Guidelines for Design and Construction of Artificial Islands

May 1998

D.J. Zwemmer

Guidelines for Design and Construction of Artificial Islands

D.J. Zwemmer

Final examination committee:
prof. ir. K. d'Angremond
dr. ir. N. Booij
dr. ir. J. van de Graaff
ir. G.J. Schiereck
ir. C. Stigter

May 25, 1998

Preface

This thesis forms the last report for, and completes this final examination project, entitled 'Guidelines for Design and Construction of Artificial Islands'. The feasibility study involved, was performed at Delft University of Technology, Department of Coastal Engineering. This report was accomplished over a period of nine months. During this period, several preliminary reports have been written. The most important and interesting sections of the previous reports have been transformed to this final thesis.
During the study, the engineering aspects of artificial islands have proven to be a very interesting and innovative area of civil engineering. Which certainly justify future research!

If one is only interested in the results of the study, it is sufficient to read the conclusions and recommendation of each chapter plus the summary. The assumptions and limitations of the results can be found in the preceding paragraphs of each chapter. Details of the study are presented in the appendixes.

Acknowledgements:
The following persons have contributed largely to the realization of this thesis.

- prof. ir. K. d'Angremond, Department of Coastal Engineering
- dr. ir. N. Booij, Department of Hydraulic Engineering
- dr. ir. J. van de Graaff, Department of Coastal Morphology
- ir. J. van 't Hoff, engineering company Van 't Hoff bv
- ir. G.J. Schiereck, Department of Coastal Engineering
- ir. C. Stigter, Boskalis Westminster NV

Dirk-Jan Zwemmer

Delft, May 1998

Extensive summary

This report is an extensive summary of a report, entitled 'Guidelines for Design and Construction of Artificial Islands'. Both reports were realized within the framework of a final examination project at Delft University of Technology, Department of Hydraulic Engineering.

1. Introduction

The main objective of this examination project is to present guidelines, which may accelerate the initial phase of artificial island design. The guidelines are restricted to the field of hydraulic engineering as only artificial islands in open sea are considered.

Initially an extensive literature research was performed, not only with regard to the purposes to construct artificial islands but also to differentiate between possible types of artificial islands. From this literature research some general guidelines could be drawn up. The literature research also revealed the most frequently occurring problems, related to artificial island construction. These problems gave cause for more guidelines, e.g. guidelines in relation to sea defences, guidelines in relation to landfill and guidelines in relation to morphological impact.

1.1 Purposes for artificial islands

From history it can be concluded that artificial islands have been built for centuries. As an example, the batteries in Tokyo Bay, constructed on man-made islands during the 1850's to cope with intruders, can be mentioned. From the second half of this century, however, the construction of artificial islands has taken on large proportions. Ever since, artificial islands have been employed in deeper and more exposed waters. Lack of extension possibilities on the mainland, location related activities and very lucrative exploitation prospects have stimulated the development of artificial islands. Artificial islands can be subdivided into single purpose islands and multi purpose islands.

Single purpose islands

These islands are built to serve only one purpose, for example to facilitate oil-exploration. Multi purpose islands, on the other hand, accommodate a variety of users. For instance an island erected to serve as an urban expansion region not only includes houses, but probably recreational facilities and some sort of energy supply as well. In general, multi purpose islands are bigger than single purpose islands and provide therefore cheaper construction costs per m^3 than smaller single purpose islands.

Table 1.1 distinguishes between eight possible users of single purpose islands. All islands mentioned in Table 1.1 have actually been built, with the exception of those for *Mariculture*, because this type has so far only existed on the drawing boards. Its users are ordered by expected island size, going from small to large.

Single purpose	**Used for:**
Ocean mining & exploration	Gas
	Oil
	Research
Recreation	
Waste handling	Waste processing
	Waste storage area
Mariculture	Farming of fish
	Oyster farm
Temporary or aiding structure	Anchoring for bridge cables
	Closure of tidal areas
	Work island
Energy generation	Atmosphere gradient
	Coal Fired plant
Port	Deepwater port
	Fishery port
	Transhipment port
Airport	

Table 1: Users of single purpose islands

Ocean mining & exploration

Natural resources are mostly mined using artificial islands. In general these islands are made out of steel or concrete (for example oil rigs). At very exposed or shallow locations reclaimed islands may prove to be more useful and more economic.

Recreation

At very densely populated areas, with only a limited amount of arable land, recreational facilities on artificial islands may be a relief. Japan has built this kind of islands.

Waste handling

Processing of waste may be more favourable on a remote island than on the mainland, for reasons of stench-annoyance, noise pollution or lack of space. Processing and storage facilities are installed on such islands.

Mariculture

Special islands may be created for mariculture, such as cultivating weeds, mussels, oysters or farming of fish for commercial use.

Temporary or aiding structure

A great variety of islands are part of this category, ranging from a temporary concrete factory to protection for bridge-piers. These islands may be small as well as big. Other examples are the working-islands for the Delta Project in the Netherlands.

Energy generation
Facilities to produce energy as well as facilities to store fuels, which are placed on an artificial island belong to this category.

Port
For deepwater ports, extension problems or extreme dredging-costs may be reason to resort to an artificial island. On the other hand, small fishery ports have been built on man-made islands too.

Airport
If properly designed, sound-contours from the island will not cross inhabited areas anymore or at least to a lesser extent. Thus environmental requirements can be fulfilled more easily. Furthermore, unobstructed approach and departure routes can be obtained. Some disadvantages of an offshore airport should be noted as well. Firstly the problems that arise in transporting passengers and goods from and to the island. Secondly, some reports proclaim that aircraft operations are affected by possible fog and thermal air currents. Moreover, environmental groups have stirred up the discussion about migratory birds resting on the island, and thereby endangering flight-traffic.

Multi purpose islands
Table 1.2 distinguishes between three possible users of multi purpose islands. All islands mentioned in Table 1.2 have actually been built. Its users are ordered by expected island size, going from small to big.

Multi purpose	**Used for:**
Waste island	Waste storage combined with future activities, like ... - Recreation - Residential areas
Industrial island	Deepwater port Industrial zones Energy generation
Complex city	Business areas Hospitals Leisure & Recreation Residential areas

Table 2: Users of multi purpose islands

Waste island
Waste material, domestic as well as industrial waste, is used as landfill for a man-made island. In order to be able to use the island for other purposes, such as recreation in future times, the use of contaminated waste should be avoided. Environmental measures are required to prevent leakage towards the surroundings.

Industrial island
If there is heavy industry on the island, it will be realistic to assume that related facilities like a port and a power plant are needed. Besides, the industrial island has to be self-supporting to some extent. It requires its own fire brigade, medical facilities, fresh water supply, waste

treatment and electricity supply. Workmen have to be housed on the island for longer periods or a properly functioning transport system from and to the mainland has to be developed.

Complex city
Most items mentioned for an industrial island also apply to complex cities.

1.2 Types of artificial islands

Half a dozen structural types of artificial islands can be distinguished. These types are shown in Figure 1.1.

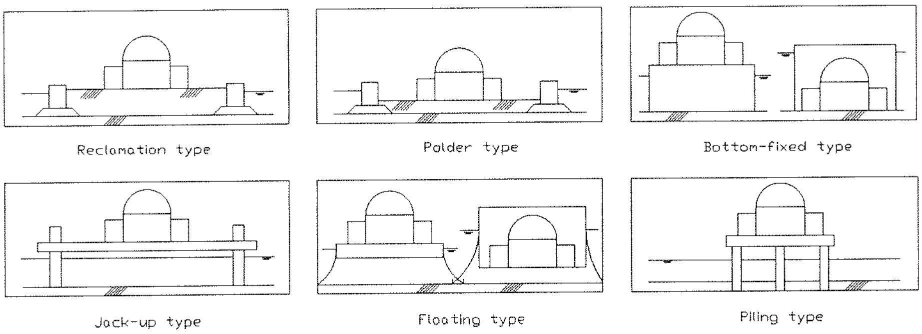


Figure 1: Structural types of artificial islands (not to scale)

Reclamation type
For this construction method an area in the sea is surrounded by a sea defence. The sea defence can consist for instance of caissons, rubble mound and concrete armour elements or double steel sheet piles. Then soil, waste, other materials or a combination of these are used as landfill.

Polder type
The polder-alternative is a variation of the reclaimed island. Designed as a polder, the ground level is situated lower than the surrounding average sea level. The greatest advantage of the polder alternative is the reduction of landfill. A disadvantage, however, is the increased rate of damage in case of failure of a dike-section. Permanent pumping is required for seepage and rainfall.

Bottom-fixed (Gravity) type
This type is the heaviest of the six construction types. It derives its stability against overturning from its weight combined with its large base. First a steel or concrete structure is created at a protected location, then this structure is towed to an ocean-site, where it is filled with sand or sea water and sunk to the bottom. It is necessary to prepare the site before the structure is sunk, because a flat sea bed is required.

Jack-up type
This structure consists of a floating pontoon which raises itself above the sea surface by jacking itself along legs which are lowered to the seabottom after the structure has been floated

into position. Since jack-up platforms can easily be moved from one place to another, they are very suitable for temporary projects. They are restricted to water locations, up to 90 - 150 metres. Many oil rigs in the North Sea use the jack-up principle.

Floating type
This type consists of a barge like structure, and the facilities are placed inside or on its upper sections. Because this is a free floating structure, anchoring is necessary to keep the structure located in a fixed position. Automatic positioning systems (Voidth-Schneider) can also be used for this purpose. Floating type structures can be located in water of any depth and are hardly affected by earthquakes but there is a drawback. They are being affected by the pitching and rolling caused by waves (including waves caused by seaquakes). To temper the effect of pitching and rolling, part of the floating-body can be submerged (called semi-submerged islands). Even floating islands have been developed for airports.

Piling type
This type of island is constructed by driving piles into the sea bottom, on top of which a platform is placed. Structures are erected on the platform.

1.3 Restriction to detail reclaimed islands only

In the remaining part of this report only reclaimed artificial islands will be discussed, for the following reasons:

- Reclaimed islands are cheap alternatives to other types of man-made islands, provided they are built in restricted waterdepths.
- Fill-material is relatively cheap, especially if the area of the island becomes larger.
- Larger areas can be realized with minimal additional effort.
- Reclaimed islands have been the most employed man-made islands so far. This implies they have proved their worth and that there is enough knowledge and experience available.

2 Literature research

Examining the literature about reclaimed islands, a collection of some 200 islands, both built and planned, was drawn up. Of course this collection is by far not complete. Nevertheless, some information can be recovered. About 130 of the total of 200 reclaimed islands were actually built. Large concentrations of man-made islands can be found in the Canadian Beaufort Sea and Japan. In the Beaufort Sea 35 reclaimed islands have been employed for the mining of oil and gas. The extremely high construction costs of these islands were justified by expected high profits. Japan also uses artificial islands as only about 30% of its surface area is available for extension.

2.1 General guidelines as derived from literature

- **In practice, waterdepths are restricted to $\pm$ 30 m, mostly on the basis of economics.**
  The main reason is, that the amount of material required for reclaimed islands will grow rapidly for increasing waterdepths. None of the built islands

from the literature collection exceeds the 30 m depth limit. If the profits of employing an island in deeper water are high enough, the waterdepth restriction may of course be abandoned.

- **In practice, an increase in island size involves a decrease in waterdepth.**
  The observed relation is a logical result from the desire to construct as economically as possible. Shallow water construction requires less landfill and material for the sea defences, consequently a cheaper design is the result. The economic benefits increase even more for larger areas.

- **A minimal amount of polder islands have been built.**
  Although the polder islands show great resemblance to the reclaimed islands, investors seem reserved to put money into these projects. Most countries in the world object to the idea of placing expensive structures and even human lives on surfaces below sea level. Assumed risk of flooding and unfamiliarity are probably the main reasons.

- **Most problems with the design and construction of artificial islands are related to the following items: earthquakes, overall costs reduction, morphological impact and settlements.**

3 Design and construction of sea defences

Generally, the largest part of a reclaimed island consists of landfill, for example sand. The total of measures needed to prevent the landfill from being eroded by waves and currents is called the sea defence. In general, sea defences account for the majority of the total island's costs. This report only discusses so-called hard sea defences, which are structures which do not move under design conditions. Hard sea defences allow islands to be employed in deeper and more exposed waters.

3.1 Design and construction of sea defences

Six types of sea defences are discussed in this report:

- Rubble mound sea defence, armoured with
  - Quarry stones
  - Dolosse
  - Tetrapods
  - Modified cubes
- Vertical composite caisson type seawall
- Horizontal composite caisson type seawall

Rubble mound sea defences

A rubble mound sea defence consists of several layers, most of which are constructed out of quarry (rubble) stones. The outer layer, called the primary armour layer, is designed to withstand the expected wave attack, and is supported by a secondary armour layer. A typical cross-section of a rubble mound sea defence is shown in Figure C.2. The dimensions used for construction are drawn as well. All four selected armour elements will be briefly discussed.

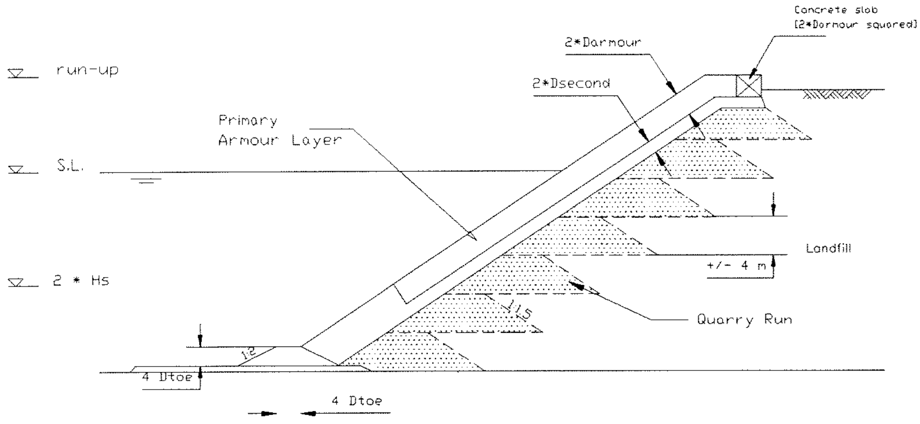


Figure 2: Cross-section of typical rubble mound sea defence

- Quarry stones
  The stones are obtained by blasting a rock-quarry, and the resulting sharp angular elements have a natural interlocking capability.

- Dolosse
  Dolosse are anchor shaped concrete armour units, without any reinforcement designed to interlock with each other. Through their high interlocking capability higher waves can be withstood compared to other armour elements, having the same weight. Dolosse should be placed in a selected order and in a double layer to assure proper working. To avoid breakage of elements, dolosse should not be applied above a mass of 20 tons.

- Tetrapods
  Tetrapods are plain concrete armour units consisting of four arms projecting from a central hub. The angular spacing between all arms of a tetrapod is the same. Tetrapods should be placed in a selected order and in a double layer as well. Furthermore, it is advised not to use tetrapods having a mass exceeding 20 tons.

- Modified cubes
  This element is evolved from the cube. In this report the modified cube, as mentioned in *Shore Protection Manual* is used. At its top side, this cube has caps on every corner, and at its bottom side a square notch is applied. Modified cubes are relatively easy to make, with only a limited need of shuttering. Consequently, modified cubes can be considered as a standard in concrete armour elements.

Concrete armour units, which are claimed to function in a single layer (e.g. accropods and core-loc) are not discussed. The main reason is the lack of information and experience with these kind of elements. Accropods have been applied only recently, and extensive information about failure mechanisms is not available.

Vertical composite caisson type seawall
Caissons are placed on top of a rubble mound berm. Attention should be paid to the height

of the rubble mound berm. The mound may cause waves to break against the caisson, generating high impact forces on the caisson. To avoid these impact forces, the mound should, according to literature, not be constructed higher than about 0.3 - 0.4 times the waterdepth above the sea bottom.

Horizontal composite caisson type seawall

If breaking waves are expected, an armoured mound can be placed in front of the caissons. The mound must absorb and break part of the wave energy effectively. Because of this protection the impact forces on the caisson are greatly reduced.

All these six defence structures make use of quarry stones. In other words, altered costs for quarry stones affects the total costs of all six examined sea defences.

3.2 Design assumptions and limitations

Natural assumptions and limitations

The local significant wave height H_s, defined at the toe of the sea defence, is used to determine the most economic type of sea defence. However, the design of sea defences depends on the wave period as well. The ratio between wave height H and the wave period T is known as the wave steepness s. The other symbols in the equation account for deepwater wave length L_0 and the acceleration of gravity g.

$$s = \frac{H}{L_0} = \frac{2\pi H}{gT^2}$$

Fortunately, the range of steepnesses is limited in nature. Steepnesses larger than 0.05 or less than 0.01 are very unlikely in nature. Both steepnesses have their own wind induced origin.

Wind induced waves can generally be divided into two groups:

- Seas: These waves are generated by a local storm, and waves reach the beach almost in the same shape in which they were generated. These waves are steep, with a typical steepness in the range of 0.05.
- Swell: These waves are generated by a distant storm, they may travel through hundreds or even thousands of miles of calm areas before reaching the shore. Wave lengths are long, and typical steepnesses are in the range of 0.01.

Of course, seas and swell may occur at the same time and location. And an intermediate wave may appear as well. However, this report is limited to these two basic conditions.

The range of wave heights used in this report, is based on these wave steepnesses as well as on the wave period limits, as found in nature. Wave periods over 15 to 20 seconds for significant wave heights require very large fetch-lengths and high wind speeds. If one assumes that 20 seconds is a maximum for significant wave periods, the maximum deepwater wave heigh is restricted, as a result of the steepness equation. A maximum wave height of 6.0 m is assumed for waves having a wave-steepness of 0.01. Steep waves, on the other hand, can reach large wave heights. A maximum deepwater wave height of 12.0 m is assumed for seas in the computations.

Furthermore, the maximum wave height is limited by the waterdepth. A general rule of thumb, widely used in coastal engineering, gives:

$$H_s = \begin{cases} H_{s0} & \text{when } H_{s0} \leq 0.5 * \text{h (h = waterdepth)} \\ 0.5 * h & \text{when } H_{s0} > 0.5 * \text{h} \end{cases}$$

Waterdepth and Sea Level: The local depths assumed in the sea defence-calculations are measured at the front of the seaward toe, rubble mound or horizontally protection layer in case of horizontally composite seawalls. Furthermore, the depth already accounts for predicted sea level rise by tides, storm conditions, wave set-up, etc. The corresponding level is called Sea Level (S.L.), see also Figure C.2.

$$\text{S.L.} = \text{M.S.L.} + \Delta\text{h (tides, storm, wind, etc.)}$$

Design assumptions and limitations

- Minimal overtopping allowed:
  Because costs per square meter are high for an artificial island, it should be possible to place structures directly behind the sea defences. However, some overtopping or spray may occur, and a drainage system is therefore required. In this report an exceedance level of 2% for the wave run-up on rubble mound sea defences is allowed. This means that 2% of the approaching design waves reach a higher wave run-up level than the calculated maximum run-up level. For caisson-type seawalls the crest height should be constructed in such a way that minimal overtopping occurs. According to literature a crest height of 1.5 times the significant wave height is advisable.
  A system could be installed to discharge the overtopped water, the spray and the rainfall. In case roads are required on the island, these roads might be built as a boulevard and at the same time be used to face the problem of overtopping water.

- Armour weight formulae:
  For rubble mound sea defences two types of formulae are used in this report. A formula by Van der Meer (1988) is used in case of quarry stone armour, the Hudson-formulae are used for concrete armour units. A more uniform outcome would result if the same formulae would be used in all cases. However, Van der Meer's formula for quarry stone armour gives more realistic results than Hudson-formulae for the same armour.
  For the caisson-type seawalls Goda's formulae are applied.

- Density of armour elements:
  The density of the quarry stones is supposed to be 2650 kg/m^3. The concrete elements are given a density in the same range, namely 2620 kg/m^3 (a standard value in *Shore Protection Manual*). This kind of density for concrete elements (without reinforcements) can be achieved by mixing the concrete with heavier aggregates, like iron ore.

- Costs of concrete armour units:
  A standard price for modified cubes is assumed of US $ 70 / ton. This is the all in price, for fabrication and placement. The overall costs for dolosse and tetrapods, however, amount to US $ 80 / ton. This is caused by the fact that the costs for fabrication and shuttering are higher in comparison to modified cubes.

- Costs of quarry stones:
  The all in costs are mentioned in Table 3.1. For quarry stones a parameter a indicates the additional costs, for example for transport, if the quarry is situated some far distance from the island site. The situation in which $a = 0$, indicates a delta-region with a suitable quarry at a distance of several hundreds of kilometres away.
  Note: This distance implies, that not the lowest possible costs for quarry stones are assumed. The value a is varied in the design, to indicate the sensitivity of the guidelines in relation to the value of a.

Gradation	D_{n50}[m]	W_{n50}[kg]	[US\$/m^3]
0.0 - 500 kg	0.32	83	(a + 36)
250 - 500 kg	0.51	361	(a + 40)
500 - 1000 kg	0.65	721	(a + 42)
1000 - 2000 kg	0.82	1443	(a + 45)
1000 - 3000 kg	0.88	1821	(a + 50)
2000 - 4000 kg	1.03	2895	(a + 55)
3000 - 6000 kg	1.18	4328	(a + 63)
6000 - 9000 kg	1.41	7399	(a + 76)
9000 - 12000 kg	1.58	10428	(a + 86)
12000 - 16000 kg	1.74	13904	(a + 100)

Table 3: Gradations and prices for quarry stones

3.3 Guidelines in relation to sea defences

For each of the afore-mentioned types of sea defences: quarry stones, dolosse, tetrapods and modified cubes defences, vertical and horizontal composite caisson type seawalls, a multitude of designs was made. The significant wave height H_s, the waterdepth h and the wave steepness were varied and of each design, a cost-estimate was made based on the earlier given assumptions. The results of all these cost-estimates are shown in several figures on the next pages.

The economically preferred solution for different waterdepths h (including predicted sea level rise) and different significant wave heights H_s is given in Figure 3.12 and Figure 3.13. In these graphs, ranges are indicated for the use of a certain type of sea defence. The wave steepness influences the total lay-out of a cross-section, and consequently two separate graphs are shown to present the guidelines, viz. one for wave steepnesses of 0.05 and one for 0.01.

- **Costs for rubble mound sea defences develop somewhat different from, and are less than the costs for rubble mound breakwaters. Reasons are the use of landfill instead of quarry run for the interior of the sea defences, moreover the protection of the backslope is superfluous for rubble mound sea defences. Caissons are designed in the same way when used as breakwater or when used as protection for a man-made island.**

- **Tetrapods, modified cubes or horizontal caisson-type seawalls are in none of these examined situations the most economic type of sea defence. It is mentioned, that the costs of a vertical composite caisson-type seawall can be used as a lower-limit for the costs of a horizontal composite caisson-type seawall, under the same conditions.** To assure proper working of dolosse, the following points must be taken into account:

- Dolosse require placement in a double layer as well as in a selected order. Random placement should be avoided in all cases.
- During the late 1970's and early 1980's a number of dolosse breakwaters failed. Over 80% of the dolosse breakages were found to originate from the fluke-shank intersection. By incorporating a large fillet extending to mid-shank, the static and dynamic tensile stresses at the fluke-shank intersection can be reduced by more than 60%.
- Special attention is required for the foundation of the dolosse. Uneven settlements of the sea defences should be avoided, because the chance of breakages is increased by uneven settlements.
- Apply dolosse only up to a mass of about 20 tons. At heavy and slender concrete units, shrinkage-cracks may occur caused by uneven cooling and hardening of the concrete. Consequently, heavy and slender concrete armour units can break at minimal cross-sections under severe wave attack.

Similar measures are required for tetrapods, namely positioning in a double layer, not using elements over 20 tons, and a preparation of the sub-soil.

- **In case there are certain doubts about the proper functioning of dolosse or tetrapods, a restriction can be made to compare only quarry stones, modified cubes and caisson type structures. In doing so the economically preferred solutions are given in Figure 3.14 and Figure 3.15.**

Only significant wave heights up to 12.0 m were examined. Therefore a dotted line is drawn at 12.0 m. Consequently, the development of the graphs over 12.0 m is theoretical. The wide area in which dolosse sea defences appear to be economic, is explained by the high stability coefficients of dolosse compared to the other armour types. Furthermore, the costs for quarry stones largely influence the outcome of the research. Therefore the value a is varied to make the guidelines applicable to altered quarry stone costs. This results in the overlapped areas in Figure 3.12, 3.14 and 3.15.

The additional quarry costs parameter a, is varied for values ranging from -15 to 0 [US $\$/m^3$]. For a value $a = 0$ the transition between quarry stones and dolosse sea defences is positioned at a depth of approximately 3 m. This applies for wave steepnesses of both 0.01 and 0.05. If a value of *-15* US $\$/m^3$ is used for the parameter a, however, quarry stone sea defences will be economical up to a depth of about 7 m, for a wave steepness of 0.05. In a similar way, the transition between dolosse and vertically caissons shift outside the field of examination. The same value of a for a wave steepness of 0.01 does not affect the transition point between quarry and dolosse. The wave steepness causes this phenomenon. According to Van der Meer's formula quarry stone armour can withstand higher waves at greater steepnesses, using the same mass.

Because dolosse and tetrapods are excluded from the graphs in Figure 3.14 and 3.15 the guidelines are totally different. The economic area of quarry stone sea defences is suddenly much wider. For a wave steepness of 0.05, a quarry stone seawall is always the most economic type for wave heights less than approximately 2 m and at depths less than about 13 m. If the costs of quarry stones are reduced, to a = -15 US $\$/m^3$, quarry stone sea defences will even be economical to a depth of about 19 m. The transition between vertical caisson-type seawalls and modified cubes sea defences is positioned at wave heights of 8 or 9 m in Figure 3.14, depending on the value of a. For wave steepnesses of 0.01 quarry stone sea defences are preferable at waterdepths less than approximately 3 m. At waterdepths over 3 m, a modified cubes sea defence is most economical at these steepnesses. If the costs for quarry stones are reduced, to a = -15 US $\$/m^3$, quarry stone sea defences will be economical both at wave heights less than about 2 m and at waterdepths less than approximately 5 m.

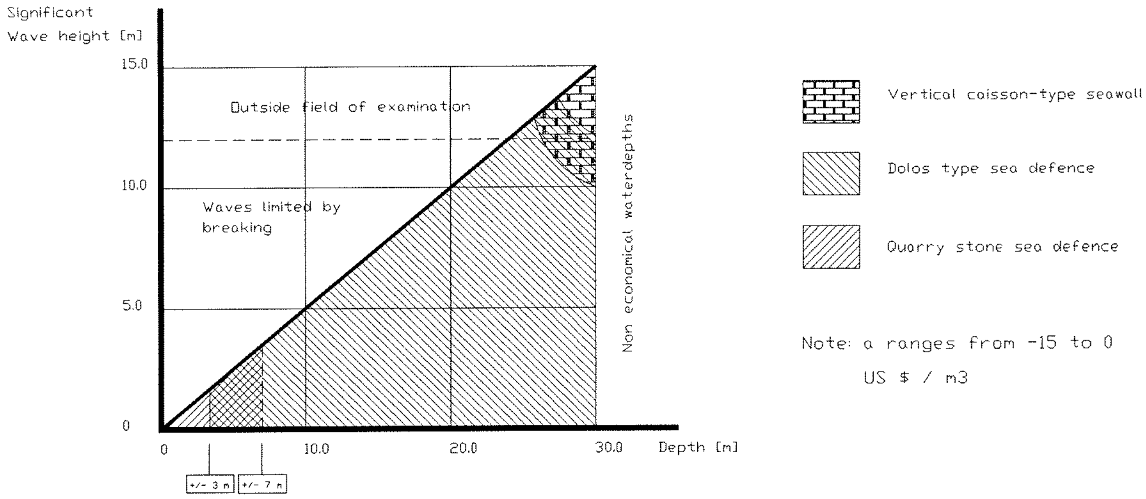


Figure 3: Ranges indicating preferred type of sea defence. Wave steepness s = 0.05. Note: tetrapods, modified cubes and horizontal caisson type seawalls are not preferred.

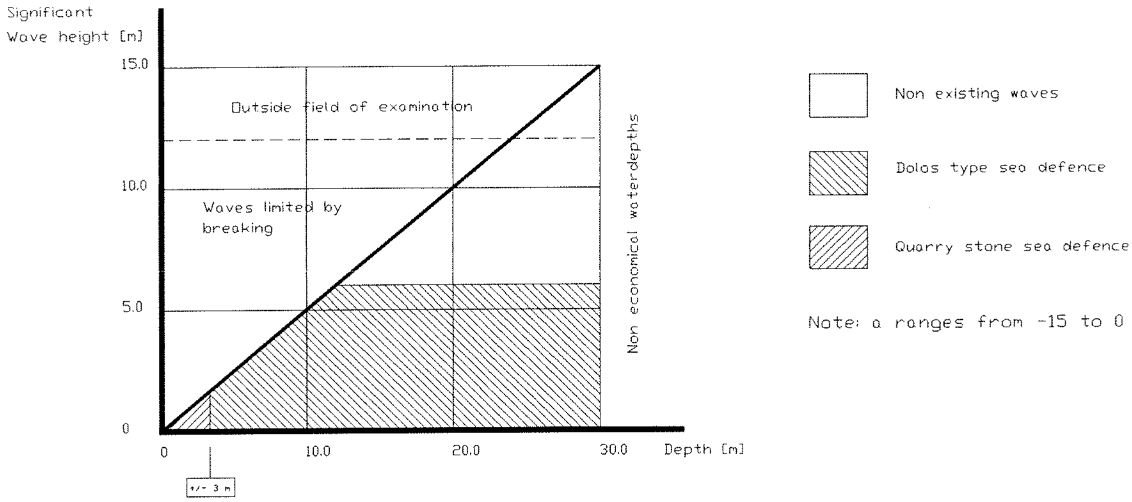


Figure 4: Ranges indicating preferred type of sea defence. Wave steepness $s = 0.01$. Note: tetrapods, modified cubes and horizontal caisson type seawalls are not preferred.

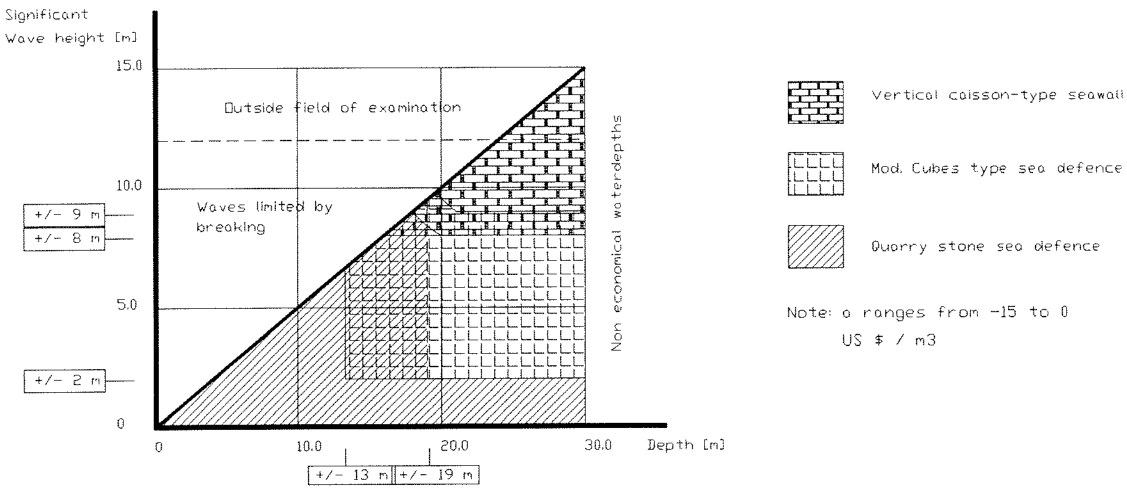


Figure 5: Ranges indicating preferred type of sea defence. Wave steepness s = 0.05. Horizontal caisson-type seawalls are not preferred. Tetrapods and dolosse are excluded.

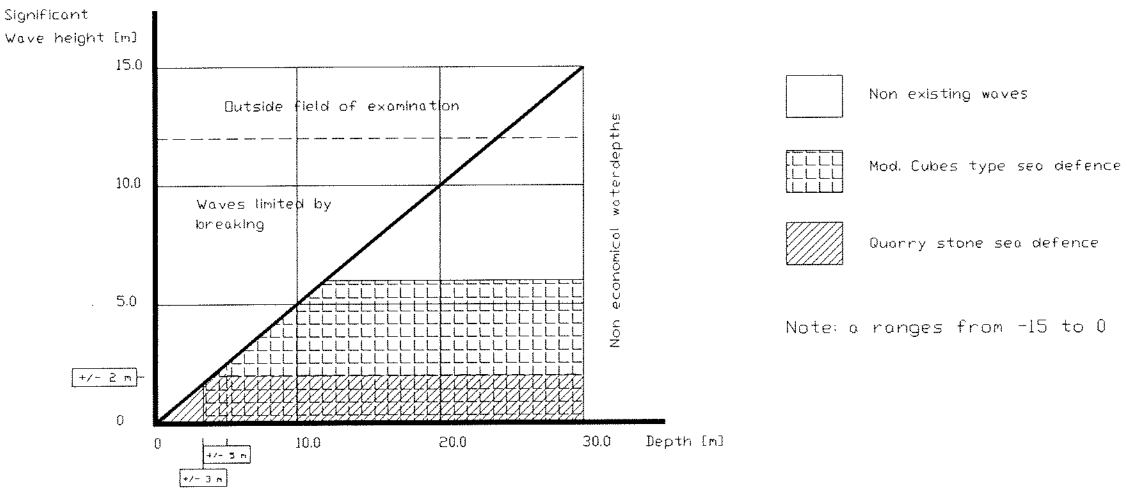


Figure 6: Ranges indicating preferred type of sea defence. Wave steepness s = 0.05. Horizontal caisson-type seawalls are not preferred. Tetrapods and dolosse are excluded.

4. Landfill of artificial islands

4.1 General landfill costs

The materials which are used to fill the interior of the island, and which do neither belong to the sea defences nor to the actual island's structures, are called landfill. Landfill can consist of soil, waste or other materials. Landfill, found on both marine and land excavation sites can be used, depending on economical preferences. Marine soils are economically excavated using trailing suction hopper dredges. Its economy is depending on the sailing distance between the excavation site and the island location.

In general, it can be stated that the bigger the size of the hopper the cheaper the costs per m^3. The cycle time, being the combined time for dredging + sailing to the dumping site + dumping + sailing to the excavation site, determines the dredging costs per m^3 for a particular trailer. If a trailing suction hopper arrives at the island location with its load, there will generally be four methods to remove the load.

- Direct dumping (occurs in a relatively short time)
- Rainbowing (medium time)
- Pumping (large time)
- Two times handling (Dredge + Stationary dredge)

Because cycle productions are less if emptying the hopper takes more time, the costs per cubic metre of landfill are higher. Direct dumping is restricted to certain waterdepths dependent on the vessel's draught. Consequently, the top parts of the island have to be filled using other methods. In other words, the landfill in top parts of artificial islands is more expensive.

4.2 Guidelines in relation to landfill

- **Large trailing suction hopper dredges are most suitable to be used in providing the fill material, especially when larger islands are considered.**
- **The overall costs of bigger islands are less as compared to smaller islands as is shown in Figure 4.3. Figure 4.3 is based on a likely relation for the costs of the fill and the costs for the sea defences. If the surface area of an island grows, the length of the sea defences grows proportional to the square root and the volume of the landfill linear. Consequently, the overall costs per square metre are less for bigger islands than for smaller islands. The graphs of Figure 4.3 also show that the proposed relation is not very sensitive to the waterdepth nor to the distance to the excavation site.**

  The relations in Figure 4.3 were made on the basis of a circular island. A circular island has the fine quality that its surface to circumference rate is very economic. Islands of other shapes, show a similar relation, as drawn in Figure 4.3, but in a less extreme way.

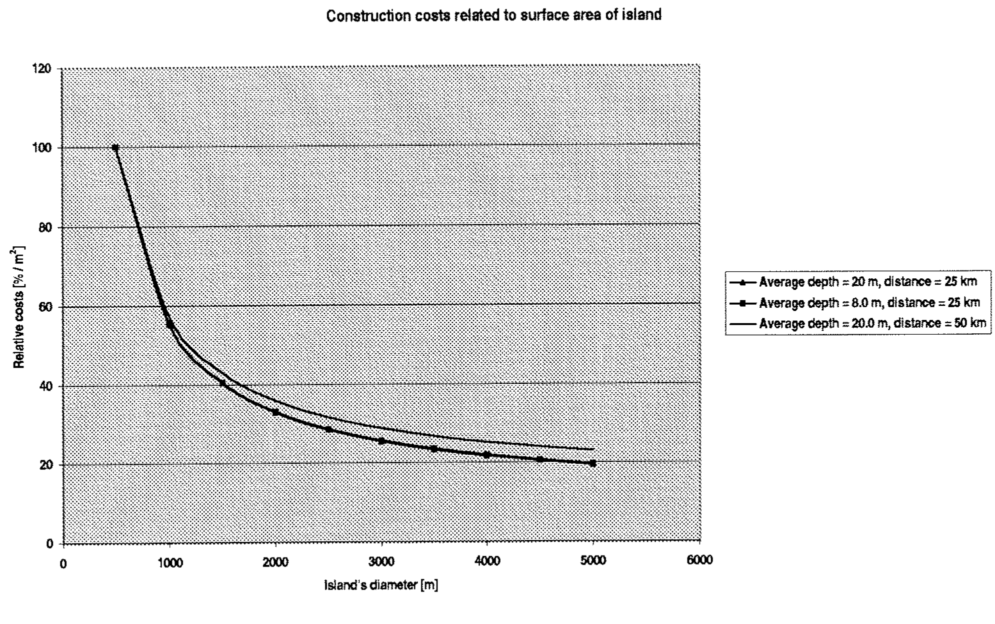


Figure 7: Constructions costs related to size of island and distance to excavation site

5 Morphological impact of artificial islands

5.1 General

A reclaimed island located some distance offshore induces a low *wave* energy zone behind it. In this so-called shadow-zone wave heights will be reduced, and consequently a reduction of longshore transport occurs. A reduction in longshore transport may cause accretion in the shadow-zone, and erosion in the adjacent areas. Whether these processes actually cause problems, will depend on the location of the island. At densely populated areas more problems may be expected than at deserted coastal areas (e.g., islands for oil & gas exploitation in the Canadian Beaufort Sea).

The predictions and statements made in this section will only be valid, during a relatively short period. For morphological processes this implies a maximum of a few years, depending on the local circumstances. Once the first signs of erosion and accretion occur, circumstances change. For example the breaker line is re-positioned, and the refraction pattern is altered as well. Nevertheless these short term influences on longshore transports are an indication of the problems to be expected in the long run. And consequently on the amount of money involved to overcome these problems during the island's lifetime. The designer may consider the extra costs of placing the island further offshore, generating less erosion, or placing the island closer to shore, and spending more money on overcoming erosion and accretion problems.

5.2 Numerical model HISWA

The influence of the island's size and its distance offshore, on longshore processes should be invested. For that reason an island model, as shown in Figure 5.2 is used. The model shows a circular island, having a diameter D, and located some distance offshore d. The deepwater waves, indicated by H_0 and T, bend around the island caused by refraction. The shadow zone can be seen too; it is clear that the width of the shadow zone is determined by refraction and also by the directional spread of the waves. Within the shadow zone the influence of the islands is most conspicuous.

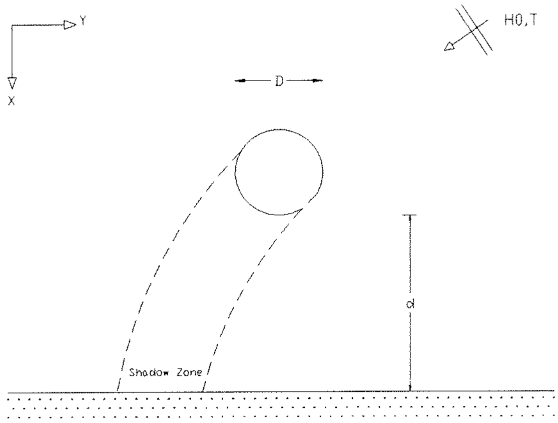


Figure 8: Schematised numerical model (two refraction-lines drawn)

The island model is used in a numerical model, called HISWA. HISWA is a numerical 2D wave propagation model to obtain realistic estimates of wave parameters in coastal areas, lakes and estuaries from given stationary wind, bottom, and current conditions. Using HISWA the wave parameters in the initial situation (before island construction) and after island construction can be computed. These wave parameters can be used subsequently to compute longshore transports, using the CERC-formula. Comparing the initial longshore transport S_0 to the longshore transport after island construction S, will give an indication of the relative influence of the island's diameter and its distance offshore on longshore transports.

The CERC-formula predicts the total longshore sediment transport through the breaker zone. The formula relates the transport to the available wave energy. Only the driving force resulting from waves approaching obliquely and having the same proportions at all points along the coast, is considered. Hence, no other driving forces than waves, such as tidal currents, are taken into account. S_x indicates the longshore transport, H_b the wave height at the breaker line, c_b the wave celerity at the breaker line and ϕ_b the angle of incidence also at the breaker line.

$$S_x = 0.025 H_b^2 c_b \sin(2 * \phi_b)$$

Despite its limitations, the CERC-formula is easy to understand, using more formulae would not increase the insight into the process.

5.3 Design assumptions and limitations

Assumptions on boundary conditions

- $\overline{H_{s0}} = 2.0$ m: An average significant deepwater wave height of 2.0 m is assumed. A suchlike wave height initiates the major part of the morphological processes, in the long term.
- Deepwater wave steepness s_0 of 0.05: This kind of steepness indicates seas; waves generated by local storms. Because steepnesses in the range of seas are the most common in nature, these have been chosen.
- No currents: HISWA only calculates the wave heights, and for these calculations, a current is only of second order influence.
- Bathymetry: For reasons of simplicity, the bottom-profile is kept simple. A uniform slope of $1:100$ to a depth of 30.0 m is assumed, beyond the sea bottom continues horizontally.

Assumption on island's lay-out

- Vertical walls: Reclaimed artificial islands are protected by hard sea defences in this report. Slopes for the sea defences vary from totally vertical (caissons) to $1:6$ (rubble mound sea defence). Related to the dimensions of the island and the influence of the morphological processes, these slopes can be taken as vertical. As a second condition, no transmission is assumed for the vertical walls.

Variables

- Diameter of circular island D: Man-made islands, which have already been built, vary in size from a few hectares to over a thousand hectares. Accordingly three diameters will vary, viz. 0.5, 1.0 and 4.0 km, representing areas of respectively 20, 78 and 1257 ha (ha = 10,000 m^2).
- Distance offshore d: The distance offshore is defined as the distance between the outmost shoreward point of the island and the transition point between the wet and the dry beach at M.S.L. This parameter varies for values of 1.0, 2.0 and 5.0 km.
- Deepwater angle of incidence ϕ_0: The angle of incidence determines the extent of the sediment transport. The transport-ratio increases for increasing wave-angle up to a deepwater angle of about 45 degrees. This statement can be 'found' in the CERC-formula. An upper-limit transport is determined for a deepwater wave angle of 45 degrees, and as a reference, the same calculations are made for a deepwater wave angle of 10 degrees.
- Directional spread of the waves: Directional spread is defined as the spread of the wave angles of incidence around the average deepwater angle of incidence. Two values are used for the directional spread of the waves, namely 5.7° and 31.5°. The second one, a wide spread, refers to the natural circumstances in which seas occur. The first one, a minimal spread, is used as a reference. The directional spread will prove to have a great influence on this section's results!

All variables are summed up once more in Table 5.2.

Diameter D [km]	Distance d [km]	ϕ_0 [degrees]	Dir. spread of waves [degrees]
0.5	1.0	10	5.7
1.0	2.0	45	31.5
4.0	5.0		

Table 4: Variable input parameters for HISWA

The influence of the parameters, mentioned at 'assumptions on boundary conditions' on the outcome of HISWA-computations indicate the sensitivity of the method used. If the outcome does not differ much for altered values of these parameters, the result of the model will be more widely applicable. The parameters mentioned in Table 5.6 are changed to test the sensitivity of the model, successively the deepwater wave height, the deepwater bottom level, the foreshore slope and the deepwater wave steepness. Without showing calculations, it is mentioned that HISWA outcomes are not sensitive at all to a minimal spread of 5.7°. For a directional spread of 31.5° the influence of the parameters of Table 5.6 becomes more remarkable, but is still small enough for the model to be widely applicable.

H_{s0}	h_0	m	s_0
1.0 m	20 m	1 : 250	0.01
3.0 m	40 m		

Table 5: Altered parameters in sensitivity-analysis

5.4 Example of computation

The influence on longshore transport is investigated of a 1 km circular island, positioned 1 km offshore. Waves approach at deepwater, with an angle of incidence of 10 degrees. Two directional spreads are taken into account, namely 5.7° (minimal) and 31.5° (large). Figure 9 shows the results varied for two directional spreads.

The relative transport rate S/S_0 indicates the relation between the initial longshore transport and the longshore transport directly after island construction. The centre of the circular island is positioned at point Y = 10,000. Because the deepwater waves are travelling towards the negative Y-direction, a positive rate of S/S_0 indicates longshore transports from right to left. What catches the eye first, are the relative transports exceeding 100% and the negative transports, both for a large directional spread. The causes of these phenomena and other conclusions will be discussed in the section below. For a minimal directional spread the minimal relative transport amounts to zero, for a large directional spread the relative transport becomes negative. Consequently the longshore transport is totally blocked in both cases, this process will most likely form eventually a tombolo, which can develop into a connection from the original shoreline to the island.

5.5 Guidelines in relation to morphological impact

The effects of artificial islands on longshore transport may be severe subordinate on the island's size and its distance offshore. The directional spread plays an important role.

If deepwater angles of incidence are small, and a large directional spread is used also, negative transports may result. In some areas at the lee-side of the island, the contribution of

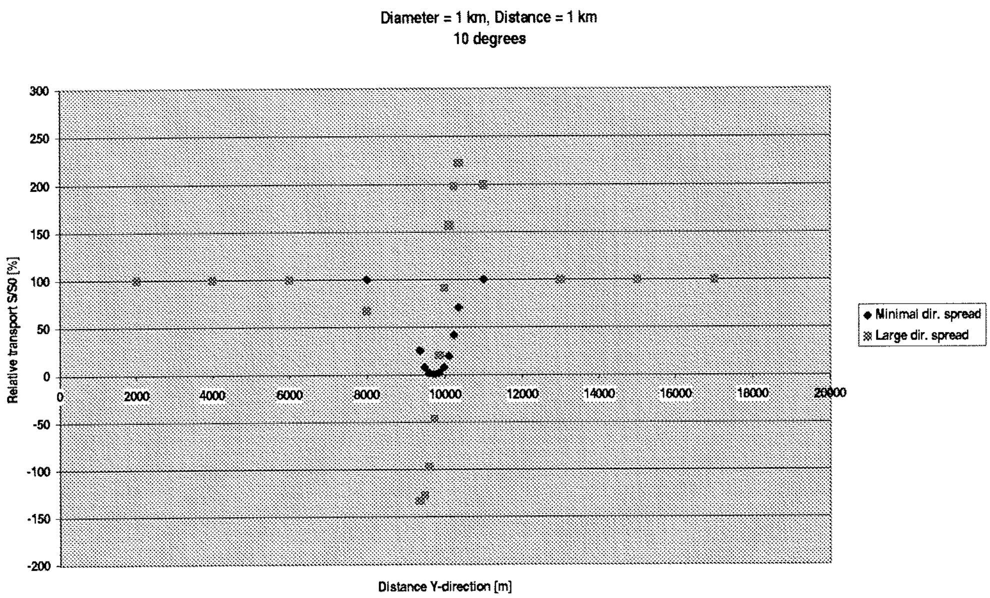


Figure 9: Relative changes in longshore transport after island construction

the waves having a large angle of incidence is blocked, and only the waves having a small or even a negative angle of incidence contribute to the transport. These waves, originating from a deepwater wave with a negative angle of incidence, will cause the longshore transport to move into a negative direction.

A similar situation exists at the exposed-side of the island. If a large directional spread is chosen, the waves having a smaller angle of incidence will be blocked by the island. This results in a higher average angle of incidence at the weather-side of the island. When the CERC-formula is used to predict sediment-transports, the maximum transport-rate is found for angles of incidence of about 45°. In some cases the local sediment transport after the construction of an island may be higher than in the initial situation. For example when the initial angle of incidence amounted to say 10 degrees, and the local average angle becomes 30 degrees.

The following guidelines can be given based on the directional spread of the incoming waves.

Large directional spread:

- **For small deepwater angles of incidence, relative transports may exceed 100%.**
- **Table 6 shows the maximum blocking of longshore transport, related to the island's distance offshore *d* and its diameter *D*. Maximum blocking occurs at**

the centre of the shadow-zone. 100% blocking indicates a minimal transport of zero or less.

- Figure 10 also shows these relations. For four situations the influence of d and D on longshore transport is drawn. All graphs represent S-curves, and consequently the missing parts may be extended to $S/S_0 = 100\%$.

Max. Blocking	d/D – 10°	d/D – 45°
50%	± 5	± 2
100%	± 2 - 2.5	± 0.5

Table 6: Maximal blocking of longshore transport related to d/D and deepwater angle of incidence. Directional spread is large

Minimal directional spread:

- Influence of deepwater angle of incidence is still noticeable, but is much smaller compared to a large directional spread.
- Figure 5.9 is a graphical reproduction of the relation between the minimum relative longshore transport and the islands diameter D plus its distance offshore d. The curves are averaged for deepwater angles of incidence of 10 and 45 degrees. It may be clear, that Figure 5.9 represents the same type of curves as shown in Figure 10, only displayed differently!
- Table 7 shows the maximum blocking of longshore transport, related to the island's distance offshore d and its diameter D. Maximum blocking occurs at the centre of the shadow-zone. 100% blocking indicates a minimal transport of zero or less.

Max. Blocking	d/D – 10°	d/D – 45°
50%	± 5.5	± 4
100%	± 1 - 1.5	± 1 - 1.5

Table 7: Maximal blocking of longshore transport related to d/D and deepwater angle of incidence. Directional spread is minimal

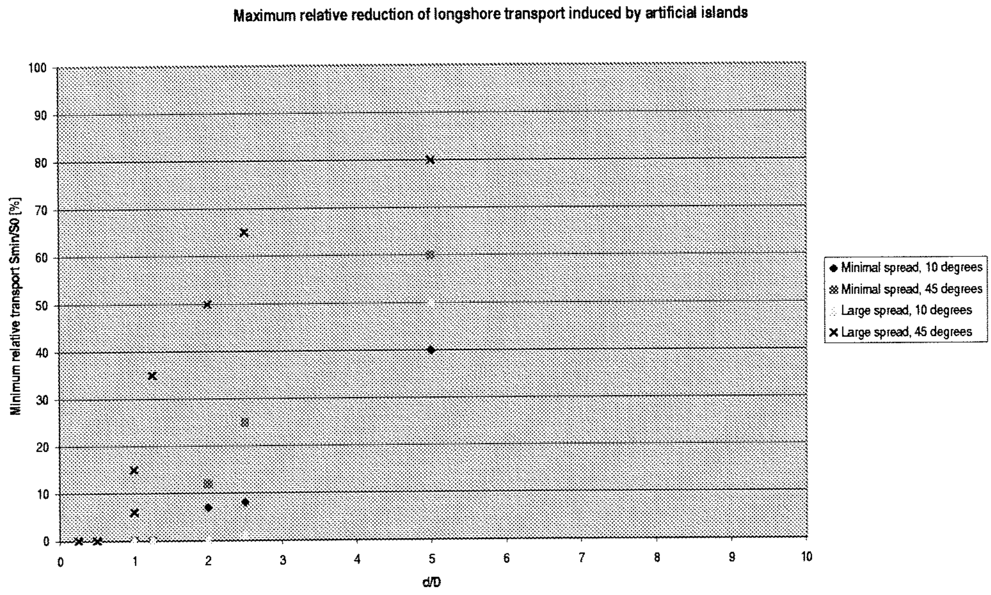


Figure 10: Minimum relative longshore transports, related to deepwater angle of incidence and directional spread

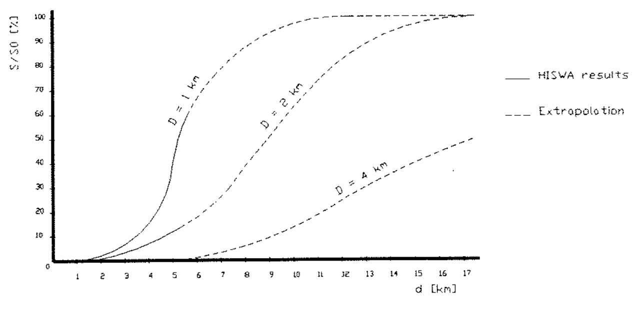


Figure 11: Minimum relative longshore transports, shown for each island's diameter individually. Directional spread is minimal

Contents

1 Introduction **1**

1.1 What are artificial islands and why are they built? 1

1.1.1 Reasons to construct artificial islands 2

1.1.2 Advantages and disadvantages of artificial islands 2

1.2 Purposes for artificial islands 3

1.2.1 Single purpose islands 4

1.2.2 Multi purpose islands 6

1.3 Types of artificial islands 7

1.3.1 Description of structural artificial island types 7

1.3.2 Comparison of structural artificial island types 9

1.3.3 Restriction to detail reclaimed artificial islands only 14

1.4 Problem description 15

1.5 Research objective 15

1.6 Problem approach 15

2 Experience gained from constructed islands **17**

2.1 Introduction 17

2.2 Similar structural properties of artificial islands 17

2.3 Main construction and design problems 18

2.3.1 Earthquakes 19

2.3.2 Morphological impact 20

2.3.3 Overall cost reduction 20

2.3.4 Settlements 20

2.4 Remarks 20

3 Design of the sea defence structures **21**
3.1 Introduction 21
3.2 Rubble mound sea defences 22
3.3 Caisson-type seawalls 26
3.3.1 Measures to increase effectiveness of caisson-type seawalls 26
3.4 Guidelines for the use of sea defences 28
3.4.1 Research objective 28
3.4.2 Design restrictions 28
3.4.3 Design assumptions 29
3.4.4 Examples of calculations 31
3.5 Conclusions and recommendations - choice sea defences 33
3.5.1 Wave steepness $s = 0.01$ 33
3.5.2 Wave steepness $s = 0.05$ 34
3.5.3 General costs indication 43
3.5.4 Influence of material-costs on the use of sea defences 47
3.5.5 Recommendations for future research 47

4 Dredging activities for artificial islands **49**
4.1 Introduction 49
4.2 Dredging of landfill 50
4.2.1 Sections of the dreding process 50
4.2.2 Parameters influencing cycle production 51
4.3 Dumping of landfill 51
4.3.1 Parameters influencing dumping of landfill 54
4.4 Costs related to size of island 54
4.4.1 Costs per dredged and dumped cubic metre 54
4.4.2 Overall island costs 57

5 Morphological processes **59**
5.1 Introduction 59
5.2 Morphological processes 60
5.2.1 The actual problem 61
5.2.2 Only short term influences 62
5.3 Island model used in this report 62
5.3.1 Expected relation 62
5.4 Problem description 64
5.4.1 Research objective 64
5.4.2 Design assumptions 64

5.5 Using the numerical model HISWA . . . 66
5.5.1 Computational grid and boundary conditions . . . 66
5.6 Calculation approach . . . 69
5.6.1 The CERC-formula . . . 69
5.7 Example of computation . . . 71
5.7.1 Sensitivity of HISWA for H_{s0} and bathymetry . . . 75
5.8 Conclusions and recommendations . . . 79
5.8.1 Minimal directional spread . . . 79
5.8.2 Large directional spread . . . 82
5.8.3 Recommendations for future research . . . 82

6 Conclusions and recommendations 85
6.1 Introduction . . . 85
6.2 Conclusions . . . 85
6.2.1 Use of sea defences . . . 85
6.2.2 Overall costs related to size of island . . . 86
6.2.3 Morphological impact of artificial islands on longshore transports . . . 86
6.3 Recommendations . . . 87
6.3.1 Use of sea defences . . . 87
6.3.2 Morphological impact of artificial islands on longshore transports . . . 88

A Collection of artificial islands (1955 - 1997) 91
A.1 Survey of artificial islands in North and South America . . . 92
A.2 Survey of artificial islands in Asia and Australia (exclusive Japan) . . . 94
A.3 Survey of artificial islands in the Beaufort Sea (Canada) . . . 95
A.4 Survey of artificial islands in Europe . . . 97
A.5 Survey of artificial islands in Japan . . . 99

B Structural relations for artificial islands 103
B.1 Relation waterdepth-area (build) . . . 104
B.2 Relation waterdepth-area (planned) . . . 105
B.3 Relation waterdepth-distance (build) . . . 106
B.4 Relation waterdepth-distance (planned) . . . 107
B.5 Relation distance offshore-area (build) . . . 108
B.6 Relation distance offshore-area (planned) . . . 109

C Design of rubble mound sea defences **111**
C.1 Methods and formulae used in calculations 112
C.1.1 General equations 112
C.1.2 Rubble mound sea defences armoured with quarry stones 113
C.1.3 Rubble mound sea defences armoured with concrete elements 115
C.1.4 Equal design parts for rubble mound sea defences 116
C.2 Two examples of rubble mound sea defence design 119
C.2.1 Deepwater sea defence [h=26m] 119
C.2.2 Alternative deepwater sea defence [h=26m] 124
C.2.3 Shallow water sea defence [h=8m] 127

D Design of caisson-type seawalls **133**
D.1 Calculation method 133
D.1.1 General construction costs 133
D.1.2 Crest height 134
D.1.3 Dimenions of the caissons 134
D.1.4 Shape of the mound 135
D.1.5 Weight of rubble mound elements 137
D.1.6 Forces on caisson as a result of island's fill 141
D.1.7 Forces on caissons as a result of non breaking wave pressures 141
D.1.8 Final design 143
D.2 Example of vertical-type seawall (h=26m) 144
D.3 Example of horizontal-type seawall (h=26m) 148
D.4 Conclusions caisson-type seawalls 150

E Details of calculated sea defences **153**

F Changes in longshore transport **187**
F.1 Tables and graphs for variations in d, D and ϕ_0 187
F.2 Sensitivity analysis, minimal spread 225
F.3 Sensitivity analysis, large spread 231

List of Figures

1 Structural types of artificial islands (not to scale) vi

2 Cross-section of typical rubble mound sea defence ix

3 Indication of ranges in which to apply sea defences A xiv

4 Indication of ranges in which to apply sea defences B xv

5 Indication of ranges in which to apply sea defences C xvi

6 Indication of ranges in which to apply sea defences D xvi

7 Constructions costs related to size of island xviii

8 Schematised numerical model (two refraction-lines drawn) xix

9 Relative changes in longshore transport after island construction xxii

10 Minimum relative longshore transports, shown for each island's diameter . . . xxiv

11 Minimum relative longshore transports, shown for each island's diameter . . . xxiv

1.1 Structural types of artificial islands (not to scale) 8

3.1 Specific elements of rubble mound sea defence 23

3.2 Examples of concrete armour units [18] 24

3.3 Caisson-type breakwaters [23] 27

3.4 Relative construction costs for quarry stones sea defence (s = 0.01) 35

3.5 Relative construction costs for dolosse sea defence (s = 0.01) 36

3.6 Relative construction costs for tetrapods sea defence (s = 0.01) 37

3.7 Relative construction costs for vertical caisson sea defence (s = 0.01) 38

3.8 Relative construction costs for quarry stones sea defence (s = 0.05) 39

3.9 Relative construction costs for dolosse sea defence (s = 0.05) 40

3.10 Relative construction costs for tetrapods sea defence (s = 0.05) 41

3.11 Relative construction costs for vertical caisson sea defence (s = 0.05) 42

3.12 Indication of ranges in which to apply sea defences A 44

3.13 Indication of ranges in which to apply sea defences B 44

3.14 Indication of ranges in which to apply sea defences C 46

3.15 Indication of ranges in which to apply sea defences D 46

4.1 Example of trailing suction hopper dredge [25] 50

4.2 Loading-curve for trailing suction hopper dredge, having a fixed overflow-height [26] 52

4.3 Constructions costs related to size of island 58

5.1 Radiation shear stresses [27] 61

5.2 Refraction shown by island model (two refraction-lines drawn) 63

5.3 Qualitative relation S/S_0 and d/D, see also figure 5.2 63

5.4 Computational grid of HISWA 67

5.5 HISWA-plot, isolines for H_s and vectors for wave direction 73

5.6 Relative changes in sediment transport after island construction 74

5.7 Influence of H_{s0} and bathymetry on changes in longshore transport 76

5.8 Influence of H_{s0} and bathymetry on changes in longshore transport 77

5.9 Minimum relative longshore transports, shown for each island's diameter . . . 80

5.10 Minimum relative longshore transports, related to island's dimensions A . . . 81

5.11 Minimum relative longshore transports, related to island's dimensions B . . . 83

C.1 Parameters related to cross-section of rubble mound sea defence 112

C.2 Dimenions used to construct rubble mound sea defences 113

C.3 Area of armourlayer in cross-section according to waterdepth 117

D.1 Stability number for rubble foundation and toe protection [SPM 1984] 140

D.2 Wave pressures on a caisson according to Goda 142

List of Tables

1 Users of single purpose islands iv

2 Users of multi purpose islands v

3 Gradations and prices for quarry stones xii

4 Variable input parameters for HISWA xxi

5 Altered parameters in sensitivity-analysis xxi

6 Maximal blocking of longshore transport related to d/D and deepwater angle of incidence. Directional spread is large xxiii

7 Maximal blocking of longshore transport related to d/D and deepwater angle of incidence. Directional spread is minimal xxiii

1.1 Users of single purpose islands 5

1.2 Users of multi purpose islands 6

1.3 Different aspects of structural island types 9

1.4 Relation between island-types and natural & structural conditions 10

1.5 Relation between island-types and users 12

3.1 Gradations and prices for quarry stones 31

4.1 Influencing parameters on cycle production 52

4.2 Influencing parameters on method of dumping 54

4.3 Cycle production calculation for trailing suction hopper dredge 56

4.4 Assumed prices per cubic metre of dredged soil 57

4.5 Construction costs relation between large and small artificial islands 57

5.1 Paramaters influencing longshore transport 62

5.2 Variable input parameters for HISWA 66

5.3 Influence of grid-density on accuracy of HISWA-outcome 69

5.4 Percentage of broken waves computed by HISWA, compared to theoretical breaker line 70

5.5 Changes in longshore transport for 1 km island at 1 km offshore, $\phi_0 = 10°$. 72

5.6 Altered parameters in sensitivity-analysis 75

5.7 Minimal relative longshore transports, after island construction A 79
5.8 Minimal relative longshore transports, after island construction B 82

C.1 Gradations and prices for quarry stones 113
C.2 Design values for S_d for a two diameter thick armourlayer [18] 114
C.3 features of concrete armour elements 115
C.4 Weight of armour elements [30] 116
C.5 Cost comparison for different slopes (h=26 m, dolosse-type sea defence) . . . 122
C.6 Cost comparison for different slopes (h=8 m, quarry stone-type sea defence) . 129

Chapter 1

Introduction

The main purpose of this master thesis is to present guidelines to designers involved in the planning or realisation of artificial islands. Part of these guidelines were collected from literature, other guidelines were part of the examination executed in the framework of this project at Delft University of Technology. The latter refers to the economical comparison of sea defence structures in chapter 3, the overall costs of man-made islands related to their size in chapter 4 and the influence of artificial islands on morphological processes in chapter 5.

Before the guidelines are presented, first a description is given of artificial islands, also called man-made islands. Chapter 1 introduces the reader into the world of artificial islands. The purposes of man-made islands are discussed as well as the main types of islands. At the end of this first chapter the problem description and research objective are stated. After the introduction, the first guidelines are given in chapter 2. These guidelines were collected from literature. Chapter 3 helps the reader to choose between defence structures. The overall costs of artificial islands depend largely on the total volume of the island, this aspect is part of chapter 4. The expenditures needed to restore the shoreline, after being eroded and accreted, under the influence of an artificial island, are underestimated by most designers. A general indication of the effects of an offshore island on longshore transports is part of chapter 5. Finally conclusions and recommendations are given in chapter 6.

1.1 What are artificial islands and why are they built?

To indicate the field for which the guidelines are valid, first a definition of an artificial island is required.

In this report an *island* is defined as an area in the ocean or in the coastal zone, surrounded by water at all sides, this in contrary to a peninsula. This implies that classic land reclamation, which in most cases forms a peninsula, is not accounted for. Neither are islands constructed in closed waters, such as lakes. In this feasibility study mostly hydraulic aspects of man-made islands are examined. The adjective *artificial* means that the island is man-made. The material of which artificial islands are built, is not automatically a soil-type of material. An artificial islands can be constructed of waste material, concrete and steel (e.g. jacket-structure or a floating structure) or even out of ice (some studies have examined this possibility). In short the studied artificial islands in this report can be defined as :

Areas in the coastal zone or the ocean, surfacing totally or partly above water level and surrounded by water at all sides, designed and constructed by man. These areas are positioned in a fixed location and serve a public interest.

1.1.1 Reasons to construct artificial islands

Numerous causes that justify the construction of man-made islands can be given. Hereafter, six reasons are summed up, together they cover most causes, which eventually could lead to the erection of an artificial island.

- Lack of extension possibilities on the mainland for the required activities.
- By placing the activities on an artificial island, and removing or reducing the original activities, the original nuisances disappear within the surroundings of the former location of the activities. By doing so the livability for the community is improved.
  Note: At such a distant location as an artificial island the same environmental laws are in force as on the mainland. The one advantage is that the chance of exceeding one of those laws on an island will probably be much smaller. This is caused by the fact, that man-made islands are mostly built for a single purpose or closely related purposes, and consequently the interference with conflicting activities is minimal.
- Lack of deepwater access.
- High costs of deepening or maintaining a dredged channel access to existing ports.
- Mining of natural resources, such as oil and gas.
- Location related activities, like tidal or wind energy generation.

Whether an island is actually built, depends largely on the economical feasibility. Technical limitations seldom were the determining factors for those islands, that were never built. Concluding from the six reasons above, man-made islands are not always constructed near densely populated areas. Something which might be expected at first instance. However, the first four items are area related, because some kind of relation with the original area or port has to be conserved, either as a market, as a source of materials or due to economical dependence.

1.1.2 Advantages and disadvantages of artificial islands

Like every structure built, a man-made island has advantages as well as disadvantages. The following list is a literally quotation from [1], and certainly worth mentioning. It refers especially to industrial islands, a type of island explained in paragraph 1.2.2. Some items mentioned below, are very dependent on factors like the distance from the island to the shore and the size of the island. Nevertheless an indication of the benefits and drawbacks of artificial islands can be obtained.

Advantages of artificial islands

- Market accessibility; if the extension limitations have reached a certain level, moving the activities to an artificial island might increase market accessibility. An island site might be only 5 - 10 miles from the heart of a city as opposed to a land site, possibly 35 - 40 miles from the city centre.
- Site utilization; an island site is likely to be unencumbered with hills, valleys, streams, highways and other physical barriers to ideal site layouts.
- Fewer environmental constraints because of a dearth of interfaces with people, plants and creatures.
- Unlimited availability of cooling water, when placed far enough offshore.
- Removal of bad-neighbour image, 'out of sight out of mind', etc.

Disadvantages of artificial islands

- Labour force transportation problems.
- Industry on island must be self-supporting in public utility services.
- Hazards from storms and other adverse events.
- Island to shore transportation of products and raw materials.
- Sociological implications of work environment.

1.2 Purposes for artificial islands

To understand for which purpose(s) an island is actually built, one must first be aware of the possible users of an artificial island. When one has assembled this information, a start can be made with the design of a man-made island, which complies with the demands of a specific user. For example, an island used as (deepwater) port demands completely different requirements and facilities than an island used as an airport. An important factor for an island used as (deepwater) port is the waterdepth around the island, where on the other hand waterdepth is of minor use for an airport-island.

The purposes for which an island is designed, are split up in two categories: single purpose islands and multi-purpose islands. Single purpose islands are built only to serve one purpose, for example to facilitate a fishery port. Multi purpose islands accommodate a variety of users, and are for that reason mostly larger than single purpose islands. For instance an island erected to serve as urban expansion region, not only holds houses, but probably recreational facilities and some sort of energy supply as well. At present, mostly single purpose islands have been constructed. In the future new economical developments are expected, resulting into renewed attention for multi-purpose islands. Especially because these islands are more attractive for investors than single purpose islands. The required size of a multi-purpose island and the associated costs are probably the reason that until now only a few of these islands are built. In paragraph 1.2.1 the users of single purpose islands are mentioned and in paragraph 1.2.2 the same is done for multi-purpose islands.

The tables shown in both paragraphs are by no extent complete. Because of the simple fact, that one can always come up with some very rare purpose for a man-made island. Nevertheless the classification of the tables was based on existing man-made islands. All mentioned types of islands have been designed and constructed, with the exception of those for *Mariculture*, as this type only exists on the drawing boards. If a purpose does not fit one of the other categories it is classified as 'Temporary&Aiding'.

1.2.1 Single purpose islands

Eight categories of single purpose islands are distinguished in this report, as can be seen in table 1.1. The categories are ordered by the expected island surface which is required, ranging from small to large. Furthermore, each category is given an abbreviation. These abbreviations are used in appendix A to indicate what kind of island is meant. Finally, examples are given where possible.

Some remarks for each category listed in table 1.1 are given, hereafter.

<u>Ocean mining & exploration</u>
Applications, such as the mining of natural resources from the ocean-floor by oilrigs are well known. In most cases these oilrigs are made out of steel and concrete. However at some areas in the world the drilling-equipment is placed on artificial islands, filled with soil or gravel. For example the mining of oil and gas in the Canadian Beaufort Sea is done from reclaimed islands, to withstand natural loads such as ice-forces. For a long-term research of an ocean-area a man-made island can also serve as an exploration-station.

<u>Recreation</u>
One would expect this type of single purpose user to be part of an urban zone island or maybe an industrial island. Nevertheless Japan has developed, and even constructed islands for this sole purpose. Countries with very limited arable land, like Japan can develop recreational facilities on an artificial island to provide leisure facilities for their citizens.

<u>Waste handling</u>
Waste handling can occur in two different ways. Firstly, the processing of waste can be more favourable on a remote island as compared to the mainland, for reasons of stench-annoyance, noise pollution or lack of space. This first purpose installs processing and storage facilities on the island. This method is called 'waste handling'. Secondly, waste material, domestic as well as industrial waste, can be used as landfill for a man-made island. When the artificial island is completely filled, it can be used for a variety of purposes, like recreation and housing. Environmental measures should be taken to prevent leakage to the surroundings. This latter sort of use is part of the multi-purpose islands, and is more detailed in paragraph 1.2.2.

<u>Mariculture</u>
Special islands can be created for mariculture, such as cultivating weeds, mussels, oysters or farming of fish for commercial use.

<u>Temporary or aiding structure</u>
These types of islands range from very small to large, as they can be constructed for a

Single purpose	Code	Some examples
Ocean mining&Exploration	M&E	Gas Oil Research
Recreation	Rec	
Waste handling	WH	Waste processing Waste storage area
Mariculture	Mar	Farming of fish Oysterfarm
Temporary or aiding structure	T&A	Anchoring for bridge cables Closure of tidal areas Work island
Energy generation	EG	Atmosphere gradient Coal Fired plant
Port	Port	Deepwater port Fishery port Transhipment port
Airport	Air	

Table 1.1: Users of single purpose islands

variety of purposes. One can think of islands, which are built to ease the closing of a (tidal) channel or an island to be used as a temporary concrete factory [2]. Aiding constructions are mostly smaller than temporary structures. If one considers islands in use as protection for bridge-piers [3], as a ventilation shaft for an undersea coalmine or even as a pig quarantine storage, it seems justified to define them as aiding structures.

Energy generation

This very distinct group of users can favour from a location on a man-made island. The environmental laws concerning noise- and smell-pollution can be coped with more effectively on an artificial island. For power plants using sea-water for cooling purposes the difference in temperature between intake and outgoing water is presumably allowed to be greater than plants situated at rivers or harbours.

- For nuclear power plants the 'not in my backyard' motive is effective. This especially is in force for densely populated areas. In such cases construction on an island would be a suitable solution. But as a result of the change in public opinion, it is questionable whether nuclear power plants will be built at all in the future.

- An 'artificial' Pump Storage Plant will only be economical in countries without a large natural difference in height-levels, such as the Netherlands. And in case the available area to construct such an immense structure is scarce, construction of a pump storage plant on an artificial island could be a solution.

- An island constructed as an oil terminal storage is considered part of this category as well, as it has a close relation with energy generation. Furthermore, safety aspects are very similar.

Port

When the economical growth of a country requires the expansion of its harbour activities, it can face expansion-problems due to lack of area. As a solution one can choose for an artificial island, nevertheless a shore connected expansion is an alternative as well! In some cases the deepening or maintenance costs of a dredged channel connecting the existing ports to the deep seas is too expensive, hence a decision can be made to erect a deep-water port on an artificial island.

Airport

Till this moment all airports built or planned on artificial islands are expansions or displacements of existing airports. These airports seek a new home on man-made islands, either because of expansion problems on the mainland or because they can cope with environmental requirements more easily on an island. When properly designed, sound-contours from the island do not cross inhabited areas any more or at least to a lesser extent, thus the environmental requirements can be fulfilled more easily. Furthermore, unobstructed approach and departure routes can be obtained. Disadvantages of an offshore airport can be noted as well. For example problems in transporting passengers and goods from and to the island. Secondly, some reports proclaim aircraft operations being affected by possible fog and thermal air currents. Moreover environmental groups have freshened the discussion about migratory birds resting at the island, and as a result endangering flight-traffic. This discussion is still open for debate. Furthermore, salt sprays can disorder sensitive equipment on the island. Although many plans have been made and are still being made, only few airports have actually been erected on artificial islands, i.e. some airports in Japan (e.g. Kansai) and Chek Lap Kok, Hong Kong.

1.2.2 Multi purpose islands

Three categories of multi purpose islands are distinguished in this section, as can be seen in table 1.2. The categories are ordered by the expected island surface which is required, ranging from small to large. Furthermore, each category is given an abbreviation. These abbreviations are used in appendix A to indicate what kind of island is meant. Finally, examples are given where possible.

Multi-purpose	Code	Possible devided into ...
Waste island	WI	Waste storage combined with future activities, like ...
		- Recreation
		- Residential areas
Industrial island	II	Deepwater port
		Industrial zones
		Energy generation
Complex city	CC	Business areas
		Hospitals
		Leisure & Recreation
		Residential areas

Table 1.2: Users of multi purpose islands

Waste island

Of all users only waste handling is mentioned both for single purpose and multi-purpose

islands. Nevertheless two different purposes are meant. In case of a single purpose island the island is used for either waste handling or permanent waste storage. In this case the waste is moderate or heavy contaminated, in contrary to waste handling on a multi-purpose island.
In case of a waste island the waste is of such a quality, that future use of the island is still allowed after the island is completely filled with waste. For example for recreational purposes or as a housing area. Filling with waste materials takes more time than conventional landfill, because it takes many years before enough waste is produced to fill an entire island. Sometimes the waste has to be temporarily stored on the main-land, because investors are reluctant to spend money on building the surrounding sea defences, and prefer to wait several years for the island to be built.

Industrial island

Constructing an island for use as industrial zone can be a welcome solution to cope with expansion-problems and to comply with environmental requirements. When heavy industry uses the island, it is a realistic assumption that related facilities are desired like a harbour and a power plant. Besides, the industrial island has to be self-supporting to some extent. It requires its own fire fighting force, medical facilities, fresh water supply, waste treatment and electricity supply. Furthermore, legal matters have to be examined. Workmen have to be either housed on the island for longer periods or a functioning transport system from and to the mainland has to be developed.

Complex city

The items mentioned for an industrial island apply also for complex cities.

1.3 Types of artificial islands

Many structures and objects can comply with the definition of an artificial island as described in paragraph 1.1. All these structures and objects can be categorized in half a dozen structural types. Paragraph 1.3.1 discusses these six types, and for each structural type advantages and disadvantages are mentioned. Although not very relevant for this report, some information about the resistance to earthquakes is given as well.
These pro's and con's are related to a variety of desired and undesired qualities, in paragraph 1.3.2. This way a deliberated choice for a structural type of island is possible, provided that the purpose of the island is known.

1.3.1 Description of structural artificial island types

Figure 1.1 displays the six structural types of man-made islands.

Reclamation type

For this construction method an area in sea is surrounded by a sea defence. The sea defence can consist for instance of caissons, rubble mound and concrete armour elements, double steel sheet piles or an other structure. Then soil, waste, other materials or a combination of these is used as landfill. Two types of sea defences are especially detailed in this report, viz. caisson-type seawalls and rubble mound sea defences. These two types are described in chapter 3. The defence structures generally account for the major part of the islands total construction costs. This type of island is vulnerable to strong earthquakes but resists strong

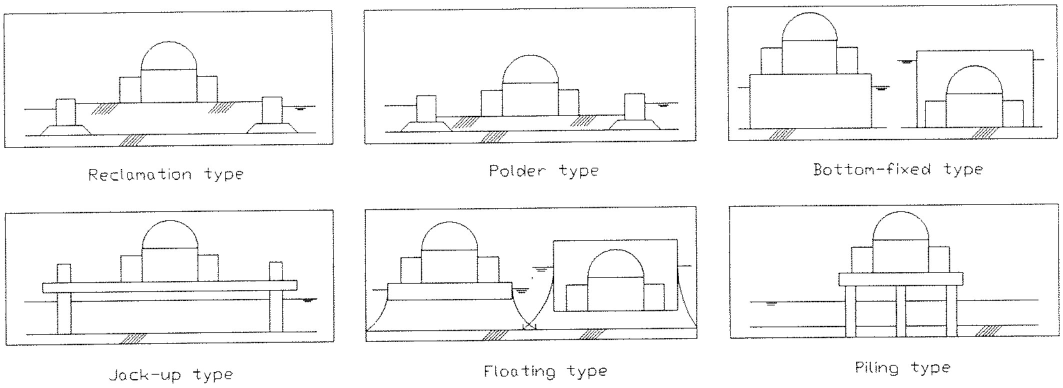


Figure 1.1: Structural types of artificial islands (not to scale)

waves.

Polder

The polder-alternative is a variation of the reclaimed-island. Designed as a polder, the ground level is situated lower than the surrounding average sea level The great advantage of the polder alternative is the reduction of landfill. A disadvantage, however, is the increased rate of damage in case of failure of a dike-section. Consequently, a polder is even more vulnerable to earthquakes, and especially exposed in case of ship collisions or explosions. A two-dike system can be a solution. The benefits of less landfill and a shorter construction time should well be weighed by an increased damage-rate in case of failure. Permanent pumping is required for seepage and rainfall.

Bottom-fixed (Gravity) type

This type is the heaviest of the six construction-types. It derives its stability against overturning from its weight combined with its large base. First a structure is created at a protected location, then this structure is towed to a ocean-site, where it is filled with sand or sea water and sunk to the bottom. It is necessary to prepare the site before the structure is sunk, because a flat sea bed is required. This type is vulnerable to earthquake-loadings as well. Repairs are very difficult and costly.

Jack-up type

This structure consists of a floating pontoon which raises itself above the sea surface by jacking itself along legs which are lowered to the seabottom after the structure has been floated into position. Since jack-up platforms can be easily moved from place to place they are best suited for temporary projects. They are restricted to relatively shallow water locations, up to 90 - 150 metres. Many oil rigs in the North Sea use the jack-up principle. A jack-up type has moderate resistance to earthquake-loadings.

Floating type

This type consists of a barge or ark like structure, and the facilities are placed inside or on its upper sections. Because this is a free floating structure, anchoring is necessary to keep the structure located into a fixed position. Automatic positioning systems (Voidth-Schneider) can be used for this purpose. Floating type structures can be located in water of any depth and are hardly affected by earthquakes but they have the disadvantage of being affected by the pitching and rolling caused by waves (including waves caused by seaquakes). In order to cope with this, measures can be taken. For example, the structure can be surrounded with a breakwater. Or to dampen the effect of pitching and rolling part of the floating-body can be submerged (called semi-submerged islands). Depending on changes in the weight of the load, the drift will change. At present, the Japanese are world-leaders in the field of floating and semi-submerged islands.

Piling type
This type of island is constructed by driving piles into the sea bottom, on top of which a platform is placed. Structures are erected on the platform. This type is also affected by earthquakes.

Table 1.3 shows some construction aspects of the above mentioned artificial island-types, like typical waterdepths and the estimated lifetime of the structures, as well as some economical features of the islands. Although this information is retrieved from a report dating back to 1976 [4] it gives an indication about the usability of the structural types. The original report stated actual minimum and maximum prices per m^2 island, of course these prices are out of date now. For that reason the maximum price per m^2 for the reclamation type has been set at 100%, taking its typical waterdepths into account. All other prices are percentage related to the maximum price for the reclamation island. Remarkable is the low cost for the polder type compared to its estimated annual maintenance costs.

Structural type	**Typical Waterdepth [m]**	**Approximate Relative Costs (Ranges) per m^2**	**Estimated Lifetime [years]**	**Estimated Annual Maintenance, % of original costs**
Reclamation	0 - 30	28 - 100%	50 - 100	1 - 2 %
Polder	0 - 20	0.8 - 16 %	50 - 100	2 - 5 %
Bottom-fixed				
- Tower(steel)	30 - 250	2000 - 20,000%	20 - 30	5 - 10%
- Pedestal (conc.)	100- 300	2000 - 20,000%	25 - 40	- 5%
Jack-up	90 - 150	2000 - 20,000%	20 - 25	5 - 10%
Floating[1]				
- Tensioned to Base structure	300 - 600	1200 - 48,000%	20 - 30	5 - 15%
- Platforms up to 400 * 400 m^2 and floating (moored)	15 - 300	480 - 1200%	20 - 40	5 - 10%
- Moored to anchors	150 - 6000	1200 - 48,000%	20 - 40	5 - 10%
- Automatic position	300+	1200 - 48,000%	20 - 25	5 - 15%
Piling	0 - 90	144 - 1440%	25 - 50	5 - 10%
[1] Semi-submerged for most cases				

Table 1.3: Different aspects of structural island types (does not include access or facilities) [4]

1.3.2 Comparison of structural artificial island types

This section checks the suitability of the six island types on the basis of several parameters. These parameters are split in two distinguished categories. The first category examines the influence of natural and structural conditions on the different types of islands. The second category examines, if a type of island is suitable to house one of the island-users, as mentioned in paragraph 1.2.1 and paragraph 1.2.2.

