CHAPTER 1

GEOTEXTILE SAND CONTAINERS FOR SHORE PROTECTION

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This chapter aims at providing a brief overview on geotextile sand containers applied in coastal engineering for shore protection. First, the engineering properties required for the geotextile used for sand containers as well as the durability and the life time prediction issue are discussed. Second, some example applications are provided to illustrate the versatility of geotextile sand containers as an appropriate soft shore prediction alternative to conventional hard coastal structures made of rock and concrete units. However, the major part of the paper is aimed to address the hydraulic stability of the containers constituting a shore protection structure subject to wave attack. For this purpose, simple formulae are first proposed for the stability of the slope and crest containers. The processes which may affect the hydraulic stability are then discussed to highlight the necessity of developing more process-based stability formulae. New stability formulae are finally proposed which can also account explicitly for the effect of deformation of the containers. Finally, a discussion is provided on comparative analysis of the stability of the slope and crest containers with and without consideration of the deformation effect.

1. Introduction

In view of the increasing storminess associated with climate change and its effect on coastal flood and erosion, more versatile materials and solutions are required for the design of new, cost effective shore protection structures as well as for the reinforcement of existing threatened coastal barriers and structures, including dune reinforcement and scour protection. In search for low cost, soft and reversible solutions, the concept of Geotextile Sand Containers (GSCs) as “soft rock” for coastal defence structures was introduced for the first time in the 1950's in form of "sand bags" for the construction of a dike to close the inlet "Pluimpot" in the Netherlands (Van Santvoort, 1994). Later, GSCs have mainly been used as temporary shore protection measures due to the difficulties associated with the assessment of the long-term performance. Meanwhile, significant advances have been made with respect to the following issues:

- Improvement of the long-term performance of geotextiles (additives and stabilizers against UV-radiation, coating against abrasion, etc.),
- Assessment of the durability and life time prediction (accelerated testing, standards etc.),
- Survey of GSC-built structures and analysis of past experience with respect to the degradation mechanisms and
- Understanding of the mechanisms of failure, including hydraulic instability under severe wave action.

These advances, together with numerous advantages of GSCs as "soft rock", have contributed to extend the use of GSCs to permanent coastal defences, including a wide range of types of structures such as seawalls, revetments, groins, artificial reefs, offshore breakwaters, perched beaches, dune reinforcement, core of rubble mound structures, scour protection, etc. (Fig. **1**).

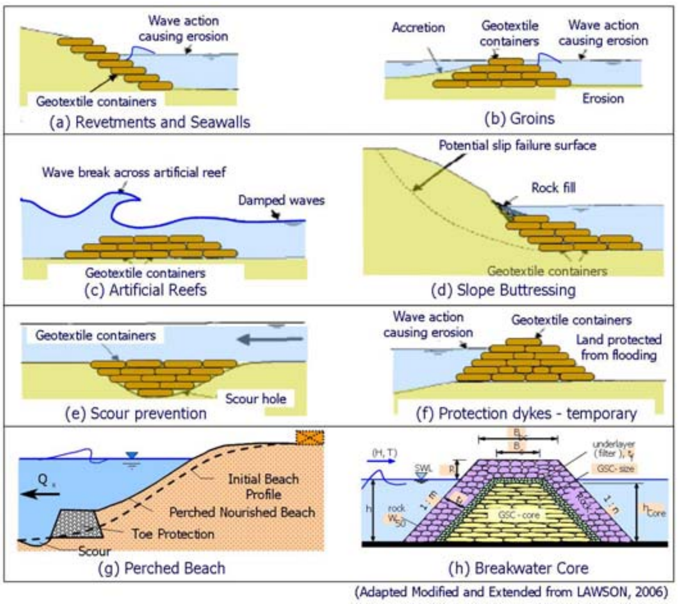


Fig. 1 Coastal Engineering Applications of Geotextile Sand Containers

Generally, in coastal engineering any containment of sand encapsulated in geotextile to build flexible and erosion-resistant gravity structures is called "geotextile sand container". A variety of size and shapes of geotextile sand containers have been used, including geotubes, geocontainers and geobags. The latter have a volume of about 0.05-5m^3 and are generally filled offsite. Geocontainers are much larger (sausage shape up to 700 m^3) and generally filled in split bottom barges. Geotubes have a diameter up to 5.5m and are filled directly at the location where they are built (see Fig. 2 and Table 2).
In most cases, it is advantageous to use smaller volume containers, because:

(i) they are more versatile and can be adapted to build any type of structures;
(ii) they can better fulfill any requirements with respect to structure slope and geometry (better tolerance);
(iii) maintenance and remedial work are much easier in case of vandalism or failure by wave action;
(iv) less tensile strength is required and less change of shape will be experienced, thus resulting in longer life time;
(v) higher density of the sand fill can be achieved;
(vi) less risk of liquefaction of the sand fill is expected and thus less GSC-deformations; and
(vii) simpler mobilisation of the required equipment.

However, larger containers may be required, for instance in the case of higher waves forces and for temporary structures. A brief illustration of the container sizes used in coastal engineering is given in Fig. 2.

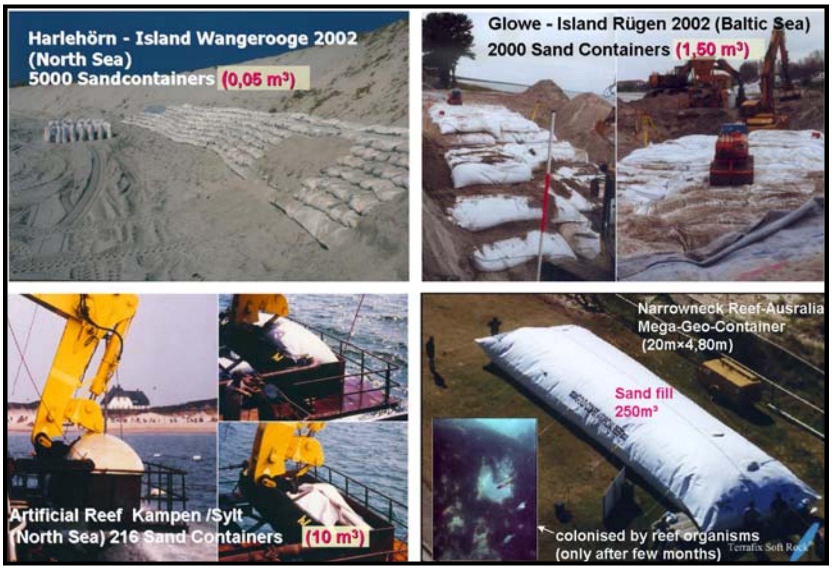


Fig. 2 Range of Geotextile Sand Container Sizes Applied in Coastal Engineering

In the following, some basic information on geotextiles, including a discussion on the durability issue are first provided. Second, the versatility of the use of geotextile sand containers as a soft shore protection alternative to conventional hard structures made of rock and concrete units is illustrated by some example applications. The major part of the paper will, however, focus on the hydraulic stability of the geotextile sand containers under wave action. For this purpose, simple formulae for the stability of slope and crest GSCs will first be proposed which don't take explicitly into account the effect of GSC-deformation and friction between containers. A detailed description of the processes, which may lead to failure under wave action, is then provided to illustrate the necessity of developing more process-based stability formulae. Finally, the new detailed stability formulae and the simple formulae are comparatively analysed to stress the effect of GSC-deformation on the hydraulic stability.

2. Geotextile Properties and Durability of Geotextile Sand Containers

2.1. *Required Properties of the Geotextile for Sand Containers*

Geotextiles and geomembranes, including their related products, are called geosynthetics; i.e. fabrics that are specially manufactured for civil and environmental engineering applications. While geomembranes are impermeable to water, geotextiles are permeable. The most widely used polymer for geotextiles is polypropylene (PP> 90%), followed by polyester (PET≈ 5%) and polyethylene (≈ 2%). Based on the manufacturing process two major categories of geotextiles may be distinguished: non-woven (≈ 60%) and woven geotextiles (≈ 40%).

Non-woven geotextiles are composed of directionally or randomly oriented fibres which are mechanically (needle punching), chemically or thermally bonded into a loose web.

Woven geotextiles are obtained by interlacing two or more sets of yarns (one or several fibres), using conventional weaving processes with a weaving loom. The yarns can be monofilament, slit film, fibrillated or multifilament. Besides these two main groups there are also knitted and stitched geotextiles. The main engineering properties of non-woven and woven geotextiles are comparatively summarized in Table 1.

Table 1 Properties of Non-Woven and Woven Geotextiles (After Lawson and Kemplin, 1995).

Types of Geosynthetics	Tensile strength (kN/m)	Extension at max. load (%)	Apparent opening size (mm)	Water flow Rate (volume Permeabiltiy) (litres/m²/s)	Mass per unit area (g/m²)
Nonwovens					
Heat-bonded	3-25	20-60	0.02-0.35	10-200	60-350
Needle-punched	7-90	30-80	0.03-0.20	30-300	100-3000
Resin-bonded	5-30	25-50	0.01-0.25	20-100	130-800
Wovens					
Monofilament	20-80	20-35	0.07-4.0	80-2000	150-300
Multifilament	40-1200	10-30	0.05-0.90	20-80	250-1500
Flat tape	8-90	15-25	0.10-0.30	5-25	90-250
Knitted					
Weft	2-5	300-600	0.20-2.0	60-2000	150-300
Warp	20-800	12-30	0.40-1.5	80-300	250-1000
Stitch-bonded	30-1000	10-30	0.07-0.50	50-100	250-1000

As shown in Table 1, non-woven and woven fabrics have significantly different properties which can be exploited to produce the best solution for each specific need, including composite fabrics to combine the advantages of both types.

The *lower tensile strength* of non-woven geotextiles as compared to woven geotextiles might be a serious drawback, if the containers have to accommodate very large stresses during installation and in service without failure.

The *higher capacity for elongation* of non-woven geotextiles can to some extent compensate the disadvantage of lower tensile strength in the sense that it allows to accommodate large strains without failure. This is particularly important when the container is required to reshape during installation and in service (adaptation to scour development and settlement, self-healing effect). However, this might also be a serious drawback as the higher capacity for elongation will allow the material to creep significantly over time, so that the container may fail to retain the design shape (loss of height of the structure).

The *hydraulic permeability* of geotextile used for sand containers is particularly important when subject to cyclic wetting and drying (e.g. in tidal regime). Water should be drained from the sand container fast enough to avoid excess pressure build up and to ensure overall stability. Therefore, it is generally required that the geotextile should have a much higher permeability (≥ 10 times) than the sand fill without loosing the finer fractions. Alternatively, the geotextile can be selected to fulfill the commonly used filter criteria. Non-woven geotextiles have a higher permeability and a higher fine retention capability than its woven counterparts, but it should be stressed that the permeability is a function of the fabric thickness, and thus depends on its compressibility under normal stresses.

The abrasion resistance is particularly important in the surf environment where coarse angular sand, shell fragments or coral debris are present. The larger thickness of non-woven geotextiles and the ability of their structure to retain sand material make them more resistant.

The puncture resistance is important in the case of vandalism, drift ice, drift wood or dropped rock material during construction. Due to their high elongation capacity and to the retained sand within the fabric structure, non-woven geotextiles are more susceptible to limit damage from puncture, including vandalism.

A higher *friction angle between sand containers* is desirable to enhance the hydraulic stability against wave and current actions. Due to their structure, non-woven geotextiles provide a higher friction.

The resistance against UV-radiation still represents one of the most unresolved issues to ensure a satisfactory long-term performance of exposed geotextile sand containers. Despite the significant progress in the use of UV-stabilizers, coating or/and armouring of the exposed geotextile containers still remain the sole alternative to achieve a satisfactory life time without damage.

Simultaneously, the coating/armouring will also protect the geotextile against abrasion, vandalism, drift ice and drift wood.

The resistance against chemical and microbiological attack of polymers (e.g. polypropylene) is very high, so that no significant loss of strength is expected during the design life time.

The *ability of geotextiles to enhance marine growth* and to attract/support diverse invertebrate communities also becomes an important issue, if maximizing the biodiversity of the recruiting communities is relevant for the choice of the type of geotextile. First results of investigations comparing woven and non-woven geotextiles in Australia have shown that the latter are more favourable in this respect (Edward and Smith, 2003). The marine growth may represent an enhancement of the resistance against UV-radiation and abrasion. However, it is still not clear whether it may cause serious changes or detrimental effects on the mechanical properties of the geotextile over life time.

2.2. *Durability and Lifetime Prediction*

Unlike conventional construction materials such as rock and concrete, synthetic geotextiles are relatively new to coastal engineering. Therefore, their degradation and long-term performance are still not well-understood. Instead of trying to answer the most frequently asked question: "How long will a geotextile structure last?", it is more practical from the engineering point of view to ask, how long must a geotextile structure last. The expected lifetime when designing "permanent" shore protection structures is typically 20-100 years. Modern geotextiles are designed to be resistant to degradation from UV-radiation, chemical/biological attack, abrasion and hydraulic loading. Generally, a life time in the order of 20-25 years can be expected if damages during construction and through vandalism are avoided.

However, not all applications in coastal engineering require such a level of life time, for instance in the case in temporary protection measures. Although lifetimes up to 100 years have been suggested based on accelerated testing and extrapolation, the following question still remains unanswered: "How to predict/achieve a 100 years or more lifetime for geotextile structures applied for shore protection?"

Since much is known about the degradation mechanisms and degradation rates of polymer materials, and how these can be reduced or prevented (Fig. 3), it seems reasonable to use this knowledge

- to develop further "index" tests similar to those proposed in Annex B of the European Standards EN 13249-13257 and EN, 13265 which are believed to ensure a minimum durability of 25 years and
- as a basis for planning and interpreting site monitoring (see ISO 13437), including the development of procedures and techniques to reduce the degradation rates.

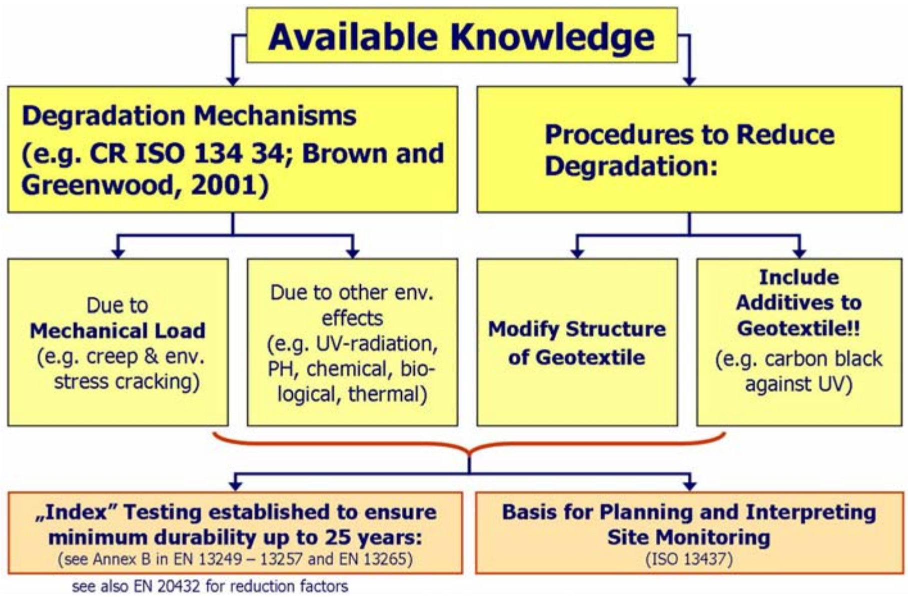


Fig. 3 Degradation Mechanisms and Reduction Procedures

Ideally, a degradation curve over the design lifetime for each relevant property of the geotextiles such as tensile strength, specific mass in g/m², elongation capacity and hydraulic permeability should be determined, together with the associated acceptable degradation limits at which the geotextile cannot perform its primary function (Fig. **4**).

