DESIGN ASPECTS

OF

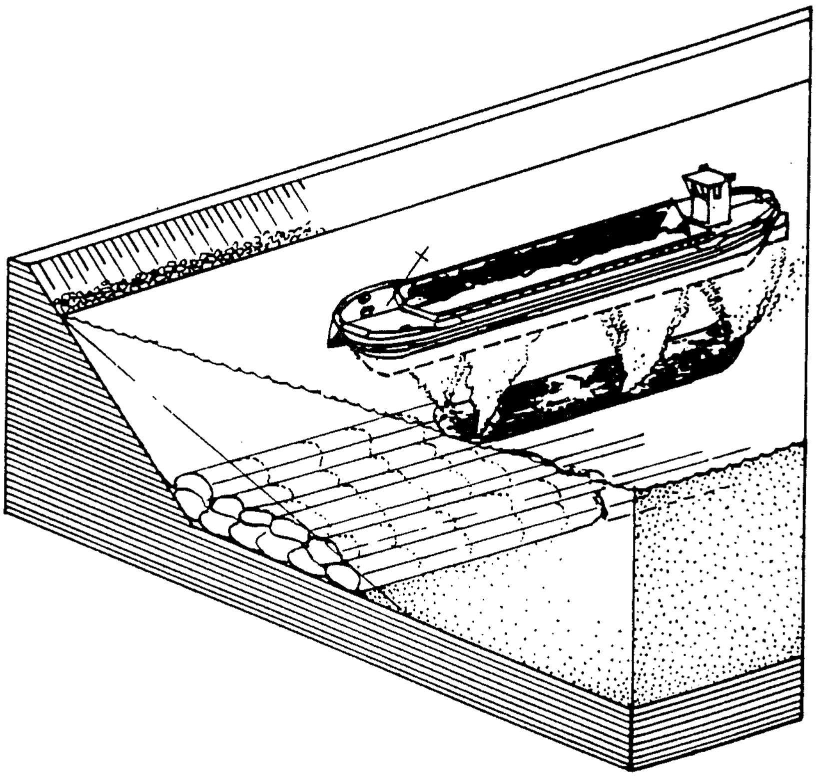
GEOTUBES AND GEOCONTAINERS

What we know and what we don't know

(discussion note)

by

Krystian W. Pilarczyk



HYDROpil

Zoetermeer, Netherlands

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General information on Geotubes and Geocontainers

Geotubes and geocontainers (Nicolon patent) hydraulically and/or mechanically filled with (dredged) materials have been successfully applied in coastal engineering in recent years. They can also be used to store and isolate contaminated materials from harbour dredging. Some informations on these systems are given below.

*** Tube system.**
Geotube is a sand/dredged material filled geotextile tube made of permeable but soil-tight geotextile. The desired diameter and length are project specific and only limited by installation possibilities and site conditions. The tube is delivered to the site rolled up on a steel pipe. Inlets and outlets are regularly spaced along the length of the tube. The tube is filled with dredged material pumped as a water-soil mixture (commonly a slurry of 1 on 4) using a suction dredge delivery line (Figure 1). The choice of geotextile depends on characteristics of fill material.

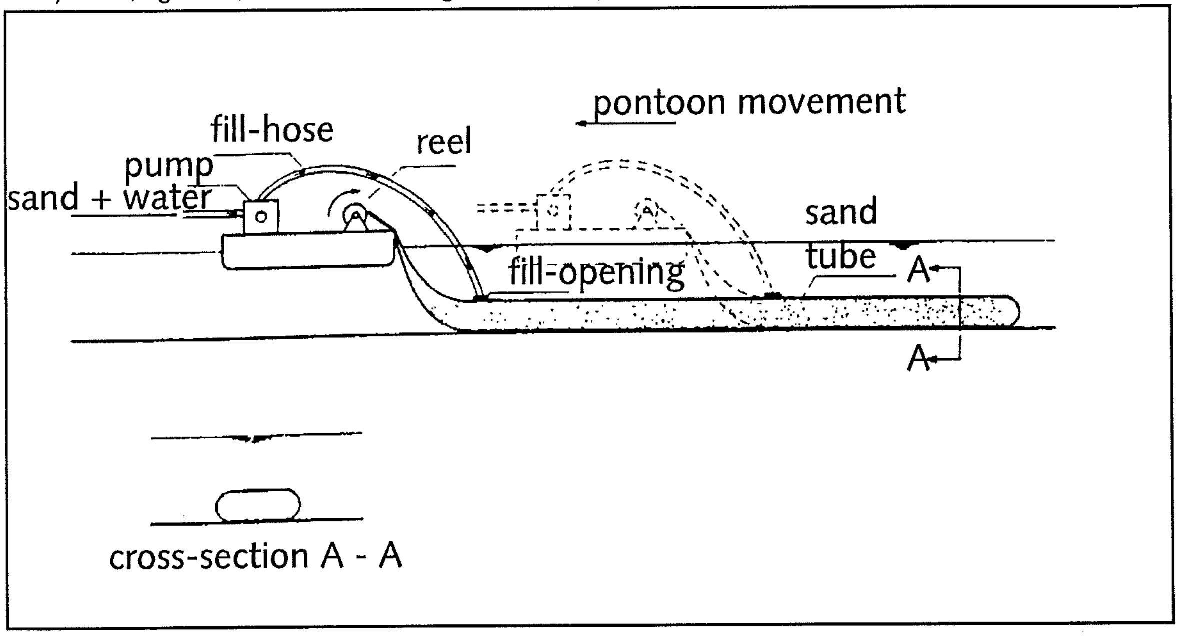


Figure 1 Filling procedure of Geotube

The tube will achieve its desired shape when filled up to about 80%; a higher filling grade is possible but it diminish the friction resistance between the tubes. The major design considerations include sufficient geotextile and seam strength to resist pressures during filling and placement impact, and fabric/soil compatibility. Additionally, long-term U.V. resistance, resistance to abrasion, tearing and puncturing (including vandalism), and tube flattening resulting from the consolidation of sediments within the tube.

Tubes can be filled on land (e.g. as dikes for land reclamation, bunds, toe protection or groyns) or in water (e.g. offshore breakwaters, sills of perched beaches, dikes for artificial islands or interruption of gullies caused by (tidal)currents). The tube is rolled out along the intended alignment with inlets/outlets centered on top. When a tube is to be placed in water, the effects of buoyancy on the tube geotextile prior to filling as well as on the dredged material's settling characteristics must be considered. In order to maximize inlet/outlet spacing, an outlet distant from the inlet may be used to enhance the discharge of dredged slurry and thereby encourage and regulate the flow of fill material through the tube so that sufficient fill will flow to distant points.
Commonly, the filter geotextile (against scour) and flat tube are fully deployed by floating and holding them in position prior to beginning the filling operation. The filter geotextile is often furnished with small tubes at the edges when filled with sand holds the filter apron at place. This apron must also extend in front and behind the unit, commonly 1-2 times the filled unit width.

*** Container systems.**

Geocontainer is a mechanically-filled geotextile and "box" or" pillow" shaped unit made of a soil tight geotextile. The containers are partially prefabricated by sawing mill widths of the appropriate length toegether and at at the ends to form an elongated "box". The "box" is then closed in the field, after filling, using a sewing machine and specially designed seams. Barge placement of the site-fabricated containers is accomplish using a specially configured barge-mounted crane or by bottom dump hoppers scows, or split barges. The containers are filled and fabricated on the barge and placed when securely moored in the desired position. Positioning of barge for consistent placement - a critical element of constructing "stacked" underwater structures - is accomplished with the assistance of modern surveying technology.

These large containers are applied, among others, for foreshore erosion control along the river Old Meuse in the Netherlands (200 m^3 site-fabricated, sand-filled geotextile containers) (Rijkswaterstaat-Nicolon, 1988). A similar solution is recently also applied for stabilization of Mississippi underwater banks.

Recently (1994), stability tests have been carried out by the Delft Hydraulics using a linear scale (n_L) equal to 20. The tested structures consisted of several layers of parallel geotubes or geocontainers. The so-called 4-3-2 structure had four containers or tubes in the bottom layer, three in the next layer and two in the top layer.

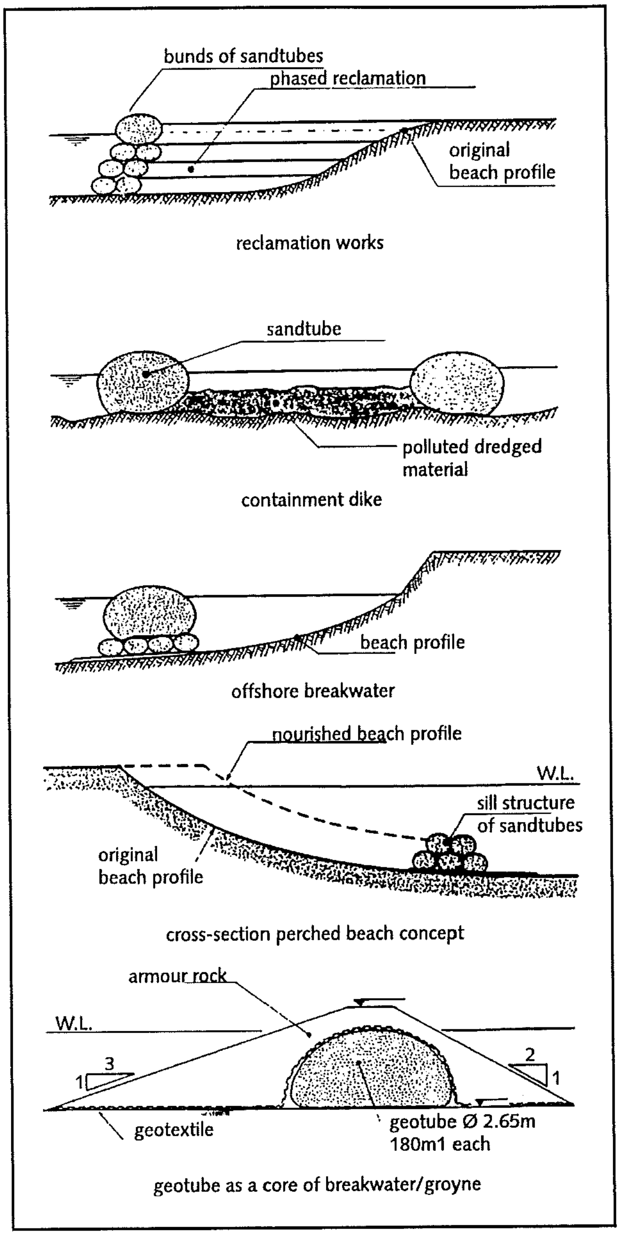


Figure 2. Applications of geotubes

The advantage of these large barge-placed containers include:

* Containers can be filled with locally available soil which may be available from simultaneous dredging activites.

* Containers can be relatively accurately placed regardless of weather conditions, current velocities, tidal movements, or water depths (one of the main advantages in comparison with Longard tubes).

* Contained material is not subjest to erosion during and after placing.

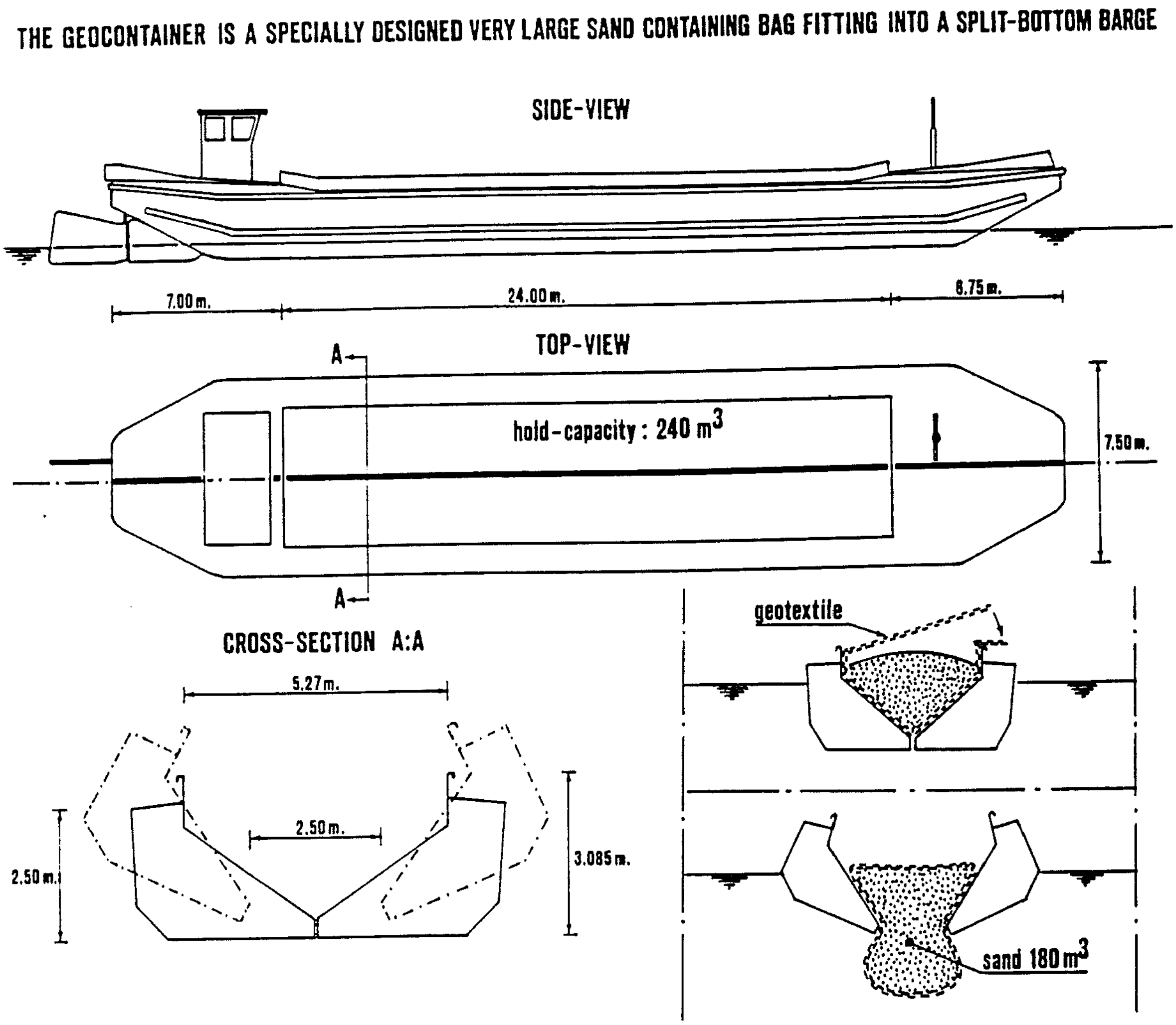
* Containers can provide relatively quick system build-up.