The items in both categories are rated from 'not suitable at all' to 'very suitable'. Consequently *++* means 'very suitable' and −− means 'not suitable at all'. The symbol *0* means indifference. For example, the reclamation type is given *++* for the area-criterion, in table 1.4. This implies that reclamation types are very suitable to construct large areas with.

Natural and structural conditions

Six specific parameters have been selected to evaluate the structural types of islands. They can help a designer decide which type of island is preferred. The following items are chosen *area of island, construction costs/m^2, construction time, the morphological impact on the coastline, the possible waterdepth and the influence of waves on the island.* Table 1.4 gives the pro's and con's for the man-made island types in relation to the aforementioned conditions.

Reclamation
Characteristics of reclaimed islands:

- Larger areas can be constructed relatively easy.
- Restricted to waterdepths of about 30 m. In deeper water the amount of material increases rapidly, causing the island to be less economical. However, if waterdepths are moderate this type is very suitable, especially because landfill can be obtained rather cheaply. Sea defence structures generally account for the majority of the costs.
- Construction takes some time, as large quantities of material are required. Time is also required for the settlements of landfill and subsoil. Furthermore, reclaimed islands may be difficult to demolish.
- Because of its large and massive volume, blocking of waves is considerable and consequently the morphological impact as well.
- Reclaimed islands are not subject to movement and vibration under wave and wind loadings.

Polder
Characteristics of polder islands:

- Larger areas can be constructed relatively easy.
- Restricted to waterdepths of about 20 to 30 m. In deeper water the amount of material increases rapidly, causing the island to be less economical. However, if waterdepths are moderate this type is even more economical than the reclamation type, as less landfill is required.

Parameter/type	Reclamation	Polder	Bottom-fixed	Jack-up	Floating	Piling
Area	++	++	0	−−	+	0
Construction costs/m^2	+	++	-	-	−−	0
Construction time	−−	-	0	+	+	0
Morphological impact on coast	−−	−−	−−	+	-	-
Waterdepth	-	−−	+	0	++	0
Wave influences	++	+	+	0	−−	+

Table 1.4: Relation between island-types and natural & structural conditions

- Construction takes some time, as large quantities of material are required. This applies also for the settlements of landfill and subsoil. Furthermore, polder islands may be difficult to demolish.
- Because of its large and massive volume, blocking of waves is considerable and consequently the morphological impact as well.
- Polder islands are not subject to movement and vibration under wave and wind loadings.

Bottom-fixed

Characteristics of bottom-fixed islands:

- When large areas are required the dimensions of the structure increase. Nevertheless, till a certain limit, enlarging the area does not induce problems. Also, larger waterdepths can be reached using this type.
- Because the foundation on the seabed requires preparation, the costs increase. In addition bottom-fixed structures consist mainly out of steel and concrete, which are relatively expensive materials.
- The structure can partly be prefabricated, the top structure, however, has to be built in-situ.
- Blocking of waves is considerable.
- Bottom-fixed islands are subject to moderate vibration from wave and wind loadings.

Jack-up

Characteristics of jack-up islands:

- Jack-up structures are designed to bear moderate loads on small surfaces. In other words increasing the area would request a severe modification of the structure. This fact makes the jack-up less suitable for large areas. An option would be to place more jack-ups next to each other to increase the area.
- If the area is small costs are in range, but if the area grows this type becomes very uneconomical.
- Because the total cross-section of its legs is small, the effects on waves are relatively small, resulting in a minor effect on longshore transports.
- Waterdepth, up to 90 - 150 m.
- Jack-up islands are influenced by waves forces on the legs of the structure, and by wind as well, because the top structure levels far above the sea surface.

Floating

Characteristics of floating islands:

- The possible area is almost unlimited, as is the waterdepth. Anchor forces grow with increasing area, and require suitable ground conditions.
- Some kind of protection is required to reduce the effects of waves. For example a breakwater, which increases total costs.

- These islands lend themselves to prefabrication and mass production, and are relatively easy to remove at the end of their operational life.
- The blocking of waves by floating objects is slightly less than with structures connected to the bottom. Still the influence on longshore transport is severe. item Floating islands are subject to vibration from wave and wind loadings. This can be reduced by a protection in front of the structure (breakwater) or by lowering the structures center of gravity.

Piling

Characteristics of piling islands:

- The number of piles grows if the area is increased.
- Piling structures consist mainly out of steel and concrete, which are relatively expensive materials.
- The top structures can be built using prefabricated elements. The piles are driven into the seabed and the prefabricated elements are placed on the piles. Construction time is moderate.
- Because the total number of piles is large, the effects on waves are considerable. This will consequently result in a considerable effect on longshore transports.
- Waterdepth, up to about 90 m.
- Piling islands are influenced by waves forces on the piles of the structure, and by wind as well, because the top structure levels far above the sea surface. In case the top structure is partly submerged, additional wave forces have to be encountered.

Users

Table 1.5 gives the pro's and con's for the man-made island types in relation to the ten types of users.

Ocean mining & Exploration

Waterdepth has a major influence in the choice of a type of structure. A significant feature of mining platforms is the small area on which the equipment is placed. When greater depths are considered reclamation is unfavourable, a relatively large volume would be needed in proportion to the surface-area, but for limited waterdepths the reclamation or polder types can be very suitable. For example in the Canadian Beaufort Sea, reclamation islands were used

Parameter/type	Reclamation	Polder	Bottom-fixed	Jack-up	Floating	Piling
Ocean mining& Exploration	0	0	++	++	0	+
Recreation	++	+	0	-	++	0
Waste handling	+	+	0	-	0	0
Mariculture	++	-	--	0	--	+
Temporary&Aiding	+	++	0	-	-	-
Energy generation	+	+	+	+	+	+
Port	++	-	0	--	+	+
Airport	++	+	-	--	+	--
Waste island	++	++	+	--	-	--
Industrial island	++	0	0	--	+	0
Complex city	++	+	0	--	+	0

Table 1.5: Relation between island-types and users

in order to withstand the heavy loading by ice and enabled work for 12 months per year. Bottom-fixed islands are very appropriate for mining, the interior can be used as a storage for oil and gas. Also the jack-up type can be put on the job temporarily. The floating type can work in greater depths (using automatic positioning systems, to maintain in position) but must stop their production in bad weather conditions.

Recreation

A large variety of recreational purposes exists, therefore it is hard to give judgement about the suitability of the island types. Nevertheless, the types which can provide large areas are given a higher ranking.

Waste handling

To function properly, a waste handling island has to be positioned near major producers of waste, being either urban areas or industrial zones. This probably means that waterdepths will be moderate, because of its location close to shore. Reclaimed and polder type islands will be most suitable.

Mariculture

Mostly small and cheap area are required. In addition, the island type must be suitable both for surface activities and underwater activities (farming of fish).

Temporary&Aiding

In general, areas are small for temporary islands and aiding islands. When the island is used as a construction-dock or a launching-base for a tunnel-drilling-machine [7] the reclamation type is most suitable. When closure of channels is involved, the main property of the island is to resist the currents. Bottom-fixed and reclamation are considered for in this case.

Energy generation

The immense variety of possibilities for energy generation on an island complicates a good choice of a structural island type. A windmill park requires a total different lay-out than a Pomp Accumulation Plant, for example. In principle, all six structures can be applied for energy generation, like table 1.5 shows.

Port

A port demands an unconstrained access to its berths, and in most cases a large area for logistic operations as well. All structures are eligible for deepwater access; the reclamation and polder types require a vertical sea defence or a protected harbour instead of slopes on the island-side were transhipment is considered. A polder is less appropriate, because goods for transhipment has to be raised and lowered to a higher or lower level inside the polder, which takes extra time. A jack-up is uneconomical when large areas are needed, the same holds for the bottom-fixed type, however, to a lesser extent.

Airport

An airport requires a considerable amount of space, and is preferably situated some distance offshore to minimise noise pollution. One should keep in mind that the distance offshore is also limited by the transportation time from the airport to the main-land. An airport built on a reclaimed island has already proved to be technical possible, viz. Kansai International

Airport, Nagasaki Airport and Chek Lap Kok, as a large area can be constructed. This also holds for the polder type, but some minor hindrance can be expected from the dikes or other structures surrounding the lower positioned airstrips. Airports need to be constructed close to inhabited areas, so waterdepths will probably be limited. A limited waterdepth makes the reclaimed and polder types very suitable and costs minimal. Other types will be more expensive in limited waterdepths. Especially the jack-up type, since more jack-ups are needed to require a sufficient large area. The bottom-fixed method requires expensive materials and such will not be economical.

Waste island

Waste will be used as construction material for these islands. Reclamation and polder islands make excellent use of this waste as fill material. The interior of a bottom-fixed structure can be used as waste storage as well, but will be more expensive. The remaining three types do not offer great possibilities for the storage of waste.

Industrial island

Considering an industrial island certainly needs a (deepwater) port, the same remarks count for this multi-purpose island as for a port-island. Moreover this type of islands requires a large amount of building-space. Because of the large area that is required the difference in level mentioned for the polder becomes less important. An extension of a floating structure requires the same price per square metre as the first square metre. On the other hand extended square metres of reclaimed islands are cheaper, because the sea defences account for the majority of the costs (see also chapter 4).

Complex city

A complex city requires a great area and a fast transport connection to the urban areas on the mainland. The parameter area is the main factor determining the ratings in Table 1.4 for the purpose of a man-made island as a complex city.

1.3.3 Restriction to detail reclaimed artificial islands only

Reading paragraph 1.3.1, unconsciously preference could be given to a certain type of island. This should be avoided, and the choice for a type of island should depend strongly on its final purpose, as described in paragraph 1.2.

This report chooses to investigate *reclaimed artificial islands* only. The following reasons can be brought up. Reclaimed islands are cheap alternatives to other types of man-made islands, provided that they are employed in restricted waterdepths (refer to table 1.3). Fill-material is relatively cheap, especially when the area of the island increases. Furthermore, larger areas can be realised with minor additional effort. Another reason is the fact that reclaimed islands are the most employed man-made islands so far. This implies that they have proven their worth and experience is available (see references).

Although polder islands are related to the reclaimed islands, they are not discussed in this report. The reason is not of a technical nature, but results from its social viability. Most countries in the world object to idea of placing expensive structures and even human lives on surfaces below sea level. Assumed risk of flooding and unfamiliarity are the main reasons to drop this type of island, and certainly not any kind of technical limitations.

1.4 Problem description

The initiation for this report was given by C. Stigter M.Sc, of Boskalis Westminster NV, and also member of the final examination committee. As an engineer, he was, and still is involved in the design of several artificial islands. Within the framework of mister Stigter's work at Permanent International Association of Navigation Congresses (P.I.A.N.C.) this report could provide some added value. The bulletin published by P.I.A.N.C. has discussed artificial islands several times already.

The problem description in this paragraph and the research purpose in the next are given in a general way, in order to cover the contents of this report. Both chapter 3 and chapter 5 have their own problem description and research purpose, because the subjects of these chapters are too detailed to describe in this introductory chapter.

The problem description is defined as follows:
Many man-made islands were constructed in recent times all over the globe. However, some islands were never built and only exist on drawing boards. A catalogue of these islands was never made. Furthermore, useful time is wasted in the initial phase of the design, specifically with regard to the assessment of sea defence structures. Useful guidelines to help designers and constructors in this initiation phase of artificial-island design do not exist.

1.5 Research objective

Designers of artificial islands should be able, with the aid of this report, to make a first assessment with regard to sea defence structures. Moreover, they should be able to estimate the short term consequences of a man-made island on longshore transports. Finally, they should be provided with some design limits, collected from literature.

1.6 Problem approach

The guidelines, which will help future designers are obtained in the following way.

1. First a literature research was carried out.
2. Using the literature, the general introduction was made, combined with the choice for reclaimed artificial islands.
3. This research resulted in several (economic) limitations in the design and construction of man-made islands. Furthermore, a collection of severe problems related to artificial islands was formulated.
4. Two of these problems, which are mostly related to the field of coastal engineering, are more detailed in this report. Namely, the choice of different types of sea defence structures and the influence of man-made islands on longshore transports.
5. Together with an extensive list of references, these items form the lion's share of this report, and -more important- fulfil the research purpose as formulated in paragraph 1.5.

The guidelines were drawn up using knowledge, collected from literature. This knowledge was extended, by making use of existing theories and methods, especially in the field of sea defences and morphology. This resulted in guidelines with regard to economic evaluation of sea defence systems, and guidelines that provided an indication of the short term influence of artificial islands on longshore transports.

Chapter 2

Experience gained from constructed islands

2.1 Introduction

During the initiation-phase of this project, a thorough literature research was executed. Two main purposes were aimed at, viz. to come up with a collection of artificial islands, and to investigate the main problems involved in the construction and design of man-made islands. The first purpose resulted in a summary of world-wide constructed and planned islands. Examining these islands, already reveals some guidelines, which are discussed in paragraph 2.2. The latter purpose, discussed in paragraph 2.3, also serves the realization of guidelines. Knowing the main construction and design problems, a more useful correspondence can be made between the presented guidelines, and the desires and demands of this report's users.

2.2 Similar structural properties of artificial islands

Appendix A shows a collection of man-made islands, categorized by continent or region. Both constructed islands and islands, which never passed the design phase are included in the collection. The islands are sorted by construction or design period. Moreover, for each island several structural features are mentioned, such as waterdepth, the surface area and the distance offshore. The column *Code* uses abbreviations to indicate what type of user exploits the island. The same abbreviations are used, as stated in table 1.1 and table 1.2.

Examining the structural features of the islands, guidelines can be given about the maximum waterdepths to employ reclaimed islands in, the relation between surface area and the distance offshore, etc. Some of the data presented in appendix A is incomplete. Nevertheless, if it was possible to draw a relation between features, this was done. The possible relations are displayed in appendix B, subdivided for built and planned islands. Since the data in appendix A does not include all existing man-made islands, the relations drawn in appendix B should be considered as indicative. The curves, drawn in the graphs indicate either upper-boundary limits or a generalised relation between the waterdepth and the distance offshore.

Below, the guidelines resulting form the collection of artificial islands and their relations are summed up.

- In general, economical waterdepths are restricted to $\pm$ 30 m
  Appendix B.1 and appendix B.3 show, that built islands hardly exceed waterdepths of 30 m. Though artificial islands are planned in waterdepths up to 200 m, the majority of planned islands still does not exceed the 30 m limit. The prediction stated in table 1.3 is now verified. However, if the profits of employing an island in deeper water are high enough, the waterdepth restriction can of course be abandoned.

- Generally, an increase in island size involves a decrease in waterdepth
  In appendix B.1 and appendix B.2 imaginary curves have been drawn, to indicate the upper-boundaries. The observed relation is a logical result form the desire to construct as economical as possible. Shallow water construction, requires less landfill and material for the sea defences, consequently a cheaper design results. The economical advantages increase even more for larger areas.

- Artificial islands are preferably built close to shore (< 5 - 10 km)
  Built islands are mainly deployed at distances offshore of less than 5 to 10 km. Future islands are, however, already planned at distances offshore of up to 50 km. One reason to keep distances offshore short, is the costs involved in the approach facilities. Furthermore, the relation between waterdepth and distance offshore is obvious.

- Generally, an increase in island size involves a decrease in the distance offshore
  This relation is shown in appendix B.5 and appendix B.6, upper-boundary curves have been included as well. Because of the relation between waterdepth and distance offshore, the remarks stated in the second bullet are in force as well. Greater distances offshore, indicate flatter bottom slopes in most cases.

- A minimal amount of polder islands have been built
  In the collection of appendix A limited polder islands can be found. Although the polder islands shows great resemblance with reclaimed islands, investors seems reserved to put money into these projects. Most countries in the world object to the idea of placing expensive structures and even human lives on surfaces below sea level. Assumed risk of flooding and unfamiliarity are probably the main reasons. Some polder islands have been built in the Netherlands.

2.3 Main construction and design problems

Literature published on artificial islands over the years, brings to light problems, in the construction and design of artificial islands. The problems range from very specific to general and from severe to minor. Since one of the goals of this project is to present guidelines, the most common problems were selected, some of which are detailed in this report. Four main problems are summed up below, in a random order. Two of the mentioned problems are more detailed in the continuation of this report. These two problems relate to the field of coastal engineering, while the other two problems are more related to the field of soil-mechanical engineering. All four problems are briefly described in paragraphs in this chapter.

Earthquakes	Damage to sea defences
	Liquefaction
	Tsunamis
Morphological impact	Erosion and accretion of coastline
	Erosion and accretion of island
	Consequences of soil removal at borrow sites
Overall cost reduction	Choice of sea defence structures
	Equipment
	Obtaining fill-material
	Distance to borrow site
	Availability of fill- and armour-material
Settlements	Undesirable settlements
	Time involved, before required settlements are reached

2.3.1 Earthquakes

Depending on the region where the island will be built, earthquakes can cause serious problems. Precautions must be taken to minimise the effect of earthquakes. Influences of earthquakes on artificial islands are subject of a soil-mechanical study and therefore no part of this report, however some remarks are given.

Earthquakes can cause damage to both the sea defences and the structures placed on the island. Three types of failures are to be distinguished:

- Soil liquefaction
- Horizontal acceleration or lateral movement of ground
- Vertical acceleration

To avoid damage, especially liquefaction some measures can be taken:

- Apply ground improvements:
  - Sand drain works
  - Sand compaction piles
  - Deep mixing
- Use of coarse material, which can more easily dewater, thus the change of liquefaction is much smaller.
- Removal of soft deposit layers. The thicker the soft deposit, the greater the ground shaking will be, caused by the effect of amplification.

Sea-quakes are able to generate tsunamis. American experience shows that waves induced by tsunamis only reach heights of a few metres along most East and Gulf Coast areas. The wave height is most likely less than the design wave height ([8] *third part, p21*). Nevertheless, the wave period can differ severely, resulting in a more severe wave impact.

Research data about earthquakes and ground improvements are among others mentioned in [9], [10], [11], [12] and [13].

2.3.2 Morphological impact

An artificial island placed some distance offshore induces a wave energy low zone behind it. In this zone, wave heights will be lower than would be the case in a situation without the island. Due to the reduced wave heights longshore sediment transports are reduced as well, and accretion will occur. Extra expenditures are required to overcome the effects due to accretion and erosion. The morphological aspects of man-made islands are discussed in chapter 5.

2.3.3 Overall cost reduction

The total construction costs of artificial islands depend both on the costs for the sea defences and the costs for the landfill. The construction costs for sea defences are detailed in chapter 3. Extremely large quantities of material are needed to fill an island. Even when a polder-island is considered, still a great potential of landfill has to be found. The economical distances from the island-site to the borrow-area, depend largely on the used equipment, size of the island and workable periods per year. A short description of the dredging-process is given in chapter 4. The same chapter discusses the relation between total construction costs and the surface area of the island as well.

2.3.4 Settlements

These kind of problems are part of a soil-mechanical study as well, and are not studied in this report. Depending on the original bottom-soils, undesirable settlements can occur, e.g. Kansai International Airport [11], [14] and [15]. Removal of low quality layers before construction is recommendable. Secondly, the settlements have to achieve a certain level before structures can be placed on the island. To speed up these settlements, ground improvements can be applied as well (see paragraph 2.3.1). Apart from the already mentioned literature, some information is also mentioned in [12], [13], [16] and [17].

2.4 Remarks

This chapter and the introductory chapter, have introduced the reader in the various aspects of reclaimed artificial islands. The limits in design and construction of this island-type were mentioned. These limits are applied in the continuation of this thesis. For example, in chapter 3 an economic evaluation is made for sea defences. The maximum waterdepth in this evaluation is set to 30 m, according to the value found in paragraph 2.2. Furthermore, the surface areas used in the morphological study are related to realistic areas, found in appendix A.

Chapter 3

Design of the sea defence structures

3.1 Introduction

Generally, the largest part of a reclaimed island consists of landfill. Landfill is generally composed of relative fine material, for example sand. If the landfill would be just dumped at a certain location offshore, without further paying any attention to it (not applying any nourishment), it would soon be eroded and transported by the movements of waves and currents. To maintain the landfill at its original location, several measures can be taken. The total of measures needed to prevent the landfill from eroding, is called the sea defence system.

Sea defence structures for artificial islands can be distinguished in three types:

1. Soft defence systems
2. Hard defence systems
3. Combined defence systems.

A soft defence systems consist solely out of fine, mostly natural material, like sand and gravel and is suitable for moderate wave-climates. After the construction of a soft defence, nature reshapes and erodes the defence system under the influence of morphological processes. A prediction of the magnitude of these processes is made in advance. By means of periodical beach nourishment's, the defence systems can be kept operational. Under moderate wave attack the capitalised costs for construction and maintenance may prove to be an economical alternative. An example is an artificial beach constructed out of sand or gravel.

A hard defence system derives it protective use from its overall inability to move. This type of defence literally forms a boundary between the landfill and the attacking waves. The majority of its lifetime costs are determined by the initial construction costs. A hard defence is very suitable in locations attacked by severe wave action. Examples of hard defences are a rubble mound defence, a caisson-type seawall and sheet-piles.

A combined system couples the two previous systems. To reduce the amount of material used for nourishment hard structures are added. For example groines can be erected along the surroundings of the island or a detached breakwater can be placed in front of the island.

This report only discusses hard sea defence structures. Using these defences, islands can be constructed in deep water and in more severe wave-climates.

Two types of sea defences are described in the following paragraphs, viz. the rubble mound sea defence and the caisson-type seawall. These types have proven to resist severe wave attack and have been used widely in the construction of artificial islands. At the end of the chapter guidelines are presented, when to use which type of seawall.

3.2 Rubble mound sea defences

A rubble mound sea defence consists of several layers, most of which are constructed with quarry (rubble) stones. The outer layer, called the primary armour layer, must be designed to withstand the expected wave attack. And as a general rule, each layer of the rubble mound sea defence must be constructed in such a way that the adjacent layer of finer material can not escape by being washed through its voids. A typical cross-section of rubble mound sea defence is shown in figure 3.1. The most significant elements of the cross-section are dealt with below.

Filter layer

A bottom filter layer prevents the erosion of fine materials, such as sand, from underneath the sea defence. And consequently the bottom filter layer guards the overall stability of the sea defence. A filter layer is most necessary when the bottom consists of easily eroded material. If the bottom is rock, a filter becomes superfluous. The bottom filter layer can be constructed of small quarry stones. When the quarry run is too large, the bottom material too fine or the wave-climate too severe, special woven mattresses can be applied. These mattresses can be made sandtight by using woven geotextiles.

Toe structure

The function of the toe is to support the lower portion of the primary armour layer and to protect the filter from direct wave attack. Furthermore, a toe constructed similar to the one drawn in figure 3.1, eases the construction of the sea defence. The berm of the toe enables a certain tolerance in placing of the armour elements. The weight of the toe-stones is determined primarily by the waterdepth in relation to the wave height. One can understand that the weight diminishes with greater depths.

Dikes

These small dikes are made out of quarry run. Quarry run consists of the finest parts in the total grading of a quarry output. The dikes have a height of about 3 to 4 metres. The purpose of these dikes is twofold. Firstly they serve as core for the sea defence, in other words they prevent the landfill from washing out through the voids of the secondary and primary armour layers. And secondly, during the filling process of the island landfill is dumped or pumped between these surrounding dikes.

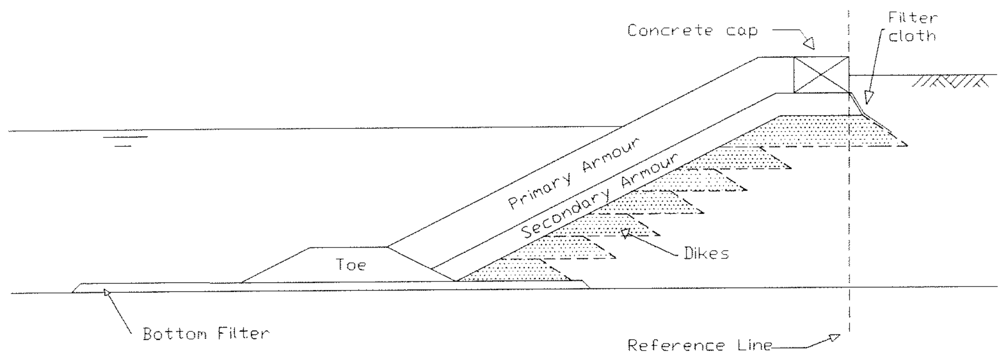


Figure 3.1: Specific elements of rubble mound sea defence

Concrete cap

A concrete cap can be placed in the superstructure. The cap provides support for the layer of armour elements. Furthermore, the cap can provide access for vehicles (cranes for repair-works) and accommodation for pipelines (industrial island).

Filter cloth

Although the sea defence is to be designed in such a way that overtopping will be limited, still some amount of water has to be discharged to the open sea. This water can come also from either rain-fall or spray under unfavourable wind-conditions. A filter cloth is applied for the coarser parts of the sea defence (secondary armour), to prevent washing out of landfill.

Reference line

This imaginary line indicates the boundary between the actual sea defence and the island's landfill. At the island side of the reference line, it would be preferable to construct a road first, and start the actual buildings somewhat more inland. In that case, overtopping and spray can be easily dealt with.

Secondary armour

This layer is built up out of quarry stones. The secondary armour acts as a transition between the primary armour layer and the core material or the dikes in this case. The stone size of this secondary armour layer depends on the weight of the primary armour elements. Sometimes the secondary armour layer has to be built up of two layers, different in stone size.

Primary armour

The primary armour layer consists of material, which must be able to withstand the designed wave-attack. This material could be, just like the rest of the sea defence, quarry stones. Quarry stones are relatively cheap. But a disadvantage of a quarry stone armour layer under severe wave attack, is the enormous weights required as well as the required thickness of the outer layer. Therefore artificial concrete armour units have been developed over the years. Concrete elements may prove useful in areas where only small or too few large quarry stones can be produced. A collection of types of concrete armour units is shown in figure 3.2. Only four types of armour units are used in this report. All four types are listed below, and for each type a short description is given.

1. Quarry stone armour
   At sites where the quarry is relatively nearby as well as in shallow water sites, a rubble mound sea defence armoured with quarry stones is an economical alternative. Because the stones are obtained by blasting at a rock-quarry, sharp angular elements result with a natural interlocking capability. Nonetheless, this interlocking capability is less than with certain concrete armour units.

2. Dolosse
   Dolosse are anchor shaped concrete armour units, without any reinforcement, designed to interlock with each other. Due to their high interlocking capability higher waves can be withstood compared to other armour elements, having the same weight. The dolos was first developed in South Africa, in 1963. During the late 1970's and early 1980's a number of dolosse breakwaters failed. Resulting from these failures, research was initiated to prevent future failures. Below, some measures are stated to minimize the risk of dolosse-failure:

   - Placement
     Attention should be paid to accurate placement. If the dolosse are placed in a selected order and in a double layer, their working is more guaranteed and the risk of failure is reduced. Random placement should be avoided at all times.
   - Damage coefficient
     Early values for dolosse damage coefficients K_D were in the range of 20 - 25. The symbol K_D indicates the stability of the armour elements, see appendix C. Present day K_D-values used in Hudson-formulae amount to 15.8.
   - Fluke-Shank intersection
     Several papers have been published on failure-mechanisms, for example [19], [20] and [21]. These papers identified the fluke-shank intersection of the dolos as a region of structural weakness. Over 80% of the dolosse breakages were found to originate from this point. From tests it was concluded that the static and dynamic tensile stresses at the fluke-shank intersection can be reduced by more than 60% by incorporating a large fillet extending to mid-shank. Although the hydraulic

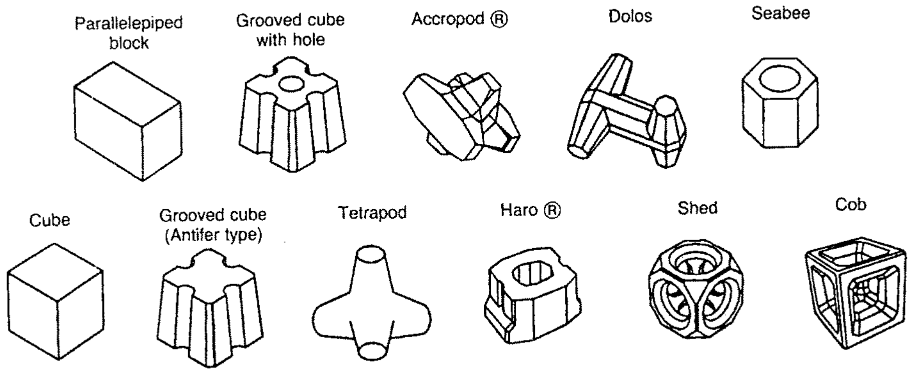


Figure 3.2: Examples of concrete armour units [18]

stability of the dolos is expected not to be influenced by the alteration, this has still to be tested.

- Maximum weight of dolosse
  Because dolosse are slender elements, it is advised not to apply them to their maximum available weight. At heavy and slender concrete units, shrinkage-cracks may occur due to uneven cooling and hardening of the concrete. Consequently, heavy and slender concrete armour units can break (brittle) at minimal cross-sections under severe wave attack. For that reason, dolosse (and tetrapods) are applied to a maximum weight of 20 tons in this study. Moreover, special attention and care for the hardening-process is required during the fabrication of these type of concrete units. More information about the maximum weight of dolosse is found in paragraph 3.4.2.

When attention is payed to the above mentioned measures, dolosse provide great interlocking with a greatly reduced change of failure. For that reason they have been selected in this thesis.

3. Modified cubes
   This element is evolved from the cube. Many different models of modified cubes have been developed. But whatever modifications have been made, all models still resemble a cube. Modifications differ from holes to additional caps, but all are applied to increase the damage coefficient value and to save on the amount of plain concrete. The first modified cube was applied in the United States around 1959. Damage coefficients are in the range of 6 to 7. In this paper the modified cube, as mentioned in *Shore Protection Manual 1975* is used. At its top side, this cube has caps on every corner, and at its bottom side a square notch is applied (this type of modified cube is unfortunately not shown in figure 3.2).
   Modified cubes are relatively easy to make, with only a limited need of shuttering. Consequently, modified cubes can be considered as a standard in concrete armour elements.

4. Tetrapods
   Tetrapods are plain concrete armour units consisting of four arms projecting from a central hub. The angular spacing between all arms of a tetrapod is the same. The tetrapod was developed by SOGREAH in France in 1950. Damage coefficient for tetrapods range from 7 to 8. The same can be said for tetrapods as for dolosse, they should be placed in a selected order, and not be applied up to their maximum available weight [22]. When the measures are taken into account, a tetrapod makes a well usable type of armour.

Above mentioned armour units should be placed in a double layer to assure their proper working. However, some concrete armour units are claimed to be placed in a single layer, e.g. accropods and core-loc. The individual units are more expensive, but it is obvious that the total costs are reduced when elements are placed in a single layer. The question may arise, why these elements have not been chosen in this report. Not enough information and experience is available to come up with a considered examination of these elements. Accropods have been applied only recently, and extensive information about failure-mechanisms is not available.

3.3 Caisson-type seawalls

Caisson-type seawalls, also called vertical wall-type seawalls, generally consist of a caisson placed on a mound of quarry stones. In some cases the exposed side of the caisson is protected by concrete armour units. The inner part of the caisson is divided into cells. These cells can be partly or totally filled with sand or some other material to increase the stability of the caisson. The caisson itself is constructed out of reinforced concrete. Three different caisson-type breakwaters can be distinguished, as can be seen in figure 3.3. Similar structures can of course function as sea defences for artificial islands as well.

In areas where the quarry stones are scarce, sub-type (a) may be a good alternative. To reduce costs, the caissons can be placed on a rubble mound, a suchlike structure is called a vertical composite breakwater. If breaking waves are expected, the caisson needs to be protected by a sloping mound of armour elements, this is called the horizontal composite breakwater.

Experience with caisson type breakwaters have been gained in Italy and especially in Japan. Most of the rules of thumb and the design-equations, used in this report, have been derived from Japanese literature. In Japan, the caisson type, particularly the caisson type composite breakwater, has almost always been used for larger breakwaters since about 1900 when caissons were first used as breakwaters. They were used in Japan because of the often encountered difficulty of obtaining large quantities of stones , the reliability of work performed by the caisson type and the need for speedy construction.

Vertical composite breakwater

This type becomes attractive when the waterdepth increases. However, it has one potential danger. The mound can cause waves to break against the caisson, resulting in high impact forces on the caisson. Many caissons have failed due to these forces in the past. To prevent excessive forces the mound should not be too high and the width of the seaward berm should not exceed 1/20 * L. L is the length of the steepest design wave which can occur at the breakwater. Though, if the mound is too low the weight of the caissons will fail to be evenly dispersed.

Horizontal composite breakwater

If breaking waves are expected, an armoured mound can be placed in front of the caissons. The mound must absorb and break part of the wave energy effectively. Because of this protection the impact forces on the caisson are greatly reduced.

3.3.1 Measures to increase effectiveness of caisson-type seawalls

The design of caisson-type breakwaters over the years, has resulted in several measures to reduce the wave loads or to reduce material, some of these measures are summed up below.

Sloping top

A caisson constructed with a sloping top is called a *Hanstholm*-caisson, named after the place in Denmark where this type of caisson was first used in the 1960s. The use of a sloping top from about S.W.L. to the crest is very effective in almost eliminating impact forces from breaking waves and in reducing the load from non-breaking waves. This solution, however,

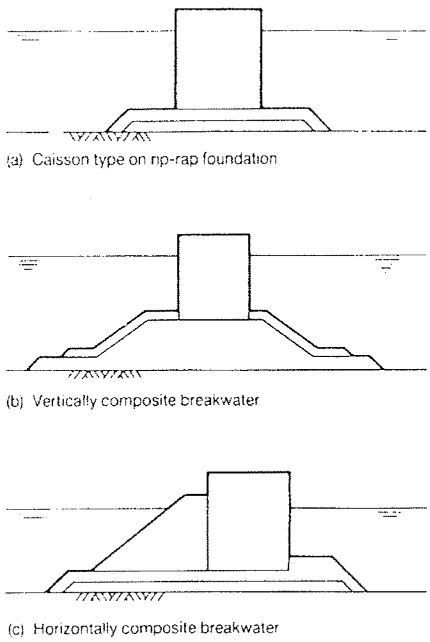


Figure 3.3: Caisson-type breakwaters [23]

is not feasible where large water-level variations occur because the sloping front must start approximately at S.W.L. Normally, the slope is constructed having an angle of 45 degrees. In general, the wave transmission over the sloping top caisson is larger than that over the ordinary vertical caisson, when the crest height remains the same. Secondly, a sloping top increases the effective weight of a caisson. When a wave runs up the sloped part, its resultant force points downwards, instead of horizontally when the wall would be vertical, and thus increases the resistant force against sliding. At the same crest height considerably more overtopping is encountered with a sloping top caisson.

Perforated front wall
A vertical wall almost totally reflects the incoming waves. If reflection is undesirable or if wave forces need to be reduced, the caisson's front wall can be slit or perforated and an interference chamber is build behind it. Some wave energy is dissipated when the waves pass through the holes or slits in the front wall. Consequently the reflection and wave force is reduced. The width of the interference chamber is about 0.15 to 0.25 of the local wave length for optimum reflection absorption. Two or more interference chambers may be used to increase dissipation. For longer wave lengths, these chambers require large widths of the caisson.

Wave energy reducing structures
Structures such as (submerged) breakwaters can be placed in front of the caisson. This way the impacting forces on the caissons are reduced, and the caissons can be constructed less heavy.

3.4 Guidelines for the use of sea defences

The final objective of this chapter is to come up with general guidelines for the use of sea defences. At the end of this chapter suggestions are given, to help designers in their choice between sea defences, dependant on certain natural conditions. Of all possible types of sea defences only the rubble mound and the caisson-type seawalls are discussed, because of considerations already mentioned in the introduction of this chapter. This paragraph deals successively with the research objective, and the design starting-points.

3.4.1 Research objective

When the local wave conditions at the island are known, this report helps designers in their choice between defence structures. The users of this report have to know the waterdepth at the toe (including additional rise due to wave setup ,etc.) and the wave height at this point. The choice between sea defences is based on the desire to construct the cheapest possible cross-section.

To be able to make a considered deliberation between the types of sea defences, the same starting-points have to be assumed for each design. For that reason several restrictions and assumptions are enforced, which form the basis for the final designs. These considerations are discussed in the following paragraphs.

3.4.2 Design restrictions

Natural restrictions

This report uses the local significant wave height H_s, given by the user, to come up with the most economic type of sea defence. Moreover, the design of rubble mound sea defences and caisson-type seawalls depends on the wave period as well. The ratio between wave height and the wave period is known as the wave steepness s.

$$s = \frac{H}{L_0} = \frac{2\pi H}{gT^2} \tag{3.1}$$

Fortunately, the range of steepnesses is limited in nature. Steepnesses steeper than 0.05 or flatter than 0.01 are very unlikely in nature. Thus, if only the local wave heigths generated by deepwater waves having a steepness of 0.01 and those of 0.05 are examined, the two extremes are accounted for. Both steepnesses have their own wind induced origin, as stated below.

Wind induced waves can generally be devided into two groups:

- Seas: These waves are generated by a local storm, and waves reach the beach in nearly the shape in which they are generated. These waves are steep, a typical steepness is in the range of 0.05.
- Swell: If waves are generated by a distant storm, they may travel through hundreds or even thousands of miles of calm areas before reaching the shore. Wavelengths are long, and typical steepnesses are in the range of 0.01.

Of course, seas and swell can occur at the same time and location. And an intermediate wave can appear as well. However, this report is limited to the two basic conditions.

Now, that the steepnesses are known, realistic assumptions need to be made for the significant wave heigths. Another limited condition by nature can be used, namely the wave period. Wave periods over 15 to 20 seconds for significant wave heigths require very large fetch-lengths and high wind-speeds. If one assumes that 20 seconds is a maximum for significant wave periods, the maximum deepwater wave heigth can not be too high, according to equation 3.1. A maximum wave heigth of 6.0 m is assumed for waves having a wave-steepness of 0.01. Steep waves, on the other hand, can reach high wave heigths. A maximum wave heigth of 12.0 m is assumed for seas.

Furthermore, the maximum wave height is limited by the waterdepth. A general rule of thumb, widely used in coastal engineering, is applied here as well.

$$H_s = \begin{cases} H_{s0} & \text{when } H_{s0} \leq 0.5 * \text{h (h = waterdepth)} \\ 0.5 * h & \text{when } H_{s0} > 0.5 * \text{h} \end{cases} \tag{3.2}$$

3.4.3 Design assumptions

These assumptions are enforced by the designer himself. According to the use for which an artificial island is constructed, the design has to answer certain design-limitations. Furthermore, assumptions are made to overcome some uncertainties, like local soil conditions, availability of material, etc.

Natural assumptions

- Bottom-slope: The bottom-slope indicates the economic possibilities to construct a man-made island. If the slope is steep, a certain depth is reached at a distance closer to shore compared to a flat slope. Consequently, the required surface area of the island needs to be extended, sooner, parallel to the shore-line to obtain this area. Furthermore, the bottom-slope determines at which distance offshore waves start to break. Again a flat slope is more attractive, since the surfzone is extended further offshore and reduced wave heigths are expected over a larger area compared to steep slopes. Goda's method, used in the computations of caisson-type seawalls, requires a given bottom-slope. For the design of rubble mound sea defences the bottom-slope is unimportant in this report, and even for the caisson computations its influence is limited. Nevertheless a uniform bottom-slope of 1 : 250 is chosen.
- Local soil conditions: It is assumed that the soil layers, on which the defence structures will be build, have enough bearing capacity and no ground improvements are necessary. Settlements of the soil layer and cavities in the underlayers are excluded as well.
- Waterdepth and Sea Level: The local depths and wave heights in the sea defence-calculations are measured at the front of the toe, rubble mound or horizontal protection layer in case of horizontal composite seawalls. Furthermore, the depth already accounts for predicted sea level rise by tides, storm conditions, wave setup, etc. The corresponding level is called Sea Level (S.L.), thus:

$$\text{S.L.} = \text{M.S.L.} + \Delta\text{h (tides, storm, wind, etc.)} \tag{3.3}$$

Design-limitations

- Minimal overtopping allowed: Because costs per square meter are high for an artificial island, it should be possible to place structures directly behind the sea defences. However, some overtopping or spray can occur, and a drainage system is therefore required. For rubble mound sea defences, overtopping is related to the wave run-up. If the crest-height is higher than the maximum wave run-up, no overtopping will occur at all. However, this is an uneconomical condition, therefore it is normal practice to indicate a maximum exceedance level for the run-up. In this thesis an exceedance level of 2% for the wave run-up on rubble mound sea defences is allowed. This means that 2% of the approaching design waves reach a higher wave run-up level than the calculated maximum run-up level. For caisson-type seawalls the crest height should be constructed in such a way that minimal overtopping occurs. A system could be placed to discharge the overtopped water, the spray and the rain-fall. In case roads are required on the island, these roads could be built as a boulevard and at the same time used to face the problem of overtopping water.

- The total costs for each cross-section are determined, under the condition of perpendicular approaching waves. Perpendicular waves cause the maximum force on a sea defence.

- Concrete armour units: Concrete armour units, except modified cubes, should not be used if their weight exceeds 20 tons. This limitation is enforced, because heavy concrete units can break (brittle) under severe wave attack. For example, the weight of a broken arm of a tetrapod is reduced to one fourth or one fifth of the original tetrapod's weight. Due to its reduced weight this arm will be moved by the waves. When the arm hits other elements these can break as well, resulting in further damage and in the worst case causing total failure of the sea defence structure. This disadvantage does not occur using modified cubes, because their minimal cross-section is much larger than that of the majority of the concrete armour units. Consequently greater forces can be withstood.

- Armour weight formulae: For rubble mound sea defences two types of formulae are used in this report. A formula by Van der Meer is used in case of quarry stone armour, the Hudson-formulae are used for concrete armour units. A more uniform outcome would result if the same formulae were used in all cases. However, no formula by Van der Meer could be found for dolosse, and the Van der Meer's formula for quarry stone armour gives more realistic results than Hudson-formulae for the same armour. The weight of the protective layer in case of a horizontal composite caisson-type is computed using the Hudson relation as well.

- Density of armour elements: The density of the quarry stones is assumed to be 2650 kg/m^3. The concrete elements are given a density in the same range, namely 2621 kg/m^3 (a standard value in *Shore Protection Manual 1975*). This kind of density for concrete elements (without reinforcements) can be achieved by mixing the concrete with heavier aggregates, like iron ore.

Economical assumptions

- Concrete armour units: A standard price for modified cubes is assumed of US \$ (b + 70) / ton. This is the all in price, for fabrication and placement. The overall costs for dolosse and tetrapods, however, amount to US \$ (b + 80) / ton. This is caused by the fact that the cost for fabrication and shuttering are more expensive as compared to modified cubes. A parameter b is introduced to indicate additional costs, for example extra transportation costs. For normal manufacturing conditions $b = 0$, which means that concrete armour units can be relatively easy fabricated. If the fabrication of concrete armour units faces some difficulties, this can be indicated by a value of $b > 0$ (e.g. remote area, high cement price, etc.).

- Quarry stones: A similar approach can be applied for quarry stones. For quarry stones a parameter a indicates the additional costs. The all in costs are mentioned in table 3.1. The original situation where a = 0, indicates a delta-region with a suitable quarry at a distance of several hundreds of kilometres away.
  Note: This distance implies that not the cheapest possible costs for quarry stones are assumed. In paragraph 3.5 the value for a is varied.

- Relative costs: A ratio (a - b) >> 0, indicates that additional costs for quarry stones will be more expensive than those for concrete units. And a ratio (a - b) << 0, indicates that additional costs for concrete units will be more expensive than those for quarry stones.

- The costs for a certain type of sea defence are determined per length metre. The reference line, as shown in figure 3.1 indicates the boundary between the actual rubble mound sea defence and the landfill. Costs for rubble mound sea defence are calculated as the total costs for all parts at the seaward side of the reference line. For caisson-type seawalls an imaginary vertical reference line can be drawn at the island-side of the caisson. However, the total costs for the mound should be included.

- Collective cross-section parts: Items, like bottom-filters, filter cloths and construction details, which are the same for all sea defences or contribute minor to the total costs are not calculated in the final costs of the cross-section.

3.4.4 Examples of calculations

The calculation methods as well as the interpretation of the restrictions plus assumptions can be found in appendixes C and D. In these appendixes, design examples for each type of sea defence are included as well. Because of the large variety of sea defences as well as the differences of natural conditions, calculations were made using a computer model, written in

Gradation	D_{n50}[m]	W_{n50}[kg]	[US\$/m^3]
0.0 - 500 kg	0.32	83	(a + 36)
250 - 500 kg	0.51	361	(a + 40)
500 - 1000 kg	0.65	721	(a + 42)
1000 - 2000 kg	0.82	1443	(a + 45)
1000 - 3000 kg	0.88	1821	(a + 50)
2000 - 4000 kg	1.03	2895	(a + 55)
3000 - 6000 kg	1.18	4328	(a + 63)
6000 - 9000 kg	1.41	7399	(a + 76)
9000 - 12000 kg	1.58	10428	(a + 86)
12000 - 16000 kg	1.74	13904	(a + 100)

Table 3.1: Gradations and prices for quarry stones

Tpascal. The results are shown in appendix E, shown as a multitude of tables. The tables should be interpreted using the remarks stated below. The conclusions of these calculations are given in paragraph 3.5.

- Although the costs per metre of sea defence are calculated using US Dollars, no currency related costs are presented in appendix E. All costs are related to the costs of a modified cubes sea defence at the same depth and under the same circumstances, because these elements can be seen as standards in armour units. They can be relatively easy constructed and placed. For each combination of waterdepth, wave height and wave steepness the costs of a rubble mound seadefence armoured with modified cubes is 100%. The costs for the other defence structures under the same conditions are related to these costs. In other words, the costs for a modified cubes sea defence are always 100%, at any point.

- No costs-relations were made for the horizontal composite caisson-type seawall. Design details for this type of seawall can be found in appendix D. For this type of seawall only a very general cost-indication was made. However, it can be said with some confidence that horizontal composite caisson-type seawalls are more expensive than vertical ones. For that reason the construction costs for a vertical composite caisson-type seawall can be seen as a lower-limit for the construction costs of a horizontal caisson-type seawall.

- All costs relations were made using a value of zero for both the parameters a and b. These parameters indicate the additional costs for rubble stones and concrete armour units. a and $b = 0$, means that concrete armour units can be constructed relatively easy and the quarry is at a distance of several hundreds of kilometres away from the proposed island site.

- The maximum waterdepth is 30 metres. Examining already constructed artificial islands, it becomes clear that waterdepths over 30 metres are uneconomical for the construction of man-made islands. See also chapter 2.

3.5 Conclusions and recommendations for the choice of sea defences

These conclusions result from the assumptions and limitations made at the beginning of this chapter as well as the graphs drawn in this paragraph. Some remarks, about the interpretation of the graphs are stated in paragraph 3.4.4. When the wave height, the waterdepth and the wave steepness are known, the most economic type of sea defence can be chosen, using the remarks below. The conclusions are categorized by wave steepness.

The graphs shown for every situation, indicate the relative construction costs of the sea defence-type compared to a rubble mound sea defence armoured with modified cubes. The relative costs are displayed in ranges of 20%. Consequently, a construction costs range of 90% - 110%, implies that the costs are relatively close to the costs of a modified cubes sea defence.

Remark: The graphs displayed in figure 3.4 through figure 3.11 sometimes show strange developments in the cost-curves. These developments do not represent the actual situation, but are caused by imperfect transitions between the formulae and methods used in the sea defences design. Despite of these strange developments, more natural costs curves have resulted. These are shown and elaborated in paragraph 3.5.3.

3.5.1 Wave steepness $s = 0.01$

Rubble mound sea defence armoured with quarry stones

No sea defence of this type can be constructed in waterdepths over 10 m, attacked by a significant wave height of 6.0 m. Examining figure 3.4, it becomes clear that at waterdepths shallower than about 3 metres (using $a = 0$), quarry stone sea defences are extremely economic. For increasing wave heights and outside the limited waterdepths the construction costs grow. This is explained by the fact that armour weights grow rapidly for increasing wave heights.

Rubble mound sea defence armoured with dolosse

In the examined range, the construction costs for dolosse sea defences amount to 50% - 90% of the construction costs of a modified cubes sea defence. If the waterdepth increases or the significant wave height enlarges the difference in costs grows. This can be simply explained, by the larger K_D-value of dolosse compared to cubes. Attacked by the same waves, the weight of dolosse is less than the weight of the modified cubes.

Rubble mound sea defence armoured with tetrapods

The construction costs for a tetrapod sea defence, are all in the range of about 95%. This result can also be explained by the K_D-values. Because the difference between the K_D-values of tetrapods and modified cubes is only about *1*, the limited reduction of costs is clarified. See also figure 3.6.

Vertical composite caisson-type seawall

At limited depths the construction costs for caisson-type seawalls are extremely high. The construction costs decrease at greater depths and for higher significant wave heights. Caissons require a minimal width to be stable, which is uneconomical at shallower depths. At these depths the dimensions of the caisson are not determined by the wave forces, but by minimal structural demands. Furthermore, they are constructed out of reinforced concrete which is also expensive.

3.5.2 Wave steepness $s = 0.05$

Rubble mound sea defence armoured with quarry stones

No sea defence of this type can be constructed if the significant wave height in front of the structure exceeds 8.0 m. Examining figure 3.8, the same conclusions can be drawn as for $s = 0.01$, namely at waterdepths shallower than approximately 3 metres (using $a = 0$) quarry stone sea defences are extremely economic. For increasing wave heights and outside the limited waterdepths the construction costs grow. This is explained by the fact that armour weights grow rapidly for increasing wave heights.

Rubble mound sea defence armoured with dolosse

In the examined range, the construction costs for dolosse sea defences amount to 50% - 90% of the construction costs of a modified cubes sea defence. If the waterdepth increases or the significant wave height enlarges the difference in costs grows. However, at depths over about 16 m and for wave heights exceeding about 8 m, the difference diminishes slightly again (this development is not visible in the graph). This is explained by the fact that the required weight of the dolosse in this range exceeds 20 tons for a slope 1 - 1.5, and a smaller slope has to be chosen. This consequently implies an increase in costs.

Rubble mound sea defence armoured with tetrapods

The construction costs for a tetrapod sea defence, are almost all in the range of 95%. This result can also be explained by the K_D-values. Because the difference between the K_D-values of tetrapods and modified cubes is only about *1*, the limited reduction of costs is clarified. See also figure 3.6. However, at depths over 12 m and wave heights over 8 m, the costs for a tetrapod sea defence suddenly exceeds the costs for modified cubes sea defences. Again the tetrapod's weight exceed 20 tons in this range and flatter slopes have to be chosen. If the wave heights grow even more, a tetrapod sea defence can not be constructed at all.

Vertical composite caisson-type seawall

At limited depths the construction costs for caisson-type seawalls are extremely high. These costs decrease at greater depths and for higher significant wave heights. Caissons require a minimal width to be stable, which is uneconomical at shallower depths. Furthermore, they are constructed out of reinforced concrete which is also expensive.

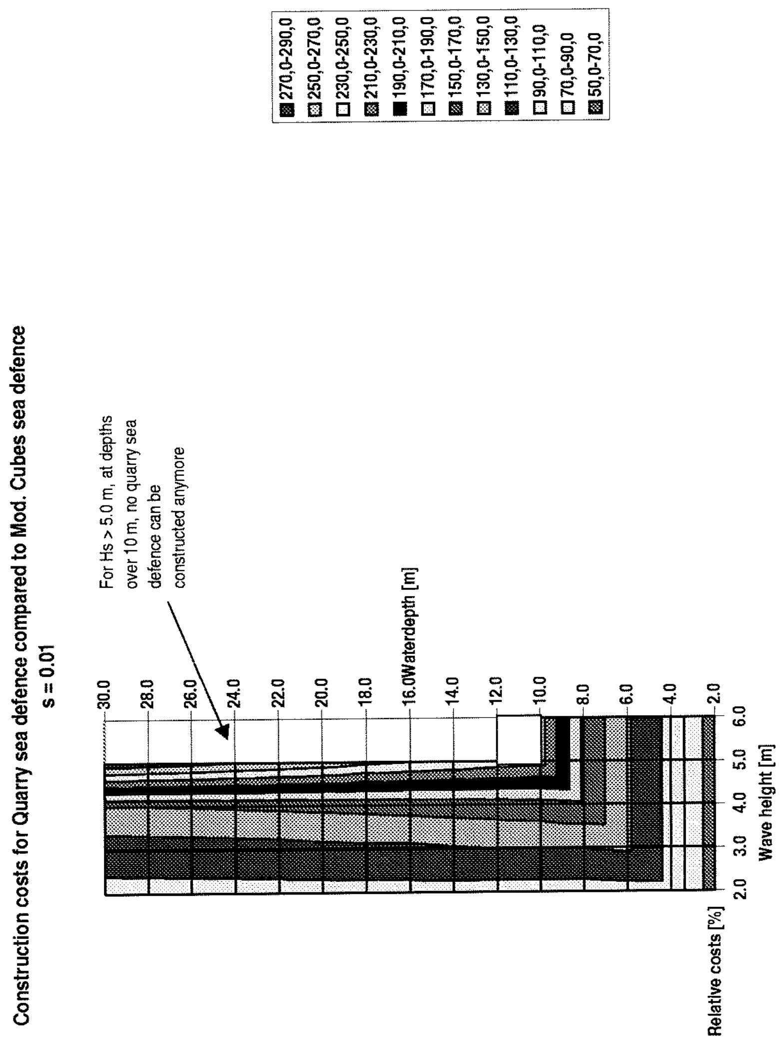


Figure 3.4: Relative construction costs for rubble mound sea defence armoured with quarry stones (s = 0.01)

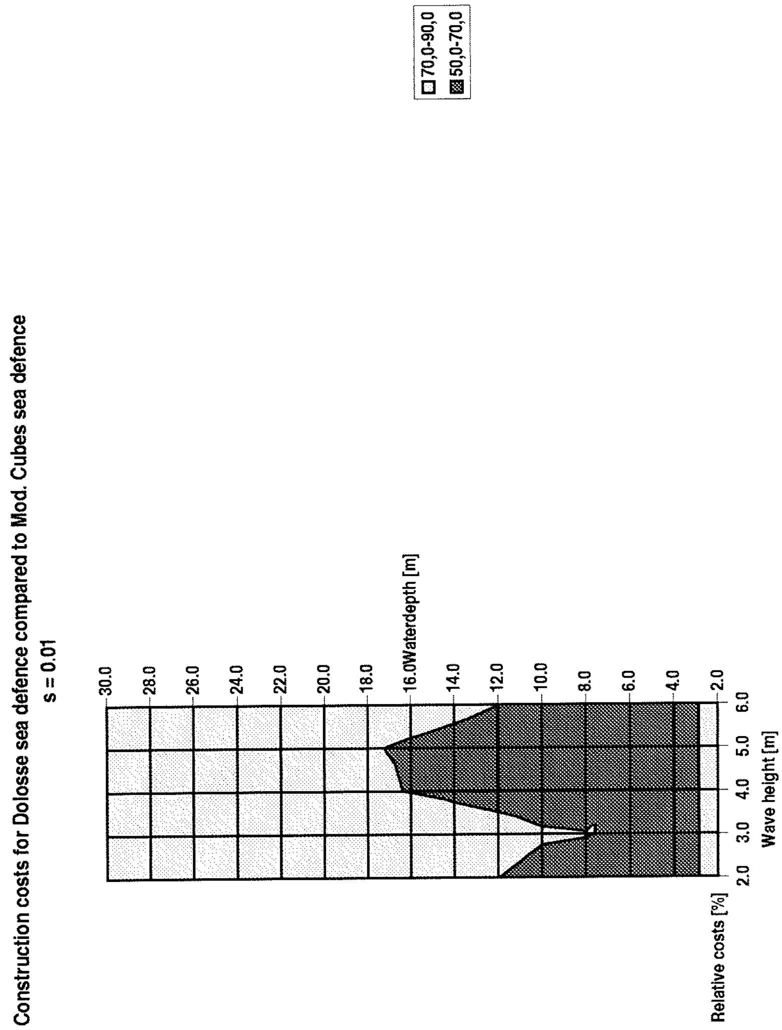


Figure 3.5: Relative construction costs for rubble mound sea defence armoured with dolosse (s = 0.01)

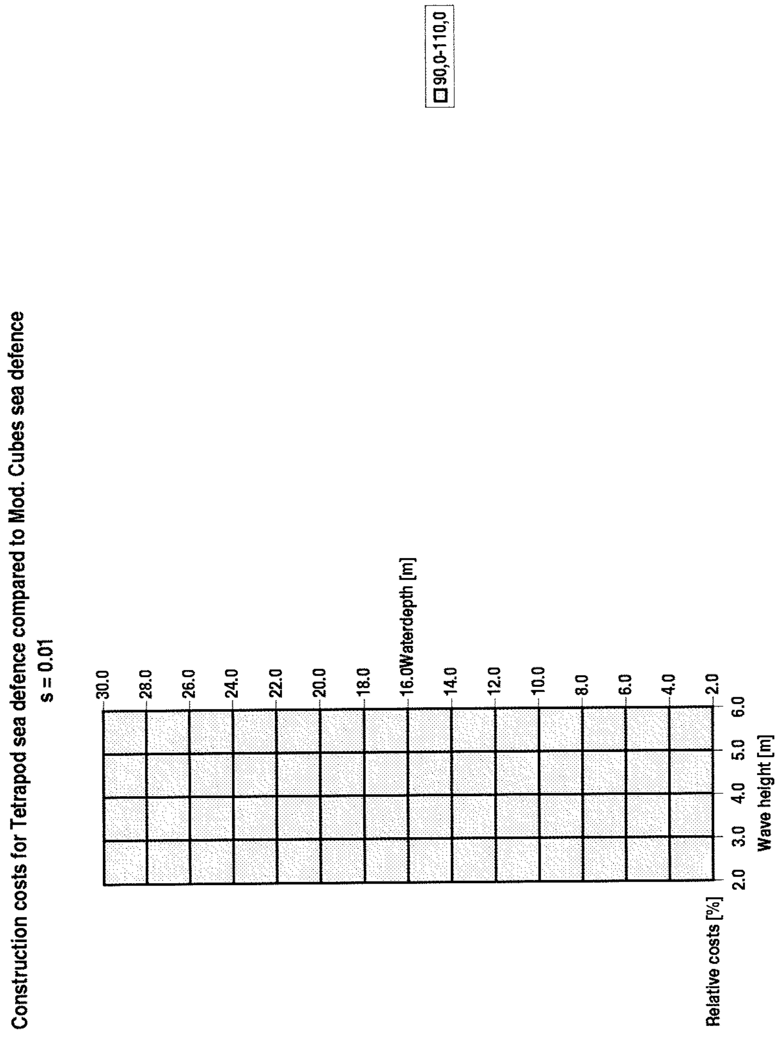


Figure 3.6: Relative construction costs for rubble mound sea defence armoured with tetrapods ($s = 0.01$)

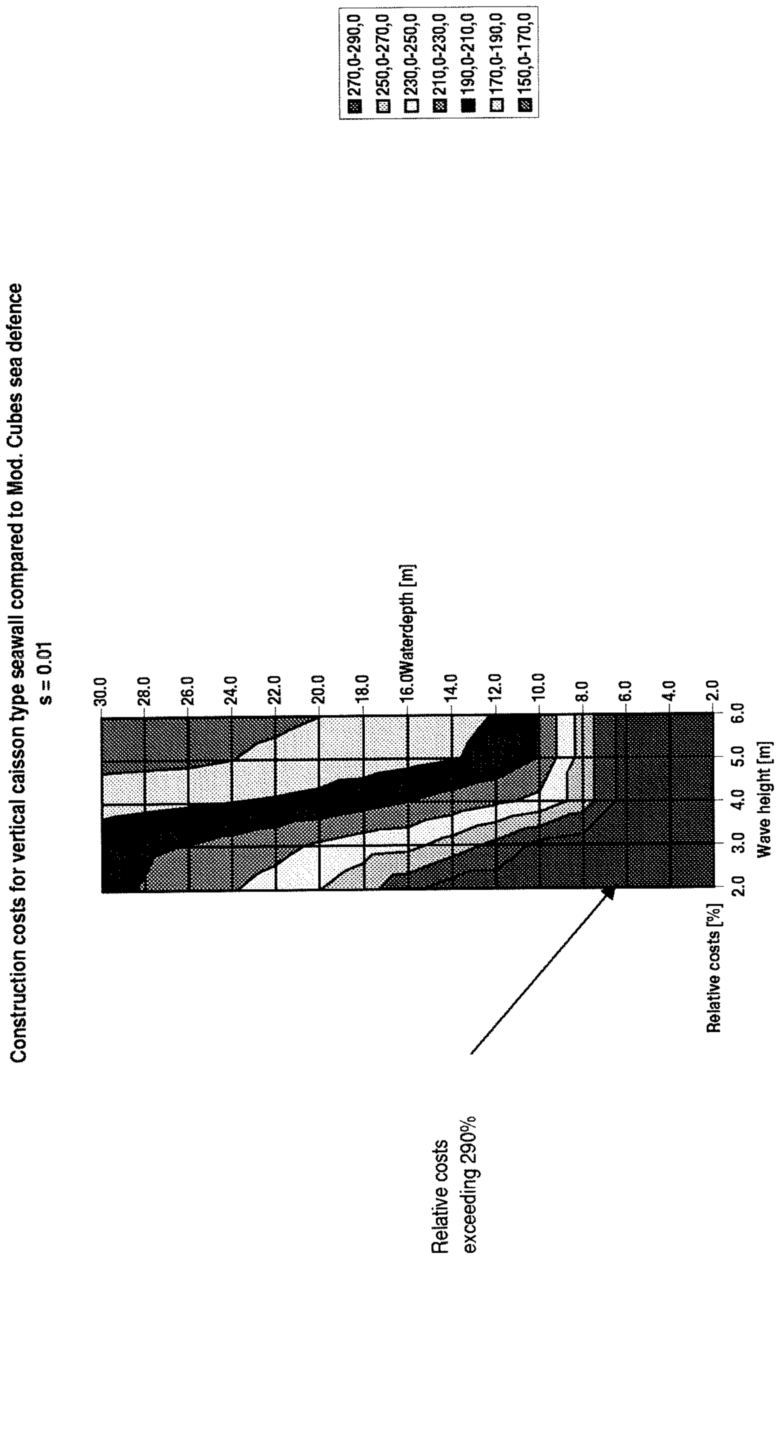


Figure 3.7: Relative construction costs for vertical composite caisson-type seawall (s = 0.01)

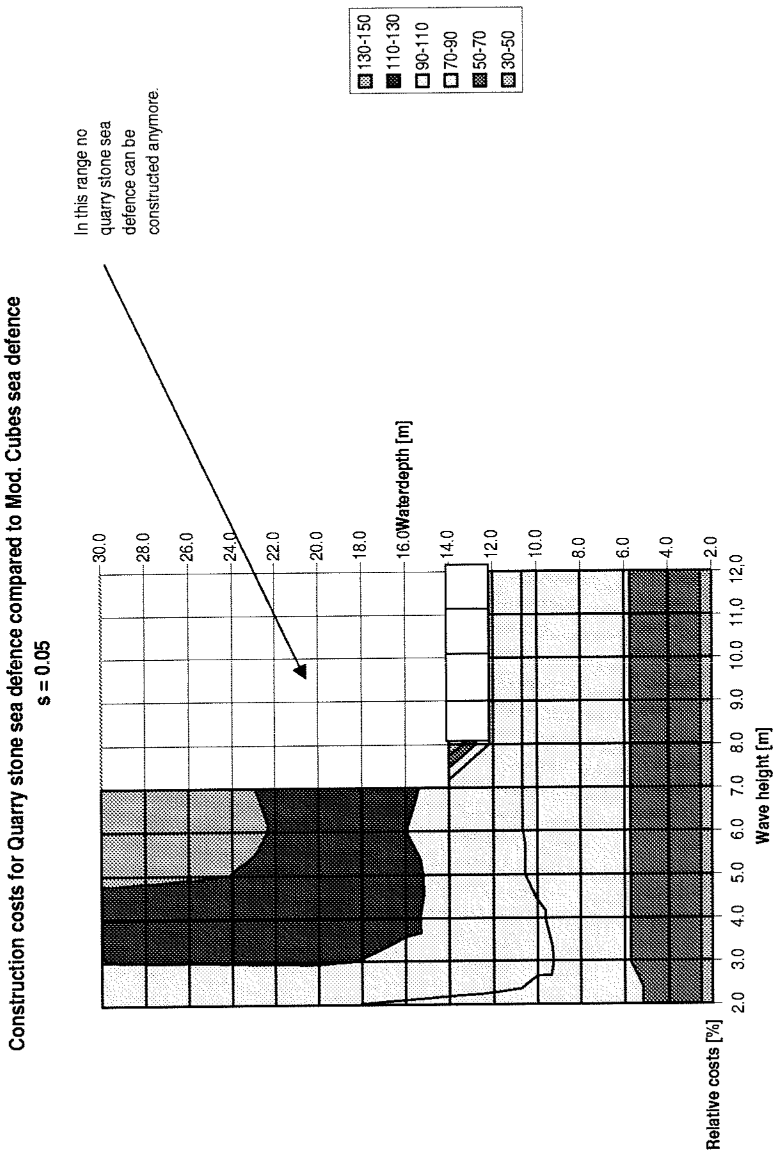


Figure 3.8: Relative construction costs for rubble mound sea defence armoured with quarry stones (s = 0.05)

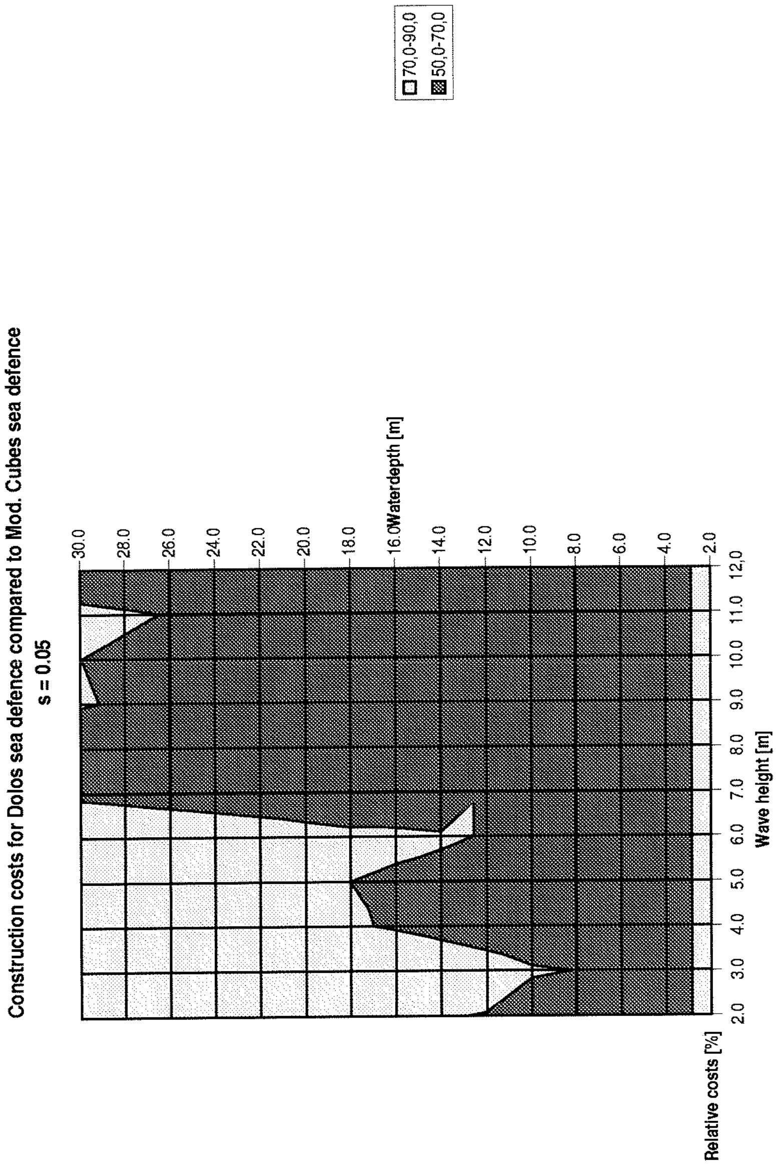


Figure 3.9: Relative construction costs for rubble mound sea defence armoured with dolosse (s = 0.05)

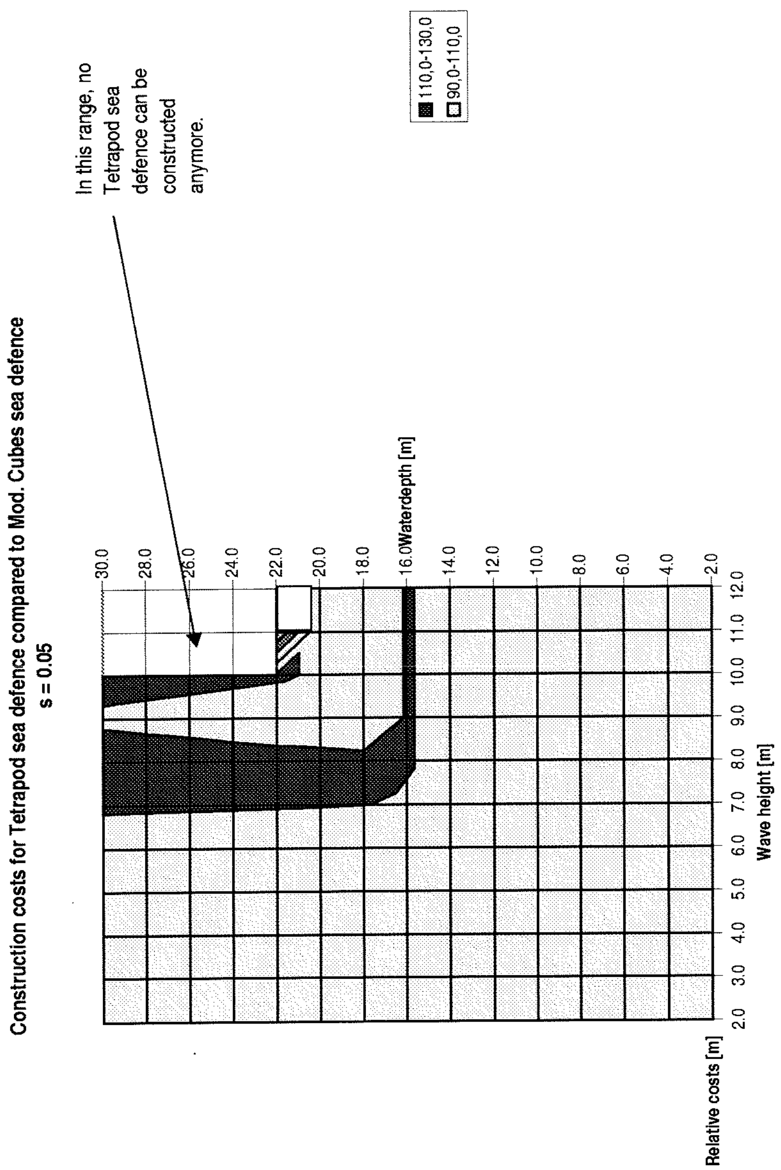


Figure 3.10: Relative construction costs for rubble mound sea defence armoured with tetrapods (s = 0.05)

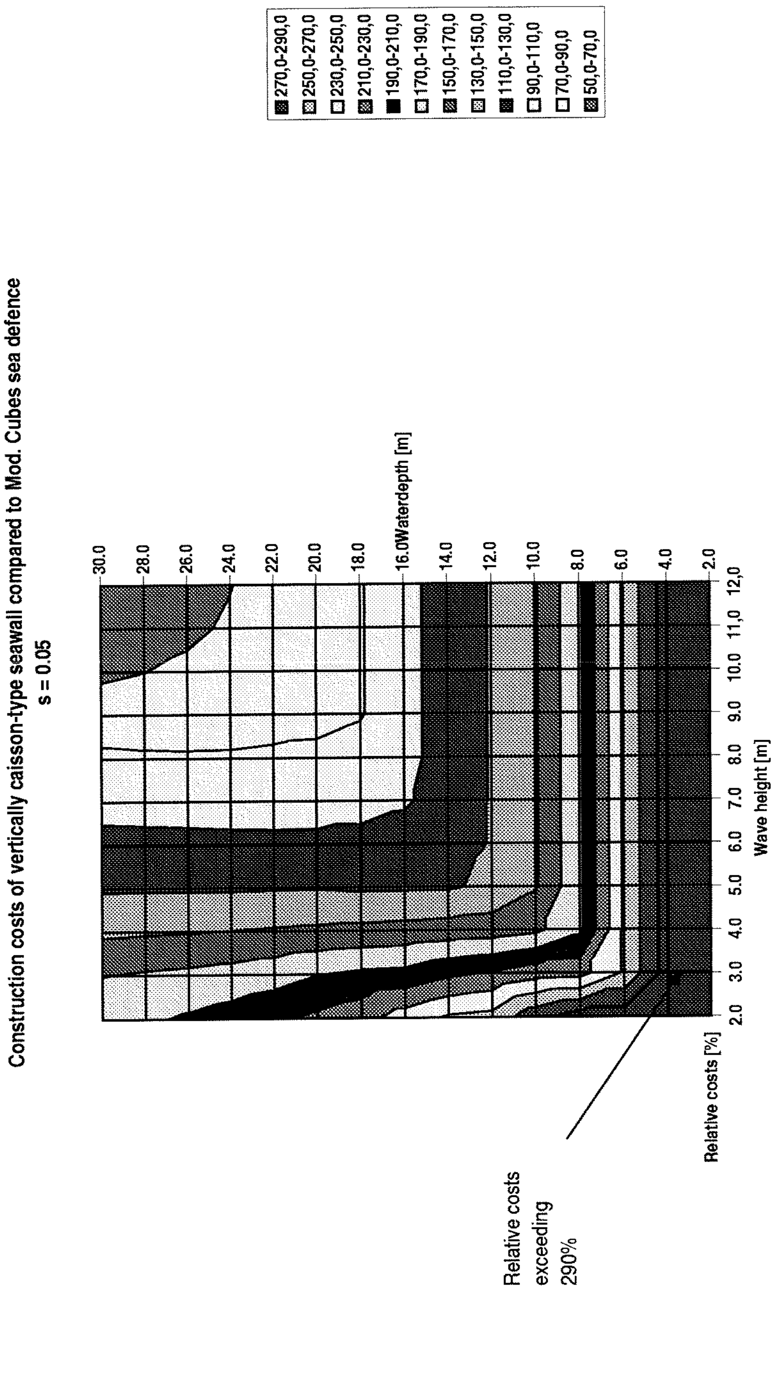


Figure 3.11: Relative construction costs for vertical composite caisson-type seawall (s = 0.05)

3.5.3 General costs indication

All graphs discussed in paragraph 3.5 show a cost relation for an individual type of defence structure. The purpose of this report, is to indicate what type of defence structure to use under certain circumstances. The guidelines of this chapter are divided in two. According to the demands of the reader, one of the two underlined items can be used. The first underlined item, gives guidelines for all five examined sea defence structures. The latter underlined item only provides guidelines for quarry stone and modified cubes sea defences plus vertical caisson-type seawalls. By doing so, the suggested problems with specialised armour elements, as mentioned in paragraph 3.2, are avoided. If a more conservative design is required, one should refer to this item.

Guidelines for all five examined sea defences
In figure 3.12 and figure 3.13 graphs are drawn, displaying the waterdepth h (including predicted sea-level rise) and the significant wave height H_s. In these graphs, ranges are indicated for the use of a certain type of sea defence. The wave steepness influences the total layout of a cross-section, and consequently two separate graphs are shown to present the guidelines, viz. one for wave steepnesses of 0.05 and one for 0.01.

Only significant wave heights up to 12.0 m were examined. Therefore a dotted line is drawn at 12.0 m. Consequently, the development of the graphs over 12.0 m is theoretically. The wide area in which dolosse sea defences appear to be economic, is explained as follows. The K_D-value of 15.8 is extremely high in comparison to the other types of armour. Furthermore, the costs for quarry stones largely influence the outcome of the calculations. Therefore the value a is varied to make the guidelines usable for altered quarry stone costs. This results in the overlapped areas in figure 3.12.

The additional quarry costs parameter a, is varied for values ranging from -15 to 0. For a value $a = 0$ the transition between quarry stones and dolosse sea defences is positioned at a depth of approximately 3 m. This applies for wave steepnesses of both 0.01 and 0.05. If a value of *-15* is used for the parameter a, however, quarry stone sea defences are economical till a depth of about 7 m, for a wave steepness of 0.05. In a similar way, the transition between dolosse and vertical caissons shift outside the field of examination by the altered value of a. The same value of a for a wave steepness of 0.01 does not affect the transition point between quarry and dolosse. The wave steepness causes this phenomena. According to Van der Meer's formula quarry stone armour can withstand higher waves at greater steepnesses, using the same weight. Tetrapods or modified cubes are in none of these examined situations the most economic type of sea defence.

If the Hudson-method is used for quarry stones as well, the transition between dolosse and quarry is still found at waterdepths of about 3 m at both graphs, assuming that $a = 0$. According to the Hudson-equation, taking into account the used layout of the sea defences, higher waves can be withstood using the same weights compared to Van der meer's Method, at steep slopes. Although the rubble mound sea defences become more economic using Hudson, still dolosse are more economic.

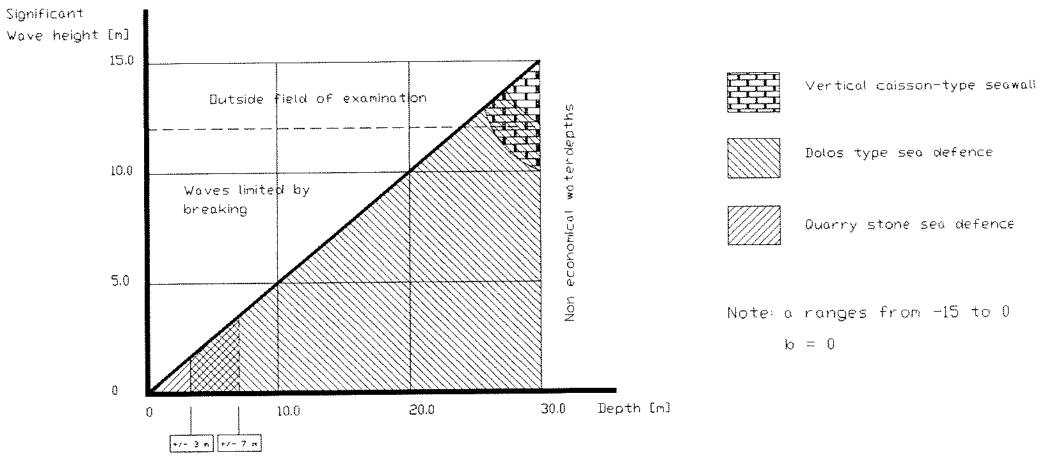


Figure 3.12: Indication of ranges in which to apply the five examined types of sea defences, s = 0.05. Refer to text for details

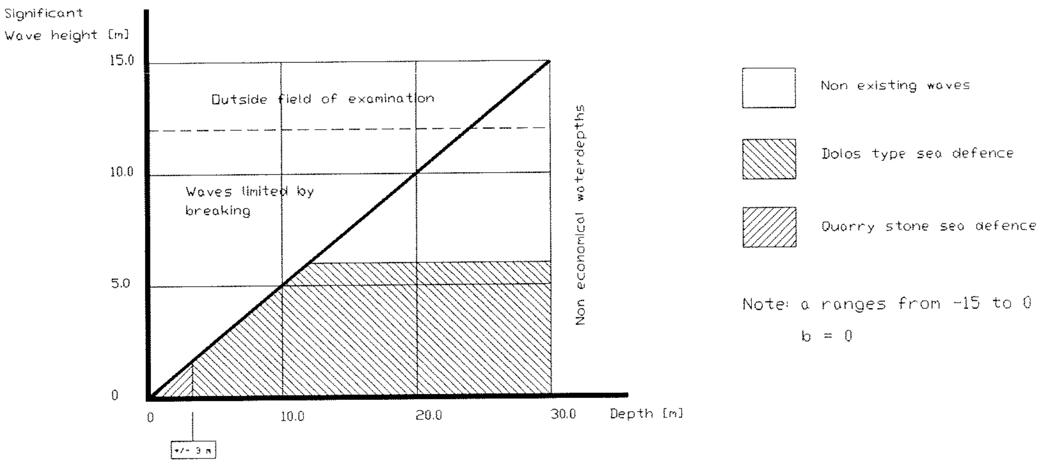


Figure 3.13: Indication of ranges in which to apply the five examined types of sea defences, s = 0.01. Refer to text for details

Guidelines for three certain types of sea defences

The three selected types (quarry stone sea defence, modified cubes sea defence and a vertical caisson type seawall) are all structures with a minimal risk of failure. Risk of fracture of one of the armour elements, like a dolos or tetrapod, is excluded. On the other hand, this extra safety has its price, because the presented guidelines do not represent the most economic possible solution. The guidelines are presented in figure 3.14 and figure 3.15. The wave steepness influences the results of the examinations, and consequently two figures are presented.

Because dolosse are excluded from these graphs the guidelines are totally different. The economic area of quarry stone sea defences is suddenly much wider. For a wave steepness of 0.05, a quarry stone seawall is always the most economic type for wave heights less than approximately 2 m as well at depths shallower than about 13 m. If the costs of quarry stones are reduced, till a = -15, quarry stone sea defences are even economical till a depth of about 19 m. The transition between vertical caisson-type seawalls and modified cubes sea defences is positioned at wave heights of 8 or 9 m in figure 3.14, depending on the value of a.

For wave steepnesses of 0.01 quarry stone sea defences are preferable at waterdepths shallower than approximately 3 m. At waterdepths over 3 m, a modified cubes sea defences is most economical at these steepnesses. If the costs for quarry stones are reduced, till a = -15, quarry stone sea defences are economical both at wave heights less than about 2 m and at waterdepths shallower than approximately 5 m.

If the Hudson-method is used for quarry stones at wave steepnesses of 0.05, the economic range for quarry stone sea defences shifts to wave heights less than about 3 m, instead of less than 2 m. On the other hand rubble mound sea defences should already be replaced by modified cubes sea defences at waterdepths of approximately 9 to 10 m, assuming $a = 0$. The foregoing is related to the slope of the sea defence. At steep slopes Hudson permits less heavy stones than Van der Meer, under the same conditions and taking into account the present layout of the sea defences (P = 0.4, N = 5000). However, at deeper water and consequently higher waves, flatter slopes are necessary for quarry stone sea defences. In case of flatter slopes, Van der Meer permits less heavy stones than Hudson, under the same conditions and taking into account the present layout of the sea defences.

