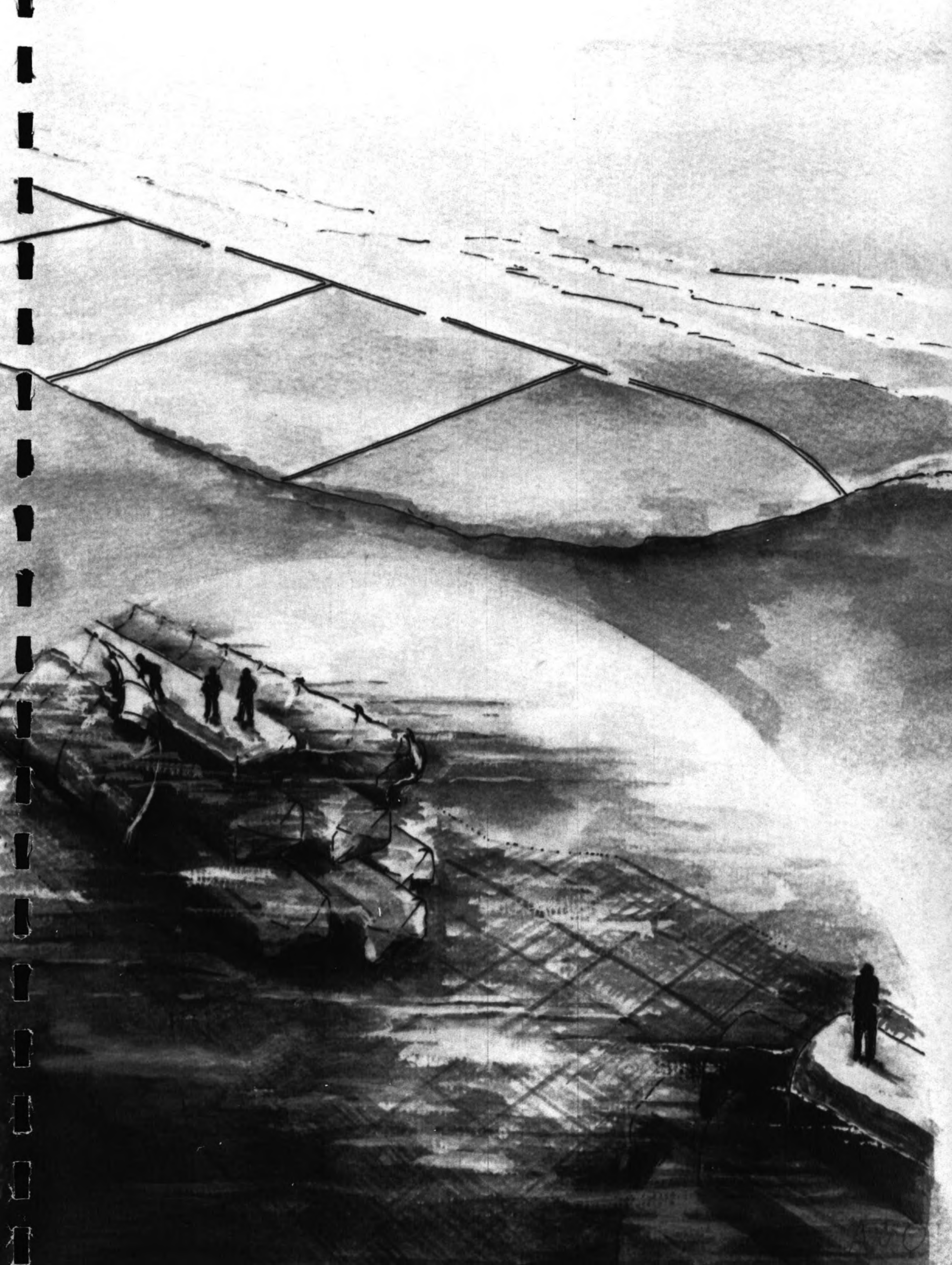


ROTTERDAM PUBLIC WORKS
Harbour Engineering Division

FEASIBILITY STUDY
LANDRECLAMATION SHANGHAI

RESPONSE AND STABILITY
of geotextile soil-filled units

Technical cooperation
Shanghai-Rotterdam

October 1987

6099E/s.939/IC

0. SUMMARY AND CONCLUSIONS

0.1 General

0.1.1 This report comprises the results of a study concerning the acceleration of sedimentation along the coast of the Cao Jing-district near Shanghai. Sedimentation can be stimulated by construction of a system of low dikes consisting of fine local sediments, packed by geotextile.

With respect to this matter S.B.W.C. (Shanghai) assigned Rotterdam Public Works (R.P.W.), on January 27th, 1987, to study the following aspects:

the required properties of geotextile tubes exposed to waves and currents:
- "size of the mesh, strength and durability (at least five years) of the geotextile cloth;
- the stability of the geotextile tubes filled with soil;
- the best construction processing on beaches including filling equipment, fullnes of tubes, the maximum water content when in filling and construction method".

0.1.2 Starting point has been a lay-out given by S.B.W.C. (Fig. 0.1). This lay-out has been detailed by Van den Berg in a thesis for the Technological University at Delft [lit (1)(2)]. From this thesis it appears that in order to stimulate sedimentation, the penetration of waves should be prevented. A longitudinal dike with openings in the longshore stretch, combined with cross-dams forms the best lay-out, in order to stimulate accretion (Fig. 0.2).

For the calculations next data were used:

Table 0.1	Normal conditions	Extreme conditions
- water-level (Wusong)	+ 0.0 - + 4.0 m	+ 5.54 m
- water-depth at the toe of the dike	0.0 - 3.0 m	4.54 m
- wave-height	0.25 - 0.80 m	3.60 m
- current velocity	0.30 - 2.0 m/s	1.0 m/s
- tidal range	3.90 - 4.20 m	5.00 m

Construction of the dam will take place during the non-flood period (October-April).

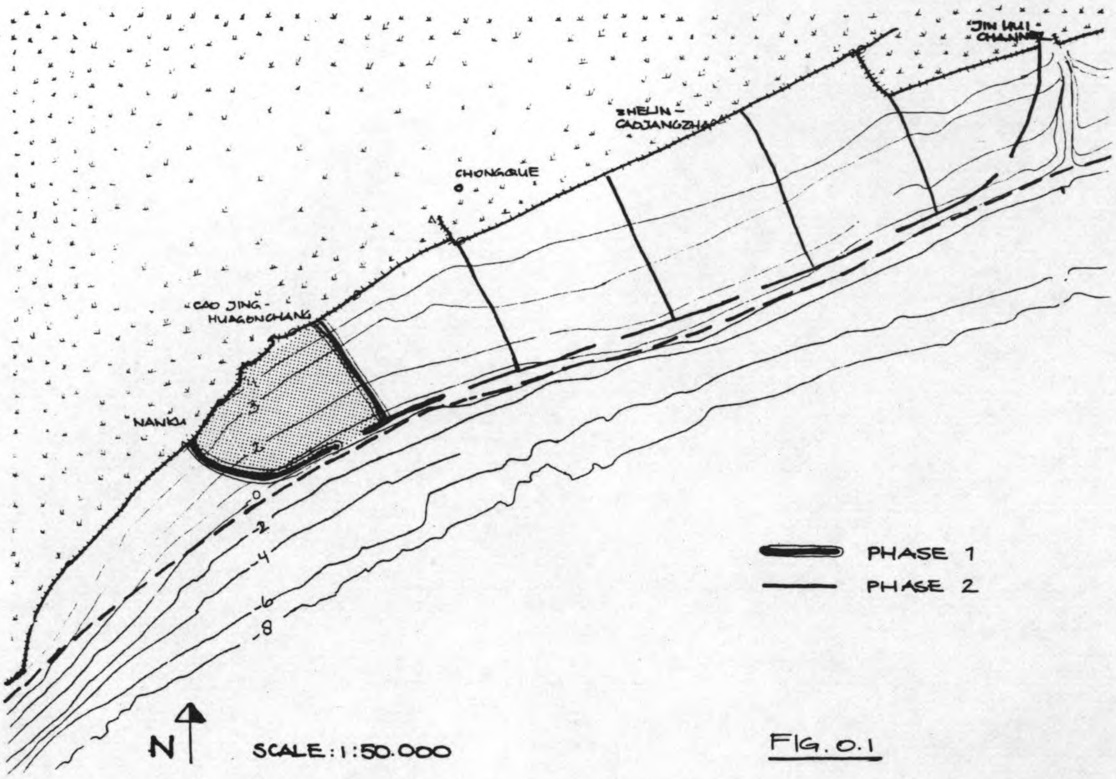
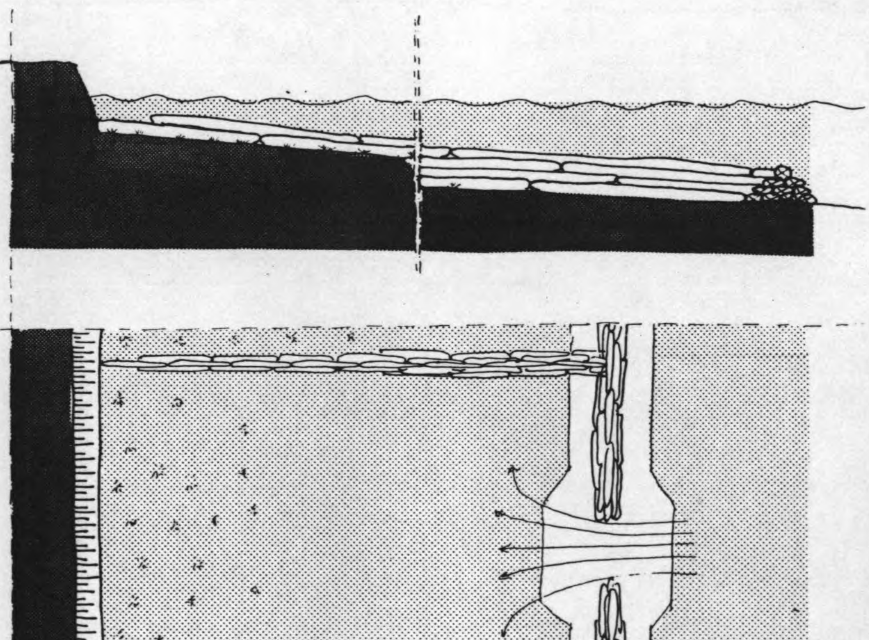


FIG. 0.1



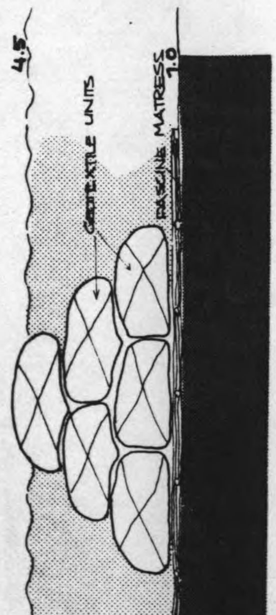


FIG. 0.2

FIG. 0.3

System of landreclamation

0.2 Wave loadings

0.2.1 The wave loadings will determine the size of the construction and the stability of the units (Fig. 0.3).
In the analysis a distinction has been made between wave attack on relative small bodies (diameter/wave-length < 0.2), and relative large bodies (diameter/wave-length > 0.2).

In case of small bodies the Morison-equation has been used. Using this theory it has been calculated the horizontal force on the construction (F_{hor}) and the vertical lifting force caused by vortex shedding (F_{vert}). It has been assumed that the tube, or the tubes, of which the dams are constructed can be considered as horizontal cylinders. Inertia forces play a more important role than the drag forces.

In the case of large bodies the influence of diffraction increases. The inertia forces are always dominant and the wave loadings are calculated using the potential flow theory.
The horizontal and vertical forces on the construction have been calculated this way. Coefficients have been determined taking into account that the construction is resting on the bottom.

0.2.2 In addition to the loadings mentioned above (drag forces, inertia forces and lift forces) the wave slamming loadings have been calculated. These loadings are caused by extreme local pressure on the units of the structure, which are alternately exposed to the air and next submerged in the water. From various investigations the study of RAMKEMA fits best with measurements in practice.

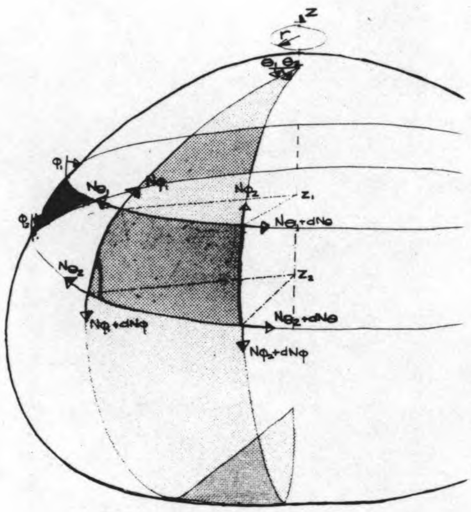
0.2.3 Summarizing the following forces, mentioned in table 0.2, occur during normal and during extreme wave attack. The calculations have been performed for a longitudinal dam, shaped like a horizontal cylinder with a diameter D of 3.50 m. The smallest units of which the dam possibly can be constructed, have been considered tubes with a diameter of 2 m. The forces have been calculated per running meter.

dike		normal conditions	extreme conditions
- D = 3.50	max. horizontal	35 kN/m'	100 kN/m'
	max. vertical	25 kN/m'	150 kN/m'
- D = 2.00	max. horizontal	15 kN/m'	40 kN/m'
	max. vertical	10 kN/m'	55 kN/m'
- slamming	maximum pressure	80 kN/m2	100 kN/m2

Table 0.2: wave-loadings on the construction

In the analysis the effect of currents has not been taken into account.

0.3 Response

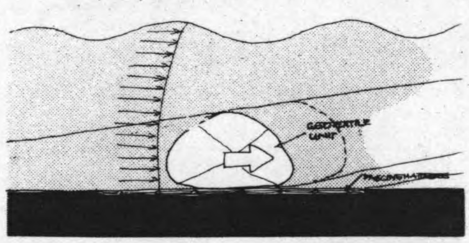


0.3.0 General

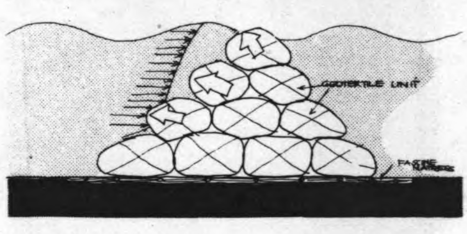
The response is the reaction of the dike to the loadings. These loadings are:

- static loadings caused by the pressure of the fill. They result in tensile stresses in the geotextile. The static load is always working in the same direction and is ever present. For this reason the safety factor of the geotextile against these gravitational loadings must be at least $\gamma = 2$;

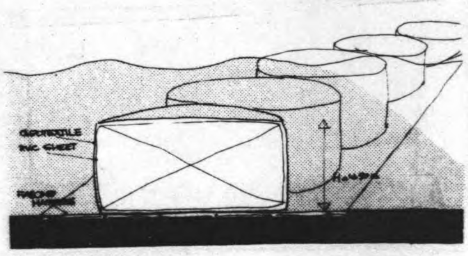
- dynamic loadings are caused by waves. The response includes displacements and small deformations of the dike, and extra stresses in the geotextile. Dynamic loadings have a random nature and the extreme circumstances are supposed to occur once in 10 years. In addition rather unfavourable starting points are used for calculations. For this reason the safety factor of dike and geotextile may be $\gamma = 1$.



Calculations are performed in chapter 3 for tube elements as well as cylinder-elements. Tubes have a horizontal axis; the diameter varies from 1 m in a dike composed of a number of several tubes until 7 m of diameter in case of the dike consisting of one single tube (n = 1). Cylinders have a vertical axis with a diameter of 5.5 to 9.0 m.

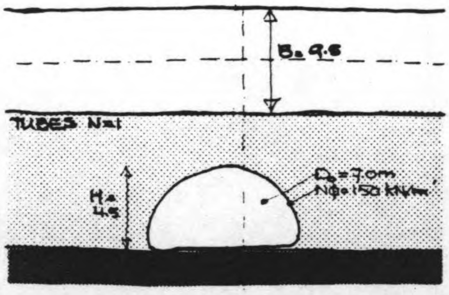


Since the top of the dike must be at a level of + 4.50 m and considering a settlement of the subsoil of about 1.0 m the construction height must be 4.50 m (the bottom of the dike is situated at 1+ m).

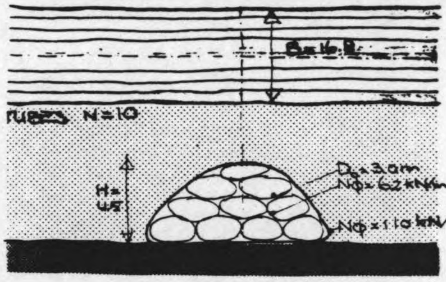


0.3.1 Tensile forces in the geotextile

In chapter 3.1 calculations have been performed. In the following table 0.3, some major tensile stresses under static and dynamic conditions are given for:

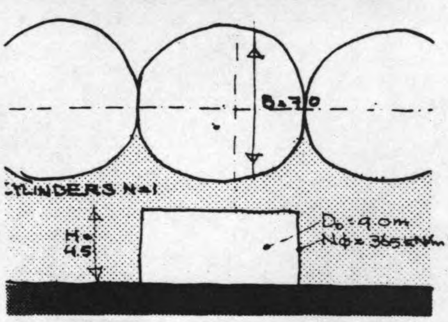


1. <u>single tube (n = 1)</u>
   - original diameter D_o = 7.0 m
   - height after deformation h = 4.5 m
   - fill A' = 35 m3/m', filling degree 75-80%



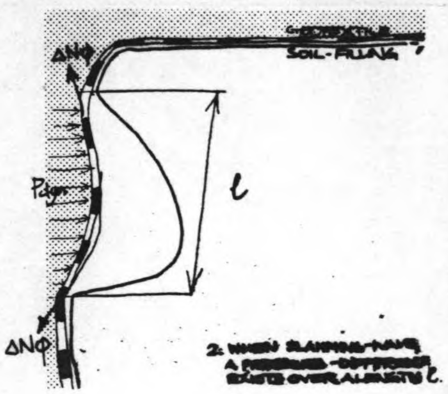
2. <u>Bottom tube in case of ten tubes (n = 10)</u>
   - original diameter D_o = 3 m
   - height after deformation h = 1.5 m
   - fill A' = 5 m3/m', filling degree 70%

3. <u>Envelope of 10 tubes</u>
   - original diameter = 9.0 m
   - height after deformation h = 4.5 m
   - fill A' = 50 m3/m', filling degree 70% (the same as under 2)



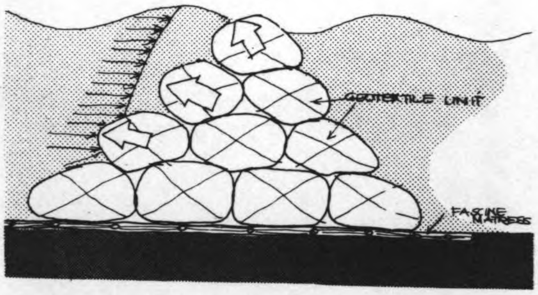
4. <u>Cylinders</u>
   - original diameter D = 9.0 m
   - height after deformation h = 4.5 m
   - fill A' = 32 m3/m'

Table 0.3	TENSILE FORCES				
	static kN/m'	ref. page	dynamic kN/m'	ref. page	strength of textile kN/m'
1. single tube	Nϕ = 125	56 + 90	Nϕ = 100	81/83/90	400
2. 10 tubes (bottom)	Nϕ = 62	60 + 90	-		62
3. envelope	Nϕ = 125	61 + 90	Nϕ = 100	81/83/90	320
4. cylinder	NΘ = 365	65 + 90	NΘ = 100	100 + 91	830



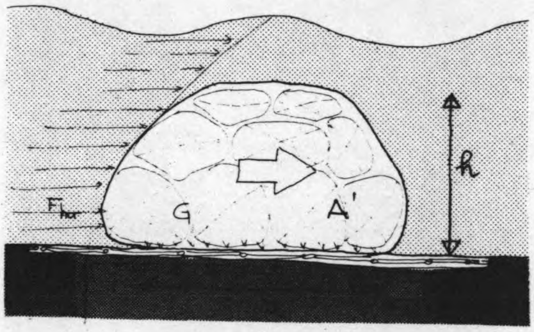
COMMENT

- Dynamic forces in the envelope are dependent on the non-supported length of a geotextile. This length will be about 1.5 m in case of n = 10. Due to the fact that an envelope consists of open-woven textile, dynamic pressures will be reduced. The applied reduction coefficient of 1/3 should be confirmed by laboratory investigations.



- The maximum tensile strength of the geotextile at the moment is, in case of:
  . polypropylene: 200 kN/m';
  . polyester : 1,000 kN/m'.

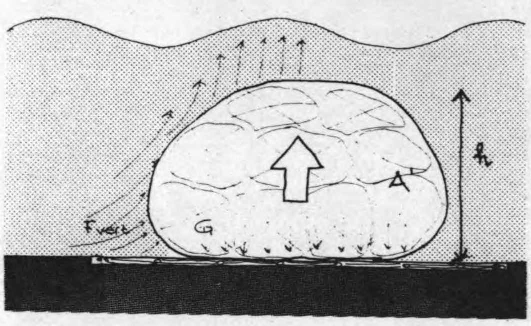
- Sewings are supposed to be feasible up to 200 kN/m'. Realized sewings have a strength up to 100 kN/m'.
  Some successive investigations have to be done in order to improve the strength of the sewings.



0.3.2 Equilibrium of the construction

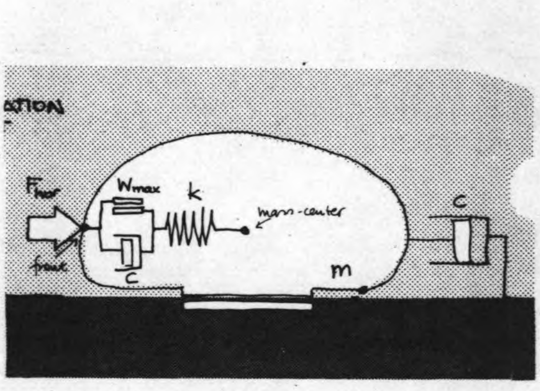
The responses of the components of the structure are approximated by calculating an upper limit or a lower limit on the basis of criteria:

- <u>The Hudson criterium</u> results in a tube diameter $D_o = 1.9$ m using a $K\Delta = 4$ and a very restricted co-operative length.
  The criterium also leads to a minimum content of one single unit of 5 m3.



- <u>Horizontal dislocations</u> are caused by the horizontal force component of wave loadings. This force must be smaller than the friction force. In order to prevent dislocation, the fill of the total dike must be at least about 30 m3/m'.

- <u>Lifting</u> of a unit is caused by the vertical component of wave loadings. To prevent lifting the minimum content of a total dike must be about 19 m3/m'.



- With a <u>spring mass</u> model the actual response to dynamic wave attack can be determined. The calculations result in minor cyclic deformations only. It becomes clear that softening of the filling by penetrating water is not allowed. For safety reasons it is important to construct a cross-section which consists of more than one unit.

- Erosion of the subsoil should be prevented by a fascine mattress.

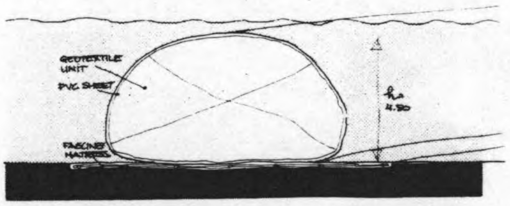
0.4 Possible types of constructions

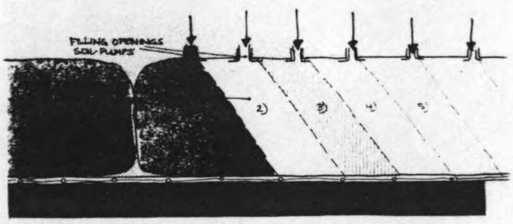
0.4.1 Based on the foregoing a number of possible constructions have been considered. Starting points for each of the designs have been:

- use of as little as possible geotextile cloth (the number of square meters influences the final costs considerably);

- a flexible straightforward way of building is necessary (the first 4 km of dike must be constructed during the first season).

0.4.2 The next constructions have been considered:

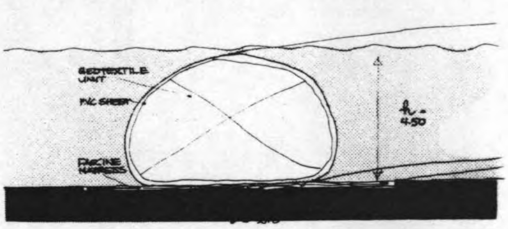


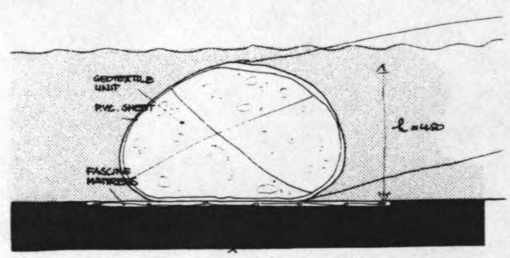


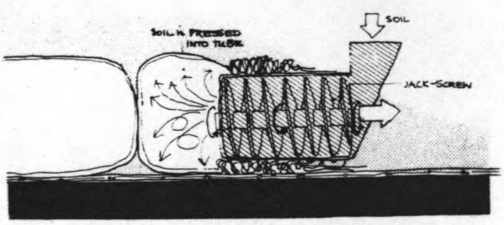
1. Cubatao method
A tube (with D = 8 à 9 m) is filled by several soil pumps working successively.

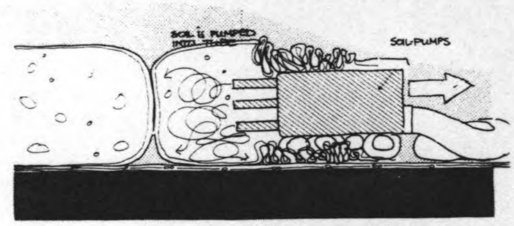
2. Jack-screw method
A movable jack-screw presses the soil (supplied by cranes) into the tube (D = 8 à 9 m) and moves itself.

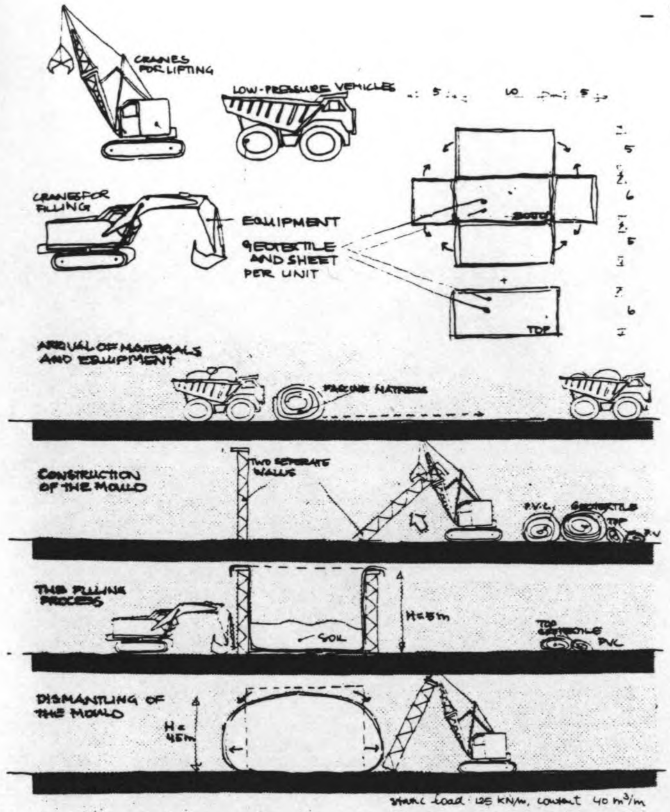
3. Pumping battery method
The tube (D = 8 à 9 m) is filled by a movable torpedo with a battery of soil pumps.







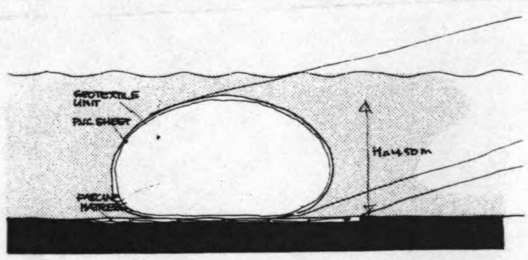


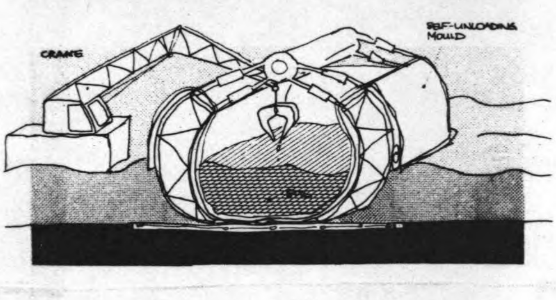


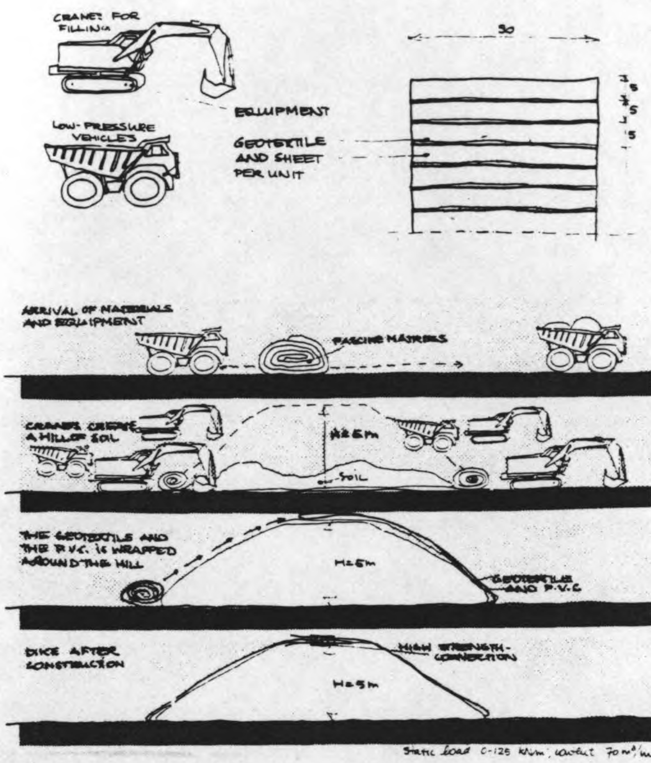
4. Mould method ▲
A mould, constructed from two separate walls which can be connected, is set up. Geotextile that is placed inside is sewed together after cranes have done the filling.
Length 10 à 30 m, width 8 m, height 5 m.

5. Self-unloading mould ▼
The mould unloads itself when lifted by a crane.

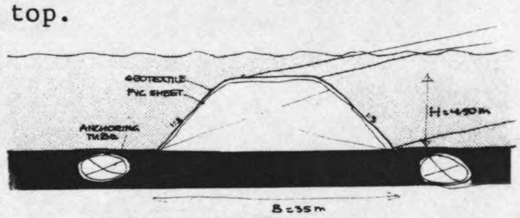
6. Injection method ▶
An anchored geotextile cloth is pumped up from underneath with a soil/water mixture.
Width about 50 m; height varying up to 3.50 m.

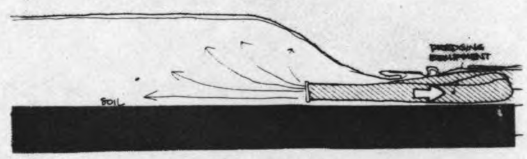


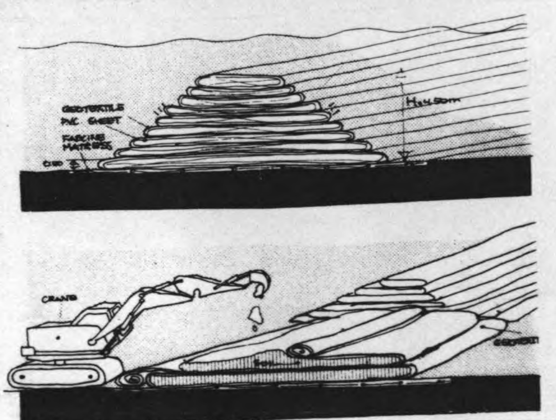


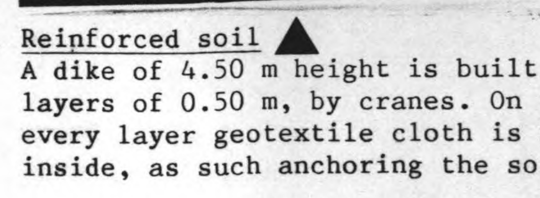


7. Packed soil ▲
Cloth of geotextile is placed and bulldozers and cranes push up a hill of soil upon the cloth. Afterwards cloth is tensed and tightened at the top.

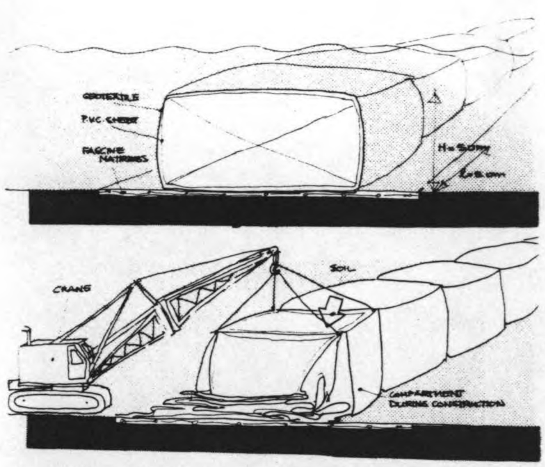




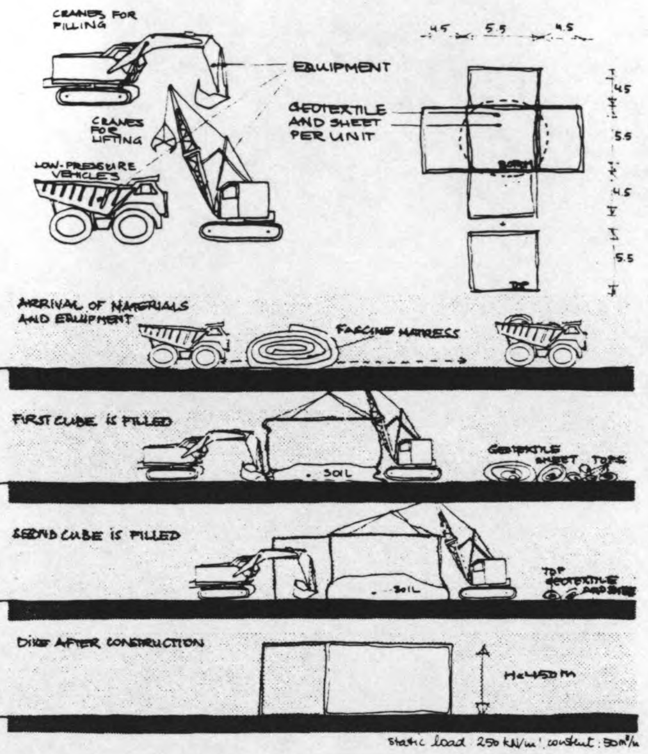




8. Reinforced soil ▲
A dike of 4.50 m height is built up in layers of 0.50 m, by cranes. On top of every layer geotextile cloth is folded inside, as such anchoring the soil.



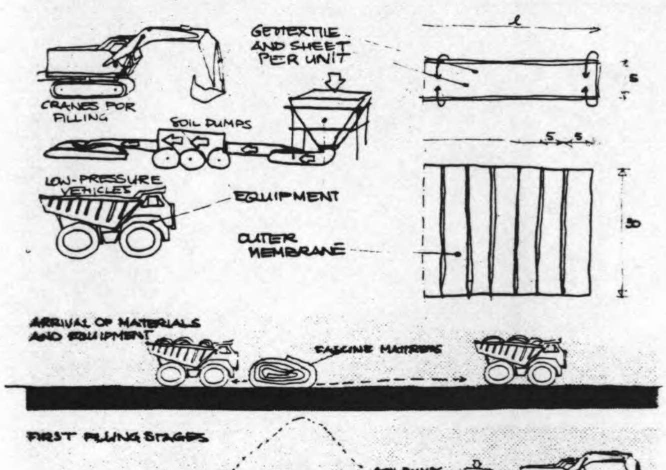
9. Compartment dam ▲
One crane fills a vertical cylinder, while another crane holds up the cylinder. The compartments are closed by sewing a cover.
Width 6 m, length 5 m, height 5 m.

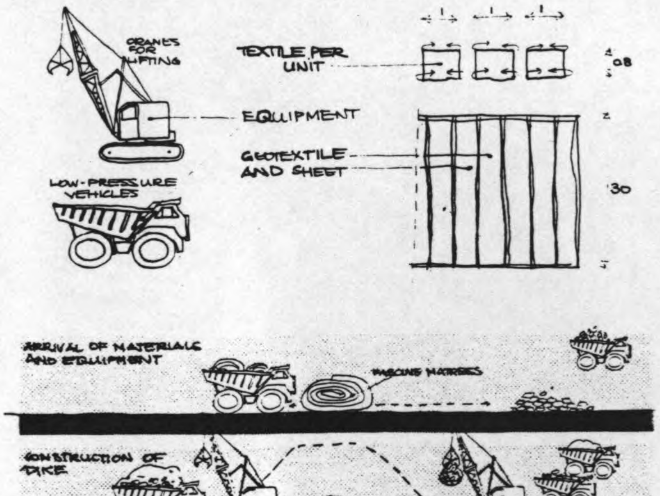


10. Cubes ▶
Instead of cylinders, cubes can be sewn in a more practical way. Filling is done in the same way as under 9.

11. Small tubes ▼
Prefabricated tubes are successively placed, one upon another, and filled with soil by pumping. When the proper height is reached a geotextile cloth is tightened around the pile and sewed.

12. Soil bags ▼
A similar construction as under 11, composed of soil filled bags.





0.5 Conclusions

0.5.1 Due to the (moderately) heavy wave attack, the longitudinal stretch with openings has a relatively important effect on (the increase of) the siltation. Calculations show a sedimentation being 6x as much as without such dams.

Cross-dams are necessary to prevent longitudinal currents in the bassin.

0.5.2 Overlooking the results of the foregoing, it seems technically possible to construct a system of cross-dams and longitudinal dams using geotextile soil filled units. It must be kept in mind that such a system is rather complicated to realize due to the following reasons:

- the tidal range of 4.0 m is rather large. This implicates that the seaward part of the system is exposed to wave attacks daily. Moreover the period of the tide which is available for constructing dams in-the-dry near the line of + 1.00 m is only 2 hours;

- the in-situ material is very fine-graded (average grain diameter about 50 μm). A geotextile which is open to water and tight enough to prevent loss of fine material doesn't exist at the moment. For this reason an inlay of very tight non-woven, or a waterproof plastic sheet is necessary in order to prevent loss of particles and softening of the fill;

- a construction consisting of wet soil of this type, will not have enough stability, so one has to use rather dry soil for the filling;

- S.B.W.C. has suggested that it was desirable to finish the western part of the system in one season, implicating a speed of construction of 30 m' per day, since most of the stretch consists of the longitudinal dam.

0.5.3 The examined constructions as described before can be compared as following:

	GEOTEXTILE		CONSTRUCTION				SAFETY		COST	FEASI-BILITY
	Nϕ	m2	↑↓	t	?	→	d	s		
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1. Cubatao-method	+	++	+	0	---	0	--	0	+++	-
2. Jack-screw method	+	++	--	-	---	0	---	0	+	-
3. Pumping battery	+	++	--	-	---	0	---	0	+	-
4. Mould method	+	+	+	+	0	0	-	0	+	+
5. Self-unloading mould	+	+	--	+	--	0	--	0	0	-
6. Injection method	0	-	+	-	0	0	-	---	-	-
7. Packed soil	+	0	+	--	+	0	-	0	+	++
8. Reinforced soil	++	--	+	+	0	+	+	--	-	-
9. Compartment dam	---	0	0	+	--	--	--	0	+	-
10. Cubes	---	0	+	0	+	--	-	0	+	+
11. Small tubes	++	--	++	0	++	++	++	++	-	+++
12. Soil bags	+++	---	++	0	++	++	+++	++	--	+++

Table 0.4: valuation of the alternatives

--- = inadequate
-- = very unfavourable
- = unfavourable
0 = moderate
+ = favourable
++ = very favourable
+++ = excellent

Interpreting this table one has to make a distinction between the cross-dams and longitudinal dams. The following conclusions and remarks can be made:

0.5.3.1 - <u>cross-dams</u>

. Cubatao, jack-screw, pumping battery and self-unloading mould are too complicated;

. the injection system and the reinforced soil system suffer from softening of the fill. Therefore they are not feasible;

. the compartment dam and the cubes are not interesting due to the restricted height of the larger part of the cross-dams, and the uneconomical required strength of the geotextile;

. remaining construction types are:
packed soil solution
small tubes solution
soil bags solution;

. at first sight the <u>small tubes</u> and <u>packed soil</u> seem to be favourites, as they both comprise relatively simple construction methods. In case of the packed soil system, it may be difficult to create watertight sealings in-situ;

. the soil bag type may be interesting near the LW-line using big bags of about 1 m3;

0.5.3.2 - <u>longitudinal dams</u>

. longitudinal dams are situated along the + 1 m-line. The construction height is 4.5 m. The compartment dam and the cubes are considered not feasible due to extreme tensile forces in the geotextile;

. using the mould or the packed soil type, the required static strength of the geotextile must be very high (350 kN/m') due to the static loadings. Geotextiles of this strength are available in Western Europe. Up till now the sewings are the weak point. They are realized up to 100 kN/m', perhaps 200 kN/m' is feasible. (A solution might be the use of double layers of geotextile. This might reduce the tensile forces of geotextile and sewing to 50%.)
As in addition the available daily working period is restricted to about 2 hours per tide, it will be almost impossible to mobilize sufficient equipment at the construction-site. This implicates that the mould solution and packed soil solution may be too difficult to create successfully;

. as possible solution there remain the small tubes solution and the soil bags solution. They could be filled at the higher part of the mud flat near the seadike. They can be transported to the site by low-pressure vehicles during LW Again the use of big soil bags can be considered.

0.5.3 Recommendations are:

- to check hydraulic and morphological boundary conditions;
- to check the response of the construction;
- to investigate the construction methods more profoundly;
- to examine the possibilities for the geotextile and the sewings;
- to perform intensive testings (see table 0.5).

The total project can be described, in time, by the following tabel

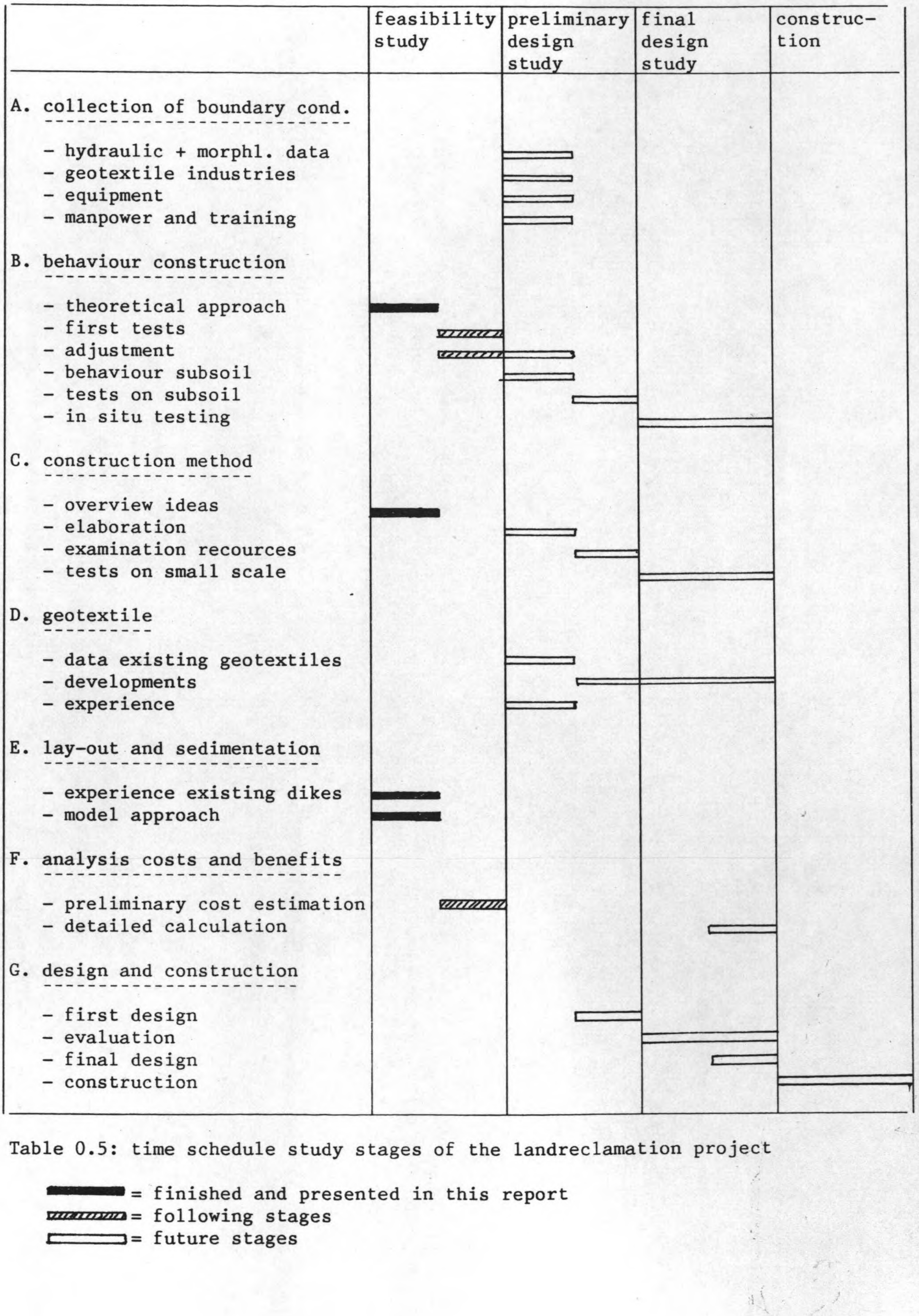
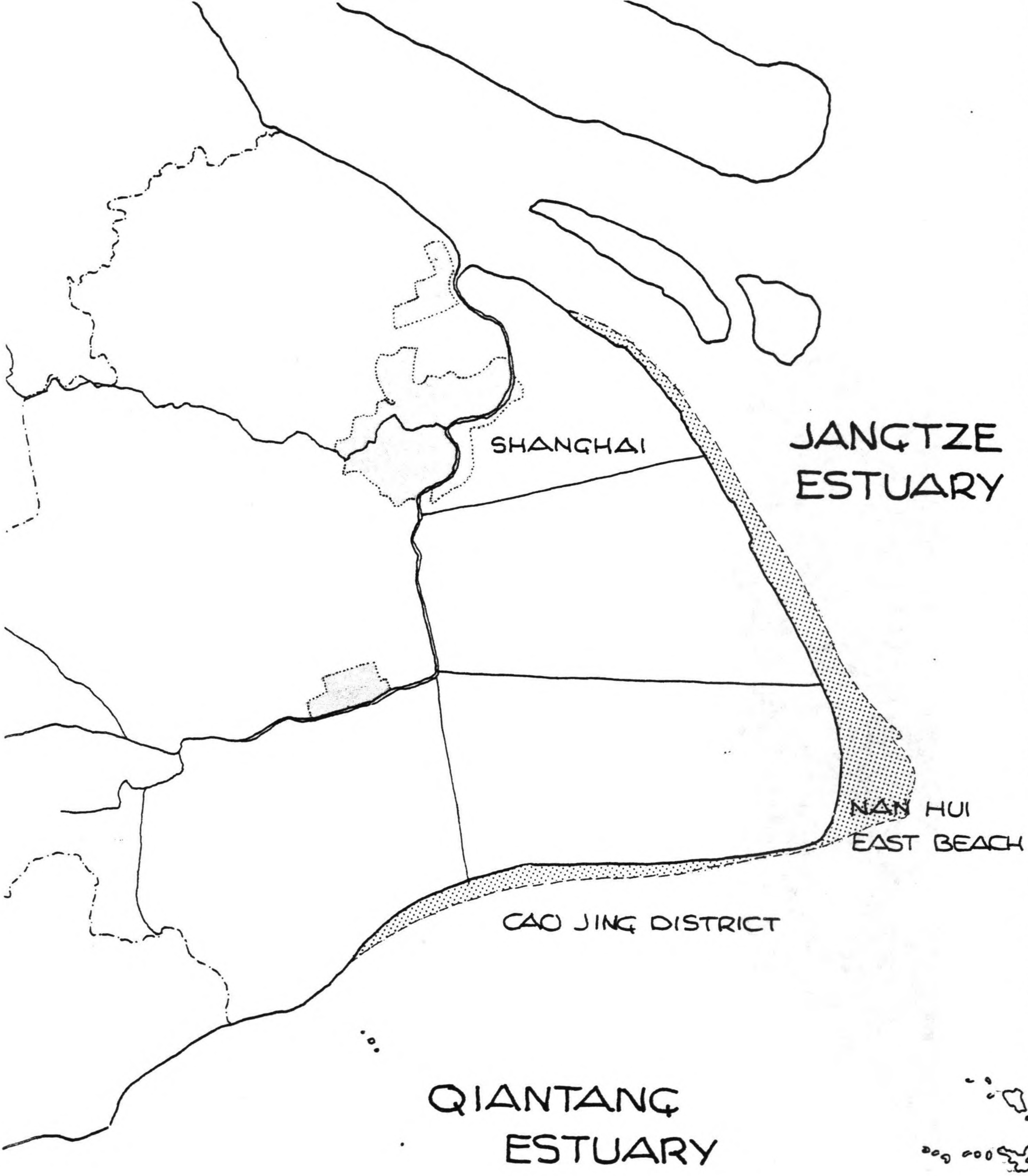


Table 0.5: time schedule study stages of the landreclamation project

= finished and presented in this report
= following stages
= future stages

CONTENT



1.1

Overview area

1. DESIGN

1.1 Problem

Along the coast of the Hangzhou-Bay at Shanghai Province a reclamation project is being prepared concerning particularly the Cao-Jing district (see fig. 1.1).

In the foregoing study "study about landreclamation" (lit. [3]) it was suggested to apply a system of cross-dams and longitudinal dams to stimulate the natural accretion. These dams then consist of geotextile membranes filled with soil (see fig. 1.2).

The next phase in this project is to perform a feasibility study resulting in a preliminary design of the landreclamation system. This study comprises the following three items:

A. LAY-OUT:

the location of the dams in relation to the expected sedimentation.
Relations might be found between the rate of siltation and the total length of dams.

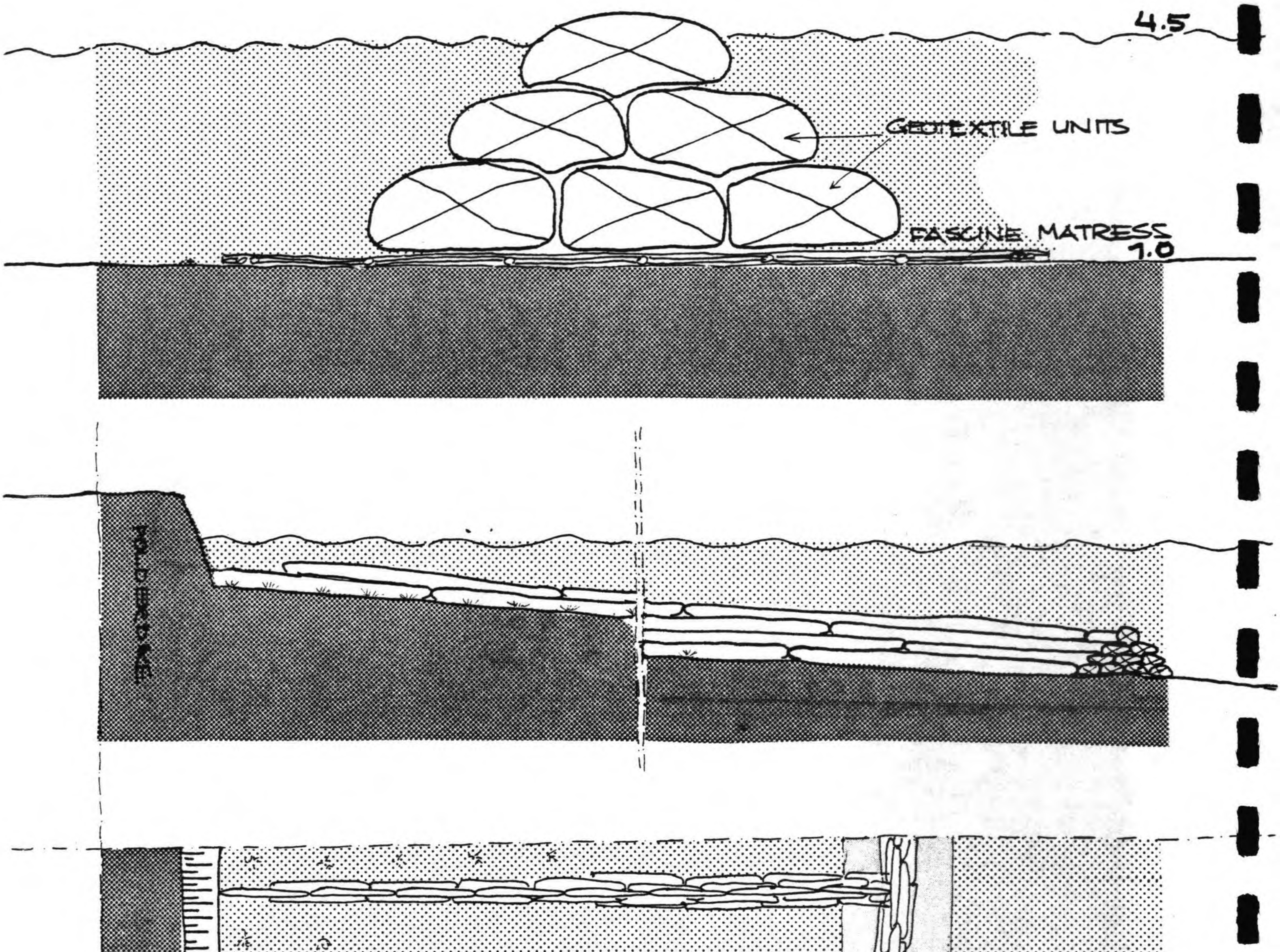
B. CONSTRUCTION DESIGN:

the dimensions of the soil filled tubes, the kind of geotextile required to resist loadings caused by waves, wind, currents and tidal action.
A relation might be found concerning the costs per m' cross-section and the chance of failure.

C. CONSTRUCTION METHOD:

ways to fill and handle the tubes in a way that they can be used as construction elements.
An optimization might be possible concerning the costs of the project related to the way of construction.

This report focusses on the second item: to determine the response of possible constructions to external loadings in order to design a stable landreclamation-dams.



1.2

System of landreclamation

1.2 Overview of the project

The mud flat at the Cao Jing-district will be the first in a series of landreclamation-projects over the province of Shanghai. The proposed area is bordered at the west-side by a prospective harbour entrance near to the petrochemical complex at Jinshanwei. At the east-side a very important water-supply channel discharges at the sea: the Jinhui Channel (see fig. 1.3).
As a result the projected area is limited and the original advice, to start the project directly eastward of Jinshan and to gradually extend it in eastward (and seaward) direction during following years, avoiding sharp edges and following the natural accretion pattern, can't be followed.

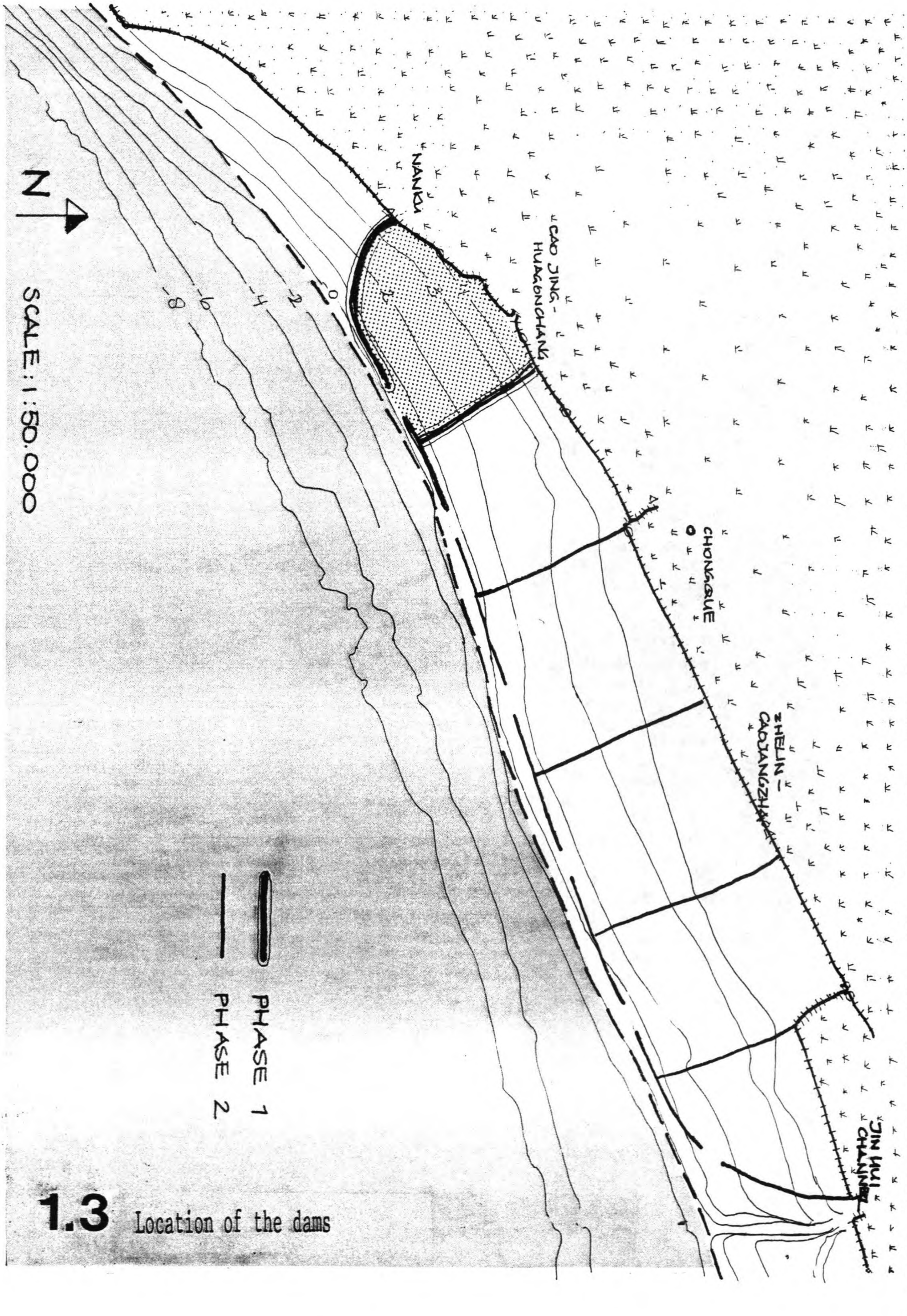
Studies and practical experience showed that, although the tidal movement carries the sediments along the coast with a longshore direction, a first requirement to stimulate accretion is to prevent the wave-action to penetrate into the reclamation zone resulting in a longitudinal dike. Cross-dams must be added to guide the longshore current away from the reclamation zone.
That way "basins" or reclamation-fields are formed in which the wave action is moderate, and sedimentation will occur.
The report "lay-out part 1" (lit. [1]) shows that it is economical to apply an inner-area of the basins, as big as possible. An upper limit is a distance of 1 à 1.5 km between the cross-dams, otherwise the watervelocities at the openings will cause damages. Inside the basins, large eddies will stimulate an equable accretion of 0.5 to 1.5 m per year. During the rising tide water, rich in sediment due to the strong tidal current and wave action, will enter the basins. The sediments will settle inside, leaving mainly water coming from the basins during declining tide.

Lit. [2], "lay-out part 2" shows that the top of the dike should be at least at a level of +4.50 m (Wusong*), in order to reduce the wave action effectively. The openings in the basins may not cover over 10% of the total stretch, in order to prevent the wave action from penetrating into the basins (see fig. 1.3).
Starting-point for the design of the lay-out is the requirement that the dike has to be constructed at the bottom contour of 1+ m or higher.

The project will be completed in two phases, starting with a stretch of 4 km at the west-side of the proposed area (see fig. 1.3); in following phases another 12 km of longitudinal and cross-dams must be constructed.

The total project comprises a length of 16 km longitudinal dikes and cross-dams. Construction will take place during the "non flood" period of the year: from October to April (moderate wave action and moderate tidal differences).

*): Wusong-level: the local reference level, at the Cao Jing-district this is comparable to the low water-level.



1.3 Location of the dams

1.3 Design criteria

1.3.1 Data from SBWC

The Shanghai Bureau of Water Conservancy (SBWC) has determined following requirements:

1. the construction of the dams will not take place below the bottom contour of 1+ m (Wusong-level);

2. data for design:

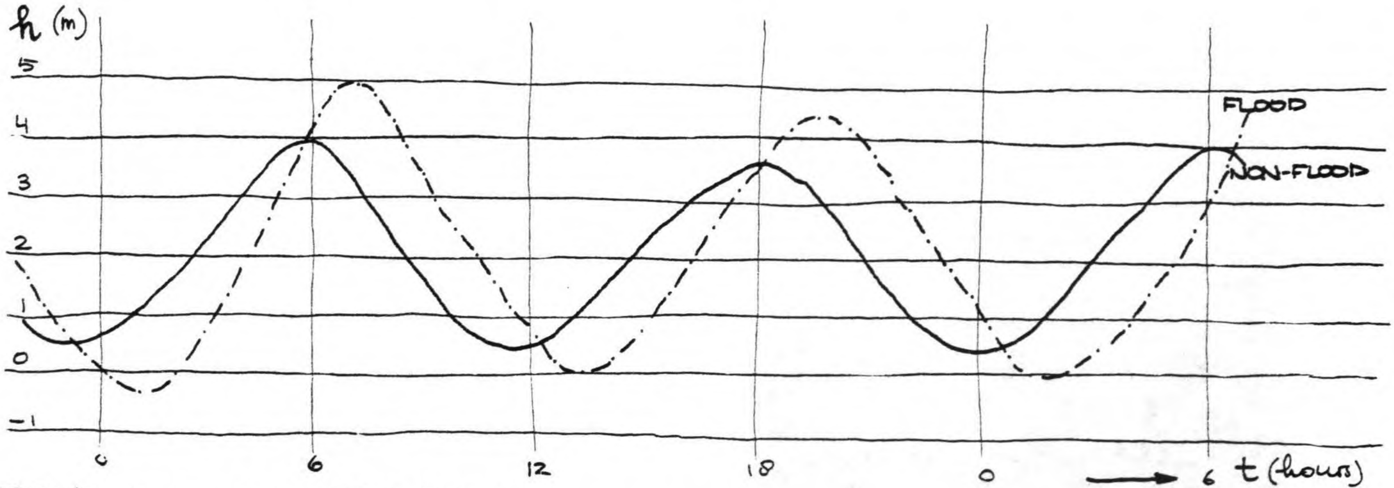
   - the water-level at this bottom contour which is exceeded once every five years is 5.37 m (Wusong-level); once every ten years is 5.54 m (Wusong-level);
   - the wave height at this bottom contour that is exceeded once every five years is 2.12 m; once every ten years is 2.54 m;
   - the average current velocity at this bottom contour is 0.43 m/s, the maximum current velocity is 1.0 m/s;
   - the average saltconcentration at this bottom contour is 0.01026 m3/m3;

3. considering a lifetime of at least five years, following characteristics have to be determined:

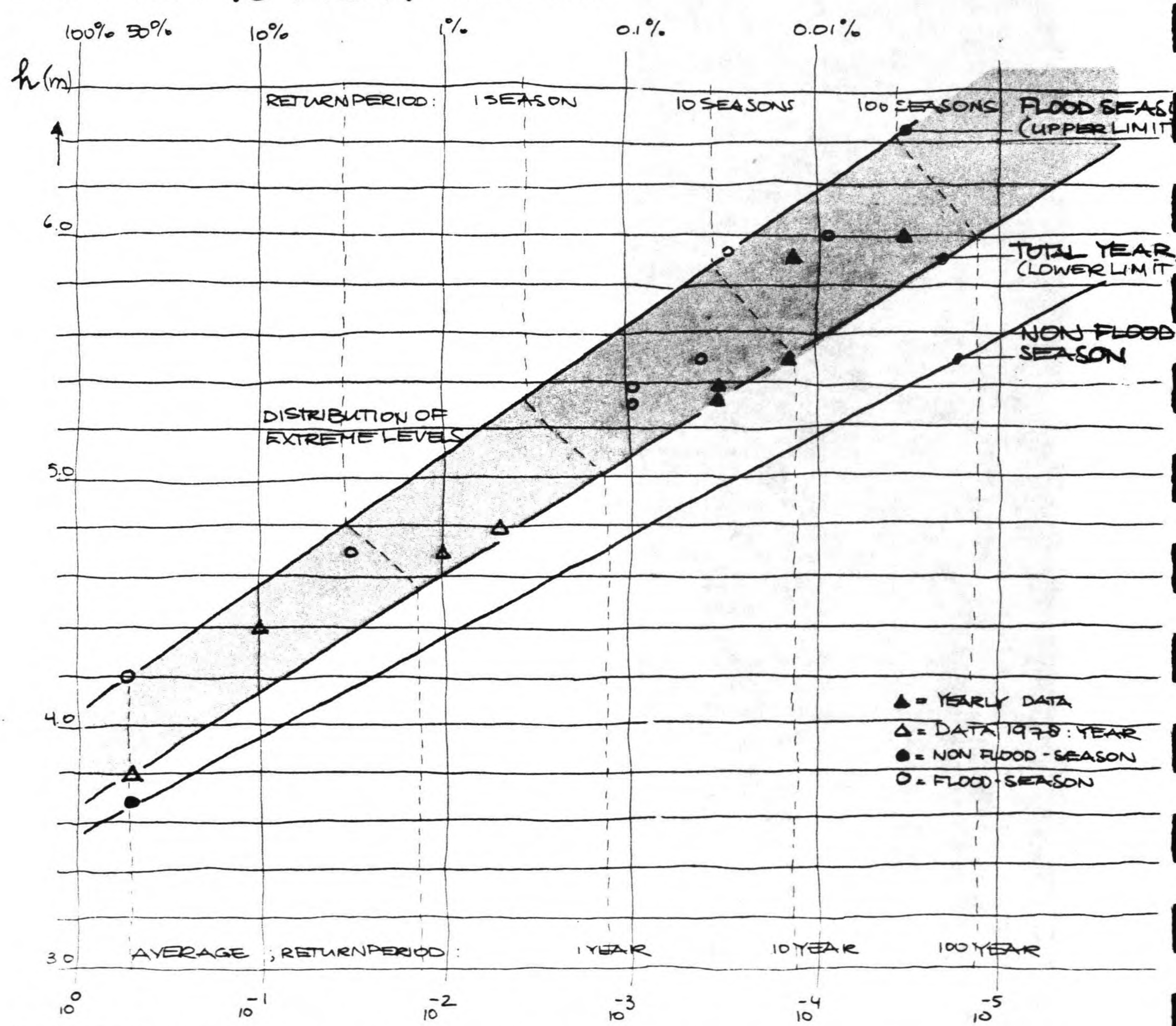
   - the dimensions of the geotextile units in order to guarantee stability;
   - the tensile strength of the geotextile;
   - the size of the mesh of the geotextile;
   - the durability of the geotextile cloth;
   - the best construction processing, including filling equipments, filling degree of the units and water-content of the filling.

These data and requirements originate from the telex d.d. 27-1-1987, about the agreement between Rotterdam Public Works and the Shanghai Burau of Waterconservancy on the subject of landreclamation.

Information about tidal motion current velocities and sediment concentrations can be found in the Chinese reports (lit. [4], [5], [6], [7]), which have been placed at the disposal of R.P.W. by S.B.W.C. (see also fig. 1.4 A and 1.4. B).



TIDAL MOTION
SOURCE : SEWC



DISTRIBUTION OF MAX. TIDAL LEVELS
SOURCE : FEASIBILITY STUDY, LAY-OUT PART 2

PROBABILITY OF EXEEDANCE

1.4a

No detailed data are available, however, for a first feasibility analysis it is sufficient to use the preliminary data given by SBWC and resulting from the studies in lit. |2| and |3|.
The final construction must be recalculated using up to date wave and tide measurements.

The longitudinal stretch of the dam (see fig. 1.3) will be subjected to wave action, wind and currents.

1.3.2 Extreme conditions

The tidal levels are distributed as shown in fig. 1.4.A. The criterion for construction the dike (lifetime at least 5 years) is chosen to be the fact that is has to survive a design-storm that occurs with a return period of 10 years.
This storm is caused by extreme winds (typhoon-conditions) so it is expected that in combination with a high water level, extreme wave action and currents will occur.

- WATER LEVEL:

the tidal level having a return period of 10 years: $h = +5.54$ m (Wusong)
(the waterdepth at the toe of the dike is: $d = 4.54$ m)
this level is given by SBWC.

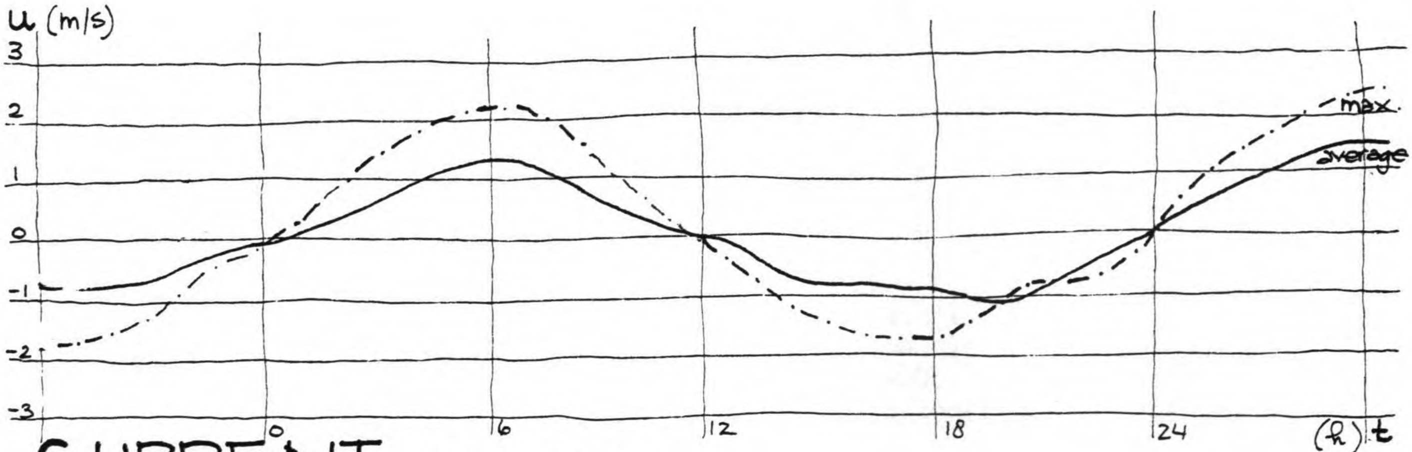
- SIGNIFICANT WAVE HEIGHT:

according to measurements and theory it is found that the significant wave height caused by strong wind, approaching a dike along a flat slope, is dependent only on the waterdepth:
$H_s \cong 0.5\ d$ resulting: $H_s = 2.50$ m.

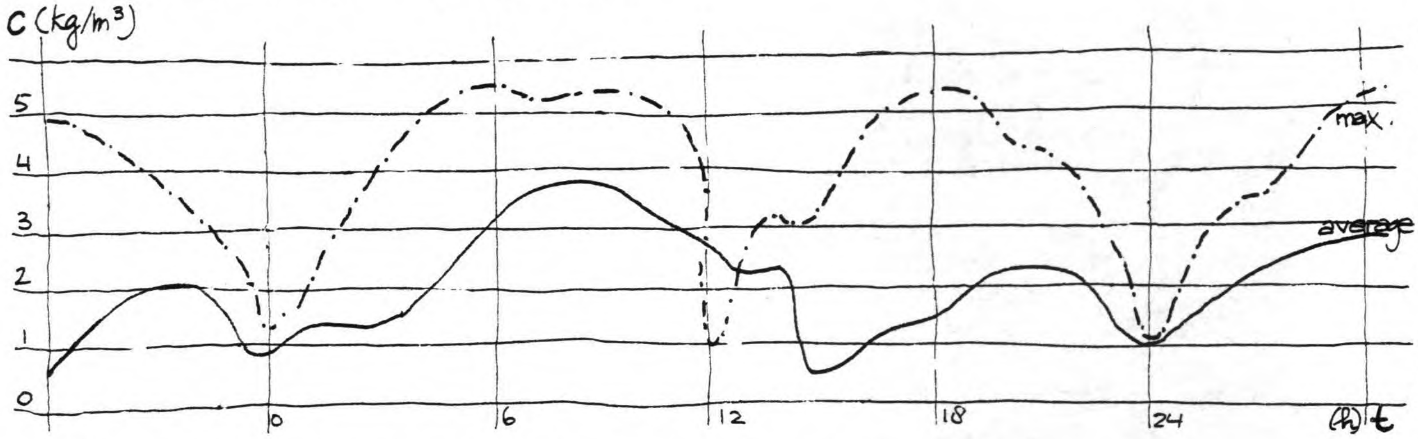
- MAXIMUM WAVE HEIGHT:

the maximum wave height is limited by the breaking criterion for waves in shallow water.
For beaches with a slope of 1:50 to 1:200 this breaking-criterion results in a maximum wave height dependent on the waterdepth:

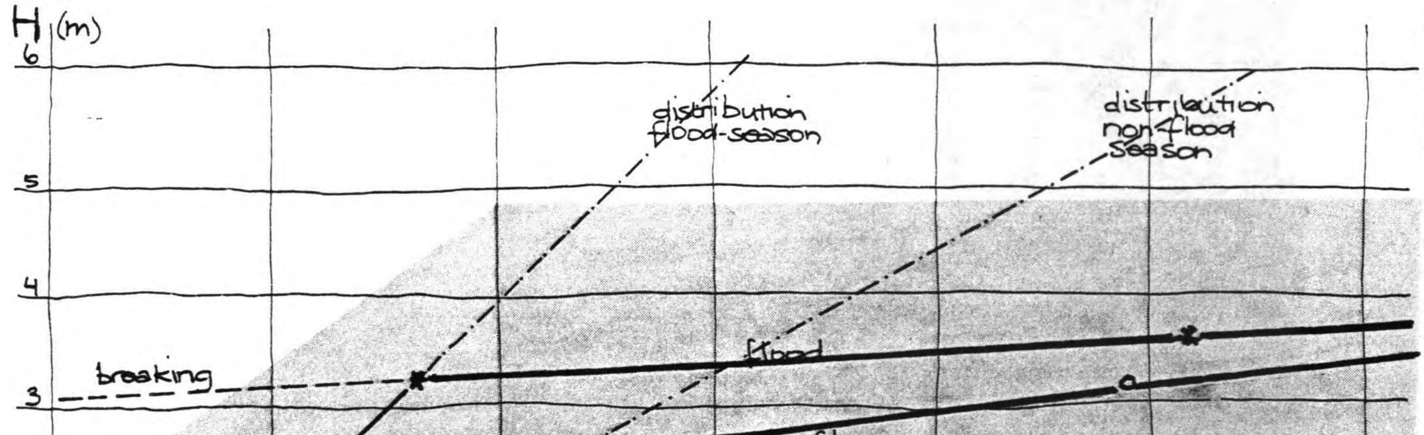
$H_{max} = 0.78\ d$ resulting: $H_{max} = 3.60$ m.



CURRENT SOURCE : SBWC



SEDIMENT CONCENTRATION SOURCE : SBWC



DISTRIBUTION OF WAVEHEIGHTS AT THE BOTTOM CONTOUR OF 1+M (WUSONG)
PROBABILITY OF EXEEDANCE SOURCE: FEASIBILITY STUDY, LAY-OUT PART 2

1.4 b

- CHANCE OF OCCURING:

the design-storm is a storm with a return period of 10 years: waterlevel $h = +5.54$ m.

