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GEOTEXTILES AS A STRUCTURAL MEMBER FOR LOAD REDISTRIBUTION

LES GEOTEXTILES COMME ELEMENT CONSTRUCTIF POUR LA REDISTRIBUTION DE LA CHARGE

GEOTEXTILIEN ALS KONSTRUKTIONSELEMENT FÜR DIE REDISTRIBUTION DER BELASTUNG

In some cases the load distribution at the soil structure contact can be influenced by the application of geotextiles. The paper formulates general conditions of the practically applicable stress redistribution due to geotextiles. It presents two examples of a structural analysis (raft foundation, pipe under embankment) in which the use of a geotextile can result in the achievement of more favourable stressing of the structure which can be designed more economically.

In einigen Fällen kann man die Spannungsverteilung am Kontakt Erdreich-Konstruktion bei Benützung von Geotextilien beeinflussen. Im Beitrag sind die allgemeinen Bedingungen der praktisch ausnützbaren Redistribution der Spannung durch Einwirkung von Geotextilien formuliert. Es sind zwei Beispiels der statischen Berechnung der Konstruktion (Flächenfundament, Rohrleitung unterhalb der Aufschüttung) angeführt, wo bei Benützung eines Geotextil-Redistributionselements eine günstigere Konstruktionsbelastung zu erreichen wäre, die dan wirtschaftlicher entworfen werden könnte.

GEOTEXTILES AND LOAD REDISTRIBUTION

Non-wowen polypropylen geotextiles, subjected to tests, had a rated weight 200 g/m2, thickness 2,5 mm, elongation min. 20% longitudinally and min. 50% transversally, with the CBR puncture resistance according to DIN 54 307 E min. 1,5 kN.

The friction between two geotextile layers can be expressed by the angle of friction between adjacent geotextiles δ_g whose magnitude does not change practically with normal stresses, changes of temperature and humidity (5, 13). Only the period of primary soil consolidation has been considered in the cases, when the water flow across the geotextiles was negligible. In these cases the adhesion a_g between adjacent geotextile layers is negligible and the shear strength of the geotextiles contact is

$$\tau_g = a_g + \sigma' \tan \delta_g \doteq \sigma' \tan \delta_g \qquad (1)$$

The shear strength of soils is

$$\tau_f = c' + \sigma' \tan \phi' \qquad (2)$$

and friction between the soil and the struture

$$\tau = a + \sigma' \tan \delta \qquad (3)$$

where c' - effective cohesion
ϕ' - effective angle of internal friction
a - wall adhesion, $a \leq c'$
δ - angle of wall friction, $\delta \leq \phi'$

Since the decisive factor of soil failure is the shear stress, stress redistribution in the soil takes place, as a rule, because of local shear deformations. The condition of stress redistribution due to geotextiles has the form

$$\tau_g < \tau_f \qquad (4)$$

or at least

$$\tau_g < \tau \qquad (5)$$

because $\tau \leq \tau_f$. The double layer of tested geotextiles is a structural member which, when suitably located in the soil, enables permanent shear deformation in the selected surface. Thus a state of stress originates in the soil, in which the tensile stress in the geotextile may, but need not be significant for the distribution of the load applied to the structure. Examples of new applications of geotextiles to structural design of building structures follow.

RAFT FOUNDATION

The mean load of foundation soil under large-sized raft foundations is small, as a rule. In the case of rigid rafts the bearing stresses in the proximity of their periphery are high. Consequently, the raft is subjected to the considerable bending moments and shearing forces. The placing of two layers of geotextiles complying with condition (4) at a suitable depth H below the foundation base reduces the bearing capacity of the foundation soil. The maximum bearing stresses near the periphery are thus reduced and the bearing stresses in the centre of the raft increase (11, 18). The raft is then subjected to more regularly distributed bearing stresses than in the case without the redistribution geotextiles.