If the Hudson-method is used for quarry stones at wave steepnesses of 0.01, quarry stone sea defences are preferred both at wave heights less than approximately 2 m and at waterdepths shallower than approximately 6 m, instead of 3 m. According to the Hudson-equation, taking into account the used layout of the sea defences, higher waves can be withstood using the same weights compared to Van der meer's Method, at steep slopes.

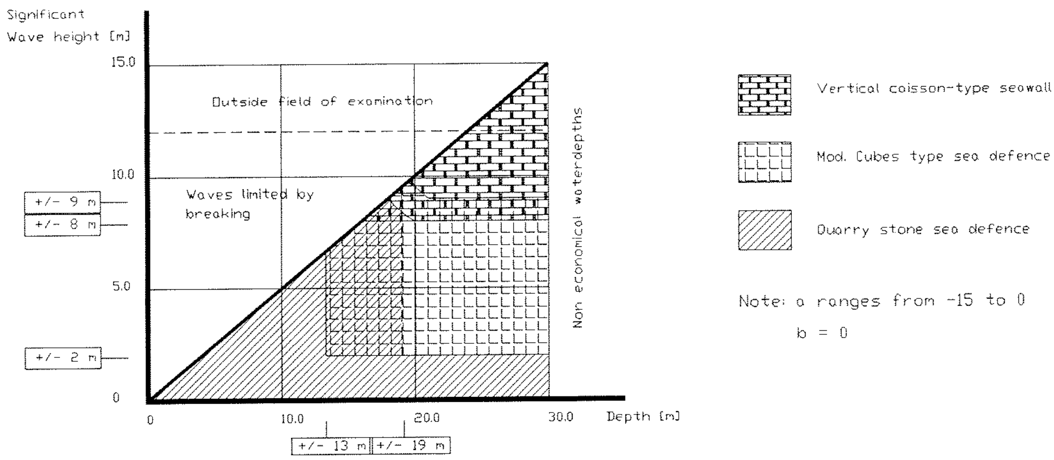


Figure 3.14: Indication of ranges in which to apply three types of sea defences, for s = 0.05. Refer to text for details

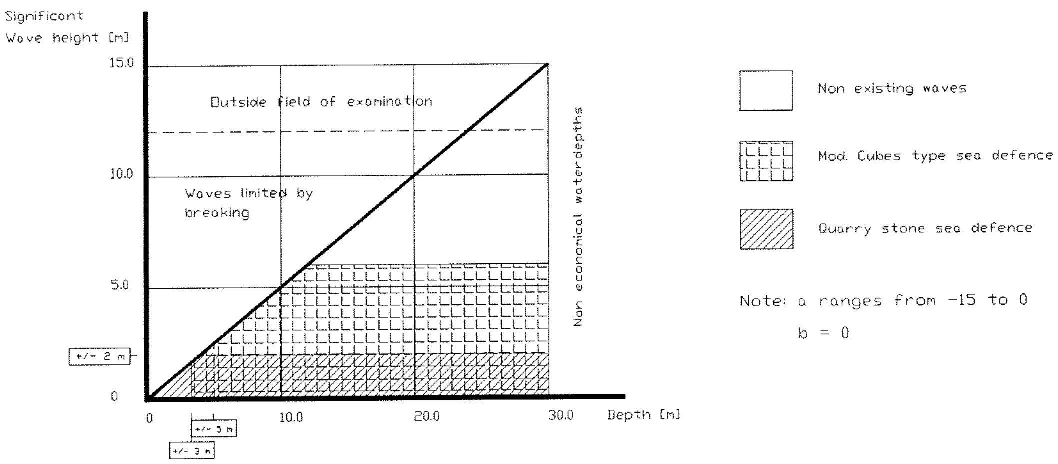


Figure 3.15: Indication of ranges in which to apply three types of sea defences, for s = 0.01. Refer to text for details

3.5.4 Influence of material-costs on the use of sea defences

In paragraph 3.5 conclusions are stated about the use of sea defences as defence structures for artificial islands. In this section the influence of a few items on the constructions costs are mentioned. Only qualitative indications are given. The costs of material for quarry stones and concrete armour units have been selected for one specific location. Consequently, it has no use to determine the remarks below quantitative, because at a different location or at a different time the initial costs of material can be different.

- Influence of increased costs for quarry stones
  If the costs of quarry stones rise dramatically, because of great distances between a suitable quarry and the construction-site or because of another reason, the guidelines as mentioned in this chapter maintain valid in general. Only at depths shallower than 3 m a dolosse sea defence may become more economical, for a certain increase in quarry stone costs. Because most of a rubble mound sea defence consists of quarry stones, only concrete armour layers are unaffected by the increase in price, and become even more economical. The caisson-type seawalls, however, may become more economical at somewhat shallower water and for reduced wave heights, because the cost for concrete are relatively reduced.

- Influence of reduced costs for quarry stones
  Only if the quarry is very close to the site, and its output holds sufficient big stones, a rubble mound sea defence armoured with quarry stones becomes economical in deeper water (compare the different values for a). However, under severe wave attack the heaviest quarry stones still can not withstand the wave attack, and an other type of defence structure is required.

- Influence of concrete costs
  The parameter b indicates additional costs for concrete. Although its value is kept constant at zero, some qualitative remarks can be given about its influence. For b ¿¿ 0, rubble mound sea defences armoured with concrete elements become more expensive. And caisson-type seawalls become even more expensive, because larger volumes of reinforced concrete are required. Consequently, both the transition between quarry stones and dolosse and the transition between dolosse and caissons can shift to deeper waters. For b ¡¡ 0, these transitions can shift to shallower water.

- Difference between sea defences and breakwaters
  The design of a caisson is exactly the same if it is used as a breakwater or as a seawall. A rubble mound sea defence, however, is more economical than a rubble mound breakwater under the same conditions. A rubble mound breakwater requires additional materials to protect the inner-slope. A rubble mound sea defence only requires sand in these parts. Moreover, rubble mound sea defences can use sand in many parts, where rubble mound breakwaters require quarry stones. Consequently the development of rubble mound sea defence-costs differs.

3.5.5 Recommendations for future research

Hereafter, some recommendations for future research or deepening of the executed study are suggested.

- Using modern formulae for armour calculations
  The study was performed using both Hudson and Van der Meer formulae. The Hudson formula is somewhat outdated, and new insights are not included. Therefore, the use of new formulae, and applying of new methods is desirable. Furthermore, a more uniform result is obtained in this manner.

- Examining the economic applicability of 'new' armour elements
  Newer armour elements, like accropods and core-loc, which can be applied in a single layer mat prove to be more economical than dolosse for instance. However, the failure mechanisms of these type of elements needs to be thoroughly examined.

- Examining non-hard sea defences
  Hard sea defences, like rubble mound and caissons, were used in the performed study. Soft sea defences, or combined defences were not taken into account. These types of sea defences, however, may prove very useful at limited depths or under less severe wave attack (for instance at the lee-side of the island).

Chapter 4

Dredging activities for artificial islands

4.1 Introduction

The materials which are used to fill the interior of the island, and which do not belong to the sea defences nor to the actual island's structures, are called landfill. Landfill can consist of soil, waste or other materials. Landfill, found on both marine and land excavation sites can be used, dependent on economical preferences. In this chapter, only marine excavation methods are discussed. This implies that soil-material is used as landfill. Marine soils are economically excavated using trailing suction hopper dredges. The processes involved are explicated in paragraph 4.2.

A short example is given, to indicate the amount of landfill required. Let us consider a mediocre airfield of 500 ha (= $5 * 10^6$ m^2), to be build on an artificial island. The average depth at the island's location reaches 10 m. Examining appendix A, these values appear to be moderate estimates. In case the ground-level is raised up to a level of 5 m above M.S.L., the total amount of landfill measures some 75,000,000 m^3, without even considering any settlements. If the island is positioned in the North Sea, suitable layers of Holocene deposits are available, in thickness ranging from one to five metres [24]. An average excavated layer thickness of 2 m would require an area of 6 * 6 km^2, for dredging operations!

The methods of filling an island are subject of paragraph 4.3. The same paragraph also discusses the parameters, which influence the filling process. Finally, paragraph 4.4 details an example, which calculates the costs, required to fill a man-made island. On the basis of this example, the economic position of larger islands in relation to smaller islands is explained. This chapter only explains the very basics of dredging techniques. A more thorough discussion can be found in additional literature, for example [26].

4.2 Dredging of landfill

Excavation of soil from marine excavation sites (the sea bottom) is done with the aid of trailing suction hopper dredges (or short, trailers). A trailing suction hopper dredge is a self-propelled vessel, used for inland or marine shipping. It is equipped mainly with two backward positioned suction tubes. While travelling slowly (a few knots), the bottom soil is caught by the dragheads and pumped via the suction tubes into the vessels hopper. Trailing suction hopper dredges can operate in moderate to high seas, especially when swell-compensators are used. This device enables the dragheads to maintain in contact with the sea bottom, even if the vessel itself is subjected to swell. A picture of a trailing suction hopper dredge is shown in figure 4.1.

The operation procedure of a trailing suction hopper dredge is a repeating process. The parts of this process are summed up below.

1. The dredging itself
2. Loaded transport to the dump-site
3. Dumping of the load
4. Empty transport to excavation site

The loaded and empty transports do not require to be elaborated. However, the dumping of the landfill does, and is explained in paragraph 4.3. This paragraph continues with the sections of the dredging process.

4.2.1 Sections of the dreding process

The loading of a trailing suction hopper dredge is executed by means of a chute. The sand-water mixture is pumped through the chute, at the end of which it is evenly distributed by a diffusor. At the opposite side of the hopper, an overflow mechanism is located. Its height is adjustable. If soil, having a low density is loaded, a greater volume can be loaded for a given tonnage than would be the case if heavier soil is transported. The volume can be adjusted by means of the overflow-height. During the filling of the hopper, three stages are distinguished.

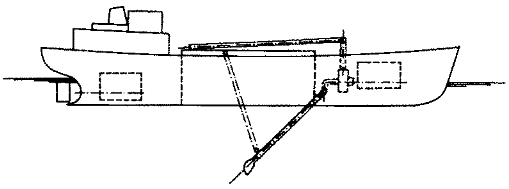


Figure 4.1: Example of trailing suction hopper dredge [25]

1. Loading of sand-water mixture till overflow level is reached
2. Set up of water-film above overflow level
3. Continue loading with overflow losses

Figure 4.2 shows the loading-curve for a trailing suction hopper dredge, having a fixed overflow-height. The tonnage is related to the time. The loaded and empty transport times (t_V and t_V) can be distinguished. As well as the loading time of the hopper, during t_Z. When the overflow-level is reached at T_{OV}, the curve starts to bend. This is caused by the overflow losses. As more soil is dumped in the hopper, more soil gets in suspension and consequently more soil vanishes through the overflow. How long should this process continue, and when should the loading stop?

The cycle production reaches its maximum value, when the transported amount of settled soil in relation to the cycle time t_C is maximal. The relation can be visualised by drawing a tangent at the loading-curve. In particular for the part, where loading is continued with overflow losses (point B in figure 4.2). Because the relation refers to the transported amount of soil and not to the transported amount of water, the tangent should start at the tonnage, for which the hopper is completely filled with water (point A in figure 4.2).The total cycle time t_C greatly influences the position of point B. If the cycle time grows, point B is shifted to a position further in time. In words this means, that for large sailing distances with a long cycle time it pays to continue loading over a longer period of time.

The loading-curve operates the following tonnages and times.

T_M	maximum tonnage	(N)
T_S	tonnage after water-film has flown off	(N)
T_{OV}	tonnage at start overflow	(N)
T_W	tonnage till overflow-level, when only water is loaded	(N)
T_{DO}	tonnage when bottomdoors are open	(N)
T_L	tonnage empty ship	(N)
t_Z	loading time	(s)
t_{OV}	loading time till overflow-level is reached	(s)
t_D	dump time	(s)
t_{VL}	loaded sailing time	(s)
t_{VE}	empty sailing time	(s)
t_C	cycle time ($t_{VL} + t_D + t_{VE} + t_Z$)	(s)

4.2.2 Parameters influencing cycle production

The basics of the cycle production of trailing suction hopper dredges were discussed in the previous paragraph. On the basis of the acquired knowledge, parameters can be determined which influence the cycle production. The most important parameters, which influence this production over a longer period, are stated in table 4.1.

4.3 Dumping of landfill

Dumping of the landfill, or in other words the filling of the island, requires a specialised approach. It should not be taken lightly, and thoroughly studies and preparations are essential.

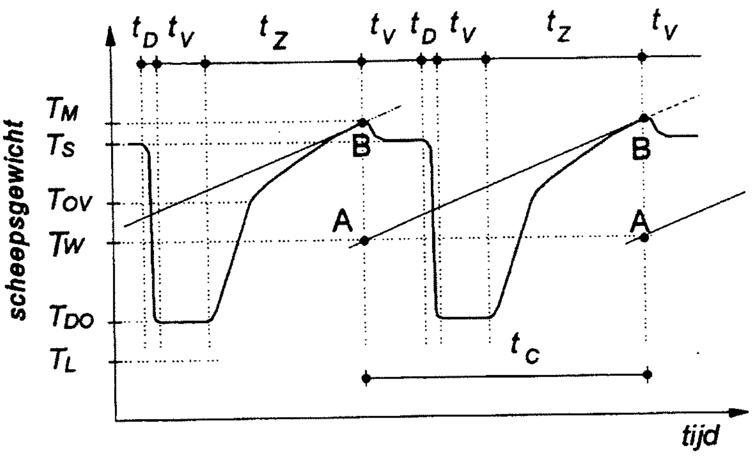


Figure 4.2: Loading-curve for trailing suction hopper dredge, having a fixed overflow-height [26]

Vessel	Soil	Meteorological	Other
Max. tonnage Sailing speed Draught Suction capacity Dumping time	Density Bulking factor Depth D_{50}, D_{90}, etc.	Workable periods Average wave length Average steepness	Sailing distance

Table 4.1: Influencing parameters on cycle production

The method of filling depends on numerous factors, which are elaborated on at the end of this paragraph. Usually, the method of filling is changed, dependent on the continuation of the construction works.

If a trailing suction hopper arrives at the island location with its cargo, their are generally three methods to remove the cargo. Each of the three methods is discussed below.

- Direct dumping
- Pumping
- Rainbowing

Direct dumping

This is by far the fastest method of unloading. If the bottomdoors are opened, large volumes of soil are dumped on the sea bottom. If waterdepths are moderate the impact of these volumes may even increase the compaction of the subsoils. However, if waterdepths become to large, the soil volumes may either be spread over a too large area or the impact on the sea bottom is of such a magnitude, that even erosion occurs directly underneath the trailer. This procedure takes about 5 to 10 minutes.
If the elevation of the island has reached a level of about 10 m, depending on the trailers draught, direct dumping is no longer possible. Split trailers and trailers with sliding doors can dump to a higher level. Split barges can reach a level of up to 5 m. At these or shallower depths, one of the methods described below should be used.

Pumping

This method empties the vessel hydraulically. The soil in the hopper is mixed with jetwater and pumped through a booster pump and floating pipeline, to the dump site. This method takes more time, about one hour to an hour and a half, for an average vessel. The estimated time, includes time required for connecting and disconnecting of pipes as well.

Rainbowing

This is a hydraulically method, as well. However, no floating pipelines are used, but the soil-water mixture is rainbowed (spouted) over the bow of the trailer, directly on the dump site. Rainbowing takes about 30 to 45 minutes.

If direct dumping is no longer possible, because of the elevation of the island or other reasons, a rehandle pit may prove to be useful. The trailer dumps it load in the rehandle pit, which is located near the island site, and returns to its excavation site again. At the rehandle pit one or more permanent suction cutter dredges are positioned. They excavate the soil from the rehandle pit, and pump it on the island. This way, the cycle time of the trailing suction hopper dredges is reduced, and consequently the total costs per excavated cubic metre of soil. However, certain conditions have to be fulfilled. Because, cutter dredges make use of floating pipelines, wave heights should be limited to a maximum of several tens of centimetres, otherwise the pipelines may break. A reduction of the wave heights can be achieved by erecting parts of the sea defences above the water level, already at the start of construction works. This way, a breakwater is realised, which reduces wave heights. Of course, the economic value of a rehandle pit depends on the expenditures for the trailer and the cutter dredges, the size of the island, etc.

4.3.1 Parameters influencing dumping of landfill

A short description of three dumping methods was given in the previous section. The most important parameters influencing these methods are summed up in table 4.2. Some parameters, which influence the cycle production, influence the method of dumping as well.

4.4 Costs related to size of island

This section proves that multi-purpose islands, which are generally larger, can be more economically constructed than smaller, single purpose islands. First a very basic example is given to calculate the cycle production of a trailing suction hopper dredge. If the costs for rental and amortisation of the trailer are known, the costs per dredged and deposited cubic metre of soil are known as well. Consequently the costs per square metre, for two different island sizes can be computed.

4.4.1 Costs per dredged and dumped cubic metre

The following characteristics, are assumed for the trailing suction hopper dredge in this example (some of which are not used, but are only mentioned to give an indication of the trailer). Well settling soil (e.g. normal sand) is assumed for the example.

Hopper volume	8,000	m^3
Effective load	11,000	ton
Draught (max.)	8.50	m
Suction pipes	2 * ø 0.90	m
Sailing speed	13	knots
Dredging depth (max.)	40	m

Table 4.3 gives an example of a cycle production calculation, using the above characteristics. In the top box of the table, some assumption-parameters are included. The characteristics of the trailer are shown, as well as some assumptions about its cargo. The parameters are shortly described below.

- Mixture velocity
  The celerity, with which the sand-water mixture is transported through the suction pipes. If the diameter and the number of suction pipes is known, the mixture discharge can be computed easily.

Vessel	Soil	Meteorological	Other
Draught	Density	Workable periods	Island's volume
Dumping time	D_{50}, D_{90}, etc.	Average wave length	Fuel prices
Pumping time		Average steepness	
Rainbowing time		Wave height	
Available dredges			

Table 4.2: *Influencing parameters on method of dumping*

- Bulking factor
  The voids ratio for in-situ soils is mostly small, because of the high rate of compaction. When the in-situ soils are excavated the compaction decreases, and the voids ratio increases. In other words, the excavated volume of in-situ soil is dissimilar to the volume of soil in the hopper. The relation between hopper soil volume and in-situ soil volume is called the bulking factor.

- Suction concentration
  Indicates the volume percentage of the sand-water mixture which actually consists of soil.

- Loading
  Indicates the volume percentage of the hopper which consist of soil.

- Total overflow losses
  Indicates the cumulative percentage of soil that is lost during the overflow-processes.

- Turning time
  During its production cycle, a trailer needs to turn from time to time. Either because the end of its dredging-path is reached or because it has to turn after the load is dumped. From the example, one can conclude that five turns are included.

- Volume in hopper
  The volume of soil and water in the hopper, depend on the maximum effective tonnage of the trailing suction hopper dredge. During the loading of the trailer the weight of the soil and water may not exceed the maximum effective tonnage. In our example this results in the following equation.

$$(0,9 * 1900 + 0,1 * 1030) * V_{soil+water} = 11,000,000$$
$$V_{soil+water} = 6067 m^3$$

  The volume of soil in the hopper amounts to 0.9 * 6067 = 5461 m^3.

- Suction time
  This, obviously, indicates the time required to fill the hopper with soil. The following equation is used.

$$t_{suction} = \frac{V_{in-situ}}{(1 - overflow\ losses) * suction\ concentration * Q_{mixture}} \tag{4.1}$$

Table 4.3 shows the weekly production for the three methods of dumping. Obviously, the direct dumping method leads to the highest cycle production. If the total weekly expenditures for the trailer are known, the costs per cubic metre of soil can be calculated. Because the soil will be used as landfill for the island, the hopper production is required instead of the in-situ production. Total weekly expenditures for the trailer are for this example assumed to be US $ 500,000. These costs account for the following costs: rent + write-off hopper, crew, fuel, down time, additional materials, etc. The total costs are only assumed costs, for an exact calculation more thorough computations are required. Consequently, the prices as stated in table 4.4 result. These prices differ from the value (e.g. US $ 3) used in the costs estimates of the landfill in the sea defences as used in appendix C. The prices in table 4.4 are only used as an example!

Cycle production for Trailing suction hopper dredge

Initial parameters			
Hopper volume [m^3]	*8000*	Density in hopper [kg/m^3]	1900
Effective tonnage [ton]	*11000*	Density water [kg/m^3]	1030
Sailing speed [kn]	*13*	Bulking factor	1,05
Sailing distance [km]	*25*		
Suction pipe diameter [m]	0,9	Suction concentration [%]	25
Mixture velocity [m/s]	4,0	Loading [%]	90
Number of suction pipes	2	Total overflow losses [%]	20
		Turning time [min]	5

Intermediate computations			
Volume in hopper (sand + water) [m^3]	6067	Mixture discharge [m^3/s]	5,09
Volume sand in hopper [m^3]	5461	Density load [kg/m^3]	1813
Volume sand in-situ [m^3]	5201		

Suction time	
Suction time [min]	85

Cycle-time and -production			
Direct dumping			
suction time [min]	85	hours per week	168
sailing time [min]	125	repair-works [hours]	12
turning [min]	25	workability	0,95
dumping [min]	5	effective hours per week	148,2
mooring [min]	0	number of trips per week	37,09
	240	Situ production [m^3/week]	**192872**
		Hopper production [m^3/week]	**202515**
Pumping			
suction time [min]	85	hours per week	168
sailing time [min]	125	repair-works [hours]	12
turning [min]	25	workability	0,85
Pumping, connecting,		effective hours per week	132,6
disconnecting and mooring [min]	110	number of trips per week	23,08
	345	Situ production [m^3/week]	**120012**
		Hopper production [m^3/week]	**126013**
Rainbowing			
suction time [min]	85	hours per week	168
sailing time [min]	125	repair-works [hours]	12
turning [min]	25	workability	0,90
rainbowing [min]	90	effective hours per week	140,4
mooring [min]	0	number of trips per week	25,94
	325	Situ production [m^3/week]	**134897**
		Hopper production [m^3/week]	**141642**

Table 4.3: Cycle production calculation for trailing suction hopper dredge

Dumping method	Situ price US $/m^3	Hopper price US $/m^3
Direct dumping	2.59	2.47
Pumping	4.17	3.97
Rainbowing	3.71	3.53

Table 4.4: Assumed prices per cubic metre of dredged soil

4.4.2 Overall island costs

Using the assumed prices of table 4.4 a total costs indication per square metre island can be made. However, some extra assumptions are necessary as well here. Because the method of dumping depends on the elevation of the island, so does the price for the landfill. The landfill in the top section of the island is more expensive than in the section, that was dumped directly. This remark is made for a proper understanding, however, an overall landfill price of US $ 1.46 is maintained in the following examples. These examples detail two circular man-made islands at the same average depth, but having a different surface area. Furthermore, the price per lineair metre of sea defence is kept constant as well. This is an unrealistic assumption, because the most exposed sides of islands require other defence structures than the lee sides of the islands.

Assumptions:

Costs sea defence structures per lineair metre	US $	55,000
Average landfill costs per cubic metre	US $	2.47
Average waterdepth both islands	m	20.0

The calculations for both islands, one small and one large, are summed up in table 4.5. The table shows that the difference in price per square metre are considerably between small and larger islands. Figure 4.3 shows similar relations, varied for the distance to the excavation site as well. The figure also shows, that the proposed relation is not very sensitive for the waterdepth nor for the distance to the excavation site.

<u>Remark:</u> All examples in this paragraph were made on the basis of a circular island. A circular island has the fortunate quality that its surface to outline rate is very economic. Islands of other shapes, show a similar relation, as drawn in figure 4.3, but less extremely.

Diameter [km]	Surface area [m^2]	Volume [m^3]	Costs defences [US $]	Costs landfill [US $]	Totall costs [US $/m^2]
1.0	$0.8 * 10^6$	$15.7 * 10^6$	$173 * 10^6$	$40 * 10^6$	266
5.0	$19.6 * 10^6$	$392.7 * 10^6$	$864 * 10^6$	$968 * 10^6$	93

Table 4.5: Construction costs relation between large and small artificial islands

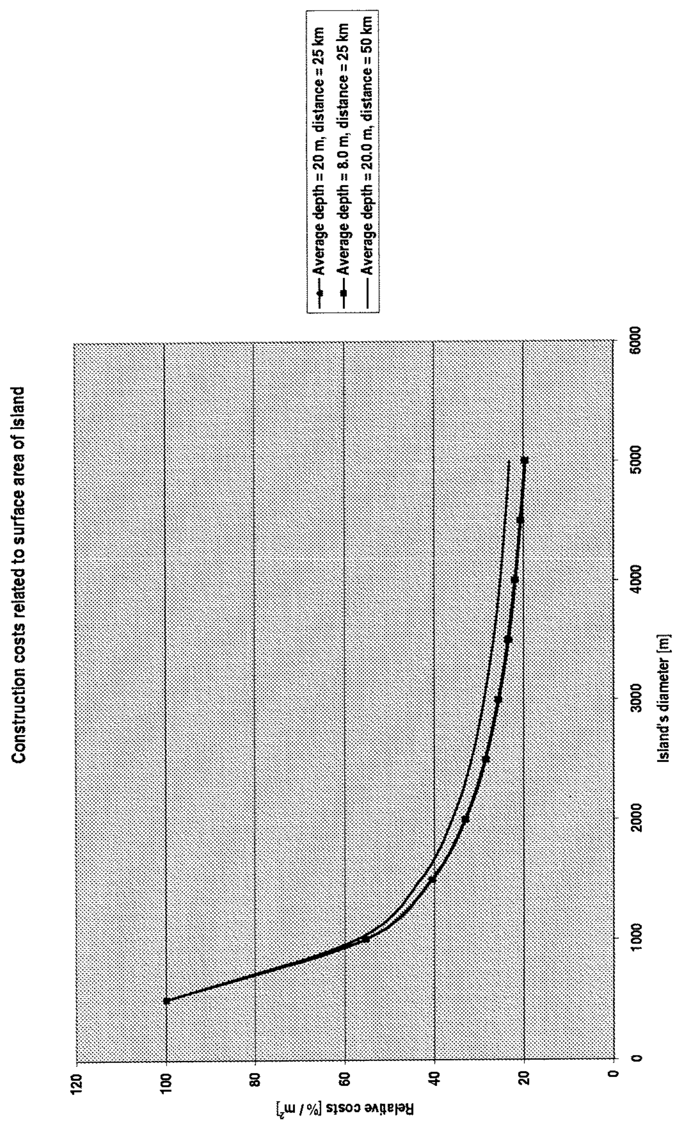


Figure 4.3: Constructions costs related to size of island and distance to excavation site

Chapter 5

Morphological processes

5.1 Introduction

Any obstacle placed in the water, causes a disturbance in the propagating wavefield. The magnitude of this disturbance depends on the dimensions and shape of the obstacle. In a similar way an artificial island causes a disturbance as well. A reclaimed man-made island placed some distance offshore induces a *wave* energy low zone behind it, the so-called shadow-zone. In this zone, wave heights will be lower than would be the case in a situation without the island. If the influence of the shadow-zone extends into the surfzone, sediment-transport is influenced as well. Accretion will occur in the energy low region of the surfzone and erosion in the neighbouring regions of the surfzone. When the changes in wave parameters are known, predictions can be made about the changes in sediment-transport, using transport-formulae like CERC and Bijker. During the island's lifetime, additional expenditures must be taken into account to restore the erosion regions by means of bypassing, sand nourishments or other measures.

Realising the consequences of the island's size and its position in the coastal zone on sediment transports, enables the designer to make a more deliberated choice for these two variables. Moreover, insight is gained in the lifetime costs caused by the effects of erosion and accretion. In this report the numerical model HISWA (see paragraph 5.5) is used to provide understanding in the morphological influences on coastal areas induced by artificial islands. This model was chosen, because its does not require an extremely powerful computer, a standard PC is sufficient. Although HISWA was not especially developed to predict sediment-transports, its outcome suites well in the field of this report, viz. to come up with guidelines.

It is not intended to give a universal solution for the problems caused by offshore islands; this would not even be possible! Because so many factors influence the transport, the situation differs from place to place. Nevertheless, this report aims to map the effects of the location and the dimension of an island on longshore transports. It is still advisable to undertake thorough site-investigations, as well as a study of the historical data regarding transport rates in the area. Testing the effects in scale models is another alternative.

Before any conclusions are presented, the most important aspects of morphological processes are presented in paragraph 5.2. Subsequently, paragraph 5.3 introduces a circular island model, which is used in the calculations. The actual goal is formulated in paragraph 5.4 as well as the assumptions and limitations imposed on the study. Paragraph 5.5 and paragraph 5.6.1 discuss the used model HISWA and the CERC-formula respectively. Finally conclusions and recommendations are given in the last paragraph of this chapter.

5.2 Morphological processes

The physical processes, which actually cause the longshore transport are subject of this paragraph. This paragraph was added to increase the understanding of this chapter's results. A short summary of the physical processes is given below.

First of all the situation without any obstacles is considered. In this situation, an initial longshore transport S_0 exists. Which phenomena cause this initial longshore transport? Most parts of this section were taken from [27].

In general, the sediment transport processes can be divided into three stages:

- The stirring-up of bottom material, bringing it into suspension, or the loosening of this material on the bottom.
- The horizontal displacement of these particles by the water.
- The re-sedimentation of these particles once again.

Waves mainly loosen material on the bottom and stir it up, while currents mainly transport material to another place. Of course, these stages are not exclusive. Waves may cause wave-driven sediment transport and currents certainly also pick up and stir up material from the bottom. Consequently, the combination of both waves and currents (coastal areas) results in a relatively large longshore transport-rate.

The driving force for many coastal phenomena can be explained by the radiation stress. Radiation stress is a pressure force in excess of the hydrostatic pressure force caused by the presence of waves. In reality, the radiation stress is neither a true stress (force/area) nor a true force (as implied in the previous sentence) but a force per unit length.

Unlike hydrostatic pressure, the radiation stress is not isotropic; indeed, just as with stresses, it is associated with a given direction or plane. Here, these planes are vertical and perpendicular to the two horizontal axes, X oriented in the direction of wave propagation and Y along the wave crest (see figure 5.1). This will yield the principal stresses; S_{XX} and S_{YY}.

If we consider a rectangular element of water enclosed by four vertical principal planes, shown in plan in figure 5.1. Then, if the wave and depth conditions at all four planes 1, 2, 3 and 4 are identical, the radiation stress components on opposite sides of the 'block' shown in the figure are identical and there is no resulting force. Only if the wave conditions vary between planes 1 and 2 or 3 and 4, there will be a resultant force. Thus, we can expect the radiation to influence physical processes only in areas, where wave conditions change. Such areas would, therefore, be at locations where wave refraction, diffraction, shoaling or breaking occur. With

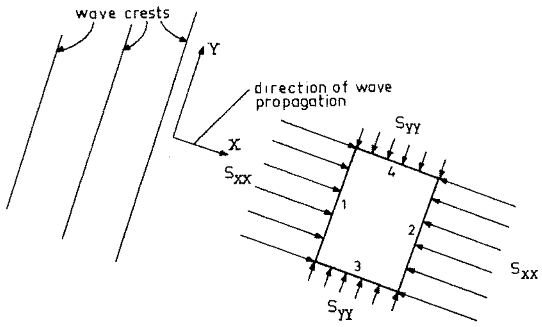


Figure 5.1: Radiation shear stresses [27]

the radiation stresses, the wave-induced currents can be explained.

Within the surfzone, waves tend to break. Due to the breaking and the increased effect on the bottom caused by the shallower water, wave stir up material on the bottom within the surfzone. Moreover, wave-induced currents act in the surfzone, because of reasons mentioned in the previous section. This explains why the major part of the longshore sediment transport takes places within the surfzone.

Let us consider the same situation, but this time an artificial island is placed some distance offshore. The island causes a wave energy-low zone or shadow zone behind it; in other words, it reduces the wave height. The magnitude of this energy-low zone depends on the size of the island, and the influence on longshore sediment transport relies on the distance offshore as well. The effect of these two variables is the essential issue of this chapter. The waves in the energy-low zone are much lower than was the case before the construction of the island, and so is the wave-induced current (the radiation stresses are less). These two effects combined with a reduced width of the breaker zone, because lower waves break at a distance closer to shore, result in a local fall in sediment transport.

5.2.1 The actual problem

The reduced transport causes accretion in the shadow zone and erosion in the 'downstream' zone directly bordering the shadow zone. The erosion is logically caused by the fact that more sand is removed than supplied. Normally, some erosion is measured at both sides of the energy-low zone, due to the changing angles of incidence.

Whether the erosion and accretion actually cause a problem, depends on the local circumstances. If the island is built in a deserted coastal area (for instance a production-island in the Canadian Beaufort Sea) a certain amount of erosion is acceptable. However, erosion in a densely populated coastal area is much more troublesome. Furthermore, the order of magnitude of the erosion depends on the bottom-material. Gravel causes less erosion and accretion than sand, and a rock bottom none whatsoever.

Before any morphological effects can be predicted, some data needs to be acquired. At most locations in the world data is available about wave height-distribution, wave direction, bathymetry, etc. If this data is not available or insufficient, already in the planning-phase data collection should commence. In addition, even when construction has already begun, it should be continued to be able to adjust the design. The preceding makes perfect sense, because the design phase may take several years and so does the construction phase as well. Using statistical predictions, the essential design conditions can be obtained. Some of the parameters, which longshore transports depend on, are summed up in table 5.1.

5.2.2 Only short term influences

The predictions and statements made in this chapter will only be valid, during a relatively short period. For morphological processes this implies a maximum of a few years, depending on the local circumstances. Once the first signs of erosion and accretion occur, circumstances change. For example the breaker line is repositioned, and the refraction pattern is altered as well. Nevertheless these short term influences on longshore transports give an indication of the problems to expect in the long run. And consequently on the amount of money involved to overcome these problems during, and after the island's lifetime. The designer can deliberate the extra costs of placing the island further offshore, generating less erosion, or placing the island closer to shore, and spending more money on overcoming erosion and accretion problems.

5.3 Island model used in this report

The required parameters, mentioned in table 5.1, are too complex, especially since only guidelines are required for this project. Consequently, a simple model is assumed, for which the distance offshore and the surface area of the island are varied. Such a model is drawn in figure 5.2.

The model shows a circular island, having a diameter D, and placed some distance offshore d. The deepwater waves, indicated by H0 and T, bend around the island due to refraction. The shadow zone can be seen as well; it is clear that the width of the shadow zone is determined by refraction. Within the shadow zone the influence of the islands is most noticeable. Therefor the reduction of the longshore transport is most severe in this region. Although not shown in figure 5.2, the width of the shadow zone depends on the directional spread of the waves and the shape of the island as well. In the continuation of this chapter the influence of the island on transport reduction is presented. More information about the actual calculation-methods can be found in paragraph 5.6.

5.3.1 Expected relation

Without making any calculations, a prediction can be given about the short term change in longshore sediment transport related to the distance offshore and the diameter of the island. A qualitative relation is shown in figure 5.3. The influence of the deepwater angle of incidence, the grain size diameter or any other parameter were not taken into account. It

Island	Waves	Transport	Geographical
shape of island	wave height-distribution	tidal-currents	bathymetry
surface area	wave direction-distribution	tidal movements	D_{50}, D_{90}, etc. of soil
sea defences	wave period-distribution	initial longshore transport	
distance offshore			

Table 5.1: Paramaters influencing longshore transport

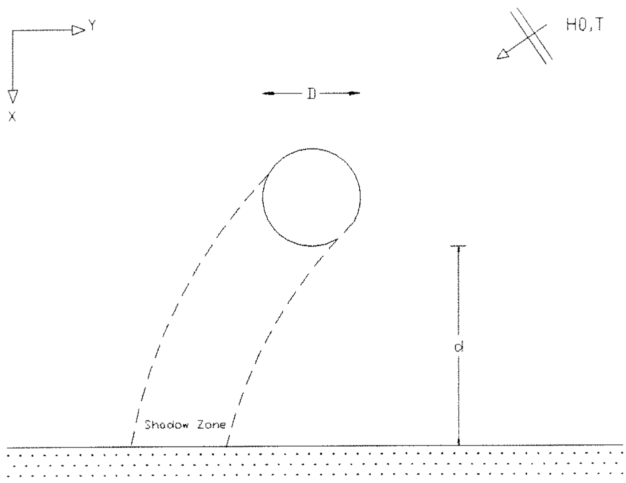


Figure 5.2: Refraction shown by island model (two refraction-lines drawn)

is expected, that the influence of the island grows for increasing diameter, and diminishes when the distance offshore increases. If the island is positioned at a very long distance from the shore, no influence on the longshore transport is expected anymore. In the continuation of this report, this relation is checked and more detailed.

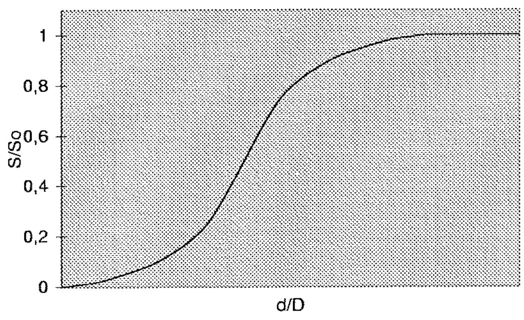


Figure 5.3: Qualitative relation S/S_0 and d/D, see also figure 5.2

5.4 Problem description

In the first chapter of this report, a general problem description and research objective were given. Because the morphological problem, discussed in this chapter, originates a very specific problem, also a specific problem description and research objective are required. On the other hand, to avoid the risk of creating a too complex matter, some limitations are added as well as a few assumptions.

5.4.1 Research objective

The short term influence of an artificial island on longshore transport should be generally explained. Only the influence of waves on longshore transports is taken into account, not the influence of currents. The reader gains insight in the effects on transports, related to the island's size and its distance offshore.

5.4.2 Design assumptions

These assumptions are enforced by the designer himself, either to overcome some gaps in the data or to limit the area of investigation. The assumptions are divided into categories, and a description is given for each assumption.

Natural assumptions

- $\overline{H_{s0}} = 2.0$ m: An average significant deepwater wave height of 2.0 m is assumed. This wave height initiates the major part of the morphological processes, in the long term. A much higher wave height may result in a temporarily upturn of the sediment transport, but the occurrence of this wave height is less, and consequently its contribution to the total transport over a period of years is limited. In paragraph 5.7.1 the influence of $\overline{H_{s0}}$ on the change in sediment transport is examined.
- Deepwater wave steepness s_0 of 0.05: This kind of steepness indicates seas; waves generated by local storms. Because steepnesses in the range of seas are the most common in nature, these have been chosen.
- No currents: HISWA only calculates the wave heights, and for these calculations, a current is only of second order influence for the HISWA-calculations. However, the effect of currents on the longshore transport-rate is severe.
- Bathymetry: For reasons of simplicity and to retain insight in the results of the study, the bottom-profile is kept simple. A uniform slope of 1 : 100 till a depth of 30.0 m is assumed, beyond that the sea bottom continues horizontally. This may seem rather steep, and consequently a narrow surfzone will result. Nevertheless, most transport-formulae only use the breakerdepth and the breaking wave height as parameters related to the surfzone. Consequently, the width of the surfzone becomes unimportant, and so does the bottom-slope. In other words, the transport density per metre surfzone, measured perpendicular to the shoreline, will be less for a wide surfzone than for a narrow one, under the same conditions. Nevertheless, the influence of the bathymatry on the change in sediment transport is discussed in paragraph 5.7.1.

Island's layout assumptions

- Circular island: a circular island holds the most economical relation between area surface and surroundings. The island's size is represented by its diameter D.
- Vertical walls: reclaimed artificial islands are protected by hard sea defences in this thesis. Slopes for the sea defences vary form totally vertical (caissons) to $1:6$ (rubble mound sea defence). Related to the dimensions of the island and the influence of the morphological processes, these slopes can be presumed as vertical. As a second condition, no wave transmission is assumed for the vertical walls.

Variable assumptions
The following parameters are varied during the use of HISWA. Finally, conclusions are drawn for the influence of artificial islands on the longshore sediment transport based on these varied parameters.

- Diameter of the island D: Man-made islands, which have already been built, vary in size from a few hectares to over a thousand hectares. Accordingly three diameters are varied, viz. 0.5, 1.0 and 4.0 km, representing areas of respectively 20, 78 and 1257 ha.
- Distance offshore d: The distance offshore is defined as the distance between the outmost shoreward point of the island and the transition point between the wet and the dry beach during M.S.L. This parameter is varied for values of 1.0, 2.0 and 5.0 km.
- Deepwater angle of incidence ϕ_0: The angle of incidence determines the extent of the sediment transport. If waves approach perpendicular to the shore and no currents exists, no longshore transport will be generated. The transport-ratio increases for increasing wave-angle up to a deepwater angle of about 45 degrees. After that, it diminishes again. This statement can be 'seen' in the CERC-formula, refer to paragraph 5.6.1. An upper-limit transport is determined for a deepwater wave angle of 45 degrees, and as a reference, the same calculations are executed for a deepwater wave angle of 10 degrees.
- Directional spread of the waves: Two values are used for the directional spread of the waves, namely 5.7° and 31.5°. The latter, a wide spread, refers to the natural circumstances in which seas occur. The first one, a minimal spread, is used as a reference. A spread in the wave direction or the existence of seasonal dominant directions, are very common factors in nature. Below, some effects of wide and minimal spreads are explained. These items are also recognisable in the graphs of appendix F.

    1. If deepwater angles of incidence are small, and a large directional spread is used as well, negative transports can result. At the lee-side of the island, the contribution of the waves having a large angle of incidence is blocked, and only the waves having a small or even a negative angle of incidence contribute to the transport. These waves, originated from a deepwater wave with a negative angle of incidence, cause the longshore transport to move in negative direction.
    2. A similar situation exists at the weather-side of the island. When a large directional spread exists, the waves having a smaller angle of incidence are blocked by the island. This results in a higher average angle of incidence at the weather-side of the island. When the CERC-formula is used to predict sediment-transports, the maximum transport-rate is found for angles of incidence of about 45°. In

some cases the local sediment transport after the construction of an island can be higher than the initial situation. For example when the initial angle of incidence amounted to say 10 degrees, and the local average angle becomes 30 degrees.

All variables are summed up once more in table 5.2.

5.5 Using the numerical model HISWA

Quoting its manual, HISWA is a numerical 2D wave propagation model to obtain realistic estimates of wave parameters in coastal areas, lakes and estuaries from given stationary wind, bottom, and current conditions. In this report, HISWA, acronym for HIndcast Shallow WAves, is used to predict the changed wave parameters in the surfzone after an artificial island is constructed some distance offshore.

However, HISWA has some limitations, two of which are mentioned below. More information about HISWA can be found in [28] and [29].

- HISWA is not able to take into account diffraction. In irregular wave fields, it seems that the effect of diffraction is small, except in a region less than one or two wave lengths away from the tip of the obstacle. In other words, as long as the influence of the island, on wave heights and angles of incidence, is measured at distances greater than two times the wave length, the lack of diffraction is not essential anymore. Since wave lengths are short (several tens of metres) in the examined cases, this is viable.
- Waves can only be computed in a sector narrower than 180°. If the deepwater angle of incidence is not too large and the bottom-profile does not vary too much this should be no problem. A value of $\Theta = 120°$ for the total directional sector is often used.
- HISWA is only able to cope with single-peaked spectra. Referring to the research purpose of this report, this limitation is of no influence.

The next paragraph details the working of HISWA, the choice of the computational grid and its boundary conditions.

5.5.1 Computational grid and boundary conditions

HISWA uses a grid to compute the changing wave-parameters. The computational region is a rectangle covered with a rectangular grid. This grid is defined by a Cartesian coordinate system, i.e. in a flat plane and not on a spherical earth. One of the axes (called the X-axis) is chosen in the down-wave direction (i.e. roughly the mean wave direction); lateral (normal) to the X-axis is the Y-axis. The computation starts at the up-wave boundary x = 0, and

Diameter of island D [km]	Distance offshore d [km]	Deepwater angle of incidence ϕ_0 [degrees]	Dir. spread of waves [degrees]
0.5	1.0	10	5.7
1.0	2.0	45	31.5
4.0	5.0		

Table 5.2: Variable input parameters for HISWA

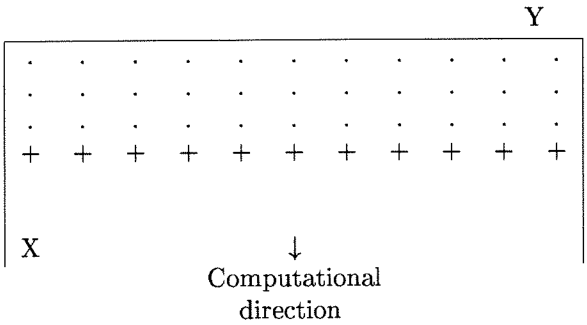


Figure 5.4: The computational grid
. points determined previously; + points being computed

proceeds in the positive x-direction. After the states in all points on a line in y-direction have been determined, the computation proceeds with the next line in the grid, see figure 5.4.

The state in a point is determined only by what happens in the up-wave direction of this point. For the propagation in x-y space an explicit scheme is used (leap-frog), in Θ-direction a fully implicit scheme is used (backward Euler). This has certain consequences. Since the numerical scheme is explicit in x-y space, the computation is only stable under the condition that the ratio of the forward step (i.e. the step in x-direction) and the lateral step (the step in y-direction) is smaller than a certain limit. In cases without currents, HISWA operates the following limit.

$$\frac{\Delta x}{\Delta y} \leq \cot(\Theta) \tag{5.1}$$

where Θ is the discrete spectral wave propagation direction. A consequence of this condition is that waves can be computed only in a section narrower than 180° (i.e. 90° to either side of the positive x-direction). In HISWA, a value of 120° for the total directional sector is often used. According to equation (5.1). Δx must be approximately half of Δy.

The accuracy of the computed values by HISWA depends largely on the chosen grid-density and the boundary conditions. First the dimensions of the computational grid are required, followed by the density of the grid. The dimensions and the grid-density are maintained for all situations throughout this chapter.

Dimensions of the computational grid

The up-wave boundary (x = 0) should be chosen in such a way that refraction influences are not yet noticeable on the propagating waves. This consequently holds that the up-wave boundary should be chosen seaward of the island and at a point where the maximum depth of 30 m has already been reached. Because the maximum distance offshore amounts to 5 km and the maximum diameter reaches 4 km, the length of the X-axis is set to 10 km (taking into account 1 km extra length).

The breadth of the computational grids is dependent on a few factors. First, the outmost point, which is theoretically still influenced by the island, should be inside the borders of the computational grid. Moreover, this point should be away far enough, to avoid influence from the grid's borders. Thus the breadth in y-direction must be larger than the area of interest, because along each lateral side of the grid (if there is an open boundary along that side) a region exists where the wave field is disturbed by an import of zero energy from the lateral boundaries.

The determinative situation is given for an island 5 km offshore with a diameter of 4 km and waves approaching with an angle of incidence of 45 degrees. See figure 5.2, when we consider the influence of the outmost left point of the island, we know the outmost left point of the shadow zone. Without considering any refraction (making a conservative approximation) nor the influence of the directional spread, this shoreline-point is at a distance 5 km + 2 km = 7 km away from the symmetry-axis of the island in y-direction. Still the problem of the influence of the lateral boundaries exists, and a much wider y-axis would be required to overcome this problem. Fortunately, HISWA possesses the command BOUNDARY LEFT/RIGHT EXTENDED. With this option, the user indicates that the incoming waves at the (lateral) boundary come from a region where the situation is independent of the coordinate normal to the boundary. This region is assumed to extend to infinity. Using this command, the disturbed area becomes much smaller. An additional 3 km is added to the 7 km to allow disturbances and to be able to measure the exact length of the shadow zone. The shadow zone ends at those points where the wave heights have the same magnitude, as was the case before the construction of the island. Although not fully necessary, a symmetrical grid is desired, and consequently the total breadth of the grid amounts to 20 km.

<u>Density of the grid</u>

Now that the dimensions of the grid are known, the density of the grid can be determined. In the beginning of this section a relation between Δx and Δy was given, namely $\Delta x / \Delta y \leq 0.5$. Δx and Δy indicate the length and breadth of a rectangle, in x- and y-direction respectively. If the length and breadth of an individual rectangle are chosen smaller, the density of the computational grid increases and more accurate calculations are made. Nevertheless, it is needless to make the grid as dense as possible. If the density is increased and the outcome does not vary too much, it was unnecessary to increase the density. A compromise has to be found between the accuracy of the outcome and computing-time. Table 5.3 shows the results for several densities; 150 meshes in both x- and y-direction, and $\Theta = 120°$ were chosen as a reference, and the average computed wave heights were set to 100%. The same is done for the angle of incidence at the breaker line ϕ_b. The average wave height was taken at several depths and under several conditions. 50 meshes in both directions do not give an accurate enough average wave height or angle of incidence. The difference in outcome between 100 and 200 meshes in both directions, on the other hand, is small. Because the results of 150 meshes in both directions approximate to hand-calculations most closely (see also paragraph 5.6.1), this option is chosen.
In the bottom-section of table 5.3 one can see the influence of the discrete spectral wave propagation direction Θ. Its influence on the results of these HISWA-calculations is small.

Summarising, this results in the following grid-characteristics, which are used in all situations.

- Grid: 10,000 * 20,000 m^2
- Δ x = 66.7 m; Δ y = 133.3 m
- $\Theta = 120°$

Δ x[m]	Δ y[m]	Θ	Number of meshes in x-direction	Number of meshes in y-direction	Relative H_s	Relative ϕ_b
200	400	120	50	50	126%	116%
100	200	120	100	100	110%	106%
66.7	133.3	120	150	150	100%	100%
50	100	120	200	200	92%	95%
66.7	133.3	90	150	150	103%	93%
66.7	133.3	100	150	150	101%	97%
50	100	90	200	200	95%	89%
50	100	100	200	200	94%	93%

Table 5.3: Influence of grid-density on accuracy of HISWA-outcome

5.6 Calculation approach

Over the years numerous formulae and methods have been developed to predict sediment transports. Some examples are: CERC-formula, BIJKER-method and Van RIJN-method. They differ in the complexity of parameters, which are taken into account. In this report only one formula is used, namely the CERC-formula. It is one of the first transport formulae, and easy to understand. Using more formulae, would not increase the insight in the purposes for which the formula is used in this report. Consequently, the CERC-formula is solely used here.

5.6.1 The CERC-formula

This method was developed soon after World War II by the Beach Erosion Board, the predecessor of the US Army Coastal Engineering Research Center. The CERC-formula predicts the total longshore sediment transport through the breaker zone. The formula relates the transport to the available wave energy. Only the driving force resulting from waves approaching obliquely and having the same proportions at all point along the coast is considered. Hence, no other driving forces than waves, such as tidal currents, are taken into account.

The CERC-formula can be written in many forms, one of which is stated below. Both deepwater conditions and conditions at the breaker line can be used in the CERC-formula. But all CERC-formulae have at least one breaker line-parameter. In this report only conditions at the breaker line are used.

$$S_x = 0.025 H_b^2 c_b \sin(2 * \phi_b) \tag{5.2}$$

Examining equation (5.2) one thing is remarkable:

- The longshore sediment transport is computed, only by using data at the breaker line. In the formula itself, this is indicated by the threefold subscripts b. As said, some CERC-derivatives use also deepwater conditions. But it is not clear, in which way an island influences the correctness of the outcome of this type of derivatives. Most likely, these type of CERC-formulae assume an undisturbed path, in which the waves travel from deep to shallow waters. In other words, the outcome of these CERC-formulae would be false, because the island changes the deepwater waves, in such a way, not accounted for by these formulae. For that reason a derivative of the CERC-formula is used, which is only influenced by the conditions at the breaker line and not by deepwater conditions.

Determining the position of the breaker line

To determine the values of the breaker line parameters, we first need to locate the breaker line itself. If waves would be monochromatic, if no directional spread would be present and if the bathymetry is even and smooth, all waves would break at the same depth. However, the natural situation is manifold and different waves break at different depths. Still the CERC-formula desires one criterion for which waves break and which can serve as a reference to come up with the breaker line. The following relation exists at the theoretical breaker line

$$H_b = \gamma * h_b \tag{5.3}$$

To overcome this problem, the theoretical breaker line is determined by hand first and related to the percentage of broken waves Q_b calculated by HISWA, all under the same conditions. The breakerdepth h_b is taken as the point of correspondence between the theoretical breaker line and the HISWA-outcomes. The hand-calculation assumes monochromatic waves, a uniform significant deepwater wave height and a constant deepwater angle of approach. The bathymetry is, as assumed in paragraph 5.4.2. Refraction theory is used and the results are listed in table 5.4.

In words this means that for a deepwater angle of approach of 10 degrees and a wave height of 2.0 m, 5.35% of the non-monochromatic waves, having a directional spread of 5.7° and computed by HISWA are broken at the depth of the theoretical breaker line, computed by hand. The difference between Q_b of the two angles of incidence is so small, that the breaker line is positioned, where 5% of the incoming waves are broken. It would be expected that Q_b for both angles of incidence is the same, because HISWA operates the same value for γ as used in the hand-calculations, namely 0.6. Why this is not the case, is not clear. In some cases, HISWA uses γ-values of over 0.6, which is even more odd. Most likely the use of non-monochromatic waves and the small directional spread, is the cause.

Method of calculation the change in sediment transport

The actual sediment transport is not of interest in this report, only the change in sediment transport is important. Therefore, the yearly sediment transport after the construction of an island is compared to the yearly sediment transport in the initial situation. If this is done for a variety of island's surface areas and distances offshore, an indication can be given about the influence of these two parameters on longshore sediment transports.

The change in transport can be computed for a single cross-section over the breaker line or for a larger area within the surfzone. The first option requires only one point on the breaker line. This point should be chosen at the centre of the shadow-zone, because the island's influence is most severe at that point. However, the determination of such a point is easily liable to errors. Furthermore, the influence of the angle of approach on the computed transport is rather large, which increases the change of errors even more. Hence, it seems logically to determine the change in transport over a larger area and consequently reduce the effect of individual errors.

ϕ_0	H_0 [m]	γ	h_b [m]	ϕ_b	H_b [m]	Q_b HISWA
10	2.0	0.6	3.19	6.5	1.91	5.35 %
45	2.0	0.6	2.85	25.9	1.71	4.87 %

Table 5.4: Percentage of broken waves computed by HISWA, compared to theoretical breaker line

The sediment transport is computed in several cross-sections along the 20,000 m Y-axis. The interspace between these cross-section is set to 2,000 m. At positions near the centre of the shadow-zone, where the reduction in transport is most severe, the interspace is reduced. This way, a more detailed view is obtained of the changes in longshore transports. The results are presented in several graphs, and are shown in appendix F. An example of a transport calculation is given in paragraph 5.7. Conclusions and recommendations, resulting from these graphs are presented in paragraph 5.8.

5.7 Example of computation

This section discusses the influence on longshore transports of a 1 km circular island, positioned 1 km offshore. Waves are approaching at deepwater, with an angle of incidence of 10 degrees and a minimal directional spread of 5.7°. At the end of this paragraph, the influence of the deepwater significant wave height H_{s0} and the bathymetry is examined as well.

Initial longshore transport S_0

The longshore transport, present in the period before the construction of the island, is called the initial transport S_0. The changed sediment transports are compared with this situation. To determine the initial transport, the theoretical breaker line is related to the percentage of broken waves, as explained in the previous section. Table 5.5 shows the initial transport, and its parameters. The breaker depth h_b differs a little from its value, calculated in table 5.4. This is caused by the fact, that not exactly 5.35% is used for Q_b but the average value of 5%. Furthermore, it is striking that the parameters ϕ_b and H_b vary from the values calculated by hand. This is caused by the use of non-monochromatic waves in HISWA, with a, although small, directional spread of 5.7°. The wave length L is used to compute the wave celerity c_b at the breaker line, as shown in equation (5.4. The deepwater wave height amounts to 2.0 m, and the wave period to 5.06 s, because the wave steepness is kept at a value of 0.05.

$$c_b = \frac{L_b}{T} \tag{5.4}$$

The initial transport S_{x0} = -1.86 $Mm^3/year$, moving in a negative direction (from right to left), is used as a reference and set to 100%.

Present longshore transport S

The longshore transport, present in the period after the construction of the island, is called the present transport S. For several cross-sections the breaker line is found, using trial and error. The transport is computed at the breaker line in the same way as was done for the initial situation. At locations near the place of minimum transport, the interval between the cross-section is reduced. This way, a more accurate view is obtained of the changes in sediment tranport. The relative changes can be seen in table 5.5. The centre of the island is located at position Y = 10000. At Y = 9735, about 300 m to the left of the island's centre, the minimum transport-rate is found. For this situation, with a man-made island very close to the shore, a complete blocking of longshore transport occurs. This process will most likely form a tombolo, which can eventually develop into a connection from the original shoreline to the island. Figure 5.5 displays iso-lines of the wave height and vectors of the wave direction. The reduction of the wave height at the lee-side of the island, is clearly visibly. The iso-lines, which cross the island's surface are caused by the chosen grid-density in HISWA. Using a more dense grid would make these physical impossibilities disappear. The development of the present longshore transport is shown in figure 5.6.

	Y [m]	h_b [m]	H_b [m]	ϕ_b [°]	L [m]	S_x [Mm^3/year]	S_x/S_0 [%]
Initial	N.A.	3.25	1.315	-7.67	26.08	-1.86	
Present	2000	3.25	1.315	-7.67	26.08	-1.86	100
	4000	3.25	1.315	-7.67	26.08	-1.86	100
	6000	3.25	1.315	-7.67	26.08	-1.86	100
	8000	3.25	1.315	-7.67	26.08	-1.86	100
	9360	2.35	0.943	-4.26	22.69	-0.47	25.06
	9485	1.80	0.684	-2.78	20.14	-0.14	7.65
	9610	1.20	0.411	-1.54	16.65	-0.02	1.27
	9735	0.65	0.232	-2.70	12.32	-0.01	0.52
	9860	0.65	0.258	-7.09	12.32	-0.03	1.68
	9985	1.25	0.435	-8.16	17.03	-0.14	7.59
	10110	1.65	0.633	-8.82	19.35	-0.37	19.69
	10235	2.10	0.856	-9.04	21.63	-0.77	41.23
	10360	2.65	1.076	-8.83	23.95	-1.31	70.51
	11000	3.25	1.315	-7.67	26.08	-1.86	100
	13000	3.25	1.315	-7.67	26.08	-1.86	100
	15000	3.25	1.315	-7.67	26.08	-1.86	100
	17000	3.25	1.315	-7.67	26.08	-1.86	100
	19000	3.25	1.315	-7.67	26.08	-1.86	100

Table 5.5: Changes in longshore transport for 1 km island at 1 km offshore, $\phi_0 = 10°$

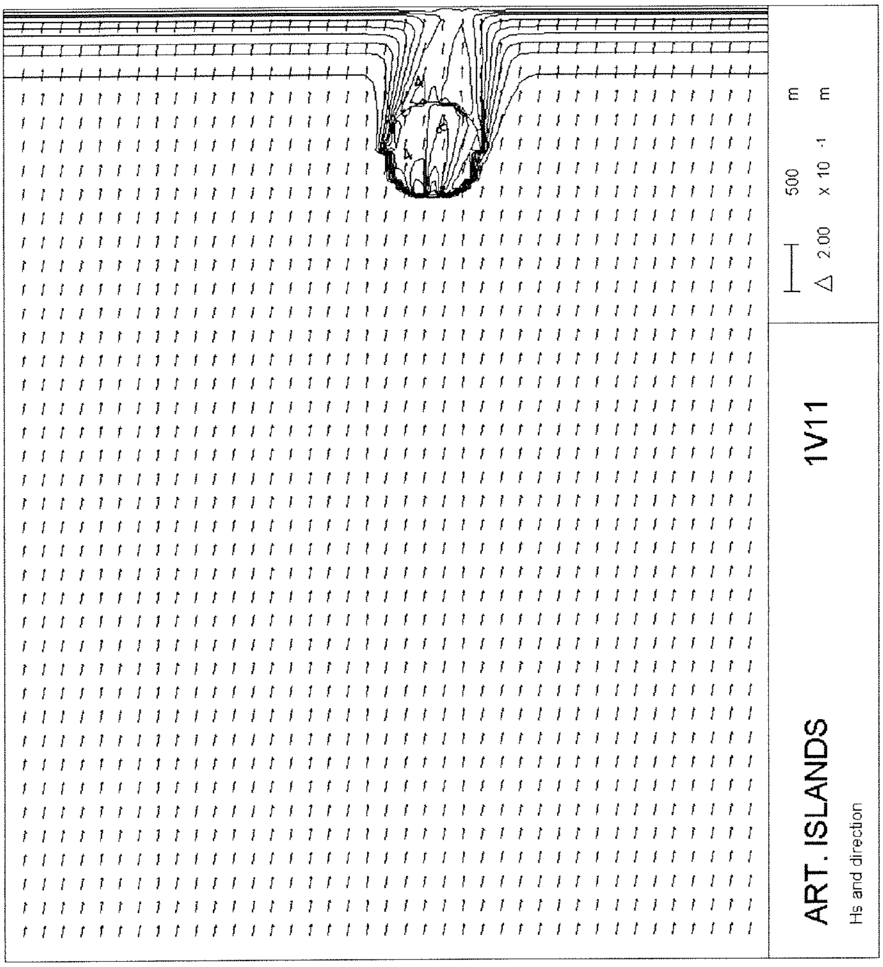


Figure 5.5: Iso-lines for H_s and vectors for wave direction; $D = 1$ km, $d = 1$ km and $\phi_0 = 10°$

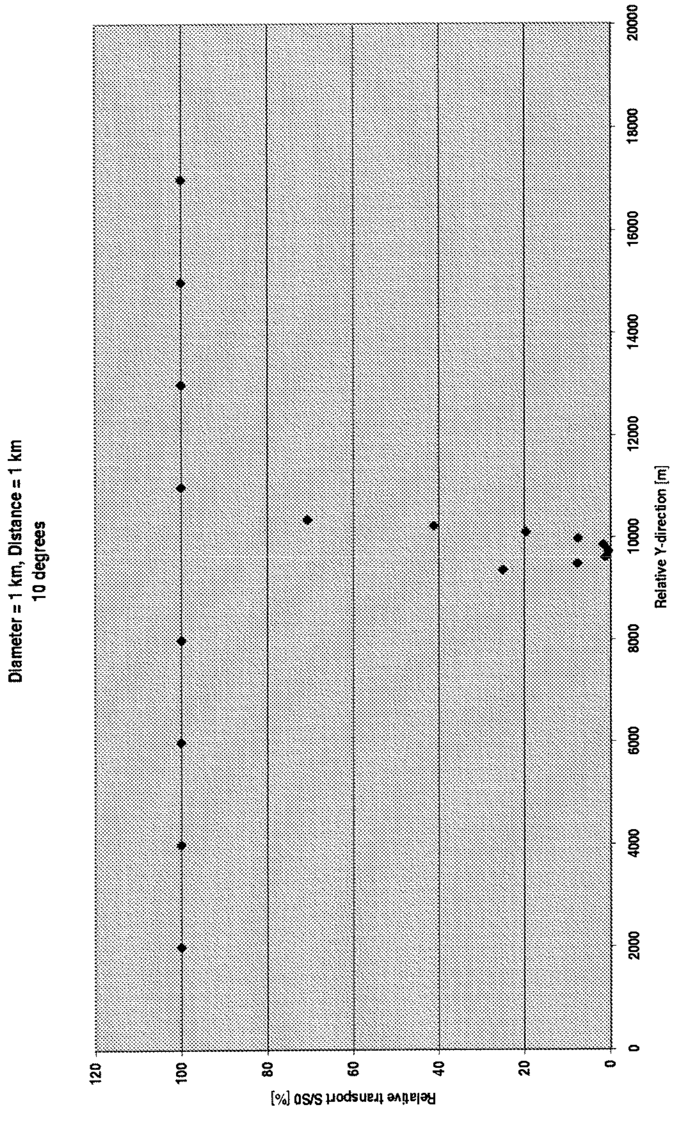


Figure 5.6: Relative changes in sediment transport after island construction, for details see example in paragraph 5.7

5.7.1 Sensitivity of HISWA for H_{s0} and bathymetry

For the same example as discussed in paragraph 5.7 the influence of several parameters on the changes in longshore transport are examined. Furthermore, these influences are examined as well, for the same example using a large directional spread of 31.5°. The parameters and their altered values are summed up in table 5.6.

H_{s0}	h_0	m	s_0
1.0 m	20 m	1 : 250	0.01
3.0 m	40 m		

Table 5.6: Altered parameters in sensitivity-analysis

All other parameters from the original example remain unchanged, and only one of the parameters, mentioned in table 5.6, is changed. Because one of the assumptions was to keep the wave steepness at a value of 0.05 (except of course for the sensitivity-analysis of the wave steepness), the wave period changes when the deepwater wave height is altered. The influence of the parameters is shown in figure 5.7 plus figure 5.8, and as a reference the relative transport of the original example is drawn as well. The measured dots are connected by lines to emphasize the differences between them, this is, of course, not the natural development of the longshore transports. For a more detailed look at the influence of the parameters, appendix F.2 and appendix F.3 both show seven tables with exact information.

Figure 5.7 and figure 5.8 show, that the influence of the mentioned parameters on the change of sediment transport is minimal. The difference between the original situation and the situation using new parameters, however, is greater in figure 5.8, because of the larger directional spread of the waves. The four parameters are discussed separately, hereafter.

- Deepwater wave height H_{s0}
  Although the absolute longshore transport is changed severely, when the deepwater wave height changes, the relative changes in transport remain almost unaltered. HISWA operates the same kind of transformations on the propagating waves when traveling toward the shore. Despite of the absolute change in wave length, the relation between the local wave height and the deepwater wave height is only changed minor.

- Deepwater bottom level h_o
  The influence of the bottom level on the propagating waves would only be noticeable if the wave height is higher than 0.5 times the water depth. In that case, the bottom level limits the wave height. This is by far not the case and consequently the changes in longshore transport are exactly the same for all three depths .

- Foreshore slope m
  The foreshore slope influences the offshore distance, where the waves break. However, it does not account for the depth at which the waves break, this depth remains unchanged. At this depth, the same relation exist between the percentage of broken waves in the initial situation and the situation after construction of the island. There is some little difference noticeable between the changes in longshore transport induced by the two slopes, because the refraction-pattern develops different under the influence of a flatter slope.

- Deepwater wave steepness s_0
  The wave steepness indicates the relation between wave height and deepwater wave

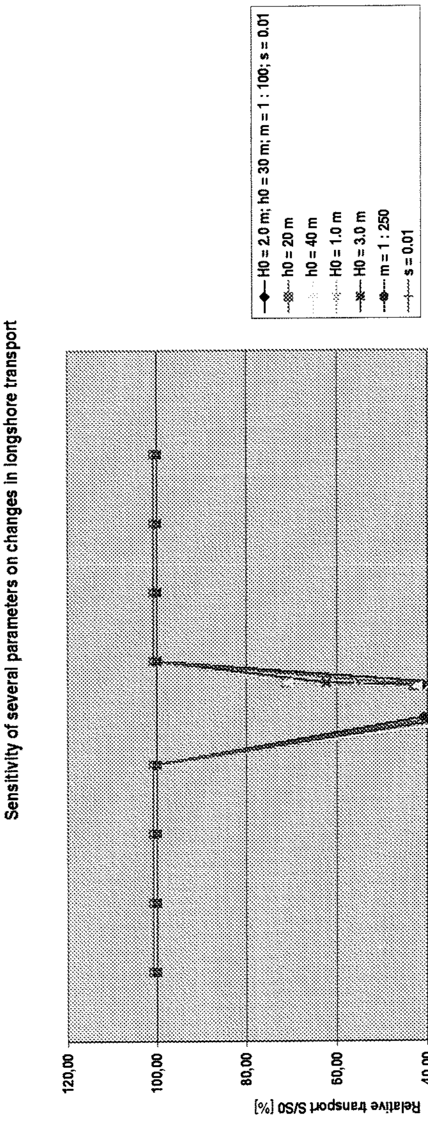


Figure 5.7: Influence of H_{s0} and bathymetry on changes in longshore transport, $\phi_0 = 10^\circ$ and directional spread is minimal

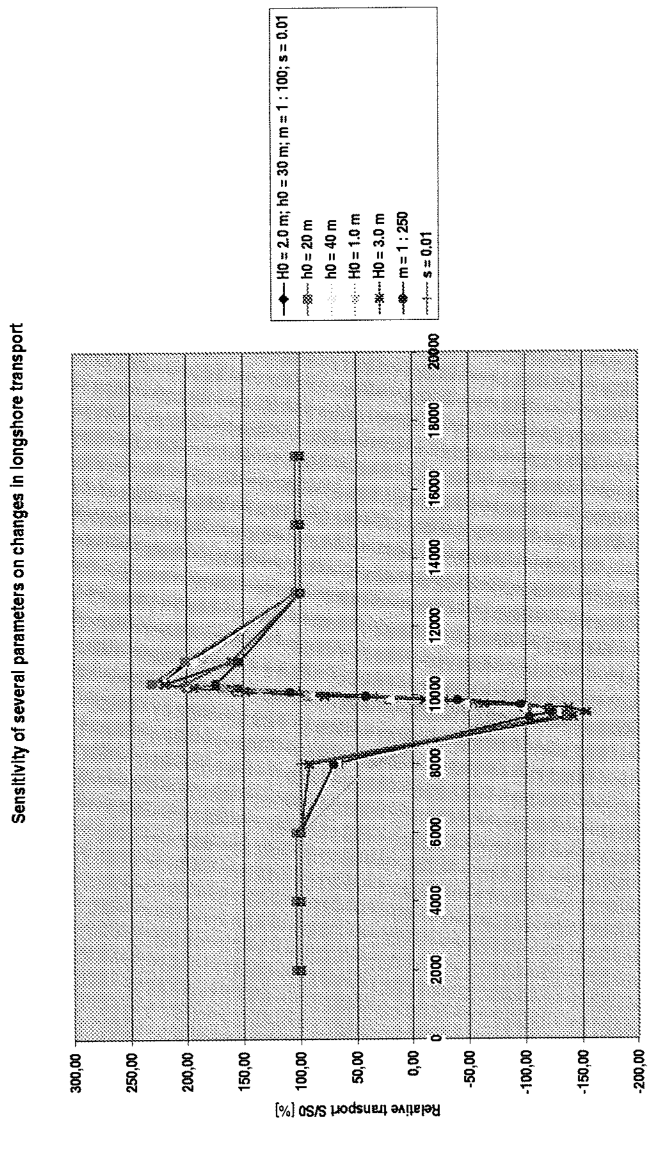


Figure 5.8: Influence of H_{s0} and bathymetry on changes in longshore transport, $\phi_0 = 10^\circ$ and directional spread is large

length. And consequently the wave period is influenced as well by the wave steepness. The wave period, the wave length and the position at which waves break are altered, Nevertheless, the ratio between initial wave heights and wave heights, after the island is constructed remains almost the same.

5.8 Conclusions and recommendations

Because the results of the computations differ severe for a minimal and a large directional spread, the conclusions are discussed in two separate paragraphs, viz. paragraph 5.8.1 and paragraph 5.8.2. The academic relation, as displayed in figure 5.3, is verified in this chapter's final section, for both spreads. A more quantitative approach to the proposed relation is suggested. Recommendations for future research are presented in paragraph 5.8.3.

5.8.1 Minimal directional spread

Appendix F shows the outcomes of the computations and the accompanying graphs. The results for a minimal directional spread are drawn separately from those with a large directional spread. None of these graphs show a value below zero, and no negative transports occur. Relative transport rates hardly exceed values of 100%. The most important reasons for these effects have been mentioned in paragraph 5.4.2, at the item *variable assumptions*.

The minimum value of the altered longshore transport is used to determine the guidelines. In some cases, this minimum value amounts to zero, which implies that total blocking of the longshore transport occurs. In other cases, a certain reduction is noticeable, but transport is still taking place. Table 5.7 shows, for all examined situations, the maximum reduction of longshore transport. The table indicates, that an island with a diameter of 4 km reduces the longshore transport to zero, in all examined cases. Consequently, islands of this size should be placed further offshore to minimize the effect on longshore transports. Furthermore, it is remarkable that the reduction for a deepwater angle of incidence of 45 degrees is less than for an angle of 10 degrees. The relative change in wave heights is for both angles of incidence in the same order of magnitude. The change in angle of incidence at the breaker line, however, is much less for deepwater angles of incidence of 45 degrees.

Figure 5.10 displays the relation between the relative minimum transport and the rate d/D; distance offshore and island's diameter, for minimal directional spread. Although the maximum computed relative transport amounts to about 60%, the academic relation of figure 5.3 can be seen (with some imagination) in figure 5.10. The figure reveals, that for a rate between the distance offshore and the islands diameter of less than about 1.25 to 1.5 , the longshore transport is totally blocked.

Diameter D [km]	**Distance d** [km]	**$\Delta S_{xmin} - 10°$** [%]	**$\Delta S_{xmin} - 45°$** [%]
1.0	1.0	None	None
	2.0	± 7%	± 12%
	5.0	± 40%	± 60%
2.0	1.0	None	None
	2.0	None	None
	5.0	± 8%	± 25%
4.0	1.0	None	None
	2.0	None	None
	5.0	None	None

Table 5.7: Minimal relative longshore transports, after island construction for minimal directional spread

The relation shown in figure 5.10 can be split up for each island's diameter individually. This relation is shown in figure 5.9. The picture shows more explicitly, that larger islands continue blocking longshore transports at greater distances offshore. In the same way, a larger distance offshore is required, for islands having greater surface areas, before the initial transport is reached again. The above described relation, is shown by the flatter S-curves, for greater diameters.

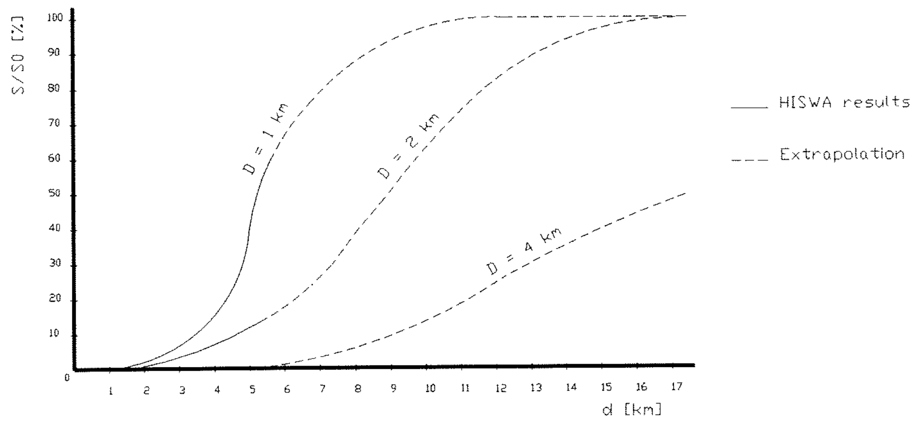


Figure 5.9: Minimum relative longshore transports, shown for each island's diameter individually. Directional spread is minimal

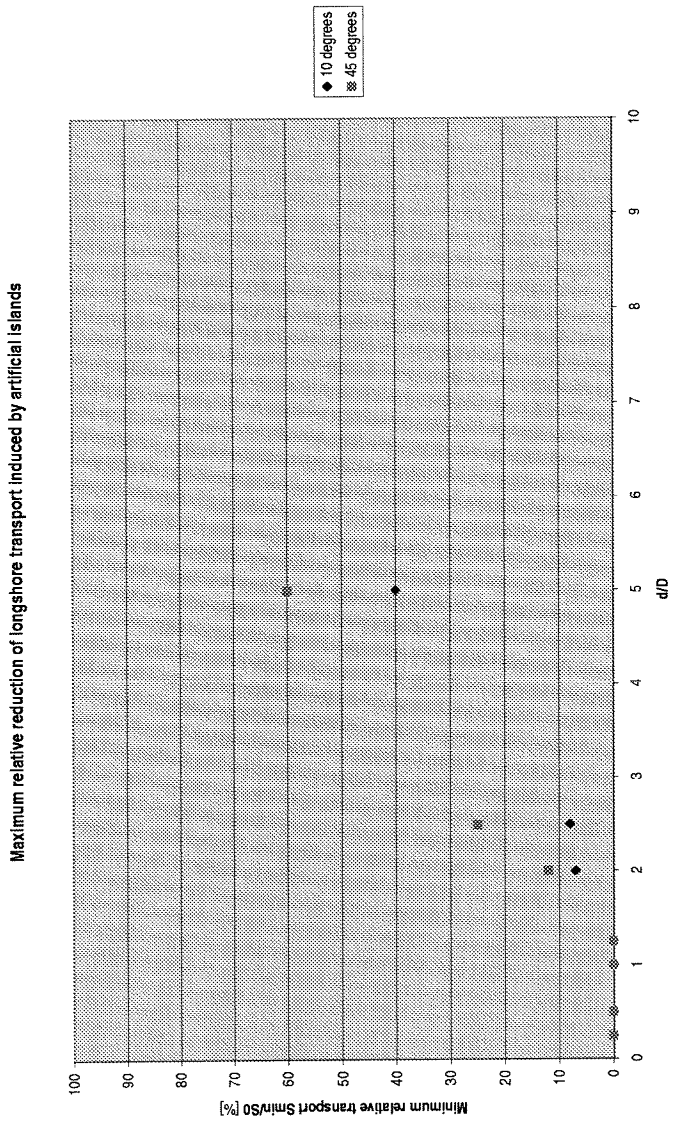


Figure 5.10: Minimum relative longshore transports, related to island's diameter D and distance offshore d. Directional spread is minimal

5.8.2 Large directional spread

A similar approach is applied here, as was done for the waves having a minimal directional spread. In this case, appendix F shows a totally different image. Relative transport rates change to a negative value frequently. Moreover, transport rates exceed 100%. This implies that the transport rates after the construction of the island, exceed the values in the initial situation. Both phenomena are explained in paragraph 5.4.2, at the item *variable assumptions.*

Again, the minimum values of the altered longshore transport are used to determine the guidelines. If a value is negative, it is considered zero. A negative transport will result in total blocking of the longshore transport as well. Table 5.8 shows, for all examined situations the maximum reduction of longshore transport. Total blocking hardly occurs for deepwater angles of incidence of 45°, and only at distances close to shore. This is caused by the fact, that these large angles do not generate negative angles of approach, even if the directional spread is large. This in contrary to a deepwater angle of incidence of 10°. Blocking occurs for all situations, except two. The large directional spread is also responsible for the fact that less blocking occurs at 45° for a large spread than at 45° having a minimal spread. Due to the large directional spread the change of waves reaching the lee-side of the island is much larger.

Figure 5.11 displays the relation between the relative minimum transport and the rate d/D; distance offshore and island's diameter, for large directional spread. Although the maximum computed relative transport amounts to about 80%, the academic relation of figure 5.3 can be seen (with some imagination) in figure 5.11. Compared to figure 5.10, the 45°-curve runs much steeper and the 10°-curve much flatter. In words this means, that in case of a large directional spread and a deepwater angle of approach of around 45° man-made islands can be placed closer to shore. On the other hand, if waves have a small deepwater angle of approach and a large directional spread, blocking of longshore transport is more severe. Total blocking still occurs for a rate d/D of about 2 - 2.5!

5.8.3 Recommendations for future research

It is clear, that the above executed study is by far not complete. Consequently, room for future research is available. Below, some research suggestions are summed up.

Diameter D [km]	**Distance d** [km]	**$\Delta S_{xmin} - 10°$** [%]	**$\Delta S_{xmin} - 45°$** [%]
1.0	1.0	None	± 6%
	2.0	None	± 50%
	5.0	± 50%	± 80%
2.0	1.0	None	None
	2.0	None	± 15%
	5.0	± 1%	± 65%
4.0	1.0	None	None
	2.0	None	None
	5.0	None	± 35%

Table 5.8: Minimal relative longshore transports, after island construction for large directional spread

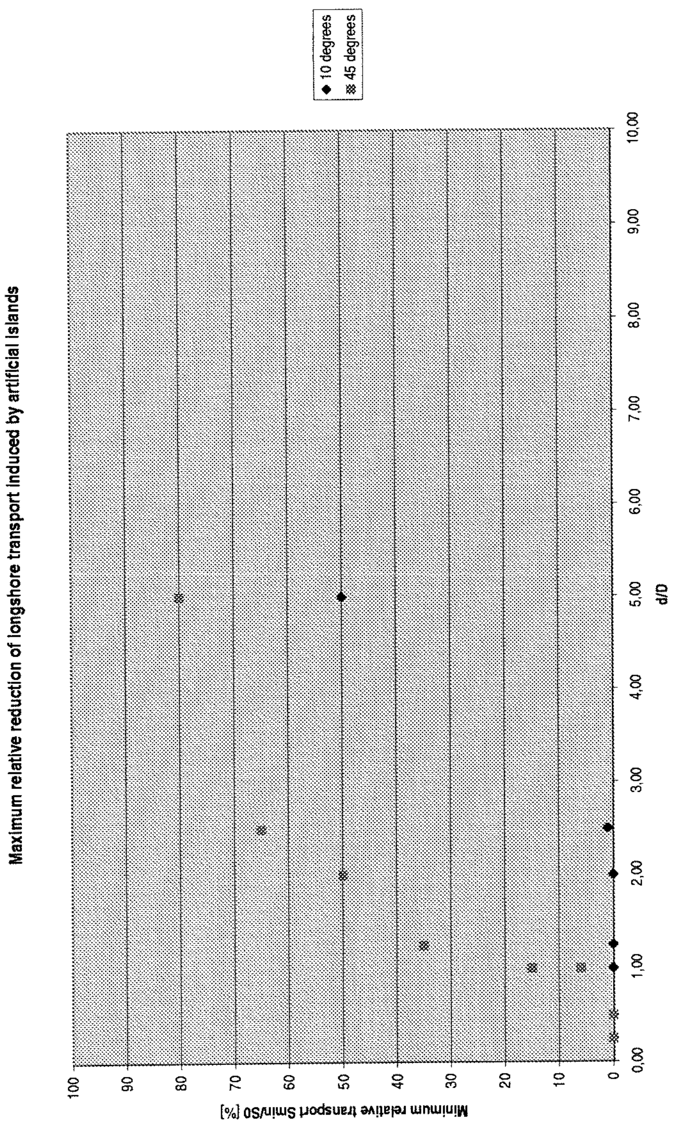


Figure 5.11: Minimum relative longshore transports, related to island's diameter D and distance offshore d. Directional spread is large

- Influence on transport for larger distances offshore
  Because the maximum relative transport value is reached at values of 60% and 80%, the development of the areas above can only be given hypothetically. Exact values for the relation between the distance offshore and the island's diameter are required in these regions.