The chance that the significant wave height $H_s = 2.50$ m occurs during this storm, can be expected to be 100%.

The chance that the maximum wave height $H_m = 3.60$ m occurs during this design-storm:

- wave heights during storm are Rayleigh-distributed;
- the high level of 5.54 remains during about 1 hour;
- average wave period during storm = $\bar{T} = 8$ sec.;
- number of independent waves during the high level N = number of waves or: $N \approx 400$

during any wave: $P\ (H \geq H_s) = e\ [-\ 2 \frac{H}{H_s}{}^2] = 0.016$

during 400 independent waves:
chance $= 1 - [1 - P\ (H \geq H_s)])^{400} \qquad = 0.998$

- CONCLUSION: the maximum wave height of 3.60 m will almost certainly occur during the design-storm.

Design-wave: $H = 3.60$ m (chance: 50% per 10 years);
design-level: $d = 4.54$ m (chance: 50% per 10 years);
design-current: $u = 1.0$ m/s (source: SBWC).

1.3.3 Normal conditions

Under "normal" conditions the longitudinal dikes will also be subjected to wave-acion, currents etc. (see fig. 1.4B). The figures show a substantial difference in waterlevels and wave-action during flood periods (May to September) and non-flood periods (October to April). This is mainly due to the fact that the prevailing wind-direction is south during flood periods and north during non-flood periods (seaward).

- WATERLEVEL (see fig. 1.4A)

The tidal difference during NON-FLOOD is about 3.5 m
(LW: +0.50 m, HW: +4.00 m (Wusong level))

the tidal difference during FLOOD is about 5 m
(LW: 0+00 m, HW: +5.00 m (Wusong level)).

- WAVE HEIGHT (see fig. 1.4B)

Outside the reclamation fields (seaward of the longitudinal dike) the wave climate (neglecting tidal motion):

during NON-FLOOD 90% of the waves are smaller than 0.75 m;
30% of the waves are smaller than 0.25 m;
the average wave height is 0.40 m;

during FLOOD 60% of the waves are smaller than 0.75 m;
16% of the waves are smaller than 0.25 m;
the average wave height is 0.60 m.

Inside the reclamation fields the wave height is mainly dependent on the windspeed and the height of the dams. A wave height higher than 0.25 m causes disturbances in the sedimentation. Summarizing the results of the report (lit. [3]) lay-out part 2 it can be found: including wave transmission, wave diffraction, and the tidal motion, the wave climate inside the reclamation fields (landward of the longitudinal dike) is, expressed in percentages of the total time:

during NON-FLOOD
if dike top at +4.50 m : 6 % of the time waves are higher than 0.25 m;
if dike top at +8.00 m : 4.2% of the time waves are higher than 0.25 m;
natural situation (+1.00 m): 28.8% of the time waves are higher than 0.25 m;

during FLOOD
if dike top at +4.50 m : 12.0% of the time waves are higher than 0.25 m;
if dike top at +8.00 m : 2.6% of the time waves are higher than 0.25 m;
natural situation (+1.00 m): 44.3% of the time waves are higher than 0.25 m.

On the basis of this report, the chosen level at which the dike top will be constructed, is +4.50 m. The height of the construction will be 3.50 m.

- CURRENT (see fig. 1.4B)

The current velocity is dependent on the tidal motion and fluctuates harmonically. The maximum tidal fluctuation occurs at springtides and the minimum fluctuation at neap tides (source: lit. (4)).

Spring tide: flood-current velocity = 2.0 m/s;
ebb-current velocity = 1.0 m/s;

Neap tide : flood-current velocity = 0.7 m/s;
ebb-current velocity = 0.4 m/s;

the average flood-current velocity is 0.4 m/s;
the average ebb-current velocity is 0.3 m/s.

- SEDIMENTTRANSPORT (see fig. 1.4B)

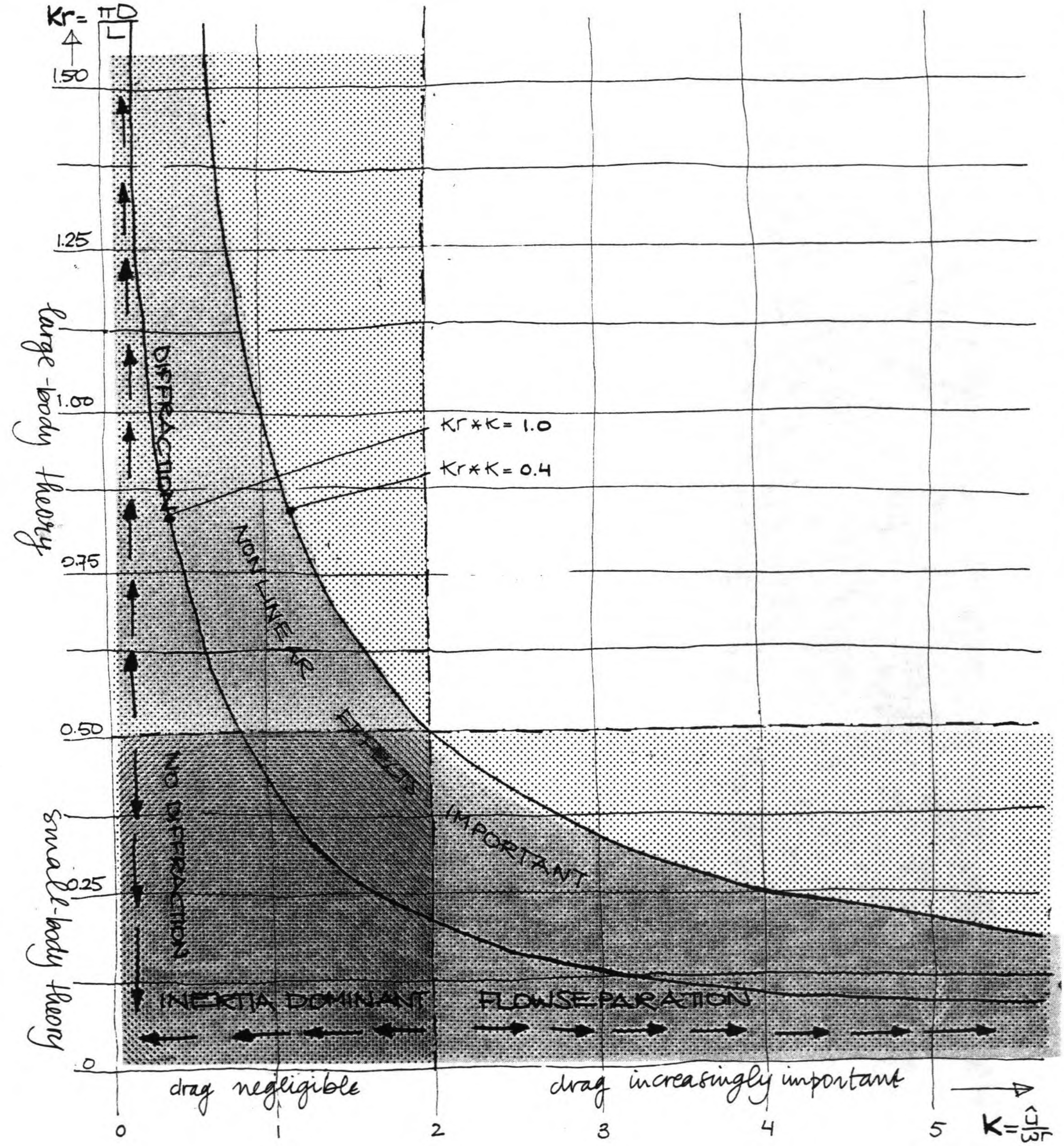
The sediments are carried along the coast by the tidal currents, orginating from the Yangtze river. The concentrations are more or less dependent on the season, and are dramatically changing at spring tides and neap tides (source: lit. (4)).

Spring tide flood sediment-concentration = 3.0 kg/m3;
ebb sediment-concentration = 2.0 kg/m3.

Neap tide flood sediment-concentration = 0.5 kg/m3;
ebb sediment-concentration = 0.05 kg/m3

the average sediment-concentration is 0.70 kg/m3, during spring tides the sediment-concentrations can reach up to 6 kg/m3.

More specific information can be found in the Chinese reports: "Report on geological survey at the North bank of the Hangzhou-Bay" (lit. [7]) and "Erosion and sedimentation processes at the North bank of the Hangzhou-Bay" (lit. [5]) and "Hydrological and morphological characteristics of the North bank of the Hangzhou-Bay" [lit. 6]) and "Analysis on the charasteristics of the sedimentation proces in Cao Jin and conditions for stimulation of accretion" (lit. [4]).



2.1

Forces on structures: classification

2. WAVE-LOADINGS

2.1 Determination of wave-loadings

In order to determine whether a possible dike will survive the design-storm, as described in chapter 1, it is important to determine the loadings on the construction. The dike is subjected to the following loadings: the gravitational loading, which has a static nature; furthermore the dike is subjected to wave-loadings, which have a dynamic character. It is expected that the wave-loadings will determine the final size of the construction, and the necesary stability of the units.

2.1.1 Classification

In this analysis, the wave-theory that has been used to determine the particle-kinematics, is a linear theory: the "airy-wave theory". In fig. 2.1 it shows that higher order theories (such as Stokes, cnoidal theory) are necessary in the range of the "steep shallow-water waves" (or Kr * K > 0.4). It will be shown that our design-waves are not within this range.

In general, wave-forces on constructions can be classified according to fig. 2.1,

1. the small-body-theory:

   the waves are not diffracted when passing a relatively small construction.
   Forces can be calculated using the "Morison-equation".
   Flow separation (development of vortices etc.) becomes increasingly important for relatively larger waves.
   The forces consist of drag forces and inertia forces;

2. the large-body-theory:

   the construction forms an obstacle for the waves, which are diffracted by the obstacle. Drag and flow separation are negligible, for increasing size, the flow can be approached as being a potential flow. The force is only an inertia force.

The classification of wave-loadings is determined by the following important wave-parameters (see <u>fig. 2.1</u>).

I. <u>Relative size:</u>

a parameter which describes the dimensions of the construction relative to the wave-length:

$$Kr = \frac{\pi D}{L} \;\ldots\ldots\ldots \left(\text{or } \frac{D}{L}\right) \ldots\ldots \left(\text{or } \frac{2\pi . \tfrac{1}{2}D}{L} = Kr\right) \ldots\ldots\ldots\ldots (1)$$

D = representative structure-diameter [m]
L = wave-length [m]

if Kr << 0.5: structure is small compared with the wave, it doesn't obstruct the wave, the watermovement can be considered to be quasi-uniform

if Kr $\simeq$ 0.5 : the structure does cause some dispersion of the waves, wave diffraction and transmission become important features, the forces can be determined by a non-linear diffraction approach and Morison-approach

if Kr >> 0.5: the structure is very big compared with the waves, the forces can be determined by a combined potential/diffraction analysis.

II. <u>Relative motion:</u>

a parameter which describes the displacements of the water (wave-amplitude) relative to the size of the structure:

$$K = \frac{\hat{u}}{\omega r} \;\ldots\ldots\ldots\ldots \left(\text{or } KC = \frac{\pi \hat{u}}{\omega r}\right) \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$\hat{u}$ = amplitude of watervelocity [m/s]
ω = angular velocity of watermovement [rad/s]
r = representatieve radius of the structure (½ D) [m]

this number K (or Keulegan-Carpenter number $KC = \frac{\pi \hat{u}}{\omega r}$)

can also be written as $K = \frac{\hat{u}/\omega}{r} = \frac{a}{r} = \frac{H}{D}$ .... (3)

(a = wave-amplitude = ½ H) to illustrate the above mentioned) if $K << 2$: the motion of the waves is much smaller than the structure-dimensions. The effect of drag (forming of wakes, vortex shedding) is negligible, inertia is far dominant and the mass-coefficient, C_m, can be appointed 2

if $K \simeq 2$: both drag and inertia-effects are important, flow separation is not occurring yet, but becomes increasingly important with upgoing K

if $K >> 2$: the motion of the waves is much larger than the construction, the flow becomes quasi-stationair, drag forms the main force on a structure.

III. Reynolds number:

a parameter which describes the inertia-forces compared with the viscous shear forces

$$Re = \frac{\hat{u}\, D}{\nu} \qquad (4)$$

D = representative structure-diameter [m]
$\hat{u}$ = velocity amplitude [m/s]
ν = dynamic viscosity $\frac{\eta}{\rho}$ (-) [m2/s]

this Reynolds number Re can also be written as:

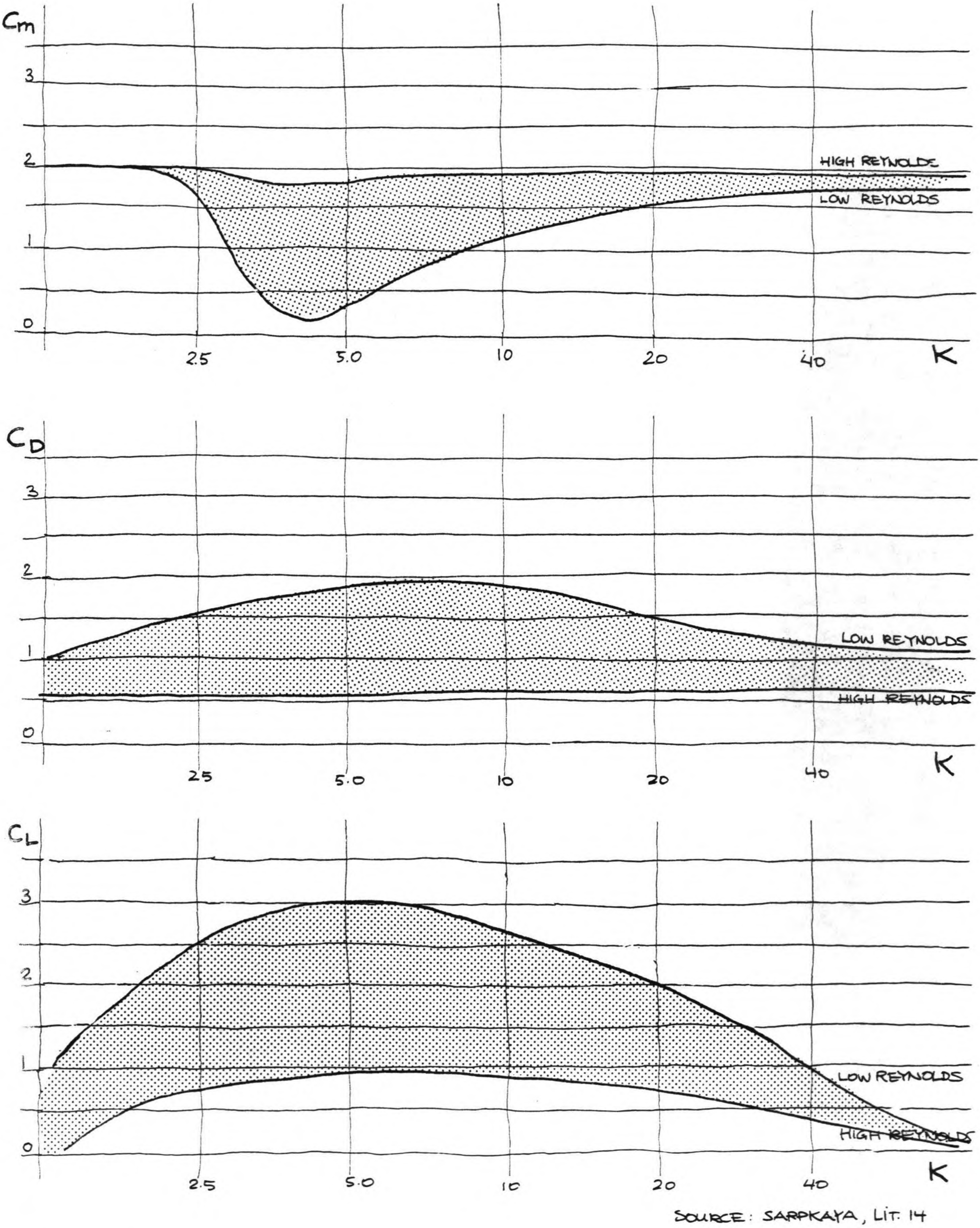
$$\frac{\hat{u}L}{\nu} = \frac{\rho L^3 \text{ (order of mass)} \cdot \frac{u^2}{L} \text{ (order of convective acceleration)}}{\eta \frac{u}{L} \text{ (viscous shear stress)} \cdot L^2 \text{ (surface)}} = \frac{\text{inertia}}{\text{viscous shear}}$$

in which L is a representative length over which the velocity changes significantly ($L \simeq D$) $\qquad (5)$

if $Re < 100$: the boundary layer as well as the vortex street of the flow around the cylinde is laminar: viscous forces are dominant

if $Re < 10^5$: the boundary layers are laminar, but it exists as turbulent vortex trail (if $Re > 10^5$ then also the boundary layer becomes turbulent)

if $Re > 1.5.10^6$: both turbulent boundary layer and regular vortex shedding



2.2

C_m, C_D, C_L versus K

The relative size number Kr is related to the relative motion number K via the maximum wave-velocity. An upper limit forms the fact that the maximum water-velocity cannot exceed the wave propagation velocity (otherwise waves will break).

$$u_{max} \leq C = \frac{\omega}{k} \quad \ldots\ldots (6)$$

$$K_{max} = \frac{\omega L}{2\ \pi\ \omega r} = \frac{L}{\pi\ D} = \frac{1}{Kr} \quad \ldots\ldots (7)$$

C = wave propagation velocity [m/s]
k = wave number (= 2 π/L) [rad/m].

Thus the maximum theoretical value of Kr * K is limited: 1 (see fig. 2). Practice shows that waves with a Kr * K-value of over 0.4 hardly exist. Non linear wave-theory must then be applied to calculate the forces (see Sarpkay, lit. (14)).

2.1.2 Small bodies

The forces on structures subjected to waves are classified according to fig. 2.1. In the case of "small bodies" (D/L < 0.2) the "Morison-equation" can be applied to calculate the in-line forces:

$$F_{hor} = \underbrace{C_D\ \tfrac{1}{2}\rho\ A_p |u| u}_{\text{drag force}} + \underbrace{C_m\ \rho\ \forall \frac{\partial u}{\partial t}}_{\text{inertia force}} \quad \ldots\ldots (8)$$

u = horizontal particle-velocity [m/s]
C_D = drag coefficient (see fig. 2.2) [-]
C_m = mass coefficient (see fig. 2.2) [-]
Ca = added mass coefficient (= cm-1) [-]
ρ = density of fluid [kg/m3]
A_p = projected frontal area [m2]
$\forall$ = displaced volume of structure [m3]
F_{hor} = horizontal force [N].

The Morison-force decomposes the in-line forces into two components: a drag force (increasingly important for decreasing Reynolds numbers and increasing K numbers) and an inertia force (increasingly important for decreasing K numbers) (see fig. 2.2).

If the boundary-layers are becoming turbulent, vortices will develop behind the construction. If the water-velocity is high enough, these vortices will leave the body in alternating direction (vortex-shedding). That way a time-dependent force is working on the body: a lift-force. This force is directed perpendicular to the stream-direction.

$F_{lift} = C_L \, ½\rho \, A_p u^2 \sin 2\pi f t$ .... (9)

C_L = lift coefficient [-]
u = horizontal particle-velocity [m/s]
f = frequency of the vortex-shedding [s^{-1}].

If we take into account that the vortex-frequency will be twice the wave-frequency, the lift force can also be written as:

$F_{lift} = C_L \, ½\rho \, A_p \, u|u|$ .... (10)

In the case the body is assumed a cylinder, with the length-axis horizontal and perpendicular to the wave-direction, the Morison-equation will become:

$F_{hor} = ½\rho \, C_D \, \pi D\ell \, u|u|$ .... (drag force)

$+ ¼\rho \, C_m \, \pi D^2 \ell \, \frac{\partial u}{\partial t}$ .... (inertia force) .... (11)

The next step is to choose an appropriate wave-theory to determine the particle kinematics.
Since the waves in this analysis are classified in the linear region the "AIRY-WAVE-THEORY" is applied;

$u = \hat{u} \sin \omega t$ (hor. velocity) .... (12A)

$\frac{\partial u}{\partial t} = \hat{u}\omega \cos \omega t$ (hor. acceleration) .... (12B)

u = horizontal velocity of the water-particle [m/s]
$\hat{u}$ = amplitude of the horizontal particle-velocity [m/s]
ω = angular velocity of the wave: $2\pi/T$ [rad/s]
t = time-variable [s].

Then the Morison-equation can be re-written:

$F_{hor} = ½\rho C_D \ell \hat{u}^2 \sin\omega t|\sin\omega t|$ ...(drag)

$+ ¼\rho C_m D^2 \ell \hat{u}\omega \cos\omega t$ ......(inertia) .... (13A)

$F_{vert} = ½\rho C_L D\ell\hat{u}^2 \sin\omega t|\sin\omega t|$ (lift) .... (13B)

D = diameter of cylinder [m]
ℓ = length of cylinder [m]
ρ = density of water [kg/m3].

Thus the forces are determined by the coefficients C_m, C_D and C_L and the particle kinematics.

2.1.3 Large bodies

In the case of "large bodies" (D/L > 0.2) diffraction becomes an increasingly important factor in the wave forces which are calculated using the potential flow theory:

$$\phi = \phi_w + \phi_s \quad (14)$$

ϕ = velocity potential [m]
ϕ_w = incident wave potential [m]
ϕ_s = scattered wave potential (diffraction!) [m]

according to airy-wave theory:

$$\phi_w = - \frac{igH}{2\omega} \cdot \frac{\cosh (k(z+d))}{\cosh (kd)} e^{i(kx-\omega t)} \quad (15)$$

ϕ_s = dependent on the problem

k = wave number [rad/m]
ω = wave frequency [rad/s]
d = water depth [m]
z = vertical coordinate [m]
H = wave height [m]
g = acceleration of gravity [m/s^2]
i = unity of imaginary system: $\sqrt{-1}$ [-].

In order to solve the force-problem, following boundary-conditions can be introduced:

$$1) \ \frac{\partial \phi s_n}{\partial n} = - \frac{\partial \phi w}{\partial n} \text{ at body surface} \quad (16)$$

and since the force has to damp out at infinity:

$$2) \ \lim_{r \to \infty} \sqrt{r} \left[\frac{\partial \phi s}{\partial r} - ik \ \phi s \right] = 0 \quad (17)$$

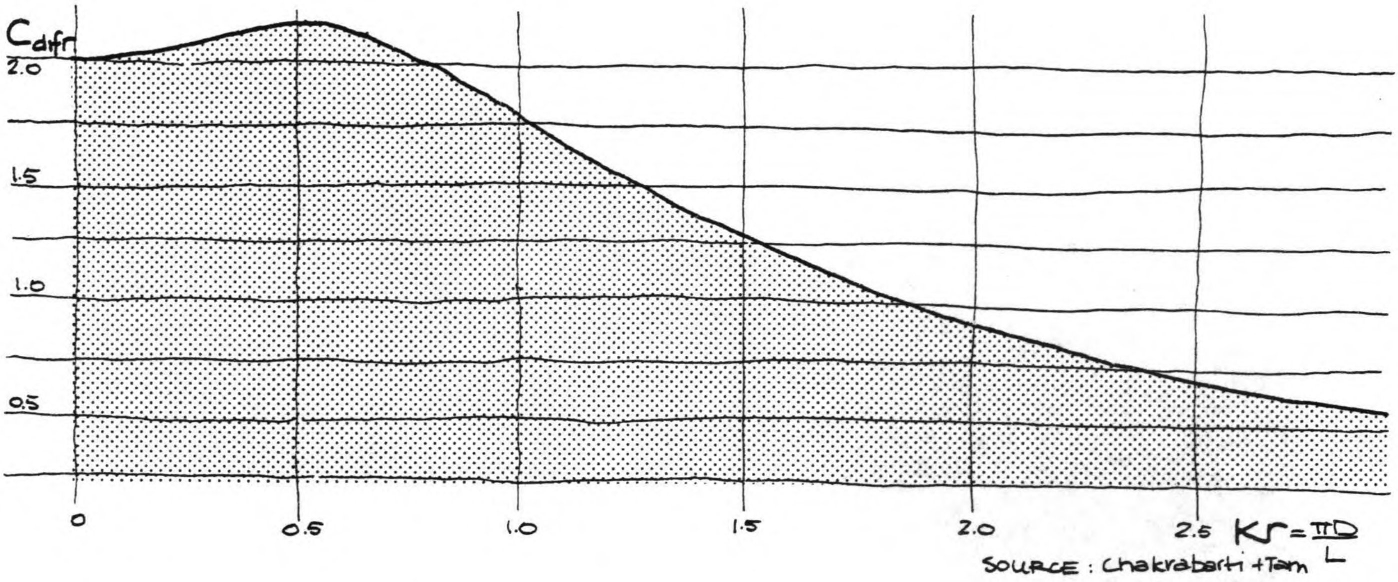
r = distance from the origin of the x and z-axes [m].

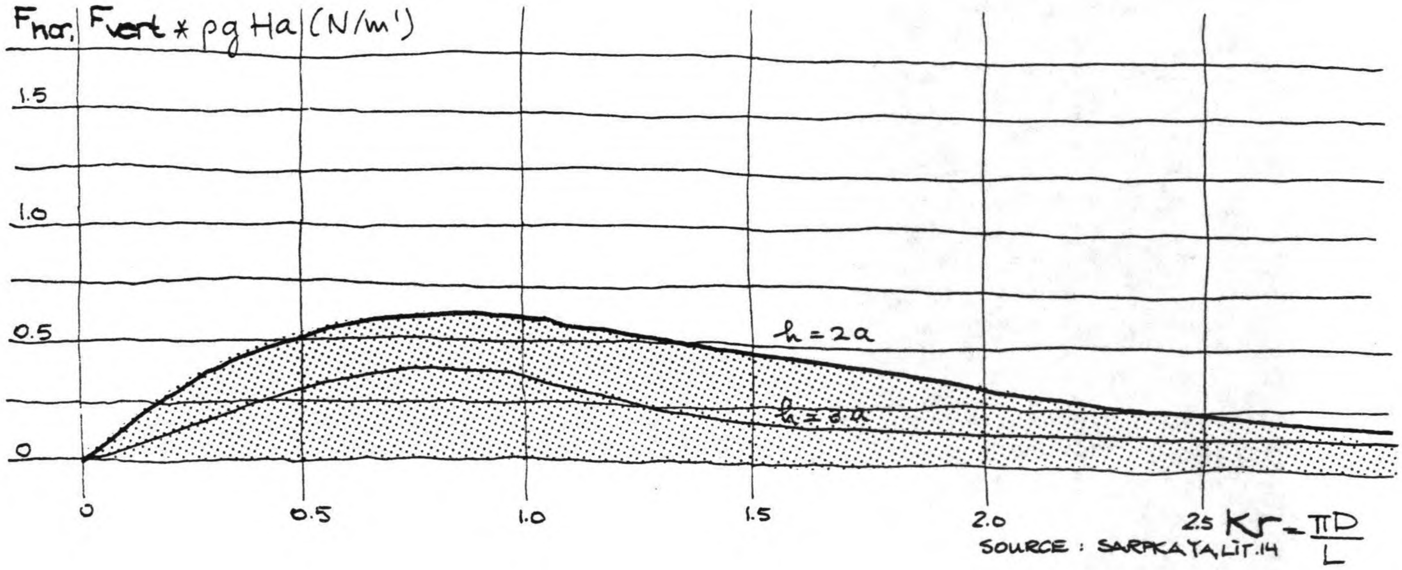
In order to calculate the force on a structure, the wave kinematics (u and v) are found from the total (= incident + diffracted waves) potential: $u = - \frac{\partial \phi}{\partial z}$ and $v = - \frac{\partial \phi}{\partial x}$

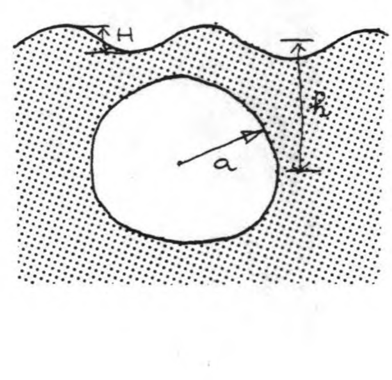
For large bodies the inertia force is allways dominant (see fig. 2.1), this way a new coefficient is introduced:

C_m = effective inertia coefficient (see fig. 2.3)

$$F_{hor} = C_m \ \rho \ S\left(\frac{\partial u}{\partial t}\right)_{max} \cdot \cos(\omega t - \delta) \quad (18)$$







2.3

Forces on large bodies

This case is very much like the Morison-case without the drag-component:

$$\left(\frac{\partial u}{\partial t}\right)_{max} \cos(\omega t - \delta) = \text{fluid acceleration (comparable to } \frac{du}{dt}\text{)} \; [m/s^2]$$

S = sectional area of structure (comparable to $\forall$) [m3]

δ = phase-angle between incident and scattered waves [rad].

For cylinders in vertical plane flows (an infinite horizontal cylinder, axis parallel to y-axis) the scattered waves do not decay at infinity. In three dimensions:

$$\phi_w = \frac{-igH}{2\omega} \cdot \frac{\cosh(k(z+d))}{\cosh(kd)} e^{i(ky\cos\alpha + kx\sin\alpha - \omega t)} \quad \ldots\ldots\ldots\ldots (19)$$

and

$$\phi_s = \phi_s' \, e^{i(ky\sin\alpha - \omega t)} \quad \ldots\ldots\ldots\ldots (20)$$

and the relation:

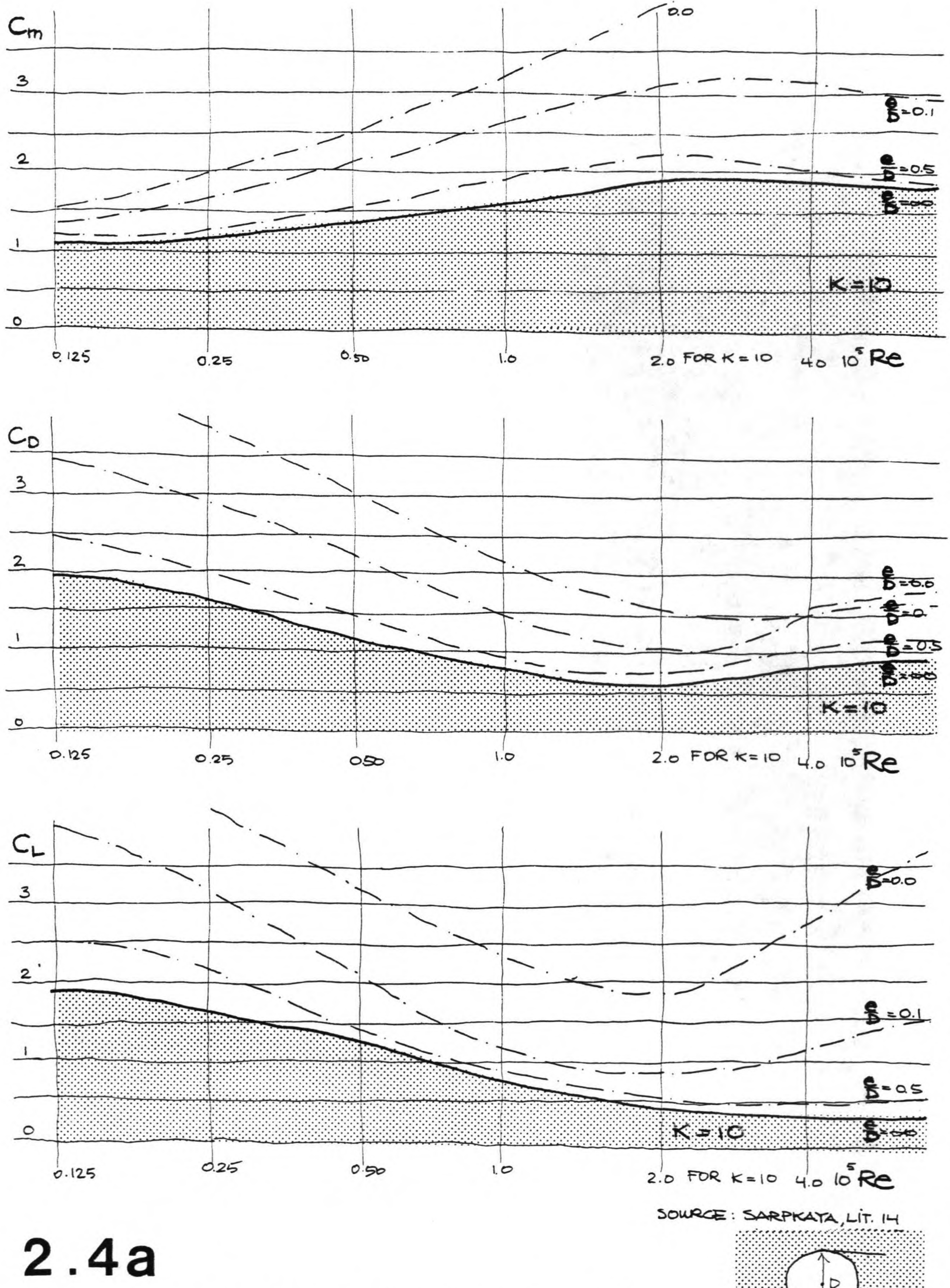
$$\frac{\partial^2\phi'}{\partial x^2} + \frac{\partial^2\phi'}{\partial z^2} - K^2 \sin^2 \alpha\phi'_s = 0 \quad \ldots\ldots\ldots\ldots (21)$$

y = coordinate along the length-axis of the cylinder [m]
x = coordinate across the cylinder [m]
α = angle between x-axis and incoming waves [rad].

This set of equations (19,20 ,21) can be solved for different conditions (see Sarpkaya, lit. (14)).
In the case of $\alpha = 0$ (wave direction perpendicular to length-axis of the cylinder), and a cylinder in deep water, the results for the force-amplitudes, F_{hor} and F_{vert} are given in <u>fig. 2.3</u>.

h = distance between axis and still-water-level [m]
d = water depth [m]
a = radius of cylinder [m].

Most times the force F is expressed in a factor multiplied by ρg Ha (in the case of horizontal cylinders) or by ρg Ha^2 (in the case of vertical cylinders).
The reason for this can be found from appendix B.



2.4a

C_m, C_D, C_L versus Reynolds-number for bottom mounted cylinders

2.1.4 Influence of the bottom

When a structure is approaching the bottom, the stream pattern is also influenced. Experiments (Sarpkaya, lit (14)) have shown, that the effect becomes increasingly important for $e/D < 0.5$ (see fig. 2.4a).

As well the drag- as the inertia coefficient are increased by the pressure of the bottom. Both coefficients depend on the Reynolds number, the K-number and of course the relative spacing e/D.

The lift-coefficient is most influenced by the relative spacing-number, and the bottom-influenced lift coefficient is significantly larger than the free cylinder lift (see fig. 2.4a).

a. The inertia-coefficient C_m

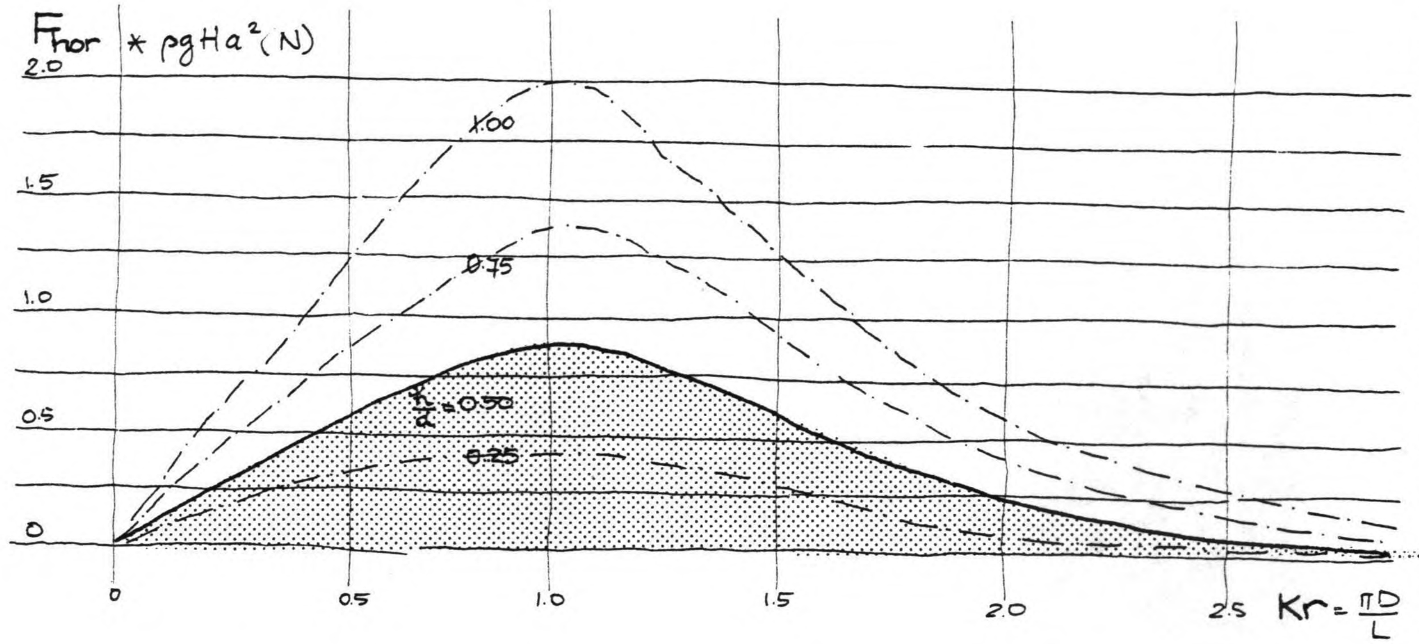
The inertia coefficient is not so much dependent on the Re-number or the roughness of the construction, and increases with increasing K. For very small values of K, where the separation-effects are negligible, C_m approaches its theoretical potential-flow value of $C_m = 3.29$.

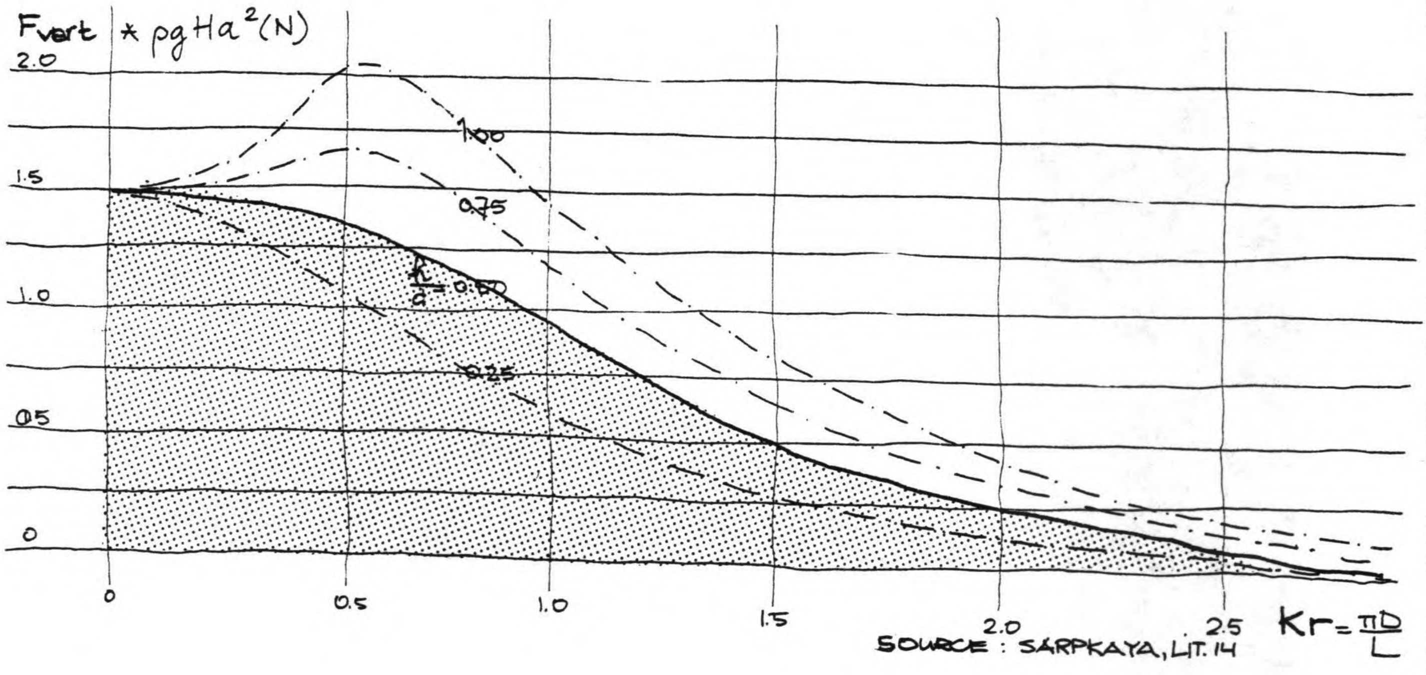
b. The drag-coefficient C_D

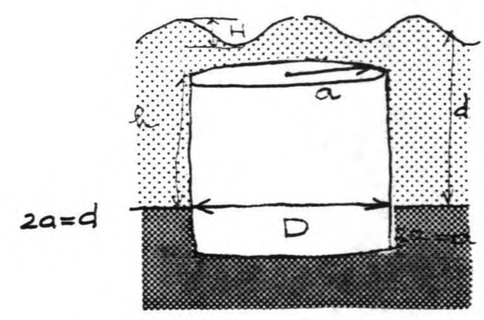
The effect of roughness on C_D is quite significant (fig. 2.4.a is based on rough cylinders).
C_D can reach very high values in case $e/D = 0$ and is a function of the Reynolds number and K.
For very small values of K (and large Re-number) C_D approaches the value of 2.

c. The lift coefficient C_L

The lift coefficient reaches very high values at relatively small K-values. The potential flow-value of C_L, in case $e/D = 0$, is given by von Muller, $C_L = 4.49$.
For K approaching zero, it is expected that the lift-coefficient will reduce to about 4.5.







2.4 b

Forces on large bodies resting on the seabed

CONCLUSIONS

In the foregoing it is shown that the wave-forces can be calculated, either using the Morison-equation, or a comparable equation for large bodies.

The pseudo-statistic determistic analysis of the extreme loading conditions is based on a design wave (chosen statistically with specified height, period and direction) and a wave-theory to calculate the fluid velocities and acceleration of each structural element. The wave is assumed to be long-crested and to propagate without change of form. The instantaneous sectional force is calculated (in-line force through Morison's equation, together with Cd and C_m values appropriate in that section, i.e. Cd (K, Re, K, e/D) and C_m (K, Re, Kr, e/D) and the transverse force through the appropriate lift coefficient C_L (K, Re, Kr, e/D). The instantaneous force on the element is obtained through the integration of the sectional forces.

The extreme conditions are those which give rise to the largest hydrodynamic loadings and low-cycle fatigue and in general. The normal conditions are those which give rise to high-cycle fatigue under operational conditions and stern form the waves and currents that constitute the vast majority of the structure's exposure to fluid loading.

DESIGN-FORCES

Following assumptions have been used calculating the wave-forces on the prospective dike:

. the dike is considered a horizontal cylinder, axis parallel to the wave-crests (and parallel to the coast), having a representative diameter D*) equal to the height of the construction. Thus D = 3.50 m for the total construction;
. the smallest possible unit is also considered a horizontal cylinder, with a diameter D* = 2.00 m;
. all forces are calculated per unit of length, so $\ell = 1$ in the Morison-equation;
. since the dike is situated in shallow water, the force is calculated by the maximum sectional force multiplied by the height, instead of integrating the sectional forces, since this simplifies the calculation (in the Morison-equation the total A_p and $\forall$ are applied, and the maximum horizontal velocity $\hat{u}$).

N.B.: the "representative" diameter D* (see par. 2.1.1) is actually the size representing the dimension of the construction, that is blocking the flow. In this case D is the height of the construction, since this part of the construction is subjected to the wave-forces (see also appendix B).

The forces now become:

$F = C_D \, ½\rho \, D\ell \, . \, \hat{u}^2 \, |\sin \omega t| \sin \omega t$ drag force . . . . (22)

$+ \, C_m \, ¼\rho \, D^2 \ell \pi \hat{u} \, \omega \cos \omega t$ inertia force . . . . (23)

$F = C_L \, ½\rho \, D\ell \, \hat{u}^2 \, |\sin \omega t| \sin \omega t$ lift force . . . . (24)

D = height of construction [m]
ℓ = length of the construction [m], chosen: $\ell = 1$ m
ρ = density of fluid [kg/m3]
$u = \hat{u} \sin \omega t$: AIRY WAVE THEORY [m/s]
$\hat{u}$ = max. horizontal wave velocity [m/s]

. C_D, C_m, C_L can be found from fig. 2.4a in the case of small bodies (e/D = 0);
. in the case of large bodies the force can be found from fig. 2.3, compared with fig. 2.4b.

For the horizontal forces, the results from fig. 2.3 can be multiplied with a factor 2 (since the free-cylinder inertia coefficient is $C_m = 2$, and the bottom-mounted coefficient $C_m \simeq 4$).

For the vertical forces, all the forces from fig. 2.3 can be multiplied with a factor 4.5 (since the free cylinder lift coefficient, $C_L = 1$, and the bottom-mounted lift coefficient $C_L = 4.5$). These factors can be adapted to agree with fig. 2.4b.

The results are shown in the following pages (see table 2.2 and 2.3).

N.B: Especially in the calculation of the liftforce for large objects, a high percentage of uncertainty exist. The magnitude of the liftforce used in this analysis is probably much too high. It is a very conservative estimation of the actual liftforces. Also the exact attitude of the liftforce with respect to time is still very unknown. The assumptions made in this analysis are based on the most up to date information. However, it is expected that the reliability of the figures is in the range of 50%. Measurements at a prototype of the dike will show to what extend the assumpionts have been right.

. EXTREME:

the design wave occurs in the design storm once every 10 years, its period is uncertain:

$H = 3.60$ m	$T = 6$ s	$\alpha = 90°$	$h = 4.54$ m
$H = 3.60$ m	$T = 8$ s	$\alpha = 90°$	$h = 4.54$ m

. NORMAL:

the average wave occurs all year through, flood and non-flood season

$H = 0.50$ m	$T = 3$ s	$\alpha = 90°$	$h = 2$ m
$H = 0.50$ m	$T = 5$ s	$\alpha = 90°$	$h = 2$ m

. the representative height of the construction as a whole is 3.50 m (situated at the 1^+ level) and the representative size of the smallest unit is 2 m.

Using airy-wave theory the most important wave characteristics can be determined:

characteristic	$H = 3.60$ m $T = 6$ s	$H = 3.60$ m $T = 8$ s	$H = 0.50$ m $T = 3$ s	$H = 0.50$ m $T = 5$ s
$L_o = 1.56\ T^2$ deep waterlength	56	99	14	39
$L = L_o \tanh (kh)$ wave-length	38	53	9.1	30
H/L (< 0.14) wave steepness	0.095	0.068	0.036	0.013
$\omega = 2\pi/T$ frequency	1.05	0.79	2.1	1.26
$\hat{u} = \frac{\omega H}{2} \cdot \frac{\cosh kh}{\sinh kh}$	2.66	2.68	0.66	0.59
$\frac{d\hat{u}}{dt} = \hat{u} \cdot \omega$ max. acceleration	2.79	2.12	1.19	0.74

Table 2.1: wave-characteristics

Now the wave force coefficients can be determined:

for D = 3.50 m

parameter	H = 3.60 T = 6	H = 3.60 T = 8	H = 0.50 T = 3	H = 0.50 T = 5
Kr = $\pi D/L$ relative size	0.29	0.21	1.20	0.37
K = $2\hat{u}/\omega D$ relative motion	1.45	1.94	0.18	0.27
Re = $\hat{u}D/\nu$ turbulence	$9{,}310.10^3$	$9{,}380.10^3$	$1{,}990.10^3$	$2{,}070.10^3$

and for D = 2.00 m

parameter	H = 3.60 T = 6	H = 3.60 T = 8	H = 0.50 T = 3	H = 0.50 T = 5
Kr = $\pi D/L$ relative size	0.17	0.12	0.70	0.21
K = $2\hat{u}/\omega D$ relative motion	2.53	3.39	0.31	0.47
Re = $\hat{u}D/\nu$ turbulence	$4{,}030.10^3$	$5{,}360.10^3$	$1{,}140.10^3$	$1{,}180.10^3$

Table 2.2.: wave-parameters

Except for the case that H = 0.50 m and T = 3 s, this classifies all calculations to "small" body-problems, and inertia will be dominating the wave forces (especially since te problem concerns a construction resting on the sea bed).

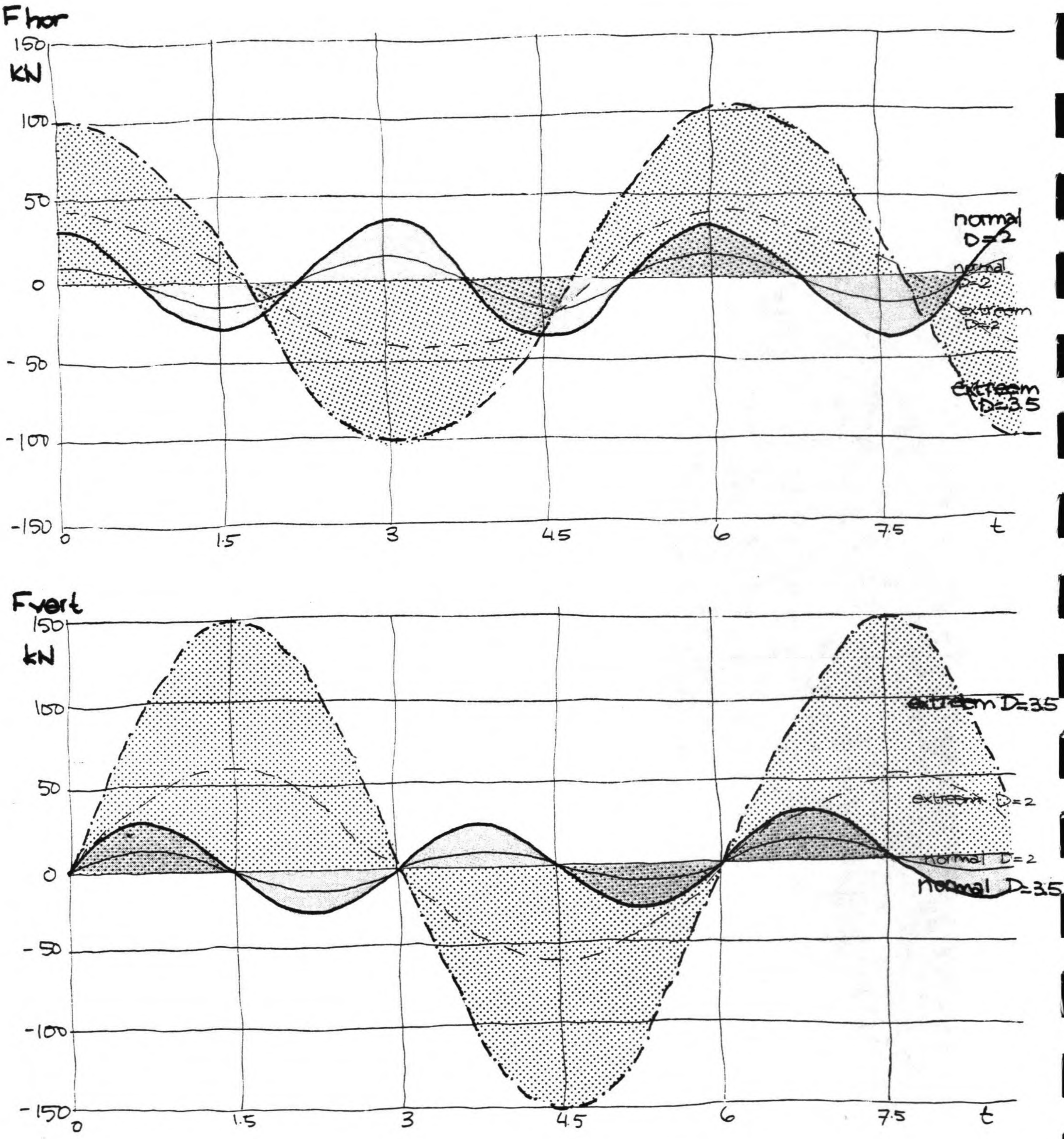
In fig. 2.2 t/m 2.4 the subsequent values for Cd, C_m àn C_L can be found, and the amplitudes of drag component and inertia component can be calculated (see table 2.3).

D = 3.50 m	H = 3.60 T = 6		H = 3.60 T = 8		H = 0.50 T = 3		H = 0.50 T = 5	
	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
C_D (drag)	*	1.2	*	1.5	*	1.2	*	1.2
C_m (mass)	4	3.5	4	4.0	4	3.5	4	3.3
C_L (lift)	4.5	5.0	4.5	5.0	4.5	5.0	4.5	4.0
$\frac{1}{2}\rho\ D\ell\hat{u}^2$[kN/m']	12.4		12.7		0.7		0.6	
$\frac{1}{4}\rho\ \pi D^2\ell\hat{u}\omega$[kN/m']	26.8		20.4		13.3		7.1	
$F_{inertia}$ [kN/m']	63	93.8	40	81.6	21	34.2	18	23.4
F_{drag} [kN/m']	–	14.9	–	19.05	–	0.9	–	0.7
F_{lift} [kN/m']	142	62.0	90	63.05	25	3.0	25	2.4

D = 2.00 m	H = 3.60 T = 6		H = 3.60 T = 8		H = 0.50 T = 3		H = 0.50 T = 5	
	(1)	(2)	(1)	(2)	(1)	(2)	(1)	(2)
C_D (drag)	*	1.2	*	1.5	*	1.2	*	1.2
C_m (mass)	4	4.0	4	4.2	4	3.5	4	3.3
C_L (lift)	4.5	5.0	4.5	4.0	4.5	4.0	4.5	4.0
$\frac{1}{2}\rho\ D\ell\hat{u}^2$[kN/m']	7.1		7.2		0.4		0.3	
$\frac{1}{4}\rho\ \pi D^2\ell\hat{u}\omega$[kN/m']	8.8		6.7		4.3		2.3	
$F_{inertia}$ [kN/m']	36	35.2	18	28.1	7.0	13.0	6.0	7.6
F_{drag} [kN/m']	–	14.2	–	18	–	0.5	–	0.3
F_{lift} [kN/m']	55	35.5	40	28.8	15	1.6	15	1.2

(1) large body theory
(2) Morison-theory

Table 2.3: wave-forces



2.5

Forces on the construction

The forces have been calculated according to the Morison-theory and the "large-body-theory". One finds that the result differ. The Morison-theory tends to under-estimate the lift forces.

Taking into account the fact that the Morison-theory is expected to be more accurate at small K-values, on the otherhand the Morison-theory does not explicitely result in a reliable lift coefficient; the following loadings are selected (see fig. 2.5):

extreme: $F_{horizontal} = \hat{F} \cos \omega t$ .... (23A)

$\omega = 1.05$ rad/s
$T = 6$ s
$\hat{F} = 100$ kN (D = 3.50 m)
$\hat{F} = 40$ kN (D = 2.00 m)

$F_{vertical} = \hat{F} \sin \omega t |\sin \omega t|$ .... (23B)

$\omega = 1.05$ rad/s
$T = 6$ s
$\hat{F} = 150$ kN (D = 3.50 m)
$\hat{F} = 55$ kN (D = 2.00 m)

normal: $F_{horizontal} = \hat{F} \cos \omega t$ .... (24A)

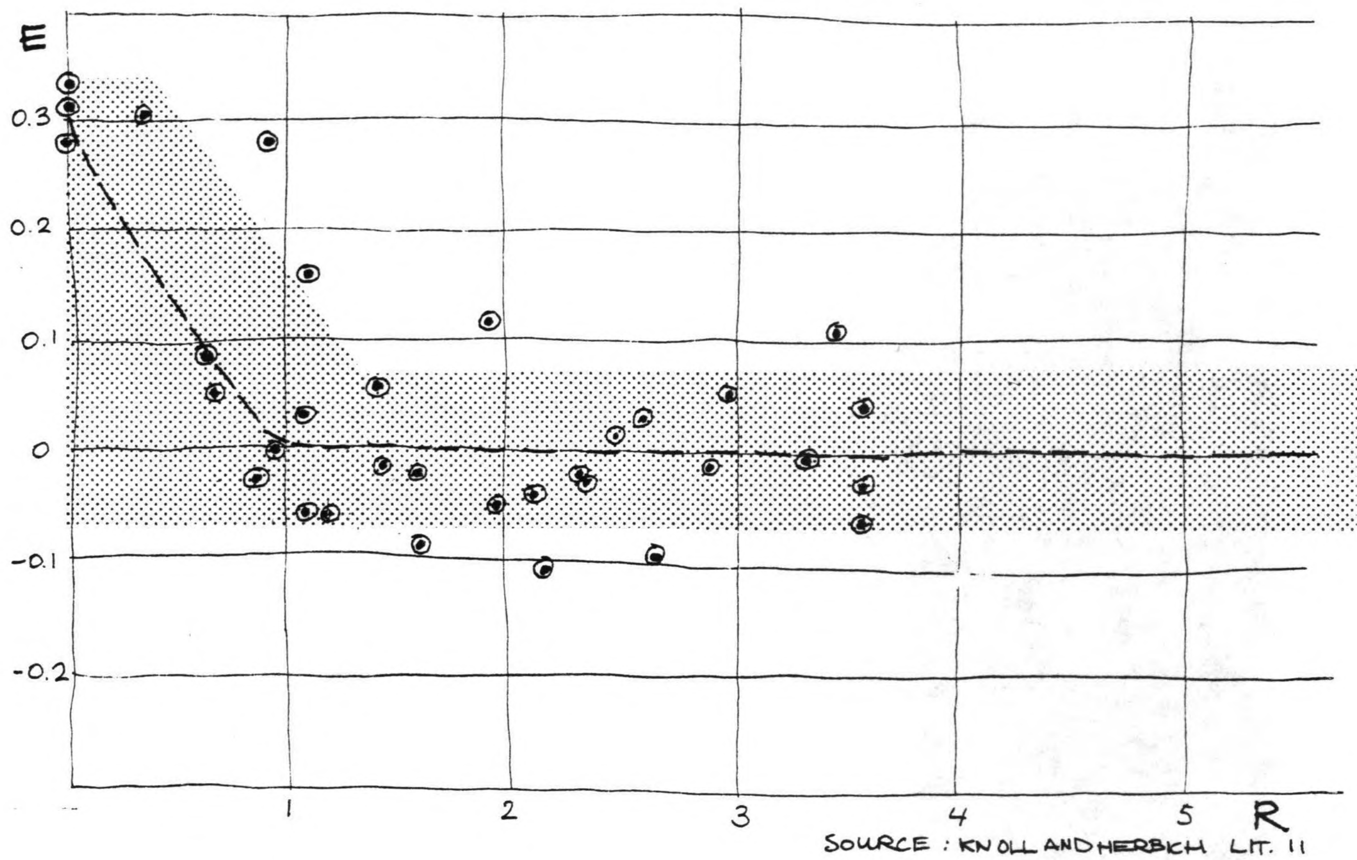
$\omega = 2.1$ rad/s
$T = 3$ s
$\hat{F} = 35$ kN (D = 3.50 m)
$\hat{F} = 15$ kN (D = 2.00 m)

$F_{vertical} = \hat{F} \sin \omega t |\sin \omega t|$ .... (24B)

$\omega = 2.1$ rad/s
$T = 3$ s
$\hat{F} = 25$ kN (D = 3.50 m)
$\hat{F} = 10$ kN (D = 2.00 m).

The extreme forces determine the dimensions of the dike and the strength of the geotextile used: the dike is desigend to withstand the 1-in-10-years-storm, (safety factor is 1.0).

The "normal" forces are used to check fatigue problems (of the geotextile and sewings). These loads should be carried by the dike with an appropriate safety factor (in the order of 2).



2.6

Influence of current on particle kinematics

2.2 Influence of currents

Forces on coastal structures can be determined by the Morison-equation, as described in the forgoing. The major input parameters are from one side the particle kinematics: velocity and acceleration, from the other side the coefficients of drag and inertia.

For the determination of the force components it thus is vital to predict the horizontal velocity of the water particles in the wave (with or without current) and its acceleration in a proper way.

INFLUENCE OF CURRENTS

It can be expected that the presence of currents has some influence on the particle kinematics. The waves may become steeper, the maximum velocity may increase. Some studies on this subject have been performed. Here the theory as developed by Knoll and Herbich (lit. (11)) is regarded:

if a uniform current is assumed, the velocity potential of airy-wave theory* ϕ_1 (derived form Laplace-equation and linearized boundary conditions) and the velocity potential of the uniform flow field ϕ_2 can be added directly to obtain the velocity potential of the combined wave current condition since both potentials are solutions of the Laplace-equation.

$$\phi = \phi_1 + \phi_2 \qquad (25)$$

and

$$\phi_1 = C_o \cosh k_o(z + d) \sin(k_o x - \omega_o t) \qquad (26)$$

$$\phi_2 = Ux \qquad (27)$$

U = uniform - current velocity [m/s]
d = waterdepth (still water) [m]
z,x = coordinates [m]
t = time variable [s]
C_o = celerity of wave in still water [m/s]

*) Airy-wave-theory: the displacement of the water-surface is of a sinusoidal nature, also the velocity of the particles and the acceleration:

$$u = \hat{u} \sin \omega t, \; \hat{u} = \omega a \frac{\cosh k(d + z)}{\sinh kd}$$

a = amplitude of vertical displacement [m]
a = ½ H, wave height [m]
u = horizontal velocity [m/s]
$\hat{u}$ = amplitude of velocity [m/s]
ω = wave frequency [rad/s]
k = wave number [rad/m]
d = water depth [m]

the total horizontal velocity is given by:

$$u = \frac{\partial \phi}{\partial x} = \frac{\partial \phi_1}{\partial x} + \frac{\partial \phi_2}{\partial x} =$$

$$C_o k_o \cosh k_o(z + d) \cos(k_o x - \omega_o t) + U \qquad (28)$$

$K_o = 2\pi/L$ = wave number in still water
$\omega_o = 2\pi/T$ = frequency in still water

or

$$u = \frac{H}{2} \omega_o \frac{\cosh k_o (z + d)}{\sinh k_o d} \cos (k_o x - \omega_o t) + U \qquad (29)$$

and the horizontal acceleration

$$u = \frac{\partial u}{\partial t} = \frac{H}{2} \omega_o^2 \frac{\cosh k_o (z + d)}{\sinh k_o d} \sin (k_o x - \omega_o t) \qquad (30)$$

theoretically this is not valid for steep waves (non-linear boundary conditions), if there are shear currents present, or non-uniform currents.
It is also assumed that the direction of wave propagation is the same as the uniform current direction.

Knoll and Herbich (see lit. (11)) tried to find the error associated with the prediction of the combined horizontal velocity field related to the error in the measured combined velocity, by measuring forces on submerged pipelines, or in other words to find the validity of the superposition principle (see fig. 2.6)

dimensionless difference:

$$E = \frac{U_{occ} - U_{tot}}{\frac{\pi H}{T}} \qquad (31)$$

velocity ratio:

$$R = \frac{U}{U_a} \qquad (32)$$

U_{occ} = occuring horizontal velocity
U_{tot} = algebraic sum of the horizontal particle velocity predicted by airy-wave theory and uniform current velocity $= U + U_a$
U_a = horizontal velocity predicted by airy-wave theory in absence of currents.

It showed that for velocity ratios ($R = \frac{U}{u}$) greater than one the superposition principle predicts the horizontal velocity rather well, for ratios less than one the velocities are underpredicted (see fig. 2.6).

CONCLUSION:

When the wave-propagation has the same direction as the current; and for velocity-ratios greater than one, the superposition principle is valid. For velocity-ratios smaller than one the combined effect is difficult to predict.
Since the current-direction at the Cao Jing-coast is perpendicular to the direction of the waves, and the velocity-ratio is in the order of 0.4; the combined effect is uncertain, and has to be found by (prototype) tests. In this analysis the effect of currents is not taken into account.

2.3 Influence of wave-direction

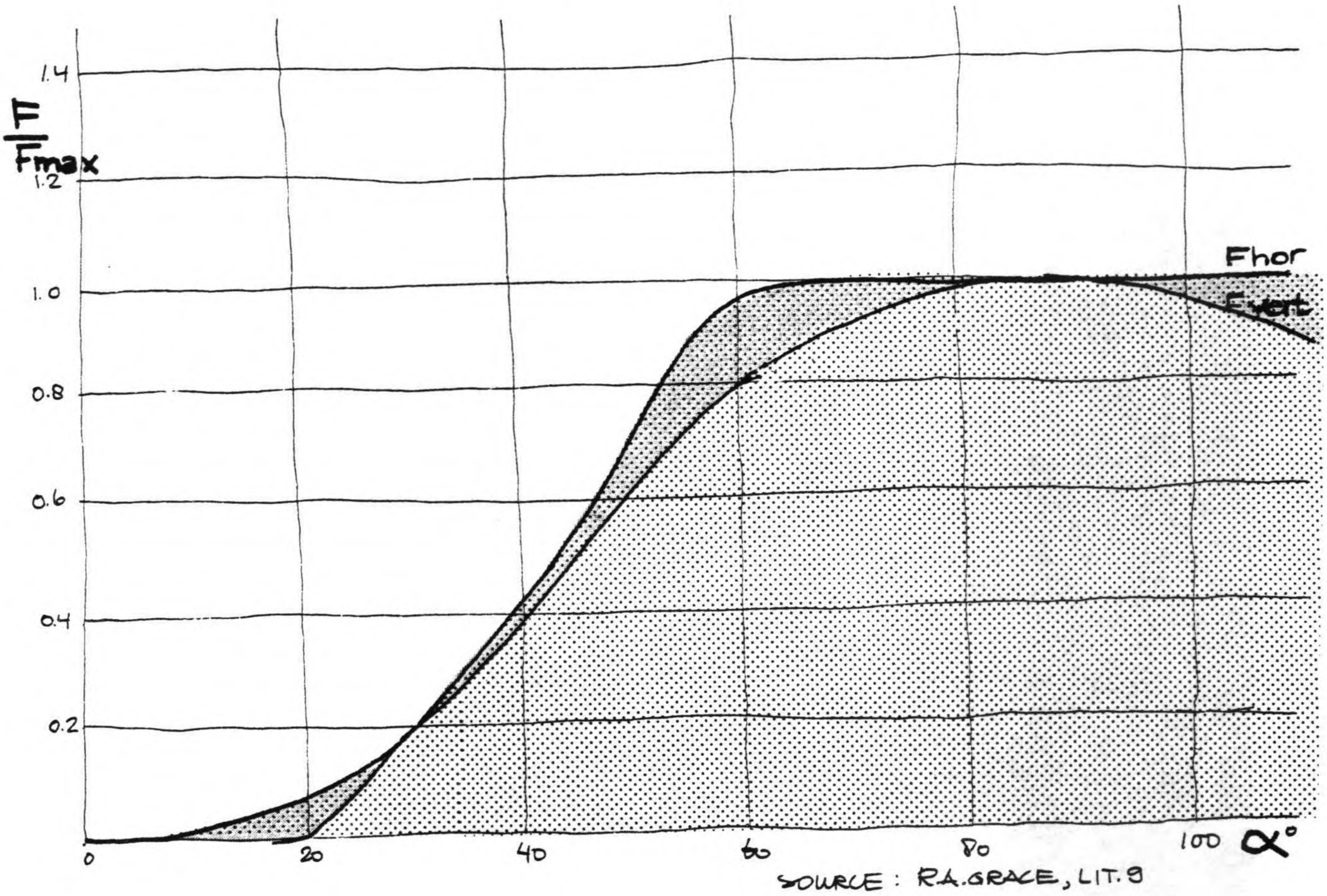
In the Morison-equation (and in the large-body theory) the waves have been assumed to be long-crested and to propagate without change of form. In case of horizontal cylinders (axis parallel to the coastline), the waves are also assumed to approach the construction perpendicular to the coastline: the wave-forces reach their maximum at the same moment along the total dike.
In reality this is of course not the matter. Wave-crests always have a limited length, the wave direction is not completely perpendicular to the construction along the total dike, and some reduction of the wave-forces can be taken into account.

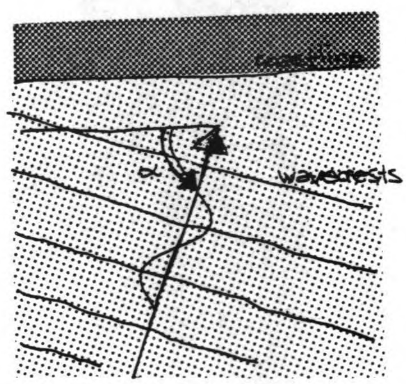
INFLUENCE OF WAVE-DIRECTION

R.A. Grace (lit. (9)) has done some researches on the subject of the wave forces, dependent on wave-direction, and clearance from the bottom. The results for wave-direction are shown in fig. 2.7.
The horizontal force seems rather insensible to changes in wave-direction, in the range of $60° < \alpha < 120°$ (see fig. 2.7).
During the design-storm, the wind direction is south, and nearly perpendicular to the coast at Cao Jing. The waves are therefore assumed to approach the coastline under the same angle, an angle of about 90°.

CONCLUSION:

The force-components (F_{hor} and F_{vert}) are not sensible to small (local) changes in wave-direction. The crest of a wave can be expected to be at least that long, that the critical cross-section of the construction is subjected to the total maximum wave-force-components of the design-wave. A reduction of the force components due to limited crest length and local deviations from the main ($\alpha = 90°$) wave direction is not considered in this analysis.





2.7

Influence of wave direction on force components

2.4 Wave slamming loadings

Offshore structures and sea walls are passed by waves near the still water level. The members of the structure are altenately exposed to the air and then submerged in the water. In some situations, the water which approaches the member is "trapped" by the structure; the water surface hits the member with an enormous force. These slamming forces, or pressures, exceed the "ordinary" inertia forces. They have the nature of (local) shock-waves comparable to small "water-hammers".

The phenomenon of slamming pressures has been a subject of many studies and researches. Up to now no consensus of opionion has been reached about reliable design-forces and pressures.

THEORY

Miller (lit. (12)) and Sarpkaya (lit. (14)) approached the problem from a theoretical point of view (see fig. 2.8a).

A) Miller (lit. (12))

Miller tackled the problem according to Von Karman's expanding plate analogy;
the slamming-load F_s is therefore expressed analogeous to the other wave-force-components:

$$F_{slam} = C_s \cdot \tfrac{1}{2}\rho\, D\, \ell\, \hat{u}^2 \qquad (33)$$

C_s = wave slamming coefficient [-]
D = diameter of cylinder (or representative diameter of structure) [m]
$\hat{u}$ = max. horizontal fluid velocity [m/s]
ℓ = length of construction (taken unity) = 1 [m].

The theoretical value of C_s (when the structure impacted by a wave is compared with a flat horizontal plate is litting a horizontal water surface) is 3.2 (π).

Then he measured (slamming) forces on semi-submerged cylinders, and also the rise-time of the load. Results:

the wave slamming coefficient is dependent on the response of the structure (elastic or plastic, tough or weak) and was found to be:

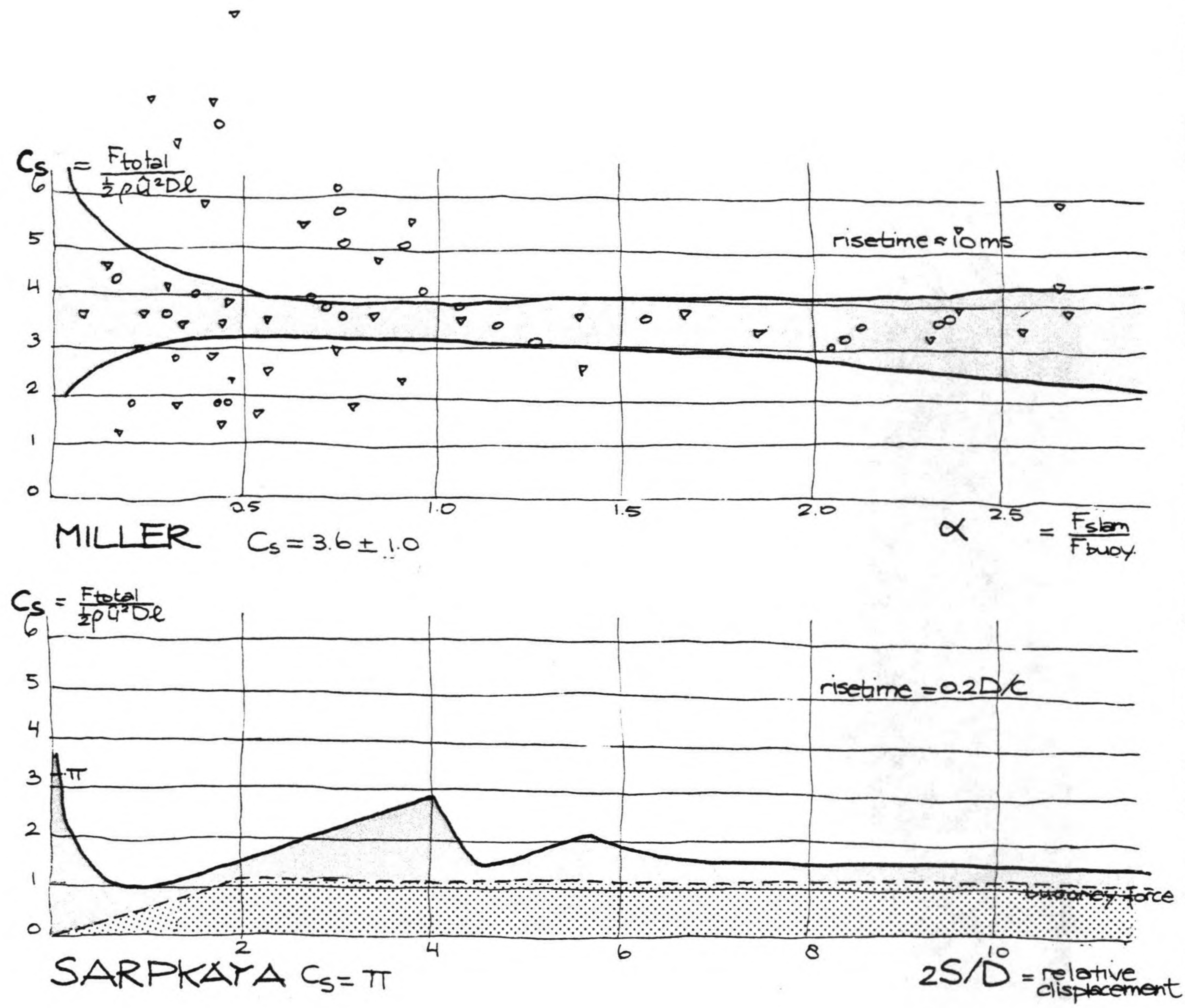
$$C_s = 3.6 \pm 1.0 \text{ (95\% reliable)} \qquad (34)$$

B) Sarpkaya (lit. (14))

Sarpkaya followed an analogeous approach. He advises to use:

C_s = 5.2 if response is unknown
C_s = 3.2 if response is determined.

The rise time of the slammingload is in the order of 15 ms for coastal structures.



2.8a

Wave slamming forces: according to Miller, Sarpkaya

EXPERIMENTS

Considering several theoretical approaches, the results are more or less the same. Other investigators approached the problem from the experimental side: they placed pressure-measuring-equipment in a prototype structure, and measured the pressures caused by wave-slamming. The results of Kamel (lit. (10)) are shown here: (see fig. 2.8b).

C) Kamel (lit. (10))

Kamel (lit. (10)) compared slamming-loads with shock pressures travelling through a medium. If the enclosure of air (which is a major parameter in the magnitude of the shock pressure) is excluded, he found a theoretical value of the shock pressure:

$$P_T = \frac{\rho_s C_s}{\rho_s C_s + \rho_w C_w} \cdot \rho_w C_w \hat{u} \quad \ldots\ldots (35)$$

ρ_w = density of water (= 1,000 kg/m3) [kg/m3]
C_w = celerity of sound in water (= 1,500 m/s) [m/s]
ρ_s = density of structure material (= 1,800 kg/m3) [kg/m3]
C_s = celerity of sound in structure material [m/s]
P_T = theoretical slamming-pressure [N/m2]
$\hat{u}$ = maximum horizontal velocity [m/s].

