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Estimation of pipe friction from shear box tests with different loading control

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Los sistemas de tuberías para propósitos especiales, tales como cañerías de calefacción o cañerías a presión, se encuentran normalmente enterradas y por lo tanto se encuentran expuestas a interacciones entre la tubería y el suelo. Debido a estados cíclicos de temperatura y presión, las tuberías enterradas pueden moverse, lo cual puede resultar en una redución de la fricción entre el suelo y la pared exterior de la tubería. Se realizaron ensayos con una caja de corte para investigar las principales influencias que afectan la degradación de la fricción. Diferentes tipos de solicitaciones fueron empleadas para obtener simulaciones realistas del comportamiento del suelo que rodea a la tubería. En ensayos de carga normal constante y monotónica el coeficiente de fricción fue aproximadamente constante. Por otro lado ensayos cíclicos de rigidez normal constante mostraron una dependencia significativa entre la degradación de la fricción, el número de ciclos, la rigidez del suelo circundante y el nivel de tensiones iniciales. Se incluyen en el artículo de los resultados experimentales una descripción del equipo de ensayo y la determinación de la rigidez del resorte normal para los ensayos de rigidez normal constante.

Palabras claves: ensayos de corte, interfaz polietileno - arena, solicitación cíclica, rigidez normal constante

Pipe systems for special purpose, such as district heating pipelines or pressure pipelines, are often buried in the ground and subsequently exposed to a significant pipe – soil interaction. Due to cyclic temperature and pressure states these pipes shift in the ground, which lead to a reduction of skin friction between pipe and soil. Shear box tests were carried out to investigate the main influences that affect the friction degradation. Different loading concepts were used to obtain a realistic simulation of the surrounding soil behaviour. In monotonic constant normal load tests the friction coefficient was approximately constant. Cyclic constant normal stiffness tests showed a significant dependency among the friction degradation, the number of cycles, the stiffness of the surrounding soil and the initial stress level. The experimental set up, the determination of the normal spring stiffness for constant normal stiffness tests and tests results are presented.

Keywords: shear tests, sand polyethylene interface, cyclic loading, constant normal stiffness (CNS)

INTRODUCTION

For an economic and safe design of buried pipe systems the earth pressure acting on the pipes, the soil properties and the pipe characteristics are important parameters. District heating pipes are subjected to significant axial displacement due to changes in the operation temperature or pressure. Different internal pressures and the friction force between soil and pipe are decisive design parameters (European Standard EN 13941, 2003). The course of axial stress, the maximum displacement and the development of the gliding section depend on the axial friction force.The friction force F_R can be calculated by multiplying the resultant normal force N, which is the integral of the normal (radial) stresses σ_r acting along the outer pipe perimeter U, with a friction coefficient μ .

$$F_R = \mu \int_U \sigma_r \, dU = \mu N \qquad (1)$$

The maximum displacement $u_{\max}$ and the length of the gliding section l_0 are dependent on the friction force according to equations (2) and (3) (Achmus, 1995).

$$u_{\max} = \frac{F_R \, l_0^2}{2 \, E \, A_s} \qquad (2)$$

$$l_0 = \frac{E \, A_s \, \alpha_T \, \Delta T}{F_R} \qquad (3)$$

Where E is the Young's Modulus of the pipe, A_S is the area of the cross section, α_T is the coefficient of thermal expansion and ΔT is the temperature increase. According to the European Standard (EN 13941, 2003), the normal force is calculated depending on the overburden weight of the soil, the diameter, the pipe weight and an earth pressure coefficient.

From that, a constant friction force is obtained. However, in experimental tests it was proved that friction forces decrease with cyclic axial displacement (Weidlich 2008; Weidlich & Achmus 2008). This effect is considered in actual European design directives by halving the coefficient of friction μ between pipe and soil (EN 13941, 2003). In the investigation presented here modified shear box tests were carried out to investigate the pipe – soil interaction. In the shear box tests soil was sheared against the pipe coating material with constant velocity and the acting shear stresses were measured. For monotonic loading only one shear displacement in one direction was done.

