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Test Areas with Vertical Drainage Systems

Champs d'essai aux systèmes de drainage vertical

In recent years the supply of different types of prefabricated drains for vertical drainage systems has been considerably increased. At the same time laboratory test methods have been developed to analyse the essential characteristics of drain materials and of the drains themselves. However the explanation of the measured difference between laboratory test results i.e. the interpretation of laboratory test results into actual practice, confront the research-worker often with difficult problems. In order to support and verify the laboratory test results of prefabricated drains, six test sites have been equipped between 1979 and 1982 for this purpose in the sub-grade of the State Highway Nr.1 south-east of Amsterdam. In these test areas the behaviour of five types of prefabricated vertical drains have mutually been compared and also to the conventional sand-drains. This paper considers the most important results of these in-situ tests. Both the measurement of settlements as well as the piezometer readings indicate a difference of behaviour amongst the test areas i.e. a difference between the functioning of the different drain types.

Dans les dernières années l'offre de divers types de drains préfabriqués pour le drainage vertical a augmenté considérablement. En outre des essais de laboratoire pour la détermination des qualités des matériaux drainantes et des drains considérées essentielles ont été développés. Toutefois, l'interpretation des différences de comportement en laboratoire des matériaux et des drains, c'est à dire la traduction des résultats d'essais de laboratoire en performance pratique, souvent pose de grands problèmes au chercheur. Pour donner assistance aux recherches et pour vérifier les résultats des essais de laboratoire obtenus avec des drains préfabriqués, entre 1979 et 1982 une demidouzaine de champs d'essai a été installée sur la route nationale no.1 au Sud-est d'Amsterdam. Dans ces champs d'essai on a comparé la performance pratique de cinq types de drain préfabriqué tout à titre mutuel qu'auprès du fonctionnement des drains de sable. Les mesurages de tassement si bien que ceux de la pression interstitielle révèlent des différences du comportement pratique des champs d'essai et, par conséquent, des drains préfabriqués.

1. INTRODUCTION.

In the field of vertical drainage systems, a number of important developments have been presented in recent years. The prefabricated band-shaped drain has been recognized more and more as an alternative for the conventional vertical sand-drain for vertical drainage projects. The increasing supply of different types of prefabricated drains is the first indication of this fact. The usual way of in-situ testing to establish the suitability of new drains is not only time-consuming, but also very costly and therefore not very attractive. Therefore producers and users of drains proceeded to develop laboratory test methods to analyse and detect a number of properties of prefabricated drains, which are considered to be essential. At present a good impression of the usability of new materials for drainage systems can be obtained with laboratory tests, by means of permeability tests on the filter and discharge capacity tests of the drains. However the main problem is still to implement the differences found with laboratory test results of different materials and drains and the consequences thereof for the usability in practice. This of course, because the circumstances during testing in a laboratory differ considerably compared to those of a drain in-situ, buried into the ground. Therefore in situ-testing will remain indispensable on a modest scale, to support and verify laboratory investigations.

The State Road Engineering Division, in cooperation with Delft Soil Mechanics Laboratory and producers of prefabricated drains have prepared six test sites. Except for the conventional sand-drain, five prefabricated drains have been installed in these test-sites, of which three drains have been equipped with a non-woven fabric filter. The aim of these tests is in first instance, to mutually compare the behaviour of the installed drains in situ. In a later stage it will be investigated whether the recorded differences of the drain behaviour in situ can be related to the laboratory test results of certain material properties and drainage properties.

In the present paper however, only the results of the in situ-tests at the different test sites will be discussed.

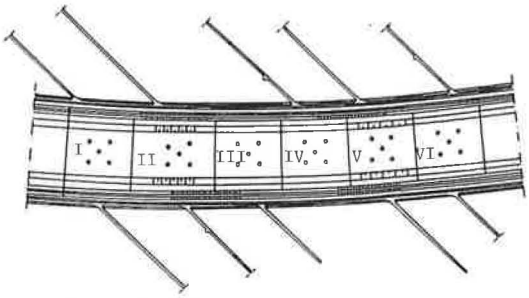
2. DETAILS OF THE TEST-SITE AND THE INSTALLED DRAINS.