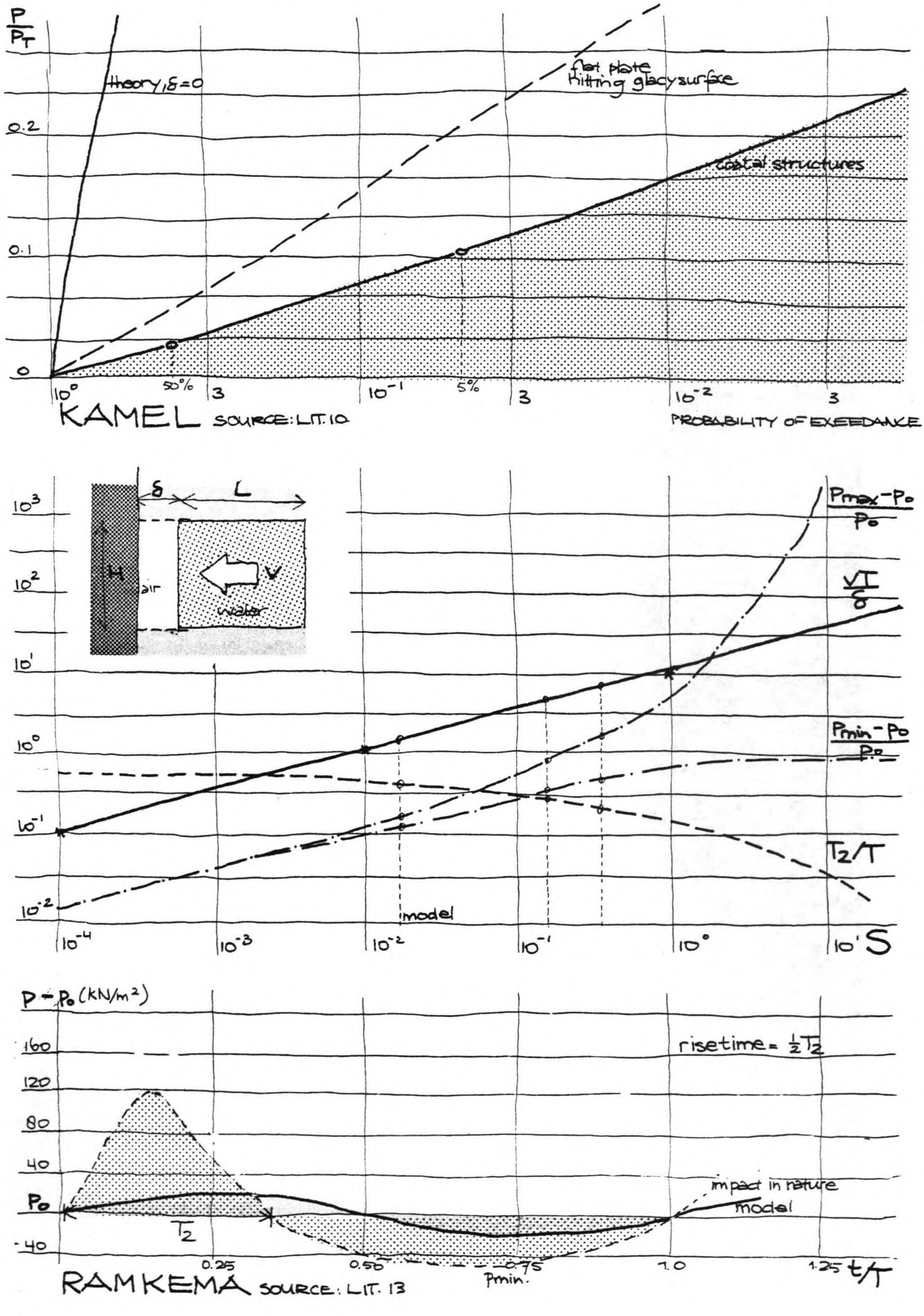
This theoretical pressure is reduced by a factor, dependent on the thickness of the entrapped air-layer, δ. His experiments were carried out by hitting a fluid-surface with a (rough) plate. Dependent on the roughness of the plate, the roughness of the fluid-surface and the entrapped air layer δ, he found a relation between P_T and δ. However, his measurements at coastal structures showed that the shock pressure is rather independent on the wave height (or $\hat{u}$), but only depends on the thickness of the entrapped air layer (of course, when the wave is large, also the impact surface is bigger and thus the total slamming force). Results for coastal structures:

50% of the pressures exceeded 95 kN/m2
5% of the pressures exceeded 316 kN/m2.

Also the rise time (which is directly related to the shock pressure)

50% of the rise times were shorter than 2 msec
5% of the rise times were shorter than 0.2 msec.

Thus the slamming-pressures behave as a stochastic variable following an exponential distribution (see fig. 2.8b).



2.8b Wave slamming forces

D) Ramkema (lit. (13))

Ramkema performed an extensive study on the subject of wave-slamming forces for the Easternscheldt-project (lit. (13)). A model of a caisson-section was built, and a theory for the expected slamming-pressures was developped, starting from the piston-model of Bagnold. After many measurements he developed a model-law, by which the slamming-pressure on the construction can be predicted. The main parameter is formed by the impact-number S:

$$S = \frac{1}{P_o} \cdot \tfrac{1}{2}\rho\hat{u}^2 \, \frac{L}{\delta} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (36)$$

P_o = atmospheric pressure (100 kN/m2) [kN/m2]
δ = thickness of entrapped air layer [m]
L = length of water column involved [m]
ρ = density of water [kg/m3]
$\hat{u}$ = maximum horizontal velocity [m/s] = v.

For the impact-number S, a scaling factor was determined. "Because the size of the caisson and water surface is determinant for the air layer thickness and the hydraulic mass involved, the scaling factor for δ and L is linear ($n_\delta = n_l$; $n_L = n_l$). The external water movement obeys Froude's law, so $n_v = \sqrt{n_l}$, where the athmospheric pressure is equal in model and nature ($n_{po} = 1$). This results in a linear scale for the impact number S, or $n_s = n_l$."
In fig. 2.8b the resulting parameters are given as a function of the impact number S. The impact number of the model was 0.015, the still-water level 0.23 m. In our case the still-water level is 5.00 m, resulting in an impact number S = 0.33:

$P_{max} - P_o$ = 110 kN/m2
$P_{min} - P_o$ = 50 kN/m2
f = 20 Hz (T = 50 ms)
and T_2/T = 0.35 (rise time: 9 ms)

T_2 = time between upward and downward zero crossing (see fig. 2.8)
T = slamming period, or time between two upward zero crossings.

For "normal" conditions, this model law results in an impact number of S = 0.17, or:

$P_{max} - P_o$ = 80 kN/m2
$P_{min} - P_o$ = 35 kN/m2
f = 50 Hz (T = 20 ms)
and T_2/T = 0.40 (rise time: 4 ms).

One finds that the slamming pressures are not very dependent on the wave-height (for normal conditions the slamming pressure is hardly lower than for extreme conditions). The total slamming load on the construction is of course bigger for higher waves, since the surface over which the load is working is bigger.

The results of Ramkema and Kamel are in the same order of magnitude, also they both found the relative unimportance of the wave height (or $\hat{u}$). Possibly the air-layer δ increases for larger waves, compensating the bigger hydraulic mass (see lit. (10, (13)).

If we compare the results of the theoretical approaches, C_s = 3.2 to 3.6, with the experimental results, $P_{max} \simeq$ 100 kN/m2 we find:

$F_{slam} = C_s \cdot \frac{1}{2}\rho D \ell \hat{u}^2$
$P_{slam} = C_s \cdot \frac{1}{2}\rho \hat{u}^2$
According to theory P_{slam} = 13 kN/m2 ($\hat{u}$ = 2.7 m/s)
According to experiments, Pslam = 100 kN/m2
or, written as a slamming-coefficient:

C_s = 27!!!

Thus the results of the theoretical approach, and the experiments are differing from each other with a factor of almost 10.
A reason that the theoretici Miller and Sarpkaya did not find such high slamming-forces, could be the fact that they measured the slamming coefficient by measuring the wave-loading response in the cylinders. In chapter 3 and appendix B it will be shown that the total cylinder hardly responds to forces having such high ($T_{slam} \simeq$ 50 ms) frequencies. So by measuring the forces at the cylinders, one can not retrace the slamming pressures, since their frequency is too high. Only pressure-measuring equipment of which the adjustment-time is much shorter than the slamming-period (say, in the order of 1 ms) will be able to record these loadings.

CONCLUSION:

The wave-slamming pressures as determined by Ramkema will be used in this analysis, since they are based on solid experiments and prototype measurements.

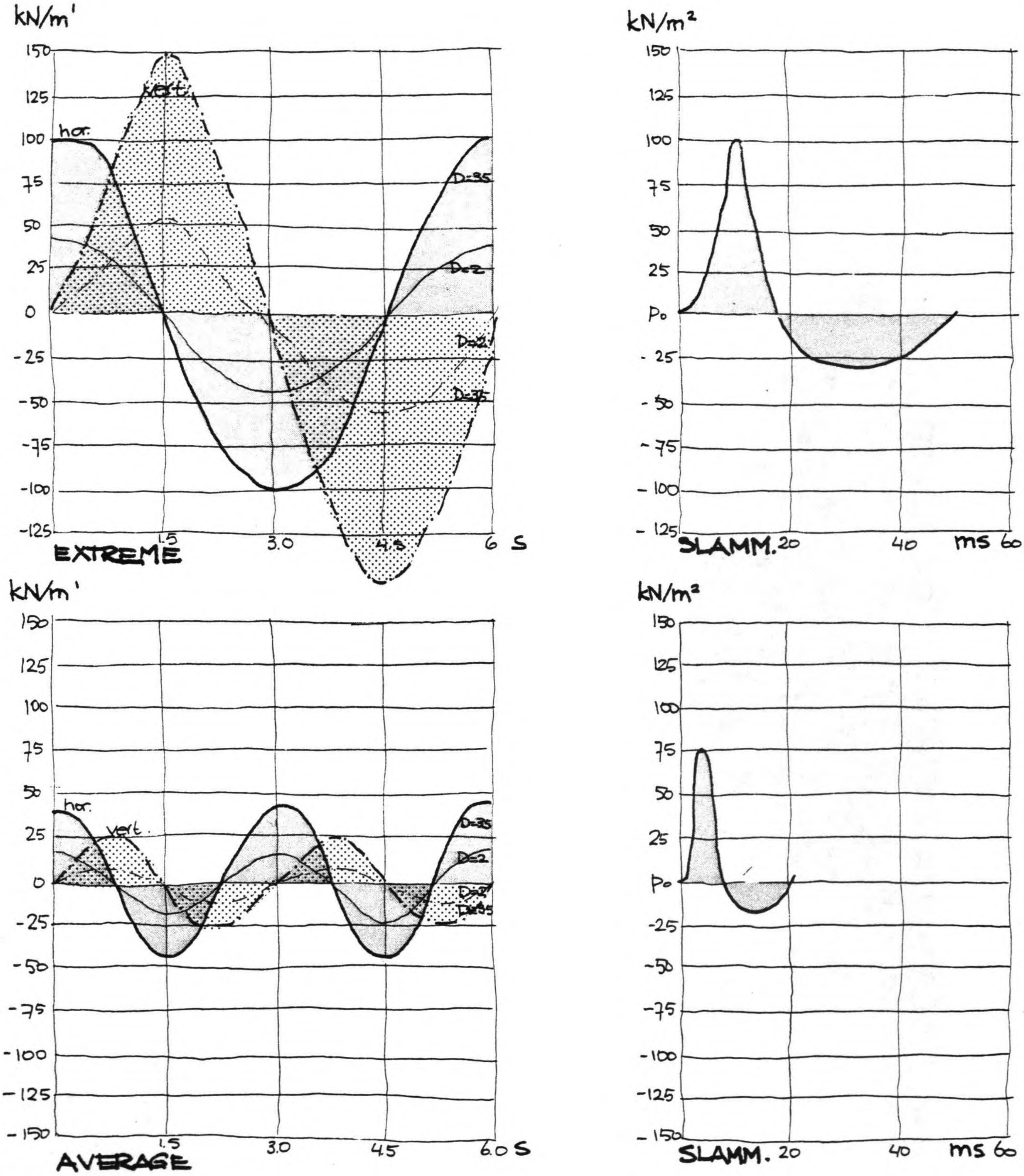
Later tests (the prototype-dike) in the laboratory will have to confirm whether these pressures are existing in reality.

The selected wave-slamming pressures originate from fig. 2.8b:

extreme conditions: $\hat{p} = 100$ kN/m2
$T = 50$ ms
$t_{rise} = 9$ ms

normal conditions: $\hat{p} = 80$ kN/m2
$T = 20$ ms
$t_{rise} = 4$ ms.

It should be stressed that the frequency of the wave-slamming pressure is so high, that the construction as-a-whole will not react to this loading (see appendix B).
The geotextile membrane of the units will react to this slamming pressure. By the slamming-load a (local) pressure-difference over the geotextile will develop, thus a tensile force is introduced in the geotextile. The results of the slamming-load is discussed in chapter 3.2.



2.9

Overview of loadings

2.5 Overview of wave-loadings (see fig. 2.9)

1. Extreme conditions:

return period design-storm = 10 years
chance of occurance during storm: 50 %

$H = 3.60$ m $\quad \omega = 1.05$
$T = 6$ sec $\quad \hat{u} = 2.7$ m/s.

	$D_{constr} = 3.50$ m	$D = 2.00$ m
$\hat{F}_{hor}$	100 kN/m'	40 kN_{m}·
$\hat{F}_{vert}$	150 kN/m'	55 kN/m'
$\hat{P}_{slam}$	100 kN/m2 $\quad t_{rise} = 10$ ms T = 50 ms	100 kN/m2 $\quad t_{rise} = 10$ ms T = 50 ms

2. Normal conditions:

medium waterdepth is 2.00 m
during flood period the waves are somewhat higher than during non-flood periods.

$H = 0.50$ m $\quad \omega = 2.10$
$T = 3$ sec $\quad \hat{u} = 0.7$ m/s.

	$D_{constr} = 3.50$ m	$D_{constr} = 2.00$ m
$\hat{F}_{hor}$	35 kN/m'	15 kN_{m}·
$\hat{F}_{vert}$	25 kN/m'	10 kN/m'
$\hat{P}_{slam}$	75 kN/m2 $\quad t_{rise} = 4$ ms T = 20 ms	75 kN/m2 $\quad t_{rise} = 4$ ms T = 20 ms

3. RESPONSE

General

The stability of the dike is determined by the way the dike will react to the loadings; the so-called response. The loadings consist of static loadings and of dynamic loadings. Static loadings are caused by gravitation, working on the filling and thus (partially) on the geotextile. Dynamic loadings are caused by the wave-loadings as determined in chapter 2.

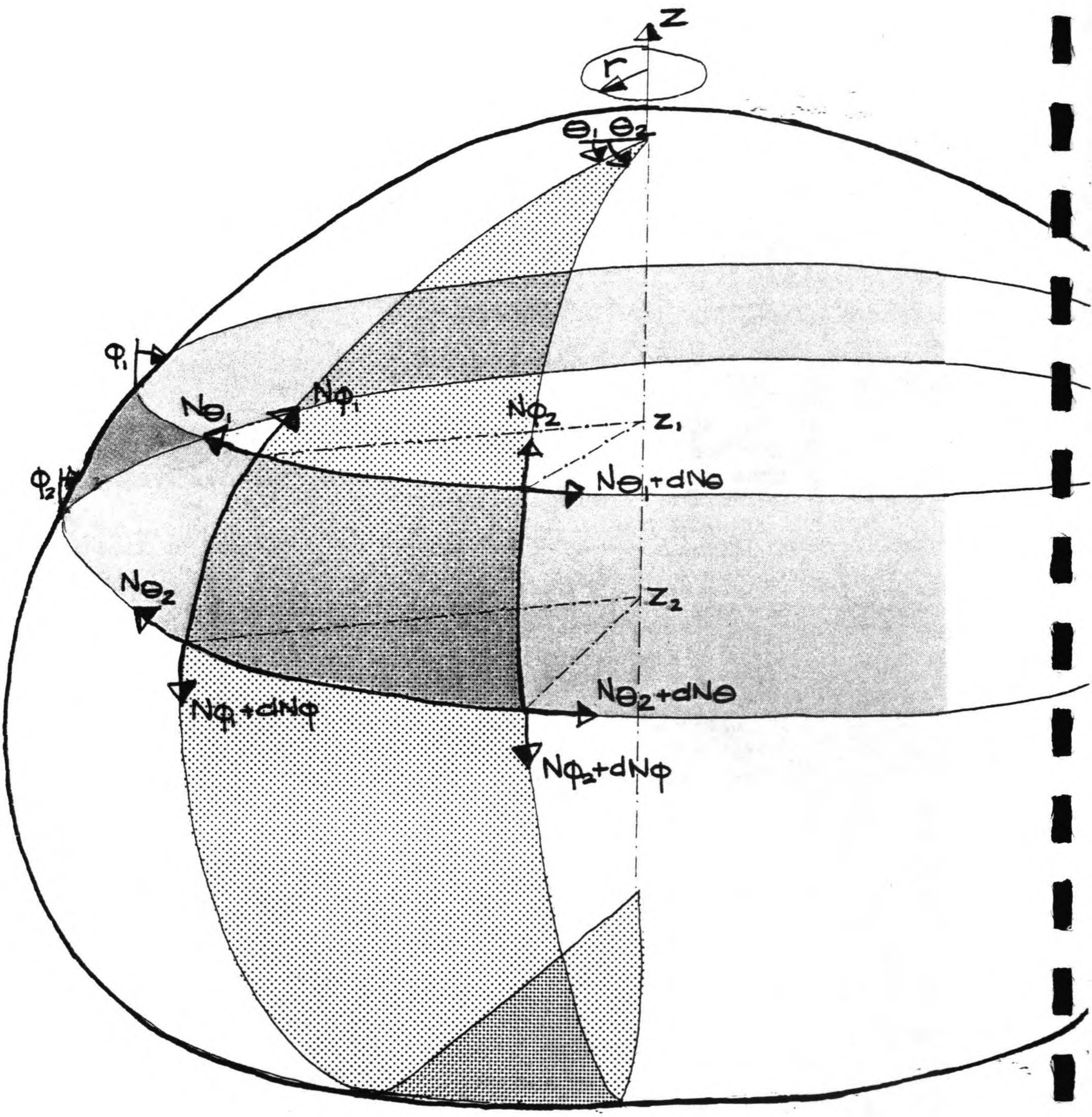
The response on the static load differs from the dynamic loads, since the first one is ever-present and working only in one direction. The displacements caused by gravitation are irreversible and will increase in time: the displacements caused by dynamic loadings are partially reversible: dynamic (wave) loadings are of a cyclic nature, working in alternating directions.

1. STATIC LOADINGS

Static loadings are caused by gravitation: the filling has to be carried (partially) by the geotextile, resulting in tensile stresses in the geotextile. The response to the static loadings is formed by these stresses. Since the static stresses are always present, the safety-factor of the resistance of the geotextile against gravitational loading must be at least $\gamma = 2$.

2. DYNAMIC LOADINGS

Dynamic loadings are caused by the waves. The dike will respond by small deformations, causing some extra stresses in the geotextile. The response of the dike to dynamic loadings is formed by the extra stresses in the geotextile, and the displacement of the dike along the bottom, plus the deformation of the dike due to the loadings. Since the dynamic loadings have a random-nature, and the extreme cirumstances are supposed to occur only once in 10 years, the safety-factor of the resistance of the eventual dike and geotextile against these loadings may be $\gamma = 1$.



3.1

Membranes, directions an defenitions

3.1 Statical behaviour

Due to the weight of the silt-water mass that is stored inside the geotextile, a tensile load is introduced, which causes elongation of the strings and decrease of the total height of the construction. This is a so-called static load, independent on the dynamic (wave-) loadings, and completely dependent on the shape of the construction and the weight of the fill.

The geotextile, which is a flexible material, has no resistance against curvature or sideward expansion. It has only resistance against elongation. In general membranes satisfy the following equation: (see fig. 3.1)

$$\frac{N\phi}{r\phi} + \frac{N\theta}{r\theta} = P_r \qquad (37)$$

ϕ = angle between membrane-meridian and vertical axis [rad]
θ = angle between membrane-hoop and horizontal axis [rad]
$N\phi$ = tensile force in ϕ-direction [kN/m']
$N\theta$ = tensile force in θ-direction [kN/m']
$r\phi$ = radius of curvature of membrane in ϕ-direction [m]
$r\theta$ = radius of curvature of membrane in θ-direction [m]
P_r = load (pressure) on the membrane [kN/m2].

In the case if a statical analysis of the soil-filled construction, the fill must be considered as a liquid, since the exact properties of the fill (internal friction, water content) are not constant, the worst case occurs during the filling process. A mixture of silt and water is put into the geotextile and the eternal shape of the construction is developped. So the fill is regarded as a heavy fluid, having a specific gravity of γ = 18,000 N/m3, this forming and upper limit of the static load. This way the load on the geotextile in (37) is expressed as:

$$P_r = \gamma \cdot z + P_o \qquad (38)$$

γ = specific gravity of fill = 18,000 N/m3 [N/m3]
z = vertical coordinate (at top: z = 0) [m]
P_o = external pressure [kN/m2].

It should be stressed that the assumption of the filling behaving like a heavy fluid, is not unlikely to occur.
Under severe conditions the wave impacts will cause the filling to soften locally. Sooner or later the construction will take its equilibrium shape: the shape in which the membrane forces will balance with the gravity forces of the filling, in other words the shape that is found from the heavy-liquid approach.

In principle there are three different ground shapes of units that a possible dike could be composed of:

A. TUBES : the membrane forces mainly concentrate in the vertical fibres of the geotextile.

B. CYLINDERS: the membrane forces mainly concentrate in the horizontal circular fibres of the geotextile.

C. CUBES : an intermediate solution: the weight is carrried partially in horizontal, partially in vertical direction, dependent of the direction of the minimal equivalent circle.

In the following these shapes are discussed, and the tensile stresses in the geotextile that are caused by gravitation.

First it is discussed the relation between the original size of the (empty) unit, and the height that is established after filling. Then the influence of the filling degree (suppose some filling is lost) on the construction is discussed, and the influence of elongation of the geotextile (creep!).

This way, the dimensions of the dike under natural circumstances can be predicted. Also the required tensile strength of the geotextile can be found.

3.1.1 Tube-elements

The main characteristic of tubes is the fact that the curvature lenghtwise (in x-direction) is (much) smaller than the curvature in vertical direction (z-direction) (see fig. 3.2A): $r\theta = \infty$.

The membrane equation (37) then degenerates to:

$$\frac{N\phi}{r\phi} = \gamma z + P_o \qquad (39)$$

γ = specific weight of the filling [kN/m3]
z = vertical coordinate [m]
P_o = external pressure [kN/m2]
$N\phi$ = tensile force in geotextile [kN/m']
$r\phi$ = radius of curvature [m].

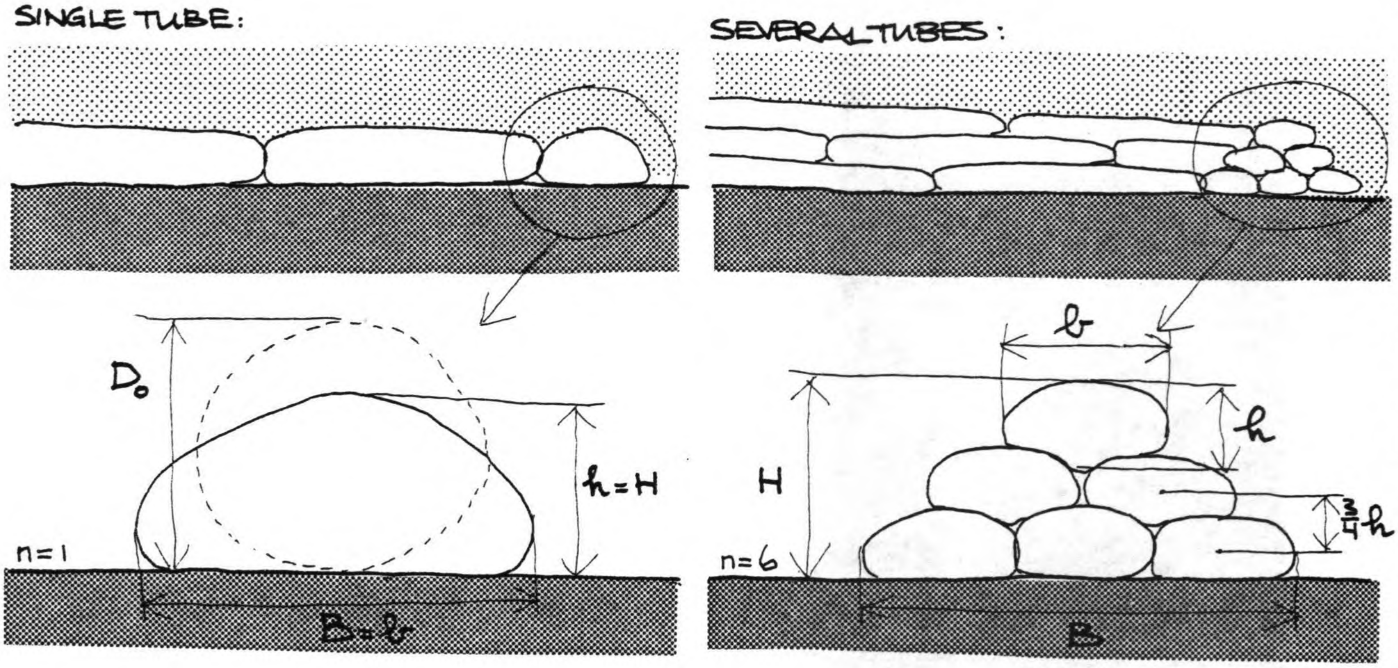
So the shape and height of a tube is determined only by its content and circumpherence. Due to the fact that the load increases towards the bottom of the tube, it's shape will not be completely circular, but some kind of a "drop shape": flat on top and curved near the bottom (see fig. 3.2A).
In appendix A the differential equations and theoretical background of this shape are discussed.

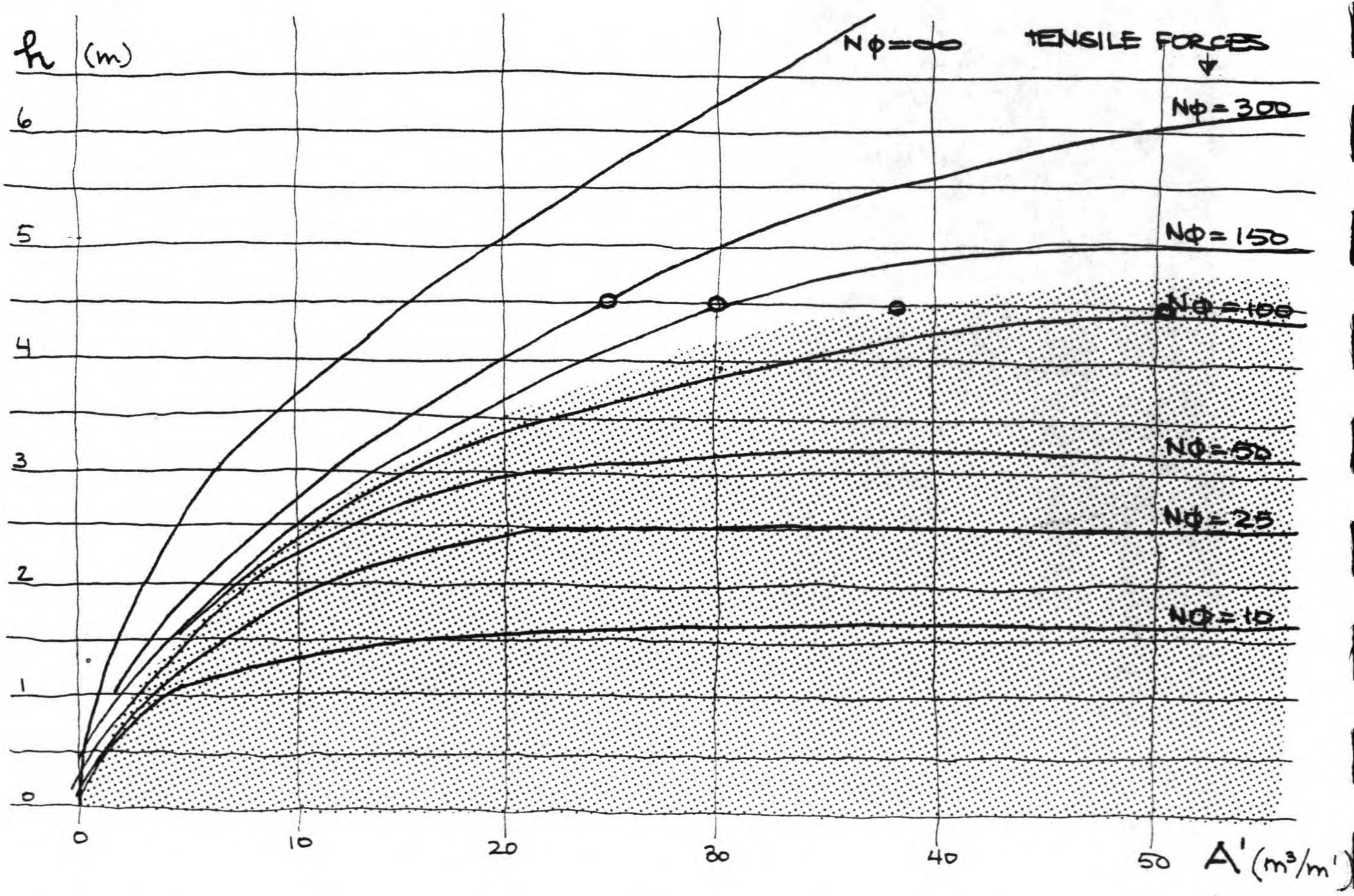
Filling degree, elongation of the geotextile and tension in the fibres are mutually related.
For a complete circular shape of a geotextile tube, the filling degree should be 100%, the elongation 0% and the tension $N\phi = \infty$.
In reality, of course, the fibres will elongate, $N\phi$ will decrease, and as a result the height will also decrease (the relative filling degree acts in the same manner).

Introducing following parameters:

b = width of the tube [m]
h = height of the tube [m]
D_o = diameter of an equivalent circle having the same content [m]
B = width of the dike [m]
A' = content of the tube per running meter [m3/m']
P' = circumpherence of the tube per running meter [m2/m']
$N\phi$ = tension in geotextile per running meter [kN/m']
H = height of the dike [m].

The relation between height of the tube, and tensile stress in the geotextile is given in fig. 3.2A. The influence of filling-degree and elongation is given in fig. 3.2B in dimensionless numbers (h/D_o).





3.2a

Tube shaped elements : tensile forces

As illustrated in fig. 3.2A the dike can be built of one single tube, or of several tubes piled up each other.

The top level of the dike must be at a level of +3.50 m (see chapter 1), considering the expected settlement of the dike into the subsoil, the construction-height will be taken 4.50 m:

$H_{construction}$ = 4.50 m.

This allows the dike to loose some content by damages, and forms an appropriate safety factor for unexpected construction-problems by which some of the dike-units could be filled unsufficiently.

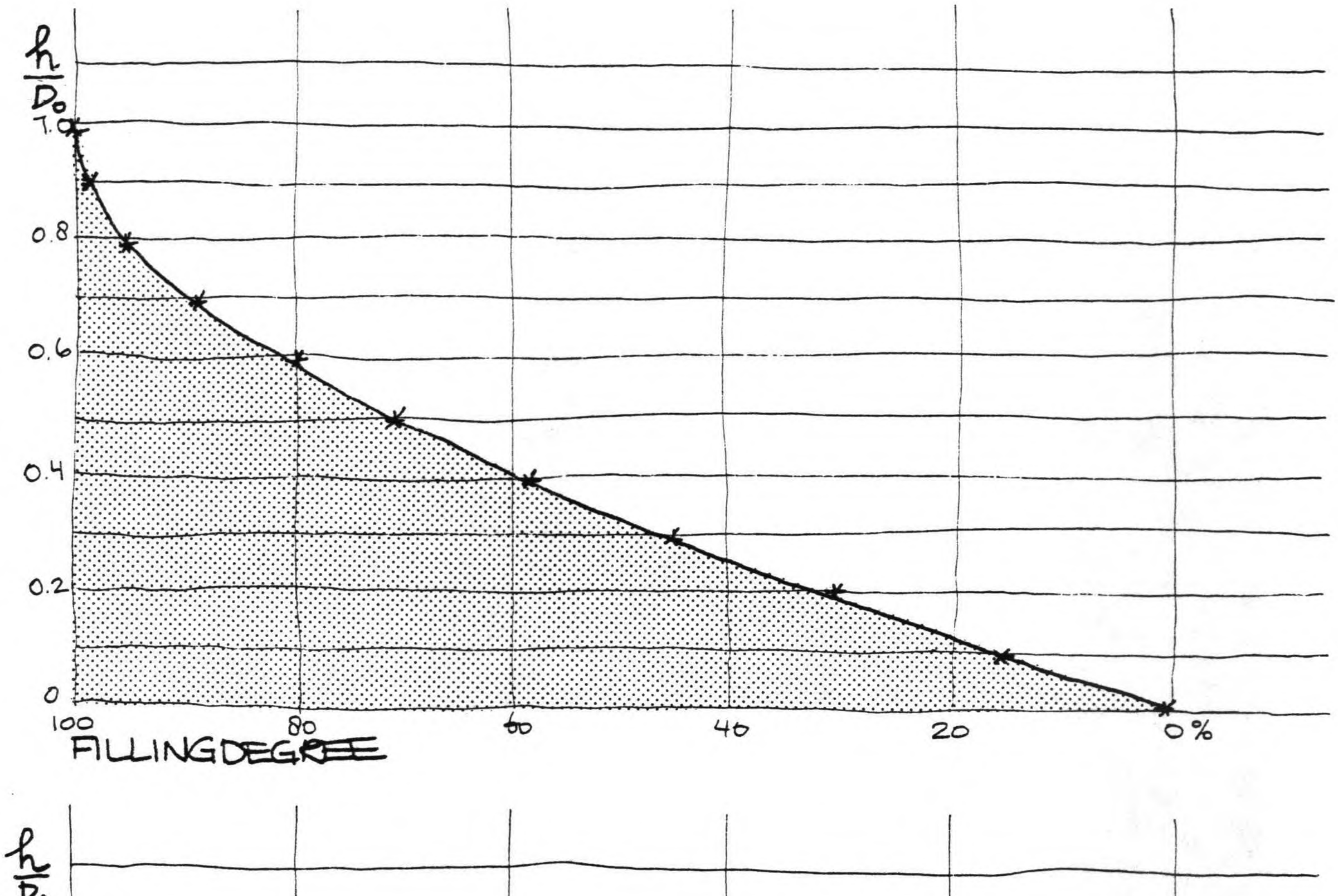
SINGLE TUBES

According to fig. 3.2A the minimum tensile force required to reach a height of 4.50 m, using one single tube, is $N\phi$ = 110 kN. If a higher tensile force is introduced in the geotextile, the accompanying diameter of the tube will decrease, the required filling degree will increase (see also Appendix A).

Possible tube-diameters and resulting filling degree, plus the tensile force required in the geotextile are given in table 3.1, assuming the height of the tube is 4.50 m.

D_o [m] n = 1	content A' [m3/m]	filling degree %	circumpherence P' [m2/m']	width B [m]	force [kN/m']
10.0	51.0	65%	31.4	15.5	110
8.0	38.0	75%	25.1	11.5	125
7.0	30.0	80%	22.0	9.5	150
6.0	25.0	90%	18.8	7.5	300
↓	↓	↓	↓	↓	↓
4.50	15.9	100%	14.1	4.50	∞

Table 3.1: single tubes, H_{constr} = 4.50 m



3.2b

filling degree and elongation

SEVERAL TUBES per cross section

In the case the dike is built out of more tubes piled on each other, only a limited number of configurations is possible. The total number of tubes necessary depends on the number of layers. The minimum size of the tubes is D_o = 1.50 m (the width of a geotextile cloth is 5.0 m).

The total height of the piled tubes again is 4.50 m, the contribution of each individual tube to the total height can be determined by (see fig. 3.2A):

$$H_{tot} = a \cdot \tfrac{3}{4}h \qquad (40)$$

a = number of layers [-]
H_{tot} = dike height (is 4.50 m) [m]
h = individual tube height [m]
n = number of tubes per cross section [-].

The reason only ¾ h is taken into account, is that the tubes settle in the holes of the underlaying tubes. A mathematical approach shows that the remaining volume, if the contribution per layer would be taken 1 h, is $(1 - \pi/4) = 0.21$ times the original volume. Thus the upper tube will utilize this volume, resulting of a decrease of the total height of the pile, in the order of 0.21 h per tube-layer. The contribution of each tube-layer to the total pile-height is thus 0.79 h, or ¾ h.

The tensile stress in the geotextile depends on the location of the tube.
For the upper tube, the geotextile-stress can be found from fig. 3.2A, dependent on filling degree and size of the tube (the upper tube has a drop-shape).
For the bottom tubes, the geotextile-stress is increased due to the load from the tubes resting on these tubes. Since the total height of the construction is 4.50 m, the load, or pressure on the tubes is $4.5 * \gamma = 81$ kN/m2 = Pr [kN/m2].
The bottom tubes do not have a drop shape, they are pressed by the rest of the tubes, and will take a shape, as flat as possible, considering the content. The maximum radius of curvature of the bottom tubes is $r\phi = \tfrac{1}{2}h$, and the subsequent tensile stress in the bottom tubes is: $N\phi = Pr \cdot r\phi = 81 \cdot \tfrac{1}{2} h = 40.5\ h$ [kN/m']

$$N\phi_{bottom\ tube} = 40.5\ h \qquad (41)$$

For different compositions of the cross section, the results for the necessary square meters of cloth, the filling degree of the tubes etc. is given in table 3.2.

D_o [m]	h [m]	filling degree %	content A [m3/m']	total A' [m3/m']	circump. P [m2/m']	total P' [m2/m']	width b [m]	total B [m]	top tube Nϕ [kN/m']	bottom Nϕ [kN/m']
a = 2 n = 3										
3.00	3.0	100%	7.1	21.3	9.4	28.2	3.0	6.0	∞	∞
4.00	3.0	88%	11.3	33.9	12.6	37.8	4.8	9.6	75	125
5.00	3.0	80%	15.7	47.1	15.7	47.1	6.7	13.4	55	125
a = 3 n = 6										
2.00	2.0	100%	3.1	18.6	6.2	37.2	2.0	6.0	∞	∞
3.00	2.0	85%	6.0	36.0	9.4	56.4	3.8	11.4	30	81
4.00	2.0	70%	8.8	52.8	12.6	75.6	5.6	16.8	25	81
a = 4 n = 10										
1.50	1.5	100%	1.8	18.0	5.0	50.0	1.5	6.0	∞	∞
2.00	1.5	90%	2.8	28.0	6.2	62.0	2.4	11.2	25	62
3.00	1.5	70%	4.9	49.0	9.4	94.0	4.2	16.8	12	62
a = 5 n = 15										
1.50	1.2	92%	1.6	24.0	5	90.0	1.7	8.5	20	50
2.00	1.2	80%	2.5	37.5	6.2	93.0	2.7	13.5	15	50

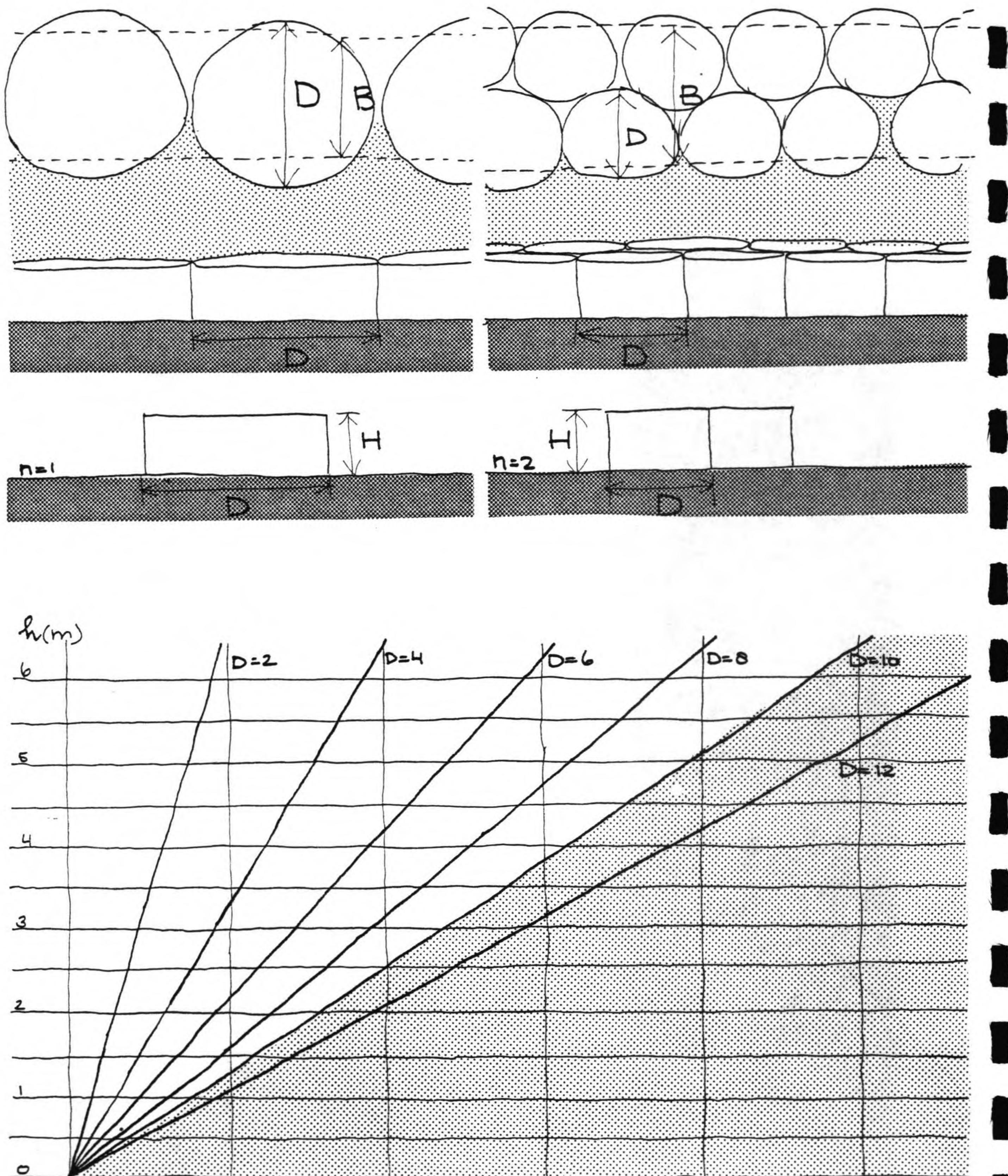
Table 3.2: several tubes per cross-section, H_{constr} = 4.50 m

In this table following expressions are used:

D_o = diameter of original (empty) tube [m]
h = height of individual tube [m]
% = filling degree of individual tube [-]
A = content of each individual tube [m3/m']
A' = total content of the pile of tubes = n * A [m3/m']
P = circumference of individual tube [m2/m']
P' = total square meters of cloth per running meter of dike
= n * P [m2/m']
b = width of individual tube [m]
B = width of the pile = total dike [m]
$N\phi_{top\ tube}$ = tensile force in geotextile of the top tube [kN/m']
$N\phi_{bottom}$ = tensile forces in geotextile of bottom tubes [kN/m'].

One finds that the tensile stress in the geotextile decraeses drastically with an increasing number of tubes per cross-section. The tensile strength of this geotextile can be less, so in spite of the increasing number of square meters of geotextile cloth, this solution could be cheaper than applying one single tube.

In time, the pile of tubes will not survive the various weather-circumstances. Sooner or later the bottom tubes will move (a bit) sideward, and uplaying tubes will sink in between the lower ones, the pile will lose height. In order to avoid this, measures must be taken in order to support the pile in sideward direction. The best way to do so, is to surround the pile by a net, or an extra geotextile.
This auxiliary "membrane" will, in time, take over the gravitational load. So, in time, the stress in this net or geotextile will be, dependent on the content of the pile, (see fig. 3.2A) $N\phi$ = 110 kN/m' to a maximum of $N\phi$ = 150 kN/m' (A' = 30 m3/m').



3.3a

Cylinder shaped elements

3.1.2 Cylinder-elements

The main characteristic of cylinders is the fact that the curvature in horzizontal direction is dominant, the curvature in vertical direction can most times be described by $r\phi = \infty$ (see fig. 3.3A).

The membrane equation (37) now degenerates to:

$$\frac{N\Theta}{r\Theta} = \gamma z + P_o \quad \text{...} (43)$$

Since the load is constant in horizontal direction, the tensile force in the geotextile fibres can be calculated directly from the radius of curvature.
The actual shape of the unit is not so much dependent on filling degree and elongation of the geotextile, but the tension in the membrane is.

Introducing following parameters:

h = height of cylinder [m]
D = diameter of cylinder [m]
A = total content of unit [m3]
A' = equivalent content per running meter [m3/m']
P = circumpherence of cylinder [m2]
P' = equivalent circumpherence per running meter [m2/m']
NΘ = tension in geotextile [kN/m']
B = width of the dike [m].

In the case of cylinders, the relation between shape and content is simple:

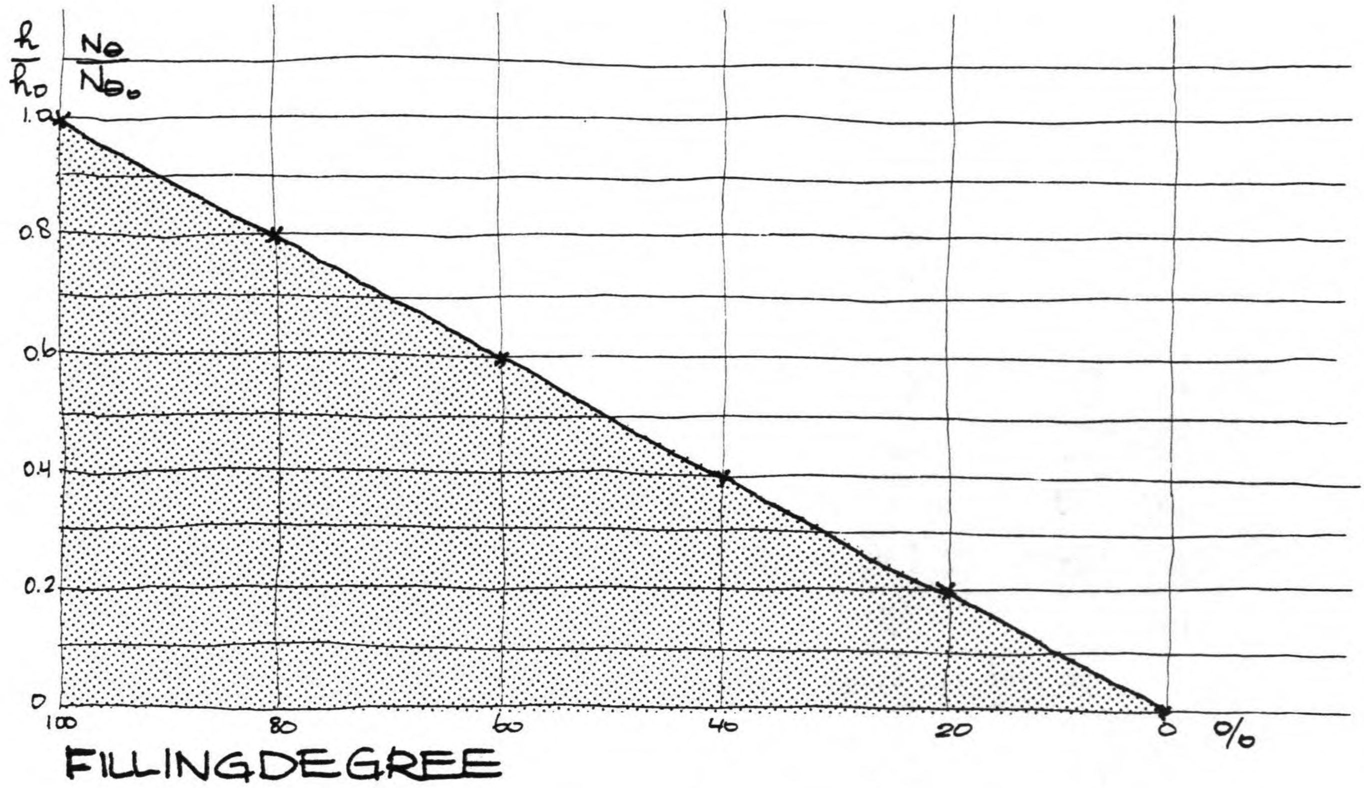
$$A = \tfrac{1}{4}\pi D^2 H \quad \text{...} (44)$$

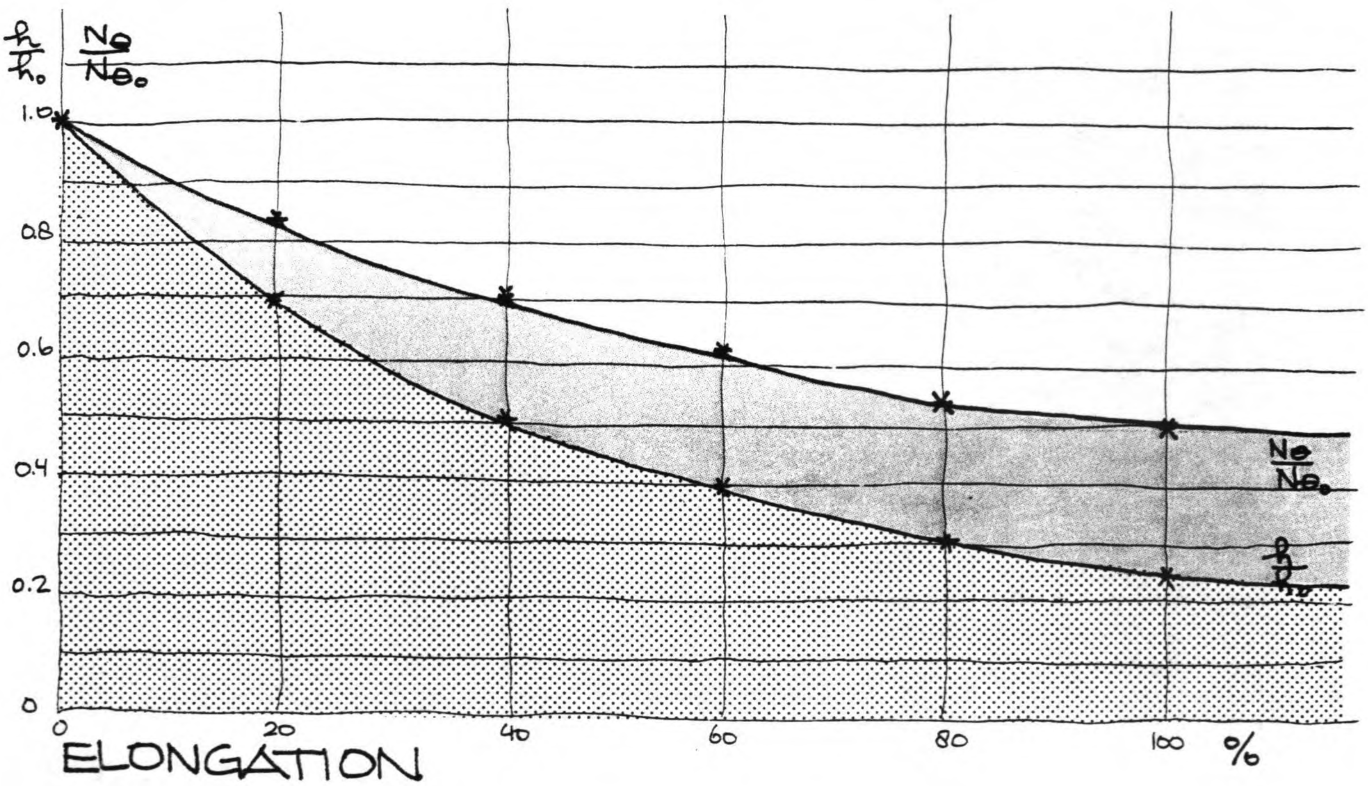
the tensile force can be found by

$$N\Theta = \tfrac{1}{2}D\,(\gamma z + P_o) \quad \text{...} (45)$$

Fig. 3.3A illustrates the relation between A, D and h resulting in NΘ, fig. 3.3B shows the influence of filling degree and elongation.

If cylinders are considered as units to build the dike of, it can be done in one single row of cylinders, or two rows of cylinders (see fig. 3.3.A). The tensile force in the geotextile and the size of the cylinders, the necessary number of square meters of cloth, is given in table 3.3.



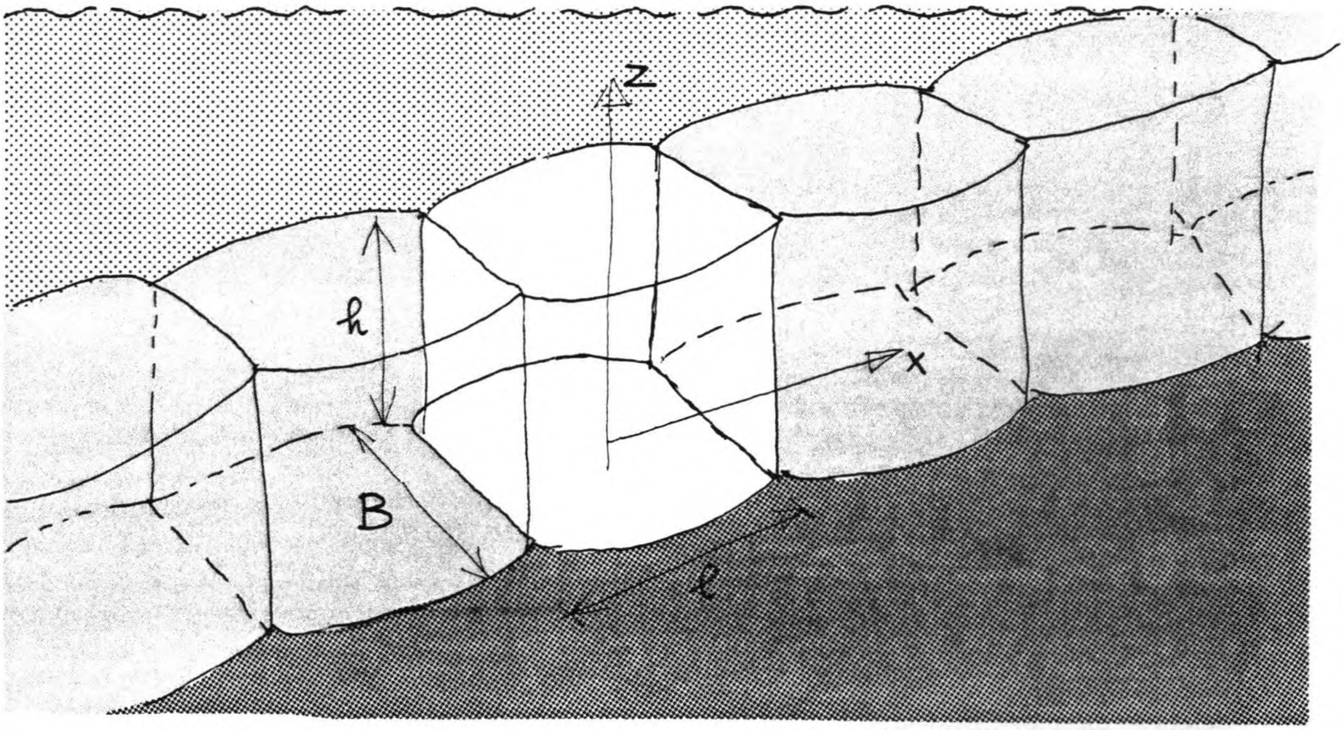


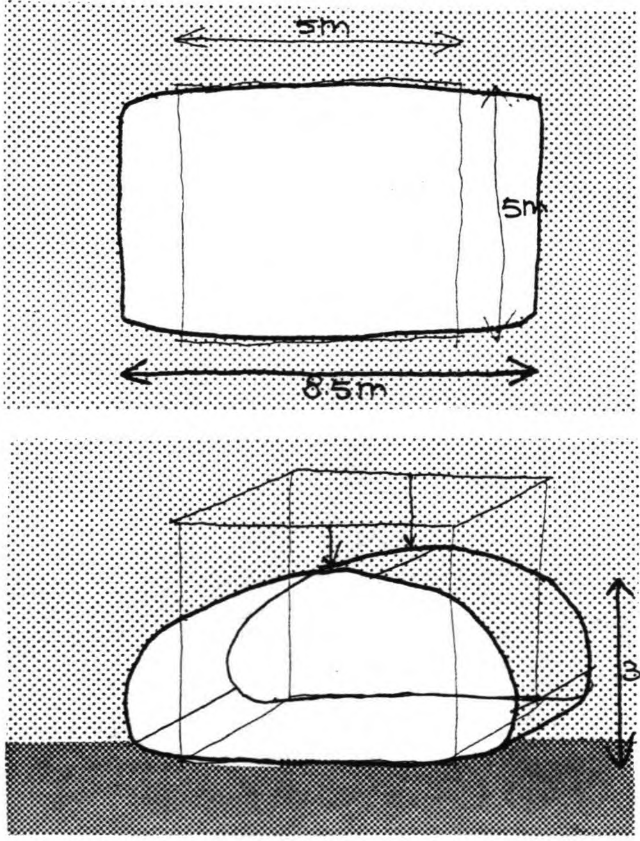
3.3b

filling degree and elongation

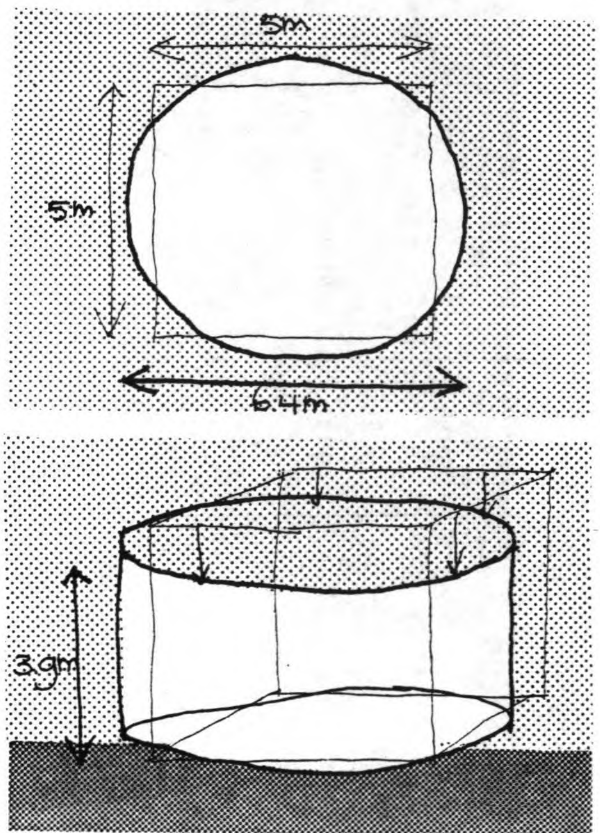
D [m]	h [m]	content A/unit [m3]	total* A' [m3/m']	circumph. P/unit [m2]	total* P' [m2/m']	width* B' [m]	tensile force NΘ [kN/m']
n = 1							
6.00	4.5	127.2	21.2	141	23.5	4.7	243
7.00	4.5	173.1	24.7	176	25.1	5.5	284
8.00	4.5	226.3	28.3	213	26.6	6.3	324
9.00	4.5	286.3	31.8	254	28.7	7.0	365
n = 2							
4.00	4.5	56.5	28.3	81.8	40.0	6.3	162
5.00	4.5	88.4	35.4	110	44.0	7.8	203
6.00	4.5	127.2	42.4	141	47.0	9.4	243

Table 3.3: cylinder-elements, one or two cylinders per cross-section





TUBE-ANALOGY



CYLINDER-ANALOGY

3.4

Cubes

3.1.3 Cube-elements

In the case of cubes, the curvature in horizontal direction is in the same order of magnitude as the curvature in vertical direction. The direction in which the membrane will carry the load of the content, is dependent on the direction in which the radius is limited.

This can be illustrated by the example of a cube with a height of 5 m (see fig. 3.4)
This cube has a content of 125 m3 and a circumference in horizontal and vertical direction of 20 m.

TUBE ANALOGY

The circumference of a tube is $\pi . D_o = 20$ m (in vertical direction) resulting in $D_o = 6.40$ m.

The cube would form a tube of 5 m length, so the content is 25 m3/m'. Compared with $D_o = 6.40$ m, the filling degree is 77%. According to fig. 3.2B then:

$h = 3.80$ m
$N\phi = 100$ kN/m'
$B = 8.50$ m
$\ell = 5.00$ m.

This would result in a maximum circumference in horizontal direction of 2(8.50 + 5.00) = 27 m. In reality the circumference is only 20 m, deformation towards a tube-shape is limited in horizontal direction.

CYLINDER ANALOGY

The circumference of a cylinder is $\pi . D_o = 20$ m (in horizontal direction) reslulting in $D_o = 6.40$ m.

The cube would form a cylinder with a diameter of $D_o = 6.40$ m. Taking into account the total content I = 125 m3, it follows:

$h = 3.93$ m
$N\Theta = 226$ kN/m' (bottom)
$D = 6.40$ m
$B = D$
$\ell = D$.

The maximum circumference in vertical direction would be 2(6.40 + 3.93) = 20.7 m, which is approximately the actual circumference, 20 m.
The deformation towards a cylindric shape is not limited.

CONCLUSION: a cube with a height-length-width-ration of 1:1:1 behaves nearly as a cylinder, the forces and height relation can be determined accordingly.

Table 3.4 gives an overview of the appropriate analogies for several height-width-length ratios (resulting from calculations as has been done for a cube here):

height-width-length-ratios			analogy	
h	B	ℓ	TUBE	CYLINDER
1	1	$\frac{1}{\infty}$	--++	++--
1	1	1/3	--++	++--
1	1	½	--++	++--
1	1	1	---+	+++-
1	1	2	-+++	+---
1	1	3	++++	----
1	1	∞	++++	----
$\frac{1}{\infty}$	1	1	--++	----
1/3	1	1	--++	----
½	1	1	--++	+---
1	1	1	---+	+++-
2	1	1	----	++++
3	1	1	----	++++
∞	1	1	----	++++

Table 3.4: overview of analogies in case of several height-width-length ratios

Thus any shape can be reduced to a cylinder or tube

For the intermediate solutions (not totally tube- or cylinder-analogy, see table 3.4) the total membrane-equation (37) must be solved. In most cases however either a tube-analogy or cylinder-analogy will satisfy to calculate the expected stresses in the geotextile. For example, according to table 3.4, if the h-B-ℓ-ratio is 1:1:2, a tube-analogy is appropriate, if the h-B-ℓ-ratio is 1:1:1, a cylinder-analogy. The cases in which neither the tube- or cylinder-analogy is appropriate (for example h-B-ℓ is 1:1:½) are not occuring in practice (such shapes would be very uneconomical).

3.2 Dynamic behaviour

General

Except for the static or gravitational loading, also dynamic loadings are acting on the dike, caused by waves, wind and currents. In chapter 2 the loadings under normal and extreme conditions have been determined.

The forces due to the gravitational effects, are always acting on the dike (the subsequent tensile stress in the geotextile forms a long term static loading on the membrane).
Even if the dike is situated under the watersurface (high tide) and the weight is reduced according to Archimedes'law, the tensile stress in the geotextile remains the same. This can be explained by considering the filling of a stiff mass. Since the shape of the mass remains the same under water, the specific elongation of the geotextile at every spot remains the same, and thus also the tensile stresses in the geotextile will remain the same.

Dynamic loadings in the geotextile are caused by waves slamming against it, therefore causing pressure differences between out- and innerside of the cloth with subsequent tensile stresses in the geotextile.

Dynamic loadings in the filling and at the dike as a whole are caused by wave loadings, which have a cyclic character.

The response of the structure components is not easy to determine, therefore some simple approaches have been followed, which form either an upper limit of the response (the actual response must be less) or a lower limit (this response is the least that can be expected).

In the following it will be discussed:

A. THE HUDSON-CONDITION: (lower limit)

armour units of breakwaters are submitted to wave loadings. Hudson developped an equation by which the necessary weight of these units can be calculated.

B. THE SLIDING CONDITION: (lower limit)

like other constructions, the contact-surface between the dike and the subsoil should have a certain size to prevent sliding of the construction under horizontal loadings.

C. THE LIFT CONDITION:

the wave loadings cause a lift force on the construction. It can be put quite simply that the weight of the construction must exceed this lift force in order to guarantee stability.

D. THE STRESS STRAIN BEHAVIOUR OF THE GEOTEXTILE UNDER DYNAMIC LOADINGS:

wave slamming can cause high local pressures on the geotextile and subsequent extra tensile stresses. A very rough appproach is discussed to calculate these extra dynamic stresses.

E. THE SPRING-MASS SYSTEM:

actually in A to D the dynamic loading is approached in a quasi-static way. A construction reacts on dynamic loadings by means of (small) deformations. This way the loadings are spread over the construction and eventually the loading energy is spoilt into the subsoil plus environment (by heat or deformation). The system of geotextile-filling-subsoil can be described by a spring mass system (if necessary a damped system).

3.2.1 The Hudson-condition

Hudson developped an equation to determine the minimum necessary weight of an armour unit used to protect breakwaters (discussed in lit. (8)). It is an empirical formula, having some significant limitations, but which is simple to use (a more appropriate formula has been developped by Iribarren, 1938, which separates the uprush forces and backwash forces) (see fig. 3.5).

$$M \geq \frac{\rho s \; H_s{}^3}{K_\Delta (\frac{\rho_s}{\rho_w} - 1)^3 \cot\alpha} \qquad \text{.......(46)}$$

H_s = maximum significant wave height [m]
ρ_s = specific mass of unit [kg/m3]
ρ_w = specific mass of water [kg/m3]
K_Δ = damage coefficient [-]
α = angle of slope, $\alpha < 33°$, $\cot\alpha > 1.5$ [rad]
M = mass of the unit [kg]
A = volume of the unit [m3].

Using H_s = 2.50 m, ρ_s = 1,800 kg/m3
K_Δ = 4, $\cot\alpha$ = 1.5
we find: $M \geq$ 9,155 kg, or $A \geq$ 5.4 m3.

For the unit, the mass is determined by:

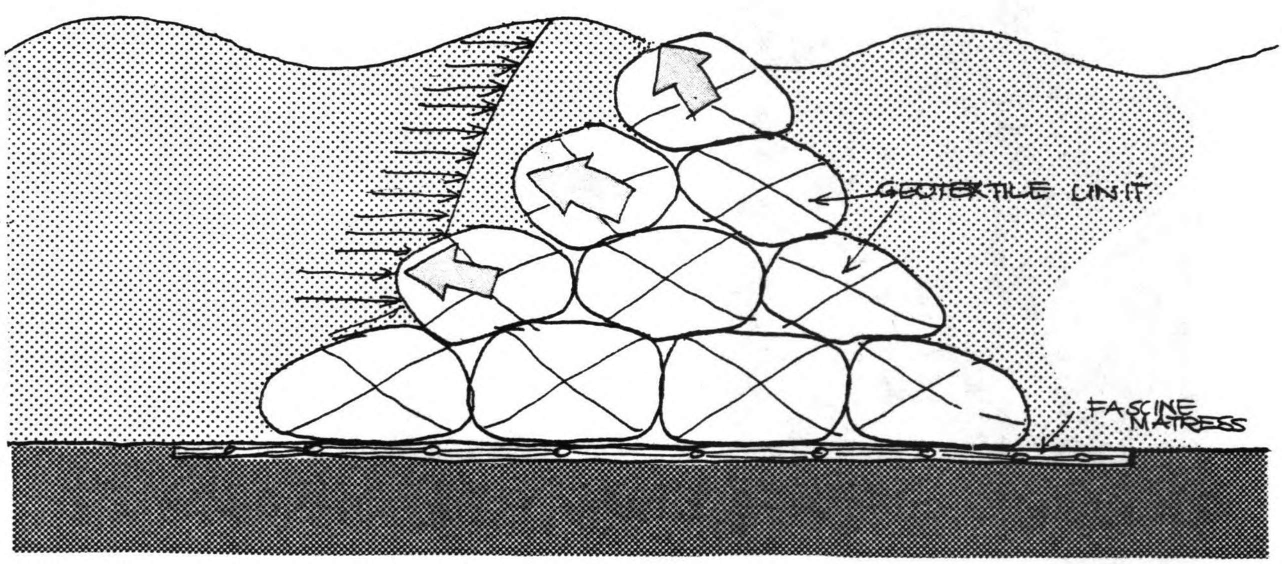
$$M = \tfrac{1}{4}\pi D_o^2 \ell . \rho_s \qquad \text{.......(47)}$$

D_o = diameter tube or cylinder [m]
ℓ = length of the tube, heigth of the cylinder [m].

In the case of a tube it is not realistic to suppose that all of the length contributes to one equivalent "unit" in the Hudson-formula. Therefore it is assumed that the cooperative length ℓ can be taken the original diameter, analogeous to the case of cubes.

$M = \tfrac{1}{4}\pi D_o{}^3 . \rho s \geq 9,155$ kg
and
$D_o \geq 1.90$ m

CONCLUSION: the diameter of D_o = 1.90 m can be considered the minimum requirement of a unit to be used. (The dike units are not under the same circumstances as armour units; armour units are carried by a massive core as for the dike units form a core selves. Furthermore the slope-angle α is hard to interpretate, it is somewhere between α = 0 and α = 45°.)



3.5

The Hudson-analogy

3.2.2 The sliding condition

The horizontal force component of the wave loadings (see chapter 2) causes a horizontal dislocation of the unit. This dislocation is resisted by the friction force which exists between the contract surface of the unit and the subsoil. This way a minimum size of the contract surface can be determined: the condition is that the maximum horizontal force must be smaller than the maximum friction force (see fig. 3.6)

$$F_{hor} \leq W \quad (48)$$

$\hat{F}_{hor}$ = 100 kN/m' (see chapter 2).

W is determined by the friction force of the subsoil that is accumulated along a possible slidingsurface. In this case the slidingsurface is the contact surface between dike and subsoil (this is the minimum sliding-surface).

$$W = \tau S \quad (49)$$

τ = friction between two layers (shear stress) [kN/m2]
S = contact surface [m2].

For soils the shear stress τ can be determined by following relation

$$\tau = \bar{\sigma}_k \tan \phi + Cu \quad (50)$$

$\bar{\sigma}_k$ = grain stress [kN/m2]
ϕ = angle of internal friction [rad]
Cu = cohesion of the soil, for silt Cu $\simeq$ 0 [kN/m2].

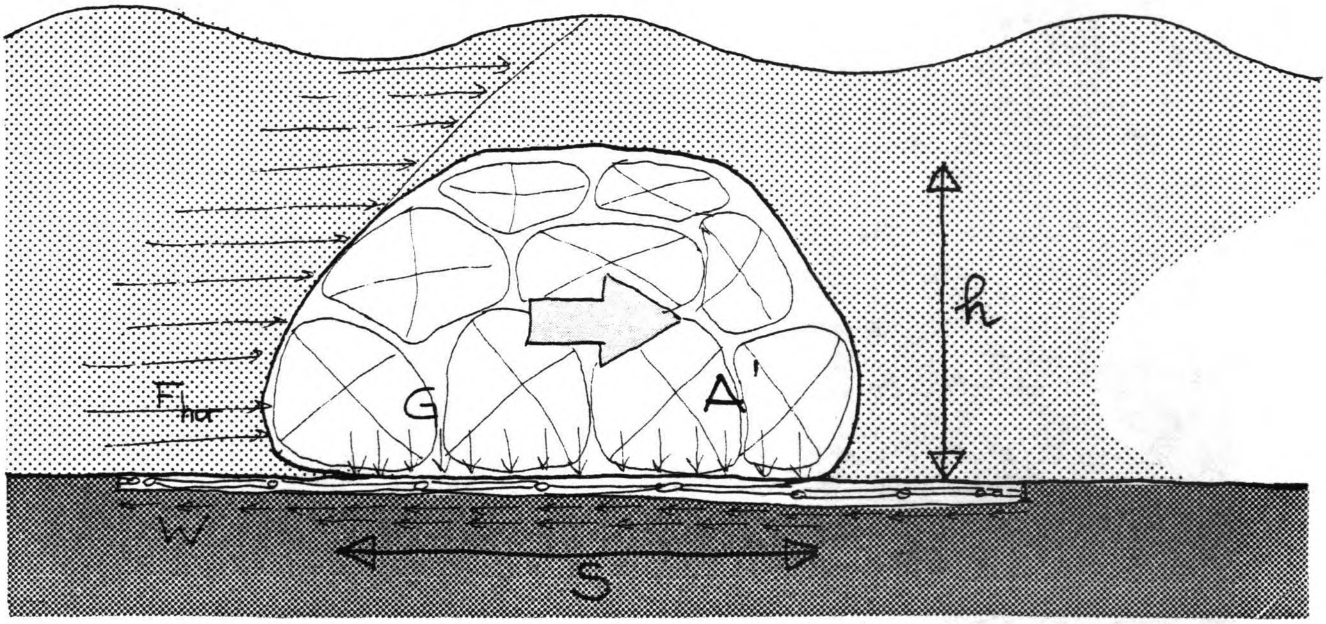
The average grain stress $\bar{\sigma}_k$ can be found by:

$$\bar{\sigma}_k = \frac{G}{S} \quad (51)$$

G = weight of the unit = A.γ [kN]
S = contact surface [m2]
A = volume of unit [m3]
γ = specific weight [kN/m3].

Then eq. 48 can be transformed, independent of the contact surface it follows:

$$\hat{F}_{hor} \leq G \tan \phi \quad (52)$$



3.6

The sliding condition

DIKE BUILT OF TUBE-UNITS

the top of the dike is at a level of 3.50 m above the subsoil

$\hat{F}_{hor}$ = 100 kN/m'
G = A'γ kN/m'.

In the most critical case , the tube(s) is (are) situated under water, so the specific weight γ (= $(\rho_s - \rho_w) * g$) is 8 kN/m3

$A' \geq 31.25$ m3/m' (compare par. 3.1.1).

DIKE BUILT OF CYLINDER-UNITS

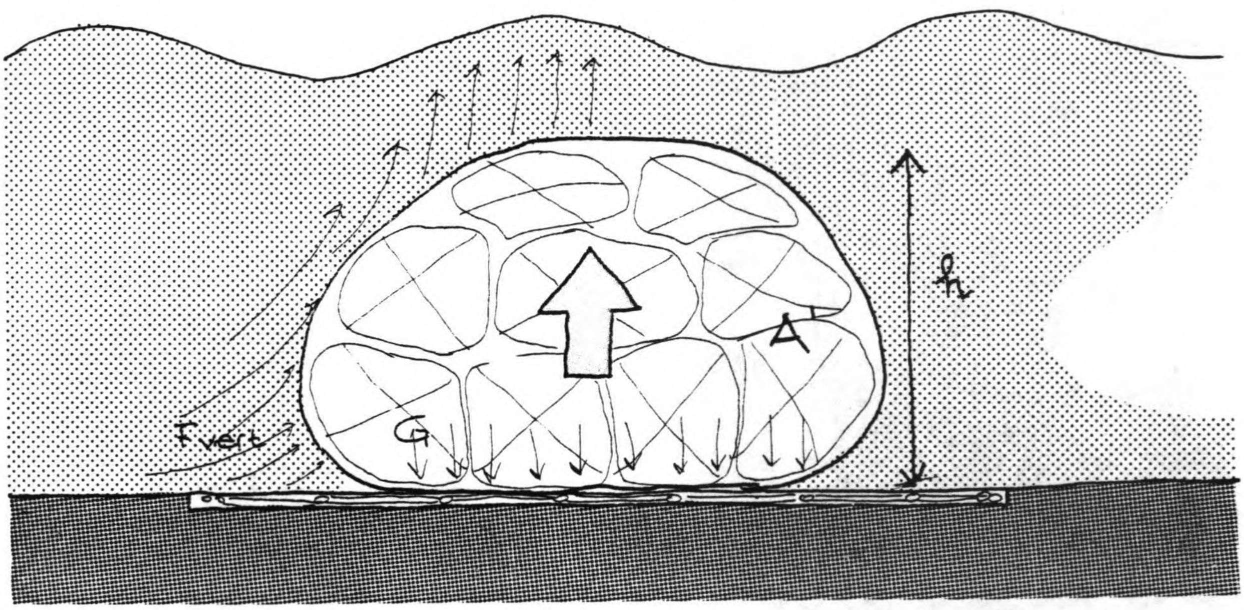
if we consider a cylinder as a whole:

$\hat{F}_{hor}$ = 100.D kN
G = $\frac{1}{4}\pi D^2 \gamma h$ kN
and $D \geq 11.4$ m (total dike)
or $A' \geq 31.2$ m3/m' (compare par. 3.1.2).

