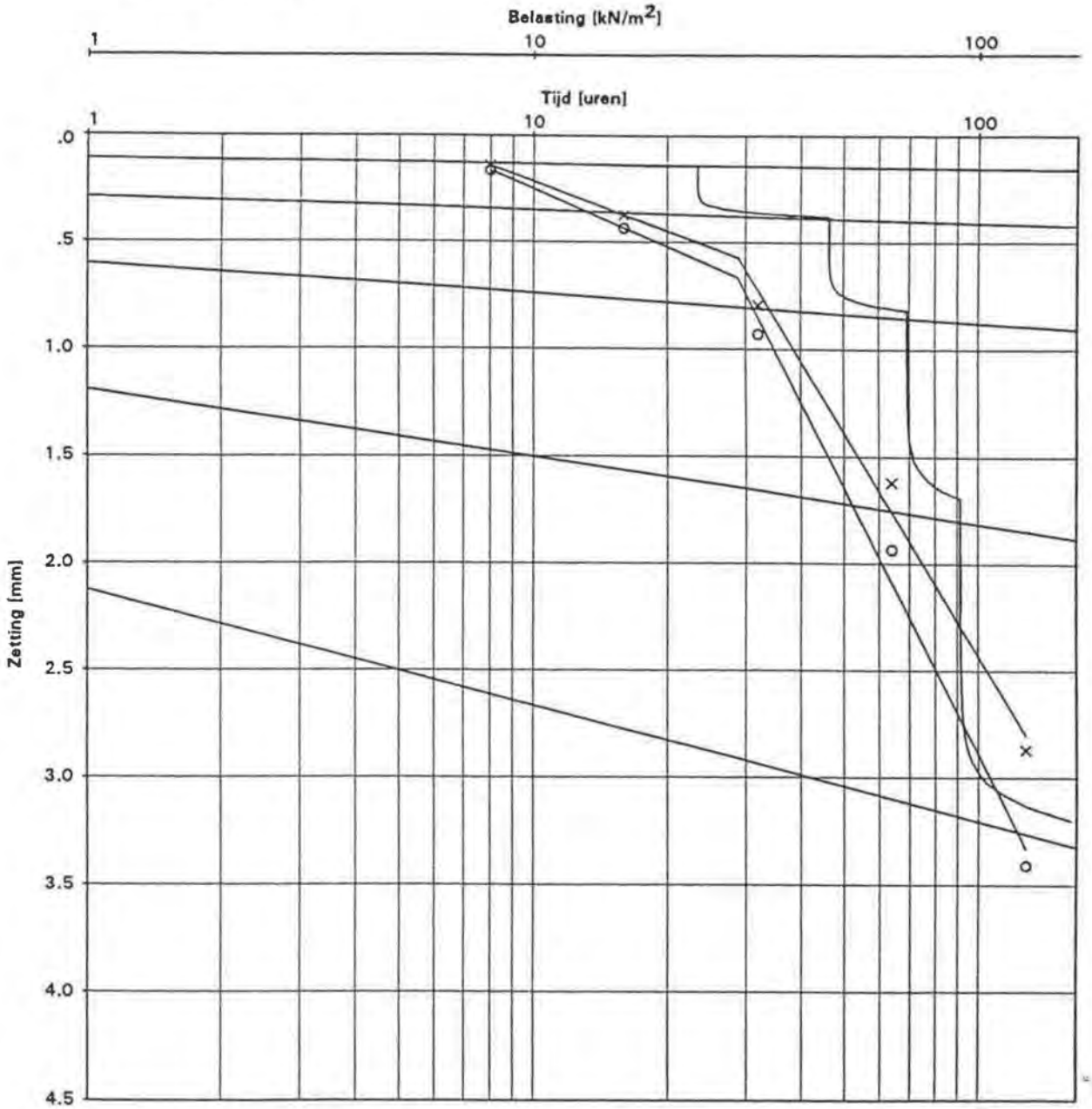
Level of design acc. to Eurocode 7
Level 1,2 and 3.
In case of level 2, properties are taken from the table.
In case of level 3, properties are taken from pre-investigation of soil and site.
In case of level 3, installation monitoring of anticipated input shall be performed.

Link with installation documents.
Yes, with prEN1046, especially table A1 and A2.
Further a firm link to the database 'PiPer' has been established, as this database provides information on variability of installation.

Frans Alferink
1/ 11/1997

Enclosure 6.1 : Oedometer result

Depth : 1.80 m, Clay



Boring	: B1	
Monster	: 2	
Diepte-MV	: 1.80 m	
Grondsoort	: KLEI matig siltig grijs	

Vg nat	=	14.8	kN/m^3
Vg droog	=	8.0	kN/m^3
Watergehalte	=	85.5	%

C1	=	34.5	
C2	=	7.2	
Pg	=	29	kPa
1/Cp1	=	.0177	
1/Cs1	=	.0028	
1/Cp2	=	.0789	
1/Cs2	=	.0152	
Hoogte	=	19.0	mm
Diameter	=	49.9	mm

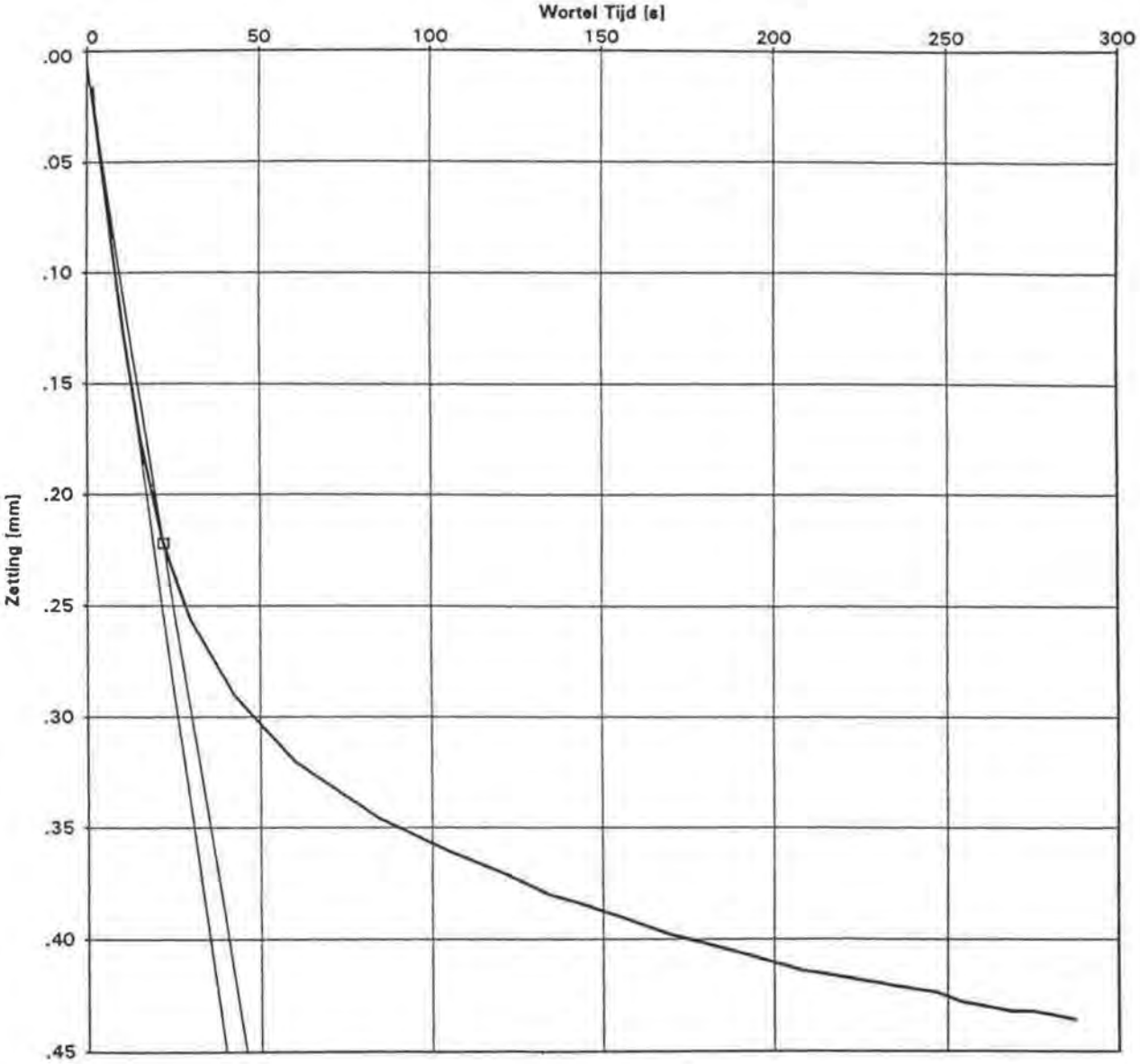
Made by: dd: 1-oct-97 Checked by: dd: 1-10-97

AHM Oedo 00.20 /11:18:52/12.OED

Samendrukkingsproef methode KEVERLING BUISMAN

Proefveld kunstofleidingen te Wons

Enclosure 6.1 : Oedometer result
Depth : 1.80 m, Clay



Boring	: B1
Monster	: 2
Diepte-MV	: 1.80 m
Grondsoort	: KLEI matig siltig grijs

Belastingstrap	:	3	
Belasting P	:	32.	kN/m^2
Belasting dP	:	16.	kN/m^2
Hoogte	:	18.620	mm

Consolidatie		50	90	%
H	=	.121	.217	mm
H100	=	.241	.241	mm
t	=	115	499	s
c_v	=	1.5E-07	1.5E-07	m^2/s
m_v	=	8.1E-01	8.1E-01	m^2/MN
k_v	=	1.2E-09	1.2E-09	m/s

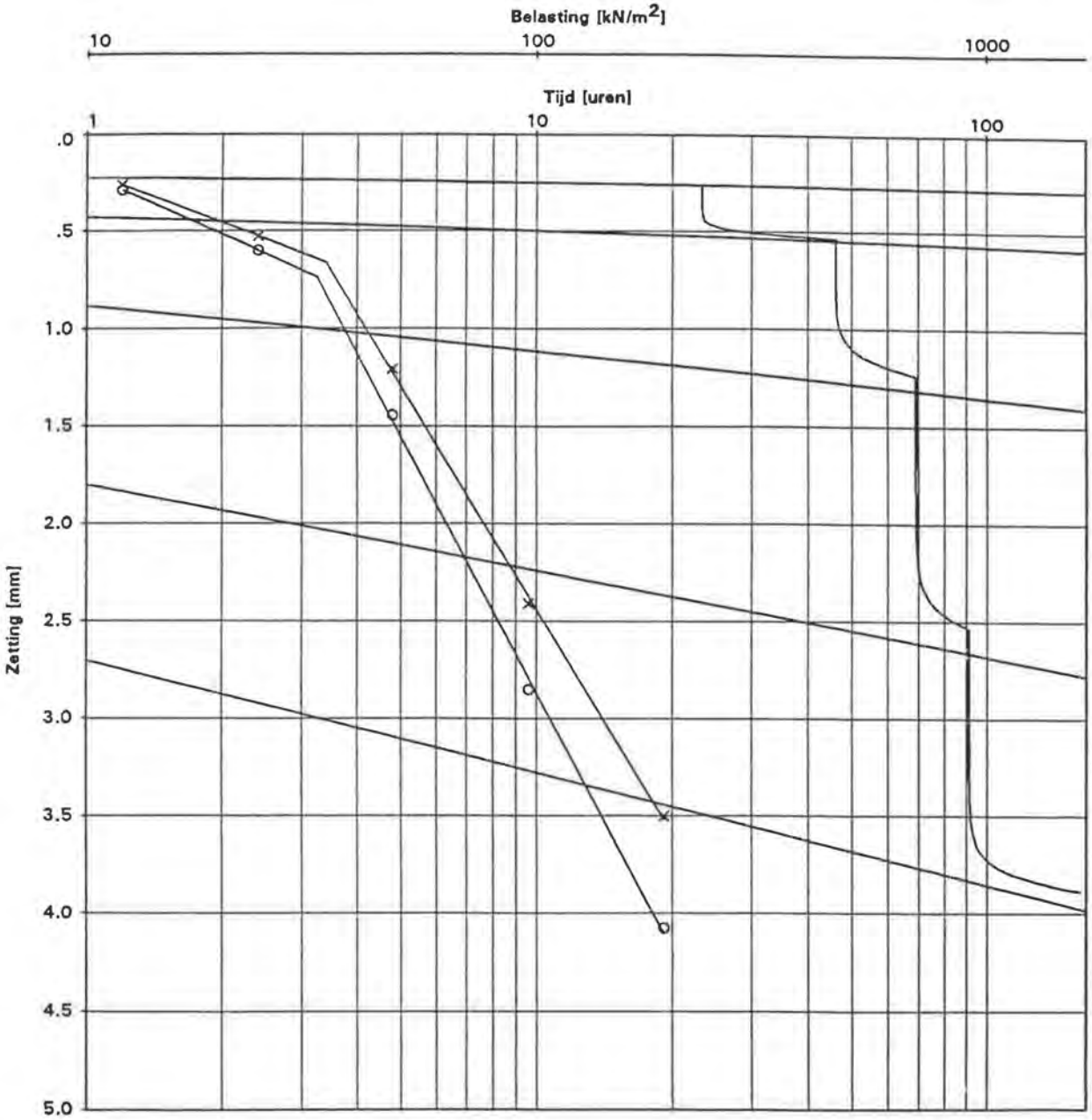
Made by: Wdd: 1-oct-97 Checked by: Vdd: 1-10-97

AHM OEDO 00.20 /11:17:38/ V2.OED

Cv bepaling d.m.v. TAYLOR methode

Proefveld kunstofleidingen te Wons

Enclosure 6.1 : Oedometer result
Depth : 3.20 m, Clay



Boring	: B1	
Monster	: 4	
Diepte-MV	: 3.20 m	
Grondsoort	: KLEI matig siltig grijs met zandlensjes	
Vg nat	= 15.5	kN/m^3
Vg droog	= 9.0	kN/m^3
Watergehalte	= 71.9	%

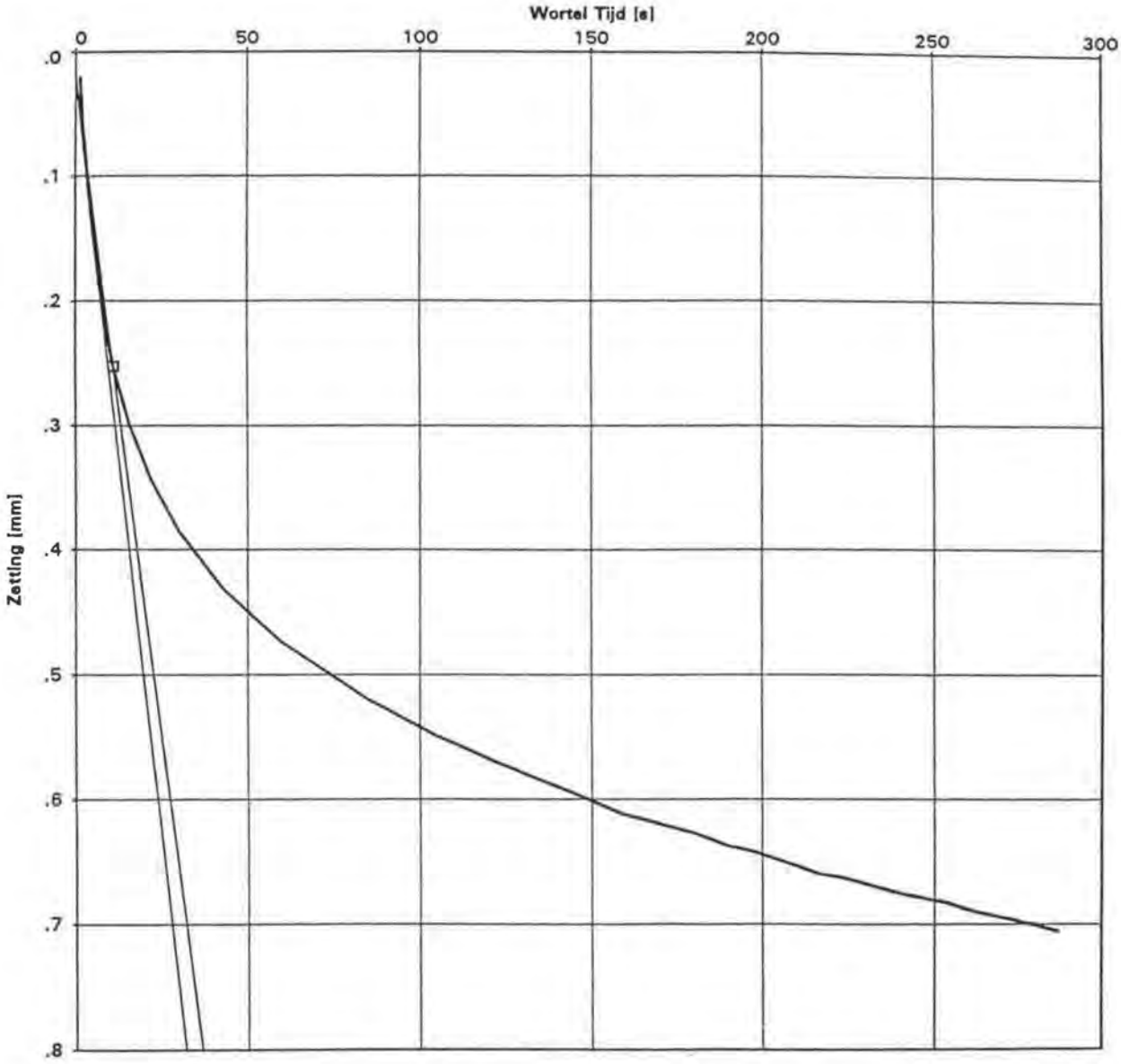
C1	=	29.3	
C2	=	7.2	
Pg	=	33	kPa
1/Cp1	=	.0202	
1/Cs1	=	.0035	
1/Cp2	=	.0870	
1/Cs2	=	.0129	
Hoogte	=	19.0	mm
Diameter	=	50.0	mm

Samendrukkingsproef methode KEVERLING BUISMAN

Proefveld kunstofleidingen te Wons

Enclosure 6.1 : Oedometer result

Depth : 3.20 m, Clay



Boring	:	B1
Monster	:	4
Diepte-MV	:	3.20 m
Grondsoort	:	KLEI matig siltig grijs met zandlensjes

Belastingstrap	:	3	
Belasting P	:	48.	kN/m^2
Belasting dP	:	24.	kN/m^2
Hoogte	:	18.464	mm

Consolidatie		50	90	%
H	=	.131	.235	mm
H100	=	.261	.261	mm
t	=	27	121	s
c_v	=	6.1E-07	6.0E-07	m^2/s
m_v	=	5.9E-01	5.9E-01	m^2/MN
k_v	=	3.5E-09	3.5E-09	m/s