For cyclic loading the direction of displacement was reversed after reaching the maximum displacement of about 5.5 mm. In standard shear box tests the normal load was kept constant (Constant Normal Load CNL). To obtain a realistic simulation of the surrounding soil behaviour in cyclic shear tests Constant Normal Stiffness tests (CNS) were carried out. These tests with a different loading control are explained here.

EXPERIMENTAL SET - UP

A standard shear box device for constant normal load tests was modified for the investigation of the interface behaviour between soil and the pipe coating material (High Density Polyethylene = HDPE). The lower shear frame was replaced by a rigid support beam, which was displaced horizontally.

The polyethylene plates were fixed on this beam. Figure 1 shows schematically the experimental set-up for the monotonic constant normal load tests.

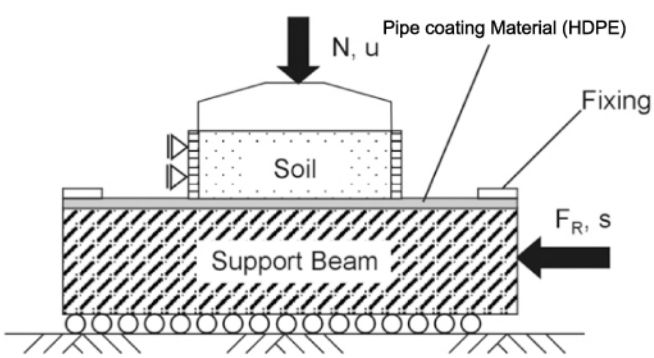

Figure 1: Scheme of the constant normal load friction tests

The upper shear frame was filled with a volume of sand of approximately 200 cm³. The contact area between the pipe coating material HDPE and the sand was 10 cm by 10 cm. In the testing device a constant shearing velocity of 2 mm/min was used. The acting forces were measured with an accuracy of 1/1000 kN and the displacement was measured with an accuracy of 1/1000 mm.

The sand used in the tests was a poorly graded quartz sand. Its relevant parameters are given in Table 1.The sand is a typical backfill material for pipeline trenches. The sand was poured in the shear box layer by layer. The required relative density (D_R = 0.7) was achieved by tamping.

Property	Unit	Value
Uniformity index U_i	-	2.21
Index of curvature C_c	-	1.0
Minimum porosity n_{min}	-	0.333
Maximum porosity n_{max}	-	0.442
Density of particles ρ_s	g/cm³	2.634
Minimum density ρ_{min}	g/cm³	1.757
Maximum density ρ_{max}	g/cm³	1.470

Table 1: Index properties of the sand used

For the tests, HDPE plates of 3.2 mm thickness were used, which have the same properties as the coating material of district heating pipes (Shore - D Hardness H_D = 61, unit weight γ_{PE} = 0.955 g/cm³).

EXPERIMENTS WITH CONSTANT NORMAL LOAD CNL

For the determination of the friction coefficient between interfaces and soil under monotonic loading interface shear boxes are common testing devices due to their know principles apart from other known principles (*e.g.* Kishida and Uesugi, 1987).

For the design of district heating pipes the friction coefficient is set to μ = 0.4 for monotonic loading according to EN 13941. However, interface tests with sand and construction materials have shown, that a dependency between the relative density D_R and the coefficient of friction is possible (Malkus, 2000). Because of that, a variation of the relative density in CNL tests with HDPE plates and soil were carried out. For typical overburden heights of pipes different normal loads were studied. In fact, a slight dependency between the relative density and the coefficient of friction could be observed. Three stress levels were studied. First a low stress level of 10, 20, 40 kN/m², after that a middle stress level of 20, 40, 80 kN/m² and finally a high stress level of 40, 80, 160 kN/m² was investigated. Figure 2 shows a dashed straight line, wich was obtained using a linear regression analysis.

$$\mu = 0.06\, D_R + 0.41 \qquad (4)$$

In figure 2 equation (4) obtained by a linear regression can be observed a slight tendency of increasing μ values with the increase in relative density of the sand for the whole stress field.

