Preface

This report is the Master thesis of Judit Heerdink, student at Delft University of Technology, Faculty of Civil Engineering and Geosciences. This thesis project is the last part of my studies in Civil Engineering, specialisation Hydraulic Engineering. The project has been carried out under guidance of Delta Marine Consultants, Gouda. The subject of this report is to determine the effects of wave climate on the shoreline response to offshore breakwaters.

I would like to thank the members of my graduation committee for their comments and support during my graduation project. The members of this committee are:

Prof. Dr. Ir. M.J.F Stive (DUT)
Dr. Ir. J. van de Graaff (DUT)
Dr. Ir. W.S.J. Uijttewaal (DUT)
Ir. J.S. Reedijk (DMC)

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Judit Heerdink

Summary

Offshore breakwaters are used more and more as shore protection since 1960. Especially in Japan, Italy and the United States a lot of these breakwaters are constructed. In the past many people studied the effects of offshore breakwaters on the shoreline, but it turned out to be very difficult to set up general design guidance: too many factors play a role in determining the shoreline response. But researchers tried to develop some design guidance: they used former projects to deduct e.g. limits for the ratio structure length – distance offshore resulting in a salient or a tombolo (a salient reaching the breakwater). However, as mentioned before, in reality the shoreline response depends on many other parameters. This creates problems when using these limits in situations with other parameter values. Especially in lakes these problems can develop: most former projects (where the researchers derived their limits from) are constructed in oceans or deep seas and there the waves will be longer and higher than in lakes. Therefore the focus in this study is on the effects of wave climate on shoreline response. Knowing the relation between wave climate and shoreline response can help to improve the design guidance and make it possible to predict shoreline response in lakes in a better way than before.

After a short description of the hydrodynamics and sediment transport in coastal areas with and without offshore breakwaters, the main problem is analyzed. This makes clear that the major problem arises when a tombolo develops in a situation where a salient is preferred, because this makes the downdrift erosion worse and people have access to the breakwater which can be very dangerous. So it is very important to know which dimensions lead to salient formation and which to tombolo formation.

The so-called diffraction method gives a limit deducted from a, at first sight, reliable theory: equilibrium is reached when waves approach the new shoreline perpendicularly. However, in this theory the effects of an alongshore gradient in breaking wave height and gradients in wave setup are not taken into account while this is an important phenomenon in the vicinity of offshore breakwaters: diffraction leads to much lower waves behind the breakwater than next to it.

The consequence of these gradients is that an alongshore current develops, transporting sand from the exposed area towards the sheltered area behind the breakwater. This leads to a reorientation of the shore: waves will approach the new shoreline under an opposing angle which causes a opposing force (from the sheltered area towards the exposed area).

Equilibrium is expected to be reached when the force due to the gradient in wave setup is balanced by a force due to waves approaching the new shoreline under an angle. It turned out that the equilibrium angle of the shoreline depends on the value of the gradient in breaker depth. A larger gradient causes a larger equilibrium angle which means a more extended shoreline compared with a situation with a small gradient (and a small equilibrium angle). This means that the chance of tombolo formation increases when this gradient is larger. The equation resulting from the momentum balance is validated qualitatively and quantitatively. It could be concluded that this equation is qualitatively right, but quantitatively the equation misses a factor 1.5. Some assumptions are made why this factor is missing (momentum transfer, directional spreading, changing bottom and water level); these assumptions need to be investigated in following studies.

Empirical relationships differ too much from each other to improve these relationships unambiguous. But it can be concluded that the chance of a good prediction of the shoreline response increases when the gradient in breaking wave height is approximately the same as in the project the researcher deducted his limits from. This gradient turned out to be most important in determining the shoreline response.

The main conclusions from this study are the following:

- The equilibrium plan form of the shoreline behind an offshore breakwater depends on the value of the gradient in breaker depth.
- Higher waves cause higher gradients in a situation with a given layout, which leads to more sand accumulation.
- To obtain the same equilibrium plan form, breakwaters in situations with higher waves should be constructed further offshore.
- Empirical relationships can be a good guide in predicting shoreline response, but they should be used with care.

Recommendations for further research are:

- Further research is needed to investigate the factor 1.5 which is missing between practice and the equation resulting from the momentum balance.
- Study the effects of other parameters to the shoreline response, in order to improve the design guidance. Monitoring upcoming projects can help to investigate the effects of parameters.
- Use a computer model, like Delft3D, to examine if the theory used in this study agrees with computer model results.

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Figures and tables

1. Introduction

1.1 Offshore breakwaters

In general, detached offshore breakwaters are shore parallel structures. Sometimes they are parallel to the wave crests of the dominant incident waves. They might provide protection to the area on the leeward side of the breakwater and are usually constructed as rubble mound structures. If a very small area has to be protected a single structure satisfies, but if an extended length of shoreline has to be protected a multiple segment system is used with gaps between the segments. See Figure 1.1 for a visual explanation.

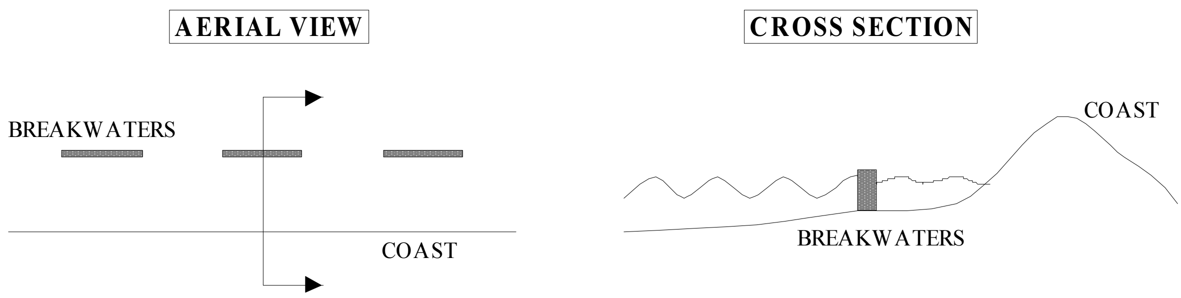


Figure 1.1: Offshore breakwaters (CUR97-2 A, 1997)

Although other shore protection structures, like groins, can help to prevent beach erosion too, offshore breakwaters are increasingly used for this objective in the last decennia. Some people see offshore breakwaters as the solution for all coastal problems, but this is not true. It depends case by case whether offshore breakwaters are the best solution or another measure is better: offshore breakwaters have disadvantages too. Some of these disadvantages are the limited design guidance, high construction costs and the still limited prediction ability of hydraulic phenomena such as rip currents, scour near the structure and adjacent beach erosion.

The main purpose of offshore breakwaters is reducing the amount of wave energy in the lee of the breakwater by reflecting, dissipating and diffracting incoming waves. This is similar to a natural reef, bar or nearshore island. The reduction of wave energy leads to a decrease of transport capacity and sediment may be deposited in the sheltered area behind the breakwater. The deposited sediment can form a salient or a tombolo (salient extended out to the breakwater), mainly dependent on the amount of wave energy transmitted into the area in the lee of the breakwaters. The difference between salient and tombolo as well as the parameters affecting the plan form will be discussed in Chapter 3.

1.2 History

Offshore breakwaters as shore protection are used more and more since 1960. In Japan already 4000 breakwaters are part of the coastline and in the United States more than 75 projects with offshore breakwaters have been constructed. Other countries where this kind of coastal protection is applied are South Africa, Israel, Spain and Italy. In Figure 1.2 an offshore breakwater project in Pennsylvania (United States) is shown.

Figure 1.2: Offshore breakwater project near Presque Isle, Pennsylvania (Coastal Engineering Manual)

In the past many people studied the effects of offshore breakwaters on the shoreline, but it turned out to be very difficult to set up general design guidance: too many factors play a role in determining the shoreline response. However, researchers tried to develop some design guidances. They used former projects to set up a design guidance of a very simple form: e.g. the ratio structure length – distance offshore is assumed to determine the shoreline response. The relationships used nowadays are summarized in Chapter 4 (and Appendix B).

1.3 Problem definition

Like mentioned above, most design guidances for offshore breakwaters are deducted from former projects and are very simple: e.g. only the structure length – distance offshore ratio determines the shoreline response. But there are more parameters affecting this response. Because these parameters are not taken into account, the empirical relationships are too simple and too site specific to be used in every place in the world. Especially in lakes, problems can rise using these relationships, because most former projects are constructed in oceans or deep seas and there are longer and higher waves than in lakes. Therefore, the focus in this study is on the effects of wave climate on the shoreline response.

1.4 Objectives

The objective of this study is investigating the effects of wave climate on the shoreline response due to the use of offshore breakwaters. Knowing the relation between wave climate and shoreline response might improve the design guidance and can help to predict shoreline response in lakes (with a different wave climate than in oceans and in seas) in a better way than before.

1.5 Outline of the report

The theoretical background considering the hydrodynamics and sediment transport in cases with and without offshore breakwaters is described in Chapter 2. Chapter 3 gives the possible applications of offshore breakwater and the problems rising in these situations. In Chapter 4 prediction methods will be summarized and one of them (the diffraction method) will be discussed in detail, including its limitations. These limitations are the basis for Chapter 5 and 6. In Chapter 5 the gradients in breaking wave height and in wave setup in alongshore direction will be quantified and the consequences of these gradients will be treated. The expected equilibrium of the shoreline (depending mainly on these gradients) is studied in Chapter 6. Then, in Chapter 7 an evaluation and improvements of the empirical relationships is given and in the last chapter conclusions and recommendations are described.

In Figure 1.3 the structure of the report is presented in a flow diagram.

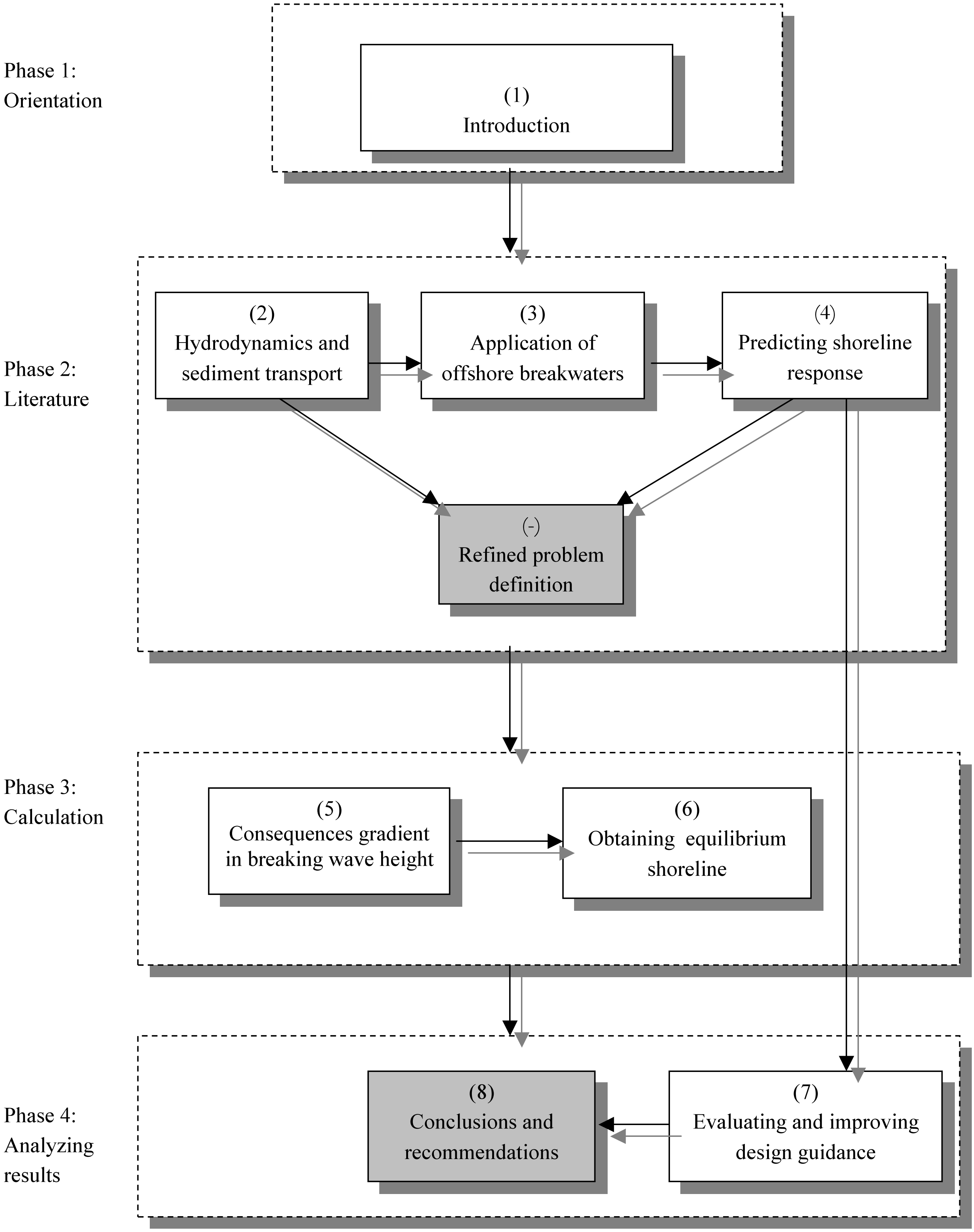


Figure 1.3: Flow diagram of structure report

2 Hydrodynamics and sediment transport

Knowledge of coastal processes is important for the understanding of the effects of offshore breakwaters. First, this chapter will discuss the coastal processes without interruption of an offshore breakwater. After that, the effects of offshore breakwaters will be discussed in terms of waves, currents and sediment transport.

2.1 Coastal processes without offshore breakwaters

The two most important hydraulic phenomena in the coastal area are tides and waves. Tide will not be treated in this report, because the focus is on lakes and tide does not occur there.

2.1.1 Waves in the nearshore

For deep water (water depth (h)/wavelength (L) > 0,5) it is generally assumed that the effect of the seabed on waves is negligible. In shallow water however, there are three important phenomena that play a role in wave propagation: shoaling, refraction and breaking of waves. These phenomena will be discussed briefly. More information can be found in D'Angremond & Pluim-Van der Velden (2000).

Shoaling
Shoaling and refraction are often mentioned together, but actually they are quite different from each other. For both, the cause of the phenomenon is the reduced depth, but refraction can only occur if the waves approach the coast under an angle while shoaling will also occur when waves approach the coast perpendicular. The idea of shoaling is that the propagation speed decreases due to the reduced depth and, because of energy conservation, the wave height will change. See formula 2.1 en 2.2.

$$\frac{1}{8}\rho g H_1^2 n_1 c_1 = \frac{1}{8}\rho g H_0^2 n_0 c_0 \tag{2.1}$$

$$\frac{H_1}{H_0} = \sqrt{\frac{c_0}{c_1}\frac{1}{2n_1}} = k_{sh} \tag{2.2}$$

where:
ρ = density of the water (kg/m^3)

g = acceleration of gravity (m/s^2)
$H_{1/0}$ = wave height in limited water depth / in deep water (m)
n_1 = ratio between group velocity and celerity of wave in limited water depth (-)
n_0 = ration between group velocity and celerity of wave in deep water = 0.5 (-)
$c_{1/0}$ = wave celerity in limited water depth / in deep water (m/s)
K_{sh} = shoaling coefficient (-)

Refraction
Refraction of waves can also be understood by spatial variations in propagation speed influenced by water depth. Formula 2.3 shows that the propagation speed in shallow water is dependent on the water depth: the shallower the water, the slower a wave will propagate.

$$c = \sqrt{gh} \tag{2.3}$$

where:
h = water depth (m)

When waves approach the shore at an angle, it is obvious that the part of the wave in deeper water travels faster than the part in shallower water. This makes the wave crest to bend and causes the angle between the wave crest and the depth contours to diminish. Wave refraction not only influences the propagation direction, but also the wave height. Because of energy conservation the wave height decreases when waves are spread (concave coastline, like bays) and wave height increases when waves are focussed (convex coastline, like headlands). See formula 2.4.

$$\frac{H_2}{H_1} = \sqrt{\frac{b_1}{b_2}} = K_r \tag{2.4}$$

where:
$H_{2/1}$ = wave height at depth belonging to point 2 / 1 (m)
$b_{2/1}$ = distance between the orthogonals at point 2 / 1 (m)
K_r = refraction coefficient (-)

Examples of refraction patterns are shown in Figure 2.1.

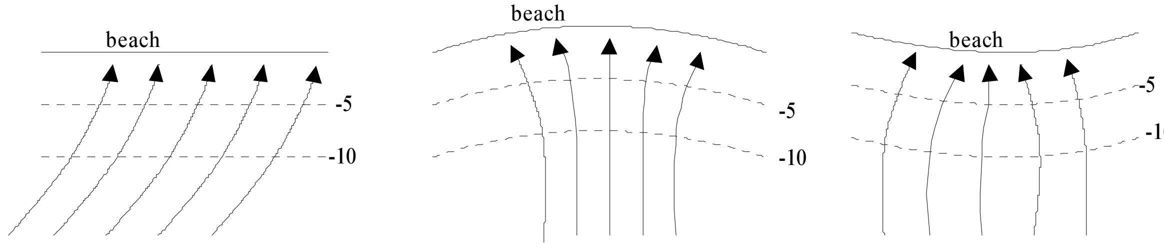


Figure 2.1: Wave refraction near straight and curved coasts (CUR 97-2A, 1997)

Breaking
Shoaling and refraction are both based on the conservation of energy. Therefore these two coefficients can only be used if there is no dissipation of energy, thus, before breaking occurs (and bottom friction is neglected). Breaking occurs either due to high steepness ($H/L > 0.14$) or due to shallow water ($H/h > 0.78$). These are the limits where the particle velocity in the crest exceeds the wave celerity and the particles tend to leave the wave profile. This implies that energy will be dissipated and wave height will be reduced considerably. The area, where this dissipation of energy takes place, is called the surf zone.

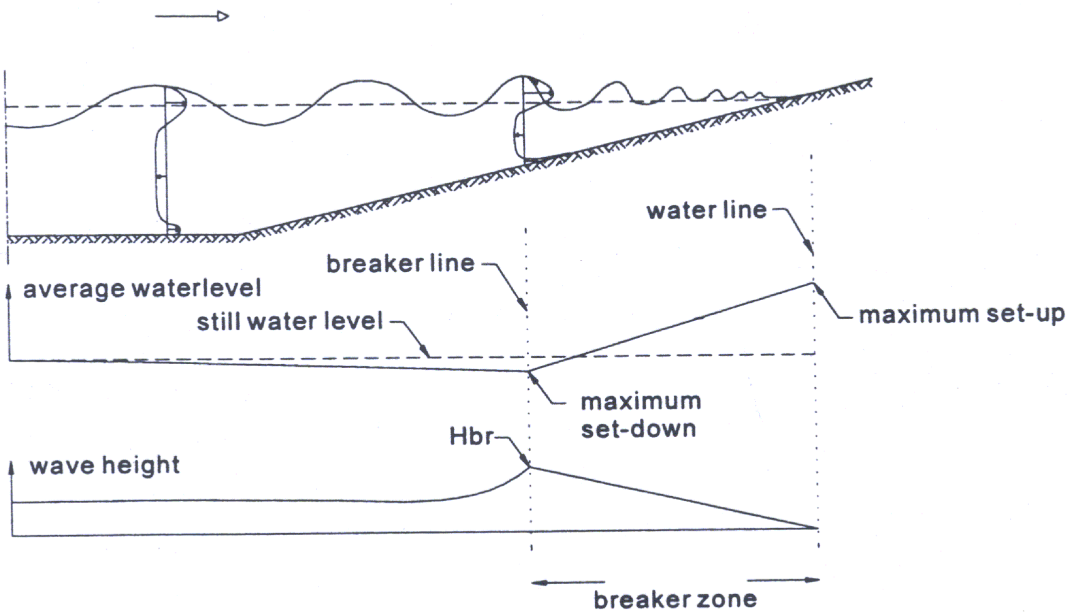


Figure 2.2: Setup and set-down in the surf zone

Another effect of breaking waves is the setup of the mean water level in the surf zone. An increase in the momentum flux in onshore direction results in compensating forces on the water column and this leads to a decrease of the mean water level relative to the still water level. The maximum depression will be at the breaking point. From this point the water level slopes upward to the point of intersection with the shore and leads to a setup of the mean water level. The setup by waves can contribute considerably to the water level changes near the coast. In Figure 2.2 this setup/setdown process is shown.

2.1.2 Currents in the nearshore

How waves are transformed in the nearshore is discussed above. Here, the nearshore currents will be treated. Two aspects are important for sediment transport: stirring up of sediment and wave driven currents.

Stirring up of sediment

In deep water, particles have a circular motion in a vertical plane perpendicular to the line of the wave crest. This motion however does not reach deep enough to affect sediment on the bottom. In depths where waves are affected by the bottom, the circular motion becomes elliptical and the water at the bottom begins to move (see Figure 2.3). In shallow water, the ellipses elongate into nearly straight lines. At breaking, particle motion becomes complicated, but also in the surf zone the water moves close to the bed forward and backward in almost horizontal paths.

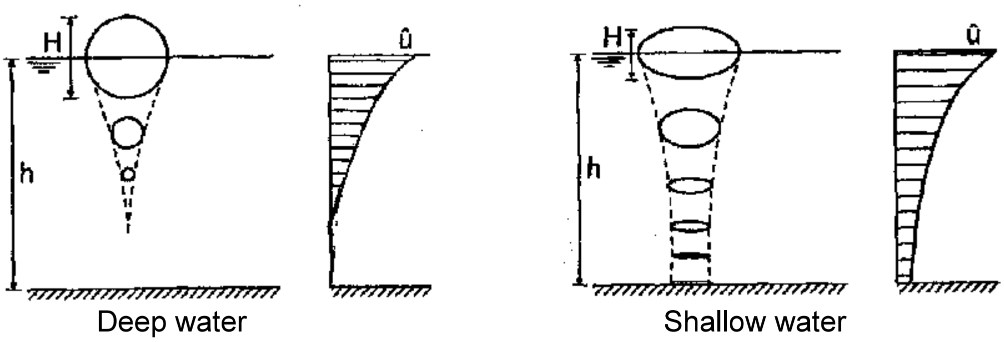


Figure 2.3: Orbital versus elliptical motion of waves

Wave driven currents

In the surf zone, where oblique incoming waves break, three wave driven currents can be distinguished.

Longshore current

Due to breaking of oblique incoming waves, the radiation shear stress component in onshore direction decreases. Because of this, a longshore current can develop. This longshore current can only exist there, where energy dissipation takes place, this means in the surf zone. The strength of the longshore current especially depends on the wave height and the angle of approach. If they increase, the longshore current increases as well. See Figure 2.4.

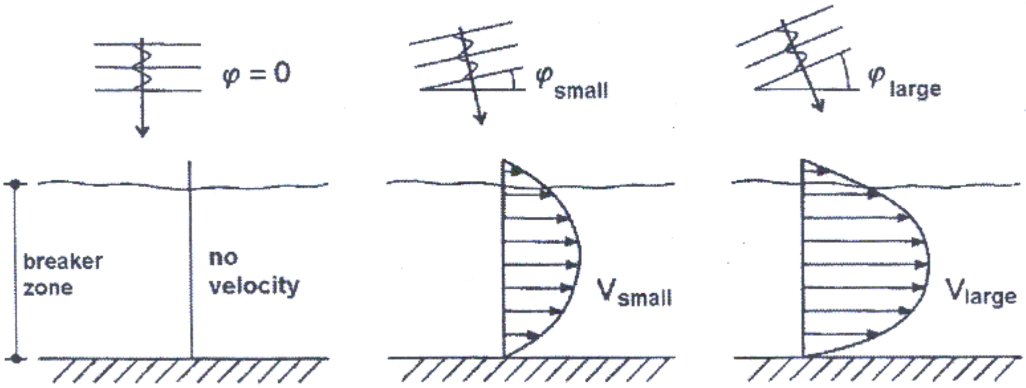


Figure 2.4: Variation in longshore current due to angle of approach

Return current
In addition to wave setup, propagating waves initiate a circulation current in the surf zone. In the upper part of the water column an onshore mass flux develops. A mass flux in the deeper part of the water column (near the bottom) brings the netto flux back to zero. This mass flux in the deeper part of the column is called the return current or undertow. Figure 2.5 shows this circulation current in the surf zone.

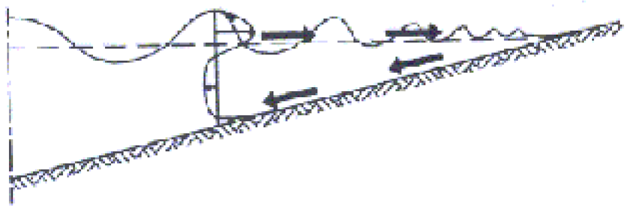


Figure 2.5: Circulation current in the surf zone

Rip current
Rip currents are concentrated jets that carry water seaward through the surf zone. They appear most noticeable when long, high waves produce wave setup on the shore. The rip channels develop most of the time on regular distance from each other. Rip currents are also called horizontal return currents.