By way of example, let us consider a rigid box-

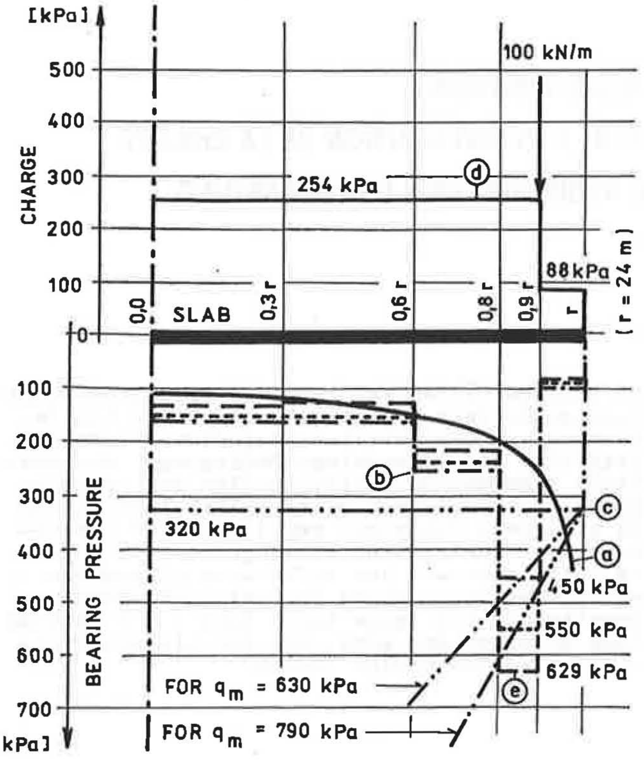

Fig. 1. Loads and bearing pressures applied to the rigid circular box-type raft foundation of a diametr 2r = 48 m.

shaped foundation of a dia. 48 m ammonia reservoir, founded at a depth D = 4.2m, and loaded as per Fig.1. The bearing stresses were calculated according to Gorbunov-Posadov (a in Fig.1), and from the conditions of equal foundation settlement and structure deflection (e in Fig.1) according to Myslivec and Kysela (16) by the method derived from Grasshof (3). The fill next to the foundation consists of the excavated sandy clay (ϕ = 22°, ϕ_r = 20°, c_r ≐ 0, γ = 17kN/m3). Below the foundation there is a layer of gravel-sand of a thickness H_1 = 0,65 m (ϕ_r = 35°, γ = 17kN/m3) on solid sandy clay (ϕ_u ≐ 0, c_u = 70 kPa, γ = 19 kN/m3). Ground water table is about 2m below the ground level; there is practically no ground water flow. The two layers of geotextiles are placed on top of the sandy clay and considered as a redistribution and simultaneously separation layer.

The bearing capacity of the soil was calculated by the method published previously (16). For this reason the calculation is only briefly described. The soil layers are numbered consecutively from top to bottom. The soil of the 1st layer reaching to the foundation depth D is considered as homogeneous, with the mean volume weight γ_1 determined with the consideration of the ground water uplift.

First the bearing capacities $q_{m2,0}$, $q_{m3,0}$, $q_{m4,0}$ are calculated for every layer below the foundation as for a homogeneous subbase for D=0.

Afterwards the two-layer subbase is considered, the second layer reaching from the foundation base to the depth H_1. Below this level there is only the soil forming the third layer. Using Table 1 the depth T_1 is determined, to which the slip surface below the foundation would reach, which would originate, if the bearing capacity of the soil, consisting in the soil of the upper layer No. 2 only, were attained or if the further layer No.3 were at a great depth. The bearing capacity of layers Nos.2 and 3 is determined according to the following formulae:

A. If $q_{m2,0} > q_{m3,0}$, then it is for (6a)

a) $H_1 \leq 0{,}18.T_1 \ \ldots\ q_{m2,3} = q_{m3,0} + \gamma_1.D.N_{q1}.s_{q1}.d_{q1}$

b) $0{,}18.T_1 < H_1 \leq 0{,}9.T_1 \ \ldots\ q_{m2,3} = q_{m3,0} + 1{,}25.(q_{m2,0}-q_{m3,0}).(H_1/(0{,}9.T_1)-0{,}2) + \gamma_1.D.N_{q1}.s_{q1}.d_{q1}$

c) $H_1 > 0{,}9.T_1 \ \ldots\ q_{m2,3} = q_{m2,0} + \gamma_1.D.N_{q1}.s_{q1}.d_{q1}$

B. If $q_{m2,0} < q_{m3,0}$, then it is for (6b)

a) $H_1 \leq 0{,}9.T_1 \ \ldots\ q_{m2,3} = q_{m3,0} - H_1.(q_{m3,0}-q_{m2,0})/(0{,}9.T_1) + \gamma_1.D.N_{q1}.s_{q1}.d_{q1}$

b) $0{,}9.T_1 < H_1 \ \ldots\ q_{m2,3} = q_{m2,0} + \gamma_1.D.N_{q1}.s_{q1}.d_{q1}$

The lowest point of the slip surface corresponding with the calculated bearing capacity $q_{m2,3}$ is situated at a depth T_2 below foundation. The ratios of T_2/B, when B is the foundation width or diameter, for various values of ϕ_2, ϕ_3 and different relative depths H_1/B of the boundary of both layers below the foundation, are shown in Table 1.