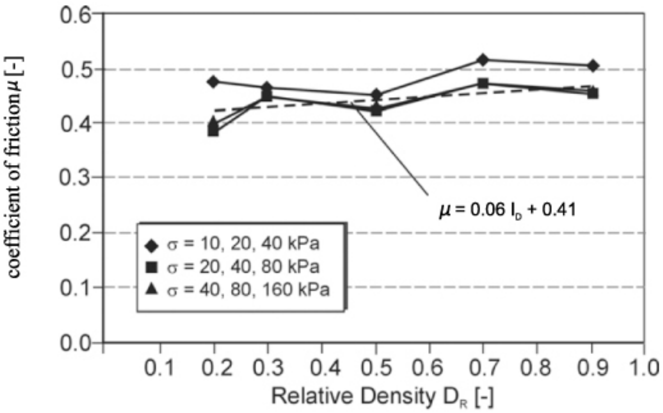

Figure 2: Coefficient of friction between HDPE and soil dependent on relative density

Consequently, for in situ conditions the most relevant relative densities are of middle dense (D_R = 0.4) to dense (D_R = 0.7) leading to a coefficient of friction between μ = 0.43 and 0.45 for the sand used and HDPE pipe. The difference is small, therefore, in case of adequate compaction of the back fill material a constant coefficient of friction μ = 0.4 according to EN 13941 is justified with sufficient accuracy. Other influences, such as temperature, hardness of the material and roughness of the surface stay approximately constant during pipe operation and play, concerning the design of district heating pipes, at least a minor role. In cyclic CNL tests no reduction of friction, which was the scope of this work, could be observed. A slight increase of friction forces with increasing number of cycles could be observed. That is why it was assumed that cyclic CNL tests do not lead to realistic results concerning buried and shifted pipes.

EXPERIMENTS WITH CONSTANT NORMAL STIFFNESS CNS

To reproduce more realisticaly the interface behaviour between pipe and soil more realistic Constant Normal Stiffness CNS shear box tests were carried out with the most relevant dense state of the sand. A CNS test can be seen as a refinement of the above described CNL test. CNS direct shear tests to study interface behavior between HDPE and sand have not been investigated. Previous studies in CNS shear test were mostly dedicated to the interface behaviour of cyclic axial loaded piles and surrounding soil (*e.g.* Boulon & Foray 1986; Airey *et al.* 1992; Tabucanon *et al.* 1995). According to Boulon & Foray (1986) the classical boundary condition of a constant normal load CNL cannot be assumed for the interface between cyclic axial displaced piles and soil. In the same way, the other extreme boundary assuming no volume change (constant volume) is not realistic, because it considers the mass of the surrounding soil to be radially perfect rigid. The reality is situated between the two extreme conditions (Boulon & Foray 1986).

This behaviour can be reproduced by a radial stiffness represented by a pressuremetric spring stiffness *K*, which assumes linear elastic soil behaviour. Dependent on the chosen spring stiffness, the initial normal load is being regulated by taking the volume changes of the soil sample into account according to equation (5). Dilatancy and contractancy of the soil can be simulated in a more realistic manner.

$$K = \frac{\Delta \sigma_N}{\Delta u} \quad (5)$$

In Figure 3 the scheme of a constant normal interface test is shown.

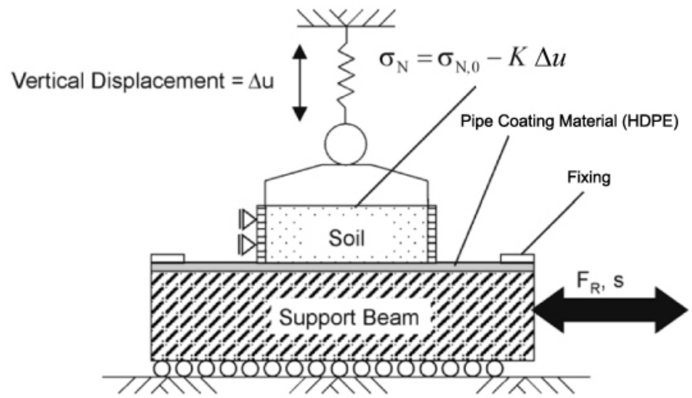

Figure 3: Scheme of the constant normal stiffness friction tests

The extreme boundary conditions of constant normal load CNL and constant volume can be reproduced by a constant stiffness of $K = 0$ (CNL) and $K \rightarrow \infty$ (CV).