2.1.3 Sediment transport in the surf zone

Sediment transport takes place when the shear stress exceeds the critical shear stress. The amount of sediment transport (at time 't') can be derived from the following formula:

$$S(t) = \int_0^{h+\eta} \left(u(t) \cdot c(t)\right) dz \tag{2.5}$$

where:
h = local water depth (m)
η = instantaneous water level (m)
u(t) = local velocity on height z above the bottom (m/s)
c(t) = local sediment concentration on height z above the bottom (-)

The currents, mentioned above, are of great importance for the amount of sediment transport. Due to high velocities, more sediment will be transported. Like currents, also sediment transport will be treated on two aspects: stirring up of sediment and wave driven currents.

Stirring up of sediment
Stirring up of sediment is defined as bringing sediment into suspension. This happens when the critical shear stress is exceeded. The causes for this process are the developed velocities at the bottom due to

the elliptical pattern of the wave near the bottom. Because of that, a shear stress develops. If the shear stress at the bottom exceeds the critical shear stress, the grains on the bottom will move, roll or come into suspension and sediment transport is initiated.

Wave driven currents
Breaking of waves causes sediment transport in longshore and cross-shore direction. Longshore transport is caused by the longshore current. Thus for sediment transport in longshore direction the same rule as for longshore current applies: the higher the waves and the larger the angle of approach, the more sediment will be transported

Cross-shore transport is caused by the wave itself and the return and rip current. The wave itself and wave asymmetry bring sediment to the shore (onshore), while the return and rip current take the sediment offshore.

2.2 Effects of offshore breakwater

As mentioned in the introduction, the main purpose of offshore breakwaters is to reduce the amount of wave energy in the lee of the breakwaters. In this paragraph a more detailed description of this process will be given.

2.2.1 Changed waves in the nearshore

When waves propagate towards a coastline provided with offshore breakwaters, they generally will be subjected to wave refraction processes first. Also wave breaking may have started; this depends on the location of the breakwaters relative to the coastline. When the waves run at and into the breakwaters, wave energy is dissipated by wave breaking on the breakwaters. Part of the energy will be reflected, another part will be transmitted by overtopping or by waves penetrating through the structure. See Figure 2.6.

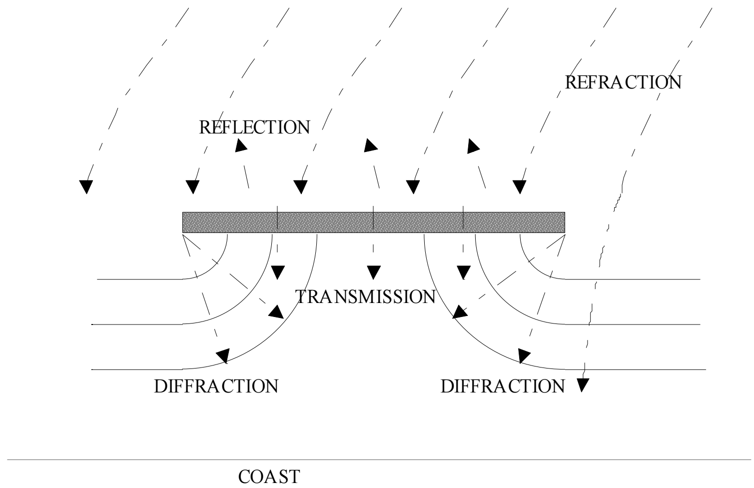


Figure 2.6: Wave processes near offshore breakwaters (CUR 97-2A, 1997)

Waves in the lee of the breakwater can also be a result of diffraction around the tips of the breakwaters and through the gaps between them. The diffracted and transmitted waves will continue to propagate towards the coast, still being affected by refraction and/or breaking (depending on the depth). The sheltered parts of the coast will also have a reduced wave setup.

The principle of offshore breakwaters is to reduce the wave conditions behind them. The magnitude of this modification depends on a large number of parameters. The most important are: wave height, wave period, water depth, gap distance between the breakwaters and the porosity and crest level of the breakwaters.

2.2.2 Changed currents in the nearshore

Like in the previous paragraph the currents in the nearshore will be treated in two parts: stirring up of sediment and wave driven currents with (again) three kinds of wave driven currents.

Stirring up of sediment
Offshore breakwaters affect the stirring up by the reduced wave energy behind the breakwaters. This leads to lower velocities. Due to the turbulence of breaking waves and the overtopping of waves plunging into the water behind the breakwater, however, the effect of these lower velocities will diminish.

Wave driven currents
Offshore breakwaters affect both the longshore current and the cross-shore currents.

Longshore current
As a result of the modification of wave conditions behind the breakwaters the longshore current is influenced. The reduction of wave height results in a decrease of the magnitude of the longshore current. Diffraction causes another effect: the wave directions in the lee of the offshore breakwaters will be different from the direction without the breakwaters.

Return current
The wave setup in the sheltered area behind the breakwater will be reduced, but the setup just behind the gaps or next to the breakwaters will remain the same. Therefore a circulation current can develop (see Figure 2.7).

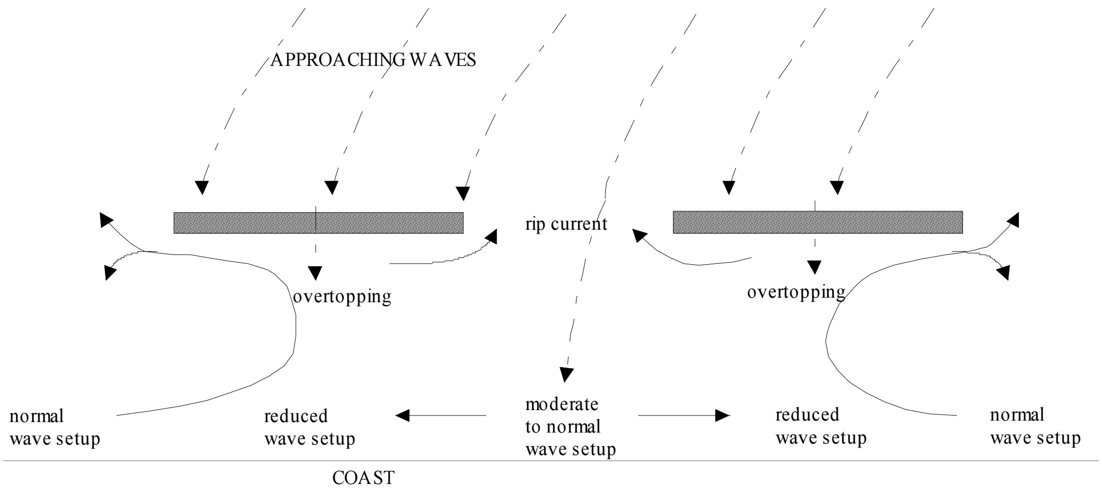


Figure 2.7: Wave induced currents around offshore breakwaters (CUR 97-2A, 1997)

Another flow of water behind the breakwaters is generated by overtopping. When the waves break on the breakwaters, large volumes of water can fall behind them. Without breakwaters a return current compensates this shoreward movement of water. The construction of breakwaters prevents the development of an undertow. Instead, the water will flow laterally towards the tips of the breakwaters and forms a concentrated rip current

Rip current

As mentioned above: a rip current can develop through the gaps between the breakwaters, because the breakwater prevents the development of an undertow. This rip current can have large velocities, because the current is so concentrated.

2.2.3 Changed sediment transport in the surf zone

The stirring up of sediment is treated above: the reduced wave height causes less stirring up, but overtopping and rip currents cause more stirring up of sediment, due to turbulence. It depends on the dimensions of the breakwater(s) and the wave climate which of the processes prevails.

Longshore transport

As mentioned before, the longshore current transporting sediment in longshore direction will be larger when the waves are higher and the angle of approach is larger. In a situation with offshore breakwaters the wave height behind the breakwaters is reduced, which means that the longshore current becomes smaller. It depends on the angle of approach and the layout of the project what the effect will be on the sediment transport, but often some sediment is deposited behind the breakwater and a new shoreline feature develops. In the next chapters the features of the new shoreline will be discussed.

Cross-shore transport

Breakwater construction can reduce offshore transport by presenting a physical barrier to offshore transport. Due to this physical barrier offshore sand losses are reduced, but in a multiple breakwater system, some sand can be lost by the rip currents through the gaps.

3 Application of offshore breakwaters

In Chapter 2 the changes in coastal processes due to offshore breakwater were treated so it became clear what the effects of offshore breakwaters are on waves, currents and sediment transport. In this chapter the application of offshore breakwaters will be discussed: how to use the characteristics of offshore breakwaters in practice.

3.1 Coastal erosion and prevention

There are two different types of coastal erosion:

- Erosion due to storm surges (see Figure 3.1). This type of erosion is often temporary, because during storms sand will be transported from the dunes and the upper part of the beach to deeper water, but under normal wave conditions the sand will return and equilibrium will be restored. The cause of this erosion is cross-shore transport.

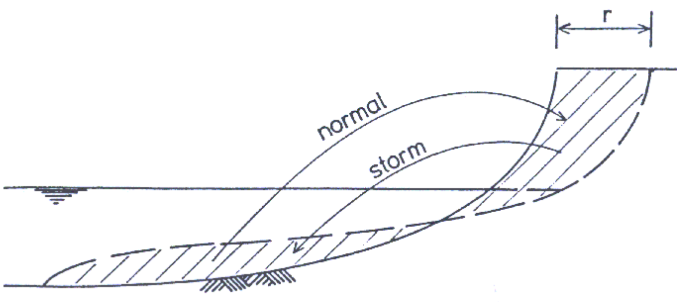


Figure 3.1: Erosion due to storm surges (Velden, van der, 2000)

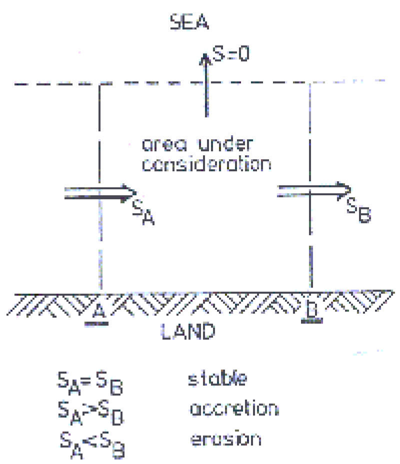


Figure 3.2: Structural erosion(Velden, van der, 2000)

- Structural erosion (see Figure 3.2). In this case the lost volume of sand will (often) not return: it is a structural loss of sand due to a gradient is longshore transport. The shoreface will first erode, but sooner or later also the upper part of the beach will be lost.

As mentioned above, erosion due to storm surges is often temporary and is therefore not as dangerous for the coastline as structural erosion. Structural erosion starts to be a problem when it hampers the use of the coastal zone (for example no recreational beach is left) or even threatens the safety of the land behind the coastline (when the upper part of the beach also starts to erode). When this happens, measures have to be taken to avoid further erosion.

In the past several measures have been used:

- Beach nourishment: the lost sediment is replaced on the beach, but the origin of the erosion is not affected, so the beach nourishment will erode again and new beach nourishment is necessary.

- Shoreface nourishment: almost the same as beach nourishment, but now the sand will not be replaced on the beach but on the shoreface. The waves transport the nourished sand to the beach and the same result as with beach nourishment will occur (with the advantage that the recreation on the beach is not disturbed and only water base equipment is needed). But again the beach will erode again and new shoreface nourishment is necessary.

- Groins: the groins interrupt the longshore transport so erosion will stop where the groins are placed. Due to this interruption of longshore transport, the downdrift side of the groins will suffer severe erosion, thus groins simply displace the problem.

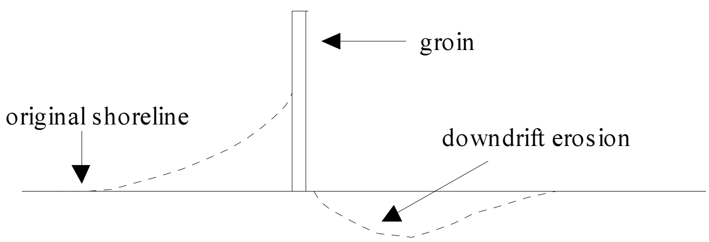


Figure 3.3: groin with downdrift erosion

- Seawalls/revetment: this solution prevents dune erosion and failure of houses and/or roads on these dunes. But with this measure, erosion of the beach will continue till no beach is left and erosion of the toe of the structure makes sudden failure of the whole structure possible.

The last decennia another measure is increasingly used: offshore breakwaters. They have some promising results in preventing beach erosion, but also offshore breakwaters have their disadvantages. They are usually more expensive than other measures, need water based equipment for construction and design experience and guidance is limited. The subject of this report is to improve the design guidance (U.S. Army Corps of Engineers, 1984).

3.2 Offshore breakwaters as shore protection

First some terms used in this paragraph (and the rest of this report) will be explained, see Figure 3.4.

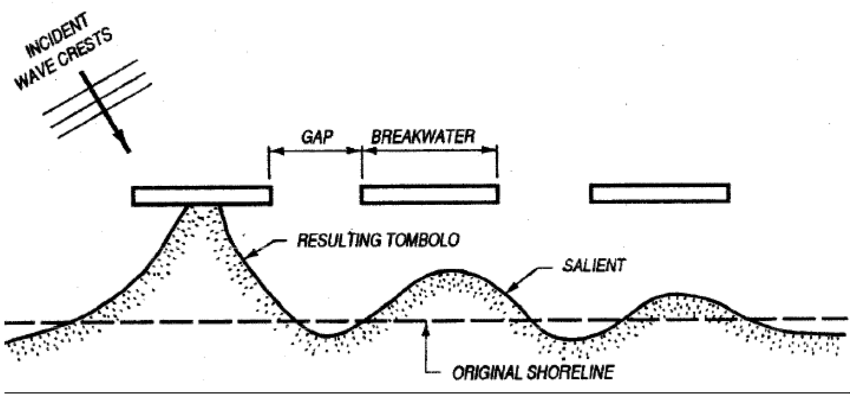


Figure 3.4: Definition of shoreline changes by offshore breakwaters (Coastal Engineering Manual)

A salient will develop when sand is disposed behind the breakwater. This bulge in the shoreline can develop till it reaches the breakwater: this plan form is called tombolo. When a salient is formed, longshore transport can still go on (although on a lower level); a tombolo will act as a total barrier (like a groin) for longshore transport.

Offshore breakwaters can be used for all kinds of shore protection, but there are two situations where offshore breakwaters are used most:

1) An eroding coastline, where a structural loss of sediment occurs.
2) A new recreational beach to attract more tourists.

In this paragraph the effects of offshore breakwaters in these situations will be discussed as well as the limitations that appear.

3.2.1 An eroding coastline

As mentioned in the previous paragraph, a structural eroding coastline is often caused by a gradient in longshore transport. Looking at Figure 3.2 one can say that when the longshore transport rate in A is smaller than in B, the section A-B will erode. Offshore breakwaters can help to reduce or even stop this process.

Application
If a local part of the coastline erodes, a single breakwater will probably satisfy, but for a longer section of the coastline, a segmented system will be necessary. The breakwater(s) reduce(s) the wave energy in the lee of the breakwater, the transport capacity of the water may decrease and erosion of this part

of the coastline can be stopped or even deposition of sand in the lee of the breakwater can occur. The effects of offshore breakwaters on the coastline in this situation depend most on the layout of the project and the angle of approach of waves (every case reacts different). In Japan most offshore breakwaters are used for this situation. An example of an eroding coastline protected by offshore breakwater is shown in Figure 3.5.

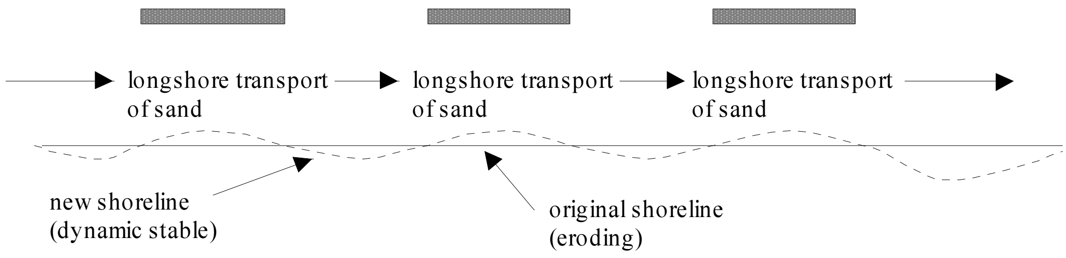


Figure 3.5: Eroding coastline

Limitations
A problem with using offshore breakwaters is downdrift erosion. At this point it makes some difference whether a salient or a tombolo is the final plan form of the beach.

- Salient: longshore transport can partly continue to move through the project area (behind the breakwaters) to downdrift beaches. This means that salients will have negative effects on downdrift beaches, but these effects are less than in a case where a tombolo developed

- Tombolo: a tombolo acts as a total barrier for longshore transport through the project area (behind the breakwaters) to downdrift beaches. The longshore transport may continue offshore of the breakwater(s), but this sediment is not immediately available to the downdrift shore and severe erosion of the downdrift beaches might occur.

In most cases downdrift erosion is not wanted. Therefore, a salient seems a better shoreline response in this situation than a tombolo.

3.2.2 A new recreational beach

A new recreational beach is usually situated on a narrow coastline or at a totally new location where no beaches are sited. To achieve a wide recreational beach, a beach fill is needed. Offshore breakwaters can reduce the volume of sand needed for the beach fill and/or prevent that the beach fill is transported elsewhere, because this new beach is not in natural equilibrium.

Application
As mentioned above, there are two reasons to use offshore breakwaters for recreational beaches:

- Reducing the volume of sand needed for the beach fill: this can be achieved by trapping sediment. When the offshore breakwaters are close enough to the shore and enough sand is available (by longshore transport), tombolos will form behind the offshore breakwaters providing a wide recreational beach/bay. At these places (where a tombolo will develop), one can save on the volume of sand needed for the beach. Especially in Spain a lot of these "pocket"-beaches have been constructed. See Figure 3.6 fro an example.

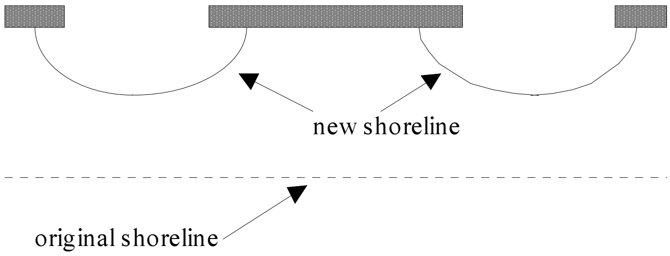


Figure 3.6: Pocket beach

- Preventing transport of the beach sand (to elsewhere): without protection, the beach fill may be transported along the coast (or offshore) and the new beach will disappear. To avoid this transport of sand, offshore breakwaters can be used. Examples of this application are Lakeview Park and Presque Isle in the United States and Muiderburght in the Netherlands. See Figure 3.7 for an example.

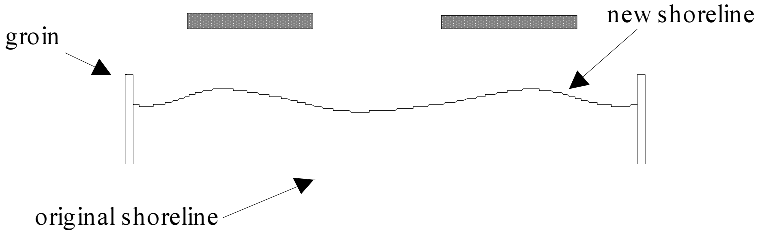


Figure 3.7: New recreational beach

Limitations

If a new recreational beach is built at a sand starving coast, downdrift erosion is not an important issue, because the downdrift beaches already suffer from a lack of sand. Thus for both situations it does not matter whether a salient or a tombolo will form: the (extra) effect on downdrift beaches is minimal for both plan forms, since the coast already suffers from a lack of sand. However, in the second situation another criterion does determine the preferred plan form: the question if access to the structure should be allowed. Advantage of this access is that monitoring and maintenance of the structure are facilitated. However, this means that also the beach users have access to the structure and the area immediately adjacent to it: this can be very dangerous. So if no sediment needs to be trapped, a salient is the preferred beach response for a recreational beach.

3.3 Offshore breakwater application in this report

First a summary will be given of the different situations mentioned in the previous paragraph, with their intended effects and their limitations. After that, the focus of this report on offshore breakwaters will be described.

Eroding coastline:
- Stops structural erosion
- Downdrift erosion possible
- Reduced downdrift erosion if a salient develops

Trapping sediment for new beach:
- Traps sediment behind the breakwaters
- Reduces volume of sand needed for beach fill
- Tombolos will form a wide beach

Keeping recreational beach wide enough:
- Prevents transport of beach fill to other places
- Tombolo or salient can form
- Salient preferred because of safety

In the first and third situation a salient is the preferred beach response. In the second situation a tombolo is preferred. To assure tombolo formation, the offshore breakwaters have to build close enough to the shore. However it is not a big problem if a salient will form instead of a tombolo: the beach will be a bit narrower, but no further problems will develop. This is different for the other two situations. If a tombolo develops instead of a salient, the downdrift beaches will have more downdrift erosion and beach users can reach the breakwater which can be very dangerous. So especially for these situations it is important to predict the beach response well: if the breakwaters are too short, the beach can (still) erode, but if the breakwaters are too long, a tombolo can develop resulting in negative effects. In the table below these effects are summarized for both cases.

Table 3.1: Breakwater effects on shoreline

Preferred beach respons	**Breakwater(s) too small**	**Breakwater(s) too large**
Tombolo	Salient : narrower beach	Large tombolo: more beach
Salient	Erosion: project failed or yearly beachfill is needed	Tombolo:downdrift erosion/dangerous swimming

So especially for situations where a salient is the preferred beach response, prediction of this beach response is very important, because the effects of a wrong prediction are worse than for the situation where a tombolo is preferred. Therefore this thesis report will focus on the situations where a salient is the preferred beach response.

4 Predicting shoreline response

In Chapter 3 the importance of a good prediction of the shoreline response was discussed briefly. In this chapter different methods for predicting this response will be treated and one method will be described in detail.

4.1 Prediction methods

In general, three tools can help to predict the shoreline response behind offshore breakwaters: physical and numerical models, prototype tests and empirical relationships. With physical and numerical models some information about the effects of a particular design can be measured. But this tool can be very expensive and time consuming, and due to model instability or inaccurate input the user can get wrong impressions of the effects of offshore breakwaters on the shoreline response. Prototype tests of a proposed design are the best method in detailing of the final design, although it can be very expensive and is not applicable in preliminary design (or it will cost even more). Empirical relationships are quick, inexpensive methods that can evaluate the beach response to a proposed design. Often, however, the relationships are site specific and simple. Therefore problems can occur when using them at a different site or under different conditions. In Table 4.1 the advantages and disadvantages of the three methods are shown.

Table 4.1: methods to predict; advantages and disadvantages

Method	**Advantages**	**Disadvantages**
Physical and numerical models	Information about effects of a particular design	Expensive Time consuming Risk of numerical instability
Prototype tests	Best for detailing final design	Not applicable in preliminary design
Empirical relationships	Quick Inexpensive	Site specific Too simple

To ensure a successful project Dally and Pope (1986) suggested a combination of the three tools: first, a desk study with various empirical relationships to identify design alternatives. Second, a physical or numerical model study fro detailing the alternatives. And finally a prototype test to verify the proposed design and to see how the shoreline adjusts to the site conditions.

In this report the emphasis is on the empirical relationships: they are often rather simple (e.g. only a ratio length structure –distance offshore is assumed to determine the shoreline response) and they depend on site specific conditions. This thesis report tries to overcome these problems by investigating the effect of wave climate on the shoreline response. When you know these effects, the wave climate can also be a part of the empirical relationships and shoreline response may be predicted better.

Although the empirical relationships differ from each other, they have one thing in common. Their formulation is the same (see Table 4.2).

Structure Length (L_s)/ Distance Offshore (Y) > *a* → tombolo formation

Structure Length (L_s) / Distance Offshore (Y) < *b* → salient formation

Table 4.2: Empirical relationships (Coastal Engineering Manual)

Condition	Comments	Reference
Conditions for the Formation of Tombolos		
$L_s/Y > 2.0$		*Shore Protection Manual* (1984)
$L_s/Y > 2.0$	Double tombolo	Gourlay (1981)
$L_s/Y >$ 0.67 to 1.0	Tombolo (shallow water)	Gourlay (1981)
$L_s/Y > 2.5$	Periodic tombolo	Ahrens and Cox (1990)
$L_s/Y >$ 1.5 to 2.0	Tombolo	Dally and Pope (1986)
$L_s/Y > 1.5$	Tombolo (multiple breakwaters)	Dally and Pope (1986)
$L_s/Y > 1.0$	Tombolo (single breakwaters)	Suh and Dalrymple (1987)
$L_s/Y > 2\ b/L_s$	Tombolo (multiple breakwaters)	Suh and Dalrymple (1987)
Conditions for the Formation of Salients		
$L_s/Y < 1.0$	No tombolo	*Shore Protection Manual* (1984)
$L_s/Y <$ 0.4 to 0.5	Salient	Gourlay (1984)
$L_s/Y =$ 0.5 to 0.67	Salient	Dally and Pope (1986)
$L_s/Y < 1.0$	No tombolo (single breakwater)	Suh and Dalrymple (1987)
$L_s/Y < 2\ b/L_s$	No tombolo (multiple breakwater)	Suh and Dalrymple (1987)
$L_s/Y < 1.5$	Well-developed salient	Ahrens and Cox (1990)
$L_s/Y <$ 0.8 to 1.5	Subdued salient	Ahrens and Cox (1990)

As can be seen from Table 4.2, all these relationships have other values for *a* and *b*. This is due to the site specific character of the relationships; one relationship is deducted from Japanese projects, another from laboratory results, and a third one from theory in combination with US-projects (in Appendix B a short description of the origin of the relationships is given (Rosati, 1990)). Because it is infeasible to evaluate all these different relationships with their different wave climates and site conditions, one relationship is chosen for this thesis report: the empirical relationship from the Shore Protection Manual (1984), called the diffraction method. This is because this relationship is deducted from a theory (instead of former projects) and therefore it can be evaluated more easily.