- Using other transport formulae
  This report only used the CERC-formula to come up with the changes in longshore transports. The CERC-formula only takes into account the effects of waves on transports. Other formulae should clarify the effects of currents and the effects of a combination of waves and currents. Moreover, the results of this report can be verified.

- Using a more natural approximation of nature
  In this report two angles of incidence are assumed, both with a minimal directional spread and a large directional spread. It would be interesting to examine the effects of an even more realistic approach, for example testing the effects of seasonal dominant wave directions.

- Varying the shape of the island
  Only a circular island has been assumed. It is very likely that some other shape, having the same surface area, will result in less reduction of the longshore transport. Which dimension, or combination of dimensions of the island influences longshore transports most? The width, the length, etc.

Chapter 6

Conclusions and recommendations

6.1 Introduction

The objective of this report, as stated in chapter 1, is to present useful guidelines to designers, involved in the design and construction of reclaimed artificial islands. After having executed a thorough research of literature, the guidelines could be narrowed down to several subjects, all related to the field of coastal engineering. Throughout this report guidelines and recommendations are presented. This chapter summarises all these guidelines and recommendations once again. The guidelines are discussed in paragraph 6.2, and the recommendations in paragraph 6.3.

6.2 Conclusions

Some guidelines are derived from literature. Missing knowledge, and most common problems with the design and construction of artificial islands gave cause for the study performed in this report. The most important guidelines, derived from literature are:

- In practice, waterdepths are restricted to $\pm$ 30 m, mostly based on economics
- In practice, an increase in island size involves a decrease in waterdepth
- Hardly any artificial polder islands have been built yet

The guidelines formulated on the basis of this report's study are presented in three paragraphs, hereafter.

6.2.1 Use of sea defences

Five types of sea defence structures are included in the study, namely rubble mound sea defences armoured with quarry stones, dolosse, tetrapods and modified cubes as well as vertical caisson-type seawalls. Using these types of defences, islands can be employed in deeper water and in more severe wave climates.

All sea defence structures are designed, fulfilling the same demands, for instance the demand of minimal overtopping. Furthermore, the mass of dolosse and tetrapods is limited to 20 tons. This way, the risk of concrete fractures is avoided. If designers know the waterdepth at the toe and the accompanying wave height, guidelines are presented for two kinds of wave steepnesses, viz. seas and swell. The most important conclusions resulting from these guidelines are:

For seas:

- Quarry stone sea defences are the most economical examined structures, at waterdepths shallower than 3 to 7 m, depending on the relative costs of the quarry stones.
- Vertical caisson-type seawalls result to be most economic, at waterdepths over approximately 25 to 28 m, and wave heights exceeding 10 to 12 m, depending on the relative costs of the quarry stones.
- Within the large intermediate range, dolosse type sea defences are economically preferred. Their range depends also on the relative costs of the quarry stones.

For swell:

- Because quarry stones are more severely loaded by swelling waves, quarry stone sea defences are economically only up to waterdepths of about 3 m, even if the relative costs of the quarry stones is reduced.
- Because waves heights over about 6 m hardly occur at swell, higher wave heights are not accounted for. For waterdepths over approximately 3 m, dolosse type sea defences are economically preferred.

6.2.2 Overall costs related to size of island

Larger reclaimed islands can be constructed more economic than smaller islands. Landfill for reclaimed artificial islands can originate from marine and land excavation sites. This report only details landfill mined from marine excavation sites. Marine landfill can best be excavated using trailing suction hopper dredges, if the sailing distance between excavation site and island location is large. The cost for marine landfill depends, among others, on the cycle time of the trailing suction hopper dredge.
Costs for the sea defence structures generally account for the majority of the total island's costs. If the surface area of an island grows, the length of the sea defences grows proportional to square root and the volume of the landfill linear. Consequently, the overall costs per square metre are less for larger islands than for smaller islands.

6.2.3 Morphological impact of artificial islands on longshore transports

A reclaimed island placed some distance offshore induces a low *wave* energy zone behind it. In this, so-called shadow-zone wave heights will be reduced, and consequently a reduction of longshore transport occurs. A reduction in longshore transport can cause accretion in the shadow-zone, and erosion in the adjacent areas.

For a circular artificial island the relative influences on longshore transport have been examined, varied for the island's diameter D and its distance offshore d. Only the influence of waves is accounted for, the influence of currents is left out of consideration. Seas have been assumed present at deepwater. For two conditions guidelines are presented, namely a large directional spread (which approximates seas best) and a minimal directional spread. On the basis of these guidelines, designer can deliberate the extra costs of placing the island further offshore, generating less erosion, or placing the island closer to shore, and spending more money on overcoming erosion and accretion problems. The following guidelines result:

Large directional spread:

- For a deepwater angle of incidence of 10 degrees, relative transports exceed 100%.
- Total blocking of longshore transports hardly occurs for deepwater angles of incidence of 45 degrees, and only at rates $d/D < \pm\ 0.5$.
- At a rate $d/D = \pm\ 2$, a minimal relative longshore transport of about 50% is present behind the island, for deepwater angles of wave incidence of 45 degrees.
- For deepwater angles of incidence of about 10 degrees, total blocking occurs even when the island is placed far offshore, up to a rate $d/D < \pm\ 2$ - 2.5. Because the angle of incidence is small and the directional spread large, negative transports can result. The waves having a large angle of incidence are blocked by the island at its lee-side. For this same reason, islands should be placed further offshore to avoid total blockage in comparison to situations with a minimal directional spread.
- At a rate $d/D = \pm\ 5$, a minimal relative longshore transport of about 50% is present behind the island, for deepwater angles of wave incidence of 10 degrees.

Minimal directional spread:

- Longshore transport is totally blocked for a rate $d/D < 1$ - 1.5.
- At a rate $d/D = \pm\ 5$, a minimal relative longshore transport of about 50% is present behind the island, depending on the deepwater angle of wave incidence.

6.3 Recommendations

The above presented guidelines can be enlarged, by means of further research. Suggestions for future research are given for two of the three subjects.

6.3.1 Use of sea defences

- Changing unit price costs
  The study revealed a very large influence of the quarry stone and concrete prices on the results of the guidelines. Although a value a was introduced to account for additional quarry stone costs, altered composition of the quarry stones, as stated in table 3.1, may result in a more widely usability of the guidelines.

- Using modern formulae for armour calculations
  The study was performed using both Hudson and Van der Meer formulae. The Hudson formula is somewhat outdated, and new insights are not included. Therefor, the use of new formulae, and applying of new methods is desirable. Furthermore, a more uniform result is obtained in this manner.

- Examining the economic applicability of 'new' armour elements
  Newer armour elements, like accropods and core-loc, which can be applied in a single layer mat prove to be more economical than dolosse for instance. However, the failure mechanisms of these type of elements needs to be thoroughly examined.

- Examining non-hard sea defences
  Hard sea defences, like rubble mound and caissons, were used in the performed study. Soft sea defences, or combined defences were not taken into account. These types of sea defences, however, may prove very useful at limited depths or under less severe wave attack (for instance at the lee-side of the island).

6.3.2 Morphological impact of artificial islands on longshore transports

- Influence on transport for larger distances offshore
  Because the maximum relative transport value is reached at values of 60% and 80%, the development of the areas above can only be given hypothetically. Exact values for the relation between the distance offshore and the island's diameter are required in these regions.

- Using other transport formulae
  This report only used the CERC-formula to come up with the changes in longshore transports. The CERC-formula only takes into account the effects of waves on transports. Other formulae should clarify the effects of currents and the effects of a combination of waves and currents. Moreover, the results of this report can be verified.

- Using a more natural approximation of nature
  In this report two angles of incidence are assumed, both with a minimal directional spread and a large directional spread. It would be interesting to examine the effects of an even more realistic approach, for example testing the effects of seasonal dominant wave directions.

- Varying the shape of the island
  Only a circular island has been assumed. It is very likely that some other shape, having the same surface area, will result in less reduction of the longshore transport. Which dimension, or combination of dimensions of the island influences longshore transports most? The width, the length, etc.

Bibliography

[1] Various; *International Conference on Industrial Islands* (November 1981).

[2] Ferguson, Hugh; 'Dutch meet the causeway challenge', *New Civil Engineer,9/16* (December 1982).

[3] Hansen / Rordam Thomsen / Sjogren / Sterndorff; 'Artificial Islands as Protection Against Ship Impact', *Proceedings of the Fourth (1994) International Offshore and Polar Engineering Conference Osaka, Japan* (1994).

[4] Michael Z. Sincoff / Jarir S. Dajani / Editors; *Planning and Evaluation Parameters for Offshore Complexes*(September 1976).

[5] Paul R. Ryan, and Michael A. Champ; 'Oceanic Architecture and Engineering in Japan', *Oceanus 12* (1986).

[6] Kondo, Takeo; 'Technological Advances in Japan's Coastal Development: Land Reclamation and Artificial Islands', *Marine Technology Society Journal. v29 n3* (1995).

[7] Eguchi, Hajime; 'Highlights of marine Civil Engineering in Japan', *Coastal Ocean Space Utilisation III.*

[8] *An Evaluation of Multi-Purpose Offshore Industrial port Islands, Final Technical Report to NSF-RANN* (May 1975).

[9] Fukue, Masaharu / Kawakami, Tetsutaro; 'Damage to Man-Made Islands caused by the 1995 Hyogoken Nanbu Earthquake', *Marine Georesources and Geotechnology v14 p 237-250* (1996).

[10] Cheng, Bao-rong /Zheng, Zhao-chang / Su, Jian; 'The Seismic Response Analysis for an Artificial Island Using a Pseudo-Static Method', *Proceedings of the Third (1993) International Offshore and Polar Engineering Conference Singapore, 6-11 June 1993 p 258-262.* ISBN 1-880653-05-2

[11] Yamakawa, Masafumi / Tsuruoka, Tatsuhiko; 'Sand Drain Works for a Large-Scale Man-Made Island', *Marine Georesources and Geotechnology v14 p 251-262* (1996).

[12] Uchida, Keinosuke / Iida, Yutaka / Yoshida, Yoshitaka / Imai, Koji; 'Special Ground Improvement Methods: Ground Improvements Associated with the Trans-Tokyo Bay Highway', *Marine Georesources and Geotechnology v14 p 47-63* (1996).

[13] Foott, Roger / Koutsoftas, Demetrious C. / Handfelt, Leo D.; 'Test Fill at Chek Lap Kok, Hong Kong', *Journal of Geotechnical Engineering v113 n2 Feb. p 106-126* (1987). ISSN 0733-9410/87/0002-0106/...

[14] Tohma, Toshiaki / Yamamoto, Shuji; 'Construction of the Kansai International Airport', *Civil Engineering in Japan* (1990).

[15] Gehbauer, Fritz; 'Der Bau des neuen Kansai Flughafens in Japan', *Die Bautechnik Ausgabe A. v70 n4* (1993).

[16] Terashi, M. / Tanaka, H.; 'Settlement Analysis for Deep mixing Method', *Proc. 8th European Conf. on SMFE v2 p 955-960* (1980).

[17] Solymer, Z.V. et al.; 'Ground Improvement by Compaction Piling', *Proc. ASCE v112 nGE12 p 1069-1083* (1986).

[18] Burcharth, H.F.; 'The design of breakwaters', *Coastal Estuarial and Harbour Engineers' Reference Book p 381 - 424* (1994).

[19] Luger, S.A. / Phelp, D.T. / van Tonder, A. / Holtzhauzen, A.H.; 'Increased Dolos Strength by Shape Modification', *Coastal Engineering 1994 - Proc. of the twenty-fourth int. conf. v2 p 1388 - 1396* (1994).

[20] Melby, Jeffrey A. / ASCE, A.M. / Turk, George F.; 'The Core-Loc: Optimized Concrete Armor', *Coastal Engineering 1994 - Proc. of the twenty-fourth int. conf. v2 p 1426 - 1438* (1994).

[21] Phelp, D / Luger, S / van Tonder, A / Holtzhausen, A; 'Results of Extensive Field Monitoring of Dolos Breakwaters', *Coastal Engineering 1994 - Proc. of the twenty-fourth int. conf. v2 p 1511 - 1525* (1994).

[22] d'Angremond, K. / van der Meer, J.W. / van Nes, C.P.; 'Stresses in Tetrapod Armour Units induced by Wave Motion', *Coastal Engineering 1994 - Proc. of the twenty-fourth int. conf. v2 p 1713 - 1726* (1994).

[23] 'Caisson-type breakwater', *CIRIA Special Publications 83/CUR Report 154 p 369 - 273* (1991).

[24] Symposium; *De zee als bouwlocatie - fictie of noodzaak? TUD April 24th* (1996).

[25] Massie, W.W.; *Coastal Engineering v1 Introduction TUD* (1982).

[26] van der Schrieck, G.L.M.; *Baggertechniek TUD* (April 1997).

[27] Velden, van der E.T.J.M.; 'Coastal Engineering', *F7, collegehandleiding TU Delft* (January 1995).

[28] N. Booij, L.H. Holthuijsen; *HISWA User Manual*, Delft University of Technology (1996).

[29] J.D. den Adel, H.D. Niemeyer, A.F. Banken, N. Booij, J. Dekker, J.A. Vogel; 'Wave model application in a wadden sea area', *Coastal Engineering p 530-543* (1990).

[30] Coastal Engineering Research Center; *Shore Protection Manual* (1984).

[31] Tomotsuka Takayama; 'Development of new caisson type breakwaters in Japan and a probabilistic design approach for composite breakwaters', *Wave Forces on Inclined and Vertical Wall Structures* (1995).

Appendix A

Collection of artificial islands (1955 - 1997)

A.1 Survey of artificial islands in North and South America

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Treasure Island				-1900		Anchor for Oakland Bay Bridge	T&A	San Francisco, USA
Government Island		271		-1913		Life Guard base	T&A	Oakland Estuary, USA
Rincon Island	12.4 - 14.6	25.5	5	1955-1958	Tetrapods and rock revetments	Oil drilling base	M&E	Santa Barbara, USA
Delaware Bay Island	24	38800		1959-1981		Iron ore & coal Transfer Terminal	Port	Delaware Bay, USA
Belmont Island + Esther Island		5	2.4	-1965		Oil drilling base	M&E	Long Beach, USA
THUMS Islands (#4)	7.6 - 12.2	40	< 1	1965-1966	Quarry waste and armor rock for protection	Oil drilling base	M&E	Long Beach, USA
A Nuclear Power Plant		167	0.85	Study (1965)		Nuclear power plant	EG	Long Beach, USA
No Name (#4)				1967-1968		Ice movement control for navigability	T&A	Lac St. Pierre, Canada
Atlantic Generating Station	10		5	1970 -	Floating structure surrounded by dolosse breakwater	Nuclear power plant	EG	Atlantic City, New Jersey, USA
North Atlantic Deepwater Oil Terminal	15 - 18	810	13	Study (1971)		Oil transfer terminal	Port	Delaware Bay, Long Branch, Raritan Bay, USA
Areia Branca	7 - 16.5		13	-1974	Sheetpiles filled with dredged sand, Riprap protection	Salt transshipment terminal	Port	Areia Branca, Brazil
Lower Delaware Bay Industrial Port Isla	12.2 - 20	3480-14000	14	Study (1977)	Build on a shoal	Industrial port island	II	Delaware Bay, USA
ICONN Island Project	23	15000	19	Study (1977)	Dredged material from Erie Canal as fill	Industrial Island	II	Lower N.Y. Bay, USA
Zapata Coal Export Island		1220-3240	5	Study (1977)		Coal export terminal	Port	Delaware Bay, USA
Sohio's Resolution Island	3 - 12	± 10	9	± 1980		Oil drilling base	M&E	Alaska North Slope
Endeavor Island	3 - 12	± 10	5	± 1980		Oil drilling base	M&E	Alaska North Slope
Exxon Island	3 - 12	± 10	9	± 1980		Oil drilling base	M&E	Alaska North Slope
Prudhoe Bay				-1984		Seawater treatment plant	WH	USA
No Name (#6)				1982-1984		Oil production platform	M&E	Norman Wells, USA
Governors Island				?				N.Y. Harbor, USA
Hoffman & Swinburne Islands				?	Waste construction materials used as fill	Undeveloped	WI	N.Y. Harbor, USA
Hart-Miller Island				?		Storage for dredged material	WH	Chesapeake Bay, USA
Great Lakes Islands				?		Erosion prevention	T&A	Great Lakes, USA

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Aero-Isla				Study (1995)		Airport	Air	Buenos Aires, Argentina

A.2 Survey of artificial islands in Asia and Australia (exclusive Japan)

Name	Waterdepth [m]	Area [1,000 m^2]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Work Island Bahrain Causeway	0 - 5	300	4	1981-1983	Seawalls + sandfill	Work Island for Bahrain Causeway	T&A	Saudi Arabia
Zubaya	shallow	72.2	15	1980 - 1982	Slopes 1:3, using slope bags and concrete blocks	Exploratory oil drilling base	M&E	Abu Dhabi, U.A.E.
Halat Hail	shallow	72.2	30	1980 - 1982	Slopes 1:3, using slope bags and concrete blocks	Exploratory oil drilling base	M&E	Abu Dhabi, U.A.E.
Pulau Busing				1983		Oil terminal storage	EG	Singapore
Terembu Pesek				1985		Pig quarantine storage	T&A	Singapore
Pulau Busing		500		-1983		Oil storage terminal	M&E	Singapore
Pulau Sakra + Pulau Bakau		1550		-1984	Two natural islands extended to one man-made island			Singapore
Terumba Pesek				-1985		Holding station for imported pigs	T&A	Singapore
Chek Lap Kok	10 - 12	9300	1	1992 -	Reclamation	Airport	Air	Hong Kong
Islands (#4)	10 - 25	1000	1	Study (1996)	Reclamation	Commercial and industrial zones	CC	Haifa, Tel-Aviv Ashdod, Gaza, Israel
Macao Airport				1994-		Airport	Air	Macao
No Name				Study		Airport	Air	Australia
Industrial island		< 15000		Study	Seawalls + fill	Industrial zones	II	Lookang, Taiwan

A.3 Survey of artificial islands in the Beaufort Sea (Canada)

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Exploration Operator	Code	Location
Immerk B-48	3.0	± 10	± 10	-1973	SBI	Esso Resources	M&E	
Adgo F-28	2.1	± 10	± 3	-1973	SRI	Esso Resources	M&E	
Pullen E-17	1.5	± 10	± 8	-1973	GHI	Esso Resources	M&E	*All structures*
Unark L-24	1.3	± 10		-1974	GHI	Sun Oil	M&E	*are situated*
Pelly B-35	2.0	± 10		-1974	BCI	Sun Oil	M&E	*in the Beaufort Sea,*
Netserk B-44	4.6	± 10		-1974	SRI	Esso Resources	M&E	*Canada*
Adgo P-25	1.5	± 10	± 8	-1974	SRI	Esso Resources	M&E	
Adgo C-15	1.5	± 10	± 10	-1975	GHI	Esso Resources	M&E	
Netserk F-40	7.0	± 10	± 6	-1975	SRI	Esso Resources	M&E	
Sarpik B-35	4.3	± 10		-1975	GHI	Esso Resources	M&E	
Ikkatok J-47	1.5	± 10		-1975	SRI	Esso Resources	M&E	
Kugmallit H-59	5.3	± 10		-1976	SRI	Esso Resources	M&E	
Adgo J-27	1.8	± 10		-1976	SRI	Esso Resources	M&E	
Arnak L-30	8.5	± 10		-1976	SBI	Esso Resources	M&E	
Kannerk G-42	8.5	± 10		-1976	SBI	Esso Resources	M&E	
Isserk E-27	13.0	± 10		-1977	SBI	Esso Resources	M&E	
Issungnak 0-61	19.0	± 10	± 36	-1979	SBI	Esso Resources	M&E	
Issungnak 2-061	19.0	± 10		-1980	SBI	Esso Resources	M&E	
Alerk P-23	10.5	± 10	± 36	-1981	SBI	Esso Resources	M&E	
N. Protection Island	4.6			-1981		Dome Petroleum	T&A	
W. Atkinson L-23	7.5	± 10		-1981	SBI	Esso Resources	M&E	
Tarsiut N-44	21.0	± 10	± 64	-1981	CRI	Gulf Canada	M&E	
Uviluk P-66	29.7	± 10	± 53	-1982	SSDC	Dome Petroleum	M&E	
Itiyok I-27	15.0	± 10		-1982	SBI	Esso Resources	M&E	
Nerlerk B-67	45.1	± 10		-1983	SSDC	Dome Petroleum	M&E	
Kogyuk N-67	28.1	± 10		-1983	SSDC	Gulf Canada	M&E	
Kadluk 0-07	14.0	± 10		-1983	CRI	Esso Resources	M&E	

LEGEND:

SRI: Sandbag Retained Island
GHI: Gravel Hauled Island
SBI Sacrificial Beach Island
CRI: Caisson Retained Island
BCI & SSDC: Ballasted Barge Island

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Amerk P-09	26.0	± 10		-1984	CRI	Esso Resources	M&E	
Adgo H-29	3.0	± 10		-1984	SBI	Esso Resources	M&E	
Nipterk I-19	11.7	± 10		-1984	SBI	Esso Resources	M&E	*All structures*
Tarsiut P-45	26.0	± 10		-1984	CRI	Gulf Canada	M&E	*are situated*
Minuk I-53	14.7	± 10		-1985	SBI	Esso Resources	M&E	*in the Beaufort Sea,*
Amauligak I-65	31.0	± 10		-1985	CRI	Gulf Canada	M&E	*Canada*
Kaubvik I-43	17.9	± 10		-1985	CRI	Esso Resources	M&E	
Amak K-06	7.2	± 10		-1985	SBI	Esso Resources	M&E	
Amauligak F-24	32.0	± 10		-1987	SBI	Gulf Canada	M&E	

LEGEND:

SRI: Sandbag Retained Island
GHI: Gravel Hauled Island
SBI Sacrificial Beach Island
CRI: Caisson Retained Island
BCI & SSDC: Ballasted Barge Island

A.4 Survey of artificial islands in Europe

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Work Island Haringvliet	5 - 8	784	2	1957-1959		Temporary sluice construction site	T&A	Haringvliet, the Netherlands
Work Island				1958	Reclamation	Temporary sluice construction site	T&A	Haringvliet, the Netherlands
Work Island Ballastplaat	5	600	3	1961-1964		Construction site for sluice-gates and part of the Lauwerszee closure dam	T&A	Lauwerszee, the Netherlands
Work Island Roggenplaat	7	350	2.5	1969-1971		Aid for the closure of the East Scheldt	T&A	East Scheldt, the Netherlands
Arneplaat	0.5	17.5	0.7	1970	Constructed on a shoal	Marina and recreation site	Rec	Veerse Meer, the Netherlands
Work Island Neeltje-Jans	5	600	3.5	1970-1972		Aid for the closure of the East Scheldt	T&A	East Scheldt, the Netherlands
Work Island Noordland	5	350	2	1970-1972		Construction site for sluice-gates and part of the East Scheldt closure dam	T&A	East Scheldt, the Netherlands
De Hondt				1971	Reclamation	Exploratory gas drilling base	M&E	Eems Estuary, the Netherlands
Work Island Brouwersdam	7.5	90	1.5	1974-1976		Construction site for sluice-gates and part of the Brouwersdam	T&A	Grevelingen, the Netherlands
Industrial Island	20 - 25	3000	45	Study (1975)	Reclamation	Industrial zone	II	Rotterdam, the Netherlands
Coxyde-Panne			3.25	Study (1975)		Waste-processing & Nuclear power plant	II	La Panne, Belgium
Plaster storage area	3 - 4	350	0.1	1977-1987	Seawalls + fill	Plaster storage	WH	Landskrona, Germany
Oysterfarm	< 10	3500	5	Study (1978)		Mariculture	Mar	Western Scheldt, the Netherlands
LNG & LPG Terminal	11 - 12	300	9	Study (1979)		LNG & LPG Terminal	Port	Europoort, the Netherlands
Pump Accumulation Plant	15 - 20	13000	16	Study (1986)	Reclamation	Pump Accumultion Plant	EG	IJmuiden, the Netherlands
Rotterdam Slufter	5	3000	<0.5	Study (1986)	Sanddike + polluted dredged material for fill	Waste storage area	WH	the Netherlands

Maritime Island Depot	5 - 10	4900-8500	10 - 29	Study (1987)		Waste storage area	WH	8 possible locations Continental Shelf, Belgium
Waste Island	< 10	> 7050	± 6	Study (1988)		Waste storage and energy generation [PAC]	WI	Outer Western Scheldt, the Netherlands
Islands around bridge-piers	7 - 10	30	<3	Study (1994)	Armor stones imbedded in impermeable stones	Protection against ship impact for Great Belt Link-bridges	T&A	Denmark
Øresund Fixed Link Island		1300	1.5	± 1995-1997		Connection between tunnel and bridge. Part of Øresund Fixed Link	T&A	Copenhagen, Denmark
Schiphol IJpoort	10	14600	6	Study(1996)	Reclamation	Airport	Air	IJmuiden, the Netherlands
Shiphol new, designed by Delft Hydraulics	10 - 15	24000	10	Study (1997)	Reclamation	Airport	Air	IJmuiden, the Netherlands
Neuwerk Scharhöm				Study		Deep sea industrial port	Port	Elb Estuary, Germany
North Sea Port				Study		Oil terminal, nuclear power plant and desaltinization plant	II	Zeebrugge, Belgium
l'Ile de Parfond				Study		ULCC port	Port	Le Havre, France

A.5 Survey of artificial islands in Japan

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Osaka South Port	10	9370	0	1958-1984	Seawall + clayfill	Harbor and commercial facility, urb. dev. site exchange facility	CC	Japan
Anezaki Kaigan		5450		-1967	Reclamation	Industrial zone	II	Chiba, Japan
Nagasaki Mitsui-Miikie Island (#3)	10	6	6	1969-1970	Seawall + soft soil	Vertical ventilation shaft for undersea., coal mine	T&A	Japan
Yokohama, Daikoku	12	3210	0.5	1963-1985	Seawall+siltfill	Harbor facility, coastal park	Port	Japan
Nagoya Kinjo	0 - 5	1910	1.4	1963-1985	Seawall + soft soil	Harbor, foreign trade facility exhibition hall, park	CC	Japan
Kobe Port Island	10 - 12	4360	0.2	1966-1981	Seawall + clayfill	Port city	II	Japan
Yokkaichi, Kasumigaura	4.5 - 12	3870	0.1	1967-1988	Seawall + soft soil	Harbor facility, industrial land	II	Japan
Nagasaki Airport	10 - 18	1630	1.5	1971-1974	Seawall + clay-basalt-fill	Airport	Air	Japan
Ohgishima Reclamation Project	0 - 15	5150	0.4	1971-1975	Concrete caissons + fill	Industrial Island	II	Japan
Rokko Island	10 - 14	5800	0.2	1971-1992	Seawall + clayfill	Port city and urban development site	CC	Japan
Higashi, Ogishima	1 - 10	4340	0,7	1972-1984	Seawall+siltfill	Harbor and transportation facility	II	Japan
Osaka North Port	10	6150	0.45	1972-1988	Seawall + clayfill	Harbor and waste disposal facility, industrial land	II	Japan
Central Breakwater in the Port of Tokyo	5 - 10	3140	1	1974-1994	Waste-Soil-Sandwich	Urban development site	CC	Japan
Kumamoto		1370		1974-	Reclamation	Ferry terminal, and facilities for recreation	Port	Kumamoto, Japan
Nagoya Port Island	6 - 7.5	1140	1.2	1975-1987	Seawall + clay-silt-fill	Port	Port	Japan
Kanda Earth Dump	7.5	1530	3.5	1977-1986	Seawall + soft soil	Dredged sand dump, park	WI	Japan
Ishigaki		730		1979-	Reclamation	Facilities for recreation	Rec	Ishigaki, Japan
Goboh Fossil Fuel Power Plant Project	5 - 18	350	0.2	1980-1983	Caissons, 20t blocks + fill sandstone	Electrical power plant	EG	Hidaka, Japan
Osaki		6310		-1981	Reclamation	Industrial zone	II	Mikawa, Japan
Ooi Futo		6810		-1986	Reclamation	Container terminal	Port	Tokyo, Japan
No. 13		3010		-1986	Reclamation	Container terminal	Port	Tokyo, Japan
Kansai International Airport	16,5 - 19	5110	5	1985-1994	Seawall + soft soil	Airport	Air	Japan
Shibushi-wan oil storage	6 - 12	1960	0.5	1985-1993	Reclamation	National oil storage basin	EG	Namimi, Japan
Tamashima E-II		1960		1986-	Reclamation	Foreign trade wharf	Port	Mizushima, Japan

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Port Island		340		1987-	Reclamation	Foreign trade wharf	Port	Hachinohe, Japan
Marinas	10 - 15	370	0.1	Study (1988)	Flaoting & Pile & Soft Ground	Complex city	CC	Tokyo Bay, Japan
Marination	100	70650		Study (1988)	Reclamation by drainage	R&D of Marine Resources	M&E	Pacific Ocean
Kinjo-Tuto		1910		-1989	Reclamation	Foreign trade wharf	Port	Nagoya, Japan
Futsu		1680		-1989	Reclamation	Industrial zone	II	Kisalazu, Japan
Kunnui Fishing Port	6	14	0.274	1988-?	Seawalls + fill	Fishing Port	Port	Japan
Kawasaki Manmade Island	28	30	5	1989-1996	Seawalls + fill	Ventilation tower for Trans-Tokyo Bay Highway	T&A	Japan
Kisarazu Manmade Island	25	75	4.5	1989-1996	Caissons + steel sheet pile cells	Ventilation tower + Tunnel launching base for Trans-Tokyo Bay Highway	T&A	Japan
Minami 5 ku		1110		1989-		LNG terminal and Industrial waste disposal site	WI	Nagoya, Japan
Blue Bay Plan			Tokyo Bay	Study (1989)	Reclamation	Recreation	Rec	Japan
Dyna City	15 - 30	1		Study (1989)	Caisson foundation	Sewerage Disposal & Energy Plant	II	Tokyo Osaka, Japan
Marilin		200	1 - 2	Study (1989)	Reclamation	Recreation & Sport	Rec	Japan
Waste Disposal on the Sea	10 - 15	1.1		Study (1989)	Soft Ground	Waste Disposal	WH	Tokyo Bay, Japan
Marine Polis	10 - 15	4		Study (1989)	Caisson foundation	Infrastructure & Complex City	CC	Tokyo Bay, Japan
The Road on Sagami Bay	30 - 40	7500	1- 3	Study (1989)	Reclamation & Pile & Floating	Infrastructure	T&A	Sagami, Japan
ARC Airport	20	910		Study (1989)	Reclamation & Caisson Foundation	Offshore Airport	Air	Yokohama, Japan
Airo Polis 2001		1100	10	Study (1989)	Reclamation	Complex city	CC	Japan
Pacific Airport 21	100	572	9	Study (1989)	Reclamation & Floating	Airport	Air	Hura, Japan
Minami-Honmoku		2170		1990-	Reclamation	Container terminal & Complex terminal	Port	Yokohama, Japan
X-SEED 4000		5000 - 7000	coastal zone	Study (1990)	Reclamation Super High-Building 4000 m	Complex city	CC	Japan
Deep Ocean Frontier 21	3000 - 5000		Deep Ocean Depth	Study (1990)	Soft Ground	Deep Sea Probing	M&E	Japan
Never Never Land	15 - 30	2.4		Study (1990)	Soft Ground	Leisure	Rec	Japan

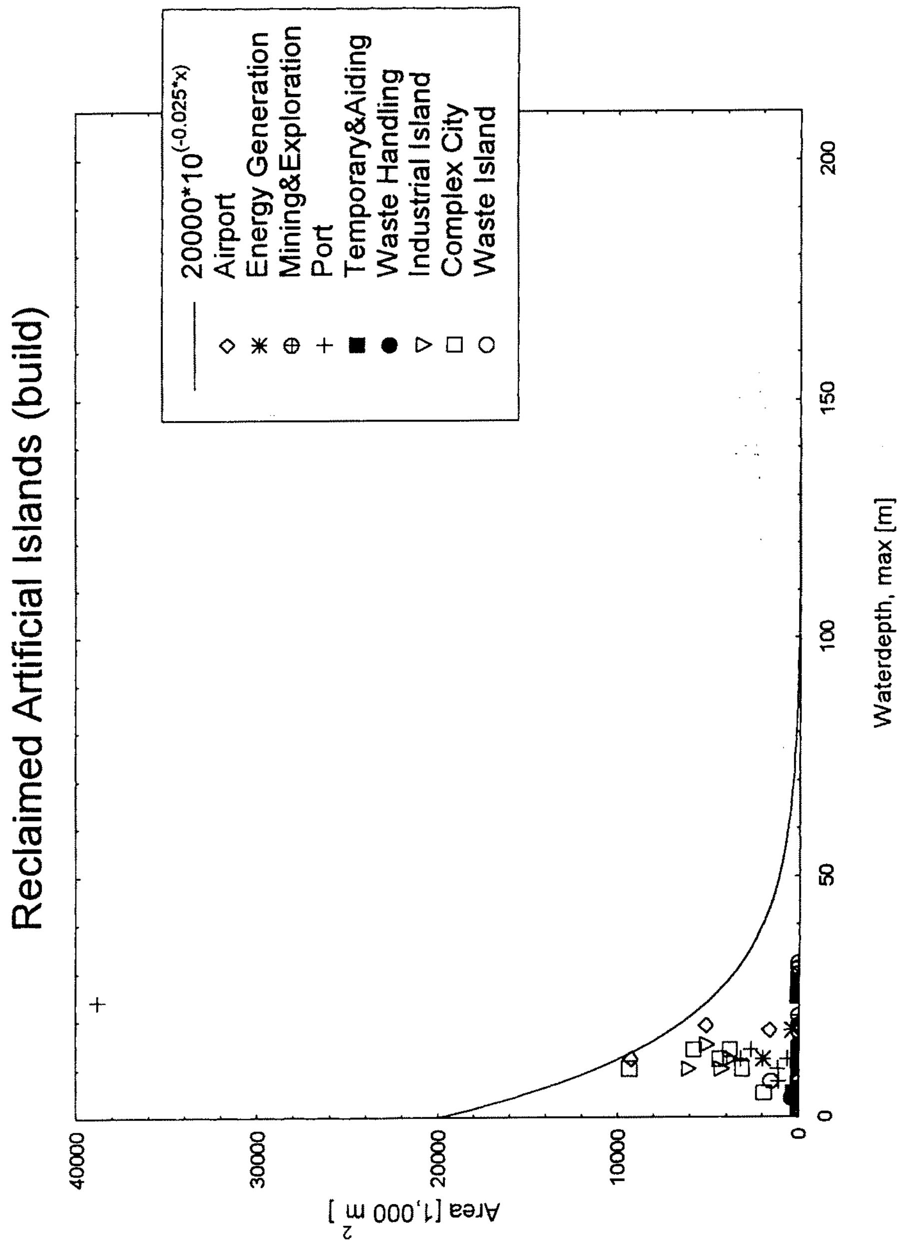
Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Soft Landing Island	10 - 15	600	1 - 2	Study (1990)	Floating & Soft Ground	Leisure & City	CC	Tokyo Bay, Japan
Marine Uranus		45.1	10	Study (1990)	Floating & Soft Ground	Complex city	CC	Japan
Honeycomb Island	10 - 15	94.7	1	Study (1990)	Reclamation & Floating & Caisson foundation	Complex city	CC	Tokyo Bay, Japan
Romantic City 2020		200	15	Study (1990)	Reclamation	Offshore Airport & Leisure	Air	Japan
Super Roof		5026.5		Study (1990)	Reclamation	Marine Ranching & Leisure	Mar	Japan
Osaka-Bay Green Belt		1700	Osaka Bay	Study (1990)	Reclamation	Infrastructure	T&A	Japan
Tokio 21 Project Bass	20 - 25	2400	10	Study (1990)	Reclamation	Offshore Airport	Air	Japan
Bay Area Canal		5000	Waterfront	Study (1990)	Reclamation	Infrastructure	T&A	Japan
Penta H-SST			Offshore	Study (1990)	Soft Ground	Recreation	Rec	Japan
Marine Community 21		50	5	Study (1990)	Reclamation	Infrastructure	T&A	Osaka bay, Japan
DIB-200		150	Waterfront	Study (1990)		Business & Residential	CC	Japan
Kuzyukuri 7-Island	20 - 25	1000	10	Study (1990)	Reclamation	Complex city	CC	Kuzyukuri, Japan
Marine colosseum	100 - 200	4	coastal zone	Study (1990)	Reclamation & Soft ground & Caisson Foundation	Recreation	Rec	Japan
Hakusukinoe	14	2640		1990-	Reclamation	Container terminal	Port	Kitakyushu, Japan
Millennium Tower		20	coastal zone	Study (1991)	Reclamation	Super high-rise Building	CC	Japan
Genesis teleport		3.1	Continental Shelf	Study (1991)	Reclamation & Floating	International Information City	CC	Japan
Flying Triton 21			5	Study (1991)	Soft Ground	Offshore Airport	Air	Japan
TRY 2004		8800	Waterfront	Study (1991)	Reclamation Super High-Rise Building 2004 m	Complex city	CC	Japan
Harmony-21		100	0.5	Study (1991)	Soft Ground	Complex city	CC	Japan
Top of the Japan Sea Rim		314	Japan Sea	Study (1991)	Floating & Soft Ground	International Ocean City	CC	Japan
Tokyo Neo Atlantis		314		Study (1992)	Reclamation	Office & City	CC	Tokyo Bay, Japan
Recycle Long Island		240	coastal zone	Study (1992)	Reclamation	Waste Disposal	WH	Japan
Hakkeijima Sea Paradise		240		-1993	Reclamation	Facilities for recreation	Rec	Yokohama, Japan
Tokyo International Airport		12650		-1993	Reclamation	Airport	Air	Tokyo, Japan
Ariake No. 10, 11		3390		-1993	Reclamation	Foreign trade wharf	Port	Tokyo, Japan
Green Island		80		-1994	Reclamation	Green zone	Rec	Hakodate, Japan
Maiskima		2250		-1994	Reclamation	Container terminal	Port	Osaka, Japan
Okinawa	10	1160		-1994	Reclamation	Container terminal	Port	Kamatsujima, Japan
No. 3, 4		6950		-1995	Reclamation	Industrial zone	II	Oita, Japan

Name	Waterdepth [m]	Area [1,000 m²]	Distance off Shore [km]	Construction Period	Construction method	Purpose	Code	Location
Wakayama Marina City		490		-1995	Reclamation	Facilities for recreation	Rec	Wakayama-shimotsu, Japan
Hakata Island City				1994-	Reclamation	Container terminal, urban facilities as houses and research centre	CC	Hakata, Japan
Hibikku Land	12	630		1995-	Reclamation	Container terminal, and Ferry terminal	Port	Shimonoseki, Japan
Maishima (North port South)	14	3810		1997-	Reclamation	Container terminal, urban facilities and houses	CC	Osaka, Japan
No Name		3740		1997-	Reclamation	New Kitakyushu Airport, dredged material disposal site	Air	Kitakyushu, Japan
Genesis Project		15		Study	Floating & Soft Ground	Recreation	Rec	Tranquil Sea
Tokyo Bay City Island	10	23000	Tokyo (River)	Study	Reclamation	Complex city	CC	Japan
Marine Plantation Island			coastal zone	Study	Reclamation	Leisure	Rec	Japan
Eco-Land		1000		Study	Reclamation	Complex city	CC	Osaka, Japan
Higashimiya Artificial Island		480		Study (1997)	Reclamation	Foreign trade wharf, and Facilities for recreation	Port	Shiogama, Japan
Higashiko Port Island		2050		Study (1997)	Reclamation	Foreign trade wharf	Port	Onahama, Japan
Kashima offshore Artificial island		1560		Study (1997)		Waste disposal site	WH	Kashima, Japan
Fukiinoura		900		Study (1997)		Foreign trade wharf, and Industrial zone	II	Tsumatsuzaka, Japan
Shinokina Yama		2140		Study (1997)		Dredged material disposal site	WH	Ube, Japan
Yatsushire Artificial Island		2040		Study (1997)		Foreign trade wharf	Port	Yatsushiro, Japan
Kagashima Artificial Island		670		Study (1997)		Terminal for sight seeing boat, Convention center	T&A	Kagoshima, Japan
Nakagusukuwan Artificial Island		1780		Study (1997)		Industrial zone, and Urban facilities as houses	CC	Nakagusukuwan, Japan

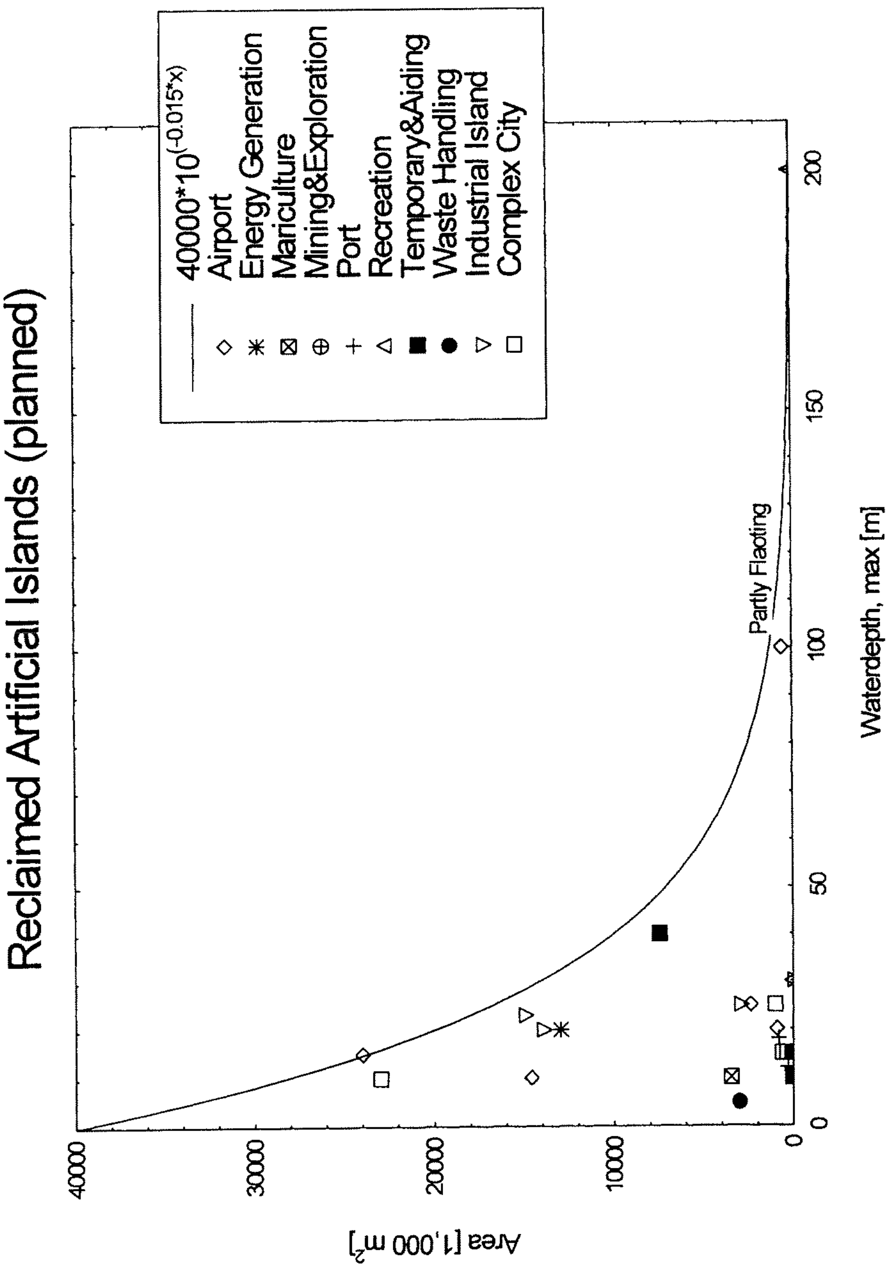
Appendix B

Structural relations for artificial islands

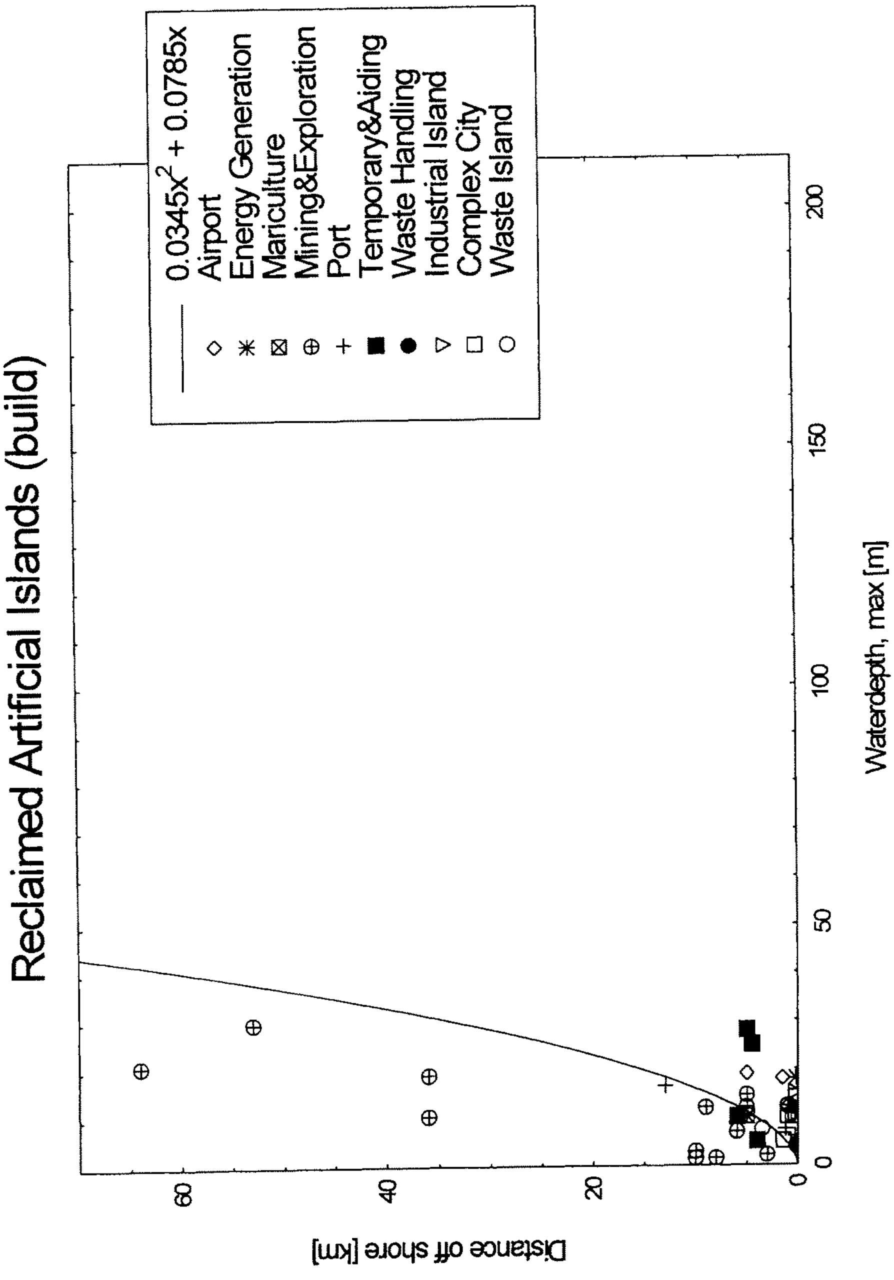
B.1 Relation waterdepth-area for artificial islands (build)



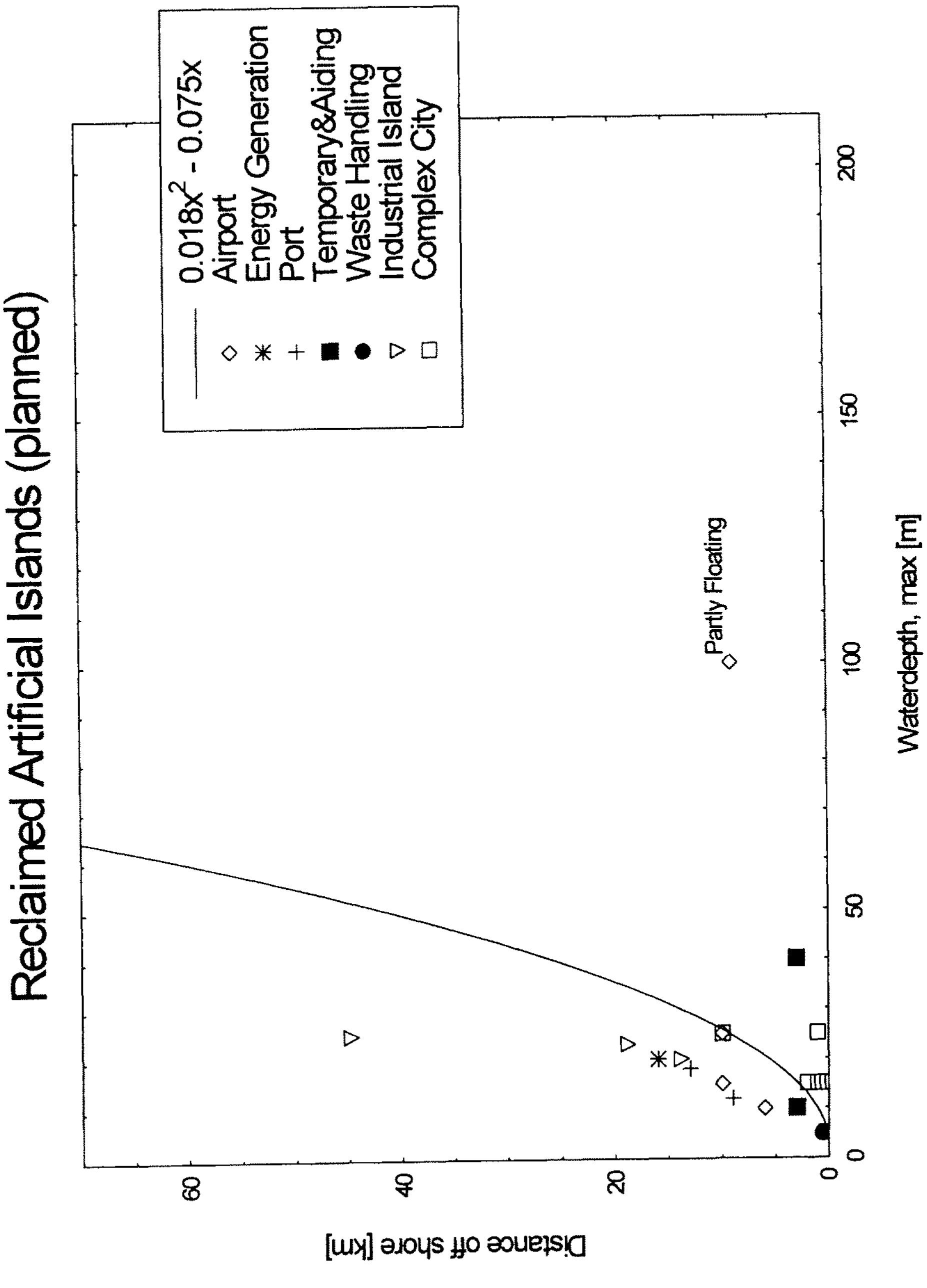
B.2 Relation waterdepth-area for artificial islands (planned)



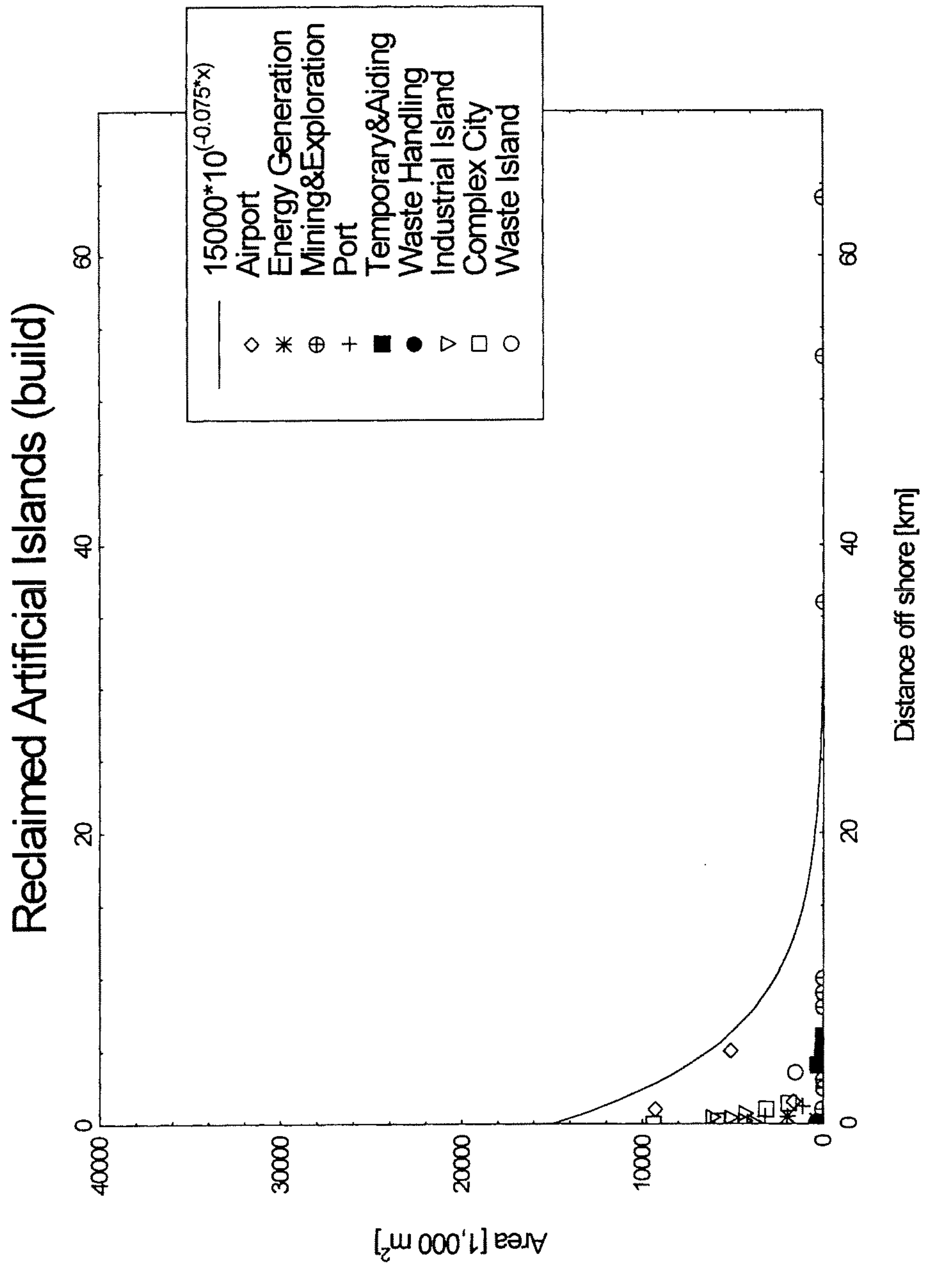
B.3 Relation waterdepth-distance offshore for artificial islands (build)



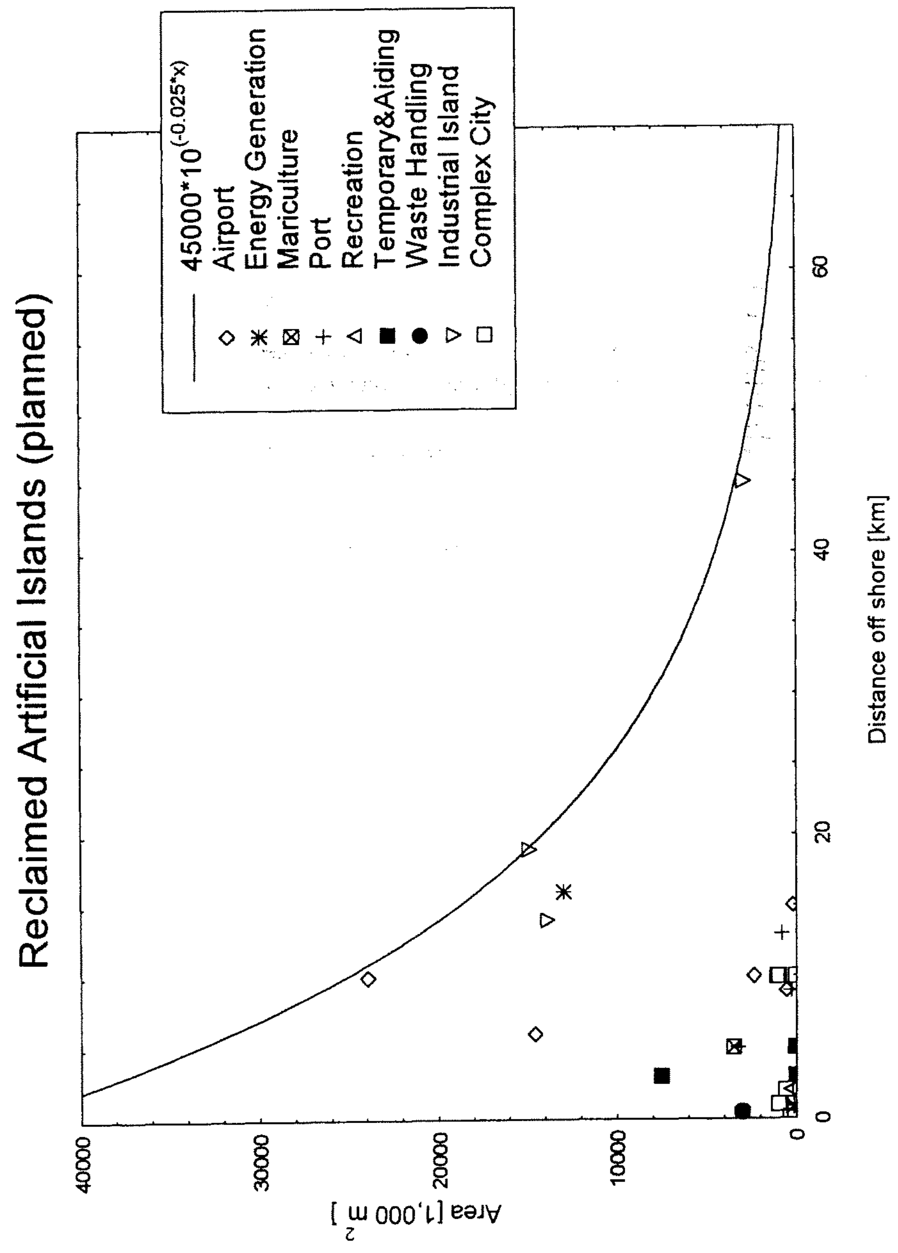
B.4 Relation waterdepth-distance offshore for artificial islands (planned)



B.5 Relation distance offshore-area for artificial islands (build)



B.6 Relation distance offshore-area for artificial islands (planned)



Appendix C

Design of rubble mound sea defences

One of the possibilities to protect an artificial island against storm conditions is to construct a sea defence totally around the island. Another possibility is to construct a sea defence only at the wave exposed side of the island and construct the lee-side as an artificial beach. The main function of the sea defence will be to prevent the wave actions from eroding the island. A typical cross section of a rubble mound sea defence, as protection for a man-made island, is shown in figure C.1 . The accentuated triangular shaped parts in the centre of the sea defence are small quarry run berms used in the filling-process of the island.

Parameter	Description
L_f	Length of filter, relative to toe
R_c	Crest freeboard, relative to sea level
h	Waterdepth
h_t	Depth of the toe below sea level
α	Angle of structure slope
t_a	Thickness of armourlayer
t_s	Thickness of secondary armourlayer
B	Crest width
h_c	Hight crest relative to seabed

Two design methods are given in this appendix, one for rubble mound sea defences armoured with quarry stones, and one for rubble mound sea defences armoured with concrete units. The two methods show a lot of resemblance, but are treated separately. At first some general equations are mentioned, followed by the calculation of a quarry stone armoured sea defence and a concrete units armoured sea defence. These design-parts, which are equal for both sea defences are mentioned subsequently.
At the end of this paragraph two examples of calculations are given, viz. a rubble mound sea defence armoured with dolosse at a depth of 26 m, and a rubble mound sea defence armoured with quarry stones at a depth of 8 m.

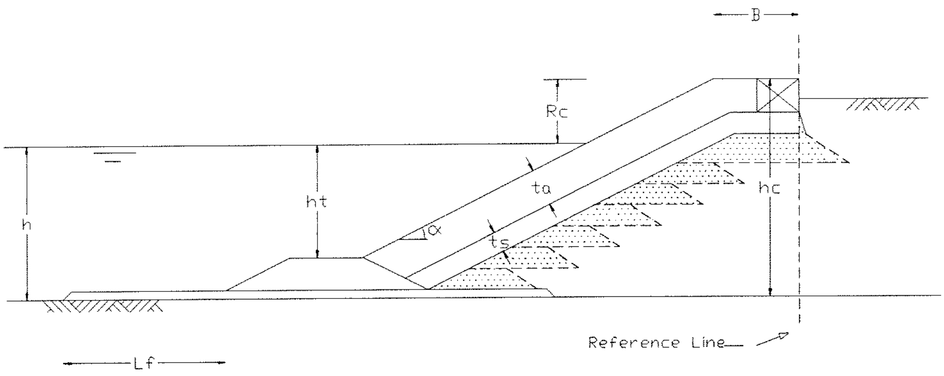


Figure C.1: Parameters related to cross-section of rubble mound sea defence

C.1 Methods and formulae used in calculations

C.1.1 General equations

Some equations, which are used in numerous relations are summed up below, without further explanation.

Deepwater wave length:

$$L_0 = gT^2/2\pi \tag{C.1}$$

Peak wave period:

$$T_p = 1.2 * T \tag{C.2}$$

Wave steepness:

$$s = H/L_0 = 2\pi H/gT^2 \tag{C.3}$$

Surf similarity parameter:

$$\xi = \tan\alpha/\sqrt{s} \tag{C.4}$$

- ξ_m is based on the normal wave period
- ξ_p is based on the peak wave period

Relative density:

$$\Delta = (\rho_{armour} - \rho_{water})/\rho_{water} \tag{C.5}$$

The most important dimensions of a rubble mound sea defence are shown in figure C.2. These dimensions are also mentioned in the text, but are visualised for a better understanding.

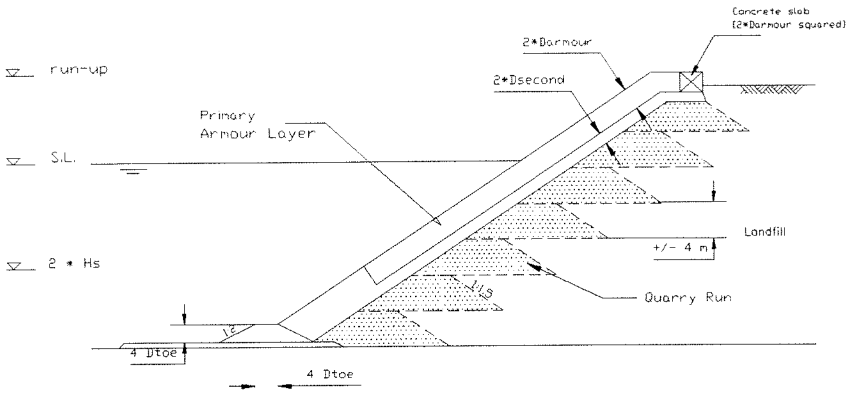


Figure C.2: Dimenions used to construct rubble mound sea defences

C.1.2 Rubble mound sea defences armoured with quarry stones

In this case the entire cross section of the defence structure is build up out of quarry stone, varying in weight from several tonnes to only a few kilograms. These weights can be divided into gradations, depending on the quarry used. For this thesis the gradations and costs as stated in table C.1 are assumed. In this paragraph, the separate steps in the design are detailed. The parameter a indicates the additional costs for transport-distances, etc.

Run-up calculations

When the crest freeboard is as high as or higher than the maximum run-up, no overtopping will occur. This condition is interpreted as 2% run-up allowed. In other words this means, that only 2% of the incoming waves may cause overtopping. Consequently, when the maximum run-up is known the height of the crest above S.L. can be determined.

In case of armoured rubble slopes the 2% run-up level is given by van der Meer (1988a) [18].

$$R_{u2\%}/H_s = 0.96\xi_m \qquad \text{for} \xi_m < 1.5 \tag{C.6}$$

Gradation	D_{n50}[m]	W_{n50}[kg]	[US$/m^3]
0.0 - 500 kg	0.32	83	(a + 36)
250 - 500 kg	0.51	361	(a + 40)
500 - 1000 kg	0.65	721	(a + 42)
1000 - 2000 kg	0.82	1443	(a + 45)
1000 - 3000 kg	0.88	1821	(a + 50)
2000 - 4000 kg	1.03	2895	(a + 55)
3000 - 6000 kg	1.18	4328	(a + 63)
6000 - 9000 kg	1.41	7399	(a + 76)
9000 - 12000 kg	1.58	10428	(a + 86)
12000 - 16000 kg	1.74	13904	(a + 100)

Table C.1: Gradations and prices for quarry stones

$$R_{u2\%}/H_s = 1.17\xi_m^{0.46} \qquad \text{for}\xi_m > 1.5 \tag{C.7}$$

The run-up for permeable structures (P > 0.4) is limited to a maximum:

$$R_{u2\%}/H_s = 1.97 \tag{C.8}$$

Armourstones calculations
The ratio H_s/Δd indicates the stability of the armour stones under wave attack. Van der Meer contrived some equations to determine this ratio. Van der Meer draws a distinction between the required weight of the stones when plunging breakers occur and when surging breakers occur. The diameter of the stones can be calculated, using the following two equations:

$$\frac{H_s}{\Delta d} = 6.2P^{0.18}(\frac{S_d}{\sqrt{N}})^{0.2}\xi_m^{-0.5} \qquad \text{for}\xi < \xi_t \qquad \text{(plunging breakers)} \tag{C.9}$$

$$\frac{H_s}{\Delta d} = 1.0P^{-0.13}(\frac{S_d}{\sqrt{N}})^{0.2}\xi_m^{P}\sqrt{\cot\alpha} \qquad \text{for}\xi > \xi_t \qquad \text{(surging breakers)} \tag{C.10}$$

The transition between the two types of breakers is given by equation (C.11). In practise, for cot ≤ 4 surging waves do not exist and only the expression for plunging waves is recommended for use.

$$\xi_{transition} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{(\frac{1}{P+0.5})} \tag{C.11}$$

The symbol S_d stands for the amount of damage caused by a certain wave height during a certain time. S_d depends on the slope and the allowable amount of damage which the breakwater is designed for, as can be seen in table C.2. Of course, the criterion of initial damage is taken in the design calculations.

Thickness primary armour layer
The thickness of the primary armour layer is determined as two times the nominal diameter of the primary armour stones, written as an equation:

$$t_{armour} = 2 * D_{n50Armour} \tag{C.12}$$

Slope	Initial damage	Intermediate damage	Failure
1 : 1.5	2	3 - 5	8
1 : 2	2	4 - 6	8
1 : 3	2	6 - 9	12
1 : 4	3	8 - 12	17
1 : 6	3	8 - 12	17

Table C.2: Design values for S_d for a two diameter thick armourlayer [18]

C.1.3 Rubble mound sea defences armoured with concrete elements

The same series of calculations as was used for the quarry stones, is used for the concrete elements. Only different equations are used. Some features of the concrete armour elements, which will be used further on, are mentioned in table C.3. The costs for modified cubes have been set to 70 US \$ / ton, and the costs for dolosse and tertapods are both US \$ 80 / ton. These prices include the total costs for fabrication and placing in position. Because the latter two elements are more complicated to fabricate, they are more expensive.

Run-up calculations

When the crest freeboard is as high as or higher than the maximum run-up, no overtopping will occur. This condition is interpreted as 2% run-up allowed. In words this means, that only 2% of the incoming waves may cause overtopping. Consequently, when the maximum run-up is known the height of the crest above S.L. can be determined.

Ahrens (1981) produced several somewhat conservative equations to calculate the run-up on smooth slopes, using the surf similarity parameter based on the peak wave period. For non-smooth slopes the reduction factor r has to be applied [18].

$$R_{u2\%}/H_s = 1.6\xi_p \qquad \text{for} 0 < \xi_p < 2.5 \tag{C.13}$$

$$R_{u2\%}/H_s = 4.5 - 0.20\xi_p \qquad \text{for} \xi_p > 2.5 \tag{C.14}$$

Armour stones calculations

To calculate the weight of the concrete armour elements, the more simple formula of Hudson is used. No factor like S_d is included to indicate the damage percentage. Generally, it is assumed, that the weight of the armour stones are calculated for a 5% damage level, although the correctness of this number is not clear.

$$W = \frac{\rho_a H^3}{K_D \Delta^3 \cot\alpha} \tag{C.15}$$

The K_D - values have already been mentioned in table C.3. Compare the large K_D - value of 15.8 for dolosse to the values of the other elements!

Van der Meer also derived relations for two concrete armour units, viz. cubes and tetrapods. Because no Van der Meer relation exists for the dimensioning of dolosse, only the Hudson-formulae are used to determine the weight of concrete armour units. However, the two Van der Meer-relations are mentioned below. N_{od} indicates the relative damage level, N the number of waves and s_m the wave-steepness. For the no-damage criterion N_{0d} becomes zero.

$$\text{Cubes:} \qquad H_s/\Delta D_n = (6.7 N_{od}^{0.4}/N^{0.3} + 1.0) s_m^{-0.1} \tag{C.16}$$

Armour-elements	run-up reduction factor r	K_D	n_v	K_Δ
Dolosse	0.45	15.8	0.56	0.94
Modified cubes	0.55	6.5	0.47	1.10
Tetrapods	0.50	7.0	0.50	1.04

Table C.3: features of concrete armour elements

$$\text{Tetrapods:} \quad H_s/\Delta D_n = (3.75 N_{od}^{0.5}/N^{0.25} + 0.85) s_m^{-0.2} \tag{C.17}$$

The weight of the concrete elements depends not only on the size, but also on the density of the concrete used. The standard weights of concrete elements are summarised in table C.4. In the continuation of this appendix the density of the armour stones is assumed to be 2621 kg/m^3. A density close to the density of quarry stones. This density can be obtained by mixing the concrete with heavier aggregates, like iron ore.

Thickness primary armour and number of units

To ensure the workability of the concrete elements they have to be placed on the secondary armour layer in a double layer. The thickness of the armour layer can be calculated using equation (C.18).

$$t_{armour} = nK_{\Delta}(W/\rho)^{1/3} \tag{C.18}$$

And the number of units per m^2 is given by:

$$N_a = nK_{\Delta}(1 - n_v)(\rho/W)^{2/3} \tag{C.19}$$

Where:
- t_{armour} = thickness of armour layer
- n = number of layers
- K_{Δ} = layer thickness coefficient
- n_v = volumetric porosity
- ρ = density of armour material

C.1.4 Equal design parts for rubble mound sea defences

Below those parts of the cross section design, which are similar for both the quarry stone sea defence and the concrete elements sea defence are summed up.

End of armourlayer

Because the costs of the armourlayer contributes significantly to the total costs of the sea defence, any decrease in the amount of armour used is welcome. According to rules of thumb the armourlayer needs to be extended till the toe in shallow water. However, if the distance between the crest of the toe and the mean sea level exceeds 1.5 times H_s the armourlayer can be replaced by the secondary armour layer. Throughout this report a water level (S.L.) is assumed, taking into account set-up by tidal-, wind- and storm-influences. Consequently,

Volume of individual armour elements [m^3]

Density kg/m^3	0.20	0.40	0.81	2.02	4.050	4.550	6.070	7.430	8.090	10.11	12.14	14.16	16.18
	Weight of individual armour elements[ton]												
2473	0.50	1.00	2.00	5.00	10.00	11.25	15.00	18.35	20.00	25.00	30.00	35.00	40.00
2621	0.53	1.07	2.14	5.34	10.68	12.00	16.02	19.60	21.36	26.70	32.04	37.38	42.71
2743	0.56	1.11	2.23	5.57	11.14	12.48	16.71	20.38	22.29	27.86	33.43	39.00	44.57
2867	0.58	1.16	2.31	5.79	11.57	13.04	17.36	21.30	23.14	28.93	34.71	40.50	46.29

Table C.4: Weight of armour elements [30]

Mean sea level will be somewhat lower, hence a distance of *2 times* H_s is taken (also including safety), instead of 1.5 times H_s. See figure C.3.

Secondary armour calculations

Underneath the primary armourlayer a layer, or sometimes several layers, of smaller stones is/are placed to support the armour layer and to prevent the core from washing out. A relationship between the weight of the armour elements and the secondary armour elements is given in the Shore Protection Manual [30]. Secondary armour elements are always rubble stones. Weight and dimensions as stated in table C.1.

$$W_{n50Secondary} = \frac{W_{n50Armour}}{10} \tag{C.20}$$

Thickness of secondary armourlayer

The thickness of the secondary armour layer is determined as two times the nominal diameter of the secondary armour stones, written as an equation:

$$t_{second} = 2 * D_{n50Second} \tag{C.21}$$

In case the size of the secondary armour layer is not able to prevent the core from being washed out, a third layer has to be applied.

Core calculations

In most cases the quarry run (stones having an average weight of less than 500 kg) can be used as core material. However, this has to be checked using filter formulae, the so called filter rules for geometrically closed filters are used. Two demands have to be fulfilled, namely stability and permeability between the filter layer and the base layer. If these two conditions are satisfied then the grains can not move out of the base layer.

$$\text{Stability:} \quad \frac{d_{15F}}{d_{85B}} < 5 \quad \text{(provided internally stable: } \frac{d_{60}}{d_{10}} < 10) \tag{C.22}$$

$$\text{Permeability:} \quad \frac{d_{15F}}{d_{15B}} > 5 \tag{C.23}$$

The subscripts F and B stand for filter and base respectively. The filter layer consists of the secondary or third armour and the core acts as the base layer. A more wide grading can be assumed for the core material. According to Van der Meer a sea defence having a

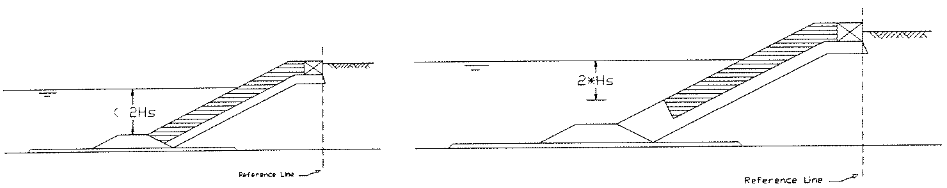


Figure C.3: Area of armourlayer in cross-section according to waterdepth permeability P of 0.4 has to possess the following relations: $D_{n50Armour}/D_{n50Secondary} = 2$ and $D_{n50Secondary}/D_{n50Core} = 4$.

Toe calculations

Van der Meer formulated an equation to calculate the Dn50 of the toe [18]. The following equation is based on the 90% confidence level, h_t indicates the distance between the top of the toe and the water level. The formula is safe for h_t / h > 0.5, for lower values the normal armour calculations should be used.

$$h_t/h = 0.253(H_s/\Delta D_{n50})^{0.7} \tag{C.24}$$

Referring to general rules of thumb, the toe will be designed having a height of about 4*D_{n50} and a crest-width of about 4*D_{n50}. The slopes have been set on 1 : 2. In deepwater this will be no problem, however in shallow water the thickness of the toe may become too large relative to the waterdepth. Consequently, a thickness of 3 or 2 times D_{n50} is required. Secondly, the toe's function is to support the armour layer. If a slope 1 : 2 is applied, using a small D_{n50}, it is possible that the toe does not completely support the layer of armour. In such cases a flatter slope should be chosen, e.g. 1 : 3. An interpretation of the designer is required, but above mentioned remarks can be useful for an initial design.

Additional remarks

The above mentioned design rules and equations are very general ones. For every location the circumstances are different and therefore different designs will result. Every construction and design should as least be preceded by a thorough site and soil mechanical investigations as well as some computer or scale models of the sea defences. The above mentioned equations can serve to give an indication of the relative final costs for each type of armour used in the sea defence. Resulting in a preference for a type of armour for a certain waterdepth, based on the cost criterion.

C.2 Two examples of rubble mound sea defence design

Using the design rules, stated in appendix C a technically functional sea defence can be constructed. However, the art is to design a technically functional and an economical sea defence. One can imagine that steeper slopes require less material in a cross section and thus will be less costly. Steeper slopes increase weight of the armour and the amount of wave run-up, which raises the crest height and thus the total volume in a cross section. However, the cheapest total cross section should be chosen. Secondly, the usable types of armour, are limited by their weight. The maximum weight of the quarry stones is limited by the quarry output. And the maximum weight of the concrete elements depends on the type of concrete used, the shuttering and the risk of fractures in the concrete. The maximum weight for dolosse and tetrapods is 20 tons in this report. Modified cubes can be applied till their maximum available weight. Things will become more clear in an example. Two examples are listed below, one for deepwater and one for shallow water.

C.2.1 Deepwater sea defence [h=26m]

It is assumed that the depth at the toe of the sea defence amounts to 26 metres. Furthermore, the significant wave height H_s is 8.0 m. Locally generated waves exist and the (local) wave steepness has a value of 0.05 (seas).

Design description
At a depth of 26 metres a sea defence is to be designed to protect a man-made island, using an armour of dolosse. No overtopping may occur and the cheapest possible cross-section will be chosen.

Additional information: perpendicular approaching waves ($\beta = 0$).
Both a and b are zero.

General calculations
Because the significant wave height and the wave period are related to each other by means of the wave steepness, the mean wave period T_m can be derived. According to $s_m = \frac{H_s}{L_0} = \frac{H_s * 2 * \pi}{g * T_m^2}$, follows $T_m = 10.12$ s.
Having calculated the mean wave period, the peak wave period and peak wave steepness can easily be computed.

$$T_p = 1.2 * 10.12 = 12.14 \text{ s.}$$
$$s_p = 2\pi H / g T_p^2 = 0.035$$

Relative density Δ for sea defence elements:
Dolosse : (2621-1030)/(1030) = 1.54
Quarry stones: (2650-1030)/(1030) = 1.57

Armour calculations
The maximum slope which can be applied depends on the maximum available armour stones, see table C.4. If one assumes a density of 2621 kg/m3 then the maximum safe tonnage amounts to 19.60 ton. If the calculated weight of the dolos would exceed this tonnage, a less steep slope is necessary. Using Hudson's formula,equation C.15, with a slope 1 : 1.5 the following weight results.

$$W = \frac{2621 * (8.0)^3}{15.8 * (1.54)^3 * 1.5} = 15.50 \quad \text{ton}$$

This result in a standard dolos of 16.02 ton. This weight is less than 20 ton, and consequently the slope of 1 : 1.5 can be maintained. Knowing the weight of the dolosse the thickness and number of units of the double layer of dolosse can easily be calculated.

$$t_{armour} = 2 * 0.94 * (16{,}020/2621)^{1/3} = 3.44 \quad \text{m}$$

$$N_a = 2 * 0.94 * (1 - 0.56) * (2621/16{,}020)^{2/3} = 0.25 \quad 1/m^2$$

The armour layer ends at a distance 2 * 8.0 = 16.0 below the sea level and the secondary armour layer continuous at that location, providing the toe allows this. But generally the height of the toe will not be higher than 26.0 - 16.0 = 10.0 m, above the sea bottom!

Run-up

Now that the slope is know the run-up can be calculated using equations (C.4) and (C.14) and, as a consequence the crest height as well.

$$\xi_p = \tan\alpha/\sqrt{s_p} = 3.56$$

$$R_{u2\%}/8.0 = 4.5 - 0.20 * 3.56 \qquad \text{for } \xi_p > 2.5$$

The total run-up, regarding the reduction amounts to $R_{u2\%}$ = 30.30 * 0.45 = 13.64 m. This same number accounts for the crest height above sea level

Secondary armour

The weight of the secondary armour stones amounts to one tenth of 16 .02 ton, thus W_{second} = 1.602 ton. Resulting in a D50 of 0.88 m and a layer thickness of 1.76 m, see table C.1.

Core calculation

Normally the gradation, which contains the stones having the least weight, is used for the core. This gradation called quarry run results in stones varying from 0.0 to 500 kg, with a nominal diameter of 0.32 m. To check if this is correct, the filter rules (equations (C.23) and (C.23)) are applied, using a wide grading for the core and a grading of 2 for the secondary armour layer. So the D_{15F} = 0.88 / $\sqrt{2}$ = 0.62 m.

Following these rules, the criterion for permeability is not fulfilled, and consequently a third layer has to be applied. According to the *Shore Protection Manual*, however, the core can consist of material with a weight of 1/200 to 1/6000 of that of the armour layer. Resulting from this rule the lowest gradation can be used for the core. Throughout this report this Shore Protection Manual-relation is maintained, on the one hand for reasons of simplicity and on the other hand to reduce calculation-time.

Toe calculation
By trial and error the diameter of the toe stones is chosen. Trying a diameter of 0.88 m results in a $h_t = 22.49$ m. When a toe thickness of 4 * 0.88 = 3.52 m is chosen, the toe will be exactly at the calculated depth and consequently stable.

$$\frac{h_t}{26} = 0.253(\frac{8.0}{1.57 * 0.88})^{0.7}$$

Concrete top slab
To give support to the layer of dolosse, a concrete slab is placed. The slab's dimensions are related to the thickness of the armour-layer. Its height and width are equal to the thickness of the armour layer. Thus the width of the crest results:

$$\text{crestwidth} = t_{armour} + (t_{armour} + t_{second}) * \sin\alpha = 6.32 \text{ m}$$

Cost calculation
To make sure the sea defence at a depth of 26 m is the most economical one, using the previously designed dimensions, an indication of the total costs for each meter of sea defence needs to be made. It is mentioned, that some small simplifications in the calculation of the layer volumes have been made. Because the island is filled using small berms(see figure C.2), the core consists of quarry run as well as sand. It is presupposed that 50% of the core volume contains sand and the remaining part of the volume consists of quarry run. The quarry run used as core, functions as berms during the filling process of the island. The costs of sand are set to US \$ 3 / m^3.