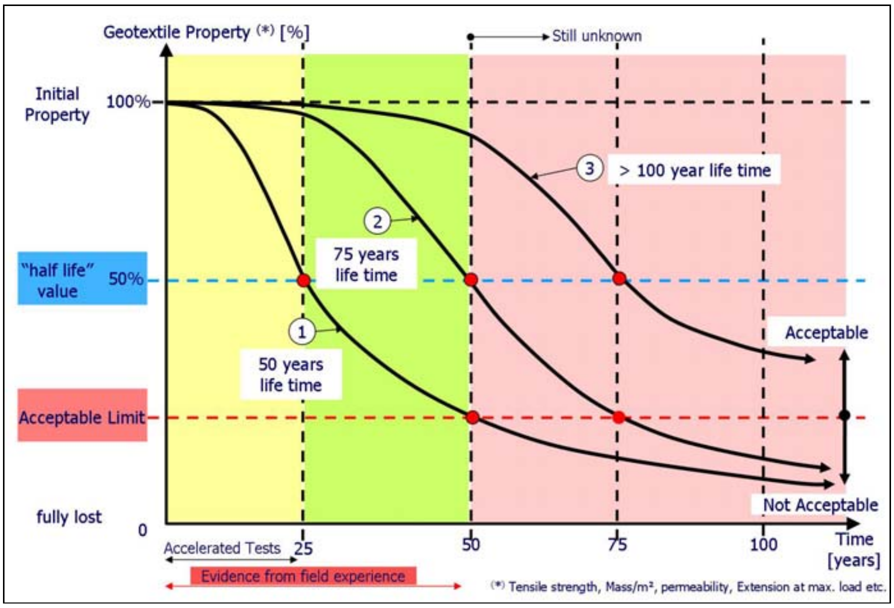


Fig. 4 Degradation Curves (Principle Sketch)

The most authoritative evidence for long-term durability is generally obtained from monitoring of the degradation under real service conditions.

In fact, the empirical evidence of long term durability form retrieved (non-exposed) geotextiles has shown that the reduction of tensile strength and other important engineering properties strongly depends on the prevalent service conditions and therefore significantly differs from one site to another. Samples of PP and PET non-woven geotextiles retrieved from 25 sites in France lost up to 30% of their tensile strength after 10-15 years service time as filters, separators and drains, while no chemical/biological effects were identified (Solton et al, 1982).

A further interesting case was reported by Lefaive (1988), showing that the reduction in tensile strength after 17 years of the same PET straps embedded in a concrete facing wall and in the backfill was completely different: 2% in the backfill with PH=8.5 and up to 40% at the transition between the wall and the backfill where PH-values of 13-14 and temperature up to 30°C prevailed. This case well illustrates the contribution of alkaline surface attack (25%), internal hydrolysis (5-10%) and mechanical damage to degradation.

The effect of UV-radiation is illustrated by a case reported by Troots et al (1994), where woven PET samples were retrieved after 13 years from an earth embankment: while along the section of the slope covered by vegetation and bitumen to provide protection against UV-attack no significant changes of the geotextile properties occurred, a reduction up to 50% of the ensile strength was identifies in the non-protected part.

These and further numerous examples from the literature show that the results/data from retrieved (essentially non-exposed) fabrics, although very valuable, have serious limitations when intended to be used for the prediction of long-term durability and lifetime of permanent structures. Moreover, the obtained data are often incomplete and relate to conditions that are generally far from those for which the prediction/assessment is being made (Fig. 5).

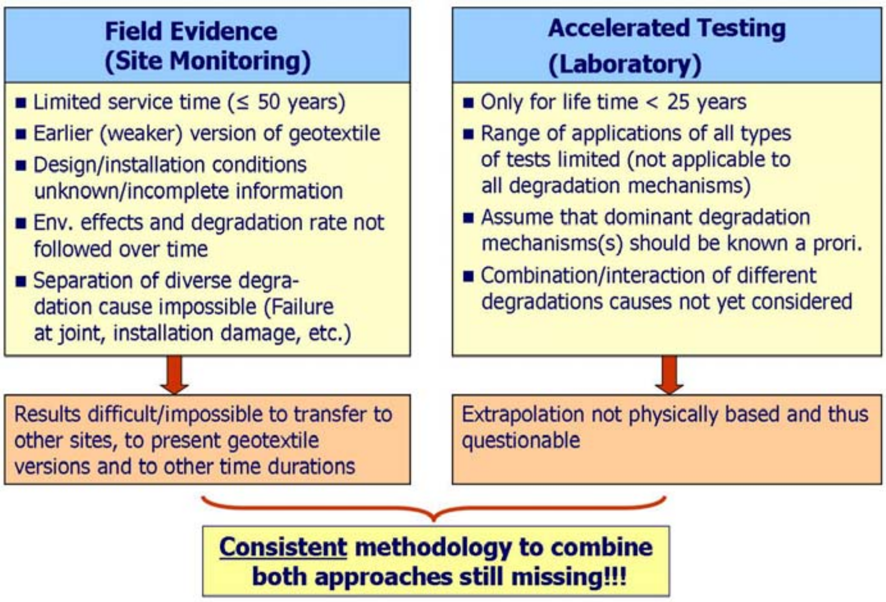


Fig. 5 Limitations of Present Approach to Predict/Assess Lifetime of Permanent Geotextile Structures for Shore Protection

In order to make the best use of field data some brief recommendation for future site monitoring are given in Plate 1.

Plate 1 Recommendations for Future Site Monitoring

- **Define system to be monitored:**
  - Material structure, compositions and properties
  - Environmental actions (mechanical loads, PH and saturation of soil, chemical contamination, biological effects, temperature and light)
  - Design and installation conditions
  - Functions (primary and secondary)
  - End of life criterion
  - Necessary maintenance and other measures.
- **Separate considerations of:**
  - Failure at joints from those of bulk material
  - Mechanical installation failure from those due to long-term degradation
  - Weathering failures from those due to chemical degradation
- **Install geotextile samples for future extractions and testing (ISO 13437):**
  - Sizes and placement of samples
  - Method of extraction
  - Close monitoring of environmental effects
  - etc.
- **Extrapolation to other sites, duration , etc.:**
  - only based on good understanding of degradation mechanisms.

These limitations and the urgent necessity for both users and manufacturers to predict life time of geotextiles have led to the development of accelerated tests which also have serious limitations (Fig. **5**).

The principle of accelerated testing is briefly summarized in Fig. **6**, showing that

(i) generally only one dominant degradation mechanism can be considered, thus ignoring the interaction with other mechanisms and

(ii) the approach cannot be applied to all degradation mechanisms.

Moreover, if two degradation mechanisms occur sequentially (e.g. UV-radiation followed by mechanical degradation), then the two mechanisms are analysed separately and the predicted lifetimes simply added. Further difficulties arise when extrapolating the short lifetime obtained from accelerated tests at increased load frequency/intensity and increased temperature to predict longer lifetime at service conditions. Power laws are often used for extrapolation. Fore more details refer to Greenwood and Friday (2006).

Ideally, the results of both site monitoring and accelerated tests in combination are expected to provide the best basis for the prediction of long term durability and lifetime. However, a consistent methodology to combine both approaches still needs to be developed (Fig. **6**).

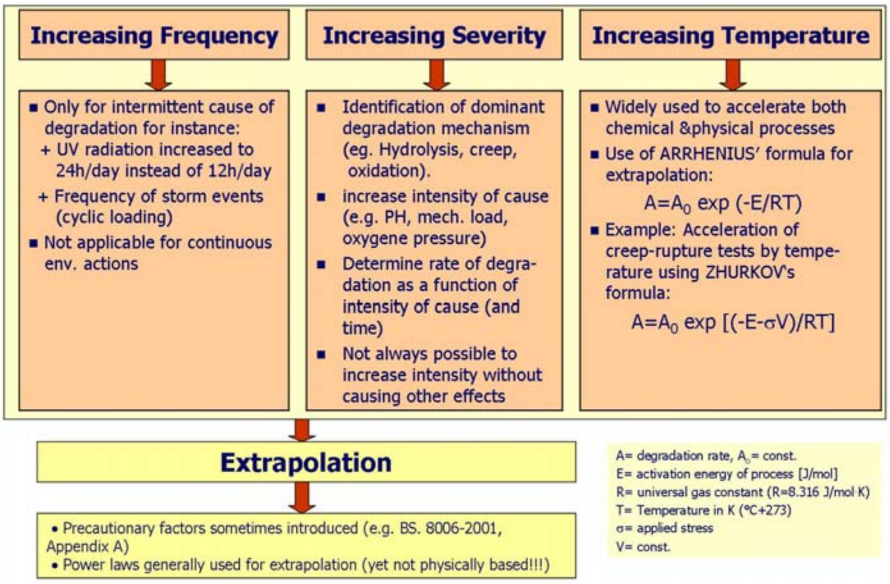


Fig. 6 Principle and Limitations of Accelerated Tests

In summary, it can be stated that geotextile applications, although using previous weaker versions of polymer material, performed relatively well over many decades when not exposed to UV-radiation. Most failures observed were rather due to faulty design, incorrect choice of material and poor quality of installation. The knowledge available on the degradation mechanisms, although still limited, allows to predict rationally lifetime up to about 25 years. Since rational prediction cannot foresee problems for which there is no empirical evidence or scientific basis, the primary goal of future research towards the assessment of durability and lifetime is to improve the understanding of all relevant degradation mechanisms –separately as well as in combination- by making use of both site monitoring and laboratory testing. Future significant improvement of UV-resistance and aesthetical aspects as well as the improvement of the long term performance of geotextile sand containers against large wave loads represent further R&D challenges towards avoiding to cover or armour geotextile structures.

3. Example Applications of Geotextile Sand Containers for Shore Protection

The types of geotextile sand containers used in coastal engineering as temporary or permanent structures are generally referred to in the literature as geo-tubes, geo-containers or geo-bags (see Table 2 and Section 1). For permanent coastal structures, small volume containers offer more advantages and are therefore often preferred (see Section 1).

Table 2 Types of Geotextile Containments Used in Coastal Engineering

Type	Volume [m³]	Sand Fill	Shape	Applications
1. Geo-Tubes	Generally > 700 m³	on site	cylindrical (D=1-55m)	• Groins • Containment dikes • Non-permanent structures
2. Geo-Containers	Generally 100 - 700 m³	split bottom barge	cylindrical/pillow (D<5m)	• Reef structures (surf zone) • defence structure against tsunami
3. Geo-Bags	0.05 - 5 m³	off site	pillow, box, mattress	As soft rock units to build any type of coastal structures. Also for scour protection and dune reinforcement.

In particular, they are more versatile in application and are used for different class of structures, including dune reinforcement, sea walls and revetments, detached breakwaters and artificial reefs, groins, etc. (see Fig. 5). Comprehensive large-scale model investigations on GSCs used for the scour protection of monopile foundation of offshore wind turbines have also been conducted by Grüne et al (2006). The results are described more detail in a final report by Oumeraci et al (2007).

Reviews of example applications related to geotextiles in general but, also including geocontainers, can be found in Heerten (1984), van Santvoort et al (1994) and Pilarczyk (2000). Comprehensive reviews on the applications of geosynthetics in hydraulic engineering and for the protection of land fill (including coastal areas) which may also provide valuable information and inspiration for the application in coastal engineering are given by Heibaum et al (2006) and Kavazanjian et al (2006), respectively. More specific reviews related to geocontainers are provided by Fowler and Trainer (1998); Lenze et al (2002); Lawson (2006) and Saathoff et al (2007). Rather than trying to duplicate the examples from the aforementioned reviews and to provide a further comprehensive review, it is attempted in the following to select only few examples from three classes of structures and applications: a) long-shore barriers in form of seawalls, revetments and dune reinforcement, b) cross-shore barriers in form of sea groins and c) a new possible application of GSCs as a core of rubble mound structures. The latter type of application may also become particularly important when armouring the GSC-structure is required due to too severe wave attack, abrasion, UV-radiation and vandalism.

3. 1 *Revetments, Seawalls and Reinforcement of Beach-Dune System*

Most of the applications of geotextile containment in coastal engineering belong to this type of shore protection; i.e. the containment is built directly along the shoreline to prevent erosion and to stabilize a beach-dune system during storm surge (Fig. 7). For this purpose, different types of containments

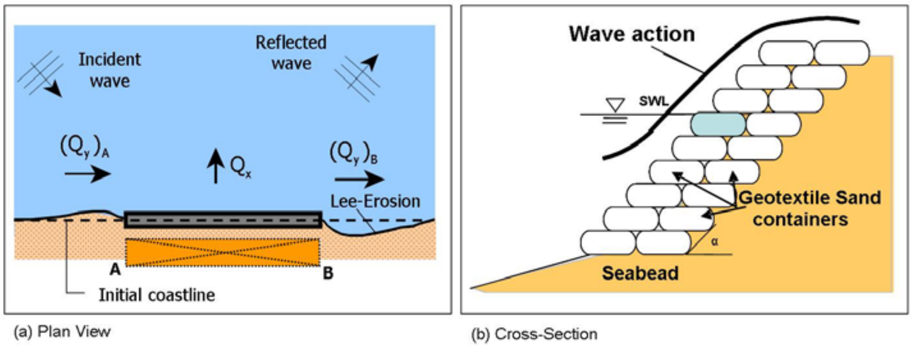


Fig. 7 Seawall Made of Geotextile Sand Containers (Principle Sketches)

have been applied, very often as a last defence line in combination with beach nourishment.

An impressive example of the performance of such a last defence line behind a beach nourishment is the wrapped sand containment needle-punched composite geotextile (woven PP slit film and non-woven PET) to reinforce a dune on the island of Sylt (North Sea, Germany) is shown in Fig. 8

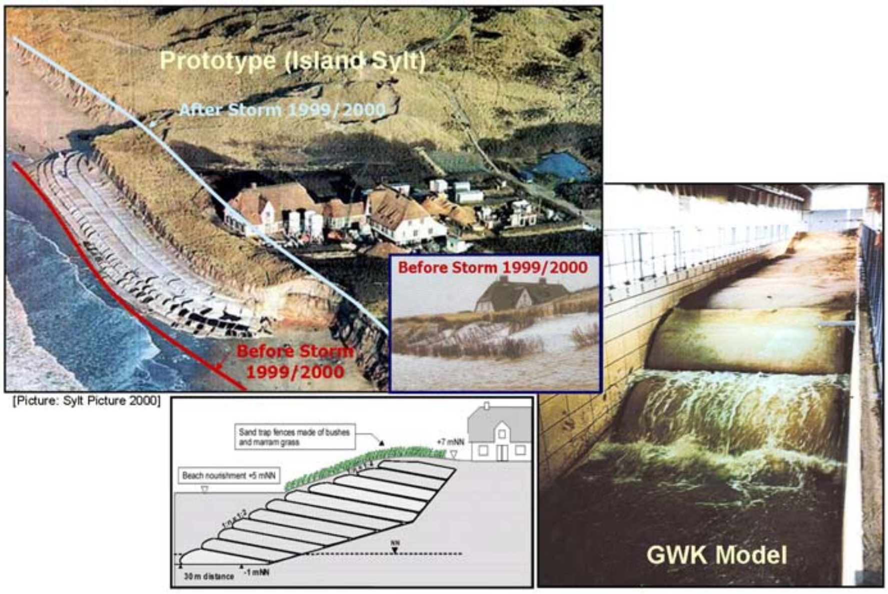


Fig. 8 Geotextile Containment for Dune Reinforcement, Sylt/Germany (Extended and Modified from Nickels and Heerten, 2000)

The stability of this stepped barrier was successfully tested in the Large Wave Flume (GWK) of Hannover. It survived since 1990 several storm surges with water levels of about 2.5m above Normal and wave heights up to 5m. Only the sand cover was removed, confirming that the nickname “Bulletproof Vest” commonly given to this type of construction is appropriate. More details on the design and construction of this shore protection are given by Nickels and Heerten (2000) and Lenze et al (2002). Similar geotextile sand containments have also been used successfully at many other sites. Some of them are well documented by ACT (2006).