4.2 Diffraction method

The diffraction method can be divided in two parts: general theory and practical application. They will be treated in this paragraph successively

4.2.1 Theory for diffraction method

The basis for this method is the following (cited from Shore Protection Manual):

"If the incident breaking wave crests are parallel to the original shoreline (which is a condition of no longshore transport): the waves diffracted into the offshore breakwater's shadow will transport sand from the edges of this region into the shadow zone. This process will continue until the shoreline configuration is essentially parallel to the diffracted wave crests and the longshore transport is again zero. In this instance the cuspate spit will have a symmetric shape. See Figure 4.1

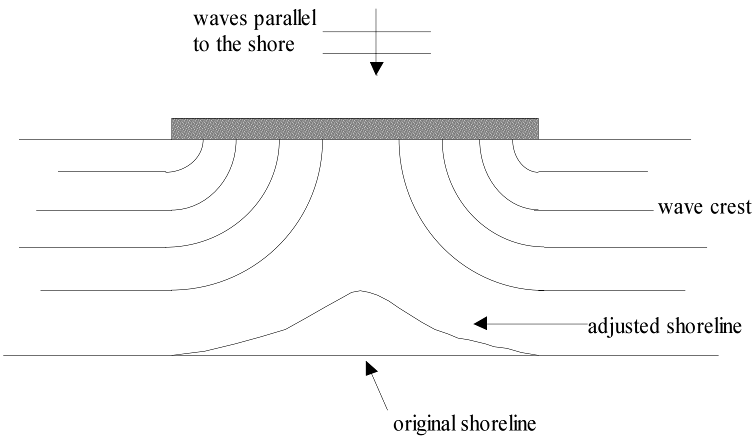


Figure 4.1: Adjusted shoreline to parallel incoming waves (Chasten, Rosati, McCormick, 1993)

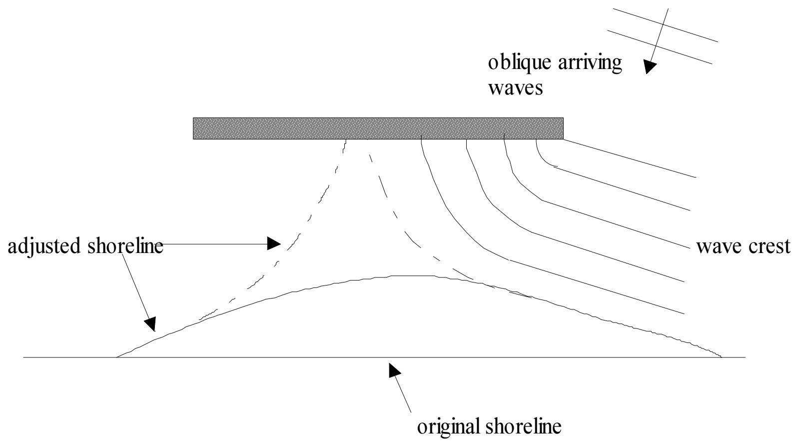


Figure 4.2: Adjusted shoreline to oblique arriving waves (Chasten. Rosati, McCormick, 1993)

For oblique incident waves the longshore transport rate in the lee of the breakwater will initially decrease, causing deposition of the longshore drift. A cuspate spit is formed which will continue to grow until either the longshore transport rate past the structure is reestablished or a tombolo is formed. The cuspate spit that results from oblique wave attack can be expected to be asymmetric. See Figure 4.2."

In this report only situations with parallel incident waves will be treated. This means situations where the final shoreline configuration only depends on the wave pattern and not directly on the longshore transport rate (which is most important for a situation with oblique incident waves).

4.2.2 Practical application: determining shoreline response

The above mentioned theory assumes that the final shoreline configuration is reached when the diffracted wave crests are parallel to this new shoreline. In this way, the final shoreline configuration can be approximated by drawing the pattern of the diffracted wave crests. The approximate location of the cuspate spit (or salient) is found at the point where the diffracted wave crests (around each end of the breakwater) intersect each other at the time that the undisturbed part of the wave reaches the shoreline. In Figure 4.3 this is shown.

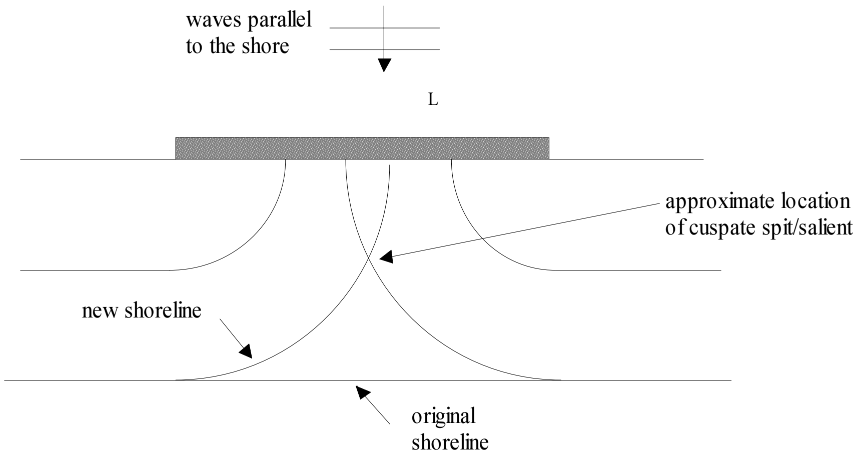


Figure 4.3: Approximate location of cuspate spit apex according to diffraction method

The limits for assuring and preventing tombolo formation can be deducted for a simplified case where the depth behind the structure is constant. In case of a bottom slope two effects play a role in the pattern of the diffracted wave crests. First, the wavelength becomes smaller in the shallower part of the water which leads to a change in the diffraction pattern. This means that the part of the undisturbed part of the wave has smaller wavelengths than the diffracted part of the wave, because this last part travels in water with a larger depth. The effect of this changed diffraction pattern is that the shoreline will be less extended in case of a bottom slope than in case without a bottom slope. Another effect of reduced wavelengths is that the wave height increases in shallow water (until the waves break). This is called shoaling, but these increased wave heights do not affect on the diffraction pattern, only on the wave heights along a diffracted wave crest.

Another effect not taking into account in this simplified case is refraction. Due to refraction the diffracted waves will bend towards the coastline and the shoreline will also be less extended. From these two effects, reduced wavelengths and refraction it can be concluded that in practice the shoreline will be less extended than the diffraction method pretends.

However, for the simplified case without bottom slope the limits for tombolo or salient formation are:

Length Structure / Distance Offshore > 2 assuring tombolo

Length Structure / Distance Offshore < 2 preventing tombolo

To assure tombolo formation, the length of the structure should be at least two times the distance offshore. In this way the two diffracted wave crests behind the breakwater do not intersect before the undisturbed part of the wave reaches the shoreline. If the length of the structure is less than two times the distance offshore, the diffracted wave crests behind the breakwater do intersect before the undisturbed part of the wave reaches the shoreline. See Figures 4.4 and 4.5 for a visual explanation of these limits.

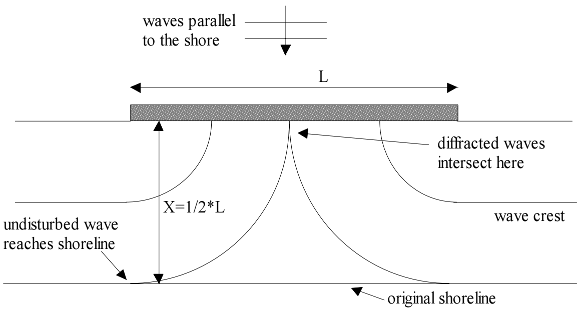


Figure 4.4: Limit for tombolo formation

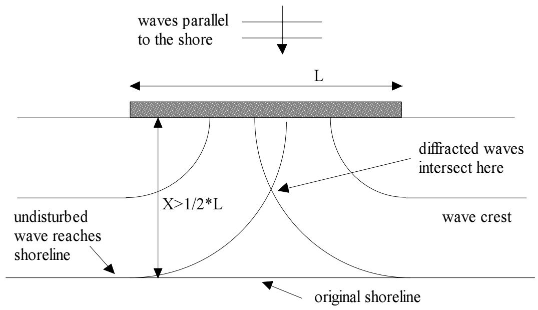


Figure 4.5: Limit for salient formation

4.3 Evaluation of the diffraction method

The diffraction method gives a rule of thumb which determines the shoreline response. This method is very simple and looks quite reliable. However, this method has two main limitations. First, in this method only the distance offshore and the length of the breakwater are taken into account, while more parameters determine the shoreline response. Second, the assumed process of sand transport towards the area behind the breakwater is too simple. There is another, maybe even more important sand transporting process. These limitations are the basis for Chapter 5 and 6.

4.3.1 Effect other parameters on shoreline response

As most empirical relationships, the diffraction method is of the simplest form: only the ratio length structure – distance offshore is assumed to determine the shoreline response. But there are more parameters which determine this response. Hanson & Kraus (1990) mention fourteen parameters influencing the shoreline response:

Breakwater properties:
- Length of the structure segment (L)
- Distance of segment from original shoreline (Y)
- Structure segment transmissivity (K)
- Gap distance between two segments (G)

Beach properties:
- Depth at structure segment (h)
- Variations in water depth, as from tide (δh)
- Beach median grain size (d_{50})

Wave properties:
- Wave height (H)
- Wave period (T)

- Predominant wave angle (θ)
- Orientation of the structure (θ_s)
- Standard deviation of wave height (σ_H)
- Standard deviation of wave angle (σ_θ)
- Standard deviation of wave period (σ_T)

All these parameters affect the final response of the shoreline behind an offshore breakwater, but these parameters are not taken into account in the diffraction method. In Figure 4.6 these parameters are shown.

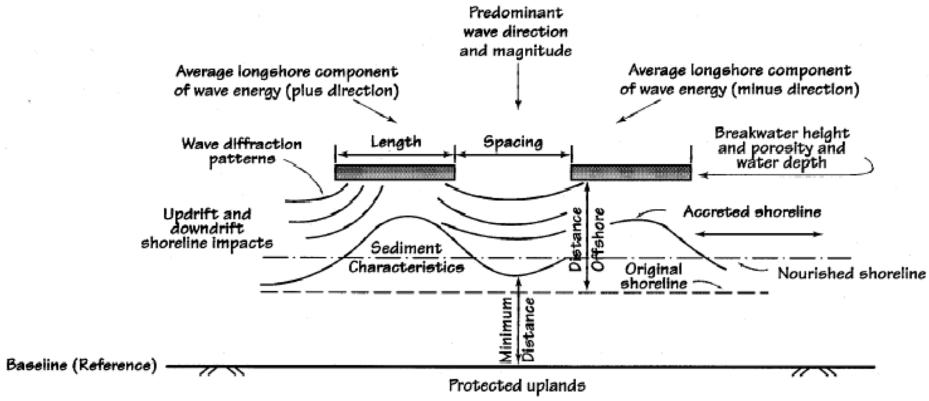


Figure 4.6: Parameter definition offshore breakwaters and shoreline response (Coastal Engineering Manual)

The initiative of this study was the project Muiderburght (see Appendix A), where three offshore breakwaters had to be designed. Here the question rose what the effect of this location would be on the shoreline response. Most empirical relationships are namely deducted from former projects, constructed in oceans, while this project was constructed in a lake. Because wave climate is the main difference between lakes and oceans, this parameter will be studied in this report. The effect of wave climate on the shoreline response will get clear in the next paragraph.

4.3.2 Sand transporting process

The diffraction method developed a simple explanation of sand transport towards the area behind the breakwater, forming a tombolo or a salient. The waves diffracted into the offshore breakwater's shadow will transport sand from the edges of this region into the shadow zone. This is because the diffracted waves approach the original shoreline under an angle directed towards the middle of the sheltered area. This process will continue until the shoreline configuration is essentially parallel to the diffracted wave crests (see Figure 4.1). In that situation the longshore transport is zero and the assumed equilibrium is reached. However, this process of sand transport is too simple. Another sand transporting process takes also place in the vicinity of offshore breakwaters and this process may be even more important for the transport of sand into the shadow zone behind the breakwater. However, the diffraction method does not take this process into account.

In Chapter 2 it has already been explained that the (breaking) wave height behind the breakwater is smaller than the (breaking) wave height next to the breakwater. This difference leads to a difference in wave setup. The wave setup behind the breakwater is smaller than the wave setup next to the breakwater. This alongshore gradient in wave setup leads to an alongshore current from the area next to the breakwater towards the area behind the breakwater. See Figure 4.7 for a visual explanation.

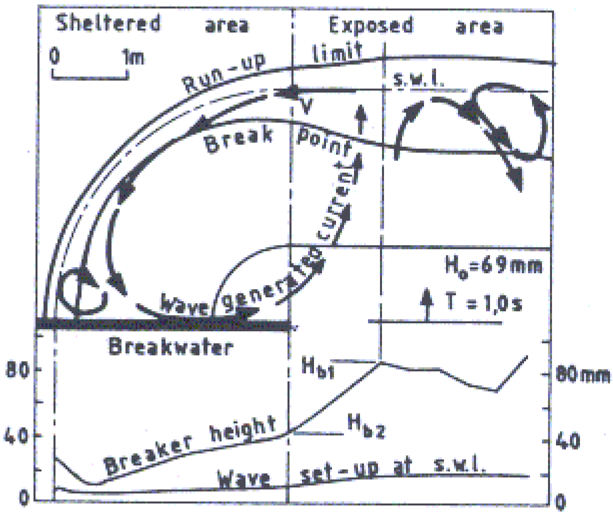


Figure 4.7: Alongshore current due to gradient in wave setup (Gourlay, 1978))

This alongshore current leads to sand transport from the area next to the breakwater towards the area behind the breakwater. This process of sand transport also exists when waves approach the new shoreline perpendicularly, because also in that situation the breaking wave height and the wave setup behind the breakwater are smaller than next to the breakwater. The alongshore gradients in breaking wave height and in wave setup depend on the incoming wave height. This means that a different wave height will cause a different alongshore current velocity and leads to a different volume of transported sand. This last effect has consequences for the final plan form and makes clear that wave climate does affect the shoreline response.

The consequence of this alongshore current transporting sand from the area next to the breakwater towards the area behind the breakwater is that the shoreline starts to rotate. Waves will not approach the new shoreline perpendicularly anymore (like assumed in the diffraction method), but approach the new shoreline under an angle. The effect of this rotation of the shoreline is that the oblique incoming waves cause an opposing current from the area behind the breakwater towards the area next to the breakwater. When this opposing current balances the current due to the gradient in wave setup, equilibrium of the shoreline can be expected. Figure 4.8 shows these two opposing currents.

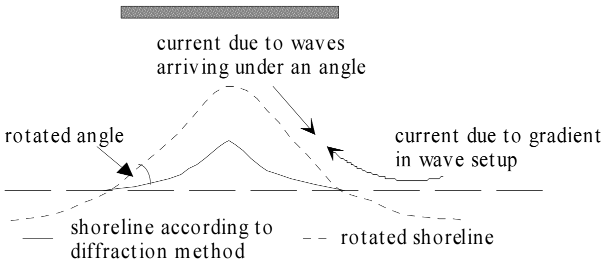


Figure 4.8: Opposing currents in the vicinity of offshore breakwaters

Looking at this (other) sand transporting process, it can be concluded that the diffraction method is too conservative in predicting the volume of sand transported into the shadow zone behind the breakwater. In a start situation with the original shoreline, the sand transporting process due to diffracted waves approaching the original shoreline under an angle will be reinforced by a sand transporting process due to a gradient in wave setup. In a situation where the diffracted waves approach the new shoreline perpendicularly (this means no transport due to oblique arriving of diffracted waves), which is seen as the final plan form by the diffraction method, the sand transporting process caused by the gradient in wave setup will still transport sand into the shadow zone. This extra volume of sand is not taken into account in the diffraction method. Finally, equilibrium is reached when the diffracted waves approach the new shoreline under an opposing angle and work against the sand transporting process due to the gradient in wave setup. Thus, at the beginning the two mechanisms of sand transport reinforce each other, while at the end (when equilibrium is supposed to be reached) these two mechanisms cancel each other out.

In Chapter 5, the gradients in breaking wave height and in wave setup will first be quantified. After that the effect of this gradient, an alongshore current leading to sand transport towards the area behind the breakwater, will be treated. This will be done with the help of a momentum balance for parallel arriving waves (Gourlay, 1978). Chapter 6 describes when equilibrium can be expected. Therefore the momentum balance introduced in Chapter 5 will be extended with the effects of a rotated shoreline. This new momentum balance can help to find out when the current due to the gradient in wave setup is balanced by the current due to oblique incoming waves. The effect of wave height is integrated in both chapters.

5 Consequences gradient in wave setup

In the previous chapter the effect of gradients in breaking wave height and in wave setup on the sand transporting process in the vicinity of offshore breakwaters is described. In this chapter, these gradients will be quantified and the effects will be discussed more detailed.

First a global description of wave setup will be given after which the wave setup around offshore breakwaters will be discussed more specifically. How this gradient leads to an alongshore current will be described in paragraph 5.3. The method in that paragraph is also the basis for Chapter 6. At the end of this chapter, the effect of the alongshore current on the shoreline will be treated.

5.1 Theory wave setup

As mentioned in Chapter 2, wave setup can considerably contribute to the water level changes near the coast. The water level drops near the break point (with a maximum depression at the break point) and from this break point the mean water level slopes upward to the shore. The theory behind this phenomenon will be discussed in this paragraph (Battjes, 2000; Velden, van der, 2000).

Figure 5.1 shows the terms used in the following equations

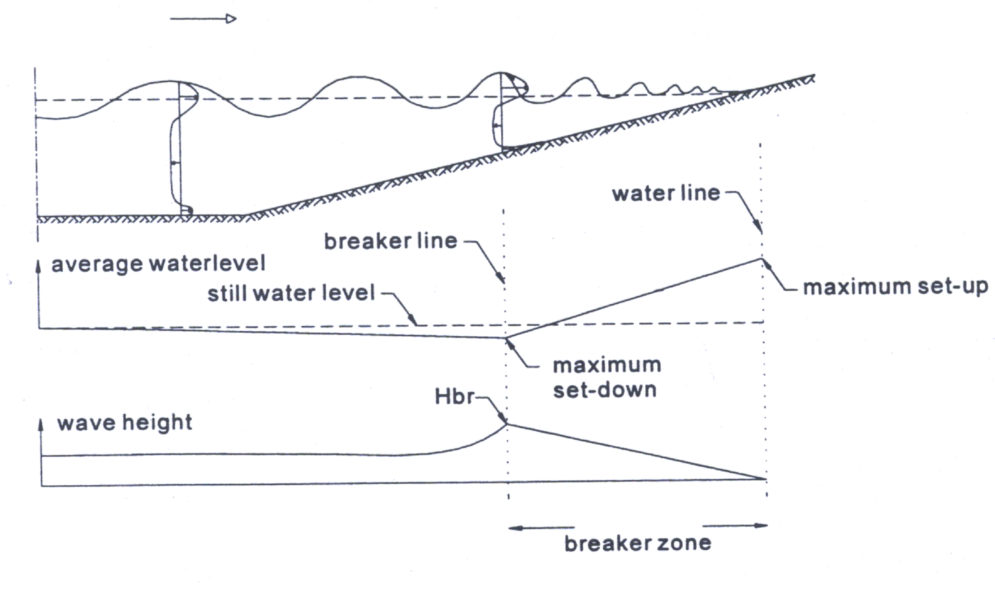


Figure 5.1: Term definition of the wave setup and setdown

Wave setup is the super elevation of the mean water level caused by wave action. This elevation of the mean water level balances the gradient in the cross-shore directed radiation stress. The equilibrium between this radiation stress change and the average water level slope yields the following equation:

$$\frac{d\bar{\eta}}{dx} = -\frac{1}{\rho g h}\frac{dS_{xx}}{dx} \qquad (5.1)$$

where:

η = mean water surface elevation above still-water level (m)
ρ = mass density of water (kg/m^3)
g = acceleration due to gravity (m/s^2)
h = local still-water level (m)
x = horizontal coordinate cross-shore to the coast (m)
S_{xx} = cross-shore component of the cross-shore directed radiation stress (N/m)

Using the boundary condition for deep water ($\eta = 0$), a formula for setdown can be deducted. At the breaker line, where maximum setdown occurs and $H = \gamma h$, the setdown will be:

$$\bar{\eta} = -\frac{1}{16}\gamma^2 h_{br} \qquad (5.2)$$

where:

γ = breaker depth index (-)
h_{br} = water depth at break point (m)

Using the assumption that $H = \gamma h$ from break point to the coastline and supposing that shallow water wave theory can be applied in this situation, the total water elevation of the water level between break point and shoreline will be:

$$\Delta\eta = \frac{3}{8}\gamma^2 h_{br} \qquad (5.3)$$

where:

Δη = setup between break point and shoreline (m)

To obtain the maximum setup at the shoreline, this total water elevation should be reduced with the setdown at the breaker line (see Figure 5.2)

$$\eta_s = \eta_b + \Delta\eta = -\frac{1}{16}\gamma^2 h_{br} + \frac{3}{8}\gamma^2 h_{br} = \frac{5}{16}\gamma^2 h_{br} \qquad (5.4)$$

This equation shows that the maximum wave setup is linearly dependent on the breaking wave height (or breaker depth). The next paragraph shows the effects of this relation for the diffraction zone behind an offshore breakwater.

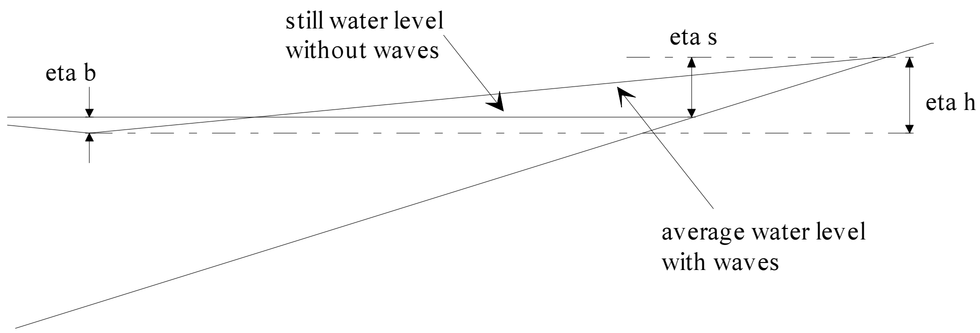


Figure 5.2: Average water level with waves

It is empirically found (Dally&Pope, 1986) that the average wave height is most important for developing a final plan form. Therefore, this study will use the average wave height in the calculations.

5.2 Wave setup in the vicinity of offshore breakwaters

Also mentioned in Chapter 2 is the difference in wave setup next to and behind a breakwater. The wave height in the sheltered part behind the breakwater is smaller than the wave height next to the breakwater. This makes that the wave setup behind the breakwater is smaller than the wave setup next to the breakwater. This paragraph will quantify these differences by using the theory described in the previous paragraph and using the diffraction diagram from the Shore Protection Manual (1984).

5.2.1 Diffraction pattern

Diffraction occurs when there is a sharp variation in wave energy along a wave crest. In case of an offshore breakwater, in first instance there are no waves in the lee of the breakwater. Therefore a gradient in wave energy develops along the wave crest and energy will be transported to the part behind the breakwater. Bending waves are the result.

The degree of diffraction depends on the ratio of the dimensions of the obstacle (the length of the offshore breakwater) and the wavelength. A thin pile will cause almost 100% diffraction, which implies that the wave field is almost the same as when there is no pile. While a long breakwater has a large zone with a disturbed wave field. The diffraction coefficient shows the degree of diffraction:

$$K_d = \frac{H_d}{H_i} \tag{5.5}$$

where:
K_d = diffraction coefficient (-)
H_d = diffracted wave height (m)
H_i = wave height of the undisturbed incident wave (m)

Diffraction diagrams show the lines of equal diffraction coefficients behind and next to obstacles. The diffraction diagram in Figure 5.3 shows a semi-infinite breakwater. In case of an offshore breakwater the diffraction diagram can be obtained by superimposing two semi-infinite breakwater diagrams. In this study however, the diffraction pattern behind the breakwater is simplified by a mirror placed in the middle of the breakwater.