Price armour layer:	{(run-up + 2 * H_s) / sin α + (t_{armour} + t_{second}) * sin α }* N_a * W_{n50} / 1000 * Unitprice = {(13.64 + 2*8.0) / sin α + (3.44+1.76) * sin α } * 0.25 * 16,02 * 80 =	*US \$ 20,124*
Concrete slab:	$(t_{armour})^2$ * ρ_c / 1000 * Unitprice = $(3.44)^2$ * 2400/1000 *70 =	*US \$ 1,988*
Price sec. layer:	{(depth + run-up - Dtoe - t_{armour}) / sin α + (crestwidth - 0.5 * (2*D_{second}) * tan α)} *(2*D_{second}) * Unitprice + (depth - 2 * H_s - 2 * Dtoe) * t_{armour} * Unitprice = {(26 + 13.64 - 0.88 - 3.44) / sin α + (6.32 - 0.5*(2*0.88)* tan α)} * (2*0.88) * 50 + (26 - 2*8.0 - 2*0.88) * 3.44 * 50	*US \$ 1,985*
Price core:	(depth + run-up - tarmour - 2 * D_{second}) * { (crestwidth - tan α * (t_{armour} +2*D_{second})) + 0.5 * (depth + run-up - tarmour - 2 * D_{second}) / tan α}* Unitprice = (26 + 13.64 - 3.44 - 2*0.88) * { (6.32 - (t_{armour} + 2*D_{second})*tan α) + 0.5 * (26 + 13.64 - 3.44 - 2*0.88) / tan α} * (0.5*36 + 0.5*3)	*US \$ 19,261*
Price toe:	{4*D_{toe} * (4*$D + toe$ + 2*4*D_{toe}} * Unitprice = 48 * 0.88 * 0.88 * 50	+ *US \$ 2,112*
TOTAL PRICE:		*US \$ 45,470*

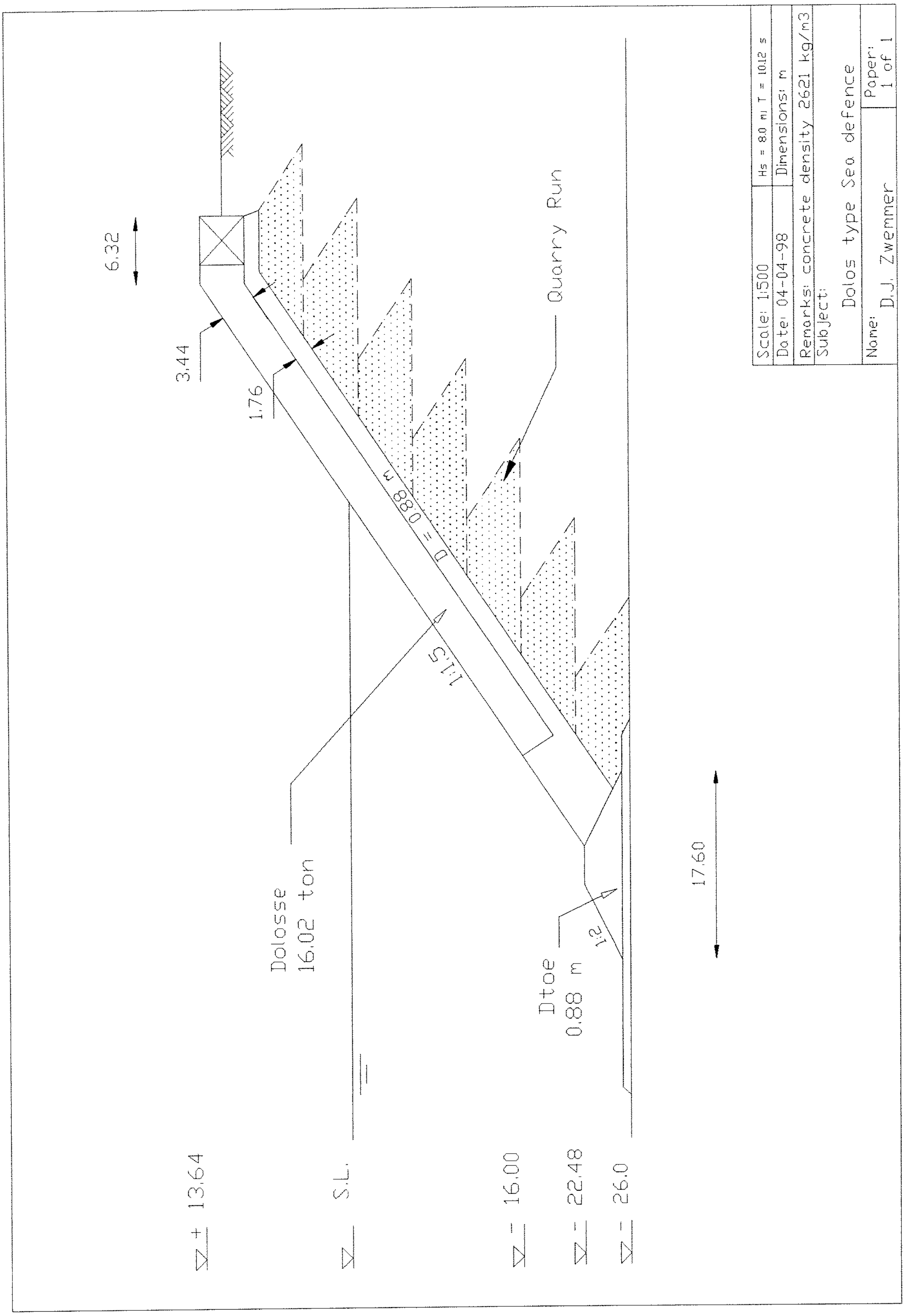
The question now is, whether or not this cross section is more economical than a cross section having a flatter slope? Without showing the calculations of all the dimensions again the result for slopes 1 : 2 through 1 : 6 under the same conditions are summed up below.

Concluding from table C.5 a slope 1 : 1.5 results in the cheapest cross section under these conditions.

Slope	Depth [m]	H_s[m]	Run-up [m]	W_{armour}[ton]	D_{second}[m]	D_{toe}[m]	P_{total}[1/m]
1 : 1.5	26	8.0	13.64	16.02	0.88	0.88	45,470
1 : 2	26	8.0	14.27	12.00	0.82	0.88	57,673
1 : 3	26	8.0	10.30	10.68	0.82	0.88	66,939
1 : 4	26	8.0	7.73	10.68	0.82	0.88	75,948
1 : 5	26	8.0	6.18	5.34	0.65	0.88	77,791
1 : 6	26	8.0	5.15	5.34	0.65	0.88	86,893

Table C.5: Cost comparison for different slopes (h=26 m, dolosse-type sea defence)

The sea defence as calculated above with a slope of 1 : 1.5 is drawn on the next page. It is mentioned that the picture shows the dimensions as calculated. A thickness of 1.76 m would be too exact to construct in practise. In practise these dimensions should be rounded off to approximately 1.80 m.


6.32
3.44
1.76
Quarry Run
D = 0.88 m
1:1.5
Dolosse
16.02 ton
Dtoe
0.88 m
1:2
17.60
+ 13.64
S.L.
- 16.00
- 22.48
- 26.0
Scale: 1:500
Hs = 8.0 m; T = 10.12 s
Date: 04-04-98
Dimensions: m
Remarks: concrete density 2621 kg/m3
Subject:
Dolos type Sea defence
Name:
D.J. Zwemmer
Paper:
1 of 1

C.2.2 Alternative deepwater sea defence [h=26m]

When looking at the cross section of the deepwater sea defence, one could imagine that this design is rather expensive. To reduce costs the position of the toe can be raised, and the underlying area filled with quarry run, which is much cheaper. The height to which the toe is raised depends on the wave height. In the previous design the armourlayer was stopped at two times the wave height underneath the sea level. Likewise the toe starts at two times the wave height in this new design.

<u>Toe calculation</u>
Again Van der Meer's formula is used to calculate the dimension of the toe stones. Now h_t is known, viz. 2 * 8.0 = 16.0 m.

$$\frac{16.0}{26} = 0.253\left(\frac{8.0}{1.57 * D_{toe}}\right)^{0.7}$$

This results in D_{toe} = 1.43 m, and consequently a stone diameter of 1.58 m is chosen.

<u>Remark:</u> No filterlayer is needed between the toe and the core according to the filter equations.

To compare the two cross sections a costs calculation is made for the new design. It is noted that the price of the core is divided into two parts. The part of the core under the toe has a greater percentage of quarry. The quarry percentage is set at 70% and the percentage sand at 30%. The bigger amount of quarry is needed to prevent the sand from washing out and secondly it is used to support the toe. Nevertheless, the remaining part of the core has a quarry-sand ratio of 40% - 60%, because less quarry is needed now in this part of the core.

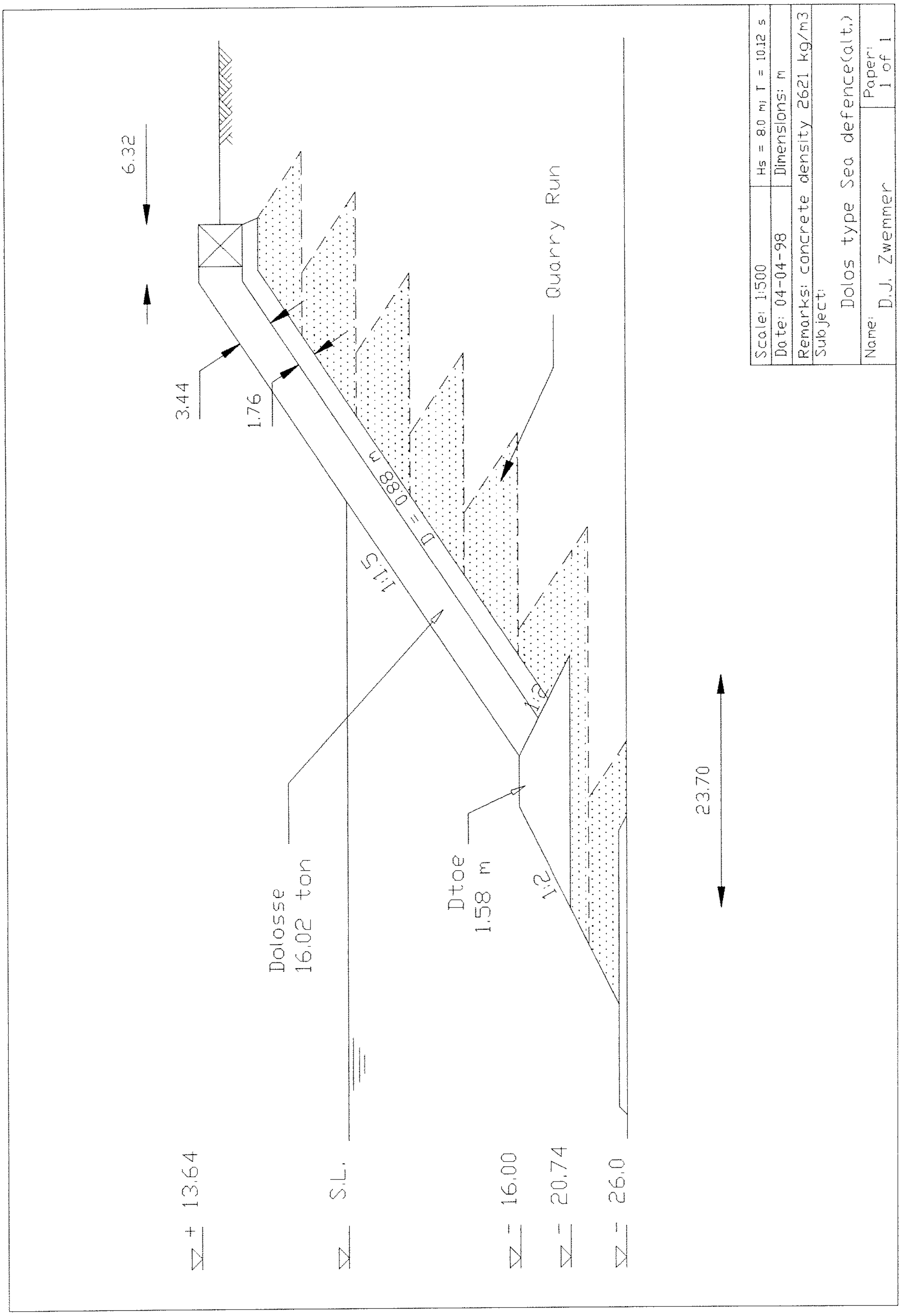
<u>Costs calculation</u>

Price armour layer:	the same as in the previous design	*US $ 20,124*
Concrete slab:	the same as in the previous design	*US $ 1,988*
Price sec. layer:	{(depth2+ run-up - $D + toe$ - t_{armour}) / sin α + (crestwidth - 0.5 * (2*D_{second}) * tan α)}* (2*D_{second}) * Unitprice = {(20.74 + 13.64 - 1.58 - 3.44)/ sin α + (6.32 - 0.5*(2*0.88)* tan α)} * (2*0.88) * 50	*US $ 5,162*
Price core 1:	{(depth + run-up - t_{armour} - 2 * D_{second}) * (crestwidth - tan α * (t_{armour} +2*D_{second}) + (depth2 + run-up - t_{armour} - 2 * D_{second})*tan α * {0.5*(depth2 + run-up - t_{armour} - 2 * D_{second}) + (depth- depth2)) } * Unitprice = {(26 + 13.64 - 3.44 - 2 * 0.88) * (6.32 - (t_{armour} + 2*D_{second})*tan α) + (20.74+ 13.64 - 3.44 - 2*0.88)*tan α *((0.5*(20.74 + 13.64 - 3.44 - 2*0.88) + (26-20.74))} * (0.4*36 + 0.6*3)	*US $ 7.066*
Price core 2:	(depth-depth2) * (15*D_{toe} + 0.5*2*(depth-depth2)) * Unitprice = (26-20.74) * (15*1.58 + 0.5*2*(26-20.74)) * (0.7*36 + 0.3 *3)	*US $ 3,976*
Price toe:	{3*D_{toe} * (3*D_{toe} + 2*3*D_{toe}} * Unitprice = 3 * 1.58 * 1.58 * 9 * 86	+ *US $ 5,797*
TOTAL PRICE		*US $ 44,113*

The costs of the alternative deepwater sea defence is less. This is mainly explained by the fact that the toe-section requires less heavy stones. Instead of a thickness four times D_{50toe} a thickness of three times D_{50toe} is taken, because of a supposed additional support by the quarry run underlayers. If this is totally justified, is questionable. However, when the same dimensions for the toe-structure would have been applied for both deepwater sea defences, the total costs would be nearly equal. For that reason and for reasons of simplification, both in understanding as in computer modelling, the first cross section is maintained throughout the comparisons. A cross section of the alternative design for the deepwater sea defence is shown on the next page.

The cross section with the raised toe may be easier to construct. In the first cross section it is necessary to change from the secondary armour layer to the dolosse, as outer layer, at an underwater position. Something similar is the case for the second design, but here the toe helps the positioning of the dolosse underwater.

Special attention has to be given to the change from secondary armour to the primary armour layer when constructing a sea defence designed like the first cross-section. Because proper placement of the dolosse and support by the toe is essential for the functioning of the sea defence as a whole, and the dolosse in particular.


6.32
3.44
1.76
D = 0.88 m
1:1.5
1:2
Quarry Run
Dolosse
16.02 ton
Dtoe
1.58 m
1:2
23.70
+ 13.64
S.L.
- 16.00
- 20.74
- 26.0
Scale: 1:500
Hs = 8.0 m; T = 10.12 s
Date: 04-04-98
Dimensions: m
Remarks: concrete density 2621 kg/m3
Subject:
Dolos type Sea defence(alt.)
Name:
D.J. Zwemmer
Paper:
1 of 1

C.2.3 Shallow water sea defence [h=8m]

For reasons of comparison a sea defence is constructed under the same conditions at was done in appendix C.2.1 only at shallow water and having an armour layer of rubble stones. This time the wave height is limited by the waterdepth. Thus,

$H_s = 0.5 * 8.0 = 4.0$ m.

$T_m = 7.16$ s. (Remember, wave steepness is still 0.05!)

Again no overtopping may occur and the cheapest possible cross section will be chosen. Both a and b are zero again.

Armour calculations

The maximum slope which can be applied depends on the maximum available weight of the rubble stones, see table C.1. The maximum tonnage amounts to $W_{n50} = 13.904$ ton. If the calculated weight of the rubble stone would exceed this tonnage, a flatter slope is necessary. Using Van der Meer formulae, equation (C.11), with a slope $1 : 1.5$ and P = 0.4 the following $\xi_{transition}$ results.

$$\xi_{transition} = [6.2P^{0.31}\sqrt{\tan\alpha}]^{(\frac{1}{P+0.5})} = 4.421 \xi_m = \tan\alpha/\sqrt{s} = 2.98$$

$\xi_m < \xi_{transition}$, this means that plunging breakers occur and equation (C.9) should be used, with $S_d = 2$ for a slope $1 : 1.5$. N = the number of waves, in this case N = 5000, which means that during a time of 10.12 * 5000 = 14 hours and 3 minutes waves attack on the sea defence. This seems a long time but N = 5000 is not even an abnormal number in Van der Meer's formulae, normally N = 7000 is taken.

$$\frac{H_s}{\Delta d} = 6.2P^{0.18}(\frac{S_d}{\sqrt{N}})^{0.2}\xi_m^{-0.5}$$

The required nominal diameter of the rubble armour stones adds up to 1.71 m, which is slightly smaller than the maximum available diameter, see table refapp-quarry stones. Resulting from the gradation a $D_{n50} = 1.74$ having a $W_{n50} = 13.904$ ton needs to be chosen. The thickness of the primary armour layer is 2 * 1.74 = 3.48 m.

Run-up

Now that the slope is know the run-up can be calculated using equation (refeq-run up rubble2) and hence the crest height.

$$R_{u2\%}/H_s = 1.17\xi_m^{0.46} \qquad \text{for} \xi_m > 1.5$$

This results in $R_{u2\%} = 1.17 * (2.98)^{0.46} * 4.0 = 7.73$ m. This same number accounts for the crest height above sea level

Secondary armour

The secondary armour layer will consist of quarry stones as well, positioned in a double layer. Using the design rule the weight of the secondary armour stones amounts to 1.390 ton, hence a $W_{n50} = 1.443$ ton is chosen, having a $D_{n50} = 0.82$ m.

Core calculations

Using the filter equations for geometrically closed filters the nominal diameter of the core is calculated. Most times the stones having the finest grading can be used as core material. This has to be checked, however. A grading of 2 is assumed for the secondary armour layer.

Secondary armour: $D_{15F} = 0.82 \ / \ \sqrt{2} = 0.58$

Core: $D_{15B} = D_{50B} \ / \ \sqrt{grading}$; $D_{85B} = D_{50B} * \sqrt{grading}$

$$\frac{d_{15F}}{d_{85B}} < 5 \quad \text{and} \quad \frac{d_{15F}}{d_{15B}} > 5$$

If the lowest gradation of $D_{n50} = 0.32$ m would be used as core the filter rules would be satisfied, provided that a wide grading is used. If the grading is not wide enough a third layer has to be applied.

Toe calculation

Normally the thickness of the toe is defined by four times the diameter of the toe stones, this diameter can be found by trial and error. This would result in a toe-height of half the water-depth. So a thickness of three times Dtoe is applied. A D_{50} of 0.51 m fulfils equation (C.24), h_t becomes 6.0 m.

$$\frac{h_t}{8} > 0.253(\frac{4.0}{1.47 * 0.51})^{0.7}$$

The minimum value for h - h_t amounts to 1.76 m and three times D_{toe} is 1.53 m. In other words, the condition is satisfied. Furthermore, a slope 1 : 3 is chosen and the top-berm of the toe is enlarged. All these measures are necessary to insure that the toe properly supports the armour layer.

Concrete top slab

To reduce the amount of armour needed on the crest of the sea defence and to give support to the armour layer a concrete slab is placed. The slab's dimensions are related to the thickness of the armour-layer. Its height and width are equal to the thickness of the armour layer. Thus the width of the crest results:

$$\text{crestwidth} = t_{armour} + (t_{armour} + t_{second}) * sin\alpha = 6.89 \text{ m}$$

Costs calculations

To make sure the sea defence at a depth of 8 m is the cheapest possible sea defence, using the previously designed dimensions, an indication of the total costs for each meter of sea defence needs to be made. It is mentioned that some small simplifications in the calculations of the layer volumes were made. Because the island is filled using small berms, the core consists of quarry stones as well as sand. It is presupposed that 50% of the core volume contains sand and the remaining part of the volume consists of quarry stones. The costs of sand are set to US \$ 3 / m^3.

Price armour layer: {(depth + run-up - 3*D_{toe}) / sin α + (crestwidth - 0.5 * (2*D_{armour}) * tan α) } * (2*D_{armour}) * Unitprice
= {(8 + 7.73 - 3 * 0.51) / sin α + (6.89-0.5*tan α * (2*1.74)) } * (2 * 1.74) * 100
US $ 10,903

Concrete slab: $(t_{armour})^2$ * ρ_c / 1000 * Unitprice
= $(1.74)^2$ * 2400/1000 *70 =
US $ 509

Price sec. layer: {(depth + run-up - D_{toe} - 2*D_{armour}) / sin α + (crestwidth - 0.5 * (2*D_{second}) * tan α)}* (2*D_{second}) * Unitprice
= {(8 + 7.73 - 0.51 - 2*1.74) /sin α + (6.89 - 0.5 * (2*D_{armour})*tan α)} * (2*0.82) * 45 =
US $ 1,984

Price core: (depth + run-up - 2 * D_{armour} - 2 * D_{second}) * { (crestwidth - tan α * (2*D_{armour} + 2*D_{second})) + 0.5 * (depth + run-up - 2 * D_{armour} - 2 * D_{second}) / tan α }* Unitprice
= (8 + 7.73 - 2*1.74 - 2*0.51) * { (6.89 - (2*D_{armour} + 2*D_{second})*tan α) + 0.5 * (8 + 7.73 - 2*1.74 - 2*0.51) / tan α}* (0.5*36 + 0.5*3) =
US $ 2,606

Price toe: {(3*D_{toe} * (6*D_{toe} + 9*D_{toe})} * Unitprice
= 45 * (0.51)2 * 40 =
+ US $ 468

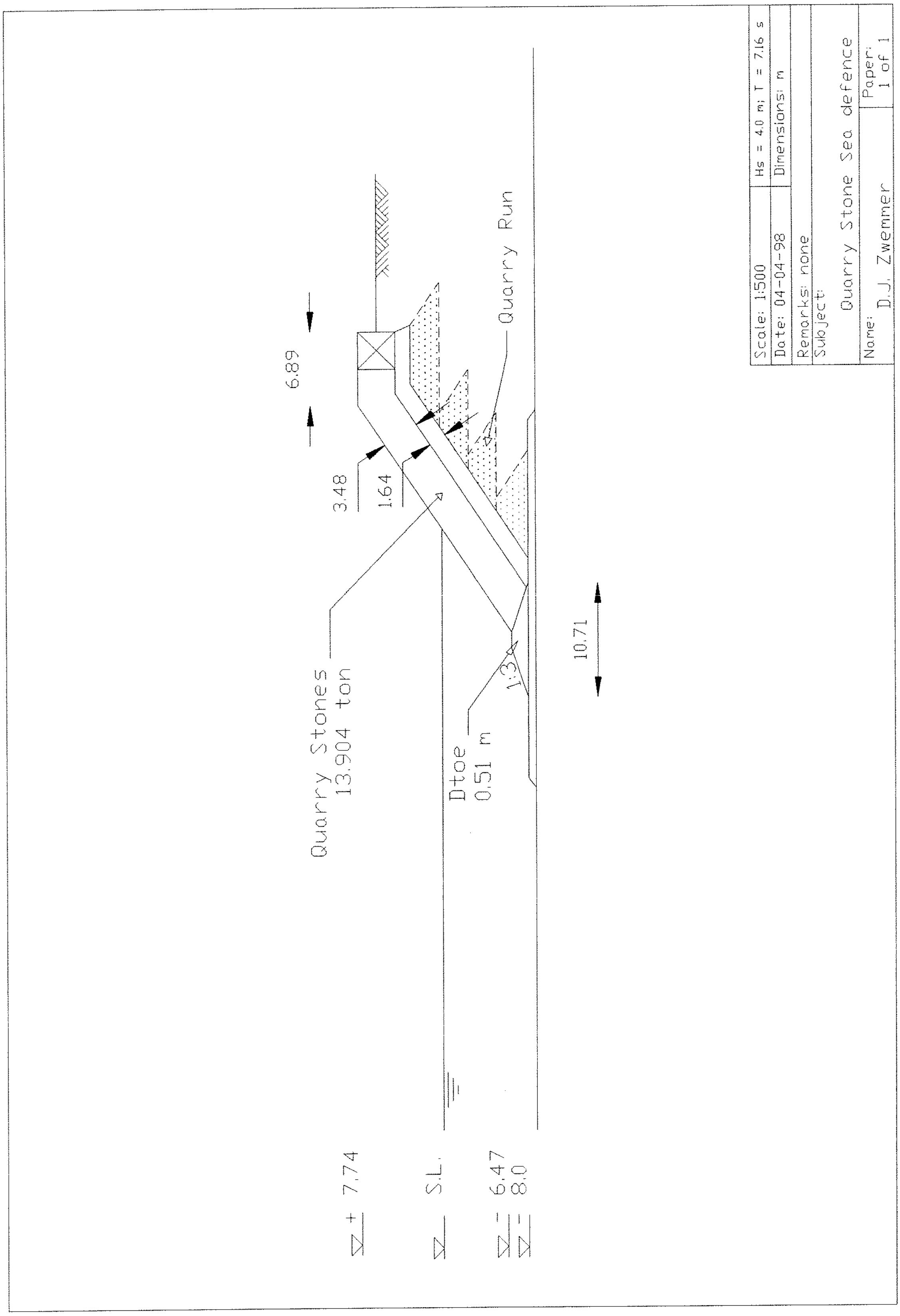
TOTAL PRICE *US $ 16,470*

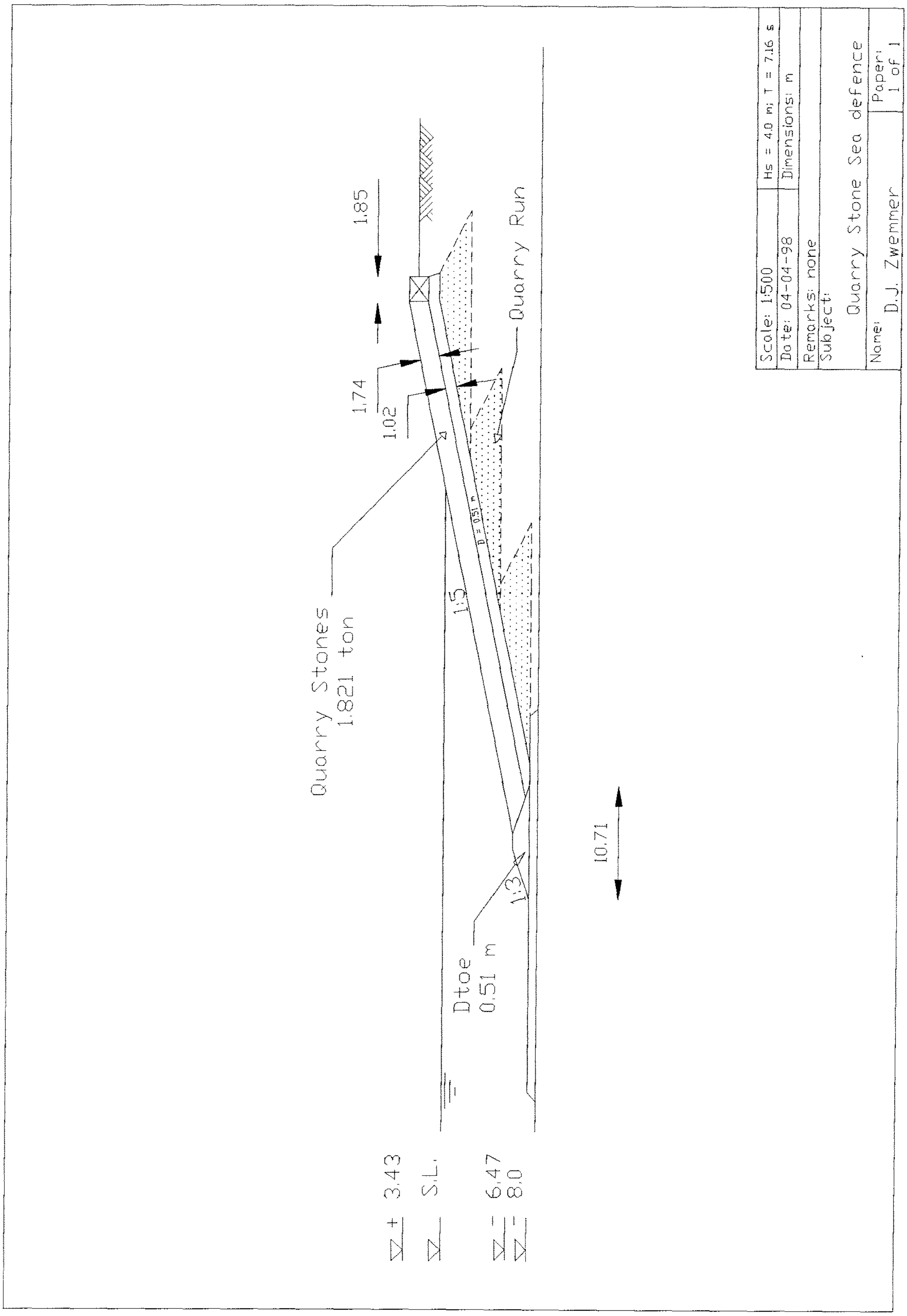
The question now is, whether or not this cross section is more economical than a cross section having a flatter slope? Without showing the calculations of all the dimensions again the result for slopes 1 : 2 through 1 : 6 under the same conditions are summed up in table (C.6).

Apparently, a slope 1 : 5 is the most preferable under these conditions. Due to the reduced run-up and armour weight the total costs for each meter of sea defence is less than would be expected at first. Furthermore, it becomes clear, that the total amount of material for a 1 : 5 slope is less than for a 1 : 6 slope, due to the somewhat steeper slope. On the next pages two cross-sections of rubble mound-sea defences are drawn, one having a slope 1 : 1.5 and the other having a slope 1 : 5.

Slope	Depth [m]	H_s[m]	Run-up [m]	D_{armour}[m]	D_{second}[m]	D_{toe}[m]	P_{total}[1/m]
1 : 1.5	8.0	4.0	7.74	1.74	0.82	0.51	16,470
1 : 2	8.0	4.0	6.78	1.58	0.82	0.51	15,727
1 : 3	8.0	4.0	5.72	1.41	0.82	0.51	16,378
1 : 4	8.0	4.0	4.19	1.03	0.51	0.51	11,797
1 : 5	8.0	4.0	3.43	0.88	0.51	0.51	11,497
1 : 6	8.0	4.0	2.86	0.82	0.51	0.51	11,655

Table C.6: Cost comparison for different slopes (h=8 m, quarry stone-type sea defence)


6.89
Quarry Run
3.48
1.64
Quarry Stones
13.904 ton
1:3
Dtoe
0.51 m
10.71
+ 7.74
S.L.
- 6.47
- 8.0
Scale: 1:500
Hs = 4.0 m; T = 7.16 s
Date: 04-04-98
Dimensions: m
Remarks: none
Subject:
Quarry Stone Sea defence
Name:
D.J. Zwemmer
Paper:
1 of 1


Scale: 1:500
Hs = 4.0 m; T = 7.16 s
Date: 04-04-98
Dimensions: m
Remarks: none
Subject:
Quarry Stone Sea defence
Name:
D.J. Zwemmer
Paper:
1 of 1
1.85
Quarry Run
1.74
1.02
Quarry Stones
1.821 ton
D = 0.51 m
1:5
10.71
1:3
Dtoe
0.51 m
+ 3.43
S.L.
- 6.47
- 8.0

Appendix D

Design of caisson-type seawalls

D.1 Calculation method

The construction of two caisson-type seawalls is detailed below, viz. the vertically and the horizontally composite caisson-type seawall. According to literature both types can be, and have been used in various waterdepths. For a description of both types refer to section 3.3. When one is not absolutely sure about the possibility of excessive impacting wave forces it is advisable to construct a horizontally composite caisson-type seawall. In this report, the horizontally composite caisson is partly protected by a dolosse mound. This design is somewhat more expensive than the vertically composite seawall.

Below, the phases of the design are explained. When no remarks are given, the relations can be applied for both the horizontally and the vertically composite seawalls. Otherwise a distinction is made between the two.

D.1.1 General construction costs

A caisson needs to be prefabricated in a construction dock or at a construction site. Furthermore, the transportation- and sinking-costs have to be taken into account. These general costs depend on a variety of circumstances, e.g. distance from island location to construction dock, difficulty of transport, etc. The following assumptions have been made:

Construction dock:	US $	50,000 / caisson (26,25 m)
Transport and sinking:	US $	170,000 / caisson (26,25 m)

Of course, the first item can not be related to a single element, but because it is not known how many elements are needed nor how much the construction costs or rent of a construction dock amount to, a price of US $ 50,000 per element is maintained. Since the length of a single caisson is set to 26.25 m (explaned further on) the fixed costs per linear metre are *US $ 8,400 /* m^1. Other costs are:

Shuttering for concrete:	US $	2 / m^3
Reinforced concrete:	US $	400 / m^3
Sand fill:	US $	3 / m^3
Dolosse:	US $	80 / ton

D.1.2 Crest height

Japanese caissons are mostly constructed having a crest height of 0.6 * H_s above the mean monthly highest water level. Using this value for a crest height will cause serious overtopping, which will certainly not be allowed for most kinds of man-made islands. Various equations are mentioned in literature to compute a non-overtopping crestheight. In this report a crestheight above Sea Level, which results in minimal overtopping, is assumed of:

$$h_c = 1.25 * H_s \tag{D.1}$$

D.1.3 Dimenions of the caissons

A caisson is constructed out of reinforced concrete (ρ_c = 2500 kg/m^3). The inner part of a caisson is divided into cells. These cells can be fully or partly filled with material (sand (ρ_s = 1800 kg/m^3), gravel, etc.) to increase the weight of the caisson. The size of the inner cells is limited to 4 * 4 m^2 or less in ordinary design. The outer wall is 0.40 to 0.50 m thick, the partition walls are 0.20 to 0.25 m thick, and the bottom slab 0.50 to 0.70 m thick. As the upright caisson-type seawall withstands the wave force mainly with its own weight, the use of prestressed concrete is not advantageous in ordinary situations.

The caissons in this report are assumed to have standard dimensions, being:

Bottom slab thickness:	0.70 m
Outer walls thickness:	0.50 m
Partition walls thickness:	0.25 m
Top slab thickness:	0.50 m
Inner cells:	max 4 * 4 m^2

In the calculations the height and the width of the caisson is part of the optimisation process. However, the length of the caisson is fixed to 26.25 m, this is the length necessary for six inner cells. This is a normal length for caissons, and will most likely not cause any troubles in fabrication or when passing sluices. The width is mostly more of a problem at sluices.

Normally, the width of the caisson is adjusted according to the calculated weight, as are the dimensions of the inner cells. In this report a standard weight per linear metre is assumed, and consequently the weight of the caisson develops proportional with the width. Below, this weight is estimated per linear metre for a given width of the caisson B and a given height $h_{caisson}$.

Weight of concrete parts per linear metre: [kN/m]

Top slab:	0.5 * B * 25 =	12.5 * B
Bottom slab:	0.7 * B * 25 =	17.5 * B
Front wall:	0.5 * ($h_{caisson}$ - 1.2) * 25 =	12.5 * ($h_{caisson}$ - 1.2)
Back wall:	0.5 * ($h_{caisson}$ - 1.2) * 25 =	12.5 * ($h_{caisson}$ - 1.2)
Outer walls (#2):	2 * 0.5 * (B - 1.0) * ($h_{caisson}$ - 1.2) * 25 / 26.25 =	0.9 * ($h_{caisson}$ - 1.2) * (B - 1.0)
Partition walls:	0.10 * 25.25 * (B - 1.0) * ($h_{caisson}$ - 1.2) * 25 / 26.25 =	2.4 * ($h_{caisson}$ - 1.2) * (B - 1.0)

Weight of inner cells filled with sand per linear metre: [kN/m]

Inner cells:	0.90 * f_s * 25.25 * (B - 1.0) * ($h_{caisson}$ - 1.2) * 18 / 26.25 =	15.6 * f_s * ($h_{caisson}$ - 1.2) * (B - 1.0)

TOTAL: W_c = ($h_{caisson}$ - 1.2) * {3.3 (B - 1.0) + 25} + 30B
W_s = ($h_{caisson}$ - 1.2) * (B - 1.0) * 15.6 * f_s

The inner part of the caisson, 25.25 * (B - 1.0) *($h_{caisson}$ - 1.2) is assumed to be divided for 10% into a concrete part (partition walls) and for 90% in an inner cells-part (used for sand-fill). For caissons having a small width the value of 10% is somewhat overestimated, for wider caissons it is more realistic. For a first design this is allowed. The symbol f_s indicates the percentage of the inner cells that is filled with sand.

D.1.4 Shape of the mound

Vertically composite caisson-type seawall

The slopes are normally 1 : 2 - 1 : 3 on the outside and 1 : 2 on the inside. The width of the seaward berm should be at least 5 m in rough waters but should not exceed 1/20 * L. L is the length of the steepest design wave which can occur at the breakwater. In case of vertically composite breakwaters, the mound should be as high as possible from an economic point of view but still low enough to prevent waves breaking on the caisson's front wall. Literature is not exactly clear about the height of the mound. But from studying both constructed caisson-type breakwaters and some flume tests, values between 0.3 and 0.4 times the waterdepth h seem safe for the height of the mound. In this report a value in-between is assumed,

$$h_{mound} \leq 0.35 * h \tag{D.2}$$

This value should certainly be tested in scale models of the caisson-type seawalls. But from this point on it is assumed that no excessive forces impact on the caisson, if equation (D.2) is fulfilled. It is also believed that this value for the mound height excludes excessive breaking waves at lower water levels than S.L. as well. Therefor, the equations by Goda for non breaking wave pressures and forces can be used to determine the dimensions of the caisson, as listed in the section 'Forces on caissons as a result of non breaking wave pressures', paragraph D.1.7.

Horizontally composite caisson-type seawall

The protective elements in front of the caisson should cause the waves to loose their energy and avoid excessive impact forces on the structure. This implies, that all design waves should already be broken before they reach the front wall of the caissons. Consequently, the protective layer must be raised till or above S.L. (the design water-level). Furthermore, the same conditions are applied here for the berm width, as for the vertically composite caisson-type seawall, to force waves to break before they reach the front wall of the caisson. Again the equations by Goda for non breaking wave pressures and forces can be used to determine the dimensions of the caisson because no excessive forces impact on the caisson, as listed in section 'Forces on caissons as a result of non breaking wave pressures', paragraph D.1.7.

Because protection blocks avoid breaking waves, impacting on the caisson, the height of the rubble mound can be freely chosen now, and the limited height of 0.35*h is abandoned. Since the concrete of the caisson attends for the major part of the total costs, the caisson part should be chosen as small as possible (for regions with limited quarry stones this is not very logic, so in that particular case, a vertically composite seawall or an alternative design may prove to be more useful).

A minimum caisson height can be achieved by not completely finishing the caisson up to its designed height, before the caisson is towed. After the caisson is placed into position the remaining part of the caisson can be constructed on site. This way, the draught of the caisson is reduced, and consequently its final height as well. The minimum caisson height during the floating transport depends on its draught and the necessary extra height to prevent waves from flooding the caisson.

Several problems come to light at this moment. Positioning and towing of the caisson will not take place during storm conditions, but during normal sea levels and moderate wave heights. Since this report calculates dimensions of seawalls according to S.L. (the mean sea level plus wave set-up, rise due to storm conditions, etc.) and the significant wave height, some relations have been drawn to come up with the M.S.L. and a moderate wave height. Of course, these relations are very theoretical and they have to be considered as global indications only. Using these relations to derive the moderate wave conditions, a safe environment is created to transport and finish the construction of the caissons in.

M.S.L.	S.L. - 1/3 H_s
Moderate wave height	1/3 H_s

The maximum mound-height is restricted by two conditions.

1. Sufficient keel clearance during positioning: $> 1/3\ H_s$.
2. Extra height above M.S.L., to prevent the partly finished caisson from flooding during towing or during construction of the remaining part: $> 1/3\ H_s$.

The minimum draught d_z of the caisson amounts to:

$$\rho_c * (1/3H_s + d_z + 1/3H_s) * B = \rho_w * d_z * B$$

The first 1/3 H_s prevents the caisson from flooding during transport and the second 1/3 H_s prevents flooding during the construction stage, when the caisson is lowered over a distance 1/3 H_s (the keel clearance) on top of the mound. The symbol ρ_c refers to the average density of the caisson, which is obviously much lower than the normal concrete density, because of the hollow space produced by the inner cells.
The minimum height of the rubble mound under M.S.L. follows: $d_z + 1/3\ H_s$. If the caisson is placed in its final position it will have a height of 1/3 H_s higher than M.S.L. From this position the construction of the caissons can be finished up to the designed height, without fear of waves overtopping or flooding the inner cells. The second condition accounts for the rise in tides as well, therefor it should be made sure, that the maximum wave height during high water, while constructing the remaining parts, should be somewhat lower than 1/3 H_s. Because the caissons are placed during M.S.L. the total height of the caisson becomes:

$$h_{caisson} = 1/3H_s + d_z + h_c + 1/3H_s$$

The slopes of the mound are normally 1 : 2 on the inside. The outside slopes are adapted to the slope of the protective armour units. The width of the seaward berm should be at least 5 m in rough waters, but should not exceed 1/20 * L. L is the length of the steepest design wave which can occur at the breakwater. This seaward berm is required to force all waves to break before they reach the caisson's front wall.

D.1.5 Weight of rubble mound elements

Vertically composite caisson-type seawall

The centre of the mound has to be protected against the scouring by wave action. Foot protection blocks are placed in front of the upright section. The rest of the berm and slope are covered by heavy stones. A formula for the weight of armour stones on the berms of rubble mounds, has been proposed by Tanimoto et al. [1982], as the result of systematic model tests with irregular waves. The minimum weight W of armour stones can be calculated by a formula of the Hudson-type:

$$W = \frac{\gamma_r H_s^3}{N_s^3 \Delta^3} \tag{D.3}$$

N_s indicates the stability number, the value of which depends on the wave conditions and mound dimensions. For waves of normal incidence, Tanimoto et al. [1982] gave the following function for armour stones:

$$N_s = \max\{1.8, 1.3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_s} + 1.8\exp[-1.5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_s}]\} \tag{D.4}$$

in which

$$\kappa = [2kh' / \sinh 2kh'] \sin^2(2\pi B_m/L) \tag{D.5}$$

B_m indicates the berm width.

However, if Tanimoto's method is checked with Van der Meer's method, the calculated weights are much smaller. Because of this fact and the tendency to construct conservatively, the method is abandoned and Van der Meer is used instead.

Horizontally composite caisson-type seawall

The weight of the quarry stones can be computed using the above mentioned method. The weight of the protective elements, in front of horizontally composite caisson-type seawalls, results from a similar approach. From the calculations, done for the rubble mound seawalls, it appeared that dolosse are the most economical types of concrete armour elements. Preferably, these units will be used as protective elements in horizontally composite seawalls.

From literature, no explicit relations to calculate the weight of the dolosse in front of caissons could be found. However, Brebner and Donnelly [SPM 1984] studied stability criteria for random-placed rubble of uniform shape and size, used as foundation and toe protection at vertical-faced, composite structures. In their experiments, the shape and size of the rubble units were uniform, that is, subrounded to subangular beach gravel of 2650 kg/m^3. The examined slopes all had an angle of 1 : 2. A uniform equation was derived, similar to equation (D.3), to calculate the mean weight of the individual armour units.

$$W = \frac{\gamma_r H^3}{N_s^3 \Delta^3} \tag{D.6}$$

The choice of a value for the parameter H, indicated as the design wave height, depends on the use of the structure. Brebner and Donnelly advise to use the local average height of the highest 1 percent of all waves, H_1 for critical structures at open exposed sites where failure would be disastrous, and in absence of reliable wave records. For less critical structures, where some risk of exceeding design assumptions is allowable, wave heights between H_{10} and H_1 are acceptable.

A relation between the relative height of the toe and the stability number N_s was derived from the experiments, see figure D.1. The one problem is that the calculated N_s only applies for rubble toe protection on a 1 : 2 slope. Because equation (D.6) is of the Hudson-type, it seems justified to make a relation between the N_s for rubble and the N_s for dolosse based on their K_D-values. Mind that this is only a very theoretical relation! Calculated weights for dolosse should certainly be tested in scale models or otherwise. If the same slope (1 : 2) is maintained for dolosse, the following relation for the stability number for dolosse is derived.

$$N^3_{s-Dolosse} = \frac{K_{D-Dolosse}}{K_{D-Rubble}} * N^3_{s-Rubble} \tag{D.7}$$

Because the layer of dolosse is raised till S.L., the ratio d_l / d_s = 0, and a N_s^3 for rubble stones results of 8. Using equation (D.7) a $N^3_{s-Dolosse}$ results of (15.8 / 2) * 8 = 63.2. If the crest of the layer of dolosse was chosen lower than S.L. again danger of excessive breaking waves exists. Moreover, a crest level higher than S.L. seems uneconomical.

Brebner and Donnelly advise a sea-ward berm width of B = 0.4 * d_s, this seems a bit high. This ratio results in deepwater in an extremely large amount of dolosse, for that reason again a mound-berm width of 5.0 m or limited by 1/20*L is chosen. The protection of dolosse is at least constructed in a double layer, just like the Brebner and Donnelly experiments.

Compared with the 'normal' Hudson relation for computing the weight of armour elements, the modified Brebner and Donnelly relation gives higher weights. This is caused by the fact, that a higher wave height is used than the significant wave height ($H_s = H_{1/3}$). It would be expected that the relation would result in lower waves, because more damage should be allowed for this type of secondary structures. Concluding, the following can be said..

> *In theory the modified Brebner and Donnelly relation may seem a good method to calculate the weight of concrete protection units. Still this method is abandoned because its has too many limitations and uncertainties. Like, it is not exactly clear what wave height to chose for equation (D.6), and the slope of the dolosse is not accounted for. Furthermore, the relation was explicitly derived for rubble. Finally the modified Brebner and Donnelly relation gives higher weights than the Hudson-equation, which does not seem logically.*

Consequently, the well known Hudson equation is used again, as stated in equation (D.8).

$$W = \frac{\gamma_r H^3}{K_\Delta \Delta^3 \cot\alpha} \tag{D.8}$$

The thickness of the dolosse-layer can be computed using equation (D.9).

$$t_{armour} = nK_\Delta (W/\rho)^{1/3} \tag{D.9}$$

And the number of units per m^2 is given by:

$$N_a = nK_\Delta(1 - n_v)(\rho/W)^{2/3} \tag{D.10}$$

Where:

t_{armour} = thickness of armour layer
n = number of layers
K_Δ = layer thickness coefficient
n_v = volumetric porosity
ρ = density of armour material

Just like rubble mound seawalls, the layer of dolosse is ended at a waterdepth of 2 times H_s, and the secondary armour layer is continued at that location.

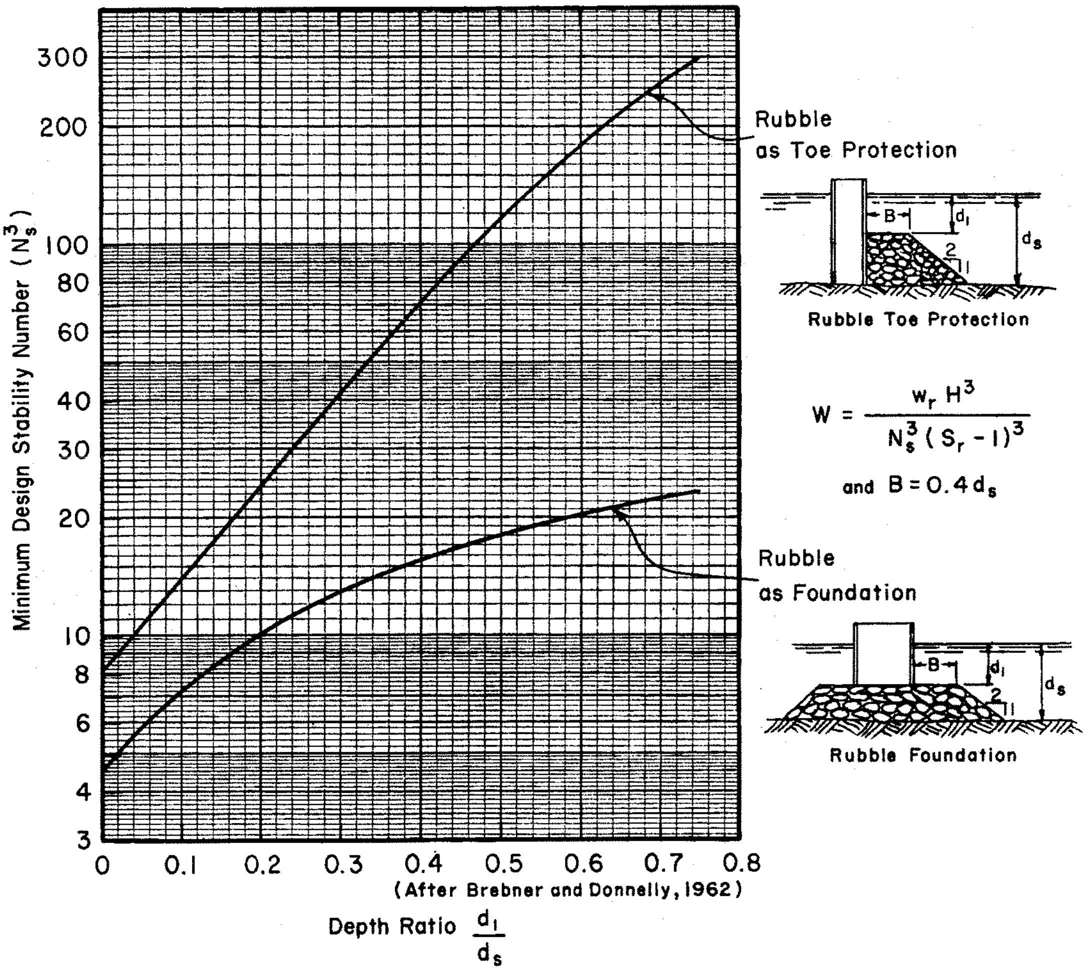


Figure D.1: Stability number for rubble foundation and toe protection [SPM 1984]

D.1.6 Forces on caisson as a result of island's fill

One difference between caisson-type breakwaters and caisson-type seawalls is that the latter has to withstand the forces induced by the island's fill material. When the caisson does not move, neutral ground pressures act on it. The neutral groundpressure can be determined by using the coefficient K_0, which should have a value between 0.5 and 1.0. A correlation exists, viz. $K_0 = 1 - \sin\phi$. The symbol ϕ indicates the angle of internal friction of the backfill material. If the caisson moves outward active groundpressures take over. Using the coefficient $K_a = \frac{1-\sin\phi}{1+\sin\phi}$ these can be computed. The neutral and active ground pressures are computed as follows, provided that no surface load exists.

$$F_{v(ertical)} = \frac{1}{2}\gamma_{dry}h_{dry}^2 + (\gamma_{dry} - h_{wet})\{h_{dry}h_{wet} + \frac{1}{2}h_{wet}2\}$$

$$F_{h,neutral} = K_0 * F_v \quad \text{(D.11)}$$
$$F_{h,active} = K_a * F_v \quad \text{(D.12)}$$

The safety against overturning and sliding towards the seaside under normal conditions (Low Water) is the determinative situation.

D.1.7 Forces on caissons as a result of non breaking wave pressures

The distribution of wave pressures according to Goda (1974) is shown in figure D.2 and the parameters in this figure can be calculated according to the following formulae.

$$\begin{aligned}
p_1 &= 0.5\lambda_1(1+\cos\beta)(\alpha_1 + \lambda_2\cos^2\beta)\rho_w g H_{max} && \text{(D.13)}\\
p_2 &= \frac{p_1}{\cosh(kh)} && \text{(D.14)}\\
p_3 &= \alpha_3 p_1 && \text{(D.15)}\\
p_4 &= p_1(1 - h_c/\eta^*) \qquad : \eta^* > h_c && \text{(D.16)}\\
&= 0 \qquad\qquad\qquad : \eta^* \leq h_c && \text{(D.17)}\\
h_c^* &= \min\{\eta^*, h_c\} && \text{(D.18)}\\
p_u &= 0.5(1+\cos\beta)\alpha_1\alpha_3\rho_w g H_{max} && \text{(D.19)}
\end{aligned}$$

in which

$$\begin{aligned}
\alpha_1 &= 0.6 + 0.5[2kh/\sinh 2kh]^2 && \text{(D.20)}\\
\alpha_2 &= \min[\frac{h_b - d}{3h_b}(\frac{H_{max}}{d})^2, \frac{2d}{H_{max}}] && \text{(D.21)}\\
\alpha_3 &= 1 - \frac{h'}{h}\{1 - \frac{1}{\cosh(kh)}\} && \text{(D.22)}\\
\eta^* &= 0.75\lambda_1(1+\cos\beta)H_{max} && \text{(D.23)}\\
H_{max} &= 1.8H_s \qquad \text{which is an upper-limit-approximation} && \text{(D.24)}
\end{aligned}$$

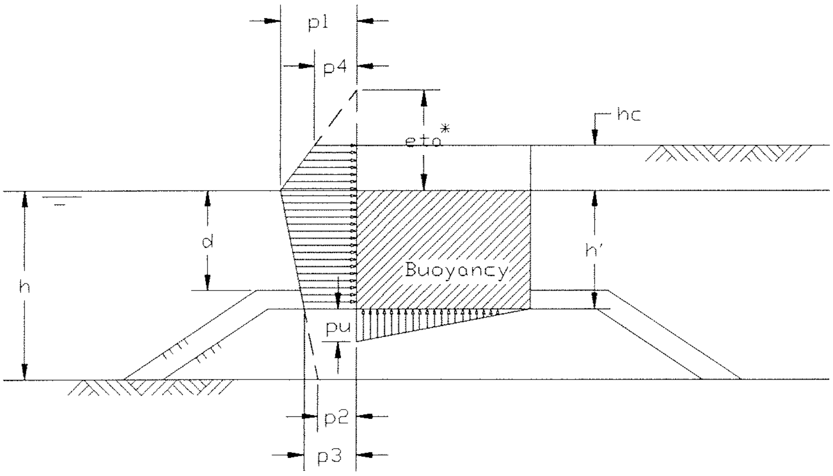


Figure D.2: Wave pressures on a caisson according to Goda

where h_b denotes the waterdepth at the location at a distance $5H_{1/3}$ seaward of the caisson-type seawall.

Takahashi et al. (1990) [31] introduced two parameters λ_1 and λ_2 to account for the effect of protective elements in front of a caisson, e.g. horizontally composite seawall. When no protective elements are placed, in other words when a vertically composite caisson-type breakwater exists, the original Goda equations are in force and both λ_1 and λ_2 have a value of 1.0.

When a horizontally composite caisson-type breakwater is constructed, the concrete units placed in front of the caisson are effective in dissipating incident wave energy and also reducing the wave force, especially the impulsive force on the caisson. The distribution of wave pressure is the same as that of the vertical caisson. The following formulae of the modification factors are formulated empirically:

$$\lambda_1 = \begin{cases} 1.0 & H_{max}/h < 0.3 \\ 1.2 - 2(H_{max}/h)/3 & 0.3 \leq H_{max}/h \leq 0.6 \\ 0.8 & H_{max}/h > 0.6 \end{cases}$$

$$\lambda_2 = \quad 0$$

Knowing the pressure distribution, the resulting forces on the caisson can be calculated. Below, the formulae are given for the total horizontal force P and the uplifting force U. For both, the momentum is given as well.

$$P = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^* \tag{D.25}$$

$$
\begin{aligned}
M_p &= \frac{1}{6}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h'h_c^* + \frac{1}{6}(p_1 + 2p_4)H_C^{*2} && \text{(D.26)} \\
U &= \frac{1}{2}p_u B && \text{(D.27)} \\
M_u &= \frac{2}{3}UB && \text{(D.28)} \\
W &= \text{Underwater weight of upright section per unit extension} && \text{(D.29)}
\end{aligned}
$$

D.1.8 Final design

The caissons must be designed in such a way that overturning and sliding is prevented. For both mechanisms a safety factor of 1.2 is in force. The friction factor μ between the concrete and the rubble stones is empirically taken as 0.6.

$$
\begin{aligned}
\text{Sliding} &= \mu\frac{W-U}{P} \leq 1.2 && \text{(D.30)} \\
\text{Overturning} &= \frac{W-M_u}{M_p} \leq 1.2 && \text{(D.31)}
\end{aligned}
$$

A third condition to be answered is the maximum bearing pressure at the heel of the upright section and the interface between the rubble mound and the foundation. The allowable bearing pressure was taken in the past as a relatively small value like 400 and 500 kN/m^2, but now is taken as 700 kN/m^2 [31].

The optimum design is that design which is cheapest and still answers to the above mentioned conditions.

D.2 Example of deepwater vertically composite caisson type seawall [h=26m]

It is assumed that the depth at the toe of the mound amounts to 26 metres. Furthermore, the significant wave height H_s is 8.0 m. Locally generated waves exist and the wave steepness has a value of 0.05 (seas).

Design description

At a depth of 26 metres a vertically composite caisson-type seawall is to be designed to protect a man-made island. No overtopping may occur and the cheapest possible cross-section will be chosen.

Additional information: perpendicular approaching waves ($\beta = 0$).
a uniform foreshore with a slope Θ of 1 : 250.
Both a and b are zero.

General calculations

Because the significant wave height and the wave period are related to each other by means of the wave steepness, the mean wave period T_m can be derived. According to $s_m = \frac{H_s}{L} = \frac{H_s*2*\pi}{g*T_m^2}$, follows $T_m = 10.12$ s.

The following parameters can be computed in this stage as well:

$H_{1/3} = H_s = 8.0$ m
$H_{max} = 1.8 * 8.0 = 14.4$ m.
$h_b = 26.0 + 5/250 * 8.0 = 26.16$ m.
$\eta^* = 0.75(1 + \cos 0)* 14.4 = 21.6$ m.
Wave length at 26 m: using dispersion-relation, for short waves

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi h}{L}$$

Using trial and error L = 134.23 m is found.
k = 2π/L = 0.0468;

Shape of the mound

To avoid excessive wave forces impacting on the caisson the height of the mound is set on:

$$h_{mound} = 0.35 * h = 9.1m$$

The slopes of the mound are constructed having an angle of 1 : 2. The width of the seaward berm should be at least 5 m in rough waters but should not exceed 1/20 * L = 6.7 m. Consequently the total width at the top of the berm amounts to the width of the caisson plus 5.0 m.

Dimensions of caisson

To provide the caissons and the rubble mound with some extra protection against the scouring by wave action as well as extra stability, it is normal practise to somewhat dig in the caisson into the rubble mound. In this design a difference of 1.5 m between the top of the rubble mound and the lowest part of the caisson is maintained (see also the drawing of the cross-section).

The length of the submerged part of the caisson h' is now known, viz. (26.0 - 9.1) + 1.5 = 18.40 m below S.L. The part of the caisson above S.L. is determined by the overtopping-criterion.

To prevent overtopping a crest height h_c of 1.25 * H_s = 10.00 m above S.L. is desired. The total height results:

$$h_{caisson} = 28.40m$$

Now that the total height of the caisson is known the horizontal pressure P and its momentum M_p can be computed independent of the caisson's width B. Below, the parameters α and the pressures p are listed.

parameters α	pressures p [kN/m^2]
$\alpha_1 = 0.692$	$p_1 = 115.350$
$\alpha_2 = 0.086$	$p_2 = 62.695$
$\alpha_3 = 0.677$	$p_3 = 78.086$
	$p_4 = 61.947$
	$p_u = 69.485$

These values result in P = 2666kN/m and M_p = 37722 kNm/m.

Using trial and error a suitable width B has to be found, at which the criteria for sliding, overturning as well as maximum pressure are fulfilled. The percentage of sandfill in the inner cells is varied as well. The optimum width is the minimum width of the caisson at which the safety and stability factors are just satisfied. When a width of 16.90 m is chosen these conditions are just met, provided that the inner cells of the caisson are totally filled with sand (f_s = 100 %). The following characteristics are valid:

W_c = 2614 kN/m	W_s = 6747 kN/m
$W_{caisson}$ = 9361 kN/m	U = 1/2ρB = 587 kN/m
$W_{submerged}$ = 6158 kN/m	M_u = 2/3UB = 6615 kNm/m

The safety factors are met, like:

S.F.sliding:	$0.6\frac{6158-587}{2666} = 1.25$	> 1.2	Agreed!
S.F.turning:	$\frac{6158*0.5*16.90-6615}{37722} = 1.21$	> 1.2	Agreed!
foundation pressure:	$\frac{6158}{16.90} = 364kN/m^2$	< 700	Agreed!

Without showing further calculations, it is mentioned that the caisson withstands the island's fillpressure during Low Water (chosen as S.L. - 2/3 H_s).

Weight of rubble mound elements

The centre of the mound has to be protected against the scouring by wave action. The rest of the berm and slope are covered by heavy stones. The weight of these stones is calculated using Van der Meer's formulae. The berm width B_m is set on 16.90 + 5.0 = 21.90 m.

$$h_t/h = 0.253(H_s/\Delta D_{n50})^{0.7}$$

The symbol h_t indicates the distance from the water level to the top of the toe, in this case the top of the rubble mound. The relation is safe for h_t / h > 0.5. Because the height of the rubble mound is limited to 0.35*h this condition is fulfilled. Using van der Meer's equation a D_{n50} of 1.32 m for the protective stones of the mound is found. This values corresponds to a weight range of 6000 - 9000 kg. Consequently a secondary layer is necessary between the protective stones and the quarry run. This layer is, however, not applied in the calculations for similar reasons as mentioned for the rubble mound sea defences.

Core calculations

Also for a mound the rules for geometrically closed filters need to be fulfilled. We assume a grading of 2 for both the protective layer of $D_{n50} = 1.32$ m and the centre layer of $D_{n50} = 0.32$ m.

$$\text{Stability:} \quad \frac{d_{15F}}{d_{85B}} = \frac{1.32/\sqrt{2}}{0.32\sqrt{2}} < 5 \quad \text{Agreed!}$$
$$\text{Permeability:} \quad \frac{d_{15F}}{d_{15B}} = \frac{1.32/\sqrt{2}}{0.32/\sqrt{2}} > 5 \quad \text{Agreed!}$$

Thus no extra filter layer is necessary to maintain the stability and the permeability of the mound.

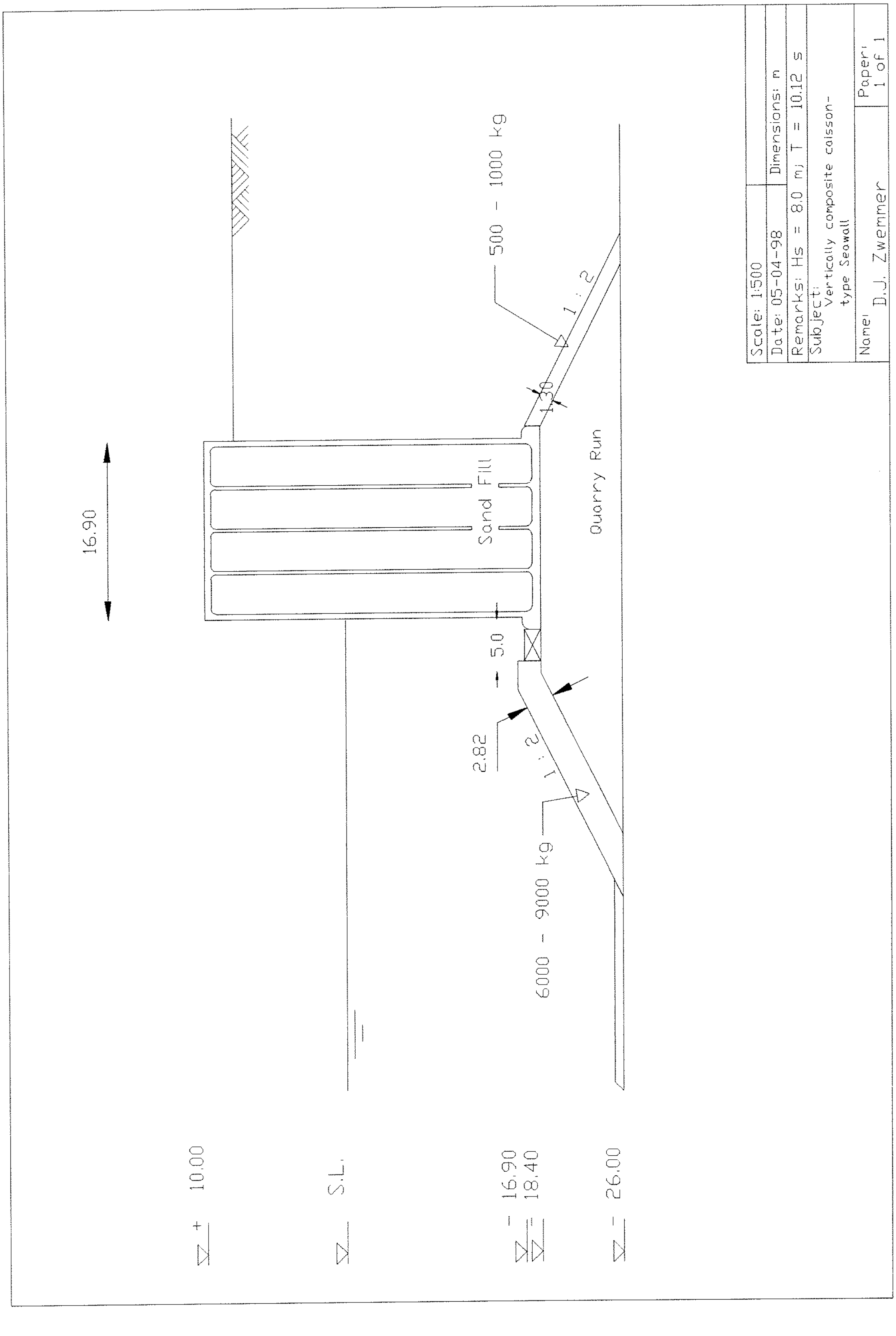
The shore-ward side of the mound is only exposed to wave action during construction. When construction is finished, this part of the mound will be covered with landfill. Consequently, no extra berm length is applied at the shore-ward side of the caisson-type seawall. Stones having a weight of about 500 - 1000 kg ($D_{n50} = 0.65$) will protect the shore-ward side from eroding during construction. Both the sea-ward and the shore-ward protection layers are constructed in a double layer.

In front of the upright section, concrete foot-protection blocks are placed weighing about 8 tons (the same weight as the rubble armour). The five meter in front of the caisson is constructed using two concrete blocks having a height of 1.5 meter and a density of about 2400 kg / m^3 (plain concrete). The desired weight of 8 tons can easily be achieved.

Costs for caisson-type seawall

Volume of caisson:	16.9 * 28.40 =	480 m^3 / m
Volume centre of mound =	(0.35*h -1.5) * { 21.9 + 2*(0.35*h - 1.5)} =	282 m^3 / m
Volume sea-ward protection layer =	(2 * 1.41) * (0.35*h - 1.5)$\sqrt{5}$ =	48 m^3 / m
Volume shore-ward protection layer =	(2 * 0.65) * (0.35*h - 1.5)$\sqrt{5}$ =	22 m^3 / m
Quarry stones:		
	0.0 - 500 kg	US $ 36 / m^3
	500 - 1000 kg	US $ 42 / m^3
	6000 - 9000 kg	US $ 76 / m^3

Fixed costs per linear metre:	US $	8,400 / m	12.3 %
Shuttering for concrete: 2 * 480 =	US $	960 / m	1.4 %
Concrete costs: (2614/25) * 400 =	US $	41,824 / m	61.1 %
Sand fill costs: (6747/18) * 3 =	US $	1,125 / m	1.6 %
Quarry stone mound (0.0 - 500 kg) =	US $	10,152 / m	14.9 %
Quarry stone mound (500 - 1000 kg) =	US $	924 / m	1.3 %
Quarry stone mound (6000 - 9000 kg) =	US $	3,648 / m	5.3 %
Concrete protection blocks	US $	1,500 / m	2.1 %
TOTAL COSTS	US $	68,533 / m	


Scale: 1:500
Date: 05-04-98
Dimensions: m
Remarks: Hs = 8.0 m; T = 10.12 s
Subject:
Vertically composite caisson-type Seawall
Name:
D.J. Zwemmer
Paper:
1 of 1
500 - 1000 kg
1 : 2
1.30
Sand Fill
Quarry Run
16.90
5.0
2.82
1 : 2
6000 - 9000 kg
+ 10.00
S.L.
- 16.90
- 18.40
- 26.00

D.3 Example of deepwater horizontally composite caisson type seawall [h=26m]

It is assumed that the depth at the toe of the mound amounts to 26 metres. Furthermore, the significant wave height H_s is 8.0 m. Locally generated waves exist and the (local) wave steepness has a value of 0.05 (seas).

Design description

At a depth of 26 metres a horizontally composite caisson-type seawall is to be designed to protect a man-made island. No overtopping may occur and the cheapest possible cross-section will be chosen.

Additional information: perpendicular approaching waves ($\beta = 0$).
a uniform foreshore with a slope Θ of 1 : 250.
Both a and b are zero.

General calculations

Because the significant wave height and the wave period are related to each other by means of the wave steepness, the mean wave period T_m can be derived. According to $s_m = \frac{H_s}{L} = \frac{H_s * 2 * \pi}{g * T_m^2}$, follows $T_m = 10.12$ s.

The following parameters can be computed in this stage as well:

$H_{1/3} = H_s = 8.0$ m
$H_{max} = 1.8 * 8.0 = 14.4$ m.
$h_b = 26.0 + 5/250 * 8.0 = 26.16$ m.
$\eta^* = 0.75(1 + \cos 0)* 14.4 = 21.6$ m.
Wave length at 26 m: using dispersion-relation, for short waves

$$L = \frac{gT^2}{2\pi} \tanh \frac{2\pi h}{L}$$

Using trial and error L = 134.23 m is found.
k = 2π/L = 0.0468;

Shape of the mound

Because the concrete part of the caisson itself accounts for the major part of the costs (see vertically composite seawall), the height of the caisson should be kept as small as possible to reduce total costs. One factor, however, that is fixed, is the crest height.

$$h_c = 1.25 * Hs = 10.0m$$

Now the minimum height of the caisson is determined by the crest height and its draught. The minimum draught d_z can be computed using the following equation:

$$\rho_c * (1/3H_s + d_z + 1/3H_s) * B = \rho_w * d_z * B$$

A keel-clearance of $1/3H_s$ is applied to improve manoeuvrability and tolerance during placing. The density of an empty caisson ρ_c is set on 500 kg/m^3 (which implies that about 80% of the caisson's volume consists of hollow space) and the water density is still 1030 kg/m^3. A draught d_z of 5.03 m satisfies the previous equation. Stability of the empty caisson against overturning during floating transport is not accounted for, but will probably be all right, because the length that raises above water about equals the length that is submerged. The floating-stability and the assumed ρ_c can be achieved by partly ballasting the caisson during transport and is improved by ballasting the inner cells. Nonetheless, this increases the draught again.

$$h_{caisson} = 1/3H_s + 5.03 + 10.0 + 1/3H_s = 20.36m$$

Consequently, the top of the rubble-mound is positioned at a level of 10.36 m below S.L. The caisson is 1.5 m dug in the rubble mound. Again the slopes of the mound are constructed having an angle of 1 : 2. The width of the seaward berm should be at least 5 m in rough waters but should not exceed 1/20 * L = 6.7 m. Consequently, the total width at the top of the berm amounts to the width of the caisson plus 5.0 m.

Dimensions of the caisson

To compute the pressures on the caisson according to Goda, first of all the reduction factors λ_1 and λ_2 have to be known. H_{max} / h = 0.55, thus

$$\lambda_1 = 0.83 \text{ and } \lambda_2 = 0$$

Now that the total height of the caisson is known the horizontal pressure P and its momentum M_p can be computed independent of the caisson's width B. Below, the parameters α and the pressures p are listed.

parameters α	pressures p [kN/m^2]
$\alpha_1 = 0.692$	$p_1 = 85.273$
$\alpha_2 = 0.580$	$p_2 = 46.348$
$\alpha_3 = 0.818$	$p_3 = 69.743$
	$p_4 = 37.753$
	$p_u = 83.950$

These values result in P = 1419 kN/m and M_p = 13370 kNm/m.

Using trial and error a suitable width B has to be found, at which the criteria for sliding, overturning as well as maximum pressure are fulfilled. The percentage of sandfill in the inner cells is varied as well. The optimum width is the minimum width of the caisson at which the safety and stability factors are just satisfied. When a width of 11.60 m is chosen these conditions are just met, provided that the inner cells of the caisson are totally filled with sand (f_s = 100 %). The following characteristics are valid:

W_c = 1498 kN/m	W_s = 3171 kN/m
$W_{caisson}$ = 4669 kN/m	U = 1/2ρB = 487 kN/m
$W_{submerged}$ = 3429 kN/m	M_u = 2/3UB = 3765 kNm/m

The safety factors are met, like:

S.F.sliding:	$0.6\frac{3429-487}{1419} = 1.24$	> 1.2	Agreed!
S.F.turning:	$\frac{3429*0.5*11.60-3765}{13370} = 1.21$	> 1.2	Agreed!
foundation pressure:	$\frac{3429}{11.60} = 296kN/m^2$	< 700	Agreed!

Without showing further calculations, it is mentioned that the caisson withstands the island's fillpressure during Low Water (chosen as S.L. - 2/3 H_s).

Dimensions of dolosse-layer

Using the Hudson-formula, with a slope 1 : 1.5 the individual weight of the dolosse can be calculated.

$$W_{Dolos} = \frac{\gamma_r H_s^3}{K_\Delta \Delta^3 \cot\alpha} = \frac{2621 * 8.0^3}{15.8 * 1.54^3 * 1.5} = 15,503kg$$

Consequently, a standard dolosse weight of 16.02 tons is chosen. Furthermore, the thickness of the double layer of dolosse and the number of units per m^2 is required.

$$t_{armour} = nK_\Delta (W/\rho)^{1/3} = 2 * 0.94 * (16020/2621)^{1/3} = 3.44m$$

$$N_a = nK_\Delta(1 - n_v)(\rho/W)^{2/3} = 2 * 0.94 * (1 - 0.56) * (16020/2621)^{2/3} = 2.77/m^2$$

Weight of rubble mound elements

The protective quarry stones positioned on the slopes of the mound are allowed to have a reduced weight, because the wave energy is already diminished by the layer of dolosse. The weight of the secondary layer is computed using rules of thumb.

$$W_{secondary} = 1/10 * W + dolos = 1,602kg.$$

A grading of 1000 - 3000 kg is chosen for the secondary armour, having a $D_{n50} = 0.88$, thus the thickness of this layer amounts to 1.76 m. The shore-ward slope of the mount only requires protection during construction, again quarry stones having a grading 500 - 1000 kg are placed. A foot-protection block 1.5 * 3.0 m^2 is placed in front of the caisson.

Costs for caisson-type seawall

Volume of caisson:	11.6 * 20.36 =	236 m^3 / m
Volume centre of mound:	(26.0-10.36)* {11.6+10.36*1.5 + 1.75*(26.0-10.36)} =	853 m^3 / m
Volume sea-ward protection layer:	(2 * 0.88)*(26.0 - 10.36)*$\sqrt{3.25}$ + 3.44*6.48 =	72 m^3 / m
Volume shore-ward protection layer:	(2 * 0.65) * (26.0 - 10.36)$\sqrt{5}$ =	45 m^3 / m
Toe volume:	4*4*0.88^2 + (4*8*0.88^2) =	37 m^3/m
Effective length of dolos layer:	16.0*1.5 + 5 =	29 m / m
number of dolosse:	29 * 2.77 =	80 / m
Quarry stones:		
	0.0 - 500 kg	US $ 36 / m^3
	500 - 1000 kg	US $ 42 / m^3
	1000 - 3000 kg (also for toe)	US $ 50 / m^3
Dolosse:		US $ 70 / ton

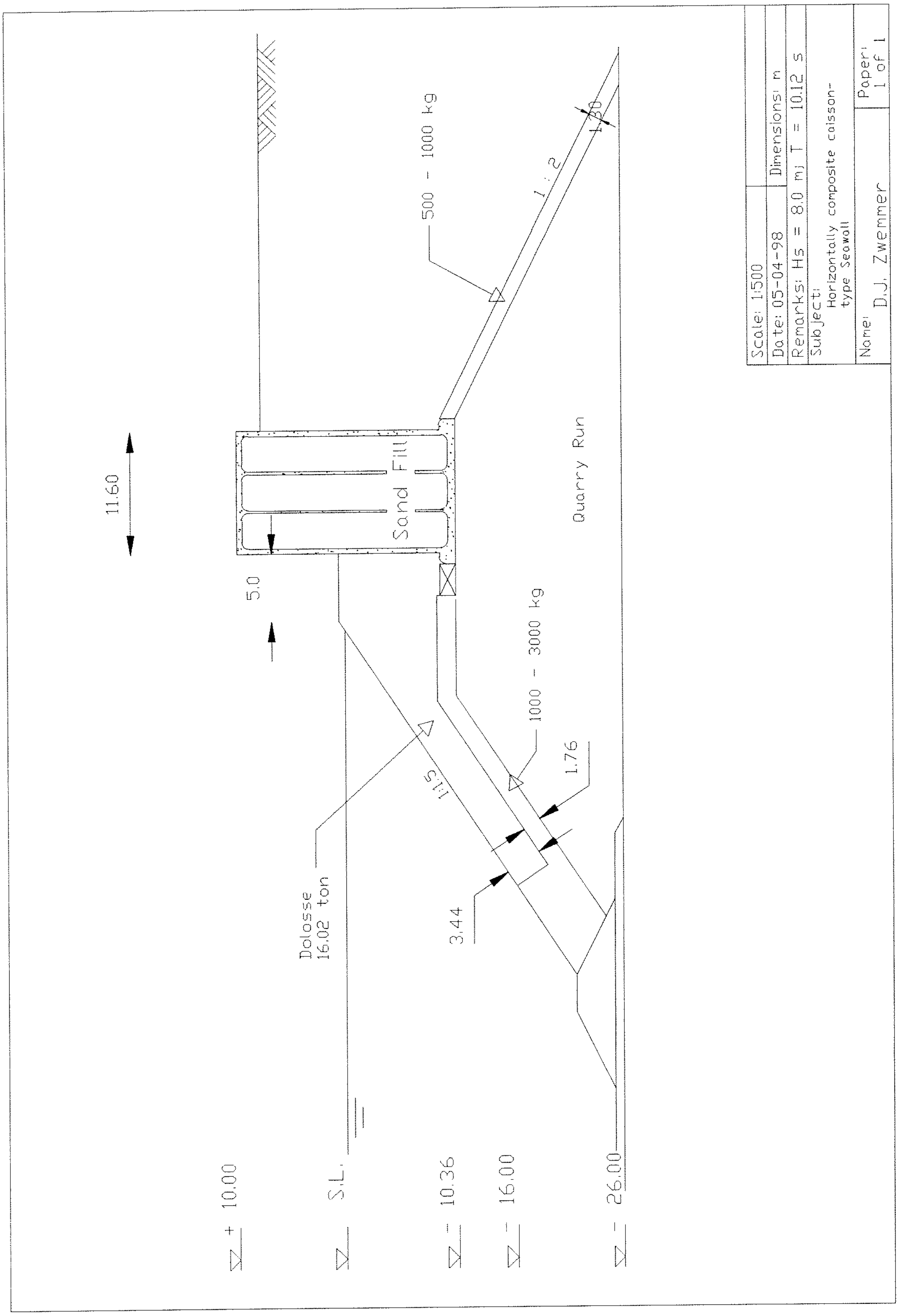
D.4 Conclusions caisson-type seawalls

Two different caisson-type seawalls were treated in this appendix. One of which was the horizontally composite caisson-type seawall. The design and calculation methods for this seawall type are not always evenly clear. Within the framework of this report, not enough time

Fixed costs per linear metre:	US $	8,400 / m	4.0 %
Shuttering for concrete: 2 * 236 =	US $	472 / m	0.3 %
Concrete costs: (1498/25) * 400 =	US $	23,968 / m	13.4 %
Sand fill costs: (3171/18) * 3 =	US $	529 / m	0.3 %
Quarry stone mound (0.5 - 500 kg) =	US $	34,704 / m	19.2 %
Quarry stone mound (500 - 1000 kg) =	US $	1,890 / m	1.9 %
Quarry stone mound (1000 - 3000 kg) =	US $	3,600 / m	2.0 %
Toe =	US $	1,850 / m	1.0 %
Dolosse =	US $	102,528 / m	57.1 %
Concrete protection blocks	US $	1,500 / m	0.8 %
TOTAL COSTS	US $	179,441 / m	

is available to go into the details of the horizontally composite caisson-type seawall-design. Hence, the design of this seawall-type, as stated in this report, should be considered as an introduction. Due to similar reasons no costs-curves for horizontally composite caisson-type seawalls are drawn in the graphs of section 3.5.

However, some conclusions can be drawn. It can be said with some confidence, that horizontally composite caisson-type seawalls are more expensive than vertically composite seawalls. Therefor, the construction costs of vertical composite seawalls can be seen as a lower-limit for the construction costs of horizontally composite caisson-type seawalls.