2.1. Location.

The test-sites comprise a part of the fill for the 2x2-lane state highway, which is under construction in the neighbourhood of Amsterdam. The test-sites are located in such a way in the alignement of the road, that in each site an area of 30x30 m^2 is not intersected by (former) open boundary-ditches. The width of each test-site varies between 50 to 55 m and the length of each site varies between 65 and 80 m. The six test-sites are bordering each other and occupy approximately a total length in the road alignement of 500 m (see figure 1).

2.2. Applied prefabricated drains.

From the considerable supply of prefabricated drains five drain types have been selected, based on laboratory test results and on the cost price, in such a way that the most used types of drain-constructions are represented. The five selected drain types and the conventional



o settlement plates

figure 1. Location of settlement plates and test-sites

sand-drain are specified in more detail in table 1. For reasons of construction the principal of the road project did not allow to prepare a reference site without vertical drains at all.

Due to the small differences in stratigraphy of the compressible layers, an arbitrary lay-out of the position of different drain types is possible. However, for historical reasons a choice has been made in such a way that conventional sand-drains could directly be compared to paper filter type drains and the latter to non-woven filter drains. Both drains with a non-woven fabric filter have been installed in such a way, that a straight comparison between these two drain types was possible. Finally the lay-out of test sites as shown in table 1 has been achieved.

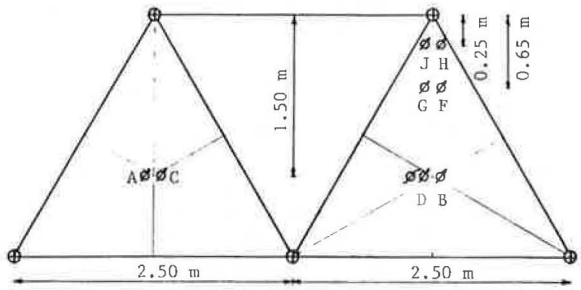
2.3. Instrumentation of test-sites.

All measuring equipment, necessary to record the behaviour of the drains in situ, has been installed and concentrated within an area of 20x20 m^2 for each test-site. The centre of this area coincides with the centre of the test-site, so that, in view of the thickness of the compressible layers, the influence of bordering test-sites with another drain type on the one hand and the sloping edges of the sandfill on the other hand, can be regarded negligible.

Settlement plates have been installed directly on the ground surface , which was N.A.P.-1.40 m (N.A.P. is the national reference level), in each corner and in the centre of the measuring area. These settlement plates have been constructed in such a way that they can also be used as open piezometers in order to record the groundwater table in the fill. Besides, an open piezometer with its filter reaching down into the sand formation below the compressible layers, with a thickness of approximately 9 m, has been installed in the centre of the test site.

Per test-site eight piezometers of the Kistler type have been installed close to the centre of the measuring area These piezometers reaching down to different depth have been installed at distances from the drains of respectively 1.50 m, 0.65 m and 0.25 m. The configuration of the locations of piezometers in conjunction with the position of the drains and the depth of their filters are indicated on figure 2. In order to restrict as much as possible the influence of negative skin friction and hence the possibility of pushing down the piezometers, the stand pipes of each piezometer have been cased over the height of the sandfill.

Recording of the settlement plates and piezometer-readings will be continued up to mid August 1982.



depth of installation of water pressure meter filters:

- A 3.2 à 3.3 m -N.A.P.
- B, F, H 4.7 à 4.9 m -N.A.P.
- C 7.0 à 7.1 m -N.A.P.
- D, G, J 8.6 à 8.8 m -N.A.P.

● drain

figure 2. Location of water pressure meters in relation to drain positions.

In order ro record the waterpressure potential over the drain filter, three drains per drain type have been equipped with miniature piezometers at both sides of the filter. Due to a progressive zero-point fluctuation after installation of these miniature piezometers, recording of these measurements had to be stopped in an early stage.

Table 1. Applied drains and lay-out of test sites.