Made by: Wdd: 1-oct-97 Checked by: Vdd: 1-10-97

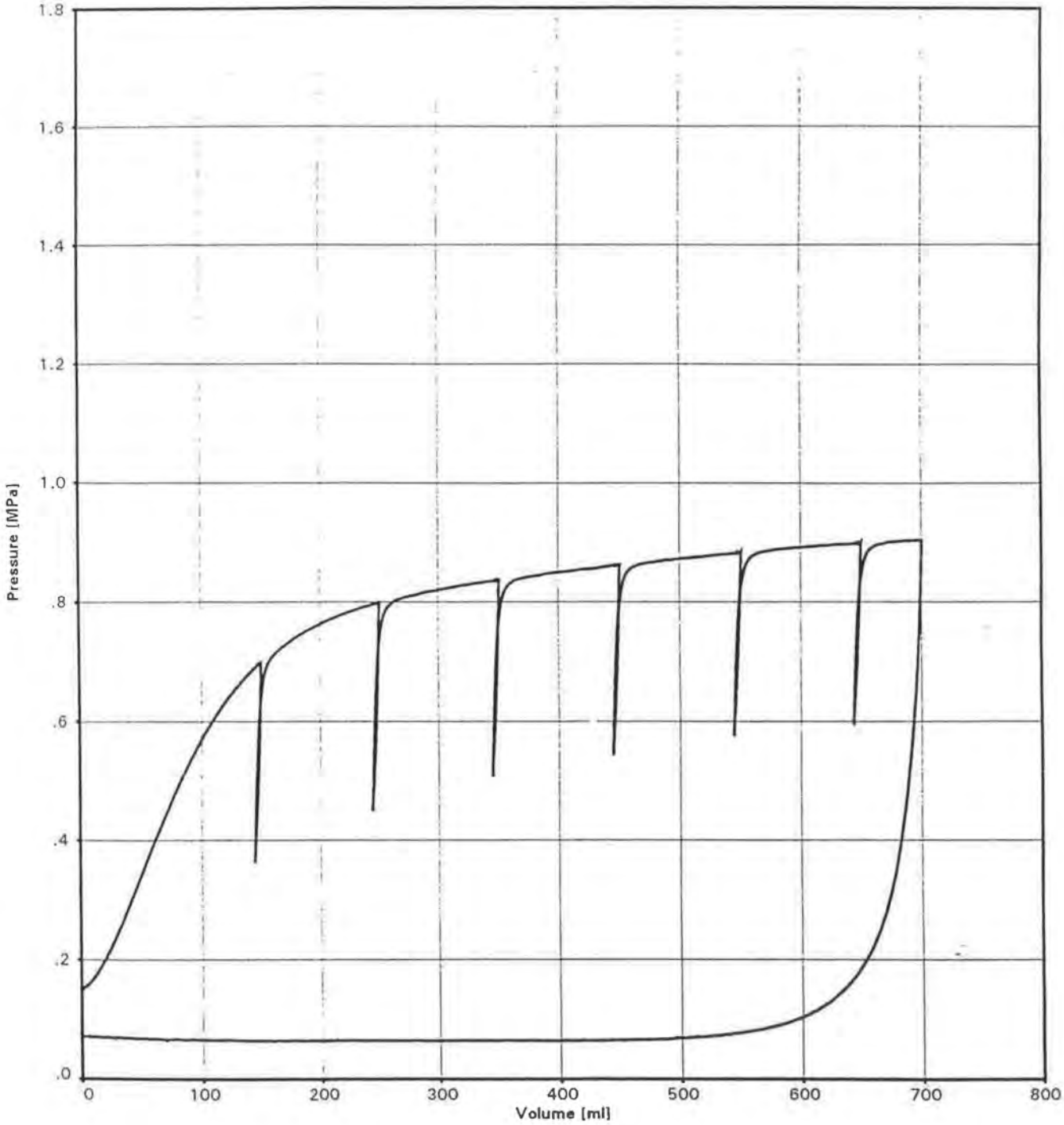
AHM OEDO 00.20 /11:18:38/ V4.OED

Cv bepaling d.m.v. TAYLOR methode

Proefveld kunstofleidingen te Wons

Enclosure 6.1 : CPM in compacted sand

Depth : 1.87 m

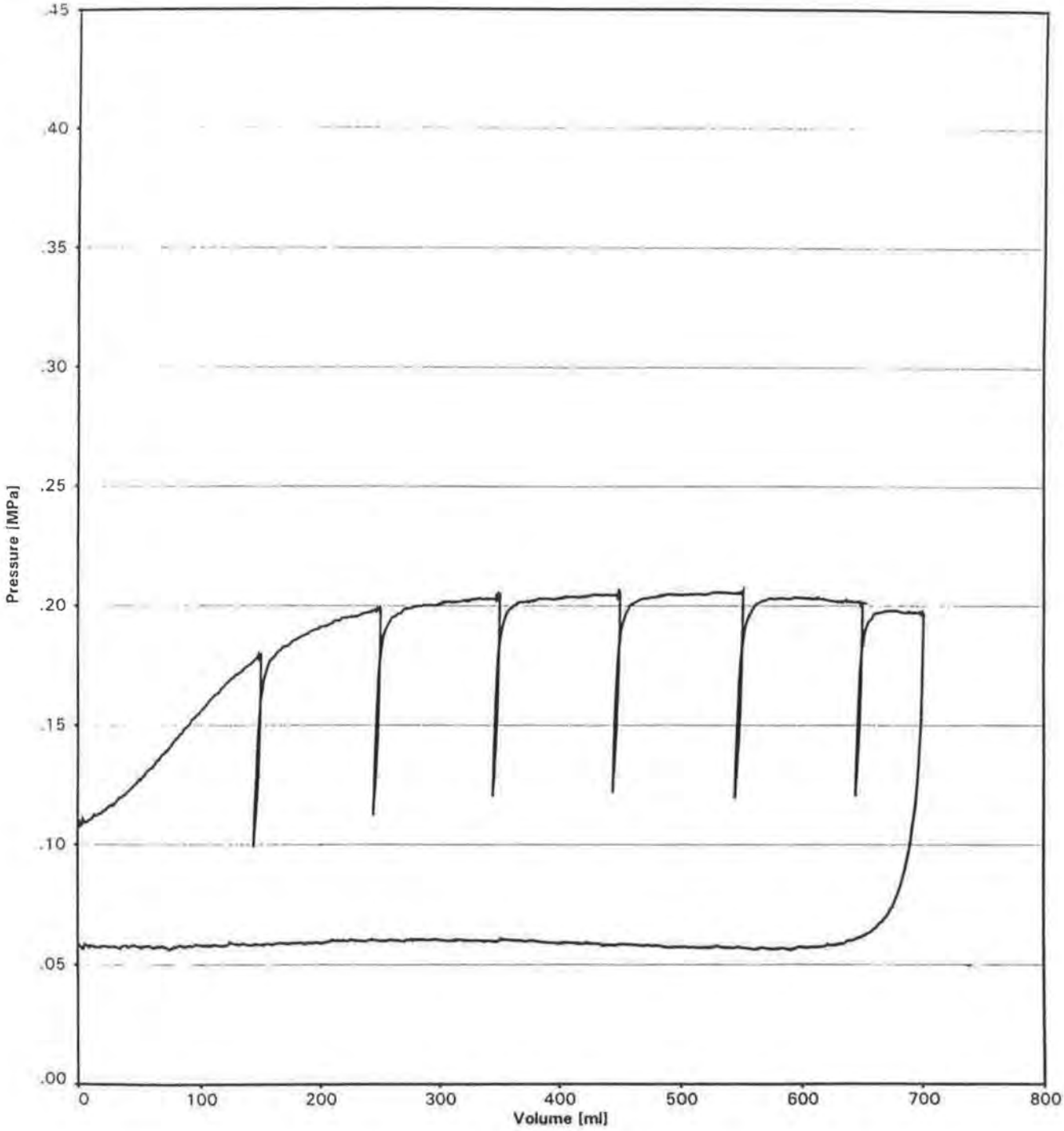


location	:	CPM 5.6
date of testing	:	26-Nov-1996
CPM test elevation	:	-1.87 m
ground surface elevation	:	.00 m
datum	:	Ground surface

CONE PRESSUREMETER TEST

PROEFVELD KUNSTOF LEIDINGEN TE HAARLE

Enclosure 6.1 : CPM in loose sand
Depth : 1.86 m



location	:	CPM 5.2
date of testing	:	26-Nov-1996
CPM test elevation	:	-1.86 m
ground surface elevation	:	+.00 m
datum	:	Ground surface

CONE PRESSUREMETER TEST
PROEFVELD KUNSTOF LEIDINGEN TE HAARLE

Enclosure 6.1 : Details about clay (1)
Depth : 1.75 m

ALGEMENE INFORMATIE

Boring	: B1	Proefstuk	: Ongeroerd
Monster	: 2	Monsterklasse	: 1
Diepte	: 1.75 m	Test Methode	: CUMS isotroop

VISUELE CLASSIFICATIE

KLEI matig siltig grijs

INITIELE EIGENSCHAPPEN	TRAP 1	TRAP 2	TRAP 3	
Hoogte	76.0			mm
Diameter	38.0			mm
Volumiek gewicht	15.7			kN/m^3
Droog volumiek gewicht	9.3			kN/m^3
Vochtgehalte	69.4			%
B-factor	0.99			-
Dichtheid van het korrelmateriaal (geschat)	2.65			t/m^3
NA VERZADIGING				
Verzadigingsspanning	300	300	300	kPa
Droog volumiek gewicht	9.3			kN/m^3
Vochtgehalte	69.4			%
B-factor	0.99			-
NA CONSOLIDATIE				
Horizontale consolidatie spanning	15	30	45	kPa
Verticale consolidatie spanning	15	30	45	kPa
Droog volumiek gewicht	9.3	9.4	9.6	kN/m^3
Vochtgehalte	69.4	67.6	65.5	%
AFSCHUIFFASE				
Axiale reksnelheid	6.0	6.0	6.0	%/uur
Bij maximale deviator spanning				
Effective horizontale spanning	2	10	14	kPa
Effective verticale spanning	40	58	74	kPa
Axiale rek	2.7	2.6	3.8	%
f_{undr}	19	24	30	kPa
ε_{50}	0.2	0.4	0.4	%
$E_{undr;50}$	10.6	5.4	7.5	MPa
Bij maximum hoofdspanningsverhouding σ_1'/σ_3'				
Effective horizontale spanning	2	10	12	kPa
Effective verticale spanning	38	57	69	kPa
Axiale rek	2.3	2.7	6.9	%
f_{undr}	18	24	29	kPa
ε_{50}	0.1	0.4	0.4	%
$E_{undr;50}$	12.6	5.4	7.8	MPa
EIND CONDITIES				
Bezwijkvorm proefstuk			Opgestuikt	
Droge dichtheid			9.6	kN/m^3
Vochtgehalte			65.5	%

BEZWIJK OMHULLENDE	maximale deviator spanning	maximale spannings-verhouding	maximale rek alle belastingtrappen	
Effectieve hoek van inwendige wrijving	29°	30	26	°
Effectieve cohesie	10	9	11	kPa

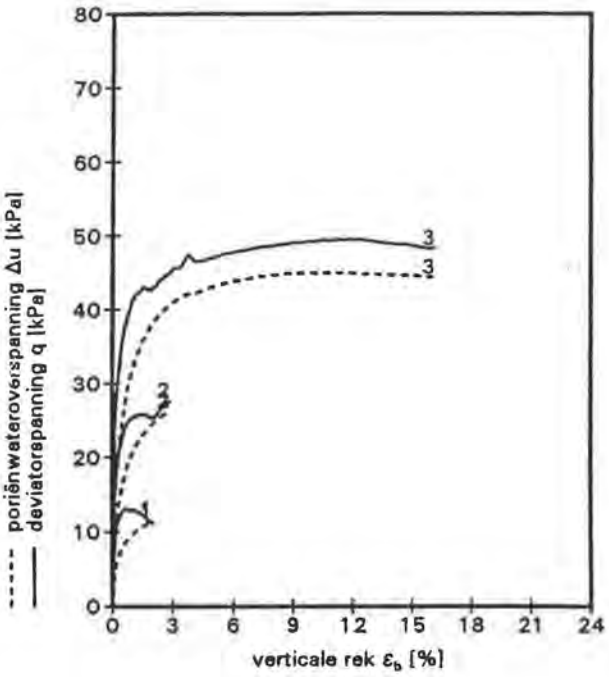
Opmerkingen:

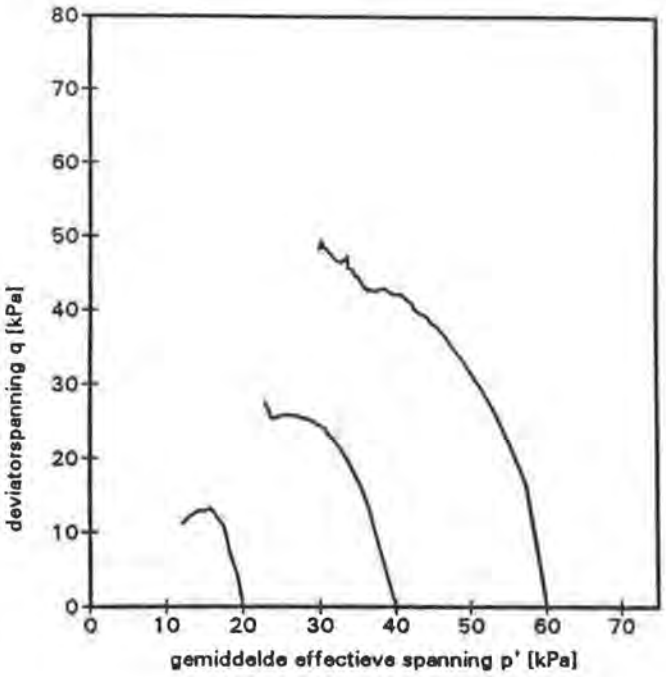
GECONSOLIDEERDE ONGEDRAINEERDE
TRIAXIAAL PROEF

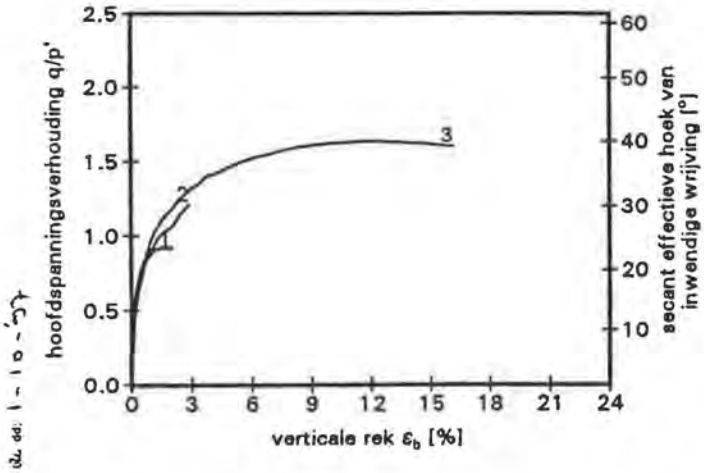
Proefveld kunstofleidingen te Wons

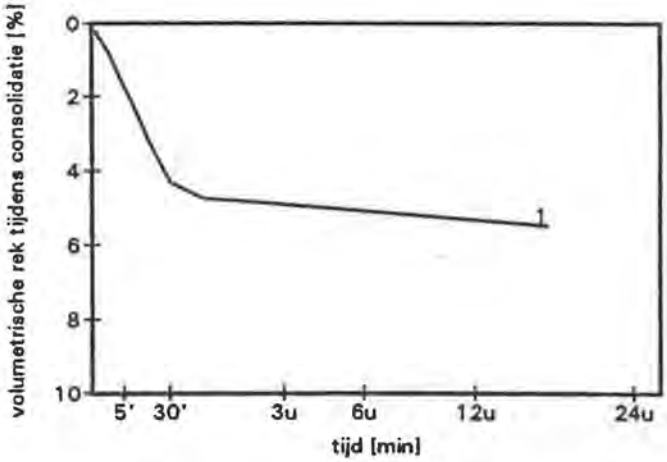
Enclosure 6.1 : Details about clay (2)

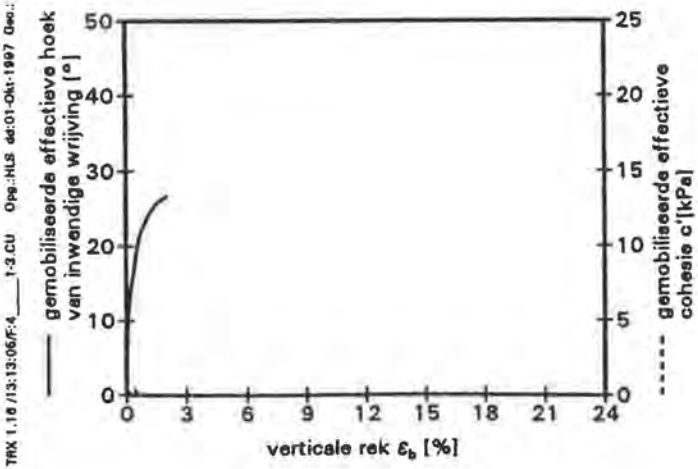
Depth : 1.75 m











Boring	: B1
Monster	: 4
Diepte	: 3.05 m
Grondsoort	: KLEI matig siltig grijs met zandlensjes

AHM TRX 1.16 /13:13:06/F:4 1-3.CU Ops.:HLS dd:01-Okt-1997 Geo.: 1 - 1 o - 1 ...

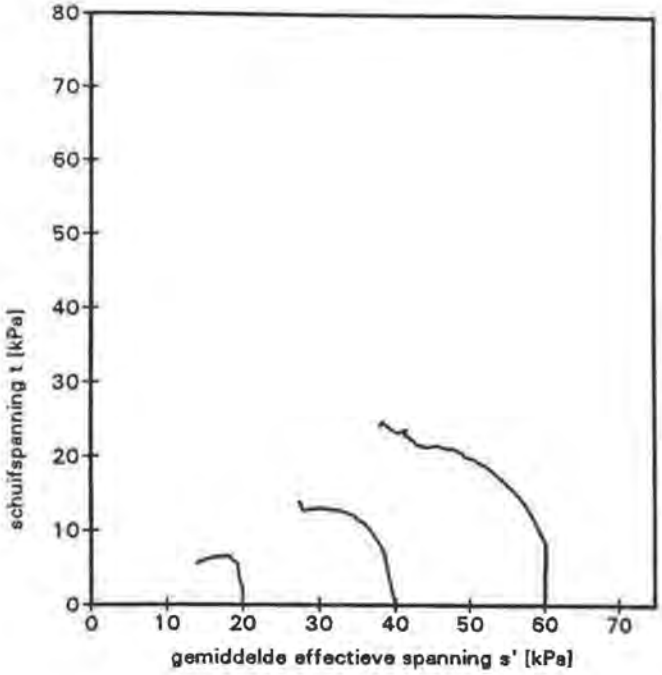
GECONSOLIDEERDE ONGEDRAINEERDE TRIAXIAAL PROEF

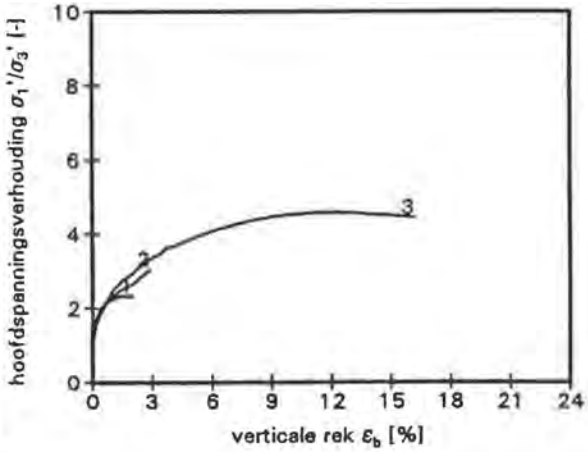
Proefveld kunstofleidingen te Wons

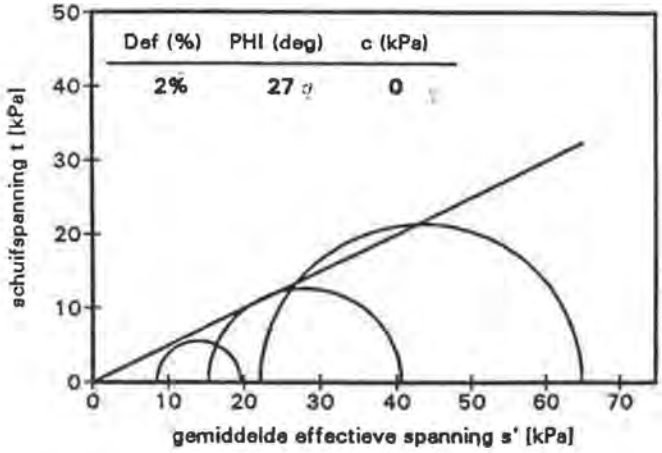
Enclosure 6.1 : Details about clay (3)

Depth : 1.75 m

Axiale rek	Eff. hoek van inwendige wrijving [φ']	Eff. cohesie [c'] kPa
0.5%	19	0
1.0%	24	0
1.5%	26	0
2.0%	27	0
3.0%	-	-
4.0%	-	-
6.0%	-	-
Max Def 2.0%	27	0







Boring : B1
Monster : 4
Diepte : [redacted]
Grondsoort : KLEI matig siltig grijs met zandlensjes

AVM TRX 1.18 /13:13:09/F:4 1-3.CU Opg.:HLS dd:01-Okt-1997 Gec.: 5 dd: 1 - 10 - 97

GECONSOLIDEERDE ONGEDRAINEERDE
TRIAXIAAL PROEF

Proefveld kunstofleidingen te Wons

Enclosure 6.2 :

Classification of soils, as shown in prEN1046

A.1 In this annex various types of soil are considered, namely non-cohesive, mixed grained, fine grained and organic. Each of these has subgroups, which for granular material are based on particle size and granulation and for cohesive material are based on levels of plasticity.