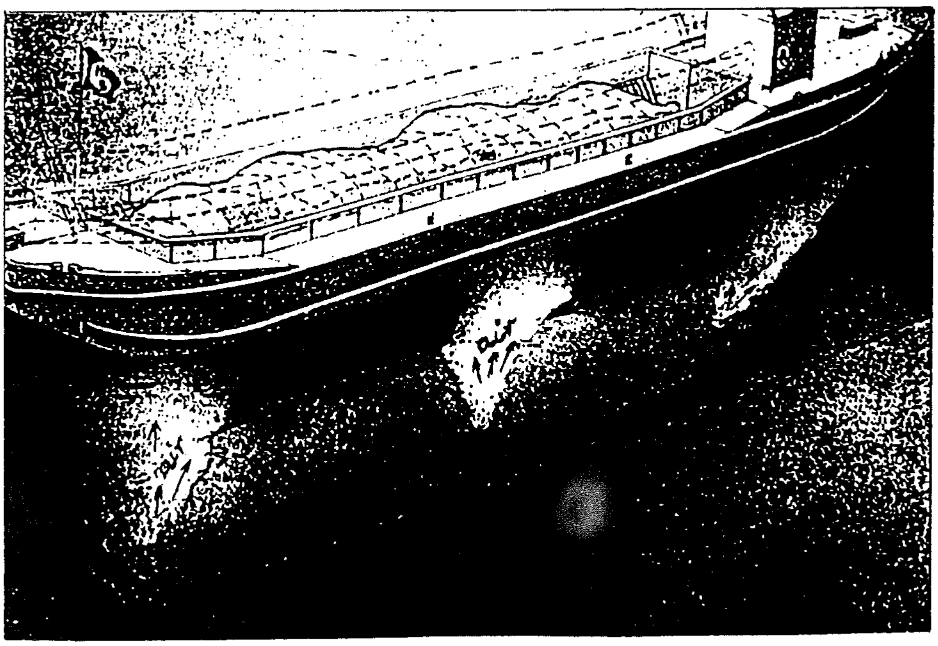
* Containers are, therefore, very cost competetive (for larger works).

When applying Geocontainers the major design considerations/problems are related to the integrity of the units during release and impact (impact resistance, seam strength, burst,abrasion, durability etc.), the accuracy of placement on the bottom (especially at large depths), and the stability. When applying this technology the manufacturer's specifications should be followed. The installation needs an experienced contractor.

Note: More informations on these systems can be found in: K.W. Pilarczyk, "Novel systems in coastal engineering; geotextile systems and other methods", June 1995, Rijkswaterstaat, Road and Hydraulic engineering division, the Netherlands.

Design considerations of geocontainers





Split hull scow is used to place GeoContainers

Figure 3. Geocontainers

Installation and dumping process of geocontainers

In respect to the structural design of geocontainers the following design phases can be distinguished.

Phase I:	preparation
Phase II:	installation and filling conditions
Phase III:	releasing/dumping of container
Phase IV:	impact at the bottom
Phase V:	reshaping of geocontainer and stabilization (final position and shape)

Phase I: Preparation

In preparation phase special attention should be paid to such items as: project requirements incl. environmental aspects (i.e. acceptance of damage of geocontainers in respect to its consequences), type of fill-material and specifications, choice of geotextile in respect to the soil tightness, permeability and strength, installation equipement and fill-procedure (hydraulic or conventional), transport, positioning system, collecting information from previous experiences, consideration in respect to model/prototype testing, etc. The basic installation procedure of geocontainers is outlined schematically in Figure 4 and illustrated in Figure 5.

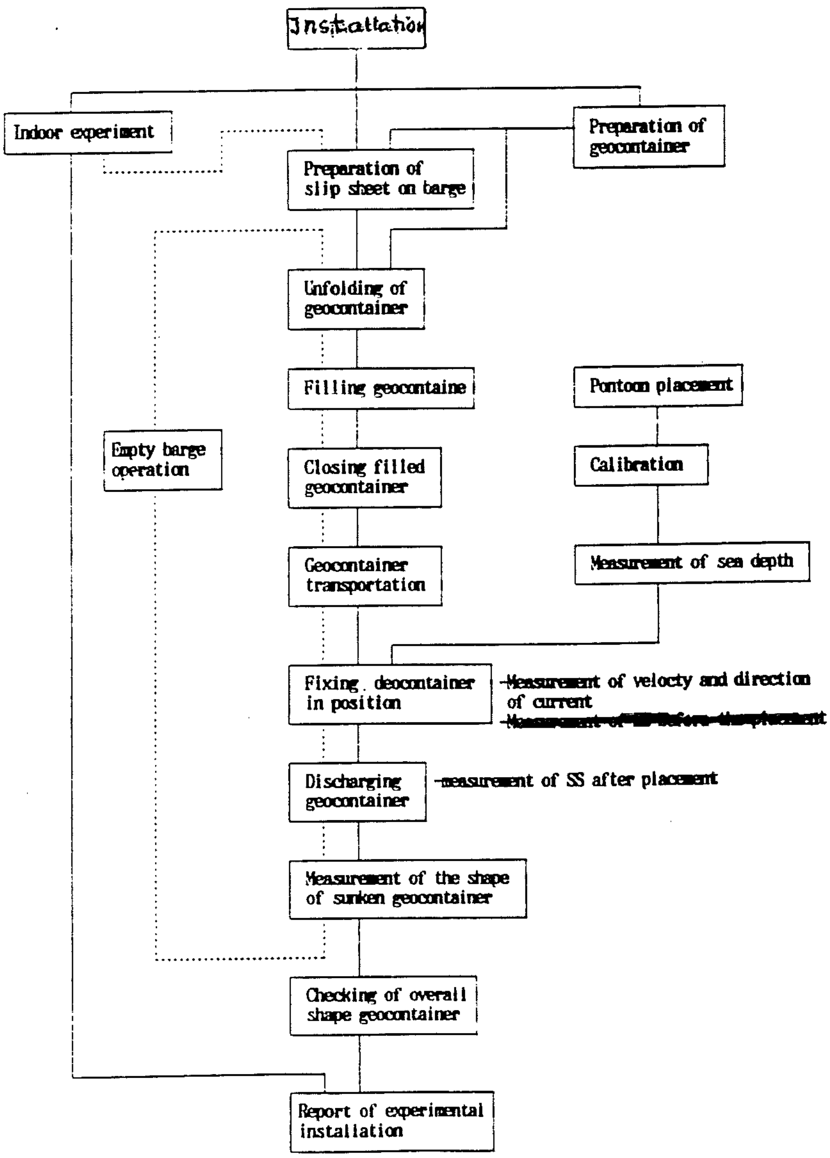
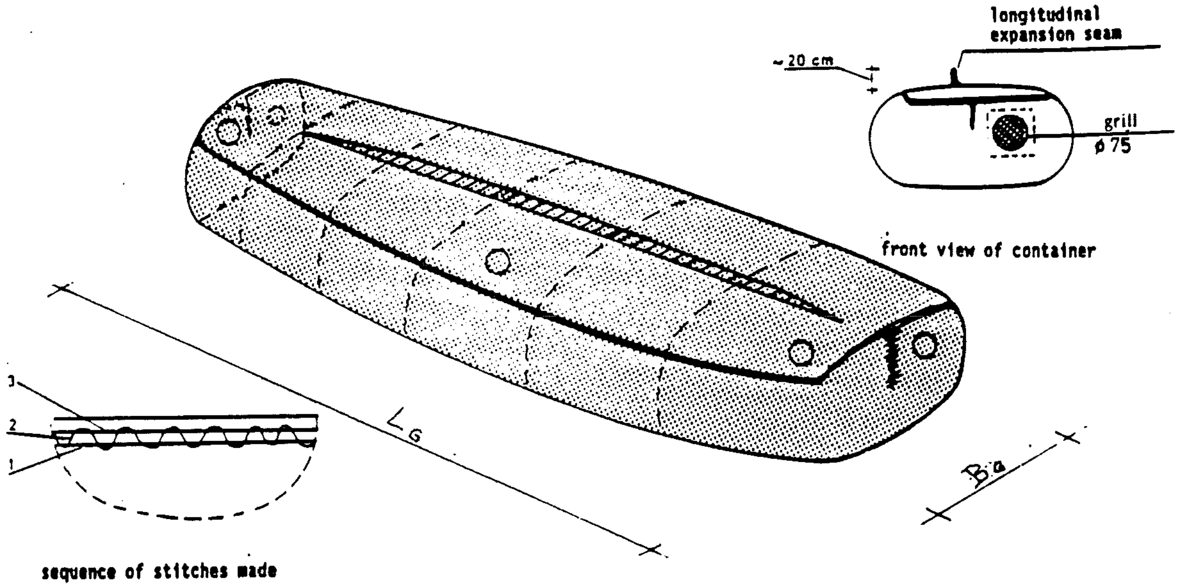


Figure 4. Flow chart of installation of geocontainers



container with 5 grills and longitudinal expansion seam

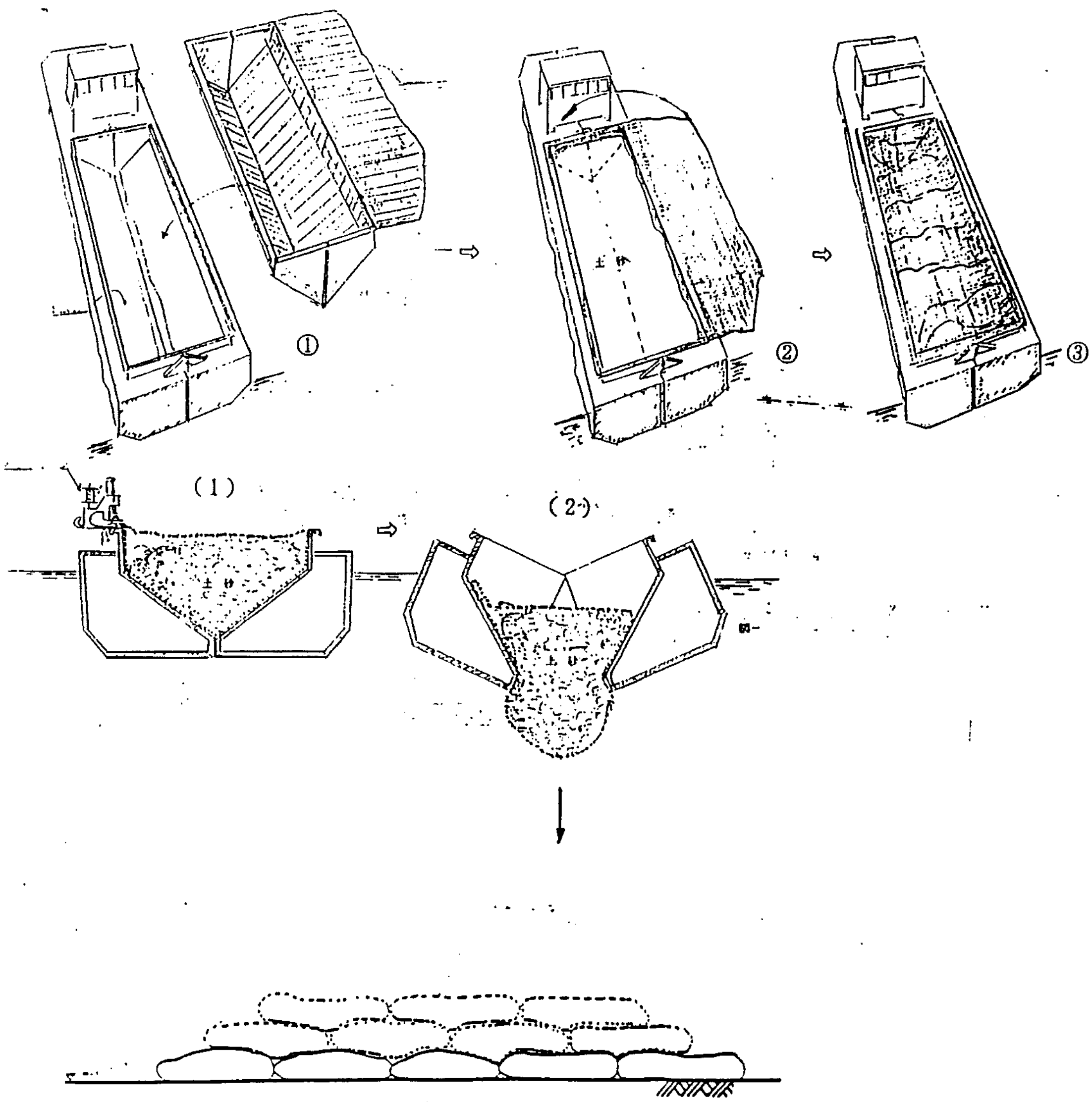


Figure 5a. Illustration of installation procedure of geocontainers

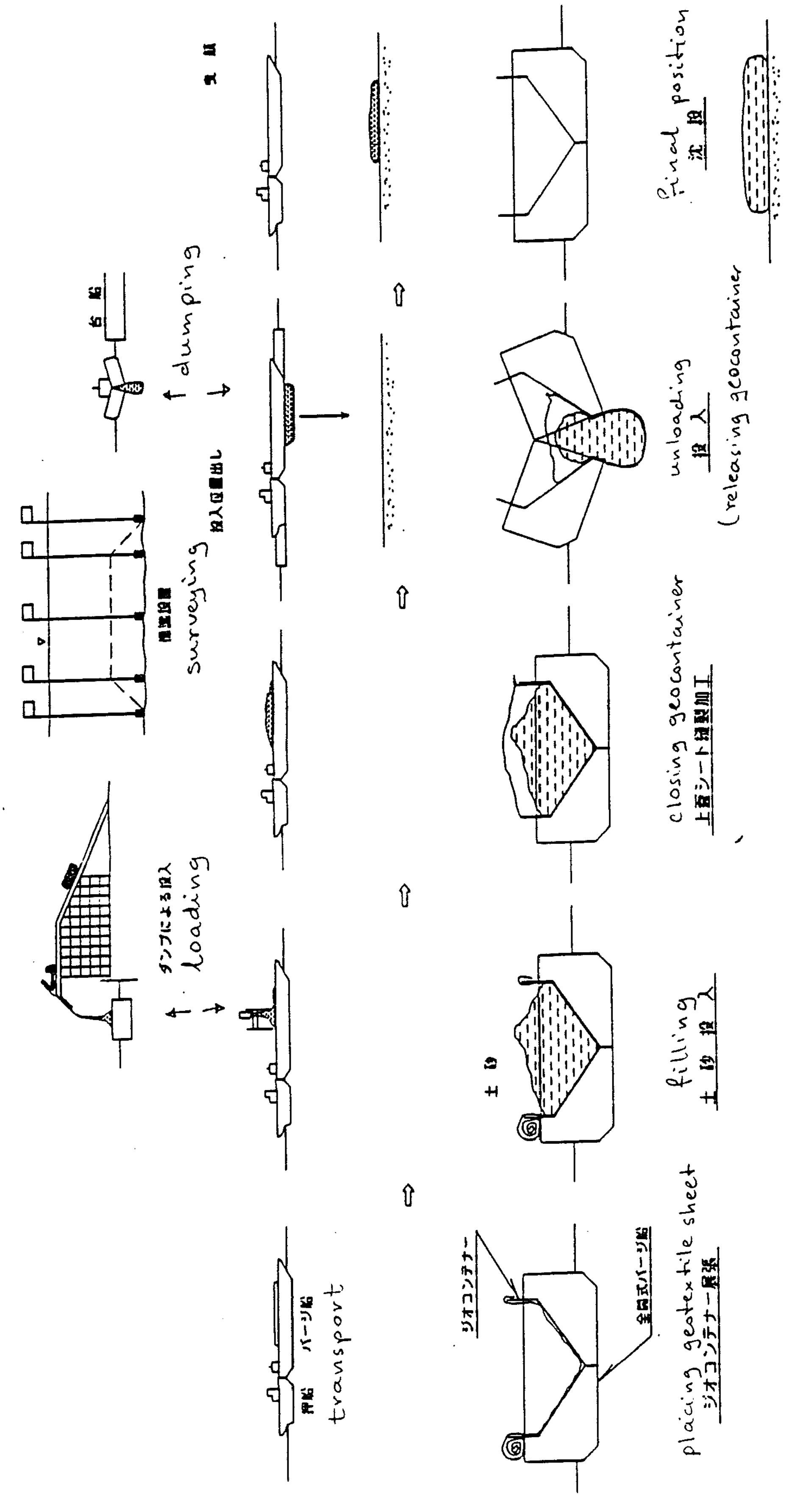


Figure 5b. Installation procedure of geocontainers

Phase II: installation and filling conditions

Container can be loaded in a barge in a harbour by using a tip truck and a loading chute and transported to the dumping site, or directly at the dumping site from the positioning pontoon equipped with loading facilities (i.e. crane) where the filling material (soil) is provided by barges. In specific projects the direct hydraulic filling with dredged material is also possible.
Soil is loaded onto the barge with a geocontainer sheet already spread out in the bin.
The soil shoud be distributed as evenly as possible along the bin.
After the filling is completed, the cover sheet of the geocontainer will be sewn using a hand sewing machine. Sewing should be done very carefully to prevent the geocontainer from being torn, which would result in the spillage of the soil during dumping and settling on the bed.
The seam strength is normally the weakest link in the design and, depending on the seaming technique specified, this value may be only 50 to 70 percent of the fabric's ultimate strength. Therefore, the strength of seams should be used as a reference strength in the design in respect to possible exerted forces.