The decisive spring stiffness for a certain situation $0 < K < \infty$ is dependent on the stiffness of the surrounding soil. Because of the geometric similarity between pipes and piles and the similar loading form (axial and cyclic), the described procedure can be applied for buried pipes under cyclic axial displacements too. It is to be assumed that for the study of the interface behaviour between shifted pipes and soil, CNS tests are even more suitable than for piles. This is because than of the constant overburden height of pipes, which leads to an almost constant radial contact pressure. However, the contact pressure between piles and soil increases with depth. For one pipe trench situation the boundary conditions can be reproduced by only one CNS test. The transferability of results from CNS tests to pile-soil interaction problems is considered however as permissible (Johnston *et al.* 1987).

ESTIMATION OF THE NORMAL SPRING STIFFNESS K

For carrying out CNS tests the normal spring stiffness *K* is to be defined. *K* represents the surrounding soil and depends on its stiffness.

For pipes the spring acts in a radial direction. According to the elasticity theory the spring stiffness can be calculated by a cylindrical expansion of a hole in a two dimensional disk model. This expansion represents the dilatancy of the soil in the interface zone between the buried shifted pipe and the soil. For the regulation of the incremental change in normal stress $\Delta\sigma_N$ according to elastic theory equation

$$\Delta\sigma_N = \frac{E}{1+\nu} \; \frac{\Delta r}{r} \tag{6}$$

The oedometric stiffness modulus E_S is related to the Young's modulus E by the Poisson's ratio ν. The normal spring stiffness K results dependent on the oedometric spring stiffness E_S according to equation (7).

$$K = E_s \; \frac{1-\nu-2\,\nu^2}{1-\nu} \; \frac{1}{r} \tag{7}$$

In the same manner the spring stiffness K for pile - soil interaction is estimated assuming linear elastic soil behaviour (*e.g.* Airey *et al*. 1992; Tabucanon *et al*. 1995). However, it is well known that stress - strain behaviour of soils can be extremely non linear. Respecting non-linear behaviour the oedometric stiffness modulus E_S is stress dependent and for loading and unloading a different modulus has to be chosen.

To take into account non-linear soil behaviour Finite Element calculation with the program system Plaxis, version 8.2, were carried out to estimate a realistic range of the spring stiffness K for the situation of a buried pipe in a trench.

The actual boundary conditions of the situation (geometry of the pipe trench, the stress level and dense state of the back-fill material) were considered. The soil was modelled with the constitutive law "Hardening Soil", which takes into account a stress dependent and loading and unloading modulus (Vermeer and Schanz 1997).

Based on different numerical models a variation of pipe diameter D and relative overburden height H was done. The parameters necessary for the constitutive law of the used sand were determined in the laboratory at the Institute of Soil Mechanics, Foundation Engineering and Waterpower Engineering, Leibniz University of Hannover. For dilatancy and contractancy at the pipe-soil interface a defined radial change in diameter (contraction increment) was applied. From the incremental changes in the contact pressure a radial spring stiffness was derived. Figure 4 shows, as an example, results from a numerical model for the relative overburden height of H/D = 3, the diameter D = 140 mm (nominal width DN65) and a contraction increment of - 0.2%, which corresponds to a relative diameter expansion of Δu/D = 0.5 ‰.

The initial system is shown at the left, the deformed system is shown at the right, which is scaled with a scale factor of 100. As expected due to non-linear soil behaviour, in the numerical investigation a different spring stiffness $K = K_D$ for dilatancy than for contractancy $K = K_C$ was observed. Moreover, the geometry of the system and the related contact stress and displacements affect the values of K. In Figure 3 the values of K around the pipe perimeter are shown for radial expansion and contraction (H/D = 3, D = 140 mm, Δu/D = 0.5‰ and 2.5‰).