Test site	Drain name	Type	Particular details	Dimensions
I	Bidim	monolith	polyester non-woven fabric 550 g/m^2	150x4 mm^2
II	Desol	monolith	perforated polyethylene tubes	95x2 mm^2
III	Colbond CX 1000	composite	core : polyester Enkamat filter: polyester non-woven fabric 130 g/m^2	150x5 mm^2
IV	Mebra-ester	composite	core : grooved polyethylene filter: polyester non-woven fabric 120 g/m^2	100x3 mm^2
V	Mebra-paper	composite	core : grooved polyethylene filter: paper 115 g/m^2	100x3 mm^2
VI	Conventional sand-drain	monolith	drainage sand: k= 12 m/24 hrs	Ø 260 mm

Table 2. Composition of the compressive formation.

soil description	γ	Consolidation coëfficients ($x10^{-7}$ m^2/sec) and compressibility constants								thickness of layers per test-site (m)					
		Δp = 10à20 kN/m^2 (I)				Δp = 50à90 kN/m^2 (II-VI)									
	$\frac{kN}{m^3}$	C_v	C_h	C_p	C_s	C_v	C_h	C_p	C_s	I	II	III	IV	V	VI
peaty clay	14.5	-	-	50	350	-	-	30	240	0.35	0.40	0.40	0.40	0.25	0.50
peat	10.0	100 25	50 15	15à25	80à100	3 1	1 1	6	54	2.00	2.00	2.00	2.00	2.00	2.00
peat	10.2	50 15	25 5	15à25	80à100	2.5 0.5	1 0.5	6	54	1.90	1.85	2.10	1.60	2.10	1.90
clay with remnants of vegetation	15.0	2.5	2.5	25	180	1.5	1.5	12	150	1.35	1.50	1.10	1.35	1.50	1.10
sandy clay	16.5	30	7	50à60	250à300	30	6	25	160	2.10	1.65	1.05	2.25	1.40	1.40
peaty clay with shell fragments	14.0	1	1	20	60à90	0.25	0.4	9	80	1.25	1.65	2.20	1.15	1.35	1.30
base-peat	-	1	1	15à25	80à100	4	2.5	6	54	0.35	0.35	0.45	0.20	0.30	0.40

3. SUBSOIL INSTRUMENTATION.

3.1. General

In the centre of each test-site one medium-range cone penetration test and one boring, continuous sampling Ø 66 mm, down to a depth of 12.0 m below original grade, have been performed. Laboratory tests, concerning the compressibility, the friction parameters and horizontal- and vertical permeability, have been performed on selected samples of this borings. From the results of these borings and cone penetration tests a geotechnical cross-section, from ground level down to about N.A.P. - 25 m, in longitudinal direction over the road-section, in which the test-sites are situated, has been made.

3.2. Subsoil description.

The original ground level of the test-sites decreases gradually from N.A.P.-1.40 m at test-site I to N.A.P.-1.45 m at test-site VI.
Below ground level the Holocene formation consists of a thin peaty top clay layer, resting on a peat layer with a thickness of approximately 4 m. These two layers belong both to the so called Holland-peat-formation. Between N.A.P.-5.5. m and N.A.P.-6.0 m this peat layer gradually changes into a clay formation belonging to the Calais-deposits. This clay formation can be subdivided in three layers, based on the nature of their mixtures. The thickness of each of these layers varies slightly from one test-site compared to another. At the top, between N.A.P. -5.80 m and N.A.P.-7.20 m the admixtures consist of remnants of vegetation. Between N.A.P.-7.20 and N.A.P.-8.80 m sandy enclosures are present. The lowest layer of this clay formation between N.A.P.-8.80 m and N.A.P.-10.50 m is peaty and contains shellfragments. Between N.A.P.-10.50 m and N.A.P.-10.80 m appears a thin layer of Base-peat, forming the conjunction to a sand layer, belonging to the Pleistocene formation, in which not only thin loam layers but also thin peat layers occur . Continuing downwards, between N.A.P.-14.00 m and N.A.P.-15.00 m the top-horizon of a clay formation can be distinguished, which contains thin sand layers to a depth of N.A.P.-18.00 m to N.A.P.-19.00 m and deeper down to approximately N.A.P.-23.00 m, contains much shells. Below this formation a dense sand formation occurs. The stratigraphy and structure of the compressible layers resting on top of the intermediate sand layer at approximately N.A.P.-10.80 m has been recorded in table 2. In this table also some laboratory test results, from tests performed on undisturbed samples from the distinctive layers, are given. Based on the results of the Dutch Cone Penetration Tests, the continuous borings and the laboratory test results, it could be concluded that there is little difference in structure and nature of the compressible layers between the different test sites.