In order to prevent tearing of the geocontainer in contact with some projections inside the bin and to facilitate the smooth unloading (dumping) of the container from the split barge, a slipsheet of geotextile is mostely placed inside the barge to decrease friction. Hovever, there will be always some friction providing some forces on geocontainer-sheet during opening of split barge and releasing the geocontainer.

When precise dumping is required, the split barge will be fixed to the mooring pontoon and will be moved to the required dumping position using the positioning facilities on the pontoon. After positioning is fixed, the split barge will be open and the geocontainer will be released. An example of positioning system is shown in Figure 6.

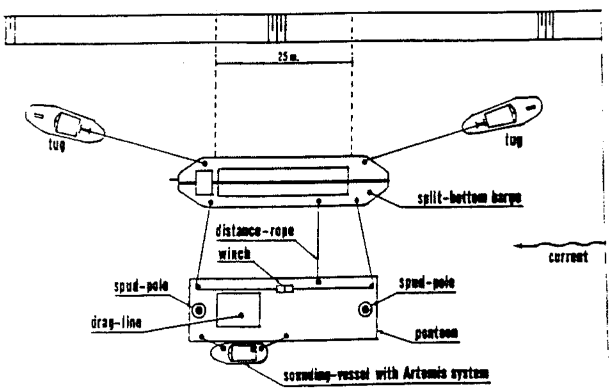


Figure 6. Positioning of split barge

The reqired perimeter of geotextile sheet must be sufficient enough to release geocontainer through the given split width b_o for a required cross-sectional area of material in the bin of barge A_f (or filling-ratio of fill-material in respect to the max. theoretical cross-section). The derivation of the required minimum length of perimeter of geotextile sheet is shown below.

Required perimeter of geotextile sheet is S_o = ?

A_o = total cross-sectional area (theoretical max.)

ϕ = filling-grade ratio ≤ 1

A_f = required cross-section of fill-sand

(or Volume = A_f L; L = bin-length of a barge)

Consider a unit passing an opening 'b_o' (Figure 7):
(assuming a rectangular form of a passing unit)

$A_f = b_o\, a \rightarrow a = A_f/b_o$

and the perimeter is equal to:

$S = 2\,(a + b) = 2\,(A_f/b_o + b_o) = S_{minimum}$

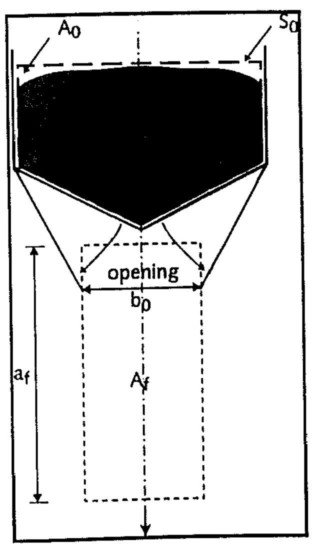


Figure 7

Practical requirement:

$S_o > S_{min}$

It is recommended to use:

$S_o = (1.25 \text{ to } 2)\ S_{min} = (2.5 \text{ to } 4)\ (A_f/b_o + b_o)$

Factor 1.25 (as minimum) up to 2 is necessary (it means somewhat longer perimeter = more free space) to avoid jamming of the unit during passing of the opening due to the discrapancy between the assumed rectangular (theoretical) form and the real one (Figure 8), but also because of uneven filling and/or friction along the bin length. Therefore, for a given perimeter S_o, the required filling-ratio ϕ will be less than 0.8 (mostly 0.3 to 0.5).

A part of this additional length is used to make pleats along the bin sides for easier sliding of geocontainer. However, ther must be enough free length (pleats) at the top of geocontainer to avoid jamming in the last phase of releasing.

The real form of falling unit is influenced by the width of opening b_o. For large (and quick) opening the falling shape will be close to a rectangular one. For small opening the shape will be similar to the shape of water-drop or cone shape (see Figure 8).

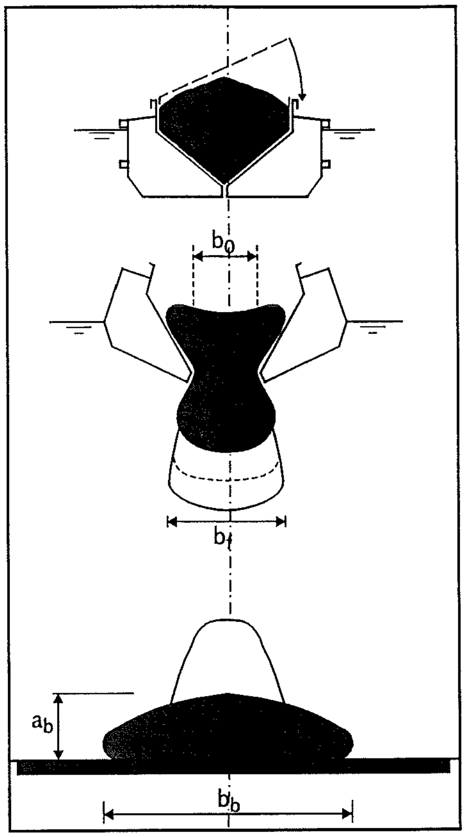


Figure 8

The bottom width (b_f) of a falling unit can be described by:

$1 \leq b_f/b_o \leq 2$

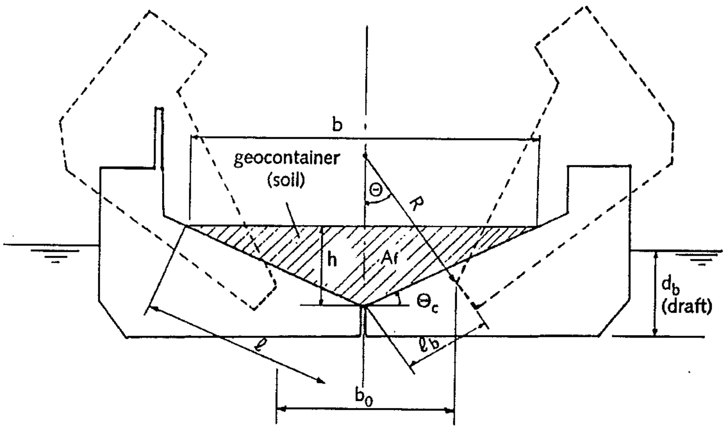
Phase III: releasing/dumping of geocontainer

During passing the split opening the geocontainer sheet must resist the clinching forces exerted at the split edges due to the weight of the already passed soil and the friction and jamming forces of the remaining upper part of soil (see Figure 8). The magnitude of such force can be in order of the total weight of geocontainer per perimeter of split opening or $[0.5\ g\ (\rho_s-\rho_w)\ A_f]$.

However, an uneven friction distribution during dumping (especially in the length direction) may increase these forces considerably. On the other side, a sudden opening of split barge up to the final width can effectively reduce these forces. Of course, the top of the geocontainer, which incorporates a large surplus of geotextile will be tensioned only (not till) at the last moment of release of geocontainer. This surplus of geotextile sheet might be larger than needed for release of the upper part of geocontainer. In such case, the free space consists only air and can perform as a balloon during sinking.

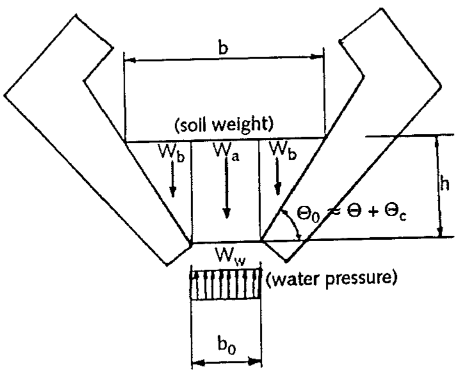
A schematic description of friction and tensile forces in geotextile during the release of container from the bin of the barge is given hereafter. For small geocontainers (and proper provisions inside the bin) these forces are mostly lower than the impact forces. However, in case of larger geocontainers these forces can be decesive for the proper choice of strength of geotextiles. This is illustrated in Figure 11 for a geocontainer with capacity (volume) of 500 m^3 of soil.

Friction and tensile forces in geotextile during release of geocontainer



Capacity (volume) of geocontainer: $V = 0.5\ L\ b\ h$
where: L = total length of geocontainer, b = top-width , and h = depth of soil.
The touch (side) length at the bottom 'l' is equal to: $l = \sqrt{h^2 + (0.5\ b)^2}$
and the split-opening width b_o as function of radius of rotation (R) and angle of opening (Θ) is given by: $b_o = 2\ R\ \sin\Theta$

Figure 9. Schematization of bin for calculation of forces



$$W_a = b_o\ h\ \gamma_s$$

$$W_b = 0.5\ (b - b_o)\ h\ \gamma_s = 0.5\ h^2\ \text{ctg}\Theta_o\ \gamma_s$$

$$h = 0.5\ (b - b_o)\ \tan\Theta_o$$

$$A_f = V/L = 0.5(b + b_o)\ h = 0.25(b^2 - b_o^2)\ \tan\Theta_o$$

or

$$b = \sqrt{4\ A_f\ \text{ctg}\Theta_o + b_o^2}$$

Equilibrium of the rectangle part in vertical direction is:

$$T_v + P_o \tan\phi = 0.5(W_a - b_o w_w)$$

or

$$T_v = 0.5(W_a - b_o w_w) - P_o \tan\phi$$

and

$$T = \{0.5(W_a - b_o w_w) - P_o \tan\phi\}\ \text{cosec}\Theta_o$$

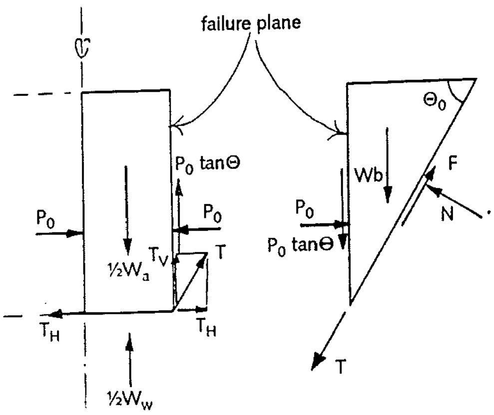


Figure 10.
Acting forces in the bin

Equilibrium of the triangle in direction of friction force F is:

$$F = W_b \sin\Theta_o - P_o \cos\Theta_o + P_o \tan\phi \sin\Theta_o + T$$

Equilibrium of the triangle in direction of normal force N is:

$$N = W_b \cos\Theta_o - P_o \sin\Theta_o + P_o \tan\phi \cos\Theta_o$$

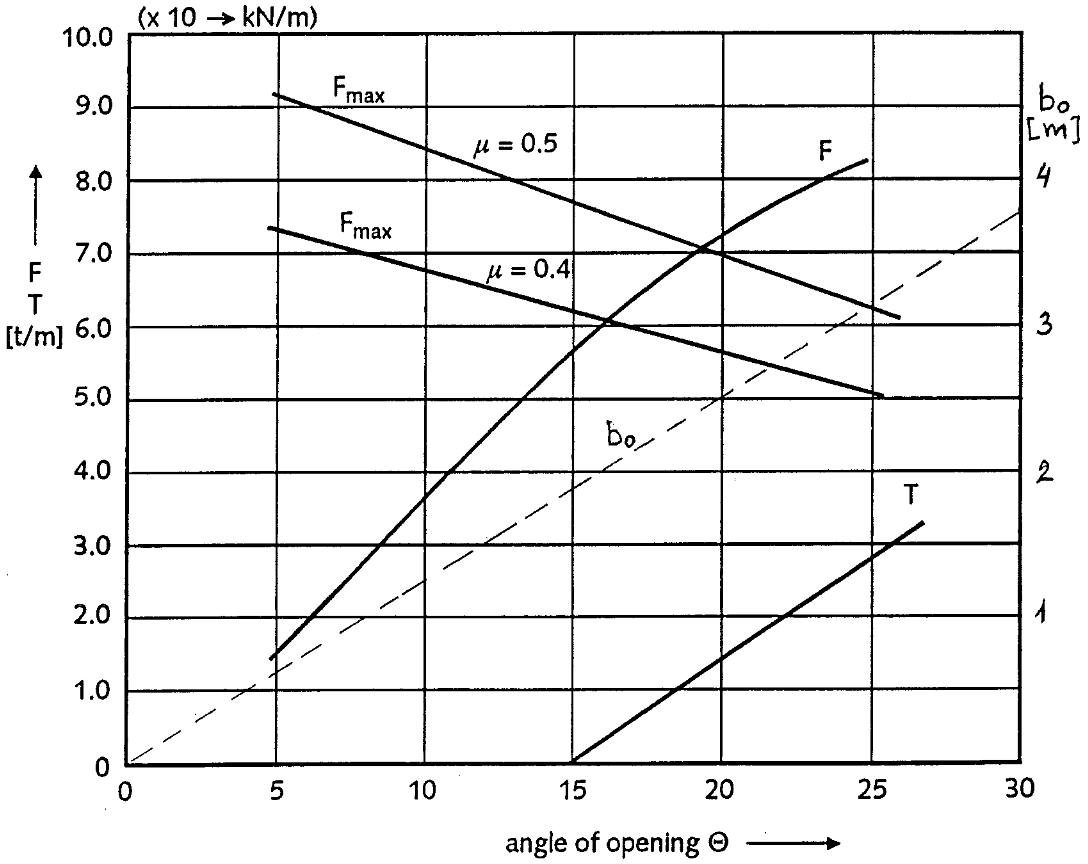
where P_o is the static earth pressure equal to: $P_o = 0.5\ K\ \gamma_s\ h^2$, W_w is the water pressure in function of loaded draft d_b, F is the friction force, T is the tensile force in geotextile, K = coefficient of static earth pressure, and ϕ the angle of internal friction.