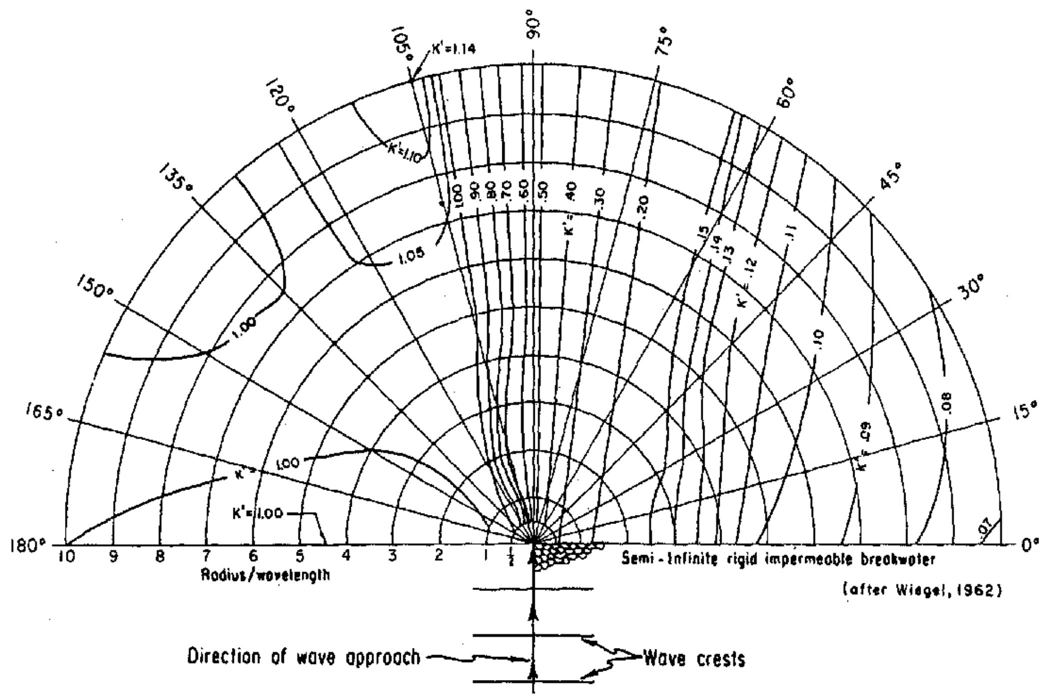


Figure 5.3: Diffraction pattern around offshore breakwater (Shore Protection Manual)

Figure 5.3 shows that in case of an offshore breakwater, the diffraction coefficient at the boundary between exposed and sheltered area is 0.55, which means that the wave height at that location is only 0.55 times the wave height of the waves in front of the breakwater. Towards the middle of the breakwater (where the mirror is put) the wave height gets smaller, while on the other side (in the exposed area next to the breakwater) the wave height climbs up to the original wave height in front of the breakwater. The consequence of this is that in a relatively small area there are quite large gradients in wave height.

Attention should be paid to the scale of the diagram, because this scale is the ratio radius – wave length (at the breakwater). This means that the diffraction pattern behind a breakwater and, more important, the distances between two lines with different diffraction coefficients change when the wave period changes. The distance between two points measured in the diagram will remain the same for different wave lengths. But due to the scale of the diagram this distance has to be multiplied with the wavelength (at the breakwater) to obtain the real distance between these two points. It will be clear that a different wavelength (at the breakwater) will lead to different distances. This has an effect on the magnitude of the gradients in breaking wave height and in wave setup. These gradients can be calculated by dividing the differences in breaking wave height and wave setup between two certain points by the distance between these two points. When this distance changes, the gradient will also change. Quantification of the effect of the scale of the diffraction diagram will be shown in the next paragraphs.

5.2.2 Wave setup in the diffraction zone behind an offshore breakwater

As seen in the previous paragraph, offshore breakwaters cause large gradients in wave height along the coast. This has consequences for the breaking wave height and therefore for the wave setup. In the diffraction zone two important phenomena occur:

1. There is a considerable alongshore gradient in breaking wave height: the lowest breaking wave height behind the offshore breakwater is much smaller than the breaking wave height in the exposed area next to the breakwater.

2. As a consequence of a large variation in breaking wave height, there is also a corresponding variation in the width of the surf zone. A larger breaking wave height corresponds with a greater breaker depth and with a given bottom slope the breaker line is located further offshore.

The first characteristic leads to a gradient in maximum wave setup, according to formula 5.6. By quantifying the gradients in both breaking wave height and wave setup, the earlier derived formulas are used. With these formulas the gradients in breaking wave height and wave setup can be calculated.

$$\eta_s = \frac{5}{16}\gamma^2 h_{br} \qquad (5.6)$$

where:
η_s = setup at the shoreline (m)
γ = breaker index = 0.8 (-)
h_{br} = water level at the break point: (m)

$$h_{br} = \frac{H_{br}}{\gamma} = \frac{K_s K_d H_0}{\gamma} \qquad (5.7)$$

where:
H_{br} = wave height of breaking waves (m)
K_s = shoaling coefficient, determined iteratively (-)
K_d = diffraction coefficient (-)
H_0 = wave height at deep water (m)

The locations where the breaking wave height and the wave setup will be calculated are derived from the diffraction diagram. In the diffraction diagram, the layout of a project will be drawn with the adjusted shoreline according to the diffraction method. At the locations where the lines of equal diffraction cross this shoreline, the wave height and wave setup will be calculated. See Appendix C.

As one can see, refraction (K_r) is not taken into account. According to the diffraction method, waves approach the new shoreline perpendicularly. Assuming that the other bottom contours adjusted themselves to this new shoreline, the diffracted waves will approach all bottom contours perpendicularly and the waves will not be subjected to refraction.

The gradient in wave setup is:

$$gradient = \frac{\Delta \eta_s}{\Delta y} \qquad (5.8)$$

where:
$\Delta\eta_s$ = difference in wave setup between two locations (m)
Δy = distance between these two locations alongshore (m)

The gradient in breaking wave height is:

$$gradient = \frac{\Delta H_{br}}{\Delta y} \qquad (5.8b)$$

where:
ΔH_{br} = difference in breaking wave height between two locations (m)
Δy = distance between these two locations alongshore (m)

The locations where the breaking wave height and wave setup are calculated are described above. The distance between these two locations is quite simple to obtain. The distance between two locations should first be measured in the diffraction diagram as a ratio distance – wavelength. Multiplying this ratio with the wavelength (at the breakwater) gives the real distance between these two locations (see Appendix C).

The consequence of these gradients in breaking wave height and in wave setup is an alongshore current directed from the exposed area next to the breakwater towards the sheltered area behind the breakwater (as explained in Paragraph 4.3). This alongshore current will be treated in Paragraph 5.3.

5.2.3 Quantification of gradient in wave setup

A quantitative evaluation of the gradients in wave setup and in breaking wave height is done for the project Lake View Park. This project will be used in Chapter 6 as validation project. By quantifying these gradients, the development of both gradients along the shoreline will become clear. And the effect of wave height on these gradients can be investigated.

The main dimensions of this project are shown in Figure 5.4 with the assumed equilibrium shoreline according to the Shore Protection Manual drawn in it: wave crests parallel to the coast. These dimensions are from Pope & Rowen (1983). They studied the shoreline response of this project and therefore they obtained many data about this project. Here, only the start dimensions of the project are used. In Paragraph 6.3, more data will be used from this study.

This situation (with the dimensions of Lake View Park) is subjected to waves of 1 m height and a wave period of 5 seconds. To investigate the effect of wave height in this study, it is chosen to use wave heights of 1, 2 and 3 meters. The wave period (for a wave height of 1 m.) is chosen to be 5 seconds. With this combination of wave height and wave period, the wave steepness for deep water is 0.026, which is a realistic value.

The breaking wave height and wave setup along the shore are shown in Figure 5.5, with the x-axes from the break point of the top of the salient (in the middle of the sheltered area), along the breaker line, to the break point of the place where the diffraction coefficient is 1 (in the formulas this is the y-direction). Appendix C (Figure C.1 and Table C.1) shows how the values in Figure 5.5 are calculated.

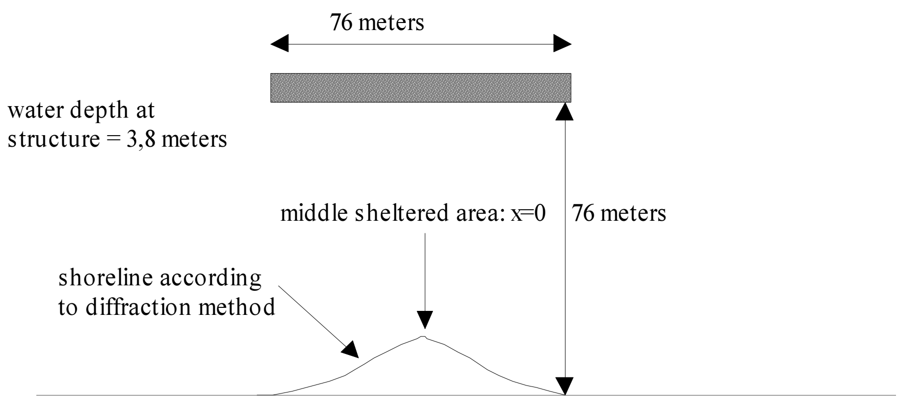


Figure 5.4: Dimensions Lake View Park

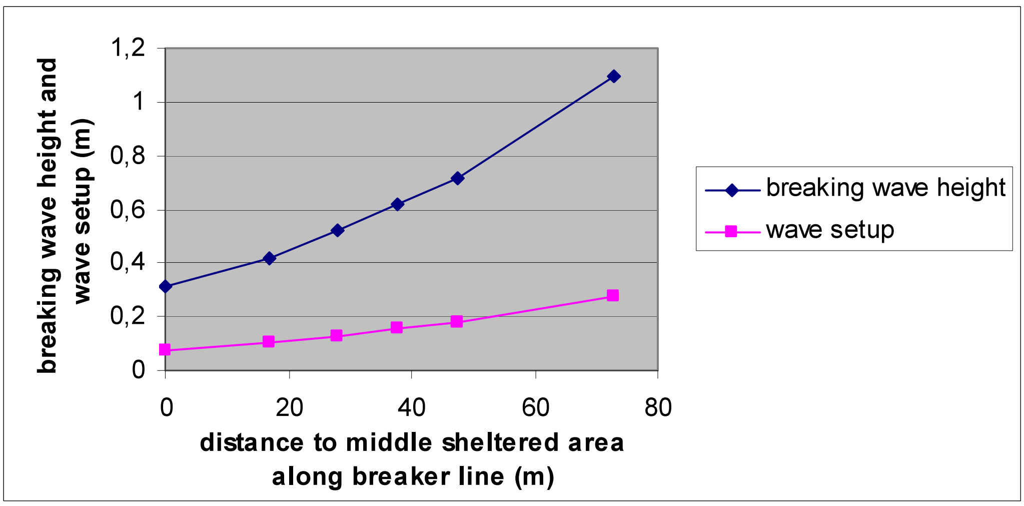


Figure 5.5: Wave setup and breaking wave height along shoreline

From Figure 5.5 it can be concluded that the variation in both the breaking wave height and the wave setup are large along the shoreline. The values at the top of the salient behind the breakwater are approximately 20% of the values next to the breakwater, where the diffraction coefficient is 1. These large variations lead to large gradients in breaking wave height and in wave setup. The effect of these gradients will be treated in Paragraph 5.3.

5.2.4 Effect wave height

Doing the same calculation with larger waves can give insight in the effect of wave height on the gradients in breaking wave height and in wave setup. The steepness of the waves is chosen to be the same for all simulations. This is done because in this study the wave height is the subject of investigation. By keeping the wave steepness constant, this wave steepness will not affect the results of that investigation. The results of the calculation are given in Figure 5.6. Appendix C shows how these gradients are calculated and what the values of Δh_{br}, $\Delta\eta$ and Δy are, leading to the diagrams in Figure 5.6 and 5.7.

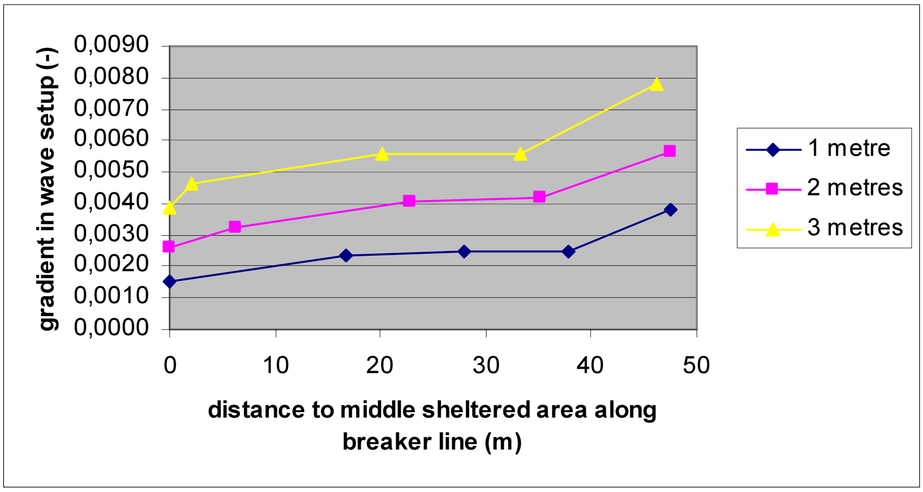


Figure 5.6: Effect wave height on gradient in wave setup

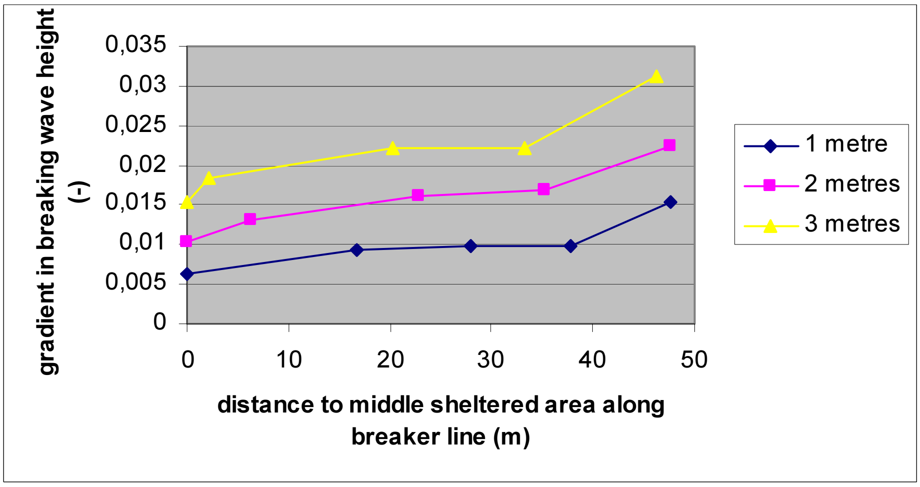


Figure 5.7: Effect wave height on gradient in breaking wave height

From Figure 5.6 and 5.7, it can be observed that higher waves cause a considerable larger gradient in both breaking wave height as wave setup. In principle, Δh_{br} and $\Delta\eta$ are linearly proportional to the incoming wave height (see Appendix C). However, because the diffraction diagram scales with the wavelength (the scale is the ratio radius – wavelength) it is found that the gradients in breaking wave height and in wave setup are smaller than linearly proportional. It can also be observed that both gradients are largest next to the breakwater and get smaller towards the middle of the sheltered area.

5.3 Gradient generated current

The gradients in wave setup and in breaking wave height lead to a current towards the sheltered area, as mentioned in Paragraph 4.3.2. Gourlay (1978) developed a method to calculate the velocity of this alongshore current. With the help of the differential equation of motion in alongshore direction he set up a momentum balance for a certain control volume and from this momentum balance he derived an expression for the alongshore current velocity. This momentum balance is also the basis for Chapter 6. In that chapter, the momentum balance of Gourlay (applicable for parallel arriving waves) will be extended with the effects of a rotated shoreline. And with the help of this extended momentum balance equilibrium of the shoreline is tried to find. In this paragraph the background of this method of Gourlay is described and the alongshore current velocity is calculated with this method.

5.3.1 Introduction method of Gourlay

From observations in the laboratory, Gourlay divided the area around offshore breakwaters in two regions. First, there is the inflow region within the exposed area next to the breakwater extending up to the geometric shadow of the breakwater (the boundary between exposed and sheltered area). In this region the primary alongshore current is developed. Second, there is the region behind the breakwater, where momentum transfer occurs to a secondary induced eddy inside the overall circulation pattern.

Gourlay concentrated his study on the first region (see Figure 5.8). In this region the gradients in breaking wave height and in wave setup are large(st) and surf zone characteristics predominate the inflow. This region of interest (concentrated on by Gourlay) is interesting for this report, because in this region the alongshore current develops and the effect of this current is investigated in this report. Furthermore, the effect of wave height is investigated in this report too, and according to Gourlay this characteristic affects the alongshore current.

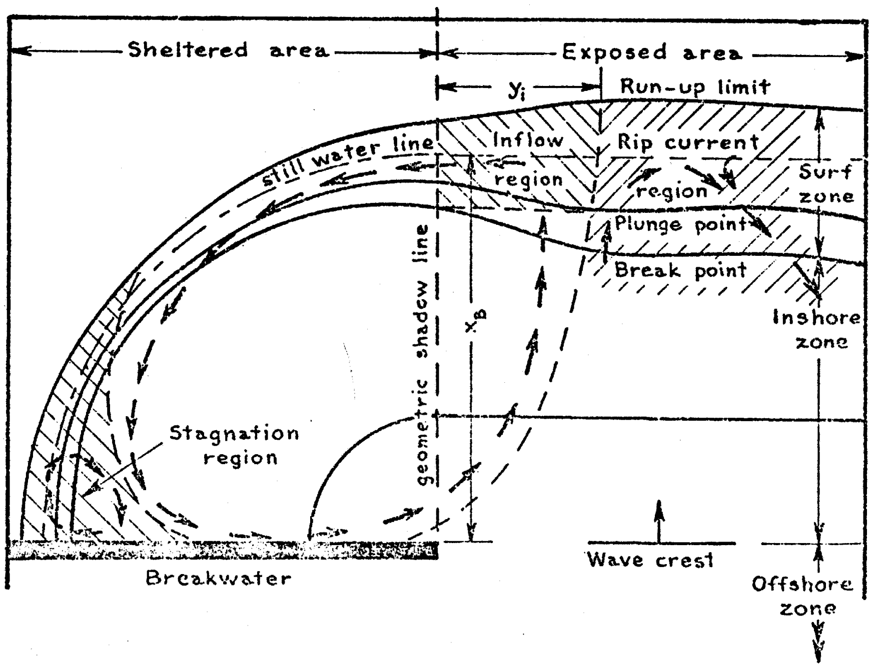


Figure 5.8: The two regions in the vicinity of an offshore breakwater (Gourlay, 1978)

With the help of the differential equation of motion (and some assumptions) a simple analytical expression for the alongshore current velocity can be formulated. The two main assumptions are:

- From observations in the laboratory he assumed that inflow into the alongshore current ceases at the geometric shadow line (where K_d is 0.55). This means that the alongshore current velocity has its maximum at this point.

- Wave setdown at the breakpoint is sufficiently small in comparison with the breaker depth, so this setdown can be neglected.

As mentioned before, Gourlay used the differential equation of motion to set up a momentum balance. This differential equation of motion in alongshore direction is:

$$\rho(\eta + h)v\frac{\partial v}{\partial y} + \tau_b + \tau_L = \frac{\partial S_{yy}}{\partial y} + \rho g(\eta + h)\frac{\partial \eta}{\partial y} \qquad (5.9)$$

where:

first term	= effect of a gradient in the alongshore component of the radiation stress
second term	= effect of the gradient in wave setup in alongshore direction
third term	= acceleration of the flow in alongshore direction
fourth term	= bottom friction
fifth term	= lateral mixing

For a simple engineering solution he chose a control volume type of analysis and the effects of bottom friction and lateral mixing were initially neglected, hence τ_b and τ_L are equal to zero, which means no shear stresses working against the direction of flow. An overview of the chosen control volume is given in Figure 5.9.

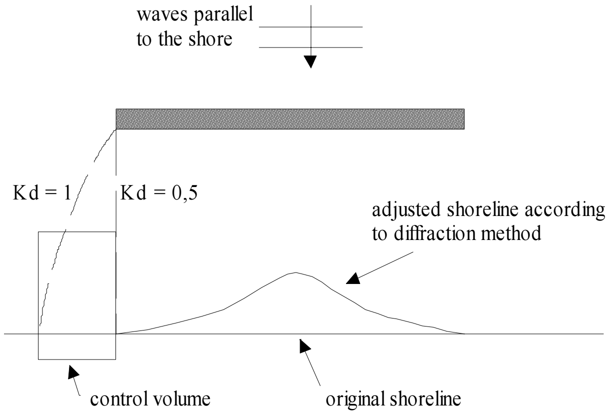


Figure 5.9: Overview of the control volume (Gourlay, 1978)

This control volume is assumed to be bounded in upstream direction at the point where the alongshore current velocity (V) is assumed to be zero (because at this point no gradient in wave setup occurs) and the diffraction coefficient is 1. This upstream boundary is called section 1. The downstream end of the control volume is at the geometric shadow of the breakwater, where the alongshore current velocity reaches its maximum value. This downstream boundary is called section 2. The offshore limit of the control volume is at the breaker line of the upstream boundary; this applies for both sections. The limits of the control volume are shown in Figure 5.10. In Figure 5.11 the form of the water surface at section 1 and 2 is shown.

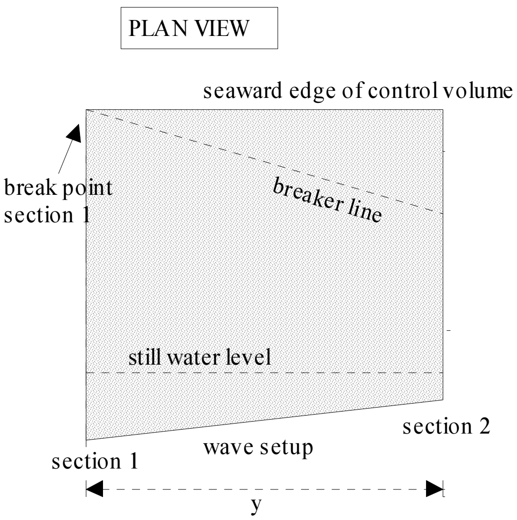


Figure 5.10: Plan view control volume (Gourlay, 1978)

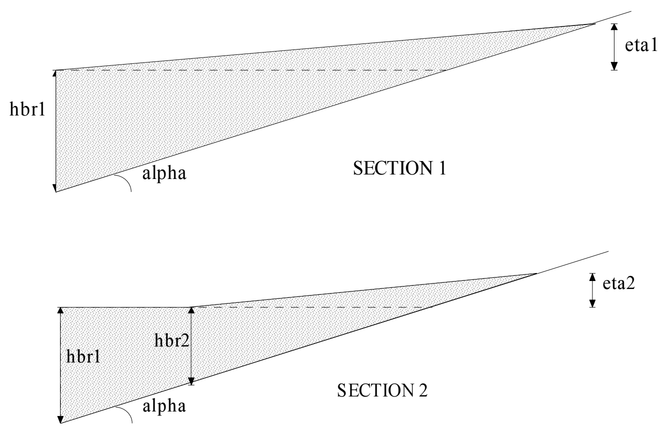


Figure 5.11: Cross-sectional areas (Gourlay, 1978)

5.3.2 Momentum balance

Application of the momentum principle (the integral form of equation 5.9, integrated over y) at this control volume, and neglecting the terms for bottom friction and lateral mixing gives the following expression:

$$\rho A_2 V_2^2 = P_1 - P_2 + S_1 - S_2 \tag{5.10}$$

where:

ρ = density of the water (kg/m³)

A_2 = cross-sectional area of flow, at section 2 (m^2)

V_2 = mean velocity of the flow through section 2 (m/s)

$P_{1/2}$ = water pressure term at section 1/2 (N)

$S_{1/2}$ = term due to radiation stress S_{yy} at section 1/2 (N)

The momentum balance of equation 5.10 shows which forces cause and affect the velocity of the alongshore current at section 2. Due to a resultant of the forces on the right side of the momentum balance, water will be accelerated between section 1 and section 2 and the velocity at section 2 can be calculated.

This momentum balance can only be applied in the inflow region, because in the other region (behind the breakwater) momentum transfer occurs to a secondary induced eddy (as Gourlay observed in the laboratory). This means that the momentum balance used for the inflow region is not complete for the region behind the breakwater. The resultant forces do not convert totally into acceleration of the flow: part of the forces will be converted into a secondary induced eddy. Therefore, the momentum balance from equation 5.10 is not applicable for the region behind the breakwater.

The terms on the right side of the equation can be expressed in terms of the surf zone parameters at the appropriate section (1 and 2). These sections are also shown in Figure 5.11. A complete deduction can be found in Gourlay (1978); here the final equations are given:

$$P_1 - P_2 = \frac{\rho g \left(h_{br1}^2 \eta_1 - h_{br2}^2 \eta_2 \right)}{6 \tan \alpha} \tag{5.11}$$

$$S_1 - S_2 = \frac{\rho g \gamma^2 \left(h_{br1}^2 \eta_1 - h_{br2}^2 \eta_2 \right)}{48 \tan \alpha} \tag{5.12}$$

The order of magnitude of these terms is respectively 10^5 and 10^4 for a wave height of 1 metre. This means that the water pressure term due to the gradient in wave setup affects the alongshore current velocity most.