More flexible and much simpler in both engineering design and installation, but equally efficient are smaller volume containers (Fig. **9**).

(a) Beach reinforcement in Australia

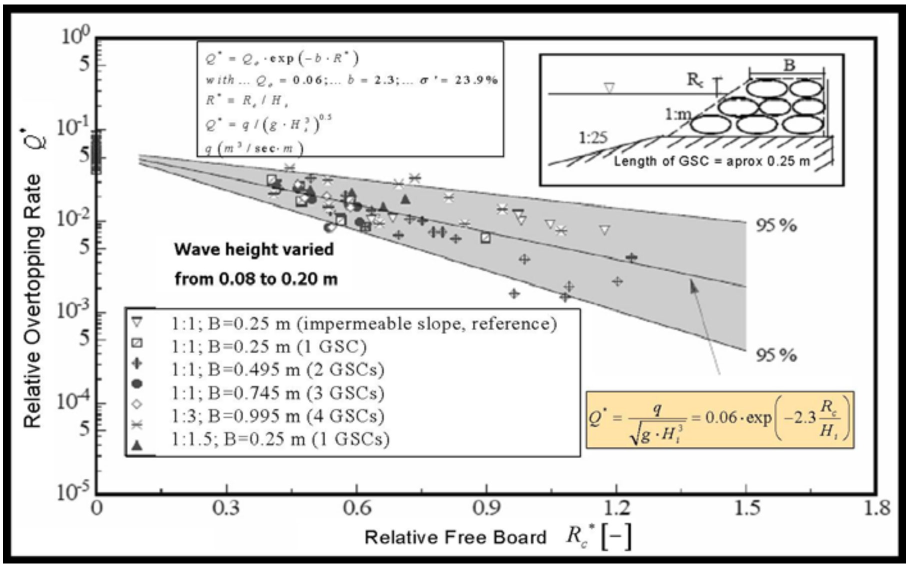
b) Stockton Beach Revetment (Saathoff et al, 2007)

c) Dune Reinforcement in Wangerooge, North Sea/Germany (Vohlken et al, 2003)

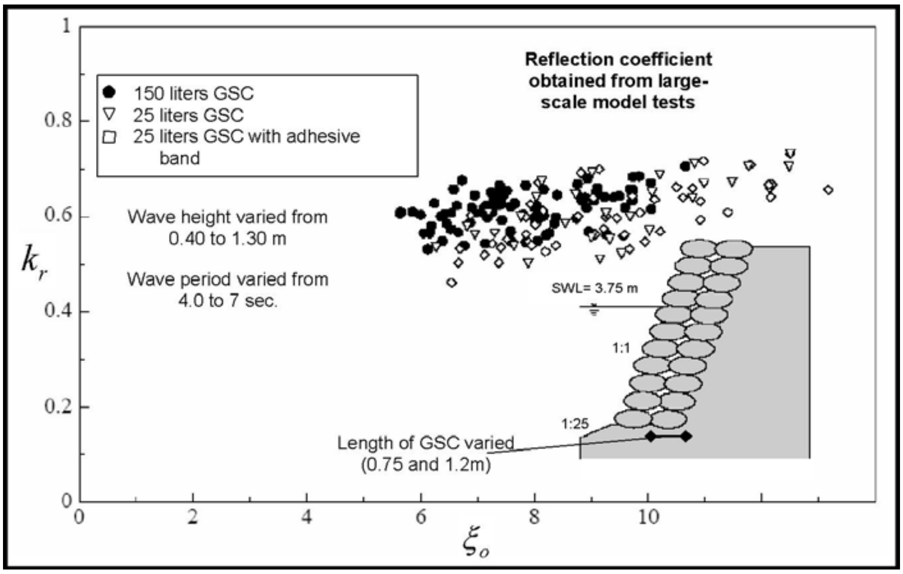
Fig. 9 Beach and Dune Reinforcement with Geotextile Sand Containers

Moreover, smaller containers have many advantages over larger containers and tubes (see Section 1 and Buckley and Hornsey, 2006). A comparative analysis of containers with $V=0.75m^3$ and containers with a 30 times larger volume for dune reinforcement which confronted the pros and cons of both methods from the client and contractor view point clearly resulted in the selection of the smaller containers (Buckley and Hornsey, 2006).

Using geotextile containment for a long-shore barrier in form of seawalls, revetment or beach-dune reinforcement has several advantages over hard barrier such as rock structures (see Section 1), but there are also some drawbacks. The most important is that the sand cover has to be fully or partially rebuilt after each important storm surge, because a naturally recovery is not always possible. Due to their lower permeability and their larger slope steepness, GSC-structures have generally higher refection coefficients and higher wave overtopping rates than rubble mound structures (Fig. **10**).



(a) Overtopping Performance



(b) Reflection Performance

Fig. 10 Wave Reflection and Overtopping performance of GSC-seawalls (Oumeraci et al, 2003)

Moreover, it should be kept in mind, that a long-shore GSC-barrier also induces a down drift erosion similar to those induced by hard structures (Fig. **11**), including erosion of the foreshore.

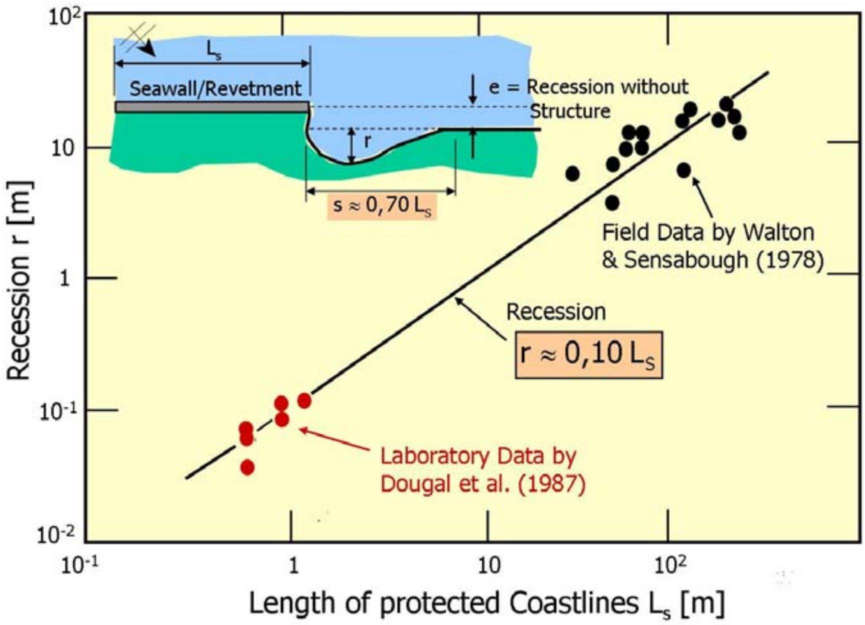


Fig. 11 Down Drift Erosion Induced by Seawalls

3. 2 *Sea Groins*

Geotextile sand containers, including geotubes, have often been used for emerged and submerged groins (Fig. **12**)

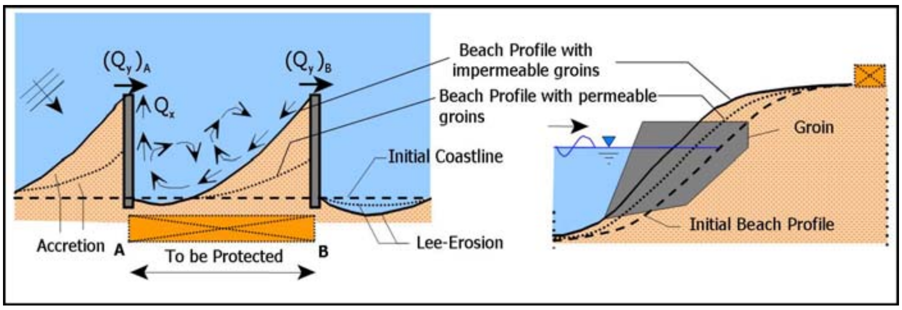


Fig. 12 Sea Groins (Principle Sketches)

Generally, the containers are uncovered and directly exposed to wave impact, abrasion and UV-radiation. Therefore, the fabric should be heavily treated for UV-stability and should consist of an inner layer for strength and an outer layer for robustness, durability and abrasion resistance (Geocomposite). Sometimes, the fabric is made in a colour that blends naturally with the beach environment (McClarty et al, 2006). Heavy-duty UV stabilized non-woven needle-punched geotextiles with high tenacity polyester thread in all seams (min. 80% of basic fabric strength) have also been successfully used (Restall et al, 2002). Despite large displacement that occurred during storms, the groin continued to provide protection and even withstood abrasion and UV attack over 10 years (Saathoff et al, 2007). To reduce the costs in the case of larger groins and larger projects the, groin core can be made of smaller container with lower requirements while strengthening the outer protective layer (McClarty et al, 2006). Further example applications are provided by Saathoff et al (2007) and Restall et al (2002, 2005).

3. 3 *Core of Rubble Mound Structures*

There are several reasons which might lead the engineer and other decision makers in practice to use sand instead of conventional quarry run for the core of rubble mound breakwaters and structures, including among others:

(i) Non-availability of rock material in sufficient quantities and at affordable costs.

(ii) Sediment infiltration through rubble mound structures which may result in the shoaling of navigation channels and harbour basins, and thus in higher maintenance dredging costs.

(iii) Reduction of wave transmission through the structure which might particularly be crucial in the case of long waves.

On the other hand, the use of a sand as a quasi-impervious core instead of quarry stone would result in an increase of

(i) wave set–up and run-up at the structure

(ii) wave overtopping

(iii) wave reflection

which might be detrimental to the stability of the structure, to the operation on and behind the breakwater (due to excessive overtopping) as well as to navigation and seabed stability.

Moreover, serious difficulties arise in practice when trying to design and construct the filter to protect the sand core against wash out by wave action.

Applying geometrically closed filter criteria would result in very complex, multiple and relative thin filter layers which will not only be very costly and very difficult to build in larger water depths, but also might certainly fail due to the almost unpredictable very complex loading conditions of the sand core under cyclic pulsations by waves and entrained air at the interface with the last filter layer. Such failures have indeed been observed under both laboratory and field conditions in the past. Laboratory evidence has also shown that introducing the so-called "geometrically open filter" criteria to design a "hydraulic sand-tight filter" may reduce the number of filter layers. However, the main practical difficulties mentioned above will remain, including those associated with the long-term stability of the sand-core due to the high complexity of the loading and its uncontrollability during the entire storm duration and the life cycle of the structure.
Geotextile filters might present themselves as an alternative to the very complex, costly and uncertain filter made of multiple layers of granular material. However, geotextile mats are not only difficult to install under waves and currents, but also may introduce a shear surface which might be detrimental to the stability of the armour layer.
A more feasible alternative is to use a core made of geotextile sand containers. This will not only allow to overcome the aforementioned core stability problems, but also to provide (i) a better erosion stability of the core and (ii) an increased stability against seismic loads as compared to a core simply made of loose sand. However, many of the drawbacks mentioned above remain with respect to wave set-up, run-up, overtopping, reflection and armour stability in comparison to a conventional core. Therefore, an extensive research programme has been initiated at Leichtweiss-Institute to study both hydraulic performance and armour stability, including the processes involved and the development of prediction formulae for the design of a class of rubble mound structures with a core made of geotextile sand containers (Fig. 13).

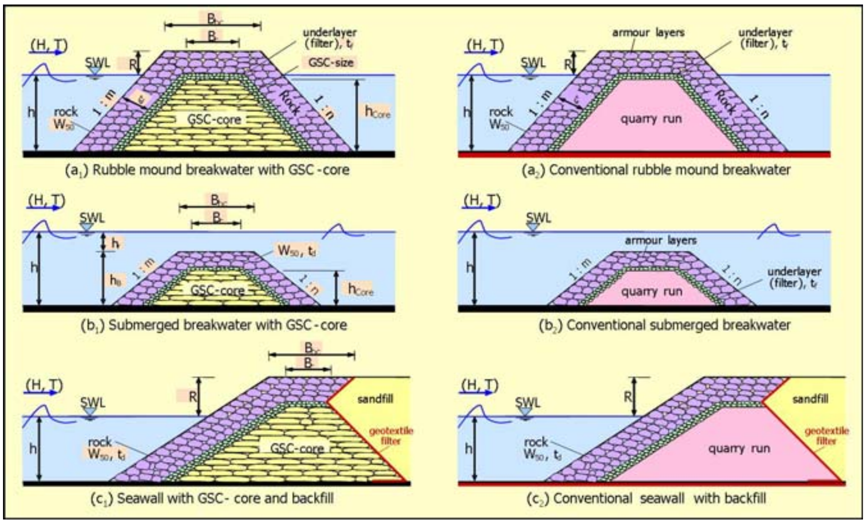


Fig. 13 Class of Geocore Structures in Comparison to Conventional Rubble Mound Structures

The first phase of this research programme which is concerned with hydraulic model tests to study in the twin-wave flumes of LWI the hydraulic performance and the armour stability of a rubble mound breakwater made of geotextile sand containers as compared to its conventional counterpart with a core made of quarry stones, has been completed and the results are published by Oumeraci et al (2007). Prediction formulae for wave reflection, wave run-up and overtopping, wave transmission and armour stability have been determined for the non-conventional breakwater in comparison with the conventional rubble mound breakwater (Oumeraci et al, 2007).

4. Hydraulic Stability of Geotextile Sand Containers

Depending on the prevailing wave loads and degradation mechanisms, geotextile sand containers may experience different types of failure modes: (i) hydraulic failure modes, (ii) geotechnical failure modes and (iii) failure modes related to the geotextile itself (Fig. 14). In the following, only the first type which is related to hydraulic stability under wave loads will be addressed.

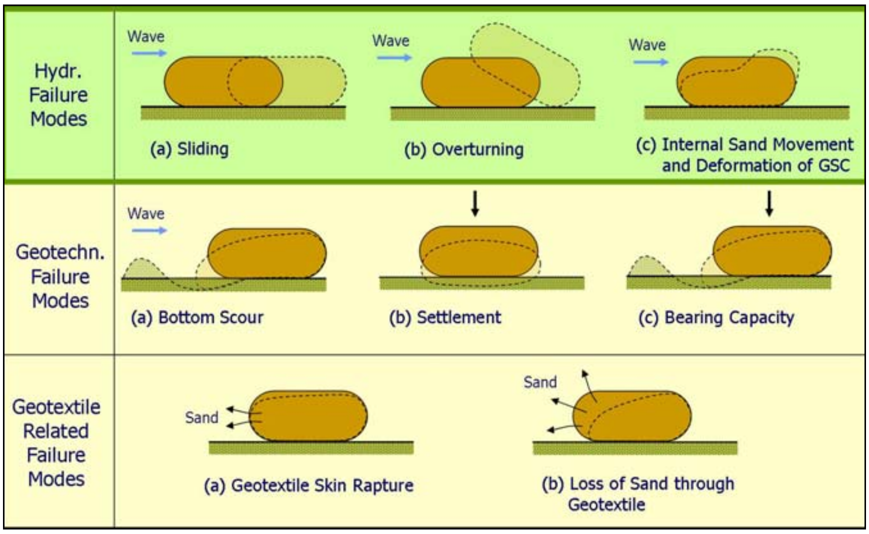


Fig. 14 Class of Geocore Structures in Comparison to Conventional Rubble Mound Structures

First, simple stability formulae without explicit account of the effect of deformation will be proposed separately for the containers on the slope and the containers on the crest of the structure (called hereafter "slope containers" and "crest containers").

Second, the necessity of a better understanding of the processes responsible for the deformation of the sand containers under wave loads as well as their effect on the hydraulic stability is illustrated by some selected results from recent research.