11.60
5.0
Sand Fill
Quarry Run
500 - 1000 kg
1 : 2
1.30
1000 - 3000 kg
1.76
1:1.5
Dolosse
16.02 ton
3.44
+ 10.00
S.L.
- 10.36
- 16.00
- 26.00
Scale: 1:500
Date: 05-04-98
Dimensions: m
Remarks: Hs = 8.0 m; T = 10.12 s
Subject:
Horizontally composite caisson-type Seawall
Name:
D.J. Zwemmer
Paper:
1 of 1

Appendix E

Details of calculated sea defences

Construction costs for Rubble mound seawall armoured with modified cubes — H_s= 4.0 m; s = 0.05

				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.08	0.53	0.00	0.32	0.32	100	2.18	0.53	0.00	0.32	0.32	*117,62*	1.57	0.53	0.00	0.32	0.32	*126,46*
Costs [%]			60.5	0.0	12.3	7.7			63.1	0.0	13.7	6.5			67.0	0.0	11.5	6.1	
6	3,0	6.24	2.14	0.51	0.32	0.51	100	6.54	2.14	0.51	0.32	0.51	*122,37*	4.72	1.07	0.51	0.32	0.51	116,546
Costs [%]			55.0	10.7	19.2	6.3			55.5	10.6	21.5	5.1			53.8	12.9	23.2	5.4	
10	4,0	8.33	5.34	0.65	0.32	0.51	100	8.72	5.34	0.65	0.32	0.51	*123,45*	6.30	5.34	0.65	0.32	0.51	*138,67*
Costs [%]			54.2	11.6	22.8	3.1			54.3	11.4	25.1	2.6			57.3	11.7	22.8	2.3	
16	4,0	8.33	5.34	0.65	0.32	0.51	100	8.72	5.34	0.65	0.32	0.51	124,40	6.30	5.34	0.65	0.32	0.51	*143,7*
Costs [%]			42.0	15.4	33.8	2.4			41.8	14.4	36.7	2.0			42.9	14.7	36.3	1.7	
20	4,0	8.33	5.34	0.65	0.32	0.51	100	8.72	5.34	0.65	0.32	0.51	125,04	6.30	5.34	0.65	0.32	0.51	*147,27*
Costs [%]			35.7	16.6	40.2	2.1			35.3	15.3	43.4	1.7			35.5	15.4	44.0	1.4	
26	4,0	8.33	5.34	0.65	0.32	0.51	100	8.72	5.34	0.65	0.32	0.51	125,91	6.30	5.34	0.65	0.32	0.51	*152,39*
Costs [%]			28.2	17.4	48.4	1.6			27.7	15.8	51.7	1.3			27.2	15.5	53.4	1.1	
30	4,0	8.33	5.34	0.65	0.32	0.51	100	8.72	5.34	0.65	0.32	0.51	*126,44*	6.30	5.34	0.65	0.32	0.51	*155,51*
Costs [%]			24.4	17.5	53.0	1.4			23.9	15.7	56.4	1.1			23.0	15.3	58.4	0.9	

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.18	0.53	0.00	0.32	0.32	*134,54*	0.94	0.53	0.00	0.32	0.32	*144,36*	0.79	0.53	0.00	0.32	0.32	*157,31*
Costs [%]			70.1	0.0	9.7	5.7			72.7	0.0	8.4	5.3			73.8	0.0	7.5	6.4	
6	3,0	3.54	1.07	0.51	0.32	0.51	*125,56*	2.83	1.07	0.51	0.32	0.51	*136,35*	2.36	0.53	0.00	0.32	0.51	*119,18*
Costs [%]			56.6	13.3	20.6	5.0			58.8	13.6	18.9	4.6			59.2	0.0	32.6	5.2	
10	4,0	4.72	2.14	0.51	0.32	0.51	*125,05*	3.78	2.14	0.51	0.32	0.51	*137,86*	3.15	2.14	0.51	0.32	0.51	*151,41*
Costs [%]			53.6	11.3	29.0	2.5			55.5	11.5	27.5	2.3			56.9	11.7	26.4	2.1	
16	4,0	4.72	2.14	0.51	0.32	0.51	*139*	3.78	2.14	0.51	0.32	0.51	*155,79*	3.15	2.14	0.51	0.32	0.51	*173,16*
Costs [%]			37.4	13.1	45.2	1.8			38.1	13.4	44.8	1.6			38.6	13.5	44.5	1.4	
20	4,0	4.72	2.14	0.51	0.32	0.51	*148,22*	3.78	2.14	0.51	0.32	0.51	*167,97*	3.15	2.14	0.51	0.32	0.51	*188,2*
Costs [%]			29.8	13.3	53.6	1.4			30.0	13.4	53.7	1.2			30.1	13.5	53.7	1.1	
26	4,0	4.72	2.14	0.51	0.32	0.51	*160,93*	3.78	2.14	0.51	0.32	0.51	*185,01*	3.15	2.14	0.51	0.32	0.51	*209,47*
Costs [%]			21.7	12.7	63.1	1.0			21.5	12.7	63.6	0.9			21.4	12.7	64.0	0.8	
30	4,0	4.72	2.14	0.51	0.32	0.51	*168,47*	3.78	2.14	0.51	0.32	0.51	*195,23*	3.15	2.14	0.51	0.32	0.51	*222,32*
Costs [%]			17.9	12.2	67.9	0.8			17.6	12.1	68.5	0.7			17.4	12.0	69.0	0.6	

Construction costs for Rubble mound seawall armoured with dolosse — H_s= 4.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.70	0.53	0.00	0.32	0.32	*75,77*	1.78	0.53	0.00	0.32	0.32	*88,858*	1.29	0.53	0.00	0.32	0.32	*96,73*
Costs [%]			57.4	0.0	13.6	10.2			60.0	0.0	15.3	8.7			63.9	0.0	13.4	8.0	
6	3,0	5.11	1.07	0.51	0.32	0.51	67,76	5.35	1.07	0.51	0.32	0.51	83,517	3.86	0.53	0.00	0.32	0.51	75,36
Costs [%]			46.4	14.3	24.0	9.2			46.7	14.3	26.7	7.5			47.9	0.0	40.4	8.3	
10	4,0	6.81	2.14	0.51	0.32	0.51	*63,89*	7.13	2.14	0.51	0.32	0.51	*80,153*	5.15	1.07	0.51	0.32	0.51	*83,24*
Costs [%]			45.2	12.4	32.3	4.9			44.7	12.1	35.2	3.9			41.3	14.0	38.5	3.8	
16	4,0	6.81	2.14	0.51	0.32	0.51	*69,21*	7.13	2.14	0.51	0.32	0.51	87,6785	5.15	1.07	0.51	0.32	0.51	*97,76*
Costs [%]			32.3	15.0	45.5	3.5			31.7	14.0	48.6	2.8			27.3	14.8	53.8	2.5	
20	4,0	6.81	2.14	0.51	0.32	0.51	*72,29*	7.13	2.14	0.51	0.32	0.51	92,0865	5.15	1.07	0.51	0.32	0.51	*106,61*
Costs [%]			26.3	15.5	52.4	2.9			25.6	14.2	55.6	2.3			21.2	14.4	61.2	1.9	
26	4,0	6.81	2.14	0.51	0.32	0.51	*76,20*	7.13	2.14	0.51	0.32	0.51	97,7204	5.15	1.07	0.51	0.32	0.51	*118,22*
Costs [%]			19.7	15.3	60.5	2.2			19.1	13.9	63.6	1.7			15.1	13.3	69.2	1.4	
30	4,0	6.81	2.14	0.51	0.32	0.51	*78,37*	7.13	2.14	0.51	0.32	0.51	*100,88*	5.15	1.07	0.51	0.32	0.51	*124,87*
Costs [%]			16.6	15.0	64.8	1.8			16.0	13.4	67.7	1.4			12.4	12.6	73.2	1.1	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	0.97	0.53	0.00	0.32	0.32	*104,04*	0.77	0.53	0.00	0.32	0.32	*112,53*	0.64	0.53	0.00	0.32	0.32	*121,66*
Costs [%]			67.2	0.0	11.7	7.4			69.9	0.0	10.6	6.8			72.1	0.0	9.8	6.3	
6	3,0	2.90	0.53	0.00	0.32	0.51	*82,34*	2.32	0.53	0.00	0.32	0.51	*90,27*	1.93	0.53	0.00	0.32	0.51	*98,66*
Costs [%]			50.7	0.0	38.6	7.6			52.9	0.0	37.4	6.9			54.6	0.0	36.5	6.3	
10	4,0	3.86	1.07	0.51	0.32	0.51	*93,04*	3.09	1.07	0.51	0.32	0.51	*103,80*	2.58	0.53	0.00	0.32	0.51	*96,192*
Costs [%]			43.2	14.5	36.7	3.4			44.6	14.9	35.5	3.0			43.0	0.3	52.1	3.3	
16	4,0	3.86	1.07	0.51	0.32	0.51	*112,58*	3.09	1.07	0.51	0.32	0.51	*128,15*	2.58	0.53	0.00	0.32	0.51	*126,07*
Costs [%]			27.7	15.1	53.6	2.2			28.0	15.4	53.4	1.9			25.5	1.1	70.7	1.9	
20	4,0	3.86	1.07	0.51	0.32	0.51	*124,89*	3.09	1.07	0.51	0.32	0.51	*143,81*	2.58	0.53	0.00	0.32	0.51	*145,83*
Costs [%]			21.2	14.6	61.5	1.7			21.2	14.7	61.7	1.4			18.7	1.2	78.1	1.4	
26	4,0	3.86	1.07	0.51	0.32	0.51	*141,39*	3.09	1.07	0.51	0.32	0.51	*165,06*	2.58	0.53	0.00	0.32	0.51	*173,07*
Costs [%]			14.8	13.3	70.0	1.2			14.6	13.3	70.5	1.0			12.5	1.3	84.9	0.9	
30	4,0	3.86	1.07	0.51	0.32	0.51	*150,98*	3.09	1.07	0.51	0.32	0.51	*177,54*	2.58	0.53	0.00	0.32	0.51	*189,24*
Costs [%]			12.0	12.4	74.0	0.9			11.7	12.3	74.6	0.8			9.8	1.2	87.9	0.7	

Construction costs for Rubble mound seawall armoured with tetrapods — H_s= 4.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.89	0.53	0.00	0.32	0.32	94,64	1.98	0.53	0.00	0.32	0.32	111,28	1.43	0.53	0.00	0.32	0.32	120,82
Costs [%]			61.6	0.0	11.8	8.1			64.2	0.0	13.2	6.9			68.0	0.0	11.2	6.4	
6	3,0	5.68	2.14	0.51	0.32	0.51	94,94	5.95	2.14	0.51	0.32	0.51	115,957	4.29	1.07	0.51	0.32	0.51	111,47
Costs [%]			56.1	10.7	18.3	6.6			56.7	10.7	20.4	5.4			54.8	12.9	22.2	5.6	
10	4,0	7.57	5.34	0.65	0.32	0.51	95,20	7.93	5.34	0.65	0.32	0.51	117,36	5.72	5.34	0.65	0.32	0.51	133,31
Costs [%]			55.4	11.7	21.9	3.3			55.5	11.5	24.1	2.7			58.4	11.7	22.0	2.4	
16	4,0	7.57	5.34	0.65	0.32	0.51	95,50	7.93	5.34	0.65	0.32	0.51	118,67	5.72	5.34	0.65	0.32	0.51	138,55
Costs [%]			42.8	15.5	33.2	2.6			42.6	14.6	36.0	2.1			43.6	14.8	35.8	1.8	
20	4,0	7.57	5.34	0.65	0.32	0.51	95,73	7.93	5.34	0.65	0.32	0.51	119,603	5.72	5.34	0.65	0.32	0.51	142,34
Costs [%]			36.3	16.7	39.8	2.2			35.8	15.5	42.9	1.7			36.0	15.5	43.7	1.5	
26	4,0	7.57	5.34	0.65	0.32	0.51	96,09	7.93	5.34	0.65	0.32	0.51	120,94	5.72	5.34	0.65	0.32	0.51	147,82
Costs [%]			28.6	17.5	48.2	1.7			28.1	15.9	51.5	1.4			27.4	15.6	53.3	1.1	
30	4,0	7.57	5.34	0.65	0.32	0.51	96,32	7.93	5.34	0.65	0.32	0.51	121,75	5.72	5.34	0.65	0.32	0.51	151,18
Costs [%]			24.6	17.5	52.9	1.5			24.1	15.8	56.2	1.2			23.2	15.3	58.4	0.9	
		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.53	0.00	0.32	0.32	129,60	0.86	0.53	0.00	0.32	0.32	139,90	0.72	0.53	0.00	0.32	0.32	151,04
Costs [%]			71.1	0.0	9.5	5.9			73.6	0.0	8.4	5.5			75.7	0.0	7.7	5.1	
6	3,0	3.22	1.07	0.51	0.32	0.51	121,29	2.58	1.07	0.51	0.32	0.51	132,67	2.15	0.53	0.00	0.32	0.51	116,42
Costs [%]			57.5	13.3	19.9	5.2			59.6	13.6	18.3	4.7			60.0	0.0	32.0	5.4	
10	4,0	4.29	2.14	0.51	0.32	0.51	121,13	3.43	2.14	0.51	0.32	0.51	134,51	2.86	2.14	0.51	0.32	0.51	148,55
Costs [%]			54.5	11.3	28.3	2.6			56.3	11.5	26.9	2.3			57.6	11.7	25.9	2.1	
16	4,0	4.29	2.14	0.51	0.32	0.51	135,05	3.43	2.14	0.51	0.32	0.51	152,34	2.86	2.14	0.51	0.32	0.51	170,14
Costs [%]			37.9	13.2	44.8	1.8			38.5	13.4	44.4	1.6			39.0	13.5	44.2	1.4	
20	4,0	4.29	2.14	0.51	0.32	0.51	144,35	3.43	2.14	0.51	0.32	0.51	164,55	2.86	2.14	0.51	0.32	0.51	185,19
Costs [%]			30.1	13.3	53.3	1.4			30.3	13.4	53.5	1.3			30.4	13.5	53.6	1.1	
26	4,0	4.29	2.14	0.51	0.32	0.51	157,24	3.43	2.14	0.51	0.32	0.51	181,73	2.86	2.14	0.51	0.32	0.51	206,56
Costs [%]			21.9	12.8	63.0	1.0			21.7	12.7	63.5	0.9			21.6	12.7	63.9	0.8	
30	4,0	4.29	2.14	0.51	0.32	0.51	164,93	3.43	2.14	0.51	0.32	0.51	192,06	2.86	2.14	0.51	0.32	0.51	219,50
Costs [%]			18.0	12.2	67.8	0.9			17.7	12.1	68.5	0.7			17.5	12.0	69.0	0.6	

Construction costs for Rubble mound seawall armoured with quarry stones											Hs = 4.0 m; s = 0.05								
				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.93	0.36	0.00	0.32	0.32	*50,56*	1.69	0.36	0.00	0.32	0.32	*53,76*	1.43	0.08	0.00	0.32	0.32	*45,26*
Costs [%]			36.5	0.0	24.2	15.2			38.5	0.0	24.6	14.3			32.0	0.0	40.4	17.0	
6	3,0	5.80	7.40	0.82	0.32	0.51	114,341	5.08	4.33	0.65	0.32	0.51	95,38	4.29	2.90	0.51	0.32	0.51	94,73
Costs [%]			51.4	16.2	12.3	5.5			47.1	15.8	18.2	6.6			45.4	14.7	23.9	6.6	
10	4,0	7.74	13.90	0.82	0.32	0.51	*118,81*	6.78	10.43	0.82	0.32	0.51	*112,72*	5.72	7.40	0.82	0.32	0.51	*120,18*
Costs [%]			57.6	12.8	16.1	2.6			53.5	14.9	19.4	2.8			50.4	17.4	22.6	2.6	
16	4,0	7.74	13.90	0.82	0.32	0.51	*115,70*	6.78	10.43	0.82	0.32	0.51	*115,23*	5.72	7.40	0.82	0.32	0.51	*129,83*
Costs [%]			45.9	17.5	25.9	2.1			40.6	19.1	31.1	2.1			36.2	20.6	36.3	1.9	
20	4,0	7.74	13.90	0.82	0.32	0.51	*113,96*	6.78	10.43	0.82	0.32	0.51	*116,77*	5.72	7.40	0.82	0.32	0.51	*135,76*
Costs [%]			39.5	19.3	31.9	1.8			34.0	20.4	37.9	1.8			29.4	21.1	43.9	1.5	
26	4,0	7.74	13.90	0.82	0.32	0.51	*111,79*	6.78	10.43	0.82	0.32	0.51	*118,8*	5.72	7.40	0.82	0.32	0.51	*143,59*
Costs [%]			31.9	20.7	40.0	1.5			26.4	21.0	46.6	1.4			22.0	20.8	53.1	1.1	
30	4,0	7.74	13.90	0.82	0.32	0.51	*110,61*	6.78	10.43	0.82	0.32	0.51	*119,96*	5.72	7.40	0.82	0.32	0.51	*148,09*
Costs [%]			27.8	21.0	44.6	1.3			22.6	20.8	51.4	1.2			18.4	20.1	58.0	1.0	
				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.08	0.00	0.32	0.32	*47,01*	0.86	0.08	0.00	0.32	0.32	*49,44*	0.72	0.08	0.00	0.32	0.32	*52,30*
Costs [%]			34.8	0.0	38.7	16.4			37.1	0.0	37.7	15.6			39.1	0.0	37.0	14.7	
6	3,0	3.22	1.44	0.51	0.32	0.51	*83,38*	2.58	0.72	0.00	0.32	0.51	*70,95*	2.15	0.72	0.00	0.32	0.51	*76,11*
Costs [%]			38.2	19.2	28.3	7.5			37.4	0.0	48.8	8.8			39.1	0.0	48.0	8.2	
10	4,0	4.29	2.90	0.51	0.32	0.51	*91,74*	3.43	1.82	0.51	0.32	0.51	*91,91*	2.86	1.44	0.51	0.32	0.51	*95,06*
Costs [%]			40.0	14.9	36.8	3.4			35.5	17.0	40.5	3.4			32.6	18.6	42.5	3.3	
16	4,0	4.29	2.90	0.51	0.32	0.51	*112,19*	3.43	1.82	0.51	0.32	0.51	*119,50*	2.86	1.44	0.51	0.32	0.51	*129,18*
Costs [%]			25.4	16.0	53.3	2.2			21.2	17.0	57.6	2.0			18.6	17.7	60.1	1.9	
20	4,0	4.29	2.90	0.51	0.32	0.51	*124,91*	3.43	1.82	0.51	0.32	0.51	*136,80*	2.86	1.44	0.51	0.32	0.51	*150,72*
Costs [%]			19.4	15.5	61.1	1.7			15.7	16.0	65.2	1.5			13.5	16.4	67.5	1.4	
26	4,0	4.29	2.90	0.51	0.32	0.51	*141,82*	3.43	1.82	0.51	0.32	0.51	*159,90*	2.86	1.44	0.51	0.32	0.51	*179,62*
Costs [%]			13.5	14.3	69.4	1.2			10.6	14.2	73.0	1.0			9.0	14.3	75.0	0.9	
30	4,0	4.29	2.90	0.51	0.32	0.51	*151,59*	3.43	1.82	0.51	0.32	0.51	*173,30*	2.86	1.44	0.51	0.32	0.51	*196,44*
Costs [%]			10.9	13.4	73.4	0.9			8.5	13.2	76.7	0.8			7.1	13.1	78.4	0.7	

Construction costs for Rubble mound seawall armoured with modified cubes — Hs = 6.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	2.08	0.53	0.00	0.32	0.32	100	2.18	0.53	0.00	0.32	0.32	117,62	1.57	0.53	0.00	0.32	0.32	126,46
Costs [%]			60.5	0.0	12.3	7.7			63.1	0.0	13.7	6.5			67.0	0.0	11.5	6.1	
6	3,0	6.24	2.14	0.51	0.32	0.51	100	6.54	2.14	0.51	0.32	0.51	122,37	4.72	1.07	0.51	0.32	0.51	116,55
Costs [%]			55.0	10.7	19.2	6.3			55.5	10.6	21.5	5.1			53.8	12.9	23.2	5.4	
10	5,0	10.41	10.68	0.82	0.32	0.65	100	10.90	10.68	0.82	0.32	0.65	123,84	7.87	5.34	0.65	0.32	0.65	114,27
Costs [%]			57.4	11.7	19.2	2.4			57.3	11.5	21.2	2.5			57.4	10.6	24.8	2.1	
16	6,0	12.49	16.02	0.88	0.32	0.82	100	13.08	12.00	0.82	0.32	0.82	114,72	9.45	10.68	0.82	0.32	0.82	126,76
Costs [%]			50.7	13.2	24.7	4.0			49.5	11.6	30.1	3.5			51.2	12.2	28.9	3.1	
20	6,0	12.49	16.02	0.88	0.32	0.65	100	13.08	12.00	0.82	0.32	0.65	115,76	9.45	10.68	0.82	0.32	0.65	130,22
Costs [%]			45.8	15.7	30.1	1.7			44.3	13.4	36.0	1.5			45.0	13.9	35.8	1.3	
26	6,0	12.49	16.02	0.88	0.32	0.65	100	13.08	12.00	0.82	0.32	0.65	116,91	9.45	10.68	0.82	0.32	0.65	134,70
Costs [%]			38.3	17.6	36.8	1.8			36.7	14.6	43.3	1.5			36.4	14.8	44.3	1.3	
30	6,0	12.49	16.02	0.88	0.32	0.65	100	13.08	12.00	0.82	0.32	0.65	117,70	9.45	10.68	0.82	0.32	0.65	137,70
Costs [%]			34.2	18.4	40.8	1.6			32.6	15.0	47.6	1.3			31.8	15.1	49.2	1.1	

		1 : 4						1 : 5						1 : 6					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	1.18	0.53	0.00	0.32	0.32	134,54	0.94	0.53	0.00	0.32	0.32	144,36	0.79	0.53	0.00	0.32	0.32	157,31
Costs [%]			70.1	0.0	9.7	5.7			72.7	0.0	8.4	5.3			73.8	0.0	7.5	6.4	
6	3,0	3.54	1.07	0.51	0.32	0.51	125,56	2.83	1.07	0.51	0.32	0.51	136,35	2.36	0.53	0.00	0.32	0.51	119,18
Costs [%]			56.6	13.3	20.6	5.0			58.8	13.6	18.9	4.6			59.2	0.0	32.6	5.2	
10	5,0	5.90	5.34	0.65	0.32	0.65	123,85	4.72	5.34	0.65	0.32	0.65	135,88	3.94	5.34	0.65	0.32	0.65	148,06
Costs [%]			60.3	10.9	22.1	1.9			62.2	11.0	20.2	2.3			63.9	11.2	18.9	2.1	
16	6,0	7.08	10.68	0.82	0.32	0.82	139,6	5.67	5.34	0.65	0.32	0.82	133,87	4.72	5.34	0.65	0.32	0.82	147,13
Costs [%]			53.5	12.6	27.0	2.8			50.4	11.2	32.7	3.0			51.7	11.4	31.8	2.7	
20	6,0	7.08	10.68	0.82	0.32	0.65	145,04	5.67	5.34	0.65	0.32	0.65	142,53	4.72	5.34	0.65	0.32	0.65	157,73
Costs [%]			46.5	14.3	34.5	1.2			42.7	12.4	41.4	1.2			43.5	12.6	40.8	1.1	
26	6,0	7.08	10.68	0.82	0.32	0.65	152,29	5.67	5.34	0.65	0.32	0.65	154,8	4.72	5.34	0.65	0.32	0.65	172,75
Costs [%]			37.1	15.1	43.8	1.2			32.9	12.7	51.5	1.1			33.2	12.8	51.4	1.0	
30	6,0	7.08	10.68	0.82	0.32	0.65	157,25	5.67	5.34	0.65	0.32	0.65	162,95	4.72	5.34	0.65	0.32	0.65	182,83
Costs [%]			32.1	15.3	49.2	1.0			27.9	12.6	57.0	1.0			28.0	12.7	57.1	0.9	

Construction costs for Rubble mound seawall armoured with dolosse													Hs = 6.0 m; s = 0.05						
				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.70	0.53	0.00	0.32	0.32	*75,77*	1.78	0.53	0.00	0.32	0.32	*88,86*	1.29	0.53	0.00	0.32	0.32	*96,73*
Costs [%]			57.4	0.0	13.6	10.2			60.0	0.0	15.3	8.7			63.9	0.0	13.4	8.0	
6	3,0	5.11	1.07	0.51	0.32	0.51	67,76	5.35	1.07	0.51	0.32	0.51	83,52	3.86	0.53	0.00	0.32	0.51	75,36
Costs [%]			46.4	14.3	24.0	9.2			46.7	14.3	26.7	7.5			47.9	0.0	40.4	8.3	
10	5,0	8.52	5.34	0.65	0.32	0.65	*64,30*	8.92	5.34	0.65	0.32	0.65	*80,23*	6.44	2.14	0.51	0.32	0.65	*75,52*
Costs [%]			50.9	12.2	26.5	3.7			50.6	12.0	29.0	3.0			47.1	11.4	35.3	3.2	
16	6,0	10.22	10.68	0.82	0.32	0.82	*71,98*	10.70	5.34	0.65	0.32	0.82	78,14	7.73	5.34	0.65	0.32	0.82	*90,32*
Costs [%]			44.9	14.0	29.9	5.5			40.4	11.6	39.6	5.1			42.3	12.0	38.4	4.4	
20	6,0	10.22	10.68	0.82	0.32	0.65	*72,83*	10.70	5.34	0.65	0.32	0.65	80,68	7.73	5.34	0.65	0.32	0.65	*95,33*
Costs [%]			40.1	16.3	36.7	1.8			35.3	13.0	47.1	1.6			36.2	13.3	46.7	1.4	
26	6,0	10.22	10.68	0.82	0.32	0.65	*75,85*	10.70	5.34	0.65	0.32	0.65	85,92	7.73	5.34	0.65	0.32	0.65	*103,93*
Costs [%]			32.2	17.5	43.9	2.3			27.7	13.4	54.5	2.0			27.8	13.4	55.2	1.7	
30	6,0	10.22	10.68	0.82	0.32	0.65	*77,37*	10.70	5.34	0.65	0.32	0.65	*88,82*	7.73	5.34	0.65	0.32	0.65	*109,07*
Costs [%]			28.2	17.9	48.3	2.0			24.0	13.5	58.8	1.8			23.7	13.3	60.0	1.4	

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	0.97	0.53	0.00	0.32	0.32	*104,04*	0.77	0.53	0.00	0.32	0.32	*112,53*	0.64	0.53	0.00	0.32	0.32	*121,66*
Costs [%]			67.2	0.0	11.7	7.4			69.9	0.0	10.6	6.8			72.1	0.0	9.8	6.3	
6	3,0	2.90	0.53	0.00	0.32	0.51	*82,34*	2.32	0.53	0.00	0.32	0.51	*90,27*	1.93	0.53	0.00	0.32	0.51	*98,66*
Costs [%]			50.7	0.0	38.6	7.6			52.9	0.0	37.4	6.9			54.6	0.0	36.5	6.3	
10	5,0	4.83	2.14	0.51	0.32	0.65	*83,08*	3.86	2.14	0.51	0.32	0.65	*91,66*	3.22	1.07	0.51	0.32	0.65	*90,93*
Costs [%]			49.6	11.8	32.9	2.9			51.4	12.1	31.3	2.6			46.4	14.1	35.3	2.6	
16	6,0	5.80	5.34	0.65	0.32	0.82	*101,14*	4.64	2.14	0.51	0.32	0.82	*98,48*	3.86	2.14	0.51	0.32	0.82	*109,18*
Costs [%]			44.2	12.4	36.9	3.9			38.5	11.2	44.9	4.0			39.4	11.4	44.3	3.6	
20	6,0	5.80	5.34	0.65	0.32	0.65	*108,40*	4.64	2.14	0.51	0.32	0.65	*109,02*	3.86	2.14	0.51	0.32	0.65	*122,04*
Costs [%]			37.2	13.6	45.8	1.2			31.4	11.8	54.5	1.2			31.8	11.9	54.1	1.1	
26	6,0	5.80	5.34	0.65	0.32	0.65	*120,20*	4.64	2.14	0.51	0.32	0.65	*125,75*	3.86	2.14	0.51	0.32	0.65	*142,02*
Costs [%]			28.1	13.6	55.2	1.5			22.7	11.3	63.7	1.4			22.9	11.3	63.8	1.2	
30	6,0	5.80	5.34	0.65	0.32	0.65	*127,49*	4.64	2.14	0.51	0.32	0.65	*136,18*	3.86	2.14	0.51	0.32	0.65	*154,65*
Costs [%]			23.7	13.3	60.4	1.2			18.8	10.8	68.6	1.2			18.8	10.8	68.8	1.0	

Construction costs for Rubble mound seawall armoured with tetrapods — Hs = 6.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.89	0.53	0.00	0.32	0.32	94,64	1.98	0.53	0.00	0.32	0.32	111,28	1.43	0.53	0.00	0.32	0.32	120,82
Costs [%]			61.6	0.0	11.8	8.1			64.2	0.0	13.2	6.9			68.0	0.0	11.2	6.4	
6	3,0	5.68	2.14	0.51	0.32	0.51	94,94	5.95	2.14	0.51	0.32	0.51	115,96	4.29	1.07	0.51	0.32	0.51	111,47
Costs [%]			56.1	10.7	18.3	6.6			56.7	10.7	20.4	5.4			54.8	12.9	22.2	5.6	
10	5,0	9.46	10.68	0.82	0.32	0.65	94,84	9.91	10.68	0.82	0.32	0.65	116,60	7.16	5.34	0.65	0.32	0.65	109,23
Costs [%]			58.7	11.7	18.3	2.5			58.9	11.6	20.3	2.1			58.5	10.7	23.8	2.2	
16	6,0	11.35	16.02	0.88	0.32	0.82	95,25	11.89	12.00	0.82	0.32	0.82	109,01	8.59	10.68	0.82	0.32	0.82	121,78
Costs [%]			51.8	13.2	23.8	4.2			50.6	11.7	29.0	3.6			52.2	12.3	28.1	3.3	
20	6,0	11.35	16.02	0.88	0.32	0.65	95,29	11.89	12.00	0.82	0.32	0.65	109,70	8.59	10.68	0.82	0.32	0.65	125,22
Costs [%]			46.7	15.8	29.4	1.8			45.4	13.6	35.3	1.2			45.8	14.0	35.2	1.4	
26	6,0	11.35	16.02	0.88	0.32	0.65	95,55	11.89	12.00	0.82	0.32	0.65	111,54	8.59	10.68	0.82	0.32	0.65	129,88
Costs [%]			39.0	17.7	36.2	1.8			37.4	14.7	42.6	1.6			36.9	14.9	43.8	1.4	
30	6,0	11.35	16.02	0.88	0.32	0.65	95,71	11.89	12.00	0.82	0.32	0.65	112,52	8.59	10.68	0.82	0.32	0.65	133,01
Costs [%]			34.8	18.5	40.4	1.6			33.1	15.1	47.1	1.4			32.2	15.2	48.9	1.2	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.53	0.00	0.32	0.32	129,6	0.86	0.53	0.00	0.32	0.32	139,90	0.72	0.53	0.00	0.32	0.32	151,04
Costs [%]			71.1	0.0	9.5	5.9			73.6	0.0	8.4	5.5			75.7	0.0	7.7	5.1	
6	3,0	3.22	1.07	0.51	0.32	0.51	121,29	2.58	1.07	0.51	0.32	0.51	132,67	2.15	0.53	0.00	0.32	0.51	116,42
Costs [%]			57.5	13.3	19.9	5.2			59.6	13.6	18.3	4.7			60.0	0.0	32.0	5.4	
10	5,0	5.37	5.34	0.65	0.32	0.65	119,62	4.29	5.34	0.65	0.32	0.65	131,55	3.58	2.14	0.51	0.32	0.65	118,36
Costs [%]			61.3	10.9	21.4	2.0			63.4	11.1	19.7	1.8			58.3	10.7	26.6	2.0	
16	6,0	6.44	10.68	0.82	0.32	0.82	135,42	5.15	5.34	0.65	0.32	0.82	130,57	4.29	5.34	0.65	0.32	0.82	144,27
Costs [%]			54.4	12.6	26.3	2.9			51.1	11.2	32.1	3.0			52.3	11.4	31.3	2.8	
20	6,0	6.44	10.68	0.82	0.32	0.65	140,78	5.15	5.34	0.65	0.32	0.65	138,70	4.29	5.34	0.65	0.32	0.65	154,73
Costs [%]			47.2	14.3	34.0	1.2			43.5	12.5	41.0	0.9			44.0	12.6	40.4	1.1	
26	6,0	6.44	10.68	0.82	0.32	0.65	148,15	5.15	5.34	0.65	0.32	0.65	151,37	4.29	5.34	0.65	0.32	0.65	169,72
Costs [%]			37.5	15.2	43.5	1.2			33.3	12.8	51.2	1.2			33.6	12.8	51.1	1.0	
30	6,0	6.44	10.68	0.82	0.32	0.65	153,19	5.15	5.34	0.65	0.32	0.65	159,55	4.29	5.34	0.65	0.32	0.65	179,82
Costs [%]			32.4	15.3	49.0	1.0			28.2	12.6	56.8	1.0			28.3	12.7	57.0	0.9	

ound seawall armoured with quarry stones												Hs = 6.0 m; s = 0.05							
				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.93	0.36	0.00	0.32	0.32	*50,56*	1.69	0.36	0.00	0.32	0.32	*53,76*	1.43	0.08	0.00	0.32	0.32	*45,26*
Costs [%]			36.5	0.0	24.2	15.2			38.5	0.0	24.6	14.3			32.0	0.0	40.4	17.0	
6	3,0	5.80	7.40	0.82	0.32	0.51	114,341	5.08	4.33	0.65	0.32	0.51	95,38	4.29	2.90	0.51	0.32	0.51	94,73
Costs [%]			51.4	16.2	12.3	5.5			47.1	15.8	18.2	6.6			45.4	14.7	23.9	6.6	
10	5,0													7.16	10.43	0.82	0.32	0.65	112,75
Costs [%]															56.0	13.7	20.8	2.8	
16	6,0																		
Costs [%]																			
20	6,0																		
Costs [%]																			
26	6,0																		
Costs [%]																			
30	6,0																		
Costs [%]																			
				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.08	0.00	0.32	0.32	*47,01*	0.86	0.08	0.00	0.32	0.32	*49,44*	0.72	0.08	0.00	0.32	0.32	*52,298*
Costs [%]			34.8	0.0	38.7	16.4			37.1	0.0	37.7	15.6			39.1	0.0	37.0	14.7	
6	3,0	3.22	1.44	0.51	0.32	0.51	*83,38*	2.58	0.72	0.00	0.32	0.51	*70,95*	2.15	0.72	0.00	0.32	0.51	*76,115*
Costs [%]			38.2	19.2	28.3	7.5			37.4	0.0	48.8	8.8			39.1	0.0	48.0	8.2	
10	5,0	5.37	7.40	0.82	0.32	0.65	*106,45*	4.29	4.33	0.65	0.32	0.65	*93,27*	3.58	2.90	0.51	0.32	0.65	*87,44*
Costs [%]			53.5	16.5	22.1	2.3			48.0	15.9	28.9	2.6			43.8	14.4	35.3	2.7	
16	6,0	6.44	10.43	0.82	0.32	0.82	*119,26*	5.15	7.40	0.82	0.32	0.82	*118,68*	4.29	4.33	0.65	0.32	0.82	*110,12*
Costs [%]			48.1	14.4	30.3	3.3			43.7	16.5	33.4	3.3			36.8	15.1	42.1	3.6	
20	6,0	6.44	10.43	0.82	0.32	0.65	*126,21*	5.15	7.40	0.82	0.32	0.65	*129,08*	4.29	4.33	0.65	0.32	0.65	*123,64*
Costs [%]			41.1	15.9	38.3	1.4			36.2	17.7	42.1	1.3			29.6	15.8	51.7	1.1	
26	6,0	6.44	10.43	0.82	0.32	0.65	*135,99*	5.15	7.40	0.82	0.32	0.65	*143,82*	4.29	4.33	0.65	0.32	0.65	*144,25*
Costs [%]			31.9	16.4	47.8	1.3			27.2	17.7	52.0	1.2			21.2	15.0	61.2	1.2	
30	6,0	6.44	10.43	0.82	0.32	0.65	*142,34*	5.15	7.40	0.82	0.32	0.65	*153,30*	4.29	4.33	0.65	0.32	0.65	*157,16*
Costs [%]			27.2	16.4	53.1	1.1			22.8	17.2	57.3	1.0			17.4	14.3	66.1	1.0	

Construction costs for Vertically composite caisson-type seawall H_s= 4.0 m; s = 0.05

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m^2]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	7.30	98.0	0,7	20.45	9.60	69.88	1.34	1.20	77.531	837,12
6,0	3,0	5,4	3,8	9.2	12.50	99.0	2,1	47.91	9.72	41.72	1.20	1.61	145.476	252,19
10,0	4,0	7,2	5,0	13.0	13.90	100.0	3,5	53.87	13.22	31.99	1.21	1.35	194.158	165,56
16,0	4,0	7,2	5,0	16.9	13.70	100.0	5,6	53.58	19.17	26.27	1.49	1.21	230.618	156,36
20,0	4,0	7,2	5,0	19.5	14.10	99.0	7,0	53.71	22.56	22.70	1.67	1.21	251.696	153,53
26,0	4,0	7,2	5,0	23.4	14.70	100.0	9,1	53.32	27.06	18.52	1.91	1.20	289.471	149,00
30,0	4,0	7,2	5,0	26.0	15.20	100.0	10,5	52.97	29.64	16.24	2.04	1.21	312.701	146,66

Construction costs for Vertically composite caisson-type seawall H_s= 6.0 m; s = 0.05

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m^2]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	7.30	98.0	0,7	20.45	9.60	69.88	1.34	1.20	77.531	837,12
6,0	3,0	5,4	3,8	9.2	12.50	99.0	2,1	47.91	9.72	41.72	1.20	1.61	145.476	252,19
10,0	5,0	9	6,3	14.2	18.10	100.0	3,5	61.52	11.67	25.56	1.21	1.81	215.772	147,91
16,0	6,0	10,8	7,5	19.4	18.20	100.0	5,6	61.57	17.05	20.01	1.20	1.33	275.271	114,91
20,0	6,0	10,8	7,5	22.0	17.70	100.0	7,0	59.96	20.57	18.10	1.30	1.20	299.211	114,67
26,0	6,0	10,8	7,5	25.9	18.20	100.0	9,1	58.70	24.87	15.03	1.50	1.21	334.174	115,50
30,0	6,0	10,8	7,5	28.5	18.60	100.0	10,5	57.84	27.38	13.35	1.62	1.21	357.365	116,15

Construction costs for Rubble mound seawall armoured with modified cubes										Hs = 8.0 m; s = 0.05									
				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.08	0.53	0.00	0.32	0.32	100	2.18	0.53	0.00	0.32	0.32	117,62	1.57	0.53	0.00	0.32	0.32	126,46
Costs [%]			60.5	0.0	12.3	7.7			63.1	0.0	13.7	6.5			67.0	0.0	11.5	6.1	
6	3,0	6.24	2.14	0.51	0.32	0.51	100	6.54	2.14	0.51	0.32	0.51	122,37	4.72	1.07	0.51	0.32	0.51	116,546
Costs [%]			55.0	10.7	19.2	6.3			55.5	10.6	21.5	5.1			53.8	12.9	23.2	5.4	
10	5,0	10.41	10.68	0.82	0.32	0.65	100	10.90	10.68	0.82	0.32	0.65	123,84	7.87	5.34	0.65	0.32	0.65	114,27
Costs [%]			57.4	11.7	19.2	2.4			57.3	11.5	21.2	2.5			57.4	10.6	24.8	2.1	
16	8,0	16.65	37.38	1.18	0.32	1.03	100	17.44	32.04	1.18	0.32	1.03	119,46	12.59	19.60	1.03	0.32	1.03	115,94
Costs [%]			54.1	14.7	19.8	3.1			53.2	15.1	22.9	2.6			54.4	13.6	24.6	2.6	
20	8,0	16.65	37.38	1.18	0.32	1.03	100	17.44	32.04	1.18	0.32	1.03	119,77	12.59	19.60	1.03	0.32	1.03	119,25
Costs [%]			50.2	15.5	22.7	4.3			49.3	15.6	26.0	3.6			49.7	14.0	28.6	3.6	
26	8,0	16.65	37.38	1.18	0.32	0.88	100	17.44	32.04	1.18	0.32	0.88	120,66	12.59	19.60	1.03	0.32	0.88	123,05
Costs [%]			44.2	18.7	28.2	2.5			43.0	18.2	31.9	2.1			42.4	16.0	36.2	2.0	
30	8,0	16.65	37.38	1.18	0.32	0.82	100	17.44	32.04	1.18	0.32	0.82	121,25	12.59	19.60	1.03	0.32	0.82	125,69
Costs [%]			40.6	20.2	31.7	1.5			39.4	19.4	35.7	1.2			38.2	16.8	40.8	1.2	
				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.18	0.53	0.00	0.32	0.32	134,54	0.94	0.53	0.00	0.32	0.32	144,36	0.79	0.53	0.00	0.32	0.32	157,31
Costs [%]			70.1	0.0	9.7	5.7			72.7	0.0	8.4	5.3			73.8	0.0	7.5	6.4	
6	3,0	3.54	1.07	0.51	0.32	0.51	125,56	2.83	1.07	0.51	0.32	0.51	136,35	2.36	0.53	0.00	0.32	0.51	119,18
Costs [%]			56.6	13.3	20.6	5.0			58.8	13.6	18.9	4.6			59.2	0.0	32.6	5.2	
10	5,0	5.90	5.34	0.65	0.32	0.65	123,85	4.72	5.34	0.65	0.32	0.65	135,88	3.94	5.34	0.65	0.32	0.65	148,06
Costs [%]			60.3	10.9	22.1	1.9			62.2	11.0	20.2	2.3			63.9	11.2	18.9	2.1	
16	8,0	9.45	16.02	0.88	0.32	1.03	117,88	7.56	12.00	0.82	0.32	1.03	119,8	6.30	10.68	0.82	0.32	1.03	127,75
Costs [%]			56.9	11.7	24.7	2.6			57.6	10.9	25.7	2.6			58.2	11.4	25.2	2.4	
20	8,0	9.45	16.02	0.88	0.32	1.03	123,42	7.56	12.00	0.82	0.32	1.03	127,41	6.30	10.68	0.82	0.32	1.03	137,24
Costs [%]			51.7	12.1	29.4	3.5			51.8	11.1	31.0	3.4			52.1	11.6	30.8	3.1	
26	8,0	9.45	16.02	0.88	0.32	0.88	129,42	7.56	12.00	0.82	0.32	0.88	135,77	6.30	10.68	0.82	0.32	0.88	147,87
Costs [%]			43.3	13.8	38.2	1.9			42.7	12.5	40.7	1.8			42.5	12.9	41.0	1.7	
30	8,0	9.45	16.02	0.88	0.32	0.82	133,69	7.56	12.00	0.82	0.32	0.82	141,77	6.30	10.68	0.82	0.32	0.82	155,49
Costs [%]			38.6	14.4	43.4	1.1			37.7	13.0	46.4	1.0			37.2	13.3	47.0	0.9	

Construction costs for Rubble mound seawall armoured with dolosse — Hs = 8.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.70	0.53	0.00	0.32	0.32	*75,77*	1.78	0.53	0.00	0.32	0.32	*88,86*	1.29	0.53	0.00	0.32	0.32	*96,73*
Costs [%]			57.4	0.0	13.6	10.2			60.0	0.0	15.3	8.7			63.9	0.0	13.4	8.0	
6	3,0	5.11	1.07	0.51	0.32	0.51	67,76	5.35	1.07	0.51	0.32	0.51	83,52	3.86	0.53	0.00	0.32	0.51	75,36
Costs [%]			46.4	14.3	24.0	9.2			46.7	14.3	26.7	7.5			47.9	0.0	40.4	8.3	
10	5,0	8.52	5.34	0.65	0.32	0.65	*64,30*	8.92	5.34	0.65	0.32	0.65	*80,23*	6.44	2.14	0.51	0.32	0.65	*75,52*
Costs [%]			50.9	12.2	26.5	3.7			50.6	12.0	29.0	3.0			47.1	11.4	35.3	3.2	
16	8,0	13.62	16.02	0.88	0.32	1.03	*61,39*	14.27	12.00	0.82	0.32	1.03	71,84	10.30	10.68	0.82	0.32	1.03	*79,61*
Costs [%]			47.7	12.8	28.8	5.0			45.9	11.4	34.5	4.3			47.7	12.1	33.0	3.9	
20	8,0	13.62	16.02	0.88	0.32	1.03	*63,65*	14.27	12.00	0.82	0.32	1.03	74,80	10.30	10.68	0.82	0.32	1.03	*84,65*
Costs [%]			43.0	13.2	32.3	6.7			41.2	11.5	38.2	5.7			42.5	12.1	37.5	5.1	
26	8,0	13.62	16.02	0.88	0.32	0.88	*65,35*	14.27	12.00	0.82	0.32	0.88	77,77	10.30	10.68	0.82	0.32	0.88	*90,26*
Costs [%]			36.8	15.5	39.7	3.8			34.9	13.0	46.1	3.2			35.0	13.4	46.5	2.8	
30	8,0	13.62	16.02	0.88	0.32	0.82	*66,17*	14.27	12.00	0.82	0.32	0.82	*79,43*	10.30	10.68	0.82	0.32	0.82	*93,76*
Costs [%]			33.5	16.6	44.5	1.7			31.4	13.7	51.0	1.4			31.0	13.9	51.9	1.2	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	0.97	0.53	0.00	0.32	0.32	*104,04*	0.77	0.53	0.00	0.32	0.32	*112,53*	0.64	0.53	0.00	0.32	0.32	*121,66*
Costs [%]			67.2	0.0	11.7	7.4			69.9	0.0	10.6	6.8			72.1	0.0	9.8	6.3	
6	3,0	2.90	0.53	0.00	0.32	0.51	*82,34*	2.32	0.53	0.00	0.32	0.51	*90,27*	1.93	0.53	0.00	0.32	0.51	*98,66*
Costs [%]			50.7	0.0	38.6	7.6			52.9	0.0	37.4	6.9			54.6	0.0	36.5	6.3	
10	5,0	4.83	2.14	0.51	0.32	0.65	*83,08*	3.86	2.14	0.51	0.32	0.65	*91,66*	3.22	1.07	0.51	0.32	0.65	*90,93*
Costs [%]			49.6	11.8	32.9	2.9			51.4	12.1	31.3	2.6			46.4	14.1	35.3	2.6	
16	8,0	7.73	10.68	0.82	0.32	1.03	*87,58*	6.18	5.34	0.65	0.32	1.03	*84,82*	5.15	5.34	0.65	0.32	1.03	*93,13*
Costs [%]			50.2	12.6	30.7	3.5			47.0	11.1	36.3	3.6			48.4	11.4	35.2	3.3	
20	8,0	7.73	10.68	0.82	0.32	1.03	*94,49*	6.18	5.34	0.65	0.32	1.03	*93,85*	5.15	5.34	0.65	0.32	1.03	*103,84*
Costs [%]			44.5	12.6	35.9	4.5			40.9	10.9	42.0	4.6			42.0	11.2	41.3	4.1	
26	8,0	7.73	10.68	0.82	0.32	0.88	*102,41*	6.18	5.34	0.65	0.32	0.88	*104,90*	5.15	5.34	0.65	0.32	0.88	*117,17*
Costs [%]			36.1	13.8	45.7	2.4			32.2	11.6	52.6	2.4			32.7	11.8	52.3	2.1	
30	8,0	7.73	10.68	0.82	0.32	0.82	*107,56*	6.18	5.34	0.65	0.32	0.82	*112,20*	5.15	5.34	0.65	0.32	0.82	*126,11*
Costs [%]			31.6	14.2	51.4	1.0			27.7	11.7	58.6	1.0			28.0	11.8	58.4	0.9	

Construction costs for Rubble mound seawall armoured with tetrapods — Hs = 8.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.89	0.53	0.00	0.32	0.32	*94,64*	1.98	0.53	0.00	0.32	0.32	*111,28*	1.43	0.53	0.00	0.32	0.32	*120,82*
Costs [%]			61.6	0.0	11.8	8.1			64.2	0.0	13.2	6.9			68.0	0.0	11.2	6.4	
6	3,0	5.68	2.14	0.51	0.32	0.51	94,94	5.95	2.14	0.51	0.32	0.51	115,957	4.29	1.07	0.51	0.32	0.51	111,47
Costs [%]			56.1	10.7	18.3	6.6			56.7	10.7	20.4	5.4			54.8	12.9	22.2	5.6	
10	5,0	9.46	10.68	0.82	0.32	0.65	*94,84*	9.91	10.68	0.82	0.32	0.65	*116,6*	7.16	5.34	0.65	0.32	0.65	*109,23*
Costs [%]			58.7	11.7	18.3	2.5			58.9	11.6	20.3	2.1			58.5	10.7	23.8	2.2	
16	8,0													11.45	19.60	1.03	0.32	1.03	*110,85*
Costs [%]															55.5	13.7	23.7	2.8	
20	8,0													11.45	19.60	1.03	0.32	1.03	*114,49*
Costs [%]															50.7	14.1	27.7	3.8	
26	8,0													11.45	19.60	1.03	0.32	0.88	*118,29*
Costs [%]															43.1	16.1	35.5	2.1	
30	8,0													11.45	19.60	1.03	0.32	0.82	*120,96*
Costs [%]															38.8	16.9	40.2	1.2	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.53	0.00	0.32	0.32	*129,6*	0.86	0.53	0.00	0.32	0.32	*139,90*	0.72	0.53	0.00	0.32	0.32	*151,04*
Costs [%]			71.1	0.0	9.5	5.9			73.6	0.0	8.4	5.5			75.7	0.0	7.7	5.1	
6	3,0	3.22	1.07	0.51	0.32	0.51	*121,29*	2.58	1.07	0.51	0.32	0.51	*132,67*	2.15	0.53	0.00	0.32	0.51	*116,42*
Costs [%]			57.5	13.3	19.9	5.2			59.6	13.6	18.3	4.7			60.0	0.0	32.0	5.4	
10	5,0	5.37	5.34	0.65	0.32	0.65	*119,62*	4.29	5.34	0.65	0.32	0.65	*131,55*	3.58	2.14	0.51	0.32	0.65	*118,36*
Costs [%]			61.3	10.9	21.4	2.0			63.4	11.1	19.7	1.8			58.3	10.7	26.6	2.0	
16	8,0	8.59	16.02	0.88	0.32	1.03	*113,79*	6.87	10.68	0.82	0.32	1.03	*113,96*	5.72	10.68	0.82	0.32	1.03	*124,85*
Costs [%]			57.9	11.7	23.9	2.7			57.4	11.2	25.8	2.7			58.9	11.4	24.6	2.5	
20	8,0	8.59	16.02	0.88	0.32	1.03	*119,58*	6.87	10.68	0.82	0.32	1.03	*121,90*	5.72	10.68	0.82	0.32	1.03	*134,51*
Costs [%]			52.5	12.1	28.7	3.6			51.5	11.4	31.2	3.5			52.8	11.6	30.3	3.2	
26	8,0	8.59	16.02	0.88	0.32	0.88	*125,51*	6.87	10.68	0.82	0.32	0.88	*130,47*	5.72	10.68	0.82	0.32	0.88	*145,00*
Costs [%]			44.0	13.8	37.6	2.0			42.3	12.7	41.1	1.9			43.1	12.9	40.6	1.7	
30	8,0	8.59	16.02	0.88	0.32	0.82	*129,77*	6.87	10.68	0.82	0.32	0.82	*136,27*	5.72	10.68	0.82	0.32	0.82	*152,57*
Costs [%]			39.1	14.4	43.0	1.1			37.3	13.2	47.0	0.8			37.6	13.3	46.6	1.0	

Construction costs for Rubble mound seawall armoured with quarry stones — Hs = 8.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.93	0.36	0.00	0.32	0.32	*50,56*	1.69	0.36	0.00	0.32	0.32	*53,76*	1.43	0.08	0.00	0.32	0.32	*45,26*
Costs [%]			36.5	0.0	24.2	15.2			38.5	0.0	24.6	14.3			32.0	0.0	40.4	17.0	
6	3,0	5.80	7.40	0.82	0.32	0.51	114,341	5.08	4.33	0.65	0.32	0.51	95,38	4.29	2.90	0.51	0.32	0.51	94,73
Costs [%]			51.4	16.2	12.3	5.5			47.1	15.8	18.2	6.6			45.4	14.7	23.9	6.6	
10	5,0													7.16	10.43	0.82	0.32	0.65	112,75
Costs [%]															56.0	13.7	20.8	2.8	
16	8,0																		
Costs [%]																			
20	8,0																		
Costs [%]																			
26	8,0																		
Costs [%]																			
30	8,0																		
Costs [%]																			

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.08	0.00	0.32	0.32	*47,01*	0.86	0.08	0.00	0.32	0.32	*49,44*	0.72	0.08	0.00	0.32	0.32	*52,298*
Costs [%]			34.8	0.0	38.7	16.4			37.1	0.0	37.7	15.6			39.1	0.0	37.0	14.7	
6	3,0	3.22	1.44	0.51	0.32	0.51	*83,38*	2.58	0.72	0.00	0.32	0.51	*70,95*	2.15	0.72	0.00	0.32	0.51	*76,115*
Costs [%]			38.2	19.2	28.3	7.5			37.4	0.0	48.8	8.8			39.1	0.0	48.0	8.2	
10	5,0	5.37	7.40	0.82	0.32	0.65	*106,45*	4.29	4.33	0.65	0.32	0.65	*93,27*	3.58	2.90	0.51	0.32	0.65	*87,44*
Costs [%]			53.5	16.5	22.1	2.3			48.0	15.9	28.9	2.6			43.8	14.4	35.3	2.7	
16	8,0													5.72	13.90	0.82	0.32	1.03	*124,55*
Costs [%]															59.0	11.3	24.3	2.5	
20	8,0													5.72	13.90	0.82	0.32	1.03	*134,16*
Costs [%]															52.9	11.6	30.0	3.2	
26	8,0													5.72	13.90	0.82	0.32	0.88	*144,63*
Costs [%]															43.1	12.9	40.3	1.7	
30	8,0													5.72	13.90	0.82	0.32	0.82	*152,19*
Costs [%]															37.7	13.3	46.3	1.0	

Construction costs for Rubble mound seawall armoured with modified cubes — Hs = 10.0 m; s = 0.05

	H_s	1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.08	0.53	0.00	0.32	0.32	100	2.18	0.53	0.00	0.32	0.32	*117,62*	1.57	0.53	0.00	0.32	0.32	*126,46*
Costs [%]			60.5	0.0	12.3	7.7			63.1	0.0	13.7	6.5			67.0	0.0	11.5	6.1	
6	3,0	6.24	2.14	0.51	0.32	0.51	100	6.54	2.14	0.51	0.32	0.51	122,37	4.72	1.07	0.51	0.32	0.51	116,546
Costs [%]			55.0	10.7	19.2	6.3			55.5	10.6	21.5	5.1			53.8	12.9	23.2	5.4	
10	5,0	10.41	10.68	0.82	0.32	0.65	100	10.90	10.68	0.82	0.32	0.65	*123,84*	7.87	5.34	0.65	0.32	0.65	*114,27*
Costs [%]			57.4	11.7	19.2	2.4			57.3	11.5	21.2	2.5			57.4	10.6	24.8	2.1	
16	8,0	16.65	37.38	1.18	0.32	1.03	100	17.44	32.04	1.18	0.32	1.03	119,46	12.59	19.60	1.03	0.32	1.03	*115,94*
Costs [%]			54.1	14.7	19.8	3.1			53.2	15.1	22.9	2.6			54.4	13.6	24.6	2.6	
20	10,0													15.74	37.38	1.18	0.32	1.41	*100*
Costs [%]															52.6	14.0	24.6	4.3	
26	10,0													15.74	37.38	1.18	0.32	1.18	*100*
Costs [%]															48.5	15.3	30.2	2.1	
30	10,0													15.74	37.38	1.18	0.32	1.18	*100*
Costs [%]															43.7	16.1	33.7	3.1	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.18	0.53	0.00	0.32	0.32	*134,54*	0.94	0.53	0.00	0.32	0.32	*144,36*	0.79	0.53	0.00	0.32	0.32	*157,31*
Costs [%]			70.1	0.0	9.7	5.7			72.7	0.0	8.4	5.3			73.8	0.0	7.5	6.4	
6	3,0	3.54	1.07	0.51	0.32	0.51	*125,56*	2.83	1.07	0.51	0.32	0.51	*136,35*	2.36	0.53	0.00	0.32	0.51	*119,18*
Costs [%]			56.6	13.3	20.6	5.0			58.8	13.6	18.9	4.6			59.2	0.0	32.6	5.2	
10	5,0	5.90	5.34	0.65	0.32	0.65	*123,85*	4.72	5.34	0.65	0.32	0.65	*135,88*	3.94	5.34	0.65	0.32	0.65	*148,06*
Costs [%]			60.3	10.9	22.1	1.9			62.2	11.0	20.2	2.3			63.9	11.2	18.9	2.1	
16	8,0	9.45	16.02	0.88	0.32	1.03	*117,88*	7.56	12.00	0.82	0.32	1.03	*119,8*	6.30	10.68	0.82	0.32	1.03	*127,75*
Costs [%]			56.9	11.7	24.7	2.6			57.6	10.9	25.7	2.6			58.2	11.4	25.2	2.4	
20	10,0	11.81	32.04	1.18	0.32	1.41	*105,26*	9.45	26.70	1.03	0.32	1.41	*107,51*	7.87	19.60	1.03	0.32	1.41	*110,82*
Costs [%]			54.0	14.9	23.1	4.1			56.2	12.5	23.9	4.0			55.1	13.6	24.8	3.9	
26	10,0	11.81	32.04	1.18	0.32	1.18	*107,57*	9.45	26.70	1.03	0.32	1.18	*111,64*	7.87	19.60	1.03	0.32	1.18	*117,1*
Costs [%]			49.3	16.2	29.3	2.0			50.9	13.5	30.9	1.9			49.3	14.5	32.2	1.8	
30	10,0	11.81	32.04	1.18	0.32	1.18	*108,39*	9.45	26.70	1.03	0.32	1.18	*113,11*	7.87	19.60	1.03	0.32	1.18	*119,61*
Costs [%]			44.1	17.0	33.3	2.8			45.3	14.2	35.4	2.7			43.5	15.1	37.0	2.6	

Construction costs for Rubble mound seawall armoured with dolosse — **Hs = 10.0 m; s = 0.05**

		1 : 1.5						1 : 2						1 : 3					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	1.70	0.53	0.00	0.32	0.32	*75,77*	1.78	0.53	0.00	0.32	0.32	*88,86*	1.29	0.53	0.00	0.32	0.32	*96,73*
Costs [%]			57.4	0.0	13.6	10.2			60.0	0.0	15.3	8.7			63.9	0.0	13.4	8.0	
6	3,0	5.11	1.07	0.51	0.32	0.51	67,76	5.35	1.07	0.51	0.32	0.51	83,52	3.86	0.53	0.00	0.32	0.51	75,36
Costs [%]			46.4	14.3	24.0	9.2			46.7	14.3	26.7	7.5			47.9	0.0	40.4	8.3	
10	5,0	8.52	5.34	0.65	0.32	0.65	*64,30*	8.92	5.34	0.65	0.32	0.65	*80,23*	6.44	2.14	0.51	0.32	0.65	*75,52*
Costs [%]			50.9	12.2	26.5	3.7			50.6	12.0	29.0	3.0			47.1	11.4	35.3	3.2	
16	8,0	13.62	16.02	0.88	0.32	1.03	*61,39*	14.27	12.00	0.82	0.32	1.03	71,84	10.30	10.68	0.82	0.32	1.03	*79,61*
Costs [%]			47.7	12.8	28.8	5.0			45.9	11.4	34.5	4.3			47.7	12.1	33.0	3.9	
20	10,0													12.88	16.02	0.88	0.32	1.41	*66,21*
Costs [%]															44.0	11.8	34.8	6.5	
26	10,0													12.88	16.02	0.88	0.32	1.18	*68,05*
Costs [%]															39.8	12.6	42.1	3.1	
30	10,0													12.88	16.02	0.88	0.32	1.18	*70,02*
Costs [%]															34.9	12.9	45.8	4.4	

		1 : 4						1 : 5						1 : 6					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	0.97	0.53	0.00	0.32	0.32	*104,04*	0.77	0.53	0.00	0.32	0.32	*112,53*	0.64	0.53	0.00	0.32	0.32	*121,66*
Costs [%]			67.2	0.0	11.7	7.4			69.9	0.0	10.6	6.8			72.1	0.0	9.8	6.3	
6	3,0	2.90	0.53	0.00	0.32	0.51	*82,34*	2.32	0.53	0.00	0.32	0.51	*90,27*	1.93	0.53	0.00	0.32	0.51	*98,66*
Costs [%]			50.7	0.0	38.6	7.6			52.9	0.0	37.4	6.9			54.6	0.0	36.5	6.3	
10	5,0	4.83	2.14	0.51	0.32	0.65	*83,08*	3.86	2.14	0.51	0.32	0.65	*91,66*	3.22	1.07	0.51	0.32	0.65	*90,93*
Costs [%]			49.6	11.8	32.9	2.9			51.4	12.1	31.3	2.6			46.4	14.1	35.3	2.6	
16	8,0	7.73	10.68	0.82	0.32	1.03	*87,58*	6.18	5.34	0.65	0.32	1.03	*84,82*	5.15	5.34	0.65	0.32	1.03	*93,13*
Costs [%]			50.2	12.6	30.7	3.5			47.0	11.1	36.3	3.6			48.4	11.4	35.2	3.3	
20	10,0	9.66	12.00	0.82	0.32	1.41	*68,46*	7.73	10.68	0.82	0.32	1.41	*74,03*	6.44	10.68	0.82	0.32	1.41	*81,09*
Costs [%]			44.7	11.0	35.7	6.3			45.5	11.6	35.1	5.8			46.9	11.9	34.1	5.3	
26	10,0	9.66	12.00	0.82	0.32	1.18	*72,25*	7.73	10.68	0.82	0.32	1.18	*79,63*	6.44	10.68	0.82	0.32	1.18	*88,37*
Costs [%]			39.8	11.6	43.8	2.9			40.1	12.1	43.6	2.7			41.0	12.3	42.9	2.4	
30	10,0	9.66	12.00	0.82	0.32	1.18	*75,17*	7.73	10.68	0.82	0.32	1.18	*83,36*	6.44	10.68	0.82	0.32	1.18	*92,77*
Costs [%]			34.5	11.8	48.1	4.1			34.5	12.2	48.3	3.7			35.2	12.4	47.9	3.3	

Construction costs for Rubble mound seawall armoured with tetrapods — **Hs = 10.0 m; s = 0.05**

				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.89	0.53	0.00	0.32	0.32	*94,64*	1.98	0.53	0.00	0.32	0.32	*111,28*	1.43	0.53	0.00	0.32	0.32	*120,82*
Costs [%]			61.6	0.0	11.8	8.1			64.2	0.0	13.2	6.9			68.0	0.0	11.2	6.4	
6	3,0	5.68	2.14	0.51	0.32	0.51	94,94	5.95	2.14	0.51	0.32	0.51	115,957	4.29	1.07	0.51	0.32	0.51	111,47
Costs [%]			56.1	10.7	18.3	6.6			56.7	10.7	20.4	5.4			54.8	12.9	22.2	5.6	
10	5,0	9.46	10.68	0.82	0.32	0.65	*94,84*	9.91	10.68	0.82	0.32	0.65	*116,6*	7.16	5.34	0.65	0.32	0.65	*109,23*
Costs [%]			58.7	11.7	18.3	2.5			58.9	11.6	20.3	2.1			58.5	10.7	23.8	2.2	
16	8,0													11.45	19.60	1.03	0.32	1.03	*110,85*
Costs [%]															55.5	13.7	23.7	2.8	
20	10,0																		
Costs [%]																			
26	10,0																		
Costs [%]																			
30	10,0																		
Costs [%]																			

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.53	0.00	0.32	0.32	*129,60*	0.86	0.53	0.00	0.32	0.32	*139,90*	0.72	0.53	0.00	0.32	0.32	*151,04*
Costs [%]			71.1	0.0	9.5	5.9			73.6	0.0	8.4	5.5			75.7	0.0	7.7	5.1	
6	3,0	3.22	1.07	0.51	0.32	0.51	*121,29*	2.58	1.07	0.51	0.32	0.51	*132,67*	2.15	0.53	0.00	0.32	0.51	*116,42*
Costs [%]			57.5	13.3	19.9	5.2			59.6	13.6	18.3	4.7			60.0	0.0	32.0	5.4	
10	5,0	5.37	5.34	0.65	0.32	0.65	*119,62*	4.29	5.34	0.65	0.32	0.65	*131,55*	3.58	2.14	0.51	0.32	0.65	*118,36*
Costs [%]			61.3	10.9	21.4	2.0			63.4	11.1	19.7	1.8			58.3	10.7	26.6	2.0	
16	8,0	8.59	16.02	0.88	0.32	1.03	*113,79*	6.87	10.68	0.82	0.32	1.03	*113,96*	5.72	10.68	0.82	0.32	1.03	*124,85*
Costs [%]			57.9	11.7	23.9	2.7			57.4	11.2	25.8	2.7			58.9	11.4	24.6	2.5	
20	10,0													7.16	19.60	1.03	0.32	1.41	*108,29*
Costs [%]															55.8	13.6	24.2	4.0	
26	10,0													7.16	19.60	1.03	0.32	1.18	*114,73*
Costs [%]															50.0	14.5	31.7	1.9	
30	10,0													7.16	19.60	1.03	0.32	1.18	*117,26*
Costs [%]															44.0	15.1	36.5	2.6	

Construction costs for Rubble mound seawall armoured with quarry stones — Hs = 10.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.93	0.36	0.00	0.32	0.32	*50,56*	1.69	0.36	0.00	0.32	0.32	*53,76*	1.43	0.08	0.00	0.32	0.32	*45,26*
Costs [%]			36.5	0.0	24.2	15.2			38.5	0.0	24.6	14.3			32.0	0.0	40.4	17.0	
6	3,0	5.80	7.40	0.82	0.32	0.51	114,341	5.08	4.33	0.65	0.32	0.51	95,38	4.29	2.90	0.51	0.32	0.51	94,73
Costs [%]			51.4	16.2	12.3	5.5			47.1	15.8	18.2	6.6			45.4	14.7	23.9	6.6	
10	5,0													7.16	10.43	0.82	0.32	0.65	112,75
Costs [%]															56.0	13.7	20.8	2.8	
16	8,0																		
Costs [%]																			
20	10,0																		
Costs [%]																			
26	10,0																		
Costs [%]																			
30	10,0																		
Costs [%]																			

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.08	0.00	0.32	0.32	*47,01*	0.86	0.08	0.00	0.32	0.32	*49,44*	0.72	0.08	0.00	0.32	0.32	*52,30*
Costs [%]			34.8	0.0	38.7	16.4			37.1	0.0	37.7	15.6			39.1	0.0	37.0	14.7	
6	3,0	3.22	1.44	0.51	0.32	0.51	*83,38*	2.58	0.72	0.00	0.32	0.51	*70,95*	2.15	0.72	0.00	0.32	0.51	*76,11*
Costs [%]			38.2	19.2	28.3	7.5			37.4	0.0	48.8	8.8			39.1	0.0	48.0	8.2	
10	5,0	5.37	7.40	0.82	0.32	0.65	*106,45*	4.29	4.33	0.65	0.32	0.65	*93,27*	3.58	2.90	0.51	0.32	0.65	*87,44*
Costs [%]			53.5	16.5	22.1	2.3			48.0	15.9	28.9	2.6			43.8	14.4	35.3	2.7	
16	8,0													5.72	13.90	0.82	0.32	1.03	*124,55*
Costs [%]															59.0	11.3	24.3	2.5	
20	10,0																		
Costs [%]																			
26	10,0																		
Costs [%]																			
30	10,0																		
Costs [%]																			

Construction costs for Vertically composite caisson-type seawall **Hs = 8.0 m; s = 0.05**

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m²]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	7.30	98.0	0,7	20.45	9.60	69.88	1.34	1.20	77.531	837,12
6,0	3,0	5,4	3,8	9.2	12.50	99.0	2,1	47.91	9.72	41.72	1.20	1.61	145.476	252,19
10,0	5,0	9,0	6,3	14.2	18.10	100.0	3,5	61.52	11.67	25.56	1.21	1.81	215.772	147,91
16,0	8,0	14,4	10,0	21.9	26.40	100.0	5,6	68.98	15.34	13.87	1.20	1.93	320.010	106,28
20,0	8,0	14,4	10,0	24.5	23.70	100.0	7,0	65.65	19.18	13.45	1.21	1.50	344.071	95,75
26,0	8,0	14,4	10,0	28.4	21.90	100.0	9,1	62.06	24.12	12.18	1.25	1.20	379.828	92,96
30,0	8,0	14,4	10,0	31.0	22.20	100.0	10,5	60.84	26.59	10.94	1.36	1.21	402.939	95,27

Construction costs for Vertically composite caisson-type seawall **Hs = 10.0 m; s = 0.05**

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m²]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	7.30	98.0	0,7	20.45	9.60	69.88	1.34	1.20	77.531	837,12
6,0	3,0	5,4	3,8	9.2	12.50	99.0	2,1	47.91	9.72	41.72	1.20	1.61	145.476	252,19
10,0	5,0	9,0	6,3	14.2	18.10	100.0	3,5	61.52	11.67	25.56	1.21	1.81	215.772	147,91
16,0	8,0	14,4	10,0	21.9	26.40	100.0	5,6	68.98	15.34	13.87	1.20	1.93	320.010	106,28
20,0	10,0	18,0	12,5	27.0	32.00	100.0	7,0	70.31	17.84	9.82	1.20	1.97	389.549	81,36
26,0	10,0	18,0	12,5	30.9	28.00	100.0	9,1	65.74	22.98	9.38	1.20	1.46	425.137	72,45
30,0	10,0	18,0	12,5	33.5	26.30	100.0	10,5	63.15	26.07	8.96	1.20	1.24	448.870	68,32

Construction costs for Rubble mound seawall armoured with modified cubes — Hs = 12.0 m; s = 0.05

		1 : 1.5						1 : 2						1 : 3								
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.08	0.53	0.00	0.32	0.32	100	2.18	0.53	0.00	0.32	0.32	117,62	1.57	0.53	0.00	0.32	0.32	126,46
Costs [%]			60.5	0.0	12.3	7.7			63.1	0.0	13.7	6.5			67.0	0.0	11.5	6.1	
6	3,0	6.24	2.14	0.51	0.32	0.51	100	6.54	2.14	0.51	0.32	0.51	122,37	4.72	1.07	0.51	0.32	0.51	116,546
Costs [%]			55.0	10.7	19.2	6.3			55.5	10.6	21.5	5.1			53.8	12.9	23.2	5.4	
10	5,0	10.41	10.68	0.82	0.32	0.65	100	10.90	10.68	0.82	0.32	0.65	123,84	7.87	5.34	0.65	0.32	0.65	114,27
Costs [%]			57.4	11.7	19.2	2.4			57.3	11.5	21.2	2.5			57.4	10.6	24.8	2.1	
16	8,0	16.65	37.38	1.18	0.32	1.03	100	17.44	32.04	1.18	0.32	1.03	119,46	12.59	19.60	1.03	0.32	1.03	115,94
Costs [%]			54.1	14.7	19.8	3.1			53.2	15.1	22.9	2.6			54.4	13.6	24.6	2.6	
20	10,0													15.74	37.38	1.18	0.32	1.41	100
Costs [%]															52.6	14.0	24.6	4.3	
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.18	0.53	0.00	0.32	0.32	134,54	0.94	0.53	0.00	0.32	0.32	144,36	0.79	0.53	0.00	0.32	0.32	157,31
Costs [%]			70.1	0.0	9.7	5.7			72.7	0.0	8.4	5.3			73.8	0.0	7.5	6.4	
6	3,0	3.54	1.07	0.51	0.32	0.51	125,56	2.83	1.07	0.51	0.32	0.51	136,35	2.36	0.53	0.00	0.32	0.51	119,18
Costs [%]			56.6	13.3	20.6	5.0			58.8	13.6	18.9	4.6			59.2	0.0	32.6	5.2	
10	5,0	5.90	5.34	0.65	0.32	0.65	123,85	4.72	5.34	0.65	0.32	0.65	135,88	3.94	5.34	0.65	0.32	0.65	148,06
Costs [%]			60.3	10.9	22.1	1.9			62.2	11.0	20.2	2.3			63.9	11.2	18.9	2.1	
16	8,0	9.45	16.02	0.88	0.32	1.03	117,88	7.56	12.00	0.82	0.32	1.03	119,8	6.30	10.68	0.82	0.32	1.03	127,75
Costs [%]			56.9	11.7	24.7	2.6			57.6	10.9	25.7	2.6			58.2	11.4	25.2	2.4	
20	10,0	11.81	32.04	1.18	0.32	1.41	105,26	9.45	26.70	1.03	0.32	1.41	107,51	7.87	19.60	1.03	0.32	1.41	110,82
Costs [%]			54.0	14.9	23.1	4.1			56.2	12.5	23.9	4.0			55.1	13.6	24.8	3.9	
26	12,0							11.33	42.71	1.18	0.32	1.41	100	9.45	32.04	1.18	0.32	1.41	104,36
Costs [%]									54.9	13.7	25.8	2.6			53.8	14.8	26.6	2.5	
30	12,0							11.33	42.71	1.18	0.32	1.41	100	9.45	32.04	1.18	0.32	1.41	105,18
Costs [%]									50.7	14.5	29.8	2.3			49.3	15.5	30.9	2.2	

Construction costs for Rubble mound seawall armoured with dolosse — Hs = 12.0 m; s = 0.05

				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.70	0.53	0.00	0.32	0.32	*75,77*	1.78	0.53	0.00	0.32	0.32	*88,86*	1.29	0.53	0.00	0.32	0.32	*96,73*
Costs [%]			57.4	0.0	13.6	10.2			60.0	0.0	15.3	8.7			63.9	0.0	13.4	8.0	
6	3,0	5.11	1.07	0.51	0.32	0.51	67,76	5.35	1.07	0.51	0.32	0.51	83,52	3.86	0.53	0.00	0.32	0.51	75,36
Costs [%]			46.4	14.3	24.0	9.2			46.7	14.3	26.7	7.5			47.9	0.0	40.4	8.3	
10	5,0	8.52	5.34	0.65	0.32	0.65	*64,30*	8.92	5.34	0.65	0.32	0.65	*80,23*	6.44	2.14	0.51	0.32	0.65	*75,52*
Costs [%]			50.9	12.2	26.5	3.7			50.6	12.0	29.0	3.0			47.1	11.4	35.3	3.2	
16	8,0	13.62	16.02	0.88	0.32	1.03	*61,39*	14.27	12.00	0.82	0.32	1.03	71,84	10.30	10.68	0.82	0.32	1.03	*79,61*
Costs [%]			47.7	12.8	28.8	5.0			45.9	11.4	34.5	4.3			47.7	12.1	33.0	3.9	
20	10,0													12.88	16.02	0.88	0.32	1.41	*66,21*
Costs [%]															44.0	11.8	34.8	6.5	
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	0.97	0.53	0.00	0.32	0.32	*104,04*	0.77	0.53	0.00	0.32	0.32	*112,53*	0.64	0.53	0.00	0.32	0.32	*121,66*
Costs [%]			67.2	0.0	11.7	7.4			69.9	0.0	10.6	6.8			72.1	0.0	9.8	6.3	
6	3,0	2.90	0.53	0.00	0.32	0.51	*82,34*	2.32	0.53	0.00	0.32	0.51	*90,27*	1.93	0.53	0.00	0.32	0.51	*98,66*
Costs [%]			50.7	0.0	38.6	7.6			52.9	0.0	37.4	6.9			54.6	0.0	36.5	6.3	
10	5,0	4.83	2.14	0.51	0.32	0.65	*83,08*	3.86	2.14	0.51	0.32	0.65	*91,66*	3.22	1.07	0.51	0.32	0.65	*90,93*
Costs [%]			49.6	11.8	32.9	2.9			51.4	12.1	31.3	2.6			46.4	14.1	35.3	2.6	
16	8,0	7.73	10.68	0.82	0.32	1.03	*87,58*	6.18	5.34	0.65	0.32	1.03	*84,82*	5.15	5.34	0.65	0.32	1.03	*93,13*
Costs [%]			50.2	12.6	30.7	3.5			47.0	11.1	36.3	3.6			48.4	11.4	35.2	3.3	
20	10,0	9.66	12.00	0.82	0.32	1.41	*68,46*	7.73	10.68	0.82	0.32	1.41	*74,03*	6.44	10.68	0.82	0.32	1.41	*81,09*
Costs [%]			44.7	11.0	35.7	6.3			45.5	11.6	35.1	5.8			46.9	11.9	34.1	5.3	
26	12,0	11.59	19.60	1.03	0.32	1.41	*64,69*	9.27	16.02	0.88	0.32	1.41	*68,11*	7.73	16.02	0.88	0.32	1.41	*75,12*
Costs [%]			43.3	13.9	36.9	4.0			44.2	11.7	38.6	3.8			45.5	12.0	37.6	3.4	
30	12,0	11.59	19.60	1.03	0.32	1.41	*65,80*	9.27	16.02	0.88	0.32	1.41	*69,87*	7.73	16.02	0.88	0.32	1.41	*77,43*
Costs [%]			39.3	14.3	41.2	3.5			39.8	12.0	43.3	3.3			40.8	12.3	42.6	3.0	

Construction costs for Rubble mound seawall armoured with tetrapods — **Hs = 12.0 m; s = 0.05**

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.89	0.53	0.00	0.32	0.32	*94,64*	1.98	0.53	0.00	0.32	0.32	*111,28*	1.43	0.53	0.00	0.32	0.32	*120,82*
Costs [%]			61.6	0.0	11.8	8.1			64.2	0.0	13.2	6.9			68.0	0.0	11.2	6.4	
6	3,0	5.68	2.14	0.51	0.32	0.51	94,94	5.95	2.14	0.51	0.32	0.51	115,957	4.29	1.07	0.51	0.32	0.51	111,47
Costs [%]			56.1	10.7	18.3	6.6			56.7	10.7	20.4	5.4			54.8	12.9	22.2	5.6	
10	5,0	9.46	10.68	0.82	0.32	0.65	*94,84*	9.91	10.68	0.82	0.32	0.65	*116,6*	7.16	5.34	0.65	0.32	0.65	*109,23*
Costs [%]			58.7	11.7	18.3	2.5			58.9	11.6	20.3	2.1			58.5	10.7	23.8	2.2	
16	8,0													11.45	19.60	1.03	0.32	1.03	*110,85*
Costs [%]															55.5	13.7	23.7	2.8	
20	10,0																		
Costs [%]																			
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.53	0.00	0.32	0.32	*129,60*	0.86	0.53	0.00	0.32	0.32	*139,9*	0.72	0.53	0.00	0.32	0.32	*151,04*
Costs [%]			71.1	0.0	9.5	5.9			73.6	0.0	8.4	5.5			75.7	0.0	7.7	5.1	
6	3,0	3.22	1.07	0.51	0.32	0.51	*121,29*	2.58	1.07	0.51	0.32	0.51	*132,67*	2.15	0.53	0.00	0.32	0.51	*116,42*
Costs [%]			57.5	13.3	19.9	5.2			59.6	13.6	18.3	4.7			60.0	0.0	32.0	5.4	
10	5,0	5.37	5.34	0.65	0.32	0.65	*119,62*	4.29	5.34	0.65	0.32	0.65	*131,55*	3.58	2.14	0.51	0.32	0.65	*118,36*
Costs [%]			61.3	10.9	21.4	2.0			63.4	11.1	19.7	1.8			58.3	10.7	26.6	2.0	
16	8,0	8.59	16.02	0.88	0.32	1.03	*113,79*	6.87	10.68	0.82	0.32	1.03	*113,96*	5.72	10.68	0.82	0.32	1.03	*124,85*
Costs [%]			57.9	11.7	23.9	2.7			57.4	11.2	25.8	2.7			58.9	11.4	24.6	2.5	
20	10,0													7.16	19.60	1.03	0.32	1.41	*108,29*
Costs [%]															55.8	13.6	24.2	4.0	
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

Construction costs for Rubble mound seawall armoured with quarry stones												Hs = 12.0 m; s = 0.05							
				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.93	0.36	0.00	0.32	0.32	50,56	1.69	0.36	0.00	0.32	0.32	53,76	1.43	0.08	0.00	0.32	0.32	45,26
Costs [%]			36.5	0.0	24.2	15.2			38.5	0.0	24.6	14.3			32.0	0.0	40.4	17.0	
6	3,0	5.80	7.40	0.82	0.32	0.51	114,341	5.08	4.33	0.65	0.32	0.51	95,38	4.29	2.90	0.51	0.32	0.51	94,73
Costs [%]			51.4	16.2	12.3	5.5			47.1	15.8	18.2	6.6			45.4	14.7	23.9	6.6	
10	5,0													7.16	10.43	0.82	0.32	0.65	112,75
Costs [%]															56.0	13.7	20.8	2.8	
16	8,0																		
Costs [%]																			
20	10,0																		
Costs [%]																			
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.07	0.08	0.00	0.32	0.32	47,01	0.86	0.08	0.00	0.32	0.32	49,44	0.72	0.08	0.00	0.32	0.32	52,30
Costs [%]			34.8	0.0	38.7	16.4			37.1	0.0	37.7	15.6			39.1	0.0	37.0	14.7	
6	3,0	3.22	1.44	0.51	0.32	0.51	83,38	2.58	0.72	0.00	0.32	0.51	70,95	2.15	0.72	0.00	0.32	0.51	76,11
Costs [%]			38.2	19.2	28.3	7.5			37.4	0.0	48.8	8.8			39.1	0.0	48.0	8.2	
10	5,0	5.37	7.40	0.82	0.32	0.65	106,45	4.29	4.33	0.65	0.32	0.65	93,27	3.58	2.90	0.51	0.32	0.65	87,44
Costs [%]			53.5	16.5	22.1	2.3			48.0	15.9	28.9	2.6			43.8	14.4	35.3	2.7	
16	8,0													5.72	13.90	0.82	0.32	1.03	124,55
Costs [%]															59.0	11.3	24.3	2.5	
20	10,0																		
Costs [%]																			
26	12,0																		
Costs [%]																			
30	12,0																		
Costs [%]																			

Construction costs for Vertically composite caisson-type seawall **Hs = 12.0 m; s = 0.05**

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m^2]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	7.30	98.0	0,7	20.45	9.60	69.88	1.34	1.20	77.531	837,12
6,0	3,0	5,4	3,8	9.2	12.50	99.0	2,1	47.91	9.72	41.72	1.20	1.61	145.476	252,19
10,0	5,0	9,0	6,3	14.2	18.10	100.0	3,5	61.52	11.67	25.56	1.21	1.81	215.772	147,91
16,0	8,0	14,4	10,0	21.9	26.40	100.0	5,6	68.98	15.34	13.87	1.20	1.93	320.010	106,28
20,0	10,0	18,0	12,5	27.0	32.00	100.0	7,0	70.31	17.84	9.82	1.20	1.97	389.549	81,36
26,0	12,0	21,6	15,0	33.4	35.90	100.0	9,1	70.56	19.90	7.35	1.20	1.82	470.867	65,14
30,0	12,0	21,6	15,0	36.0	33.50	100.0	10,5	68.04	22.72	7.13	1.20	1.56	494.318	60,59