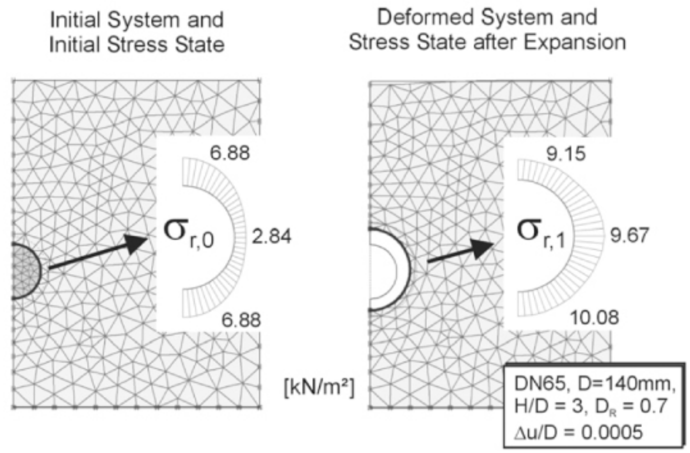

Figure 4: Stress changes due to radial expansion (scale factor of the deformed system: 100)

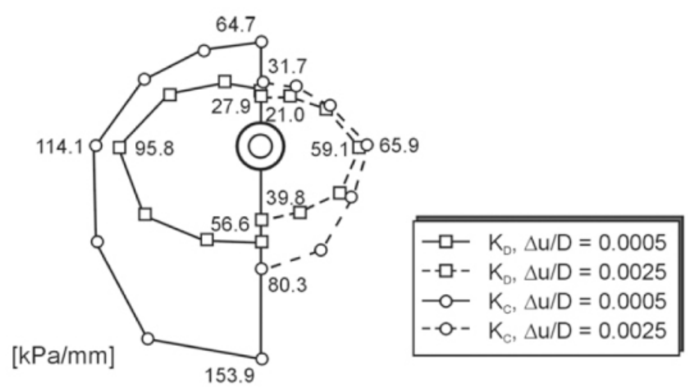

Figure 5: Values of K for radial expansion and radial contraction

The values of K obtained showed different variation around the pipe perimeter. An average K-value K_{avg} was defined according to equation (8). For all CNS tests an average value of K was chosen.

$$K = K_{avg} = \frac{\sum_{i=1}^{n} K_i}{n} \tag{8}$$

The values of average spring stiffness for expansion K_D (dilatancy), and contraction, K_C (contractancy) are summarized in Table 2 for different pipe trench situations. The variation of the diameter and the relative overburden height led to values of K between 50 and 150 kPa/mm. For the led relevant expansion the ratio of K_D / K_C was about 0.6 to 0.7.

	Δu/D	K_D [kPa/mm]	K_K [kPa/mm]	K_D/K_C[-]
	0.00050	66.5	105.1	0.63
DN 40. H/D=2	0.00125	52.0	74.9	0.69
	0.00250	36.3	53.0	0.69
	0.00050	69.0	111.7	0.62
DN 65. H/D=3	0.00125	58.1	85.6	0.68
	0.00250	44.8	60.9	0.73
	0.00050	78.4	134.8	0.58
DN 80. H/D=4	0.00125	67.7	105.3	0.64
	0.00250	53.6	74.1	0.72

Table 2: Average Normal Spring Stiffness K_{avg}

Since dense sands and interfaces in contact with dense sands show a general tendency to undergo a contractive behaviour under cyclic loading (Gudehus 1985, Porcino *et al.* 2003), the normal spring stiffness was set to K_C.

Note that the first reaction of dense granular soil subjected to shear is to dilate. However, for cyclic loading the soil behaviour changes from dilation to contraction. This occurs because the soil volume increases and the orientation of grains during the first "dilatant" shear changes. When the shear direction is inversed the previous grain orientation is modified. As a result, the shear band of the soil body is ready to contract. This leads to a contraction tendency of (initially) dense sands under cyclic loading.

EXPERIMENTAL RESULTS

In the CNS tests the testing device controlled the horizontal displacement and the back regulated normal load. An identical set up was chosen for the measurement of vertical displacements and acting shear forces and the installation method of the sand specimen for CNL and CNS tests. In the CNS tests the initial normal stress σ_0 was set to 10, 20 and 40 kPa respectively.

Each initial normal stress represents the average radial contact pressure on a buried pipe. The normal stiffness K was varied in the range between 20 up to 100 kPa/mm.