If $\gamma = 0.8$, these equations lead to:

$$P_1 - P_2 + S_1 - S_2 = \frac{1.08\rho g\left(h_{br1}^2\eta_1 - h_{br2}^2\eta_2\right)}{6\tan\alpha} \tag{5.13}$$

where:

g = acceleration due to gravity (m/s^2)
$h_{br1/2}$ = breaker depth at section 1/2 (m)
$\eta_{1/2}$ = wave setup at section 1/2 (m)
tan(α) = bottom slope (-)

Because $h_{br2} \approx \frac{1}{2} h_{br1}$ (according to Figure 5.3, where the diffraction coefficient at section 1 is 1 and at section 2 this diffraction coefficient is 0.55) and in this situation with parallel incoming waves $\eta = \frac{1}{5} h_{br}$ (see equation 5.4), combining equation 5.10 with 5.13 leads to an expression for the alongshore current velocity at section 2 (assumed that the velocity is equally divided over the section):

$$V = 0.3\sqrt{gH_{br1}} \tag{5.14}$$

where:

V = alongshore current velocity at section 2 (m/s)
H_{br1} = breaking wave height at section 1 (m)

This equation has been evaluated with experimental values of laboratory tests, by Gourlay himself (Gourlay, 1978), and the results were satisfying. This means that this simple analytical expression (with the assumptions made) gives a good impression of the alongshore current velocity at section 2.

The alongshore current velocity calculated above is the simplified case with bottom friction and lateral mixing neglected. Lateral mixing mainly redistributes the velocity in the direction normal to the shore. In practice this means that the velocity at section 2 will be larger in the one part of the section and smaller at another part of the section, but the average velocity will stay the same. And it is the average velocity which is calculated with the method of Gourlay. Bottom friction can contribute considerably in determining the magnitude of the current velocity. The bottom shear stress will cause an extra term in opposite direction and will reduce the alongshore current velocity. Bottom friction can only reduce the alongshore current velocity, but can never fade out the current, because the bottom friction term depends on the velocity itself.

5.3.3 Effect of wave height on alongshore current

To investigate the effect of wave height on this alongshore current, different wave heights are substituted in equation 5.14. Table 5.1 shows the current velocities at section 2 for these different wave heights. It can be observed that higher waves cause larger velocities, which is logical, because the gradients in breaking wave height and in wave setup are larger too (in a situation with given project layout, see Appendix C). The velocities however are very high compared to velocities usually observed in nature. This can be explained, because bottom friction is not accounted for in this formula. With a smooth bottom (concrete), bottom friction reduced the velocities with only 8% (in the study of Gourlay, 1978), but at a bottom with coarse sediment bottom friction will have a larger effect. The calculated velocities are therefore qualitatively right, but quantitatively too large.

Table 5.1: Effect of wave height on alongshore current velocities

H (m)	Hbr1 (m)	V (m/s)
0.5	0.54	0.69
1	1.09	0.98
1.5	1.63	1.20
2	2.17	1.38
2.5	2.72	1.55
3	3.26	1.70

The alongshore current velocities in Table 5.1 are the velocities at section 2 (see Figure 5.10). At this section the velocity reaches its maximum according to the method of Gourlay (and his observations in the laboratory). Like mentioned before, behind the breakwater, momentum transfer occurs from the primary alongshore current towards a secondary induced eddy inside the overall circulation pattern. Because of this momentum transfer, the momentum balance used for the control volume between section 1 and 2 is not applicable for the region behind the breakwater. However, comparing the velocities at section 2 for different wave heights can give a good insight in the effect of wave height on the alongshore current velocity. Higher waves cause larger velocities (at section 2).

5.3.4 Consequences of the alongshore current

The consequence of this alongshore current is that, compared to the theory of the diffraction method, an extra transport of sand occurs towards the sheltered area behind the breakwater. This transport of sand will lead to a rotation of the shoreline, which means that the shoreline in the exposed area regresses and the salient behind the breakwater grows. See Figure 5.12.

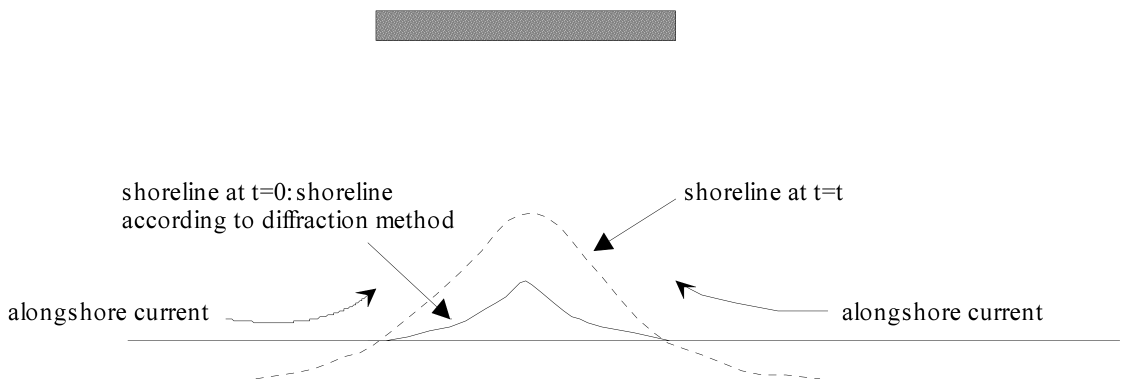


Figure 5.12: Consequence of alongshore current on shoreline

Due to transport of sand into the sheltered area behind the breakwater, the shoreline will rotate and the diffracted waves will not arrive parallel anymore, but will approach the new shoreline under an angle. This leads to an opposing current from the sheltered area towards the exposed area next to the breakwater. Another effect of this rotation of the shoreline is that the terms in the momentum balance from paragraph 5.3.2 will change. These effects will be discussed in Chapter 6 where, with the help of an extended momentum balance, equilibrium is described with the help of the terms due to the gradient in wave setup (P and S) and a term due to oblique arriving waves.

6 Obtaining equilibrium shoreline

As seen in Chapter 5, the gradient in wave setup (and breaker height) causes a current towards the sheltered area. Due to this current, sediment will be transported in the direction of the current and the shoreline will rotate: waves will approach the new shoreline under an opposing angle. In this chapter equilibrium of the shoreline is tried to find. This will be done by setting up an extended momentum balance for the rotated shoreline. Working out this momentum balance will give a relation between the gradient in breaking wave height and the equilibrium angle of the shoreline. At last, this equation will be validated and conclusions about the final plan form and the effect of wave height on this plan form will be drawn.

6.1 Waves approaching shoreline under an angle

The momentum balance deducted by Gourlay (1978) was for a situation with parallel incoming waves. The gradients in S_{yy} and in wave setup caused an alongshore current velocity transporting sand from the exposed area next to the breakwater towards the sheltered area behind the breakwater. This transport of sand leads to a rotation of the shoreline as shown in Figure 5.12.

Due to this rotation of the shoreline, the diffracted waves do not arrive parallel to the shoreline anymore, but approach the new shoreline under an angle. This leads to an opposing current and to changes in the terms of the momentum balance of paragraph 5.3.2. The basis for the momentum balance in this paragraph is this momentum balance of Gourlay, but in this paragraph the changes due to the rotated shoreline will first be explained and after that implemented in this basic momentum balance.

6.1.1 Effect rotated shoreline on radiation stress components

Figure 6.1 shows how the radiation stress components change when waves arrive under an angle instead of parallel.

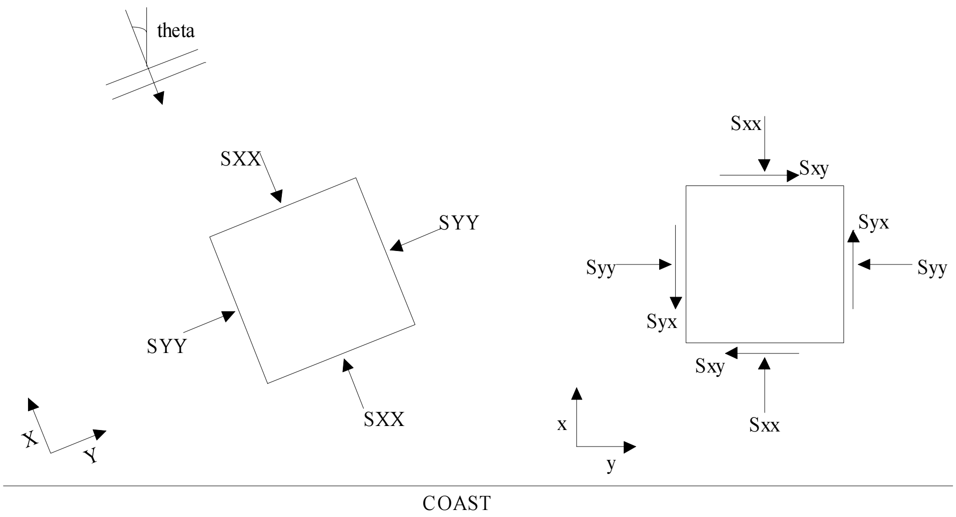


Figure 6.1: Radiation stress components in case of oblique arriving waves

The radiation stress components in the xy-system in case of oblique arriving waves in shallow water (n =1) are (Battjes, 2000):

$$S_{xx} = \left(n - \frac{1}{2} + n\cos^2\theta \right) nE = \left(\frac{1}{2} + \cos^2\theta \right) E \tag{6.1}$$

$$S_{yy} = \left(n - \frac{1}{2} + n\sin^2\theta \right) nE = \left(\frac{1}{2} + \sin^2\theta \right) E \tag{6.2}$$

$$S_{xy} = S_{yx} = nE\cos\theta\sin\theta = E\cos\theta\sin\theta \tag{6.3}$$

where:

θ = angle of approach (°)

E = wave energy = $\frac{1}{8}\rho g H^2$ (J/m²)

Due to the radiation stress component S_{xx}, which determines the wave setup it can be concluded that the water pressure term (P) in the momentum balance decreases by an increasing angle. S_{xx} depends on a factor $\cos^2\theta$ (see equation 6.1), which decreases by an increasing angle. This leads to a reduction of the wave setup and this reduction causes a reduction of the water pressure term in the momentum balance. The radiation stress term (S) in the momentum balance is affected from two sides. First, there is the radiation stress component S_{yy} which increases by an increasing angle, because of the factor $\sin^2\theta$ (see equation 6.2). But on the other side is wave setup also implemented in this term S, and that value decreases.

As one can observe from Figure 6.1, an extra radiation stress component develops in case of oblique arriving waves. The component S_{xy} is zero for parallel incoming waves and increases until an angle of

45 degrees is reached, due to the factor sinθcosθ. By larger angles, this component will decrease and become zero by an angle of 90 degrees.

In paragraph 6.1.2 the consequences of these changes in radiation stress components for the terms of the momentum balance are described more detailed and will be implemented in the basic momentum balance from Gourlay (Paragraph 5.3.2).

6.1.2 Effect rotated shoreline on momentum balance

Like mentioned before, the value of S_{xx} decreases when waves approach the shoreline under an angle and thus the wave setup decreases too. This means that the wave setup is not $0{,}2h_{br}$ anymore, but is reduced by a factor depending on θ:

$$\eta = \frac{\left(\frac{1}{2}+\cos^2\theta\right)}{\frac{3}{2}}\frac{1}{5}h_{br} = \left(\frac{1}{2}+\cos^2\theta\right)\frac{2}{15}h_{br} \tag{6.4}$$

where:

η = wave setup (m)

θ = angle of approach (°)

h_{br} = breaker depth (m)

This has an effect on the water pressure term (P). Not in terms of surf zone parameters, but by the changed value of η. The resultant water pressure term will be:

$$P_1 - P_2 = \frac{\rho g\left(h_{br1}^2\eta_1 - h_{br2}^2\eta_2\right)}{6\tan\alpha} \tag{6.5}$$

The term S_{yy} from the momentum balance will be affected by two factors: the increased value of S_{yy}, but at the same time a decreased value of η. The resultant term due to the radiation stress component S_{yy} will change to:

$$S_1 - S_2 = \frac{\left(\frac{1}{2}+\sin^2\theta\right)}{\frac{1}{2}}\frac{\rho g\gamma^2\left(h_{br1}^2\eta_1 - h_{br2}^2\eta_2\right)}{48\tan\alpha} = \left(1+2\sin^2\theta\right)\frac{\rho g\gamma^2\left(h_{br1}^2\eta_1 - h_{br2}^2\eta_2\right)}{48\tan\alpha} \tag{6.6}$$

The most important effect of oblique arriving waves is that a radiation stress component S_{xy} develops (the force due to this radiation stress is called F_{xy}). This radiation stress component is the driving force for an opposing current from the sheltered area towards the exposed area. This radiation stress component does not act on a section (1 or 2), but acts along the breaker line, from the break point in section 2 to the break point in section 1 (in Figure 5.10 this breaker line is shown). The alongshore component of this breaker line is the offshore limit of the control volume. And to obtain the force due to this radiation stress term in alongshore direction, this alongshore component of the breaker line should be taken as the distance where S_{xy} acts on.

Because this radiation stress component does not act on a section, but on the seaward boundary of the control volume, the alongshore distance between section 1 and section 2 (y_i) should be implemented in this term of the momentum balance. Then the term due to this radiation stress is:

$$F_{xy} = y_i \frac{1}{8}\rho g H_{average}{}^2 \cos\theta \sin\theta = y_i \frac{1}{8}\rho g \gamma^2 h^2_{br,average} \cos\theta \sin\theta = \\ y \ \frac{1}{16}\rho g \gamma^2 \left(\frac{h_{br1}+h_{br2}}{2}\right)^2 \sin(2\theta) \tag{6.7}$$

Now, the right side of the momentum balance in alongshore direction (y) will be:

$$P_1 - P_2 + S_1 - S_2 - F_{xy} = \left(1+0.08\left(1+2\sin^2\theta\right)\right)\left(h^2_{br1}\eta_1 - h^2_{br2}\eta_2\right)\frac{1}{6\tan\alpha} \\ - y_i \frac{1}{25}\left(\frac{h_{br1}+h_{br2}}{2}\right)^2 \sin(2\theta) \tag{6.8}$$

where:

$$h_{br2} = h_{br1} - \frac{\partial h}{\partial y} y_i , \tag{6.9}$$

$$\eta = \left(\frac{1}{2} + \cos^2\theta\right)\frac{2}{15} h_{br} \tag{6.10}$$

with:

$\frac{\partial h}{\partial y}$ = gradient in breaker depth = gradient in breaking wave height / 0.8 (-)

Observing equation 6.8 and 6.9 it will be clear that an extra parameter shows up in the extended momentum balance: the distance (y_i) between section 1 and section 2. This is because the radiation stress component S_{xy} acts along this distance. However, this extra parameter makes it more difficult to deduct a simple relation between the gradient in breaker depth and the angle of the shoreline, defining equilibrium. Therefore, the control volume for the new momentum balance will be changed. This change and the effects of it will be described in the next paragraph.

6.2 Equilibrium

The previous paragraph described how the terms of momentum balance change due to the rotated shoreline. In this paragraph equilibrium of the shoreline is described with the help of this new momentum balance. To obtain a general solution, independent of the alongshore distance y_i, a new control volume is adopted. The alongshore length of this new control volume is chosen to be Δy, to make it possible to obtain a general solution.

A visual explanation of the forces on the new control volume is given in Figure 6.2.

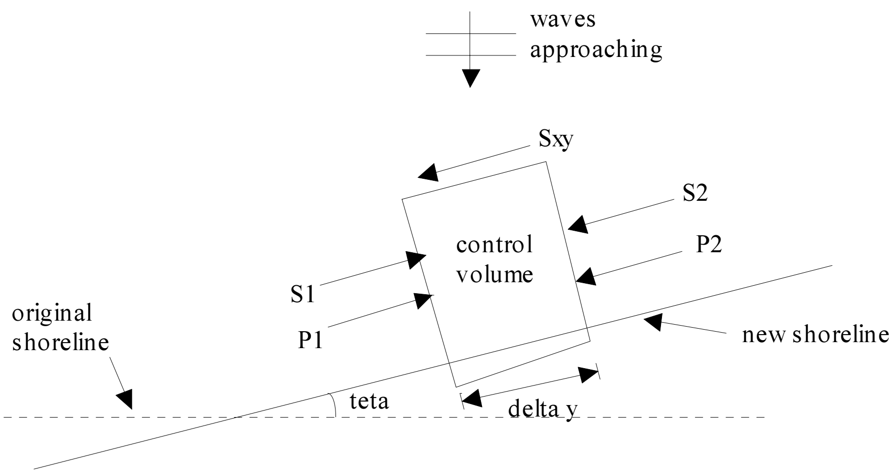


Figure 6.2: New control volume with rotated shoreline

Equilibrium can be expected if the forces acting on this new control volume balance each other. In that situation no alongshore current velocity can develop according to equation 6.11 (the new momentum balance):

$$\rho A_2 V_2^2 = P_1 - P_2 + S_1 - S_2 - F_{xy} \tag{6.11}$$

If the resultant of the terms on the right side of this momentum balance is zero, no velocity will develop. This means that the alongshore current velocity is zero and no further transport of sand will take place. When this occurs, equilibrium can be expected.

From the previous paragraph it became clear that the terms in the momentum balance depend on the angle θ. It is assumed that for a certain value of this angle θ, the resultant of these terms will become zero. This so-called equilibrium angle will depend on the parameters used in the momentum balance terms. This paragraph studies in what way this angle will be affected by these parameters.

Using equation 6.8, the equation for equilibrium of the new control volume is:

$$\left(1+0.08\left(1+2\sin^2\theta\right)\right)\left(h_{br1}^2\eta_1 - h_{br2}^2\eta_2\right)\frac{1}{6\tan\alpha} - \Delta y\frac{1}{25}\left(\frac{h_{br1}+h_{br2}}{2}\right)^2\sin(2\theta)=0 \qquad (6.12)$$

Substituting $\eta = \left(\frac{1}{2}+\cos^2\theta\right)\frac{2}{15}h_{br}$ and $h_{br2} = h_{br1} - \frac{\partial h_{br}}{\partial y}\Delta y$ in equation 6.12 leads to equation 6.13. In this equation $\partial h_{br}/\partial y$ is replaced by h': the equation will change into:

$$\left(1+0.08\left(1+2\sin^2\theta\right)\right)\left(\begin{array}{l}\frac{2}{15}h_{br1}^3\left(\frac{1}{2}+\cos^2\theta\right)- \\ \frac{2}{15}\left(h_{br1}^3 - 3h_{br1}^2 h'\Delta y + 3h_{br1}\left(h'\Delta y\right)^2 - \left(h'\Delta y\right)^3\right)\left(\frac{1}{2}+\cos^2\theta\right)\end{array}\right)\frac{1}{6\tan\alpha}$$
$$-\Delta y\frac{1}{25}\left(\frac{h_{br1}+\left(h_{br1}-h'\Delta y\right)}{2}\right)^2\sin(2\theta)=0 \qquad (6.13)$$

For small values of Δy ($\Delta y \to 0$), h'Δy is small in comparison with h_{br1}, equation 6.13 can be rewritten by neglecting the terms with $(h'\Delta y)^2$ and $(h'\Delta y)^3$ (these terms are much smaller than the other terms).

$$\left(1+0.08\left(1+2\sin^2\theta\right)\right)\left(-\frac{2}{15}\left(\frac{1}{2}+\cos^2\theta\right)\left(-3h_{br1}^2 h'\Delta y\right)\right)\frac{1}{6\tan\alpha} - \Delta y\frac{1}{25}\left(\frac{2h_{br1}}{2}\right)^2\sin(2\theta)=0 \qquad (6.14)$$

The solution of this equation can be written as follows:

$$h' = \frac{\frac{1}{25}\sin(2\theta)}{3\left(1+0.08\left(1+2\sin^2\theta\right)\right)\left(\frac{1}{6\tan\alpha}\right)\left(\frac{2}{15}\left(\frac{1}{2}+\cos^2\theta\right)\right)} \qquad (6.15)$$

This equation gives a relation between the gradient in breaker depth (h'), the bottom slope (tan(α)) and the so-called equilibrium angle (θ). For different bottom slopes (tan(α)), the relation between this equilibrium angle (θ) and the gradient in breaker depth (h') can be plotted in a figure; see Figure 6.3 for a bottom slope of 0.05. Appendix D shows the plots for bottom slopes of 0.01; 0.02; 0.1. The x-axes gives the angle in radians, the y-axes the gradient in breaker depth (h') belonging to this angle.

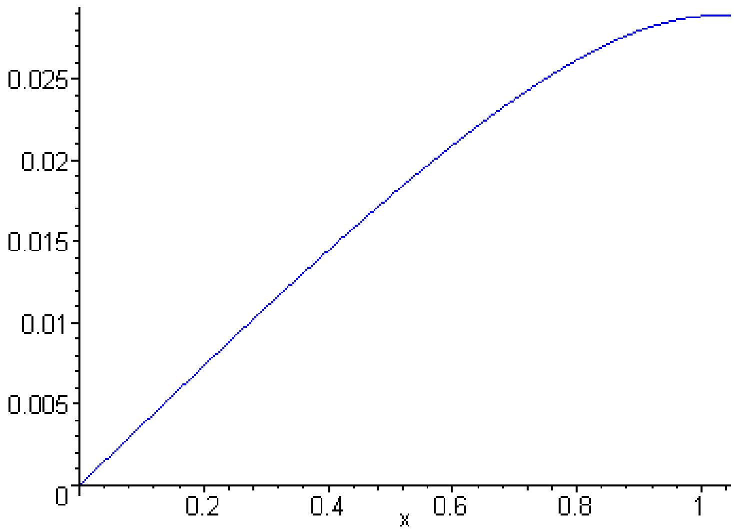


Figure 6.3: Plot of equation 6.15 for bottom slope 0,05

Equation 6.15 describes that for a certain value of the gradient in breaker depth (h'), there is an angle of the shoreline (θ) which leads to no resultant force on the control volume. This means that for this angle the alongshore current velocity is zero. In a situation where this alongshore current velocity is zero, the shoreline will not rotate any further and the final plan form is expected to be reached. The angle between this final plan form and the original shoreline is called the equilibrium angle.

From Figure 6.3 it can be concluded that for each gradient in breaker depth there is one related equilibrium angle. When the gradient in breaker depth changes, the equilibrium angle of the shoreline changes too. On the other hand it can be concluded that each equilibrium angle is the result of one certain value of the gradient in breaker depth. A change in the equilibrium angle suggests a change in the gradient in breaker depth. Observing from Figure 6.3 it can be said that if the gradient in breaker depth is larger, the equilibrium angle of the shoreline will be larger too (and reverse). This result can also be deducted from the terms in the momentum balance. If the gradient in breaker depth increases, the resultant terms due to water pressure ($P_1 - P_2$) and the radiation stress component S_{yy} ($S_1 - S_2$) increase too. To make the resultant of the forces on the right side of the momentum balance zero, the term due to S_{xy} should increase too. This can be obtained by a larger angle of the shoreline. By this larger angle the force due to the radiation stress component S_{xy} will increase and the total resultant force of the momentum balance will be zero.

It will be clear that the wave height or breaker depth is not in the equation anymore. This does not mean that the equilibrium angle (θ) does not depend of them. The effect of wave height or breaker depth will be treated in Paragraph 6.4

6.3 Validation of relation between h' and θ

To see if equation 6.15 can be used in practice, the results of that formula will be subjected to two validation criteria: qualitatively with a former research project on equilibrium bays and quantitatively also with this research project, and with Lake View Park (a project in Lake Erie (USA) with good monitoring).

6.3.1 Qualitative validation

In 1999 research is done for the SASME project. Part of this research focussed on equilibrium bays (Sweers, 1999). For gaining results Sweers used the computer program Delft3D-MOR. The study was split in two main parts. The objective and method of approach of both parts will be treated in this paragraph as well as the conclusions from both parts of the study. These conclusions will be evaluated with equation 6.15.

Because Sweers uses the terms positive and negative circulation, Figure 6.4 shows the definitions of these terms and the parameters used by Sweers. Positive circulation is defined as a current from the sheltered area behind the breakwaters towards the middle of the exposed area, while negative circulation is defined as a current from the middle of the exposed area towards the sheltered parts behind the breakwaters.