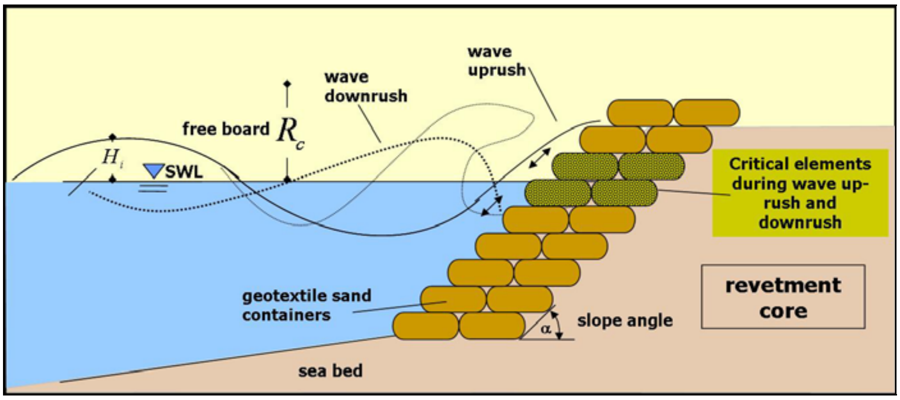
Finally, more detailed stability formulae are proposed which can also explicitly account for the deformation effects, including a comparison with the simple stability formulae.

4.1. *Simple Stability Formulae*

Due to the different wave loads and boundary conditions which prevails on the slope and on the crest of a coastal structure, a different stability behaviour and thus different stability formulae are expected for the containers on the slope and those on the crest. The following results are extracted from the research reports of too comprehensive laboratory studies: small-scale model tests performed in the wave flume of Leichtweiss-Institute, using 1-liter sand containers subject to random waves up to 20cm height and large-scale model tests in the large wave flume of the Joint Coastal Research Centre (FZK) of both Universities Hannover and Braunschweig, using 150-liter sand containers subject to random waves up to 1.6m height. (Oumeraci et al, 2002, Oumeraci et al, 2003).

4.1.1 *Stability of Slope Containers*

The sand containers on the slope which are located around the still water level are repeatedly moved up and down by the waves rushing up and down the slope, leading to an incremental seaward displacement of the containers. This dislodgement/pull out effect is illustrated by Fig. **15** as observed in the wave flume and in the field.



a) Wave up rush and down rush on slope containers

b) Pull out effect in the FZK-large wave flume

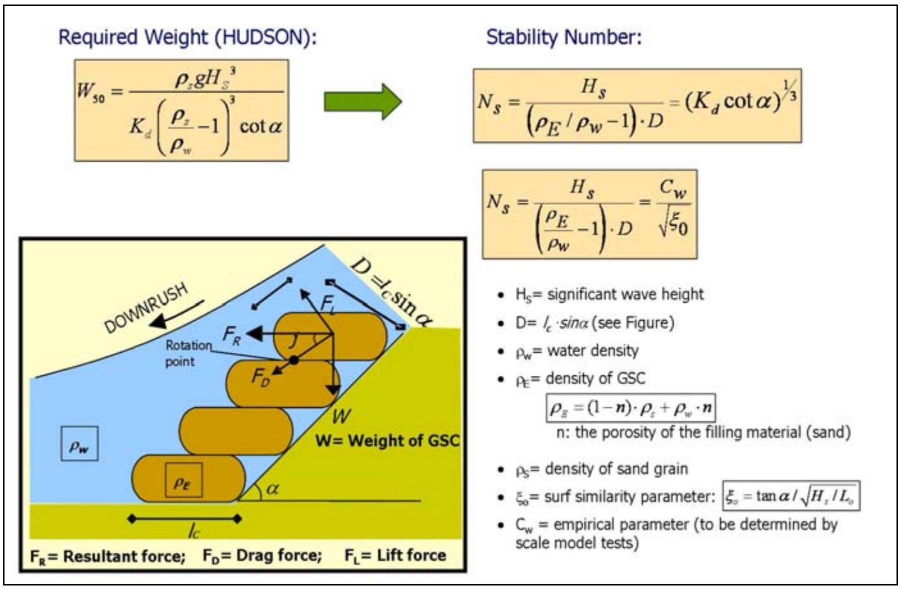
c) Pull out effect in a dune reinforcement (Courtesy by Heerten)

Fig. 15 Hydraulic Failure Modes of Slope Containers

Based on the HUDSON-formula for the hydraulic stability of rock armour units (non-deformable) and similarly to WOUTERS (1998), a stability number N_s is formulated and postulated to be a function of the surf similarity parameter ξ_0, which includes both the slope steepness $\tan\alpha$ as well as the significant wave height H_s and the wave length L_{op} (Plate 2):

$$N_s = \frac{H_s}{\left(\frac{\rho_E}{\rho_W} - 1\right) \bullet D} = \frac{C_w}{\xi_0}, \tag{1}$$

Plate 2. Stability of Slope Containers based as HUDSON-Formula



With the surf similarity parameter $\xi_0 = \tan\alpha \big/ \sqrt{H_s / L_{op}}$ expressed in terms of the deep water length $L_{op} = gT_p/2\pi$ (T_p = peak period of wave spectrum) the following stability formula is obtained in terms of the characteristic size D of the container:

$$D = \frac{H_s^{3/4} \bullet T_p^{1/2} \bullet (\tan\alpha)^{1/2}}{C_w \bullet \left(\frac{2\pi}{d}\right)^{1/4} \left(\frac{\rho_E}{\rho_W} - 1\right)} \tag{2}$$

Defining the characteristic size D as $D = l_c \sin\alpha$ according to WOUTERS (1998) and according to the principle sketch in Plate 2, Eq. (2) can be reformulated in terms of the length l_c of the slope containers as:

$$l_c = \frac{H_s^{3/4} \bullet \sqrt{T_p}}{C_w \bullet \left(\frac{2\pi}{g}\right)^{1/4} \left(\frac{\rho_E}{\rho_W} - 1\right) \sqrt{\frac{\sin 2\alpha}{2}}} \tag{3}$$

The empirical parameter C_w was determined by stability tests in a large and a small wave flume to c_w = 2.75. In Fig. **16** only the results of the large-scale model tests are plotted to illustrate that the threshold curve between stable and unstable containers is obtained for C_w = 2.75.
For C_w = 2.75 and g = 9.81m/s² Eq. (3) reduces to:

$$l_c = \frac{H_s^{3/4} \bullet \sqrt{T_p}}{1.74 \bullet \left(\frac{2\pi}{g}\right)^{1/4} \left(\frac{\rho_E}{\rho_W} - 1\right) \bullet \sqrt{\sin(2\alpha)}} \tag{4}$$

with
H_s = significant wave height [m]
T_p = Peak period of waves [s]
α = slope angle of structure [°]
ρ_E = bulk density of GSC [kg/m³]
ρ_W = density of water [kg/m³]

$$\rho_E = (1-n) \bullet \rho_s + \rho_W \bullet n \qquad \text{(with } \rho_E \approx 1800 \text{ kg/m}^3 \text{ for sand)}$$

n = porosity of fill material [-]
ρ_S = density of grain material [kg/m³]

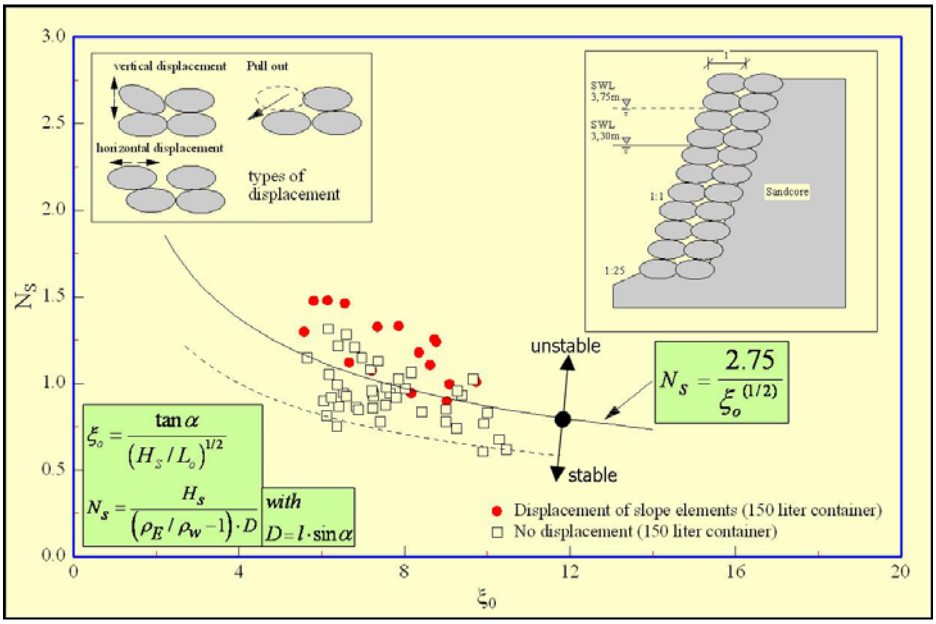


Fig. 16 Stability of Slope Containers from Large-Scale Model Tests (modified from Oumeraci et al, 2003)

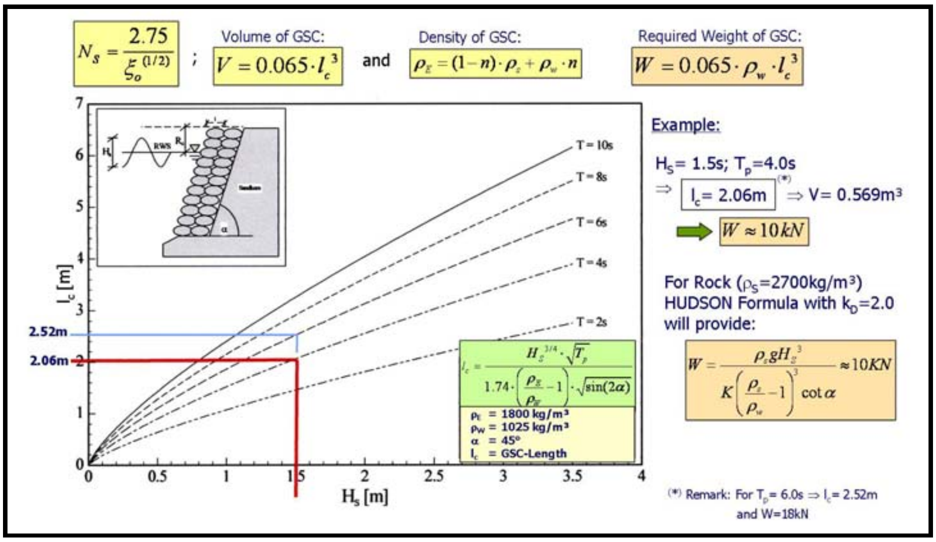


Fig. 17 Required Length of Slope Containers for Hydraulic Stability

The stability formula in Eq. (4) expressed in terms of the required container length l_c is plotted in Fig. 17 for a structure slope angle $\alpha = 45°$ in order to illustrate the
sensitivity to the significant wave height H_s and to the peak period T_p. To compare the results with those of the HUDSON-Formula for rock armour, the volume of the tested slope containers ($V = l_c \cdot 0.46\ l_c \cdot 0.14\ l_c = 0.065\ l_c^3$) is considered. For $H_s = 1.5$m and $T_p = 4$s, about the same required weight for slope containers with $\rho_E = 18000$ kg/m³ is obtained as for a rock armour with $\rho_E = 2700$kg/m³ and $k_d = 2.0$. Using a 10 times higher k_d-value ($k_d = 20$) and applying HUDSON-formula in this specific case for slope containers would indeed provide the same result as the proposed formula. However, such an approach is not applicable as the stability of slope container is very sensitive to the wave period and the k_d-value is expected to be a function of both wave height and wave period, due to the deformation effect caused by wave action. In fact, if for the same wave height $H_s = 1.5$m the wave period is increased from $T_p = 4$s to $T_p = 6$s the required length l_c will increase by more than 20 % (Fig. 17). Moreover, due to the effect of deformation of the slope containers on the long term stability it is not advisable to use unprotected slope containers for design wave heights of $H_s > 1.5$ to 2.0 m.

4.1.2 *Stability of Crest Containers*

The sand containers on the crest of the structure may fail due to two possible mechanisms (Fig. 18):

(i) uplifting during the wave up rush process and shoreward displacement by the wave overtopping flow
(ii) dislodgement and pull out effect similar to the mechanism observed for the slope containers

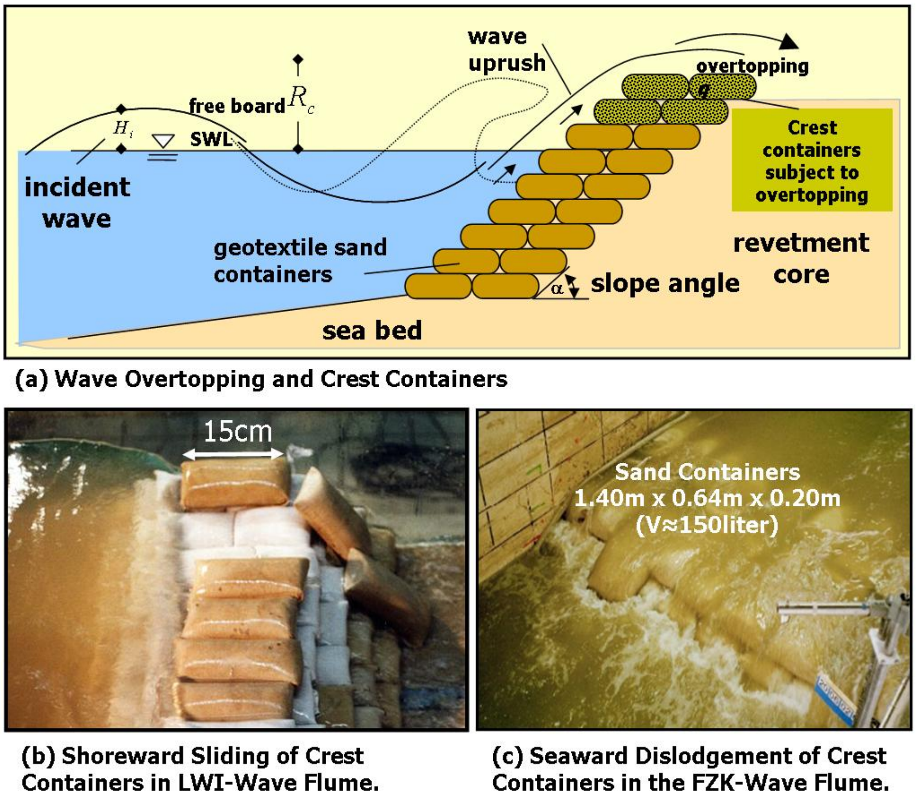


Fig. 18 Hydraulic Failure Modes of Crest Containers

Due to the boundary conditions of the crest containers which are more critical than those of the slope containers (no overburden from upper layers) it is expected that the stability of the crest containers will be more critical than that of the slope containers if the crest level of the structure is not high enough. The relative freeboard Rc/H_s therefore represents the most important influencing parameter. In fact, it was difficult to identify a noticeable effect of the surf similarity parameter ξ_0 on the stability number N_S as clearly observed for slope containers (Fig. **19**). However, plotting at the left top of Fig. **19**, the stability number N_S against the relative freeboard Rc/H_s shows that the stability number of the crest containers increases with increasing relative freeboard Rc/H_s according to the following linear relationship (Fig. **19**):

$$N_S = \frac{H_s}{\left(\frac{\rho_E}{\rho_W} - 1\right) \cdot D} = 0.79 + 0.09 \frac{R_c}{H_s}, \qquad (5)$$