CONCLUSION: in order to prevent horizontal dislocation, the minimum content of the dike must be 31.2, say:

30 m3/m'

(irrespective of the shape of the units).



3.7

The lift condition

3.2.3 The lift condition

The vertical force component of the wave loadings (see chapter 2) causes a vertical dislocation of the units. For the dike it is not permitted that any of the units are lifted.
A condition for stability is that the gravity force is larger than the lift force of the dike (see fig. 3.7)

$$\hat{F}_{vert} \leq G \qquad (52)$$

$\hat{F}_{vert}$ = 150 kN/m' (see chapter 2) [kN/m']
G = weight of the dike = $A\gamma$ [N/m3]
A = volume of the dike [m3]
γ = specific weight of the filling = $(\rho_s-\rho_w)$ g [N/m3]
g = acceleration of gravity [m/s^2].

DIKE BUILT OF TUBES

the top of the dike is situated at a level of 3.50 m above the subsoil so $\hat{F}_{vert}$ = 150 kN/m',
$A' \geq 18.7$ m3/m'.

DIKE BUILT OF CYLINDERS

also $\hat{F}_{vert}$ = 150 kN/m' and
$A' \geq 18.7$ m3/m' ($D > 3.0$ m).

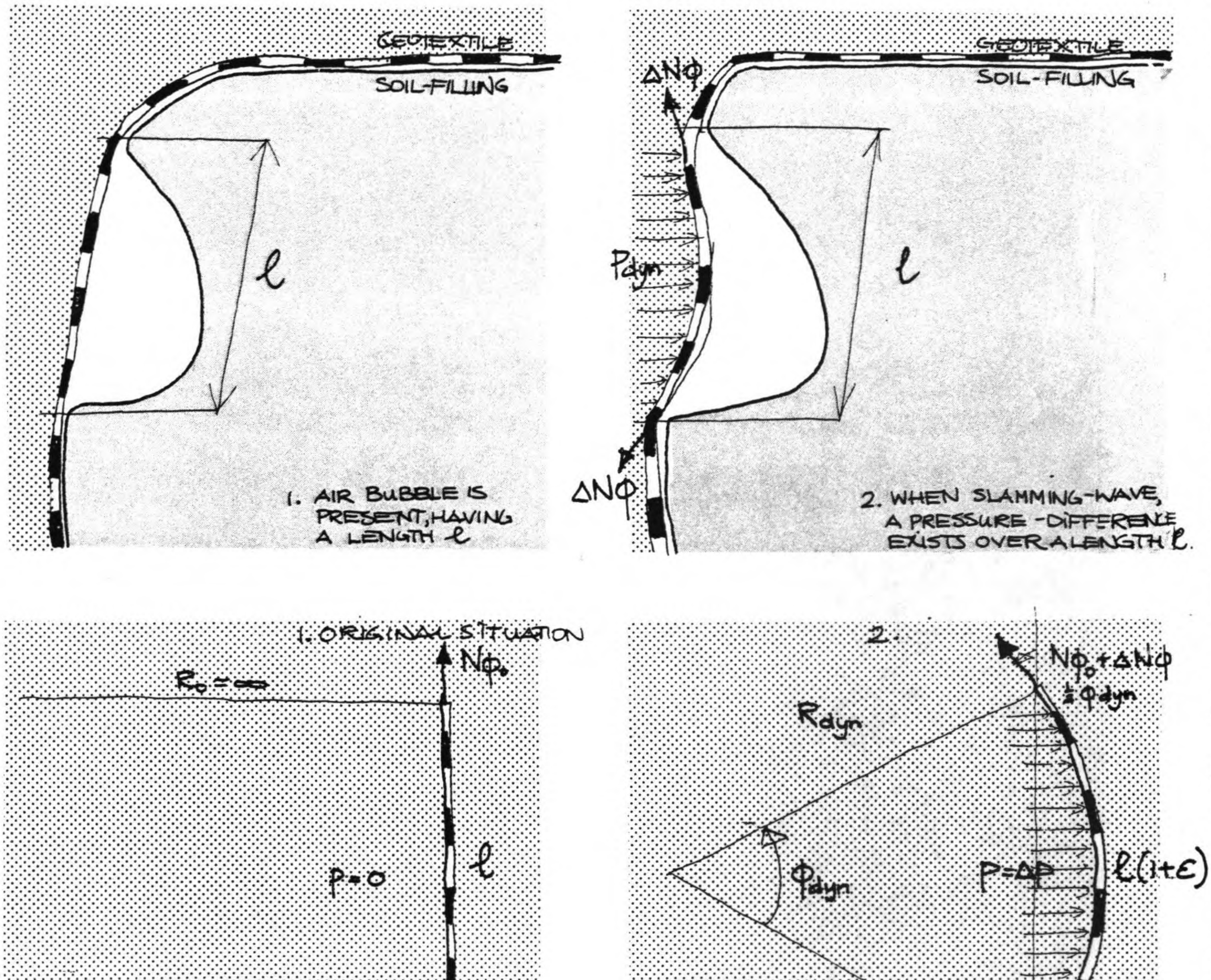
CONCLUSION: in order to prevent lifting the minimum content of the dike must be:

$A' = 18.7$ m3/m'

N.B.: fortunately the lift force is out of phase with the horizontal force, otherwise the "weight" (in eq. 51) should be substituted by the resulting vertical force: gravity minus buoyancy, minus the lift force in order to calculate the grain stress

$$"G" = A'(\rho_s - \rho_w)g - F_{lift}.$$

If the lift force and the horizontal force would be in phase: (now the lift force is low at the moment of maximum hor. force).
This would result in $A' \geq 50$ m3/m' for both tubes and cylinders.



3.8a

Extra dynamic stresses in the geotextile

3.2.4 The stress strain behaviour of the geotextile under dynamic loadings

Generally, the geotextile will have direct contact with the filling, so that the dynamic loadings will be carried by the filling. (The influence of the geotextile on the response of a whole of the dike is negligible.)

However, locally some filling can be absent, due to air bubbles non-homogenesus filling etc., and locally the geotextile will have to carry the dynamic pressure (see fig. 3.8).
The geotextile itself has no resitance against sideward pressure, extra loading has to be carried through an increase of the tensile force $N\phi$ and through curvature of the geotextile (see fig. 3.8A).

Theoretically, according to the membrane-equation:

$$N\phi_o = P_o R_o \qquad (53)$$

and

$$N\phi_o + \Delta N\phi = (P_o + \Delta P)R_{dyn} \qquad (54)$$

$N\phi$ = tensile force in geotextile [kN/m']
$\Delta N\phi$ = extra tensile force in geotextile [kN/m']
P_o = initial pressure difference [kN/m2]
ΔP = extra pressure over the geotextile [kN/m2]
R_o = initial radius of curvature [m]
R_{dyn} = "dynamic" radius of curvature [m].

The most disadvantageous case is when

- the initial radius of curvature is ∞: the geotextile is straight;
- the extra dynamic pressure is caused by wave slamming. Then the air inside the "bubble" might be absorbed in the filling: the pressure difference over the geotextile will be equal to the dynamic slamming pressure (extreme condition).

The elongation determines the increase of the tensile force (strain) in the geotextile. According to 53, 54:

$$\Delta N\phi = P_o(R_{dyn} - R_o) + \Delta P\, R_{dyn} \qquad (55)$$

If the initial curvature is zero, also the initial pressure difference P_o is zero: the membrane is in equilibrium.
ΔP = 100 kN/m2 according to chapter 2, so

$\Delta N\phi$ = 100 R_{dyn} [kN/m'].

The R_{dyn} is determined by the stress-strain relation of the geotextile (R_{dyn} is shown in Fig. 3.8.A, the algebraic relation in eq. 56 and table 3.5). If we consider a strip of geotextile with a reprensentative length $\ell = 1$ m then the length after the load has been introduced will be $\ell(1 + \varepsilon)$, ε = elongation [-] resulting in a specific curvature (R_{dyn}) and successive tensile force ($\Delta N\phi$). The laws of goniometry learn:

- if $R_o = \infty$ (initial: zero curvature) following expressions apply:

  1. $R_{dyn}\ \phi_{dyn} = \ell(1 + \varepsilon)$

  2. $\sin \tfrac{1}{2}\phi_{dyn} = \tfrac{1}{2}\ell/R_{dyn}$

  together:

$$R_{dyn} = \frac{\ell(1 + \varepsilon)}{\phi_{dyn}} = \frac{\tfrac{1}{2}\ell}{\sin \tfrac{1}{2}\phi_{dyn}}$$

  and

$$\frac{\sin \tfrac{1}{2}\phi_{dyn}}{\tfrac{1}{2}\phi_{dyn}} = \frac{1}{1 + \varepsilon} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(56)}$$

- If $R_o \neq \infty$ (initial curvature) following expressions apply:

  1. $R_{dyn}\ \phi_{dyn} + \ell(1 + \varepsilon)$

  2. $R_o\phi_o = \ell$

  3. $\sin \tfrac{1}{2}\phi_o\ .\ R_o = \sin \tfrac{1}{2}\phi_{dyn}\ R_{dyn}$

  together:

$$R_{dyn} = \frac{\ell(1 + \varepsilon)}{\phi_{dyn}} = \frac{\sin \tfrac{1}{2}\phi_o}{\sin \tfrac{1}{2}\phi_{dyn}} \cdot \frac{\ell}{\phi_o}$$

  and

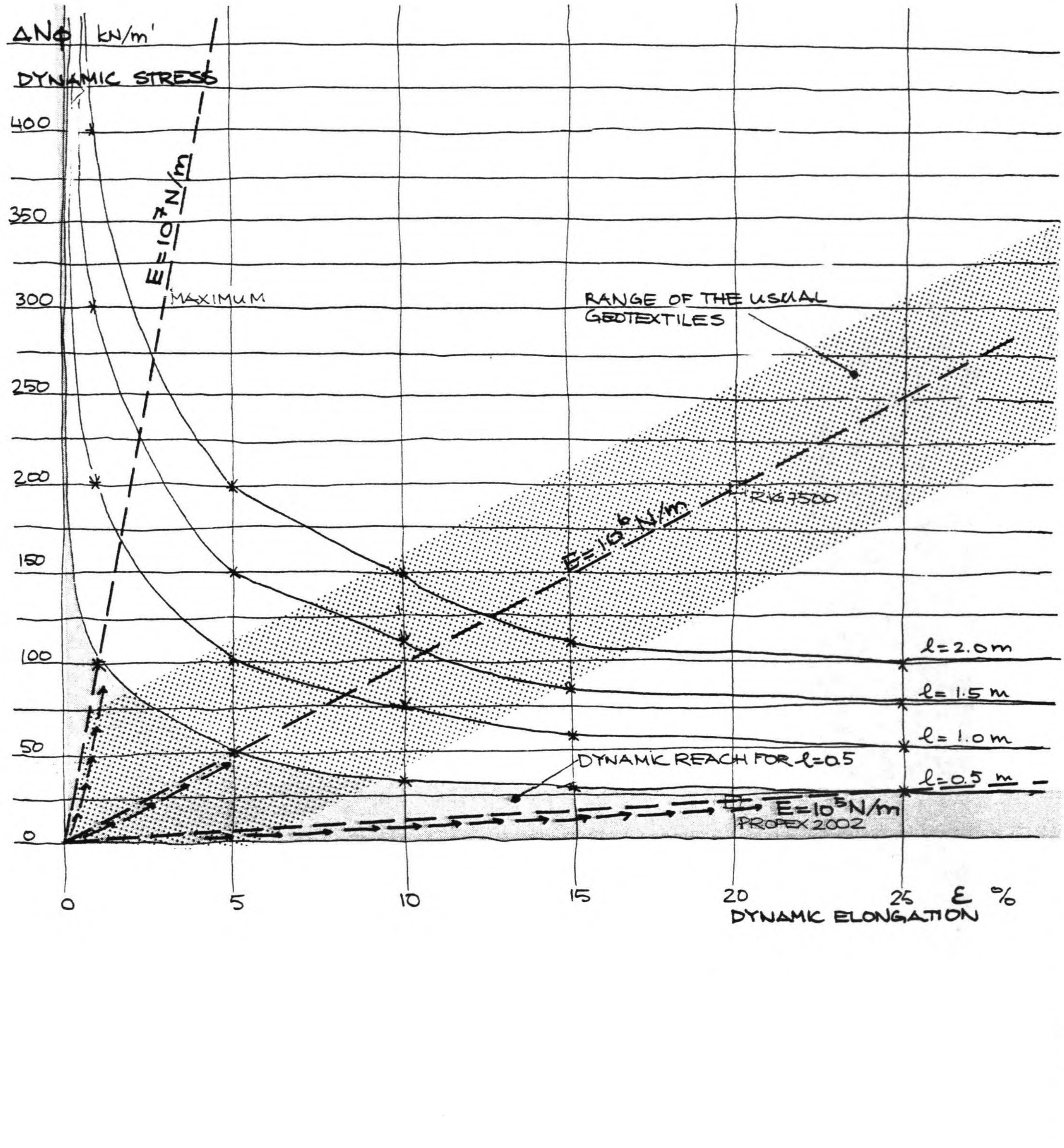
$$\frac{\sin \tfrac{1}{2}\phi_{dyn}}{\tfrac{1}{2}\phi_{dyn}} = \frac{1}{1 + \varepsilon}\ \frac{\sin \tfrac{1}{2}\phi_o}{\tfrac{1}{2}\phi_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(57)}$$

The last case is more advantageous, since the term $\left(\frac{\sin \tfrac{1}{2}\phi_o}{\tfrac{1}{2}\phi_o}\right)$ is always smaller than one ($\phi \neq 0$), thus R_{dyn} will decrease, so will $\Delta N\phi$.
The relation between ε and $\Delta N\phi$ is given in table 3.5. It is not yet determined what the real value of ε will be, this depends on the elasticity-modules of the geotextile.

ε (%)	$\frac{\sin \frac{1}{2}\varphi}{\frac{1}{2}\varphi}$	φ	R_{dyn} (m)	$\Delta N\phi$ (kN/m')
0	1.000	> 0.10	> 10.0 ℓ	1.000 . ℓ
1	0.990	0.50	2.02 ℓ	202 . ℓ
2	0.980	0.70	1.46 ℓ	146 . ℓ
3	0.971	0.84	1.23 ℓ	123 . ℓ
4	0.962	0.96	1.08 ℓ	108 . ℓ
5	0.952	1.04	1.01 ℓ	101 . ℓ
6	0.943	1.18	0.90 ℓ	90 . ℓ
7	0.935	1.26	0.85 ℓ	85 . ℓ
8	0.926	1.34	0.81 ℓ	81 . ℓ
9	0.917	1.43	0.76 ℓ	76 . ℓ
10	0.909	1.50	0.73 ℓ	73 . ℓ
15	0.870	1.80	0.64 ℓ	64 . ℓ
100*)	0.637	3.14	0.50 ℓ	50 . ℓ
∞*)	0.637	3.14	0.50 ℓ	50 . ℓ

Table 3.5: relation between elongation and extra dynamic tensile stress, dependent on striplength ℓ and elasticity of the geotextile (see fig. 3.8B)

*) N.B.: the maximum curvature, or the minimum radius is limited by the strip length ℓ, in such a way that $2R = \ell$. This can also be interpreted: the total load is $\Delta P.\ell$. So the string components should be at least $\frac{1}{2}\Delta P\ell$, in fact they are: $\Delta N\phi = \frac{1}{2}\Delta P\ell / \sin\frac{1}{2}\varphi$.



3.8b

stress-strain relations

In the foregoing a strip of a length of 1 m has been taken into account, in reality the strip will not have this length. The actual length is dependent on the size of the geotextile units.

DIKE BUILT OF TUBES

if the dike is constructed out of one single tube, the maximum strip length ℓ is determined by the size of the air-bubbles in the fill. A rough estimate is a maximum size of ℓ = 0.50 m.

When more tubes are used, they will be surrounded by an envelope. The maximum strip length is now also dependent on the height of the units (see Fig. 3.8A, $\ell \approx h$) If n is the number of units per cross section, the maximum size of a piece of cloth, not supported by filling is:

for n = 3 ℓ_{max} = 3.00 m (2 tube-layers)
n = 6 ℓ_{max} = 2.00 m (3 tube-layers)
n = 10 ℓ_{max} = 1.50 m (4 tube-layers)
n = 15 ℓ_{max} = 1.20 m (5 tube-layers)
n = 21 ℓ_{max} = 1.00 m (6 tube-layers).

DIKE BUILT OF CYLINDERS

the maximum strip length is always determined by the size of the bubbels in the filling since the units are not surrounded by an extra geotextile:

ℓ_{max} = 0.50 m.

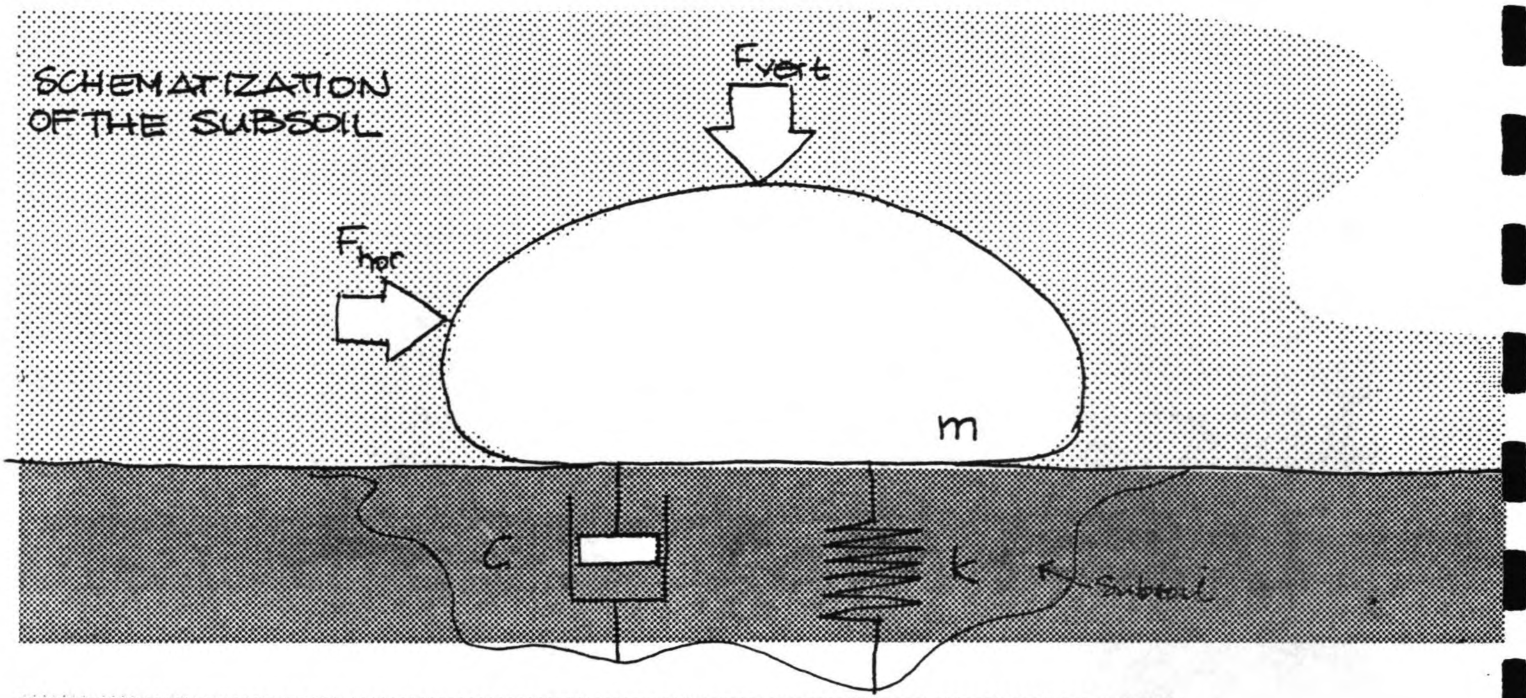
In fig. 3.8B the regions of dynamic extra tensile force are given, for different lengths of strips, and ΔP = 100 kN/m2 (the extreme case).

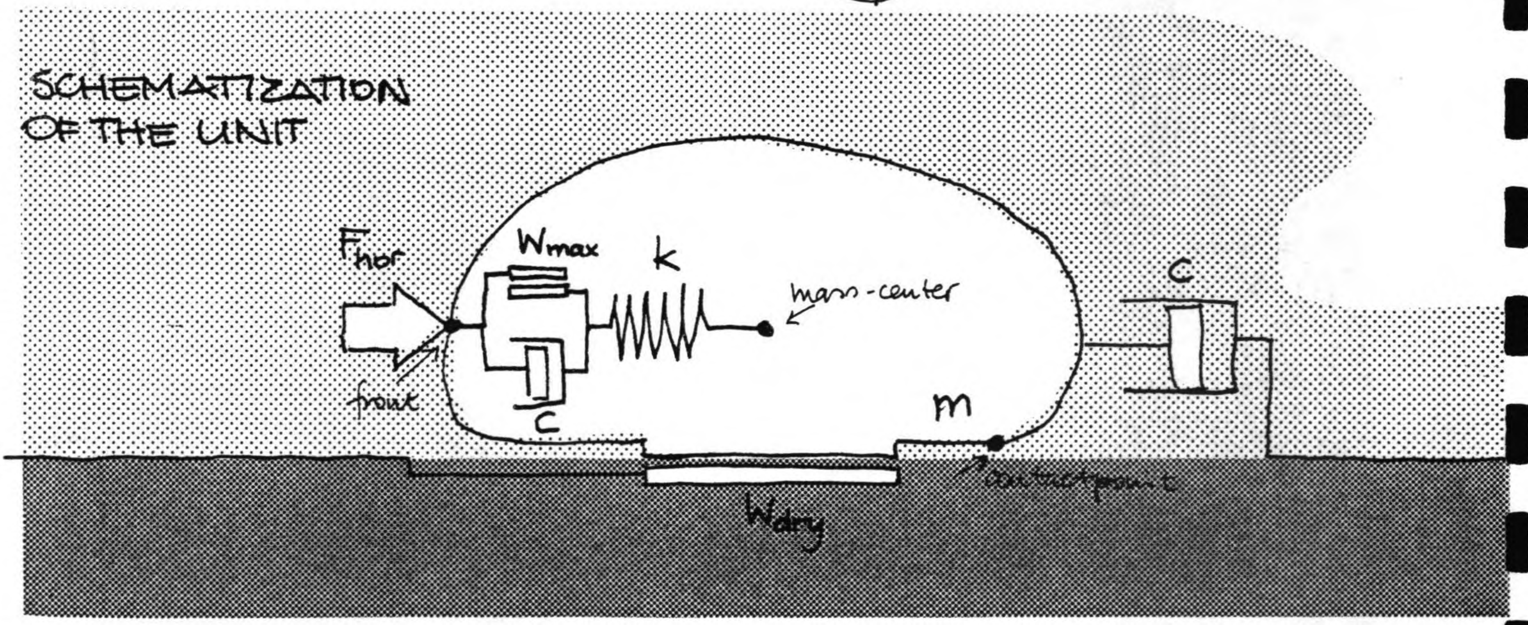
A method to prevent the dynamic extra tensile force, when several units are used to built the dike, is to fill up the interspaces after the individual tubes have been filled. Then the maximum strip length is reduced to $\ell_{max} \approx 0.5$ m.

CONCLUSION: the dynamic extra tensile stresses in the geotextile can be found from its stress-strain relation. The higher the elasticity-modulus of the geotextile, the less the textile will elongate under dynamic loadings and the higher the increase of the stress in the textile. As a maximum for the dynamic stress it is taken:

$\Delta N\phi \simeq 100$ kN/m'

since the elasticity-modulus of existing fabrics is never above $E = 10.10^6$ N/m (or 1.000 kN/m') and the strip length will be about 0.50 m.
The striplength ℓ is only larger than 0.50 m, if the dike is built of several tubes. In this case, the outer geotextile is porous, and the maximum pressure difference will not exist. As an upper limit of the dynamic stress it is assumed that also in the case $\ell > 0.50$ m: $\Delta N\phi$ = 100 kN/m'. Tests will have to verify this.





3.9a

Spring mass systems : schematizations

3.2.5 The spring mass system

An attempt to predict the actual dynamic response is given in appendix B: dynamic response.
Fig. 3.9A shows the schematizations used.
Fig. 3.9B gives the results of the computations.

The dynamic response of the construction can be found from an intensive investigation of the mechanical characteristics of the construction. This should result in a number of characteristic frequences of the units, resulting in eigen vectors, and the transmission spectrum.
The wave spectrum used as input, the output: the response.

However more information (wave spectra, soil characteristics etc.) would be necessary to perform this analysis of the response.

Simplistic schematization results in:
- characteristic frequences:

  total construction:
  $\omega = 6$ rad/s (horizontal displacement)
  $\omega = 0$ rad/s (rotation)

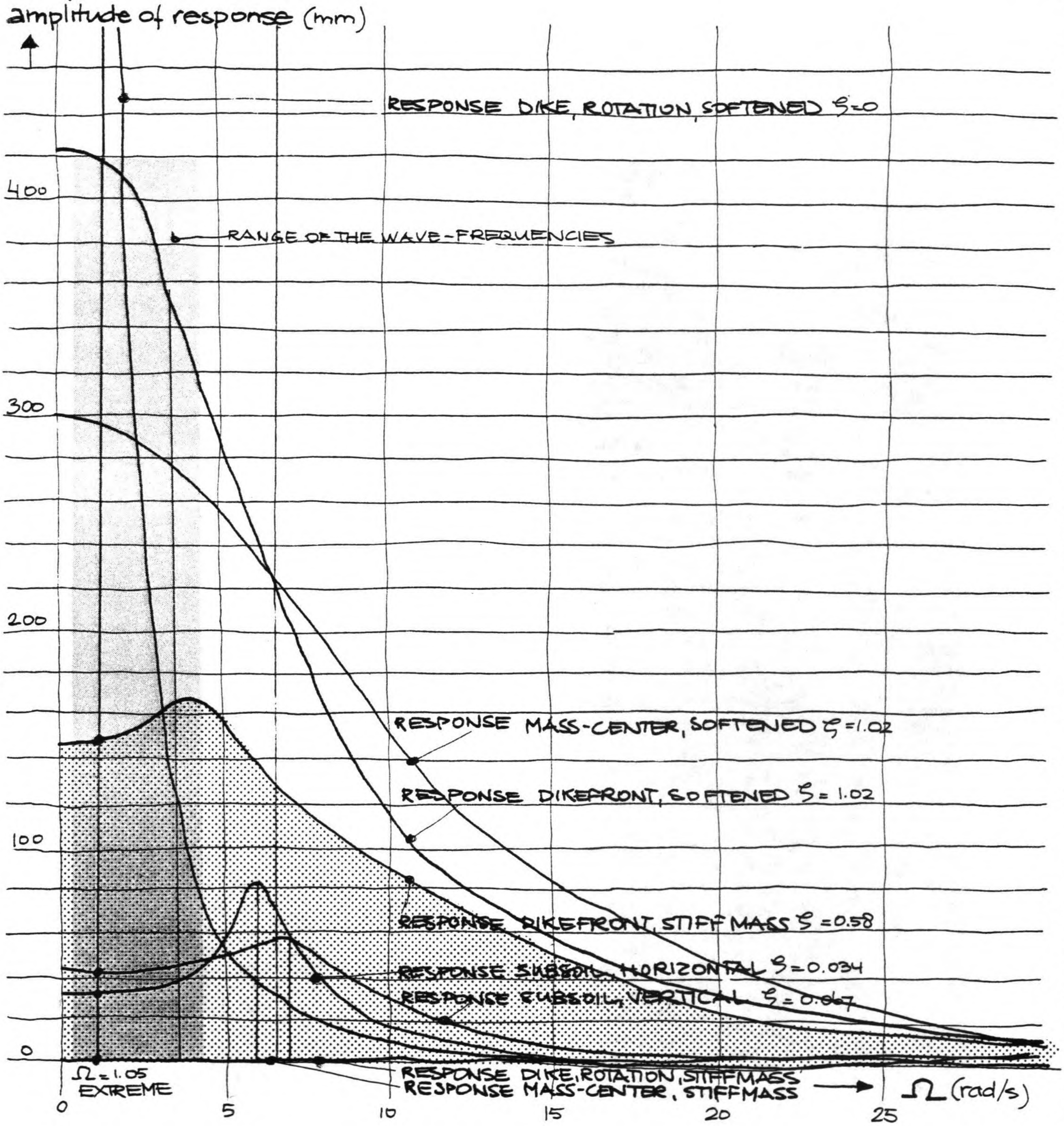
  subsoil:
  $\omega = 7$ rad/s (horizontal displacement)
  $\omega = 6.7$ rad/s (vertical displacement).

- The construction responds in a static way to wave loadings: it can be regarded as a stiff mass attacked by waves.

- The wave loadings frequencies are in the range of (see fig. 3.9B) $\Omega = 4$ to $\Omega = 0.5$, the extreme loadings occur around $\Omega = 1$ to Ω 0.62.
  So the loadings are not in the range of the eigen-frequencies, resonance will not occur (except in the case of $\Omega = 4$, but for high wave-frequencies the waves are very small, thus not causing a response of any importance).

CONCLUSION:

- The wave-loading frequencies are not in the range of the characteristic frequencies of the units.
  The response of the construction consists of small elastic and plastic deformations.



3.9b

Spring mass systems

- Softening of the filling is not allowed, it is adviced to compose a cross-section that consists out of more than one unit, for safety-reasons.

- Erosion of the subsoil is not allowed, therefore the subsoil has to be protected by a fascine matress.

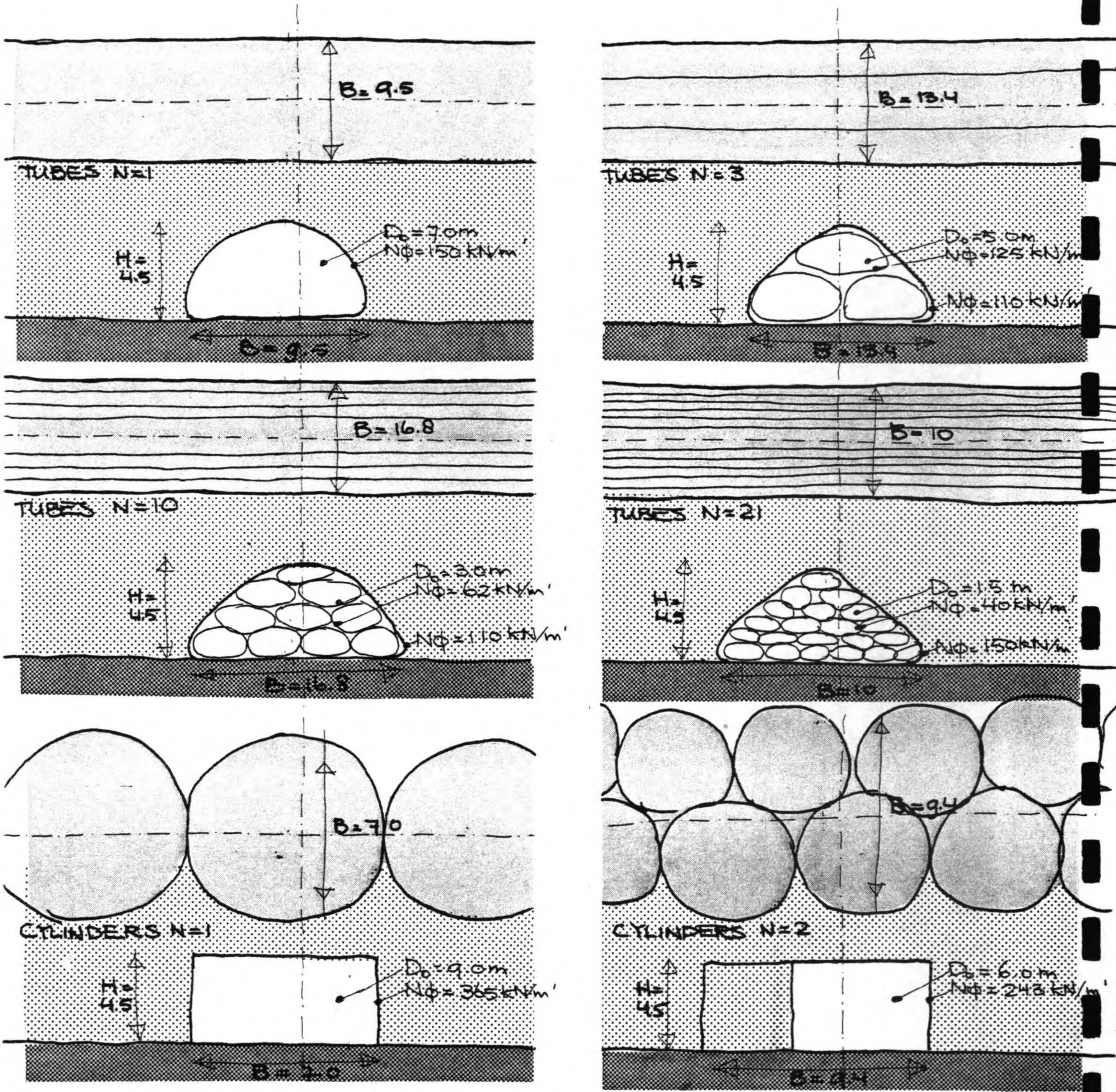
- Wave slamming (Ω = 126 rad/s) does not cause a response of the construction of any importance.
  The frequency is too high for the construction to react (see also appendix B).
  The wave slamming-load only influences the tensile stress in the geotextile membrane of the construction (see par. 3.2.4).

N.B.: in the calculations done in this paragraph and in appendix B, the mass of the construction is assumed to be 30 m3/m', since the slidingcondition results in this minimim mass (see par. 3.2.2).
A different mass would result in different characteristic frequencies.

For example: if units of D = 2.0 m are considered

ω = 10 rad/s (horizontal displacement)
ω = 0 rad/s (rotation).

If the construction with a total mass of 30 m3/m' is composed out of small units, the characteristic frequencies of the units are superimposed on the characteristic frequencies of the total construction.



3.10

Principle sketches of composition of the dike

3.3 Overview of responses

The response of the constructions depends, especially the static response, on the composition of the construction (for dynamic loadings, the construction reacts as a whole, so the total mass is determining the response, not the composition).

For each type of geotextile unit the response (expected) is given below. A division is made between tube-shaped units and cylinder-shaped units.

The parameters are following: (see fig. 3.10)

n = number of units per cross section [-]
D_o = diameter of units [m]
h = height of units [m]
% = filling degree [-]
A' = content of toal construction per running meter [m3/m']
P' = total number of necessary square meters of geotextile per running meter [m2/m']
X = maximum horizontal displacement (amplitude of movement) [m]
Z = maximum vertical displacement (amplitude of movement) [m]
$\Delta N\phi$ = maximum extra dynamic tensile stress in geotextile [kN/m']
$N\phi$ = static stress in geotextile [kN/m'].

In the cass of tube units it is assumed that the tubes (only if $n > 1$) are surrounded by an outer membrane, which carries the dynamic loadings and which holds together the units. (In the case of cylinders this is not necessary, since cylindric units are self-carrying.)

For each of the discussed units, the response is given in table 3.6 (tube-elements) and table 3.7 (cylinder-elements).

TUBES					dynamic RESPONSE				static	
n	D_o [m]	%	A' [m3/m']	P' [m2/m']	X [m]	Z [m]	ΔNφ unit	ΔNφ outer	Nφ unit	Nφ outer
1	10	60	54	34	0	0	100	*	110	*
	8	75	37	25	0	0	100	*	125	*
	7	80	30	22	0.1	0	100	*	150	*
	6	90	23	18	0.7	0	100	*	300	*
3	5	80	47	48	0	0	*	100	125	110
	4	90	34	40	0	0	*	100	125	140
	3	100	21	29	0.8	0	*	100	∞	500
6	4	70	53	76	0	0	*	100	81	110
	3	85	36	57	0	0	*	100	81	130
	2	100	19	38	1.2	0	*	100	∞	600
10	3	70	49	94	0	0	*	100	62	110
	2	90	28	62	0.5	0	*	100	62	150
	1.5	100	18	50	1.3	0	*	100	∞	600
15	2	80	38	93	0	0	*	100	50	125
	1.5	92	24	90	0.6	0	*	100	50	300
21	1.5	80	30	100	0.1	0	*	100	40	150
	1.2	95	23	80	0.7	0	*	100	40	400
28	1.5	75	37	132	0	0	*	100	35	130
	1	95	21	88	1.0	0	*	100	35	500
36	1.5	70	45	170	0	0	*	100	30	110
	1	90	25	113	0.6	0	*	100	30	300

Table 3.6: response of tube-elements

N.B. 1. Softening is assumed not to occur.

2. "Outer" refers to the "net" or extra geotextile that surrounds the units (in case n > 1). The dynamic extra tensile stress always occurs in the "outer" geotextile, since this forms the first contact between slamming-loading and construction.

CYLINDERS					dynamic RESPONSE			static
n	D_o [m]	h [m]	A' [m3/m']	P' [m2/m']	X [m]	Z [m]	ΔNΘ [kN/m']	NΘ [kN/m']
1	10	4.5	35	30	0	0	100	405
	9	4.5	32	29	0	0	100	365
	8	4.5	28	27	0.4	0	100	324
	7	4.5	25	26	0.5	0	100	284
	6	4.5	21	24	0.8	0	100	243
2	6	4.5	42	47	0	0	100	243
	5	4.5	35	44	0	0	100	203
	4	4.5	28	41	0.4	0	100	162

N.B.: 1. softening is assumed not to occur.

4. REQUIREMENTS

4.1 Ways of failure

In order to determine the requirements on geotextile and filling it is necessary to analyse the mechanisms by which the construction can fail. When the failure mechanisms are known, requirements can be based on the necessary resistance of the construction under design conditions: extreme loadings with respect to strength and normal loadings with respect to fatigue etc.

In general two major ways of failure can be distinguished for a construction built out of geotextile units:

1. failure by displacement: the construction is moved;
2. failure by fracture of the membrane: material is lost.

In fig 4.1 an overview is given of the way of failure of the construction. The mechanisms can occur at the same time, failure can be caused by a superposition of mechanisms.

CONCLUSION:

In general it can be said that the wave-loadings (extreme conditions) determine the dimensions of the construction, thus the content of the construction.
The gravitational loadings determine the necessary strength of the geotextile (normal conditions).

The safety factor for the resistance of the construction (dimensions, tensile force in the geotextile) is dependent on the circumstances:

under extreme conditions: $\gamma = 1$;
under normal conditions : $\gamma = 2$.

"Normal" conditions concern gravity and fatigue.

In the case of tube units, the safety factor for the geotextile strength $\gamma = 1$, because the small inner units only have to carry the filling during the building stage. Later on the "outer" geotextile will take over a part of the gravity forces.
So the quality of the inner geotextile may be rather poor, and the safety-factor $\gamma = 1$ against the static load is sufficient.

4.2 Requirements

The results of the analysis are given below.

DIMENSIONS OF THE DIKE

- initial height $h_o \geq 4.50$ m;
- content of the dike $A' \geq 30$ m3/m';

DIMENSIONS OF SEPARATE UNITS*

- content of units $A \geq 5$ m3;

GEOTEXTILE (and sewings)

- extra dynamic stress: $\Delta N\phi$ = 100 kN/m' (fatigue);
- safety factor of $\gamma = 2$ for the static tensile forces (see table 3.5 and 3.6);
- the lifetime of the geotexile must be at least 5 years: considering: fatigue, aging processes, UV radiation and salt-intrusion;
- no loss of filling material can be allowed: the construction must be impermeable to soil, considering the size of the fraction, D_{50} = 50 µm, this requires the use of an auxiliary plastic sheet. This sheet can be situated either on the outer membrane, then the inner elements have no requirements on permeability (only during the building stage they have to be stable). The other possibility is to apply an extra plastic-sheet-layer on eah of the inner units (then the high-quality and high-strength "outer" membrane has no requirements on permeability). Which solution is chosen, depends on the costs and the way of construction (see also chapter 5);
- the subsoil will be protected by means of a fascine mattress.

The required tensile strength for each of the building-possibilities is given in table 3.7 and 3.8.

*) separate units: if not surrounded by an auxilary geotextile, this is only valid in the case of cylinders, cubes, bags etc.

The required tensile strength in case of TUBE-ELEMENTS, parameters on page 93 (horizontal axis):

DIMENSIONS				GEOTEXTILE		
n	D_o [m]	filling %	A' [m3/m']	P' [m2/m']	INNERelements Nϕ [kN/m']	OUTER* Nϕ [kN/m']
1	9	70	45	29	*	320
	8	75	37	25	*	350
3	5	80	47	48	125	320
	4	90	34	40	125	380
6	4	70	53	76	81	320
	3	85	36	57	81	360
10	3	70	49	94	62	320
15	2	80	38	93	50	350
21	1.5	80	30	100	40	400
28	1.5	75	37	132	35	360
36	1.5	70	45	170	30	320

Table 3.7: requirements on tensile strength of tube-elements

One finds that the "outer" membrane (i.e. the "net" or geotextile that holds the tube-units together, see par. 3.1.1) always has to have a tensile strength of about 350 kN/m', regardless of the content.
The resaon for this is the fact that this "net", in time, will always carry the total load. The inner elements will slowly adjust their shape and location, so that the outer membrane will carry all the (static) load after a while. The geotextile of the inner elements will not carry anything anymore, and rests upon the outer membrane (see fig. 3.10).

However, this tensile strength of 350 kN/m' is still more advantageous than in the case of cylinder-elements, where even in the best case the required geotextile strength is 500 kN/m'. Moreover, in the case of big "tubes" only one sewing, on top of the dike, will be necessary. Whereas in the case of cylinders, all of the units will contain several sewings, thus reducing the strength of the membrane considerably.

The possibilities and requirements on a dike built out of CYLINDER ELEMENTS (or cubes, bags), parameters on page 93 (vertical axis):

DIMENSIONS					GEOTEXTILE
n	D [m]	h [m]	A' [m3/m']	P' [m2/m']	NΘ [kN/m']
1	11	4.5	38	32	1,000
	10	4.5	35	30	900
2	6	4.5	42	47	600
	5	4.5	35	44	500

Table 3.8: requirements on tensile strength of the geotextile of cylinder-elements

N.B.: the requirement "impermeability for soil particles" is caused by the fact that loss of filling material influences directly the height of the dike (see par. 3.1).
Any reduction of dike height causes an increased transmission of waves to the reclamation basins.
Furthermore loss of filling material causes a reduced stiffness of the construction and increases the chance of softening.

Thus, taking into account the required lifetime of 10 years of the construction, loss of filling material by permeability of the geotextile is absolutely not allowable. (Damages will cause some loss of material, but they can be repared.)

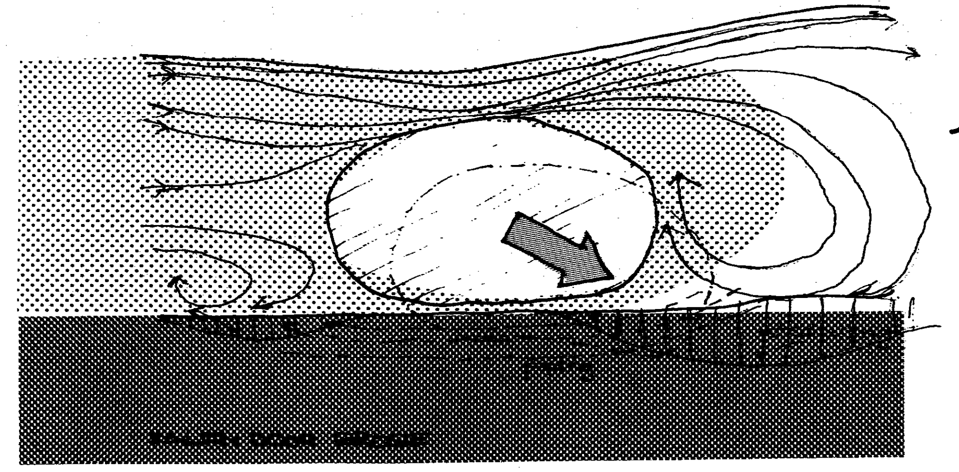
The only way the particle impermeability of the cloth is guaranteed, using the silt mixture of the Cao Jing district, is to add an inlay of p.v.c.-plastic sheet.

ultimate limit state:

cause:

requirement:

EROSIE
EROSION



PLAATSELIJKE STERKE EROSIE
strong local erosion.......
EXTREME CONDITIONS

- GOLVEN / waves → subsoil protected by fascine matress
- STROMING / currents → subsoil protected by fascine matress
- WIND / wind → subsoil protected by fascine matress
- VERVAL / diff. in waterlevel → additional testing necessary

LANGEDUUR EROSIE
long-term erosion.......
NORMAL CONDITIONS

- GOLVEN / waves → subsoil protected by fascine matress
- STROMING / currents → subsoil protected by fascine matress
- WIND / wind → subsoil protected by fascine matress
- VERVAL / diff. in waterlevel → additional testing necessary
- ZON / sun

ONDERLOOPSHEID
seepage...........
NORMAL CONDITIONS

- STROMING / currents → additional testing necessary
- VERVAL / diff. in waterlevel → additional testing necessary
- HOGE WATERSTAND / high waterlevel → additional testing necessary

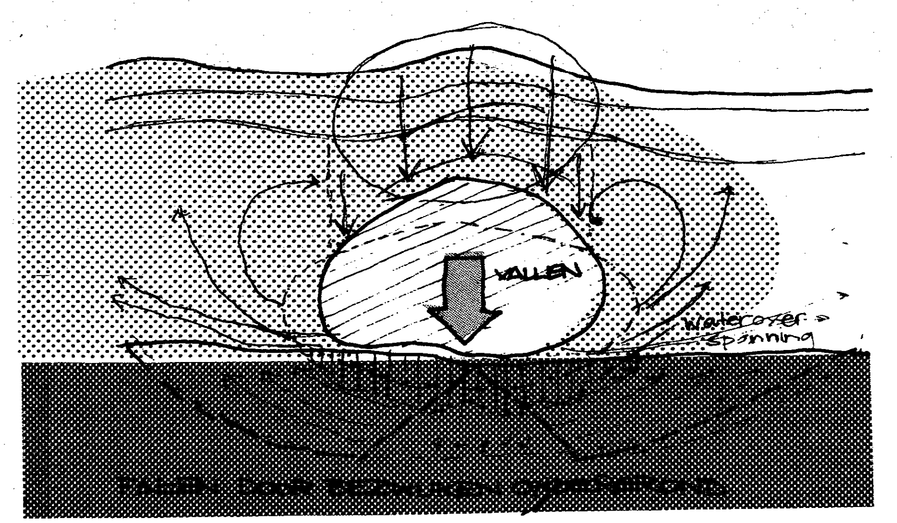
4.1a
Ways of failure construction

ultimate limit state:

cause:

requirement:

BEZWIJKEN ONDERGROND
FAILURE OF THE SUBSOIL



TE GROTE KRACHT
loadings..............
EXTREME CONDITIONS

- GOLVEN / waves → theoretical approach necessary
- STROMING / currents → theoretical approach necessary
- VERVAL / difference in waterlevel → theoretical approach necessary
- PLAATSING / stationing → theoretical approach necessary

VERWEKING
softening.......
NORMAL CONDITIONS

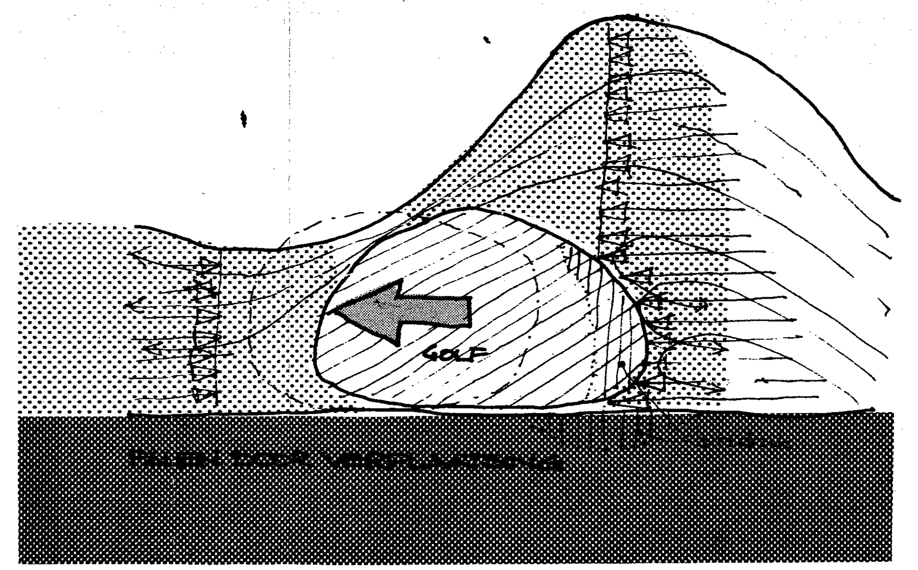
- GOLVEN / waves → additional testing necessary
- STROMING / currents → additional testing necessary
- HOGE WATERSTAND / high waterlevel → additional testing necessary

ultimate limit state:

cause:

requirement:

VERPLAATSING TUBE
DISPLACEMENT OF THE CONSTRUCTION



TE GROTE KRACHT
loadings..............
EXTREME CONDITIONS

- GOLVEN / waves → content of dike: > 30 m³/m' (sliding) content of unit: > 5 m³/m' (hudson)
- STROMING / currents → additional testing necessary (probably minor influence)
- WIND / wind → not of importance
- VERVAL / diff. in waterlevel → content of dike: > 18 m³/m' (sliding)
- PLAATSING / stationing → not of importance

VERWEKING
softening........
NORMAL CONDITIONS

- GOLVEN / waves → additional testing necessary: dike should be composed of more than one unit.
- STROMING / currents → additional testing necessary
- HOGE WATERSTAND / high waterlevel → additional testing necessary

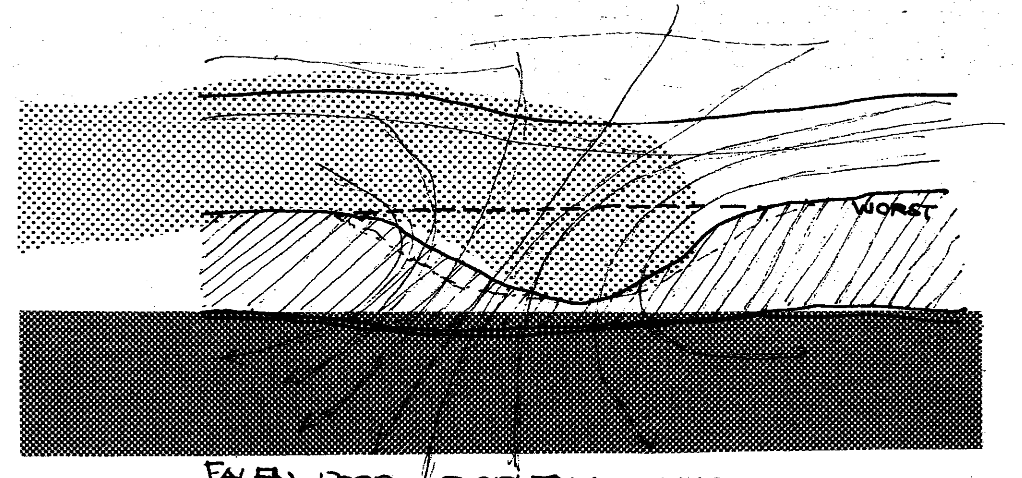
VERVORMINGEN
deformations.......
EXTREME CONDITIONS

- GOLVEN / waves → dike must respond as stiff mass (spring-mass system)
- STROMING / currents → not of importance
- WIND / wind → not of importance

ultimate limit state: | cause: | requirement:

ONTBREKEN VULLING

ABSENCE OR LOSS OF FILLING-MATERIAL



FALEN DOOR ONTBREKEN VULLING

DOORLATEN WEEFSEL
leakage through the textile..
NORMAL CONDITIONS

- GOLVEN / waves → loss is not allowed: O90 = 10 μm ⇒ use of a plastic sheet as inlay
- STROMING / currents → loss is not allowed: O90 = 10 μm ⇒ use of a plastic sheet as inlay
- ZON / sun → UV-stability of the plastic sheet should be sufficient to guarantee the water-tightness for 5 years.

INTERNE VERPLAATSING
internal movement ......
EXTREME CONDITIONS

- GOLVEN / waves → filling degree should be over 70%
- STROMING → softening is not allowed
- currents → probably of minor importance
- WIND / wind → probably of minor importance
- PLAATSING / stationing → filling should take place with care

ONTBREKEN VULLING
local absence of material ...
SPECIAL CONDITIONS

→ repair must be possible,
→ voids must be prevented
→ the construction must consist of units which can be re-filled

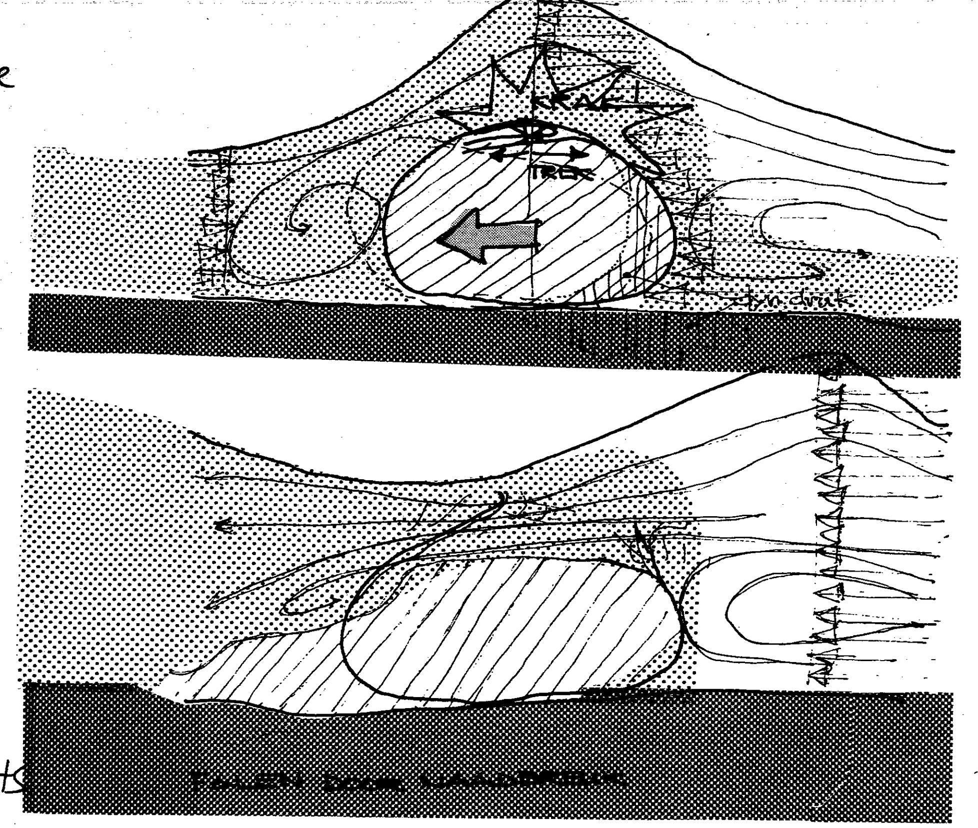
4.1b

Ways of failure geotextile

ultimate limit state: | cause: | requirement:

NAADBREUK

CRACK IN THE SEAM



TE GROTE KRACHT
loadings..........
EXTREME CONDITIONS

- GOLVEN / waves → dynamic strength: $\Delta N\phi = 100$ kN/m' ($\gamma = 1$)
- STROMING / currents → additional testing necessary (probably minor influence)
- WIND / wind → additional testing necessary (probably minor influence)
- PLAATSING / stationing → gravitational loading; static strength dependent on construction ($N\phi \approx 125$ kN/m') $\gamma = 2$!

VERMOEIING
fatigue........
NORMAL CONDITIONS

- GOLVEN / waves → additional testing necessary strength after 5 years: 350 kN/m'
- STROMING / currents → additional testing necessary strength after 5 years: 350 kN/m'
- WIND / wind → additional testing necessary strength after 5 years: 350 kN/m'
- ZON / sun → UV-stability: strength after 5 years: 350 kN/m'

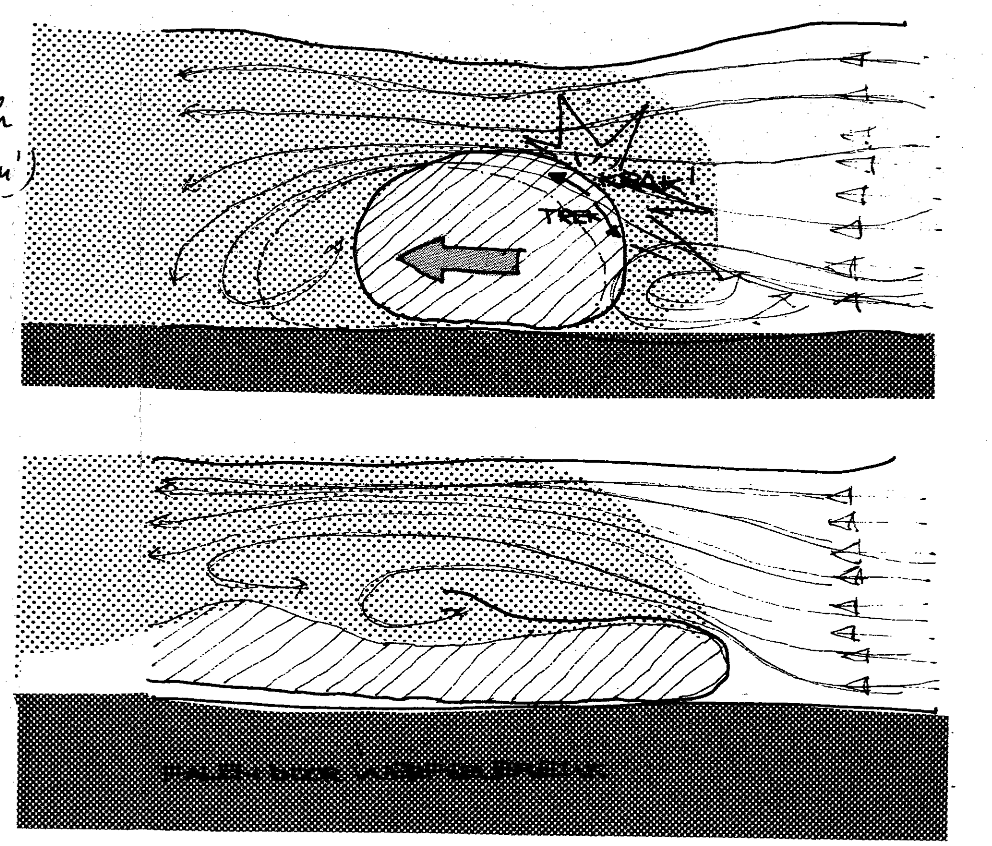
SCHERP VOORWERP
sharp object..........
SPECIAL CONDITIONS

→ influence not taken into account (repair should be possible)

ultimate limit state: | cause: | requirement:

WEEFSELBREUK

CRACK IN THE TEXTILE



TE GROTE KRACHT
loadings..............
EXREME CONDITIONS

- GOLVEN / waves → dynamic strength $\Delta N\phi = 100$ kN/m' ($\gamma = 1$)
- STROMING / currents → additional testing necessary (probably minor influence)
- WIND / wind → additional testing necessary (probably minor influence)
- PLAATSING / stationing → gravitational loading, static strength ($\gamma = 2$) dependent on construction (loading: $N\phi = 125$ kN/m')

VERMOEIING
fatigue.......
NORMAL CONDITIONS

- GOLVEN / waves → additional testing necessary; ($\Delta N\phi \approx 50$ kN/wave) strength after 5 years must be 350 kN/m'
- STROMING / currents → additional testing necessary strength after 5 years must be 350 kN/m'
- WIND / wind → additional testing necessary strength after 5 years must be 350 kN/m'
- ZON / sun → UV stability of five years: strength after 5 years must be 350 kN/m'

SCHERP VOORWERP
sharp object........
SPECIAL CONDITIONS

→ influence not taken into account (repair should be possible)

5. POSSIBLE CONSTRUCTIONS

5.1 Designs

Based on the requirements and findings described in the foregoing, some possible constructions have been developped.

It is discussed:

- Cubatao method
- Jack-screw method
- pumping battery method
- mould method
- self-unloading mould
- injection method
- packed soil
- reinforced soil
- compartment dam
- cubes
- small tubes
- soil bags.

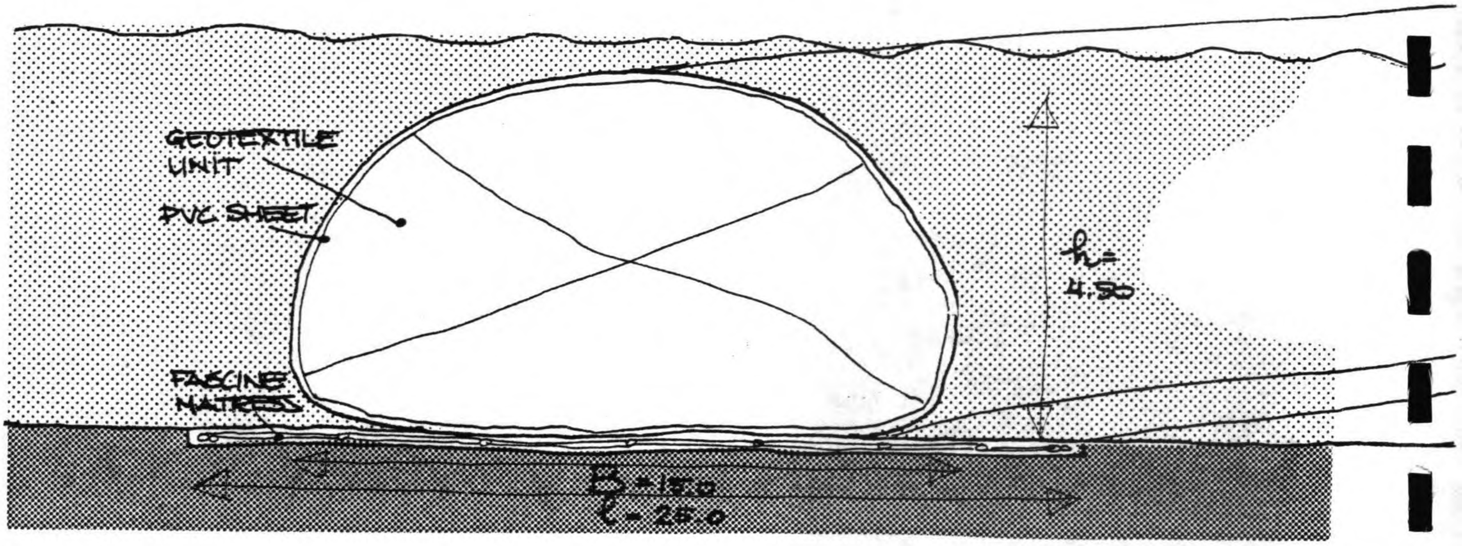
Starting points for each of the designs have been:

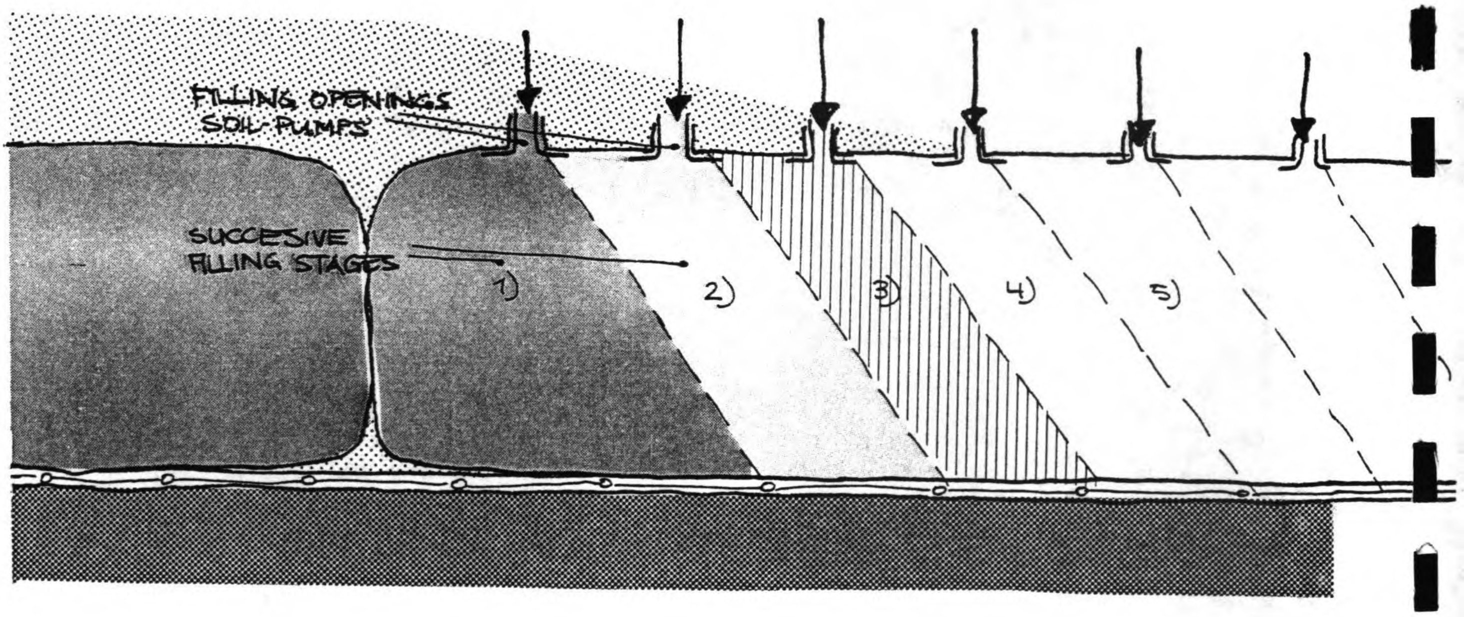
I. the most economic construction contains as little as possible geotextile cloth, since the number of square meters cloth will considerably influence the final costs ;

II. the progression of the project (the first 4 km of dike must be constructed during the first year) requires a flexible, straight forward way of building;

III. the size of the construction requires a dike of which the units will be constructed <u>in situ</u>, along the tracé of the dike.
(Smaller elements like tubes or bags can be pre-fabricated at the land strip.);

IV. the necessary capacity and available working-time are based on the most critical piece of the dike: the trace that has to be constructed at the 1+ level (see also <u>fig. 1.3</u>). Since parts of the planned tracé are situated at a higher bottom-level, the actual capacity and available working time are more advantegeous (see also par. 5.2).





5.1

Cubatão-method

5.1.1 Cubatao-method

Description

A prefabricated tube, with a diameter of 8 à 9 m, is transported, empty, towards the 1+ m level, where it is placed upon a fascine mattress. Soil-pumps fill the tubes through successive openings (see fig. 5.1) After the required height has been reached, the filling-process stops and the soil-pumps are moved to the next section.

Construction can only take place in-the-dry, thus during low-water. When the tide rises, all equipment has to be transported towards the land and construction is continued the next day.

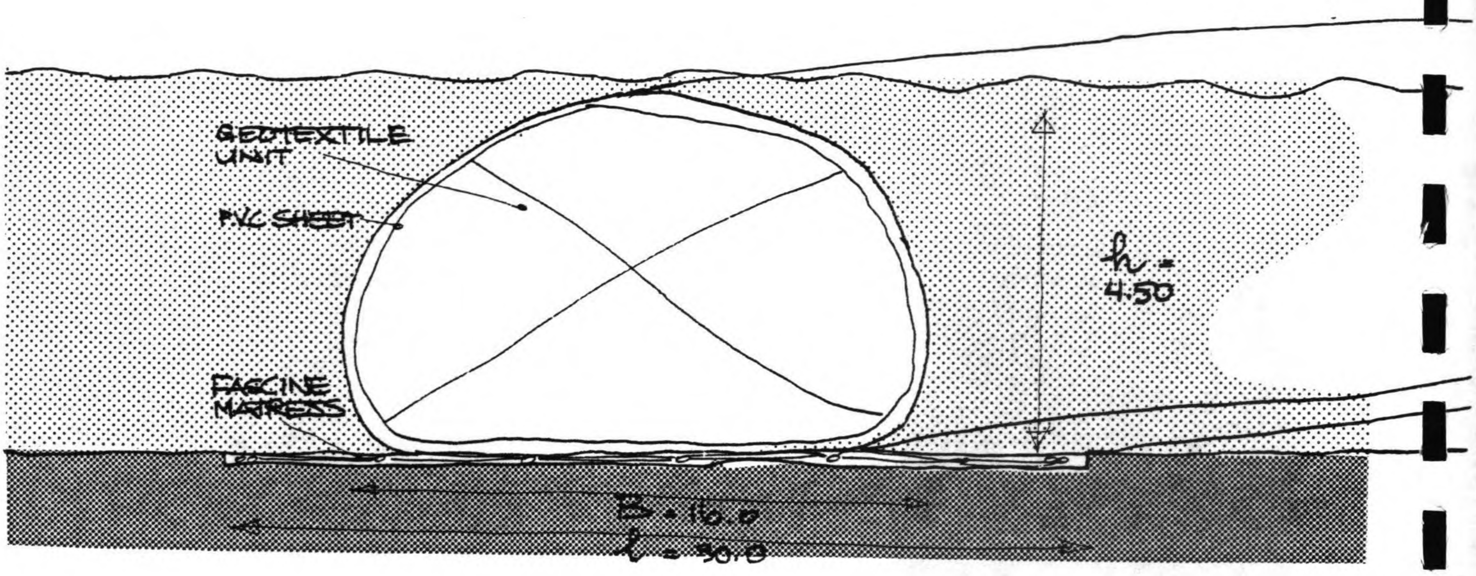
Materials (per running meter dike)

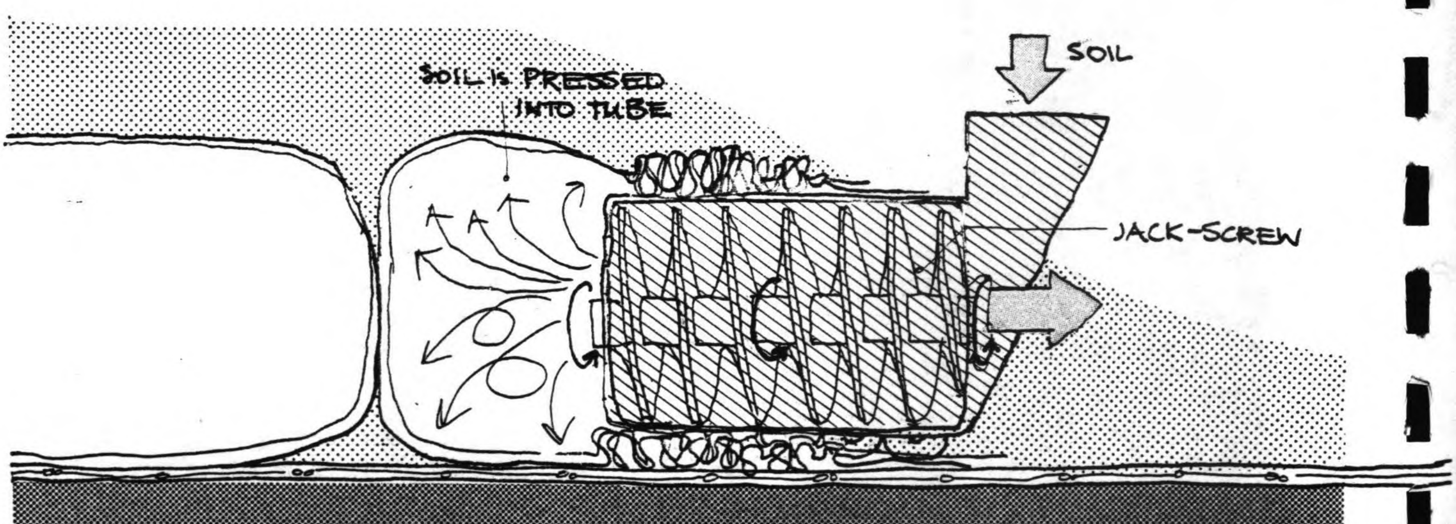
SOIL	: - the silt in situ	40	[m3/m']
GEOTEXTILE:	- cloth with a tensile strength of 350 kN/m':	30	[m2/m']
	- plastic sheet p.v.c. (inlay)	30	[m2/m']
EQUIPMENT	: - soil pumps, total capacity:	600	[m3/hour]
	- cranes to provide the reservoirs of the pumps, total capacity:	600	[m3/hour].

Advantages and disadvantages

Advantages: - simple construction-plan and low costs (very little cloth).

Disadvantages:
- the available working time per day is very short (2 hours during low tide, at the 1+ level);
- problems will arise in order to fill the tubes up to the required height. In the first filling stages, the soil-mixture will spread along the bottom, forming a flat cake. During later stages, this cake has to be re-formed to a drop-shape, thus the passive soil-pressure has to be exceeded by the pumping pressure, which will probably result in collapse of the geotextile. A measure to prevent these problems would be the use of local reinforçements, like baleens, which form a kind of frame, that carries the load;
- safety: a tear in the geotextile would result in loss of the total tube.





5.2

Jack-screw method

5.1.2 Jack-screw method

Description

A prefabricated geotextile, with a dimameter of 8 à 9 m, is transported, empty, towards the construction site (1+ level), where it is placed upon a fascine matress. The tubes are filled by means of a movable jack-screw, that presses the soil (supplied by cranes) into the tube, and moves itself (see fig. 5.2), during filling.

The construction takes place in situ, during low tide. During the rising tide the equipment has to be transported to the land. Continuation is delayed until the next day.