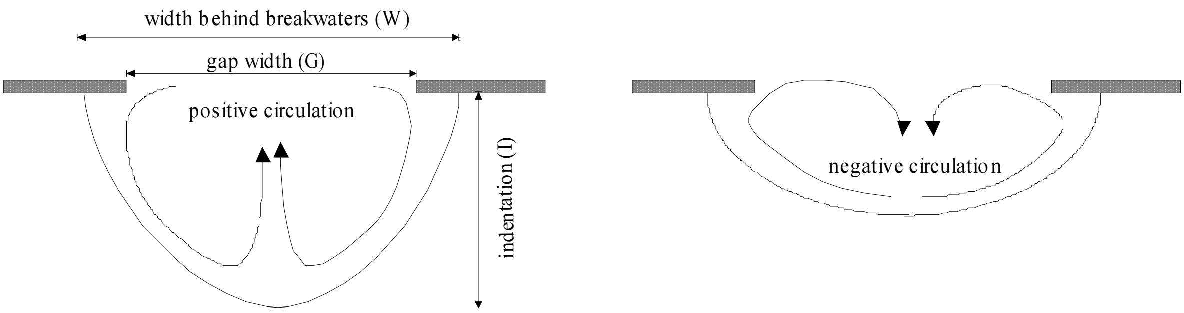


Figure 6.4: Negative and positive circulation pattern

First part: equilibrium bay and effect changing dimensions

First, Sweers did several simulations with one wave height (H = 2 m.). The dimensions of the bay were changed in every simulation and the alongshore current velocities were computed for all these bays. Finally, one bay, with a certain combination of dimensions turned out to have alongshore current velocities of almost zero: this bay was called the equilibrium bay. Bays with other dimensions (different gap width or distance offshore) turned out to have a negative or a positive circulation in the bay. Sweers used the ratios G/W and I/W to define the different dimensions. This means that for a given value of W there is a gap width (G) and an indentation (I) which belong to the so-called equilibrium bay. Changing these dimensions gives the following results (for a given width behind the breakwaters):

1. If the indentation is larger than the equilibrium indentation, a positive current develops
2. If the indentation is larger than the equilibrium indentation, a negative current develops
3. If the gap width is larger than the equilibrium gap width, a positive current develops
4. If the gap width is larger than the equilibrium gap width, a negative current develops

All these four conclusions can be evaluated with equation 6.15. If the indentation or the gap width is larger than the equilibrium indentation and gap width (conclusions 1 and 3), the effect of diffraction (differences in breaker depth along the shoreline) is spread over a larger distance. This means that the gradient in breaker depth decreases (see Paragraph 6.4 for a more detailed description of this effect). A smaller value of this gradient causes smaller terms P and S_{yy} in the momentum balance This means that the momentum balance is not in equilibrium anymore. A net momentum exists, directed from the sheltered area behind the breakwaters towards the exposed area behind the gap. This leads to a positive current.

dy ↑ ⟶ dh/dy ↓ ⟶ Syy and P ↓ ⟶ positive current

If the indentation or the gap width is smaller than the equilibrium indentation and gap width (conclusions 2 and 4), the opposite counts. This means that the effect of diffraction at the shoreline is spread over a smaller distance. This leads to a larger gradient in breaker depth and the net momentum is directed in the opposite direction: from the exposed area behind the gap towards the sheltered area behind the breakwaters. The consequence is a negative current.

dy ↓ ⟶ dh/dy ↑ ⟶ Syy and P ↑ ⟶ negative current

Second part: effect wave height on equilibrium and current velocities
Simulations were done for three different wave climates for five bays with different dimensions: the equilibrium bay, two "negative circulation" bays and two "positive circulation" bays. Changes in the alongshore velocities were measured for three sections at different locations in the bay. However, only the general conclusions are used in this validation. The general conclusions were:

5. Bays subjected to higher waves need a larger indentation and broader gaps to obtain equilibrium
6. The effect of increasing the wave height has most influence on the negative velocity, but
7. The wave height does not have affect the circulation pattern: positive circulations remain positive and negative circulations remains negative

Conclusion 5 can be explained in the same way as the conclusions of part 1. If the so-called equilibrium bay is subjected to higher waves this will cause a larger gradient in breaker depth (see Chapter 5). This leads to a negative current and equilibrium is disturbed. To obtain equilibrium again, the effect of diffraction (alongshore difference in breaker depth) at the shoreline should be spread over a larger distance. This will lead to a reduction of the gradient and the alongshore current velocity will reduce too. If the reduction of the gradient in breaker depth is large enough, the alongshore current velocity will be zero and equilibrium is expected to be reached.

Also conclusion 6 confirms equation 6.15. No matter which bay, a rise of wave height will cause a larger gradient in breaker depth (see Chapter 5), which leads to larger terms in the momentum balance due to P and S_{yy}. Then the net force directed from the exposed area behind the gap towards the sheltered area behind the breakwaters gets larger and the negative velocity increases.

$H\uparrow \longrightarrow dh\uparrow \longrightarrow dh/dy\uparrow \longrightarrow S_{yy}$ and $P\uparrow \longrightarrow$ negative current increases

According to this theory it should be possible to change a positive circulation pattern into a negative, if the increased terms due to S_{yy} and P exceed the term in the momentum balance causing the positive circulation. However, this was not observed by Sweers. This is the only aspect where equation 6.15 can not confirm the results of this project about equilibrium bays. The reason of this mismatch should be studied to know whether this mismatch is important, but for now it can be said that equation 6.15 is qualitatively proved (except this one conclusion).

6.3.2 Quantitative validation

Quantitative validation will be done with two projects: the project on equilibrium bays (Sweers, 1999), and the project Lake View Park (Pope & Rowen, 1983). The gradient in breaker depth obtained by using the diffraction diagram will be compared with the gradient in breaker depth according to equation 6.15. This paragraph will describe this quantitative validation.

Equilibrium bays project
Quantitatively the project on equilibrium bays can also help. The equilibrium bay obtained with a wave height of 2 meters (test case 4c in Sweers (1999)) has an equilibrium angle of the shoreline at the location where equation 6.15 applies: the area just next to the breakwater. The "equilibrium bay" of Sweers is shown in Figure 6.5.

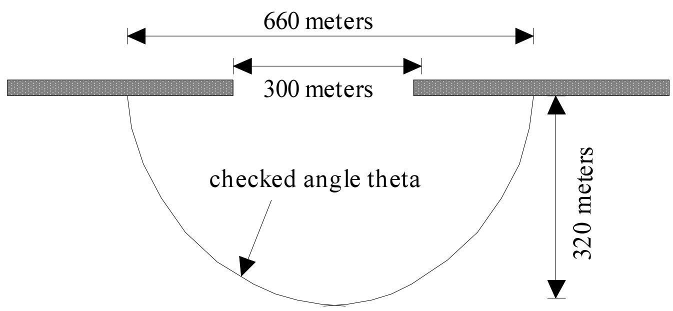


Figure 6.5: Equilibrium bay of Sweers (1999)

The checked angle corresponds with a gradient in breaker depth substituting the angle in equation 6.15. The gradient in breaker depth calculated in this way will be compared with the gradient in breaker depth for this part of the shoreline found with the help of the diffraction diagram (Shore Protection Manual). This last method of calculating a gradient is explained in Paragraph 5.2.2 and in Appendix C.

The angle of the shoreline at the location where equation 6.15 applies is measured by hand from the output of Delft3D. This angle (being about 25 degrees) corresponds with a gradient in breaker depth being 0.008 according to equation 6.15. However, using the diffraction diagram and substituting the used wave climate and geometry in it, gives a gradient in breaker depth of 0.017. Figure 6.6 shows the diagram for the calculation of this gradient in breaker depth in two different ways for the project on equilibrium bays.

DIFFRACTION DIAGRAM

H = 2 m
L = 76.4 m
bottom slope = 0.025 → hbr1 = 2.72 m
hbr2 = 1.66 m → dh = 1.06 m
dy = 65 m → **dh/dy = 0.016**

dh/dy = 0.008 ← theta = 25 degrees

EQUATION 6.15

Figure 6.6: Comparing gradient in breaker depth for project on equilibrium bays

From Figure 6.6 it can be concluded that according to the method using the diffraction diagram, the measured equilibrium angle of the shoreline is reached for a larger gradient in breaker depth than according to equation 6.15 (a factor 2 larger). Apparently, the measured equilibrium angle of shoreline belongs to a larger gradient in breaker depth than calculated with equation 6.15. This means that equation 6.15 underestimates the gradient in breaker depth belonging to a certain equilibrium angle. This can be due to several reasons. Reasons having a physical background will be described at the end of this paragraph. Other reasons can be the inaccuracy of measuring the angle by hand from the output of Delft3D and the inaccuracy of measuring distances in the diffraction diagram.

Lake View Park project

In Lake View Park, located in Lake Erie, a new recreational beach was constructed in 1977. To prevent the new beach of erosion, they used offshore breakwaters. To learn from this experience they implemented an intensive 5-year monitoring program to document the effectiveness of the breakwaters. Many data are available about this project (Pope&Rowen, 1983), and therefore this project can be used for validation.

A sketch of the project is given in Figure 6.7. The length of the breakwaters is 76 meters, the gap width is 49 meters and the distance to the shoreline (after beach fill) is 76 meters. The overall dimensions of the project are 403 meters by 150 meters.

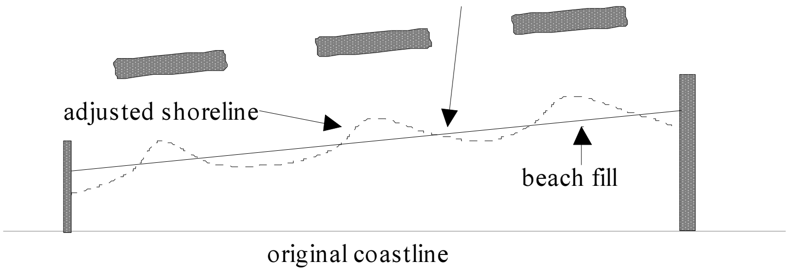


Figure 6.7: Lake View Park with adjusted shoreline

Also for this validation the equilibrium angle of the shoreline at the location where equation 6.15 applies can be measured: the area just next to the breakwater. This angle corresponds with a gradient in breaker depth substituting the angle in equation 6.15. The gradient in breaker depth calculated in this way will also in this validation be compared with the gradient in breaker depth for this part of the shoreline found with the help of the diffraction diagram (Shore Protection Manual). This last method of calculating a gradient is explained in Paragraph 5.2.2 and in Appendix C.

Monitoring of this project resulted in several aerial views. From the aerial view of May 1978 (scale 1:5000) the angle of the shoreline is measured. Substituting this angle (about 20 degrees) in equation 6.15 results in a gradient in breaker depth being 0.013. In this case the gradient calculated with the help of the diffraction diagram is 0.039 (for the average wave height). Figure 6.8 shows the diagram for the calculation of the gradient in breaker depth in two different ways for Lake View Park.

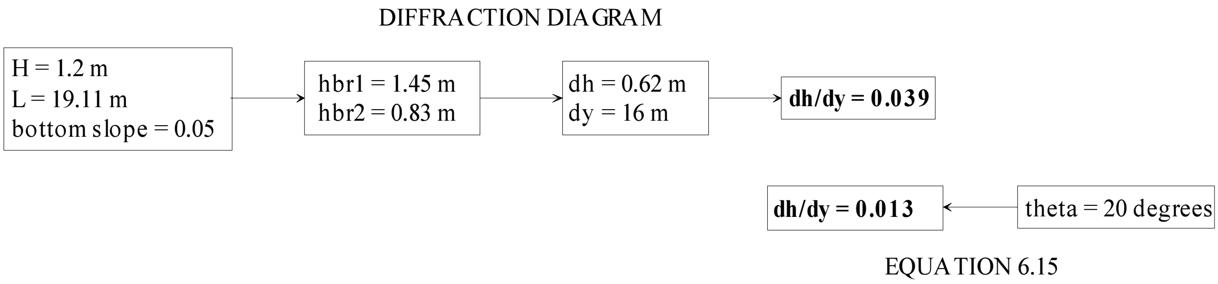


Figure 6.8: Comparing gradient in breaker depth for project Lake View Park

From Figure 6.8 the same conclusion can be drawn as from Figure 6.6. According to the method using the diffraction diagram, the measured equilibrium angle of the shoreline is reached for a larger gradient in breaker depth than according to equation 6.15 (in this case, a factor 3 larger). Apparently, the measured equilibrium angle of shoreline belongs to a larger gradient in breaker depth than calculated with equation 6.15. As already mentioned before, this means that equation 6.15 underestimates the gradient in breaker depth belonging to a certain equilibrium angle. Besides the reasons already given for the quantitative validation of the equilibrium bays project, there may be another reason for this particular project. The angle of the shoreline is measured from an aerial view from May 1978. In May the wave height is lower than the average wave height, because the average wave height is the mean value throughout the year (including storm season in winter). With a smaller wave height, the gradient in breaker depth calculated using the diffraction diagram would be smaller and the difference of the gradient in breaker depth between this method and equation 6.15 will reduce. So for Lake View Park it depends of the time of the year what the equilibrium angle of the shoreline is and what the difference is between the diffraction method value of this gradient in breaker depth and the value according to equation 6.15.

Validation diffraction diagram method

Above, equation 6.15 has been quantitatively validated. This has been done with the help of the diffraction diagram to obtain the gradient in breaker depth occurring in practice. This method (using the diffraction diagram to obtain the gradient in breaker depth) does not take into account the effect of bottom contours by calculating the gradient in breaker depth, although these bottom contours do affect the wave heights along the shore. Therefore, the method using the diffraction diagram will be validated with the help of the project on equilibrium bays to investigate the effect of bottom contours on the gradient in breaker depth. Taking the bottom contours into account gives a more realistic value of the gradient in breaker depth.

In the equilibrium bays project breaking wave heights along the shoreline are obtained. From these breaking wave heights, a gradient in breaker depth can be obtained by using equation 5.8b. The breaking wave heights of section 2 (hbr2) and 5 (hbr1) are taken and the distance between these two sections is measured by hand from the output of Delft3D.

Taking the same “equilibrium bay” as before (see Figure 6.5), the gradient in breaker depth at the location of interest, the boundary between exposed and sheltered area, is 0.007. See Figure 6.9 for the values in the calculation. The calculated gradient by using the diffraction diagram is also given to show the difference.

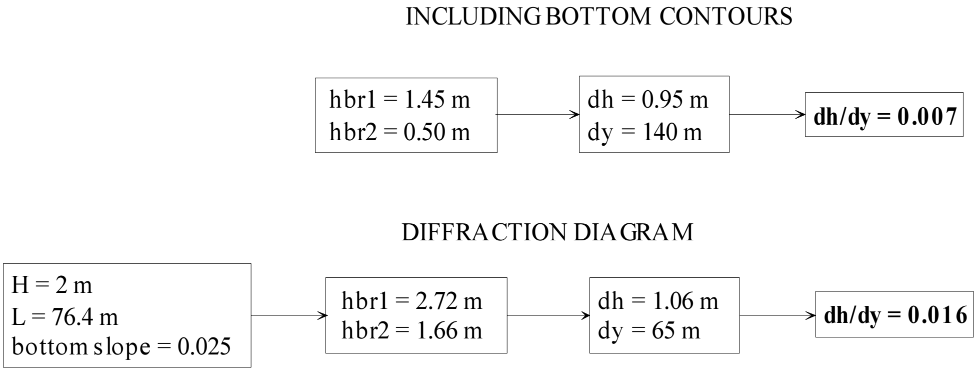


Figure 6.9: Gradient in breaker depth in Sweers (1999)

It can be concluded that when bottom contours are taken into account, the gradient in breaker depth is two times smaller than when bottom contours are not taken into account (the method using the diffraction diagram). This means that when the gradient obtained by using the diffraction diagram is corrected for bottom contours equation 6.15 agrees well with the gradient in breaker depth occurring in practice for the equilibrium bays project (see Figure 6.6). Not taking into account the bottom contours overestimates the gradient in breaker depth in practice by a factor 2.

Although Lake View Park does not have the same bottom contours as the equilibrium bays project, the conclusion about the diffraction method can be used in case Lake View Park too, because the principle is the same: taking bottom contours into account by calculating the gradient in breaker depth gives another, a two times smaller and more realistic, gradient in breaker depth than not taken these bottom contours into account. This means that if in case Lake View Park the gradient obtained by using the diffraction diagram is corrected for bottom contours the factor missing between equation 6.15 and practise would not be 3, but 1.5. The factor 1.5 missing for case Lake View Park can be seen as a margin, due to uncertainties in the validation.

It can be concluded that, although there are still uncertainties in the validation and Lake View Park still misses a factor 1.5 between practice and equation 6.15, equation 6.15 shows promising results in calculating the equilibrium angle of the shoreline.

Quantitative conclusions
It is clear that for both projects the gradient in breaker depth calculated with the diffraction diagram does not match with the values calculated with equation 6.15. After validation of the method using the diffraction diagram it has been concluded that the reason of these factors (2 – 3) can be the use of the diffraction diagram. Because this method does not take into account the bottom contours, while bottom contours are important in a coastal area. When the gradients in breaker depth resulting from the method using the diffraction diagram are corrected for bottom contours, equation 6.15 is satisfactory for the equilibrium bays project, and validated with the project Lake View Park, equation 6.15 only misses a factor 1.5 with the gradient occurring in practice. This factor 1.5 can be seen as a margin due to uncertainties in the validation. These uncertainties can be due to some physical processes which are not taken into account in equation 6.15, but do appear in practice. These processes can be:

- Due to the alongshore current, the mean water level will rise in the direction of the current. This leads to a reduction of the gradient in breaker depth (Gourlay, 1981).

- Underwater elevation of the beach within the surf zone, rising in the direction of the alongshore current. This leads to an adverse bed gradient inhibiting further sediment transport (Gourlay, 1981)

- In practice, although the mean wave direction is parallel to the coast, there will be some spreading through the year. This directional spreading will change the gradients in breaking wave height and in wave setup: these gradients get smaller.

Research should be done to investigate the consequences of these effects (see Chapter 8, recommendations). But the overall conclusion is that equation 6.15 gives promising results by calculating the equilibrium angle of the shoreline

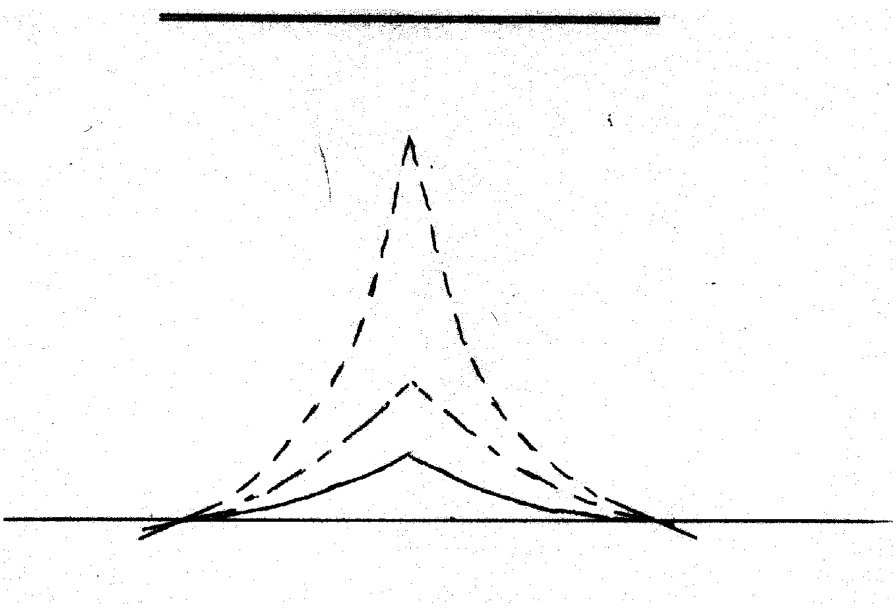
6.4 Final plan form and the effect of wave climate

This paragraph will first discuss conclusions about the final plan form and after that the effect of wave climate on this final plan form will be treated.

6.4.1 Final plan form

The main result of this chapter is equation 6.15. With the help of this equation the equilibrium angle of the new shoreline can be calculated by substituting the gradient in breaker depth at a certain location. Knowing this equilibrium angle, the final plan form can be deducted. In Figure 6.10 the final plan form is sketched for waves of 1 meter height. Besides the shoreline according to the diffraction method there are two final plan forms sketched in this figure. For the first shoreline the gradient in breaker depth resulting from the diffraction method (see Appendix C) is used to obtain the equilibrium angle. For the other shoreline, the gradient resulting from the diffraction method is corrected for bottom contours, because it turned out that this correction gives a more realistic value of the gradient in breaker depth occurring in practice.

To obtain the final plan form only the equilibrium angle is calculated at the boundary between sheltered and exposed area and from there the expected shoreline is sketched. This is because at other locations, the gradient in breaker depth changes due to the rotated shoreline: the distance between two lines of equal diffraction along the shoreline changes, which affects the gradient in breaker depth and this has consequences for the equilibrium angle and so on. This means that Figure 6.10 only gives an indication of the new shoreline: drawing the exact new shoreline needs more investigation. The equilibrium angle by using the diffraction diagram is 23 degrees, when the gradient in breaker depth is corrected for bottom contours this equilibrium angle is 12 degrees.



_ _ _ _ = calculated with gradient from diffraction diagram

__ _ __ = calculated with gradient including bottom contours

__________ = shoreline according to diffraction method

Figure 6.10: Final plan form of a shoreline behind an offshore breakwater

Figure 6.10 shows that due to the rotation of the shoreline, the shape of the salient in the sheltered area behind the breakwater will not be a circle anymore (according to the diffraction method). Sand transported from the exposed area towards the sheltered area will be deposited behind the breakwater and the salient grows. How much this salient grows depends on the gradient in breaker depth. If the diffraction diagram is used (without correction for bottom contours) to obtain the gradient in breaker depth the gradient is two times larger than when bottom contours are taken into account. From Figure 6.10 it can be observed that therefore the salient is most extended when the diffraction diagram is used and bottom contours are not taken into account.

Because in practice bottom contours are important, the middle shoreline will be the most realistic shoreline.

6.4.2 Effect wave height on final plan form

Like mentioned earlier in this chapter, the equilibrium angle of the beach and with that the final plan form do depend on the breaking wave height or breaker depth, although wave height is not in equation 6.15. The wave height (or breaker depth) is implemented in the gradient of the breaker depth (h'). As seen in Chapter 5 (Figures 5.6 and 5.7), in situations with a given layout, higher waves cause larger gradients in breaking wave height and in wave setup. This means two things for the final plan form:

1. Different wave heights in a settled breakwater area result in different equilibrium shorelines. A larger value of the gradient in breaker depth (h') will cause a larger equilibrium angle θ and a more extended salient. Figure 6.11 gives an indication of the effect of wave height.

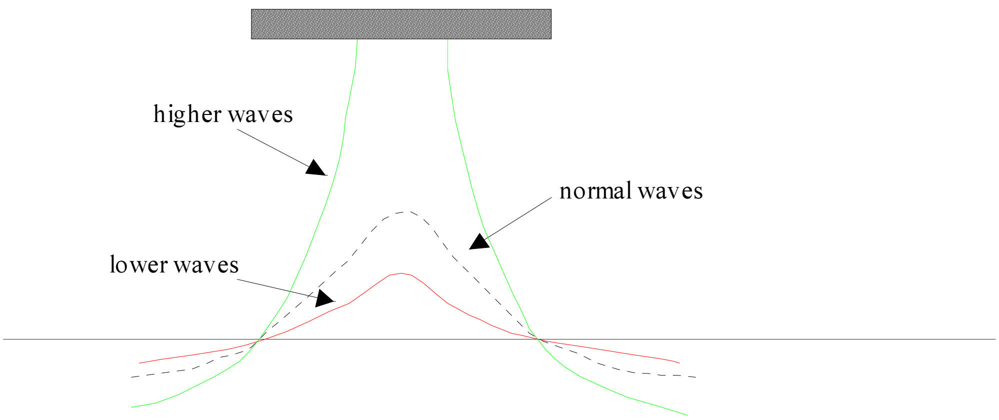


Figure 6.11: Consequence larger gradient in breaker depth (h') for final plan form

This figure shows the effect of wave height on the final plan form. It will be clear that in situations with higher waves the salient will be more extended towards the breakwater and the chance of tombolo formation increases. The sand accumulated behind the breakwater comes from the area next to the breakwater, thus this part of the shoreline regresses.

2. Obtaining the same equilibrium shoreline for different wave heights, requires a different layout of the breakwater area. It became clear in Chapter 5 (Figure 5.6 and 5.7) that in a situation with given layout higher waves cause a larger gradient in both breaking wave height and wave setup. This means that for higher waves also the gradient in breaker depth is larger (the breaker depth is the breaking wave height divided by 0.8). Due to this larger gradient in breaker depth, the equilibrium angle of the shoreline will be larger for higher waves. This effect is already shown in Figure 6.11.

   To obtain the same equilibrium angle, the gradient in breaker depth should be the same (according to equation 6.15). This can be obtained by constructing a breakwater further offshore when the shoreline is subjected to higher waves. Figure 6.12 shows the effect of constructing a breakwater further offshore in the diffraction diagram. Shoreline 1 can be seen as the shoreline in a basic situation, while shoreline 2 is a situation where the breakwaters are constructed further offshore.