Table A.1: Soil groups:

Soil type	#	Typical Name	Soil group sym-bol *	Distinguishing characteristics	Example(s)
Non-cohesive, coarse grained granular, predominantly gravel sized	1	Single sized gravel	(GE) [GU]	Steep grading curve, predominance of one grain size	Crushed rock, river and beach gravel, morainic gravel
Non-cohesive, coarse grained granular, predominantly sand sized	2	Well graded gravels, gravelsand-mixtures	[GW]	Continuously sloping grading curve, several grain sizes	Scoria, vulcanic ash
		Poorly graded gravelsand-mixtures	(GI) [GP]	Steplike grading curve, one or more absent grain sizes	
		Single sized sands	(SE) [SU]	Steep grading curve, predominance of one grain size	Dune and drift sand, valley sand, basin sand
		Well graded sands, sand-gravel mixtures	[SW]	Continuously sloping grading curve, several grain sizes	Morainic sand, terrace sand, beach sand
		Poorly graded sandgravel-mixtures	(SI) [SP]	Steplike grading curve, one or more absent grain sizes	
Mixed grained soil with low fine fraction and some cohesion	3	Silty gravel-sand-mixtures	(GU) (SU) [GWM] [GPM] [SWM] [SPM]	Gravel being the pre-dominant fraction, particle size ≤ 0,06 mm (5 to 15 %)	Morainic gravel, weathered material, hillside debris, glacial till
		Clayey gravel-sand-mixtures	(GT) (ST) [GWC] [GPC] [SWC] [SPC]		
		Silty sands	(SU0 [SWM] SPM	Sand being the pre-dominant fraction, particle size ≤ 0,06 mm (5 to 15 %)	Tertiary sand, terrace sand
		Clayey sands	(ST) [SWC] [SPC]		
	4				>>>

Table A.1: (continuation)

Soil type	Soil group				
	#	Typical Name	sym-bol *	Distinguishing characteristics	Example(s)
Mixed grained soil with high fine fraction and moderate cohesion	4	Very silty gravelsand-mixtures	(GU') (SU') [GML] [GMI] [SMH] [SMV] [SME]	Well of poorly graded gravel resp. sand being the predominant fraction (portion of silt resp. clay 15 to [35] 40 %)	Morainic gravel, weathered material, hillside debris, glacial till
		Very clayey gravelsand-mixtures	(GT') (ST') [GCL] [SCL] [SCH] [SCV] [SCE]	Well of gap graded gravel resp. sand being the predominant faction (portion of silt resp. clay 15 to [35] 40 %)	Riverine loam, sandy loess, decalcified glocial till, calcerous glocial till
		Very silty resp. clayey sand	(SU') (ST')[SML] [SCH] [SCV] [SCE]		
		Silty or clayey fine sand	(SU') (ST') [SWM] [SPM] [SWC] [SPC]	Particle size < 0,2 mm (portion of silt resp. clay 5 to [35] 40 %), low plasticity, low dry strength	Loess, lecustrine clay
		Silt of low plasticity	(UL) [MLS] [MIS]	Particle size < 0,06 mm (portion of silt > [35] 40 %)	
Fine grained cohesive soils	5	Inorganic silts, very fine sands, rock flour, silty of clayey fine sands	(UL) [ML]	Low stability, rapid reaction, nil to slight plasticity	Loess, loam
		Inorganic clay, ditinctly plastic clay	(TA) (TL) (TM) CL	Medium to very high stability, no to slow reaction, low to medium plasticity	Alluvial marl, clay
Organic	6	Mixed grained soils with admixtures of humus or chalk	[OK]	Admixtures of plant or non-plant type, decay smell, light weight, large porosity	Top soils, chalky sand, tuff sand
		Organic silt and organic silt clay	(OU) [OL]	Medium stability, slow to very quick reaction, low to medium plasticity	Sea chalk, top soil
		Organic clay, clay with organic admixtures	(OT) [OH]	High stability, nil reaction, medium to high plasticity	Mud, loam
	7				>>>

Table A.1: (concluding)

Soil type	#	Typical Name	Soil group sym-bol *	Distinguishing characteristics	Example(s)
Organic	7	Peat, other highly organic soil	(HZ) (HN) [Pt]	Decomposed peats, fibrous, brown to black coloured	Peat
		Muds	[F]	Sludges deposited under water, often inerspersed with sand/clay/chalk, very soft	Muds

* The symbols used are taken from two sources. Symbols in square brackets [..] are taken from the British Standard BS 5930. Symbols in round brackets (..) are taken from the German Standard DIN 18196.

Where a soil is a mixture of types then whichever is the predominant one present can be used for the classification.

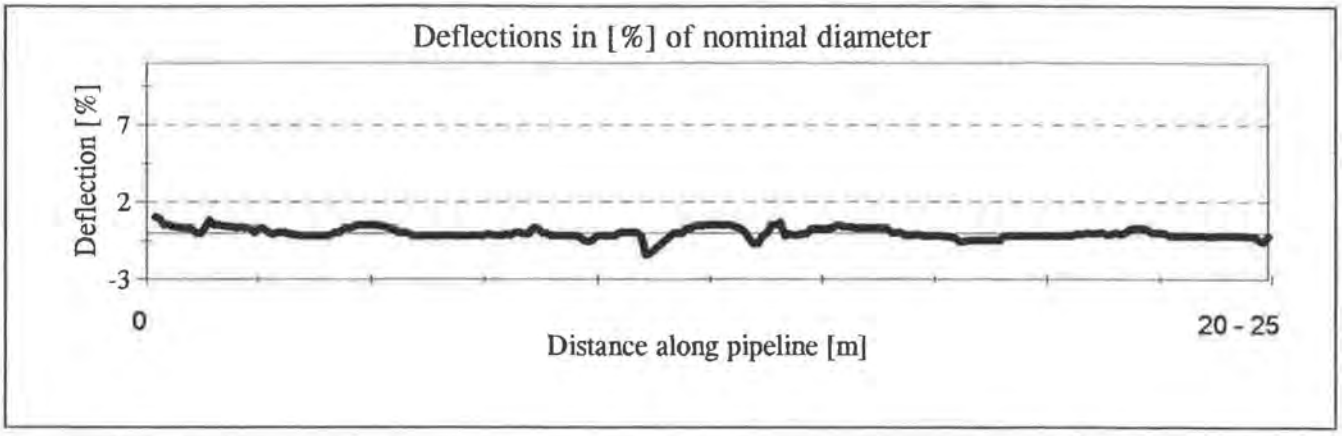
Table A.2: Consolidation class terminology

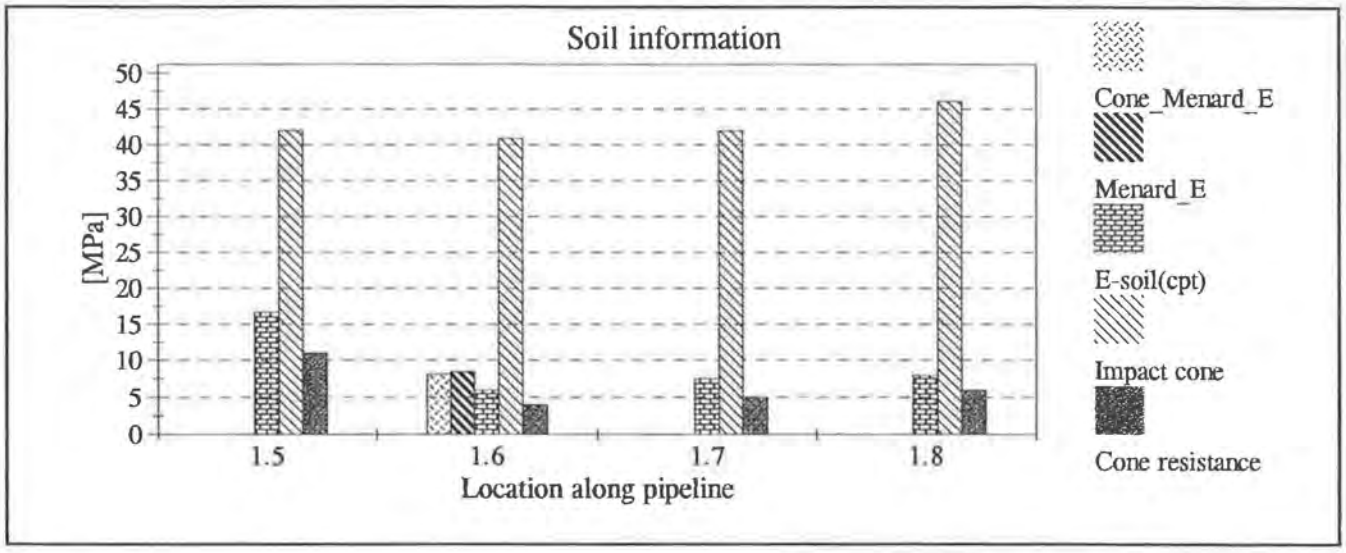
Description	Degree of consolidation			
% Standard Proctor 1)	≤ 80	81 to 90	91 to 94	95 to 100
Blow count	0 to 10	11 to 30	31 to 50	> 50
Expected degrees of consolidation achieved by the compaction classes in this standard	NOT (**N**)	MODERATE (**M**)		WELL (**W**)
Granular soil	loose	medium dense	dense	very dense
Cohesive and organic soil	soft	firm	stiff	hard

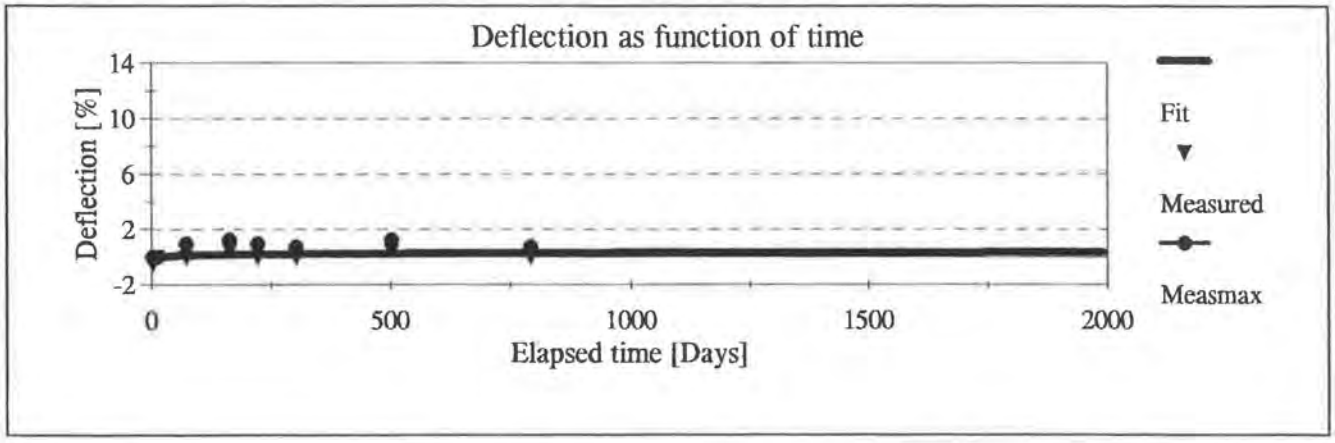
1) Determined in accordance with DIN 18127

Note: Table A.2 is meant to be an aid for interpretation of descriptions used in various sources into the terms used for the degrees of consolidation in this standard.

Where detailed information of the undisturbed native soil is not available then it is usually assumed that it has a consolidation equivalent to between 91 % and 97 % Standard Proctor Density (D_{Pr})

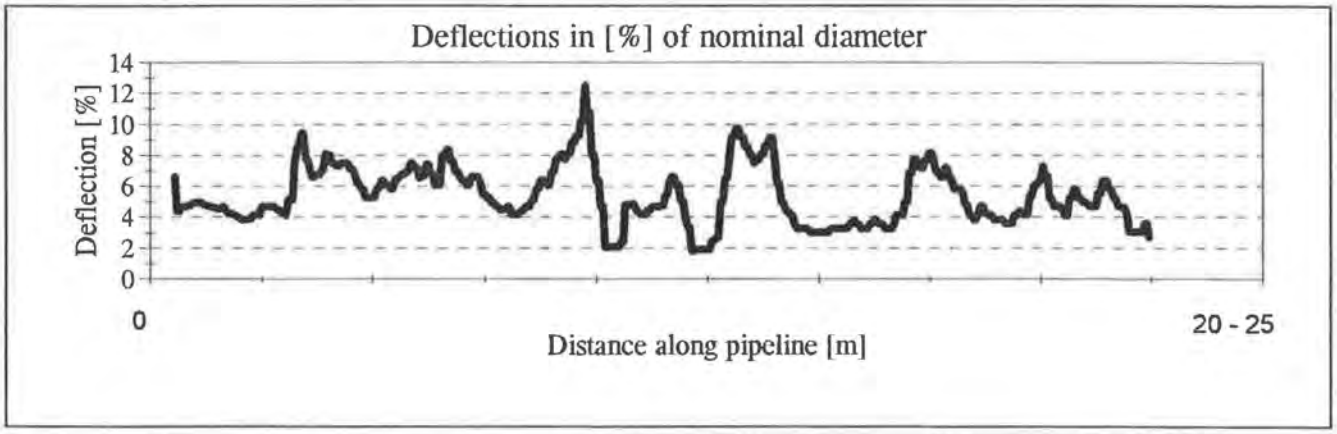


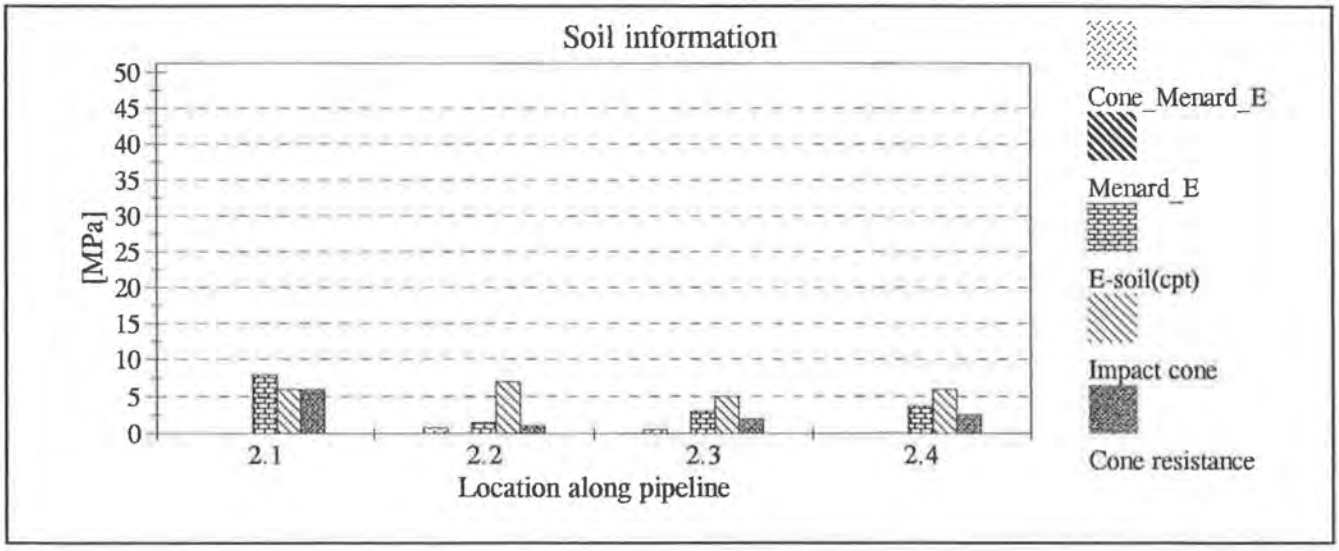


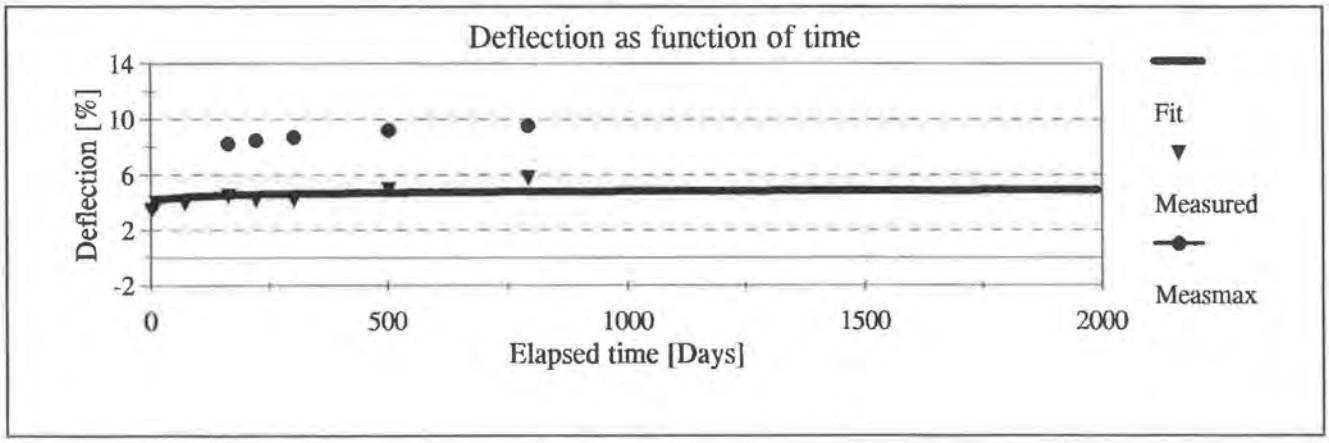


Information	Value
Case	Case 1
Pipe material	PVC
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	Well
Depth of cover [m]	1,15

Result	Value
Initial average	-0.5
Initial maximum	0.0
Final average	0.0
Final maximum	1.0
t_100[Days]	0.0

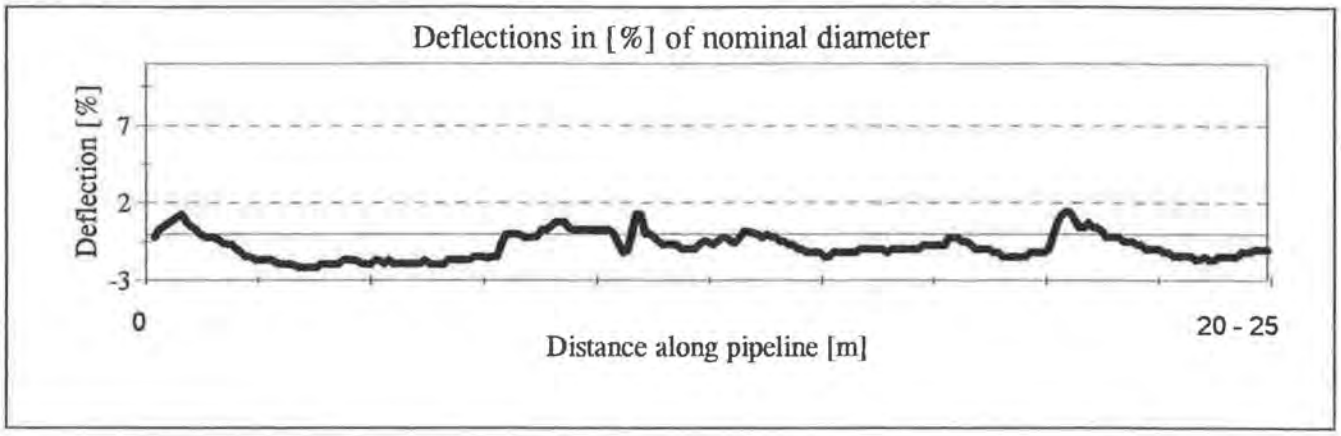


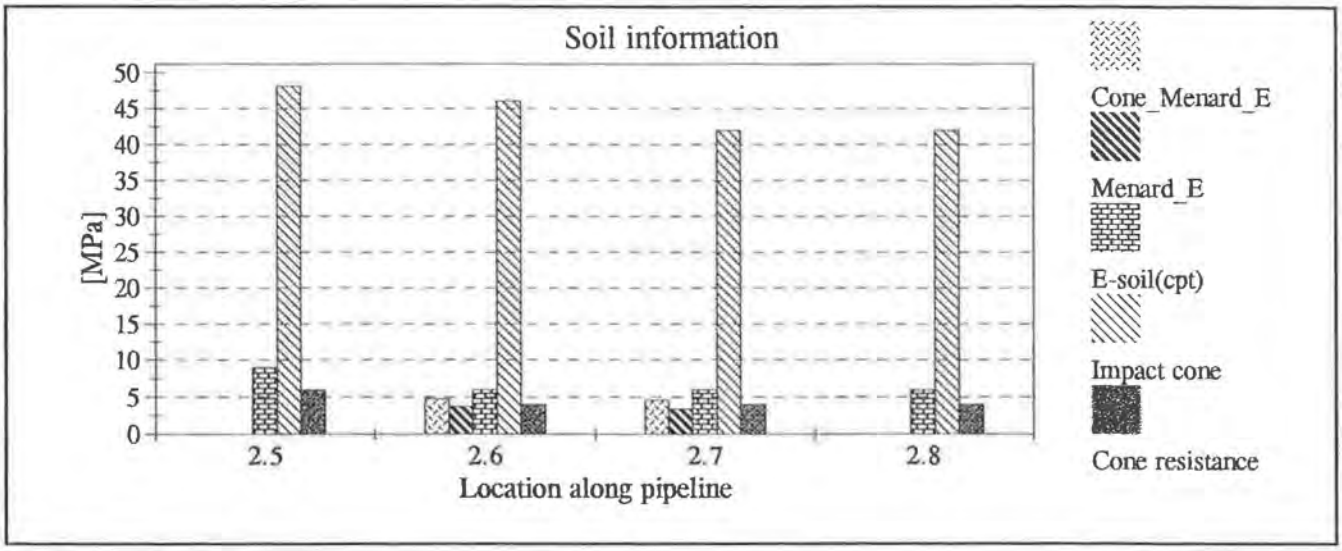


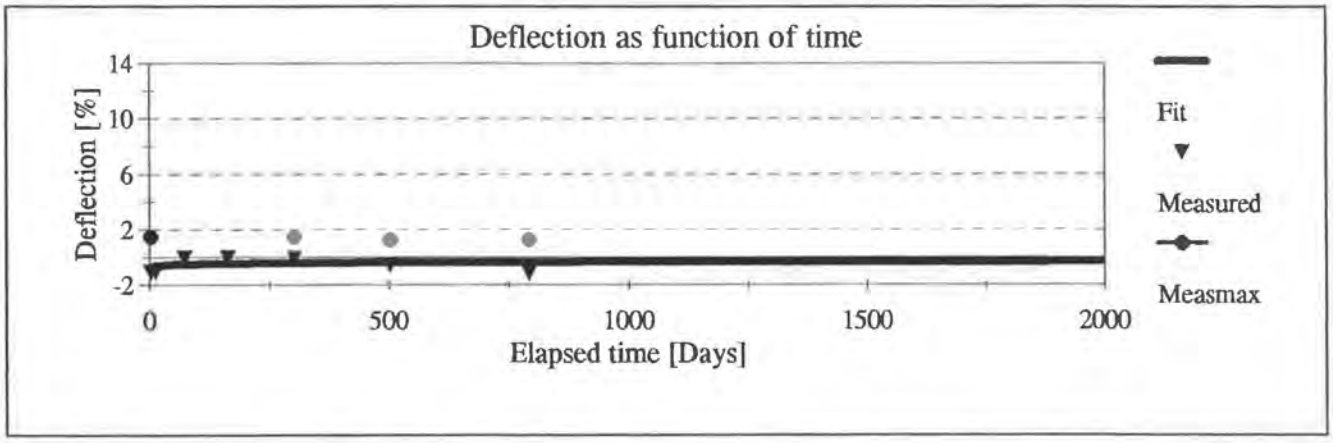


Information	Value
Case	Case 2
Pipe material	PVC
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,15