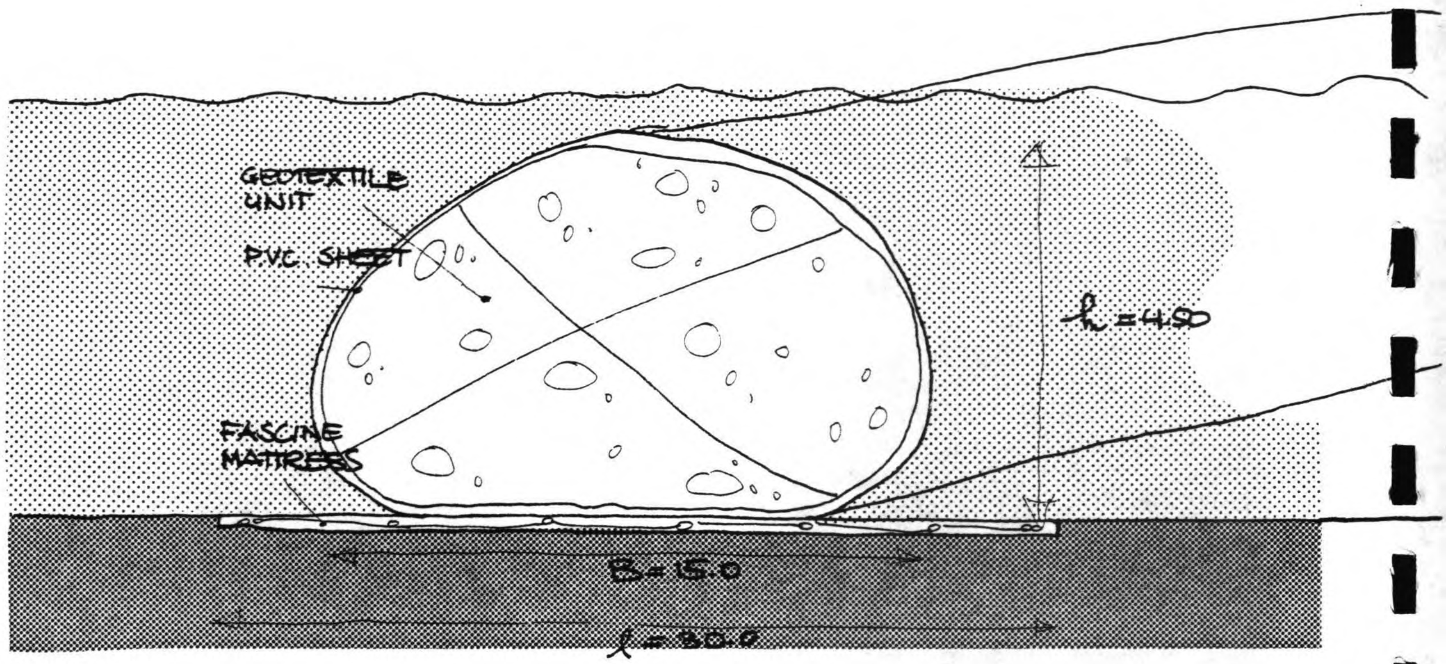
Materials (per running meter)

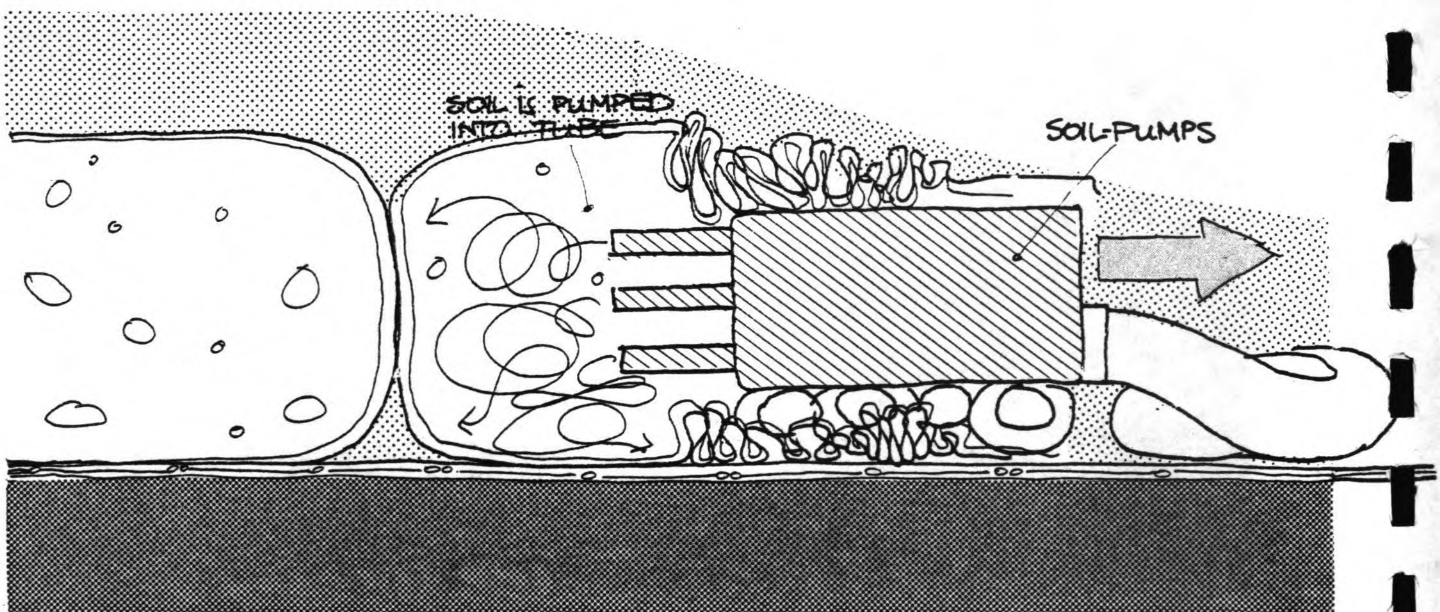
SOIL	: - the silt in situ	40	[m3/m']
GEOTEXTILE:	- cloth having a tensile strength of of 350 kN/m':	30	[m2/m']
	- plastic sheet as an inlay	30	[m2/m']
EQUIPMENT :	- the jack-screw (special design) capacity	600	[m3/hour]
	- cranes to provide the installation of soil with a total capacity of	600	[m3/hour].

Advantages and disadvantages

Advantages: - simple construction-plan and low costs (very little cloth).

Disadvantages:
- the avialable working time per day is very short (2 hours per day);
- since the jack-screw has a cylinder-shape, and the soil-filled tube a drop shape, difficulties will arise at the transition between jack-screw and tube. The geotextile will almost certainly tear at this place (enormous tensile stresses). Furthermore it will be very difficult to design a movable jack-screw of these dimensions;
- the jack-screw must be especially designed and is not functional for lower parts of the dike or other projects;
- safety: failure of the geotextile means large damage;
- the procedure is very vulnerable: damage of the jack-screw means discontinuance of the project.





5.3

Pumping battery method

5.1.3 Pumping battery method

Description

A prefabricated tube, with a diameter of 8 à 9 m, is transported, empty, towards the construction site (1+ level), where it is placed upon a fascine matress. The tubes are filled by means of a movable torpedo, which includes a battery of soil pumps discharging a soil-water mixture into the tube while moving slowly towards the end of the tube (see fig. 5.3).

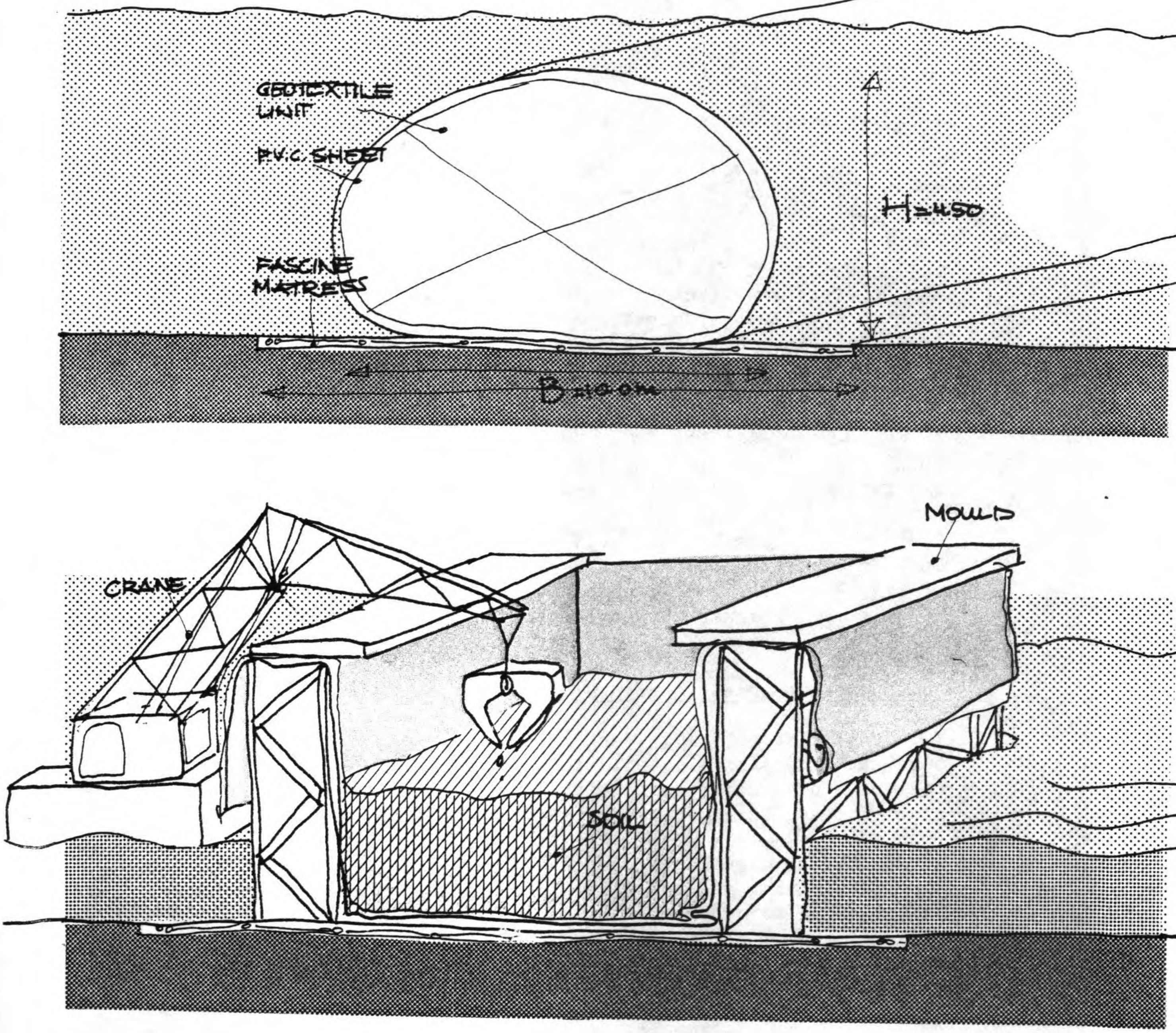
The construction takes place in situ, during low tide. Since construction is only possible in-the-dry, the equipments have to be transported to the land during up-coming water. The next day, the construction is continued.

Materials (per running meter)

SOIL :	- the silt in situ	40 [m3/m']
GEOTEXTILE:	- cloth having a tensile strength of of 350 kN/m':	30 [m2/m']
	- plastic (p.v.c.) sheet as an inlay	30 [m2/m']
EQUIPMENT :	- the pumping battery (special design) with a capacity of	600 [m3/hour]
	- cranes to provide the pumps with sufficient filling, capacity:	600 [m3/hour].

Advantages and disadvantages

Mainly simular to 5.1.2 (jack-screw), the pumping battery forms a somewhat more flexible unit (the linkage might be easier to realize), but the details of the movable torpedo are complicated also. The design can be adjusted to the drop shape of the final tube.
An additional disadvantage is the voids that are pumped into the tube, containing air and water. These voids increase the chance of softening to occur.



5.4

Mould method

5.1.4 Mould method

Description

A mould is set up at the construction site (1+ level), upon a fascine matress. It consists of two separate walls that can be connected. The dimensions of the mould are: height 5 m, width 8 m, length: 10-30 m (dependent on local circumstances). In the mould a geotextile is placed, covered by a plastic sheet. Cranes fill up the mould. After filling the geotextile is sewn together and the mould is removed.

Construction takes place mainly during low tide (mould: low tide; filling might continue during higher tide). If construction continues during rising tide, the cranes must be equiped with a pontoon.

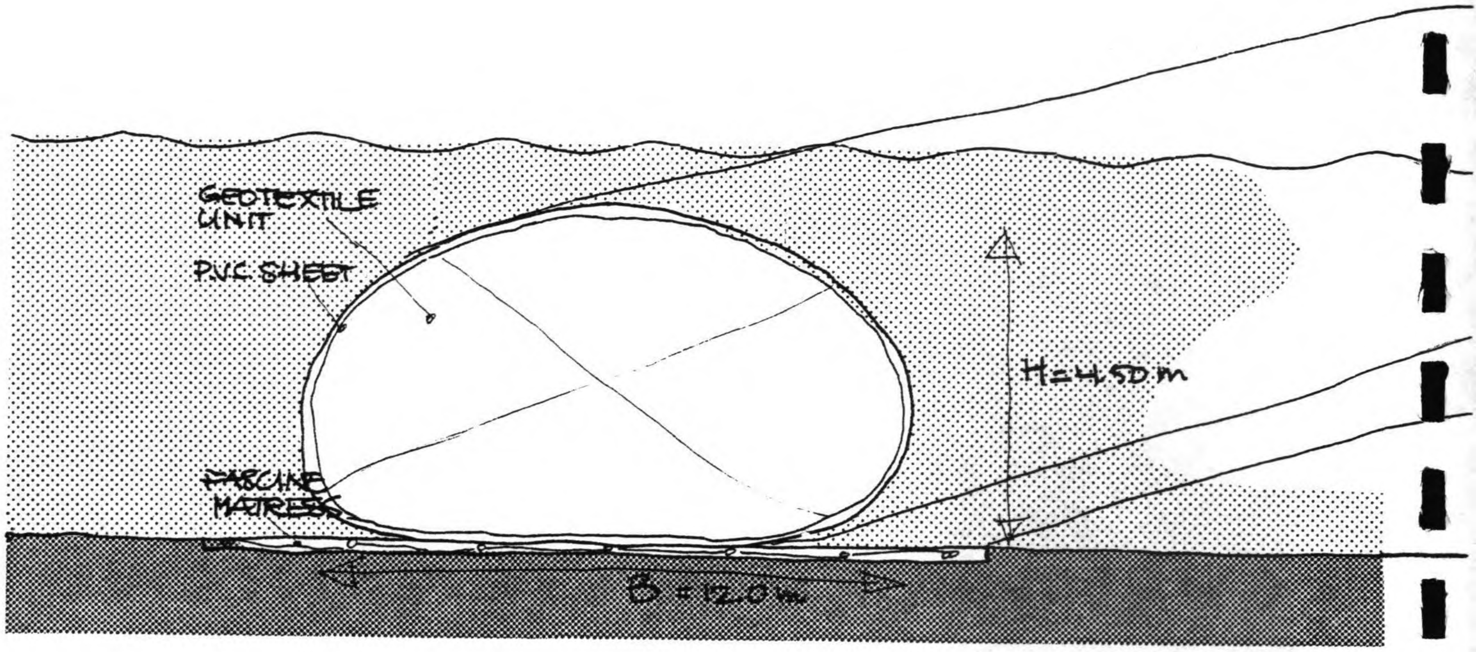
Materials (per running meter)

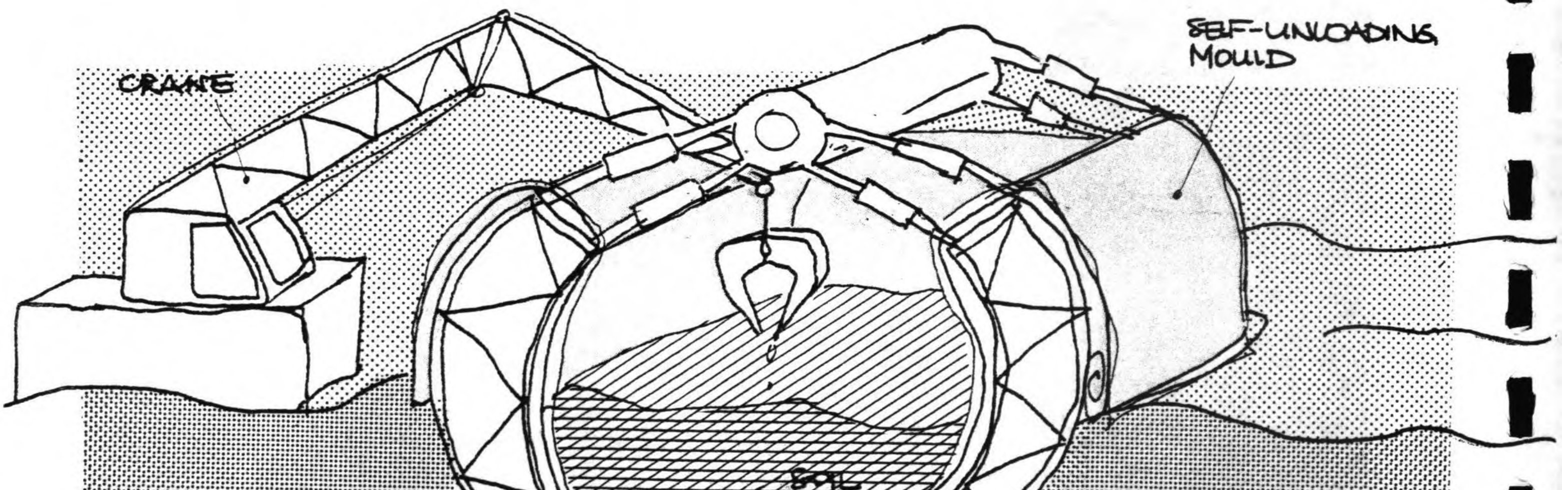
SOIL :	- the silt in situ (as dry as possible):	40 [m3/m']
GEOTEXTILE:	- cloth having a tensile strength of of 320 kN/m':	34 [m3/m']
	- plastic (p.v.c.) sheet as an inlay:	34 [m3/m']
EQUIPMENT :	- cranes to supply the moulds with sufficient filling, capacity:	600 [m3/hour]
	- the moulds (easy to assemble and dismantle, (re)movable by the cranes):	3 [pieces].

Advantages and disadvantages

Advantages:
- easy processing and little equipment (also little square meters of cloth);
- the moulds can be adjusted to the local circumstances: height, width is variable;
- the equipment is well-known, current, and can also be used for other projects.

Disadvantages:
- the available working time is rather short;
- the details of a movable mould (massive construction);
- closure of the tube must be done in situ where the conditions for an high-quality sewing are not favourable;
- safety: a tear in the geotextile causes loss of the total dike-segment.





5.5

Self-unloading mould

5.1.5 Self-unloading mould

Description

A mould is transported to the construction site (the dam-tracé), upon a fascine matress. The geotextile plus p.v.c.-inlay is placed inside, and the mould is filled by cranes. After the geotextile has been sewn together, the mould unloads itself. The diameter of the tube inside is in the order of 9 m, the length is in the order of 20 m (see fig. 5.4).

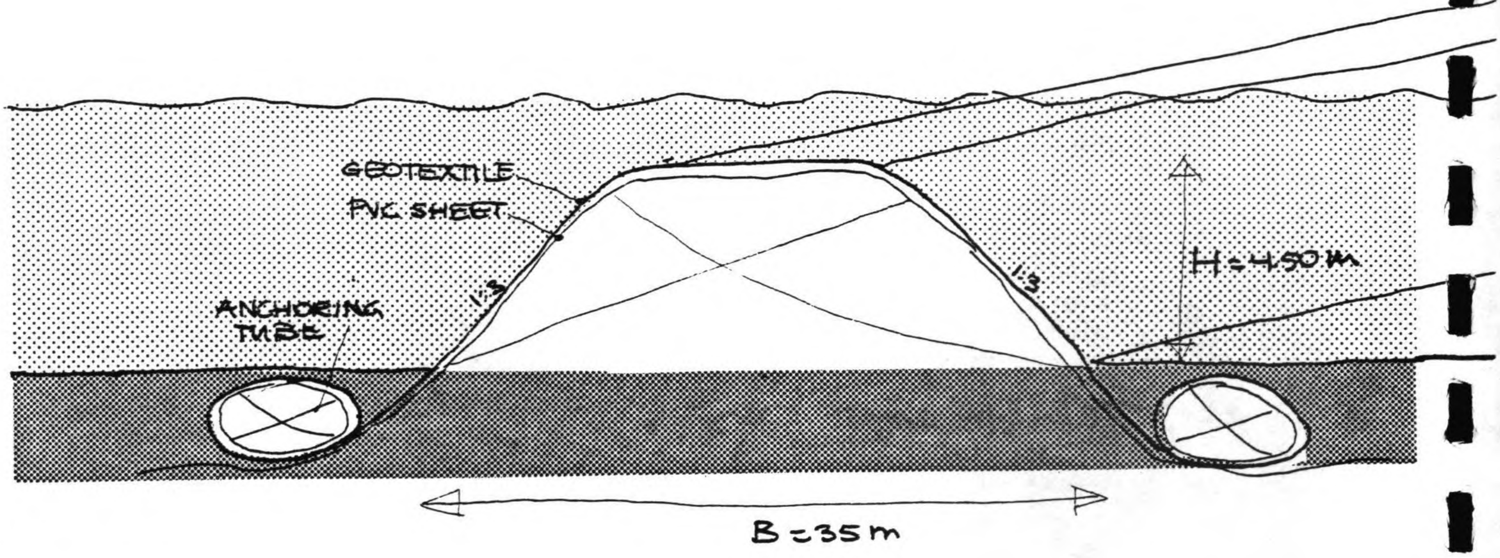
Transport of the mould, and unloading, take place, during low tide: the dry working time. Filling can continue during rising tide.

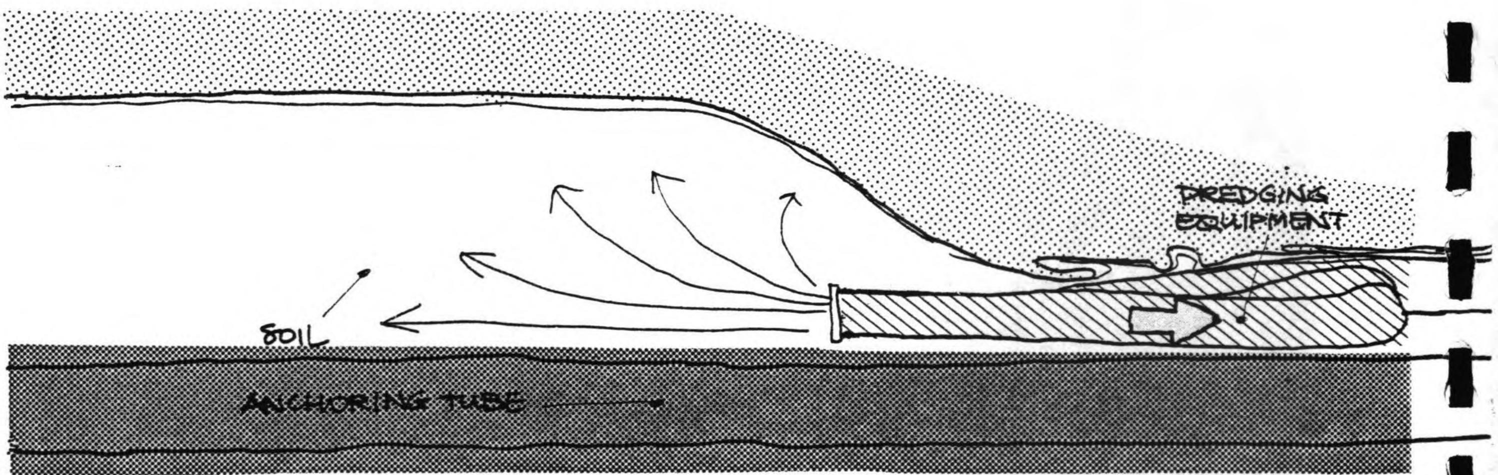
Materials (per running meter)

SOIL :	- the silt in situ (as dry as possible):	40 [m3/m']
GEOTEXTILE:	- cloth having a tensile strength of of 350 kN/m':	34 [m2/m']
	- plastic p.v.c.-sheet as inlay:	34 [m2/m']
EQUIPMENT :	- cranes to fill the mould with a total capacity of:	600 [m3/hour]
	- moulds, removable and self-unloading:	2-3.

Advantages and disadvantages

Mainly the same as the mould method 5.1.4. An additional advantage is the self-unloading capacity of the mould, an additional disadvantage is the costs of this self-unloading device due of the complicated details of such an instrument. Also the fact that this mould is standard: for sections where a lower dike height is sufficient, a special mould has to be developped.





5.6

Injection dam

5.1.6 Injection-method

Description

A geotextile cloth, anchored by two side tubes, is placed at he construction site (along the dam-tracé). Using dredging equipment or high capacity soil pumps. The cloth is blown up with a mixture of water and soil until the required height has been reached. The width of the cloth is in the order of 50 m, the length is dependent on the capacity of the filling equipment (50-100 m).

Construction can only take place during the "dry working" period, during low tide.

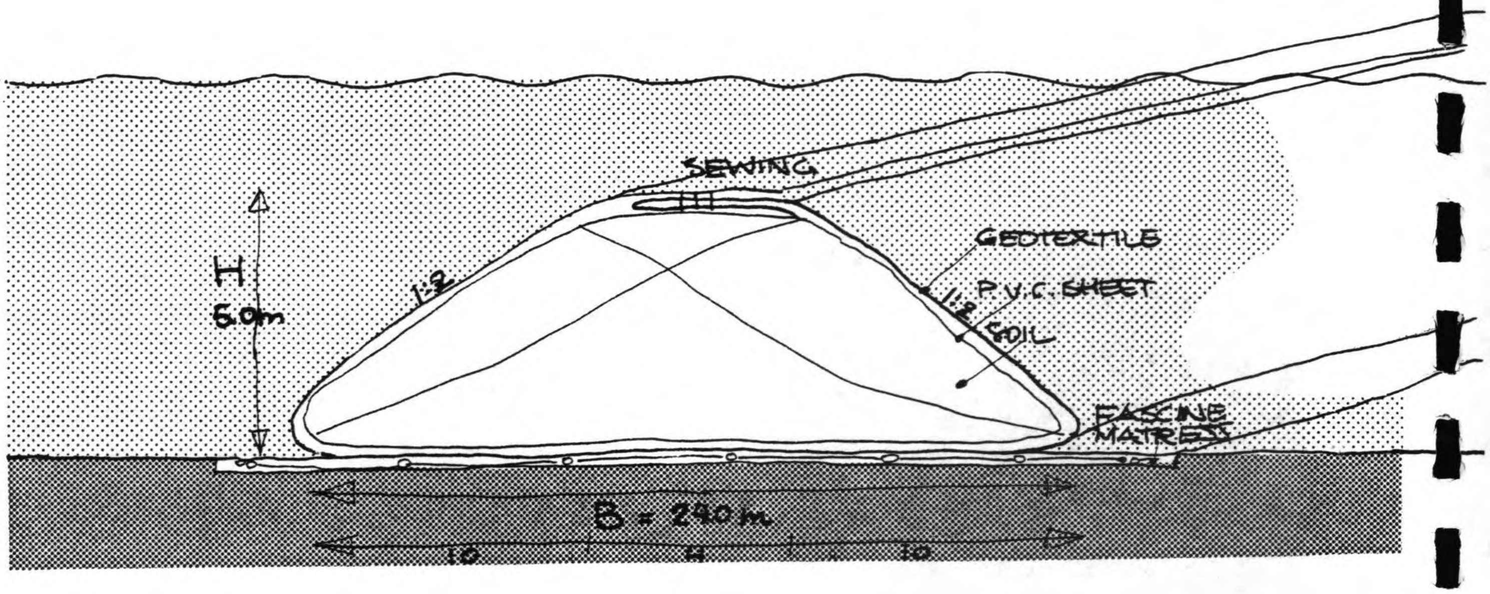
Materials (per running meter)

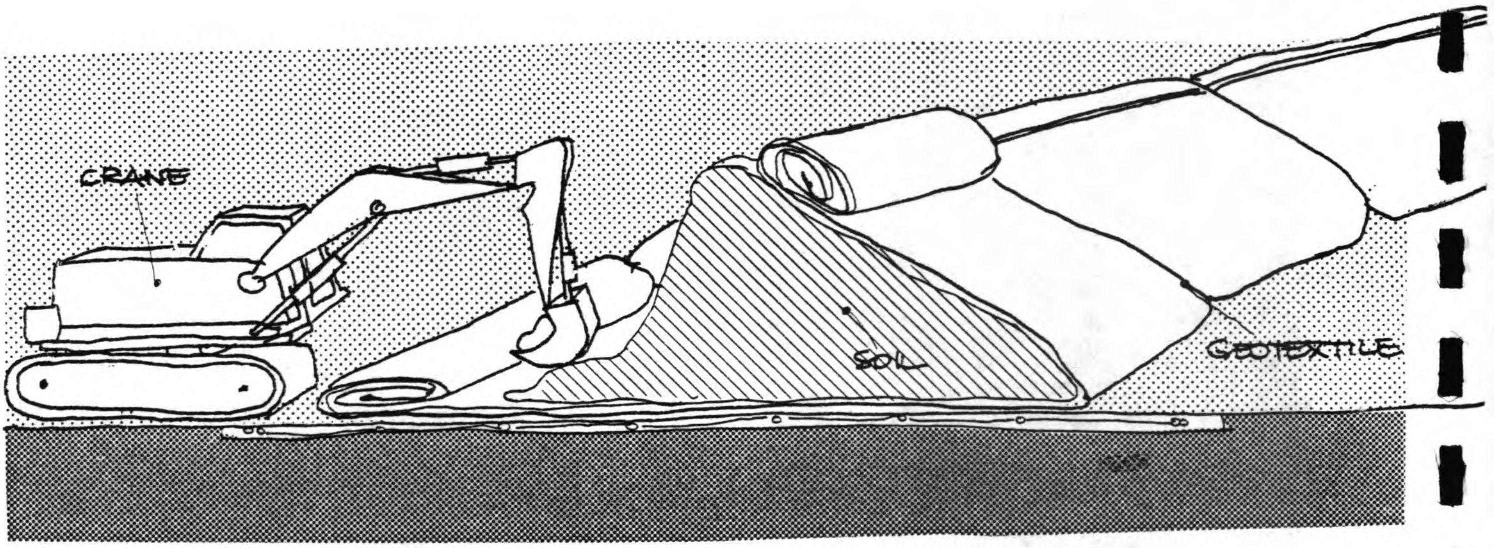
SOIL :	- the silt in situ	$\geq$	75 [m3/m']
GEOTEXTILE:	- cloth having a tensile strength of 300 kN/m'	$\geq$	60 [m2/m']
	- p.v.c.-inlay	$\geq$	60 [m2/m']
	- additional anchoring tubes (plus p.v.c.-inlay), cloth:		20 [m2/m']
EQUIPMENT :	- dredging equipment or high-capacity-soil-pumps, total capacity:	$\geq$	2.000 [m3/hour].

Advantages and disadvantages

Advantages:
- very easy filling processing;
- moderate number of square meters of cloth;
- a fascine matress is not necessary.

Disadvantages:
- the filling process increases the chance of softening, due to the high water-content of the filling;
- the construction has no resistance against softening (internal sliding causes direct failure of the dike);
- the soil in situ is actually not suitable for hydropneumatic filling, it should be improved;
- working time is short: only during low water;
- low safety: a tear in the geotextile causes loss of filling, and finally failure of the dike.





5.7

Packed soil

5.1.7 Packed soil

Description

A cloth of geotextile, with a width of 50 m, is placed upon a fascine matress. Bulldozers and cranes create a hill of soil upon the cloth, until a level of 5.0 m, with a slope of 1:2 or steeper. Then the cloth is wrapped around the hill, the geotextile is tensed and sewn together, the length of the "package" is dependent on the "filling" capacity, 50-100 m.

Construction takes place only during the dry working period: low tide. When it rains it is impossible to work, each contact with water must be prevented.

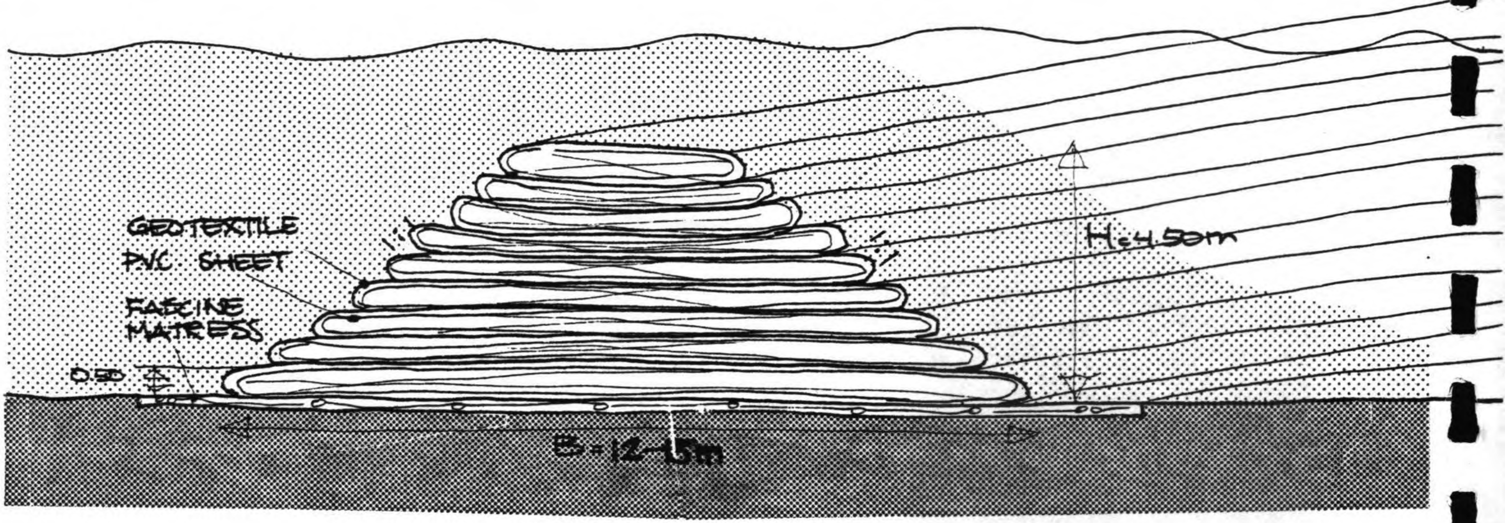
Materials (per running meter)

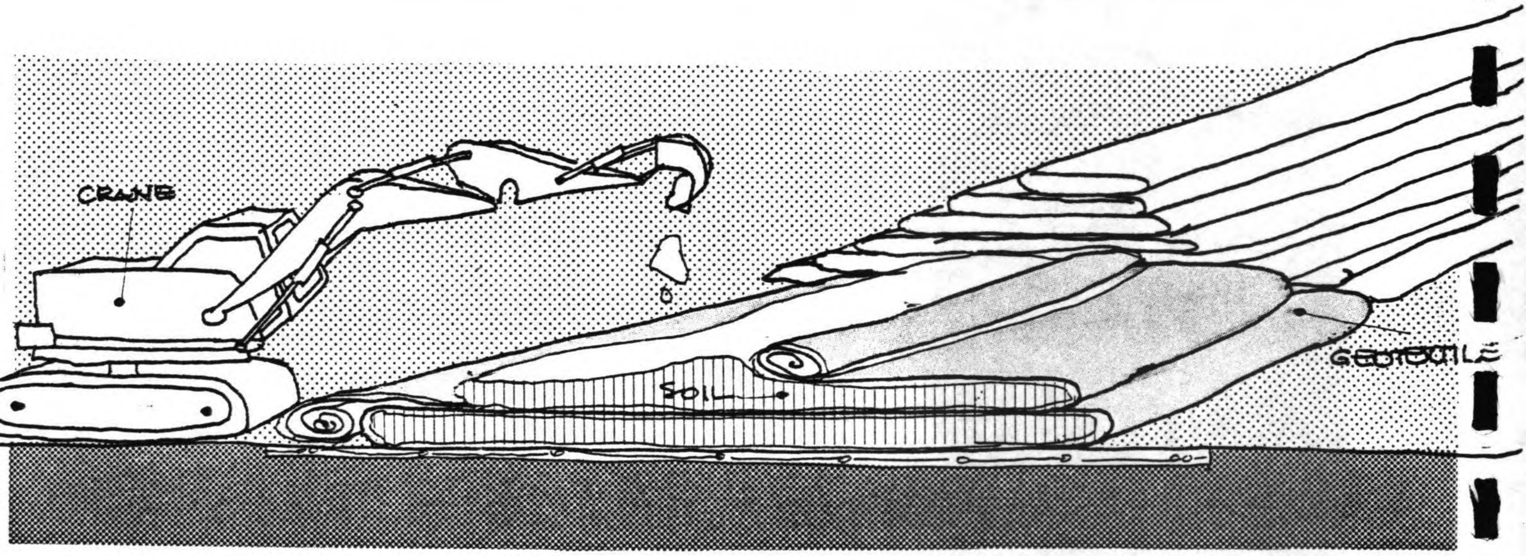
SOIL :	- the silt in situ (as dry as possible):	$\geq$	70 [m3/m']
GEOTEXTILE:	- cloth having a tensile strength of 300 kN/m':	$\geq$	50 [m2/m']
	- plastic p.v.c.-sheet (inlay)	$\geq$	50 [m2/m']
EQUIPMENT :	- bulldozers and cranes for the filling, total capacity	$\geq$	2.000 [m3/hour]
	- instruments to tense the geotextile around the soil hill and to realise a high tension sewing in situ:	5-6	

Advantages and disadvantages

Advantages:
- very easy filling process;
- moderate number of square meters of cloth;
- the equipment is current and well-known, it can also be used for other projects.

Disadvantages:
- large filling-capacity needed;
- soil must be used dry (otherwise a hill can't be formed);
- short working-time: maximum 2 hours per day;
- the details of instruments to stretch the geotextile around the hill and realize the necessary sewing, are complicated;
- moderate safety: a tear in the geotextile will cause failure of the dike-segment.





5.8

Reinforced soil

5.1.8 Reinforced soil

Description

The dike is constructed in layers of 0.50 m height each. A fascine mattress protects the subsoil from erosion. The layers are constructed by cranes: soil is spread over a geotextile cloth, starting with a width of 12-15 m. The cloth is anchored by the second layer, that rests upon the first. This process is continued until the required height of 4.50 m is reached (9 layers).
The anchoring length of the geotextile is dependent both on the density of the filling, and the shear-stress of the soil (ϕ).
The tensile stress in the geotextile depends on the height of the layers:

if $h_{layer} = 0.50$ then $N\phi \cong 50$ kN/m'.
The necessary anchoring length is in the order of 20 m, so each layer has to be sewn together, like a flat tube (see fig. 5.8).

Construction of each layer can only take place in-the-dry, each contact with water must be prevented.

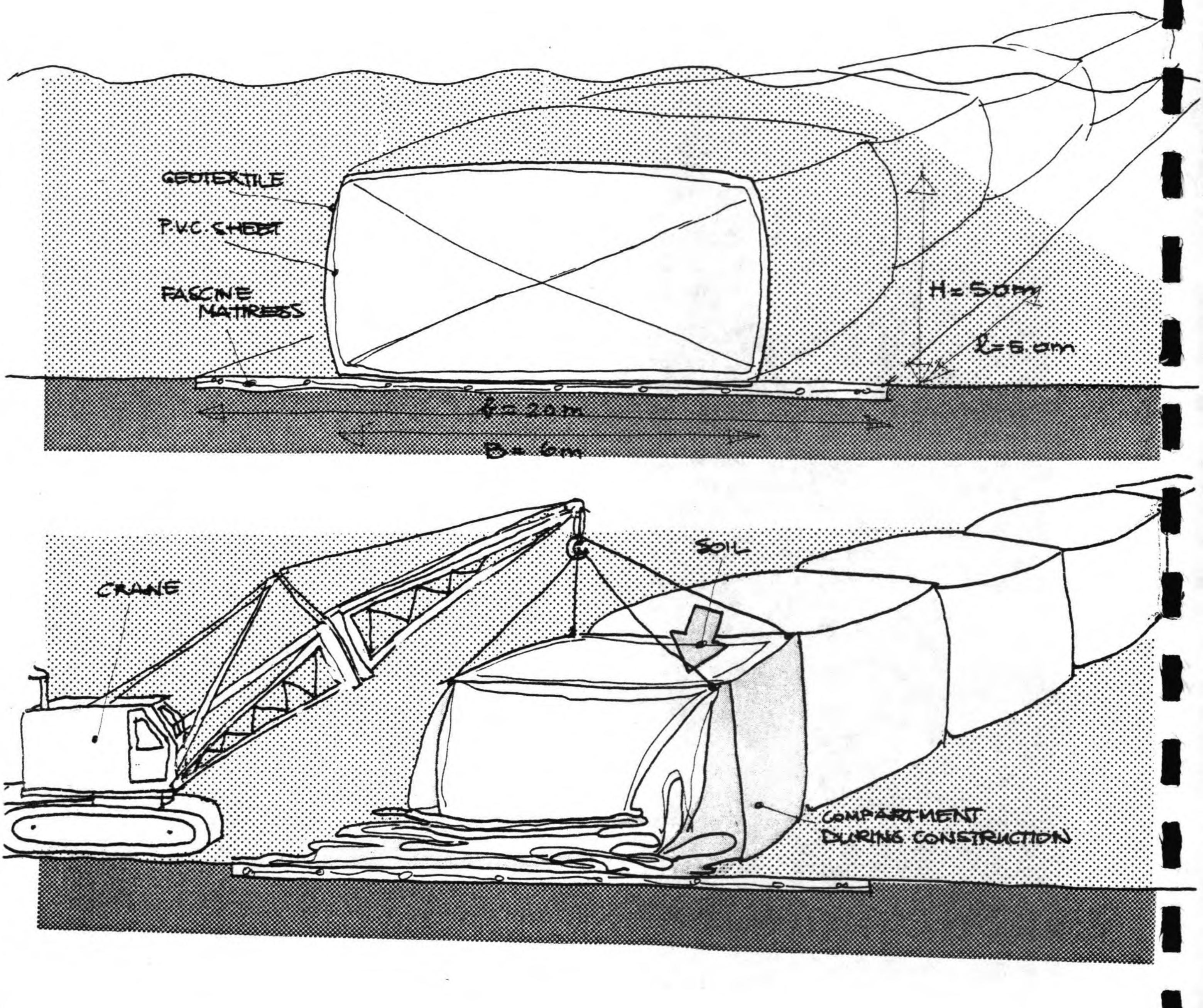
Materials (per running meter)

SOIL :	- the silt in situ (as dry as possible):	≥ 30	|m3/m'|
GEOTEXTILE:	- geotextile having a tensile strength of 200 kN/m':	≥ 150	|m2/m'|
	- p.v.c.-sheet (inlay):	≥ 150	|m2/m'|
EQUIPMENT :	- cranes to fill up the layers with dry soil, capacity:	≥ 500	|m3/hour|.

Advantages and disadvantages

Advantages:
- tensile stress in the geotextile is reduced;
- working method in layers: the soil in each layer can compact before the next layer is built, the building process can be performed in stages;
- safe structure: failure of one of the layers will not cause failure of the dike
- current and well-known equipment: it can also be used for other projects.

Disadvantages:
- the layers can, with upcoming water level, slide along each other easely, caused by the "smooth" geotextile;
- a large quantity of geotextile is needed;
- a high-quality sewing has to be realized on top of each layer. This has to be done in situ.



5.9

Compartment dam

5.1.9 Compartment dam

Description

A prefabricated "string" of compartments is transported to the construction site. After placing upon a fascine matress, a crane holds up a compartment while other cranes do the filling. Once filled, the compartment is closed by sewing a cover, and the crane can be disconnected. This cycle is repeated for every successive compartment. If necessary, several compartments can be filled at the same time.
The dimensions of each compartment: width = 6 m, length = 5 m, height = 5 m.

Construction can take place during the low tide. In this case 5 or 6 compartments must be filled at the same time. When operating during the entire tidal period, each compartment can be filled seperately. In this case the cranes must be equiped with a pontoon, and the soil must be transported towards the construction site by vessels.

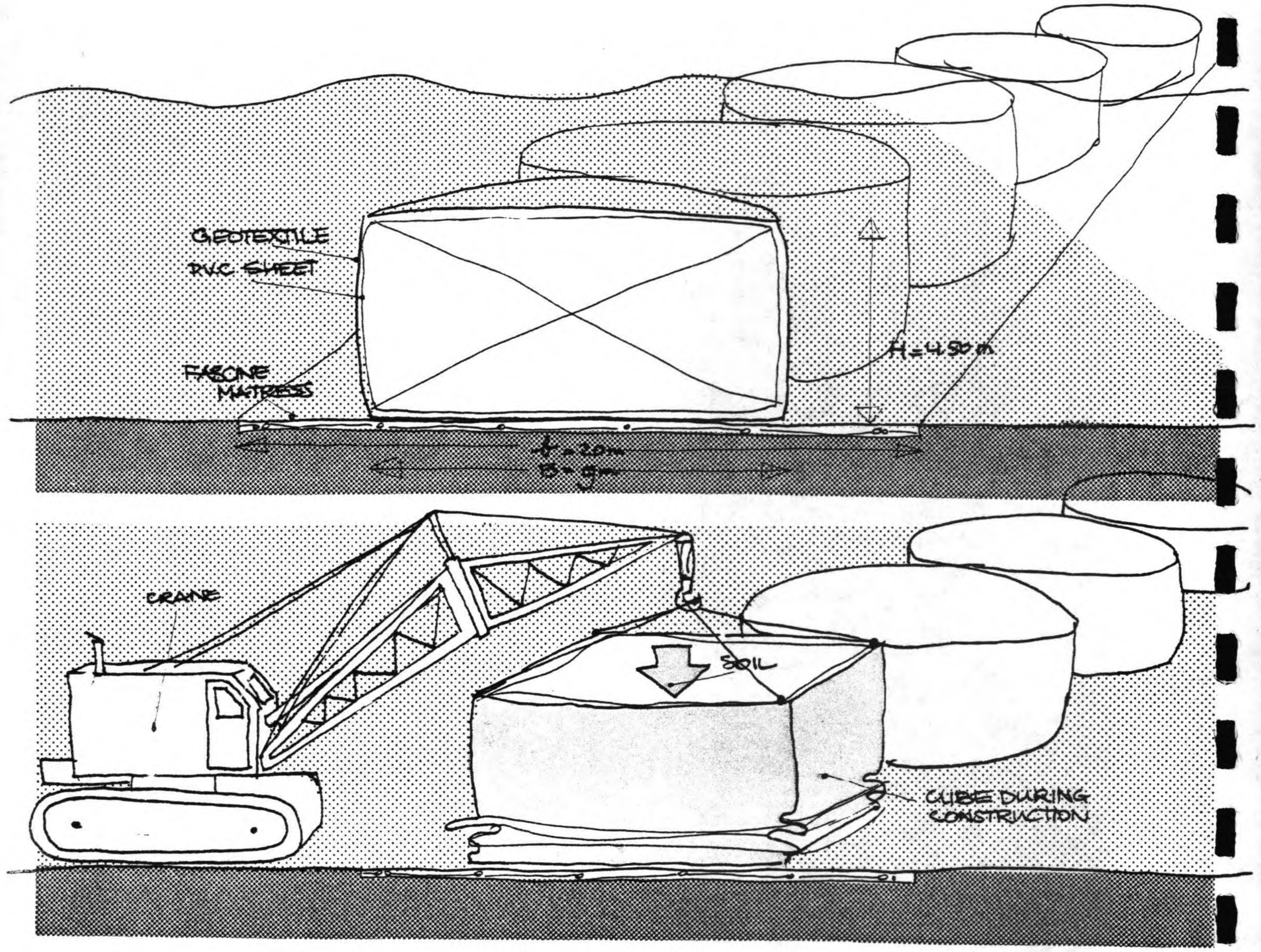
Materials (per running meter)

SOIL :	- the silt in situ: dry	≥ 30	[m3/m']
GEOTEXTILE:	- high quality cloth having a tensile strength of 600 kN/m':	≥ 35	[m2/m']
	- p.v.c.-sheet (inlay):	≥ 35	[m2/m']
EQUIPMENT :			
case 1 :	- cranes and filling equipment:	500	[m3/hour]
	- crane for lifting compartment:	1	[piece]
case 2 :	- cranes on pontoons, capacity:	125	]m3/hour]
	- crane for lifting compartment:	1	[piece]
	- vessels for soil, capacity:	800	[m3].

Advantages and disadvantages

Advantages:
- flexible construction method;
- little square meters of cloth needed;
- sewings of cover can be very simple;
- current and well-known building-equipment.

Disadvantages:
- the safety of the structure is unsufficient: damage of the geotextile will eternally cause the loss of a compartment;
- problems will rise in the sewings that connect the compartments. These sewings are more stiff than the rest of the geotextile, they will attract the loadings and finally they will collapse;
- the required tensile strength of the geotextile is very high.


GEOTEXTILE
P.V.C. SHEET
FASCINE MATTRESS
H=4.50 m
L=20m
B=9m
CRANE
SOIL
CUBE DURING CONSTRUCTION

5.10
Cubes

5.1.10 Cubes

Description

Since the sewing of perfect cylinders of geotextile cloth is not possible., the practical shape of "cylinders" consists of cubes. These cubes can be prefabricated, and transported to the construction site.There they are placed upon a prepared fascine matress. A crane, or framework, holds up the cube while filling (done in the same way as the compartment dam). Closing of each cube is realized by sewing a cover on the filled cube.
The dimensions of the cubes depend on the number of rows of cubes (see chapter 3.2). For one single row (n = 1) the width must be 9 m, the length is 9 m and the height 4.5 m. For (n = 2) two rows, the width of each cube is 5.5 m, the length is 5.5 m and the height 4.5 m (see fig. 5.10).

Construction can take place during the low-water periods only (this results in a working-time of two hours per day). When operating during the total tidal period, all cranes must be equiped with a pontoon, and the soil has to be transported to the construction-site by vessels.

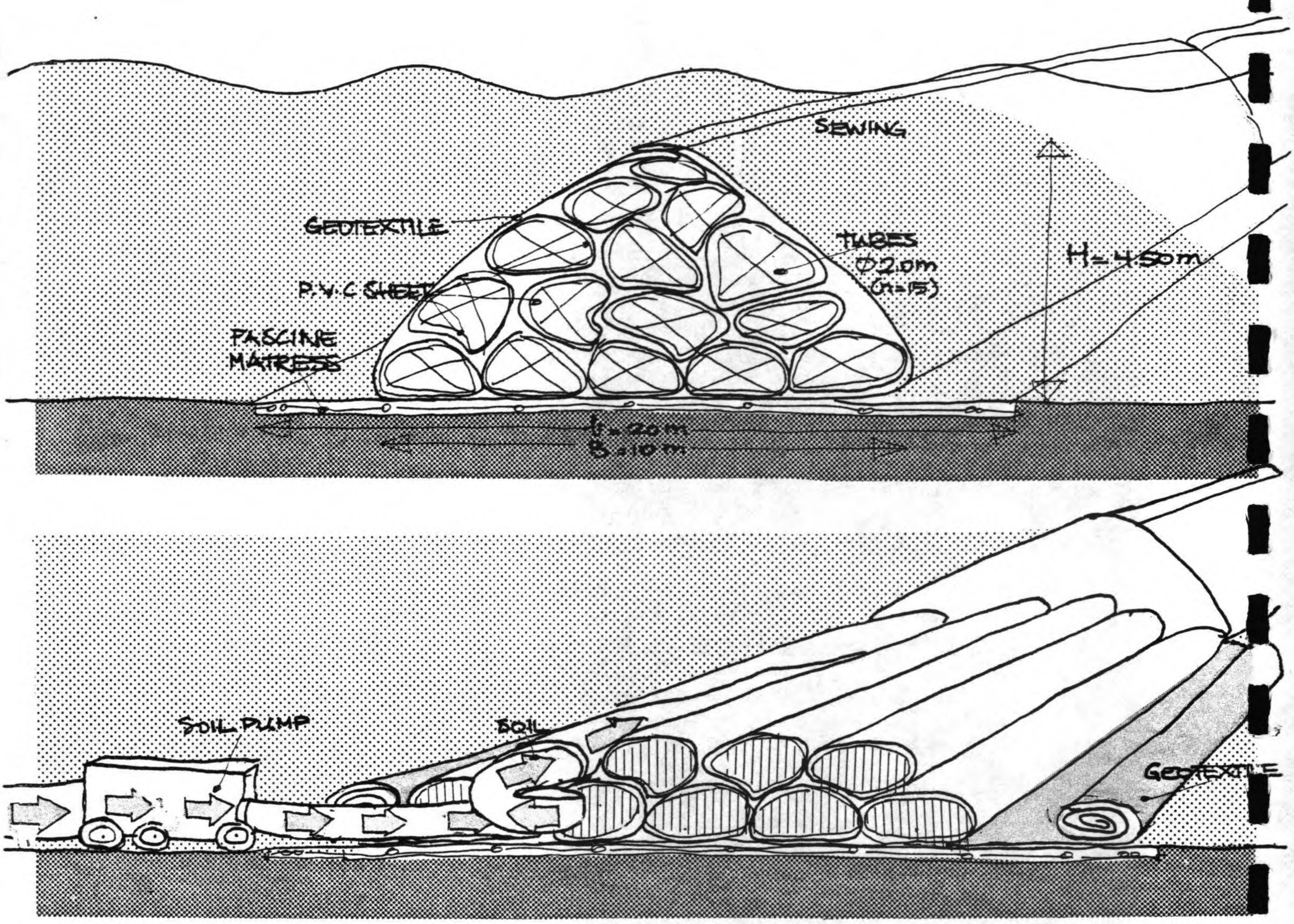
Materials (per running meter)

SOIL	: - silt in situ: as dry as possible:	
	1 row :	40 [m3/m']
	2 rows:	50 [m3/m']
GEOTEXTILE:	- high quality geotextile	
	1 row : 1,000 kN/m'	36 [m2/m']
	2 rows: 600 kN/m'	60 [m2/m']
	- p.v.c.-inlay	idem
EQUIPMENT	: the same as under 5.1.9.	

Advantages and disadvantages

Advantages:
- relatively little geotextile needed;
- building equipment is current and well-known;
- flexible construction method;
- sewings of cover are simple.

Disadvantages:
- a very high tensile strength is required in the geotextile;
- the safety of the structure is unsufficient: damage of the geotextile will eternally cause the loss of a cube;
- the openings between the units will cause severe erosion problems (unless 2 rows are applied).



5.11
Small tubes

5.1.11 Small tubes

Description

A geotextile cloth is placed upon a prepared fascine mattress at the construction site. Prefabricated tubes are transported to the construction site, and placed inside a high-quality geotextile cloth. The tubes are filled by soil pumps, which pump up the tubes to their required height, starting with the bottom tubes. The heigth that each of the tubes can reach determine the number, n, of tubes that are necessary to built the dike.
Next, the high-quality cloth is tightened around the pile of tubes. A reasonable size of the inner tubes is a diameter of 1.5 m, since the width of prefabricated cloths is 5.0 m.

Construction can take place only during the low-water period: 2 hours a day.

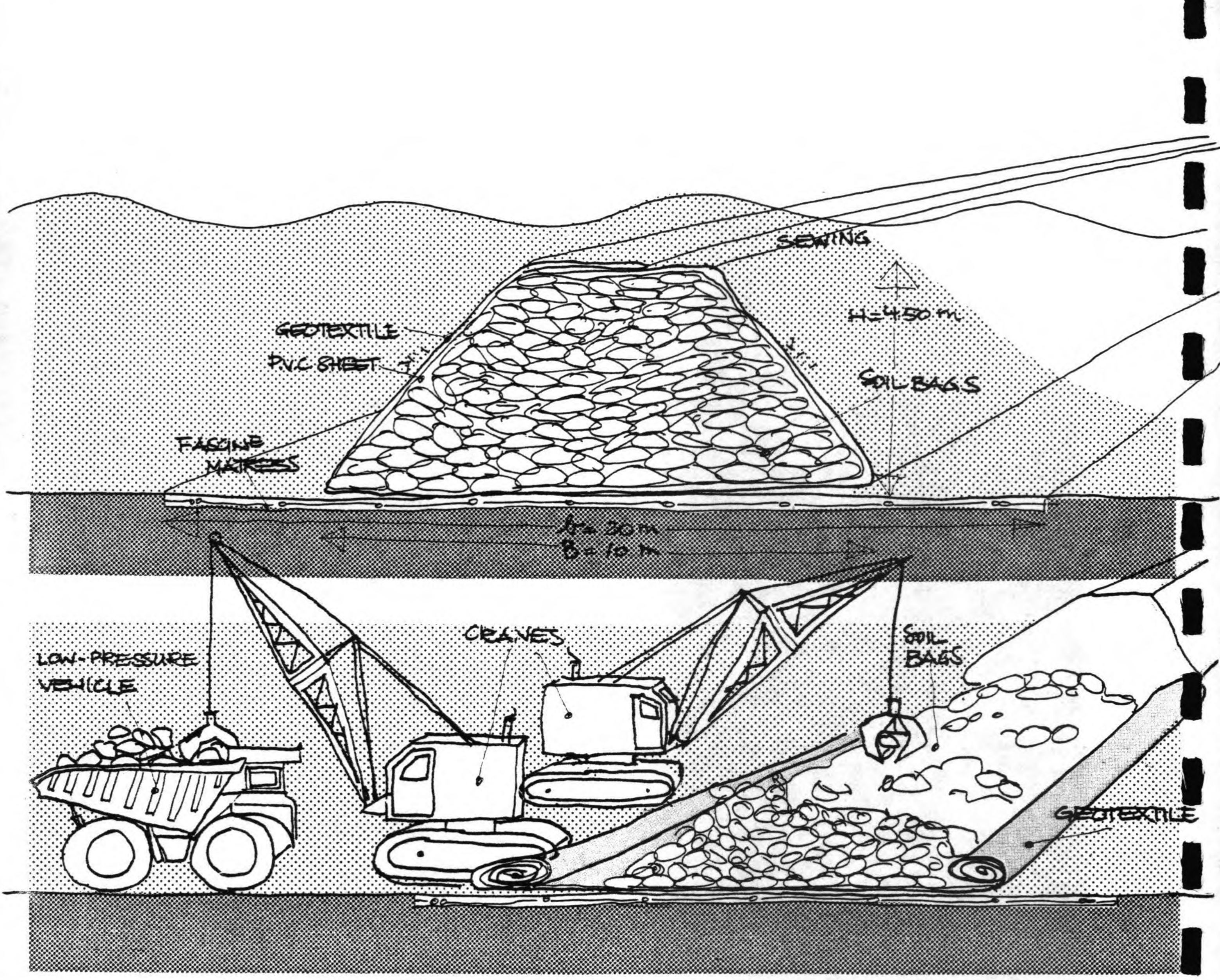
Materials (per running meter)

SOIL :	- in situ, processed through soil-pumps:	30	|m3/m'|
GEOTEXTILE:	- high quality geotextile cloth with a tensile strength of 300 kN/m':	30	|m2/m'|
	- geotextile cloth of less quality, tensile strength dependent on the dimensions of inner tubes; (if n = 21, $N\phi$ = 40 kN/m')	100	|m2/m'|
	- p.v.c.-sheet inlay (if n = 21):	100	|m2/m'|
EQUIPMENT :	- soil pumps, a total capacity:	500	|m3/hour|
	- cranes to provide the soil pumps with sufficient soil:	500	|m3/hour|.

Advantages and disadvantages

Advantages:
- high safety of the structure;
- simple construction method;
- flexible construction method.

Disadvantages:
- a large number of square meters of geotextile is needed;
- a relatively short working time is available (2 hours a day).



5.12
Soil bags

5.1.12 Soil bags

Description

The same working method as under 5.1.11 is used. However, in this case the high-quality geotextile cloth is filled with prefabricated soil bags, which are transported to the construction site by loading-trucks or vessels. The (pre)fabrication of soil bags, and the transport to the construction site continues during the entire tidal period. During low tide, the bags are placed upon the geotextile cloth and piled up to the required hight of 4.50 m. Then the geotextile is pulled around the bags, and sewed together.
The dimensions of the bags is dependent fabrication process, transport mechanism etc.

Construction of the dam takes place during low tide, but the supply of soil bags can continue during the entire tidal period.

Materials

SOIL :	- the silt in situ (or at the soil bag prefabrication site):	30	[m3/m']
GEOTEXTILE:	- high quality geotextile cloth of 300 kN/m':	30	[m2/m']
	- low-quality geotextile cloth for the soil bags, dependent on their size, ("garbage "bags"):	200 - 550	[m2/m']
	- if extra p.v.c.-sheet is necessary:	30	[m2/m']
EQUIPMENT :	- a soil-bag-fabrication-plant near the construction site:	1	[piece]
	- transport equipment to transport the bags, total capacity:	1.000 - 30.000	[bags/day]
	- cranes to place bags inside geotextile, total capacity:	1.000 - 30.000	[bags/day].

Advantages and disadvantages

Advantages:
- high safety of the structure;
- flexible construction method;
- the "high quality" geotextile cloth can also be a net, and the quality of the cloth of the bags can be relatively poor.

Disadvantages:
- the processing of the enormous number of soil bags is complicated;
- extra facilities must be created: soil bags must be fabricated, a transport system to and from the site must be developed.

5.2 Most feasible ways of construction

In order to compare the different alternatives, some points of importance have been selected:

A. concerning the geotextile:

1. the required tensile strength
2. the number of square meters of cloth required to construct the dam

B. concerning the building method:

3. flexibility of the method: the possibility to adapt the construction height and width, without changing equipment or units, the currency of the equipment
4. available working time: time during which the dam is constructed
5. expected problems during construction and the reliability of the equipment: damages to equipment, problems with sewings, complexity of the building process
6. connection between the units, and between the parts that have been constructed during each day. Insufficient linkage causes erosion problems, damages, etc.

C. concering the safety of the construction:

7. the safety against (local) damages: small tears in the geotextile, accidents with boats, etc.
8. sensitivity to softening: softening of the total filling causes failure of the dike

D. concerning the toal costs of the project:

9. the expected costs, as a result of 1 to 8

E. concerning the feasibility of the project:

10. is it feasible, as a result of 1 to 8

the total costs (9) and the feasibility (10) form the two major criteria, the project must be valued thoroughly with respect to these two items. The valuation for each of the alternatives discussed in par. 5.1 is given in table 5.1:

	GEOTEXTILE Nϕ (1)	m2 (2)	CONSTRUCTION ↑↓ (3)	t (4)	? (5)	→← (6)	SAFETY d (7)	s (8)	COSTS (9)	FEASI-BILITY (10)
1. Cubatao-method	+	++	+	0	---	0	--	0	+++	-
2. Jack-screw method	+	++	--	-	---	0	---	0	+	-
3. Pumping battery	+	++	--	-	---	0	---	0	+	-
4. Mould method	+	+	+	+	0	0	-	0	+	+
5. Self-unloading mould	+	+	--	+	--	0	--	0	0	-
6. Injection method	0	-	+	-	0	0	-	---	-	-
7. Packed soil	+	0	+	--	+	0	-	0	+	++
8. Reinforced soil	++	--	+	+	0	+	+	--	-	-
9. Compartment dam	---	0	0	+	--	--	--	0	+	-
10. Cubes	---	0	+	0	+	--	-	0	+	+
11. Small tubes	++	--	++	0	++	++	++	++	-	+++
12. Soil bags	+++	---	++	0	++	++	+++	++	--	+++

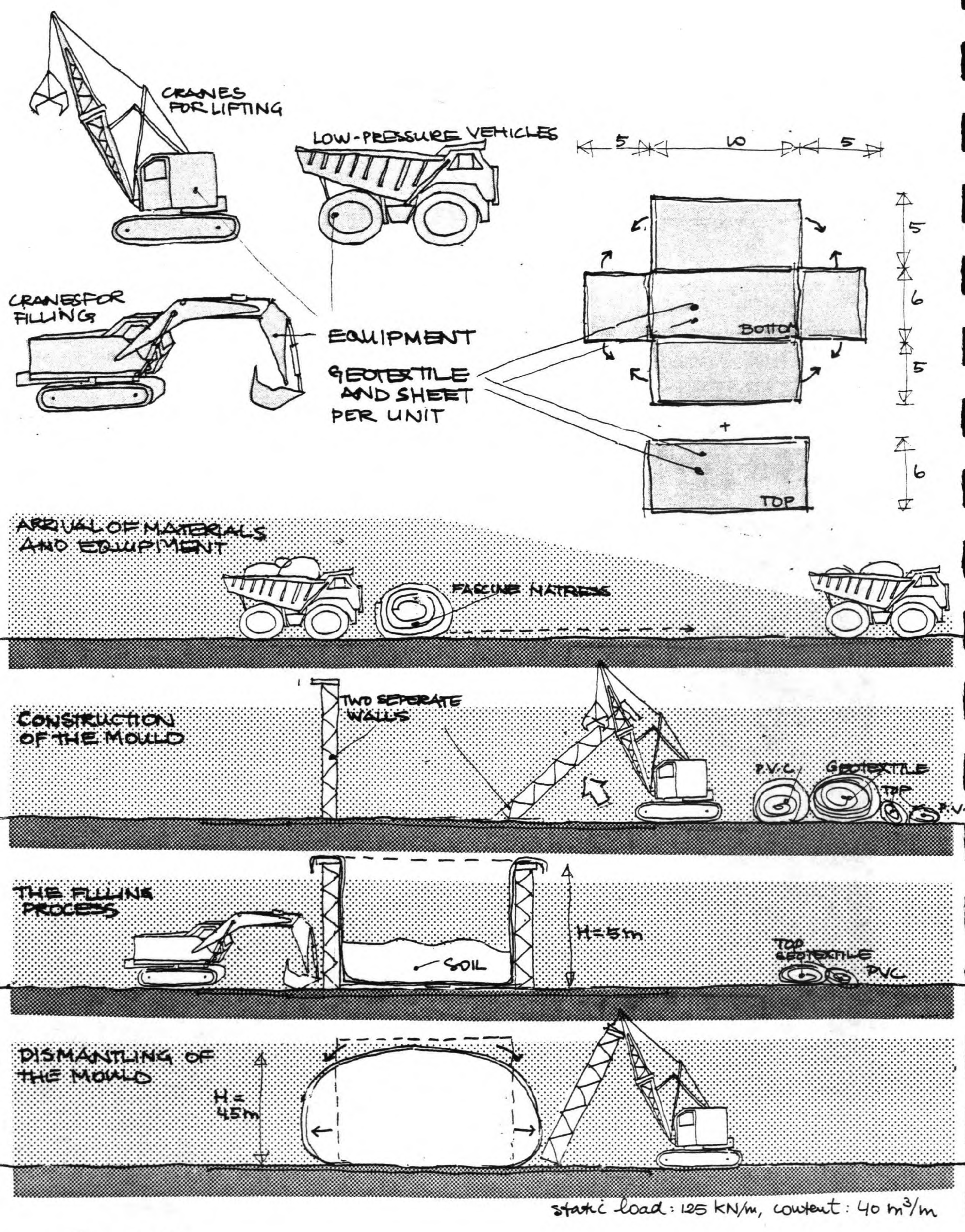
Table 5.1: valuation of the alternatives

--- = inadequate
-- = very unfavourable
- = unfavourable
0 = moderate
+ = favourable
++ = very favourable
+++ = excellent

The five alternatives that are considered feasible, are discussed in the following:

- the mould-solution
- the packed soil-solution
- cubes
- small tubes
- soil bags.

It will be described more in detail what materials are necessary, the activities during construction, specific advantages and disadvantages and the total costs of the project for each solution.



5.13

The mould solution

5.2.1 The mould-solution

The construction-phases can be described as follows (see fig. 5.13):

- at low-water low pressure vehicles arrive at the dike-tracé, containing the necesary building-materials, 3 or 4 movable moulds (easy to assemble) and the necessary manpower to conduct the construction. Also the rest of the equipment: cranes, soil-pumps etc.;

- a fascine mattress is spread along the tracé with a width of 30.0 m;

- the mould-segments, consisting of two strong frameworks and some connecting beams, are built up. As a partition between the segments a removable wall is used;

- next a pre-sewn cloth of geotextile, with a tensile strength of 350 kN/m', is placed inside the mould segments;

- a pre-sealed plastic sheet, with the same dimensions as the geotextile, is placed inside the cloth;

- cranes and/or soil-pumps fill up the segments;

- when a segment is full, first a p.v.c. sheet is sealed along the upper side. Then also a geotextile cloth is sewn to the rest of the cloth, thus forming a package of soil in the mould-segments;

- the partition-walls are removed. Next the side-walls of the mould-segments are removed by releasing the connecting beams. Cranes remove the mould-components, either to the land-side, either to a next part of the tracé;

- this cycle is repeated for the rest of the segments, dependent on the number of segments that can be constructed during one day;

- when the water-level is rising, low pressure vehicles leave the construction site, taking all materials, equipment, people etc. to the land-side. Construction is delayed until the next day.

N.B.: a variant on the foregoing is the use of floating cranes and soil pumps. Then the filling can continue during rising-tide. The dismantling and removing of the mould-segments however, can only take place during the low-water periods.

The advantage of this method is the decreased need of filling capacity, and thus the decreased number of filling equipment.

ADVANTAGES AND DISADVANTAGES:

advantages:

* simplicity of the construction: place mould, fill it, close it, then next segment;

* segmented construction: failure of one of the segments has no consequences for the total dike, it can be replaced;

* construction method is not sensible to disturbances: damaged mould segments can be replaced, the tension on the geotextile is absent during the filling, if troubles are expected the segment can be left, standing in the mould;

* during construction, the filling can settle and compact by leaving the mould some time around the filling. Air bubbles and flocks of water can be prevented;

* by using a mould, equipment can be spared. The mould can be re-used a lot of times, at different levels and dimensions (low investments in the project);

disadvantages:

* the construction is very sensitive to damages: the mould must be removed with care to avoid tears in the geotextile;

* immediately after the mould has been removed, the tension develops in the geotextile (the square package becomes a drop-shape). This requires high-quality sewings and geotextile. An extra safety-factor might be required, since a damage of any sewing results in a sudden collapse of the total package, during removal of the mould (dangerous situation for the people);

* problems will arise in the removal of the partition-walls between the segments: the pressure of the filling will become very high. A solution must be found;

* the membrane (geotextile plus plastic sheet) must be completely sand-tight: the geotextile and sheet must be placed with care, damages must be repaired immediately;

* the tension in the geotextile is very high: 350 kN/m' is the required strength. This means high-quality sewings, also to be constructed in situ.

5.2.2 The packed soil-solution

The construction-phases can be described as follows (see fig. 5.14):

- at low-water, low pressure vehicles arrive at the dike-tracé, containing the necessary building-materials and manpower. Also cranes arrive that will take care of the filling;

- a fascine mattress is spread along the tracé, with a width of 40.0 m;

- a geotextile cloth, pre-sewn with a width of 50 m, is spread over the fascine mattress;

- a plastic-sheet with the same dimensions is placed over the geotextile;

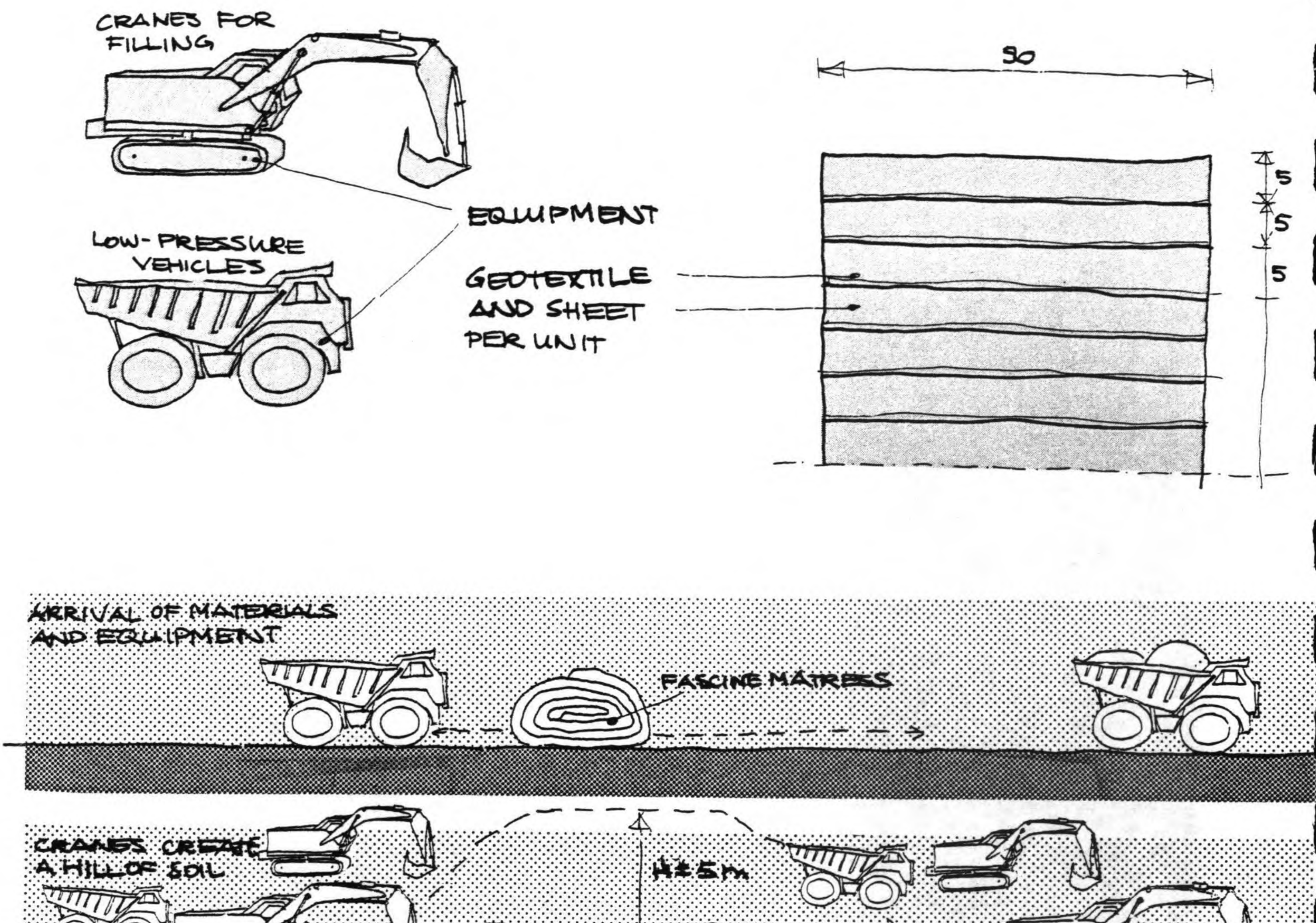
- at a higher part of the mudflat, dry soil is placed upon low-pressure vehicles by cranes. The soil is transported to the dike tracé and dumped upon the membrane;

- cranes create a hill of dry soil on top of the membrane, up to a height of 5 m. The slopes of the hill can be as steep as possible (1:2 or 1:1½);

- the plastic sheet is pulled around the hill and sealed;

- the geotextile cloth is tightened around the hill, and a high-quality connection is realized (strength: 350 kN/m');

- when the water-level is rising, the equipment and materials are removed from the construction-site. Next day the construction is continued, following the same cycle.



5.14

The packed soil solution

ADVANTAGES AND DISADVANTAGES:

advantages:

* simplicity of the construction: place geotextile, create hill, wrap hill, next section;

* construction method is not dependent on auxilary constructions like frameworks, etc.: the building-method is very simple, and the hill is self-carrying;

* construction method is not sensible to disturbances all the cranes can be replaced, tension does not develop in the geotextile then after a long time (after a storm);

* the construction is not sensible to damages: if some tears develop they can be repaired easily since there exists no tensile stress in the geotextile under "normal" conditions;

disadvantages:

* the construction is very sensible to the water-content of the filling: if it rains, the hill can't be created since the silt will behave as a fluid;

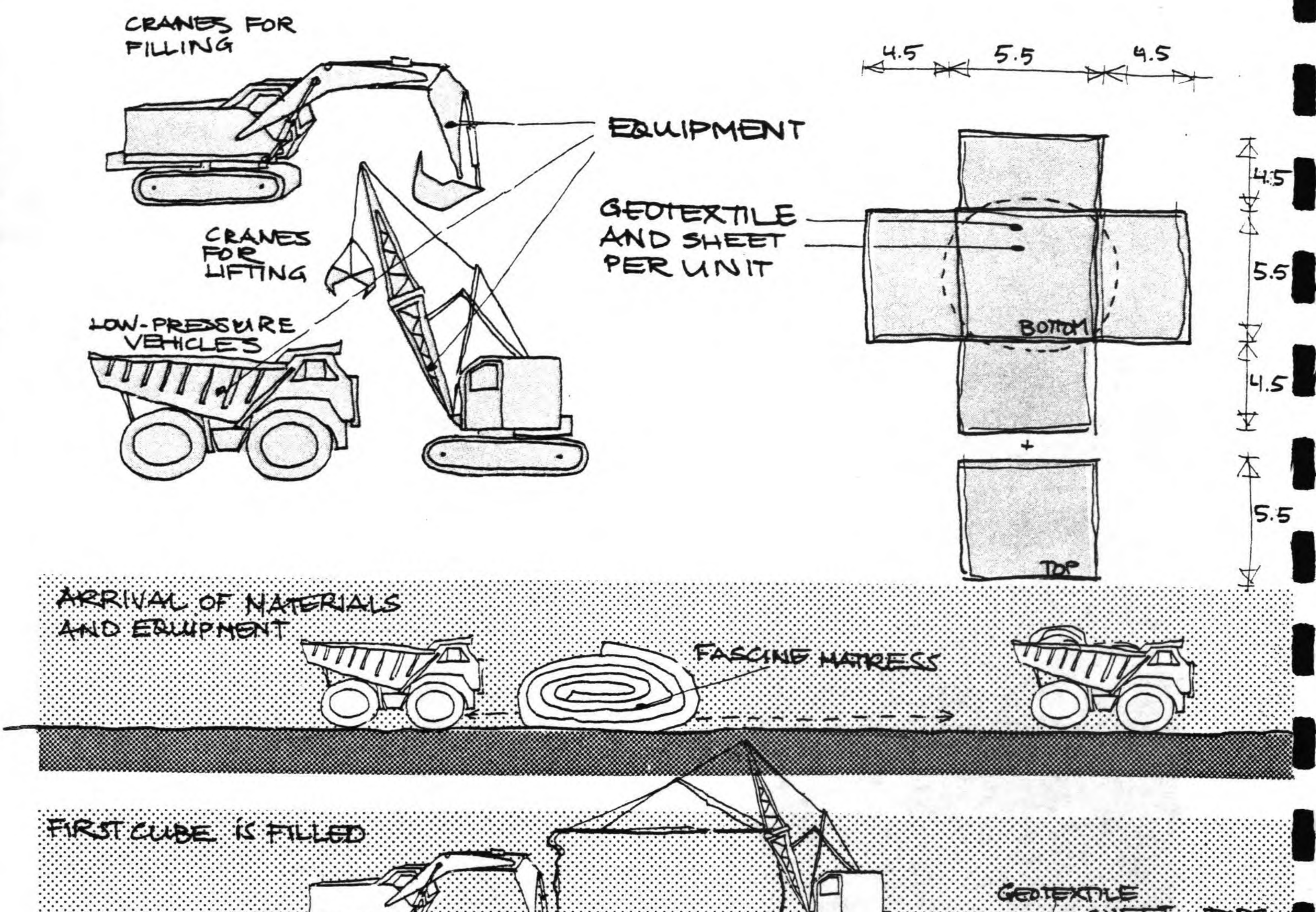
* the available working-time per day is very short, especially at the lower levels of the dike, since the construction can only take place in-the-dry;

* a high-quality connection must be realized on top of the dike: required strength of 350 kN/m' (after the first storms, the hill will also take the shape of a large drop);

* the membrane must be completely sand-tight so extra care must be taken when filling the hill, in order to prevent damages at the geotextile and the plastic-sheet;

* a relatively large number of high-quality geotextile cloth is necessary, since the total content of the dike is larger than in the other solutions. Also the dimensions of the fascine mattress are larger, for the same reason (totally: 50 m2/m');

* safety: damage of the geotextile will cause the loss of soil, and finally loss of the total segment.