Table 1. Values of T_1/B and T_2/B.

ϕ_2 →		0°	20°	40°
T_1/B →		0,70	0,95	1,25
H_1/B	ϕ_3	T_2/B		
≐ 0	0°	0,70	0,70	0,70
	20°	0,95	0,95	0,95
	40°	1,35	1,35	1,35
0,4	0°	0,70	0,80	0,85
	20°	0,85	0,95	1,05
	40°	1,10	1,25	1,35
0,8	0°	-/	0,85	1,00
	20°	-/	0,95	1,15
	40°	-/	1,05	1,35
1,2	0°	-/	-/	1,20
	20°	-/	-/	1,25
	40°	-/	-/	1,35

Note: -/ The lower layer will not influence the result.

For the calculation of the bearing capacity of raft foundations on a three-layer subbase, using the afore mentioned results, the layers Nos. 2 and 3 are replaced with a fictious homogeneous soil reaching to a depth of $H_2 > H_1$ below the foundation base. This soil has a bearing capacity $q_{m2,3}$ to a depth T_2 below the foundation base, if the third layer (at present layer No. 4) at a greater depth did not cooperate. In this way the problem is transformed into the solution of the bearing capacity of a two-layer subbase, consisting of the combined 2-3 layer and the layer No. 4. Practical calculation of the bearing capacity q_m of the three-layer subbase is carried out by means of repeated application of the afore mentioned conditions and formulae, using the values shown in the second line of Table 2 instead of those shown in the first line. In this process

$$q_{m2,3,0} = q_{m2,3} - \gamma_1 . D . N_{q1} . s_{q1} . d_{q1} \qquad (7)$$

If the ground water table is below the foundation base, the soil below the ground water table must be considered as a separate layer.

Table 2. Substitution of Quantities in the Calculation of q_m.

$q_{m2,0}$	$q_{m3,0}$	H_1	T_1	$q_{m2,3}$
$q_{m2,3,0}$	$q_{m4,0}$	H_2	T_2	q_m

Because the shear failure of the soil along the predetermined slip surface can be due to the displacement of merely several milimetres, and because the settlement of a raft foundation amounts to several centimetres, as a rule, the relative elongation of geotextiles can attain maximally several per cent. The force per unit width of the geotextiles, therefore, is negligible from the wiewpoint of the state of stress below the foundation. In our particular case it is sufficient to consider the effect of the geotextiles as the effect of thin soil layer with the characteristics of $\phi' = \delta_g = 18°$, $c' = a_g \doteq 0$ and $\gamma = 18$ kN/m3. Then we obtain for gravelsand $q_{m2,0} = 4100$ kPa, for the soil layer replacing geotextiles $q_{m3,0} = 310$ kPa, and for sandy clay $q_{m4,0} = 470$ kPa. The calculation according to the afore mentioned method of the three-layer subbase analysis yields $q_{m2,3} = 310 + 320 = 630$ kPa. Without geotextiles it would be $q_m = 470 + 320 = 790$ kPa. The reduction of the bearing capacity of the foundation due to geotextiles is of no consequence, because the mean load of the foundation soil is only q = 230 kPa. The geotextile layer, however, facilitates and accelerates the origin of local plastic failures of the soil near the foundation edge. Thus a redistribution of bearing stresses occurs which can be determined, in our particular case, like for rigid foundations (c in Fig.1). (18) In this way the maximum bearing stress is determined at approx. 550 kPa for the foundation without geotextiles below the gravelsand layer, and approx. 450 kPa for the foundation with geotextiles below the gravelsand layer. Theoretical bearing stress in the same part of the cross section amounts to 629 kPa. The redistribution of bearing stresses (b in Fig. 1) for the described use of geotextiles is most favourable for the dimensioning of the raft foundation and the financial means are used more effectively (12) than without the use of geotextiles.

The described effect of geotextiles was tested on models with the strict observation of the rules of model similarity, incl. the application of the model scale to geotextile thickness,etc.