Result	Value
Initial average	3.5
Initial maximum	6.0
Final average	5.5
Final maximum	8.0
t_100[Days]	67.0

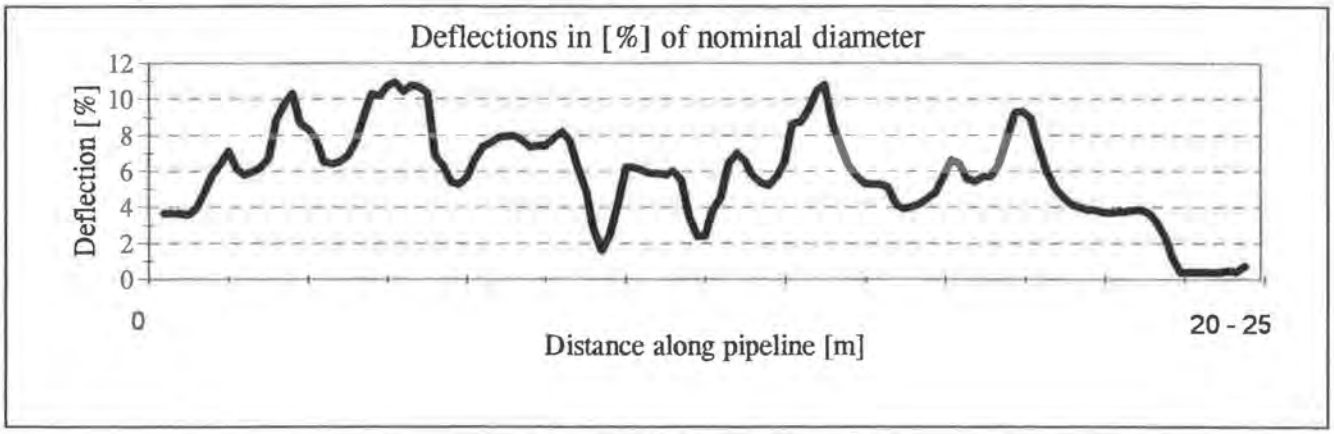


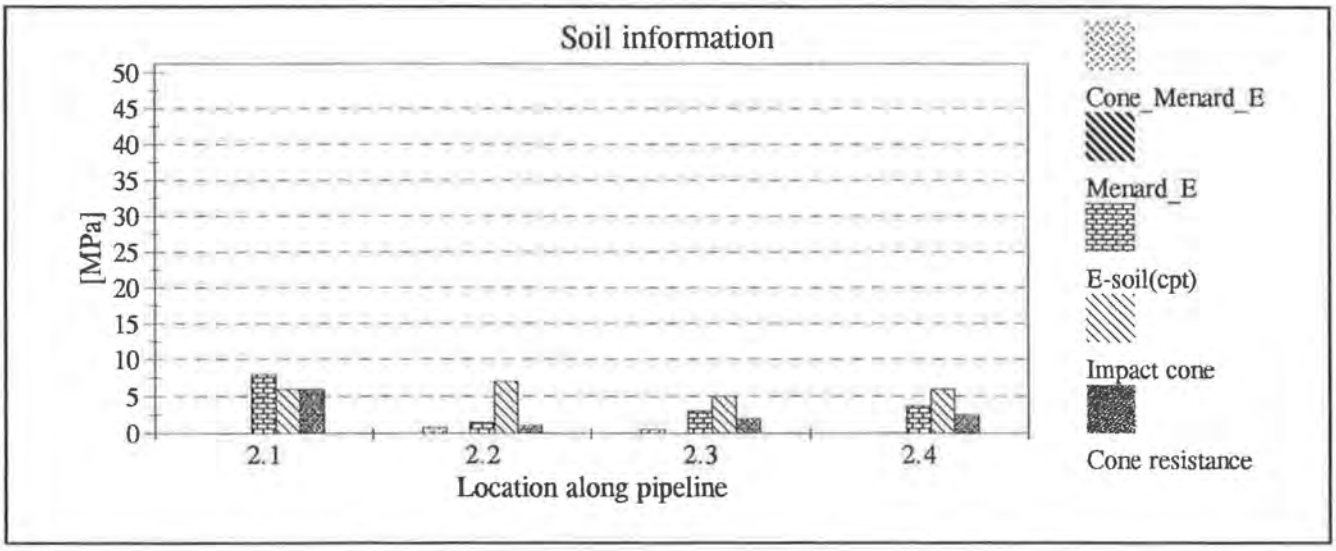


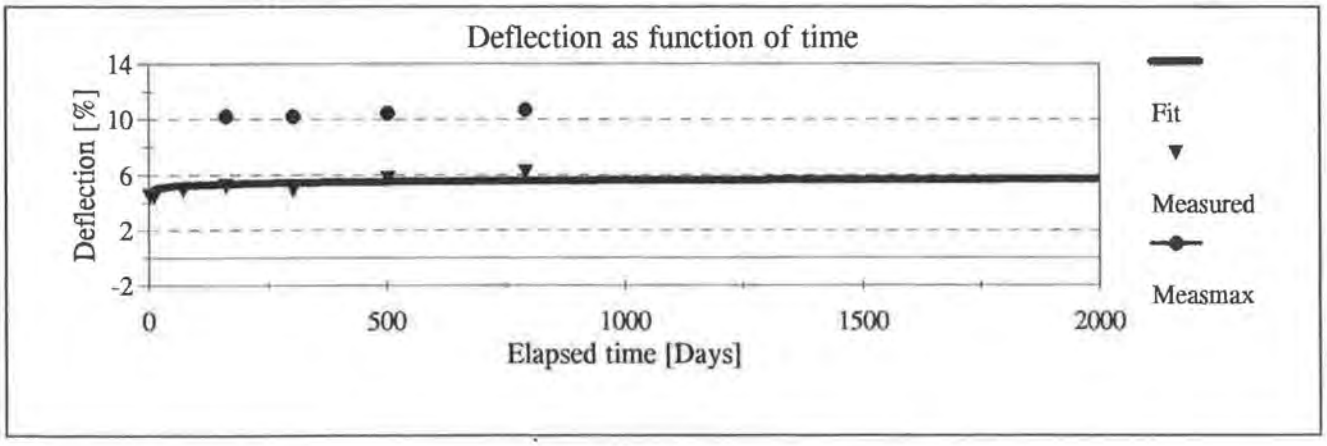


Information	Value
Case	Case 3
Pipe material	PVC
SN [kN/m3]	2,00
SN actual [kN/m3]	3,52
Embedment	Sand
Native soil	Sand
Installation	Well
Depth of cover [m]	1,15

Result	Value
Initial average	-1.0
Initial maximum	0.0
Final average	0.0
Final maximum	1.0
t_100[Days]	0.0

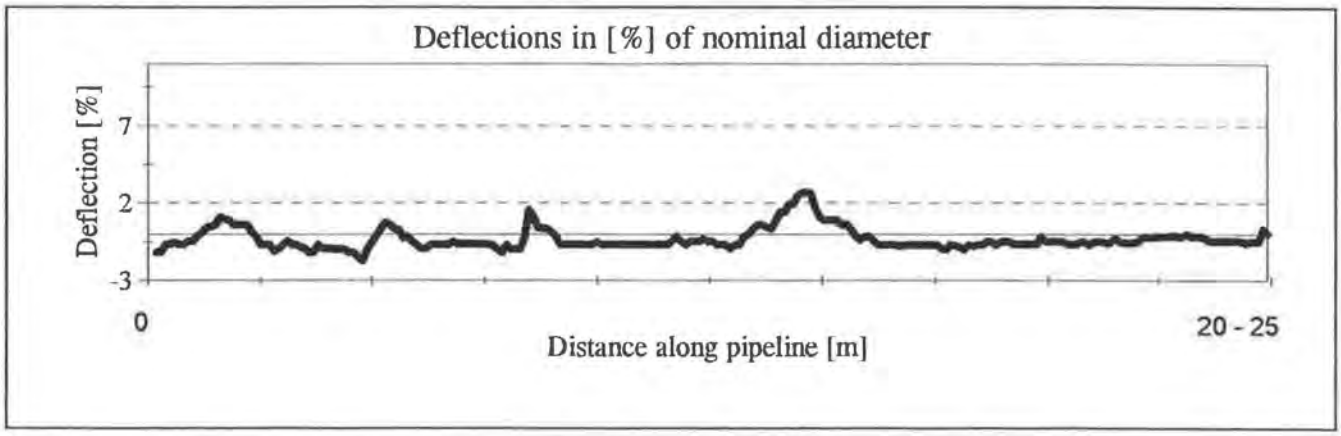


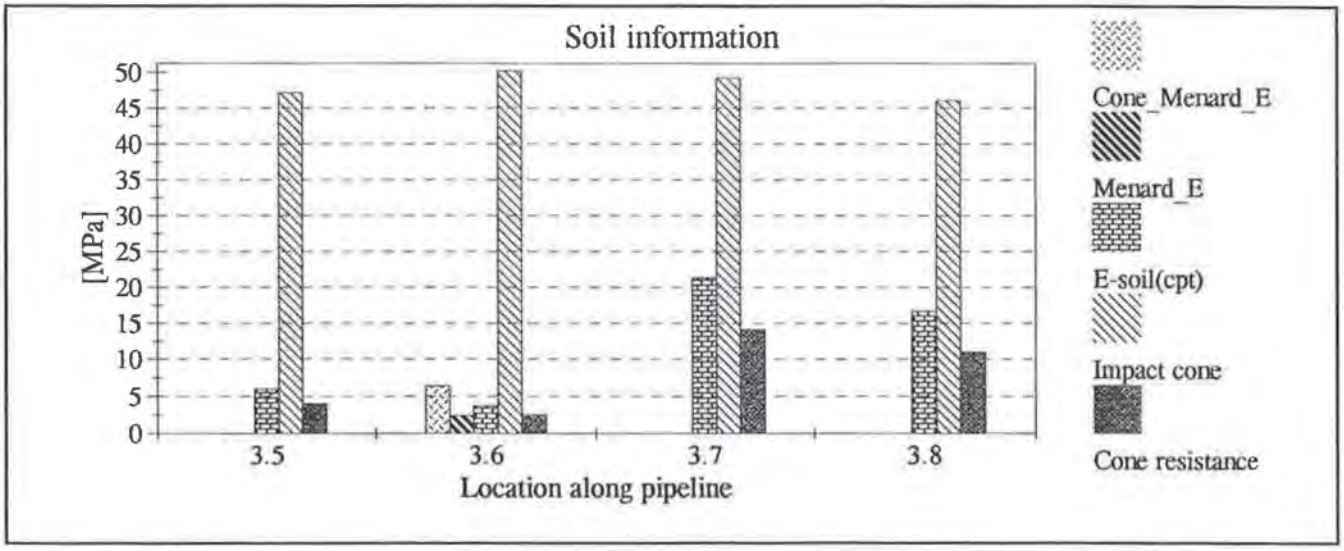


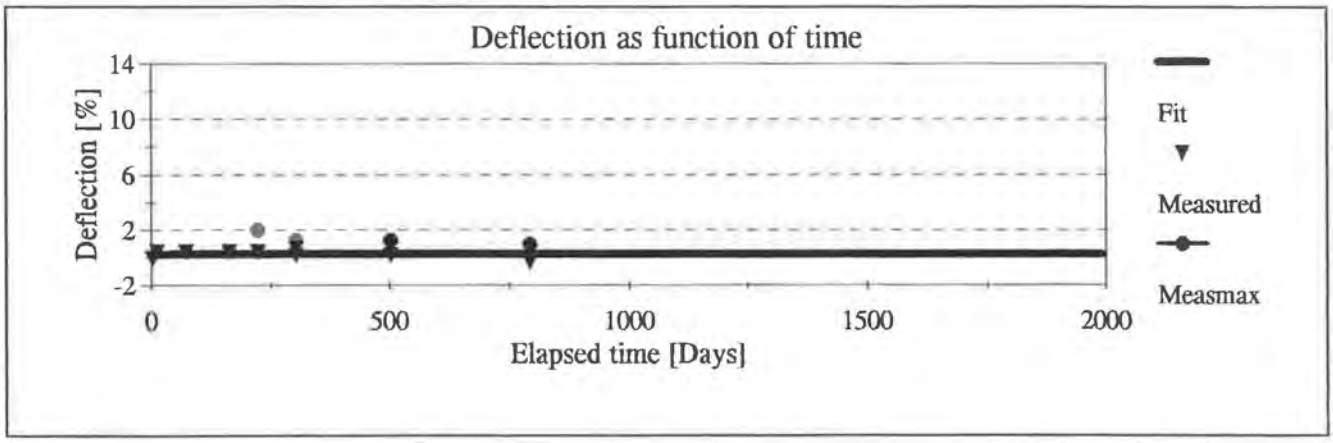


Information	Value
Case	Case 4
Pipe material	PVC
SN [kN/m3]	2,00
SN actual [kN/m3]	3,52
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,15

Result	Value
Initial average	4.5
Initial maximum	9.0
Final average	6.0
Final maximum	10.3
t_100[Days]	35.0

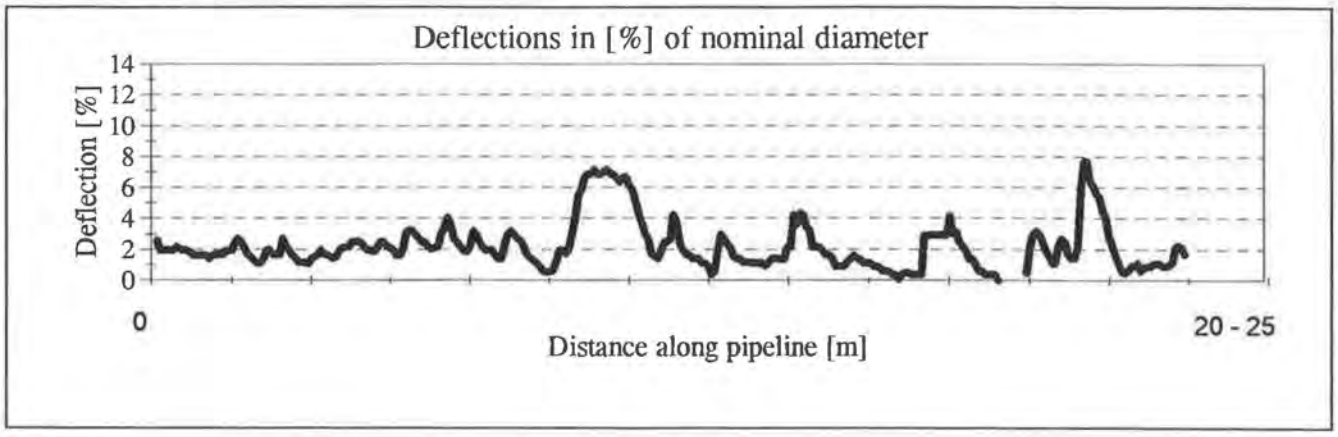


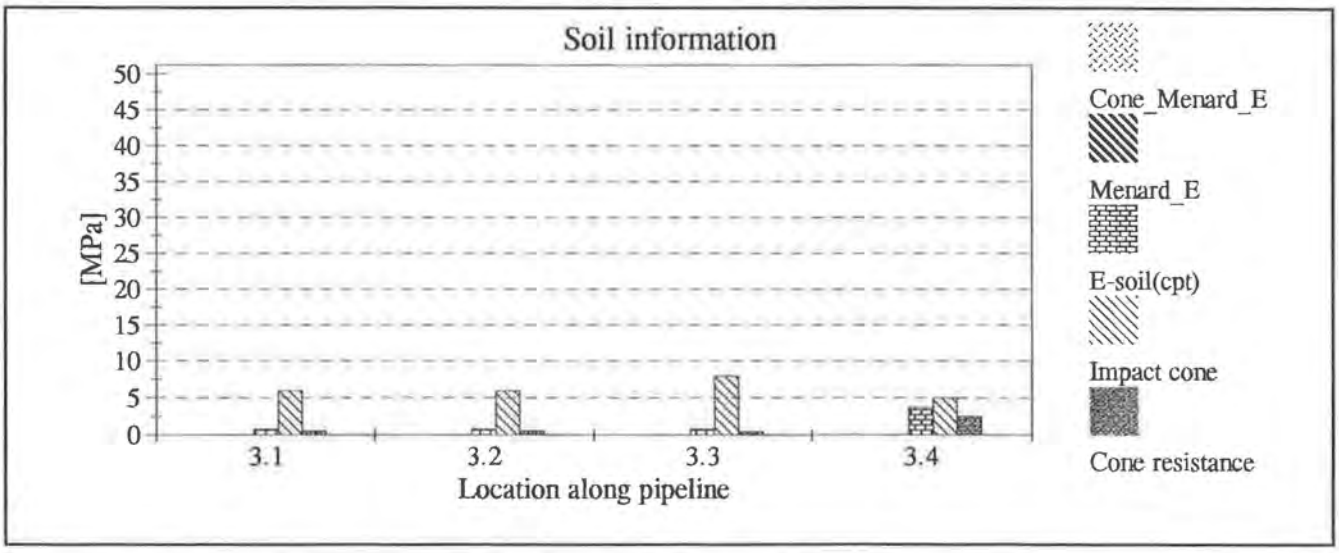


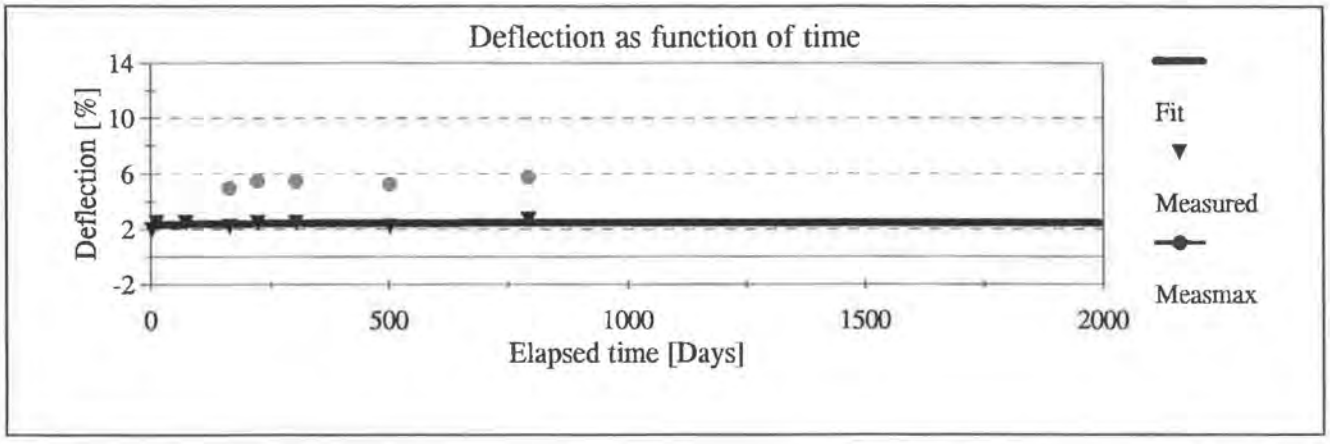


Information	Value
Case	Case 5
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Sand
Native soil	Sand
Installation	Well
Depth of cover [m]	1,15

Result	Value
Initial average	0.0
Initial maximum	2.5
Final average	0.3
Final maximum	2.75
t_100[Days]	0.0