The criterion is that the friction force F cannot exceed μN in which μ is the friction coefficient between geocontainer and bin of barge. Thus,

$$F_{max} = \mu\ N$$

At the moment that F exceeds F_{max}, sliding along the barge starts and the maximum tensile force is reached.

In the graph in Figure 11 the indicative results for forces F, F_{max} and T are presented for two values of μ. For larger friction coefficient (μ) the container will release the bin at larger opening of the split (larger Θ_o).



Note on calculation conditions and assumptions:

1) Barge: bin-width B = 19.4 m, L = 22 m, loaded draft d_b = 3.1 m, average loaded cross-section of geocontainer (soil) A_f = 22 m^2,
2) Geocontainer 500 m^3
   Angle of internal friction; ϕ = 30°
   Unit weight of soil; γ_s = 1.6 t/m^3
3) No water penetration during opening
4) Friction at both ends of geocontainer is neglected

Figure 11. Relationships between friction and tensile forces as function split opening

After start of the opening of the hold of the split barge the underside of the geocontainer starts moving through the opening of the split barge. A part of the geocontainer is hanging under the split barge. At a certain width of the split opening the whole geocontainer falls through the opening and the falling speed increases rapidly. From this moment the geocontainer is free falling. When the sand in the container is not uniformly distributed in longitudinal direction, the container will mostly drop firstly from the side with the least load. Therefore, it is advised to fill the container with a little more sand at the both ends than in the middle. It will stimulate the horizontal sinking of the geocontainer.

The soil inside a container can be of various consistency. In case of hydraulic filling it will be a fully saturated soil (ρ_s = 2000 kg/m^3). In case of filling by relatively dry sand (with normal moisture), the main soil body will have a bulk density of about 1600 kg/m^3. However, because there is always some leakage of water through the bottom split, the lower part of geocontainer soil will be probably saturated. The dumping process is rather short (a few seconds), therefore, one may assume that this initial soil consistency remains nearly the same at the moment of impact with the bottom.

In both cases, there will be during a dumping process a certain air pocket at the top of the geocontainer, providing an additional buoyancy which may have influence on fall-velocity and thus, on impact forces with a bottom.
However, this problem is much serious in case of filling by relative dry sand. In that case approximately 40 % of air is contained in the pores of the sand, and between the sand and the top fabric (i.e. container with 200 m^3 of dry sand may contain up to 80 m^3 of air). During dumping, one or two large air pockets will be formed, which very often may cause the top seams to spring open. The reason for this is that the fabric, although sandtight and water permeable, is no longer able to release such a big quantity of air momentaril, and musr therefore be required as relatively 'airtight'. It appeared to present major problems, particularly in the case of the thicker types of fabrics.

It is also possible that containers which remain intact during dumping then may collapse during impact with the bottom. The reason for that is also the large quantity of air which cannot be removed immediately during the 'change of shape' which the container undergoes after impact.
Prototype observations showed that if the container was already collapsed during dumping (mostly on of the sealing/closing seam), no further damage was found after impact. Where collapse occurred on the bed, damage was found on seams at the ends and/or in the centre; in most cases this damage was caused to the sealing seam.
The possible collapse modes during dumping are illustrated in Figure 12.

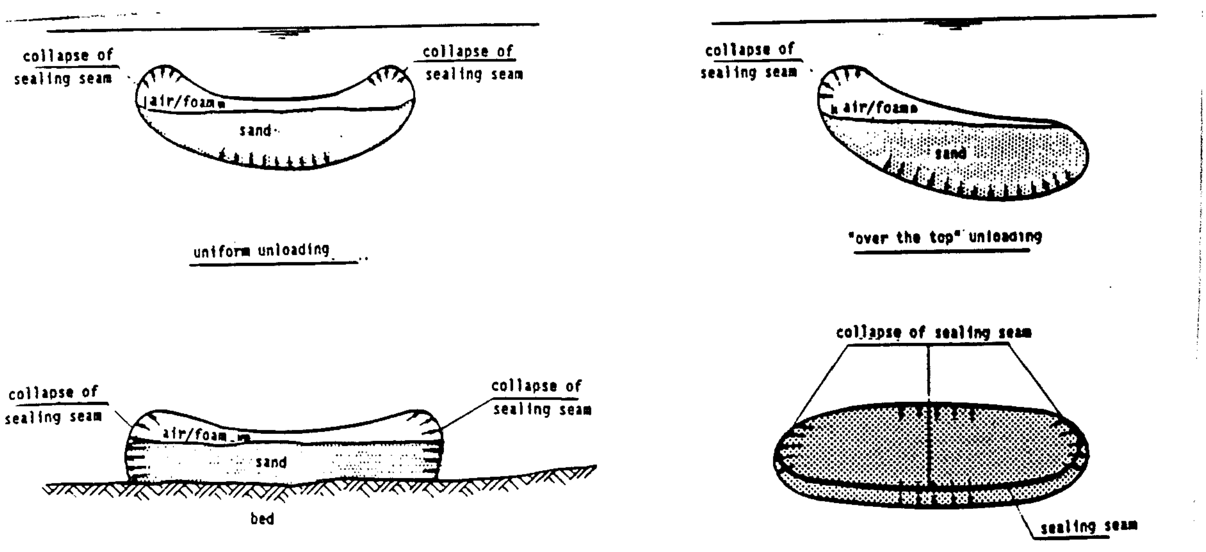


Figure 12. Possible collapse modes of geocontainers during dumping

To avoid these failures the strong seams, special grills ('outlet valves/air vents' for air release at both ends of geocontainer), and additional expansion seams with a proper capacity should be provided on the top of geocontainer sheet. It may also help to overcome the 'air problem' by placing the bin under water after sealing the container. It may help to avoid air problems during dumping, and also to create conditions for more even placing, however, the replacing of air by water will increase the rate of sinking (fall-velocity), and thus also the impact from landing on the bottom. In this case the high momentaneous impact pressure will be transmitted by water into geotextile which is too tight for immediate release of this pressure.

The impact forces with the bottom are of function of fall-velocity (dump velocity) of a geocontainer. The derivation of the fall-velocity is described hereafter.

Fall (Dump) velocity

The acting forces on the geocontainer are the gravitational force, directed downward, and the flow resistance force, directed upward.

The gravitational force:

$$F_g = Vol\ (\rho_s - \rho_w)\ g$$

with:

F_g	= gravitational force	[kN]
Vol	= volume of geocontainer	[m^3]
ρ_s	= specific density fill material	[kg/m^3]
ρ_w	= specific density water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]

The flow resistance force:

$$F_r = \frac{1}{2}\ A\ \rho_w\ C_d\ V^2$$

with:

F_r	= flow resistance force	[kN]
A	= flow catching surface area geocontainer	[m^2]
ρ_w	= specific density water	[kg/m^3]
C_d	= drag coefficient	[-]
V	= velocity of geocontainer	[m/s]

The velocity of the geocontainer will increase after the release from the split barge. The increase of the velocity is given in the following formula:

$$dV = \frac{(F_g - F_r)}{Vol\ \rho_s}\ dt$$

with:

dV	= increase of velocity	[m/s]
F_g	= gravitational force	[kN]
Fr	= flow resistance force	[kN]
Vol	= volume of geocontainer	[m^3]
ρ_s	= specific density fill material	[kg/m^3]
ρ_w	= specific density water	[kg/m^3]
g	= gravitational acceleration	[m/s^2]
dt	= time step	[s]

The equilibrium velocity is reached, when the gravitational force equals the flow resistance force. This is the maximum velocity (it can be in order of 4 to 7 m/s for common containers).

$$V_{max} = \sqrt{\frac{2\ Vol\ (\rho_s - \rho_w)\ g}{A\ \rho_w\ C_d}}$$

with:

V_{max}	= equilibrium velocity	[m/s]
ρ_s-ρ_w	= submerged (bulk) density fill material inside geocontainer	[kg/m^3]
	(ρ_s = 1600 for dry sand and 2000 for saturated sand)	

Very important with respect to the simulation of the velocity is the cross directional shape of the geocontainer during the dump. The A, C_d and free falling height are determined by the shape of the geocontainer during the dump. In this theoretical simulation a horizontal orientation of the geocontainer is assumed. The shape of the geocontainer during the release can be schematized by the following factors:

- Filling of geocontainer, A_f
- Split width, b_0
- Height of geocontainer, a_f

The falling height of the geocontainer is defined as the difference between the underside of the split barge and the sea/river bed. The free falling height is smaller, namely the distance between the underside of the geocontainer and the bed at the moment the speed of the geocontainer starts to increase rapidly. The free falling height is important in order to determine if the geocontainer reached its equilibrium velocity before touch down.

It is assumed that just before the geocontainer is free falling the whole volume of fill is hanging in the geocontainer under the split barge. Assuming a rectangular shape of the geocontainer passing the split opening of b_0 the height of the geocontainer (a_f) can be calculated. The draught of the split barge just before the geocontainer left the hold depends on barge type and its loading. As the shape of the geocontainer is not exactly known during the dump, the flow catching area of the geocontainer A (= b_f*L) and the drag coefficient C_d are not known. The initial Cd value can be approximated to 1 (or 1.2).

As it was already mentioned before the bulk density of the fill material inside the geocontainer incorporates the dry bulk density of the fill material, the water contents in the geocontainer and the air inside the geocontainer on top of the soil which acts as buoyancy. Due to water in the hold (bin) of the split barge during the filling the lowest part of the soil in the geocontainer is saturated. Also the buoyancy of the air on top of the soil in the geocontainer was not determined. Based on the available information the bulk density of the fill material inside the geocontainer may vary approximately from 1600 to 2000 kg/m3 for sand and 1500 kg/m3 for clay.

In practice the velocity of the geocontainer is influenced by several factors that are not incorporated in the model, such as rotation of the geocontainer and non-horizontal orientation during the dump. Further research is necessary on the shape of the geocontainer during dumping in order to assess the drag coefficient, flow catching area and free falling height.
Also it is recommended to assess accurately the bulk density of the fill material.

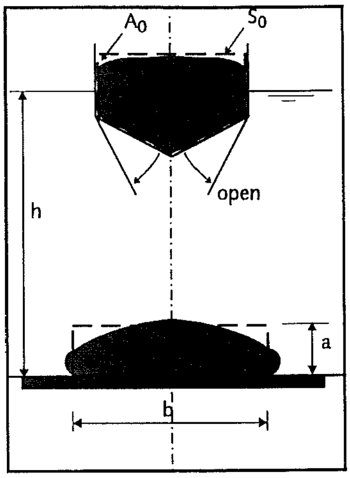
All these partly unknown factors will influence the accuracy of calculation of fall-velocity. However, the available measurements data indicate that this approach provides sufficient accuracy for an initial design.

In cases where higher accuracy is required the large model or prototype tests will provide a proper solution.

The change of shape during the dump and impact is schematized as follows Figures 8 and 13):

1. Original situation

- given perimeter of geotextile sheet $\mathbf{S} = S_o$
- filling grade $\phi \leq 1$
- max. cross-section $\mathbf{A_0} = \varphi S_o^2$



2. Final position

We assume a rectangular shape as an average one (in reality it is a semi-oval or flat triangular shape)

- perimeter $\mathbf{S}$ = constant

- cross-section $A_f = \phi A_o = \phi \varphi S_o^2 = a b$

Figure 13

Solution

$S = 2 (a + b)$

$A_f = \phi A_o = a b; \quad b = \phi A_o/a$

and
$S = 2 (a + \phi A_o/a)$

$2a + 2 \phi A_o/a - S = 0$

$2a^2 - S a + 2 \phi A_o = 0$

$\mathbf{A_0} = \varphi S^2$

$a^2 - S a/2 + \phi \varphi S^2 = 0$

$a = 0.5 \{S/2 \pm \sqrt{(S^2/4) - 4 \phi \varphi S^2}\}$

$a = S/4 (1 - \sqrt{1 - 16 \phi \varphi})$

Assume: $\phi = 0.75$, $A_o = 20m^2$ ($A_f = 15m^2$), and $S = 20m$
thus,
$\varphi = A_o/S_o^2 = 20^2/20 = 0.05 < 0.08$ (max.)
($\varphi = 0.08$ is a maximum for a circular shape)
than,
$a = S/4 (1 - \sqrt{1 - 16 \phi \varphi})$

$a = S/4 (1 - \sqrt{1 - 16\ 0.75\ 0.05})$

$a = S/4 (1 - \sqrt{1 - 0.6})$

$a = 0.0925 * S$

$a = 0.0925 * 20 = 1.85$ m

and

$b = \phi A_o/a = 0.75 * 20 / 1.85 = 8.1$ m

Similar calculation for semi-oval shape provides:

The basic equations are:

$$S = 0.25 \pi (a + b) + b = S_o = \text{const.}$$

and

$$A_f = 0.125 \pi a b = \phi A_o = \phi \varphi S_o^2$$

providing

$$a_{max} = 0.635 S_o (1 - \sqrt{1 - 14.4 \varphi \phi}) \quad \text{(max. height of semi-oval shape)}$$

and,

$$b = A_f/(0.125 \pi a) = \phi A_o/(0.125 \pi a) = \phi \varphi S_o^2/(0.125 \pi a)$$

The average height of the semi-oval shape will be: $a_{average} = A_f/b = 0.125 \pi a_{max}$

For low filling grade (ϕ), the real shape will be more close to rectangular one, while for a high filling grade more close to the semi-oval shape. Therefore the max. height of geocontainer in the final position will be:

$$0.25 S_o (1 \pm \sqrt{1 - 16 \varphi \phi}) < a < 0.635 S_o (1 - \sqrt{1 - 14.4 \varphi \phi})$$

(Close to rectangular shape) (Close to semi-oval shape)

In this way an average number of required geocontainers for a certain cross-section/volume of design structure can be estimated. It has to be stated that this is only true in case the geocontainer is released from the split barge in horizontal orientation over the whole length.
The cross sectional shape of the geocontainer during the various stages of the dump depends on the following factors:

- geometry of the hold of the split barge
- split width
- opening speed of hold
- perimeter of geocontainer
- filling grade of geocontainer
- type of fill material (clay or sand)
- bulk weight of the fill material

The exact position and or shape of geocontainers on the bottom, and the shape of realized structure can be estimated in site by sounding methods and/or divers.