In general for dense sands under cyclic loading it was observed an increase of settlements, a decrease of shear stress and a constant ratio between normal load and shear force.

In Figure 6 a typical result for a CNS test of 10 cycles is shown.

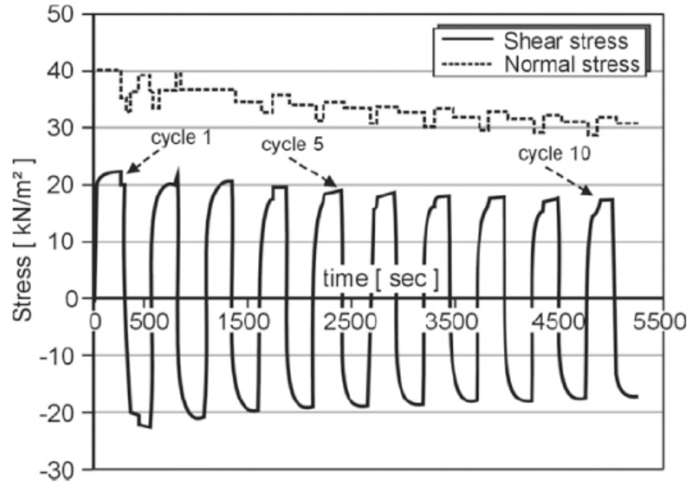

Figure 6: CNS test, σ_0 = 40 kPa, K = 60 kPa/mm, D_R = 0.7

During cycling a significant reduction of shear stress and normal stress was observed. A residual shear stress was reached after about 5 to 10 cycles. Concerning the required cycles for achieving a residual friction force Lee *et al.* (1987) obtained similar results with cyclic axial loaded model piles. Since the residual shear stress is reached after 10 cycles, as a measure for the friction and normal load degradation, following Poulos (1999) the degradation factor D_F was defined according to equation (9).

$$D_F = \frac{F_{10}}{F_0} = \frac{N_{10}}{N_0} \tag{9}$$

Due to contractive behaviour of the sand after change in shear direction a settlement is measured. As a consequence the back regulated normal load is reduced, which also leads to a reduced shear force. In Figure 7 the dominating contractive behaviour is evident for the whole test, hence the dominating settlement.

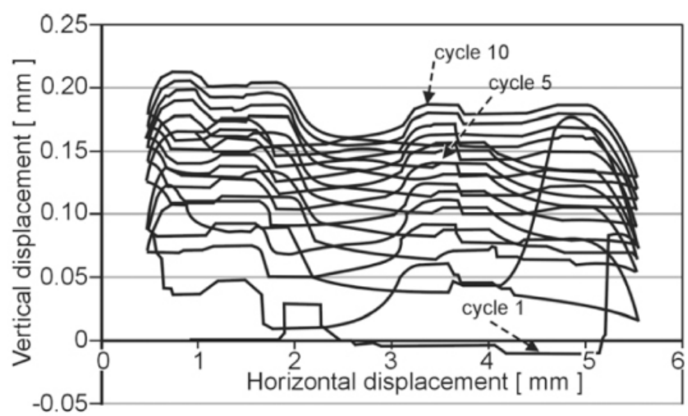

Figure 7: Vertical and horizontal displacement from a 10 cycles CNS test, σ_0 = 40 kPa, K = 60 kPa/mm, D_R = 0.7.

Evaluating the ratio between shear stress and normal stress for the 10 cycles of the test resulted in a constant coefficient of friction. The degradation of friction on a cyclic loaded buried pipe is therefore believed to occur due to a stress redistribution around the pipe, which is simulated here by changing the normal pressure, and not due to abrasion or particle crushing at the soil-pipe interface. The degradation factor D_F obtained in CNS tests after 10 cycles are shown in Figure 8.