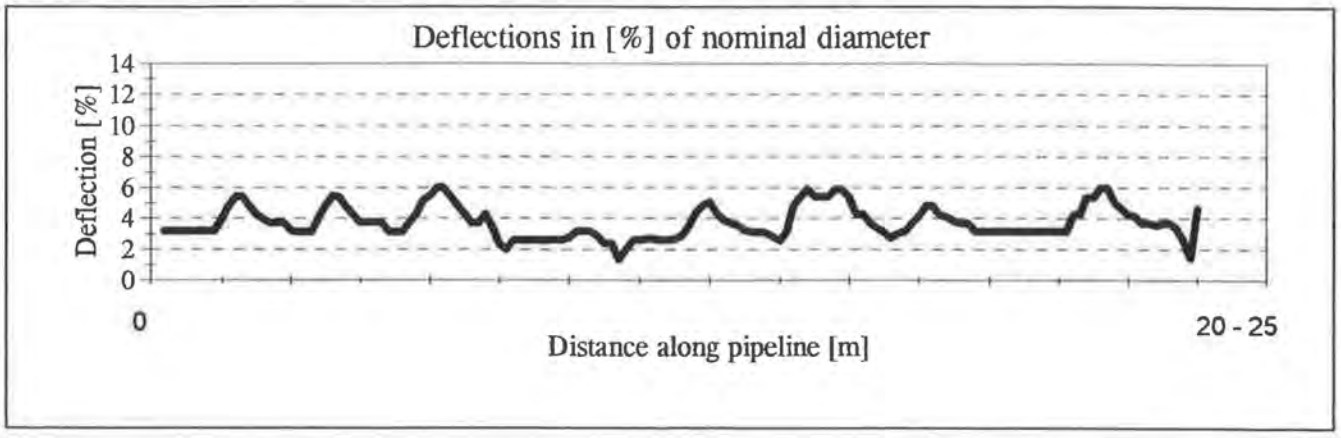


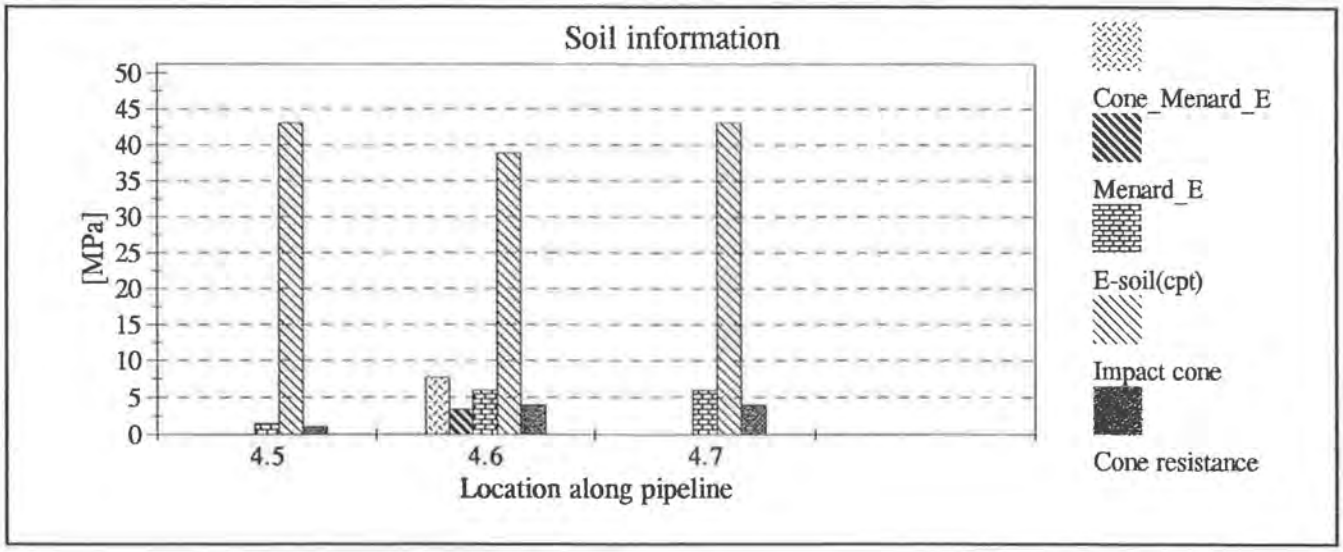


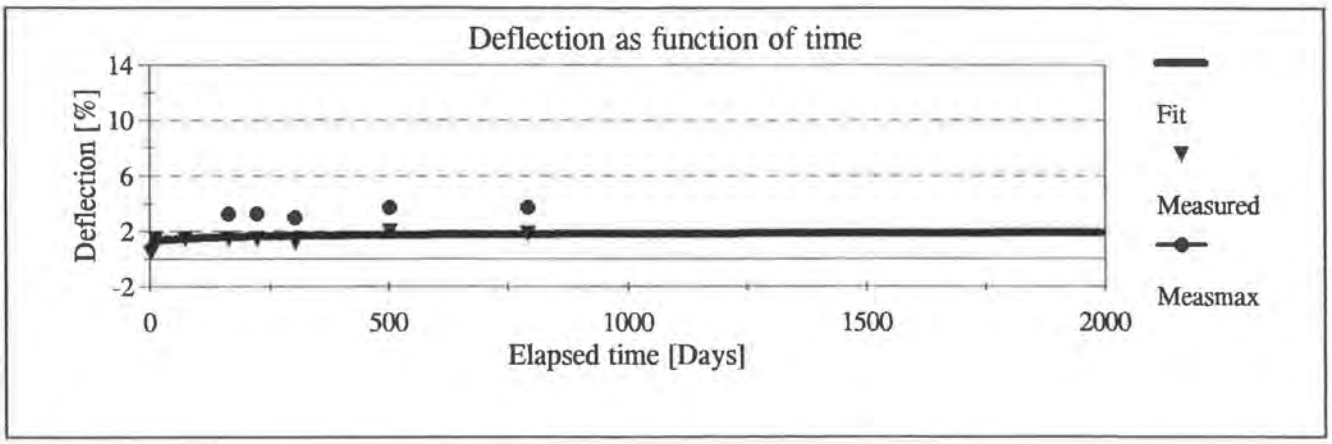


Information	Value
Case	Case 6
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,15

Result	Value
Initial average	2.0
Initial maximum	5.0
Final average	2.5
Final maximum	6.0
t_100[Days]	0.0

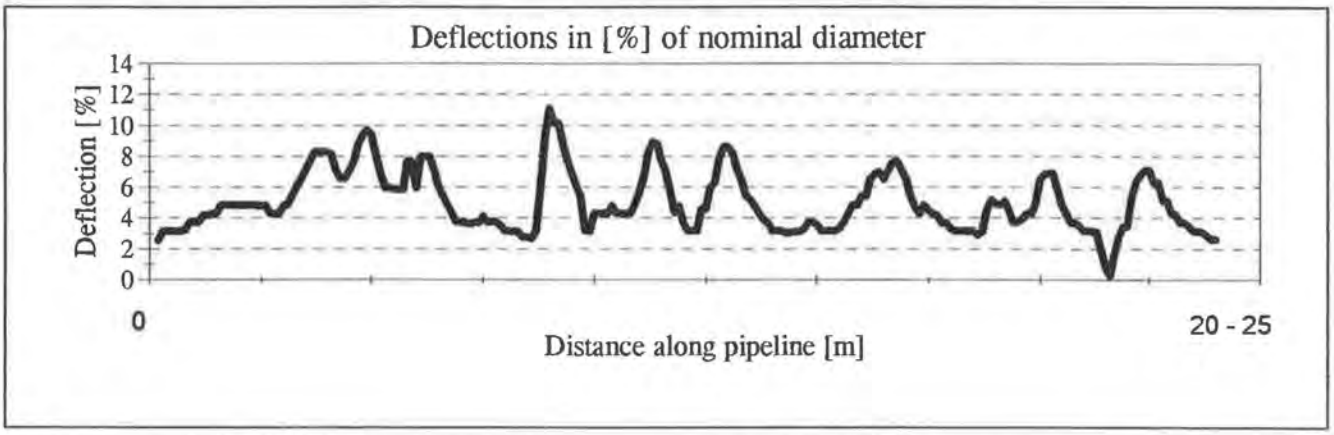


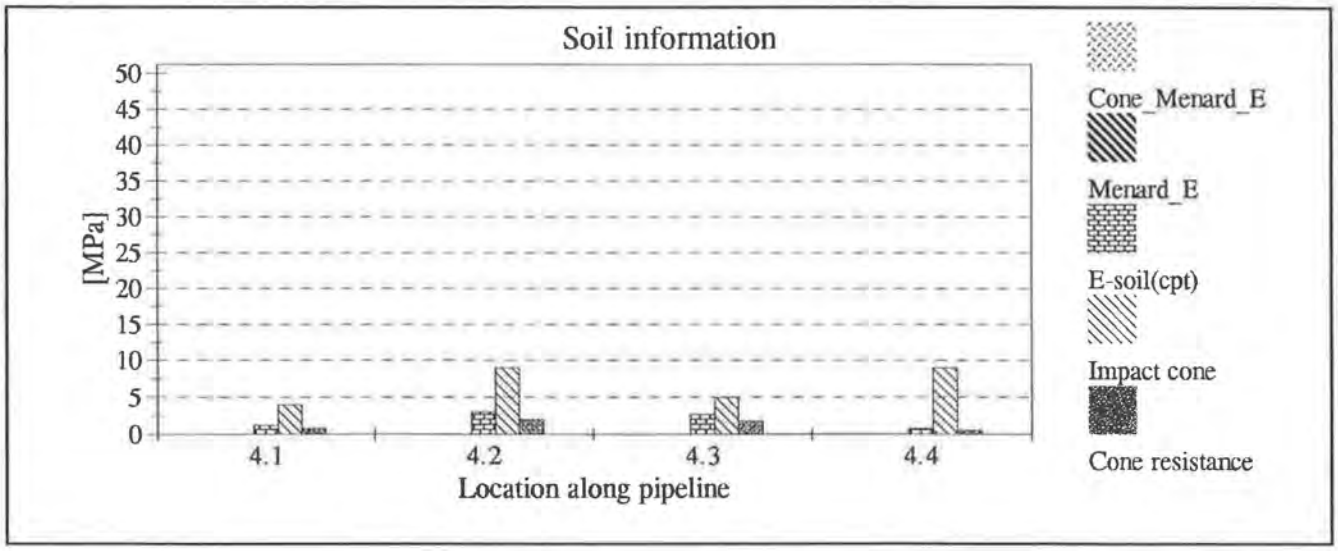


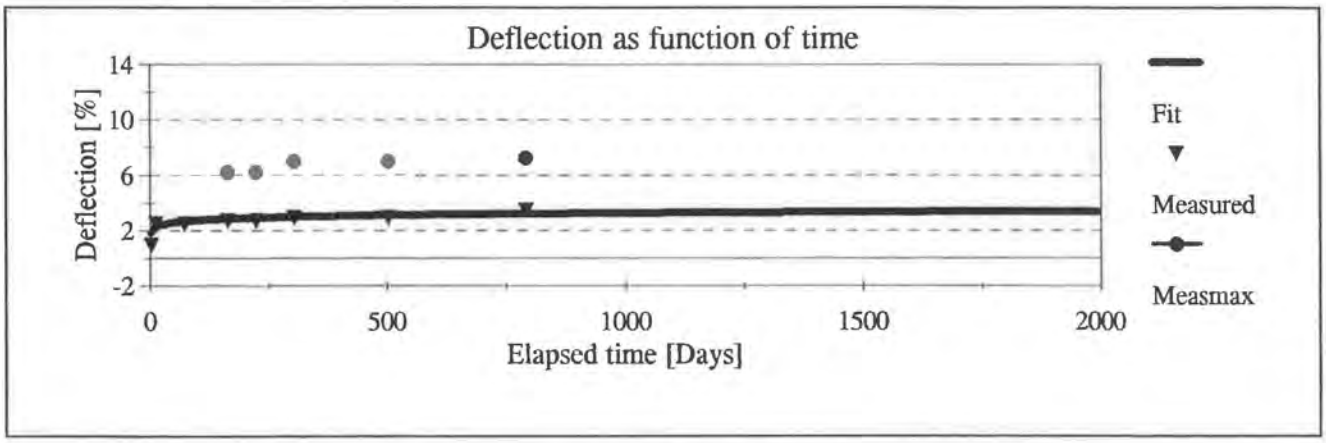


Information	Value
Case	Case 7
Pipe material	PVC
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	Moderate
Depth of cover [m]	1,90

Result	Value
Initial average	0.5
Initial maximum	1.5
Final average	2.3
Final maximum	3.5
t_100[Days]	12.0

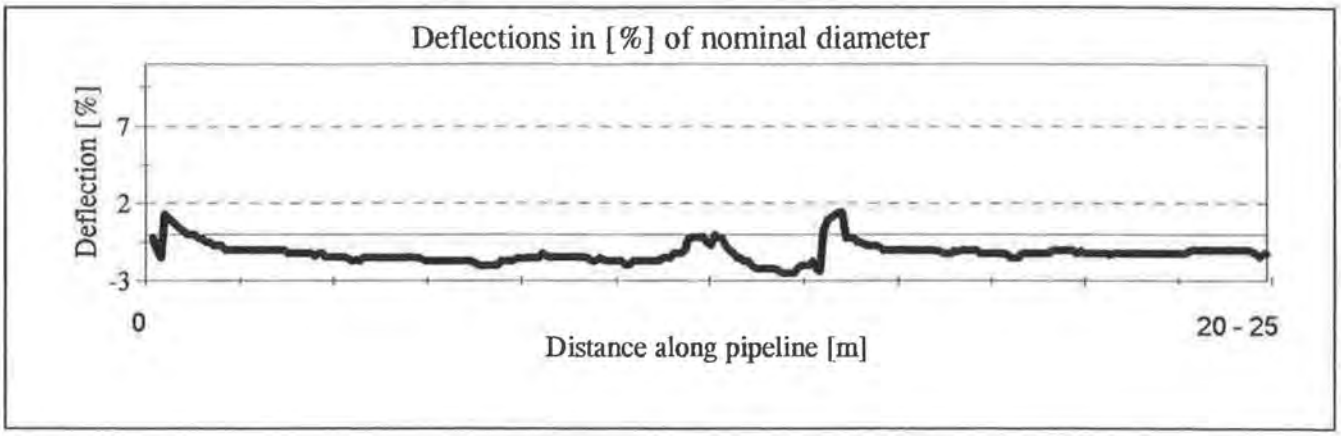


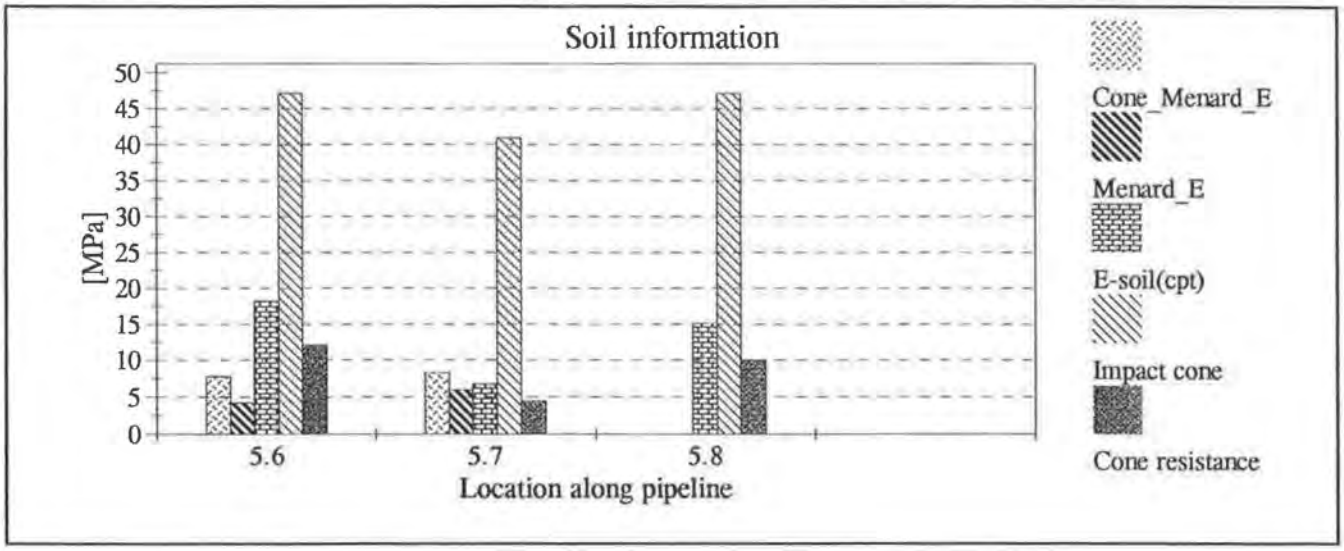


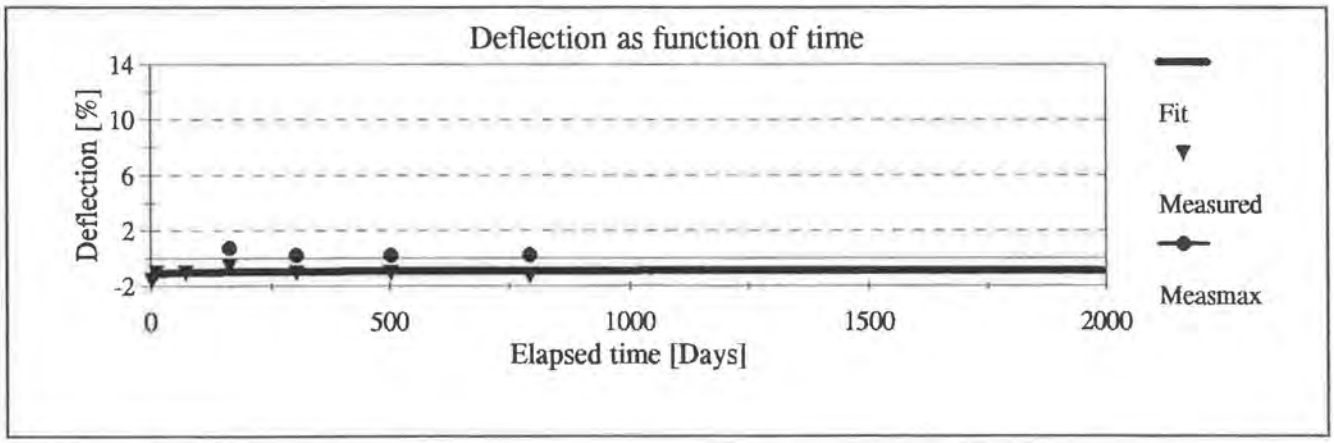


Information	Value
Case	Case 8
Pipe material	PVC
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,90

Result	Value
Initial average	1.0
Initial maximum	5.0
Final average	4.0
Final maximum	7.0
t_100[Days]	422.0