PIPES UNDER EMBANKMENTS

Measurements on models on the scales of 1:5 to 1:2 have ascertained that, if the constraint (5) has been satisfied, a two-layer geotextile mantel considerably reduces the loading of pipes by soil pressure (13). The application of geotextiles not only suppresses the local extrems of soil pressure, but also causes changes in the distribution of bearing stresses on the pipe surface. Selected results of the measurements are given in Table 3, in which 100% is the absolute magnitude of maximum peripheral stress in the wall of a cylindrical pipe, when the backfill has been compacted in layers and when it has been only moved laterally by a buldozer. The table shows that with a geotextile mantle the pipeline is stressed far less significantly than if it is placed directly into the soil.

Table 3. Comparative Values of Pipeline Stresses in Selected Cases.

Method of backfill placement	Pipe surface	Pipeline placed in the soil	
		without geotextiles	with a double geotextile mantle
Compaction in layers	int.	100%	88%
	ext.	100%	73%
Lateral movement	int.	419%	265%
	ext.	431%	258%

For the structural design of pipelines placed in the ground a new method was developed, since the traditional methods (7, 8, 19, 20) resulted in considerable differences between theoretical and actual values in a number of cases. The number of characteristic features of traditional methods of the determination of soil pressure on pipes includes the dependence of the results on very variable deformation characteristics of the soil and the difficulties connected with the expression of some technological factors in the calculation (9). An improvement can be attained by the determination of soil pressures according to the theory of plasticity and limit states in soil mechanics (1, 14, 15, 16). The fact is taken into account that the displacements in the soil in the course of the settlement of the soil in the vicinity of the pipe under an embankment are several times greater than those required for the mobilization of the shear strength of soils. Since the soil in the embankment in the vicinity of the pipe is in the state of failure for the major part of the period of its consolidation, which is the least favourable state for the dimensioning of the pipeline, as a rule, the structural analysis uses the strength parameters and volume weights of soils. These characteristics can be determined with considerably greater accuracy than the deformation characteristics of the ground. The new method described further on has been elaborated for independent pipelines, i.e. the pipelines situated at a distance of at least 3 $\bar{d}_m$ from other parallel pipelines, when $\bar{d}_m$ is the mean outside diameter of adjoining pipes. It is also assumed that the overburden of

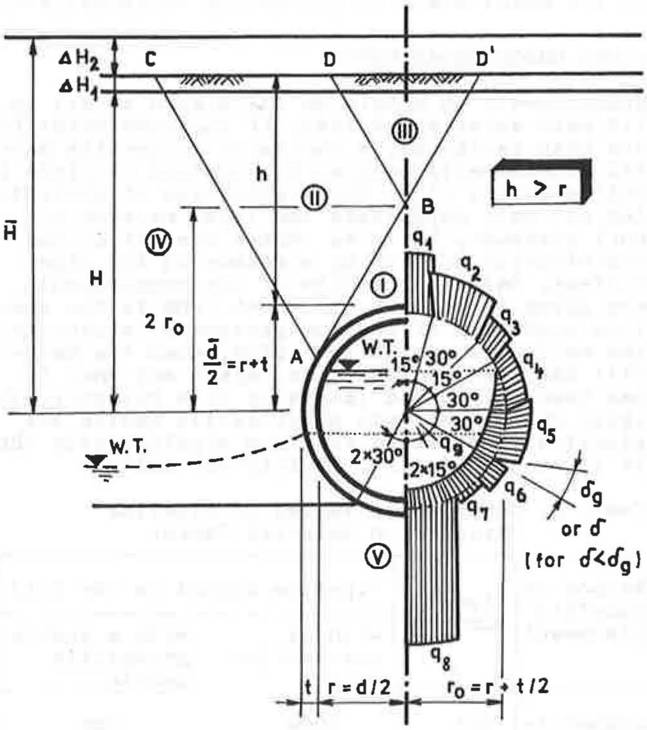

Fig. 2. Slip surfaces in the embankment near the pipeline (left) and idealized pipeline loads (right).

the pipes is thicker than the pipe radius and that the tensile strength of the unconsolidated embankment is negligible.

The bearing capacity of the pipe soil bed, adjusted to the shape of the pipes, with the angle of repose at least 60° (V in Fig.2, left) is not utilized by the pipe pressure loaded from above with the overburden and moving loads, as a rule. The soil at the sides of the pipes (IV in Fig.2, left) undergoes settlements which are larger than the compression of the pipes and their penetration into the soil bed. In this process a soil wedge originates above the pipe (I in Fig.2, left), along which the settling soil (II) slides with approximately one half of the soil volume from the area (III) above the wedge (I).