CRANES FOR FILLING
EQUIPMENT
CRANES FOR LIFTING
LOW-PRESSURE VEHICLES
4.5
5.5
4.5
GEOTEXTILE AND SHEET PER UNIT
BOTTOM
TOP
ARRIVAL OF MATERIALS AND EQUIPMENT
FASCINE MATRESS
FIRST CUBE IS FILLED
SOIL
GEOTEXTILE
SHEET
TOPS
SECOND CUBE IS FILLED
TOP
GEOTEXTILE AND SHEET
DIKE AFTER CONSTRUCTION
H=4.50 m
static load: 250 kN/m', content: 50 m³/m

5.15
Cubes

5.2.3 Cubes

The construction-phases can be described as follows (see fig. 5.15). Here the solution of two rows of cubes is chosen (n = 2) in order to prevent erosion problems and in order to decrease the tensile stress in the geotextile.

- At low-water low pressure vehicles arrive at the dike-tracé, containing pre-sewn cubes, fascine mattresses, manpower etc. Also the necessary equipment (cranes, soil pumps) arrive;

- a fascine mattress is spread along the dike-tracé, with a width of 30.0 m;

- the pre-fabricated cubes (empty), consisting of a geotextile cloth with a tensile strength of 600 kN/m', plus a plastic-sheet, are placed upon the mattress. Cranes hold up the empty cubes;

- cranes and/or soil-pumps fill the cubes;

- after the cubes have been filled a cover is sewn on each of the cubes (consisting of a plastic sheet plus geotextile);

- the cranes release the cubes carefully;

- during rising tide the construction is stopped, equipment and low-pressure vehicles leave the construction site. Next day the same process is continued.

N.B.: the cubes must have a slight conical shape, in order to avoid sideward instabilities during the building stage (when the fill is rather fluid).

ADVANTAGES AND DISADVANTAGES:

advantages:

* simplicity of the construction: place cube, fill cube, close cube, next one;

* the sewing at the cover, which has to be realized in situ, can be of a relatively poor quality, since no tensile stress exists at the top-side of the cubes;

* segmented construction: failure of one of the cubes has no consequences for the rest of the dike, it can be replaced;

* no complicated constructions are necessary to support the segments: the cubes are, in principle, self-carrying. The crane only has to keep up the empty cube to avoid side-ward instability and to enlighten the filling process;

disadvantages:

* the available working-time per day is very short: construction takes place only in-the-dry;

* the tensile stress in the geotextile is enormous: 600 kN/m' is the required strength, this is also valid for the (prefabricated) sewings at the bottom of the cubes;

* the membrane must be completely sand (soil)-tight. Filling should be executed with great care to avoid damages in the sheet and the cloth, since damages will be hard to repair (high tension in the cloth);

* lost equipment: during the filling process, the cranes that hold up the cubes are not available for other purposes;

* connection of the units: since the dike consists of separate units, the cubes must be placed with care in order to avoid openings and to ensure a good connection;

* a relatively large number of high-quality geotextile cloth is necessary, since the dike consists of two rows.

* safety: damage of the geotextile will cause enormous tears in the membrane, and loss of the cube.

5.2.4 Small tubes

The construction-phases can be described as follows (see fig. 5.16):

- at low-water, low pressure vehicles arrive at the dike-tracé, containing prefabricated (empty) tubes, manpower etc. Also the necessary equipment arrives (cranes, soil-pumps);

- a fascine mattress is spread along the dike-tracé, with a width of 30.0 m;

- a high-quality geotextile cloth is spread over the fascine mattress, with a tensile strength of 350 kN/m' and a width of 30 m;

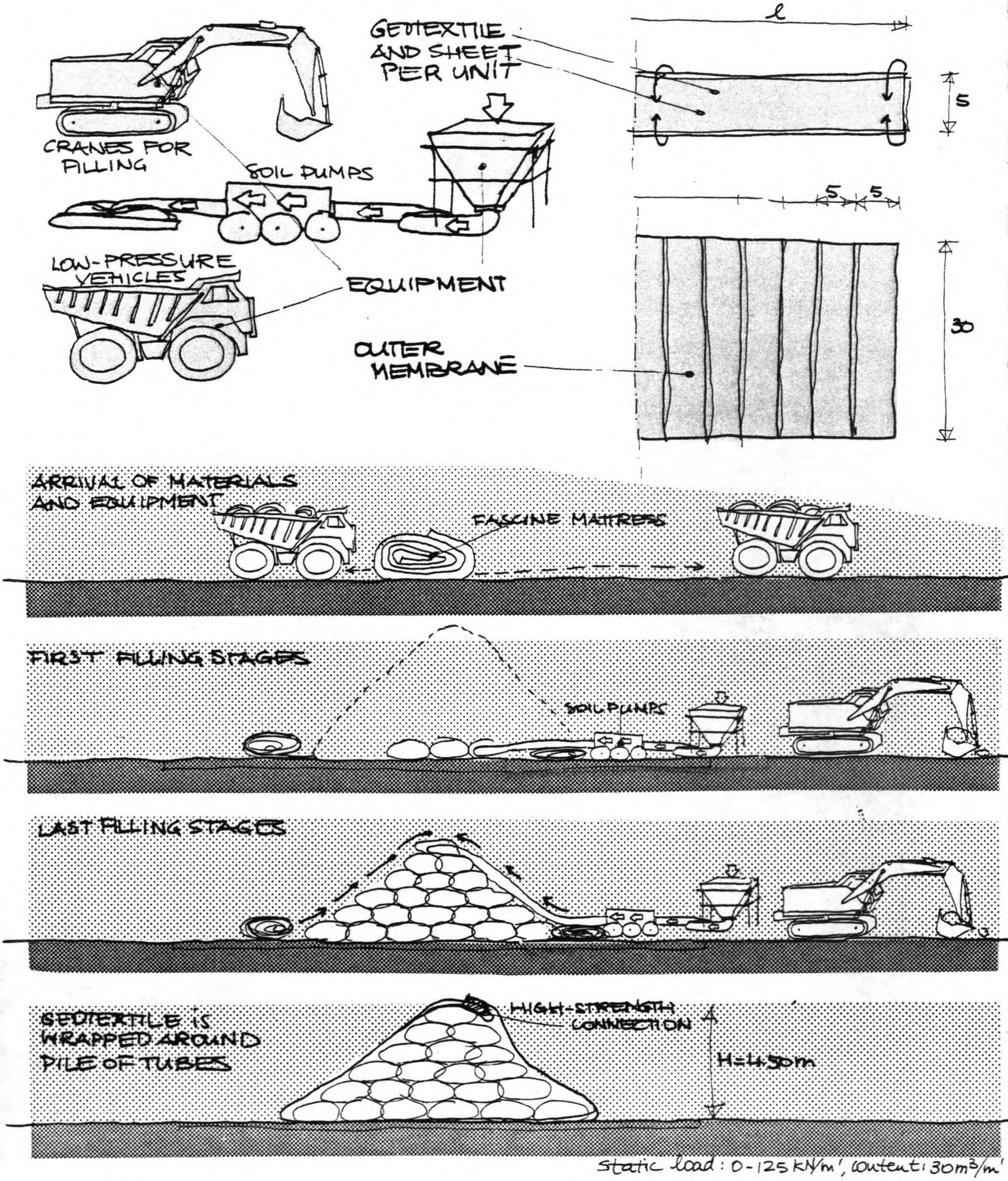
- the prefabricated tubes with a diameter of 1.5 m and a variable length, are placed in the cloth and filled by soil-pumps. The tubes consist of geotextile, with a tensile strength of 40 kN/m', and an inner layer of plastic-sheet;

- the bottom-tubes are filled first, with a filling-degree of 80%. In total 21 tubes must be filled. The tubes are closed by sewing the ends;

- after all tubes have been filled, the high-quality cloth is tightened around the pile of tubes. A high quality-connection is realized (strength: 350 kN/m');

- during rising water-level all low-pressure vehicles and equipment leave the construction site, since the work can only be executed in-the-dry. Next day this cycle is repeated.

N.B.: the filling of the tubes can continue during rising water if the soil-pumps are modified to operate also on wet silt. Also pre-filled tubes can be transported to the tracé by Vessels, in order to continue the building process during rising tide.
The connection on top of the dike does not have to be realized at the same day as the filling of the tubes is done: the pile of tubes is able to withstand (small) waves and currents.
This is advantageous for the construction process, since each building stage can be separated. (Not all the activities have to be concentrated at one spot.)



5.16

Small tubes

ADVANTAGES AND DISADVANTAGES:

advantages:

* simplicity of the construction method: place cloth, place tubes, fill tubes, close cloth, next section;

* high degree of prefabrication: in situ only the top-connection has to be realized, the rest of the sewings etc. are prefabricated: no sealing has to be done during construction (this decreases the number of necessary equipment);

* high safety of the structure: the composition of separate tubes increases the stability and unity of the construction;

* the outer, high-quality, membrane does not have to be sand-tight: the inner elements are sand tight, and small damages are allowable. This simplificates the construction in situ considerably;

* no complicated constructions are necessary to support the units: the pile is self-carrying and no equipment is lost for building-stages;

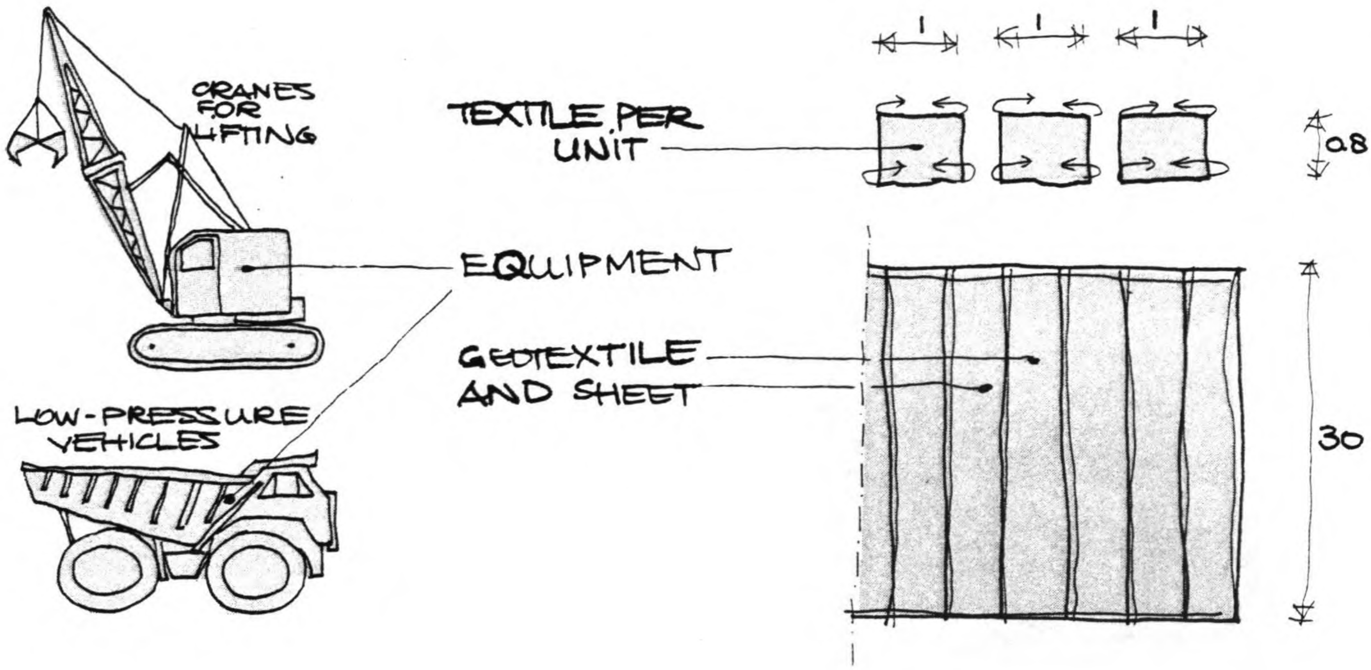
* the construction-method is not sensible to disturbances: all pumps can be replaced, tension does not develop in the outer membrane until after a long time;

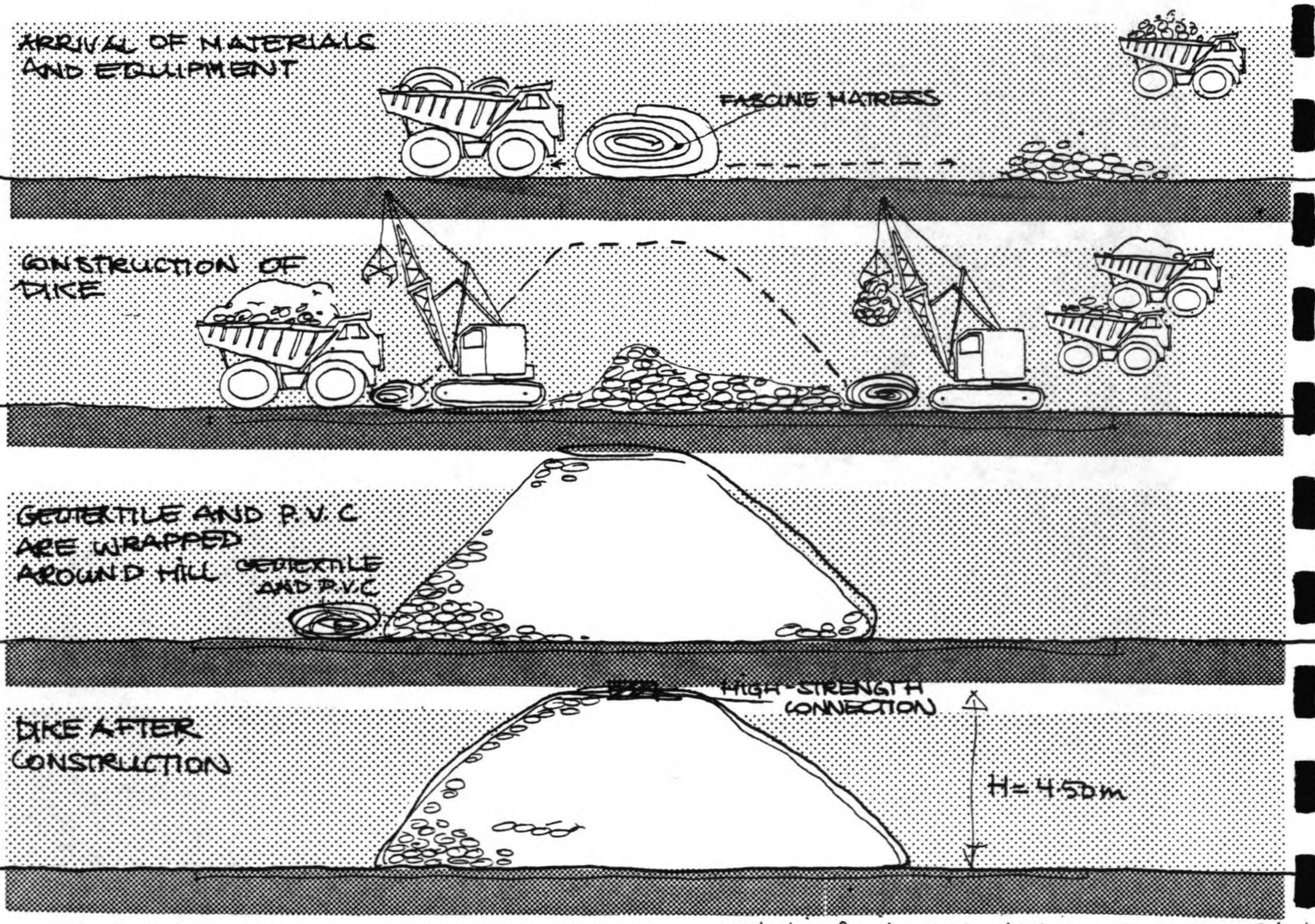
disadvantages:

* the available working-time per day is very short: construction can only take place during low-water (except in the case that special measures are taken);

* a high-quality connection has to be realized in situ (strength: 350 kN/m');

* a relatively large number of ("poor quality", strength: 40 kN/m') geotextile cloth is needed, due to the large number of tubes (totally: 105 m2/m plus 30 m2/m' high-quality cloth).





Static load: 0-125 kN/m, content: 30 m³/m'

5.17
Soil bags

5.2.5 Soil bags

The construction-phases can be described as follows (see fig. 5.17):

- at low-water, low-pressure vehicles arrive at the dike-tracé, containing prefabricated soil bags (an enormous number) and manpower. Also the necessary equipment (cranes etc.) arrive;

- a fascine mattress is spread along the dike-tracé, with a width of 30 m;

- a geotextile cloth of high-quality (strength: 350 kN/m') is spread over the fascine mattress;

- cranes create a hill of soil bags, up to a height of 4.50 m. The soil bags consist either of geotextile cloth, either of plastic sheet. In the case the bags are not sand-tight a plastic-sheet layer must be placed directly over the high-quality cloth. In the case the bags are sand-tight, this is not necessary. The tensile strength of the bags is dependent on the dimensions: in the case of 50 kg-bags the required strength is 10 kN/m'. If the bags are larger, also the strength must increase;

- the cloth is tightened around the hill of bags and a high-quality connection is realized (strength: 350 kN/m');

- the construction can continue during rising tide, since the soil bags are prefabricated, and are able to withstand (small) waves and currents. In this case the cranes must be floating.

N.B.: prefabrication of the soil-bags takes place at a higher bottom level, during the entire tidal period. Here the silt can be processed dry. The soil-bags are transported to the dike-tracé during low-tide, by low-pressure vehicles (or during high water, by pontoons).

ADVANTAGES AND DISADVANTAGES:

advantages:

* simplicity of the construction method: place cloth, place bags, close cloth, next section;

* a high degree of prefabrication: in situ only the top-connection has to be realized, the rest of the sewings, bags etc. is prefabricated;

* high safety of the structure: the composition of a lot of bags increases the stability and unity of the construction;

* the outer, high-quality, membrane or net does not have to be sand-tight: (if the units are sand tight) small damages are allowable. This simplificates the construction in situ;

* no complicated constructions are necessary to support the units: the pile is self-carrying and no equipment is lost for stability;

* the construction-method is not sensible to disturbances: damaged bags can be replaced, the cranes can be replaced, tension in the outer geotextile does not develop until after a long time, and only during storm-conditions. Damages can be repaired at any moment;

disadvantages:

* a high-quality connection has to be realized in situ (strength: 350 kN/m');

* the soil-bags have to be prefabricated at a higher level of the mud-flat. The costs of transporting all these (filled) bags can become considerably. Also a prefabrication-plant has to be developed (high investments);

* a large number of (poor quality) geotextile or p.v.c. sheet is needed to fabricate all the soil bags (in case of 50 kg-bags totally 400 m2/m, plus the 30 m2/m high-quality geotextile for the outer membrane).

5.2.6 Choice

In the foregoing, the most feasible ways of construction have been discussed.
The actual choice will be influenced by several aspects:

- the price of geotextile cloth;
- the maximum strength of the geotextile cloth;
- the maximum strength of the sewings that can be realized;
- the price of cranes, soil pumps, vessels etc.;
- the number of years in which the project has to be realized (or the required speed of construction);
- the avialable manpower, and the price of manpower;
- the required safety of the construction;
- currency of the construction-methods;
- psychological aspects;
- preference for a particular construction-method.

In table 5.2 and 5.3 a comparison of the necessary materials and activities is given, based on the most critical building stage, where the dike has to be constructed at a level of 1.00 m (Wusong-level).

N.B. 1. As a starting point it is taken, that at the most critical stage of construction (at the 1+-level) the progression of the dike must be at least 30 m'/day.

2. The available working-period is 2 hours per day (only working in-the-dry is taken into account).

3. In the case of soil-bags, the bags are made of geotextile cloth. The sand-tightness of the construction is guaranteed by a plastic sheet under the outer membrane.

A comparison can be made with respect to the necessary materials for each alternative (unity: per running meter):

MATERIALS	mould	packed soil	cubes	tubes	soil bags
basis materials:					
- geotextile cloth $N\phi$ = 350 kN/m'	34 m2	50 m2	-	30 m2	30 m2
- geotextile cloth $N\phi$ = 600 kN/m'	-	-	60 m2	-	-
- geotextile cloth $N\phi$ = 40 kN/m'	-	-	-	105 m2	-
- geotextile cloth $N\phi$ = 10 kN/m'	-	-	-	-	400 m2
- plastic sheet d = 0.5 mm	34 m2	50 m2	60 m2	105 m2	30 m2
- fascine mattress	30 m2	40 m2	30 m2	30 m2	30 m2
- soil (silt) in situ	40 m3	70 m3	50 m3	30 m3	30 m3
equipment (based on the necessary capacity and a progression of 30 m per day):					
- cranes for filling 0.5-1 m3/sewing	5	20	8	5	-
- cranes for filling capacity: 40 ton	3	-	6	-	20
- soil-pumps capacity 100 m3/hour	-	-	-	5	-
- low-pressure vehicles (40 ton)	5	40	2	2	20
construction-materials:					
- length of sewing in geotextile	-	-	8 m	5 m	-
- length of plastic-sheet sealing	3.6 m	1 m	8 m	-	1 m
- mould-segments of 10 m length	4 + 2 extra	-	-	-	-
- soil-bag prefabrication plant	-	-	-	-	1
- high-quality connection system ($N\phi$ = 350 kN/m')	3.6 m	1 m	-	1 m	1 m

Table 5.2: necessary materials per running meter, most feasible ways of construction

A comparison can be made with respect to the necessary activies, for each alternative with respect to the intensivity of labour:

ACTIVITIES	mould	packed soil	cubes	tubes	soil bags
preparation:					
- pre-sewing outer geotextile (high quality)	3	2	3	2	2
- pre-sealing plastic sheet	2	2	3	0	2
- prefabrication tubes	0	0	0	(plant)	0
- prefabrication soil bags	0	0	0	0	(plant)
construction (in situ):					
- placing of fascine mattress	1	1	1	1	1
- placing geotextile cloth	2	1	0	1	1
- placing plastic sheet	2	1	0	0	1
- placing mould construction	3	0	0	0	0
- placing empty cubes/tubes	0	0	3	2	0
- placing soil bags	0	0	0	0	3
- manning of the equipment	2	3	2	2	3
- supervision	2	2	2	2	3
- sealing of plastic sheet	3	2	3	0	2
- sewing of geotextile	0	0	2	1	0
- high-quality connection geotextile	3	2	0	2	2
- dismantling construction	3	0	0	0	0
- removal of equipment etc.	2	3	2	2	3
after-care:					
- inspection	2	1	2	1	1
- repair	2	1	2	0	0

Table 5.3: necessary activities most feasible ways of construction
0 = zero activities 2 = medium intensivity
1 = low intensivity 3 = high intensivity

The construction methods as described before, are considered the 5 principle ways in which the construction can be realized.

In general, each construction method has its specific advantages and disadvantages. However some remarks can be made:

- the mould solution:

  requires a very heavy framework (the force of the filling is considerable), this framework has to be dismantled and removed all the time;

  also the sewings of the geotextile have to be very strong: 350 kN/m'. Research must be done, whether this strength is feasible;

- the cubes-solution:

  the strength of the geotextile must be enormous: 600 kN/m'! Up to now no such geotextile is available. A solution could be to apply several layers of geotextile. Intensive research must be done, whether this will result in the required strength;

- the packed-soil solution:

  together with the tubes and soil bags; in this case only one sewing, or connection, has to be made on top of the dike. A solution can be found for this (a kind of clamping-device). Furthermore, this connection will not be under tension during normal conditions.

For reasons of safety, these constructions are not so advantageous, since damage of the geotextile will immediately result in (big) losses of filling-material. A solution could be an intensive inspection;

- the tubes solution and soil-bags solution:

  the total costs will be rather high, due to the increased number of square meters of geotextile cloth. However, these constructions seem very feasible, no problems are expected during the construction phases, since the technology is simple, straightforward and easy to survey;

- combinations:

  in order to find the most optimum way of construction, combinations of the five basic building methods are possible. For example the packed soil method combined with an outer layer of soil bags reduces the disadvantages of both systems. In the next report this item will be outlined.

The best construction method can't be determined yet, first an intensive study, as mentioned in chapter 6, will be necessary.

6. FURTHER APPROACH

In this report some assumptions have been made with respect to the response of the construction, and the required strength of the construction. As a result, some ways of construction have been developed. Still a lot of information has to be gathered before a preliminary design of the construction can be determined.

6.1 Proposals for testing

In the scope of this feasibility-study following tests are necessary in order to verify the calculations and assumptions made in this report.

A. PRELIMINARY IN SITU TESTS IN CHINA ON THE SUBSOIL:

in order to verify the bearing-capacity of the subsoil, since this study showed that the sliding condition (par. 3.2) caused the most severe condition for stability.

B. PROTOTYPE TEST OF A CONSTRUCTION-DESIGN AT THE BEACH:

in order to verify whether the proposed construction-methods can be realized in practice. Also in order to gain some experience with the use of geotextile as a building-material, and to check whether soil-pumps and cranes form the appropriate equipment for the construction of the dikes.

C. (SCALE) MODEL TEST IN THE LABORATORY:

in order to verify the response of the construction under extreme conditions, and in order to verify the behaviour of the proposed constructions under the influence of (irregular) waves.
Also the sensitiveness of the response to parameters like wave-height, currents, piping, and the sensitiveness of the filling to softening must be determined.

ad A): preliminary in situ test in China on the subsoil

Rotterdam Public Works have taken some samples at characteristic places of the mud-flat of Cao Jin. In a following report the set-up and results of this test will be discussed.
Special attention has been given on the subject of

- distribution of the grain sizes;
- content of organic material;
- bearing capacity of the soil, by means of shear-stress measurements;
- the influence of the moisture-percentage of the subsoil on the shear-stress.

ad B): prototype-test on the beach

Rotterdam Public Works will perform a prototype test on two of the most feasible ways of construction. The results and conclusions of this test will be given in a following report.
Special attention will be given to:

- the necessary strength of the geotextile;
- the optimum filling-degree of the geotextile units;
- the possibilities of soil-pumps and cranes as equipment;
- the extra requirements caused by the construction method;
- softening-aspects during filling.

ad C): (scale) model test in the laboratory

At the Deltagoot of the Delft Hydraulics Laboratory, the facilities are available to perform the necessary tests under irregular wave-loadings. However the financial means, as disposed in the agreement between Rotterdam Public Works and Shanghai Bureau of Water Conservancy, are not sufficient to perform such a study. Therefore, Rotterdam Public Works recommend to perform such a study either in China, either in Delft, at a later stage. This in the scope of a continuing study to the preliminary design of the dike-construction.

6.2 Recommendations

The conclusion of this study is that it may be technically feasible to construct a dike, rather cheap of construction, in order to stimulate the natural accretion. However the economic feasibility depends on construction costs, as well as on the amount of land that will be gained and the rate at which the land can be reclaimed.
For these reasons, following studies are recommended.

A. COLLECTION OF THE BOUNDARY CONDITIONS
- hydraulic and morphological data;
- industries with respect to geotextiles;
- the availability of equipment;
- the availability of labour and the training needs.

B. THE BEHAVIOUR OF THE DIKE CONSTRUCTION
- the theoretical approach[1)];
- scaletests and prototypetests;
- adjustment of the calculations to the results of the tests;
- behaviour of the subsoil;
- tests on the subsoil, in combination with the existance of the dike;
- in situ tests.

C. CONSTRUCTION METHOD
- overview of possible methods, ideas [1)];
- detailed elaboration: constructionplan;
- examination of local equipments and manpower;
- tests on small scale.

D. GEOTEXTILE
- data about existing geotextiles;
- developments in the industry;
- experience in sewing, sealing, etc.

E. LAY-OUT AND THE EXPECTED SEDIMENTATION
- experience with existing dikes[1)];
- model-approach[1)].

F. ANALYSIS OF THE COSTS AND BENEFITS
- preliminary feasibility study on the subject of the costs;
- detailed calculation of the expected costs.

G. DESIGN AND CONSTRUCTION
- first design;
- evaluation;
- final design;
- construction.

[1)] These studies are presented in this report.

Ad A): collection of the boundary conditions

1. Hydraulic and morphological data are still urgently required; also to ascertain the risk and quantity of the dams.
   A hydrographic study should start with particular reference to water depth and waterlevel-variations; waves; currents and sediment concentrations.
   The program should comprise:
   - a map of scale 1:2,500 with soundings every 200 m. The map will have to be updated every three months;
   - waterlevel gauges on three places near Jin-shan-wei and Jin-hui;
   - a permanent current gauge on one site near Cao Jin;
   - three buoys in order to measure contineously the wave-heights at the same places as the waterlevel-measurements;
   - measurements over the total height of the water column with respect to sediment-concentration and current-velocities;
   - detailed data of the subsoil (bearing capacity, composition, sensibility to softening) and of the suspend sediments (about 30 samples each).

2. In order to become hold on the problem, preliminary attention has been given to the way of construction in relation to the requirements of the dam and the geotextile.
   However, every effort should be made to maximize the use of local resources (manpower, materials, equipment, machines etc.).
   This will generate both cheap solutions and the utilization of local industries. In order to optimize this use, it is important to start a survey of the local industries:
   - geotextile weaving industries; the capacity, quality-control, ability to satisfy the requirements, the possibilities of adaption;
   - the strength of existing textiles, and the possibility to generate stronger ones;
   - the durability and UV-stability of the geotextiles and the possiblity to improve them;
   - the experience in sewing the geotextile, the strength and durability of the sewings;
   - the dimensions of the cloth: length and width that each industry can produce and the possibility of changing patterns etc.

2. The same as under 2, with respect to the equipments and industry of equipments. In this report a first inventarization of necessary equipment is given: cranes for lifting and filling, soil pumps, low-pressure vehicles.

If possible construction must continue during the entire tidal period; as such there is also need for: vessels, pontoons, floating cranes, pipelines etc.:
- which equipment is available and where does it has to come from, development in the industry;
- what is the capacity of the equipment at the moment;
- technical capacity, lifetime, possibilities for repair in situ, availability of mechanical parts, etc.;
- the financial budget in order to order new equipments.

4. The same as under 2 with respect to the capacity of manpower and their training, which might be determining for the construction method:
   - the availability of manpower, the planned number of people - to be - involved the costs of manpower and the technical level of manpower;
   - experience in building dikes, experience in working with cranes, soil pumps, dredging equipment, etc.;
   - possibilities for training the people; the forces in the geotextile will be rather large, therefore it will have to be handled with great care, sewings will have to be superstrong, sealings should be watertight, etc.

Ad B): the behaviour of the dike construction

1. In this report a lot of assumptions have been made on the subject of material parameters and the resulting forces and displacements. Some theories have been developed to calculate the behaviour of the geotextile tubes. Uncertain parameters in the calculation:
   - soil parameters: the elasticity, the plasticity, the shean-stress as a function of the humidity of the soil; softening aspects;
   - geotextile parameters: the exact elasticity as a function of time and stress; the effect of tears and the ability to re-spread local stresses;
   - sewing parameters: the strength of sewings and their lifetime; sensibility to small damages etc.;
   - the wave slamming pressures and the exact effect on the geotextile. This cyclic loading could become a determining parameter for the strength of the geotextile, with respect to fatigue and aging aspects.

2. In order to valuate the theoretical approach, intensive testing will be necessary. Taking into account the enormous stretch of dikes that will have to be realized, it is vital to find answers to all uncertain points of the calculations. A testing programm should comprise:
   - preliminary testing as proposed in 6.1;
   - short-term testing in situ with selected geotextiles and sizes;

- longterm testing on several lay-outs in situ with selected geotextiles;
- a profound study of the response of the dike in a full scale laboratory test;
- laboratory testing on the machinal properties of high quality geotextile, with respect to strength, UV-stability, fatigue, aging etc.

3. When the testing procedures have been performed, the theoretical approach of this report can be adjusted to the results. This can result in more detailed requirements on strength e.d.
   Also some economizing on geotextile quality etc. can be accomplished.

4. The behaviour of the subsoil has not been investigated in this report. For the stability of the structure it is vital that the dike is not undermined, eroded or based on a softened subsoil. Taking into consideration that the soil consists of silt, which erodes easily and softens easily, a good model has to be developed by which the behaviour of the subsoil can be predicted.

5. After a theoretical model has been developed, this model should be verificated by testing on prototype scale and in situ, also in combination with the dike (this can be combined with 2).

6. The best way of checking the theoretical approaches is to construct a probational compartment, in which several constructions, consisting of several alternative cross-sections (as proposed in this report). During one or two years these compartments can be observed in order to chose the best lay-out, cross-section and kind of geotextile tubes.

Ad C): construction method

1. An overview of possible methods and ideas has been given in this report.

2. A detailed elaboration can only be given when the exact cross-section and composition of the dike has been chosen on the basis of tests as proposed in A and B.
   This elaboration should comprise the items:
   - time schedule of the building stages, including preparations, investment plan, necessary equipment and manpower;
   - detailed prescription of each building stage: the design, sketches, time schedule per stage, etc.;
   - measurement in case of damage (storms, rainfall, loss of equipment, etc.);
   - alternatives for building-up each stage, in case of unexpected problems, unforseen damages to geotextile, etc.

3. Of course the optimum way of construction depends on the real potential of equipment and manpower. On the basis of studies mentioned in A2, A3 and A4 the schedule, as developed under C2, can be adjusted to the local circumstances. This way the right way of construction can be found.

4. Since the planned project is very large (the total strech of dikes will be about 12 kilometers) it is vital to apply a reliable way of construction. Thus also the construction method must be tested in situ (combined with B-6) on a small scale.
   Special attention must be given to:
   - the capacity of the equipment: in order to realize the project in a short time, enough filling capacity must be available;
   - realiability of the equipment and the construction method. There must always be an alternative construction method for each building stage, in order to avoid unallowable loss of time due to damage to a vital piece of equipment (this requires careful planning);
   - safety of the construction method:: avoid tears in the geotextile, avoid preliminary stresses in the geotextile and avoid concentration of activities on one spot;
   - the speed at which the construction progresses; if more equipment is used, the speed shuld increase;
   - the possibility to exchange equipments and manpower in case of loss etc. Each crane, pump and person should be general in use and mutually exchangable.

   It is advised to apply several alternative ways of construction, in order to compare the methods.

Ad D): <u>geotextile</u>

1. In the scope of this report, a research of literature has been performed. However, some testing and studying could be avoided, if more intensive examination on the subject of data about existing geotextiles would occur. The items which need more attention in order to determine whether the existing geotextiles satisfy the requirements:
   - tensile strength (kN/m');
   - grab strength;
   - UV-stability (chemical composition);
   - fatigue strength (cyclic load of 100 kN/m');
   - aging aspects;
   - elasticity modulus (short term and long term);
   - creep behaviour;
   - influence of salt intrusion;
   - influence of temperature;
   - properties of the material in ultimate loading conditions (strengthening, tough properties of fragile properties);
   - the strength of sewings in practice, possiblities to improve these properties, etc., etc.

2. Since the requirements on the strength of the geotextile and the sewings, as found in this report, are rather high, improvement of the properties of existing geotextiles will be necessary.
   If the project will have to be realized within 5 years, the developments in the geotextile industries must be studied with great care.
   For this project it is vital that a reliable geotextile is chosen, which retains its mechanical properties for at least five years.

3. More attention to the point of sewings and sealings. High quality connections will have to be realized. Trained people will be necessary to construct the necessary sewings and sealings (in the plastic sheet). Intensive research will have to be performed in order to improve the strength of sewings, because this will reduce the number of square meters of geotextile cloth considerably.

Ad E): lay-out and the expected sedimentation

1. As mentioned above, the economical feasibility of the project depends on the rate of sedimentation in relation to the costs of the construction of the dike. The rate of sedimentation is mainly influenced by the lay-out of the dike plan. Some experience with existing lay-outs has been gained in China. It would be advisable to check these results with the proposed lay-out.

2. The movement of the water can be simulated, just like the movement of the sediments. This can be done either by using model testing, or by computer simulation. Such a study has been performed by Mrs. van den Berg, as a part of her graduating thesis, at the University of Delft. The results of this report have been used in this analysis.

Ad F): analysis of the costs and benefits

1. The preliminary costs of each construction method can be made in the phase of the feasibility study. Before this can be done, some data have to be found (see also B2, D):
   - the price of the (existing) geotextiles and the properties (and of plastic sheets);
   - the price of the equipment; cranes, pumps, low-pressure vehicles, etc., etc. and the availability;
   - the price of manpower, taking into account the training-needs etc.;
   - the required progression speed of the project;
   - the price of large scale soil bags, or small tubes, prefabrication sites at the higher part of the mud flat.

These costs could form a determining aspect of the cross-section design and construction method.

2. The costs of the project can be determined after a design for the cross-section has been selected (B) and an exact construction plan (C) on the basis of a solid lay-out and geotextile (D, E).
   Then the prices for materials, equipment, manpower and activities can be determined, and a cost evaluation can be made (in order to determine the benefits, some long term in situ testing is necessary).

Ad G): design and construction

1. A first design is the result of the feasibility study, together with the results of research and the results of a first costs estimation.

2. Evaluation takes place during the in situ testing of some designs (B6) and the construction method on small scale (4).

3. The final design is the result of the tests mentioned under 2 and the final cost estimation.

4. All foregoing finally leads to the actual construction of the dike at the mud flat of Cao Jin.

The total project can be described, in time, by the following tabel

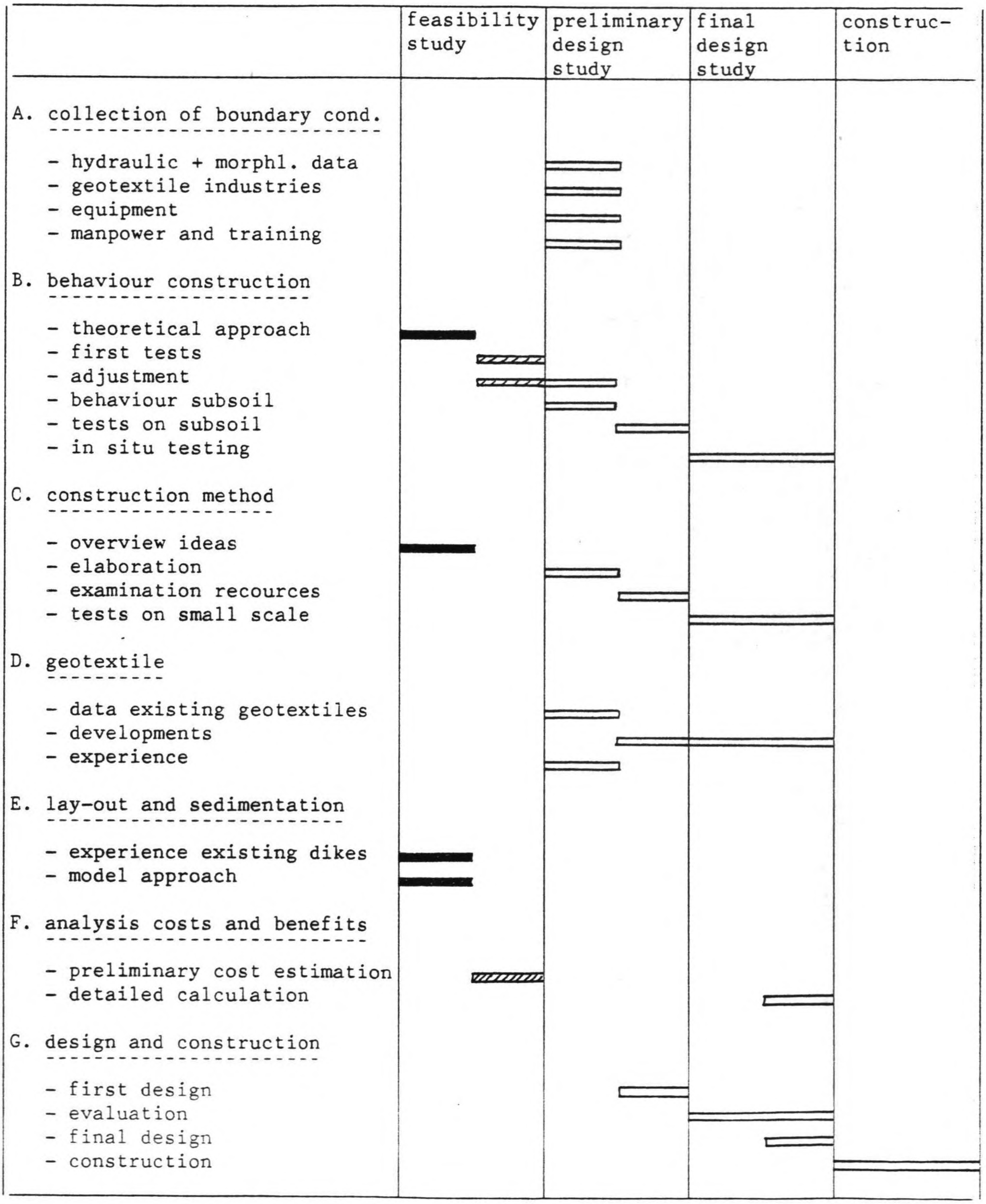


Table 6.1: time schedule study stages of the landreclamation project

= finished and presented in this report
= following stages
= future stages

NOTATIONS

parameter		unity
a	= 1. displacement of water during wave	[m]
	2. number of layers	[-]
	3. dimensionless tension-number = $N\phi/\gamma$	[-]
	4. distance from mass-centre to contact point with the subsoil	[m]
$\hat{a}$	= amplitude of water movement = ½ H	[m]
b	= width	[m]
c	= 1. celerity of wave	[m/s]
	2. sediment concentration [kg/m3],	[m3/m3]
	3. friction coefficient of damping constant	[N/s]
C_{cr}	= critical damping = 2 km	[N/s]
d	= water depth	[m]
e	= gap-width	[m]
f	= frequency = $1/T$	[s^{-1}]
g	= acceleration of gravity	[m/s^2]
h	= 1. height of unit	[m]
	2. water level	[m]
	3. initial height of liquid that forms an over-pressure of a tube or cylinder	[m]
h_o	= initial height of construction	[m]
i	= unity of imaginary direction = -1	[-]
k	= 1. wave number = $2\pi/L$	[rad/m]
	2. spring-constant or elasticity	[N/m]
ℓ	= 1. length of construction	[m]
	2. representative strip-length	[m]
m	= mass of body	[kg]
n	= 1. total number of units per cross-section	[-]
	2. normal coordinate (perp. to surface)	[m]
p	= pressure	[N/m2]
p_o	= 1. atmospheric pressure	[N/m2]
	2. initial external pressure	[N/m2]

parameter		unity
p_{min}	= minimum slamming-pressure	[N/m2]
p_{max}	= maximum slamming-pressure	[N/m2]
p_r	= pressure on membrane in r-direction	[N/m2]
p_ϕ	= pressure on membrane in ϕ-direction	[N/m2]
p_θ	= pressure on membrane in θ-direction	[N/m2]
r	= 1. radius of structure = ½ D 2. radius of curvature 3. coordinate of membranes, in horizontal direction 4. root of differential equation	[m] [m] [m] [-]
r_ϕ	= radius of curvature along meridian	[m]
r_θ	= radius of curvature along hoop	[m]
s	= coordinate along surface	[m]
t	= time-variable	[s]
t_{rise}	= rise time of slamming-impuls	[s]
u	= horizontal water-velocity in stream-direction	[m/s]
$\bar{u}$	= depth-averaged velocity	[m/s]
$\hat{u}$	= maximum water-velocity	[m/s]
u_a	= hor. water velocity predicted by airy-wave theory	[m/s]
v	= 1. hor. water velocity perpendicular to stream-direction 2. hor. water velocity in stream direction	[m/s] [m/s]
w	= vertical velocity of water	[m/s]
x	= 1. horizontal coordinate 2. horizontal displacement	[m] [m]
$\dot{x}$	= horizontal velocity	[m/s]
$\ddot{x}$	= horizontal acceleration	[m/s^2]
$\hat{x}$	= max. horizontal displacement	[m]

parameter		unity
y	= 1. coordinate in y-direction	[m]
	2. displacement in y-direction	[m]
$\dot{y}$	= velocity in y-direction	[m/s]
$\ddot{y}$	= acceleration in y-direction	[m/s^2]
$\hat{y}$	= max. displacement in y-direction	[m]
z	= 1. vertical coordinate	[m]
	2. displacement in z-direction	[m]
$\dot{z}$	= velocity vertical	[m/s]
$\ddot{z}$	= vertical acceleration	[m/s^2]
$\hat{z}$	= amplitude of vertical displacement	[m]

parameter		unity
A	= volume	[m3]
A'	= volume per running meter	[m3/m']
A_p	= projected frontal area	[m2]
$\forall$	= displaced volume of structure	[m3]
B	= width	[m]
C	= friction coefficient or damping constant for rotation-elements	[Nm/s]
C_a	= added mass-coefficient = C_m-1	[-]
C_D	= drag-coefficient	[-]
C_L	= lift-coefficient	[-]
C_m	= inertia coefficient	[-]
C_s	= 1. slamming-coefficient 2. celerity of sound in material	[-] [m/s]
C_u	= cohesion of soil	[N/m2]
C_w	= celerity of sound in water	[m/s]
D	= diameter of structure, cylinder	[m]
D_o	= initial diameter of tube or cylinder	[m]
E	= 1. energy [J] 2. dimensionless velocity-difference	[Nm] [-]
F	= force, (external)	[N]
G	= weight	[N]
H	= 1. height of unit 2. wave-height	[m] [m]
H_o	= initial height of construction	[m]
H_s	= significant wave-height	[m]
H_{max}	= maximum wave-height	[m]
HW	= high-water level (tidal)	[m]
I	= rotational moment of inertia	[kgm2]

parameter		unity
K	= 1. relative number of motion = $\hat{u}/\omega r$	[-]
	2. spring constant for rotation	[Nm/m]
K_r	= relative size-number = $\pi D/L$	[-]
$K\Delta$	= damage-coefficient	[-]
L	= 1. wave-length	[m]
	2. length of slamming water-column	[m]
LW	= low-water level (tidal)	[m]
M	= mass of structure	[kg/m'], [kg]
N	= force normal to the subsoil	[N]
$N\phi$	= tensile force in ϕ-direction	[N/m]
$N\theta$	= tensile force in θ-direction	[N/m]
P	= 1. pressure	[N/m2]
	2. circumpherence, perimeter	[m]
	3. power	[Nm/s]
P'	= circumpherence per running meter	[m2/m']
P_o	= 1. initial pressure	[N/m2]
	2. initial circumpherence	[m]
P_{min}	= minimum slamming-pressure	[N/m2]
P_{max}	= maximum slamming-pressure	[N/m2]
P_T	= theoretical max. slamming pressure	[N/m2]
R	= 1. radius of structure = ½ D	[m]
	2. radius of circle or cylinder	[m]
	3. radius of curvature	[m]
	4. velocity-ration = U/u_a	[-]
R_o	= initial radius of circle/cylinder, curvature	[m]
R_e	= Reynolds number = $\hat{u}\ D/\nu$	[-]
R_{dyn}	= dynamic radius of curvature (due to dynamic loading	[m]
S	= 1. contact-surface	[m2]
	2. sectional area of structure	[m2]
T	= 1. wave-period	[s]
	2. slamming period	[s]
T_z	= time between up- and downward zero crossing	[s]

parameter		unity
U	= current-velocity	[m/s]
U_a	= current-velocity predicted by airy-wave theory	[m/s]
U_{occ}	= occuring total horizontal water-velocity	[m/s]
U_{tot}	= total horizontal water-velocity	[m/s]
V	= enlargement-factor of dynamic motion	[m]
V_d	= enlargement-factor of damped motion	[-]
W	= friction-force along subsoil	[N]

parameter		unity
α	= 1. angle	[rad]
	2. angle incident waves parallel to structure	[rad]
	3. slope-angle	[rad]
γ	= 1. specific weight	[N/m3]
	2. safety-factor	[-]
δ	= 1. phase-angle between incident and scattered waves	[rad]
	2. thickness of air-layer	[m]
ε	= elongation = $\Delta\ell/\ell$	[-]
ζ	= 1. dimensionless vertical coordinate = z/a	[-]
	2. damping-coefficient = c/c_{cr}	[-]
η	= 1. displacement of water surface	[m]
	2. angle-number = sin ϕ	[-]
	3. viscosity of water	[kg/ms]
υ	= dynamic viscosity of water = η/ρ	[m2/s]
κ	= constant of Von Karman = 0.4	[-]
ν	= contraction coefficient of soil	[-]
ξ	= angle-number = cos ϕ	[-]
ρ	= 1. density of material	[kg/m3]
	2. dimensionless horizontal coordinate r/a	[-]
ρ_s	= density of soil, sediment	[kg/m3]
ρ_w	= density of water	[kg/m3]
σ	= stress (normal)	[N/m2]
σ_k	= grain-stress	[N/m2]
τ	= shear-stress	[N/m2]
ϕ	= 1. phase-angle	[rad]
	2. angle of internal friction of soil	[rad]
	3. angle of membrane-surface along the meridian with vertical axis	[rad]
	4. displacement in angular direction	[rad]
$\dot{\phi}$	= angular velocity = ω	[rad/s]
$\ddot{\phi}$	= angular accelaration = $\dot{\omega}$	[rad/s^2]

parameter		unity
ϕ_{st}	= angular displacement due to static load	[rad]
ϕ_{dyn}	= dynamic angle of curvature	[rad]
ω	= angular velocity, wave-frequency = 2 π/T	[rad/s]
ω_s	= slamming-frequency = 2 π/T_s	[rad/s]
Δ	= relative density = $(\rho_s - \rho_w)/\rho_w$	[-]
Δx	= 1. contraction/elongation of spring	[m]
	2. increase/decrease of mass centre in hor. direction	[m]
θ	= angle of membrane-surface along the hoop, with vertical axis	[rad]
ϕ	= velocity potential	[m]
ϕ_s	= scattered waves, velocity potential	[m]
ϕ_w	= incident waves, velocity potential	[m]
ϕ'_s	= simplified scattered wave-potential	[m]
Ω	= angular velocity or frequency of external force = 2 π/T	[rad/s]

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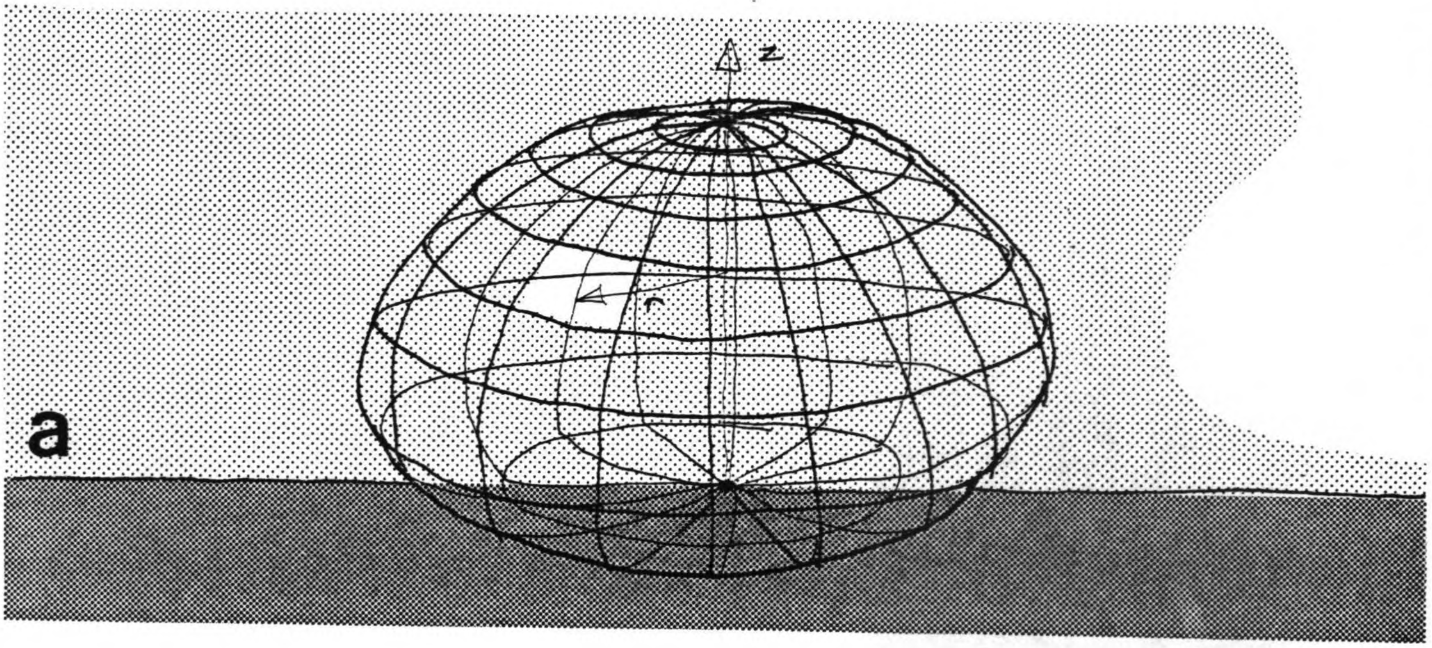
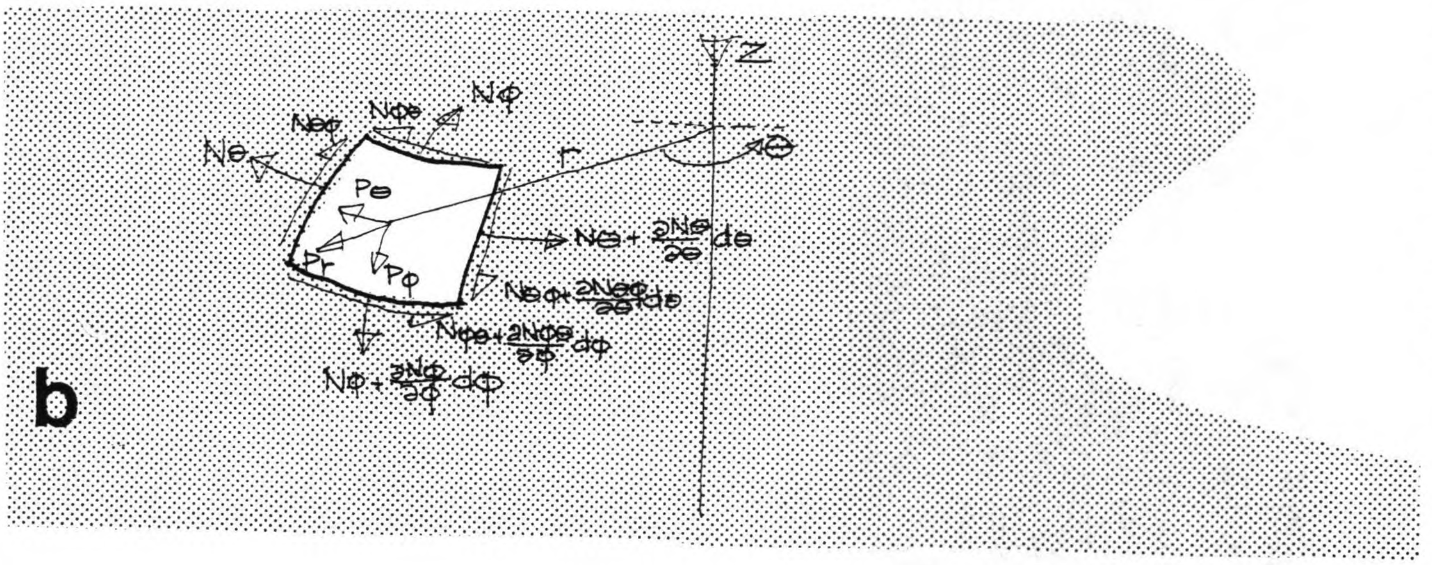
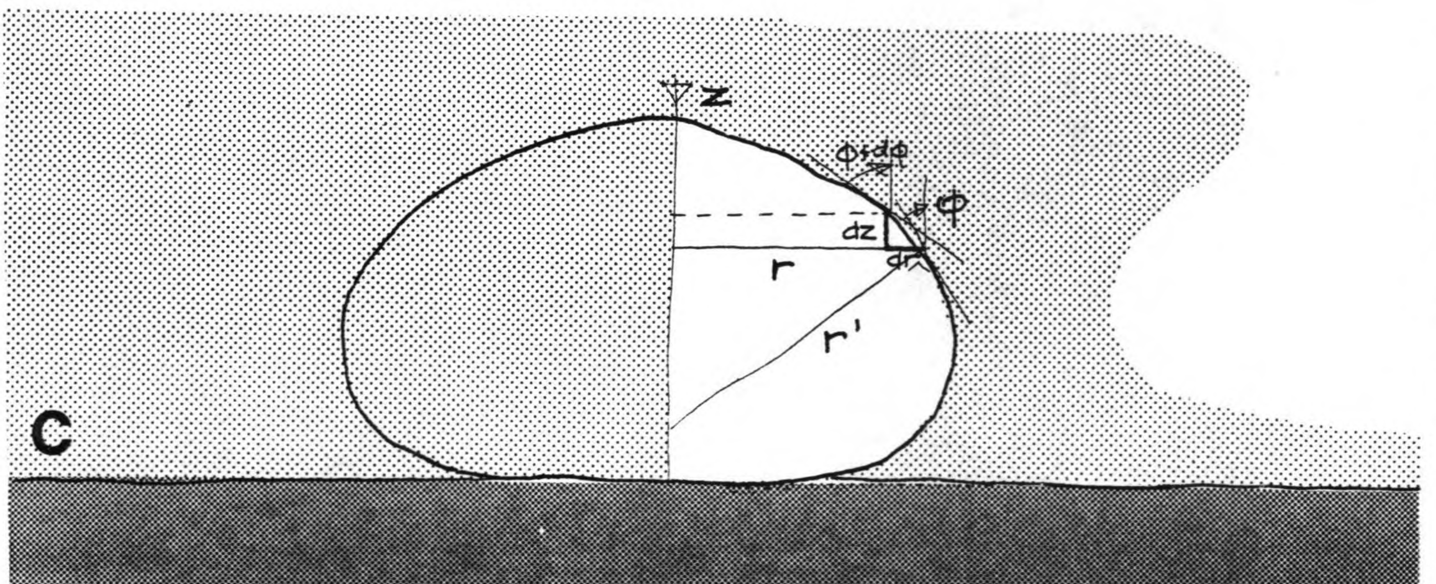
LIST OF FIGURES


z
r

a


Z
Nφ
Nφθ
Nθφ
Nθ
r
θ
Pθ
Pr
Pφ
Nθ + ∂Nθ/∂θ dθ
Nθφ+∂Nθφ/∂θ dθ
Nφθ+∂Nφθ/∂φ dφ
Nφ + ∂Nφ/∂φ dφ

b


Z
φ+dφ
φ
dz
dr
r
r'

c

1

APPENDIX A: DROP SHAPES

In general, forces and stresses in shells of revolution can be found with the membrane equations. A surface of revolution is generated by the rotation of a plane curve about an axis in its plane. This generating curve is called a meridian, and an arbitrary point on the middel surface of the shell is described by specifying the particular meridian on which it is found and by giving the value of a second coordinate which varies along the meridian and is constant on a circle around the axis of the shell. Since all these circles for different values of the second coordinate are parallel to each other, they are called the "parallel circles" (see fig. 1A), or "hoops".

When observing the shell equilibrium (see fig. 1B) the equilibrium can be described by:

$$\frac{N\phi}{r\phi} + \frac{N\Theta}{r\Theta} = Pr \qquad (1)$$

$$\frac{\partial}{\partial\phi}(rN\phi) + r\phi\frac{\partial N\Theta\phi}{\partial\Theta} - r\phi N\Theta\cos\phi + P\phi r r\phi = 0 \qquad (1A)$$

$$\frac{\partial}{\partial\phi}(rN\phi\Theta) + r\phi\frac{\partial N\Theta}{\partial r} - r\phi N\Theta\phi\cos\phi + P\Theta r r\phi = 0 \qquad (1B)$$

And the relation between r, $r\phi$ and $r\Theta$ (see fig. 1C) by:

$$r = r'\sin\phi \qquad (2)$$

$$\frac{dr}{d\phi} = r\phi\sin\phi \qquad (3)$$

$$\frac{dz}{d\phi} = r\phi\cos\phi \qquad (4)$$

$$\frac{1}{r}\frac{dr}{d\phi} = \frac{r\phi}{r'}\cot\phi \qquad (5)$$

The most economical shape of a membrane is that particular shape in which the stresses in ϕ and Θ-direction are equal and of the same magnitude all over the membrane.
In the case if a "tank" filled with a liquid of specific weight γ and an "over-pressure" at the highest pont of γh, this solution can be found (comparing with a liquid drop, where the membrane forces $N\phi$ and $N\Theta$ are formed by the capillary forces and these must be in equilibrium with the hydraulic pressure) (see lit. (18)).

For an example: in a spherical tank, the meridonial stress $N\phi/t$ is rather small in the upper part of the tank compared with the hoop stress $N\theta/t$. In a cylindrical tank, there is no meridional stress at all and the hoop stress alone carries the load.

The origin of the coordinates r, z is chosen at a level h above the top of the tank. In this notation the load acting on the shell is

$$P\phi = 0 \text{ and } Pr = \gamma z \quad \text{.......} \quad (6)$$

If we use eq (2...5) to express the radii $r\phi$ and $r\theta$ in (6) by means of r and z, we find:

$$\frac{d\sin\phi}{dr} + \frac{\sin\phi}{r} = \frac{\gamma z}{N\phi} \quad \text{.......} \quad (7)$$

$N\phi$ is the allowable, or desired, tensile stress in the membrane.

The shape of the meridian can be found together with (3, 4):

$$\frac{dz}{dr} = \tan\phi \quad \text{.......} \quad (8)$$

This set of equations (7, 8) can only be solved by numerical integration. In order to make the result more useful it is advisable to introduce dimensionless notations:

$$\eta = \sin\phi \qquad \rho = \frac{r}{a}$$

$$a^2 = \frac{N\phi}{\gamma} \qquad \zeta = \frac{z}{a}$$

The set of equations can then be described by:

$$\frac{\partial\eta}{\partial\rho} = \zeta - \frac{\eta}{\rho} \quad \text{.......} \quad (9)$$

and

$$\frac{\partial\zeta}{\partial\rho} = \frac{\eta}{1 - \eta^2} \quad \text{.......} \quad (10)$$

In order to avoid singular solutions some help must be given, or introducing following conditions:

1. for small values of r: $\rho = \frac{2a}{h} \cdot \eta$
   (z = h)

   and $\frac{d\eta}{d\rho} = \frac{h}{2a}$ .... (11A)

2. for small values of ϕ, sin ϕ is small compared with 1 and the equations become linear in η and ζ, eliminating η it gives: (to continue the calculations)

   $\frac{d^2\zeta}{d\rho^2} + \frac{1}{\rho}\frac{d\zeta}{d\rho} - \zeta = 0$ ....(11B)

3. after $\phi = 50°$, or $\eta = 0.75$, r is no longer suitable to serve as an independent variable, therefore it is introduced $\xi = \cos\phi$;

   $\frac{d\xi}{d\zeta} = \frac{1 - \xi^2}{\rho} - \zeta$ ....(12)

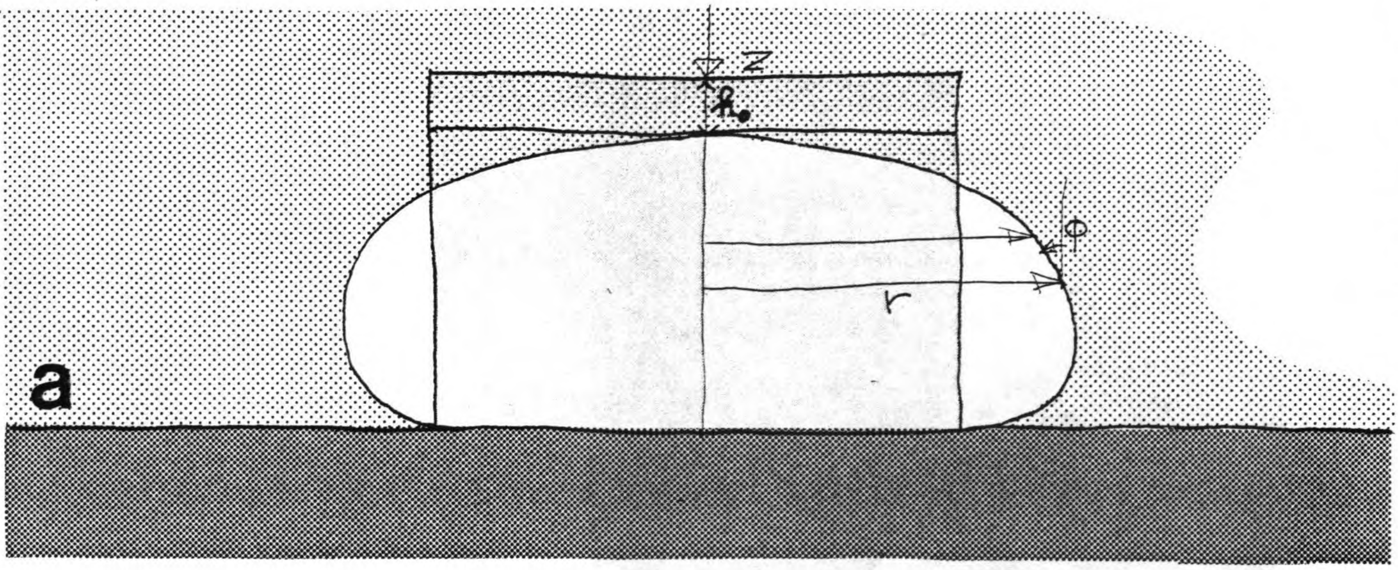
   and

   $\frac{d\rho}{d\zeta} = \frac{\xi}{1 - \zeta^2}$ .... (13)

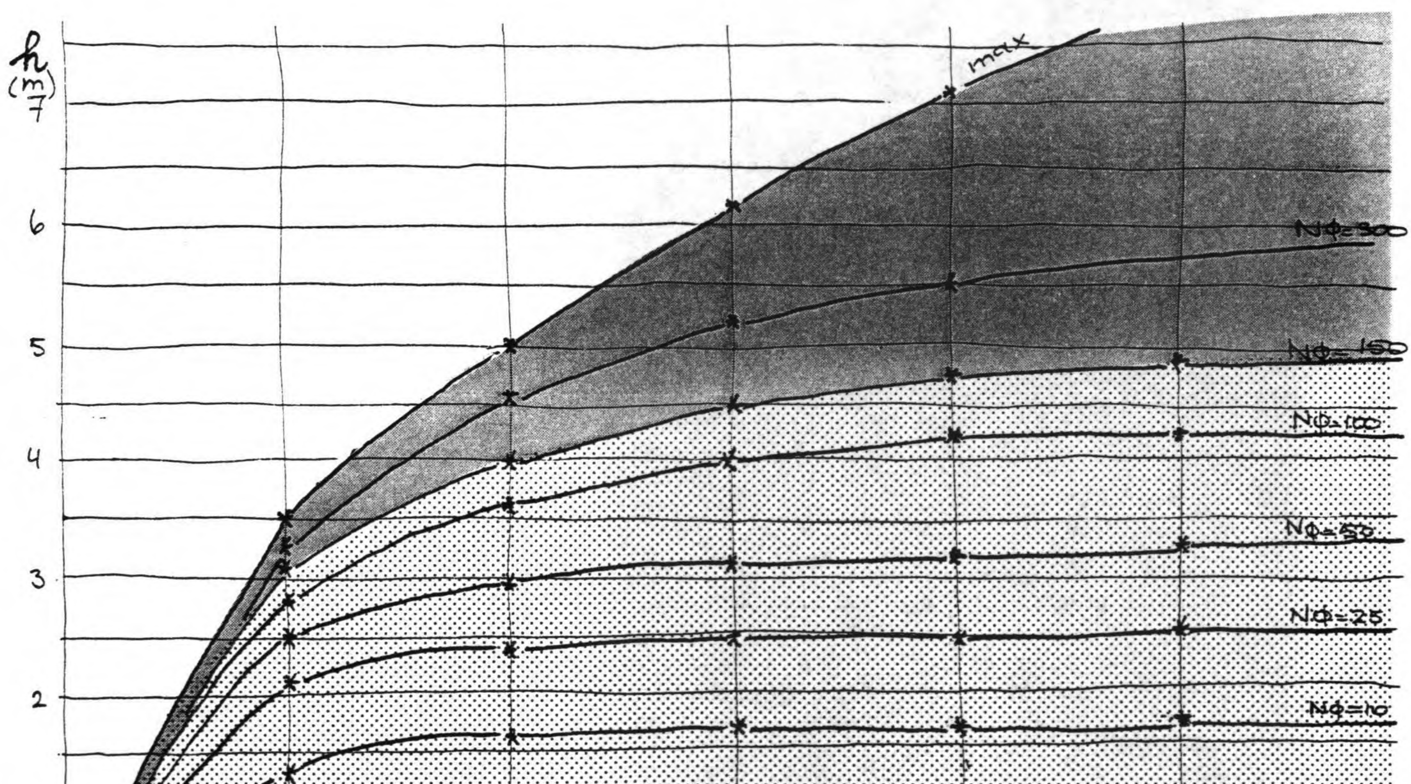
these equations continued until $\phi \simeq 140°$, then one can return to the original equations (9, 10);

4. the meridian ends at a point where $\phi = 180°$, the tank may be closed by a flat bottom in which the force $N\phi$ is introduced as a radial force.

The solutions for different values of $N\phi$, γ and h are given in the figures on page 16 etc.



a
z
h_o
φ
r



h (m)
7
6
5
4
3
2
1
b
10
20
30
40
50
A'(m³/m')
max
Nφ=300
Nφ=150
Nφ=100
Nφ=50
Nφ=25
Nφ=10

TUBE SHAPES

A drop shape is in principle a three-dimensional shape.
If we want to describe a tube shape, the meridians still exist, only the parallel circles become parallel lines, touching the original drop circles.
The tube can be described by a drop shape, cut into pieces and stretched out in x-direction.

However the relations between $N\phi$, $r\phi$, z and r still exists even when the defenition of r (2, 5) is not quite good anymore. r is no longer determined by $r\theta \sin\phi$, but the set of equations appropriate to solve a tube shape are still

$$\frac{d \sin\phi}{dr} + \frac{\sin\phi}{r} = \frac{\gamma z}{N\phi} \quad \dots\dots\dots\dots (7)$$

and

$$\frac{dz}{dr} = \tan\phi \quad \dots\dots\dots\dots (8)$$

(z, r: see <u>fig. 2A</u>).

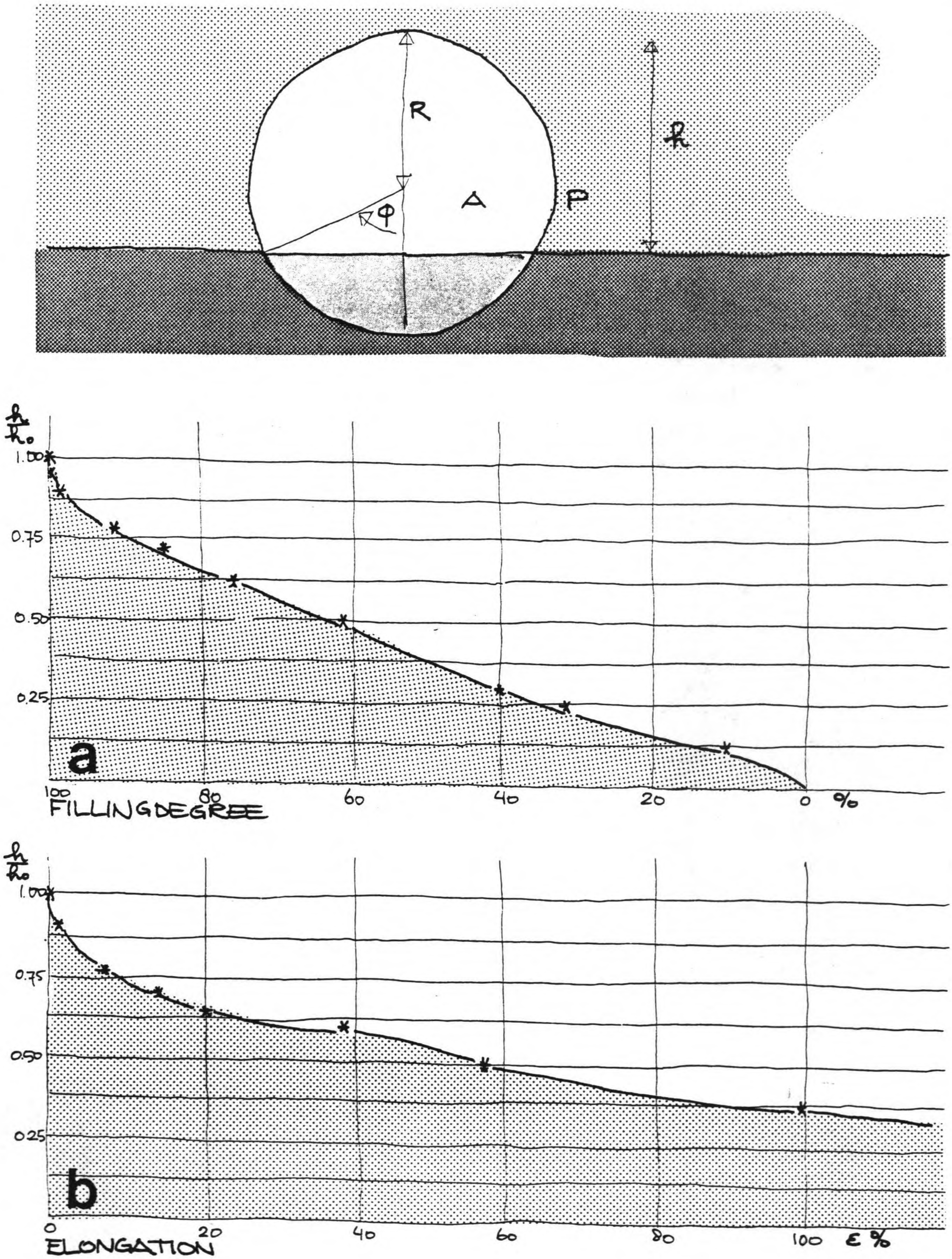
And the solution can be found according to the same method as described for drop shapes: the figures at page 16 also illustrate the meridian for tube shapes.