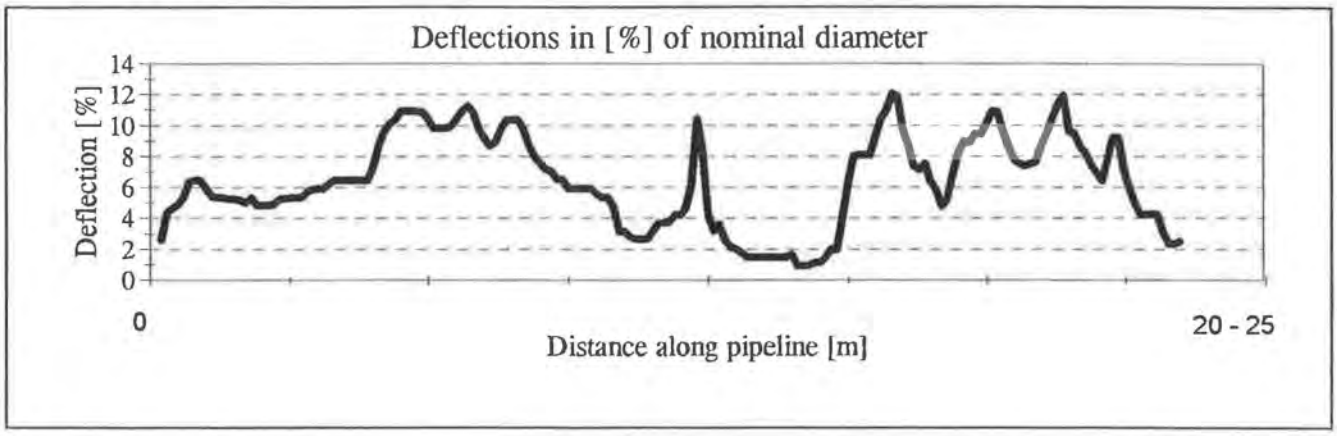


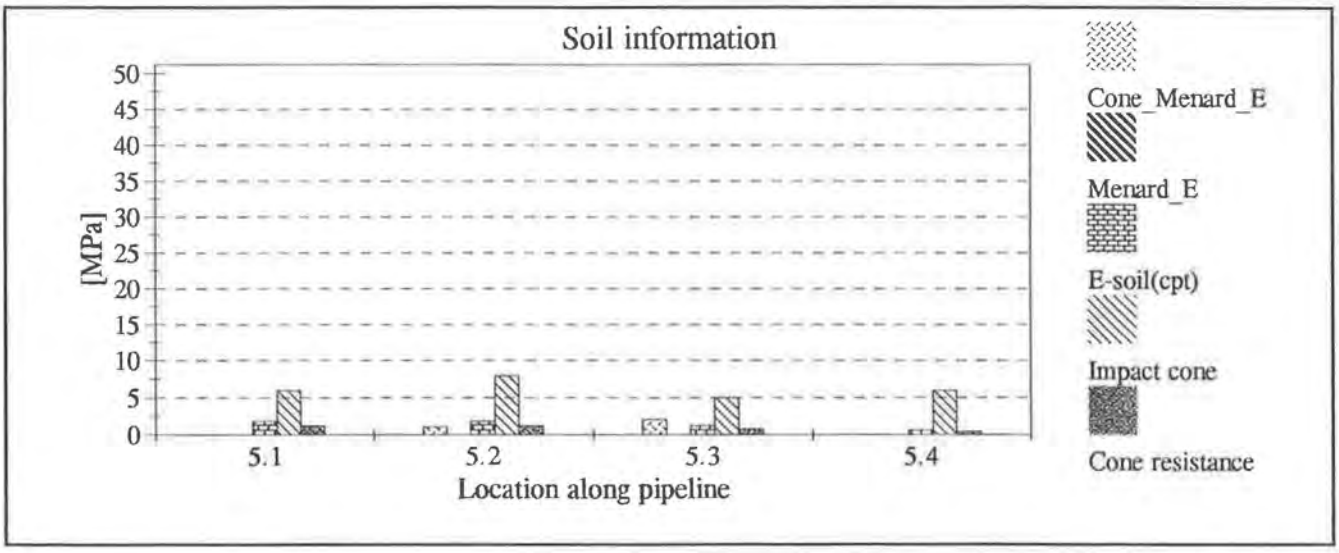


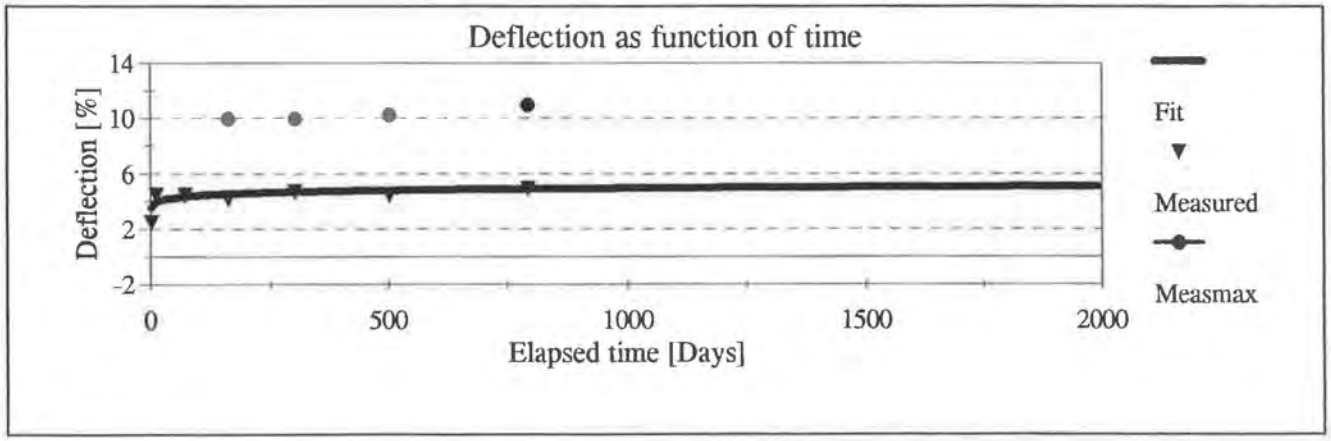


Information	Value
Case	Case 9
Pipe material	PVC
SN [kN/m3]	2,00
SN actual [kN/m3]	3,52
Embedment	Sand
Native soil	Sand
Installation	Well
Depth of cover [m]	1,90

Result	Value
Initial average	-1.5
Initial maximum	0.0
Final average	-0.8
Final maximum	1.0
t_100[Days]	0.0

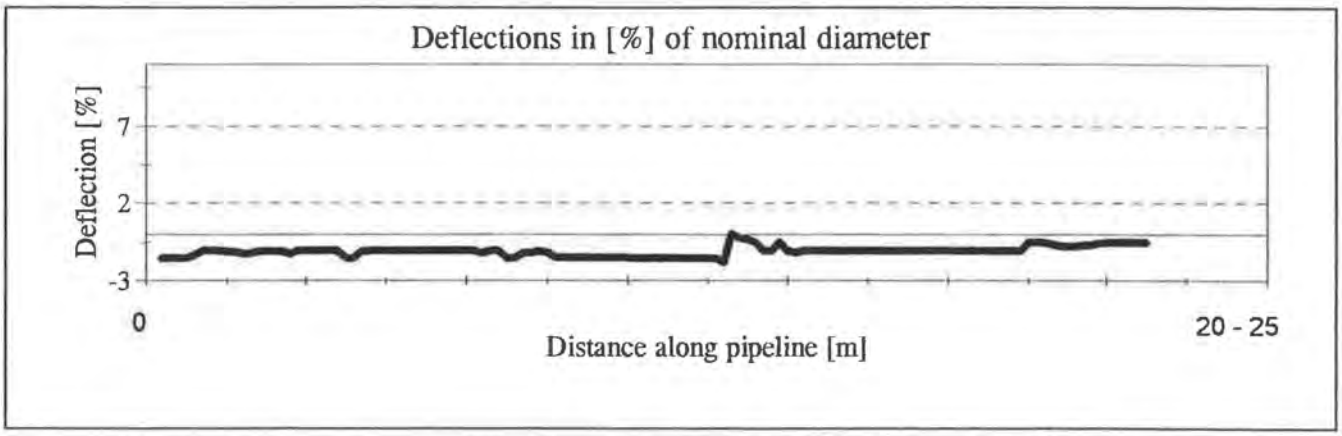


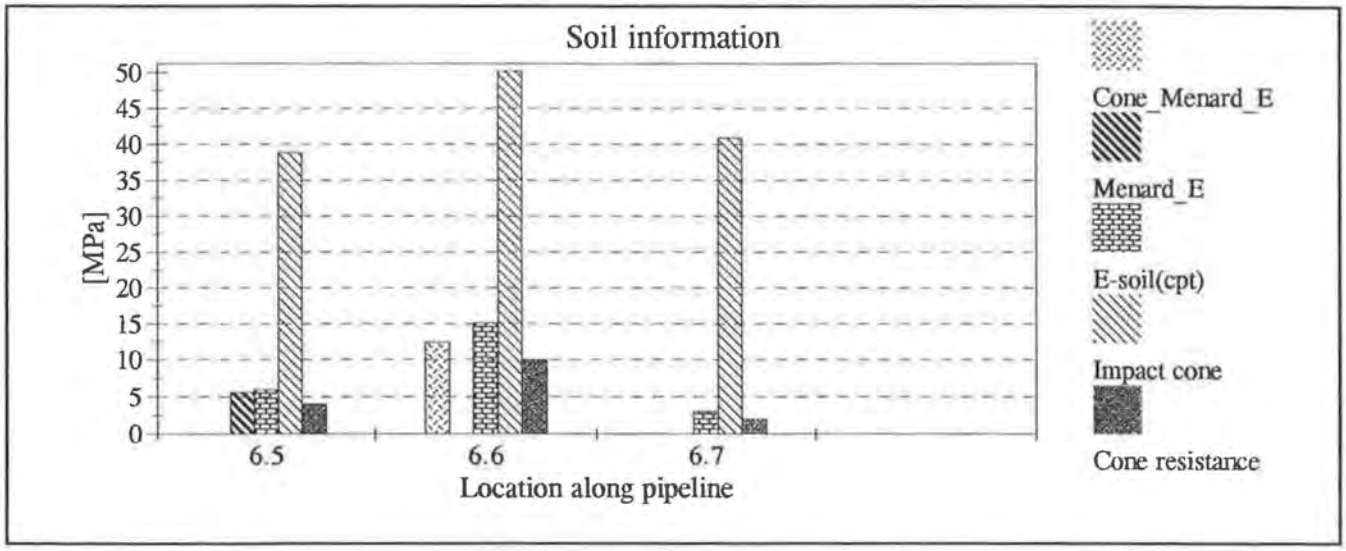


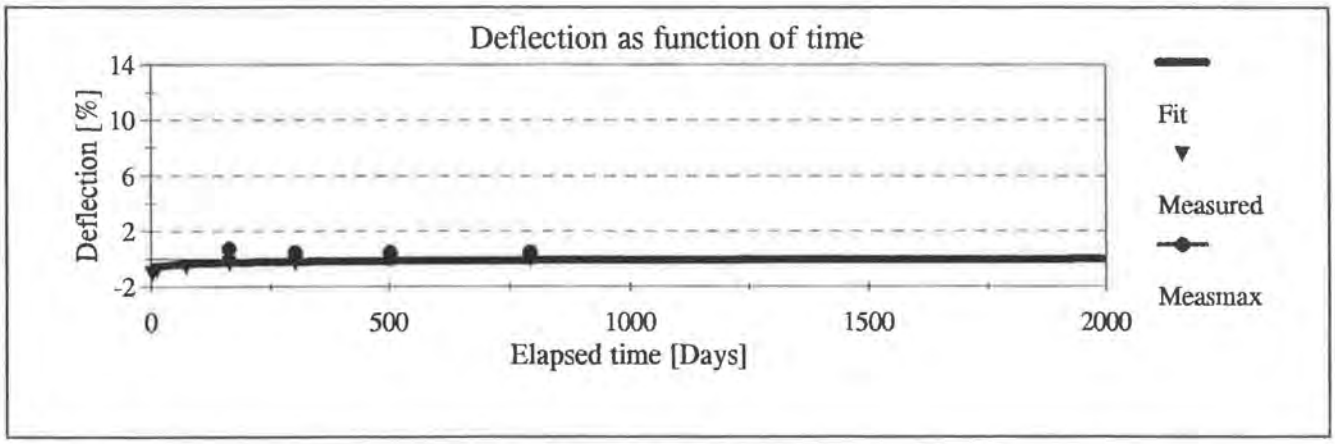


Information	Value
Case	Case 10
Pipe material	PVC
SN [kN/m3]	2,00
SN actual [kN/m3]	3,52
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,90

Result	Value
Initial average	3.0
Initial maximum	8.0
Final average	5.8
Final maximum	11.0
t_100[Days]	419.0

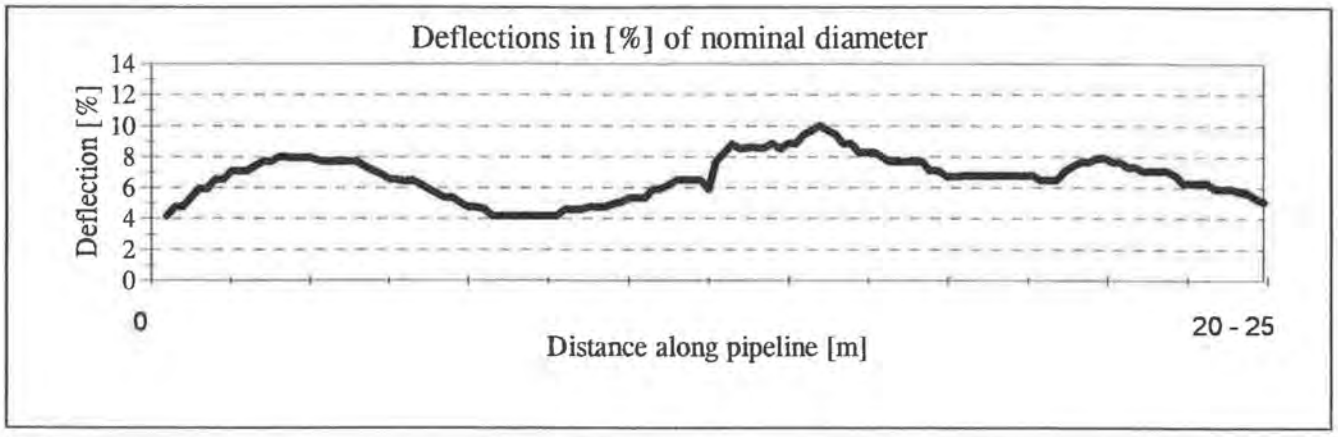


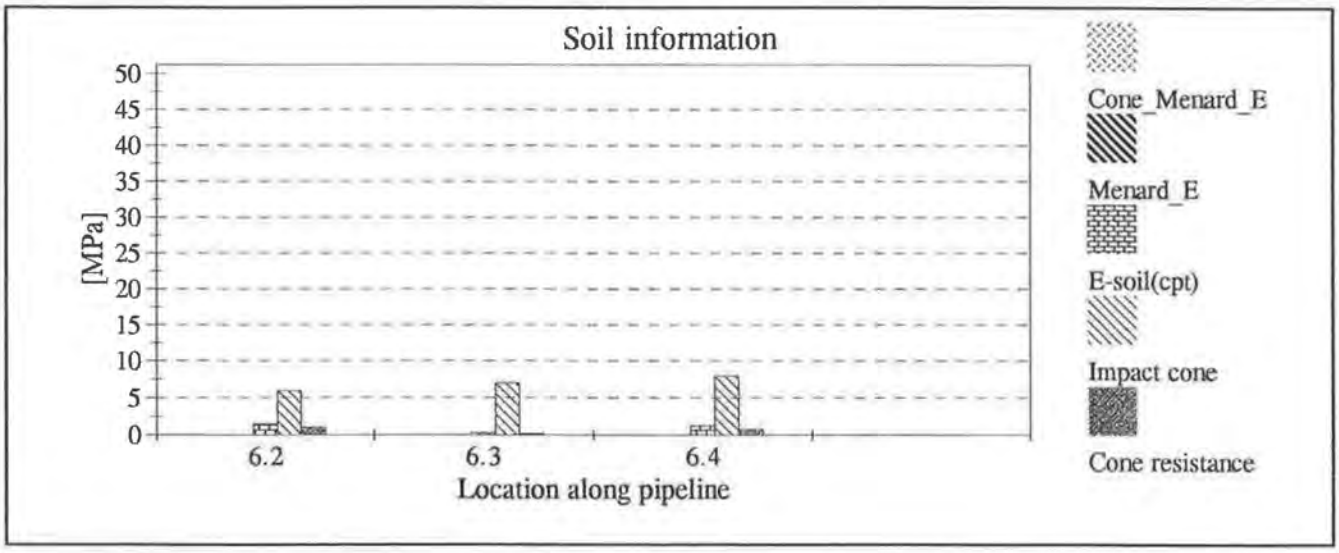


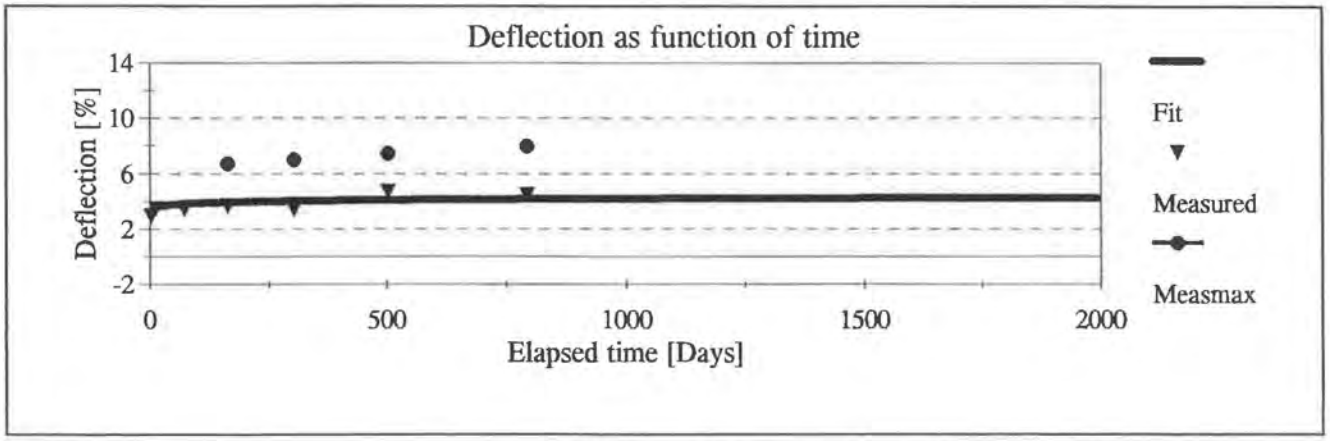


Information	Value
Case	Case 11
Pipe material	Steel
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	Well
Depth of cover [m]	1,90

Result	Value
Initial average	-1.0
Initial maximum	-0.5
Final average	0.0
Final maximum	0.5
t_100[Days]	0.0

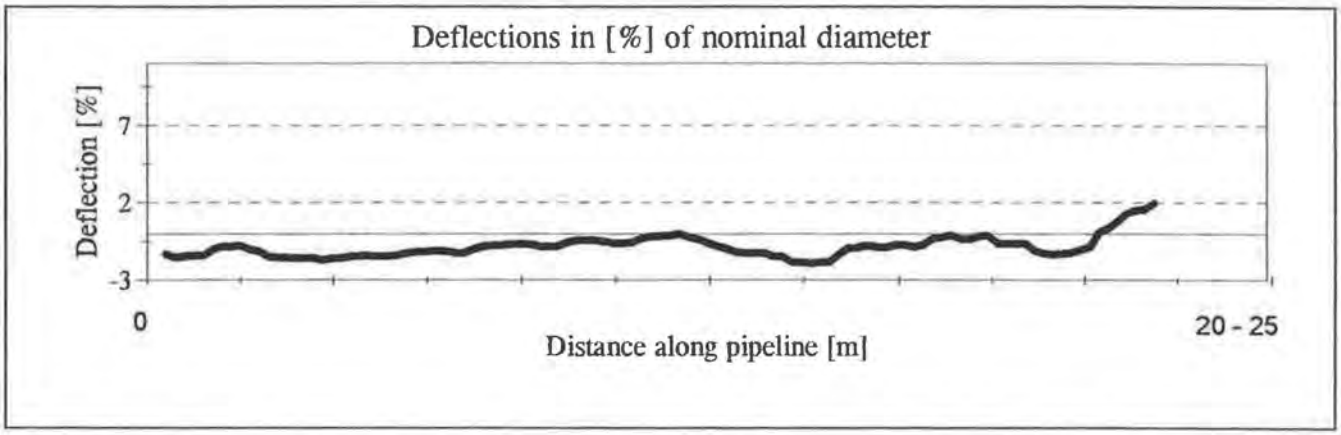


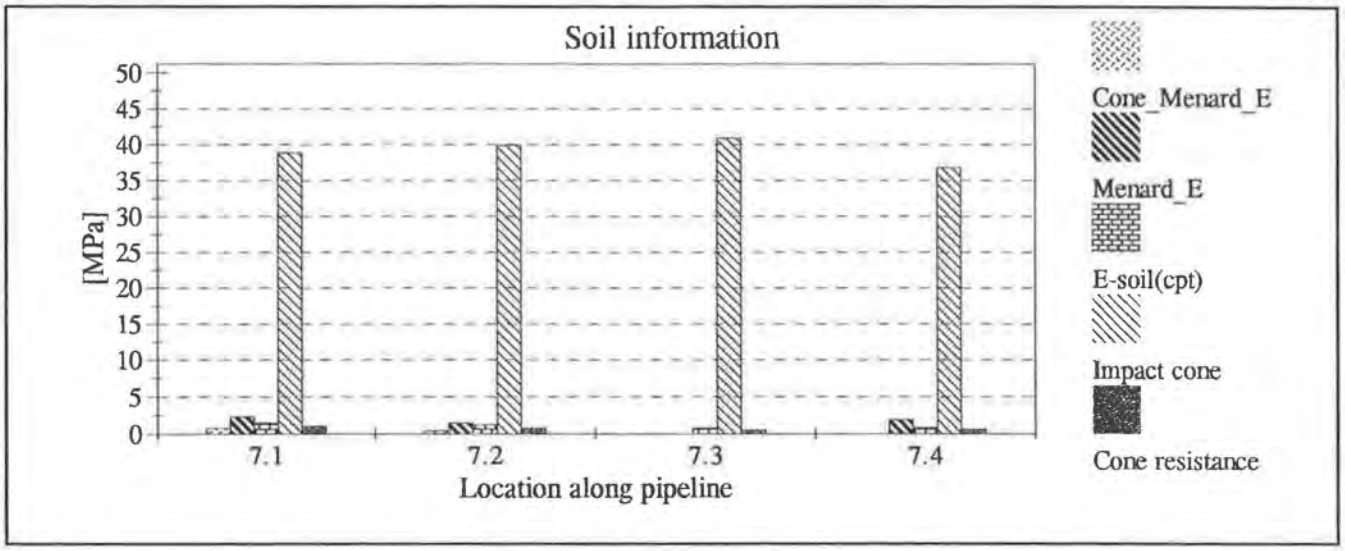


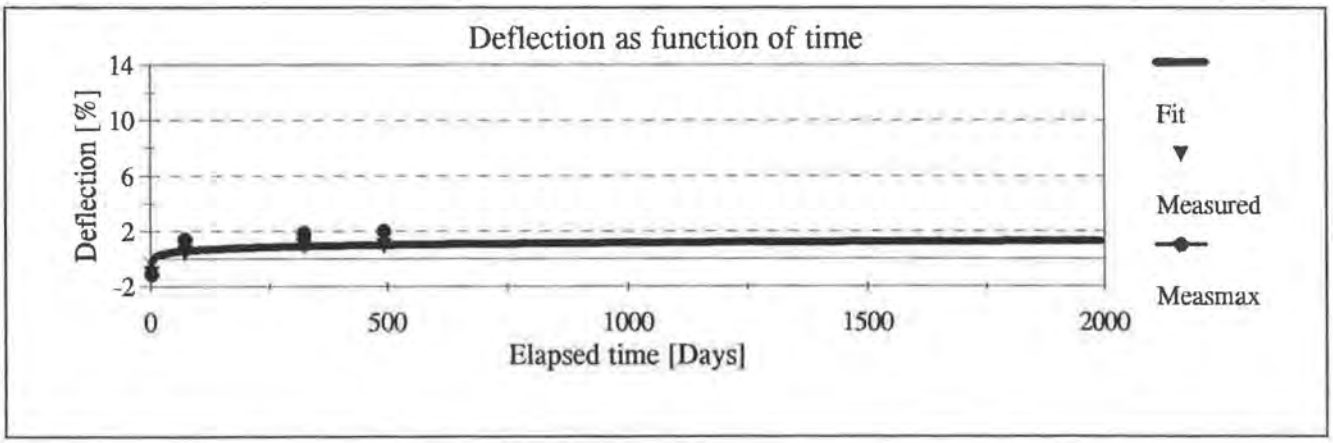


Information	Value
Case	Case 12
Pipe material	Steel
SN [kN/m3]	4,00
SN actual [kN/m3]	5,37
Embedment	Sand
Native soil	Sand
Installation	None
Depth of cover [m]	1,90