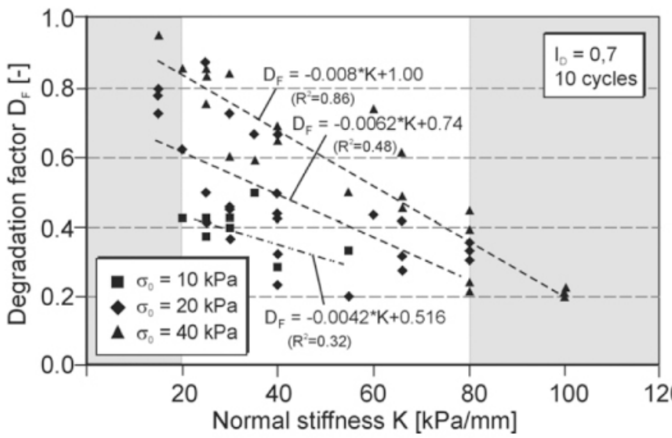

Figure 8: Degradation factor D_F from CNS tests

With increasing normal stiffness K an increasing degradation of friction was observed as a function of the initial normal stress σ_0 .Tendency lines for the relationship between degradation factors D_F and constant normal stiffness values K were estimated. Linear regression analyses lead to equations (10), (11) and (12), which are shown by dashed straight lines in Figure 8.

$$D_F(\sigma_0 = 10\ \text{kPa}) = -\ 0.0042\ K + 0.516 \qquad (10)$$

$$D_F(\sigma_0 = 20\ \text{kPa}) = -\ 0.0062\ K + 0.74 \qquad (11)$$

$$D_F(\sigma_0 = 40\ \text{kPa}) = -\ 0.0080\ K + 1.00 \qquad (12)$$

The equations are valid for the studied range of normal constant stiffness of K = 20 to 80 kPa/mm. For different boundary conditions further experimental work and statistical evaluation is needed.

CONCLUSIONS

This investigation showed that the applicability of CNS tests is not limited only to pile-soil interface problems. It is believed that for the study of the interface behaviour between shifted and buried pipes and soil CNS tests are even more suitable than for piles, due to the quite constant contact pressure over the pipe length.

Monotonic CNL tests were carried out, which showed that a coefficient of friction between the pipe coating material HDPE and sand of μ = 0.4 is reasonable. For cyclic loading and dense states of the sand sample in CNS tests a friction reduction was evident in all tests. Because of the relative smooth interface between HDPE and sand a weak dilatancy was observed in the first cycle. For the other cycles only the normal stiffness for contractancy was decisive.

It was found that the degradation of friction depends stress level $\sigma_{m,0}$ .With increasing normal stiffness K an increasing degradation of friction was observed. For normal stiffness values of 80 to 100 kPa/mm a peak degradation of 20 % of the initial friction was obtained. As a consequence in the current design directives for district heating pipes, which still consider the friction degradation byoverall halving the coefficient of friction μ, the stiffness of the soil around the buried pipe affecting significantly the degradation of friction should be taken onto account.

NOMENCLATURE

A_S	cross-sectional area of the medium pipe, m²
C_C	index of curvature
D	pipe diameter, m
D_F	degradation factor
D_R	relative density = (n_{max} - n) / (n_{max} - n_{min})
E	Young's modulus of the medium pipe material (steel), kN/m²
E_S	oedometric stiffness modulus, MN/m²
F_R	friction force, kN
F_{10}	friction force after 10 cycles, kN
F_0	peak friction force in the 1st movement, kN
H	height of backfill, m
H_D	Shore D Hardness
K	constant normal stiffness, kPa/mm
K_{avg}	average radial spring stiffness, kPa/mm
K_i	radial stiffness, kPa/mm
l_0	length of the gliding section, m
N	normal force, kN
N_0	peak normal force in the 1st movement, kN
N_{10}	normal force after 10 cycles, kN
n	number of K-values among the perimeter
n_{max}	maximum porosity
n_{min}	minimum porosity
r	radius, mm
s	horizontal displacement, mm
U	pipe perimeter, m

U_I uniformity index

u vertical displacement, mm

α_t coefficient of thermal expansion, 1/K

μ coefficient of friction

ν poisson's ratio

ρ_{min} minimum density, g/cm³

ρ_{max} maximum density, g/cm³

ρ_s density of particles, g/cm³

σ_r radial stress, kN/m²

$\Delta\sigma_n$ increment of the effective normal stress, kN/m²

Δr increment of the radial displacement, mm

ΔT temperature difference,°K

Δu increment of the vertical displacement, mm

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