According to this method, a relation between $N\phi$ and h, and the content per m' of the tube can be found: (see <u>fig. 2B</u>).

Now the parameter γh describes the over pressure of the fill within the tube, caused by the tensile force in the geotextile (or membrane).

[(<u>fig. 4</u> etc. describe the forces etc. for γ = 18 kN/m3, the specific weight of a silt-water mixture as it can be found at the Cao Jin district).]

In order to find a relation between the height of a tube, and it's filling degree and elongation, several analogies can be tried.


R
h
A
P
φ
h/h₀
1.00
0.75
0.50
0.25
a
100
80
60
40
20
0 %
FILLINGDEGREE
b
0
20
40
60
80
100 ε %
ELONGATION

3

TUBE SHAPE: CIRCLE ANALOGY

For a perfect circle, the relation between content, height and circumpherence is simple. If we aproach a partly filled tube as a circle resting on the bottom with a flat surface, we can express content height and circumpherence as a function of the angle ϕ, which describes the angle of the flat surface with the vertical axis (see fig. 3) ($0 \leq \phi \leq \pi$).

Content:

$$A = \pi R^2 \left(1 - \frac{\phi}{\pi} + \frac{\sin\phi \cos\phi}{\pi}\right) \qquad (14)$$

Circumpherence:

$$P = 2\pi R \left(1 - \frac{\phi}{\pi} + \frac{\sin\phi}{\pi}\right) \qquad (15)$$

Height:

$$h = R(1 + \cos\phi) \qquad (16)$$

FILLING DEGREE:

if the circumpherence is taken constant (say P_o) and ϕ increases stepwise, the appropriate height and content can be calculated with (9, 10, 11).
This can be interpreted as the heigth of a tube having different filling degrees.

So for the filling degree it is taken:

$P_o = 2\pi R_o$
$h_o = 2R_o$.

The influence of increasing ϕ is given in the following table (see also fig. 3A).

FILLING DEGREE

$\frac{h}{ho}$	ϕ	$1 - \frac{\phi}{\pi} - \frac{\sin\phi}{\pi}$	$1 - \frac{\phi}{\pi} - \frac{\sin\phi\cos\phi}{\pi}$	$\frac{R}{R_o}$	$\frac{P}{2\pi R_o}$	$\frac{A}{\pi R_o^2}$	filling degree
1.000	0.000	1.000	1.000	1.000	1	1.000	100 %
0.994	0.157	1.500	0.999	1.000	1	0.999	99.9%
0.978	0.314	0.998	0.994	1.002	1	0.998	99.8%
0.950	0.471	0.995	0.979	1.005	1	0.988	98.8%
0.916	0.628	0.987	0.951	1.013	1	0.976	97.6%
0.876	0.785	0.975	0.909	1.026	1	0.956	95.6%
0.831	0.942	0.956	0.852	1.046	1	0.932	93.2%
0.779	1.099	0.934	0.779	1.071	1	0.893	89.3%
0.720	1.257	0.903	0.693	1.101	1	0.850	85.0%
0.669	1.414	0.864	0.599	1.157	1	0.802	80.2%

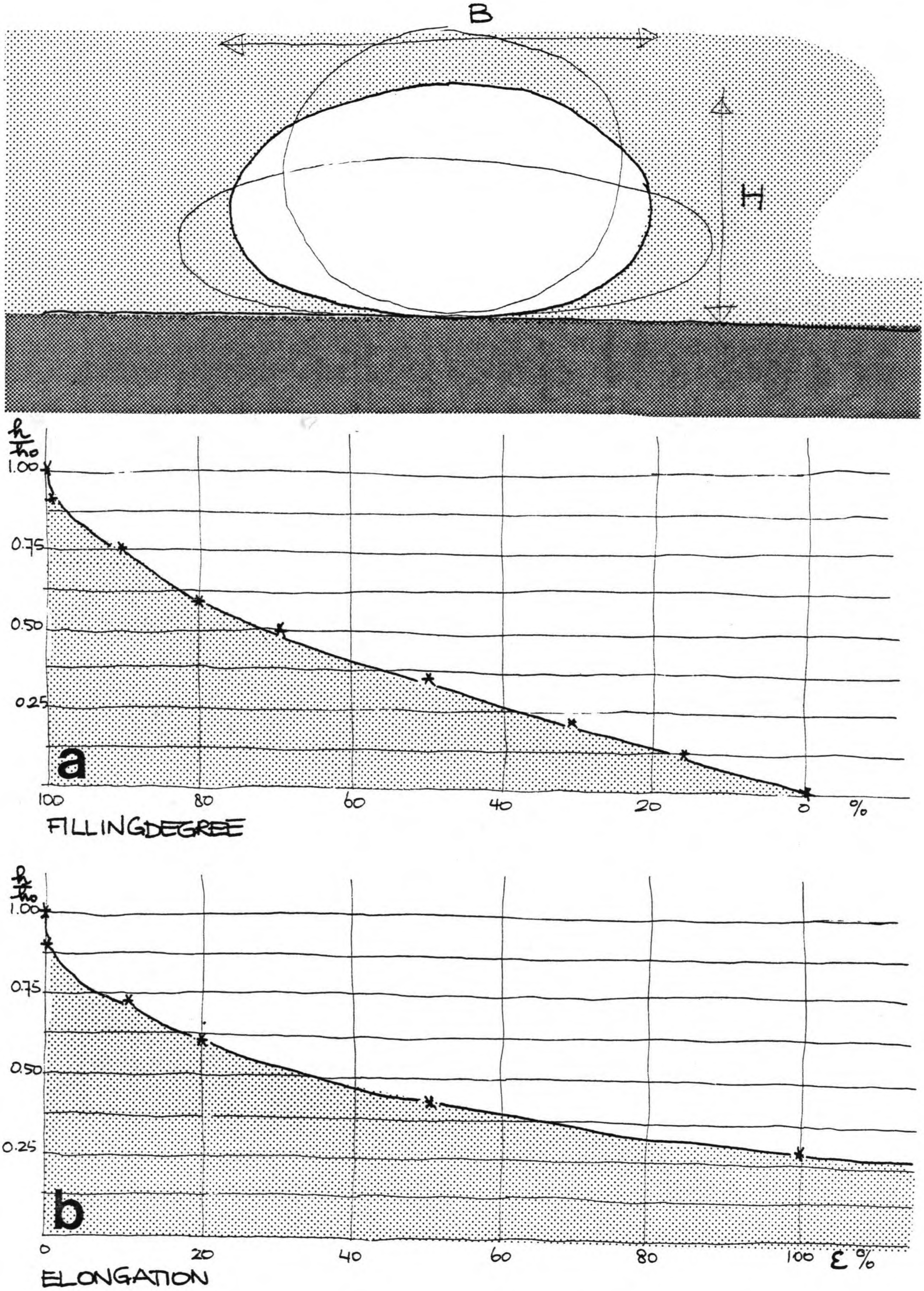
ELONGATION

$\frac{h}{h_o}$	ϕ	$1 - \frac{\phi}{\pi} - \frac{\sin\phi}{\pi}$	$1 - \frac{\phi}{\pi} - \frac{\sin\phi\cos\phi}{\pi}$	$\frac{R}{R_o}$	$\frac{A}{\pi R_o^2}$	$\frac{P}{2\pi R_o}$	elongation %
1.000	0.000	1.000	1.000	1.000	1	1.000	0.0
0.995	0.157	1.000	0.999	1.001	1	1.001	0.1
0.978	0.314	0.998	0.994	1.003	1	1.002	0.2
0.956	0.471	0.995	0.999	1.011	1	1.006	0.6
0.927	0.628	0.987	0.951	1.025	1	1.012	1.2
0.896	0.785	0.975	0.909	1.049	1	1.023	2.3
0.860	0.942	0.956	0.852	1.083	1	1.035	3.5
0.824	1.099	0.934	0.779	1.133	1	1.058	5.8
0.786	1.257	0.903	0.693	1.201	1	1.085	8.5
0.745	1.414	0.864	0.599	1.292	1	1.116	11.6

ELONGATION (continuation)

$\frac{h}{ho}$	ϕ	$1 - \frac{\phi}{\pi} - \frac{\sin\phi}{\pi}$	$1 - \frac{\phi}{\pi} - \frac{\sin\phi\cos\phi}{\pi}$	$\frac{R}{R_o}$	$\frac{A}{\pi R_o^2}$	$\frac{P}{2\pi R_o}$	elongation %
0.707	1.571	0.818	0.500	1.414	1	1.157	15.7
0.666	1.728	0.764	0.401	1.579	1	1.206	20.6
0.694	1.805	0.704	0.306	1.808	1	1.273	27.3
0.594	2.042	0.634	0.221	2.177	1	1.380	38.0
0.534	2.199	0.558	0.149	2.591	1	1.446	44.6
0.486	2.356	0.475	0.091	3.315	1	1.575	57.5
0.431	2.513	0.387	0.049	4.517	1	1.748	74.8
0.377	2.670	0.295	0.021	6.901	1	2.036	104
0.316	2.827	0.199	0.006	12.91	1	2.569	157
0.193	2.985	0.099	0.001	31.62	1	3.130	213
0	3.141	0.000	0.000	∞	1	∞	∞

according to circle analogy


B
H
h/ho
1.00
0.75
0.50
0.25
a
100
80
60
40
20
0
%
FILLINGDEGREE
h/ho
1.00
0.75
0.50
0.25
b
0
20
40
60
80
100
ε %
ELONGATION

4

The same philosophy can be followed to determine the influence of the elongation of the membrane:

ELONGATION:

the content is taken constant, this time ϕ increases stepwise, so the circumference has to increase also if the same content is reached with a lower height.

$A_o = \pi R_o^2$
$h_o = 2R_o$

(see fig. 3B), the tabel on page 10 shows the calculation according to (9, 10, 11).

Another analogy can be, to suppose that the shape of the tube remains an ellipse.

TUBE SHAPE: ELLIPSE ANALOGY

For an ellipse, the relation between content, height and circumference is a bit more complicated to determine, the variable will be the height of the ellipse: (see lit.(19)).

Content

$$A = \tfrac{1}{4}\pi BH \qquad (17)$$

Circumference

$$P = \tfrac{1}{2}\pi(B + H) \cdot \frac{64 - 3\left(\frac{B - H}{B + H}\right)^4}{64 - 16\left(\frac{B - H}{B + H}\right)^2} \qquad (18)$$

Height

$$h = H. \qquad (19)$$

The influence of filling degree (take P constant) and elongation (take A constant for decreasing h) can be found following the same method as for the circle analogy.
The results are shown on page 14 and in fig. 4A.

FILLING DEGREE

$\frac{h}{h_o}$	$\frac{B}{h_o}$	$\frac{B}{H}$	factor	$\frac{P}{\pi h_o}$	$\frac{A}{\frac{1}{4}\pi h_o^2}$	filling %
1.000	1.000	1.000	1.000	1	1.000	100 %
0.950	1.049	1.104	1.001	1	0.996	99.6%
0.900	1.096	1.218	1.002	1	0.986	98.6%
0.850	1.140	1.341	1.005	1	0.969	96.9%
0.800	1.182	1.478	1.009	1	0.946	94.6%
0.750	1.222	1.629	1.014	1	0.916	91.6%
0.700	1.260	1.803	1.021	1	0.882	88.2%
0.650	1.296	1.994	1.028	1	0.843	84.3%
0.600	1.331	2.218	1.036	1	0.798	79.8%
0.550	1.363	2.478	1.046	1	0.750	75.0%
0.500	1.393	2.786	1.056	1	0.696	69.6%
0.450	1.422	3.160	1.069	1	0.640	64.0%
0.400	1.448	3.620	1.082	1	0.579	57.9%
0.350	1.473	4.208	1.097	1	0.515	51.5%
0.300	1.495	4.983	1.114	1	0.448	44.8%
0.250	1.515	6.060	1.133	1	0.379	37.9%
0.200	1.533	7.665	1.154	1	0.307	30.7%
0.150	1.548	10.32	1.178	1	0.232	23.2%
0.100	1.500	15.00	1.205	1	0.156	15.6%
0.050	1.569	31.38	1.236	1	0.078	7.8%
0.00	1.571	∞	1.271	1	0.000	0 %

according to ellips analogy

ELONGATION

$\frac{h}{h_o}$	$\frac{B}{h_o}$	$\frac{B}{H}$	factor	$\frac{A}{\frac{1}{4}\pi h_o^2}$	$\frac{P}{\pi h_o}$	elongation %
1.000	1.000	1.000	1.000	1	1.000	0.0
0.950	1.053	1.108	1.001	1	1.002	0.2
0.900	1.111	1.234	1.003	1	1.008	0.8
0.850	1.176	1.383	1.007	1	1.020	2.0
0.800	1.250	1.563	1.012	1	1.037	3.7
0.750	1.330	1.773	1.020	1	1.062	6.2
0.700	1.429	2.041	1.029	1	1.096	9.6
0.650	1.538	2.366	1.042	1	1.140	14.0
0.600	1.667	2.778	1.056	1	1.197	19.7
0.550	1.818	3.305	1.073	1	1.271	27.1
0.500	2.000	4.000	1.092	1	1.365	36.5
0.450	2.222	4.938	1.113	1	1.487	48.7
0.400	2.500	6.250	1.136	1	1.647	64.7
0.350	2.857	8.163	1.175	1	1.884	88.4
0.300	3.333	11.11	1.183	1	2.150	115.0
0.250	4.000	16.00	1.206	1	2.560	156.0
0.200	5.000	25.00	1.227	1	3.190	219.0
0.150	6.667	44.45	1.245	1	4.240	324.0
0.100	10.000	100.0	1.259	1	6.340	534.0
0.050	20.00	400.0	1.268	1	12.71	1,171.0
0.000	∞	∞	1.271	1	∞	∞

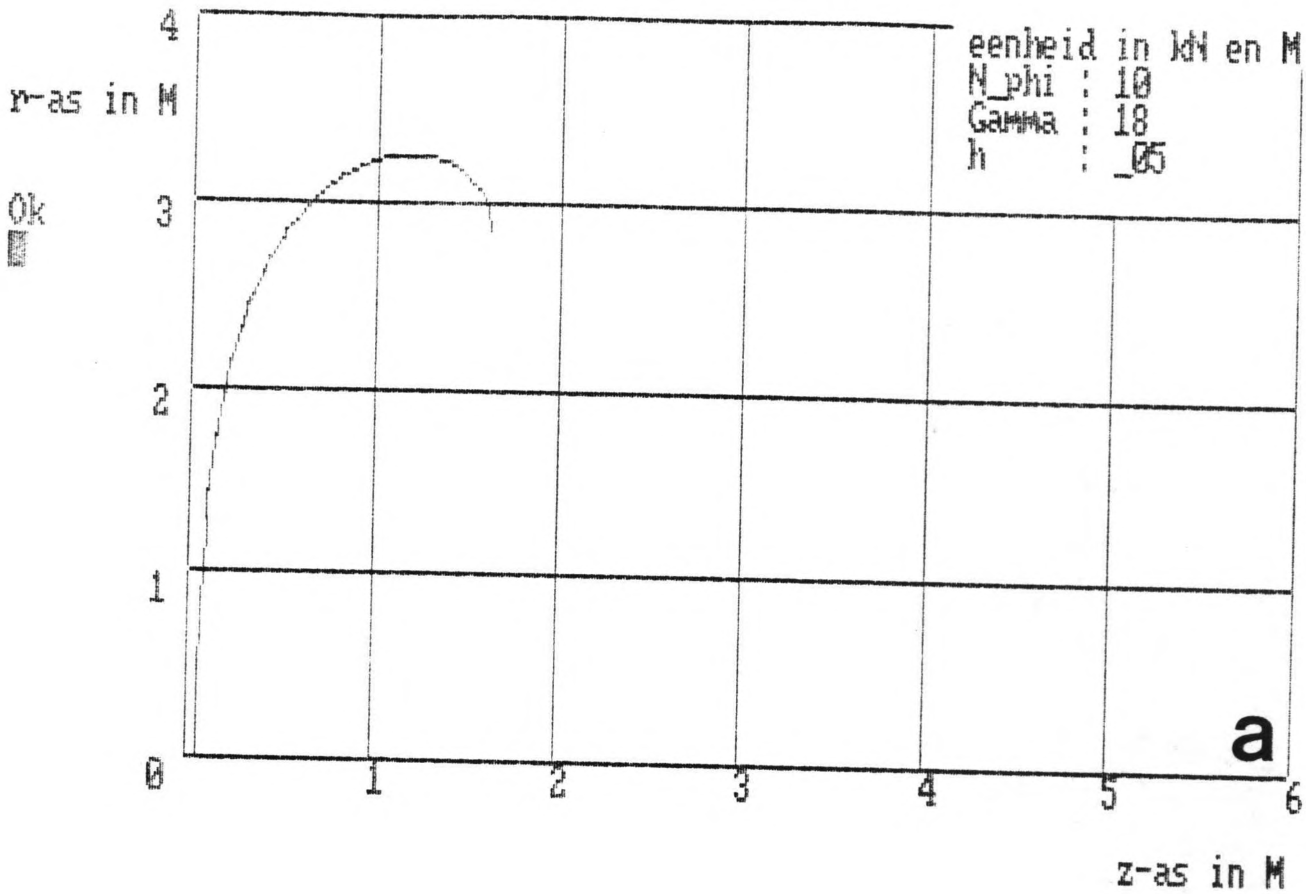
according to ellips analogy

The two approaches give more or less the same results. It is expected that the drop shape shows the same behaviour with respect to filling degree and elongation. Some comparison with the computer model results have shown that in the upper filling-degree-reach (bigger than 80%) the actual drop shape results in somewhat higher tubes. The difference is biggest at very high filling degree: a tube of 95% filling is, according to the drop shape, almost as high as a tube of 100% filling.

As in practice the filling degree will not be much more than 80%, the ellips analogy satisfies to determine the relation between height and circumpherence (or filling degree).
The subsequent tensile force can than be determined by the drop analogy, as described before and shown in fig. 2b.

In practice, an empty tube diameter will be chosen, based on economical reasons, or based on the width of the cloth knitting machine (in the order of 5 m), so integer numbers of this width will be taken to form tubes of.
Then the tubes are filled with pumps, these pumps inflate the tube with a mixture of silt and water resulting in a certain height of the tube. The height of the tube can be checked by means of checking the volume of filling that has been pumped into the tube (the filling degree determines the tensile force in the geotextile that surrounds the fill. (Also the tensile force is a measure for the height of the tube.)
It seems that the filling degree is an easier parameter to check than the tensile force (the filling can be spread unequally, local sewing-inaccuracies can also cause inaccuracies in the tensile force).

Concluding it can be said that the filling degree determines the eventual height of the tube, the filling degree therefore will be the main requirement with respect to the height of the dike.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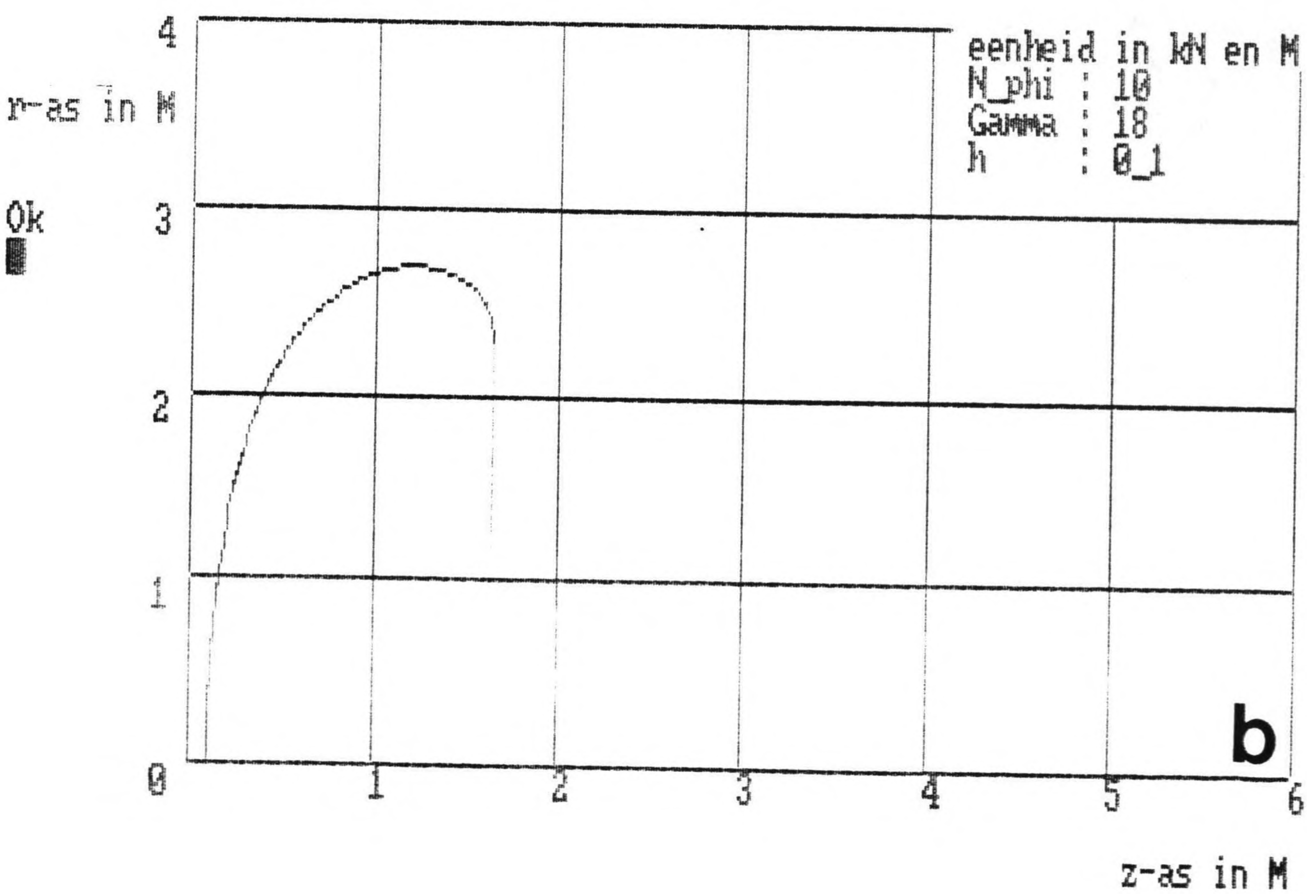
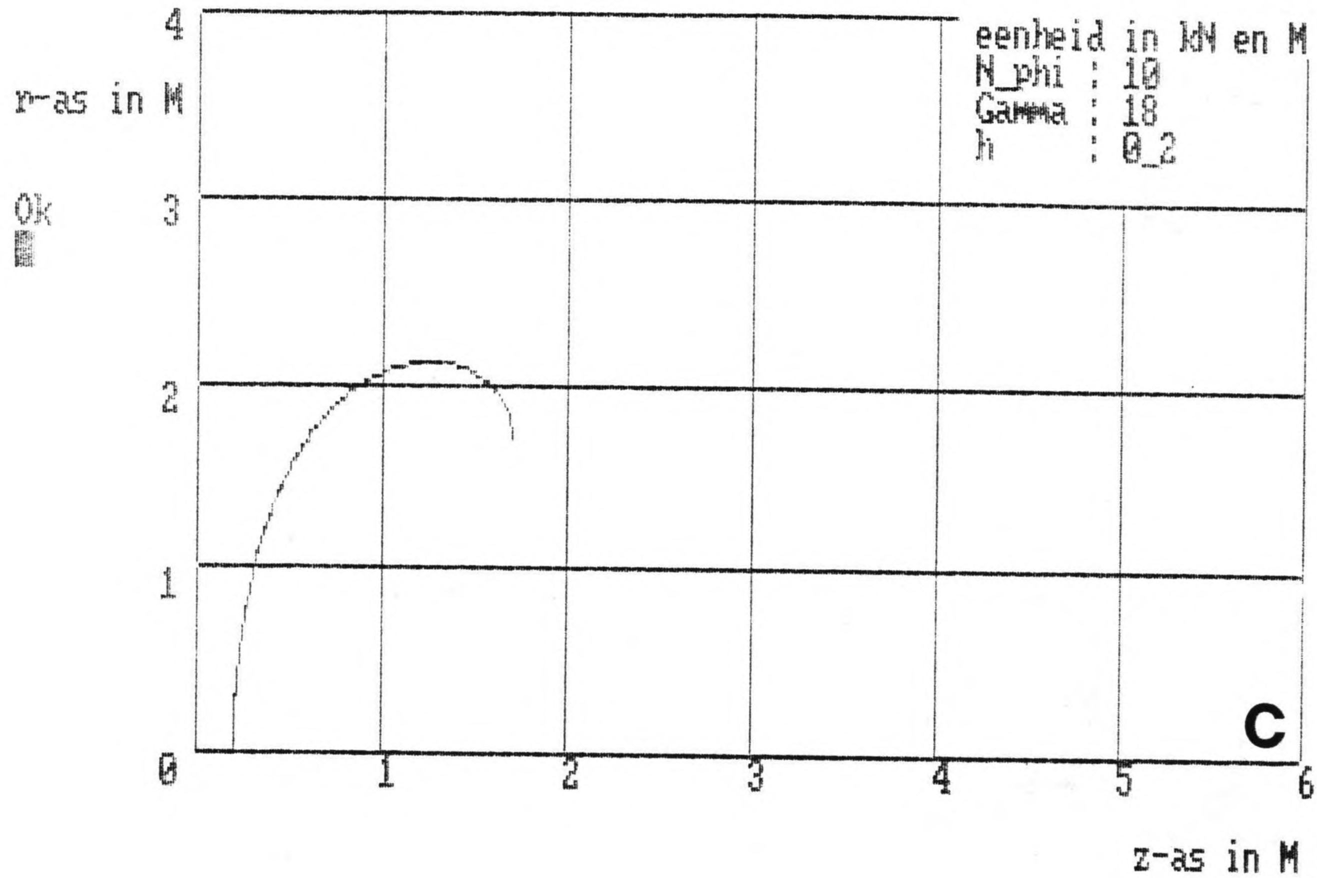
Ok
eenheid in kN en M
N_phi : 10
Gamma : 18
h : _05
4
3
2
1
0
1
2
3
4
5
6
z-as in M
a


r-as in M
Ok
eenheid in kN en M
N_phi : 10
Gamma : 18
h : 0_1
4
3
2
1
0
1
2
3
4
5
6
z-as in M
b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N_phi : 10
Gamma : 18
h : 0_2
r-as in M
Ok
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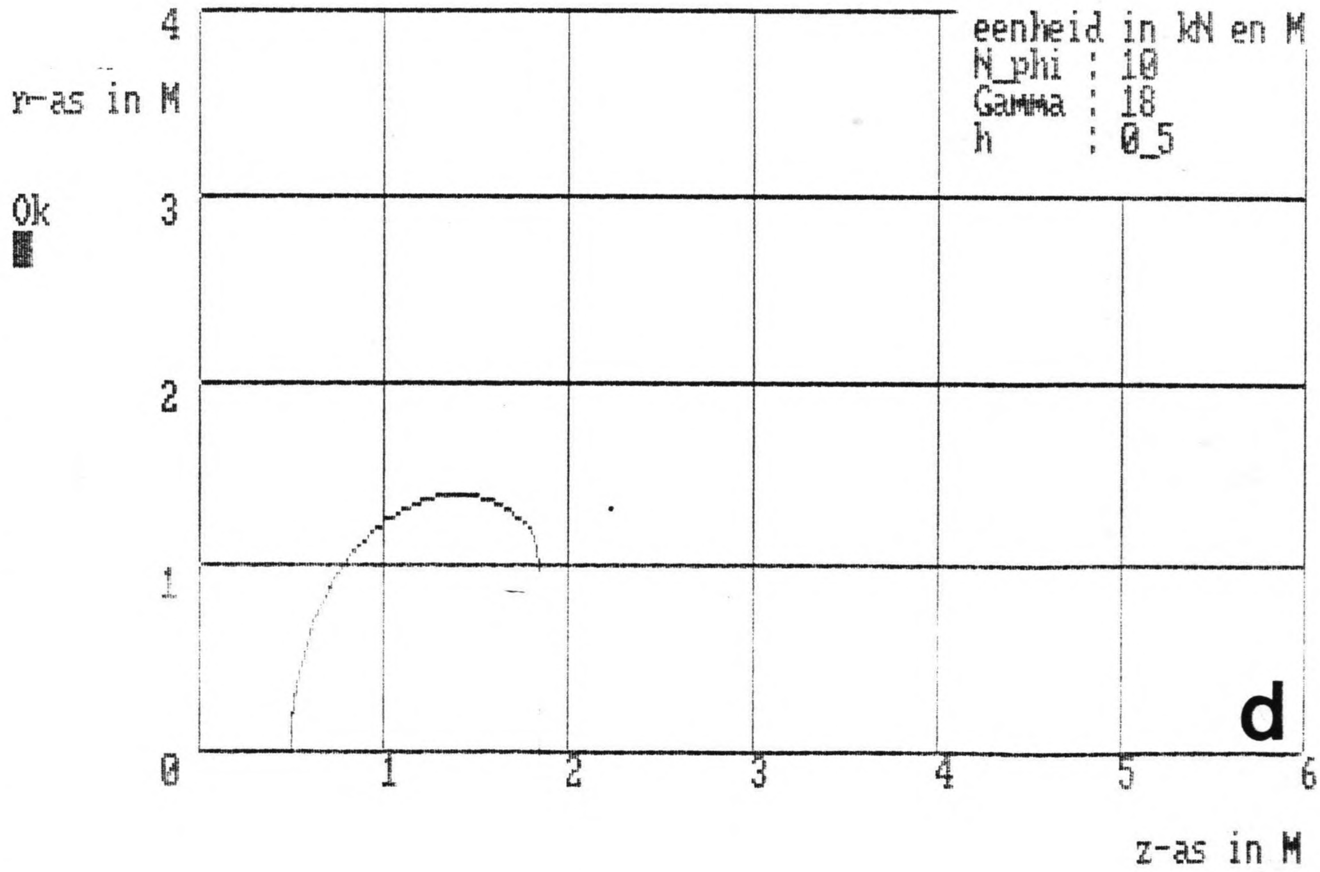
3
2
1
0
1
2
3
4
5
6
z-as in M
c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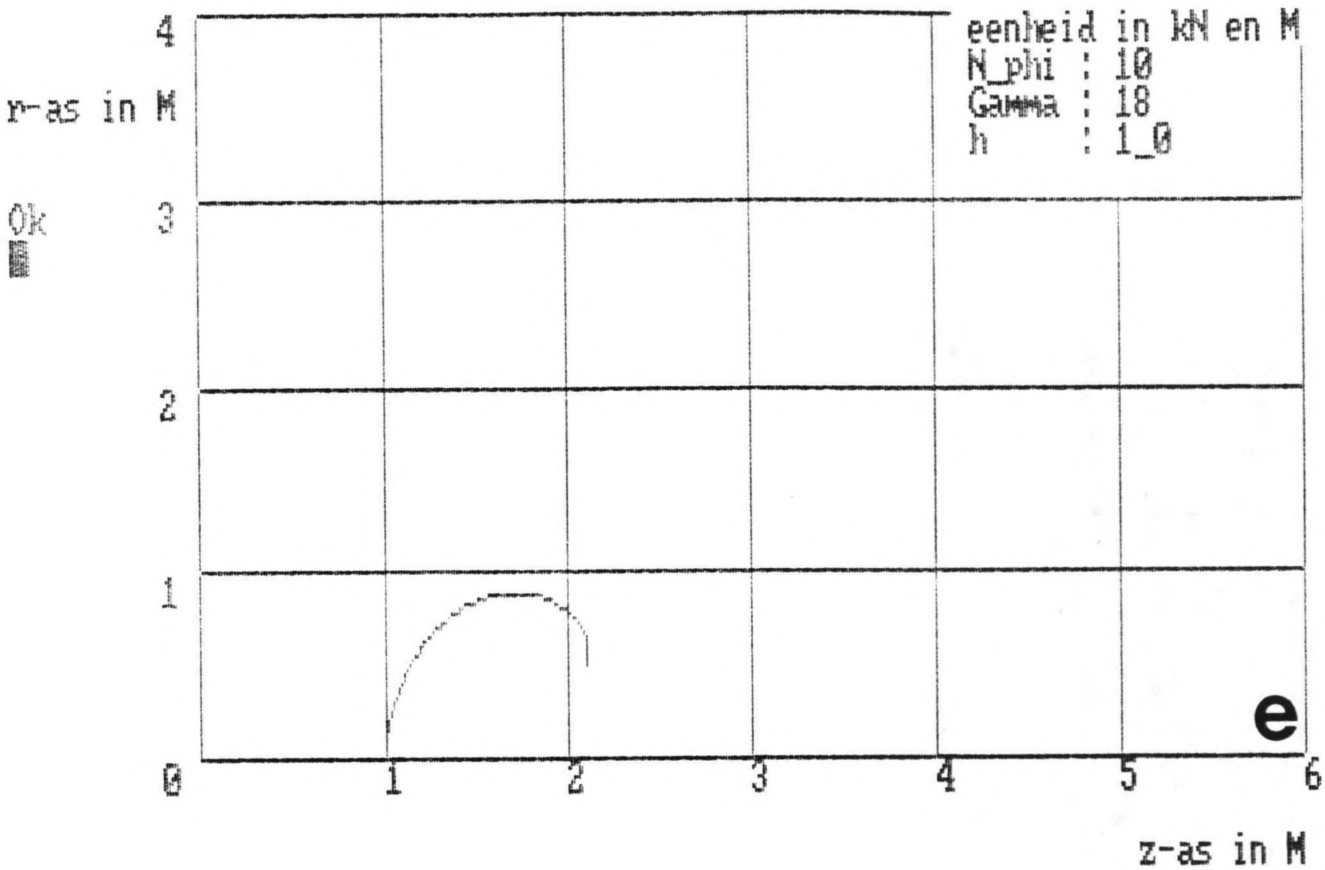
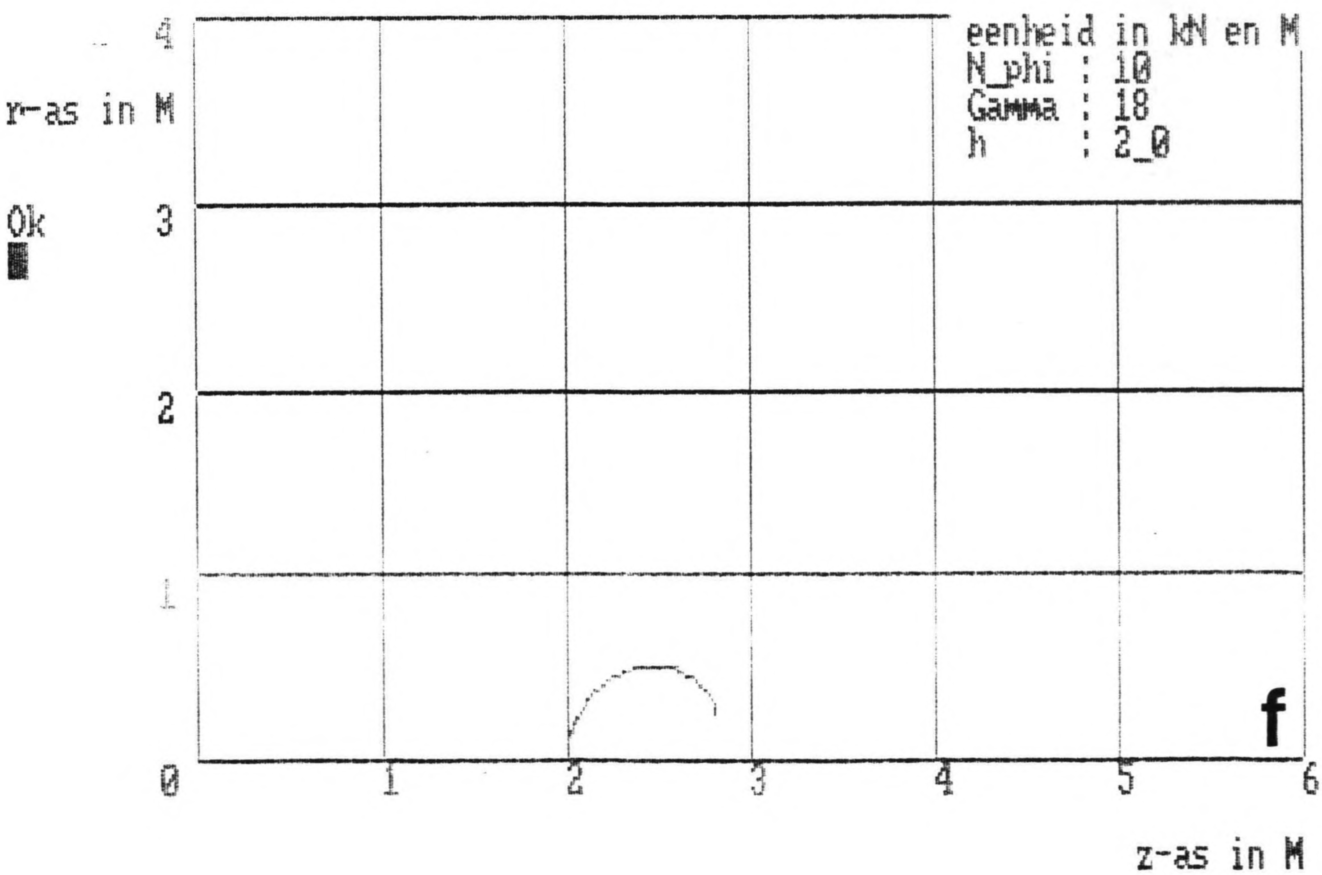
N_phi : 10
Gamma : 18
h : 0_5
r-as in M
Ok
4
3
2
1
0
1
2
3
4
5
6
z-as in M
d



eenheid in kN en M
N_phi : 10
Gamma : 18
h : 1_0
r-as in M
Ok
z-as in M
0
1
2
3
4
5
6
e



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N_phi : 10
Gamma : 18
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r-as in M
Ok
z-as in M
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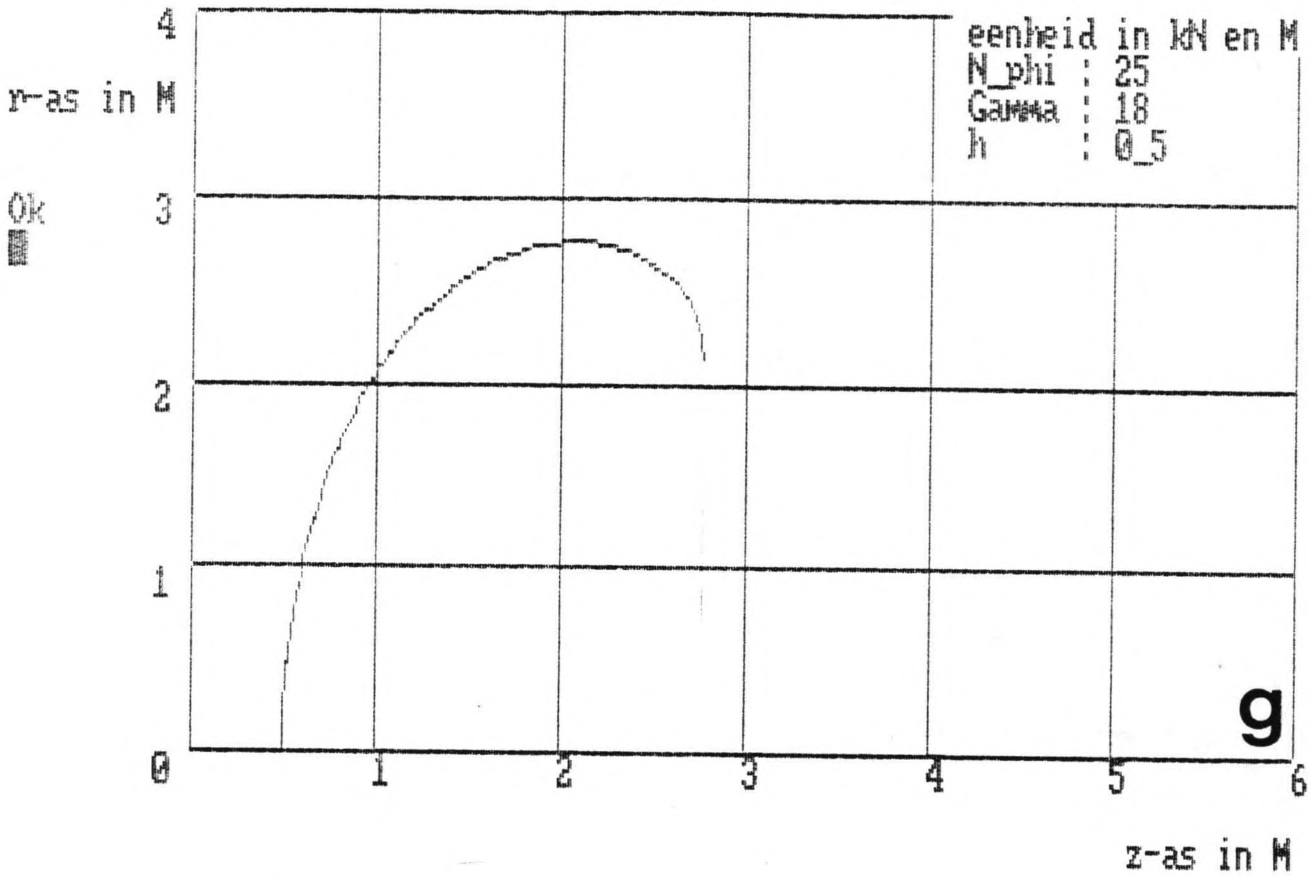
4
5
6
f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N_phi : 25
Gamma : 18
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r-as in M
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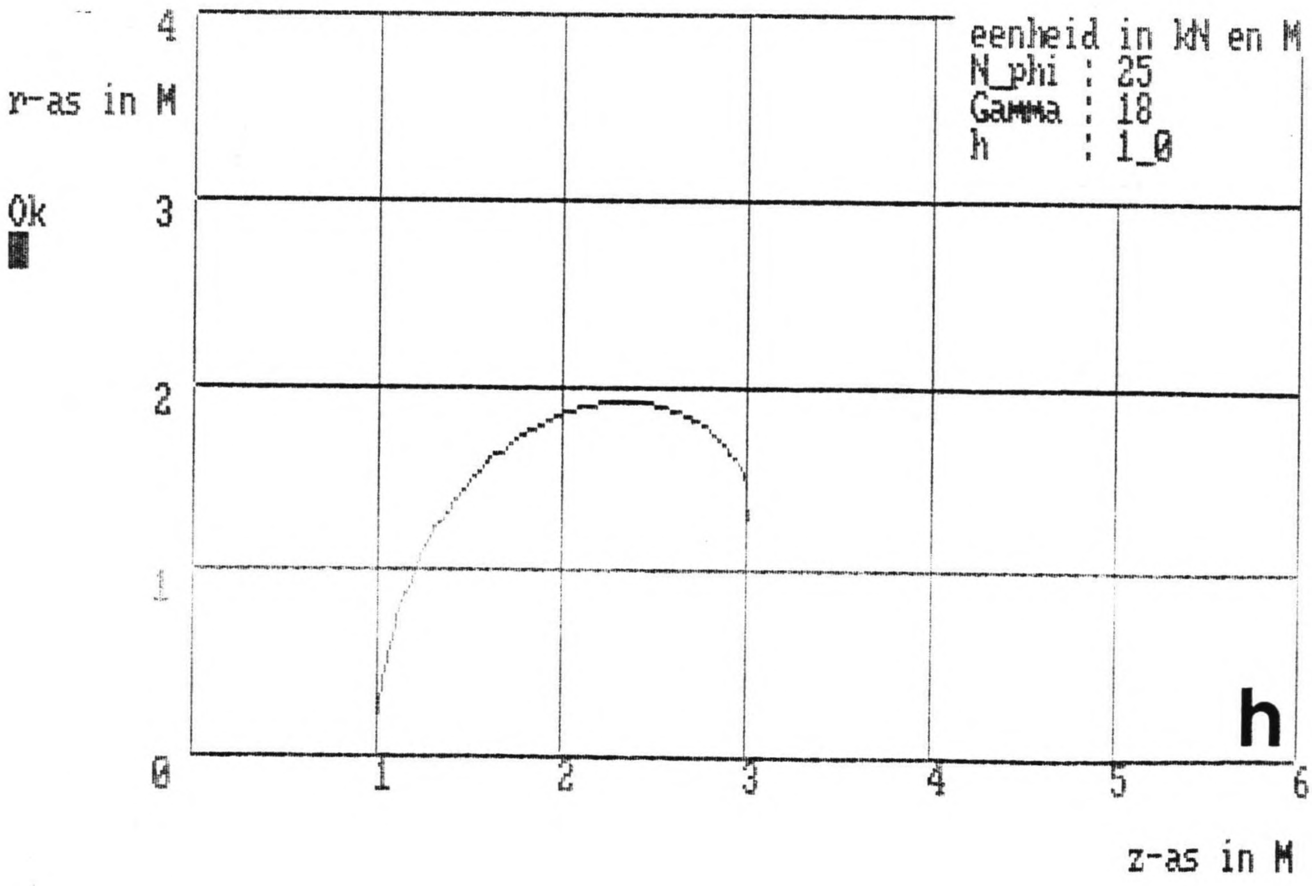
0
1
2
3
4
5
6
z-as in M
g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N_phi : 25
Gamma : 18
h : 1_0
r-as in M
Ok
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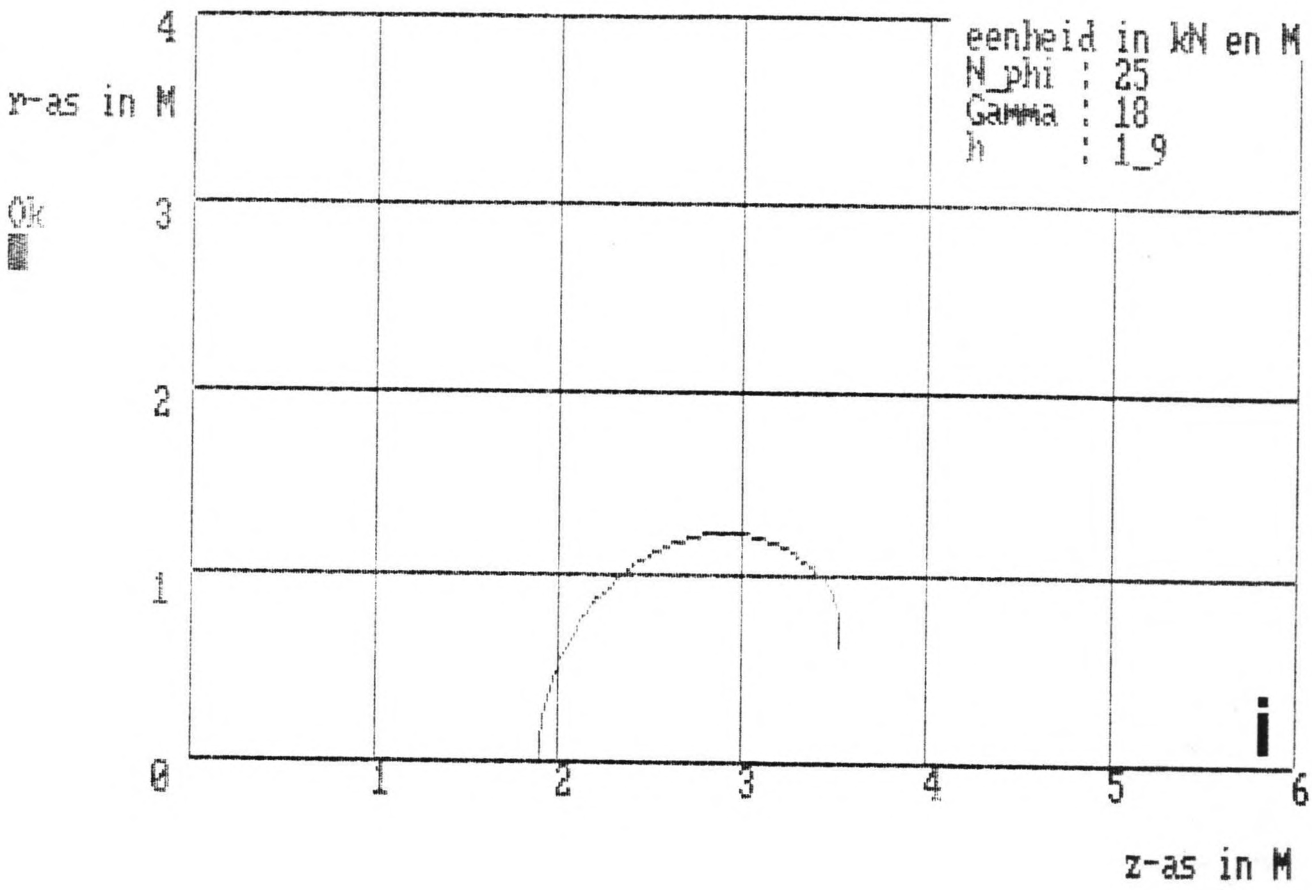
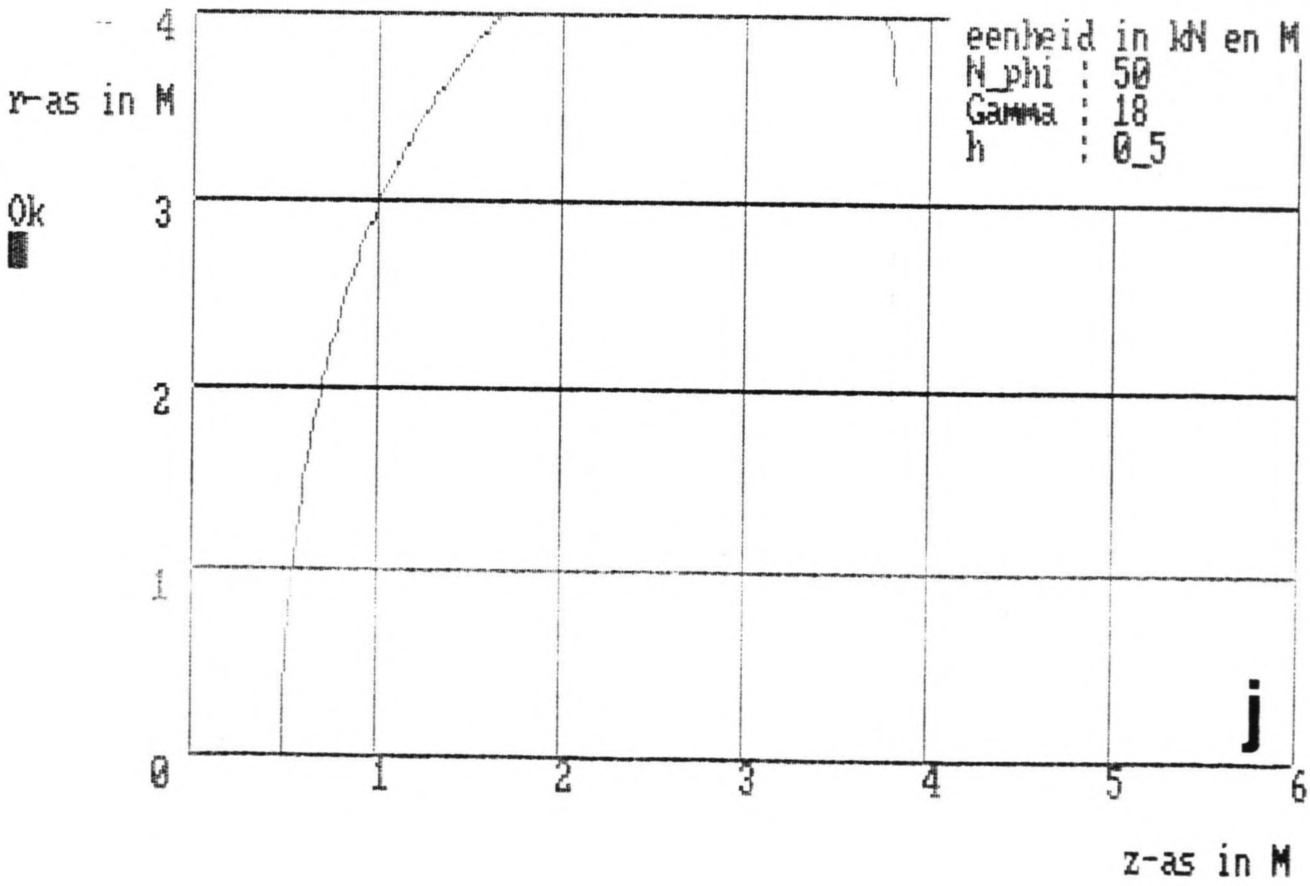
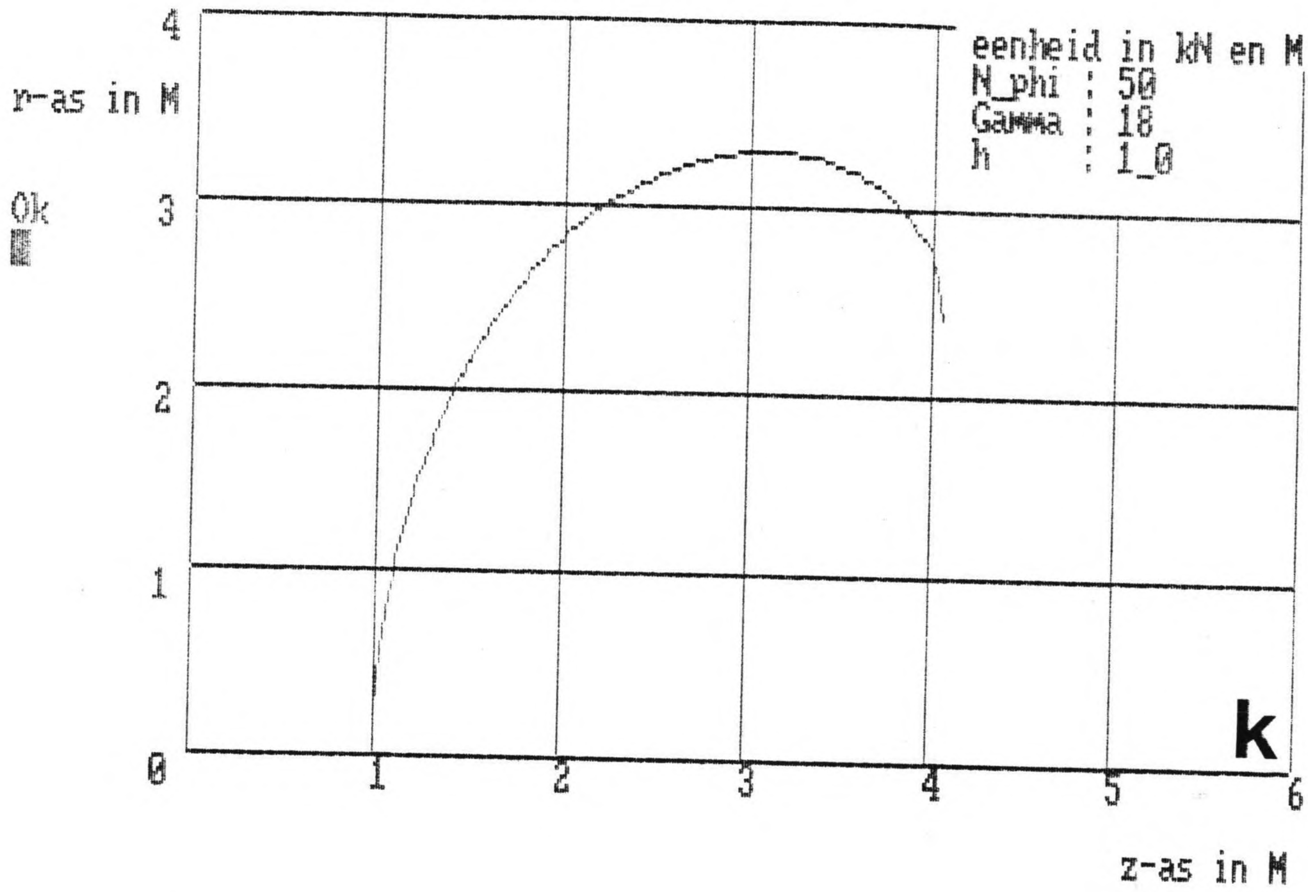
2
3
4
5
6
z-as in M
h



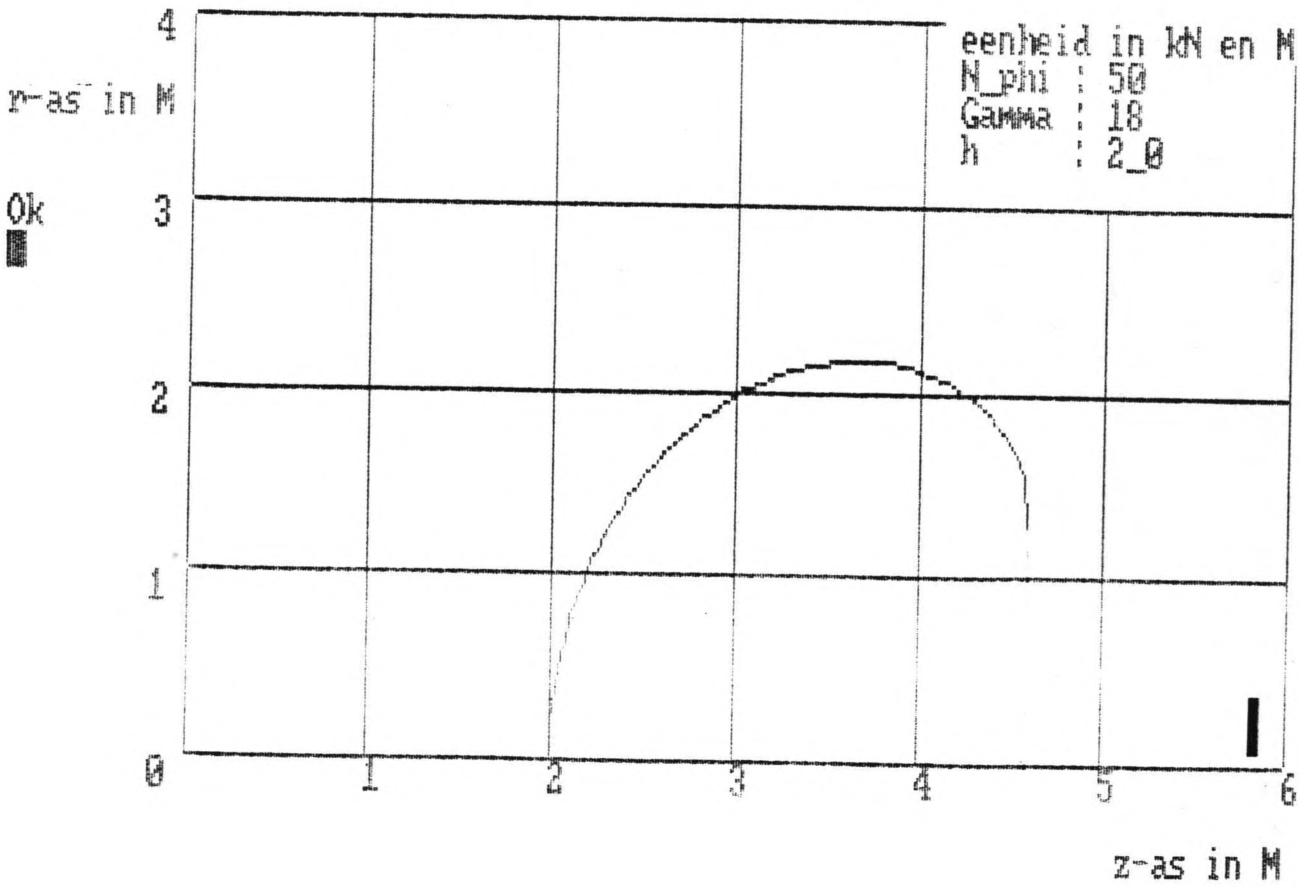
eenheid in kN en M
N_phi : 25
Gamma : 18
h : 1_9
r-as in M
Ok
0
1
2
3
4
5
6
z-as in M
i



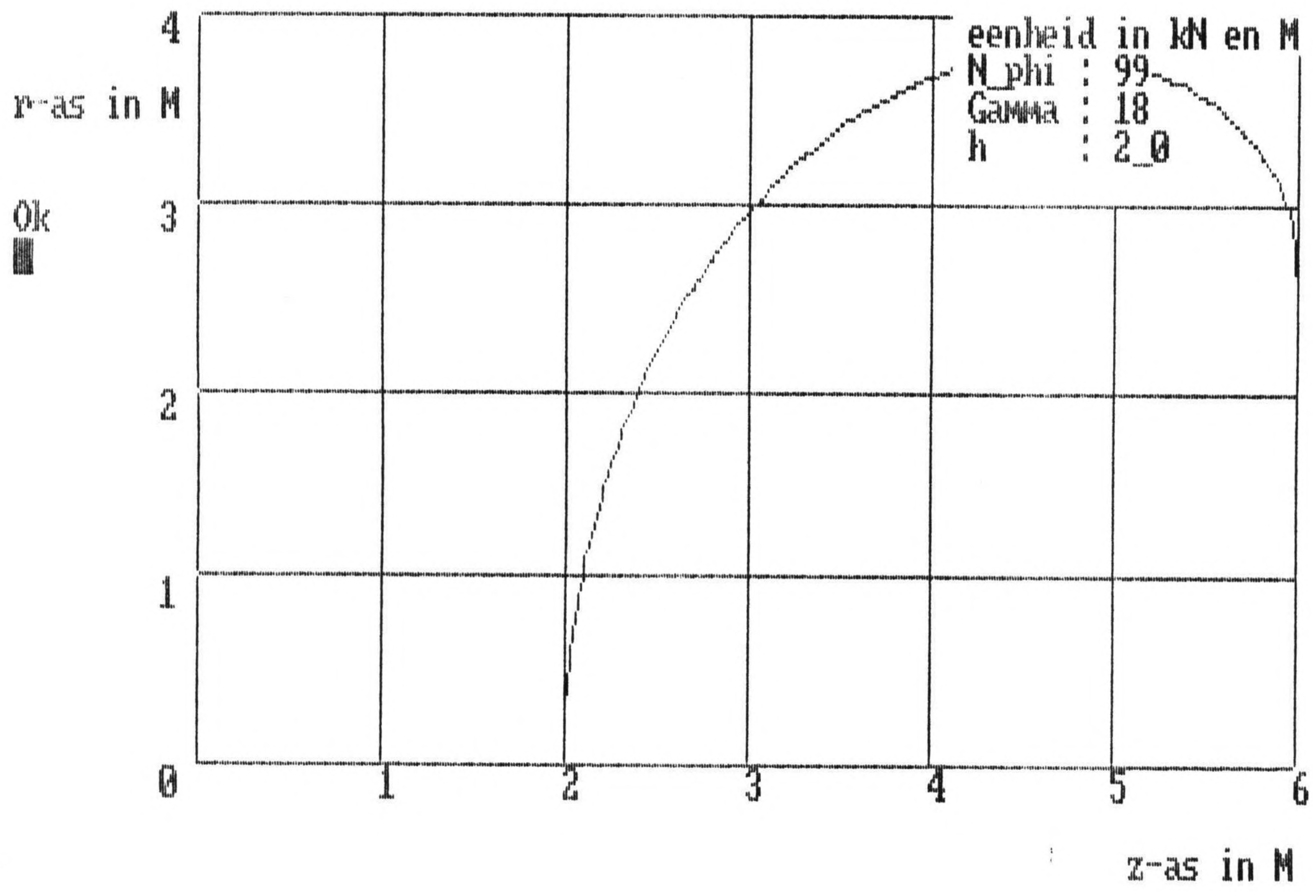
eenheid in kN en M
N_phi : 50
Gamma : 18
h : 0_5
r-as in M
Ok
0
1
2
3
4
5
6
z-as in M
j



r-as in M
Ok
eenheid in kN en M
N_phi : 50
Gamma : 18
h : 1_0
0
1
2
3
4
5
6
z-as in M
k



r-as in M
Ok
eenheid in kN en M
N_phi : 50
Gamma : 18
h : 2_0
0
1
2
3
4
5
6
z-as in M
l



eenheid in kN en M
N_phi : 99
Gamma : 18
h : 2_0
r-as in M
Ok
0
1
2
3
4
5
6
z-as in M

m

APPENDIX B: DYNAMIC RESPONSE

In this appendix, it is discussed the mechanical schemes which have been used to determine the dynamic response of the construction to wave-loadings.

First the basic concepts of dynamics are treated, and the elementary solution of the one-spring-mass system. This is the most simple system which contains all the elements of dynamics. A distinction can be made between (par. B.1):

I. the one-spring-mass system, undamped; and
II. the damped one-spring-mass system.

Next the actual wave-loadings are determined, since the specific wave-loadings are dependent on the response of the construction (par. B.2).

In the last paragraph, several schematizations of the construction are discussed. The response of the construction is calculated on the basis of these schemes. A response consists of two basic movements which are mutually coupled: rotation and translation.
It is discussed (par. B.3):

I. the response of the subsoil;
II. the response of the construction itself;
III. the finite-element method.

On the basis of this analysis, some conclusions can be drawn. These conclusions have already been mentioned in par. 3.2; the basis on which they have been calculated can be found in this appendix.

B.1 Theory

General

The response of a body to an externally imposed force is in fact the motion that each separate point, that belongs to the body, describes due to the force.

The motion of a point can be described by three parameters:

- the location (or coordinate): at each point of time — x, y, z at point t (referring to some fixed point)
- the velocity (or speed) at each point of time: — $\dot{x}$, $\dot{y}$, $\dot{z}$ at point t

  the velocity is the first derivative of the location with respect to time

  $$\dot{x} = \frac{dx}{dt}, \quad \dot{y} = \frac{dy}{dt}, \quad \dot{z} = \frac{dz}{dt}$$

- the acceleration at each point of time: — $\ddot{x}$, $\ddot{y}$, $\ddot{z}$ at point t

  the acceleration is the second derivative with respect to time

  $$\ddot{x} = \frac{d^2x}{dt^2}, \quad \ddot{y} = \frac{d^2y}{dt^2}, \quad \ddot{z} = \frac{d^2z}{dt^2}$$

If each parameter is known at each time point the motion of the body is also known. Of course the calculation of the motion of each point belonging to the body (in the membrane fibre) is not possible in practice. Responses are calculated as large scale motions of elements of the construction, and the response of each element to a load is known beforehand. So the response of the body (or construction) is a summation of the responses of the elements.

In dynamics, three basic elements can be distinguished: (the characteristics here are expressed only in one dimension, in reality it comprised three dimensions) (see fig. 1).

1. THE MASS ELEMENT: if a force is introduced on a mass element, it reacts according to Newton's law of inertia,

   parameter: m [kg] — the shape of the mass does not change

   m = mass of the body

   $\vec{F} = m\ddot{\vec{x}}$ .... (1)

   F = external force [N]
   $\ddot{x}$ = acceleration of the body [m/s^2].

   The energy that is stored during the motion by inertia, and the power

   $E = \frac{1}{2}m\dot{x}^2$ .... (1A)
   $P = Fx$ .... (1B)

2. THE SPRING ELEMENT: if a force is introduced on a spring element, it is deformed immediately, when the force is taken away, the element will take its initial shape again.

   parameter: k [N/m]

   k = spring-coefficicient or elasticity of the body

   $\vec{F} = k.\Delta x$ .... (2)

   F = external force [N]
   Δx = contraction (if elongation occurs the force is negative) [m]

   The energy that is stored into the spring, and the power:

   $E = \frac{1}{2}k(\Delta x)^2$.......................... (2A)
   $P = F(\Delta x)$ .... (2B)

3. THE FRICTION ELEMENT: if a force is introduced on a friction element, it is transformed to heat and the force energy is dissipated:

   parameter: c [Ns/m]

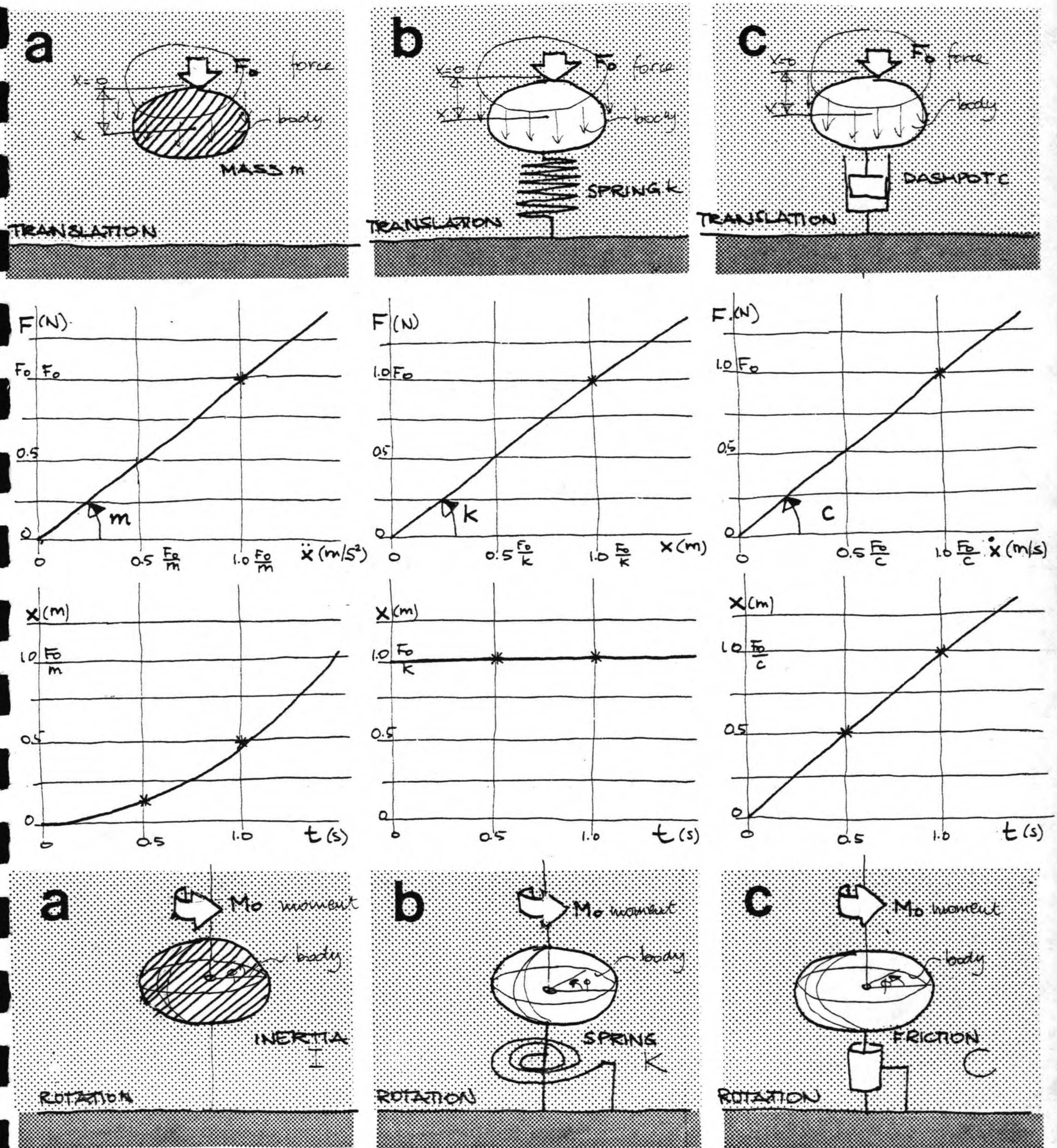
   $\vec{F} = c\,\dot{x}$ .... (3)

   c = friction constant or damping constant

   F = external force [N]
   x = velocity of the body [m/s]

   The energy that is stored into the body by friction is always zero (it is dissipated), the power:

   $E = -$ .... (3A)
   $P = c\dot{x}^2$ .... (3B)


a
F_o force
x=0
x
body
MASS m
TRANSLATION
b
F_o force
x=0
x
body
SPRING k
TRANSLATION
c
F_o force
x=0
x
body
DASHPOT c
TRANSLATION
F (N)
F_o F_o
0.5
0
m
0
0.5 $\frac{F_o}{m}$
1.0 $\frac{F_o}{m}$
$\ddot{x}$ (m/s²)
F (N)
1.0 F_o
0.5
0
k
0
0.5 $\frac{F_o}{k}$
1.0 $\frac{F_o}{k}$
x (m)
F (N)
1.0 F_o
0.5
0
c
0
0.5 $\frac{F_o}{c}$
1.0 $\frac{F_o}{c}$
$\dot{x}$ (m/s)
x (m)
1.0 $\frac{F_o}{m}$
0.5
0
0
0.5
1.0
t (s)
x (m)
1.0 $\frac{F_o}{k}$
0.5
0
0
0.5
1.0
t (s)
x (m)
1.0 $\frac{F_o}{c}$
0.5
0
0
0.5
1.0
t (s)
a
Mo moment
body
INERTIA
I
ROTATION
b
Mo moment
body
φ
SPRING
K
ROTATION
c
Mo moment
body
φ
FRICTION
C
ROTATION

1

With these three elements, a system can be described, resulting in a differential equation, which describes the response of the system, and that can be solved by means of appropriate boundary conditions, and initial conditions.

Analogue to the motion elements, rotation elements can be introduced (see fig. 1) in which the external force F is replaced by an external moment T, and the rotation is described by the angle ϕ, the rotation-velocity around the axis ϕ (or ω) and the rotation accelaration ϕ.