Result	Value
Initial average	2.8
Initial maximum	5.0
Final average	4.8
Final maximum	8.0
t_100[Days]	26.0

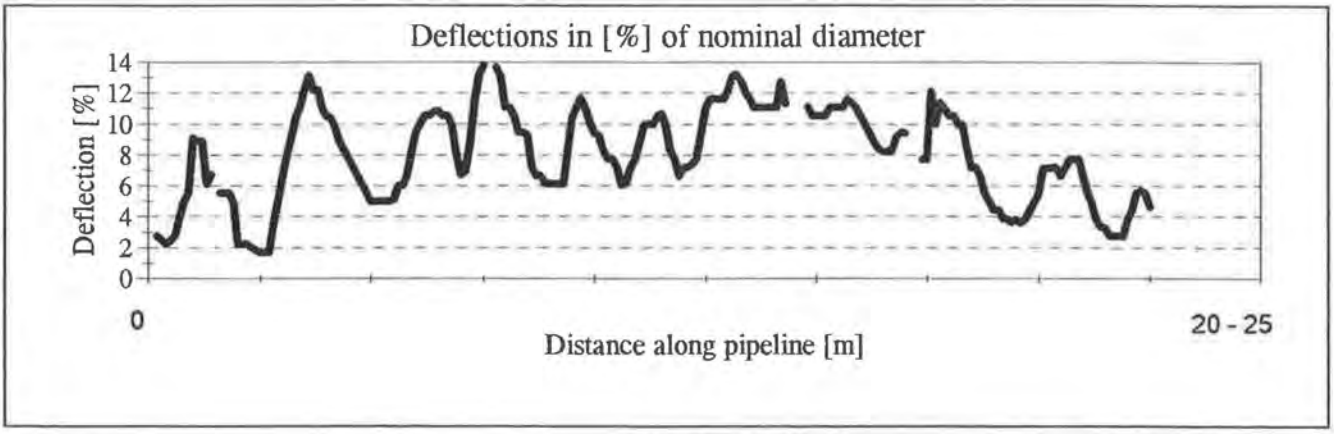


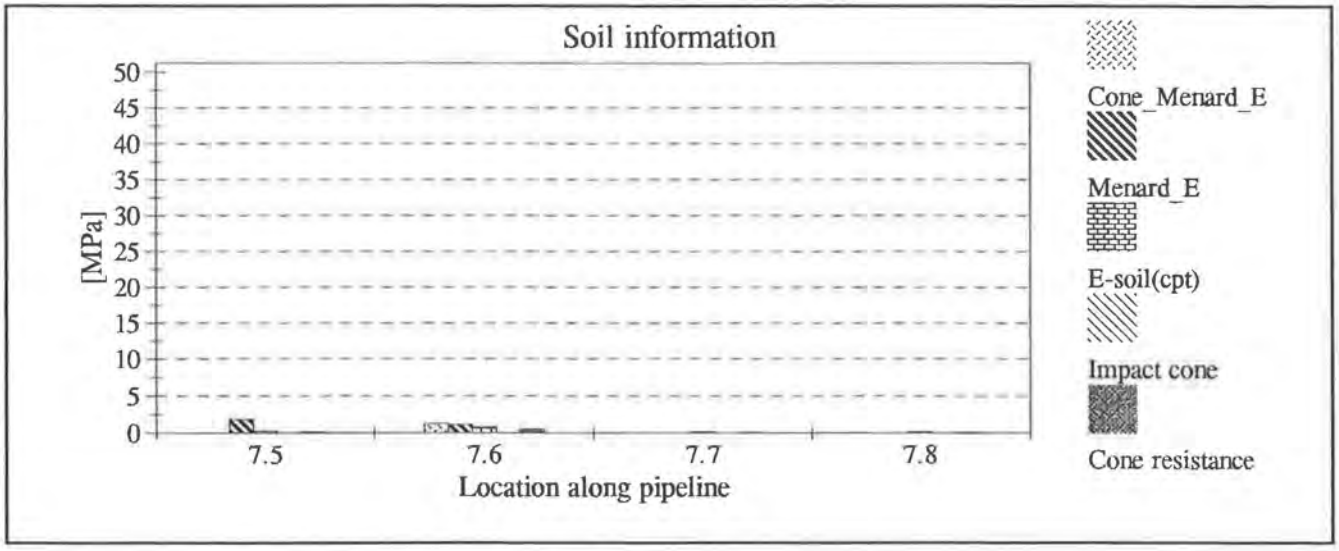


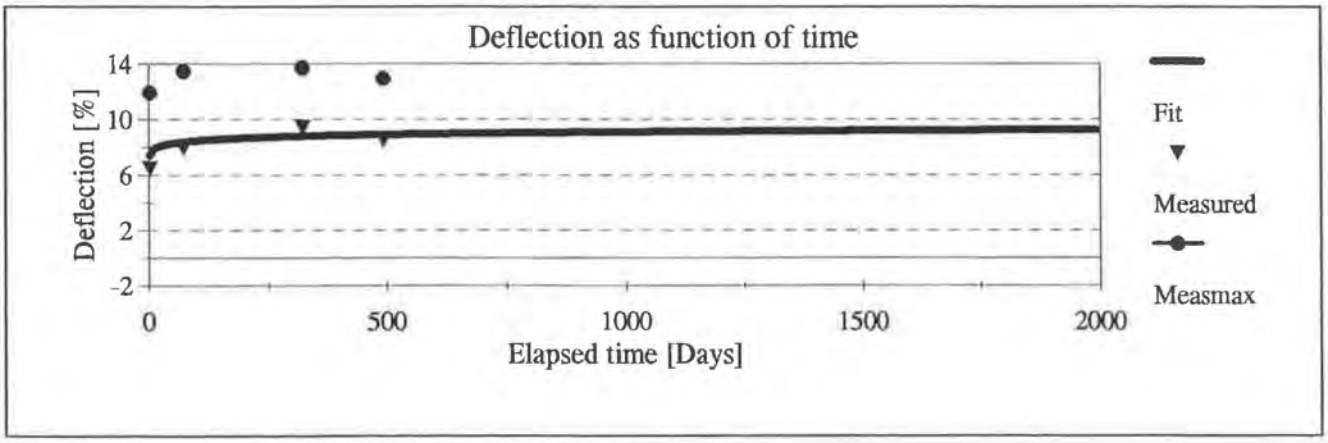


Information	Value
Case	Case 13
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Sand
Native soil	Clay
Installation	Well
Depth of cover [m]	1,15

Result	Value
Initial average	-1.0
Initial maximum	-0.5
Final average	2.0
Final maximum	3.5
t_100[Days]	388

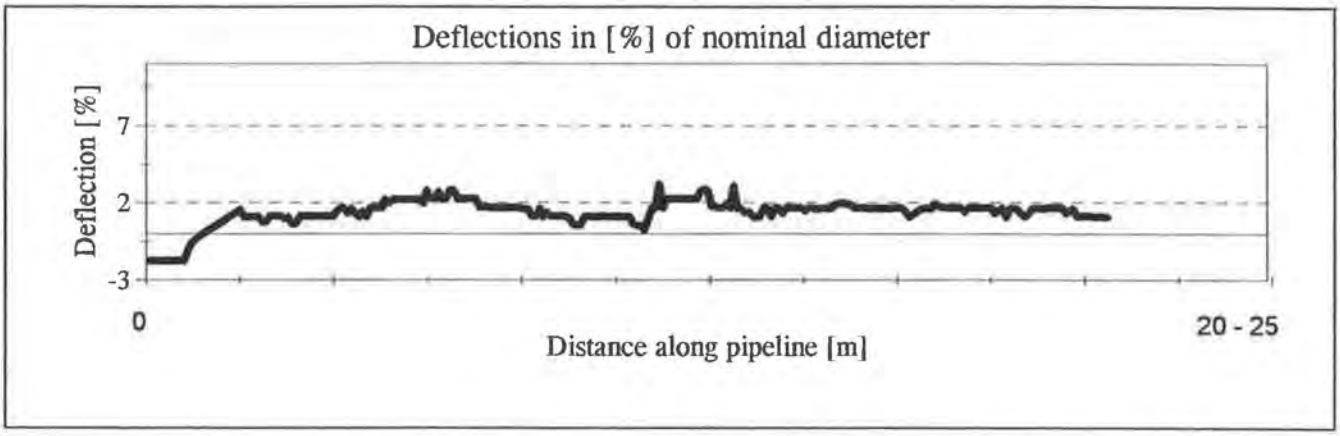


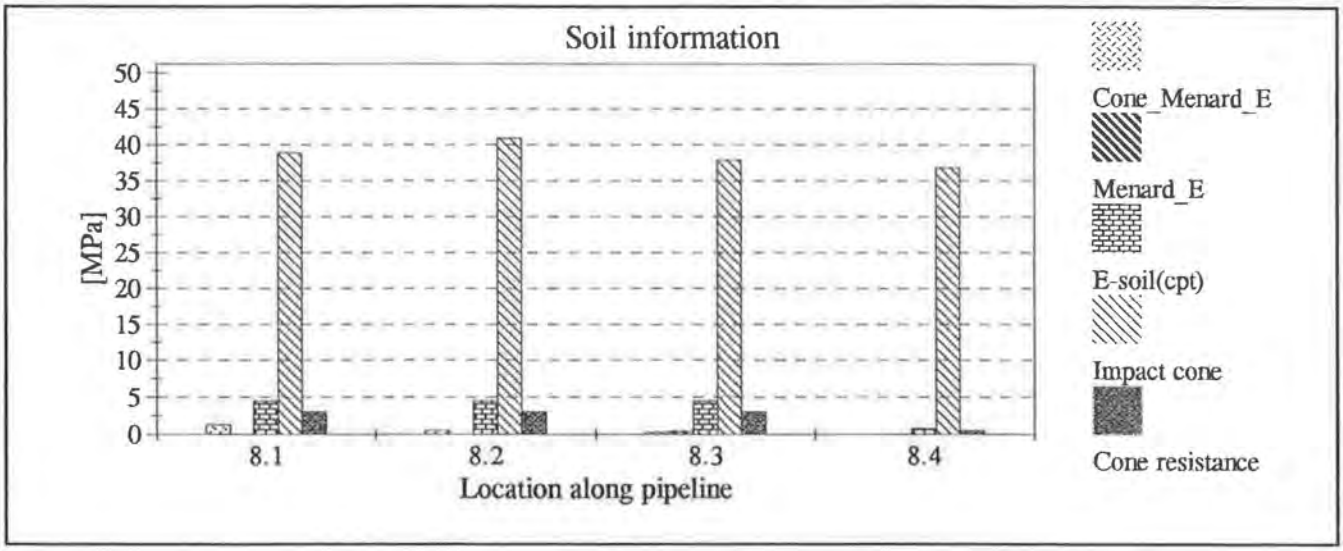


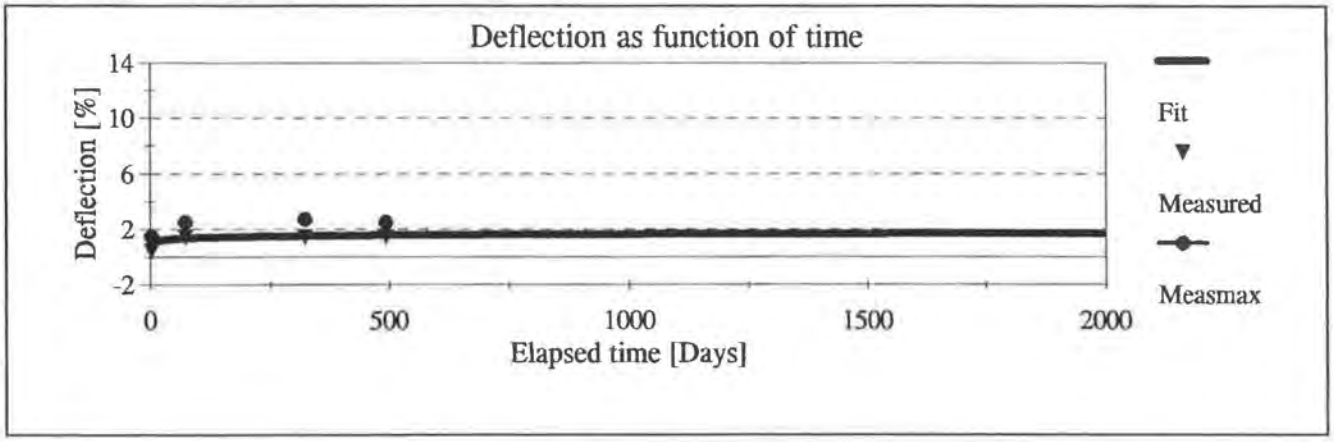


Information	Value
Case	Case 14
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Clay
Native soil	Clay
Installation	None
Depth of cover [m]	1,15

Result	Value
Initial average	6.5
Initial maximum	12.0
Final average	10.0
Final maximum	14.0
t_100[Days]	897

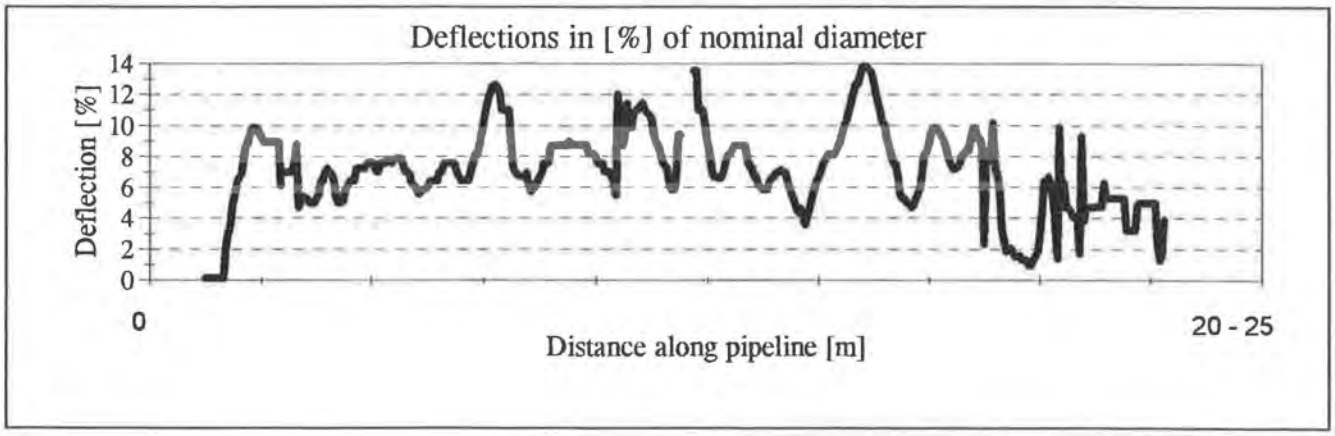


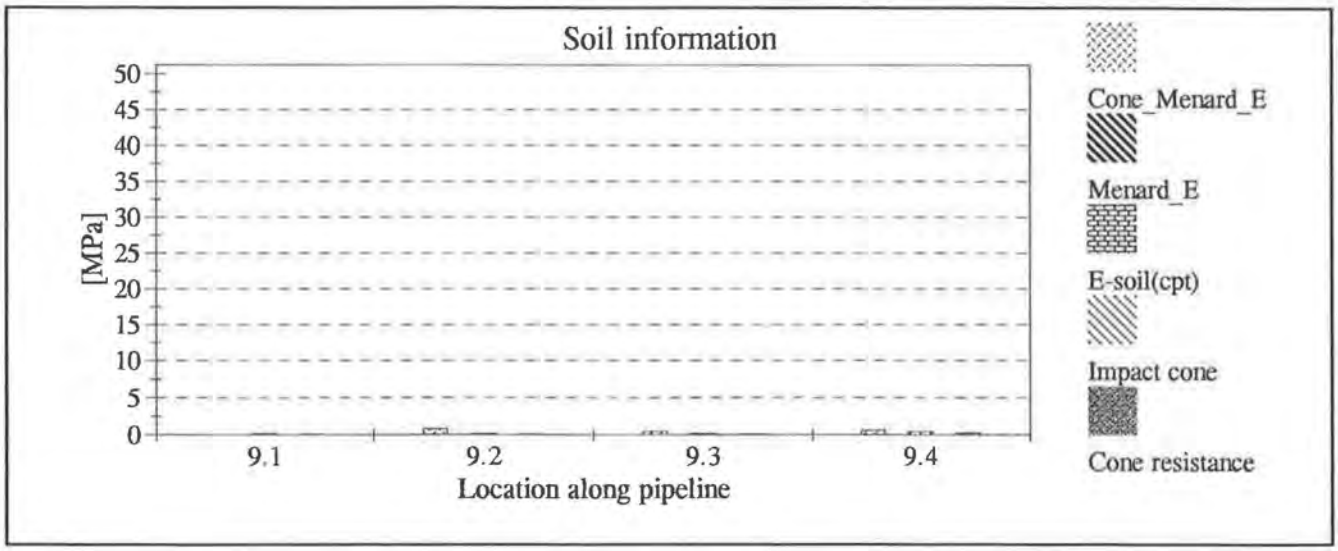


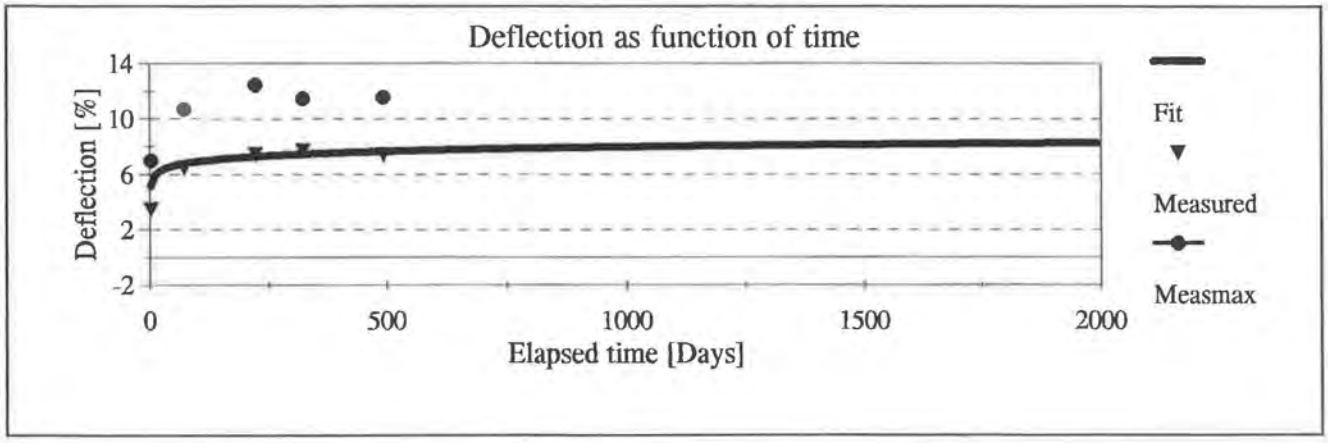


Information	Value
Case	Case 15
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Sand
Native soil	Clay
Installation	Well
Depth of cover [m]	2,80

Result	Value
Initial average	0.5
Initial maximum	1.0
Final average	2.0
Final maximum	3.5
t_100[Days]	5.0







Information	Value
Case	Case 16
Pipe material	PE
SN [kN/m3]	6,00
SN actual [kN/m3]	7,99
Embedment	Clay
Native soil	Clay
Installation	None
Depth of cover [m]	2,80

Result	Value
Initial average	3.5
Initial maximum	6.5
Final average	10.0
Final maximum	12.0
t_100[Days]	4,122.0

Enclosure 6.4:

Literature and Short summary of reference studies

Some of the references are also shortly discussed, as they provide a check of the current results with results of those who carried out comparable tests.