Construction costs for Rubble mound seawall armoured with modified cubes — **Hs = 4.0 m; s = 0.01**

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.60	0.53	0.00	0.32	0.32	100	1.82	0.53	0.00	0.32	0.32	*120,33*	2.04	0.53	0.00	0.32	0.32	*162,47*
Costs [%]			59.3	0.0	10.1	8.6			62.6	0.0	12.0	7.2			67.3	0.0	13.9	5.3	
6	3,0	4.79	2.14	0.51	0.32	0.51	100	5.45	2.14	0.51	0.32	0.51	126,21	6.11	1.07	0.51	0.32	0.51	157,441
Costs [%]			55.6	10.7	16.2	7.2			56.3	10.7	19.1	5.7			52.1	12.6	26.6	4.6	
10	4,0	6.38	5.34	0.65	0.32	0.51	100	7.26	5.34	0.65	0.32	0.51	*127,23*	8.14	5.34	0.65	0.32	0.51	*183,11*
Costs [%]			55.1	11.9	20.0	3.6			55.2	11.6	22.9	2.8			55.9	11.4	25.7	2.0	
16	4,0	6.38	5.34	0.65	0.32	0.51	100	7.26	5.34	0.65	0.32	0.51	127,86	8.14	5.34	0.65	0.32	0.51	*184,55*
Costs [%]			42.2	16.3	31.6	2.8			42.1	15.1	35.1	2.2			42.4	13.8	38.3	1.5	
20	4,0	6.38	5.34	0.65	0.32	0.51	100	7.26	5.34	0.65	0.32	0.51	128,27	8.14	5.34	0.65	0.32	0.51	*185,58*
Costs [%]			35.5	17.7	38.4	2.3			35.3	16.1	42.1	1.8			35.5	14.5	45.5	1.2	
26	4,0	6.38	5.34	0.65	0.32	0.51	100	7.26	5.34	0.65	0.32	0.51	128,85	8.14	5.34	0.65	0.32	0.51	*187,07*
Costs [%]			27.8	18.5	47.2	1.8			27.5	16.6	50.9	1.4			27.5	14.6	54.4	1.0	
30	4,0	6.38	5.34	0.65	0.32	0.51	100	7.26	5.34	0.65	0.32	0.51	*129,19*	8.14	5.34	0.65	0.32	0.51	*187,96*
Costs [%]			23.8	18.6	52.0	1.6			23.5	16.5	55.7	1.2			23.5	14.4	59.1	0.8	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.15	0.53	0.00	0.32	0.32	*205,94*	2.11	0.53	0.00	0.32	0.32	*242,69*	1.76	0.53	0.00	0.32	0.32	*254,26*
Costs [%]			70.4	0.0	14.8	4.2			72.6	0.0	14.8	3.6			74.1	0.0	12.9	4.4	
6	3,0	6.44	1.07	0.51	0.32	0.51	*207,13*	6.34	1.07	0.51	0.32	0.51	*248,8*	5.28	0.53	0.00	0.32	0.51	*212,22*
Costs [%]			52.7	12.7	28.1	3.5			53.5	12.8	28.2	2.9			53.4	0.0	41.3	3.4	
10	4,0	8.58	2.14	0.51	0.32	0.51	*201,08*	8.45	2.14	0.51	0.32	0.51	*243,03*	7.04	2.14	0.51	0.32	0.51	*254,71*
Costs [%]			49.5	10.3	35.8	1.8			50.1	10.3	36.0	1.5			52.0	10.7	33.9	1.4	
16	4,0	8.58	2.14	0.51	0.32	0.51	*211,32*	8.45	2.14	0.51	0.32	0.51	*256,27*	7.04	2.14	0.51	0.32	0.51	*272,95*
Costs [%]			36.1	11.5	49.3	1.3			36.4	11.3	49.8	1.1			37.2	11.6	48.7	1.0	
20	4,0	8.58	2.14	0.51	0.32	0.51	*217,21*	8.45	2.14	0.51	0.32	0.51	*264,05*	7.04	2.14	0.51	0.32	0.51	*284,24*
Costs [%]			29.5	11.5	56.4	1.1			29.7	11.2	56.9	0.9			30.0	11.6	56.4	0.8	
26	4,0	8.58	2.14	0.51	0.32	0.51	*224,62*	8.45	2.14	0.51	0.32	0.51	*273,94*	7.04	2.14	0.51	0.32	0.51	*299,07*
Costs [%]			22.3	11.1	64.6	0.8			22.4	10.8	65.2	0.7			22.3	11.0	65.2	0.6	
30	4,0	8.58	2.14	0.51	0.32	0.51	*228,68*	8.45	2.14	0.51	0.32	0.51	*279,43*	7.04	2.14	0.51	0.32	0.51	*307,52*
Costs [%]			18.8	10.7	68.8	0.7			18.8	10.4	69.5	0.6			18.6	10.5	69.7	0.5	

Construction costs for Rubble mound seawall armoured with dolosse — Hs = 4.0 m; s = 0.01

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.31	0.53	0.00	0.32	0.32	*76,62*	1.49	0.53	0.00	0.32	0.32	*91,63*	1.67	0.53	0.00	0.32	0.32	*122,83*
Costs [%]			56.3	0.0	11.5	11.3			59.5	0.0	13.6	9.4			64.3	0.0	15.7	7.0	
6	3,0	3.92	1.07	0.51	0.32	0.51	68,23	4.46	1.07	0.51	0.32	0.51	86,59	5.00	0.53	0.00	0.32	0.51	101,17
Costs [%]			47.2	14.5	20.8	10.5			47.5	14.5	24.3	8.3			46.2	0.0	43.7	7.1	
10	4,0	5.22	2.14	0.51	0.32	0.51	*64,06*	5.94	2.14	0.51	0.32	0.51	*82,83*	6.66	1.07	0.51	0.32	0.51	*109,85*
Costs [%]			46.3	12.9	29.4	5.6			45.7	12.4	33.0	4.3			39.8	13.4	41.4	3.3	
16	4,0	5.22	2.14	0.51	0.32	0.51	*69,69*	5.94	2.14	0.51	0.32	0.51	90,66	6.66	1.07	0.51	0.32	0.51	*124,47*
Costs [%]			32.6	15.9	43.5	4.0			32.0	14.6	47.2	3.0			26.9	14.0	55.5	2.2	
20	4,0	5.22	2.14	0.51	0.32	0.51	*72,93*	5.94	2.14	0.51	0.32	0.51	95,19	6.66	1.07	0.51	0.32	0.51	*132,81*
Costs [%]			26.2	16.4	50.9	3.2			25.6	14.8	54.6	2.4			21.2	13.6	62.3	1.7	
26	4,0	5.22	2.14	0.51	0.32	0.51	*76,99*	5.94	2.14	0.51	0.32	0.51	100,86	6.66	1.07	0.51	0.32	0.51	*143,18*
Costs [%]			19.4	16.2	59.6	2.4			18.9	14.4	63.0	1.8			15.4	12.7	69.9	1.3	
30	4,0	5.22	2.14	0.51	0.32	0.51	*79,22*	5.94	2.14	0.51	0.32	0.51	*103,99*	6.66	1.07	0.51	0.32	0.51	*148,84*
Costs [%]			16.2	15.8	64.1	2.0			15.7	13.9	67.3	1.5			12.7	12.0	73.6	1.0	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.76	0.53	0.00	0.32	0.32	*154,89*	1.73	0.53	0.00	0.32	0.32	*182,33*	1.44	0.53	0.00	0.32	0.32	*190,54*
Costs [%]			67.4	0.0	16.7	5.6			69.7	0.0	16.8	4.7			71.9	0.0	15.2	4.5	
6	3,0	5.27	0.53	0.00	0.32	0.51	*133,64*	5.18	0.53	0.00	0.32	0.51	*160,96*	4.32	0.53	0.00	0.32	0.51	*167,83*
Costs [%]			46.7	0.0	45.8	5.4			47.4	0.0	46.3	4.5			49.6	0.0	44.4	4.3	
10	4,0	7.02	1.07	0.51	0.32	0.51	*145,97*	6.91	1.07	0.51	0.32	0.51	*176,99*	5.76	0.53	0.00	0.32	0.51	*159,74*
Costs [%]			39.7	13.2	43.0	2.5			40.1	13.2	43.3	2.0			38.5	0.2	58.1	2.3	
16	4,0	7.02	1.07	0.51	0.32	0.51	*165,51*	6.91	1.07	0.51	0.32	0.51	*201,59*	5.76	0.53	0.00	0.32	0.51	*192,70*
Costs [%]			26.8	13.4	57.0	1.7			26.9	13.2	57.6	1.4			24.5	0.8	72.7	1.4	
20	4,0	7.02	1.07	0.51	0.32	0.51	*176,65*	6.91	1.07	0.51	0.32	0.51	*215,73*	5.76	0.53	0.00	0.32	0.51	*212,48*
Costs [%]			21.1	12.9	63.8	1.3			21.2	12.7	64.4	1.1			18.7	1.0	78.9	1.1	
26	4,0	7.02	1.07	0.51	0.32	0.51	*190,51*	6.91	1.07	0.51	0.32	0.51	*233,44*	5.76	0.53	0.00	0.32	0.51	*237,98*
Costs [%]			15.3	11.9	71.2	1.0			15.3	11.6	71.8	0.8			13.0	1.0	84.9	0.8	
30	4,0	7.02	1.07	0.51	0.32	0.51	*198,08*	6.91	1.07	0.51	0.32	0.51	*243,17*	5.76	0.53	0.00	0.32	0.51	*252,32*
Costs [%]			12.6	11.2	74.8	0.8			12.6	10.9	75.4	0.6			10.5	1.0	87.6	0.6	

Construction costs for Rubble mound seawall armoured with tetrapods — **Hs = 4.0 m; s = 0.01**

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.45	0.53	0.00	0.32	0.32	*95,07*	1.65	0.53	0.00	0.32	0.32	*114,31*	1.85	0.53	0.00	0.32	0.32	*154,26*
Costs [%]			60.5	0.0	9.9	9.1			63.7	0.0	11.6	7.6			68.3	0.0	13.4	5.6	
6	3,0	4.35	2.14	0.51	0.32	0.51	95,68	4.95	2.14	0.51	0.32	0.51	120,3	5.55	1.07	0.51	0.32	0.51	149,21
Costs [%]			56.7	10.7	15.5	7.5			57.4	10.8	18.2	6.0			53.3	12.7	25.3	4.8	
10	4,0	5.80	5.34	0.65	0.32	0.51	*95,92*	6.60	5.34	0.65	0.32	0.51	*121,65*	7.40	5.34	0.65	0.32	0.51	*174,56*
Costs [%]			56.2	12.0	19.3	3.8			56.4	11.7	22.1	3.0			57.0	11.4	24.7	2.1	
16	4,0	5.80	5.34	0.65	0.32	0.51	*96,16*	6.60	5.34	0.65	0.32	0.51	*122,63*	7.40	5.34	0.65	0.32	0.51	*176,55*
Costs [%]			42.9	16.3	31.2	2.9			42.8	15.2	34.5	2.2			43.1	14.0	37.7	1.6	
20	4,0	5.80	5.34	0.65	0.32	0.51	96,37	6.60	5.34	0.65	0.32	0.51	123,335	7.40	5.34	0.65	0.32	0.51	*178,04*
Costs [%]			36.0	17.8	38.2	2.4			35.8	16.2	41.7	1.9			36.0	14.6	45.0	1.3	
26	4,0	5.80	5.34	0.65	0.32	0.51	*96,70*	6.60	5.34	0.65	0.32	0.51	*124,36*	7.40	5.34	0.65	0.32	0.51	*180,22*
Costs [%]			28.1	18.5	47.1	1.9			27.8	16.6	50.7	1.5			27.8	14.7	54.1	1.0	
30	4,0	5.80	5.34	0.65	0.32	0.51	*96,90*	6.60	5.34	0.65	0.32	0.51	*124,97*	7.40	5.34	0.65	0.32	0.51	*181,55*
Costs [%]			24.0	18.6	52.1	1.6			23.7	16.5	55.7	1.2			23.7	14.5	59.0	0.9	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.95	0.53	0.00	0.32	0.32	*195,54*	1.92	0.53	0.00	0.32	0.32	*230,81*	1.60	0.53	0.00	0.32	0.32	*240,73*
Costs [%]			71.4	0.0	14.2	4.4			73.6	0.0	14.2	3.7			75.7	0.0	12.6	3.6	
6	3,0	5.85	1.07	0.51	0.32	0.51	*195,85*	5.76	1.07	0.51	0.32	0.51	*235,37*	4.80	0.53	0.00	0.32	0.51	*201,48*
Costs [%]			53.9	12.7	26.8	3.7			54.7	12.8	26.9	3.1			54.6	0.0	40.0	3.6	
10	4,0	7.80	2.14	0.51	0.32	0.51	*190,70*	7.68	2.14	0.51	0.32	0.51	*230,65*	6.40	2.14	0.51	0.32	0.51	*243,52*
Costs [%]			50.7	10.4	34.5	1.9			51.3	10.4	34.7	1.6			53.1	10.8	32.8	1.5	
16	4,0	7.80	2.14	0.51	0.32	0.51	*201,42*	7.68	2.14	0.51	0.32	0.51	*244,46*	6.40	2.14	0.51	0.32	0.51	*262,11*
Costs [%]			36.8	11.6	48.5	1.4			37.0	11.4	49.0	1.1			37.8	11.8	48.1	1.1	
20	4,0	7.80	2.14	0.51	0.32	0.51	*207,80*	7.68	2.14	0.51	0.32	0.51	*252,8*	6.40	2.14	0.51	0.32	0.51	*273,85*
Costs [%]			30.0	11.7	55.8	1.1			30.1	11.4	56.4	0.9			30.4	11.7	56.0	0.8	
26	4,0	7.80	2.14	0.51	0.32	0.51	*215,97*	7.68	2.14	0.51	0.32	0.51	*263,61*	6.40	2.14	0.51	0.32	0.51	*289,45*
Costs [%]			22.6	11.3	64.3	0.8			22.6	10.9	64.9	0.7			22.5	11.1	65.0	0.6	
30	4,0	7.80	2.14	0.51	0.32	0.51	*220,54*	7.68	2.14	0.51	0.32	0.51	*269,69*	6.40	2.14	0.51	0.32	0.51	*298,43*
Costs [%]			18.9	10.8	68.6	0.7			18.9	10.5	69.3	0.6			18.7	10.6	69.5	0.5	

Construction costs for Rubble mound seawall armoured with quarry stones Hs = 4.0 m; s = 0.01

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.97	0.36	0.00	0.32	0.32	*57,31*	1.97	0.36	0.00	0.32	0.32	*65,13*	1.97	0.36	0.00	0.32	0.32	*81,314*
Costs [%]			36.6	0.0	24.5	15.1			38.7	0.0	27.0	13.3			42.6	0.0	30.0	10.6	
6	3,0	5.91	7.40	0.82	0.32	0.51	132,747	5.91	7.40	0.82	0.32	0.51	153,87	5.91	7.40	0.82	0.32	0.51	199,323
Costs [%]			51.4	16.2	12.5	5.4			52.9	16.3	13.6	4.7			55.5	16.6	14.6	3.6	
10	4,0							7.88	13.90	0.82	0.32	0.51	*162,18*	7.88	13.90	0.82	0.32	0.51	*216,92*
Costs [%]									58.5	12.6	17.7	2.2			59.9	12.5	18.6	2.2	
16	4,0							7.88	13.90	0.82	0.32	0.51	*157,22*	7.88	13.90	0.82	0.32	0.51	*211,67*
Costs [%]									46.2	16.7	28.2	1.8			47.0	15.9	30.1	1.7	
20	4,0							7.88	13.90	0.82	0.32	0.51	*154,44*	7.88	13.90	0.82	0.32	0.51	*209,11*
Costs [%]									39.5	18.2	34.6	1.5			40.0	17.0	37.0	1.5	
26	4,0							7.88	13.90	0.82	0.32	0.51	*151,03*	7.88	13.90	0.82	0.32	0.51	*206,29*
Costs [%]									31.6	19.2	43.1	1.2			31.7	17.6	45.9	1.2	
30	4,0							7.88	13.90	0.82	0.32	0.51	*149,18*	7.88	13.90	0.82	0.32	0.51	*204,91*
Costs [%]									27.4	19.3	47.9	1.0			27.4	17.5	51.0	1.0	
		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.78	0.36	0.00	0.32	0.32	*92,03*	1.61	0.36	0.00	0.32	0.32	*100,78*	1.48	0.08	0.00	0.32	0.32	*84,988*
Costs [%]			45.8	0.0	29.9	9.4			48.5	0.0	29.3	8.6			36.8	0.0	46.7	10.2	
6	3,0	5.35	4.33	0.65	0.32	0.51	*182,01*	4.83	2.90	0.51	0.32	0.51	*172,97*	4.44	2.90	0.51	0.32	0.51	*191,31*
Costs [%]			51.2	16.4	21.0	4.0			47.8	15.2	27.0	4.2			49.3	15.5	26.1	3.8	
10	4,0	7.13	10.43	0.82	0.32	0.51	*222,28*	6.44	7.40	0.82	0.32	0.51	*222,94*	5.92	4.33	0.65	0.32	0.51	*202,2*
Costs [%]			56.0	14.7	21.7	2.2			51.6	17.2	24.7	2.2			45.2	16.3	33.3	1.8	
16	4,0	7.13	10.43	0.82	0.32	0.51	*226,54*	6.44	7.40	0.82	0.32	0.51	*237,47*	5.92	4.33	0.65	0.32	0.51	*229,68*
Costs [%]			42.1	17.6	34.6	1.6			37.1	19.4	38.9	1.5			30.5	17.2	48.9	1.2	
20	4,0	7.13	10.43	0.82	0.32	0.51	*229,47*	6.44	7.40	0.82	0.32	0.51	*246,32*	5.92	4.33	0.65	0.32	0.51	*246,11*
Costs [%]			34.9	18.2	42.1	1.3			30.1	19.5	46.6	1.3			23.9	16.8	56.6	0.9	
26	4,0	7.13	10.43	0.82	0.32	0.51	*233,55*	6.44	7.40	0.82	0.32	0.51	*257,82*	5.92	4.33	0.65	0.32	0.51	*267,23*
Costs [%]			26.8	18.2	51.3	1.0			22.5	18.8	55.9	0.9			17.2	15.6	65.2	0.7	
30	4,0	7.13	10.43	0.82	0.32	0.51	*235,96*	6.44	7.40	0.82	0.32	0.51	*264,33*	5.92	4.33	0.65	0.32	0.51	*279,08*
Costs [%]			22.8	17.8	56.4	0.9			18.8	18.1	60.7	0.8			14.1	14.7	69.5	0.6	

Construction costs for Rubble mound seawall armoured with modified cubes — **Hs = 6.0 m; s = 0.01**

				1 : 1.5						1 : 2						1 : 3			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.60	0.53	0.00	0.32	0.32	100	1.82	0.53	0.00	0.32	0.32	*120,33*	2.04	0.53	0.00	0.32	0.32	*162,47*
Costs [%]			59.3	0.0	10.1	8.6			62.6	0.0	12.0	7.2			67.3	0.0	13.9	5.3	
6	3,0	4.79	2.14	0.51	0.32	0.51	100	5.45	2.14	0.51	0.32	0.51	126,21	6.11	1.07	0.51	0.32	0.51	157,441
Costs [%]			55.6	10.7	16.2	7.2			56.3	10.7	19.1	5.7			52.1	12.6	26.6	4.6	
10	5,0	7.98	10.68	0.82	0.32	0.65	100	9.08	10.68	0.82	0.32	0.65	*128,00*	10.18	5.34	0.65	0.32	0.65	*155,23*
Costs [%]			58.4	11.8	16.3	2.8			58.3	11.6	18.9	2.8			55.3	10.4	28.2	1.8	
16	6,0	9.57	16.02	0.88	0.32	0.82	100	10.89	12.00	0.82	0.32	0.82	117,88	12.21	10.68	0.82	0.32	0.82	*167,56*
Costs [%]			51.4	13.6	22.0	4.5			50.4	11.9	27.9	3.8			49.8	11.8	31.9	2.7	
20	6,0	9.57	16.02	0.88	0.32	0.65	100	10.89	12.00	0.82	0.32	0.65	118,88	12.21	10.68	0.82	0.32	0.65	*170,42*
Costs [%]			46.3	16.5	27.6	1.9			44.9	13.9	34.2	1.6			44.0	13.1	38.3	1.1	
26	6,0	9.57	16.02	0.88	0.32	0.65	100	10.89	12.00	0.82	0.32	0.65	119,83	12.21	10.68	0.82	0.32	0.65	*172,34*
Costs [%]			38.3	18.7	34.7	2.0			36.9	15.3	41.8	1.7			36.0	13.9	46.1	1.1	
30	6,0	9.57	16.02	0.88	0.32	0.65	100	10.89	12.00	0.82	0.32	0.65	*120,50*	12.21	10.68	0.82	0.32	0.65	*173,77*
Costs [%]			34.0	19.6	39.1	1.8			32.6	15.8	46.3	1.5			31.7	14.1	50.7	1.0	

				1 : 4						1 : 5						1 : 6			
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	2.15	0.53	0.00	0.32	0.32	*205,94*	2.11	0.53	0.00	0.32	0.32	*242,69*	1.76	0.53	0.00	0.32	0.32	*254,26*
Costs [%]			70.4	0.0	14.8	4.2			72.6	0.0	14.8	3.6			74.1	0.0	12.9	4.4	
6	3,0	6.44	1.07	0.51	0.32	0.51	*207,13*	6.34	1.07	0.51	0.32	0.51	*248,8*	5.28	0.53	0.00	0.32	0.51	*212,22*
Costs [%]			52.7	12.7	28.1	3.5			53.5	12.8	28.2	2.9			53.4	0.0	41.3	3.4	
10	5,0	10.73	5.34	0.65	0.32	0.65	*205,63*	10.56	5.34	0.65	0.32	0.65	*248,8*	8.80	5.34	0.65	0.32	0.65	*258,51*
Costs [%]			55.5	10.3	29.5	1.3			56.0	10.3	29.5	1.4			58.3	10.6	27.0	1.4	
16	6,0	12.87	10.68	0.82	0.32	0.82	*220,89*	12.67	5.34	0.65	0.32	0.82	*235,32*	10.56	5.34	0.65	0.32	0.82	*246,78*
Costs [%]			50.2	11.6	33.3	2.1			45.6	9.9	40.9	1.9			47.3	10.2	38.9	1.8	
20	6,0	12.87	10.68	0.82	0.32	0.65	*225,29*	12.67	5.34	0.65	0.32	0.65	*243,89*	10.56	5.34	0.65	0.32	0.65	*257,88*
Costs [%]			44.2	12.6	39.7	0.9			39.5	10.5	47.7	0.8			40.7	10.9	46.2	0.8	
26	6,0	12.87	10.68	0.82	0.32	0.65	*227,65*	12.67	5.34	0.65	0.32	0.65	*251,45*	10.56	5.34	0.65	0.32	0.65	*268,92*
Costs [%]			36.2	13.2	47.6	0.9			31.7	10.6	55.6	0.8			32.3	11.0	54.8	0.7	
30	6,0	12.87	10.68	0.82	0.32	0.65	*229,55*	12.67	5.34	0.65	0.32	0.65	*256,48*	10.56	5.34	0.65	0.32	0.65	*276,29*
Costs [%]			31.9	13.3	52.2	0.8			27.6	10.5	60.1	0.7			27.9	10.8	59.6	0.6	

Construction costs for Rubble mound seawall armoured with dolosse — Hs = 6.0 m; s = 0.01

		1 : 1.5						1 : 2						1 : 3					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	1.31	0.53	0.00	0.32	0.32	76,62	1.49	0.53	0.00	0.32	0.32	91,63	1.67	0.53	0.00	0.32	0.32	122,83
Costs [%]			56.3	0.0	11.5	11.3			59.5	0.0	13.6	9.4			64.3	0.0	15.7	7.0	
6	3,0	3.92	1.07	0.51	0.32	0.51	68,23	4.46	1.07	0.51	0.32	0.51	86,59	5.00	0.53	0.00	0.32	0.51	101,17
Costs [%]			47.2	14.5	20.8	10.5			47.5	14.5	24.3	8.3			46.2	0.0	43.7	7.1	
10	5,0	6.53	5.34	0.65	0.32	0.65	64,49	7.43	5.34	0.65	0.32	0.65	83,10	8.33	2.14	0.51	0.32	0.65	102,10
Costs [%]			52.2	12.4	23.4	4.3			51.8	12.2	26.7	3.3			45.1	11.0	38.6	2.7	
16	6,0	7.83	10.68	0.82	0.32	0.82	72,63	8.91	5.34	0.65	0.32	0.82	80,69	9.99	5.34	0.65	0.32	0.82	118,30
Costs [%]			45.7	14.4	27.2	6.2			41.2	12.0	37.5	5.6			41.1	11.5	41.1	3.8	
20	6,0	7.83	10.68	0.82	0.32	0.65	73,44	8.91	5.34	0.65	0.32	0.65	83,29	9.99	5.34	0.65	0.32	0.65	123,52
Costs [%]			40.6	17.2	34.4	2.0			35.9	13.6	45.5	1.8			35.4	12.5	48.7	1.2	
26	6,0	7.83	10.68	0.82	0.32	0.65	76,63	8.91	5.34	0.65	0.32	0.65	88,72	9.99	5.34	0.65	0.32	0.65	131,33
Costs [%]			32.2	18.5	42.1	2.6			27.9	14.1	53.3	2.2			27.5	12.6	56.6	1.5	
30	6,0	7.83	10.68	0.82	0.32	0.65	78,20	8.91	5.34	0.65	0.32	0.65	91,68	9.99	5.34	0.65	0.32	0.65	135,83
Costs [%]			28.0	18.9	46.8	2.2			24.0	14.1	57.9	1.9			23.6	12.5	61.1	1.3	

		1 : 4						1 : 5						1 : 6					
Depth/Costs	H_s [m]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]	Run-up [m]	$W_{primary}$ [t]	$D_{50Second}$ [m]	D_{50Core} [m]	D_{50Toe} [m]	Ctotal [%]
2	1,0	1.76	0.53	0.00	0.32	0.32	154,89	1.73	0.53	0.00	0.32	0.32	182,33	1.44	0.53	0.00	0.32	0.32	190,54
Costs [%]			67.4	0.0	16.7	5.6			69.7	0.0	16.8	4.7			71.9	0.0	15.2	4.5	
6	3,0	5.27	0.53	0.00	0.32	0.51	133,64	5.18	0.53	0.00	0.32	0.51	160,96	4.32	0.53	0.00	0.32	0.51	167,83
Costs [%]			46.7	0.0	45.8	5.4			47.4	0.0	46.3	4.5			49.6	0.0	44.4	4.3	
10	5,0	8.78	2.14	0.51	0.32	0.65	136,08	8.64	2.14	0.51	0.32	0.65	164,83	7.20	1.07	0.51	0.32	0.65	157,70
Costs [%]			45.0	10.9	40.1	2.0			45.4	10.9	40.3	1.7			41.1	12.6	43.4	1.8	
16	6,0	10.53	5.34	0.65	0.32	0.82	156,49	10.37	2.14	0.51	0.32	0.82	167,72	8.64	2.14	0.51	0.32	0.82	177,55
Costs [%]			41.2	11.3	42.7	2.9			34.6	9.7	52.0	2.7			35.9	10.1	50.5	2.6	
20	6,0	10.53	5.34	0.65	0.32	0.65	164,28	10.37	2.14	0.51	0.32	0.65	179,95	8.64	2.14	0.51	0.32	0.65	192,59
Costs [%]			35.3	12.0	50.2	0.9			29.0	10.0	59.4	0.8			29.8	10.3	58.4	0.8	
26	6,0	10.53	5.34	0.65	0.32	0.65	174,36	10.37	2.14	0.51	0.32	0.65	195,64	8.64	2.14	0.51	0.32	0.65	212,16
Costs [%]			27.5	12.0	58.1	1.1			22.1	9.6	66.7	1.0			22.4	9.8	66.3	0.9	
30	6,0	10.53	5.34	0.65	0.32	0.65	180,36	10.37	2.14	0.51	0.32	0.65	204,98	8.64	2.14	0.51	0.32	0.65	224,05
Costs [%]			23.6	11.8	62.6	1.0			18.7	9.2	70.7	0.9			18.8	9.4	70.5	0.8	

Construction costs for Rubble mound seawall armoured with tetrapods — Hs = 6.0 m; s = 0.01

		1 : 1.5						1 : 2						1 : 3					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.45	0.53	0.00	0.32	0.32	95,07	1.65	0.53	0.00	0.32	0.32	114,31	1.85	0.53	0.00	0.32	0.32	154,26
Costs [%]			60.5	0.0	9.9	9.1			63.7	0.0	11.6	7.6			68.3	0.0	13.4	5.6	
6	3,0	4.35	2.14	0.51	0.32	0.51	95,68	4.95	2.14	0.51	0.32	0.51	120,30	5.55	1.07	0.51	0.32	0.51	149,21
Costs [%]			56.7	10.7	15.5	7.5			57.4	10.8	18.2	6.0			53.3	12.7	25.3	4.8	
10	5,0	7.25	10.68	0.82	0.32	0.65	95,56	8.25	10.68	0.82	0.32	0.65	121,14	9.25	5.34	0.65	0.32	0.65	147,03
Costs [%]			59.6	11.8	15.6	2.9			59.9	11.7	18.2	2.3			56.6	10.4	26.9	1.9	
16	6,0	8.70	16.02	0.88	0.32	0.82	95,97	9.90	12.00	0.82	0.32	0.82	112,7	11.10	10.68	0.82	0.32	0.82	159,57
Costs [%]			52.4	13.6	21.3	4.7			51.4	12.0	27.0	4.0			50.9	11.8	30.8	2.8	
20	6,0	8.70	16.02	0.88	0.32	0.65	95,99	9.90	12.00	0.82	0.32	0.65	113,276	11.10	10.68	0.82	0.32	0.65	162,49
Costs [%]			47.1	16.6	27.1	2.0			46.0	14.1	33.6	1.3			44.9	13.3	37.4	1.2	
26	6,0	8.70	16.02	0.88	0.32	0.65	96,21	9.90	12.00	0.82	0.32	0.65	114,96	11.10	10.68	0.82	0.32	0.65	164,85
Costs [%]			38.9	18.8	34.4	2.1			37.5	15.4	41.3	1.7			36.7	14.1	45.5	1.2	
30	6,0	8.70	16.02	0.88	0.32	0.65	96,36	9.90	12.00	0.82	0.32	0.65	115,82	11.10	10.68	0.82	0.32	0.65	166,58
Costs [%]			34.5	19.6	38.8	1.8			33.1	15.9	46.0	1.5			32.2	14.3	50.2	1.1	

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.95	0.53	0.00	0.32	0.32	195,54	1.92	0.53	0.00	0.32	0.32	230,81	1.60	0.53	0.00	0.32	0.32	240,73
Costs [%]			71.4	0.0	14.2	4.4			73.6	0.0	14.2	3.7			75.7	0.0	12.6	3.6	
6	3,0	5.85	1.07	0.51	0.32	0.51	195,85	5.76	1.07	0.51	0.32	0.51	235,37	4.80	0.53	0.00	0.32	0.51	201,48
Costs [%]			53.9	12.7	26.8	3.7			54.7	12.8	26.9	3.1			54.6	0.0	40.0	3.6	
10	5,0	9.75	5.34	0.65	0.32	0.65	194,40	9.60	5.34	0.65	0.32	0.65	234,62	8.00	2.14	0.51	0.32	0.65	206,03
Costs [%]			56.9	10.4	28.2	1.4			57.6	10.4	28.2	1.2			52.4	9.9	34.8	1.3	
16	6,0	11.70	10.68	0.82	0.32	0.82	209,99	11.52	5.34	0.65	0.32	0.82	223,2	9.60	5.34	0.65	0.32	0.82	235,78
Costs [%]			51.3	11.7	32.2	2.2			46.7	10.0	39.6	2.0			48.4	10.3	37.8	1.9	
20	6,0	11.70	10.68	0.82	0.32	0.65	214,49	11.52	5.34	0.65	0.32	0.65	231,33	9.60	5.34	0.65	0.32	0.65	246,81
Costs [%]			45.1	12.8	38.8	0.9			40.5	10.7	46.8	0.6			41.5	11.0	45.3	0.8	
26	6,0	11.70	10.68	0.82	0.32	0.65	217,47	11.52	5.34	0.65	0.32	0.65	239,94	9.60	5.34	0.65	0.32	0.65	258,31
Costs [%]			36.8	13.3	46.9	0.9			32.3	10.8	54.9	0.8			32.9	11.1	54.2	0.8	
30	6,0	11.70	10.68	0.82	0.32	0.65	219,78	11.52	5.34	0.65	0.32	0.65	245,37	9.60	5.34	0.65	0.32	0.65	266,01
Costs [%]			32.4	13.4	51.7	0.8			28.1	10.7	59.6	0.7			28.3	10.9	59.2	0.7	

Construction costs for Rubble mound seawall armoured with quarry stones Hs = 6.0 m; s = 0.01

		1 : 1.5							1 : 2							1 : 3						
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.97	0.36	0.00	0.32	0.32	*57,31*	1.97	0.36	0.00	0.32	0.32	*65,13*	1.97	0.36	0.00	0.32	0.32	*81,314*
Costs [%]			36.6	0.0	24.5	15.1			38.7	0.0	27.0	13.3			42.6	0.0	30.0	10.6	
6	3,0	5.91	7.40	0.82	0.32	0.51	132,747	5.91	7.40	0.82	0.32	0.51	153,87	5.91	7.40	0.82	0.32	0.51	199,323
Costs [%]			51.4	16.2	12.5	5.4			52.9	16.3	13.6	4.7			55.5	16.6	14.6	3.6	
10	5,0																		
Costs [%]																			
16	6,0																		
Costs [%]																			
20	6,0																		
Costs [%]																			
26	6,0																		
Costs [%]																			
30	6,0																		
Costs [%]																			

		1 : 4						1 : 5						1 : 6					
	H_s	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal	Run-up	$W_{primary}$	$D_{50Second}$	D_{50Core}	D_{50Toe}	Ctotal
Depth/Costs	[m]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]	[m]	[t]	[m]	[m]	[m]	[%]
2	1,0	1.78	0.36	0.00	0.32	0.32	*92,03*	1.61	0.36	0.00	0.32	0.32	*100,78*	1.48	0.08	0.00	0.32	0.32	*84,99*
Costs [%]			45.8	0.0	29.9	9.4			48.5	0.0	29.3	8.6			36.8	0.0	46.7	10.2	
6	3,0	5.35	4.33	0.65	0.32	0.51	*182,01*	4.83	2.90	0.51	0.32	0.51	*172,97*	4.44	2.90	0.51	0.32	0.51	*191,31*
Costs [%]			51.2	16.4	21.0	4.0			47.8	15.2	27.0	4.2			49.3	15.5	26.1	3.8	
10	5,0													7.40	13.90	0.82	0.32	0.65	*275,25*
Costs [%]															64.8	11.8	18.2	1.3	
16	6,0																		
Costs [%]																			
20	6,0																		
Costs [%]																			
26	6,0																		
Costs [%]																			
30	6,0																		
Costs [%]																			

Construction costs for Vertically composite caisson-type seawall **Hs = 4.0 m; s = 0.01**

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m^2]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	8.30	99.0	0,7	24.50	8.90	66.46	1.20	1.62	75.433	988,19
6,0	3,0	5,4	3,8	9.2	16.10	100.0	2,1	54.89	8.62	35.60	1.21	2.38	144.454	339,92
10,0	4,0	7,2	5,0	13.0	19.10	100.0	3,5	61.28	11.77	25.72	1.21	2.13	191.175	235,42
16,0	4,0	7,2	5,0	16.9	17.70	100.0	5,6	59.06	17.64	22.05	1.21	1.48	227.151	210,22
20,0	4,0	7,2	5,0	19.5	17.10	100.0	7,0	57.65	21.19	19.91	1.20	1.22	251.196	195,85
26,0	4,0	7,2	5,0	23.4	17.90	100.0	9,1	57.12	25.35	16.20	1.35	1.21	286.008	188,26
30,0	4,0	7,2	5,0	26.0	18.40	100.0	10,5	56.56	27.81	14.27	1.44	1.21	309.160	183,12

Construction costs for Vertically composite caisson-type seawall **Hs = 6.0 m; s = 0.01**

Depth [m]	H_s [m]	H_{max} [m]	h_{crest} [m + S.L.]	$h_{caisson}$ [m]	breadth [m]	sandfill [%]	h_{mound} [m]	$C_{concrete}$ [%]	C_{mound} [%]	C_{fixed} [%]	$SF_{TURNING}$	$SF_{SLIDING}$	Pressure [kN/m^2]	Ctotal [%]
2,0	1,0	1,8	1,3	4.1	8.30	99.0	0,7	24.50	8.90	66.46	1.20	1.62	75.433	988,19
6,0	3,0	5,4	3,8	9.2	16.10	100.0	2,1	54.89	8.62	35.60	1.21	2.38	144.454	339,92
10,0	5,0	9	6,3	14.2	24.10	100.0	3,5	67.35	10.49	20.65	1.20	2.61	213.863	211,24
16,0	6,0	10,8	7,5	19.4	26.20	100.0	5,6	68.05	15.06	15.17	1.20	2.14	272.097	173,02
20,0	6,0	10,8	7,5	22.0	25.20	100.0	7,0	66.15	18.24	13.89	1.20	1.80	295.501	169,84
26,0	6,0	10,8	7,5	25.9	24.20	100.0	9,1	63.57	22.55	12.18	1.20	1.46	330.607	160,28
30,0	6,0	10,8	7,5	28.5	23.70	100.0	10,5	61.96	25.17	11.20	1.20	1.29	354.025	154,87

Appendix F

Changes in longshore transport for artificial islands

F.1 Tables and graphs for variations in distance offshore, island's diameter and angle of incidence

Changes in net yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,315	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	4000	1,315	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	6000	1,315	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	8000	1,315	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	9360	0,94	2,35	-4,26	22,69	-0,47	25,06	-0,47
2,00	5,06	9485	0,68	1,80	-2,78	20,14	-0,14	7,65	-0,14
2,00	5,06	9610	0,41	1,20	-1,54	16,65	-0,02	1,27	-0,02
2,00	5,06	9735	0,23	0,65	-2,70	12,32	-0,01	0,52	-0,01
2,00	5,06	9860	0,26	0,65	-7,09	12,32	-0,03	1,68	-0,03
2,00	5,06	9985	0,44	1,25	-8,16	17,03	-0,14	7,59	-0,14
2,00	5,06	10110	0,63	1,65	-8,82	19,35	-0,37	19,69	-0,37
2,00	5,06	10235	0,86	2,10	-9,04	21,63	-0,77	41,23	-0,77
2,00	5,06	10360	1,08	2,65	-8,83	23,95	-1,31	70,51	-1,31
2,00	5,06	11000	1,32	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	13000	1,32	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	15000	1,32	3,25	-7,67	26,08	-1,86	100	-1,86
2,00	5,06	17000	1,32	3,25	-7,67	26,08	-1,86	100	-1,86

Changes in net yearly longshore sediment transports

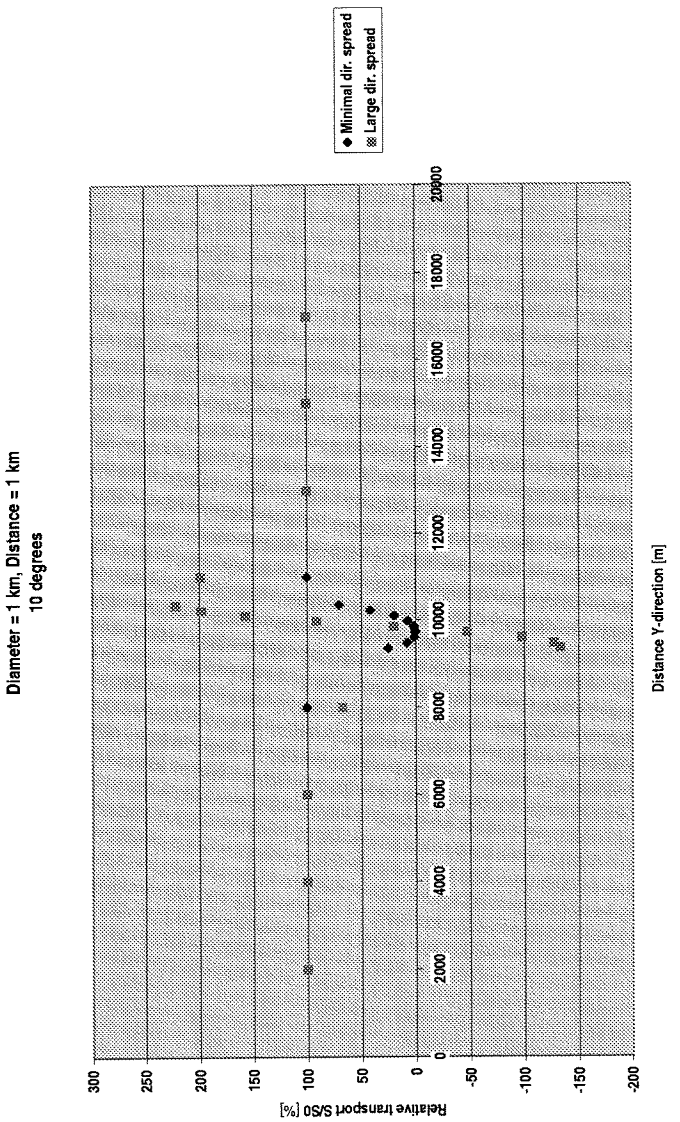
Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	4000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	6000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	8000	1,238	3,05	-2,87	25,38	-0,61	67,57	-0,61
2,00	5,06	9360	1,05	2,55	8,57	23,54	1,20	-133,34	1,20
2,00	5,06	9485	1,02	2,50	8,85	23,32	1,14	-127,36	1,14
2,00	5,06	9610	0,98	2,40	7,46	22,90	0,87	-97,34	0,87
2,00	5,06	9735	0,95	2,35	3,77	22,68	0,42	-46,65	0,42
2,00	5,06	9860	0,94	2,30	-1,70	22,48	-0,18	20,24	-0,18
2,00	5,06	9985	0,95	2,35	-7,52	22,68	-0,82	91,28	-0,82
2,00	5,06	10110	0,97	2,40	-12,52	22,90	-1,41	157,08	-1,41
2,00	5,06	10235	0,99	2,45	-14,98	23,11	-1,77	197,64	-1,77
2,00	5,06	10360	1,02	2,50	-15,76	23,32	-1,99	222,02	-1,99
2,00	5,06	11000	1,19	2,95	-9,38	25,03	-1,79	199,31	-1,79
2,00	5,06	13000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90
2,00	5,06	15000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90
2,00	5,06	17000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90



Diameter = 1 km, Distance = 1 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300
250
200
150
100
50
0
-50
-100
-150
-200
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	4000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	6000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	8000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	9240	0,945	2,35	-3,76	22,69	-0,41	22,23	-0,41
2,00	5,06	9365	0,778	1,9	-2,95	20,67	-0,20	10,78	-0,20
2,00	5,06	9490	0,632	1,6	-3,04	19,07	-0,13	6,76	-0,13
2,00	5,06	9615	0,546	1,35	-5,15	17,69	-0,15	7,90	-0,15
2,00	5,06	9740	0,581	1,4	-7,97	17,99	-0,26	13,98	-0,26
2,00	5,06	9865	0,718	1,8	-9,23	20,14	-0,51	27,56	-0,51
2,00	5,06	9990	0,864	2,1	-9,51	21,63	-0,82	44,11	-0,82
2,00	5,06	10115	1,017	2,5	-9,44	23,32	-1,22	65,42	-1,22
2,00	5,06	10240	1,128	2,75	-9,1	24,32	-1,51	81,01	-1,51
2,00	5,06	11000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	13000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	15000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	17000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	19000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

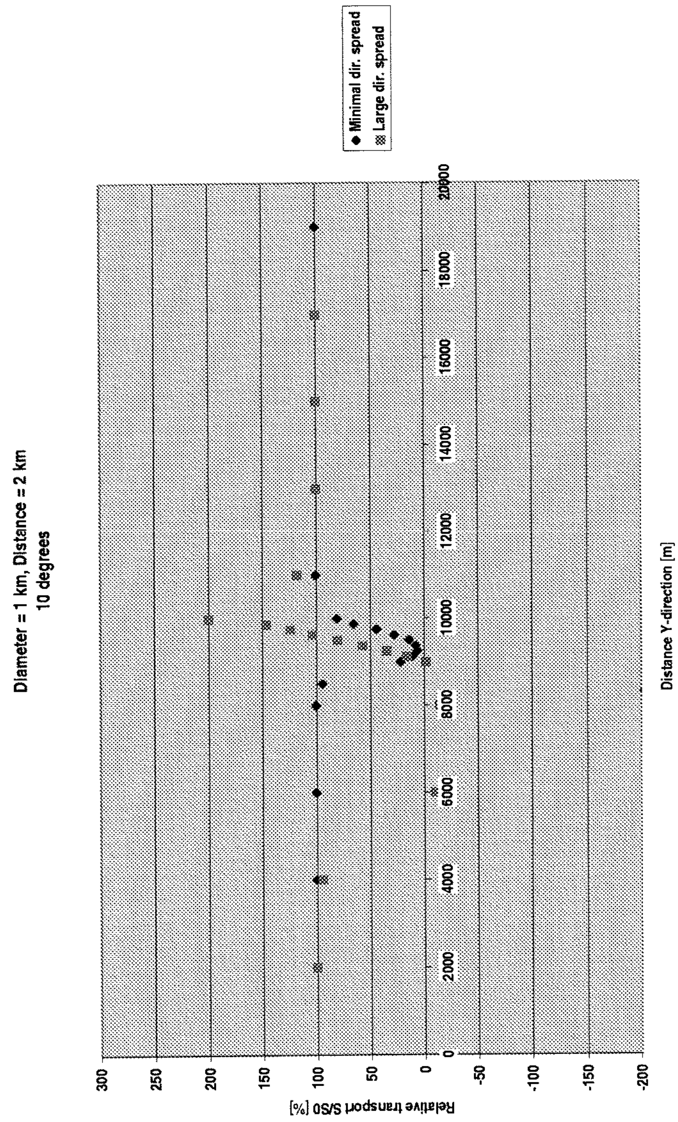
Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	4000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	6000	1,255	3,1	-3,89	25,55	-0,85	94,61	-0,85
2,00	5,06	8000	1,216	3	0,4	25,2	0,08	-9,04	0,08
2,00	5,06	9240	1,144	2,8	0,75	24,5	0,13	-14,58	0,13
2,00	5,06	9365	1,14	2,8	0,05	24,5	0,01	-0,97	0,01
2,00	5,06	9490	1,127	2,75	-0,86	24,32	-0,14	16,10	-0,14
2,00	5,06	9615	1,114	2,7	-1,94	24,15	-0,32	35,22	-0,32
2,00	5,06	9740	1,112	2,7	-3,16	24,15	-0,51	57,09	-0,51
2,00	5,06	9865	1,112	2,7	-4,46	24,15	-0,72	80,42	-0,72
2,00	5,06	9990	1,112	2,7	-5,74	24,15	-0,93	103,22	-0,93
2,00	5,06	10115	1,114	2,7	-6,91	24,15	-1,12	124,33	-1,12
2,00	5,06	10240	1,126	2,7	-7,93	24,32	-1,31	146,35	-1,31
2,00	5,06	11000	1,193	2,75	-9,43	25,03	-1,79	200,00	-1,79
2,00	5,06	13000	1,254	2,95	-4,85	25,55	-1,05	117,57	-1,05
2,00	5,06	15000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	17000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90


Diameter = 1 km, Distance = 2 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300
250
200
150
100
50
0
-50
-100
-150
-200
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 5 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	4000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	6000	1,317	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	8000	1,317	3,25	-6,49	25,72	-1,56	83,99	-1,56
2,00	5,06	9000	1,008	2,50	-5,98	23,32	-0,77	41,16	-0,77
2,00	5,06	9125	0,993	2,45	-6,76	23,11	-0,83	44,65	-0,83
2,00	5,06	9250	0,998	2,50	-7,6	23,11	-0,94	50,58	-0,94
2,00	5,06	9375	1,021	2,55	-8,32	23,32	-1,08	58,35	-1,08
2,00	5,06	9500	1,05	2,65	-8,81	23,51	-1,22	65,76	-1,22
2,00	5,06	9625	1,091	2,80	-9,07	23,95	-1,38	74,39	-1,38
2,00	5,06	9750	1,14	2,90	-9,15	24,5	-1,56	83,80	-1,56
2,00	5,06	9875	1,18	3,00	-9,07	24,85	-1,68	90,30	-1,68
2,00	5,06	10000	1,216	3,25	-8,9	25,2	-1,77	95,48	-1,77
2,00	5,06	11000	1,317	3,25	-7,75	26,08	-1,88	101,33	-1,88
2,00	5,06	13000	1,317	3,25	-7,75	26,08	-1,88	101,33	-1,88
2,00	5,06	15000	1,317	3,25	-7,75	26,08	-1,88	101,33	-1,88
2,00	5,06	17000	1,317	3,25	-7,75	26,08	-1,88	101,33	-1,88
2,00	5,06	19000	1,317	3,25	-7,75	26,08	-1,88	101,33	-1,88

Changes in net yearly longshore sediment transports

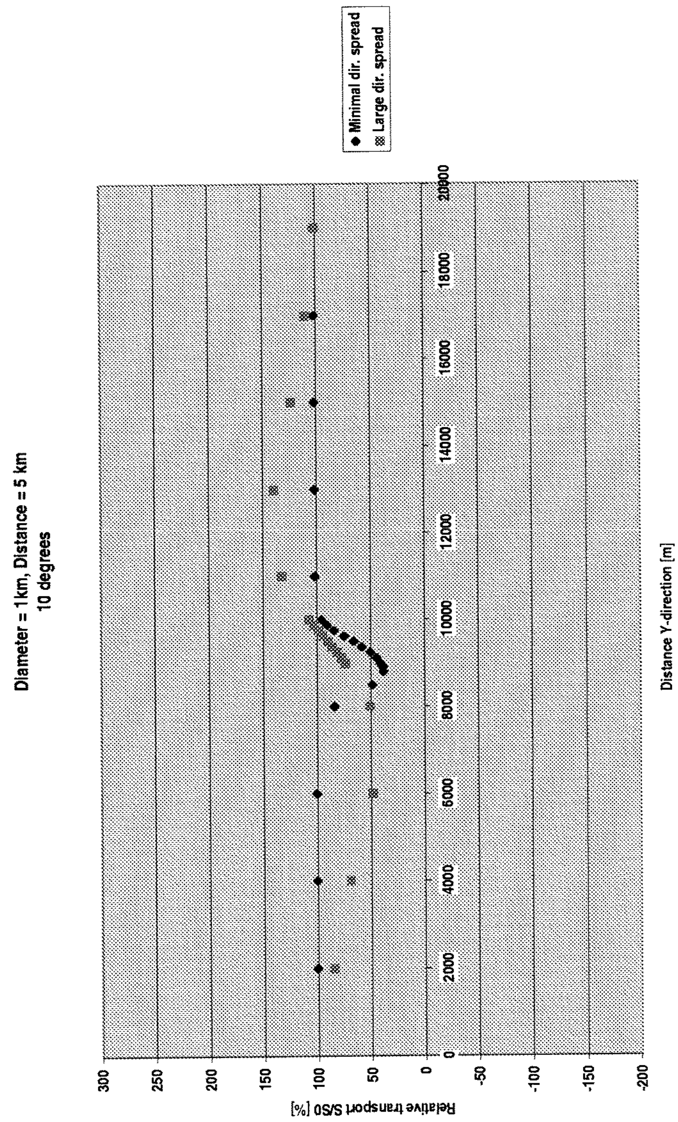
Diameter island: 1 km; φ_0 = 10 degrees

Distance offshore: 5 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,255	3,1	-3,49	25,55	-0,76	84,93	-0,76
2,00	5,06	4000	1,253	3,1	-2,85	25,55	-0,62	69,19	-0,62
2,00	5,06	6000	1,237	3,05	-2,08	25,38	-0,44	48,93	-0,44
2,00	5,06	8000	1,218	3	-2,25	25,2	-0,46	50,94	-0,46
2,00	5,06	9000	1,215	3,00	-3,27	25,2	-0,66	73,59	-0,66
2,00	5,06	9125	1,214	3,00	-3,44	25,2	-0,69	77,27	-0,69
2,00	5,06	9250	1,214	3,00	-3,62	25,2	-0,73	81,29	-0,73
2,00	5,06	9375	1,214	3,00	-3,81	25,2	-0,77	85,53	-0,77
2,00	5,06	9500	1,214	3,00	-4	25,2	-0,81	89,77	-0,81
2,00	5,06	9625	1,213	3,00	-4,2	25,2	-0,84	94,07	-0,84
2,00	5,06	9750	1,213	3,00	-4,39	25,2	-0,88	98,29	-0,88
2,00	5,06	9875	1,213	3,00	-4,58	25,2	-0,92	102,51	-0,92
2,00	5,06	10000	1,213	3,00	-4,77	25,2	-0,96	106,73	-0,96
2,00	5,06	11000	1,218	3	-5,88	25,2	-1,19	132,33	-1,19
2,00	5,06	13000	1,237	3,05	-5,97	25,38	-1,25	139,54	-1,25
2,00	5,06	15000	1,254	3,1	-5,08	25,55	-1,10	123,09	-1,10
2,00	5,06	17000	1,256	3,1	-4,51	25,55	-0,98	109,75	-0,98


Diameter = 1 km, Distance = 5 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
300
250
200
150
100
50
0
-50
-100
-150
-200
Distance Y-direction [m]
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 2 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	4000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	6000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	8000	1,31	3,25	-7,61	26,08	-1,84	98,93	-1,84
2,00	5,06	8860	0,79	1,95	-3,13	20,94	-0,22	11,82	-0,22
2,00	5,06	9110	0,37	1,20	-0,94	16,65	-0,01	0,63	-0,01
2,00	5,06	9360	0,10	0,65	0,94	12,37	0,00	-0,03	0,00
2,00	5,06	9610	0,02	0,28	0,46	6,96	0,00	0,00	0,00
2,00	5,06	9860	0,05	0,65	-7,16	12,38	0,00	0,05	0,00
2,00	5,06	10110	0,19	0,65	-6,53	12,38	-0,02	0,87	-0,02
2,00	5,06	10360	0,45	1,25	-8,52	17,02	-0,16	8,62	-0,16
2,00	5,06	10610	0,84	2,05	-9,22	21,42	-0,74	39,60	-0,74
2,00	5,06	10860	1,18	2,90	-8,73	24,85	-1,61	86,72	-1,61
2,00	5,06	11000	1,27	3,15	-8,18	25,72	-1,83	98,40	-1,83
2,00	5,06	13000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	15000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	17000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	19000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

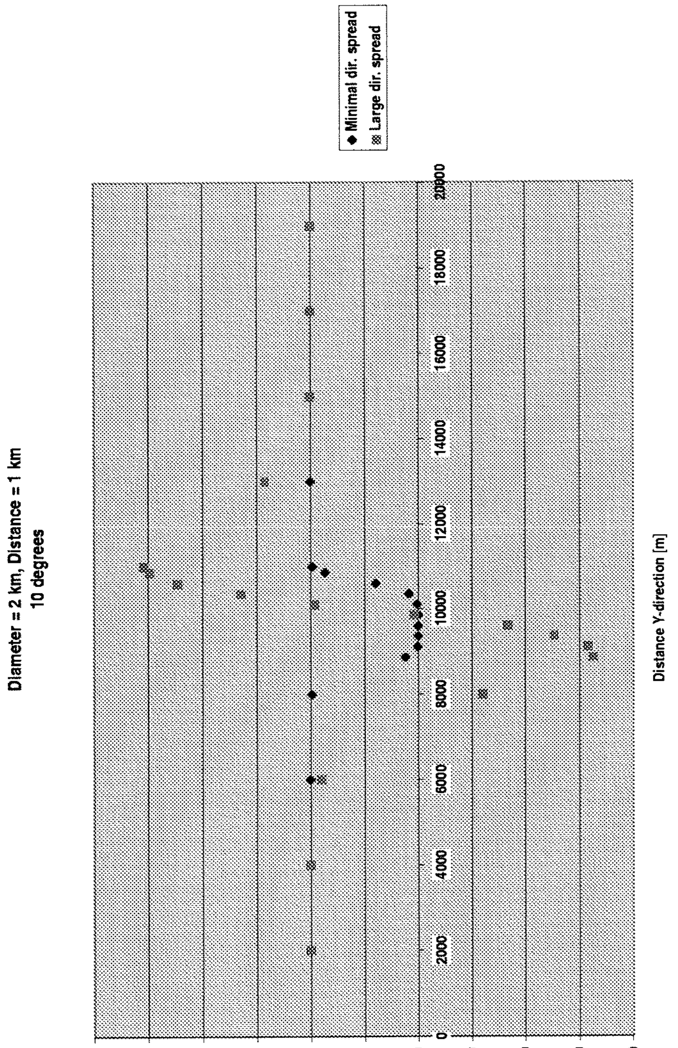
Diameter island: 2 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,26	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	4000	1,26	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	6000	1,25	3,1	-3,72	25,55	-0,81	90,35	-0,81
2,00	5,06	8000	1,18	2,9	2,88	24,85	0,54	-60,21	0,54
2,00	5,06	8860	1,01	2,50	11,59	23,32	1,46	-162,94	1,46
2,00	5,06	9110	0,91	2,25	14,79	22,26	1,42	-158,03	1,42
2,00	5,06	9360	0,81	1,95	16,30	20,94	1,14	-126,97	1,14
2,00	5,06	9610	0,73	1,85	13,06	20,40	0,75	-83,58	0,75
2,00	5,06	9860	0,71	1,80	-0,54	20,14	-0,03	3,29	-0,03
2,00	5,06	10110	0,75	1,85	-14,70	20,40	-0,87	96,53	-0,87
2,00	5,06	10360	0,83	2,00	-20,34	21,21	-1,48	164,62	-1,48
2,00	5,06	10610	0,94	2,30	-20,38	22,48	-2,00	222,84	-2,00
2,00	5,06	10860	1,02	2,50	-17,96	23,32	-2,24	249,14	-2,24
2,00	5,06	11000	1,07	2,60	-16,44	23,75	-2,28	254,46	-2,28
2,00	5,06	13000	1,24	3,05	-6,07	25,38	-1,27	142,07	-1,27
2,00	5,06	15000	1,26	3,10	-4,14	25,55	-0,90	100,81	-0,90
2,00	5,06	17000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90


Diameter = 2 km, Distance = 1 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 2 km; $\varphi_0 = 10$ degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	4000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	6000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	8000	1,29	3,2	-7,07	25,9	-1,64	88,39	-1,64
2,00	5,06	8710	0,86	2,1	-3,02	21,63	-0,26	14,04	-0,26
2,00	5,06	8960	0,54	1,35	-1,24	17,69	-0,04	1,90	-0,04
2,00	5,06	9210	0,26	0,65	0,23	12,37	0,00	-0,06	0,00
2,00	5,06	9460	0,15	0,65	-2,45	12,37	0,00	0,19	0,00
2,00	5,06	9710	0,23	0,65	-7,38	12,37	-0,02	1,34	-0,02
2,00	5,06	9960	0,42	1,25	-8,82	17,02	-0,14	7,77	-0,14
2,00	5,06	10210	0,68	1,75	-9,53	19,88	-0,47	25,16	-0,47
2,00	5,06	10460	0,96	2,4	-9,61	22,9	-1,09	58,72	-1,09
2,00	5,06	10710	1,18	2,9	-9,01	24,85	-1,65	88,96	-1,65
2,00	5,06	11000	1,28	3,15	-8,12	25,72	-1,84	98,78	-1,84
2,00	5,06	13000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	15000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	17000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

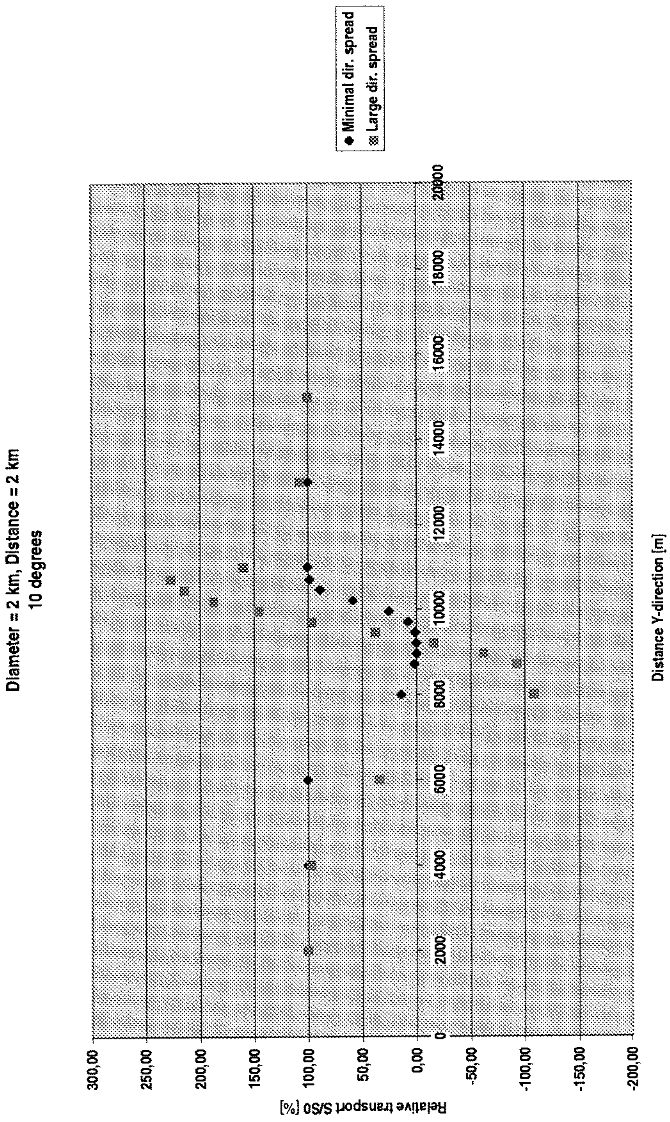
Diameter island: 2 km; $\varphi_0 = 10$ degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,26	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	4000	1,26	3,1	-4,01	25,55	-0,88	97,66	-0,88
2,00	5,06	6000	1,14	3,05	-1,7	25,39	-0,30	33,81	-0,30
2,00	5,06	8000	1,14	2,8	5,43	24,5	0,94	-104,55	0,94
2,00	5,06	8710	1,07	2,6	6,68	23,75	0,97	-108,31	0,97
2,00	5,06	8960	1,04	2,55	6,03	23,53	0,83	-92,53	0,83
2,00	5,06	9210	1,02	2,5	4,22	23,32	0,55	-61,48	0,55
2,00	5,06	9460	1,00	2,45	1,16	23,11	0,15	-16,25	0,15
2,00	5,06	9710	1,00	2,45	-2,71	23,11	-0,34	37,76	-0,34
2,00	5,06	9960	1,01	2,5	-6,73	23,32	-0,86	96,17	-0,86
2,00	5,06	10210	1,02	2,5	-10,13	23,32	-1,31	145,63	-1,31
2,00	5,06	10460	1,04	2,55	-12,47	23,53	-1,68	186,74	-1,68
2,00	5,06	10710	1,07	2,6	-13,61	23,75	-1,92	213,99	-1,92
2,00	5,06	11000	1,09	2,65	-13,58	23,95	-2,03	226,83	-2,03
2,00	5,06	13000	1,24	3,05	-6,87	25,38	-1,43	159,68	-1,43
2,00	5,06	15000	1,26	3,1	-4,44	25,55	-0,97	107,88	-0,97
2,00	5,06	17000	1,26	3,1	-4,1	25,55	-0,90	100,00	-0,90


Diameter = 2 km, Distance = 2 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 2 km; φ_0 = 10 degrees

Distance offshore: 5 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	4000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	6000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	8000	1,07	2,6	-4,14	23,75	-0,61	32,64	-0,61
2,00	5,06	8200	0,97	2,40	-3,36	22,9	-0,40	21,26	-0,40
2,00	5,06	8450	0,82	1,95	-2,56	20,93	-0,19	10,40	-0,19
2,00	5,06	8700	0,7	1,80	-2,71	20,14	-0,14	7,79	-0,14
2,00	5,06	8950	0,61	1,50	-4,9	18,53	-0,18	9,74	-0,18
2,00	5,06	9200	0,64	1,65	-8,1	19,34	-0,35	18,76	-0,35
2,00	5,06	9450	0,76	1,90	-9,66	20,67	-0,62	33,19	-0,62
2,00	5,06	9700	0,9	2,20	-10,01	22,05	-0,94	50,80	-0,94
2,00	5,06	9950	1,04	2,55	-9,85	23,53	-1,32	71,22	-1,32
2,00	5,06	10200	1,13	2,75	-9,44	24,32	-1,57	84,23	-1,57
2,00	5,06	11000	1,3	3,20	-8,08	25,9	-1,88	101,33	-1,88
2,00	5,06	13000	1,32	3,25	-7,67	25,9	-1,85	99,61	-1,85
2,00	5,06	15000	1,32	3,25	-7,67	25,9	-1,85	99,61	-1,85
2,00	5,06	17000	1,32	3,25	-7,67	25,9	-1,85	99,61	-1,85
2,00	5,06	19000	1,32	3,25	-7,67	25,9	-1,85	99,61	-1,85

Changes in net yearly longshore sediment transports

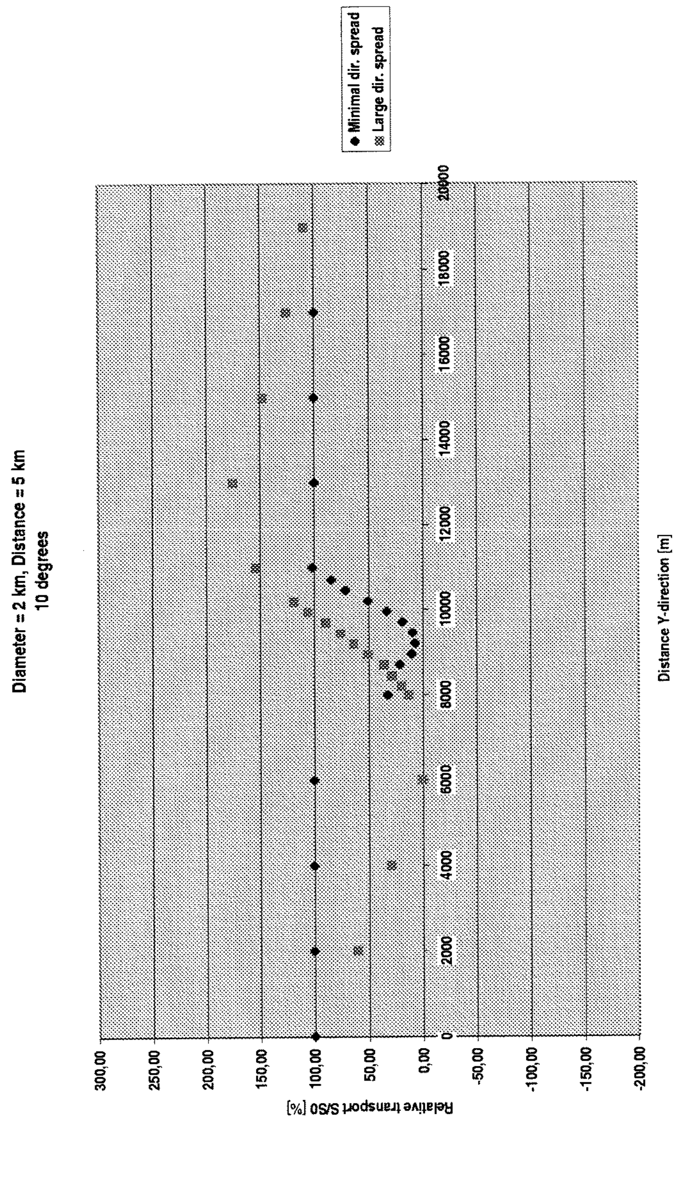
Diameter island: 2 km; φ_0 = 10 degrees

Distance offshore: 5 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	1,25	3,1	-2,51	25,55	-0,55	60,86	-0,55
2,00	5,06	4000	1,24	3,05	-1,26	25,38	-0,27	29,61	-0,27
2,00	5,06	6000	1,22	3	-0,06	25,2	-0,01	1,35	-0,01
2,00	5,06	8000	1,18	2,9	-0,66	24,85	-0,12	13,80	-0,12
2,00	5,06	8200	1,18	2,90	-0,96	24,85	-0,18	20,00	-0,18
2,00	5,06	8450	1,16	2,85	-1,41	24,68	-0,26	28,52	-0,26
2,00	5,06	8700	1,13	2,70	-1,9	24,15	-0,32	35,49	-0,32
2,00	5,06	8950	1,16	2,85	-2,55	24,68	-0,46	51,27	-0,46
2,00	5,06	9200	1,16	2,80	-3,2	24,5	-0,57	63,71	-0,57
2,00	5,06	9450	1,15	2,80	-3,9	24,5	-0,68	76,10	-0,68
2,00	5,06	9700	1,15	2,80	-4,6	24,5	-0,81	89,81	-0,81
2,00	5,06	9950	1,16	2,85	-5,29	24,68	-0,95	105,90	-0,95
2,00	5,06	10200	1,16	2,85	-5,93	24,68	-1,06	118,54	-1,06
2,00	5,06	11000	1,18	2,90	-7,46	24,85	-1,38	154,20	-1,38
2,00	5,06	13000	1,22	3,00	-7,85	25,2	-1,57	175,40	-1,57
2,00	5,06	15000	1,24	3,05	-6,3	25,38	-1,32	147,37	-1,32
2,00	5,06	17000	1,25	3,1	-5,16	25,55	-1,12	125,00	-1,12


Diameter = 2 km, Distance = 5 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	3000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	5000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	7000	1,28	3,15	-6,93	25,72	-1,57	84,21	-1,57
2,00	5,06	7870	0,07	1,80	-1,91	20,14	0,00	0,06	0,00
2,00	5,06	8370	0,19	0,65	0,25	12,34	0,00	-0,03	0,00
2,00	5,06	8870	0,02	0,38	-0,88	8,30	0,00	0,00	0,00
2,00	5,06	9370	0,00	0,00	0,00	0,01	0,00	0,00	0,00
2,00	5,06	9870	0,03	0,38	-5,99	8,30	0,00	0,01	0,00
2,00	5,06	10370	0,03	0,55	-7,65	11,08	0,00	0,02	0,00
2,00	5,06	10870	0,26	0,65	-7,85	12,34	-0,03	1,82	-0,03
2,00	5,06	11370	0,81	1,95	-9,60	20,95	-0,70	37,42	-0,70
2,00	5,06	11870	1,25	3,10	-8,52	25,55	-1,83	98,53	-1,83
2,00	5,06	12000	1,28	3,15	-8,12	25,72	-1,84	98,78	-1,84
2,00	5,06	14000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	16000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	18000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

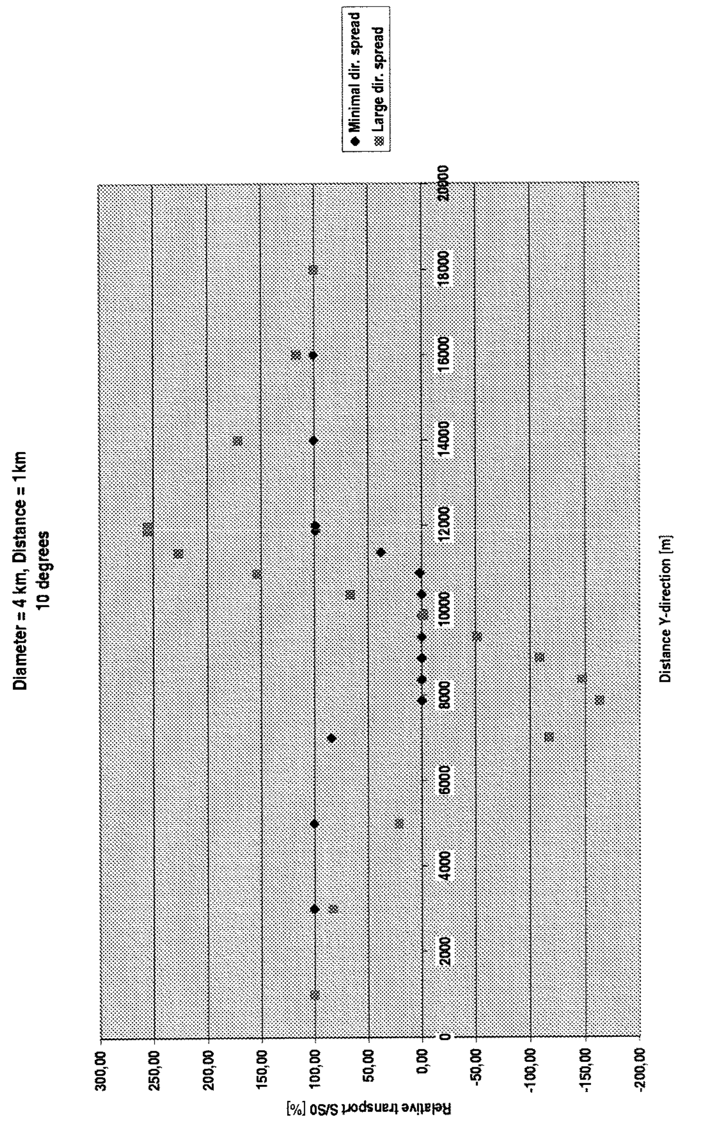
Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,26	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	3000	1,25	3,1	-3,43	25,55	-0,75	83,35	-0,75
2,00	5,06	5000	1,24	3,05	-0,89	25,38	-0,19	20,88	-0,19
2,00	5,06	7000	1,13	2,75	6,34	24,32	1,05	-117,55	1,05
2,00	5,06	7870	1,00	2,45	11,98	23,12	1,47	-163,39	1,47
2,00	5,06	8370	0,87	2,10	15,84	21,63	1,32	-147,61	1,32
2,00	5,06	8870	0,71	1,80	18,98	20,14	0,98	-109,07	0,98
2,00	5,06	9370	0,51	1,30	19,99	17,39	0,46	-51,07	0,46
2,00	5,06	9870	0,40	1,20	0,76	16,65	0,01	-1,20	0,01
2,00	5,06	10370	0,55	1,35	-22,90	17,69	-0,60	66,38	-0,60
2,00	5,06	10870	0,77	1,90	-23,33	20,67	-1,38	153,59	-1,38
2,00	5,06	11370	0,93	2,30	-20,88	22,48	-2,04	226,83	-2,04
2,00	5,06	11870	1,05	2,55	-17,38	23,53	-2,29	255,40	-2,29
2,00	5,06	12000	1,07	2,60	-16,36	23,75	-2,29	255,26	-2,29
2,00	5,06	14000	1,23	3,05	-7,39	25,38	-1,53	170,95	-1,53
2,00	5,06	16000	1,26	3,10	-4,78	25,55	-1,04	116,07	-1,04
2,00	5,06	18000	1,26	3,1	-4,13	25,55	-0,90	100,57	-0,90


Diameter = 4 km, Distance = 1 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	3000	1,32	3,25	-7,67	26,08	-1,9	100,30	-1,86
2,00	5,06	5000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	7000	1,23	3,05	-5,97	25,38	-1,24	66,58	-1,24
2,00	5,06	7620	0,87	2,1	-2,69	21,63	-0,24	12,72	-0,24
2,00	5,06	8120	0,42	1,2	-0,02	17,39	0,00	0,02	0,00
2,00	5,06	8620	0,11	0,65	1,75	12,34	0,00	-0,07	0,00
2,00	5,06	9120	0	0	0	0,01	0,00	0,00	0,00
2,00	5,06	9620	0	0	0	0,01	0,00	0,00	0,00
2,00	5,06	10120	0,11	0,65	-7,43	12,34	-0,01	0,30	-0,01
2,00	5,06	10620	0,38	1,2	-8,93	16,65	-0,12	6,28	-0,12
2,00	5,06	11120	0,81	1,95	-9,86	20,95	-0,72	38,97	-0,72
2,00	5,06	11620	1,19	2,95	-9,08	25,02	-1,73	93,03	-1,73
2,00	5,06	12000	1,29	3,2	-8,12	25,9	-1,88	101,34	-1,88
2,00	5,06	14000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	16000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	18000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

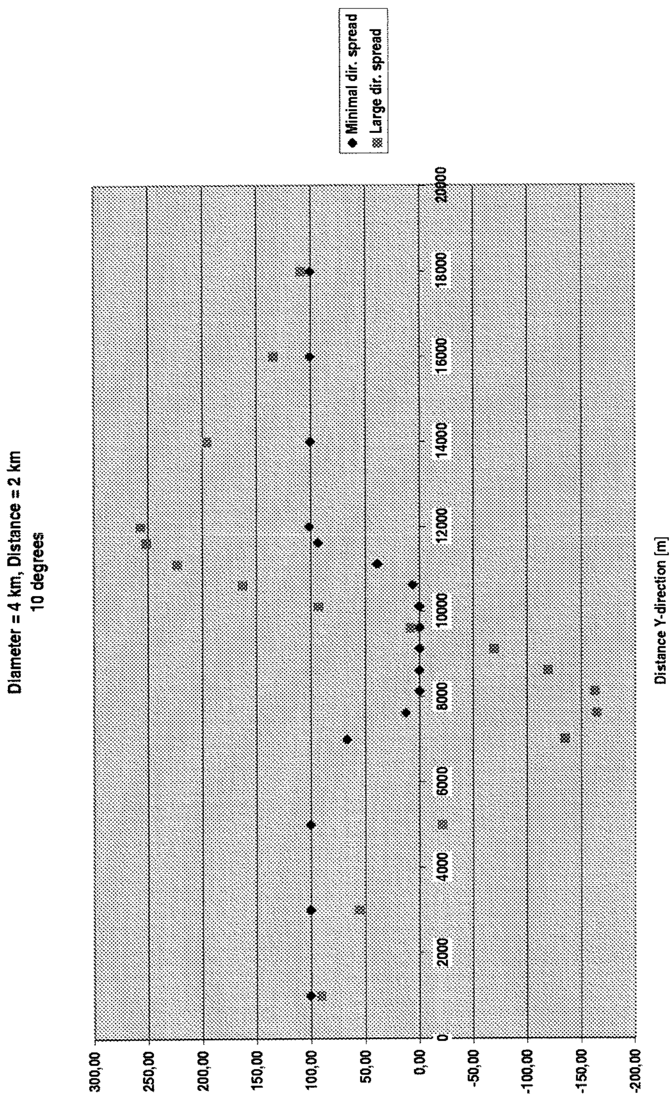
Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,26	3,1	-3,72	25,55	-0,81	90,50	-0,81
2,00	5,06	3000	1,25	3,1	-2,3	25,55	-0,5	55,69	-0,50
2,00	5,06	5000	1,22	3	0,93	25,2	0,19	-21,00	0,19
2,00	5,06	7000	1,09	2,65	7,89	23,95	1,21	-135,13	1,21
2,00	5,06	7620	1,04	2,55	11	23,52	1,48	-164,55	1,48
2,00	5,06	8120	0,96	2,35	13,36	22,79	1,46	-162,99	1,46
2,00	5,06	8620	0,88	2,15	12,02	21,84	1,07	-119,39	1,07
2,00	5,06	9120	0,81	1,95	8,44	20,94	0,62	-69,63	0,62
2,00	5,06	9620	0,8	1,95	-1,06	20,94	-0,08	8,50	-0,08
2,00	5,06	10120	0,81	1,95	-11,48	20,94	-0,83	92,85	-0,83
2,00	5,06	10620	0,87	2,1	-17,53	21,63	-1,46	162,95	-1,46
2,00	5,06	11120	0,96	2,35	-19,15	22,69	-2,00	223,21	-2,00
2,00	5,06	11620	1,03	2,5	-18,11	23,32	-2,26	251,43	-2,26
2,00	5,06	12000	1,07	2,6	-16,4	23,75	-2,30	256,29	-2,30
2,00	5,06	14000	1,22	3	-8,79	25,2	-1,75	195,46	-1,75
2,00	5,06	16000	1,25	3,1	-5,56	25,55	-1,20	134,15	-1,20
2,00	5,06	18000	1,26	3,1	-4,46	25,55	-0,97	108,54	-0,97



Diameter = 4 km, Distance = 2 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore:5 km

Directional spread : 5.7 degrees

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	3000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	5000	1,32	3,25	-7,51	26,08	-1,83	98,26	-1,83
2,00	5,06	7000	1,01	2,5	-3,45	23,32	-0,45	23,95	-0,45
2,00	5,06	7100	0,97	2,40	-3,08	22,9	-0,36	19,22	-0,36
2,00	5,06	7600	0,7	1,80	-1,19	20,14	-0,06	3,45	-0,06
2,00	5,06	8100	0,43	1,25	0,33	17,02	0,01	-0,31	0,01
2,00	5,06	8600	0,23	0,65	-0,27	12,34	0,00	0,05	0,00
2,00	5,06	9100	0,23	0,65	-7,37	12,34	-0,03	1,38	-0,03
2,00	5,06	9600	0,42	1,20	-9,56	16,65	-0,15	8,10	-0,15
2,00	5,06	10100	0,68	1,75	-10,27	19,88	-0,50	26,95	-0,50
2,00	5,06	10600	0,95	2,35	-10,15	22,69	-1,10	59,30	-1,10
2,00	5,06	11100	1,14	2,80	-9,45	24,5	-1,62	87,05	-1,62
2,00	5,06	12000	1,3	3,2	-8,09	25,9	-1,89	101,45	-1,89
2,00	5,06	14000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	16000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86
2,00	5,06	18000	1,32	3,25	-7,67	26,08	-1,86	100,30	-1,86