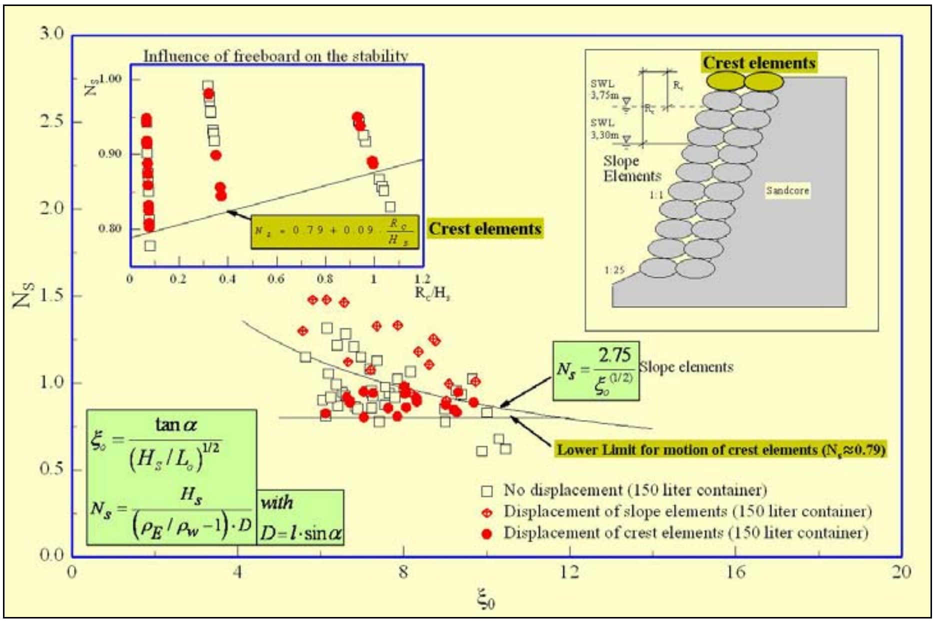


Fig. 19 Stability of crest containers form Large-Scale model tests (Oumeraci et al, 2003)

With $D = l_c \sin\alpha$ substituted in Eq. (5), a stability formula for crest containers is obtained in terms of the required container length l_c:

$$l_c = \frac{H_s}{\left(\frac{\rho_E}{\rho_W} - 1\right)\left(0.79 + 0.09\frac{Rc}{H_s}\right)\sin\alpha} \tag{6}$$

In a similar way as in Fig. 17 for slope containers, the required container length l_c in Eq. (6) is plotted against the design wave height H_s for different relative freeboard $Rc/H_s = 0\text{-}2.0$ (Fig. 20). Considering exemplarily a typical value $Rc/H_s = 1.2$ and $H_s = 1.5$m, a container length $l_c = 3.15$m and a container weight W = 36.6kN are obtained which are much larger than those obtained for the slope containers in Fig. 17.

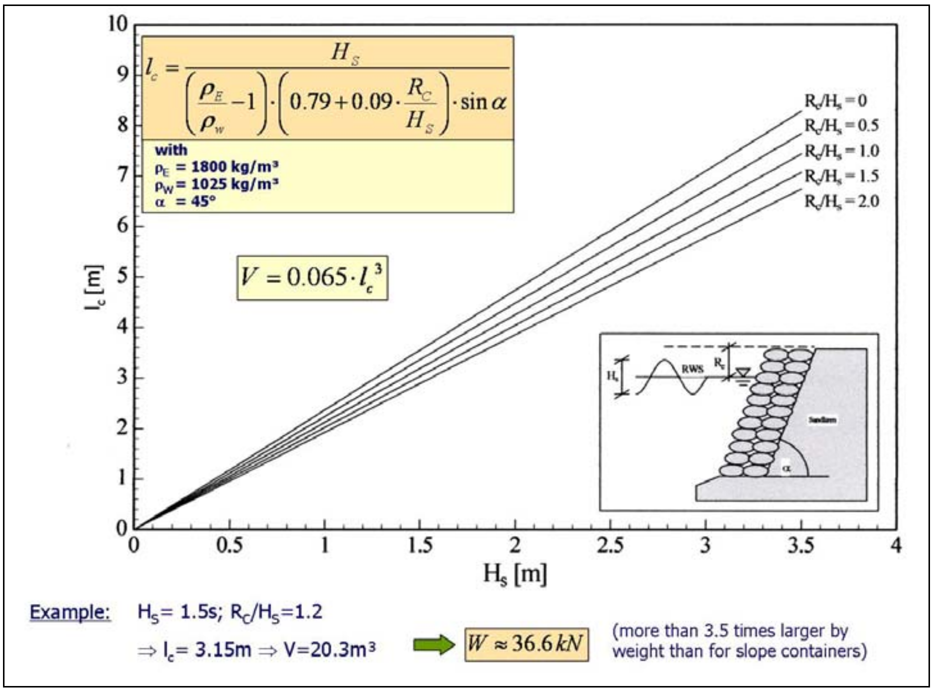


Fig. 20 Required Length of Crest Containers for the Hydraulic Stability

4.2. *Processes Affecting the Hydraulic Stability*

4.2.1 *Position of the Problem and Need to Improve Process Understanding*

As observed in the experiments and shown in Fig. **21**, the dislodgment and pull out of the slope containers by wave action, including the sliding and overturning of crest containers are strongly affected by the deformation of the sand containers. Simple stability formulae like those proposed in the previous section 4.1 cannot explicitly account for the deformation and other mechanisms affecting the hydraulic stability.

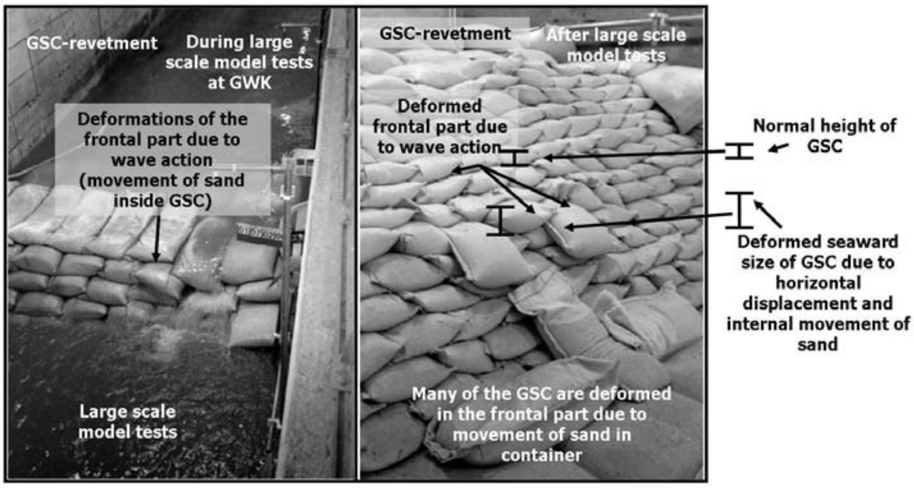


Fig. 21 Observed Failure Mechanisms Under Wave Loads

An improved understanding of the processes and mechanisms is needed in order to

(i) possibly avoid failure (engineering judgment)

(ii) develop more process-based stability formulae (see Section 4.3 below).

For this purpose it is necessary to address the following aspects:

(i) hydraulic permeability of GSC-structures and its effect on the stability, (ii) wave loads and identification of the most critical loading case and location of the containers, (iii) internal movement of sand fill and its effect on the stability, (iv) effect of the friction angle between geotextile containers on the stability and (v) effect of the container deformations on the stability.

The content of this Section 4.2 and next Section 4.3 represents a brief summary of selected key results which have essentially been obtained from comprehensive laboratory studies, including several types of experiments in combination with numerical studies using a CFD-code (RANS-VOF model COBRAS developed at Cornell University) partially coupled with a CSD-code (FEM-DEM code "UDEC" developed by Itasca). These studies were performed in the framework of the PhD-research work by Recio (2007) and the results are described in more detail in several research reports (Recio and Oumeraci, 2006a, c, d; 2007 a, b, c) and in a PhD thesis (Recio, 2007).

4.2.2 *Hydraulic Permeability of GSC-structures*

Surprisingly, no information on the assessment of the hydraulic permeability of structures made of geotextile sand containers could be found in the published literature, although permeability represents an important parameter for both hydraulic stability and hydraulic performance (wave transmission, reflection run-up and overtopping). Moreover, reliable permeability values of GSC-structures are also needed for numerical simulations. Comprehensive laboratory experiments supplemented by numerical simulations using the COBRAS-code were performed for the first time to investigate the hydraulic permeability of several types of GSC-structures for different modes of placement and sizes of the containers, and based on the achieved results then to develop a conceptual model for the assessment of the hydraulic permeability (Recio and Oumeraci, 2007a; 2008a; Recio, 2007). The key results may be summarized as follows:

(i) The permeability of a GSC-structure is essentially governed by the size of the gaps between the containers. The flow through the sand fill can therefore be neglected in the computation.

(ii) The hydraulic permeability of GSC-structure is generally more than ten times higher and ten times lower than the permeability of sand ($k \approx 10^{-3}$ m/s) and gravel ($k = 10^{-1}$ m/s), respectively (see Fig. 22).

Model Structure	Description	Darcy's Permeability Coefficient k (m/s)
	Structure made of geotextile sand containers **placed interlaid** blocking the gaps of the previous layer	**1.244×10^{-2}**
	Structure made of geotextile sand containers **placed longitudinally** to the flow	**2.274×10^{-2}**
	Structure made of geotextile sand containers **placed randomly**	**2.412×10^{-2}**
	Structure made of **gravel** (D_{50} = 2.3 cm, D_{max}=2.9cm, D_{85}/D_{15}=1.4).	**3.881×10^{-1}**

Remark: Permeability of gravel is normally higher than 10^{-2}m/s and permeability of sand is between $1\text{x}10^{-3}$ and $3\text{x}10^{-3}$m/s.

Fig. 22 Hydraulic Permeability Coefficients for Different Mode of Placement of Geo-Containers (Recio and Oumeraci, 2007a, 2008a)

(iii) The mode of placement of GSCs generally affects the permeability of the GSC-structure (Fig. 22). However, randomly placed GSCs and longitudinally placed GSCs have approximately the same permeability coefficient: k = 2.4 cm/s and k = 2.3 cm/s, respectively. To achieve a lower permeability an "interlaid" placement (gaps of the longitudinally placed containers are blocked by transversally placed containers) is required, allowing to reduce the permeability up to k = 1 cm/s and even to k = 0.5 cm.

(iv) A conceptual model and a systematic procedure to predict the hydraulic permeability is proposed by Recio and Oumeraci (2007a, 2008b) Validation by experimental data show that uncertainties less than 30 % would be expected when applied to prototype containers.

4.2.3 Wave-Induced Loads and Critical Location of Slope Containers

Based on the pressure recorded by load sensors around an instrumented container placed at different locations along the slope of the GSC-structure subject to both breaking and non-braking waves, the horizontal and vertical components of the total wave force on the instrumented containers is obtained for each time during a wave cycle. A typical result of the measurement is exemplarily provided in Fig. 23 to illustrate that the highest horizontal force in seaward direction and the highest vertical force in upward direction occur on the container located just below still water level.

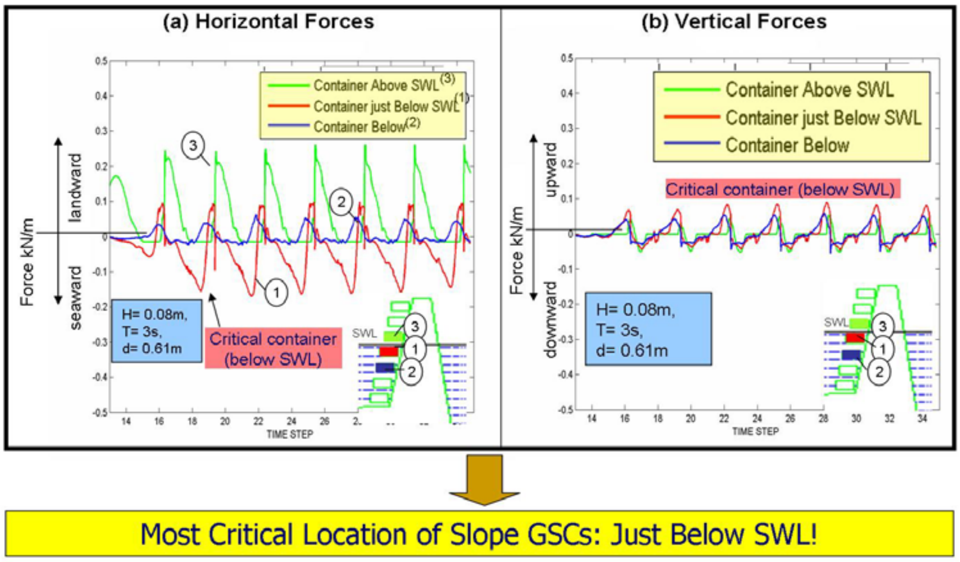


Fig. 23 Wave-Induced Force: Identification of Critical Container Location (Recio and Oumeraci, 2007, 2008b)

Using pressure measurements within the gaps between slope containers for both breaking wave impacts and non breaking waves, it is found tat the former are less critical for the hydraulic stability than the non-breaking waves rushing up and down the slope and causing more damage. In fact, the impact pressure induced by breaking waves is of much shorter duration. Moreover, it is strongly damped when propagating inside the gap (Fig. **24**).

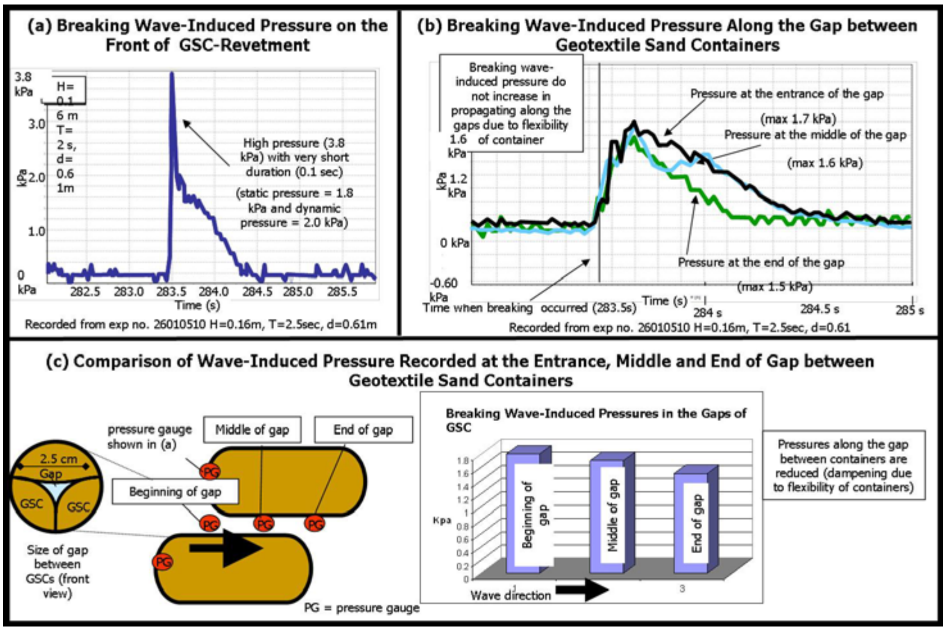


Fig. 24 Impact Pressure Propagation Inside a Gap Between Geotextile Sand Containers (Recio and Oumeraci, 2007b, 2008b)

4.2.4 *Movement and Redistribution of Sand Inside the Containers Induced by Wave Action*

The analysis of the video records of the movement of coloured sand inside a transparent permeable container built in the slope of the GSC-structure (Fig. **25**) and subject to wave action have shown that (Fig. **26**):

(i) Noticeable sand movements occur only for larger waves that are capable to substantially move the front part of the container up and downward during the wave run-up and run down process (Fig. **26** b). After about 30 wave cycles the internal sand movement decreases significantly due to the accumulated sand at the seaward front of the container (Fig. **26** c).