3.3. Groundwater tables.

The phreatic level in the polder in which the test-sites are located, is maintained at N.A.P.-1.58 m. With the aid of open-piezometers the phreatic level in the intermediate sand layer has been established and declines from N.A.P.-1.30 m at test-site I to N.A.P.-1.60 m at test-site VI.
During ample time before the start of construction works and applying the fill, it has been established with the aid of piezometers, that the groundwater pressure in the compressive layers declines almost hydrostatically.

4. CONSTRUCTION OF THE TEST-SITES.

At the location of the test-sites a sand fill with a total thickness of 5.80 m had to be put in place. Following the installation of the settlement plates in mid-July 1979, a base-layer of drainage-sand with a thickness of 0.80 m has been applied. This base-layer has been dry-filled in two layers of 0.40 m.
Using this sand layer as a working platform, the vertical drains, reaching down to approximately N.A.P.-11.00 m i.e. reaching down into the intermediate sandlayer, have been installed in September 1979. For all drain types a mutual distance, centre to centre, of 2.50 m has been chosen for all test-sites. In practice the drains have been installed on cross-lines. The distance between the cross-lines is 2.50 m and the distance centre to centre of the drains on the cross-line is also 2.50 m. The prefabricated drains have been installed using a vibrating shaft with an outside cross-section of 150x50 mm^2. The conventional sand-drain has been constructed using wash-boring techniques. Following the original time-schedule for construction, the remaining sand fill with a thickness of 5.0 m would have been applied in 5 stages in layers with a thickness of 1.0 m each, immediately after the installation of the drains. However, problems with the supply of sand have caused a delay of approximately 60 weeks in applying the first stage (stage II) of the remaining fill on top of the base-layer (stage I). In November 1980 the application of the fill (stage II) started by dry-filling methods, working

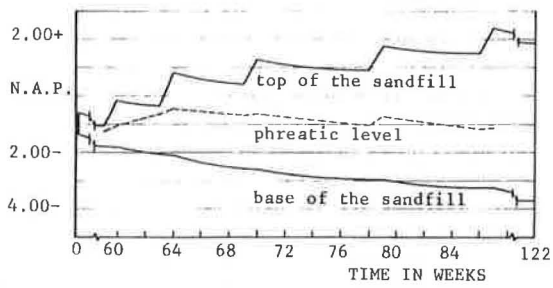


figure 3. Diagram of the application of the fill

in a direction from test-site VI to test-site I. The capacity of sand supply and transport was such that as an average two test-sites could be filled over a thickness of 1.0 m in 5 working days. In order to secure the stability of the side-slopes of the fill, a rest period was obeyed between the application stages of minimal 4 weeks (between stage II and stage III), increasing with two weeks for every following stage. Schematically the following filling schedule has been performed:

stage II 60 weeks after stage I
stage III 4 weeks after stage II
stage IV 6 weeks after stage III
stage V 9 weeks after stage IV
stage VI 8 weeks after stage V

The applied filling-schedule resulted in a loading scheme as shown in figure 3. In this loading scheme the recorded settlements of the fill and the phreatic level in the fill, measured between the application of the filling stages, are also given. The application of the fill was completed in May 1981. Hence the application of the remaining fill of the stages II-VI lasted for each test-site a period of 28 weeks.

5. MEASUREMENTS AND INTERPRETATION OF RESULTS.

5.1. Frequency of measurements.

During the execution of the work, the settlements of the fill and the phreatic levels have consistently been measured at the beginning and at the end of each working week. During the rest period, between the completion of stage I and the start of stage II, as well as in the period one month after completion of the fill, the frequency of measuring has been reduced to once per month.

Piezometer measurements and pore pressure meter readings have been performed with a frequency of three times per week, exceptfor the rest period between stage I and stage II. These readings will be continued with a frequency of once per two weeks until and including the execution of the pavement construction in August 1982.