The dominant role of the boundary slip surfaces in comparison with the volume and shape deformation of the soil structure was confirmed by radiographic methods (15). From the conditions of equilibrium on kinematically possible and probable slip surfaces AB, AC, BD, BD', determined by model tests, taking into account the effect of rigidity (14, 16) of standard steel, cast iron and concrete pipes and the friction between the soil and the pipes provided with a two-layer geotextile mantle or without it, an idealized equivalent diagram of bearing stresses (Fig.2, right) was derived. Fig. 2 shows also the symbols used.

The least favourable magnitude of lateral soil pressure on the pipes is taken into account. The settling soil produces a lower lateral pressure than the consolidated soil. The settlement cannot be fully eliminated by embankment compaction, because the attainable effect of compaction is limited by the bearing capacity of the surface. However, after the placing of subsequent layers the lower layers settle (2). After the soil consolidation has terminated, the stresses in the pipes in the transverse direction decrease, but effect of longitudonal flexure of the pipe increases (14).

$$q_1 = 2{,}15\ \bar{\gamma}\ (\bar{H} - r_o) \qquad (8)$$

$$q_2 = 1{,}73\ \bar{\gamma}\ (\bar{H} - r_o) \qquad (9)$$

$$q_3 = 1{,}30\ \bar{\gamma}\ (\bar{H} - r_o) \qquad (10)$$

$$q_4 = 1{,}15\ \bar{\gamma}\ (\bar{H} - r_o/4)\ K \qquad (11)$$

$$q_5 = 1{,}15\ \bar{\gamma}\ (\bar{H} + r_o/4)\ K \qquad (12)$$

$$q_6 = 0{,}72\ \bar{\gamma}\ (\bar{H} + r_o/2)\ K \qquad (13)$$

$$q_7 = 0{,}24\ \bar{\gamma}\ (\bar{H} + r_o/2)\ K \qquad (14)$$

With the assumption of the bearing of the pipes amounting to 85% of their length and for $q_9 = 0$

$$q_8 = 7{,}3\ \gamma_p\ t + 1{,}65\ \bar{\gamma}\ (2{,}3 \cdot (\bar{H} - r_o) + \bar{H}\ K) \qquad (15)$$

q_9 is the most unfavourable difference of the pressures inside and outside the pipes (in Fig.2 the inside water pressure is higher, because some leaks of the pipe joints are assumed)

In the equations the following symbols are used:

$$K = \tan^{2æ}(45^o - \phi'_r/2) \qquad (16)$$

$$\bar{\gamma} = \gamma\,(1 + a\ /\ (H + \Delta H_1 + \Delta H_2)) \qquad (17)$$

where

ϕ'_r - effective residual angle of internal friction of soil

γ - volume weight of temporarily loose overburden

γ_p - volume weight of pipe material

æ - coefficient of effect of compaction quality and speed of soil consolidation according to Table 4

a - coefficient expressing the dynamic effect of loads (e.g. for highways a=0,3, for railways a=0,6)

ΔH_1 - additional height of embankment during construction

ΔH_2 - equivalent (fictitious) overburden replacing the static effects of dynamic loads (e.g. 1m for highways)

Table 4. Coefficient æ.

Embankment compaction	Embankment consolidation	
	speedy (e.g. sands)	slow (e.g. loams)
in layers	5 / 8	1
none	7 / 8	1

The thus determined loads are in good agreement with the measurements of water supply pipelines and with the results of 117 measurements by various methods on the outside of a R.C. pipe of inside diameter 10,5m under a sandy backfill the height of which attained as much as 23,9m (17).

The calculation of bending moments, shearing and normal forces in the wall of the pipe is carried out as the analysis of a circular frame, e.g. by means of influence coefficients (6). If the length of the individual pipe units exceeds approx $1,5.d^{0,5}$ (for d in metres), it is necessary, as a rule, to take into account the three-dimensional state of stress of the pipes in the dimensioning. The resulting stress, decisive for the dimensioning of the pipeline from the viewpoint of strength, is multiplied by a correction coefficient k, which was determined by a probabilistic analysis of the influence of the variability of input data with regard to the simplifications used in the derivation of the design method, the influence of the angle of repose and the geotextile mentle, etc. (14).

Table 5. Coefficient k.