(ii) Due to the sand fill redistribution at the seaward front of the container, the latter deforms thus offering a larger impact surface area to the mobilizing wave forces and reducing the contact area with neighbouring containers (Fig. **26** c).

(iii) With the increased mobilizing forces and the decreased resisting forces, an incremental lateral seaward displacement of the deformed container occurs (pull out effect) which causes again the start of the internal sand movement in a similar way as during the first wave cycles (Fig. 26 a).

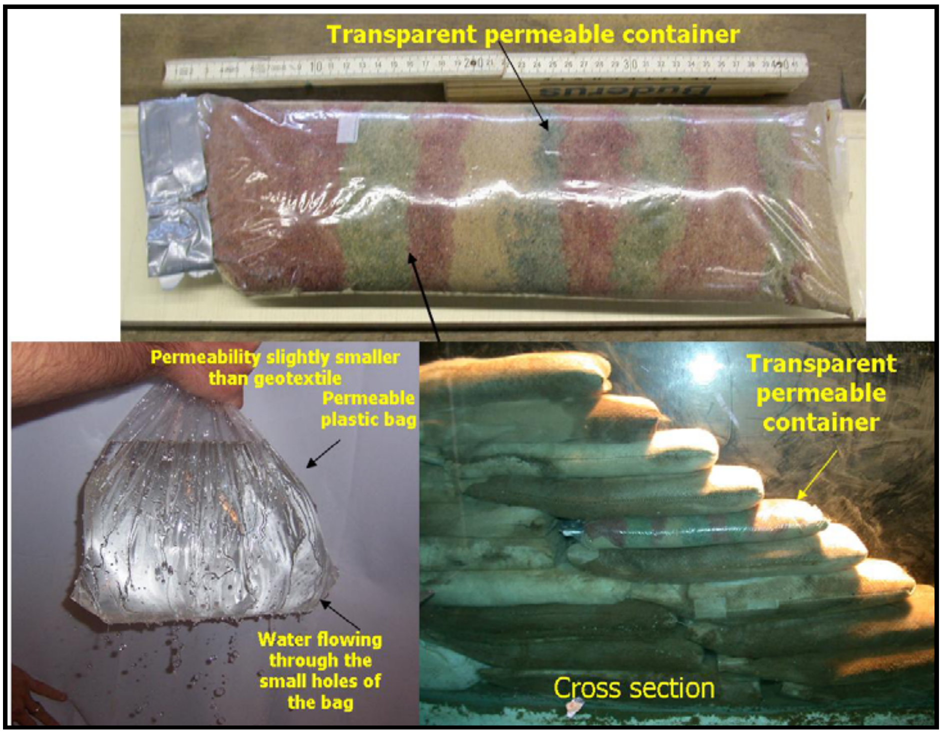


Fig. 25 Permeable Transparent Container for the Investigation of Internal Movement of Sand (Recio and Oumeraci, 2007a)

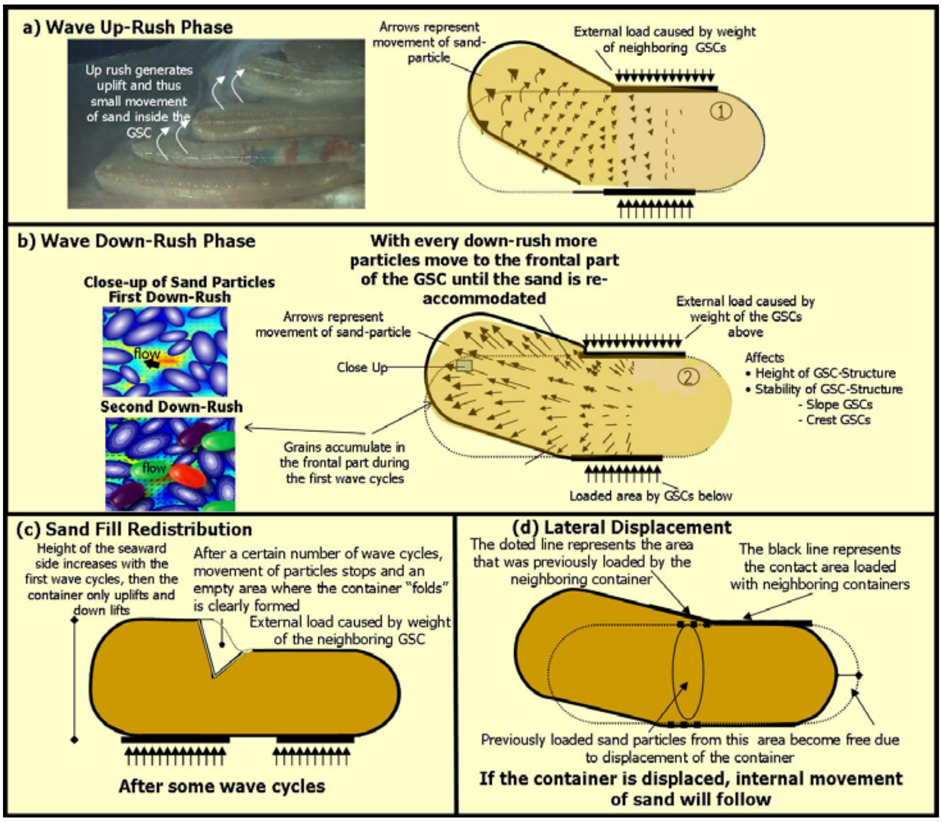


Fig. 26 Internal Movement and Redistribution of Sand Fill in the Transparent Container (Recio and Oumeraci, 2007b)

These results have considerable practical implications as the internal sand movement is responsible for the deformations of the container which affect

- the hydraulic stability by reducing the contact area between GSCs and by increasing the drag and lift forces due to the increased exposed areas,
- the crest level of a GSC-structure. Even in the laboratory it is observed that the height of sand containers is reduced by 4 % after placement under water and by further 6 % due to wave action. As a result, a total reduction of the height of the GSC-structure of about 10 % was observed (Recio and Oumeraci 2007a, 2008a)

Since these effects are strongly dependent on the adopted sand fill ratio, future research and design guidance should be directed towards the definition of an optimal sand fill ratio by accounting for the deformation properties of the geotextile and by balancing the advantages and drawbacks of high sand fill ratio. Moreover, this issue should also be explicitly addressed in future standards and guidelines due to the considerable effect of the sand fill ratio on stability and long-term performance.

4.2.5 *Friction Between Geotextile Sand Containers*

Friction angles between woven geotextiles vary from about 12° (e.g. MirafiGT500) and 20° (e.g. Geolon PP120S), while for non-woven geotextiles values of 20° to 26° (mechanical bound) or even 20° to 30° (thermal bound) are more common. The results of numerical simulations using the partially coupled CFD and CSD codes (COBRAS and UDEC) previously validated by laboratory data, as shown in Fig. 27, highlight the significant effect of friction between the GSCs on the hydraulic stability.

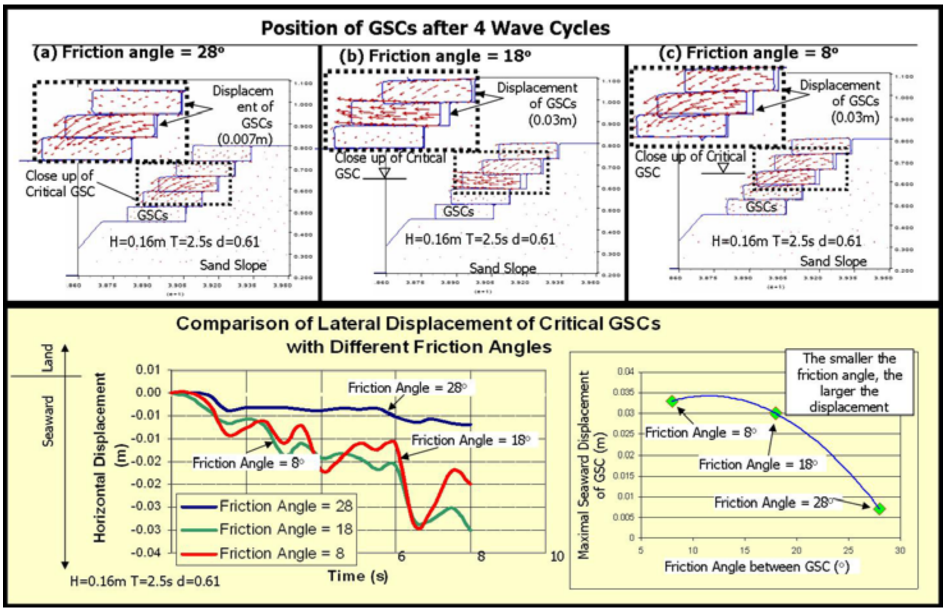


Fig. 27 Effect of Friction between GSCs on Hydraulic Stabilisation (Recio and Oumeraci, 2007b, 2008a)

The effect of the friction angle on the stability is particularly important within the range of the practically relevant values (15°-30°), implying that the friction between containers should be explicitly considered in future stability formulae.

4.2.6 *Effect of Container Deformations on the Stability*

It was shown in Section 4.2.4 that the sand fill is redistributed, resulting in deformations of the containers (Fig. **26**). These deformations affect the stability of the containers in the following manner:

(i) Reduction of the stability against sliding caused (a) by the increase of the drag and uplift forces as a result of the increased exposed areas A_S and A_T (Fig. **28**) as well as (b) by the decrease of the resisting force as a result of the decreased friction area between the containers (Fig. **28** Fig. **29**).

(ii) Reduction of the stability against overturning caused by the increase of the mobilizing drag and uplift forces as mentioned above, but also by the seaward shift of the centre of gravity of the deformed container leading to a reduction of the resisting moment (Fig. **30**).

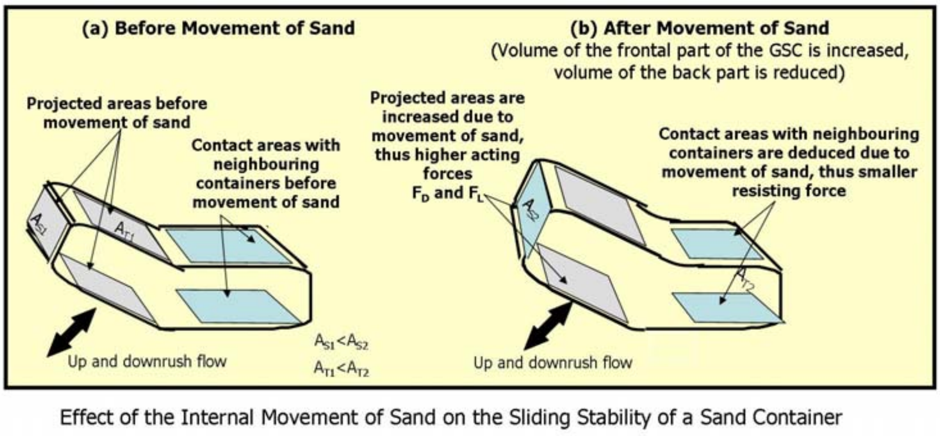


Fig. 28 Increase of Exposed Areas of Drag and Uplift Forces and Decrease of Friction Areas Due to Container Deformation

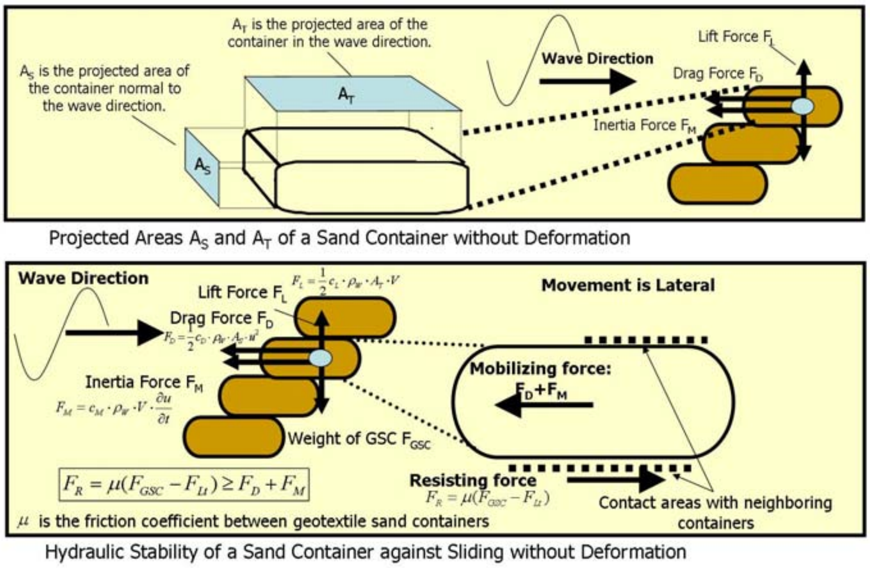


Fig. 29 Effect of Container Deformation on Sliding Stability
(Recio and Oumeraci, 2007a, 2008b)

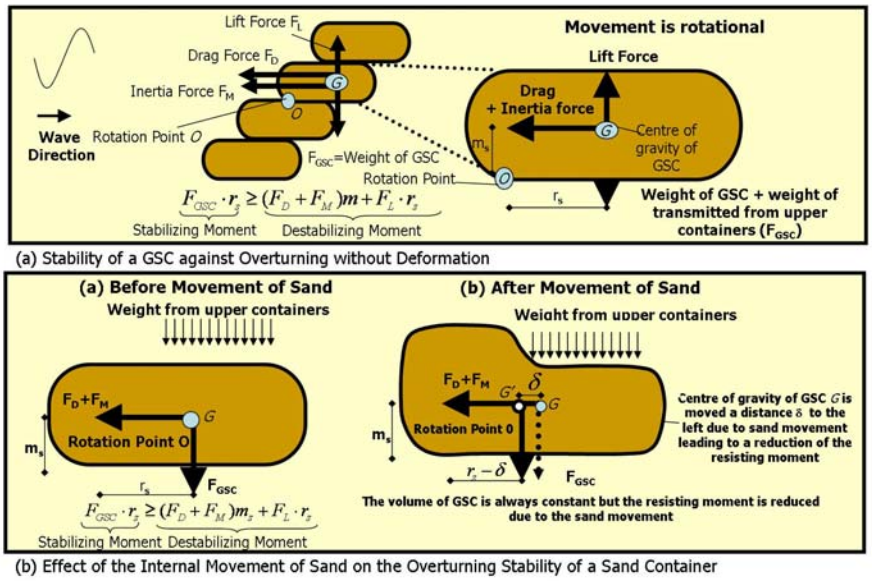


Fig. 30 Effect of Container Deformation on Overturning Stability
(Recio and Oumeraci, 2007b, 2008a)

A closer experimental and numerical examination of the variation of the "effective" contact areas between containers during wave action (i.e. those areas which contribute to the resistance against dislodgment by friction) has shown that the "effective" contact areas

(i) decrease due to the upward movement of the front part of the containers (Fig. **31**)

(ii) increase with increasing slope angle of the GSC-structure.