<u>Change of shape of round (circle) geotube on the bottom</u>

Perimeter $S = \pi D = S_o = \text{const.}$ (Figure 14)
Cross-sectional area: $A_o = \pi D^2/4$ (= 100%)

1. When 100% filled ($\phi = 1$)
- the maximum value of a ratio $\varphi = A_o/S_o^2$ is:

$$\varphi = A_o/S_o^2 = (\pi D^2/4)/(\pi D)^2 = 1/(4\pi) = 0.08$$

2. If $\phi < 1$, $\varphi = A_o/S_o < 1/(4\pi)$, the shape at the bottom will be close to the ellipse.

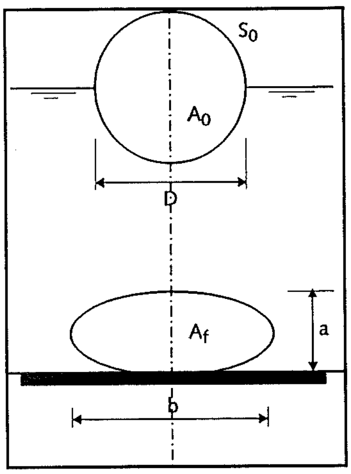


Figure 14

The basic equations are:

$$S = 0.5\,\pi\,(a + b) = \pi D = S_o = \text{const.and}$$

$$A_f = 0.25\,\pi\, a\, b = \phi\, A_o = \phi\,\pi\, D^2/4$$

These equations can be solved in respect to the dimensions of ellipse:

$$a\,b = \phi\, D^2 \rightarrow b = \phi\, D^2/a \quad \text{and} \quad (a + b) = 2D$$

$$(a + \phi\, D^2/a) = 2D \quad \text{and} \quad a^2 - 2aD + \phi D^2 = 0$$

providing $\mathbf{a = D\,(1 \pm \sqrt{1 - \phi})}$

If $\phi = 1$, $a = b = D$ (circular tube) and because $D = S_o/\pi$, thus

$$a = S_o/\pi\,(1 \pm \sqrt{1 - \phi})$$

Assume $\phi = 0.75$, than

$$a = D\,(1 - \sqrt{1 - \phi}) = D\,(1 - \sqrt{1 - 0.75}) = D\,(1 - \sqrt{0.25}) = 0.5\,D$$

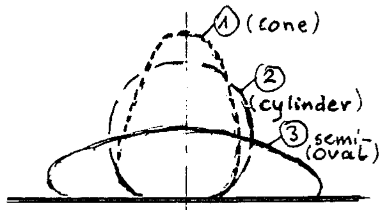
For $\phi = 0.90$ and 0.95 the respective magnitude of 'a' is 0.69 D and 0.78 D. That explains also the final shapes of geotubes and geocontainers; because the filling grade is always less than 1 (less than 100%) the shape will always be oval. Even 95% of filling grade provides 22% reduction of the hight of a unit.
For larger units and low filling grade (ϕ), the real shape will be more close to rectangular one:

$$S_o/4\,(1 \pm \sqrt{1 - 16\,\varphi\,\phi}) < a < D\,(1 - \sqrt{1 - \phi}) = S_o/\pi\,(1 - \sqrt{1 - \phi})$$
(Close to rectangular shape) (Close to elliptical shape)

Phase IV: Impact on bottom (subsoil)

A theoretical model is set up to simulate the transformation of the kinetic energy of the geocontainer into the overpressure inside the geocontainer. During the impact of the geocontainer on the sub soil the kinetic energy will be dissipated. The contributions to the dissipation are:

- internal friction and cohesion of the soil during deformation inside the geocontainer;
- tensile strain of the geotextile;
- grills and expansion seams;
- type (rock, normal soil, soft soil) and settlement of the subsoil;
- friction between subsoil and geotextile;
- escape of air-water through the geotextile during the impact;
- escape of air-water in length direction: 3 dimensional effects.



The shape of the geocontainer during the dump before and after the impact on the sub soil is schematized as in Figures 8 and 13.

During the impact on the sub soil the geocontainer is reshaped from a vertically orientated ellipse into a horizontally orientated ellipse. In the derivation of the theoretical model it is assumed that the geocontainer during the impact is at certain moment cylinder shaped (as transition from drop shape into semi-oval shape, see Figure 5). Also it is assumed that the mass inside the geocontainer is equally distributed throughout.
Just before touch down the whole geocontainer has a certain kinetic energy depending on its velocity (Figure 15).

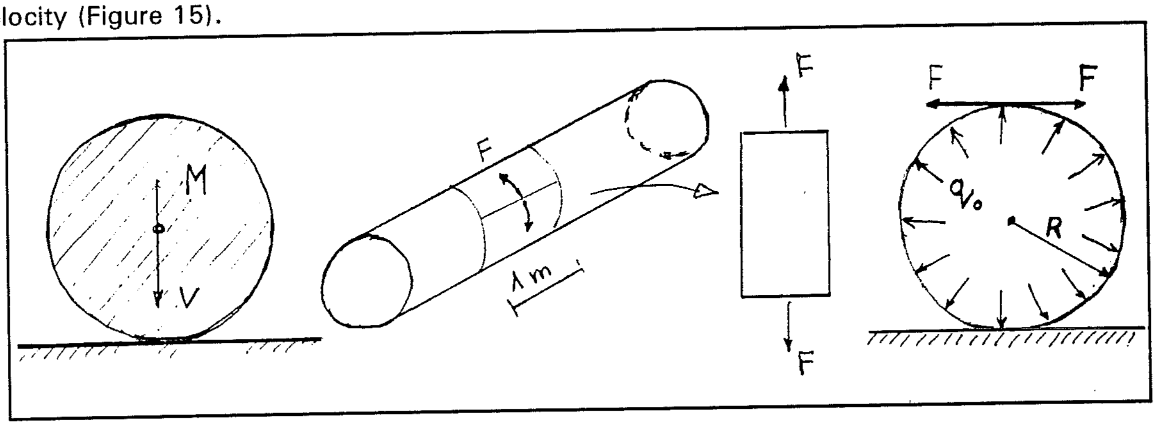


Figure 15. Schematization of impact

$$E_{kin} = \frac{1}{2} M v^2$$

with:

E_{kin}	= kinetic energy	[Nm]
M	= mass of geocontainer	[kg]
v	= fall velocity of geocontainer	[m/s]

The kinetic energy is (partly) absorbed by the strain of the geotextile. The following formula is valid for 1 m width of the geocontainer.

$$E_{abs} = \frac{1}{2} \left(\frac{S}{E}\right) F^2$$

with:

E_{abs}	= absorbed energy by strain geotextile	[Nm]
E	= elasticity modulus of geotextile	[N/m]
S	= perimeter of geocontainer	[m]
F	= tensile force in geotextile	[N/m]

The inside pressure results in a tensile force of the geotextile. Assuming a cylinder shaped geocontainer and a constant pressure along the perimeter the formula presented below is valid (for a cross section of a cylinder with 1 m width).

$$F = q_0 \ R$$

with:

F	= tangential force in cylinder	[N/m]
q_o	= inside pressure	[N/m2]
R	= radius of cylinder ($=S/2\pi$)	[m]
S	= perimeter of geocontainer	[m]

Following from the above presented formulas, the energy absorbed by the strain of the geotextile over the full length of the geocontainer can be presented as follows:

$$E_{abs} = \frac{1}{2} \ (\frac{S}{E}) \ q_0^2 \ R^2 \ L$$

with:

E_{abs}	= absorbed energy by strain geotextile	[Nm]
E	= elasticity modulus of geotextile	[N/m]
S	= perimeter of geocontainer	[m]
q_o	= inside pressure	[N/m2]
R	= radius of cylinder	[m]
L	= length of geocontainer	[m]

Only a part of the total kinetic energy will be translated into strain of the geotextile.

$$E_{abs} = K \ E_{kin}$$

with:

E_{kin}	= kinetic energy	[Nm]
E_{abs}	= absorbed energy by strain geotextile	[Nm]
K	= dissipation factor	[-]

The above formulas result in the formula presented below.

$$q_0 = K^2 \sqrt{\frac{Vol \ \rho_s \ v^2 \ E}{S \ L \ R^2}}$$

with:

q_o	= overpressure inside the geocontainer	[N/m^2]
Vol	= volume of fill inside the geocontainer	[m^3]
ρ_s	= bulk density of the fill material	[kg/m^3]
	(ρ_s = 1600 for dry sand and 2000 for saturated sand)	
v	= velocity at the touch down	[m/s]
E	= stiffness modulus of the geotextile	[N/m]
L	= length of geocontainer	[m]
R	= radius of the geocontainer ($=S/2\pi$)	[m]
S	= perimeter of geocontainer	[m]
K	= dissipation factor	[-]

Prototype verification

This theoretical model can be calibrated with the test results by means of the K-factor. From the prototype tests in the Netherlands in 1994 for two dumps of the geocontainers filled with sand (170 and 130 m^3 resp. in water depth of 18 and 13 m resp.), the K-factor is determined, according to the above given formula.
Each geocontainer had a theoretical volume of 368 m^3 with the following dimensions: length approx. 24.5 m, width approx. 5.0 m, and height approx. 3.0 m; (A_o = 368/24.5 = 15 m^2). The max. split opening was b_o = 2.5 m.
The geocontainers were fabricated from a polypropylene woven geotextile, GEOLON 120. This geotextile has the following characteristics: Mass 630 gr/m^2, Tensile strength (warp and weft direction) 120 kN/m, Young's modulus of elasticity 1000 kN/m, Opening size O_{90} 170 μm, and Permeability 17 l/m^2/s.
Based on the standard width of the geotextile, the geocontainer was constructed of geotextile sections of 5 m. The seams had a strength of 70% of the tensile strength of the geotextile.
On top of the geocontainer 3 reinforced air vents have been created to decrease the expected overpressures inside the geocontainer, which occurs during the dump. Also at the front and rear end such air vents have been constructed. Further two longitudinal expansion seams were made on top of the geocontainer with the purpose to decrease the tensile strain in the geotextile during the dump of the geocontainer.
After filling, the topside is connected to the geocontainer by means of a rope and a handstitch at the front end, the rear end and along one longitudinal side.

The following formula was calibrated:

$$q_0 = K^2 \sqrt{\frac{Vol\ \rho_s\ v^2\ E}{S\ L\ R^2}}$$

with ρ_s = 1600 kg/m^3.

For the first geocontainer the result of the calibration is: K^2 = 0.40 or K = 0.63. This would mean that 60% of the theoretical increase of the pressure is apparent. This would also mean that more than 37% of the kinetic energy is dissipated in another way.
For the second geocontainer the dissipation factor K^2 = 1.17 or K is 1.08. The reason for this higher value can be that in the first geocontainer overpressure could escape because the geocontainer was ruptured before reaching the bottom of the sand pit.
In theory the dissipation factor cannot exceed 1. This could mean that the schematizations in the model are too rough.

Once again it is stated that this theoretical model is a first step towards a more sophisticated model. Therefore it is necessary to perform more tests in order to increase the validity of the model. Possible reasons for the non-validity of the model could be:
- No cylinder shape of the geocontainer
- The fill of the geocontainer is not equally distributed over the cross sectional area
- The measured pressure is not present throughout whole geocontainer at one time but more locally
- The short term elasticity of the geotextile is larger than 1000 kN/m

In similar way the tensile force in the geotextile can be derived from the inside pressure as measured during these tests in the geocontainers.

$$F = q_0\ R$$

with:	F	= tangential force in cylinder	[N/m]
	q_o	= inside pressure	[N/m2]
	R	= radius of cylinder (=S/2π = 2.55 m)	[m]
	S	= perimeter of geocontainer (S = 16 m)	[m]

For the first geocontainer (170 m^3 of sand) an overpressure of 17 kN/m^2 results in a tensile force of 43 kN/m. The second geocontainer (130 m^3 of sand) had to withstand an overpressure of 35 kN/m^2, which results in a tensile stress of 89 kN/m. The tensile strength of the seams of the geocontainer are approximately 70% of the tensile strength of the geotextile: 70% of 120 kN/m = 84 kN/m. It has to be stated that the calculated tensile forces are impact loads. The short term tensile strength of the geotextile is larger than 120 kN/m. Both tensile forces are below the tensile strength of the confection seam.

Geocontainer 2 did not fail during the impact on the subsoil, which could be expected from the above calculation bearing in mind that the short term ultimate tensile force is higher than the 84 kN/m.
Geocontainer 1 failed at the front end during the impact on the sub soil although the tensile force theoretically does not exceed the tensile strength. From the observations during the test it can be concluded that the geocontainer failed because of a reason which is not incorporated in the theoretical model (uneven release of geocontainer resulting in failure of one of the top seams).
It should be stated that the theoretical simulation models are rough schematizations. These models can only be used to give an indication.

Practical note:
The prototype experience indicate that geocontainers with volume up to 200 m^3 and dumped in water depth exceeding 10 m have been frequently damaged (collapse of seams) using geotextile with tensile strength lower than 75 kN/m, while nearly no damage was observed when using the geotextile with tensile strength equal or more than 150 kN/m. This information can be of use for the first selection of geocontainers for a specific project (is an accidential damage acceptable or not).

Phase V: reshaping of geocontainer and stabilization (final position and shape)

In the previous section the forces and stresses in geotextile just direct after impact with bottom were calculated (assuming a cylindrical transitional shape). These are probably the maximum momemtaneous forces/stresses acting on geocontainers. However, as it was already stated before (see also Figure 8) the final shape of geocontainer is close to a semi-oval or flat triangular one. The forces during reshaping to the final position can be roughly approached in the following way (Figure 16). We assume here that the soil in the geocontainer is saturated and behaves as a very dense-fluid jet (ρ_s = 2000 kg/m^3) with the mass M and impacting a bottom with velocity **v**.

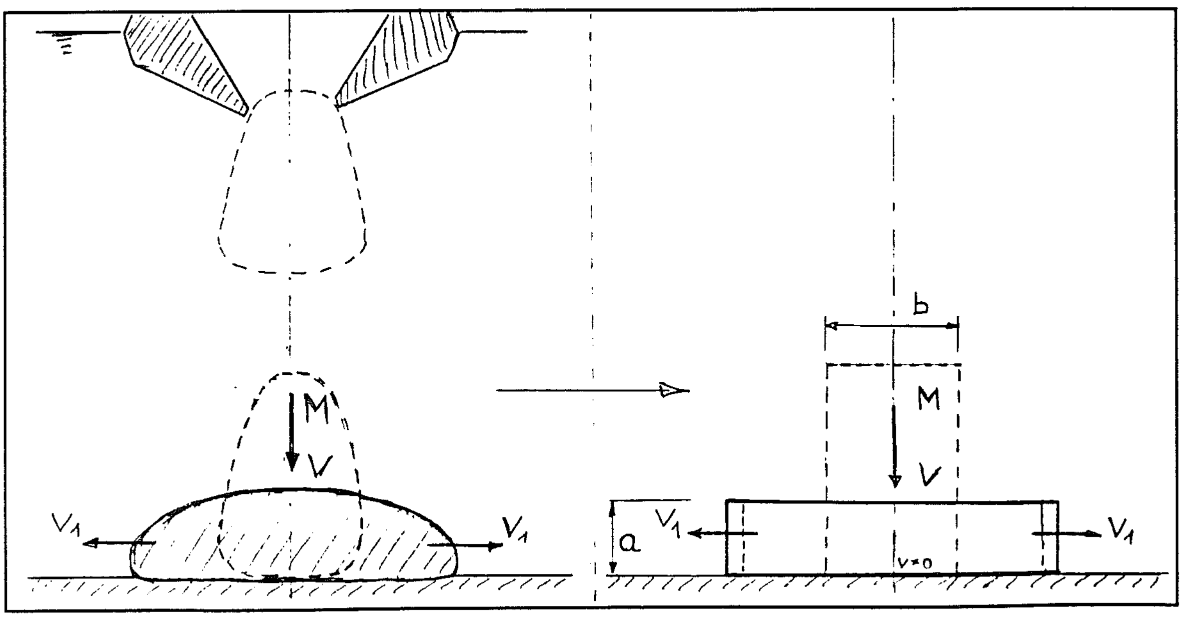
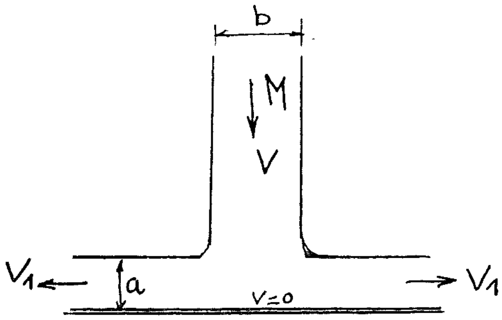


Figure 16. Mathematical schematization of reshaping of geocontainer

The impact energy is equal to (0.5 M v). This energy will be used for reshaping of geocontainer untill the stationary position is reached. Because of the liquefied conditions of soil one may assume that this process will take place nearly without initial dissipation of energy. After impact this soil will try to spread on both sides with velocity v_1 and thickness 'a', within the limitations of the perimeter of the geocontainer (see Figure 16).
Applying the energy conservation equation one may obtain for the energy in the horizontal jet the following result:



$$E_{1,left} = E_{1,right} = (1/12)\ M\ v^2_1$$

and, from the energy balance

$$E_{1,left} + E_{1,right} = E_{total}$$

$$2\ (1/12)\ M\ v^2_1 = (1/2)\ M\ v^2$$

and,

$$v_1^2 = 3\ v^2 \quad \text{or} \quad v_1 = \sqrt{3}\ v$$

On the other side, applying equation of momentum continuity, one may obtain the average thickness of a horizontal jet 'a' as function of the thickness of vertical jet 'b' (thickness of geocontainer just before impact):

$$b\ v = 2\ a\ v_1$$

or

$$a = b\ v/(2\ v_1) = b/(2\sqrt{3}) = 0.289\ b$$

(as a first approximation the thickness 'b' can be assumed equal to the split opening of the barge b_o).