Angle of repose of the pipeline	Pipeline construction	
	with an effective geotextile redistribution mantle, i.e. condition (5) applies	in some other way
→ 60°	1	7 / 6
90°–120°	5 / 6	1

The favourable influence of the geotextile redistribution mantle of pipelines is introduced into the calculation, on the one hand, by the coefficient k, on the other hand by the angle of wall friction δ_g, when $\delta_g < \delta$. The values of the coefficient k for the pipelines placed on unprepared soil bed and/or burried by lateral movement of the fill, e.g. by a buldozer, are recomended to be determined by in situ measurements and, therefore, are not given here.

In the case of pipelines with an inner overpressure $p \doteq 1$ MPa and over, placed in the ground, it is necessary to take into account also the initial curvature in the individual places of the cross section and the changes of curvature produced by external loads and by the usually variable inner overpressure. The production deviations δ_g of the pipes from circular cross section - the imperfections of pipe shape - cause that locally the radius of curvature may differ - from structural point of view significantly - from the nominal pipe radius. It is advisible to solve this problem as a stochastic problem.

Fig. 3 shows the relations of the maximum tensille stress σ_t in the wall of a steel gas pipeline placed in the ground and the inner overpressure p for the following cases:

a) ideally circular thin-walled pipe under embankment,

c) pipe of dimensions and placement as per a), but with such imperfection in the place of max. σ_t that the external load produces a curvature of the wall the radius of which does not depend on p (e.g. oval pipe with longer vertical axis),

b), d) pipes as per c) with different imperfections of the same character,

e), f), g) pipes of dimensions and placement as per a), but with such imperfections in the place of max. σ_t that a certain inner overpressure p_1 produces the same wall curvature as in the case of not loaded pipe (e.g. oval pipe with horizontal longer axis),

h) the values of σ_t due to p, regardless of external loads.

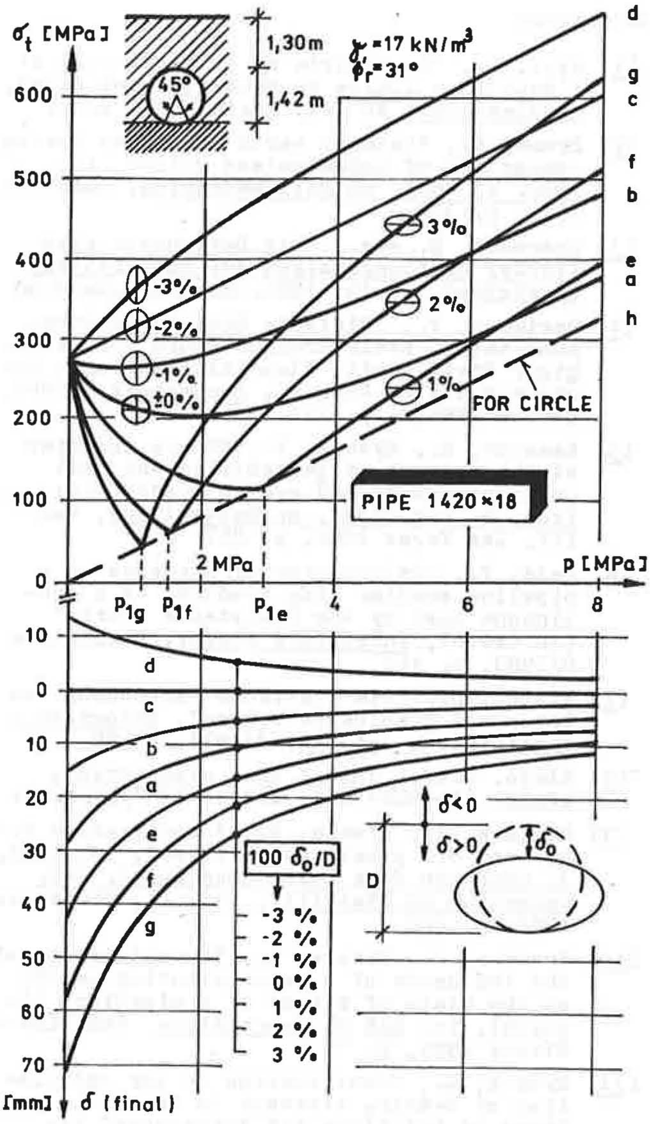

Fig. 3. Maximum tensile stress $\sigma_t > 0$ plotted against inner overpressure p of steel gas pipelines placed in the ground under an embankments. Deformations δ of the pipes plotted against inner overpressure p.

The curves a), b), c), d), e), f) express qualitatively the state after the completion of the embankment. The values of σ_t vary in time. If the soil is moist to water saturated, the embankment consolidation can be accelerated by drainage, using the geotextile pipeline mantle.

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