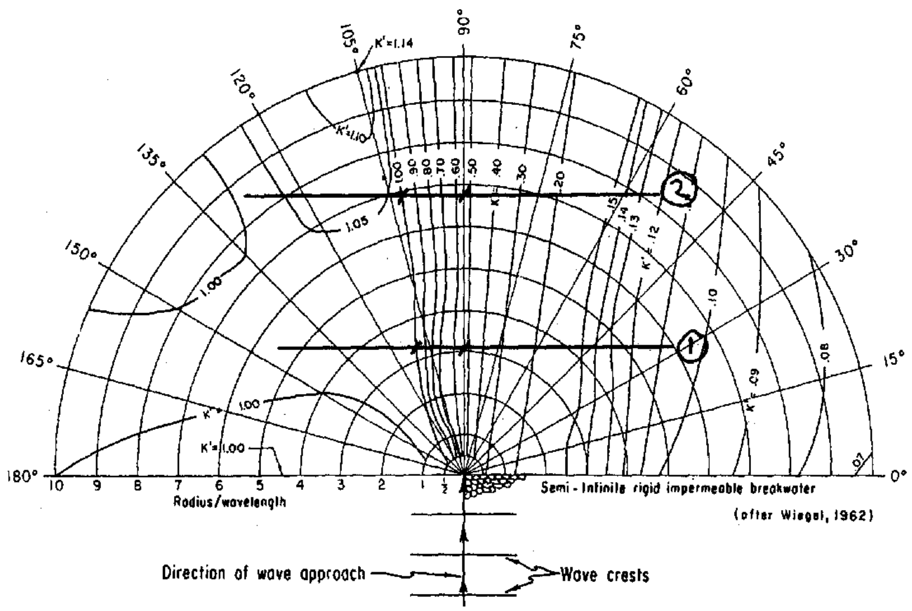


Figure 6.12: Effect diffraction pattern on gradient in breaker depth (U.S. Army Corps of Engineers, 1984)

The gradient in breaker depth is the difference in breaker depth between two locations (Δh_{br}) divided by the distance (Δy) between these locations. Taking the crossing of the lines with diffraction coefficients 1 and 0.55 with the shoreline as these two locations, the difference in breaker depth for both situations (subjected to the same wave height) will be the same. However, in situation 2, the distance between these two points is larger than in situation 1. This enlarged distance Δy leads to a smaller gradient in breaker depth comparing situation 2 with situation 1.

Concluding, it can be said that the gradient in breaker depth changes by changing the distance between the original shoreline and the breakwaters. This makes it possible to obtain the same gradient in breaker depth for different wave heights. The breakwaters should just be constructed at a different distance offshore. Breakwaters in front of a shoreline subjected to high waves should be constructed further offshore than breakwaters in front of a shoreline subjected to low waves. If the same gradient in breaker depth is obtained, the equilibrium angle of the shoreline will be the same too, according to equation 6.15.

7 Evaluating and improving design guidance

The conclusion in Chapter 6 about the final plan form and the effect of wave height on this plan form can help to evaluate the existing design guidance about shoreline response. First, empirical relations will be evaluated, including the diffraction method. After that also the project Muiderburght (the initiative for this study) will be evaluated and at last the design guidance will be tried to improve.

7.1 Evaluating empirical relationships

Before evaluating empirical relationships, the main conclusions from Chapter 6, about the final plan form will be repeated. Figure 7.1 shows a sketch of the final plan form with the effect of wave height.

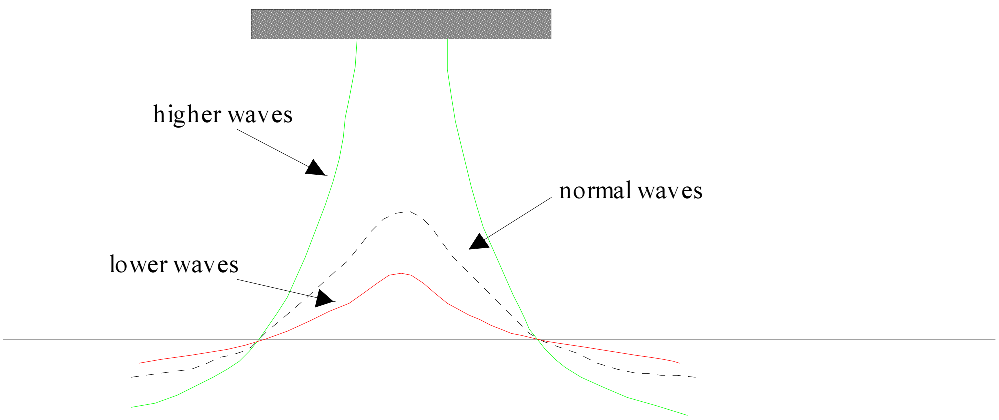


Figure 7.1: Sketch of final plan form and the effect of wave height on this plan form

Due to an alongshore current (caused by a gradient in wave setup), sand is transported from the exposed area next to the breakwater towards the sheltered area behind the breakwater. Higher waves cause a larger alongshore current velocity (due to a higher gradient in wave setup), transporting more sand towards the area behind the breakwater. This leads to a larger equilibrium angle, according to equation 6.15, and a more extended shoreline than low waves. This means that the chance of tombolo formation increases when a shoreline prevented by breakwaters is subjected to higher waves. This increased chance of tombolo formation can be avoided by using another ratio structure length – distance offshore. For situations subjected to higher waves, the ratio structure length – distance offshore should be smaller to avoid tombolo formation. Furthermore, it was concluded from equation 6.15 that each equilibrium angle of the shoreline belongs to one certain value of the gradient in breaker depth. When this gradient changes, the equilibrium angle of the shoreline changes too and this leads to a change in the final plan form.

7.1.1 Diffraction method

In Chapter 4 the diffraction method was described and some doubts about the correctness of this method came up. Due to a gradient in wave setup, the shoreline would extend more than the diffraction method predicted. Furthermore, there are more parameters affecting the shoreline response than just the ratio structure length – distance offshore. Both these doubts turned to be right. A gradient in wave setup height causes an alongshore current transporting sand towards the sheltered area. This leads to a more extended shoreline than the final plan form assumed by the diffraction method and this new shoreline is not parallel to the wave crests anymore. The effect of wave height on the shoreline response got clear too. Higher waves will cause a more extended shoreline than low waves (in a situation with given layout) and a tombolo will be formed for a ratio structure length –distance offshore smaller than 2. The limit for tombolo formation depends on the wave height and the layout of the project. Because of these two effects, the diffraction method is not a good method to predict shoreline response behind an offshore breakwater. One can use this method for a first guess of the shoreline response, but it is clear that the diffraction method underestimates the chance that a tombolo will develop. The sketched shoreline in Figure 6.8 looks more like the shoreline predicted by Silvester&Hsu (1997). This relationship will be evaluated now.

7.1.2 Silvester&Hsu (1997)

Silvester&Hsu developed a relationship determining the final plan form of the beach. This relation was based on an analysis of a large number of data obtained from prototype tests, numerical models and laboratorial studies. The main conclusion of this analysis was that the dimensionless ratio structure length – distance offshore is the most important parameter determining the final plan form. The shape of the new beach was defined to be parabolic, which was a new view, because since the introduction of offshore breakwaters, the logarithmic spiral was used most for determining the shape of the beach. The shape of the beach is determined by the value of R (metres) at any angle θ (degrees). This value can be read from Figure 7.2. The y-axes gives the ratio R_θ/R_0; the x-axes gives the angle θ in degrees.

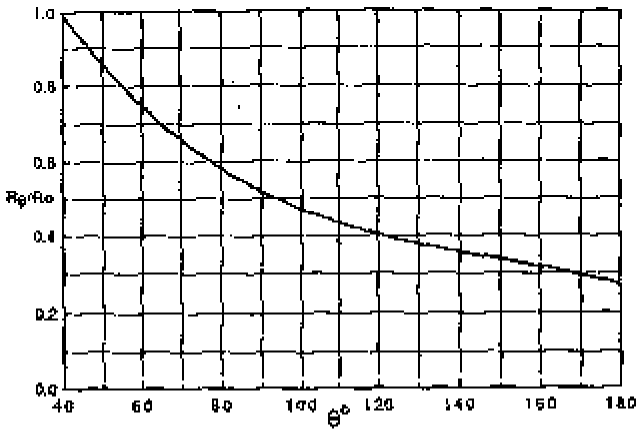


Figure 7.2: The dimensionless ratio R_θ/R_0 versus angle θ (Silvester&Hsu)

The results from the diagram in Figure 7.2 lead to a parabolic shape of the shoreline. See Figure 7.3.

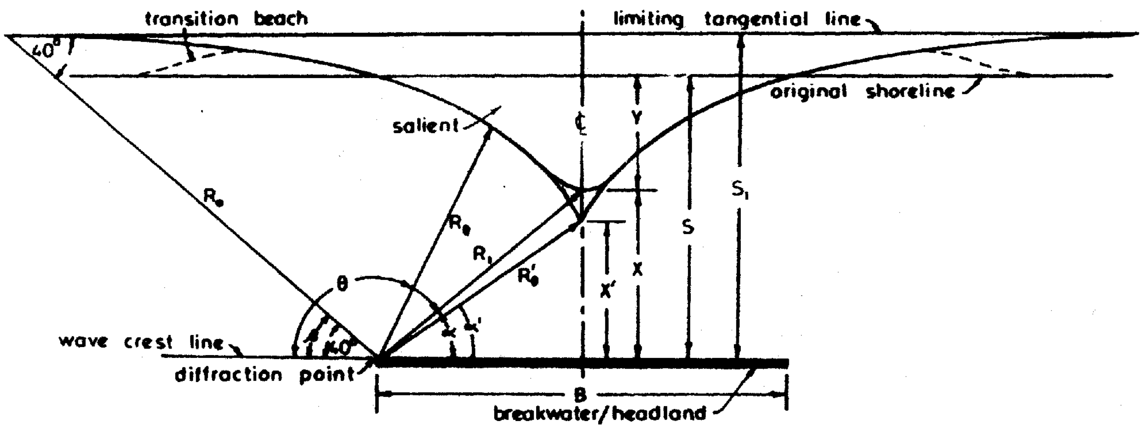


Figure 7.3: Parabolic beach shape (Silvester&Hsu)

This shape agrees well with the final plan form sketched in Figure 6.8. The shoreline next to the breakwater rotated with a certain angle (the equilibrium angle). And the transported sand from this region is deposited behind the breakwater, which leads to a salient which is larger (more extended towards the breakwater) than the salient according to the diffraction method.

However, also this relationship does not take into account the wave height, although this wave height does affect the final plan form (as seen in Paragraph 6.4.2) by a different gradient in breaker depth. According to equation 6.15 there is a relation between this gradient in breaker depth (h') and the equilibrium angle of the shoreline (θ) for a given bottom slope. This means that a certain angle of the shoreline next to the breakwater belongs to one value of the gradient in breaker depth. Another gradient in breaker depth will lead to another equilibrium angle and that leads to another shape of the final plan form. This means that the equilibrium angle of the shoreline in Figure 7.3 only applies for a certain value of the gradient in breaker depth, belonging to this equilibrium angle of the shoreline. For other gradients in breaker depth the equilibrium angle of the shoreline will change and the shape of the shoreline given in Figure 7.3 is not applicable. Thus the empirical relationship of Silvester&Hsu (1997) agrees better with the final plan form resulting from Chapter 6, but also this relationship does not take into account the wave height.

7.1.3 Other empirical relationships

The diffraction method is the only empirical relationship deducted from a theory; the other empirical relationships are all deducted from laboratory studies or constructed projects. The limits for tombolo or salient formation resulting from these studies can be very good, but they should be used with care. Wave climate and layout of a project are not the same in any place, so the limits found in these studies are only applicable in similarly projects with similar wave climate. According to equation 6.15 the equilibrium angle of the shoreline depends mainly on the gradient in breaker depth. This means that if this gradient in breaker depth is the same as in the study of the researcher, the shoreline can be predicted well.

These empirical relationships do not mention the shape of the final plan form. They only give limits for the extension of the shoreline (salient or tombolo). Therefore it is impossible to evaluate the relationships on this issue.

7.2 Evaluating Muiderburght

The project Muiderburght was the initiative for this study. This project consists of three breakwaters constructed in Lake Marken to prevent a new recreational beach to erode (Aveco de Bondt, raadgevend ingenieursbureau, 2000). The main purpose of these breakwaters was that maintenance (refilling the sand of the beach) would only be necessary once in five to ten years. Figure 7.4 shows the project without offshore breakwaters with its overall dimensions. Low waves arrive from the west; high waves arrive from north-west.

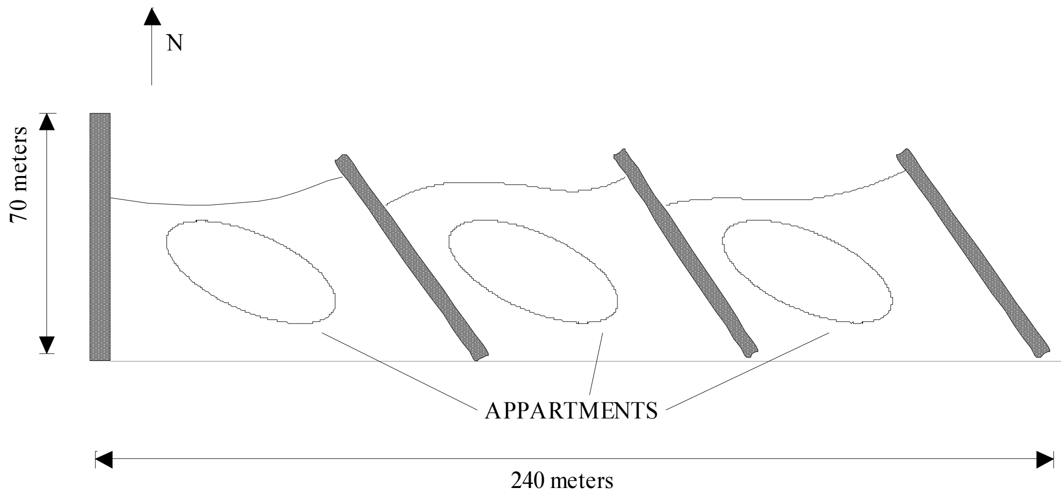


Figure 7.4: Muiderburght without offshore breakwaters

By designing these breakwaters the relation of Ahrens&Cox (1990) was used to determine the ratio structure length –gap width. And the relation of Suh&Dalrymple was used to determine the ratio structure length – distance offshore. This paragraph will evaluate the use of these relationships for the project Muiderburght.

Because the construction area available was only unto 30 meters offshore, the breakwaters had to be constructed that close to the shore. The minimum gap width was 15 m. at water level, so little boats can sail through the gaps to the beach (wish of the users of the beach). This minimum gap width is chosen as *the* gap width. This means that the dimensions of the distance offshore and the gap width were already fixed. Further on, from esthetical point of view, it had been decided that there should come three offshore breakwaters, with the middle breakwater longer than the other two breakwaters.

7.2.1 Ahrens &Cox (1990)

Ahrens & Cox used the beach response index classification scheme of Pope and Dean (1986) to develop a relationship for expected beach response:

$I_s = e^{(1,72 - 0,41L/G)}$

where:
L = Structure length
G = Gap width

I_s is the beach response index, coded with:
$I_s = 1$ Permanent tombolo formation
$I_s = 2$ Periodic tombolos ($L / G > 2.5$)
$I_s = 3$ Well developed salients ($L / G < 1.5$)
$I_s = 4$ Subdued salients ($L / G < 0.8$ a 1.5)
$I_s = 5$ No sinuosity ($L / G < 0.27$)

To be sure no maintenance would be necessary in the next five till ten years, the designers choose a tombolo as the preferred beach response. By iteration they found that the breakwater lengths should be 40 and 60 metres with a gap width of 15 metres. These dimensions lead to a beach response index (I_s) of respectively 1.9 for the offshore breakwaters on the west and east side and 1.1 for the middle breakwater. This means that behind the middle breakwater a permanent tombolo will develop and behind the west and east breakwater a periodic tombolo will develop (if enough sand is available).

7.2.2 Suh&Dalrymple (1987)

The results of their study were the following:

- For a single breakwater a tombolo will usually form when $L / X > 1$
- For a multiple breakwater this will happen for $L_g X/ L_s^2 < 0.5$

- When the ratio's are respectively smaller and larger, a salient will form

where:

$L_{(s)}$ = length of the breakwater (m)
X = distance from the breakwater to the original shoreline (m)
L_g = gap width (m)

Substituting the resulting dimensions of Muiderburght (length of the breakwaters being 40 and 60 meters, gap widths of 15 meters and a distance offshore of 30 meters) gives for the west and east breakwater a value of 0.28 and for the middle breakwater a value of 0.13. This means that behind all three breakwaters a tombolo will develop and because these values are much lower than 0.5, they are expected to be permanent, if enough sand is available.

The predicted shoreline is shown in Figure 7.5. The dimensions can be found in Appendix A, Table A.1).

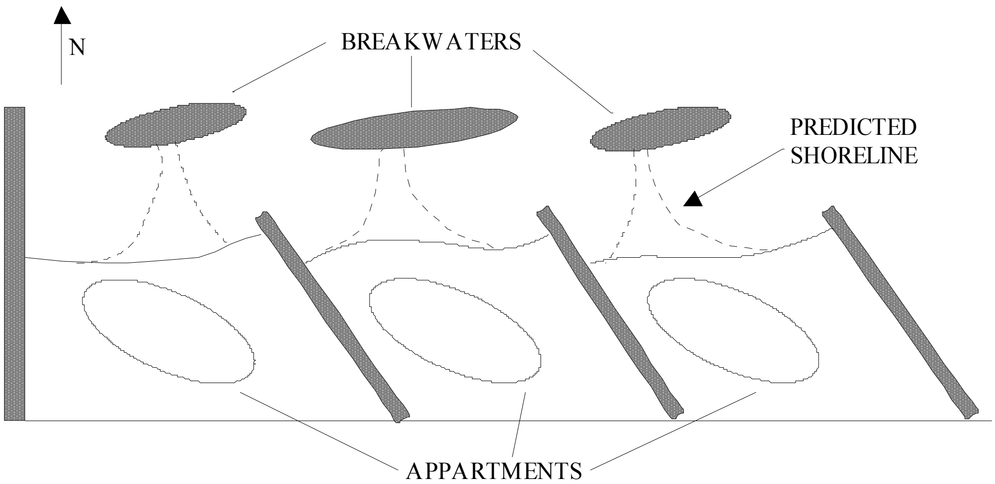


Figure 7.5: Predicted shoreline Muiderburght

7.2.3 Evaluation

After a year (December 2002) it turned out that only the beach behind the middle breakwater behaved as expected. Here a tombolo developed, but behind the east and west breakwater no major sand accumulation occurred. However, half a year later (in May 2003) also behind the east breakwater a tombolo developed. This means that the used empirical relationships predicted the shoreline response quite well. Only the beach behind the west breakwater does not behave as expected. This may be caused by the direction of the average waves (west) in combination with the lengthened groin at the west side. Due to these two facts, no/little sand will be transported to the area behind the west breakwater, while the middle and east breakwater get sand from the area behind the west breakwater, transported eastward.

However, the empirical relationships used for this project do not take into account the wave height, although it turned out (in Chapter 6) that this wave height does affect the final plan form. According to equation 6.15, the gradient in breaker depth determines the shoreline response. So, probably the gradient for the project Muiderburght was approximately the same as in the studies of the researchers. For other gradients in breaker depth the shoreline response would have been different, according to equation 6.15. This means that the well predicting of the shoreline was a coincidence and that these empirical relationships do not work well for sure in every other situation in lakes. As mentioned before, this depends on the gradient in breaker depth.

7.3 Improving design guidance

To improve the design guidance the evaluations in this chapter can help. The main conclusion is that the empirical relationships are indeed too simple, because they do not take into account the effect of wave height (and other shoreline affecting parameters). While, in Chapter 6 it turned out that wave height has a considerable effect on the shoreline response behind offshore breakwaters.

Due to the site specific character of the empirical relationships, these empirical relationships have different limits for the ratio structure length (L) – distance offshore (X). Therefore it is very difficult to improve all these different empirical relationships. But it can be concluded that the shoreline response depends most on the gradient in breaker depth, according to equation 6.15. This means that when the gradient in breaker depth is the same as in the study of the researcher, the shoreline response can be predicted well. But when this gradient is different, it will probably predict the shoreline response wrong, because a different gradient in breaker depth leads to a different equilibrium angle of the shoreline. This means that before using an empirical relationship, one should know in what conditions (read: which gradient of breaker depth) the study is executed.

In general it can be said that the ratio structure length – distance offshore should be smaller than 2 to prevent tombolo formation. This limit is the one from the diffraction method, and in Chapter 5 and 6 it turned out that this ratio is only right for waves of zero metres (which means no gradients in breaking wave height and in wave setup). If gradients in breaking wave height and in wave setup exist, the shoreline will rotate and the shoreline behind the breakwater will extend in the direction of the breakwater. This may lead to tombolo formation behind breakwaters with a ratio structure length – distance offshore smaller than 2. This means that the limit of this ratio should be reduced to be sure no tombolo will form. In Paragraph 6.4 it turned out that the limit for this ratio depends on the wave height: the higher the waves, the smaller the ratio.

Another effect of wave height on the shoreline response became clear too. To obtain the same shoreline response (same equilibrium angle) one needs the same gradient in breaker depth. This can be obtained by constructing breakwaters at a distance offshore depending on the wave height in that situation. Shorelines subjected to higher waves should have breakwaters constructed further offshore than shorelines subjected to low waves. In that way, the effect of diffraction (different breaker depths along the shoreline) will be spread over a larger distance and this reduces the gradient in breaker depth. When the gradient in breaker depth is the same for two situations (no matter what the wave height is), these situations will have the same equilibrium angle and thus the same shoreline response (although scaled). However, the value of this gradient in breaker depth depends on more parameters than just the wave height. Therefore, the required distance offshore, belonging to a certain wave height, can not be implemented unambiguous.

The conclusion from this evaluation is that empirical relationships can not be improved unambiguous, because these relationships are deducted under different conditions. Obtaining the same gradient in breaker depth as used in the study of the researcher can increase the chance that the shoreline response will be predicted well. This is because, according to equation 6.15, the equilibrium angle of the shoreline (and therefore the final plan form) depends on this gradient. The most important improvement is that the ratio structure length – distance offshore should be smaller for higher waves to avoid tombolo formation.

8 Conclusions and recommendations

The objective of this study is investigating the effects of wave climate on the shoreline response due to offshore breakwaters, and improving the existing design guidance on this area of interest. In this chapter, conclusions and recommendations regarding these objectives will be given..

8.1 Conclusions

The following conclusions can be drawn

- The theory that parallel incoming wave crests determine the equilibrium of a new shoreline (diffraction method) is wrong. Gradients in breaking wave height and in wave setup cause an alongshore current changing this "equilibrium" (Chapter 4).

- The equilibrium angle of the shoreline is determined by the gradient in breaker depth (equation 6.15). Each gradient in breaker depth is related to one equilibrium angle (and reverse). A change of the gradient in breaker depth leads to a change in the equilibrium angle of the shoreline (and reverse). A larger gradient in breaker depth causes a larger equilibrium angle; a smaller gradient a smaller angle (Chapter 6).

- The effect of a larger gradient in wave setup is the cause for differences in shoreline response due to wave height. Higher waves in a situation with given layout result in a more extended shoreline. This is because the gradient in breaker depth is larger and therefore the shoreline will have a larger equilibrium angle which causes the shoreline behind the breakwater to extend more towards the breakwater (Chapter 6).

- To obtain the same equilibrium angle for shorelines subjected to different wave heights, the gradient in breaker depth should be the same. It was investigated that in a situation with a given layout, higher waves cause a larger gradient in breaker depth. If the breakwaters are constructed further offshore this gradient can be reduced. This is because, the effect of diffraction (different breaker depths along the shoreline) will be spread over a larger distance and this reduces the gradient in breaker depth ($\Delta h_{br}/\Delta y$) (Chapter 6).

- Empirical relationships can not be improved unambiguously, because these relationships are deducted under different conditions. Obtaining the same gradient in breaker depth as used in the study of the researcher can increase the chance that the shoreline response will be predicted well. This is because, according to equation 6.15, the equilibrium angle of the shoreline (and therefore the final plan form) depends on this gradient (Chapter 7).

- Design guidance is simple (and should be simple), but practice is very complex: many parameters play a role in determining the shoreline response. This makes it very difficult to make implicit (and everywhere to use) design guidance (Chapter 7).

8.2 Recommendations

Research after the shoreline response due to offshore breakwaters is still young and further research is definitely necessary to improve the existing design guidance. Recommendations for further research are:

- Further research is needed to investigate the factor which is missing between practice and the equation resulting from the momentum balance. Some assumptions about the reason of this factor are already given, but these assumptions should be investigated. And equation 6.15 should be validated with more projects.

- Once the effect of wave height is found, it may be interesting to find out the effects of other parameters on the shoreline response too in order to improve the design guidance. A good way to study these effects is monitoring upcoming projects. The data from these projects can help to investigate the effect of some of these parameters.

- Use a computer model, like Delft3D, to examine if the theory used in this study agrees with computer model results. Check if the calculated water level variations and current velocities agree. Although refraction was not taking into account in the present study.

- It would be interesting to check the value of the gradient in breaker depth for the different empirical relationships. Knowing these values, empirical relationships can be used with more certainty of good prediction.

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Internet:

www.andes.nl

Appendices

Appendix A: Project Muiderburght

The initiative for this study was given by the project Muiderburght in Lake Marken (The Netherlands). See Figure A.1 for the location.