Knowing the elasticity characteristics (elongation vs. stress) of the geotextile the exerted force necessary to reduce the velocity v_2 to zero can be calculated.

$T = \tan\varphi \cdot \varepsilon \quad \varepsilon \leq 10\%$ Tension $\approx$ linear function ε elongation

Assuming that the geotextile is kept in position by friction with the subsoil, the force exerted by horizontal jet on geotextile of cross-area (a*1m length) will be in order of the following magnitude:

$$F_2 = 0.5 \cdot a \cdot (1\ m) \cdot \rho_s \cdot v^2_2 = 1.5 \cdot a \cdot (1\ m) \cdot \rho_s \cdot v^2_1$$

where:

ρ_s = bulk density of the fill material [kg/m^3]
(1600 for dry sand and 2000 for saturated sand)

Example:

Assume: b_o = 2.5m, ρ_s = 2000 kg/m^3 (saturated sand), a = 0.29, b = 0.725 m, and v_1 = 4 m/s, then

$$F_2 = 1.5 \cdot 0.725 \cdot 1 \cdot 2000 \cdot 4^2 = 34800 \text{ N per 1m length} \quad \text{(for saturated sand)}$$

These forces are probably lower than these for the transitional circular shape as descussed previously.

Note: the practical relevancy of this approach has not been proven yet.

Summary of dumping process and practical uncertainties

A summary of various forces during the dumping and placement process is given in Figure 17.

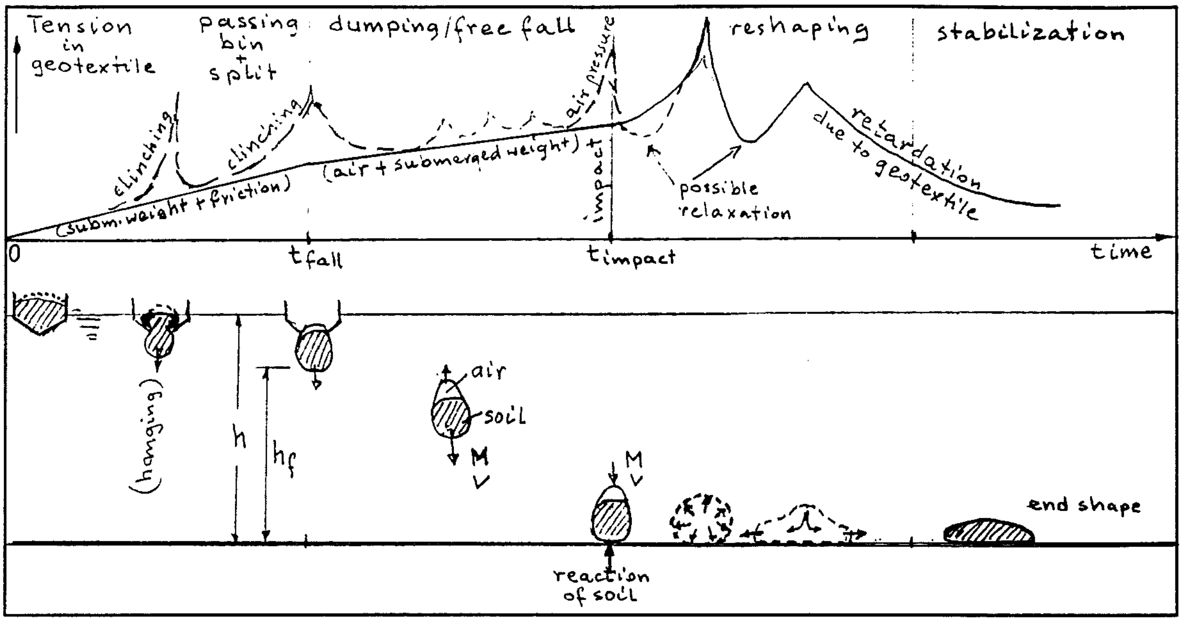


Figure 17. Development of forces during dumping process of geocontainer

1. After opening of the split of a barge the geocontainer is pulled out by the weight of soil but at the same time the friction forces along the bin side are retarding this process. Due to these forces the tension in geotextile is developing at lower part and both sides of the geocontainer. The upper part is free of tension till the moment of complete releasing of geocontainer.

Questions:
- description of friction forces in the bin (to avoid blockage of split) incl. the rol of pleats (additional folds/wrinkles) along the bin
- development of forces/stresses in geotextile during hanging in and passage of split incl. effect of initial perimeter of geotextile and fill-grade
- what is a proper filling of geocontainer in longtudinal direction allowing horizontal dumping/placement

2. Geocontainer will always contain a certain amount of air in the pores of soil and between the soil and the top of (surplus) geotextile providing an additional buoyancy during sinking. The amount and location of air pockets depends on soil consistency (dry, saturated) and uniformity of dumping. The air pockets will exert certain forces on geotextile and will influence the way of sinking.

Questions:
- how to describe the process of release of air from the soil in (short) time under incresing outside (hydrostatic) pressure, the rol of free space created by surplus of geotextile, and the permeability of geotextile/airtightness (i.e. what will be the influence of increasing of geotextile opening and percentage of openings by factor 2 on reduction of pressure and fall-velocity); influence of air pocket on fall-velocity
- description of stresses in geotextile due to air pockets/air balloon
- effect of infiltration of water in case of dry sand
- description of change of shape of geocontainer during dumping incl. the rol of speed of split opening and the final split width

3. The forces due to the impact with the bottom will be influenced by a number of factors:

* consistency of soil inside the geocontainer (dry, semi-dry, saturated, cohesie, etc.) and its physical characteristics (i.e. internal friction)
* amount of air
* permeability/airtightness of geotextile
* strength characteristics of geotextile (elasticity/elongation vs. stresses, etc.)
* fall-velocity (influenced by consistency of soil; saturated soil diminish amount of air but increases fall speed)
* shape and catching surface of geocontainer at impact incl. effect of not horizontal sinking (i.e. catching of bottom with one end)
* type of bottom (sand, clay, soft soil, rock, soil covered with rockfill mattress, etc.) and/or type of sublayer (i.e. layer of previous placed containers)

During the impact the cross-sectional shape of geocontainer will be undergoing a continous reshaping; from cone shape, first probably into a transitional cylindrical shape, and through a certain relaxation, into a semi-oval shape or flat triangular/rectangular shape dictated by soil type, perimeter of geotextile, and elongation characteristics of geotextile.

Questions:

- What do we know on the modelling (mathematical formulation) concerning the process description, exerted forces/pressures, and stresses induced in geotextiles
- How to optimize design/minimize forces and stresses (qualitatively and/or quantitatively), (i.e. dry sand vs. saturated soil, effect of amount of air, effect of shape and position of geocontainer at the impact, etc.)
- What is the performance of geocontainer during the impact when hydraulically filled (dense fluid/soil-water mixture) for different possible cases: geotextile permeable and geotextile impermeable (impermeable because of clogging/blokking or impermeable because of functional requirements, i.e. storage of contaminated dredged material); what will be performance in time (dewatering, settlement, reshaping, etc.)
- When we are able to describe the effect of the air we can formulate the additional requirements concerning the air release measures (air vents, expansion seams, etc.)
- What will be effect of impact of geocontainer on relatively sharp stone
- Can we bring already existing approaches to one consistent design line

4. In final situation the geocontainers will perform as a core material of various protective structures or as independent structure exposed to loading by currents and waves, and other loadings (ice, debris, ship collision, vandalism, etc.). In most cases the geocontainers will be filled by fine (loosely packed) soils.

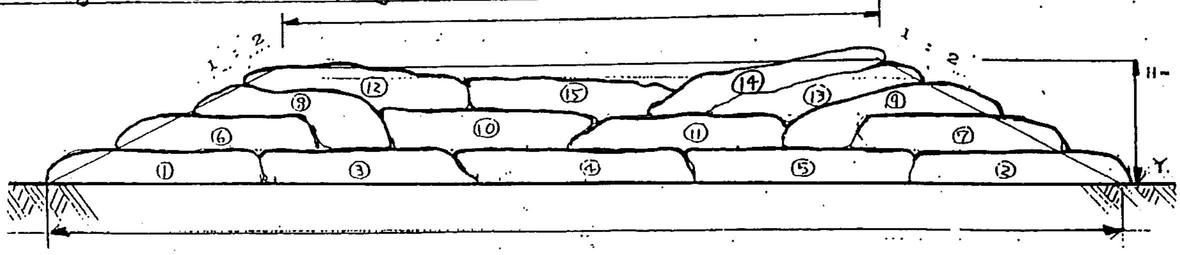
Questions:

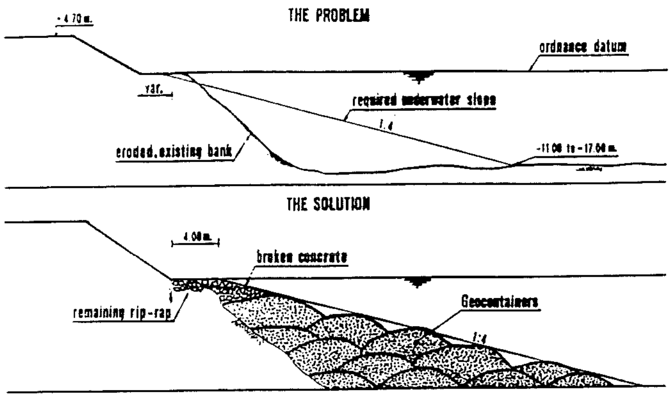
- How to determine the internal stability of geocontainers in function of hydraulic loading (migration/'rupsen', conditions of liquefaction, deformation of structure, etc.)
- Influence of armoring on the surface (rock, blockmats) on performance of geocontainers (reduction of external/internal loading)
- Performance of geocontainers under seismic loading

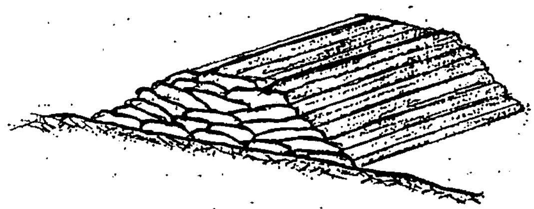
NB.
All additional suggestions on improvement of design technique of geocontainers are welcome!!!

Some examples of placing and application of geocontainers are shown in Figure 18.

Submerged mound of geocontainers

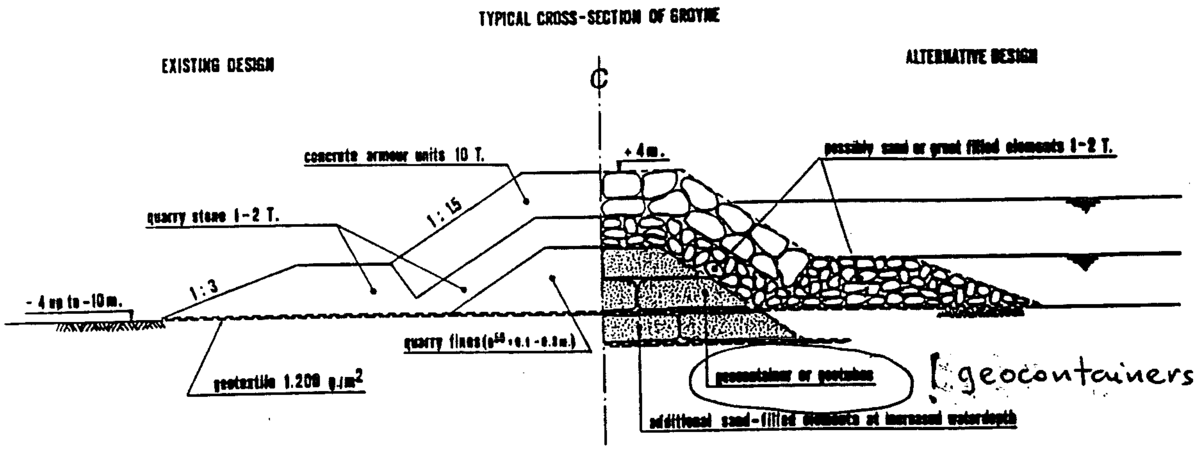




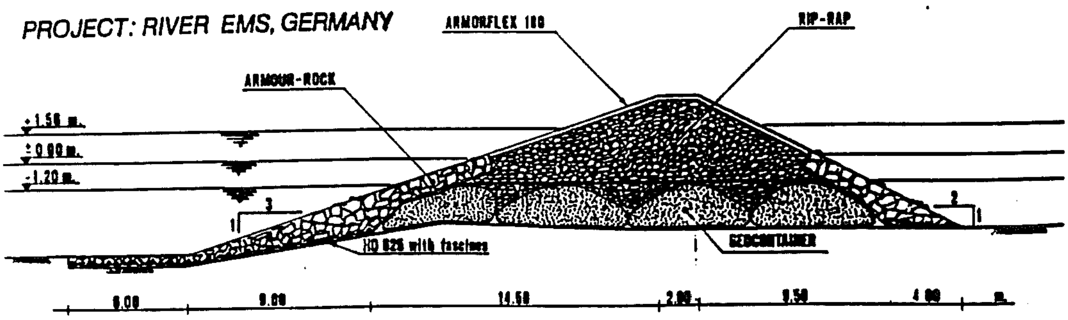


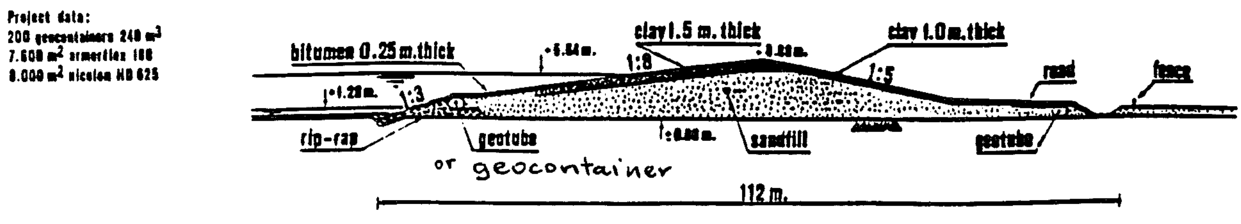
View

PROJECT:
Old Meuse, NL



PROJECT: RIVER EMS, GERMANY





Geotubes in dike design, project Leybucht, Germany (construction period 1987 - 1990)

Figure 18. Examples of placing and application of geocontainers

Recommendations on stability criteria for geosystems

* Sand and mortar filled bags

For the time being it can be concluded that the stability of sandbags with the width-length ratio not larger than 1 to 3 and properly filled (> 70%), can be computed in the similar way as riprap. It is recommended to calculate the stability acc. to Pilarczyk's formula (Pilarczyk, Coastal Protection, 1990) with stability coefficient **c** = **2.5**, nl.:

$$H_s/\Delta D = \mathbf{c} \cos\alpha\, \xi^{1/2} \quad \text{for } \xi <= 3 ,$$

(for $\xi > 3$, the values calculated for $\xi = 3$ can be used),

where: H_s = significant wave height, Δ = relative density of the bags, $(\rho_s - \rho_w)/\rho_w$, D = average thickness of bags, **c** = stability coefficient defined at $\xi = 1$, α = slope angle (it can be neglected for slopes milder than 1 on 3), ξ = surf-similarity parameter equal to $\tan\alpha/(H_s/L_o)^{1/2}$, and L_o = wave length. The density of bags (ρ_s) can be assumed 2000 and 2300 kg/m^3 resp. for sand and concrete (Δ resp. 1 and 1.3).