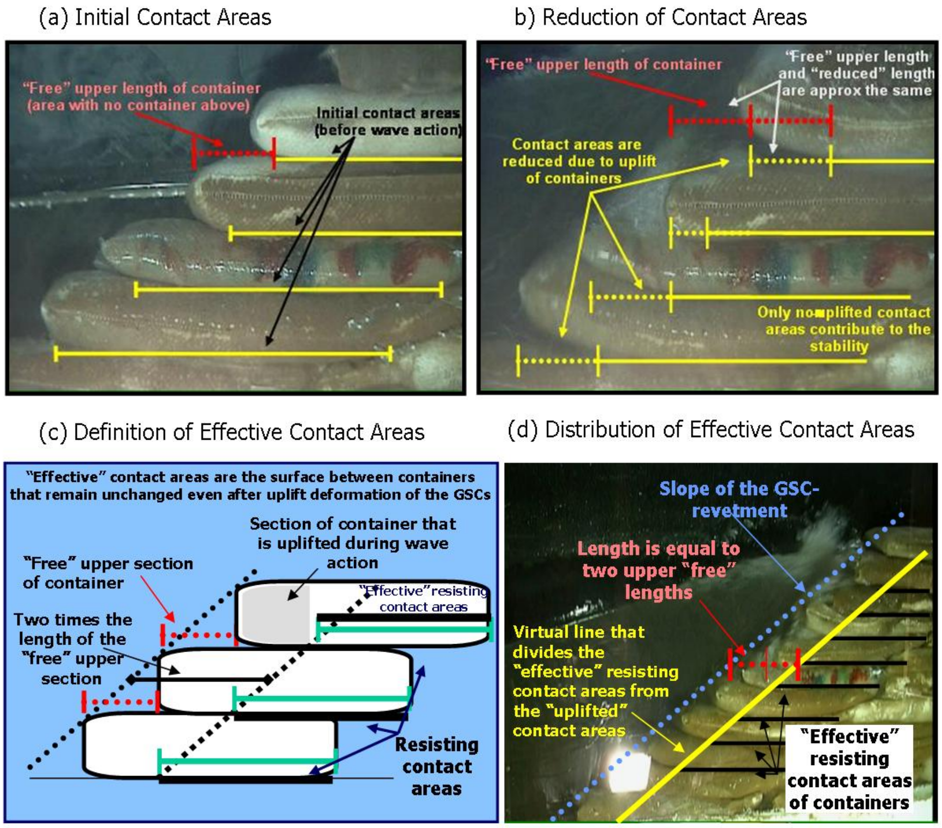


Fig. 31 Reduction of Effective Contact Areas between Containers during Wave Action (Recio, 2007)

4.3. *Process-Based Stability Formulae*

The insight in the processes affecting the hydraulic stability as described in the previous Section 4.2 has clearly highlighted the necessity of an explicit account of these processes in future stability formulae, at least those processes which mostly affect the stability such as the effect of deformations of the container and the friction between the containers.

In order to examine more closely the effect of deformation on the stability new process-based stability formulae are proposed: first without any account of the deformation effect and then correction factors are introduced to account for the deformation effect. Finally, a comparative analysis between the results of the stability formulae with and without deformation effect is provided for both slope and crest containers.

It should be underlined that the new formulae proposed below were derived analytically for the geometry of the containers commonly used in coastal structures, i.e. with a container length l_c which is twice the container width $(l_c/2)$ and five times the container height $(l_c/5)$ resulting in the following relationships for the volume of the container V and the application area of the drag and uplift forces A_S and A_T, respectively (Fig. **32**a)

$$\left.\begin{aligned} V &= 0.1\, l_c^3 \\ A_S &= 0.1\, l_c^2 \\ A_T &= 0.5\, l_c^2 \end{aligned}\right\} \tag{7}$$

These relationships provide the geometrical parameters that govern the resisting forces (weight) and the mobilizing forces (drag, inertia and uplift forces), thus allowing to express the stability formulae in terms of the container length l_c (see also Section 4.1). If, however other container geometries, and thus other relationships, which differ from those in Eq. (7) are adopted the stability formulae can be modified accordingly. Further indications on how to proceed with such modifications and on the limitations of the proposed stability formulae will also be given below.

4.3.1 *Stability Formulae without Deformation Effect*

(a) Stability against sliding: A sand container is stable as long as the resisting force $F_R=\mu(F_{GSC}-F_L)$ generated by the resulting normal force $(F_{GSC}-F_L)$ due to friction remains larger than the drag force F_D and the inertia force F_M (Fig. 32):

$$\mu\left(F_{GSC} - F_L\right) \geq \left(F_D + F_M\right) \tag{8}$$

And with the relative density parameter $\Delta = (\rho_E/\rho_W) - 1$:

$$\mu\left[\Delta gV - 0.5C_L A_T \cdot u^2\right] \geq \left(0.5C_D A_S u^2 + C_M V\frac{\partial u}{\partial t}\right) \quad (9)$$

Given the considered container geometry, relationships similar to those in Eq. (7) which provide V, A_T and A_S as a function of the container length l_c can be obtained and substituted in Eq. (9) which is then solved to obtain either the required length l_c or the required mass W_{GSC} of the container. Using for instance the geometry described by Eq. 7 the stability formulae are given in Fig. **32** in terms of the required length l_c or mass W_{GSC} of the container.

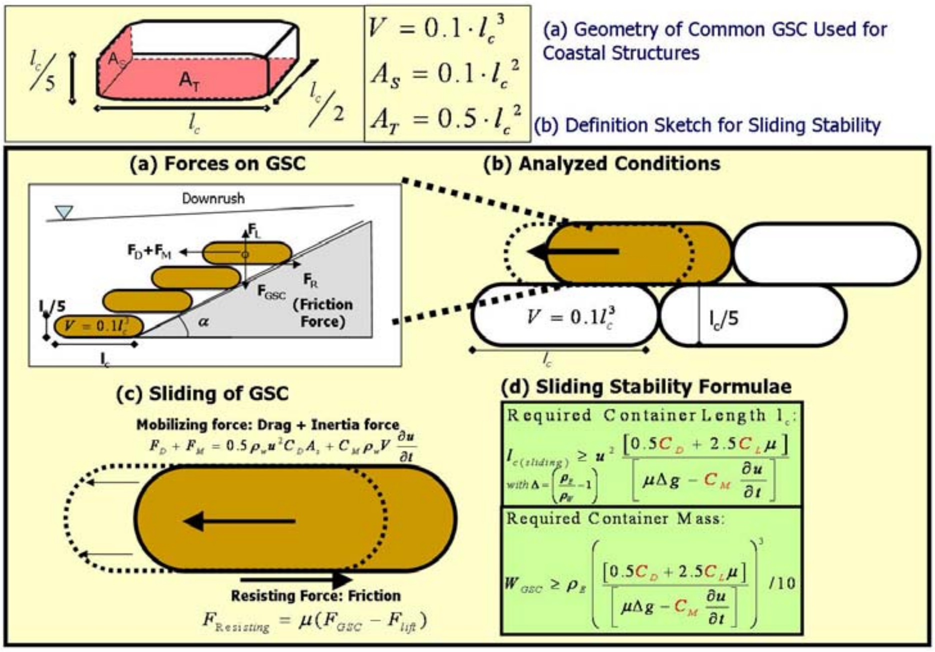


Fig. 32 Sliding Stability Formulae Without Deformation Effect

(b) Stability against overturning: A sand container is stable as long as the stabilizing moment induced by the weight of the container under buoancy F_{GSC} remains larger than the mobilizing moment induced by the drag, inertia and uplift forces F_C, F_M and F_L (Fig. **33**):

$$F_{GSC} \cdot r_s \geq F_D \cdot r_h + F_M \cdot r_h + F_c \cdot r_s \quad (10)$$

$$\left(\rho_E - \rho_w\right) gV \cdot r_s \geq 0.5\rho_w C_D u^2 A_s \cdot r_h + \rho_w C_M \frac{\partial u}{\partial t} V \cdot r_h + 0.5\rho_W C_L u^2 A_T \cdot r_s \quad (11)$$

Given the considered container geometry, relationships similar to those in Eq. (7) which provide V, A_s and A_T, but also the lever arms r_s and r_h as a function of the container length l_c can be obtained and substituted in Eq. (11). The latter is then solved to obtain either the required length l_c or the required mass W_{GSC} of the container using for instance the geometry described by Eq. (7). The overturning stability formulae are obtained in terms of the required length l_c or mass W_{GSC} of the container (Fig. 33).

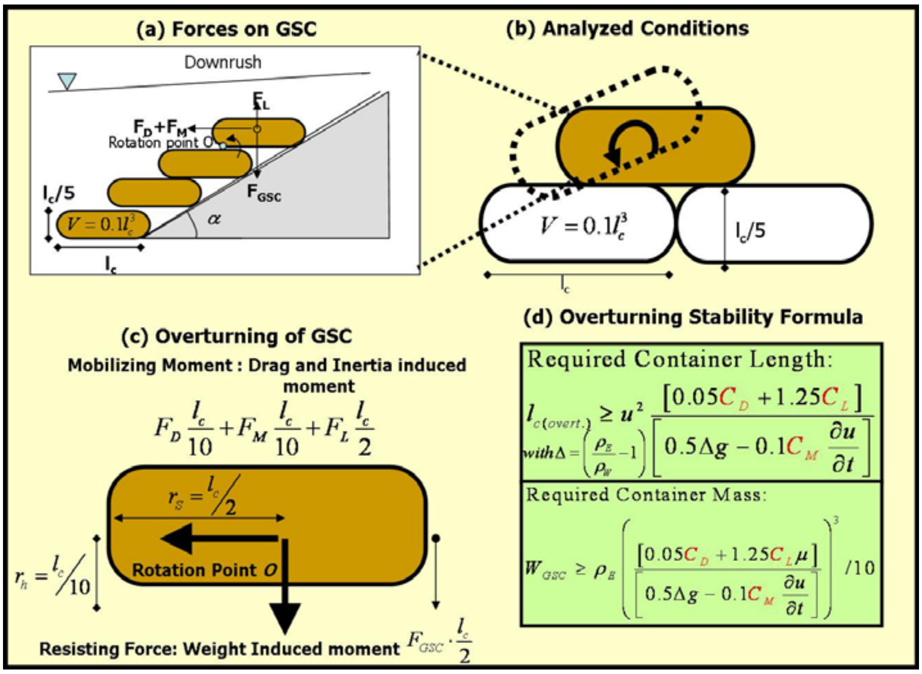


Fig. 33 Overturning Stability Formulae Without Deformation Effect

More details on the force coefficients C_D, C_M and C_L as well as on further input parameters required in the stability formulae summarized in Fig. 32 and Fig. 33 will be given in Section 4.3.2.

4.3.2 *Stability Formulae Including Deformation Effect*

The effect of the container deformations on the stability is explicitly accounted for by introducing analytically derived deformation factors into the formulae for the drag force, lift force, inertia force and resisting forces for both hydraulic failure modes: sliding and overturning. The deformation factors are obtained as correction factors, describing the changes of (Plate 3):

the resisting force F_R against sliding. The correction factor KS_R is obtained as the ratio of the effective weight contribution to the frictional force F_R with and without deformation,

the resisting moment against overturning. The correction factor KO_R is obtained as the ratio of lever arm r_s of the container weight F_{GSC} under buoyancy with and without deformation effect.

Plate 3 Stability Formulae Including the Effect of Deformation

	Sliding Stability		Overturning Stability
Required container length	$l_{c(sl)} \geq u^2 \frac{[0.5KS_{CD}C_D + 2.5KS_{CL}C_L\mu]}{\left[\mu KS_R\Delta g - KS_{CM}C_M \frac{\partial u}{\partial t}\right]}$	Required container length	$l_{c(ov)} \geq u^2 \frac{[0.05KO_{CD}C_D + 1.25KO_{CL}C_L]}{\left[0.5\Delta KO_R g - 0.1KO_{CM}C_M \frac{\partial u}{\partial t}\right]}$
Required container mass	$W_{GSC} \geq \rho_g \left(u^2 \frac{[0.5KS_{CD}C_D + 2.5KS_{CL}C_L\mu]}{\left[\mu\Delta KS_R g - KS_{CM}C_M \frac{\partial u}{\partial t}\right]} \right)^3 /10$	Required container mass	$W_{GSC} \geq \rho_g \left(u^2 \frac{[0.05KO_{CD}C_D + 1.25KO_{CL}C_L]}{\left[0.5\Delta KO_R g - 0.1KO_{CM}C_M \frac{\partial u}{\partial t}\right]} \right)^3 /10$

KS_{CD}, KS_{CL}, KS_{CM} and KS_R = Deformation factors for the drag force, lift force, inertia force and resisting forces associated with sliding stability

KO_{CD}, KO_{CL}, KO_{CM} and KO_R = Deformation factors for the drag force, lift force, inertia force and resisting forces associated with overturning stability

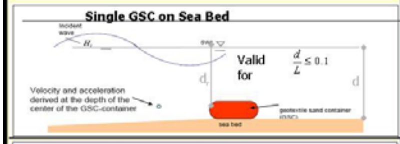
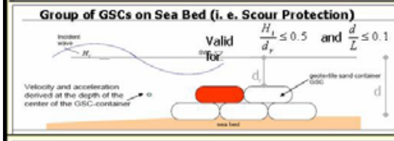
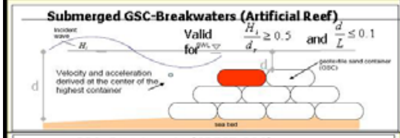
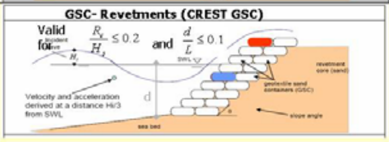
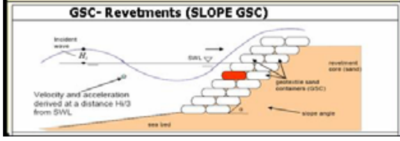
The mobilizing drag, lift and inertia forces contribute to sliding. The correction factors of the drag force F_D and lift force F_L (KS_{CD} and KS_{CL}) are obtained as the ratios of the areas A_S and A_T with and without deformation effect. The correction factor KS_{CM} is assumed to be 1.0 since the container volume V remains constant.

The mobilizing moment induced by the drag, lift and inertia forces. The correction factor for the moments induced by the drag and lift forces (KO_{CD} and KO_{CL}) area obtained as the ratio of describing the changes of both surface areas (A_S and A_T) and lever arms (r_m and r_s) of the drag and lift forces F_D and F_L. The correction factor KO_{CM} is obtained as the ratio of lever arms r_{sn} of the inertia force with and without deformation effect.

The values of the correction factors suggested in Table 3 are derived on the basis of a number of simplifying assumptions (see Recio, 2007 for more details).

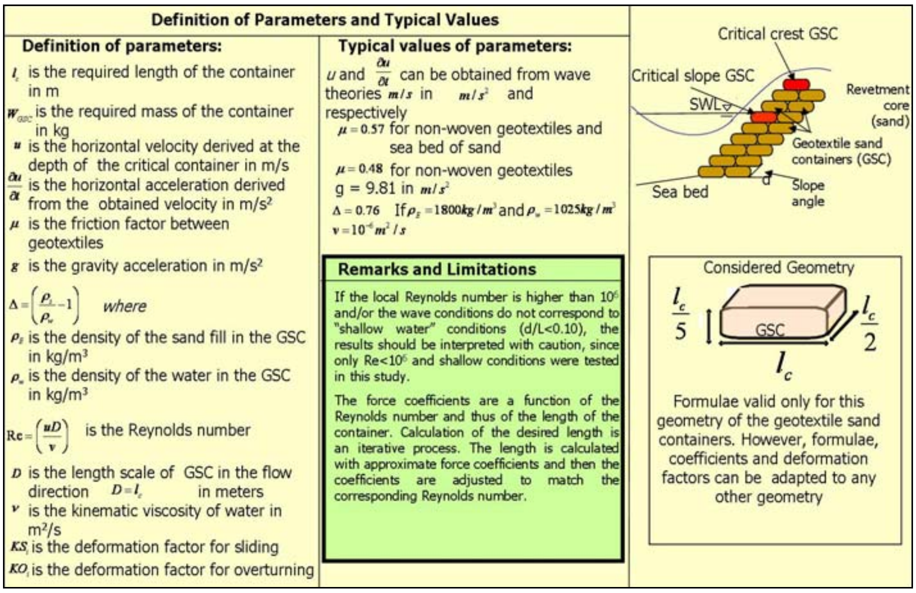
Among the most important assumptions the following are noteworthy: sand fill ratio of 80% and a slope angle of the GSC-structure of 45°. Indications to account for other slope angles and recommendations on further research to overcome most of the simplifying assumptions are given by Recio (2007).
Moreover, the force coefficient C_D, C_L and C_M are also given as a function of the Reynolds number for different locations and boundary conditions which may represent different practical applications (scour protection on the sea bed, artificial reef, slope containers and crest container of a surface piercing structure such as revetments, seawalls, groins, etc.). The proposed values of C_D, C_M and C_L have been determined on the basis of systematic laboratory experiments (Recio and Oumeraci, 2006c).