5.2. Porewater pressures.

During the rest period of approximately 60 weeks after the application of the drainage base layer, the compressive layers have had more than enough opportunity to consolidate under the applied load of 13 to 15 kN/m^2 of this base layer. No settlement increments, nor further decrements of porewater pressures, have been recorded during the last weeks of the rest period. Just before the application of stage II, almost hydrostatic groundwater pressures have been measured in the compressive layers, at all test-sites, with a phreatic level of N.A.P.-1.20 m in the drainage base layer and a phreatic level of N.A.P.-1.90 m in the intermediate sand layer. The hydrostatic water pressure line, mentioned above has been used as a reference (time-set t=0) for the recorded excess pore pressures during and after the application of the fill in stages II up to stage VI. The filling-schedule, given in figure 3 for the stages II to VI corresponds, due to the recorded settlements and phreatic levels, with a single load increase of 83 kN/m^2 applied at once at time t=0.

With the aid of piezometer readings and pore water pressure meter readings, the excess pore pressure-curve has been recorded as a function of the depth in the compressive layers, immediately after the completion of the fill at stage VI (t_1= 196 days) and also 90 days (t_2= 286 days) and 244 days (t_3= 440)days after completion date. Figure 4, for example, illustrates the curve of schematised porewater pressures in test-site IV. The points A to E in this figure are the recorded excess porewater pressures of the pore pressure meters of the same character, placed in the centre of gravity of the triangle in which the drains are installed.

Based on the open-piezometer readings, it is expected that the porewater pressure curve in the compressive layers will become stable at the end of the hydrodynamic period (t_e), resulting in an almost hydrostatic pressure line, with a phreatic level of N.A.P.-1.10 m in the fill and a phreatic level of N.A.P.-1.80 m in the intermediate sand layer.

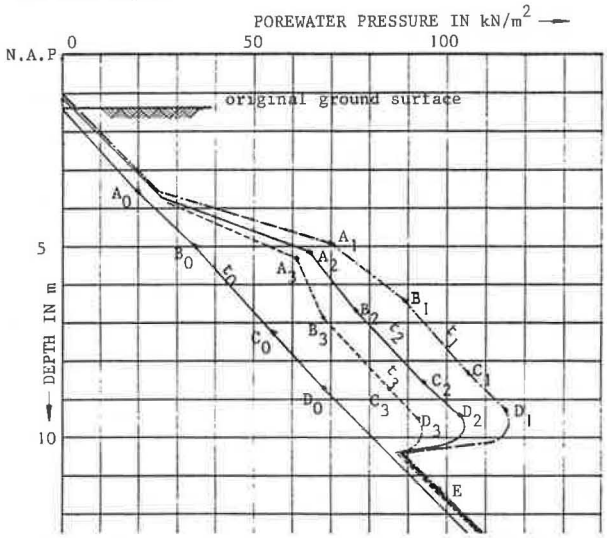


Figure 4. Porewater pressure curves of test-site IV at t_0, t_1, t_2 and t_3.

The excess pore pressures for the pore pressure meters A and C recorded at t_1, t_2 and t_3, can be expressed in percentages of the maximum excess pore pressure (which would have appeared theoretically), compared to the expected final hydrostatic pressure (see table 3). By plotting these proportional excess pore pressures as a function of time, it is possible to obtain a graphical impression of the progress of dissipation of the excess pore pressures (figures 5 and 6). These figures show clearly that for the peat-layer as well as for the clay layer, significant differences in dissipation of excess pore pressures occur for the different types of drains. Until today (January 1982), the dissipation of excess pore pressures progressed best, as expected, at test site VI, in which the conventional sand drains have been installed. The dissipation of excess pore pressures progresses very slowly at test-site II and falls clearly behind compared to the remaining test-sites.

Table 3. Excess pore pressures at different time-points (meters A and C).

test site	excess pore pressures in kN/m² and % at t_1=196 days, t_2=286 days and t_3=440 days											
	peat-layer						clay-layer					
	t_1		t_2		t_3		t_1		t_2		t_3	
	kN/m²	%	kN/m²	%	kN/m²	%	kN/m²	%	kN/m²	%	kN/m²	%
I	42.8	54.8	35.2	45.2	27.5	35.3	49.5	63.5	35.1	45.0	23.4	30.0
II	42.4	54.5	38.9	50.0	36.1	46.4	45.9	58.9	41.1	52.8	35.1	45.1
III	31.0	39.7	23.6	30.2	17.0	21.8	34.3	43.8	22.6	28.8	12.8	17.0
IV	31.6	40.2	24.0	30.5	19.0	24.1	35.9	46.0	22.4	28.8	11.7	15.0
V	36.2	47.8	30.1	39.6	26.0	34.2	42.4	55.0	35.5	46.1	29.4	38.2
VI	22.4	29.5	7.8	10.3	3.4	4.5	25.3	32.8	12.2	15.8	7.8	10.1