Note: Sand-filled units applicable till H_s = 1.5 m (max. 2 m).

*** Stability of foreshore protection mattresses incl. sand-sausage mattresses (Profix-mat)**

For the first approximation of stability of sand- or mortar-filled mattresses (i.e. ProFix or Fabriform mats) of more or less uniform thickness the formula proposed by Pilarczyk (1990) can be used:

$$H_s/\Delta D_{eq.} = \mathbf{c} \cos\alpha\, \xi^{-2/3} \qquad \text{for } \xi <= 3$$

(for $\xi > 3$, the values calculated for $\xi = 3$ can be used)
where: Δ = relative density of the mattress, $D_{eq.}$ = equivalent (average) thickness of mattress, **c** = stability coefficient defined at $\xi = 1$ (definition of other parameters is the same as above).
The value of coefficient '**c**' depends on the failure mechanism and the ratio between the permeability of the mattress and the permeability of the subsoil, k_m/k_s:

c = 4 when $k_m/k_s < 1$ with the uplift of mattress and deformation of subsoil as main failure mechanism, and
c = 6 when $k_m/k_s >= 1$ with the deformation of subsoil as the main failure mechanism.

The range of **c**-values follows from the research projects of Delft Hydraulics with placed block revetments/block-mats and different type of mattresses. It should be noted that the uplift can already start at **c** = 2, but it is so small and of such short duration that it will no result in a serious damage to the mattress protection. Therefore <u>**c = 3 to 4**</u> can be treated as a design value.
In special cases as large mattresses of temporary use and/or when some deformation of the subsoil can be accepted or the subsoil is more resistant to deformation (i.e. clay) the higher values of '**c**' can be chosen (max. 6). The research described in (Delft Hydraulics, 1975; large mattresses on circular island) can be illustration of such case. Using these high **c**-values the structure should be controlled on sliding, and in most cases it will require a special anchoring of mattresses.

Note: sand-filled units applicable till $H_s <= 1.5$ m.

*** Geotubes and Geocontainers**

Delft Hydraulics/Nicolon, 1994, H2029). The tested structures consisted of three layers of elements. The bottom layer contained four adjacent elements, the middel layer contained three adjacent elements and the top layer contained two adjacent elements. This structure is referred to as the 4-3-2 structure. The applied wave spectrum was of the Pierson-Moscowitz type. The significant wave height was increased in steps until the structure collapsed or until the highest obtainable significant wave height had been reached.

Results

Element	Water level over crest	Significant wave height periode at instability	$H_s/\Delta\ D_{eq.}$	Remarks
Geotube	0.0 m	1.7 m / 5.7 s	1.05	minor motions
(D = 2.15 m)		2.5 m / 7.1 s	1.55	0.4 m displacement
		3.1 m / 9.0 s	1.92	no further displacement
Geotube	3.5 m	4.2 m / 9.1 s	2.60	rapidly collapsed
Geocontainer	3.5 m	3.3 m / 7.3 s	1.76	minor motions
(D = 3.75 m)		4.2 m / 9.0 s	2.24	0.4 m displacement

Note: Equivalent thickness is assumed as equal to 0.75 D for geotubes and 0.5 D for geocontainers, and the relative density Δ = 1; assuming further the equivalent slope as equal to 1 on 1, the surf-similarity parameter (breaker index) will be about ξ = 5.5 (surging breaker). These informations can be of use for comparison with the stability of other systems.

*** General stability criteria for geotubes filled with sand or mortar**

Based on small scale investigations by Delft Hydraulics (Breakwater of concrete filled hoses, M 1085, 1973) and other literature informations, the following stability criteria for geotubes can be formulated:

- tubes on the crest (at S.W.L. or submerged) lying parallely to the axis of breakwater

$$H_s/\Delta\ B = 1$$

where B is the width (horizontal ovality measure) of a tube; one may roughly assume B = 1.1 D (original diameter of a tube).

Note: when the crest layer is composed of two tubes connected artificially to each other (i.e. re-bars) the equivalent width is equal to 2B.

- when the tube is placed perpendicularly to the axis of a breakwater the stability can be approximated by

$$H_s/\Delta\ L = 1$$

where L is the length of a tube.

Note: sand-filled units applicable till H_s = 1.5 m (max. 2 m)

Due to the absence of reinforcement in the mortar filled units it is very likely that for long tubes (say longer than 3D) also cracks will occur; some reinforcement should be recommended or an equivalent length should be taken equal to L < (3 to 4) D.

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K.W. Pilarczyk, 1995, "Novel systems in coastal engineering; geotextile systems and other methods", Rijkswaterstaat, Road and Hydraulic Engineering Division, the Netherlands.

VAN OORD ACZ, 1995, Report test programme Geocontainers, VAN OORD ACZ B.V. - NICOLON B.V., November 1995.

Tsunoda, N., 1995, Personal communication on friction and tensile forces in geotextile during release from the barge, Mitsubishi Kagaku Sanshi Corporation, Tokyo, Japan.

J. Wouters, 1995, Stability of geosystems, Delft Hydraulics, report H 1930/A2.95.40.

For Geotubes see:

Dov Leshchinsky and Ora Leshchinsky, 1995, Geosynthetic Confined Pressurized Slurry (GeoCops): Supplement Notes for Version 1.0, May 1995 (Copyright Nicolon US).

APPENDICES

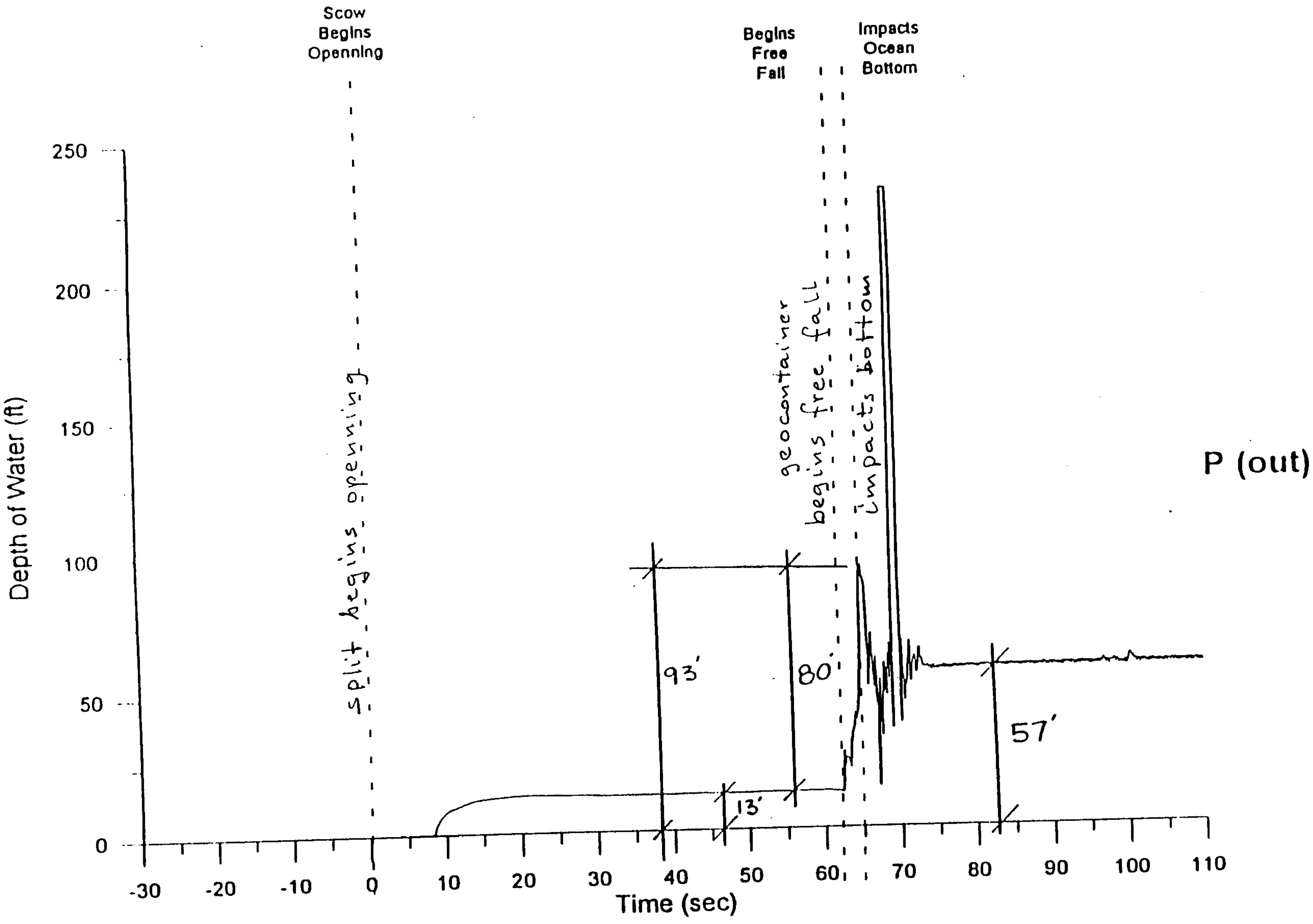


Figure I. Pressure Cell Outside the Container, Pressure vs Time

Appendix I : Example of pressure registration ; New York & New Jersey project (fill = dredged material)
= saturated conditions

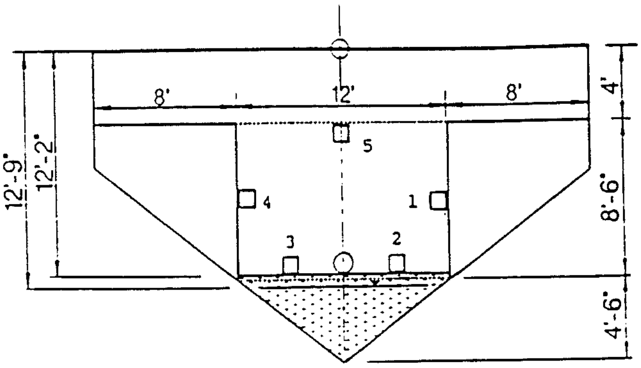


Figure A Cross Section of Split Hull Bottom Dump Barge and Strain Gage Locations.

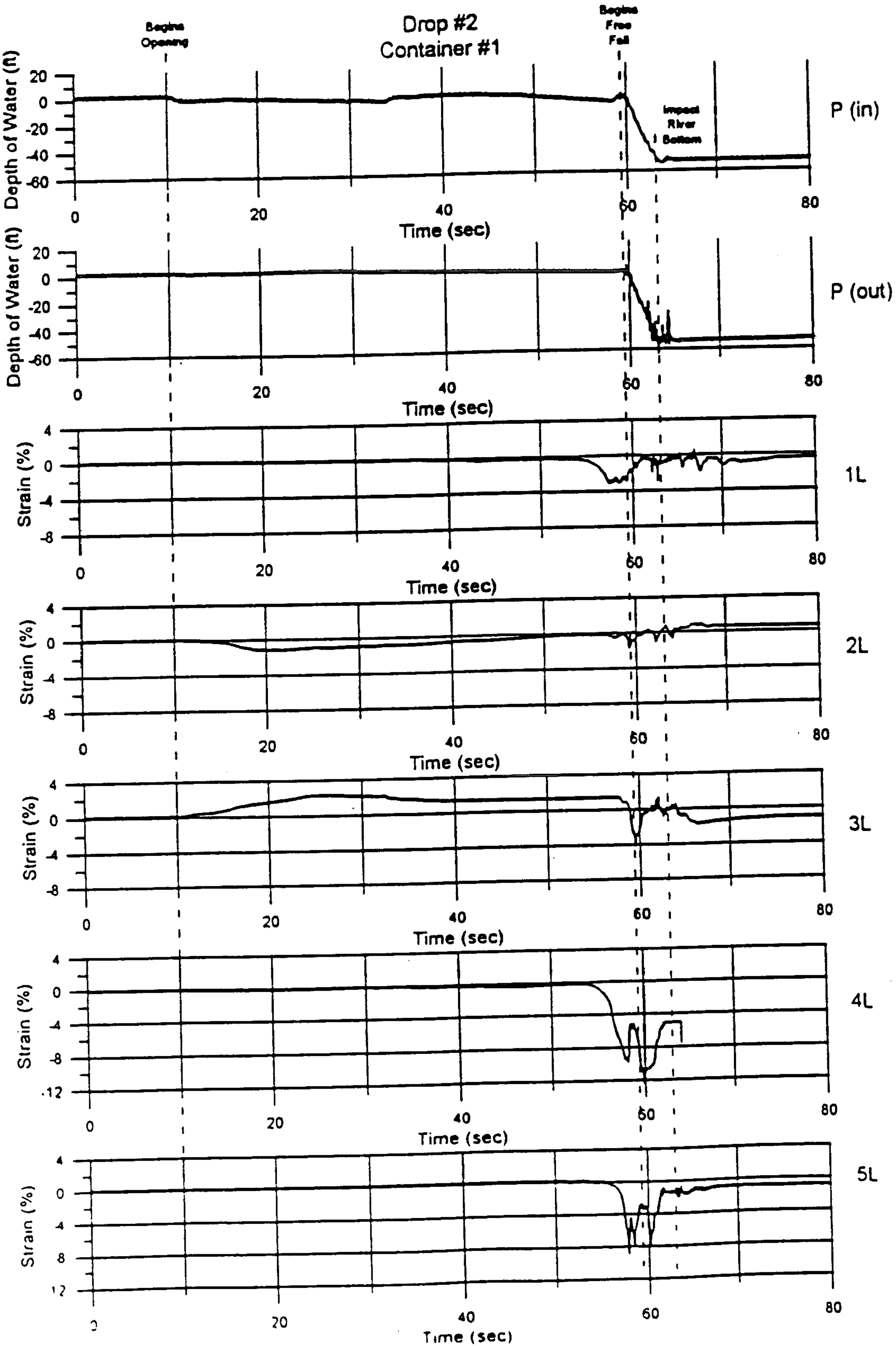
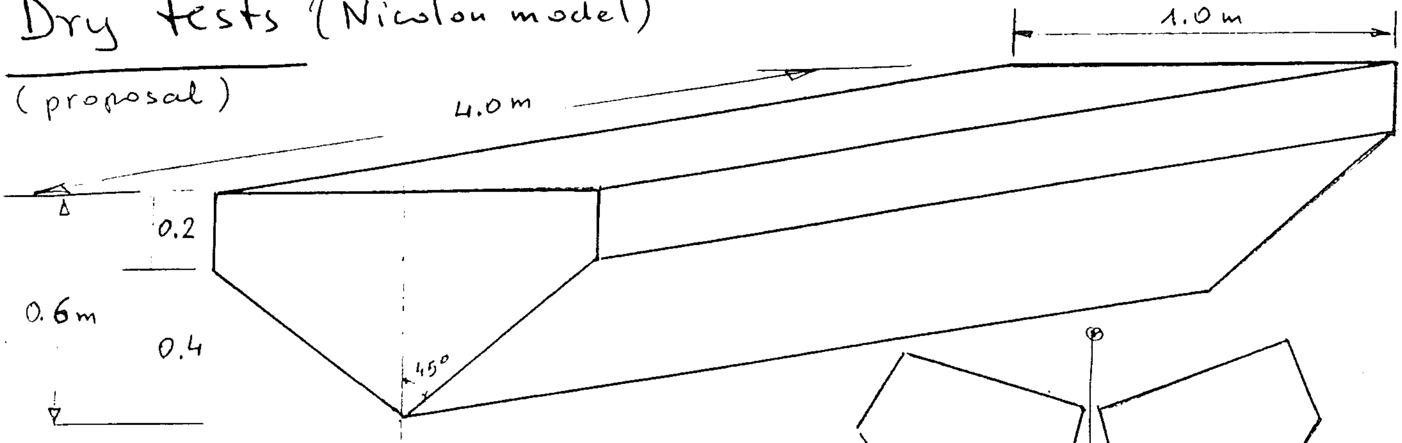