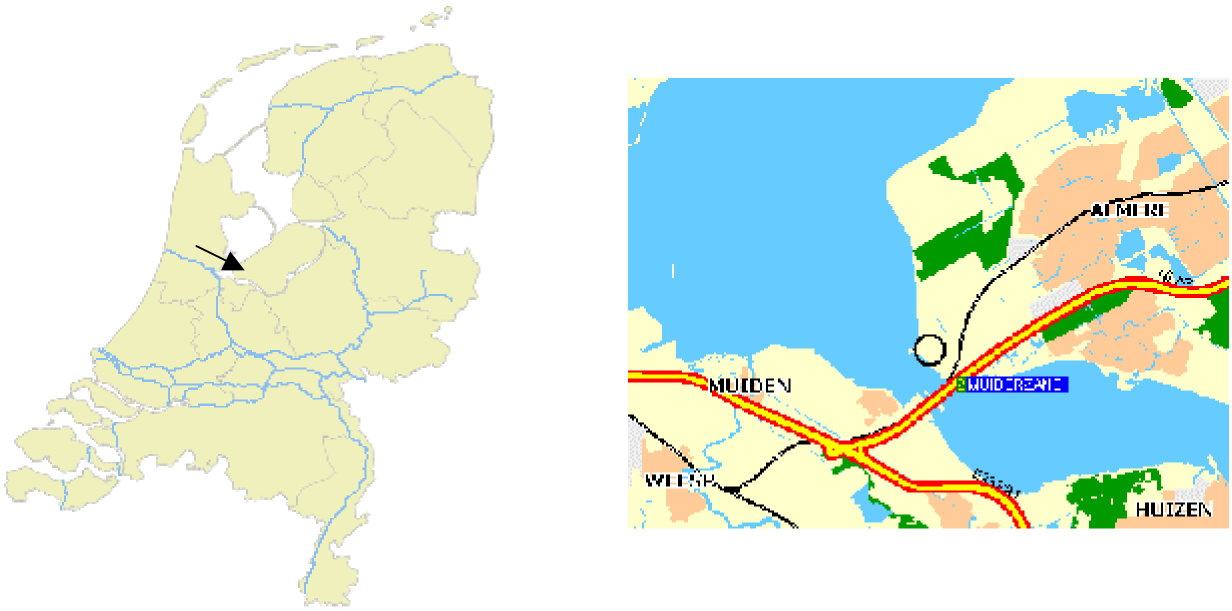

Figure A.1: Location of project Muiderburght (www.andes.com)

In fall 2001 this project was constructed on the west coast of Flevoland, near Muiden. It consisted of three buildings with in total 78 apartments and a private beach for the occupants of these buildings. This new recreational beach was not stable without protection and out of some protection alternatives the offshore breakwater was chosen. The contractor asked Delta Marine Consultants to design these offshore breakwaters (especially the dimensions and positions). The main purpose was that maintenance (refilling the sand of the beach) would only be necessary once in the five till ten years. To be sure the sand would stay at the beach, tombolos were chosen to be the preferred beach plan form. Using some empirical relationships they came with the plan of Figure A.2 (simplified). The dimensions of Figure A.2 are given in Table A.1 (Rovers, 2001). The gap width between the breakwaters is 15 meters.

Table A.1: Dimensions Muiderburght

	Breakwaters Muiderburght		
	west	middle	east
length breakwater (m)	40	60	40
distance offshore (m)	30	30	30
Water depth at structure, winter (m)	1.3	1.3	1.3
water depth at structure, summer (m)	1.5	1.5	1.5

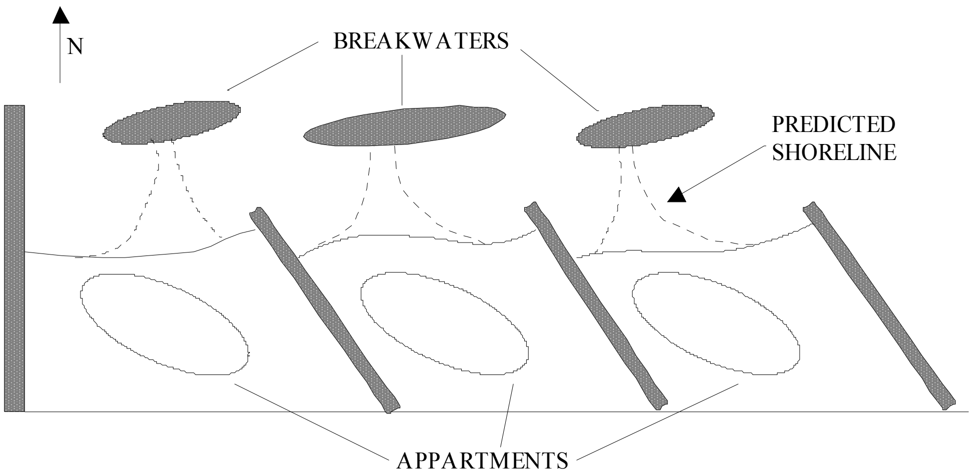

Figure A.2: design plan of Delta Marine Consultants

The designers (Delta Marine Consultants) were wondering if it was right to use the existing empirical relationships in this situation, because most empirical relationships are deducted from former projects in oceans and Muiderburght is located in a lake. But they did not have the time to investigate the effects of the location and they just used the relationships.

After all, in Muiderburght it turned out that the empirical relationships predicted the shoreline response quite well. Behind the east and middle breakwater a tombolo formed; only behind the west breakwater no tombolo formed. However, the effect of wave height (the main, and general, difference between lakes and oceans) on the shoreline response behind offshore breakwaters remained interesting. That is the reason why this study is carried out by Delta Marine Consultants.

Appendix B: Empirical relationships

Table 4.2 showed the empirical relationships most used. The background and theory of the diffraction method (Shore Protection Manual) were discussed in detail in Chapter 4. Here the background of the other relationships will be discussed. The information about these empirical relationships is obtained from Rosati (1990). For a definition of used terms, see Figure B.1.

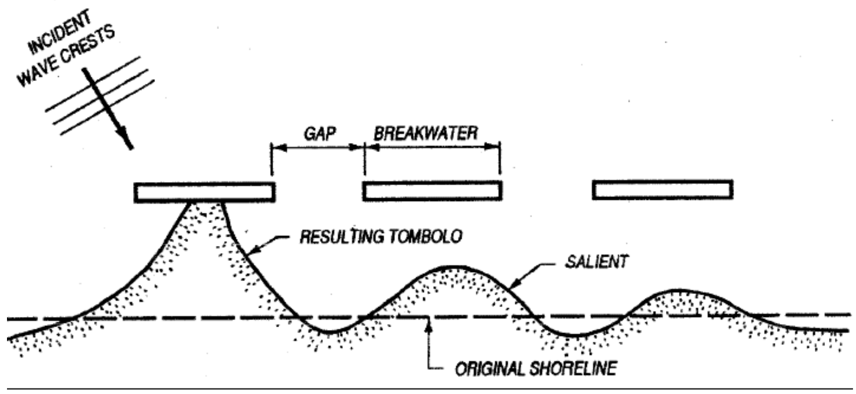

Figure B.1: Definition of used terms (Coastal Engineering Manual)

In all relationships the length of the structure is called $L_{(s)}$, the distance offshore is called X and the gap width is called G or L_g.

Gourlay (1981)

This study is based on results of a physical model and field observations. A comparison of these results with former laboratory studies gives the following conclusions:

- Tombolo's can form only if the structure is located in the surf zone. The ratio L / X should be more than 0.67
- A (double) tombolo develops when L / X > 2. This is caused by noninterfering diffraction patterns
- A salient develops when L / X < 0.4 a 0.5. Here the diffraction patterns interfere and a tombolo can not be formed

Ahrens & Cox (1990)

These researchers used the beach response index classification scheme of Pope and Dean (1986) to develop a relationship for expected beach response.

$I_s = e^{(1,72 - 0,41L/G)}$

I_s is the beach response index, coded as follows:
$I_s = 1$ Permanent tombolo formation
$I_s = 2$ Periodic tombolos ($L / G > 2.5$)
$I_s = 3$ Well developed salients ($L / G < 1.5$)
$I_s = 4$ Subdued salients ($L / G < 0.8$ a 1.5)
$I_s = 5$ No sinuosity ($L / G < 0.27$)

Dally & Pope (1986)

The results of this study are, like in the Shore Protection Manual, based on the diffracting wave pattern in the lee of the breakwaters. In this study, however, some breakwater projects in the USA are evaluated too. The conclusions of this study are:

- For a single breakwater tombolos will form when $L / X > 1.5$ to 2
- For a multiple breakwater applies $L_s / X > 1.5$
- For both a single and a multiple breakwater tombolo formation will be prevented when $L / X < 0.5$

Suh & Dalrymple (1987)

This study is based on unscaled monochromatic movable- bed laboratory tests. Combining the laboratory results with available prototype data, they come to the following relationships:

- For a single breakwater a tombolo will usually form when $L / X > 1$
- For a multiple breakwater this will happen for $L_g X / L_s^2 < 0.5$
- When the ratio's are respectively smaller or larger, a salient will form

From this summary, it becomes clear that the empirical relationships are all deducted under different conditions leading to different limits for the ratio structure length - distance offshore. This makes it very difficult to investigate and improve these relationships unambiguous.

Appendix C: Gradients in breaking wave height and wave setup

This appendix shows the calculation of the gradients in breaking wave height and wave setup for three different wave heights: 1, 2 and 3 m. Figure 5.6 and 5.7 show the results of these three calculations. The breaking wave height and wave setup are calculated with the help of the following formulas:

$$H_{br} = K_s K_d H_0 = h_{br}\gamma \qquad \text{(B.1)}$$

where:
H_{br} = breaking wave height (m)
K_s = shoaling coefficient, determined iteratively (-)
K_d = diffraction coefficient, read from diffraction diagram (-)
H_0 = wave height at deep water (m)
h_{br} = breaker depth (m)
γ = breaker index = 0,8 (-)

$$\eta_s = \frac{5}{16}\gamma H_{br} \qquad \text{(B.2)}$$

where:
η_s = wave setup

The locations where the breaking wave height and the wave setup will be calculated are derived from the diffraction diagram. In the diffraction diagram, the layout of project Lake View Park has been drawn with the adjusted shoreline according to the diffraction method. At the locations where the lines of equal diffraction cross this shoreline, the wave height and wave setup have been calculated.

Both gradients can be calculated with these formulas:

$$gradient = \frac{\Delta H_{br}}{\Delta y} \text{ and } gradient = \frac{\Delta \eta_s}{\Delta y} \qquad \text{(B.3 and B.4)}$$

where:
ΔH_{br} = difference in breaking wave height between two locations (m)
$\Delta \eta_s$ = difference in wave setup between two locations (m)
Δy = alongshore distance between these two locations (m)

The two locations where the differences in breaking wave height and wave setup are measured between, are the crossings of the lines of equal diffraction with the shoreline according to the diffraction method. The distance between these locations can be measured in the diffraction diagram. First the ratio distance – wavelength can be read from the diagram and then this ratio can be multiplied with the wavelength at the breakwater to obtain the real distance. The next pages give first the wave characteristics. Then the diffraction diagram is shown with the shoreline according to the diffraction method drawn in it (this shoreline is pointed with an arrow). And at last the tables with the calculated values are given.

Wave height at deep water = 1 metre

Period = 5 seconds
Wavelength at deep water = 39 metres
Wavelength at breakwater = 27 metres

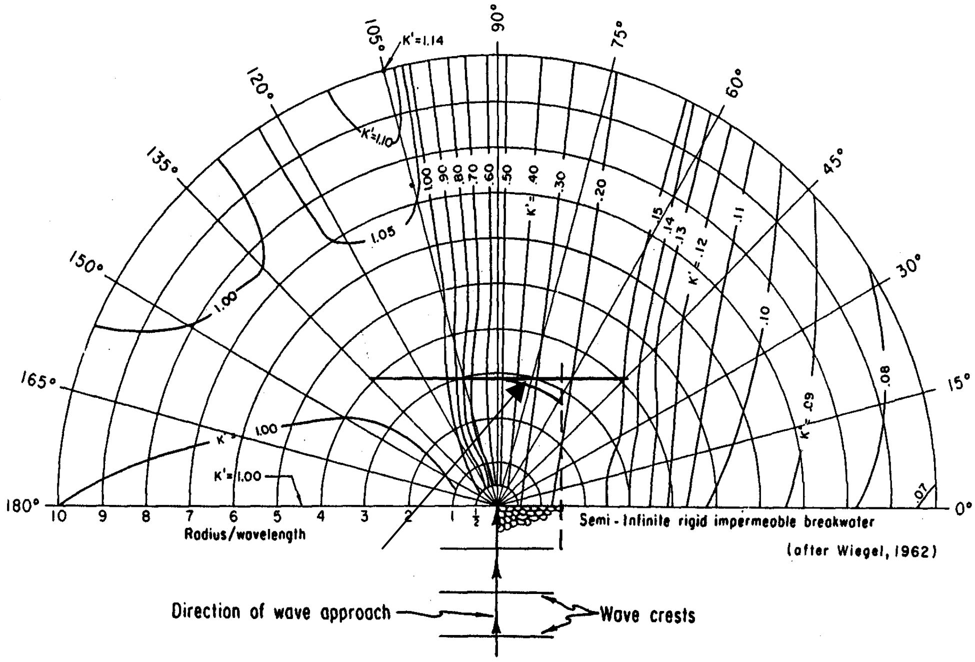

Figure C.1: Diffraction diagram with dimensions for $L_{at\ breakwater}$ = 27 m. (Shore Protection Manual)

Table C.1: Calculation gradient in breaking wave height and wave setup for H =1 m.

	Diffraction coefficient					
	1	0,6	0,5	0,4	0,3	0,22
H (m)	1	0,6	0,5	0,4	0,3	0,22
hbr (m)	1,37	0,89	0,77	0,65	0,52	0,39
Hbr (m)	1,096	0,712	0,616	0,52	0,416	0,312
setup (m)	0,274	0,178	0,154	0,13	0,104	0,078
y (m)	72,8	47,6	37,8	28	16,8	0
delta setup (m)		0,096	0,024	0,024	0,026	0,026
delta H (m)		0,384	0,096	0,096	0,104	0,104
delta y (*L at breakwater)		0,9	0,35	0,35	0,4	0,6
delta y (m)		25,2	9,8	9,8	11,2	16,8
i (setup)		0,0038	0,0024	0,0024	0,0023	0,0015
i (Hbr)		0,0152	0,0098	0,0098	0,0093	0,0062

Wave height at deep water = 2 metres

Period	= 7 seconds
Wavelength at deep water	= 76.4 metres
Wavelength at breakwater	= 40.5 metres

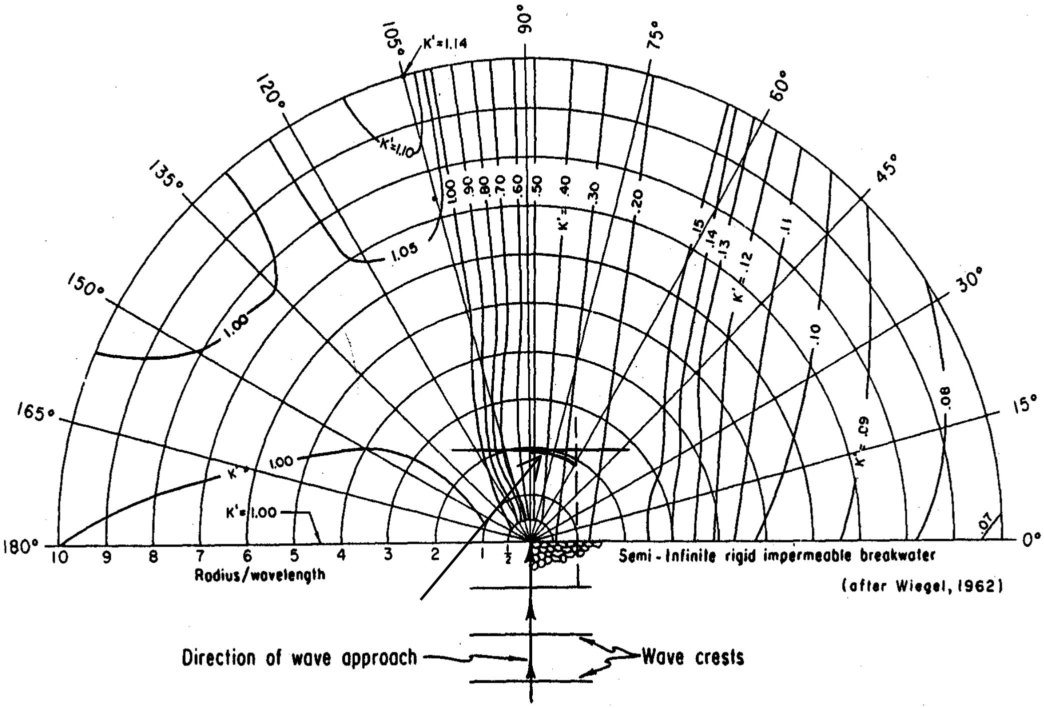

Figure C.2: Diffraction diagram with dimensions for $L_{at\ breakwater}$ = 40.5 m. (Shore Protection Manual)

Table C.2: Calculation gradient in breaking wave height and wave setup for H =2 metres

	Diffraction coefficient					
	1	0,6	0,5	0,4	0,3	0,27
H (m)	2	1,2	1	0,8	0,6	0,54
hbr (m)	2,72	1,79	1,53	1,28	1,01	0,93
Hbr (m)	2,176	1,432	1,224	1,024	0,808	0,744
setup (m)	0,544	0,358	0,306	0,256	0,202	0,186
y (m)	80,73	47,61	35,19	22,77	6,21	0
delta h (m)		0,186	0,052	0,05	0,054	0,016
delta Hbr (m)		0,744	0,208	0,2	0,216	0,064
delta y (*L at breakwater)		0,8	0,3	0,3	0,4	0,15
delta y (m)		33,12	12,42	12,42	16,56	6,21
i (setup)		0,0056	0,0042	0,0040	0,0033	0,0026
i (Hbr)		0,0225	0,0167	0,0161	0,0130	0,0103

Wave height = 3 metres

Period = 8.6 seconds
Wavelength at deep water = 115.4 metres
Wavelength at breakwater = 50.7 metres

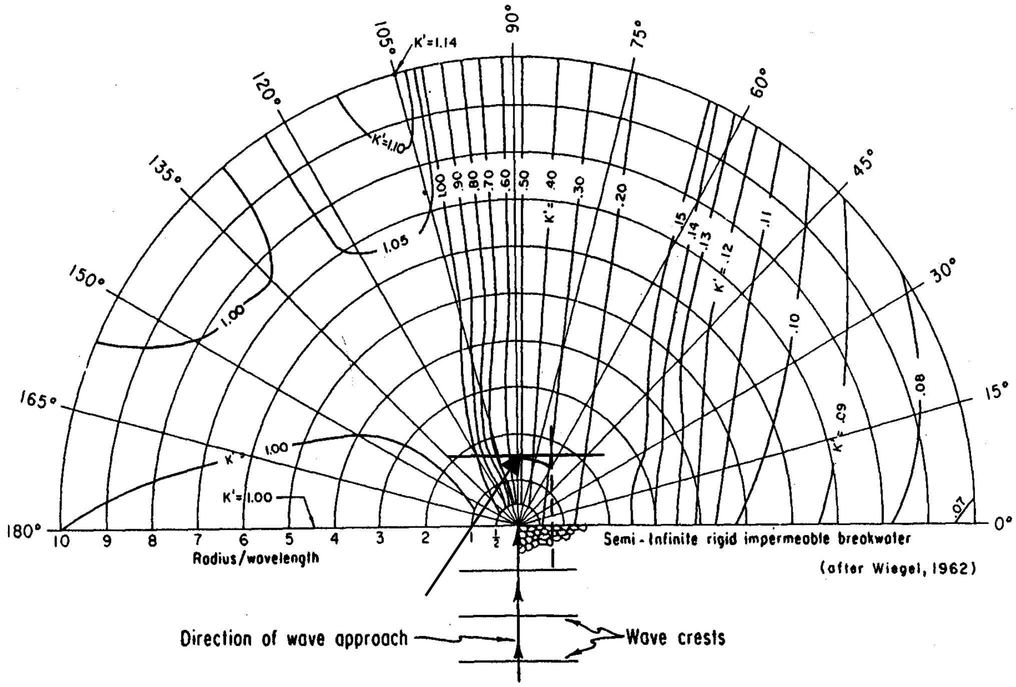

Figure C.3: Diffraction diagram with dimensions for $L_{at\ breakwater}$ = 50.7 m. (Shore Protection Manual)

Table C.3: Calculation gradient in breaking wave height and wave setup for H = 3 metres

	Diffraction coefficient					
	1	0,6	0,5	0,4	0,3	0,29
H (m)	3	1,8	1,5	1,2	0,9	0,87
hbr (m)	4,08	2,66	2,3	1,94	1,52	1,48
Hbr (m)	3,264	2,128	1,84	1,552	1,216	1,184
setup (m)	0,816	0,532	0,46	0,388	0,304	0,296
y (m)	82,521	46,191	33,216	20,241	2,076	0
delta setup (m)		0,284	0,072	0,072	0,084	0,008
delta Hbr (m)		1,136	0,288	0,288	0,336	0,032
delta y (*L at breakwater)		0,7	0,25	0,25	0,35	0,04
delta y (m)		36,33	12,975	12,975	18,165	2,076
i (setup)		0,0078	0,0055	0,0055	0,0046	0,0039
i (Hbr)		0,0313	0,0222	0,0222	0,0185	0,0154

Appendix D: Maple plots equation 6.15

This appendix gives plots of equation 6.15 for different bottom slopes.

Equation 6.15:
$$dh = \frac{\frac{1}{25}\sin(2\theta)}{3\left(1+0{,}08\left(1+2\sin^2\theta\right)\right)\left(\frac{1}{6\tan\alpha}\right)\left(\frac{2}{15}\left(\frac{1}{2}+\cos^2\theta\right)\right)}$$

For all plots: the a-axes is θ in radians, the y-axes is the gradient in breaker depth ($\Delta h_{br}/\Delta y$)

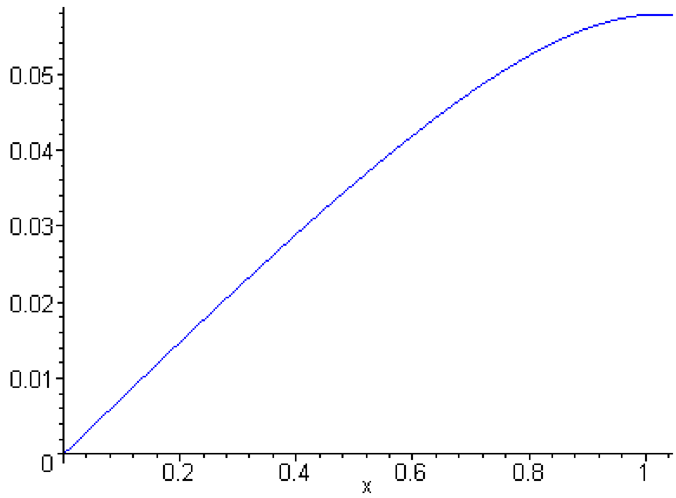

Figure D.1: Relation gradient in breaker depth and θ for tan(α) = 1/10

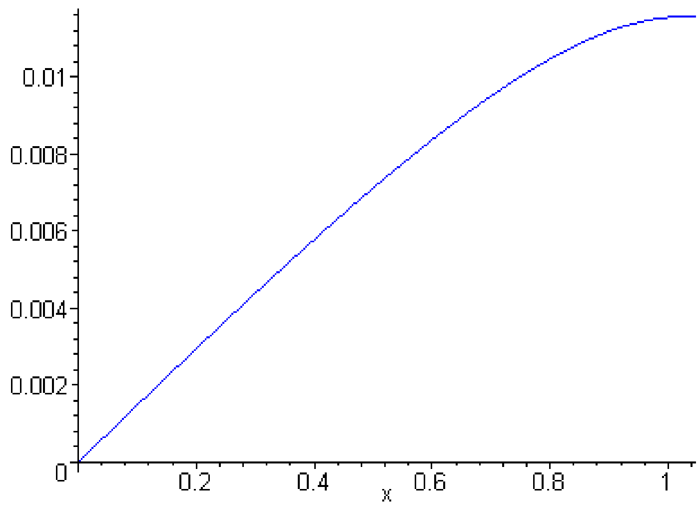

Figure D.2: Relation gradient in breaker depth and θ for tan(α) = 1/50

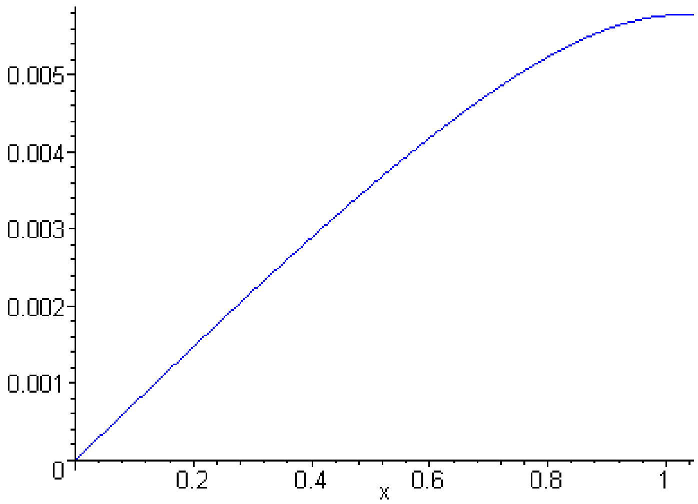

Figure D.3: Relation gradient in breaker depth and θ for tan(α) = 1/100