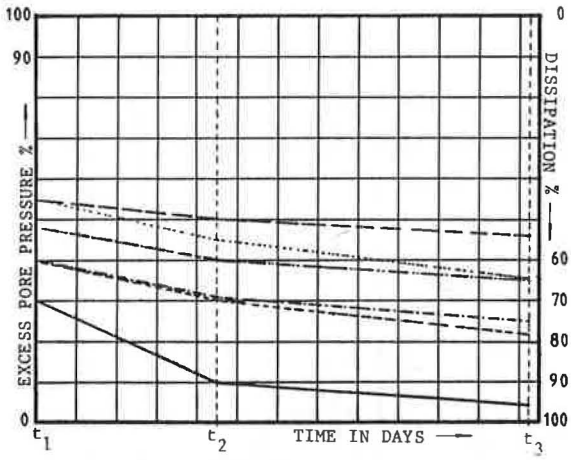


figure 5. Excess pore pressure in peat-layer.

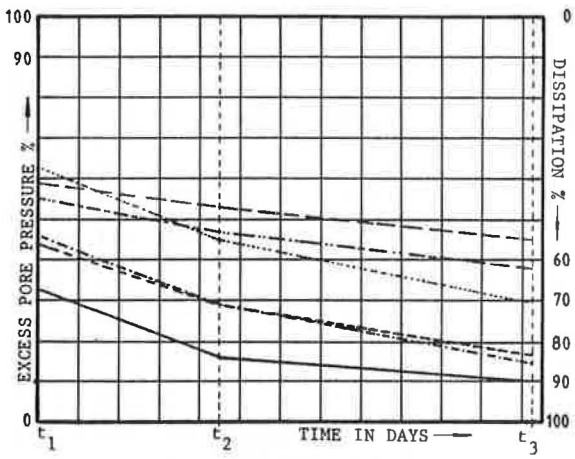


figure 6. Excess pore presssure in clay-layer.

5.3. Development of the settlements.

When the properties of compressive clay- and peat-layers, as determined in the laboratory, are compared to each other, it appears that there is only little difference between the test-sites for the distinctive layers. The variation in the expected total settlements of the different test-sites has mainly to be explained by the mutual differences in thickness of the peat- and clay-layers per test-site. As can be seen from table 4, in which the thickness of the different layers per test-site have been recorded, it appears that mainly the stratigraphy of the subsoil at the test-sites IV and VI differs from those in other test-sites. The thickness of the peat-layer at test site IV and the thickness of the clay-layer at test-site VI are smaller compared to the other test-sites, so that less settlements can be expected in these two test-sites.

Based on the soil-parameters, obtained from laboratory tests, a prognosis has been made of the total settlement for each test-site. For the test-sites IV and VI a total settlement of the compressive layers, resting on the intermediate sand layer, of approximately 3.25 m is expected and for the other test-sites the total settlment has been estimated to be between 3.40 m and 3.50 m.

The compression of the deeper clay-layers below the intermediate sand layer has been estimated to be between 0.10 m and 0.15 m for all test-sites.

In figure 7 the average of recorded settlments has been plotted for each test-site, covering the period between t_0=0 (just before the application of the fill in stage II) and t_3=440 days. The final settlements of the previously applied base-layer, stage I, achieved at t_0=0, as well as the settlements, referred to original grade, at time-points t_2= 286 days and t_3= 440 days (end of January 1982) are tabulated in table 4. In this table

Table 3. Summary of settlement data.

test site	thickness of compressive layers (m)		settlement prognosis	average of recorded settlements at time-points t_0, t_2 and t_3(m)			settlement at t_3 as a percentage of the settlement prognosis (%)
	peat	clay	(m)	t_0=0	t_2=286 days	t_3=440 days	
I	3.90	5.05	3.40	0.47	2.33	2.48 ± 0.10	73.0
II	3.85	5.25	3.45	0.46	2.13	2.22 ± 0.12	64.5
III	4.10	4.75	3.45	0.43	2.57	2.67 ± 0.15	77.5
IV	3.60	5.15	3.25	0.49	2.45	2.55 ± 0.12	78.5
V	4.10	4.50	3.40	0.50	2.10	2.20 ± 0.09	64.5
VI	3.90	4.30	3.25	0.53	2.48	2.57 ± 0.15	79.0

the settlements recorded in January 1982, are also expressed in a percentage of the total expected settlements.