Table 3 Deformation Factors and Force Coefficients

GSC-Structure	**Deformation factors**		**Force Coefficients**
	Sliding	**Overturning**	
Single GSC on Sea Bed 	$KS_{CD} = 1.40$ $KS_{CM} = 1.00$ $KS_{CL} = 0.94$ $KS_R = 0.70$	$KO_{CD} = 1.54$ $KO_{CM} = 1.1$ $KO_{CL} = 0.80$ $KO_R = 0.92$	$C_D = -2x10^{-5}\,\mathrm{Re} + 6.81$ with $1.3 \le C_D \le 6.5$ $C_M = 0.60$ $C_L = 1x10^{-5}\,\mathrm{Re} - 0.612$ with $0.2 \le C_L \le 1.4$
Group of GSCs on Sea Bed (i. e. Scour Protection) 	$KS_{CD} = 1.40$ $KS_{CM} = 1.00$ $KS_{CL} = 0.94$ $KS_R = 0.70$	$KO_{CD} = 1.54$ $KO_{CM} = 1.1$ $KO_{CL} = 0.80$ $KO_R = 0.92$	$C_D = -6x10^{-5}\,\mathrm{Re} + 14.70$ with $4 \le C_D \le 11$ $C_M = 0.50$ $C_L = 1x10^{-5}\,\mathrm{Re} - 0.669$ with $0.4 \le C_L \le 1.3$
Submerged GSC-Breakwaters (Artificial Reef) 	$KS_{CD} = 1.40$ $KS_{CM} = 1.00$ $KS_{CL} = 0.94$ $KS_R = 0.70$	$KO_{CD} = 1.54$ $KO_{CM} = 1.1$ $KO_{CL} = 0.80$ $KO_R = 0.92$	$C_D = -9x10^{-5}\,\mathrm{Re} + 23.04$ with $4 \le C_D \le 15$ $C_M = 0.30$ $C_L = 1x10^{-5}\,\mathrm{Re} - 0.587$ with $0.3 \le C_L \le 1.2$
GSC- Revetments (CREST GSC) 	$KS_{CD} = 1.40$ $KS_{CM} = 1.00$ $KS_{CL} = 0.94$ $KS_R = 0.70$	$KO_{CD} = 1.54$ $KO_{CM} = 1.1$ $KO_{CL} = 0.80$ $KO_R = 0.92$	$C_D = -2x10^{-5}\,\mathrm{Re} + 6.81$ with $1.3 \le C_D \le 6.5$ $C_M = 0.60$ $C_L = 1x10^{-5}\,\mathrm{Re} - 0.612$ with $0.2 \le C_L \le 1.4$
GSC- Revetments (SLOPE GSC) 	$KS_{CD} = 1.40$ $KS_{CM} = 1.00$ $KS_{CL} = 0.94$ $KS_R = 1.60$	Not applicable (GSC pulled out seaward)	$C_D = -3x10^{-5}\,\mathrm{Re} + 8.9$ with $2.5 \le C_D \le 9$ $C_M = 0.30$ $C_L = 1x10^{-5}\,\mathrm{Re} - 0.587$ with $0.3 \le C_L \le 1.2$

In Plate 4 the parameters used in the stability formulae described in Fig. 32-Fig. 34 are defined and typical values are also given, including some remarks on the limitations of the suggested values.

Plate 4 Defining of Parameters and Typical Values to be Used in the Stability Formulae



4.3.3 *Comparative Analysis of Stability Formulae with and without Including Deformation Effect*

In order to illustrate the effect of deformation the results of the new more process-based stability formulae with and without consideration of the deformations of the container as proposed in Sections 4.3.2 and 4.3.1 are compared in Fig. **34** and Fig. **35** for a sloping revetment with an angle of 45°, a water depth d = 4m at the structure and a peak period T_p = 6s of the waves. Moreover, the simple stability formulae for slope containers (Fig. **34**) and crest containers (Fig. **35**) proposed in Section 4.1 are also considered to illustrate the difference with the results from the more process-based formulae.

Depending on the range of the design wave height H_S, the following remarks may be drawn from the comparison of the formulae for the slope containers Fig. **34**):

(i) <u>For smaller design waves ($H_S \leq 1.5$m):</u> the simple stability formulae proposed in Section 4.2 are too conservative and the deformation effect on the stability obtained from the comparison of the new formulae proposed on Section 4.3 are relatively small.

(ii) <u>For larger design waves ($H_s \geq 2.5$m):</u> the simple stability formulae become more unsafe with increasing wave height. The effect of deformation also increases with the increase of the design wave height.

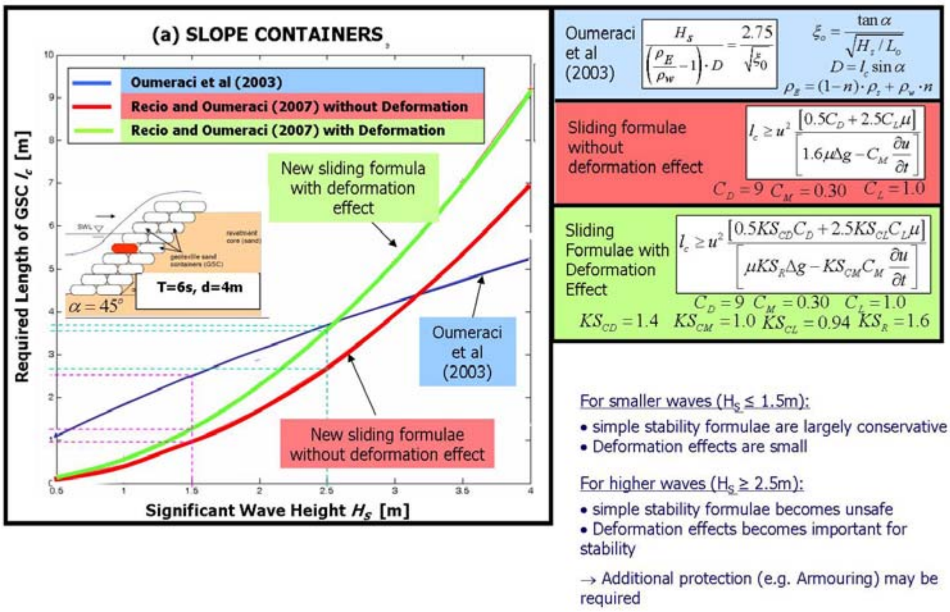


Fig. 34 Stability Formulae for Slope Containers: Comparative Analysis

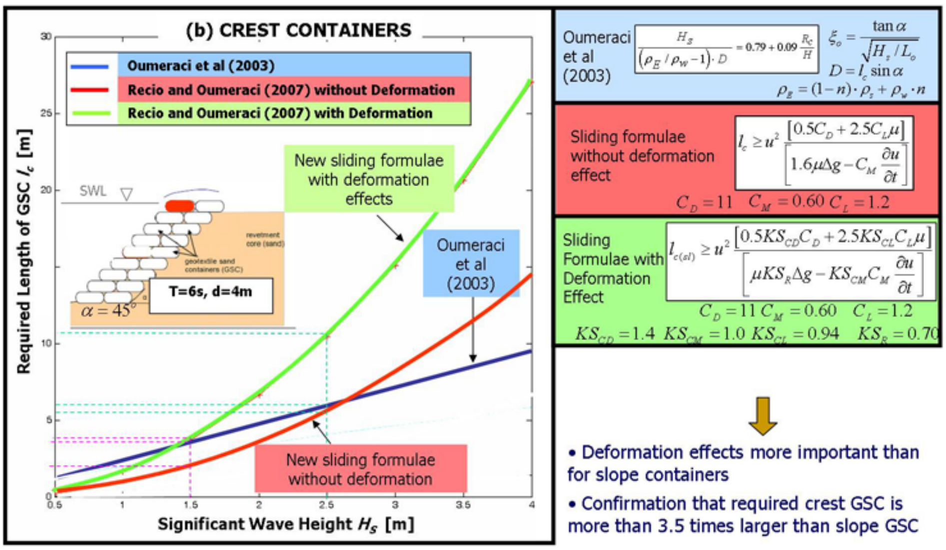


Fig. 35 Stability Formulae for Crest Containers: Comparative Analysis

Depending on the range of the design wave height H_s the following remarks may be drown from the comparison of the stability formulae for the crest containers (Fig. 35):

(i) For smaller design waves ($H_s \leq 2.5$m): the simple stability formula proposed in Section 4.2 is slightly conservative and becomes unsafe as soon as the significant wave height exceed 1.5 m. It can therefore be used instead of the more process-based formula only for $H_S \leq 1.5$m. However, the effect of deformation is higher than in the case of slope containers.

(ii) For larger design waves ($H_s \geq 2.5$m): the simple stability formula by Oumeraci et al (2003) becomes more unsafe with increasing wave height H_s. The effect of deformation on the stability also increases with increasing wave height.

Comparing Fig. 34 and Fig. 35 also confirms that for commonly used relative freeboards Rc/H_s in the order of 1.2, much larger containers are required for the crest than for the slope of the structure. Moreover, it also shows that the effect of deformation on the stability is much more pronounced for the crest containers than for the slope containers.

5. Concluding Remarks

After about 50 years of successful experience of geotextile applications in coastal engineering, applications for shore protection are well established. Most failures which have yet been experienced are rather due to bad design, bad choice of material and/or bad installation.

Geotextile sand containers (GSCs) represent nowadays a soft and low cost alternative to conventional hard structures made of rock and concrete. Moreover, GSC-made structures are environmentally more appropriate and more easily reversible as they need essentially sand as construction material which is generally available at any coastal site. As "soft rock" GSCs can be manufactured at any size and used to build any type of shore protection structure, including scour protection, dune reinforcement and repair of undermined structures.

However, several problems still need to be solved in order to make use of the full potential of GSCs, particularly including (i) the long-term durability and lifetime prediction and (ii) the hydraulic stability under severe wave action.

The facts, limitations and research needs related to the durability and lifetime issue may be summarized as follows:

(iii) Although field evidence (mostly non-exposed geotextile) is available over about 50 years, useful information extracted from non-retrieved samples is often very incomplete or very limited as the results can hardly be transferred to other sites, to present geotextile products and to time durations and conditions beyond the experienced service time/conditions.

(iv) Results of accelerated tests –even in combination with those from site monitoring and retrieved samples– are still very limited when trying to predict lifetime of more than about 25 years. In fact, a systematic methodology to combine both laboratory and field monitoring approaches for this purpose is still missing.

(v) Recommendations for future research in the mid-term and long-term should focus on two directions: (a) improvement of the understanding of the degradation mechanisms, including physical, biological and chemical processes, both isolated and in combination, and (b) development of a consistent framework for the assessment of long-term durability and lifetime (up to 100 years and more) based on the results of the above and including site monitoring, laboratory testing and theoretical/numerical modeling.

(vi) Meanwhile, in order to contribute to solve the present problems in practice, the following two recommendations might be helpful. (a) apply engineering judgment based on the present knowledge of degradation mechanisms rather than relying on "extrapolation approaches" to predict lifetime and (b) apply where feasible and necessary well established measures to enhance long-term performance, including for instance appropriate stabilizers and additives (e.g. against UV-radiation), more robust geotextiles (e.g. multi-layer) and geotextile coating (e.g. against abrasion and vandalism), sand covering (e.g. against UV-radiation and vandalism and to enhance aesthetical aspect), rock covering (e.g. against ice loads, debris, very high waves, UV and vandalism) and setup of a consistent maintenance plan.

Regarding the hydraulic stability under severe wave action, the present state of knowledge, the limitations and the needs for future research may be summarized as follows:

(i) The large experience and stability formulae available for rock and concrete armour units cannot simply be transferred to geotextile sand containers, essentially due to the deformation of the GSCs under very severe wave attack.

(ii) The deformations of the GSCs are essentially induced by the internal movement of the sand fill of the containers.

(iii) The effect of the deformations of the geotextile sand containers on the hydraulic stability rapidly increases with the severity of wave attack, depending on the size and sand fill ratio of the containers as well as on the degree of exceedance of the wave loads required for the inception of the internal movement of the sand fill.

(iv) Beside the effect on the hydraulic stability, the internal sand movement in submerged geotextile containers may lead to a substantial reduction of the height of GSC-structures (up to about 10%) when subject to severe wave attack.

(v) The deformation of GSCs affects the hydraulic stability, essentially due to two mechanisms: (a) reduction of the contact areas between GSCs caused by the uplift of the containers by wave action, thus decreasing the stabilizing forces/moments and (b) increase of the surface areas exposed to drag and lift forces which represent the main destabilizing forces/moments. Therefore, the deformation effect should be accounted for explicitly in the stability formulae.

(vi) Friction between geotextile sand containers affects the hydraulic stability much more than commonly assumed in past and present design practice. Therefore, and due to the unpredictable possible changes in service life, friction should be incorporated explicitly in stability formulae.

(vii) The hydraulic permeability of structures made of geotextile sand containers is not only important for the prediction of the hydraulic performance (e.g. wave transmission, run-up, overtopping, etc.), but also slightly affects the hydraulic stability. However, no clear correlation could be found between stability and permeability for the range of practical permeability coefficients of GSC-structures in the order of $k = 1\text{-}3$cm/s. The permeability of a GSC-structure is essentially determined by the gaps between the containers, so that the flow through the sand fill itself can be neglected. Therefore, the hydraulic permeability essentially depends on the mode of placement of the containers. For randomly placed containers and longitudinally (in wave direction) placed containers, the permeability coefficient is in the order of $k = 2.4$ cm/s and $k = 2.3$ cm/s , respectively.

(viii) The effect of breaking wave impact on sliding and overturning stability of slope containers was been found much less than expected, due to the potential of the GSCs to effectively damp impact pressure propagation inside the gaps. More efficient to destabilize the slope containers are the uprush and downrush of the longer non-breaking waves and partially breaking waves.

(ix) The proposed simple stability formulae derived in Section 4.1 on the basis of the HUDSON-formula take additionally into account the effect of the wave period for the slope containers and that of the relative freeboard (Rc/H_s) for the crest containers. In both cases, the effect of container deformations is taken into account implicitly through the empirical parameters determined from laboratory experiments. These formulae are conservative for waves up to about $H_s = 1.5$m and can thus be used for design wave heights not larger than about 2 m. For higher waves ($H_s > 2$m) the effect of deformation on the stability becomes more important and must therefore be considered more explicitly in order to ensure long-term stability performance.

(x) The more detailed stability formulae proposed in Section 4.3 do not only take into account the most relevant processes, but also allow us to better quantify the effect of deformation on the required size of the container as a function of the incident wave height for both slope containers (Fig. 34) and crest containers (Fig. 35). In fact, these new process-based formulae clearly highlight the effect of container deformation which is much more pronounced for the stability of the crest containers than for that of the slope containers. Moreover, they also show that even for the commonly used relative freeboard of about $Rc/Hs \approx 1.2$ much larger containers are required for the crest than for the slope of a GSC-structure to ensure hydraulic stability.

(xi) Although significant advance has been achieved in the understanding of the processes and mechanisms responsible for the hydraulic failure of geotextile sand containers, more systemic research is further needed to investigate and better control the friction between GSCs, the effect of the sand fill ratio, the effect of the slope steepness of the GSC-structure, the internal movement of the sand fill, the container deformations and their more explicit consideration in both stability formulae and numerical simulations. A fully coupled CFD and CSD model system well-validated by experimental data will be needed as a necessary tool in combination with new laboratory experiments to achieve these goals.

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