Changes in net yearly longshore sediment transports

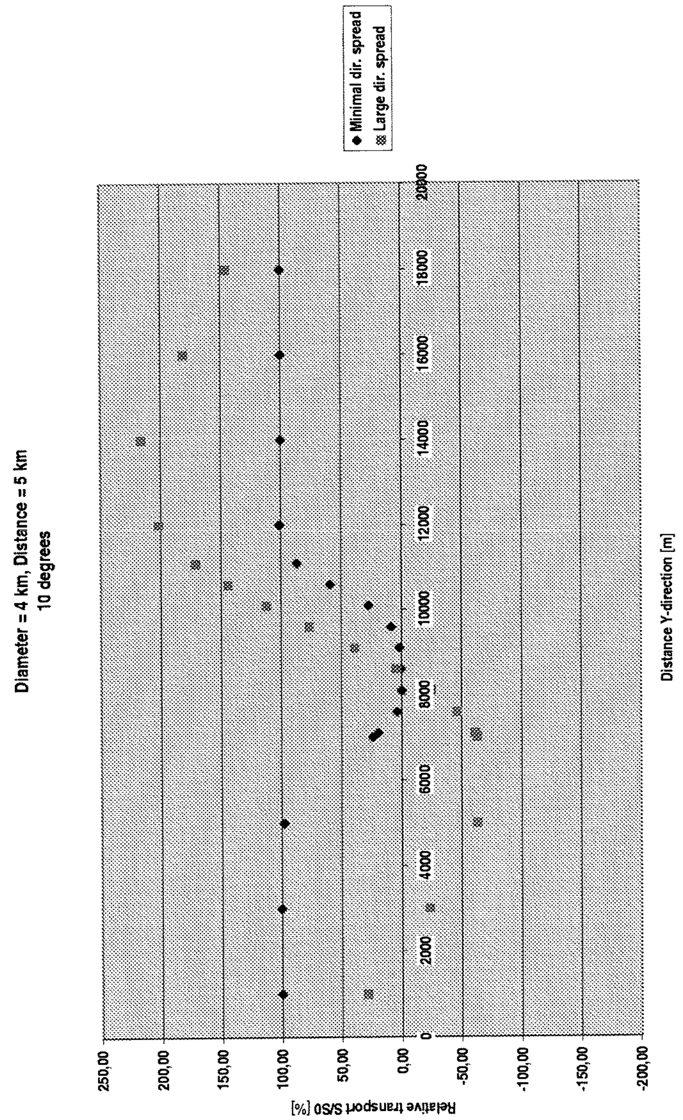
Diameter island: 4 km; φ_0 = 10 degrees

Distance offshore:5 km

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,24	3,05	-1,21	25,38	-0,26	28,48	-0,26
2,00	5,06	3000	1,22	3	1,03	25,2	0,21	-23,26	0,21
2,00	5,06	5000	1,18	2,9	3,04	24,85	0,57	-63,43	0,57
2,00	5,06	7000	1,11	2,7	3,5	24,15	0,57	-62,98	0,57
2,00	5,06	7100	1,11	2,70	3,41	24,15	0,55	-61,26	0,55
2,00	5,06	7600	1,09	2,65	2,68	23,95	0,41	-46,25	0,41
2,00	5,06	8100	1,08	2,65	1,47	23,95	0,22	-25,07	0,22
2,00	5,06	8600	1,07	2,60	-0,25	23,75	-0,04	4,11	-0,04
2,00	5,06	9100	1,06	2,60	-2,39	23,75	-0,35	38,91	-0,35
2,00	5,06	9600	1,07	2,60	-4,73	23,75	-0,69	76,89	-0,69
2,00	5,06	10100	1,06	2,60	-6,96	23,75	-1,01	112,33	-1,01
2,00	5,06	10600	1,07	2,65	-8,88	23,95	-1,30	144,45	-1,30
2,00	5,06	11100	1,08	2,65	-10,29	23,95	-1,53	170,86	-1,53
2,00	5,06	12000	1,11	2,7	-11,44	24,15	-1,81	202,01	-1,81
2,00	5,06	14000	1,19	2,95	-10,22	25,02	-1,94	216,34	-1,94
2,00	5,06	16000	1,23	3,05	-7,84	25,38	-1,62	181,11	-1,62
2,00	5,06	18000	1,25	3,1	-6,07	25,55	-1,31	146,04	-1,31


Diameter = 4 km, Distance = 5 km
10 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 1 km; $\varphi_0 = 45$ degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	4000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	6000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	8000	1,09	2,65	-30,09	23,95	-3,85	83,09	-3,85
2,00	5,06	8980	0,24	0,65	-16,02	12,34	-0,06	1,30	-0,06
2,00	5,06	9105	0,30	0,01	-19,93	15,17	-0,14	2,96	-0,14
2,00	5,06	9230	0,43	1,25	-21,96	17,02	-0,34	7,31	-0,34
2,00	5,06	9355	0,56	1,40	-24,92	17,99	-0,67	14,56	-0,67
2,00	5,06	9480	0,73	1,85	-27,25	20,41	-1,36	29,40	-1,36
2,00	5,06	9605	0,88	2,15	-29,37	21,84	-2,24	48,33	-2,24
2,00	5,06	9730	1,02	2,50	-30,94	23,32	-3,33	71,88	-3,33
2,00	5,06	9855	1,09	2,65	-31,68	23,95	-3,98	85,92	-3,98
2,00	5,06	9980	1,13	2,75	31,85	24,32	-4,34	93,75	-4,34
2,00	5,06	10000	1,15	2,80	-31,92	24,50	-4,49	97,04	-4,49
2,00	5,06	12000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	14000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	16000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	18000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63

Changes in net yearly longshore sediment transports

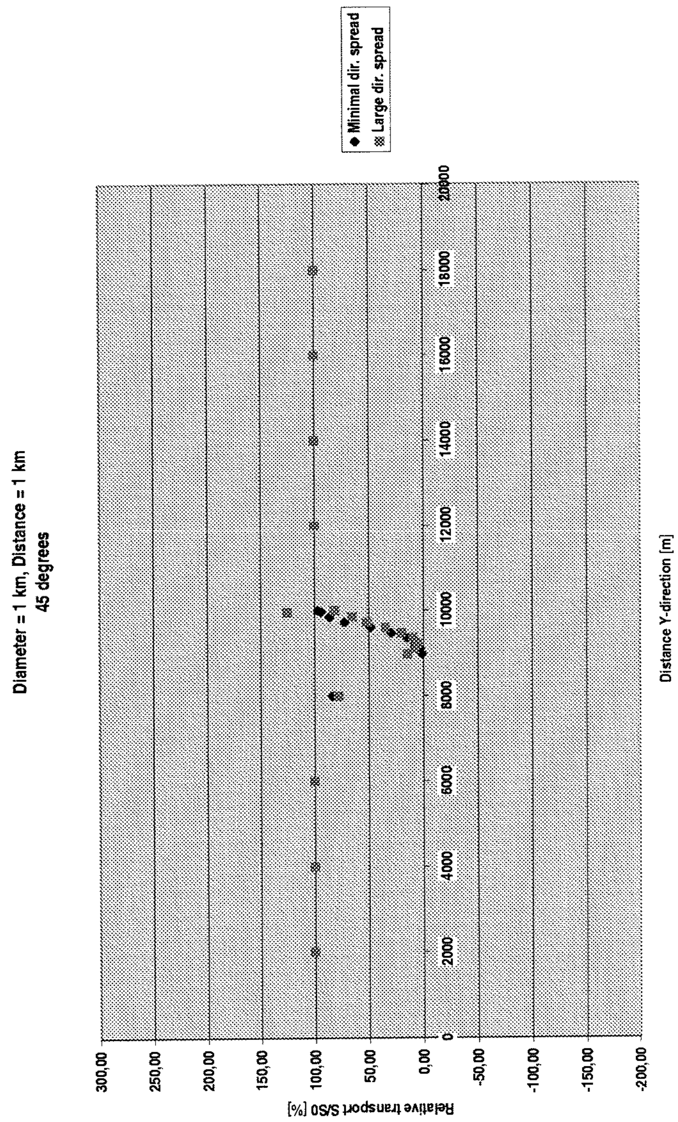
Diameter island: 1 km; $\varphi_0 = 45$ degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,24	3,05	-17,49	25,39	-3,46	100,04	-3,46
2,00	5,06	4000	1,24	3,05	-17,49	25,39	-3,46	100,04	-3,46
2,00	5,06	6000	1,24	3,05	-17,49	25,39	-3,46	100,04	-3,46
2,00	5,06	8000	1,18	2,9	-14,85	24,85	-2,69	77,77	-2,69
2,00	5,06	8980	0,99	2,45	-3,93	23,11	-0,48	13,99	-0,48
2,00	5,06	9105	0,95	2,35	-2,53	22,69	-0,28	8,07	-0,28
2,00	5,06	9230	0,89	2,20	-2,17	22,05	-0,21	6,00	-0,21
2,00	5,06	9355	0,85	2,05	-3,96	21,42	-0,33	9,63	-0,33
2,00	5,06	9480	0,84	2,00	-8,62	21,21	-0,68	19,80	-0,68
2,00	5,06	9605	0,84	2,00	-15,37	21,21	-1,19	34,55	-1,19
2,00	5,06	9730	0,88	2,15	-21,28	21,84	-1,78	51,55	-1,78
2,00	5,06	9855	0,93	2,30	-24,71	21,84	-2,25	65,20	-2,25
2,00	5,06	9980	0,98	2,40	-25,96	22,90	-4,34	125,51	-4,34
2,00	5,06	10000	1,00	2,45	-26,15	23,11	-2,82	81,57	-2,82
2,00	5,06	12000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	14000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	16000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 1 km, Distance = 1 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Distance Y-direction [m]

Changes in net yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	4000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	6000	1,15	2,8	-31,7	24,5	-4,49	97,01	-4,49
2,00	5,06	8000	0,55	1,35	-21,43	17,69	-0,56	12,12	-0,56
2,00	5,06	8740	0,99	2,45	-30,79	23,11	-3,09	66,78	-3,09
2,00	5,06	8865	1,04	2,55	-31,36	23,53	-3,54	76,57	-3,54
2,00	5,06	8990	1,09	2,65	-31,7	23,95	-3,94	85,01	-3,94
2,00	5,06	9115	1,11	2,7	-31,84	24,15	-4,16	89,77	-4,16
2,00	5,06	9240	1,13	2,75	-31,88	23,32	-4,15	89,74	-4,15
2,00	5,06	9365	1,15	2,8	-31,88	24,5	-4,49	96,97	-4,49
2,00	5,06	9490	1,15	22,8	-31,82	24,5	-4,50	97,21	-4,50
2,00	5,06	9615	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	9740	1,16	2,85	-31,83	24,68	-4,64	100,17	-4,64
2,00	5,06	10000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	12000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	14000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	16000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	18000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63

Changes in net yearly longshore sediment transports

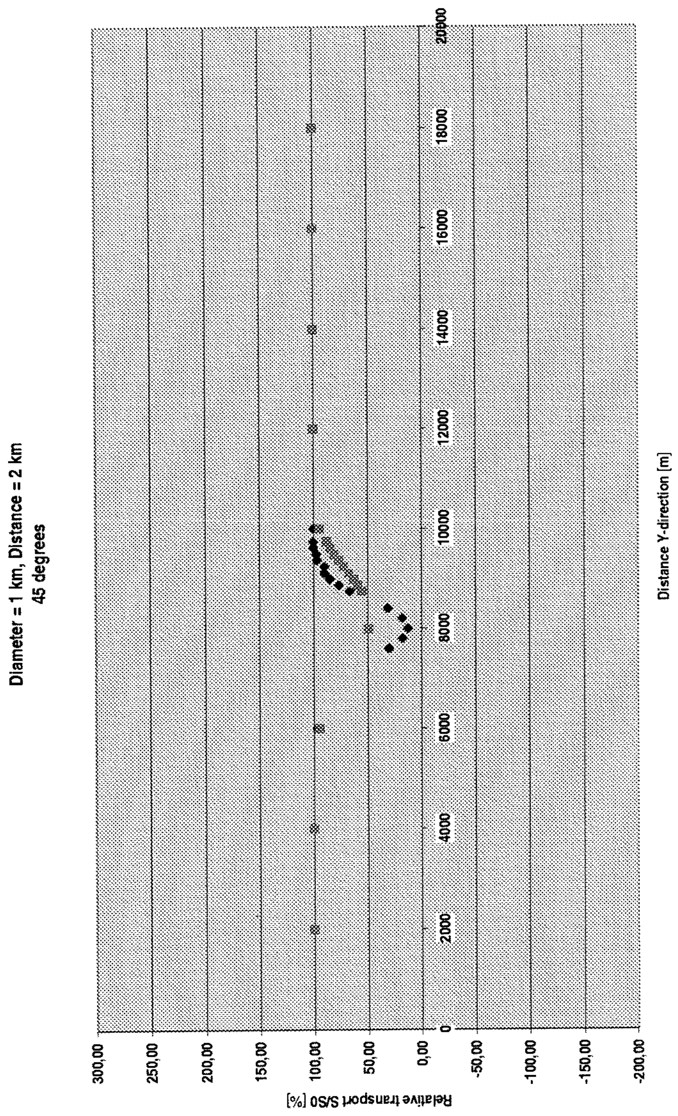
Diameter island: 1 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	4000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	6000	1,22	3,00	-16,98	25,2	-3,26	94,41	-3,26
2,00	5,06	8000	1,13	2,75	-10,43	24,32	-1,71	49,56	-1,71
2,00	5,06	8740	1,09	2,65	-12,85	23,95	-1,92	55,50	-1,92
2,00	5,06	8865	1,09	2,65	-13,8	23,95	-2,05	59,19	-2,05
2,00	5,06	8990	1,09	2,65	-14,87	23,95	-2,19	63,26	-2,19
2,00	5,06	9115	1,09	2,65	-16,03	23,95	-2,34	67,56	-2,34
2,00	5,06	9240	1,09	2,65	-17,25	23,95	-2,50	72,23	-2,50
2,00	5,06	9365	1,09	2,65	-18,45	23,95	-2,65	76,70	-2,65
2,00	5,06	9490	1,09	2,65	-19,61	23,95	-2,80	80,93	-2,80
2,00	5,06	9615	1,09	2,65	-20,63	23,95	-2,93	84,71	-2,93
2,00	5,06	9740	1,1	2,65	-21,39	23,95	-3,04	87,89	-3,04
2,00	5,06	10000	1,11	2,7	-22,16	24,15	-3,26	94,35	-3,26
2,00	5,06	12000	1,23	3,05	-17,6	25,38	-3,47	100,39	-3,47
2,00	5,06	14000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	16000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 1 km, Distance = 2 km
45 degrees
Relative transport S/S0 [%]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
Distance Y-direction [m]
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 1 km; $\varphi_0 = 45$ degrees

Distance offshore: 5 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,15	2,8	-31,73	24,5	-4,50	97,23	-4,50
2,00	5,06	3000	1,13	2,75	-30,6	24,32	-4,21	90,95	-4,21
2,00	5,06	5000	0,97	2,4	-28,16	22,9	-2,80	60,47	-2,80
2,00	5,06	7000	1,13	2,75	-31,91	24,32	-4,33	93,47	-4,33
2,00	5,06	7200	1,14	2,8	-31,93	24,5	-4,48	96,75	-4,48
2,00	5,06	7325	1,14	2,8	-31,9	24,5	-4,51	97,12	-4,50
2,00	5,06	8075	1,15	2,8	-31,83	24,5	-4,60	99,27	-4,60
2,00	5,06	8200	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	9000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	11000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	13000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	15000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	17000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	4600	1	2,45	-27,86	23,11	-2,96	64,02	-2,96
2,00	5,06	4800	0,98	2,4	-27,85	22,9	-2,81	60,78	-2,81
2,00	5,06	5200	0,96	2,35	-28,61	22,69	-2,75	59,30	-2,75
2,00	5,06	5400	0,98	2,4	-29,29	22,9	-2,89	62,53	-2,89

Changes in net yearly longshore sediment transports

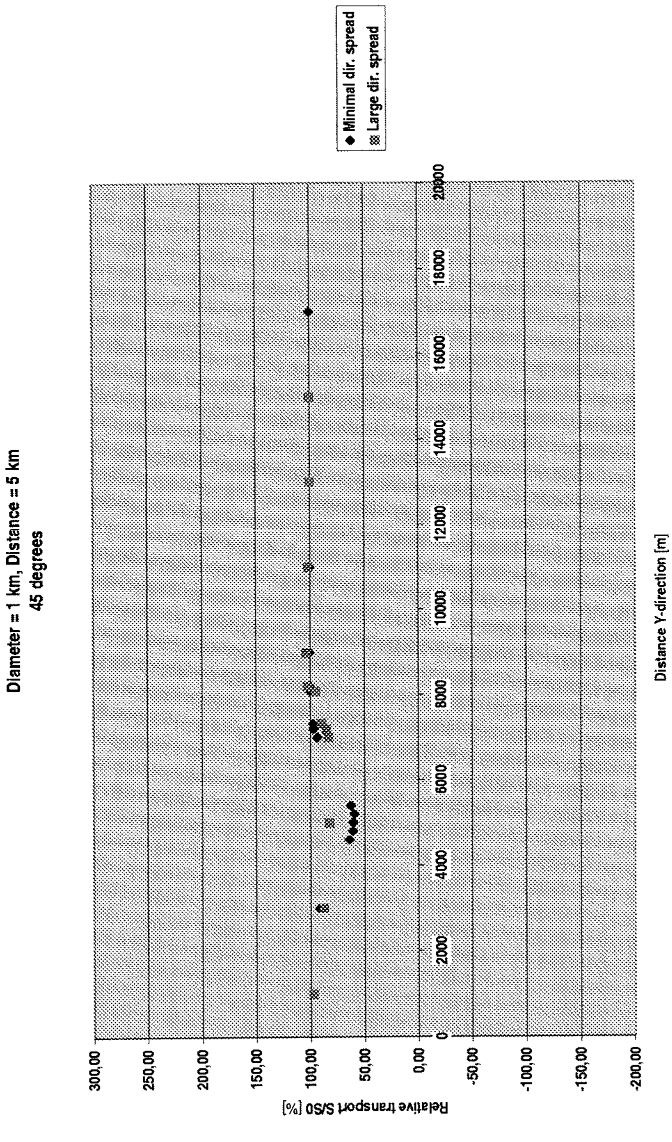
Diameter island: 1 km; $\varphi_0 = 45$ degrees

Distance offshore: 5 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,23	3,05	-17,09	25,38	-3,37	97,52	-3,37
2,00	5,06	3000	1,22	3,00	-15,62	25,2	-3,02	87,22	-3,02
2,00	5,06	5000	1,20	2,95	-15,16	25,03	-2,84	81,99	-2,84
2,00	5,06	7000	1,18	2,9	-16,08	24,85	-2,88	83,27	-2,88
2,00	5,06	7200	1,18	2,9	-16,29	24,85	-2,94	84,93	-2,94
2,00	5,06	7325	1,18	2,9	-16,44	24,85	-3,11	89,22	-3,09
2,00	5,06	8200	1,18	2,90	-17,59	24,85	-3,28	94,72	-3,28
2,00	5,06	9000	1,18	2,90	-18,63	24,85	-3,53	102,19	-3,53
2,00	5,06	11000	1,21	3,00	-19,13	25,2	-3,58	103,63	-3,58
2,00	5,06	13000	1,23	3,05	-17,9	25,38	-3,52	101,70	-3,52
2,00	5,06	15000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	17000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 1 km, Distance = 5 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Distance Y-direction [m]

Changes in net yearly longshore sediment transports

Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	4000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	6000	1,15	2,8	-31,59	24,5	-4,47	96,65	-4,47
2,00	5,06	8000	0,22	0,65	-11,08	12,35	-0,03	0,72	-0,03
2,00	5,06	8480	0,04	0,60	-9,67	11,78	0,00	0,02	0,00
2,00	5,06	8730	0,00	0,00	0,00	0,01	0,00	0,00	0,00
2,00	5,06	8980	0,06	0,65	-15,79	12,35	0,00	0,08	0,00
2,00	5,06	9230	0,21	0,65	-17,06	12,35	-0,05	0,99	-0,05
2,00	5,06	9480	0,44	1,25	-21,83	17,03	-0,35	7,52	-0,35
2,00	5,06	9730	0,74	1,85	-27,19	20,40	-1,40	30,15	-1,40
2,00	5,06	9980	1,02	2,50	-30,81	23,32	-3,30	71,29	-3,30
2,00	5,06	10230	1,13	2,75	-31,82	24,32	-4,33	93,49	-4,33
2,00	5,06	10480	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	11000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	13000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	15000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	17000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	19000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	7000	1,00	2,45	-27,37	23,11	-2,96	63,90	-2,96

Changes in net yearly longshore sediment transports

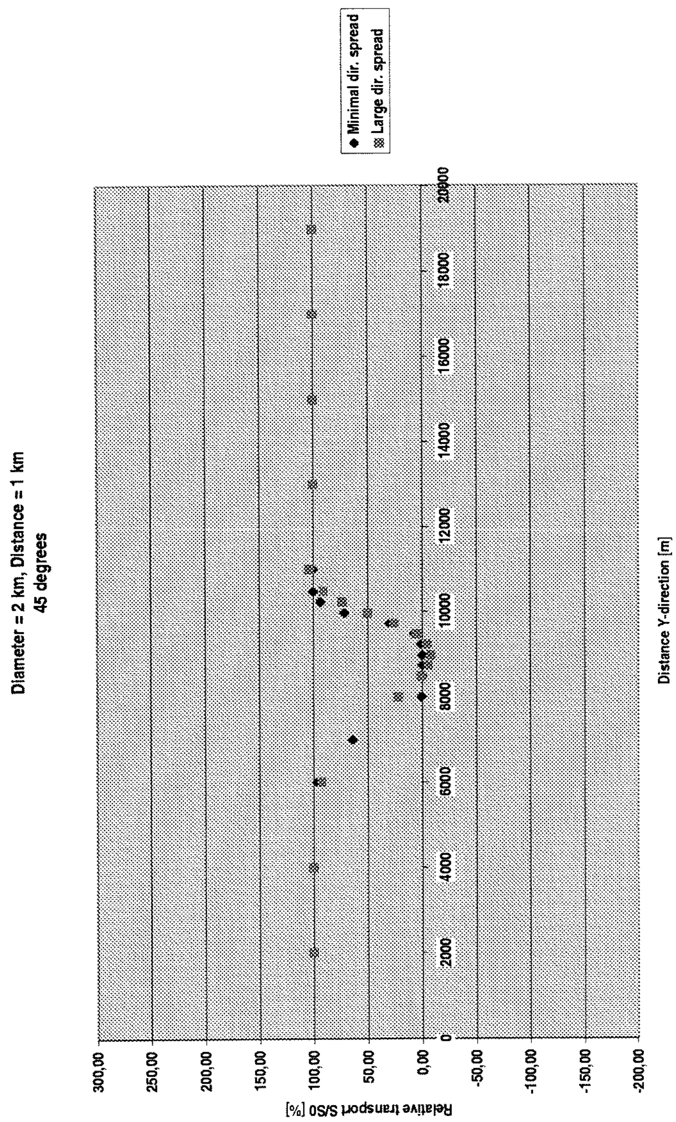
Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	4000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	6000	1,22	3,00	-16,6	25,2	-3,19	92,24	-3,19
2,00	5,06	8000	1,04	2,55	-5,72	23,53	-0,79	22,88	-0,79
2,00	5,06	8480	0,90	2,20	-0,52	22,05	-0,05	1,46	-0,05
2,00	5,06	8730	0,79	1,95	2,57	20,95	0,18	-5,32	0,18
2,00	5,06	8980	0,66	1,70	5,72	19,60	0,26	-7,63	0,26
2,00	5,06	9230	0,53	1,30	5,58	17,39	0,15	-4,21	0,15
2,00	5,06	9480	0,51	1,30	-7,73	17,39	-0,18	5,35	-0,18
2,00	5,06	9730	0,64	1,65	-23,46	19,30	-0,90	25,94	-0,90
2,00	5,06	9980	0,80	1,95	-27,55	20,94	-1,73	49,90	-1,73
2,00	5,06	10230	0,94	2,30	-27,64	22,48	-2,53	73,26	-2,53
2,00	5,06	10480	1,04	2,55	-26,14	23,53	-3,14	90,71	-3,14
2,00	5,06	11000	1,16	2,85	-21,67	24,78	-3,57	103,30	-3,57
2,00	5,06	13000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	15000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46



Diameter = 2 km, Distance = 1 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
Distance Y-direction [m]

Changes in net yearly longshore sediment transports

Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	3000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	5000	1,13	2,75	-31,06	24,32	-4,28	92,39	-4,28
2,00	5,06	7000	0,42	1,2	-15,89	16,65	-0,24	5,21	-0,24
2,00	5,06	7800	0,21	0,65	-17,28	12,35	-0,05	1,08	-0,05
2,00	5,06	8050	0,34	1,1	-21,73	15,91	-0,19	4,18	-0,19
2,00	5,06	8300	0,50	1,3	-24,5	17,39	-0,52	11,18	-0,52
2,00	5,06	8550	0,70	1,8	-27,41	20,14	-1,25	26,99	-1,25
2,00	5,06	8800	0,88	2,15	-29,66	21,84	-2,25	48,51	-2,25
2,00	5,06	9050	1,02	2,5	-31,22	23,32	-3,37	72,82	-3,37
2,00	5,06	9300	1,09	2,65	-31,79	23,95	-4,00	86,40	-4,00
2,00	5,06	9550	1,13	2,75	-31,86	24,32	-4,34	93,72	-4,34
2,00	5,06	9800	1,16	2,85	-31,86	24,68	-4,62	99,88	-4,62
2,00	5,06	10000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	12000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	14000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	16000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	18000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	6000	1,00	2,45	-27,03	23,11	-2,92	62,98	-2,92

Changes in net yearly longshore sediment transports

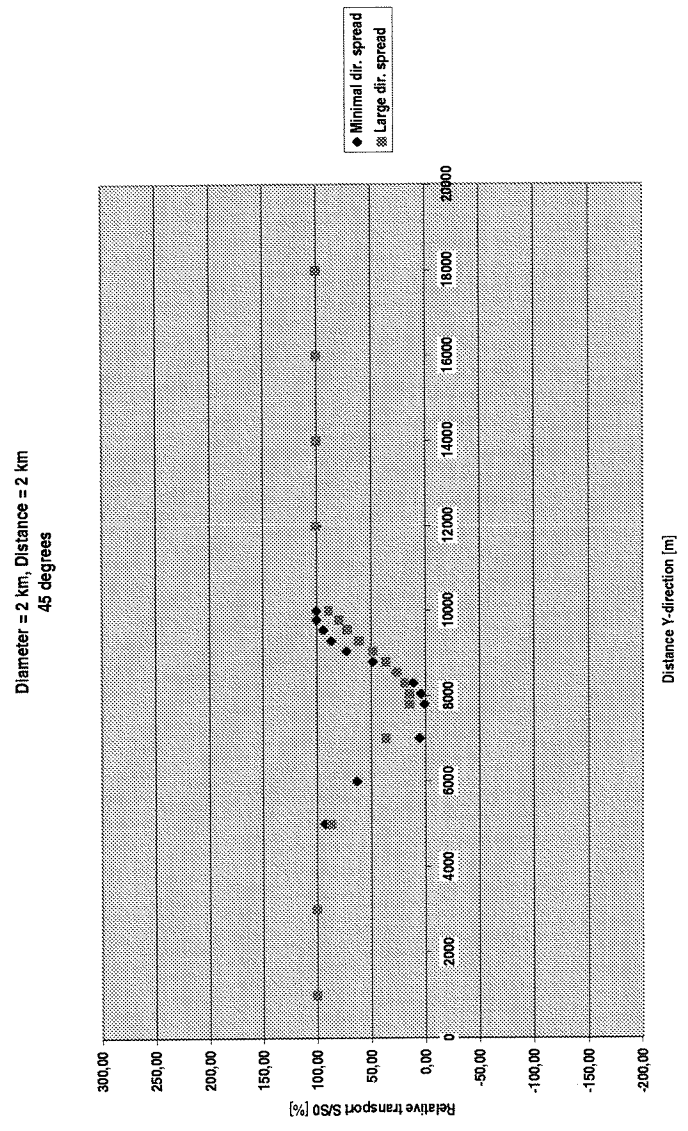
Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	3000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	5000	1,22	3,00	-15,56	25,20	-3,00	86,64	-3,00
2,00	5,06	7000	1,09	2,65	-8,22	23,95	-1,26	36,49	-1,26
2,00	5,06	7800	1,00	2,45	-4,10	23,11	-0,51	14,85	-0,51
2,00	5,06	8050	0,98	2,40	-4,27	22,90	-0,50	14,57	-0,50
2,00	5,06	8300	0,95	2,35	-5,97	22,69	-0,67	19,25	-0,67
2,00	5,06	8550	0,93	2,30	-8,63	22,48	-0,91	26,22	-0,91
2,00	5,06	8800	0,93	2,30	-12,33	22,48	-1,26	36,40	-1,26
2,00	5,06	9050	0,93	2,30	-16,7	22,48	-1,67	48,23	-1,67
2,00	5,06	9300	0,95	2,35	-20,61	22,69	-2,11	60,93	-2,11
2,00	5,06	9550	0,98	2,4	-23,33	22,90	-2,49	71,93	-2,49
2,00	5,06	9800	1,02	2,5	-24,83	22,32	-2,75	79,45	-2,75
2,00	5,06	10000	1,04	2,55	-25,22	23,53	-3,07	88,92	-3,07
2,00	5,06	12000	1,22	3,00	-18,43	25,20	-3,49	101,05	-3,49
2,00	5,06	14000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 2 km, Distance = 2 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 5 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,13	2,75	-31,05	24,32	-4,29	92,70	-4,29
2,00	5,06	3000	1,00	2,45	-26,9	23,11	-2,91	62,89	-2,91
2,00	5,06	5000	0,73	1,8	-26,56	20,14	-1,33	28,66	-1,33
2,00	5,06	6210	1,02	2,50	-31,35	23,32	-3,37	72,71	-3,37
2,00	5,06	6460	1,06	2,60	-31,68	23,75	-3,74	80,89	-3,74
2,00	5,06	6710	1,09	2,65	-31,85	23,95	-3,97	85,70	-3,97
2,00	5,06	6960	1,11	2,70	-31,92	24,15	-4,16	89,89	-4,16
2,00	5,06	7210	1,13	2,75	-31,94	24,32	-4,32	93,35	-4,32
2,00	5,06	7460	1,14	2,80	-31,94	24,5	-4,48	96,73	-4,48
2,00	5,06	7710	1,15	2,80	-31,87	24,5	-4,55	97,12	-4,50
2,00	5,06	8210	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	9000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	11000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	4600	0,70	1,8	-24,3	20,14	-1,17	25,20	-1,17
2,00	5,06	4800	0,71	1,8	-25,38	20,14	-1,22	26,32	-1,22
2,00	5,06	5200	0,77	1,9	-27,7	20,67	-1,57	33,95	-1,57
2,00	5,06	5400	0,81	1,95	-28,65	20,94	-1,80	38,82	-1,80
2,00	5,06	3600	0,87	2,1	-24,45	21,63	-1,91	41,24	-1,91
2,00	5,06	3800	0,81	1,95	-23,71	20,94	-1,58	34,13	-1,58
2,00	5,06	4000	0,78	1,9	-23,26	20,67	-1,41	30,40	-1,41

Changes in net yearly longshore sediment transports

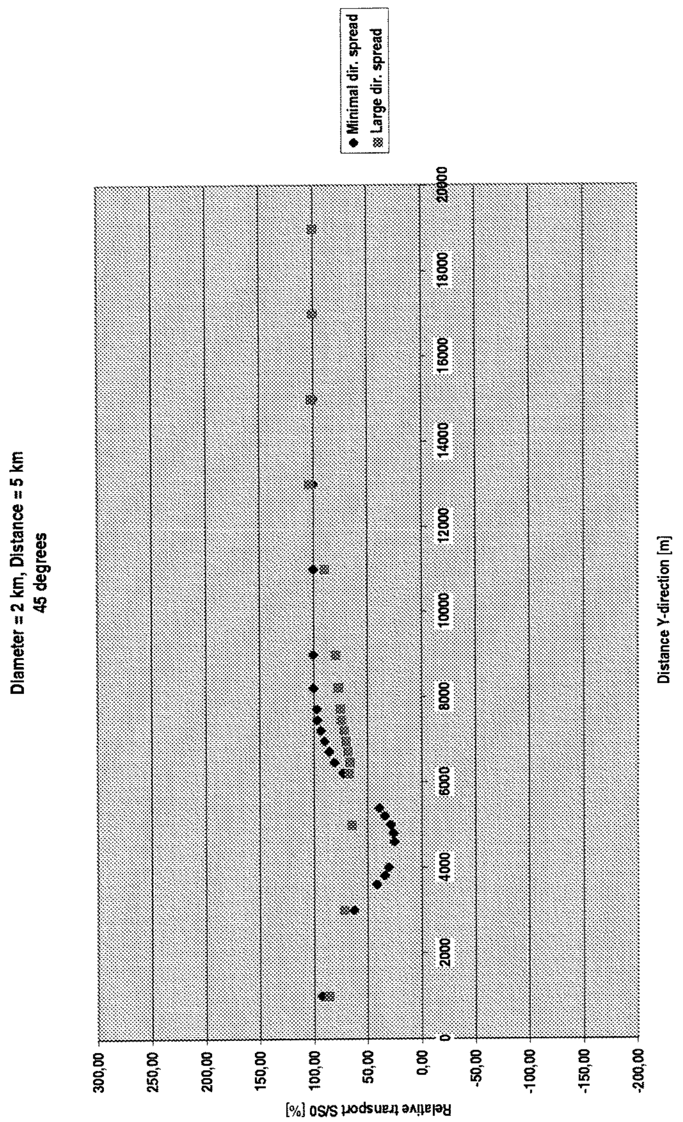
Diameter island: 2 km; φ_0 = 45 degrees

Distance offshore: 5 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	1000	1,22	3,00	-15,44	25,2	-2,98	86,18	-2,98
2,00	5,06	3000	1,19	2,95	-13,28	25,03	-2,49	71,90	-2,49
2,00	5,06	5000	1,16	2,85	-12,83	24,68	-2,24	64,80	-2,24
2,00	5,06	6210	1,14	2,80	-13,84	24,5	-2,31	66,88	-2,31
2,00	5,06	6460	1,13	2,75	-14,15	24,32	-2,29	66,34	-2,29
2,00	5,06	6710	1,13	2,75	-14,58	24,32	-2,35	67,94	-2,35
2,00	5,06	6960	1,13	2,75	-15,07	24,32	-2,42	69,89	-2,42
2,00	5,06	7210	1,13	2,75	-15,62	24,32	-2,49	72,06	-2,49
2,00	5,06	7460	1,13	2,75	-16,22	24,32	-2,57	74,40	-2,57
2,00	5,06	7710	1,12	2,70	-16,79	24,15	-2,59	74,83	-2,59
2,00	5,06	8210	1,12	2,70	-17,44	24,15	-2,68	77,37	-2,68
2,00	5,06	9000	1,12	2,70	-18,09	24,15	-2,76	79,87	-2,76
2,00	5,06	11000	1,13	2,75	-20,00	24,32	-3,10	89,63	-3,10
2,00	5,06	13000	1,18	2,90	-20,56	24,85	-3,56	103,06	-3,56
2,00	5,06	15000	1,22	3,00	-18,46	25,2	-3,50	101,36	-3,50


Diameter = 2 km, Distance = 5 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 4 km; $\varphi_0 = 45$ degrees

Distance offshore: 1 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [°]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	3000	1,145	2,8	-31,43	24,5	-4,45	96,21	-4,45
2,00	5,06	5000	0,83	2	-23,53	21,21	-1,67	36,00	-1,67
2,00	5,06	7000	0	0	0	0	0,00	0,00	0,00
2,00	5,06	7470	0,06	0,65	-14,30	12,32	0,00	0,06	0,00
2,00	5,06	7970	0,08	0,65	-14,12	12,32	-0,01	0,13	-0,01
2,00	5,06	8470	0,00	0,00	0,00	0,01	0,00	0,00	0,00
2,00	5,06	8970	0,00	0,00	0,00	0,01	0,00	0,00	0,00
2,00	5,06	9470	0,15	0,65	-15,85	12,32	-0,02	0,48	-0,02
2,00	5,06	9970	0,56	1,40	-24,93	17,99	-0,66	14,26	-0,66
2,00	5,06	10470	1,04	2,55	-31,32	23,53	-3,53	76,22	-3,53
2,00	5,06	10970	1,15	2,80	-31,83	24,50	-4,50	97,23	-4,50
2,00	5,06	11470	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	12000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	14000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	16000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	18000	1,16	2,85	-31,83	24,68	-4,63	100,00	-4,63

Changes in net yearly longshore sediment transports

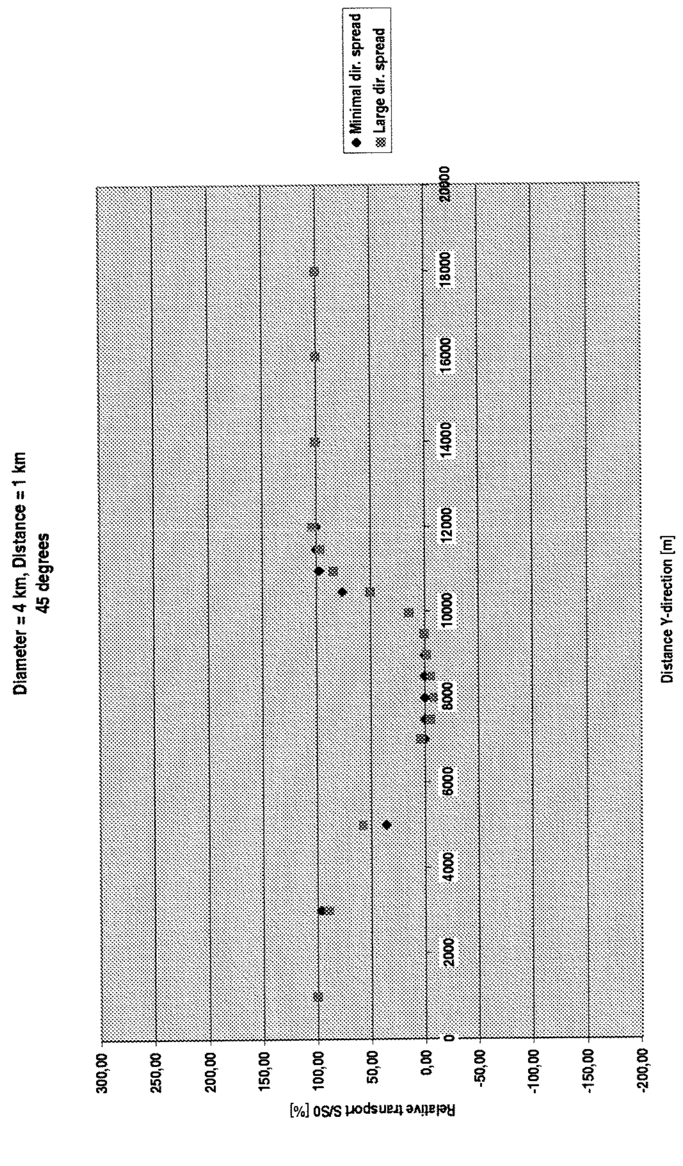
Diameter island: 4 km; $\varphi_0 = 45$ degrees

Distance offshore: 1 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [°]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	3000	1,22	3,00	-16,00	25,2	-3,08	89,12	-3,08
2,00	5,06	5000	1,16	2,85	-11,35	24,68	-2,00	57,75	-2,00
2,00	5,06	7000	0,94	2,30	-1,29	22,48	-0,14	3,99	-0,14
2,00	5,06	7470	0,83	2,00	1,98	21,21	0,16	-4,49	0,16
2,00	5,06	7970	0,68	1,75	5,62	19,87	0,28	-8,14	0,28
2,00	5,06	8470	0,49	1,30	8,89	17,39	0,20	-5,77	0,20
2,00	5,06	8970	0,28	0,70	10,66	12,95	0,06	-1,60	0,06
2,00	5,06	9470	0,20	0,65	-9,29	12,34	-0,02	0,72	-0,02
2,00	5,06	9970	0,50	1,30	-25,08	17,39	-0,52	15,10	-0,52
2,00	5,06	10470	0,80	1,95	-28,12	20,94	-1,75	50,58	-1,75
2,00	5,06	10970	1,00	2,45	-27,24	23,11	-2,91	84,25	-2,91
2,00	5,06	11470	1,09	2,65	-24,51	23,95	-3,35	96,98	-3,35
2,00	5,06	12000	1,16	2,85	-21,60	24,68	-3,57	103,15	-3,57
2,00	5,06	14000	1,23	3,05	-17,65	25,38	-3,48	100,63	-3,48
2,00	5,06	16000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46
2,00	5,06	18000	1,24	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 4 km, Distance = 1 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

Changes in net yearly longshore sediment transports

Diameter island: 4 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,147	2,80	-31,64	24,5	-4,49	96,90	-4,49
2,00	5,06	3000	1,085	2,65	-29,11	23,95	-3,73	80,67	-3,73
2,00	5,06	5000	0,445	1,25	-16,03	17,01	-0,28	6,02	-0,28
2,00	5,06	6400	0,028	0,42	-8,27	9,04	0,00	0,01	0,00
2,00	5,06	6900	0	0	0	0	0,00	0,00	0,00
2,00	5,06	7400	0,023	0,14	-14,22	4,91	0,00	0,00	0,00
2,00	5,06	7900	0,114	0,65	-16,45	12,32	-0,01	0,29	-0,01
2,00	5,06	8400	0,325	1,00	-21,65	15,17	-0,17	3,70	-0,17
2,00	5,06	8900	0,666	1,70	-27,11	19,6	-1,10	23,74	-1,10
2,00	5,06	9400	0,991	2,45	-30,95	23,11	-3,12	67,39	-3,12
2,00	5,06	9900	1,113	2,7	-31,87	24,15	-4,18	90,30	-4,18
2,00	5,06	10400	1,158	2,85	-31,87	24,68	-4,62	99,90	-4,62
2,00	5,06	11000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	13000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	15000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	17000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	19000	1,159	2,85	-31,83	24,68	-4,63	100,00	-4,63

Changes in net yearly longshore sediment transports

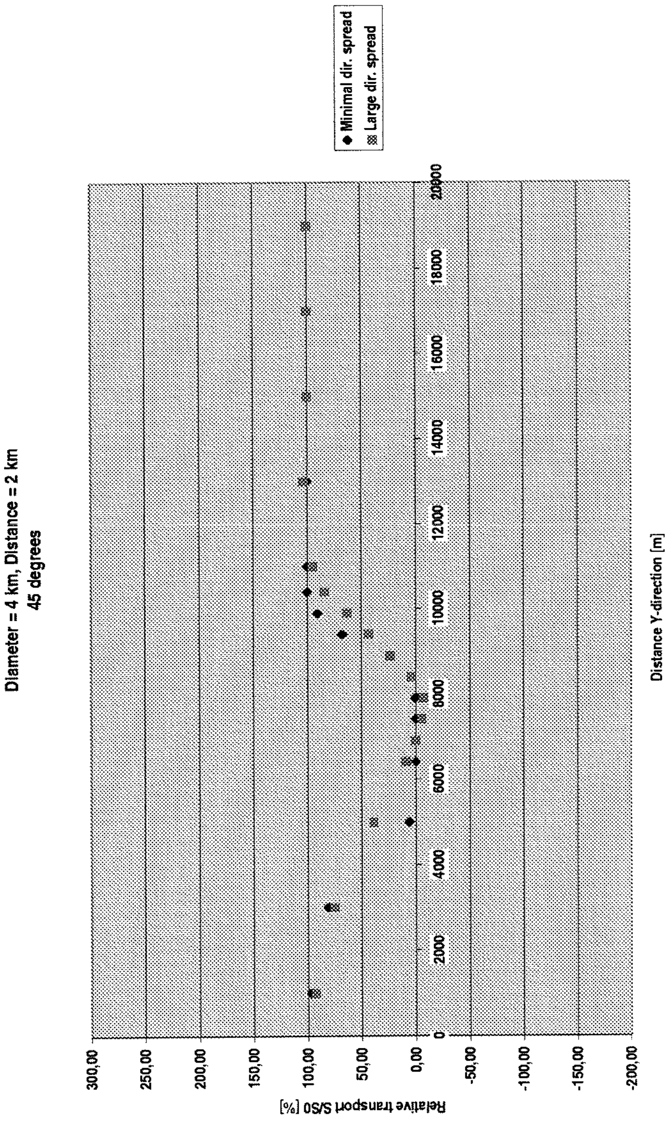
Diameter island: 4 km; φ_0 = 45 degrees

Distance offshore: 2 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	1000	1,22	3,00	-16,60	25,2	-3,20	92,55	-3,20
2,00	5,06	3000	1,20	2,95	-13,81	25,04	-2,59	74,82	-2,59
2,00	5,06	5000	1,11	2,70	-8,50	24,15	-1,35	38,99	-1,35
2,00	5,06	6400	0,98	2,40	-2,55	22,9	-0,30	8,74	-0,30
2,00	5,06	6900	0,90	2,20	0,03	22,05	0,00	-0,08	0,00
2,00	5,06	7400	0,80	1,95	2,73	20,94	0,20	-5,75	0,20
2,00	5,06	7900	0,705	1,80	4,57	20,14	0,25	-7,17	0,25
2,00	5,06	8400	0,645	1,65	-2,61	19,33	-0,11	3,30	-0,11
2,00	5,06	8900	0,69	1,75	-17,03	19,87	-0,83	23,87	-0,83
2,00	5,06	9400	0,78	1,90	-25,24	20,67	-1,49	43,15	-1,49
2,00	5,06	9900	0,88	2,15	-27,76	21,84	-2,17	62,82	-2,17
2,00	5,06	10400	0,99	2,45	-27,48	23,11	-2,90	83,73	-2,90
2,00	5,06	11000	1,07	2,60	-25,47	23,75	-3,29	95,14	-3,29
2,00	5,06	13000	1,22	3,00	-19,08	25,2	-3,59	103,74	-3,59
2,00	5,06	15000	1,23	3,05	-17,56	25,38	-3,46	100,19	-3,46
2,00	5,06	17000	1,24	3,05	-17,59	25,38	-3,47	100,50	-3,47
2,00	5,06	19000	1,24	3,05	-17,59	25,38	-3,47	100,50	-3,47


Diameter = 4 km, Distance = 2 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]

Changes in net yearly longshore sediment transports

Diameter island: 4 km; φ_0 = 45 degrees

Distance offshore: 5 km

Directional spread : 5.7 degrees

S0 =	-4,63

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	0,686	1,75	-20,47	19,88	-0,96	20,63	-0,96
2,00	5,06	4000	0,317	0,9	-20,1	14,43	-0,15	3,15	-0,15
2,00	5,06	4740	0,486	1,25	-24,3	17,02	-0,47	10,15	-0,47
2,00	5,06	5240	0,643	1,65	-26,84	19,34	-1,00	21,68	-1,00
2,00	5,06	5740	0,784	1,90	-28,77	20,67	-1,67	36,08	-1,67
2,00	5,06	6240	0,932	2,30	-30,51	22,48	-2,66	57,49	-2,66
2,00	5,06	6740	1,039	2,55	-31,5	23,53	-3,53	76,18	-3,53
2,00	5,06	7240	1,091	2,65	-31,87	23,95	-3,98	86,05	-3,98
2,00	5,06	7740	1,128	2,75	-31,94	24,32	-4,33	93,52	-4,33
2,00	5,06	8240	1,146	2,80	-31,88	24,5	-4,50	97,14	-4,50
2,00	5,06	8740	1,153	2,80	-31,81	24,5	-4,55	98,21	-4,55
2,00	5,06	9000	1,159	2,853	-31,84	24,68	-4,63	100,02	-4,63
2,00	5,06	17000	1,159	2,853	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	19000	1,159	2,853	-31,83	24,68	-4,63	100,00	-4,63
2,00	5,06	1000	0,92	2,25	-25,08	22,27	-2,26	48,71	-2,26
2,00	5,06	1500	0,806	1,95	-22,83	20,94	-1,52	32,75	-1,52
2,00	5,06	2500	0,527	1,30	-17,61	17,39	-0,43	9,38	-0,43
2,00	5,06	3000	0,409	1,20	-15,52	16,65	-0,22	4,83	-0,22
2,00	5,06	3500	0,302	0,80	-16,38	13,69	-0,11	2,27	-0,11

Changes in net yearly longshore sediment transports

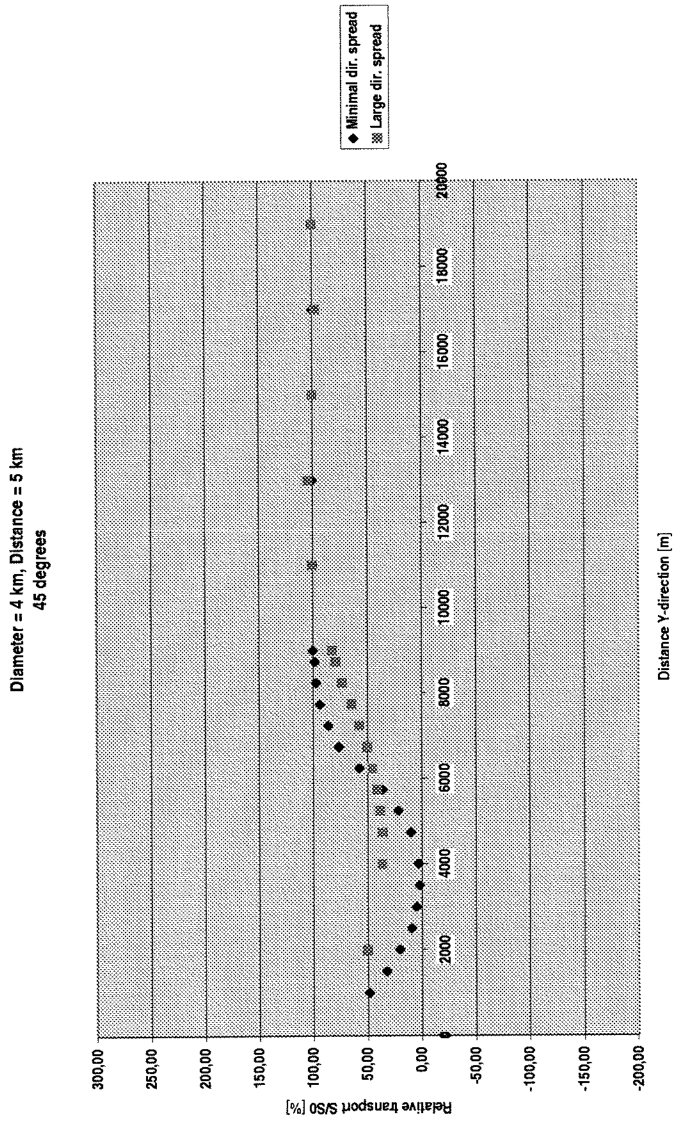
Diameter island: 4 km; φ_0 = 45 degrees

Distance offshore: 5 km

Directional spread : 31.5 degrees

S0 =	-3,46

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm³/year]	Relative	Absolute
2,00	5,06	2000	1,143	2,80	-10,21	24,5	-1,74	50,32	-1,74
2,00	5,06	4000	1,086	2,65	-8,37	23,95	-1,27	36,66	-1,27
2,00	5,06	4740	1,065	2,60	-8,75	23,75	-1,26	36,50	-1,26
2,00	5,06	5240	1,058	2,60	-9,43	23,75	-1,34	38,72	-1,34
2,00	5,06	5740	1,042	2,55	-10,42	23,53	-1,42	40,96	-1,42
2,00	5,06	6240	1,037	2,55	-11,82	23,53	-1,58	45,72	-1,58
2,00	5,06	6740	1,023	2,50	-13,58	23,32	-1,74	50,20	-1,74
2,00	5,06	7240	1,022	2,50	-15,67	23,32	-1,97	57,09	-1,97
2,00	5,06	7740	1,024	2,50	-17,88	23,32	-2,23	64,39	-2,23
2,00	5,06	8240	1,037	2,55	-20,04	23,53	-2,54	73,41	-2,54
2,00	5,06	8740	1,044	2,55	-21,74	23,53	-2,75	79,52	-2,75
2,00	5,06	9000	1,048	2,55	-22,43	23,53	-2,84	82,14	-2,84
2,00	5,06	11000	1,130	2,75	-23,06	24,32	-3,49	100,86	-3,49
2,00	5,06	13000	1,200	2,95	-20,00	25,03	-3,61	104,40	-3,61
2,00	5,06	15000	1,221	3,00	-18,10	25,2	-3,46	99,98	-3,46
2,00	5,06	17000	1,223	3,00	-17,52	25,2	-3,37	97,52	-3,37
2,00	5,06	19000	1,235	3,05	-17,49	25,38	-3,46	100,00	-3,46


Diameter = 4 km, Distance = 5 km
45 degrees
Minimal dir. spread
Large dir. spread
Relative transport S/S0 [%]
Distance Y-direction [m]
300,00
250,00
200,00
150,00
100,00
50,00
0,00
-50,00
-100,00
-150,00
-200,00
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000

F.2 Sensitivity analysis of several parameters, minimal spread

Netto yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 10 degrees

Initial situation: H0 = 2.0; m = 1:100 and h0 = 30.0 m

S0 =	-1,86

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,315	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	4000	1,315	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	6000	1,315	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	8000	1,315	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	9360	0,94	2,35	-4,26	22,69	-0,47	25,09	-0,47
2,00	5,06	9485	0,68	1,80	-2,78	20,14	-0,14	7,66	-0,14
2,00	5,06	9610	0,41	1,20	-1,54	16,65	-0,02	1,27	-0,02
2,00	5,06	9735	0,23	0,65	-2,70	12,32	-0,01	0,52	-0,01
2,00	5,06	9860	0,26	0,65	-7,09	12,32	-0,03	1,69	-0,03
2,00	5,06	9985	0,44	1,25	-8,16	17,03	-0,14	7,60	-0,14
2,00	5,06	10110	0,63	1,65	-8,82	19,35	-0,37	19,72	-0,37
2,00	5,06	10235	0,86	2,10	-9,04	21,63	-0,77	41,29	-0,77
2,00	5,06	10360	1,08	2,65	-8,83	23,95	-1,31	70,62	-1,31
2,00	5,06	11000	1,32	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	13000	1,32	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	15000	1,32	3,25	-7,67	26,08	-1,86	100,15	-1,86
2,00	5,06	17000	1,32	3,25	-7,67	26,08	-1,86	100,15	-1,86

Depth deepwater = 20.0 m

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	**Relative**	**Absolute**
2,00	5,06	2000	1,317	3,25	-7,71	26,08	-1,87	100,97	-1,87
2,00	5,06	4000	1,317	3,25	-7,71	26,08	-1,87	100,97	-1,87
2,00	5,06	6000	1,317	3,25	-7,71	26,08	-1,87	100,97	-1,87
2,00	5,06	8000	1,317	3,25	-7,71	26,08	-1,87	100,97	-1,87
2,00	5,06	9360	0,94	2,35	-4,28	22,69	-0,47	25,21	-0,47
2,00	5,06	9485	0,69	1,80	-2,79	20,14	-0,14	7,76	-0,14
2,00	5,06	9610	0,41	1,20	-1,54	16,65	-0,02	1,28	-0,02
2,00	5,06	9735	0,23	0,65	-2,72	12,32	-0,01	0,54	-0,01
2,00	5,06	9860	0,26	0,65	-7,15	12,32	-0,03	1,73	-0,03
2,00	5,06	9985	0,44	1,25	-8,23	17,03	-0,14	7,81	-0,14
2,00	5,06	10110	0,64	1,65	-8,89	19,35	-0,37	20,19	-0,37
2,00	5,06	10235	0,87	2,15	-9,11	21,84	-0,80	42,99	-0,80
2,00	5,06	10360	1,08	2,65	-8,88	23,95	-1,33	71,53	-1,33
2,00	5,06	11000	1,32	3,25	-7,71	26,08	-1,87	100,82	-1,87
2,00	5,06	13000	1,32	3,25	-7,71	26,08	-1,87	100,82	-1,87
2,00	5,06	15000	1,32	3,25	-7,71	26,08	-1,87	100,82	-1,87
2,00	5,06	17000	1,32	3,25	-7,71	26,08	-1,87	100,82	-1,87

Depth deepwater = 40.0 m

S0 =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	**Relative**	**Absolute**
2,00	5,06	2000	1,317	3,25	-7,67	26,08	-1,86	100,46	-1,86
2,00	5,06	4000	1,317	3,25	-7,67	26,08	-1,86	100,46	-1,86
2,00	5,06	6000	1,317	3,25	-7,67	26,08	-1,86	100,46	-1,86
2,00	5,06	8000	1,317	3,25	-7,67	26,08	-1,86	100,46	-1,86
2,00	5,06	9360	0,94	2,35	-4,26	22,69	-0,46	24,99	-0,46
2,00	5,06	9485	0,68	1,80	-2,78	20,14	-0,14	7,66	-0,14
2,00	5,06	9610	0,41	1,20	-1,54	16,65	-0,02	1,27	-0,02
2,00	5,06	9735	0,23	0,65	-2,66	12,32	-0,01	0,52	-0,01
2,00	5,06	9860	0,26	0,65	-7,09	12,32	-0,03	1,69	-0,03
2,00	5,06	9985	0,44	1,25	-8,16	17,03	-0,14	7,60	-0,14
2,00	5,06	10110	0,63	1,65	-8,82	19,35	-0,37	19,72	-0,37
2,00	5,06	10235	0,86	2,15	-9,06	21,84	-0,78	42,17	-0,78
2,00	5,06	10360	1,08	2,65	-8,83	23,95	-1,31	70,62	-1,31
2,00	5,06	11000	1,32	3,25	-7,66	26,08	-1,86	100,33	-1,86
2,00	5,06	13000	1,32	3,25	-7,66	26,08	-1,86	100,33	-1,86
2,00	5,06	15000	1,32	3,25	-7,66	26,08	-1,86	100,33	-1,86
2,00	5,06	17000	1,32	3,25	-7,66	26,08	-1,86	100,33	-1,86

Deepwater waveheight = 1.0 m

S0 =	-0,38

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
1,0	3,58	2000	0,692	1,6	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	4000	0,692	1,6	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	6000	0,692	1,6	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	8000	0,692	1,6	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	9360	0,47	1,20	-4,13	11,38	-0,08	20,43	-0,08
1,0	3,58	9485	0,32	0,75	-2,65	9,22	-0,02	4,97	-0,02
1,0	3,58	9610	0,20	0,60	-1,66	8,19	0,00	1,09	0,00
1,0	3,58	9735	0,13	0,60	-4,64	8,19	0,00	1,18	0,00
1,0	3,58	9860	0,17	0,60	-7,74	8,19	-0,01	3,79	-0,01
1,0	3,58	9985	0,26	0,95	-9,75	10,18	-0,05	12,66	-0,05
1,0	3,58	10110	0,37	1,00	-9,81	10,42	-0,11	27,45	-0,11
1,0	3,58	10235	0,50	1,25	-9,82	11,62	-0,21	55,07	-0,21
1,0	3,58	10360	0,57	1,30	-9,45	11,86	-0,28	72,04	-0,28
1,0	3,58	11000	0,69	1,60	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	13000	0,69	1,60	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	15000	0,69	1,60	-8,21	12,89	-0,38	100,00	-0,38
1,0	3,58	17000	0,69	1,60	-8,21	12,89	-0,38	100,00	-0,38

Deepwater waveheight = 3.0 m

S0 =	-4,59

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	j_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
3,0	6,20	2000	1,906	4,8	-7,4	38,93	-4,59	100,00	-4,59
3,00	6,20	4000	1,906	4,8	-7,4	38,93	-4,59	100,00	-4,59
3,00	6,20	6000	1,906	4,8	-7,4	38,93	-4,59	100,00	-4,59
3,00	6,20	8000	1,906	4,8	-7,4	38,93	-4,59	100,00	-4,59
3,00	6,20	9360	1,45	3,65	-4,54	34,65	-1,45	31,60	-1,45
3,00	6,20	9485	1,03	2,60	-2,93	29,82	-0,41	8,98	-0,41
3,00	6,20	9610	0,61	1,60	-1,61	23,72	-0,06	1,36	-0,06
3,00	6,20	9735	0,31	0,95	-1,88	18,25	-0,01	0,32	-0,01
3,00	6,20	9860	0,33	1,10	-6,16	19,70	-0,06	1,27	-0,06
3,00	6,20	9985	0,54	1,40	-7,52	22,34	-0,21	4,64	-0,21
3,00	6,20	10110	0,82	2,05	-8,22	26,75	-0,64	13,95	-0,64
3,00	6,20	10235	1,15	2,95	-8,62	31,55	-1,58	34,29	-1,58
3,00	6,20	10360	1,48	3,75	-8,49	34,95	-2,86	62,22	-2,86
3,00	6,20	11000	1,91	4,80	-7,40	38,93	-4,59	100,00	-4,59
3,00	6,20	13000	1,91	4,80	-7,40	38,93	-4,59	100,00	-4,59
3,00	6,20	15000	1,91	4,80	-7,40	38,93	-4,59	100,00	-4,59
3,00	6,20	17000	1,91	4,80	-7,40	38,93	-4,59	100,00	-4,59

Foreshore slope m = 1 : 250

S0 =	-1,28

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [°]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]
2,0	5,06	2000	1,126	3,20	-7,24	25,95	-1,28
2,0	5,06	4000	1,126	3,20	-7,24	25,95	-1,28
2,0	5,06	6000	1,126	3,20	-7,24	25,95	-1,28
2,0	5,06	8000	1,126	3,20	-7,24	25,95	-1,28
2,0	5,06	9360	0,89	2,52	-5,20	23,45	-0,52
2,0	5,06	9485	0,63	1,82	-3,35	20,33	-0,15
2,0	5,06	9610	0,36	1,02	-1,82	15,63	-0,02
2,0	5,06	9735	0,18	0,54	-1,14	12,07	0,00
2,0	5,06	9860	0,15	0,50	-4,58	13,02	-0,01
2,0	5,06	9985	0,24	0,74	-6,64	13,81	-0,03
2,0	5,06	10110	0,37	1,06	-7,52	16,06	-0,09
2,0	5,06	10235	0,53	1,54	-7,98	10,87	-0,13
2,0	5,06	10360	0,73	2,10	-8,00	21,68	-0,50
2,0	5,06	11000	1,13	3,20	-7,24	25,95	-1,28
2,0	5,06	13000	1,13	3,20	-7,24	25,95	-1,28
2,0	5,06	15000	1,13	3,20	-7,24	25,95	-1,28
2,0	5,06	17000	1,13	3,20	-7,24	25,95	-1,28

Relative	Absolute
100,00	-1,28
100,00	-1,28
100,00	-1,28
100,00	-1,28
40,67	-0,52
11,59	-0,15
1,55	-0,02
0,19	0,00
0,56	-0,01
2,15	-0,03
6,82	-0,09
10,28	-0,13
38,71	-0,50
100,00	-1,28
100,00	-1,28
100,00	-1,28
100,00	-1,28

Wavesteepness s = 0,01

S0 =	-3,02

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,0	11,32	2000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	4000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	6000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	8000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	9360	1,50	3,15	-3,24	61,76	-1,09	36,10	-1,09
2,0	11,32	9485	1,10	2,40	-2,29	54,06	-0,36	12,02	-0,36
2,0	11,32	9610	0,63	1,35	-1,38	40,80	-0,05	1,80	-0,05
2,0	11,32	9735	0,30	0,65	-0,90	28,00	-0,01	0,18	-0,01
2,0	11,32	9860	0,24	0,60	-2,87	26,72	-0,01	0,34	-0,01
2,0	11,32	9985	0,38	1,05	-4,50	35,65	-0,06	1,90	-0,06
2,0	11,32	10110	0,64	1,40	-5,01	41,53	-0,21	6,89	-0,21
2,0	11,32	10235	0,99	2,15	-5,36	51,25	-0,64	21,32	-0,64
2,0	11,32	10360	1,38	2,95	-5,42	59,82	-1,49	49,32	-1,49
2,0	11,32	11000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	13000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	15000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02
2,0	11,32	17000	1,937	4,00	-4,80	69,33	-3,02	100,00	-3,02

F.3 Sensitivity analysis of several parameters, large spread

Net yearly longshore sediment transports

Diameter island: 1 km; φ_0 = 10 degrees

Initial situation: H0 = 2.0; m = 1:100 and h0 = 30.0 m

Directional spread : 31.5 degrees

S0 =	-0,90

H_0 [m]	T_m [s]	Y-distance [m]	H_b [m]	h_b [m]	φ_b [°]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	4000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	6000	1,257	3,1	-4,1	25,55	-0,90	100,00	-0,90
2,00	5,06	8000	1,238	3,05	-2,87	25,38	-0,61	67,57	-0,61
2,00	5,06	9360	1,05	2,55	8,57	23,54	1,20	-133,34	1,20
2,00	5,06	9485	1,02	2,50	8,85	23,32	1,14	-127,36	1,14
2,00	5,06	9610	0,98	2,40	7,46	22,90	0,87	-97,34	0,87
2,00	5,06	9735	0,95	2,35	3,77	22,68	0,42	-46,65	0,42
2,00	5,06	9860	0,94	2,30	-1,70	22,48	-0,18	20,24	-0,18
2,00	5,06	9985	0,95	2,35	-7,52	22,68	-0,82	91,28	-0,82
2,00	5,06	10110	0,97	2,40	-12,52	22,90	-1,41	157,08	-1,41
2,00	5,06	10235	0,99	2,45	-14,98	23,11	-1,77	197,64	-1,77
2,00	5,06	10360	1,02	2,50	-15,76	23,32	-1,99	222,02	-1,99
2,00	5,06	11000	1,19	2,95	-9,38	25,03	-1,79	199,31	-1,79
2,00	5,06	13000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90
2,00	5,06	15000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90
2,00	5,06	17000	1,26	3,10	-4,10	25,55	-0,90	100,00	-0,90

Depth deepwater = 20.0 m
Directional spread : 31.5 degrees

SO =	-0,90

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93
2,00	5,06	4000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93
2,00	5,06	6000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93
2,00	5,06	8000	1,24	3,05	-2,84	25,38	-0,60	67,08	-0,60
2,00	5,06	9360	1,06	2,60	8,61	23,75	1,23	-137,20	1,23
2,00	5,06	9485	1,02	2,50	8,81	23,32	1,15	-127,80	1,15
2,00	5,06	9610	0,99	2,45	7,46	23,11	0,91	-101,08	0,91
2,00	5,06	9735	0,96	2,40	3,76	22,90	0,43	-48,27	0,43
2,00	5,06	9860	0,95	2,35	-1,75	22,69	-0,19	21,62	-0,19
2,00	5,06	9985	0,95	2,35	-7,56	22,69	-0,83	92,96	-0,83
2,00	5,06	10110	0,97	2,40	-12,28	22,90	-1,40	155,86	-1,40
2,00	5,06	10235	1,00	2,45	-15,02	23,11	-1,80	200,12	-1,80
2,00	5,06	10360	1,04	2,55	-15,86	23,54	-2,07	231,15	-2,07
2,00	5,06	11000	1,20	2,95	-9,45	25,02	-1,81	201,33	-1,81
2,00	5,06	13000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93
2,00	5,06	15000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93
2,00	5,06	17000	1,27	3,15	-4,14	25,72	-0,93	103,75	-0,93

Depth deepwater = 40.0 m
Directional spread : 31.5 degrees

SO =	-1,86

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-CERC}}$ [Mm^3/year]	Relative	Absolute
2,00	5,06	2000	1,256	3,10	-4,1	25,55	-0,90	99,84	-0,90
2,00	5,06	4000	1,256	3,10	-4,1	25,55	-0,90	99,84	-0,90
2,00	5,06	6000	1,256	3,10	-4,1	25,55	-0,90	99,84	-0,90
2,00	5,06	8000	1,238	3,05	-2,87	25,38	-0,61	67,57	-0,61
2,00	5,06	9360	1,05	2,55	8,57	25,53	1,29	-143,24	1,29
2,00	5,06	9485	1,02	2,50	8,85	23,32	1,14	-127,11	1,14
2,00	5,06	9610	0,98	2,40	7,46	22,90	0,87	-97,34	0,87
2,00	5,06	9735	0,95	2,35	3,78	22,69	0,42	-46,79	0,42
2,00	5,06	9860	0,94	2,30	-1,69	22,48	-0,18	20,12	-0,18
2,00	5,06	9985	0,95	2,35	-7,52	22,69	-0,82	91,32	-0,82
2,00	5,06	10110	0,97	2,40	-12,25	22,90	-1,38	153,90	-1,38
2,00	5,06	10235	0,99	2,45	-14,98	23,11	-1,77	197,64	-1,77
2,00	5,06	10360	1,02	2,50	-15,76	23,32	-1,99	222,02	-1,99
2,00	5,06	11000	1,18	2,90	-9,36	24,85	-1,74	193,85	-1,74
2,00	5,06	13000	1,26	3,10	-4,10	25,55	-0,90	99,84	-0,90
2,00	5,06	15000	1,26	3,10	-4,10	25,55	-0,90	99,84	-0,90
2,00	5,06	17000	1,26	3,10	-4,10	25,55	-0,90	99,84	-0,90

Deepwater waveheight = 1.0 m

S0 =	-0,21

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
1,0	3,58	2000	0,675	1,6	-4,59	12,89	-0,21	100,00	-0,21
1,0	3,58	4000	0,675	1,6	-4,59	12,89	-0,21	100,00	-0,21
1,0	3,58	6000	0,675	1,6	-4,59	12,89	-0,21	100,00	-0,21
1,0	3,58	8000	0,657	1,55	-1,74	12,72	-0,07	35,57	-0,07
1,0	3,58	9360	0,55	1,30	6,11	11,86	0,17	-80,75	0,17
1,0	3,58	9485	0,55	1,30	5,29	11,86	0,14	-69,03	0,14
1,0	3,58	9610	0,53	1,30	3,44	11,86	0,09	-43,24	0,09
1,0	3,58	9735	0,53	1,30	0,35	11,86	0,01	-4,29	0,01
1,0	3,58	9860	0,52	1,30	-3,61	11,86	-0,09	43,68	-0,09
1,0	3,58	9985	0,53	1,30	-7,68	11,86	-0,19	92,41	-0,19
1,0	3,58	10110	0,53	1,30	-11,13	11,86	-0,28	135,20	-0,28
1,0	3,58	10235	0,54	1,30	-13,51	11,86	-0,35	167,69	-0,35
1,0	3,58	10360	0,55	1,30	-14,65	11,86	-0,39	188,07	-0,39
1,0	3,58	11000	0,62	1,40	-11,06	12,24	-0,38	186,05	-0,38
1,0	3,58	13000	0,68	1,60	-4,64	12,89	-0,21	101,08	-0,21
1,0	3,58	15000	0,68	1,60	-4,59	12,89	-0,21	100,00	-0,21
1,0	3,58	17000	0,68	1,60	-4,59	12,89	-0,21	100,00	-0,21

Deepwater waveheight = 3.0 m

S0 =	-2,37

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
3,0	6,20	2000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37
3,00	6,20	4000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37
3,00	6,20	6000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37
3,00	6,20	8000	1,86	4,75	-3,67	38,77	-2,18	91,79	-2,18
3,00	6,20	9360	1,56	3,95	8,80	35,86	3,36	-141,34	3,36
3,00	6,20	9485	1,48	3,75	10,75	35,02	3,65	-153,58	3,65
3,00	6,20	9610	1,38	3,50	11,29	24,04	3,27	-137,76	3,27
3,00	6,20	9735	1,32	3,30	8,21	33,21	1,50	-63,23	1,50
3,00	6,20	9860	1,29	3,25	1,00	32,98	0,25	-10,38	0,25
3,00	6,20	9985	1,30	3,25	-7,62	32,98	-1,86	78,48	-1,86
3,00	6,20	10110	1,34	3,35	-13,95	33,43	-3,52	148,21	-3,52
3,00	6,20	10235	1,39	3,50	-16,89	34,04	-4,59	193,47	-4,59
3,00	6,20	10360	1,47	3,70	-16,98	34,85	-5,20	219,20	-5,20
3,00	6,20	11000	1,78	4,55	-7,85	38,13	-3,81	160,41	-3,81
3,00	6,20	13000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37
3,00	6,20	15000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37
3,00	6,20	17000	1,88	4,80	-3,92	38,93	-2,37	100,00	-2,37

Foreshore slope m = 1 : 250

S0 =	-0,65

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [°]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,0	5,06	2000	1,113	3,10	-3,73	26,0	-0,65	100,00	-0,65
2,0	5,06	4000	1,113	3,10	-3,73	26,0	-0,65	100,00	-0,65
2,0	5,06	6000	1,113	3,10	-3,73	26,0	-0,65	100,00	-0,65
2,0	5,06	8000	1,055	3,00	-2,92	26,0	-0,46	70,41	-0,46
2,0	5,06	9360	0,90	2,56	6,52	23,61	0,68	-103,85	0,68
2,0	5,06	9485	0,82	2,32	9,91	22,64	0,81	-123,97	0,81
2,0	5,06	9610	0,72	2,04	13,32	21,39	0,78	-120,45	0,78
2,0	5,06	9735	0,66	1,84	13,53	20,43	0,62	-95,83	0,62
2,0	5,06	9860	0,61	1,74	6,51	19,92	0,26	-40,01	0,26
2,0	5,06	9985	0,61	1,74	-6,87	19,92	-0,27	41,91	-0,27
2,0	5,06	10110	0,65	1,82	-16,11	20,33	-0,71	109,05	-0,71
2,0	5,06	10235	0,71	2,00	-18,71	21,20	-1,00	154,28	-1,00
2,0	5,06	10360	0,77	2,16	-17,16	21,95	-1,13	174,44	-1,13
2,0	5,06	11000	1,06	2,85	-6,33	26,0	-1,00	153,11	-1,00
2,0	5,06	13000	1,11	3,10	-3,73	26,0	-0,65	100,00	-0,65
2,0	5,06	15000	1,11	3,10	-3,73	26,0	-0,65	100,00	-0,65
2,0	5,06	17000	1,11	3,10	-3,73	26,0	-0,65	100,00	-0,65

Wavesteepness s = 0,01

S0 =	-1,48

H_0 [m]	T_m [s]	distance [m]	H_b [m]	h_b [m]	φ_b [º]	L [m]	$S_{x\text{-}CERC}$ [Mm^3/year]	Relative	Absolute
2,0	11,32	2000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	4000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	6000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	8000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	9360	1,54	3,25	5,20	62,74	1,87	-125,86	1,87
2,0	11,32	9485	1,41	3,00	7,10	60,30	2,04	-137,59	2,04
2,0	11,32	9610	1,23	2,60	8,66	56,31	1,78	-119,58	1,78
2,0	11,32	9735	1,08	2,35	5,52	53,50	0,84	-56,27	0,84
2,0	11,32	9860	0,98	2,10	3,90	50,68	0,46	-30,68	0,46
2,0	11,32	9985	0,97	2,05	-4,25	48,83	-0,47	31,86	-0,47
2,0	11,32	10110	1,07	2,04	-10,23	53,50	-1,50	101,02	-1,50
2,0	11,32	10235	1,22	2,60	-11,95	56,31	-2,36	158,79	-2,36
2,0	11,32	10360	1,37	2,90	-11,40	59,33	-3,00	201,87	-3,00
2,0	11,32	11000	1,842	3,90	-4,25	68,47	-2,39	161,11	-2,39
2,0	11,32	13000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	15000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48
2,0	11,32	17000	1,901	4,00	-2,44	69,33	-1,48	100,00	-1,48

List of symbols

Roman symbols

Symbol	Definition	Dimension
B	Width of caisson	m
c	Wave celerity	m
c_b	Wave celerity at breaker line	m/s
c_g	wave group celerity	m/s
d	Distance between beginning of circular island and shoreline	m
	Diameter of grains (d_{50})	m
d_z	Minimum draught of caisson	m
D	Diameter of circular island	m
D_{n50}	Nominal diameter $(M_{50}/\rho_r)^{1/3}$	m
D_{50}	Sieve diameter, diameter of stone which	
	exceeds the 50% value of sieve curve	m
D_{15}	15% value of sieve curve	m
D_{85}	85% value of sieve curve	m
g	Acceleration of gravity	m/s^2
h	Waterdepth	m
h_c	Crest height	m
$h_{caisson}$	Caisson's total height	m
h_b	Waterdepth at breaker line	m
H	Wave height	m
H_b	Wave height at breaker line	m
H_s	Significant wave height	m
H_0	Deepwater wave height	m
H_{s0}	Deepwater significant wave height	m
k	Wave number $(2\pi/L)$	1/m
K_D	Stability coefficient in Hudson's formula	–
K_Δ	Layer thickness coefficient	–
L	Wave length	m
L_b	Wave length at breaker line	m
L_0	Deepwater wave length	m
m	Beach slope	–

Symbol	Definition	Dimension
M.S.L.	Mean Sea Level	m
n	Ratio c_g to c	–
	Number of layers	–
n_v	Volumetric porosity	–
N	Number of waves in Van der Meer's formula	–
p	Pressure	N/m^2
P	Permeability in Van der Meer's formula	–
Q	Discharge	m^3/s
Q_b	Percentage of broken waves calculated by HISWA	–
R_u	Wave run-up	m
$R_{u2\%}$	Wave run-up exceeded by 2% of waves	m
s	Wave steepness	–
s_m	Wave steepness for mean period	–
s_p	Wave steepness for peak period	–
s_0	Deepwater wave steepness	–
S	Sediment transport	m^3/s
	Radiation stress	kg/s^2
S_d	Damage level in Van der Meer's formula	–
S.L.	Sea Level (= M.S.L. + set-up's)	m
S_{XX}	XX-component of the radiation stress	kg/s^2
S_{YY}	YY-component of the radiation stress	kg/s^2
S_{xx}	xx-component of the radiation stress	kg/s^2
S_{yy}	yy-component of the radiation stress	kg/s^2
S_0	Initial sediment transport	m^3/s
t	Time	s
	Layer thickness	m
T	Wave period	s
T_m	Mean wave period	s
T_p	Peak wave period	s
W	Weight of armour stones in Hudson's formula	kg
X	Coordinate direction	–
Y	Coordinate direction	–

Greek symbols

Symbol	Definition	Dimension
α	Slope angle	degrees
β	Angle of wave attack with respect to structure	degrees
γ	Breaker depth ratio (H_b/h_b)	–
Δ	Relative density $(\rho_s - \rho_w/\rho_w)$	–
Δ_X	Length of HISWA-mesh in X-direction	m
Δ_Y	Length of HISWA-mesh in Y-direction	m
Θ	Angle	degrees
μ	Friction coefficient	–
ξ	Irribarran's parameter	–
ξ_m	Irribarran's parameter based on T_m	–
ξ_p	Irribarran's parameter based on T_p	–
π	Constant (=3,141592650)	–
ρ_s	Density of soil	kg/m^3
ρ_w	Density of water	kg/m^3
ϕ	Angle	degrees
ϕ_b	Angle of wave approach at breaker line	degrees
ϕ_0	Angle of wave approach at deepwater	degrees

Index

a, 30, 32
Accropods, 25
Airport, 6, 13
Artificial islands, 1
    Advantages, 3
    Collection of, 91
    Disadvantages, 3
    Problems, 18
    Purposes, 3
    Similar properties, 17
    Types, 7
Asia, 94
Australia, 94

b, 30, 32
Beaufort Sea, 95
Bottom-fixed island, 8, 11
Brebner and Donnelly, 137
Bulking factor, 55

Caisson-type seawalls, 26
CERC-formula, 69
Collection of A.I., 91
Combined defence system, 21
Comparing types, 9
Complex city, 7, 14
Concrete cap, 23
Construction reasons, 2
Core-loc, 25
Costs
    Morphology, 62
    Overall costs, 54
Costs estimates for A.I. types, 9

Delft University of Technology, 1
Dikes, 22
Direct dumping, 53
Dolosse, 24
Dredging activities, 49
    Cycle production, 51
    Direct dumping, 53
    Dredging of landfill, 50
    Dredging process, 50
    Dumping of landfill, 51, 54
    Loading-curve, 51
    Pumping, 53
    Rainbowing, 53
Dumping of landfill, 51

Earthquakes, 19
Economical waterdepth, 18
Energy generation, 5, 13
Environmental laws, 2
Europe, 97
Experience gained, 17

Filter cloth, 23
Filter layer, 22
Floating island, 8, 11

Goda, 135
Gradations quarry stones, 31
Gravity island, 8, 11
Guidelines
    Experiences, 18
    Morphology, 79
    Sea defences, 28

Hanstholm-caisson, 26
Hard defence system, 21
HISWA, 66
    Sensitivity, 75
Horizontal composite breakwater, 26
Hudson-formula, 115

Industrial island, 7, 14

Jack-up island, 8, 11
Japan, 99

Landfill, 50, 51
Literature study, 17
Loading-curve, 51

Longshore transport, 60

Mariculture, 4, 13
Modified cubes, 25
Morphology, 59
    Bathymetry, 64
    Directional spread, 65
    Example, 71
    Expected relation, 62
    Island model, 62
    Processes, 60
    Recommendations, 82
    Sensitivity of HISWA, 75

North America, 92

Ocean mining & exploration, 4, 12
Overall island costs, 54
Overtopping, 30, 113, 115

Peak wave period, 112
Perforated front wall, 27
Piling island, 9, 12
Polder island, 8, 10, 18
Port, 6, 13
Primary armour, 23
    Accropods, 25
    Dolosse, 24
    Modified cubes, 25
    Quarry stones, 24
    Tetrapods, 25
primary armour
    Core-loc, 25
Principal stresses, 60
Problem approach
    General, 15
Problem description
    General, 15
    Morphology, 64
Problems, 18
Pumping, 53
Purposes, 3
    Multi purpose islands, 6
    Single purpose islands, 4

Quarry stones, 24

Rainbowing, 53
Reclamation island, 7, 10, 14
Recreation, 4, 13
Reference line, 23
Refraction, 63
Rehandle pit, 53
Research description
    Morphology, 64
Research objective
    General, 15
    Sea defences, 28
Rubble mound sea defences, 22

Sea defences, 21
    Caisson-type seawalls, 26, 133
    Conclusions, 33
    Design of, 111, 133
    Examples, 119, 144, 148
    Influence material-costs, 47
    Recommendations, 33
    Rubble mound sea defences, 22
Sea-quakes, 19
Seas, x, 28
Secondary armour, 23
Settlements, 20
Sloping top, 26
Soft defence systems, 21
South America, 92
Steepness of waves, 28
Surf similarity parameter, ξ, 112
Swell, x, 28
Symbols
    Greek, 239
    Roman, 237

Tanimoto et al., 137
Temporary or aiding, 13
Temporary or aiding structure, 4
Tetrapods, 25
Toe structure, 22
Trailing suction hopper dredge, 50
Types of artificial islands, 7

Van der Meer formulae, 114, 115
Vertical composite breakwater, 26

Waste handling, 4, 13
Waste island, 6, 14
Wave length, 112
Wave period, maximum, 29
Wave steepness, 28