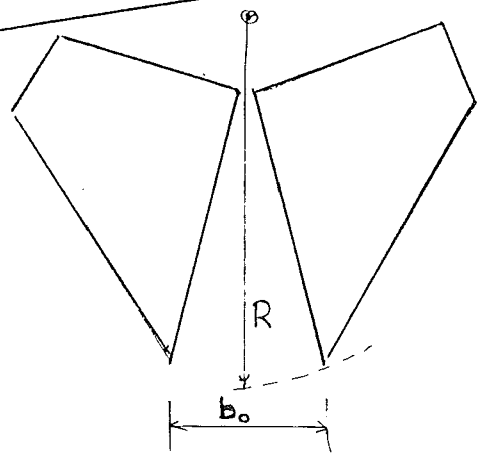
Figure B Drop #2, Container #1, Pressure and Longitudinal Strain Verses Time

App. II: Example of registration; Red Eye Crossing, Mississippi River (fill = wet sand)

Dry tests (in air) (Nicolon model)

(proposal)





Variables / parameters

1° Fill-material
- dry sand
- wet sand
- (ev. water in membrane geocontainer) to simulate 'slurry'

2° Strength geotextile : higher (i.e. 10 kN/m) } the choice will be made after desk-studies
lower (i.e 5 kN/m)

3° Application of wrinkles (locally folded geotextile) and attached threads of a proper (scaled) strength for control of strength (instead of strain gauges)

4° Opening of split-barge, b_o :
- in small steps, until the release of geotextile
- quick (continue) open

5° Circumference (perimeter) of geotextile, S ;
- $S = S_{minimum}$ } for Smin. see derivation by Pilarczyk
- $S = 1.25\ S_{min.}$
- $S = 1.5\ S_{min.}$

6° Friction of bin
- steel - geocontainer } verification friction model (see note by Pilarczyk)
- geotextile cloth on the surface of the bin

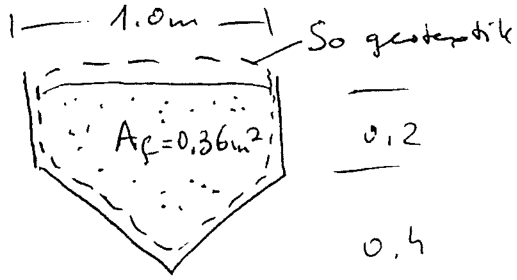
7° Fall-height : 2 m, 3 m, 4 m (model should be placed outside)

8° Bottom stiffness :
- foam plastic/rubble (soft bottom)
- sand
- concrete

9° Uneven loading of barge in longitudinal direction

[illegible] the tests

Dry drop-test (June 26, 1996, Nicolon, Almelo)



Given

$A_{o\,max} = 1 \cdot (0.2 + \frac{0.4}{2}) = 0.4\ m^2$

$A_{fill} = 0.36\ m^2$

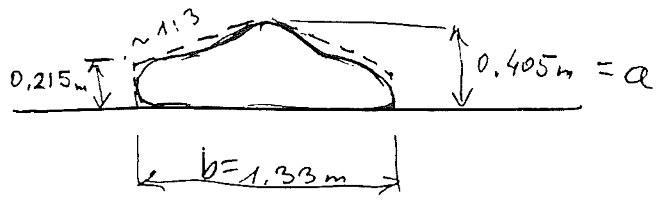
fill-grade coeff.

$\frac{A_f}{A_o} = \frac{0.36}{0.40} \cdot 100 = 90\%\ ;\ \phi = 0.9$

perimeter $S_o = 2.85 m$

$\varphi = \frac{A_o}{S_o^2} = \frac{0.4}{2.85^2} = 0.04925$

final shape



End-shape (measured) ⟶

Control: $A_{ex} = \frac{0.215 + 0.405}{2} \cdot 1.33 = 0.41 m^2$ (external dimensions)

average $\bar{A} = \bar{a} \cdot \bar{b} = 0.3 \cdot 1.2 = 0.36\ m^2$ (internal dimension)

Approximation by a trapezoidal shape provides for $\phi = 0.9$:

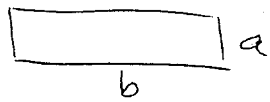
$a_{max} = 0.14\, S_o$, $\bar{a}_{average} = \frac{3}{4} a_{max}$; $b \cong 3 a_{max} = 0.42\, S_o$

$a_{max} = 0.14 \cdot 2.85 = 0.4 m$, $b = 1.2 m$; $A_f = 1.2 \cdot \frac{3}{4} \cdot 0.4 = 0.36 m^2$

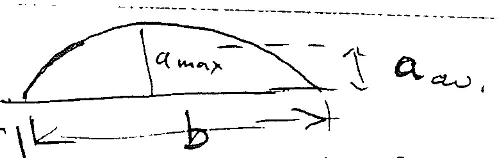
Rectangular shape: $a = \frac{S_o}{4}(1 - \sqrt{1 - 16\varphi\phi})$; $b = \frac{A_f}{a}$

$a = \frac{2.85}{4}(1 - \sqrt{1 - 16 \cdot 0.04925 \cdot 0.9}) = 0.328 m$

$b = \frac{0.36}{0.328} = 1.1 m$



Semi-oval shape



$a_{max} = 0.636\, S_o (1 - \sqrt{1 - 14.4\varphi\phi}) = 0.636 \cdot 2.85 (1 - \sqrt{1 - 14.4 \cdot 0.04925 \cdot 0.9}) = 0.72$

$b = \frac{A_f}{0.125\,\pi \cdot a_{max}} = \frac{0.36}{0.125 \cdot \pi \cdot 0.72} = 1.27 m$

$a_{average} = \frac{A_f}{b} = \frac{0.36}{1.27} = 0.283 m$

Scale rules

How to scale geocontainers [falling in air (dry tests), falling in water

length $n_L = \frac{\text{prototype}}{\text{model}}$ (i.e. $n_L = 5$; model 5 times smaller)

area n_L^2 (i.e. $n_L^2 = 5^2 = 25$; area of model 25 x smaller)

volume n_L^3 (i.e. $n_L^3 = 5^3 = 125$; volume 125 x smaller)

time $n_t = \sqrt{n_L}$ (i.e. $n_t = \sqrt{5} = 2.24$;

velocity $n_v = \sqrt{n_L}$

specific density n_ρ

specific weight n_γ

$$\gamma = \rho \cdot g \rightarrow g = \frac{\gamma}{\rho} \Rightarrow \frac{n_\gamma}{n_\rho} = n_g = 1$$ gravity

mass $n_L^3 \cdot n_\rho$

drag $C_D = 1$ the same in model as in prototype

acceleration $n_\gamma \cdot n_\rho^{-1}$

$$\text{Force} = \frac{M \cdot L}{T^2} \Rightarrow n_L^3 \cdot n_\gamma = 5^3 \cdot 1 = 125$$

mass, $m \cdot g$

pressure/stress $n_L \cdot n_\gamma$

energy $n_L^4 \cdot n_\gamma$

$$\text{Energy} = 5^4 \cdot 1 = 625$$

impuls $n_L^{7/2} \cdot n_\rho^{1/2} \, n_\gamma^{1/2}$

impact stress $\approx n_\sigma = \sqrt{n_L}$ = when model material properties are the same as in the prototype

If $n_{\gamma_a} = 1$ and $n_E = n_L \longrightarrow n_\sigma = n_L$

elasticity (Young modulus)

If $n_L = 5$ $\quad n_{weight} = n_L^3 \cdot n_\gamma \doteq n_L^3 = 5^3 = 125$ (n_γ " ~ 1)

Impact force $n_F = (n_L)^{5/2} \sqrt{n_\rho n_g \cancel{n_E}}$

and impact stress

$$n_{\sigma_i} = \frac{n_F}{n_L^2} \text{ or } n_{\sigma_i} = \sqrt{n_{\gamma_a} n_E n_L} \qquad \text{where } n_{\gamma_a} = n_{\rho_a} \cdot n_g^{\,\prime\prime 1}$$

Young modulus of elasticity of geotextile : $\sim$ 1000 kN/m

Tensile strength : i.e (prototype) 120 kN/m (strong geotextile) $= \frac{120}{125} \approx 1$ kN/m (model) ?? (125 " n_L^3)

very weak geotextile (not existing)

or it should be: $\frac{n_E}{n_L} = \frac{n_L^3 \cdot n_\gamma}{n_L} = n_L^2 \Rightarrow 5^2 = 25 \Rightarrow \frac{120}{25} \approx 5$?

density of water 1000 kg/m³ (or 1050)

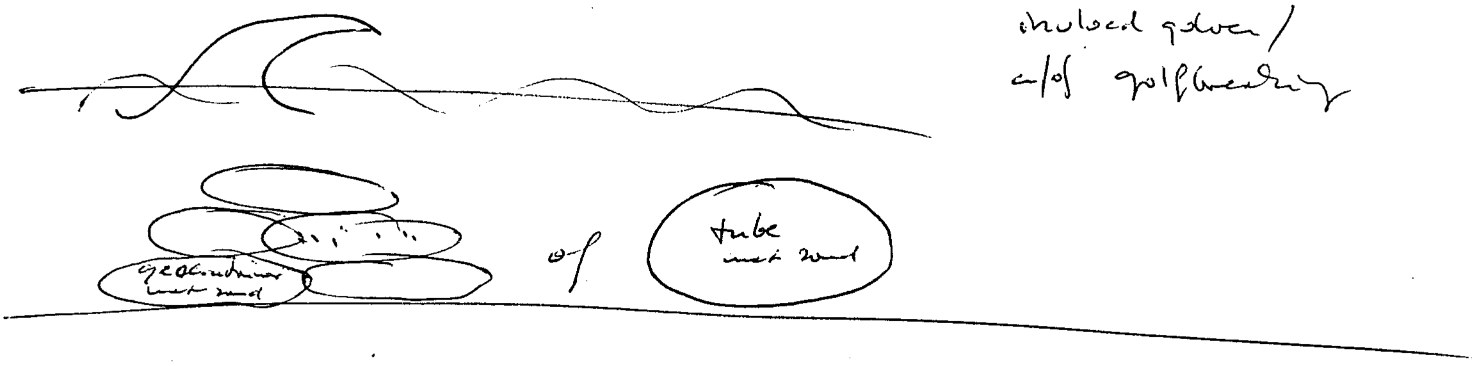
density of air 1.25 kg/m³

It is only a summary of various scaling rules; it is not easy to give a proper answer. It should be studied in more systematic way taking into account all aspects/components involved.
Therefore a desk-study on this subject is necessary.

Nog extra huiswerk mbt. geosystemen. Dit betreft de interne stabiliteit van opgesloten zand na de aanleg, bij stroom en golf aanval.

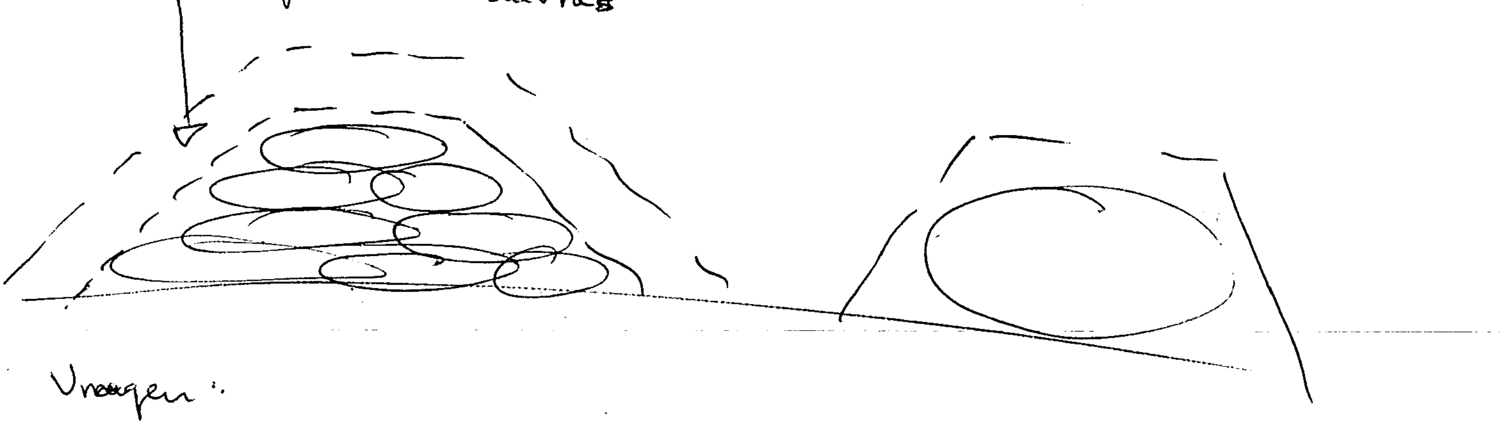
A) Zonder bescherming

invloed golven / en/of golfbreking



B) Met beschermende laag

stortsteen, of blokkenmat of mortar-matras



Vragen:

1) hoe bepaal je interne stabiliteit (geen migratie/'rupsen' van zand binnen zijn tube/container

2) hoe bepaal je wel/of niet verweking (liquefaction) gevaar → en dus sterke vervorming.

3) hoe dempt beschermende laag de interne processen

4) invloed gradatie vul-materiaal op interne stabiliteit / waar liggen de grenzen mbt. instabiliteit

5) hoe is het mogelijk gedrag bij seismische belastingen / aardbeving