4. INERTIA ELEMENT: $T = I\phi$ .... (4)

parameter I [kgm2]
- I = inertia to rotation [kgm2]
- T = external moment [Nm]
- ϕ = angular acceleration [rad/s^2]

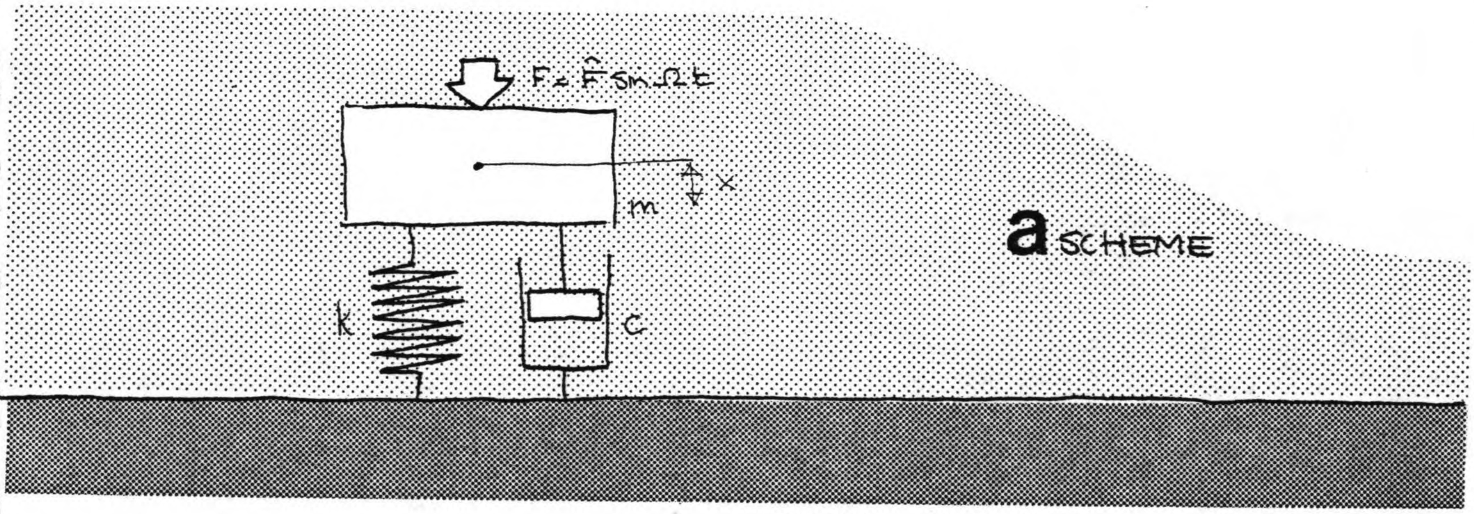
5. SPRING ELEMENT: $T = K\Delta\phi$ .... (5)

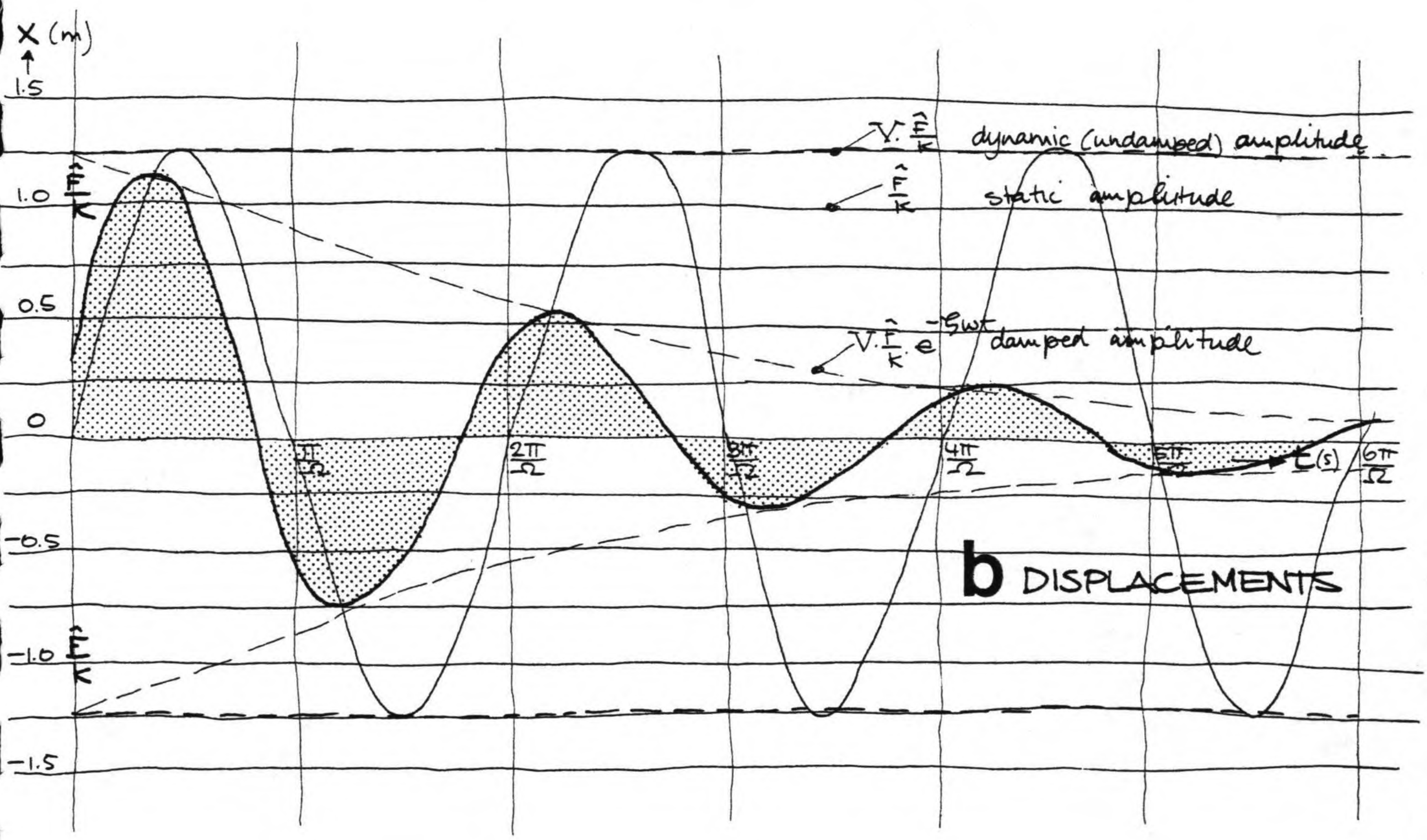
parameter K [Nm/rad]
- K = spring coefficient of rotation [Nm/rad]
- T = external moment [Nm]
- $\Delta\phi$ = angular contraction or elongation [rad]

6. FRICTION ELEMENT: $T = C\phi$ .... (6)

parameter: C [Nms/rad]
- C = friction coefficient [Nms/rad]
- T = external moment [Nm]
- ϕ = velocity of rotation [rad/s]

When using these 6 basic elements, every response of a structure can be predicted. Furthermore the response can be devided in a translation calculation (elements 1, 2, 3) and a rotation calculation (elements 4, 5, 6).


F = F̂ sin Ωt
x
m
k
c
a SCHEME


x (m)
1.5
1.0 F̂/k
0.5
0
-0.5
-1.0 F̂/k
-1.5
V · F̂/k dynamic (undamped) amplitude
F̂/k static amplitude
V · F̂/k · e^{-ζωt} damped amplitude
π/Ω
2π/Ω
3π/Ω
4π/Ω
5π/Ω
6π/Ω
t(s)
b DISPLACEMENTS

The spring-mass system

The one-spring-mass system (damped by a friction element, or not), satisfies the element structure as described before. In general the system in one dimension can be described by: (see fig. 2)

$$F(t) = m\ddot{x} + c\dot{x} + kx \quad \ldots \quad (7)$$

$F(t)$ = external force [N]
x = location of body centre [m]
$\dot{x}$ = velocity of body centre [m/s]
$\ddot{x}$ = acceleration of body centre [m/s2]
m = mass [kg]
c = damping of friction constant [Ns/m]
k = elasticity or spring constant [N/m].

A solution to this differential equation can be found from the solution from the reduced equation (which describes the characteristic motion of the system) and the solution from the "steady state" equation (for $t \to \infty$), the response to the external force.

The characteristic equation consists of (transient equation):

$$m\ddot{x} + c\dot{x} + kx = 0 \quad \ldots \quad (8)$$

of which the solution is (transient solution):

$$x = C_1' e^{r_1 t} + C_2' e^{r_2 t} \text{ or } C_1 e^{i\omega t} + C_2 e^{-i\omega t} \quad \ldots \quad (9)$$

$$r_{1,2} = -\frac{c}{2m} \pm \sqrt{\left(\frac{c}{2m}\right)^2 - \frac{k}{m}}, \qquad \omega_d = \sqrt{\frac{k}{m} - \frac{c^2}{2m}} = \sqrt{\frac{k}{m}(1 - \zeta^2)}$$

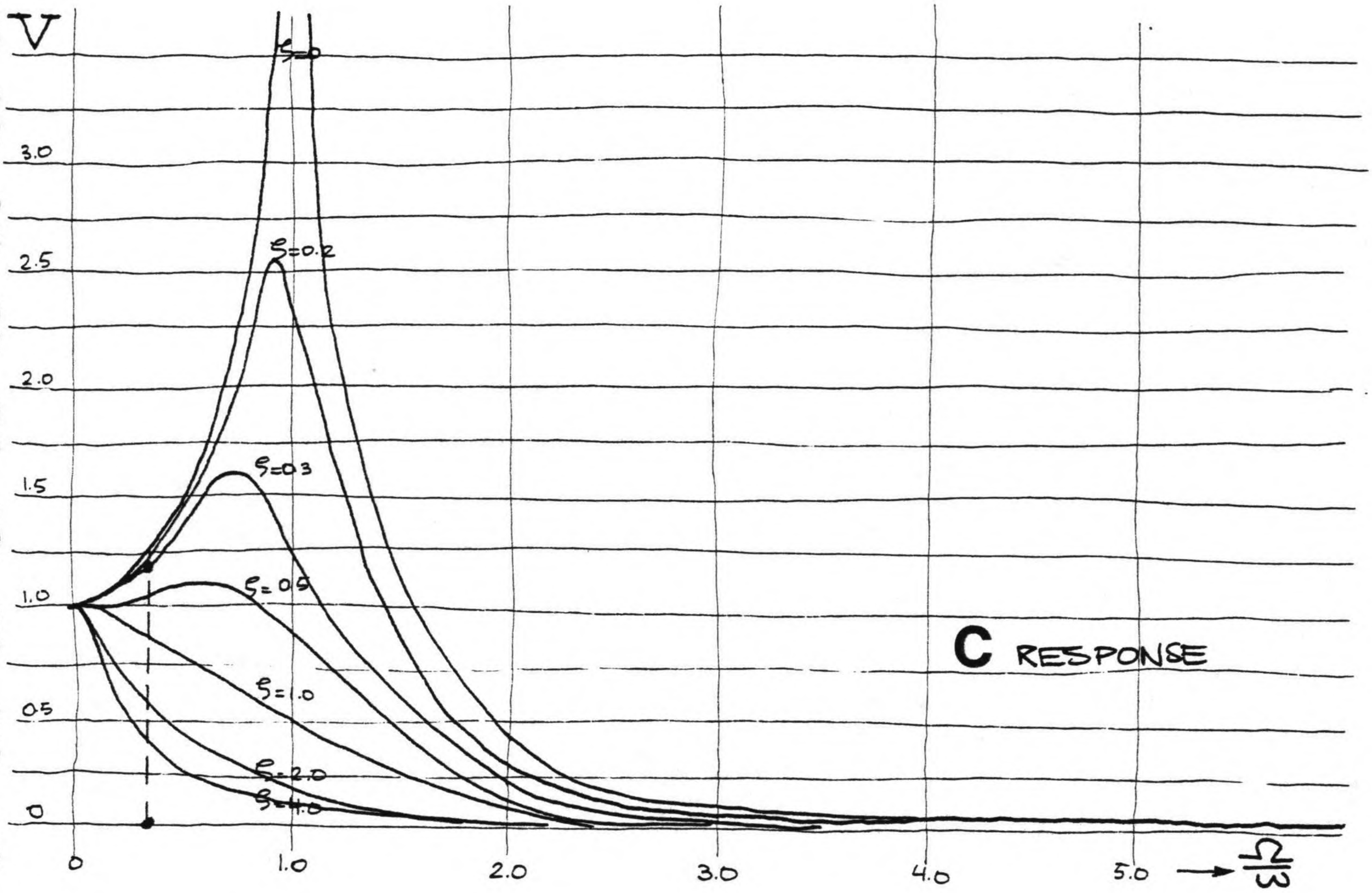
C_1 and C_2 can be found from initial and boundary conditions,

(if $c = 0$ than $r_{1,2} = \pm\, i\sqrt{\frac{k}{m}}$ and $\omega = \sqrt{\frac{k}{m}}$, the so-called characteristic frequency

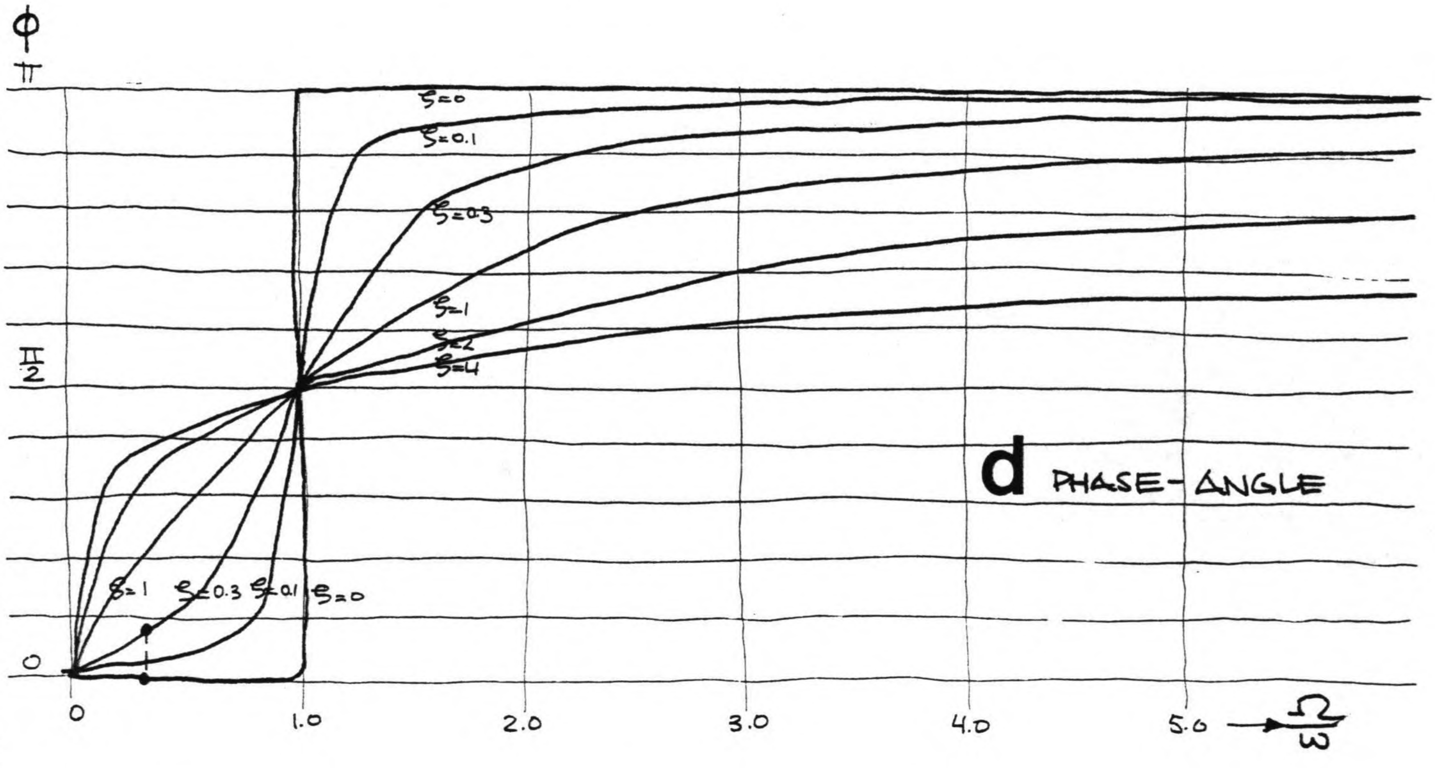
(if $c = 2\sqrt{km}$ then $r_{1,2} = -\frac{c}{2m}$ and $\omega_d = 0$, the so-called critical damping occurs.)

The steady state solution resulting from the force, depends on the function which the force has.

The response of a damped one spring mass system can be explained from the response to a undamped system:



C RESPONSE



d PHASE-ANGLE

2

I. ONE-SPRING-MASS SYSTEM, UNDAMPED

The general equation is, assuming a harmonic force:

$$\hat{F}\sin\Omega t = m\ddot{x} + kx \quad \text{.......} (10)$$

the solution of the steady state equation (for $t \to \infty$) is: (the transmission):

$$x = \hat{x}\sin(\Omega t - \phi) \quad \text{.......} (11)$$

$$\hat{x} = \frac{1}{1 - \dfrac{\Omega^2}{\omega^2}} \cdot \frac{\hat{F}}{k} = V \cdot X_{st}$$

Ω = force-frequency [rad/s]
ω = characteristic frequency $= \sqrt{\frac{k}{m}}$ [rad/s]
$\hat{F}$ = external force amplitude [N]
ϕ = phase angle (see fig. 2) [rad]

in which $\frac{\hat{F}}{k}$ is the "static" amplitude, X_{st}

(which would occur when the force would be applied slowly: $\Omega = 0$)

and $V = \dfrac{1}{1 - \dfrac{\Omega^2}{\omega^2}}$ is the "magnifying factor"

(or enlargement factor) which expresses the total amplitude related to the static amplitude.

In fig. 2 it is shown how the response of an undamped ($c = 0$) system behaves, as a function of Ω and ω.

If $\Omega = \omega$ "resonance" occurs: in the undamped version the amplitude of the response gets infinite (ω is the characteristic frequency of the system).

The solution of the complete differential equation (13) is a superposition of the transient solution and the "transmission":

$$x = C_1 e^{i\omega t} + C_2 e^{-i\omega t} + V.x_{st}\sin(\Omega t - \phi)$$

or: $x = C\cos(\omega t - \varphi) + V.x_{st}\sin(\Omega t - \phi)$ .... (12)

An oscillation in the characteristic frequency ω superposed on an oscillation in the force frequency Ω.

II. ONE SPRING MASS SYSTEM, DAMPED

The general equation is, assuming a harmonic force:

$$F \sin \Omega t = m\ddot{x} + c\dot{x} + kx \qquad (13)$$

the solution of the steady state equation (for $t \rightarrow \infty$) is:

$$x = \hat{x} \sin(\Omega t - \phi) \qquad (14)$$

$$x = \frac{1}{(1 - \frac{\Omega^2}{\omega^2})^2 + (2\zeta \frac{\Omega}{\omega})^2} \cdot \frac{\hat{F}}{k} = V_d \cdot X_{st}$$

in which $\frac{\hat{F}}{K}$ is again the static amplitude X_{st}

and the enlargement factor V_d is now dependent on a "damping ratio":

$$V_d = \frac{1}{(1 - \frac{\Omega^2}{\omega^2})^2 + (2\zeta \frac{\Omega}{\omega})^2}, \quad \zeta = \frac{C}{C_r} = \frac{C}{2\sqrt{km}}$$

$$\phi = \arctan \frac{C\,\Omega}{k - m\,\Omega^2} = \arctan \frac{2\,\zeta\,\frac{\Omega}{\omega}}{1 - \frac{\Omega^2}{\omega^2}}$$

The response of the damped system is also shown in fig. 2, dependent on $\frac{\Omega}{\omega}$ and on the damping ratio ζ.

The frequency at which the maximum amplitude occurs is no longer the characteristic frequency $\omega = \sqrt{\frac{k}{m}}$, but somewhat smaller; and the maximum amplitude is no longer infinite: (see fig. 2).

$$\Omega = (1 - 2\,\zeta^2)\omega \text{ "damped resonance"}$$

$$X_{max} = \frac{1}{2\zeta\,(1 - 2\,\zeta^2)} \cdot X_{st}.$$

The complete solution to the differential equation can be found by superposition:

$$x = e^{-\zeta\omega t}\left(C_1\, e^{i\sqrt{1-\zeta^2}\,\omega t} + C_2\, e^{i\sqrt{1-\zeta^2}\,\omega t}\right) + V_d\, x_{st} \sin(\Omega t - \phi) \qquad (15)$$

B.2 <u>The dynamic wave-loadings</u>

The force acting on a structure, is in fact dependent on the response. In chapter 2 the force has been determined by the Morison-equation:

$$F_{hor} = \underbrace{C_D \cdot \tfrac{1}{2} \rho A_p |u|u}_{\text{drag}} + \underbrace{C_m \rho \forall \frac{\partial u}{\partial t}}_{\text{inertia}} \quad \ldots\ldots \quad (16A)$$

$$F_{vert} = \underbrace{C_L \cdot \tfrac{1}{2} \rho A_p |u|u}_{\text{lift}} \quad \ldots\ldots \quad (16B)$$

It can be expected that the movement of the structure has some effect on the magnitude of the forces. In this analysis, the undisturbed water-particle velocity will be called u_w, and the velocity of the construction will be called u_c. By the movement of the construction, the Archimedes-term in the inertia-term (eq. 16A) will not change, but the term containing the added mass will:

$$F_{hor} = \underbrace{C_D \cdot \tfrac{1}{2} \rho A_p |u_w - u_c| (u_w - u_c)}_{\text{drag}} +$$

$$\underbrace{1 \cdot \rho\forall \left(\frac{\partial u_w}{\partial t}\right)}_{\text{Archimedes}} + \underbrace{C_a \rho\forall \left(\frac{\partial u_w}{\partial t} - \frac{\partial u_c}{\partial t}\right)}_{\text{added mass}} \quad \ldots\ldots \quad (17A)$$

$$F_{vert} = \underbrace{C_L \cdot \tfrac{1}{2} \rho A_p |u_w - u_c| (u_w - u_c)}_{\text{lift}} \quad \ldots\ldots \quad (17B)$$

F_{hor} = horizontal force [N]
F_{vert} = vertical force [N]
C_D = drag coefficient [-]
C_m = mass coefficient = $C_a + 1$ [-]
C_L = lift coefficient
ρ = density of water [kg/m3]
A_p = projected frontal area [m2]
$\forall$ = displaced volume [m3]
u_w = velocity of water-particles [m/s]
u_c = velocity of construction [m/s].

In terms of dynamics, these equations (17A, 17B) can be rewritten:

$$F_{hor} = C_D \cdot \tfrac{1}{2} \rho A_p |u_w - \dot{x}|(u_w - \dot{x}) \text{ drag } +$$

$$\rho\forall \frac{\partial u_w}{\partial t} + (Cm - 1)\, \rho\forall \left(\frac{\partial u_w}{\partial t} - \ddot{x}\right) \text{ inertia} \quad \ldots\ldots\ldots\ldots\ldots (18A)$$

$$F_{vert} = C_L \tfrac{1}{2} \rho A_p |u_w - \dot{x}|(u_w - \dot{x}) \text{ lift} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (18B)$$

x = hor. displacement of the construction [m]
$\dot{x}$ = hor. velocity of the construction [m/s]
$\ddot{x}$ = hor. acceleration of the construction [m/s²].

The particle kinematics can be approximated by an appropriate wave-theory. In this case the "airy-wave" theory is chosen. The force-amplitudes have also been determined in chapter 2, where it was assumed that the construction would not move. If we combine these results, and (eq. 18A, 18B) we find:

$$F_{hor} = \underbrace{\tfrac{1}{2} C_D \rho\, H.\ell\, |\hat{u} \sin \Omega t - \dot{x}|(\hat{u} \sin \Omega t - \dot{x})}_{\text{drag}} +$$

$$\underbrace{\tfrac{1}{4} \rho\pi H^2 \ell \hat{u}\Omega\cos\Omega t + (C_m - 1)\, \tfrac{1}{4} \rho\pi H^2 (\hat{u}\Omega\cos\Omega t - \ddot{x})}_{\text{inertia}} \quad \ldots\ldots\ldots (19A)$$

$$F_{vert} = \underbrace{\tfrac{1}{2} C_L \rho H \ell |\hat{u} \sin \Omega t - \dot{x}|(\hat{u} \sin \Omega t - \dot{x})}_{\text{lift}} \quad \ldots\ldots\ldots\ldots\ldots (19B)$$

H = height of the construction [m]
ℓ = length of the construction [m]
Ω = wave-frequency = $2\,\pi/T$ [rad/s]
$\hat{u}$ = maximum hor. water velocity [m/s]
u_w = $\hat{u} \sin \Omega t$: airy-wave theory [m/s].

Under extreme conditions (see chapter 2) following values are taken (forces in kN/m')

H = 3.50 m (height of the construction)*
ℓ = 1 (forces per unit of length)
Ω = 1.05 rad/s (T_{wave} = 6 s)
$\hat{u}$ = 2.7 m/s (wave height is 3.6 m)
C_m = 4.0 (bottom-mounted cylinder)
C_D = 1.2 (bottom-mounted cylinder)
$\hat{F}_{vert}$ = 150 kN/m' (see chapter 2)

$$F_{hor} = \underbrace{15|\sin\Omega t - 0.37\,\dot{x}|(\sin\Omega t - 0.37\,\dot{x})}_{\text{drag}} +$$

$$\underbrace{25\cos\Omega t + 75\,(\cos\Omega t - 0.35\,\ddot{x})}_{\text{inertia}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20A)$$

$$F_{vert} = \underbrace{150|\sin\Omega t - 0.37\,\dot{x}|(\sin\Omega t - 0.37\,\dot{x})}_{\text{lift}} \quad \ldots\ldots\ldots\ldots (20B)$$

If we neglect the drag-force with respect to the inertia-force, and if we approach "$\sin\Omega t|\sin\Omega t|$" by "$\sin\Omega t$", and neglect the term containing $\dot{x}^2$, these expressions become:

$$F_{hor} = 100\cos\Omega t - 26\,\ddot{x}\ [kN/m'] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21A)$$

$$F_{vert} = 150\sin\Omega t - 111\,\dot{x}\ [kN/m'] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (21B)$$

These are the expressions for the wave-forces that will be used in order to determine the response of the system.

N.B. In the Morison-equation, the displaced volume $\forall$ appears. In this analysis, and in chapter two, it is taken: $\forall = \frac{1}{4}\pi H^2$ in which the construction is compared with a cylinder. In reality however, the volume of the construction will be in the order of $\forall$ = 30 [m3/m']. According to Morison's formula, this should result in a maximum horizontal force of 300 [kN/m']. However, if we consider the construction an impermeable wall (upper limit), the wave-loading can be found by:

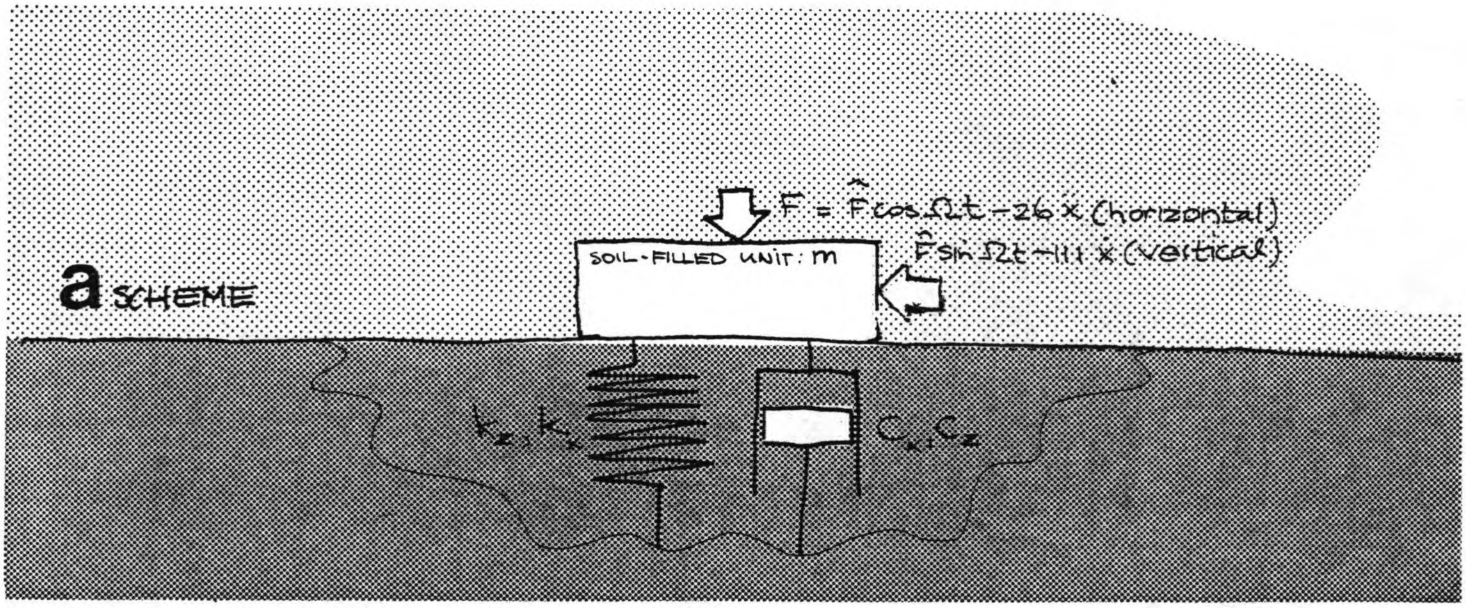
$F_{hor} = \int_0^a (P_+)\,dz$ [N/m'] in which P_+ presents the

extra wave pressure and a the height of the construction.

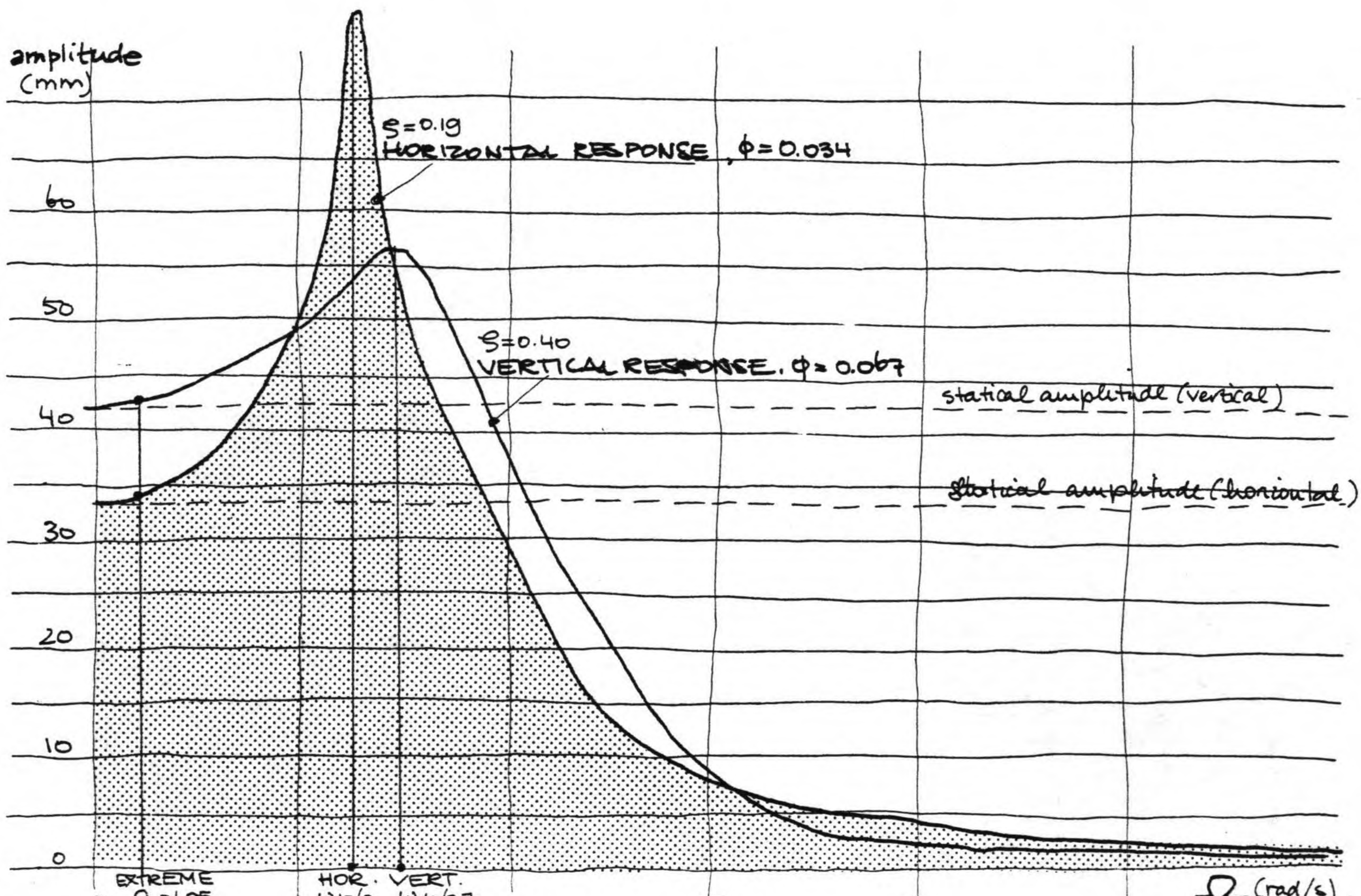
$$P+_{max} = \rho g H_{max} \frac{\cosh k\,(d + z)}{\cosh kd} \ . \ [N/m2] \quad (H_{max} = \text{max. wave height})$$

Solving this equation with a = 3.50 m, we would find a maximum hor. force of 90 [kN/m']. So the original Morison-approach, using $\forall = \frac{1}{4}\pi H^2$ is sufficiently accurate. This also explaines why the force on large bodies is most times expressed in a factor containing $\rho g H a$, since

$$\int_0^a (P+)dz = \rho g H a \ . \int_0^a \frac{\cosh k(d + 2)}{\cosh kd}\,dz \ [N/m']$$



a SCHEME



b RESPONSE

3

B.3 Response to wave-loadings

I. RESPONSE OF THE SUBSOIL: SPRING DASHPOT MASS SYSTEM

foundation analogy

If the soil filled unit is considered a homogeneous stiff mass, resting on the subsoil, an analogy can be found between this system and a one-spring-mass system in case of foundations (see lit(15)).
The unit has a mass, m, the subsoil has an elaticity, K and a damping of C. The values of K and C are dependent on the contact area with the subsoil, S (see fig. 3).

GENERAL: foundations

$$K_x = \frac{0.9\ E\sqrt{S}}{1 - \nu^2} \quad \ldots\ldots (22)$$

$$K_z = \frac{1.1\ E\sqrt{S}}{1 - \nu^2} \quad \ldots\ldots (23)$$

$$C_x = S\sqrt{\frac{0.3\ \rho\ E}{1 + \nu}} \quad \text{and} \quad \zeta_x = \sqrt{\frac{S\rho_s\sqrt{S}(1 - \nu)}{12\ m}} \quad \ldots\ldots (24)$$

$$C_z = S\sqrt{\frac{\rho E}{1 + \nu}} \quad \text{and} \quad \zeta_x = \sqrt{\frac{S\rho_s\sqrt{S}(1 - \nu)}{4.4\ m}} \quad \ldots\ldots (25)$$

E = elasticity of the soil $\simeq$ 1,000 10^3 N/m2
ρ_s = density of the soil $\simeq$ 1,800 kg/m3
ν = contraction coefficient of soil $\simeq$ 0.3

$$\zeta = \frac{C}{C_{cr}} \text{ in which } C_{cr} = 2\sqrt{km.}$$

The response of the system "soil filled unit-subsoil" can be calculated, according to the damped one spring mass system (eq. 16...18) using the practicle values mentioned above.

The force is described (drag is neglected):

$F_x = 100 \cos\Omega t - 26\ \ddot{x}$ kN/m' (eq. 21A)
$F_z = 150 \sin\Omega t - 111\ \dot{x}$ kN/m' (eq. 21B)

x stands for displacements in hor. direction
z stands for displacements in vert. direction.

Solutions can be found in horizontal and vertical direction.

HORIZONTAL: $F(t) = 100 \cos\Omega t - 26 \ddot{x}$ [kN/m']
$$(m + 26.10^3)\ddot{x} + c\dot{x} + kx = 100.10^3 \cos\Omega t$$

a single unit has a minimum content of 30 m3/m' (m = 54,000), see par. 3.2.2, and a contact surface of 9 m2/m' (S = 9.0 m2/m') due to the inertia-term in the force, an "apparent" mass develops of 80,000 kg:

calculating:

$c_x = 1.83.10^5$, $\zeta_x = 0.19$, $K_x = 2.96.10^6$

the characteristic frequency

$$\omega_d = \sqrt{1-\zeta^2}\sqrt{\frac{k}{m}} = 6 \text{ rad/s}$$

$$X_{stat} = \frac{\hat{F}}{K} = 33.8.10^{-3} \text{ m}$$

$$V_d = 1.03$$

for the complete horizontal response see fig. 3, steady state solution:

$$X = 0.034 (\cos\Omega t - \phi) \quad \ldots \ldots (26)$$

$$\phi = \arctan \frac{\frac{2\zeta\Omega}{\omega}}{1 - \left(\frac{\Omega}{\omega}\right)^2} = 0.07 \text{ rad}$$

the same can be done for the vertical direction:

VERTICAL: $F(t) = 150 \sin\Omega t - 111 \dot{x}$ [kN/m']
$$m\ddot{z} + c\dot{z} + kz = 150.10^3 \sin\Omega t - 111.10^3 (0.034\,\Omega \sin(\Omega t - \phi)$$
$$m\ddot{z} + c\dot{z} + kz = 146.10^3 \sin\Omega t - 444 \cos\Omega t$$
$$m\ddot{z} + c\dot{z} + kz = 146.10^3 \sin\Omega t$$

$\zeta_z = 0.40$, $k_z = 3.63.10^6$
$\omega_d = 6.7$ rad/s
$Z_{stat} = 41.3.10^{-3}$ m
$V_d = 1.01$
$\phi = 0.014$ rad

for the complete vertical response see fig. 3; steady state solution:

$$Z = 0.042 \sin(\Omega t - \phi) \quad \ldots \ldots (27)$$

the same procedure can be followed for the smallest units that the dike is built of: A' = 5 m3/m

then m = 9,000 m3/m'
S = 3 m2/m'

the force components are:

$F_x = 40\ (\cos\Omega t) - 10.5\ \ddot{x}\ [kN/m']$
$F_z = 55\ (\sin\Omega t) - 41\ \dot{x}\ [kN/m']$

the results: (steady state, so for $t >> \frac{2\pi}{\omega}$)

$x = 0.021\ (\cos\ \Omega t - \phi)\ ,\ \phi = 0.34\ ,\ \omega_x = 9.9$ rad/s .... (28)
$z = 0.023\ (\sin(\Omega t - \phi)\ ,\ \phi = 0.067\ ,\ \omega_z = 12.5$ rad/s .... (29)

The units show an almost static response to the wave loadings, this confirms the assumption of the unit responding as a stiff mass.

CONCLUSION: the dynamic response of the subsoil to the wave loadings (acting on the dike) results in a
- horizontal translation with amplitude 34 mm;
- vertical translation with amplitude 42 mm.

The units itself respond in a static way to the wave loading

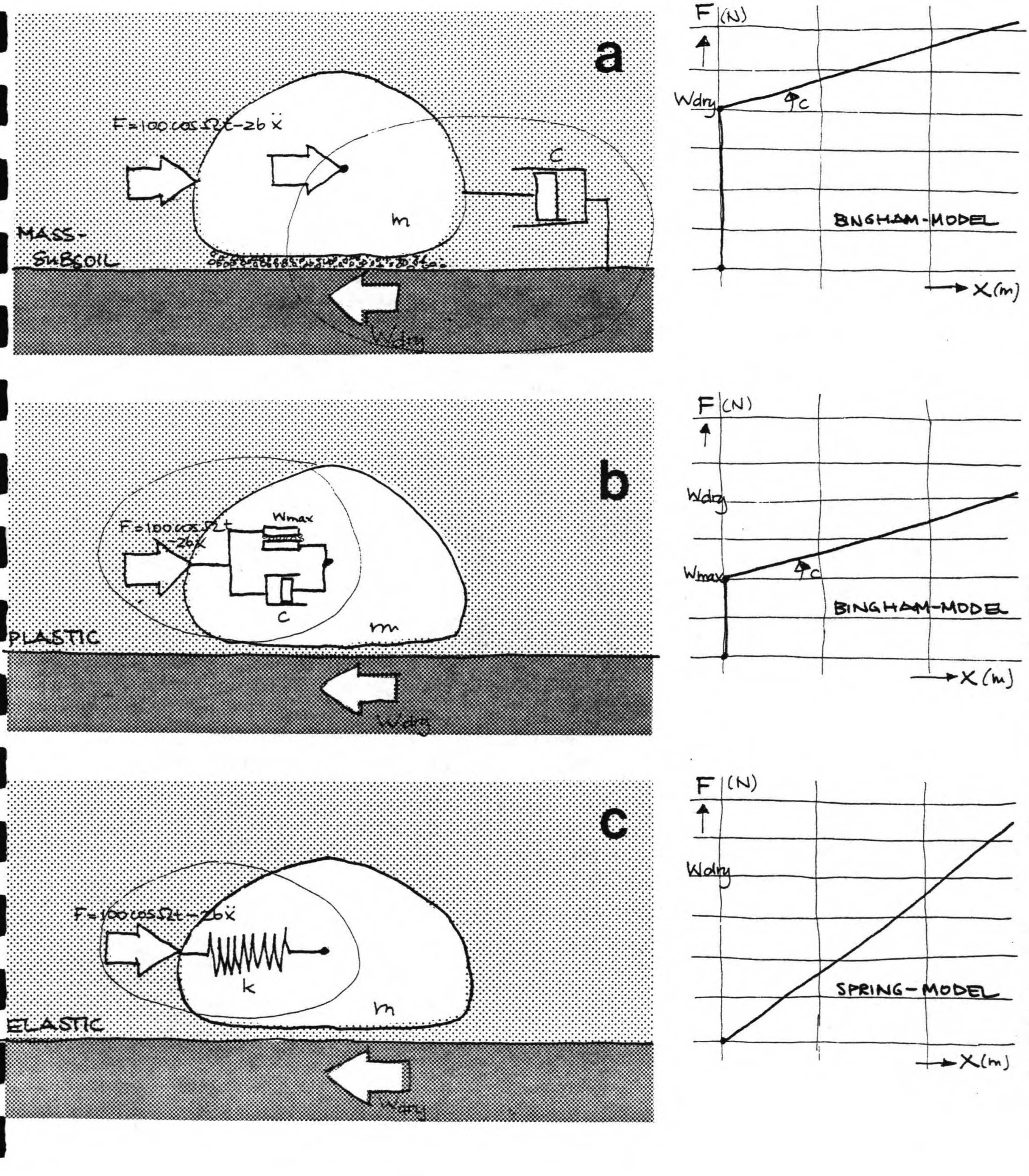
characteristic frequency: (whole dike)

hor : ω = 7 rad/s
vert: ω = 6.7 rad/s

characteristic frequency of the smallest units:

hor : ω = 13 rad/s
vert: ω = 12.5 rad/s.

N.B. fig. 3 shows that the loading of wave slamming is not in the response range of the construction, when Ω = 126 (T = 0.050 s)
the enlargement factor is about 0.003, in other words the dike as
a whole (and the subsoil) is insensitive to slamming loads.


a
F=100.cos Ωt−2b ẍ
C
m
MASS-
SUBSOIL
Wdry
F (N)
Wdry
C
BINGHAM-MODEL
X (m)
b
F=100.cos Ωt
−2bẍ
Wmax
c
m
PLASTIC
Wdry
F (N)
Wdry
Wmax
C
BINGHAM-MODEL
X (m)
c
F=100.cos Ωt−2bẍ
k
m
ELASTIC
Wdry
F (N)
Wdry
SPRING-MODEL
X (m)

4

II. RESPONSE OF THE SOIL FILLED CONSTRUCTION:

SPRING DASHPOT MASS SYSTEM

The soil filled dike itself can also be described as a system consisting of a spring: the elastical deformation due to wave loadings, a dashpot: the plastic deformation caused by a loading, and a mass: the filling of the dike.

The response to horizontal loadings can be checked by means of stability against translation and stability against rotation.

A. TRANSLATION:

The system can be devided into three subsystems, that will be studied in more detail (see fig. 4)

1. MASS SUBSOIL SYSTEM (see fig. 4A)

The item that is discussed here, is the dislocation of the mass centre of the dike due to horizontal loadings (or the dike as a whole).
It can be described as:

$$m\ddot{x} = F(t) - W(t) \quad \ldots\ldots (30)$$

The friction between mass and subsoil can be described with a bingham-model:

$$W = W_{dry} + c\dot{x} \quad \ldots\ldots (31)$$

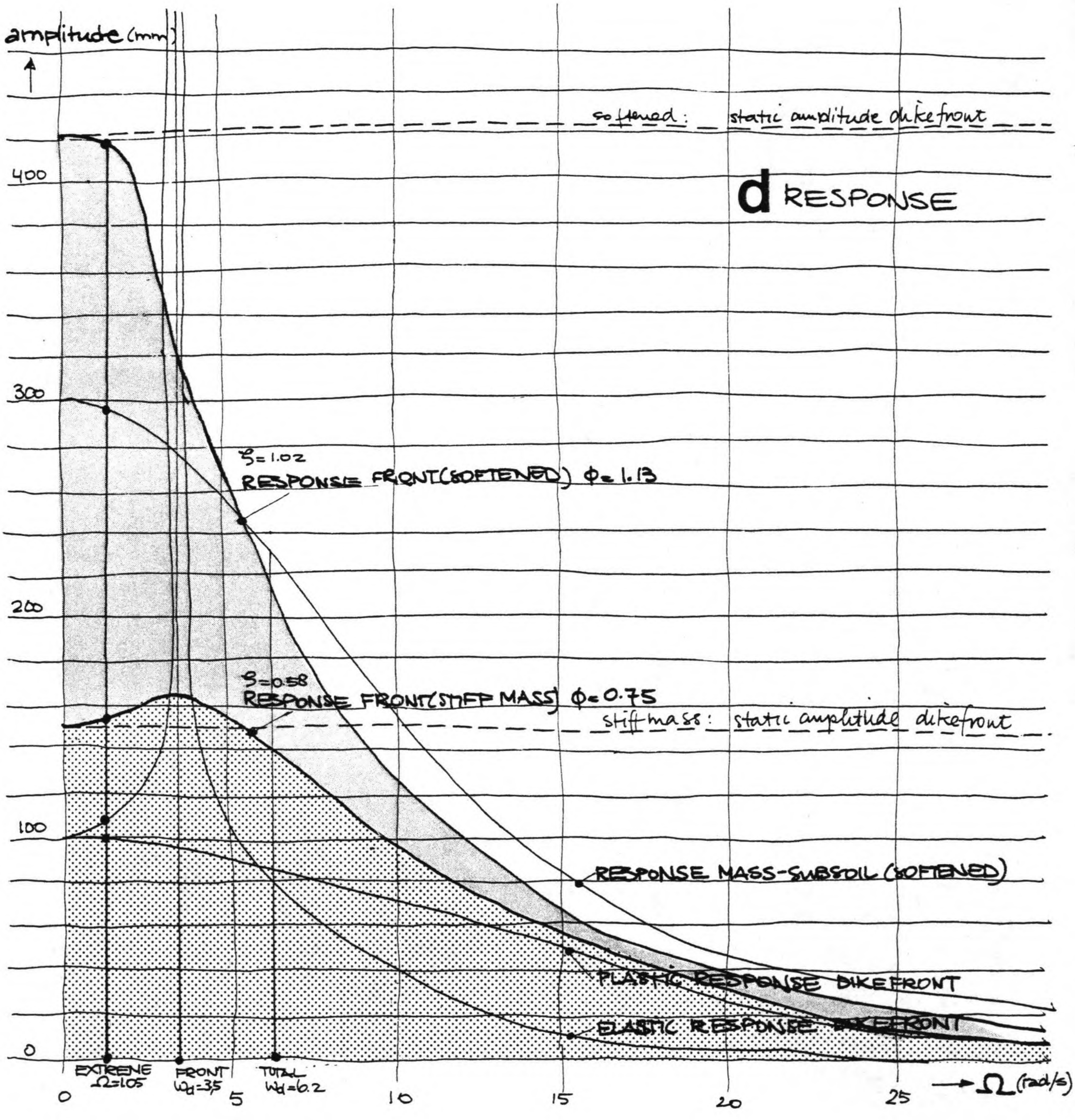
$$W_{max} = (\bar{\sigma}_k \tan\phi + Cu)S + c\dot{x}$$

The first part is also called the "dry friction". This implies that W has the same magnitude as the loading F(t) as long as W_{dry} is not exceeded, $W = F(t) \leq W_{max,\ dry}$.

Of course, the dike is designed in a way that W_{dry} is always larger than F(t), and the dislocation will be zero (see chapter 3.2.2). Anyhow, an upper- and lower limit of the response of the dike can be found by assuming $W_{dry} \geq F(t)$ and by assuming $W_{dry} = 0$.

assuming the dry friction zero (by softening of the subsoil or something simular) then,

$$m\ddot{x} = F(t) - c\dot{x} \quad \ldots\ldots (32)$$


amplitude (mm)
softened: static amplitude dikefront
d RESPONSE
400
300
ξ=1.02
RESPONSE FRONT (SOFTENED) φ=1.13
200
ξ=0.58
RESPONSE FRONT (STIFF MASS) φ=0.75
stiff mass: static amplitude dikefront
100
RESPONSE MASS-SUBSOIL (SOFTENED)
PLASTIC RESPONSE DIKEFRONT
ELASTIC RESPONSE DIKEFRONT
0
EXTREME Ω=1.05
FRONT ωd=3.5
TOTAL ωd=6.2
0
5
10
15
20
25
Ω (rad/s)

ρ = 1,800 kg/m3
E = $1{,}000.10^3$ N/m
ν = 0.3

in which
$m = 54.10^3$ (A' = 30 m3/m') kg
$F(t) = 100\ (\cos\Omega t) - 26\ \ddot{x}$ kN/m
Ω = 1.05 rad/s

$$c = S\sqrt{\frac{\rho E}{1+\nu}} = 3.3\ 10^5 \quad \text{Ns/m}$$

S = 9.00 m2

then the steady state solution ($t \to \infty$) is: (for the dislocation of the mass centre)

$X = 0.30\ (\cos\Omega t - \phi)$ (ϕ = 1.31 rad) .... (33)

N.B. the full equation (32) can be written as:

$$54.10^3\ \ddot{x} + 3.3.10^5\ \dot{x} = 100.10^3\ \cos\Omega t - 26.10^3\ \ddot{x}$$

transient equation: $m\ddot{x} + c\dot{x} = 0$, solution: $x = \hat{x}e^{\frac{-t}{T}}$

with "m" = 80.10^3, c = 330.10^3

$$T = \frac{m}{c} = 0.25 \text{ s}$$

$$\hat{\dot{x}} = \frac{\hat{F}}{c} = 0.30 \text{ m}$$

if $t_{loading} > 2T$ (= 0.5 s) then the steady state solution is also the actual solution:

$m\ddot{x} + c\dot{x} = \hat{F}\cos\Omega t$, solution $x = \hat{x}\ (\cos\Omega t - \phi)$

$$\tan\phi = \frac{1}{\Omega T} = \frac{c}{m\Omega} = 3.92 \text{ rad}, \quad \phi = 1.32 \text{ rad}$$

$$\hat{x} = \frac{\hat{F}}{\Omega c}\ \frac{1}{\sqrt{1 - \left(\frac{m\Omega}{c}\right)^2}} = 0.30 \text{ m}$$

assuming the dry friction is large enough to exceed the total horizontal loading:

$m\ddot{x} = F(t) - W(t) = 0$ .... (34)

so: x = 0: no displacements

2. PLASTIC DEFORMATION (see fig. 4B)

Deformation takes place at the front of the soil filled unit, at the place the wave attacks the dike.
The response of the soil, can also be described with a bingham-model

$$m\ddot{x} = F(t) - W(t) \quad \ldots\ldots (35)$$

in which $W(t) = W_{dry} + c\dot{x}$

If we assume a sliding surface at the half of the dike height:

$$W_{max} = S\left[\tfrac{1}{2}H(\rho_s - \rho_w)g\ \tan\phi\right] + c\dot{x} = 50.10^3 + c\dot{x}$$
$$= 50.10^3 + c\dot{x}$$

so the motion is described by:

$$c\dot{x} = 100\ (\cos\Omega t) - 26\ \ddot{x} - 50\ [\mathrm{kN/m'}]$$

solution ($c = 3.3.10^5$ Ns/m , "m" = 26.10^3 kg):

$$\underline{x = 0.10\ \cos(\Omega t - \phi)}\ ,\ \phi = 1.49\ \mathrm{rad} \quad \ldots\ldots (36)$$

($x = 0.15\ \cos(\Omega t - \phi)$ if $F(t) \geq 50$ kN/m').

3. ELASTIC DEFORMATION (see fig. 4C)

The elastic deformation also takes place at the front of the dike, where the waves tackle the dike.
The response can be described as a spring having the elasticity of the soil,

$$kx = F(t) \quad \ldots\ldots (37)$$

with $k = 1000.10^3$ N/m and $F(t) = 100\ \cos\Omega t - 26\ \ddot{x}\ [\mathrm{kN/m'}]$

so "m" = 26.10^3 kg and $\omega = \sqrt{k/m} = 6.2$ rad/s

solution: $\underline{x = 0.10\ \cos\Omega t}$ ($\phi = 0$) $\quad \ldots\ldots (38)$

CONCLUSION

The total "response" of the front can be found by superposition of the sub-displacements. (the displacement of the mass-center is independent of the properties of the "front").
stiff mass: (total dry friction larger than loading)

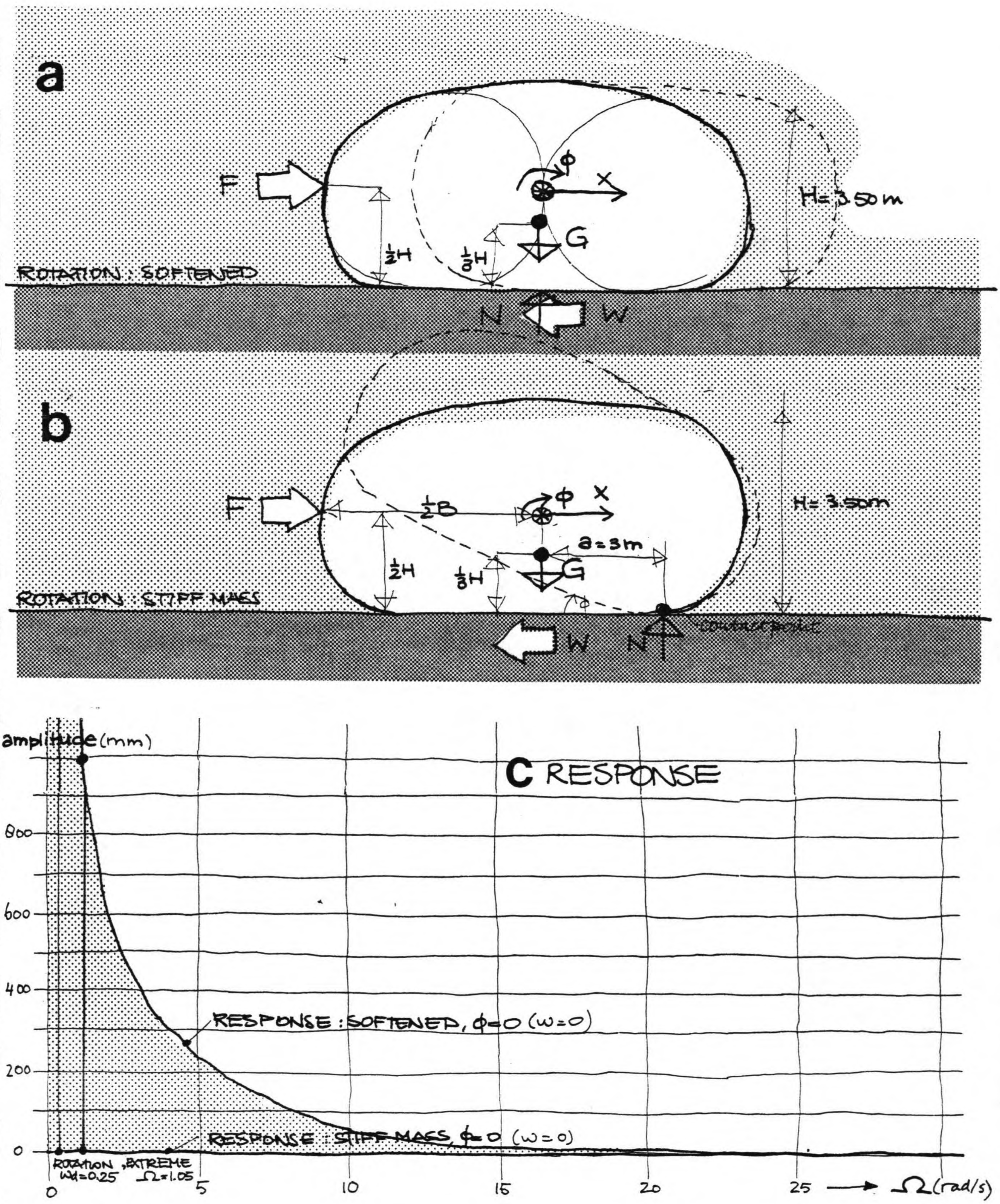
mass centre: $\underline{x = 0}$

front dike : $\underline{x = 0.15\ \cos(\Omega t - \phi)}\ \ \underline{\phi = 0.75}$ $\quad \ldots\ldots (39)$

characteristic frequency: $\sqrt{\frac{k}{m}} = 6.2$ rad/s) softened: (no dry friction)

$$\left.\begin{array}{l}
\text{mass centre: } \underline{x = 0.30 \cos(\Omega t - \phi)}\ \underline{\phi = 1.32} \\[2ex]
\text{front dike : } \underline{x = 0.42 \cos(\Omega t - \phi)}\ \underline{\phi = 1.13} \\[1ex]
\text{(characteristic frequency: } \sqrt{\frac{k}{m}} = 3.5 \text{ rad/s).}
\end{array}\right| \qquad \ldots\ldots\ldots(40)$$

Again the frequency of the wave slamming is so high that the dike is insensitive to it (Ω = 126) (see fig. 4).


a
F
φ
x
H=3.50m
G
½H
⅓H
ROTATION : SOFTENED
N
W
b
F
½B
φ
x
H=3.50m
a=3m
G
½H
⅓H
ROTATION : STIFF MASS
W
N
contact point
amplitude (mm)
C RESPONSE
800
600
400
200
0
RESPONSE : SOFTENED, φ=0 (ω=0)
RESPONSE : STIFF MASS, φ=0 (ω=0)
ROTATION ωd=0.25
EXTREME Ω=1.05
0
5
10
15
20
25
Ω (rad/s)

5

B. ROTATION:

if we consider rotation, translation of the base of the construction along the subsoil is assumed not to occur. The construction itself can be considered a stiff mass (lower limit of the response) or a heavy liquid without resistance against internal shear (upper limit of the response).

Softened:

the content of the dike is considered without friction, (see fig. 5) dry mass in x-direction:

$m\ddot{x} = F(t) - W$ .... (41)

in ϕ-direction

$2I\ \ddot{\phi} = \frac{1}{2}H.\ W$ .... (42)

and the relation between x and ϕ:

$\ddot{x} = \frac{1}{2}H\ \ddot{\phi}$ (since the membrane does noet slide along the bottom)
assuming $H = 3.50$ m, $F(t) = 100 \cos\Omega t - 26\ \dot{x}$, $I = 0$ (no internal friction) $m = 54.10^3$ kg then it would follow:

$$m\ddot{x} = F(t) = 100 \cos\Omega t - 26\ \dot{x}\ [\mathrm{kN/m'}]$$

$$\ddot{x} = \frac{100}{80} \cos\Omega t\ [\mathrm{m/s^2}]$$

$$x = -1 \cos\Omega t \text{ (amplitude of 1 m)}$$

this, of course, is not allowable. Softening of the filling should be avoided.

Stiff mass:

the content is considered a stiff mass, rotating around the contact point which is a = 3.0 m out of the centre of the dike (see <u>fig. 5</u>) in x-direction:

$m\ddot{x} = F - W$ $\qquad$ $F(t) = 100 \cos\Omega t - 26\ \dot{x}\ [\mathrm{kN/m'}]$ .... (43)

in z-direction: (under water)

$m\ddot{z} = G - N$ $\qquad$ $G = 30\ \mathrm{m3/m'}\ (\rho_s - \rho_w)g$ .... (44)

in ϕ-direction: (for small ϕ: $\sin\phi = \phi$, $\cos\phi = 1$).

$I\ddot{\phi} = \frac{1}{2}B\ F \sin\phi + \frac{1}{2}H\ W \cos\phi - 1/6H\ G \sin\phi - aN \cos\phi$ .... (45)

the relation between x, z and ϕ:

$\ddot{x} = \frac{1}{2}H\ddot{\phi}$, $\ddot{z} = -a\ddot{\phi}$

so $W = F(t) - \frac{1}{2}H\ddot{\phi}m$ and $N = G + a\ddot{\phi}m$

equation (43, 44, 45) become:

$$\left[(I + \tfrac{1}{4}H^2)m + a^2m\right]\ddot{\phi} + \left[1/6H\ G - \tfrac{1}{2}B\ F\right]\phi = \left[\tfrac{1}{2}HF - aG\right] \quad \ldots\ldots.(46)$$

or: "I" $\ddot{\phi}$ + "K" ϕ = "M".

The solution follows quite simple from a spring-mass analogy:

$\phi = \phi_{st}\ V \cos(\Omega t - \phi)$

introducing $H = 3.50$ m, $a_{max} = 3.0$ m, $G = 240$ kN,
$m = 54.10^3$ kg it follows:

$I_{tot} = \frac{1}{12}m\ B^2 + \frac{1}{4}m\ H^2 + ma^2 = 24.10^5$ kg m2
$K_{tot} = 1/6HG - \frac{1}{2}B\ F_{max} = -310$ kN/m' ("negative!")
$M_{tot} = \frac{1}{2}HF_{max} - aG = -545$ kNm/m' ('negative").

As long as the total moment is negative, the dike will not rotate (then the contact point a is less then 3 m away from the centre). If $F = 100$ kN/m', $k = 0$, $z = 0$, $\phi = 0$ and $a = 0.73$ m

solution: $\underline{\ddot{x} = 0,\ \ddot{z} = 0,\ \ddot{\phi} = 0}$

(if $F \leq 411$ kN/m'.)

CONCLUSION: the dynamic response of the dike itself to the wave loadings consist of
- an elastic deformation of about 100 mm;
- a plastic deformation of about 100 mm.

Sufficient care should be taken to prevent softening of the subsoil and softening of the filling, because it would result in unallowable displacements (no resistance against rotation and sliding). This would imply to use several separated units per cross section (a construction of tubes).

The response is mainly static, but with a considerable phase angle, due to the plastic deformation.

Characteristic frequency: (whole dike)

hor: $\omega = 6$ rad/s;
rot: $\omega = 0$ rad/s.

Characteristic frequency: (smallest units)

hor: $\omega = 10$ rad/s
rot: $\omega = 0$ rad/s.

III. RESPONSE OF THE SOIL FILLED CONSTRUCTION:

FINITE ELEMENT METHOD

It is possible to describe the soil filled geotextile as a structure consisting of a finite number of elements:

1. SOIL ELEMENTS (see fig. 6)

   the characteristics of the soil elements consist of a volume, having a certain mass, on elasticity and a plasticity (through the relation of the shearstress with the deformation, depending on the angle of internal friction of the soil ϕ, and the cohesion Cu);

2. MEMBRANE ELEMENTS (see fig. 6)

   the characteristics of the membrane elements consist of a one-dimensional element, having only an elasticity, and which is unable to resist sideward displacements; the elasticity is only valid for elongation, not for contraction.

When using an appropriate computer model, the theoretical solution of the STATIC response could be found.
The main problem, using this approach, is that the problem is propably too complicated:

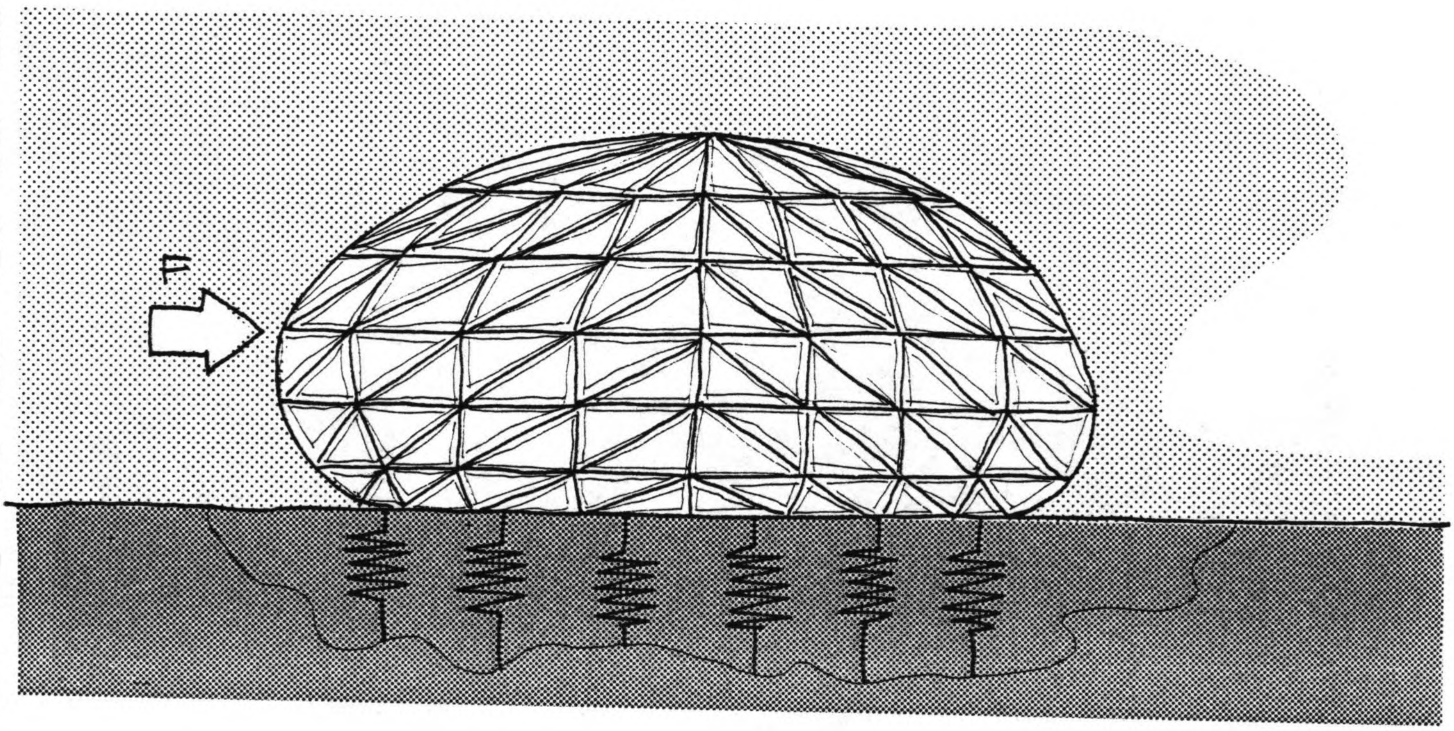
- the construction is non-linear in a geometrical sence (deformation is not proportional to the loadings) and the characteristics of the soil are non-linear (though they could be schematized as linear);

- furthermore, to find a stable solution for the membrane elements (no resistance against sideward displacements) is not easy, several "auxiliary equations" should be added to ensure that the calculations converge towards an existing shape.

In order to find the dynamic solution, the static solution can be used to determine the characteristic frequencies of the unit. The response to the actual loadings can be determined by considering the construction as "free" and impose the loading as boundary condition. The solution will be a superposition of the eigen-vectors of the construction.

At the moment, there is not an appropriate program available to perform the above-mentioned calculations.
Since problems are expected concerning the stability of the calculations and schematizing the elements, the actual response of the soil filled construction should be determined by means of a prototype test.
The calculations as done in the foregoing already give an impression of the response to be expected.

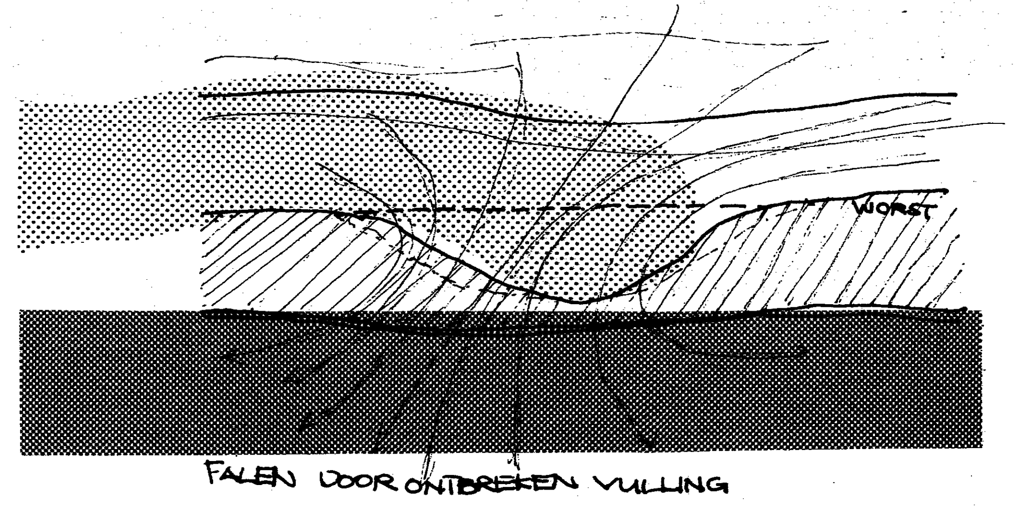
CONCLUSION: a prototype test should be performed in order to determine the actual response of the soil filled construction.


F

ultimate limit state: | cause: | requirement:

ONTBREKEN VULLING

ABSENCE OR LOSS OF FILLING-MATERIAL



4.1b

Ways of failure geotextile

DOORLATEN WEEFSEL
leakage through the textile..
NORMAL CONDITIONS

- GOLVEN (waves) → loss is not allowed: $O_{90} = 10\mu m$ ⇒ use of a plastic sheet as inlay
- STROMING (currents) → loss is not allowed: $O_{90} = 10\mu m$ ⇒ use of a plastic sheet as inlay
- ZON (sun) → UV-stability of the plastic sheet should be sufficient to guarantee the water-tightness for 5 years.

INTERNE VERPLAATSING
internal movement......
EXTREME CONDITIONS

- GOLVEN (waves) → filling degree should be over 70%
- STROMING → softening is not allowed
- (currents) → probably of minor importance
- WIND (wind) → probably of minor importance
- PLAATSING (stationing) → filling should take place with care

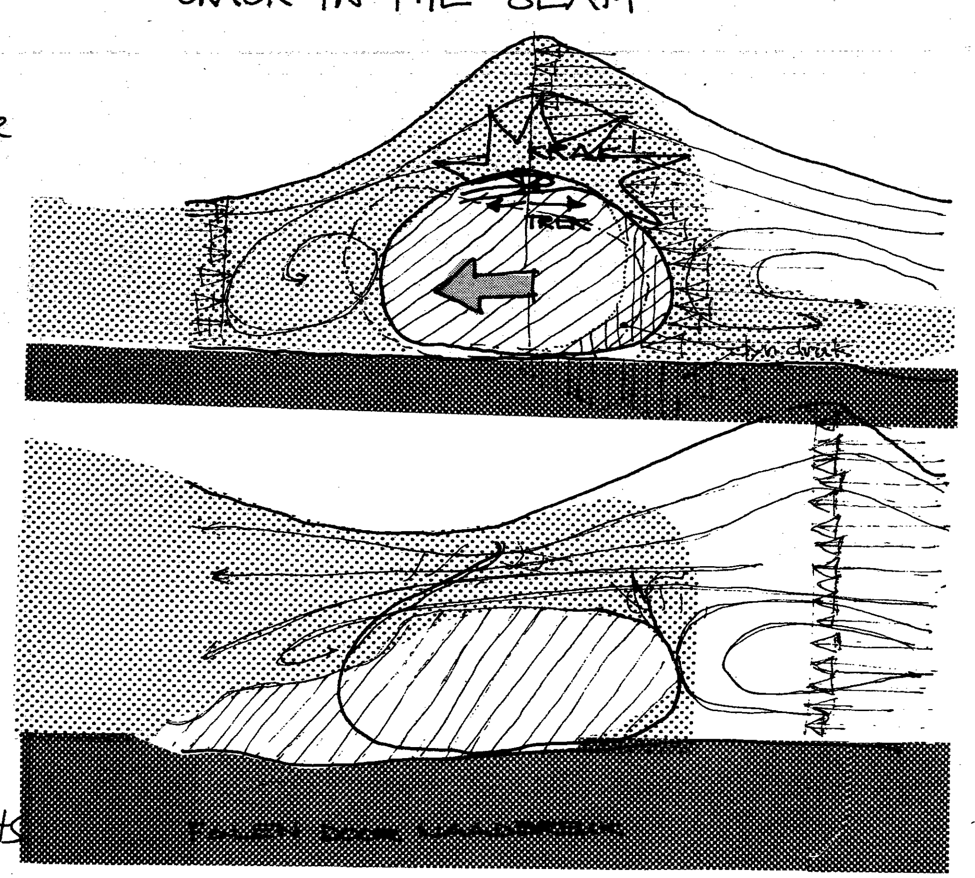
ONTBREKEN VULLING
local absence of material...
SPECIAL CONDITIONS

→ repair must be possible,
→ voids must be prevented
→ the construction must consist of units which can be re-filled.

ultimate limit state: | cause: | requirement:

NAADBREUK

CRACK IN THE SEAM



TE GROTE KRACHT
loadings...........
EXTREME CONDITIONS

- GOLVEN (waves) → dynamic strength: $\Delta N\phi = 100 kN/m'$ ($\gamma = 1$)
- STROMING (currents) → additional testing necessary (probably minor influence)
- WIND (wind) → additional testing necessary (probably minor influence)
- PLAATSING (stationing) → gravitational loading; static strength dependent on construction ($N\phi \approx 125 kN/m'$) $\gamma = 2$!

VERMOEIING
fatigue........
NORMAL CONDITIONS

- GOLVEN (waves) → additional testing necessary strength after 5 years: 350 kN/m'
- STROMING (currents) → additional testing necessary strength after 5 years: 350 kN/m'
- WIND (wind) → additional testing necessary strength after 5 years: 350 kN/m'
- ZON (sun) → UV-stability: strength after 5 years: 350 kN/m'

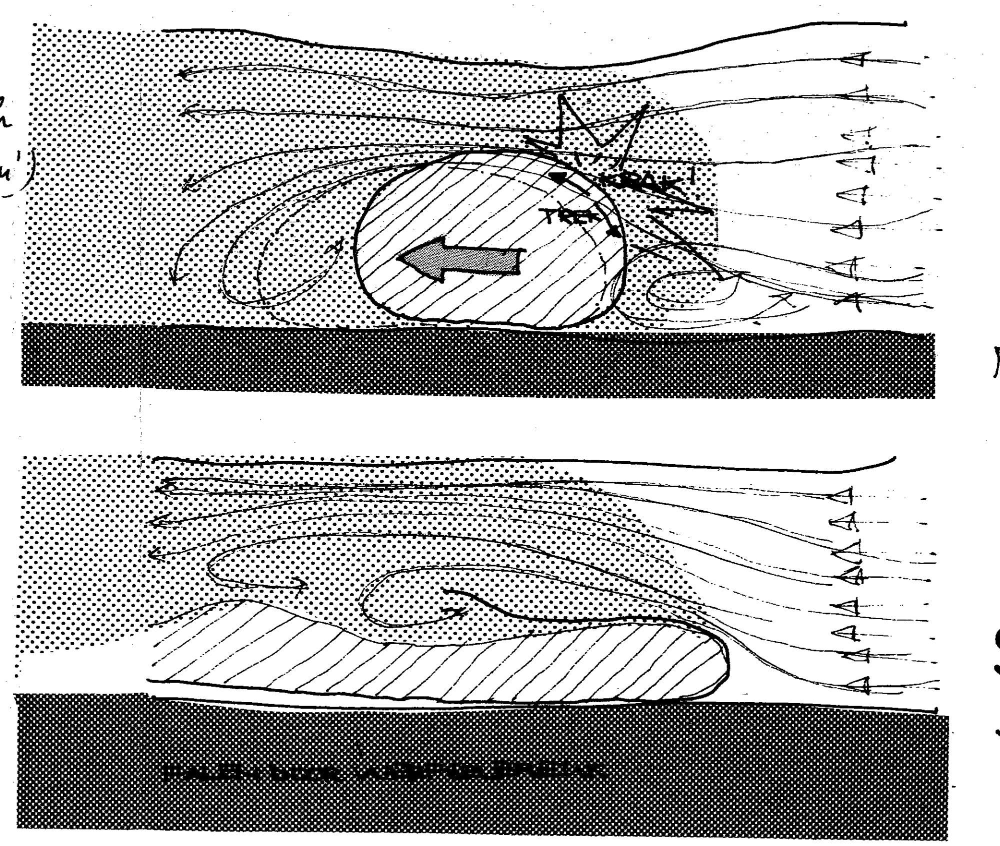
SCHERP VOORWERP
sharp object.........
SPECIAL CONDITIONS

→ influence not taken into account (repair should be possible)

ultimate limit state: | cause: | requirement:

WEEFSELBREUK

CRACK IN THE TEXTILE



TE GROTE KRACHT
loadings..............
EXREME CONDITIONS

- GOLVEN (waves) → dynamic strength $\Delta N\phi = 100 kN/m'$ ($\gamma = 1$)
- STROMING (currents) → additional testing necessary (probably minor influence)
- WIND (wind) → additional testing necessary (probably minor influence)
- PLAATSING (stationing) → gravitational loading, static strength ($\gamma = 2$) dependent on construction (loading: $N\phi = 125 kN/m'$)

VERMOEIING
fatigue.......
NORMAL CONDITIONS

- GOLVEN (waves) → additional testing necessary; ($\Delta N\phi \approx 50 kN$/wave) strength after 5 years must be 350 kN/m'
- STROMING (currents) → additional testing necessary strength after 5 years must be 350 kN/m'
- WIND (wind) → additional testing necessary strength after 5 years must be 350 kN/m'
- ZON (sun) → UV stability of five years: strength after 5 years must be 350 kN/m'

SCHERP VOORWERP
sharp object........
SPECIAL CONDITIONS

→ influence not taken into account (repair should be possible)