By comparing the achieved settlments, i.e. the development of the settlements as a function of time, to the settlement prognosis, it appears that the test-sites III, IV and VI give the best proportional results, followed by those of test-site I. The results of test-sites II and V are almost identical and fall clearly behind.

The spread of the recorded settlements for each test-site varies from one test-site to another between 0.10 m and 0.15 m. Taking this variance into account, it can be concluded that, up to January 1982, the test-site III, IV and VI have settled 0.50 m to 0.55 m more and test-site I has settled 0.30 m more than the test-sites II and V.

6. CONCLUSIONS.

Based on the results of in situ measurements combined with soil mechanical data and calculations, it has to be concluded that the established differences in behaviour in practice of the different test-sites can only be explained by the better or worse functioning of the different drain-types.

Further analyses of the different measurements have to indicate which proporties of the drainmaterials or drains have resulted to the observed differences in behaviour in practice.

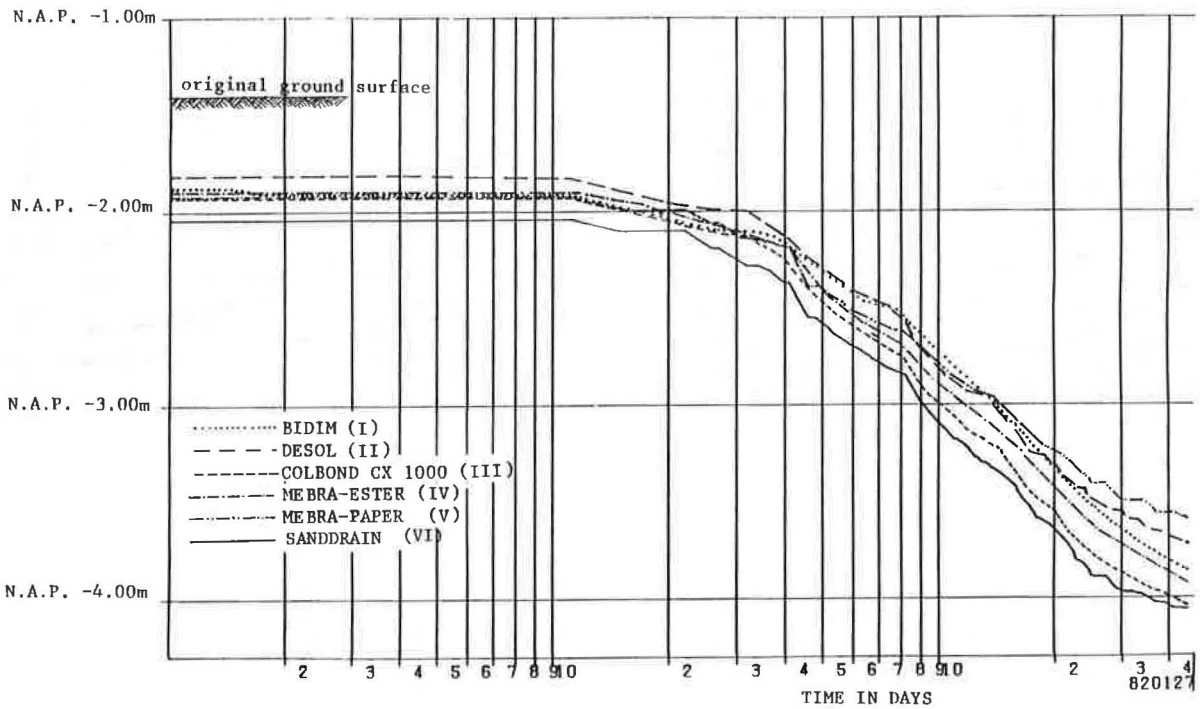


figure 7. Development of the settlements.