1. "*Plastics pipes for water supply and sewage disposal*", pp. 81-83, Borealis Handbook, 1995, Sweden, Lars Eric Janson.

2. *"Structural design of buried pipelines under various conditions of loading"*. EN1295

3. *"Static cone to compute static settlement over sand"*,Journal of the SOIL MECHANICS AND FOUNDATIONS DIVISION, May 1970, John Schmertmann.

4. *"Effect of installation conditions on buried plastics pipes: Results of specific field trials"* F.J.M Alferink and M.Wolters, Plastics Pipes 8, September 1992, Eindhoven, NL

In this paper the results from specific field trials carried out in sand, clay, gravel and peat are summarised. Next to this an analysis has been carried out into the relationship between average and maximum pipe deflection. It was shown that maximum deflections can be estimated using a factor of 1.5 to 2 to the average deflection. The factor of 2 is rather conservative. Also the effect of traffic is discussed, and measurements showed that the only effect traffic had on pipe deflection was that it speeds up the consolidation of the soil, but does not effect the pipe deflection value in the end .
The traffic loading tests are carried out directly on the soil without the effect of load spreading by pavement.

5. "*The actual performance of buried plastics pipes in Europe over 25 years.*" W.J. Elzink and J. Molin, Plastics Pipes 8, September 1992, Eindhoven, NL

This paper summarises the work carried out by Wavin, Komo and by VBB over a period of more than 40 years. It comprises over 50 km of measurements carried out in 8 European countries. The results are gained from measurements carried out on operational sewers using the Wavin sledge, the Lancier equipment or the Soini device. Most of the pipes have been measured twice, in most cases during or shortly after installation and after a time period varying between 3 months to 10 years. The study can be considered as giving the best general overview about how the European plastics sewers are performing under various conditions of loading. The data are normally sometimes not detailed enough to allow the judgement of the performance of very detailed design methods. For instance the installation is classified in three classes, whereas some design methods utilise much more detail by involving a proctor density value for the embedment as well as for the bedding and other areas in the neighbourhood of the pipe.

6. *"A study of an experimental uPVC pipeline laid beneath a major road during and after construction"*. Proc. Int. Plastics Pipes Symposium, Southampton, Sept. 1974 J.J. Trott and J. Gaunt

7. *"Some experience with 30 years old buried uPVC pipes from the viewpoint of stress and strain."*
F.J.M Alferink, Buried Plastics Pipes Technology, Dallas, September 1990, Dallas USA

Results are shown for operational gas pipes 110, 160 and 200 mm, ageing between 12 and 25 years with some excesses till 38 years. These PVC gas pipes are operated at pressures less than 200 mBar and can therefore be considered as pressureless. The depths these pipes were buried at varied between 30 cm in combination with heavy traffic !, upto 1.50 meter when buried in verges and at the countryside. The majority of the pipes were buried between 0.6 and 1.10 metre. The measurements provided deflection values as well as strain values in the buried pipes. The deflection measurements are however carried out in an incremental way, meaning that in each 10 meter length of pipe, 10 cross sections are analysed.

8. *"Load deflection field tests of 675 mm PVC pipe"*
A.K. Howard, Buried Plastics Pipes Technology, Dallas, September 1990, Dallas USA

Howard buried PVC pipes with a ring stiffness of 2.7 kPa and with a diameter of about 700 mm, each having a length of 6 metre. He utilised three installation conditions varying from dumped over 85 to 95 % compaction. The deflection measurements were of the discontinuous type. At every 6 metre 4 cross sections were measured. The test set up and measuring procedure does not allow the development of a normal installation procedure, and limits strongly the estimation of actual variability. He also shows the deflection values during backfilling related to depth of cover reached at a certain stage. He proposes an accuracy of the measurement of 0.25 mm, equals 0.03 % deflection. This is a quite tough claim since it seems to be difficult to measure that accurately with a micrometer, when laying in a 660 mm inside diameter pipe. Nevertheless the measurements at the dumped section shows that when tripling the depth of cover, also the deflection is multiplied to almost the same extend. The remark that shall be made here is that the pipes are buried in an almost fully developed 'Complete embankment condition', since the trench width is roughly 6 times the pipe diameter and next to this a moderate sloop for the trench wall was used.

9. *"Vertical deflection of buried flexible pipes"*
M.E Greenwood, Buried Plastics Pipes Technology, Dallas, September 1990, Dallas USA.

In this research work, results of field measurements are given. Low as well as high stiffness pipes are used, at depths varying between 1, 2 and 3 metre. The pipe diameters varied from 600 mm to 2 metres. All pipes used are fibreglass pipes. The author suggests dealing with variability in the same way as proposed by Molin, meaning that deflection values shall be added to the calculated one. The study does not tell which measurement method has been utilised, but the impressions is given that discontinue measurements have been performed.

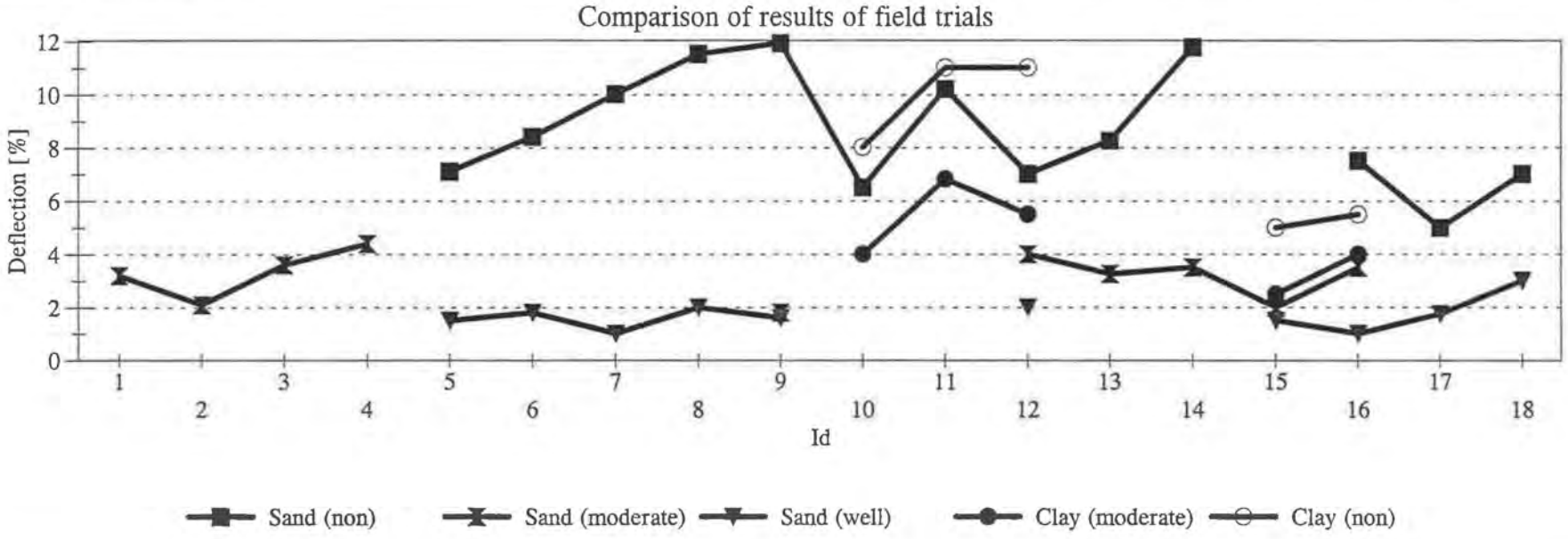
10. *"Handbook of PVC pipe"*
Design and construction. Unibell

Chapter 7

The most significant results shown there is the level of deflection and the effect of depth on pipe deflection. In the test sites of TEPPFA / APME it was shown that the effect of depth is negligible, whereas the American study shows that depth is really an issue. The TEPPFA / APME study showed that depth is not an issue and is completely overruled by the installation.
The results shown in the study are gained from tests on short pipe lengths. The minimum pipe stiffness was 5.9 kPa. As has been stated before, the installation of a short pipe in a sand box does not allow a normal installation sequence. The box width is about 1.80 meter, in which pipes up to a diameter of 600 mm can be tested. The sides of the box are stiff, by this the pipe soil system responds rather stiff in comparison to real life installations. Therefore the deflections found can be expected to be lower than in real life.
The effect of depth is simulated by increasing the load on the soil envelope. The soil envelope is limited to a depth of soil cover of 75 cm. By placing a plate connected to hydraulic activators, one can increase the load on the soil envelope, and as such monitor the effect on pipe deflection. The researchers at Utah state University claim that by following this approach they can in very short term tests create long term deflection values. It is said that by this method the load reaches the pipe within a few hours whereas in the real life this can take years.

11. *"Stresses against underground structural cylinders"*
Kaare Höeg, Journal of the SOIL MECHANICS AND FOUNDATION DIVISION, 1968"

12. *"Visco-elastic Approach to modelling Performance of Buried Pipes"*
K.M Chua, R.L.Lytton, Journal of Transportation Engineering, Vol.115,No.3, May 1989.



Id	Reference	SN	Gravel (moderate)	Sand (non)	Sand (moderate)	Sand (well)	Clay (moderate)	Clay (non)
1	Greenwd, max	1,45			3,20			
2	Greenwd,av	1,45			2,10			
3	Greenwd, max	1,50			3,60			
4	Greenwd,max	1,50			4,40			
5	Greenwd, av	1,80		7,10		1,50		
6	Greenwd, max	1,80		8,40		1,80		
7	TEPPFA,av	2,00		10,00		1,00		
8	TEPPFA,max	2,00		11,50		2,00		
9	Howard, av	2,70		11,90	1,80	1,60		
10	Alferink, av	4,00		6,50			4,00	8,00
11	Alferink, max	4,00		10,20			6,80	11,00
12	Elzink,av	4,00	3,00	7,00	4,00	2,00	5,50	11,00
13	TEPPFA,av	4,00		8,25	3,25			
14	TEPPFA,max	4,00		11,75	3,50			
15	Alferink,av	8,00			2,00	1,50	2,50	5,00
16	Elzink, av	8,00	2,00	7,50	3,50	1,00	4,00	5,50
17	TEPPFA,av	8,00		5,00		1,75		
18	TEPPFA,max	8,00		7,00		3,00		

Enclosure 8.1:

An example of an analytical method

s	= d*pow((sn1*12/(1000*Ep)),0.33)		To calculate s from nominal stiffness
Sc	= Ep/12*pow((s/d),3)	(0)	input values, Ep,s,d
n	= 0.061*Es/(8*Sc)	(3)	input value Es
c1	= (8+5*n)/(5+5*n)	(2)	
ps	= c1*c2*w*h*10*1e-3	(1)	input values w, unit weight soil input values h, depth of cover ps in kPa ps is soil load
pt	= xval/(pow(h,yval))	(4)	input values x and y, factors pt is traffic load
p	= ps+pt		

"Columbus effect" =-0.01*K*gam*D/(EI/d3),

K = 0.7 (well), K = 0.5 (moderate), K = 0.3 (none)

ymin_soil	= k*ps*(0.083)/(8*Sn1+0.061*Es*1000)*100	(8)	Input value k, deflection lag
ymin_traffic	= k*pt*(0.083)/(8*Sn1+0.061*sf*Es*1000)*100	(8b)	effect of traffic, sf for accomodating to higher soil stiffness
ymin	= ymin_soil+ymin_traffic		
Yav_init	= ymin/k+Inf		Average initial deflection
Ymax_init	= ymin/k+Bf		Maximum initial deflection
Yav_fin	= Ymin+Inf		(9)Inf from table, =installation factor
Ymax_fin	= Yav_fin+Bf	(10)	Bf from table
eps_init	= df*Ymax_init*s/d	(7a)	df from table df.db
eps_fin	= df*Ymax_fin*s/d		
sig_init	= Ep*eps_init/100	(7b)	
sig_fin	= Ep*eps_fin*retfac/100 retfac from table, is to accommodate for relaxation		

For rigid pipes

m = 0.25*df_rigid*k2alva*p*d*(1-kh)*d (14)

Beddingfactor = 0.6366/(Df_rigid*k2alva*(1-kh)) + Install.fac

Enclosure 9.1

Some guidance to analytical approaches

Already a lot of analytical approaches exists, and several comments on their validity for designing flexible pipes and moreover Thermoplastics pipes have been questioned. As a matter of fact they are able to calculate the deflection reasonable well in case of "Well" type of installations. However when one looks at the physics of the soil pipe system, and especially of that of the soil, then it is obvious that it is more a coincidence that analytical methods cover the measured results, as long as constant soil properties over the whole construction process are used. One is referred to the discussions in section 7.4.

Nevertheless, improvements on analytical methods are possible and some concepts are presented in this enclosure.

The following essential requirements have become clear during the research work:

1. The method needs to discriminate between the three different phases in the deflection process.
2. The method needs to consider the change of soil properties during these phases.
3. The method shall not involve a high level of mathematical refinement, as by experience it was shown that this refinement is by far overruled by effects during construction and as such results in a fake confidence at the customers place.

In the following these aspects will be worked out bit by bit.

The installation phase

Three possibilities to deal with the initial installation effect are identified:

A : Emperical values. Several studies provide data on it (9)
B : Use the Hoeg's model and use low soil stiffness, low load and a switch between qv and qh
C : Use the formula for ring loading, but now without vertical load, as follows :

$(\delta/D) = 0.02\ (0.3-K)\ \gamma_n\ D/SN,$

K depends on installation, as follows :
Well ->0.7, Moderate ->0.5, None -> 0.
γ_n is a unit load per m.

For the further installation, meaning the pipe is completely embedded by soil, the following options are identified:

D : $\Delta(Psoil) / (16SN + 0.122\ (Ess_i\ (Psoil / \sigma o)^n)$
In this equation the load is brought up stepwise, and the appropriate soil modulus (remember it changes during the process) is applied.
The effect of the in situ soil stress on stiffness is taken into account by $(Psoil / \sigma o)^n$

E: The same procedure but now applied to the Hoeg or any other model.
Next to this the installation factor shall be added to this deflection when one is interested in the maximum value.

The settlement phase

For the settlement phase the use of a deflection lag add on factors is most adequate. The values proposed in table 8.2 are conservative, but can be used with confidence.

The after settlement phase

This is the most simple phase, and can be set to zero.

The above is valid for flexible pipes. However when one would utilise the Hoeg model, than it would be valid for rigid pipes as well. One problem still exists for rigid pipes, which is how to involve the variability factors? It is recommended to consider the application of the variability factors as found for flexible pipes.
A further analysis for rigid pipes falls outside the scope of this report.

The total initial deflection can be calculated by a 3 steps formula as follows:
$(\delta/D) = 0.02\ (0.5\text{-}K)\ \gamma\ D/SN + Psoil\ /\ (16SN + 0.122\ Ess\ (Psoil\ /\ \sigma_o)^{n})$
$+ Plive\ /\ (16SN + 0.122\ Eu)$

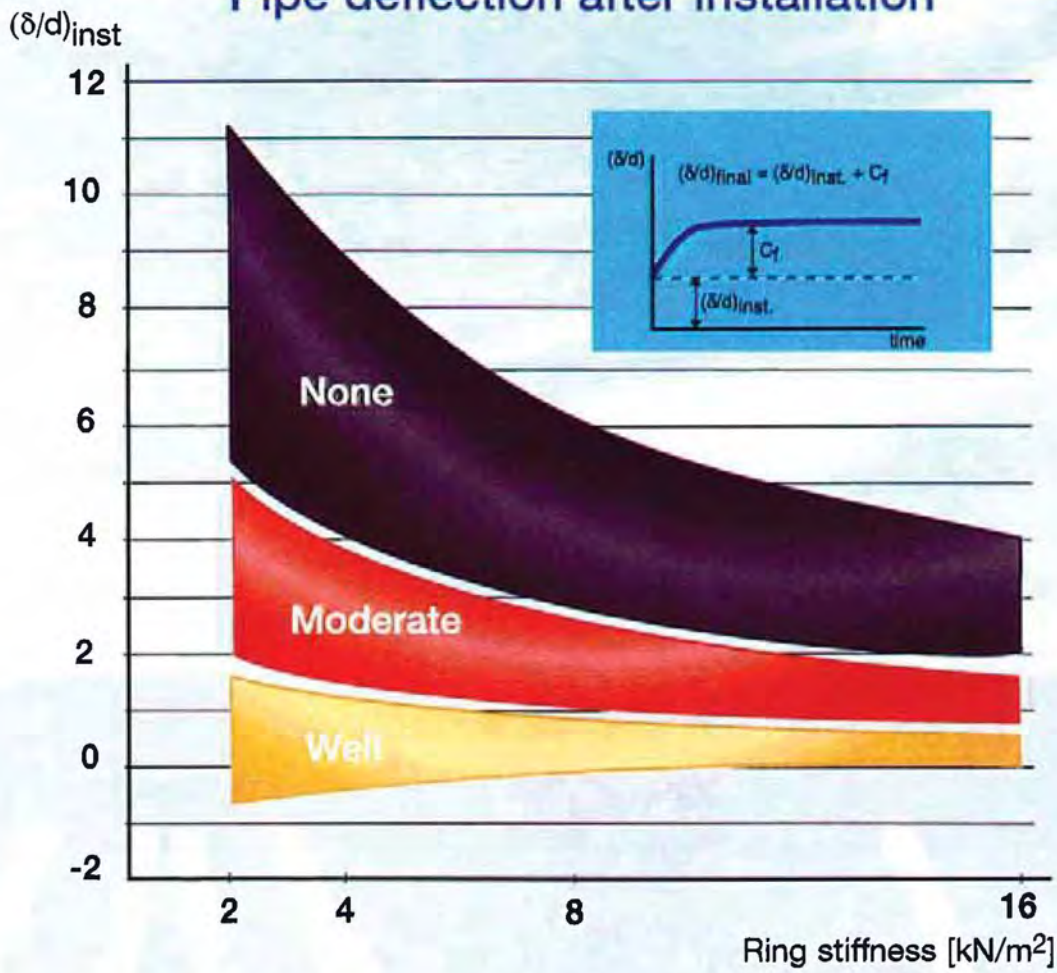
In which :

(δ/D)	= Pipe deflection [%]
K	= Load factor depending on installation (Well K=0.7, Moderate K=0.5, None
K	= 0.3)
	= Unit weight of the soil [kN/m3]
D	= Nominal pipe diameter [mm]
SN	= Pipe ring stiffness (EI/D3) [kPa]
Psoil	= Soil load [MPa]
Plive	= Live load [Mpa]
Ess	= Soil modulus valid for proctor density at installation [Mpa]
Eu	= Soil modulus valid for the ultimate density of the soil [Mpa]

The first two terms in the formula represent the achievement of the installation deflection, whereas the third term reflects the additional deflection that live loads applied after installation do cause.
Although this approach does not account for the continued change of soil modulus, it anyway discriminates between the installation phase, where the soil modulus increases from a low to higher value as soon as the soil becomes more consistent before the live load phase. This approach can be used by any of the current design methods.

Design Graph

Pipe deflection after installation



Note: ***The average deflections immediately after installation are represented by the lower boundary of each area, and the maximum values by the upper boundaries.***

Installation types and related consolidation deformation

"Well" compaction, $C_f = 1.0$

The embedment soil of a granular type is placed carefully in the hauncing zone and compacted, followed by placing the soil in shifts of maximum 30 cm, after which each layer is compacted carefully. The pipe shall at least be covered by a layer of 15 cm. The trench is further filled with soil of any type and compacted. Typical values of standard proctor are above 94%.

"Moderate" compaction, $C_f = 2.0$

The embedment soil of a granular type is placed in shifts of maximum 50 cm, after which each layer is compacted carefully. The pipe shall at least be covered by a layer of 15 cm. The trench is further filled with soil of any type and compacted. Typical values for the proctor density are in the range of 87-94%.

"None" compaction in granular soil, $C_f = 3.0$

The embedment soil of a granular type is added without compaction.

"None" compaction in clay, $C_f = 4.0$

The embedment soil of a cohesive type is added without compaction.

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The design graph is valid under the following conditions:

- Depths between 0.80 meter up to and including 6 meter.
- Designers first need to establish allowable deflections, average and maximum.
  (National requirements, product standards etc.)
- Pipes fulfil the requirements as listed in prEN 13476, prEN 12666, EN 1852 and EN 1401.
- Installation categories "well", "moderate" and "none" should reflect the level of workmanship on which the designer can rely upon.
- Sheet piles shall be removed before compaction, in accordance with the recommendations in EN 1610.
  If however the sheet piles are removed after compaction, one shall realize that the well or moderate compaction level will be reduced to the "None" compaction level.
- Pipes with diameters up to 1100 mm.
- Depth / diameter ratio at least above 2.0.
- Deflections are unlikely to be exceeded in practice for the circumstances described.
- For the deflection mentioned in the graph, the strain will be far below the design limit, and does not need to be given attention to in the design.