A Bypass Friendly Harbour

A study to the possibilities of a bypass friendly harbour

Master of Science Thesis - Rob Spruit

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01-05-2013

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PREFACE

This thesis constitutes the final part of my Master's degree programme of Civil Engineering at the Delft University of Technology. At the department of Hydraulic Engineering I have studied the possibilities of achieving a bypass friendly harbour. This topic was put forward by the consultancy firm Arcadis. Amongst others, their field of expertise is the design of harbours and their influence on the morphology of the coastline. Within this perspective they are interested in the possibilities of designing bypass friendly harbours. This means that the maximum amount of sediment possible should bypass the harbour in combination with maintaining sufficient serviceability of the harbour. This seemed me to be an interesting topic to study.

I started my research with a literature study on the topic. After having completed this, it was time to test the theory with numerical models. Arcadis' department with expertise on numerical models was based in Zwolle so I frequently worked at the office in Zwolle to set up the model. With their help I was able to quickly set up a numerical model to study the bypassing process. With the results of the numerical models, I was able to find the conditions at which as much sediment as possible bypasses the harbour in combination with maintaining sufficient serviceability of the harbour. I hope the results of this study give Arcadis more input in the design of bypass friendly harbours morphologically active coastlines.

The report is written for those people with a significant level of background knowledge on coastal engineering.

I cannot finish this preface without expressing my gratitude to the people who facilitated this research with their highly appreciated contributions. First of all, I want to thank Jan van Overeem and Rob Steijn as my supervisors at Arcadis. They frequently gave me fruitful advice on where to focus on with my research. Their support also helped me to gain better insight in the acting hydrodynamic and morphodynamic processes. I also would like to thank Bart Grasmeijer and Ivo Pasmans for their support in setting up the numerical model. I could not have done it within the set time without their help. I would also like to hank professor Stive for his critical and striking remarks during the meetings with the graduation committee. The same accounts for professor Vellinga who especially looked into the harbour related matters.

Finally, I thank my friends, D-01, my family and my girlfriend Danneke for their support and trust, especially my parents since my activities in Delft did not always fit an educational purpose.

Enjoy reading this thesis!

Rob Spruit

EXECUTIVE SUMMARY

A harbour is an interruption of the coastline and it therewith blocks the longshore sediment transport. This interruption causes accretion at the updrift side of the harbour and erosion at the downdrift side of the harbour. The downdrift erosion could harm the protective function of the coastline and the updrift accretion could eventually lead to sedimentation in the harbour basin or the harbour entrance. The sedimentation in and in front of the harbour decreases the serviceability of the harbour. Both the accretion and the erosion are thus undesirable. The mismatch of the navigational requirements and the morphological processes around the harbour defines the problem of this research. This mismatch can be decreased if the sediment is able to bypass the harbour instead of accumulation in or in front of the harbour. The sediment that is able to bypass the harbour contributes to a decrease in the downdrift erosion.

Research objective

The objective of this study is to determine for which conditions the bypass capacity is optimized in combination with a limited influence on the navigational function of the harbour. The bypass capacity is defined as the capacity to bypass the sediment around the harbour under the presence of the defined driving forces (i.e. waves and/or tide). The conditions involve varying forcing, geometry of the breakwater, bathymetry, the presence of a dredged entrance channel and the grain size. Finding such conditions is part of an Integrated Coastal Zone Approach[1]. The approach is to find conditions at which the interests of the harbour manager and the coastal zone manager are both addressed in a reliable and sustainable way.

Preliminary study

The capacity for sediment bypassing depends amongst others on the sediment transport regime and the harbour geometry. For this research both phenomena have been categorized to make up a framework of transport regimes and types of harbours. This gives 6 archetypes with specific bypass characteristics, which are explained in an overview in Appendix C. Not every archetype is equally promising in achieving the objective. Only the most promising archetype has been studied more thoroughly. Therefore the hydrodynamic and morphodynamic processes of each archetype have been studied with the help of existing literature, reference situations and a sensitivity analysis of the relevant parameters.

[1] In an Integrated Coastal Zone Approach, the interests of all the stakeholders are addressed in a sustainable way.

Based in this analysis it is concluded that a harbour which does not require a dredged channel and only services vessels with a small draft (approx. $2.5\ m - 3.5\ m$), along a coastline which experiences a large net transport is the most relevant and promising archetype to study. This is a relevant case because the downdrift coastline experiences erosion which opposes a problem for the coastal manager. The described case is a promising case because the small harbour blocks less sediment, there is no dredged channel and the contraction of the tidal current in combination with the wave-breaking-induced current is expected to give sufficient bypass capacity.

Model calculations

The most promising archetype is studied more thoroughly with the numerical model Delft3D. This gives a more accurate prediction of the bypass capacity in combination with maintaining sufficient serviceability of the harbour. From a sensitivity analysis and the literature study it was concluded that the parameters, as specified below, have significant influence on the bypass capacity.

1. The extension of the breakwater in the surf zone, related to the width of the surf zone;
2. The wave height;
3. The wave angle;
4. The shape of the breakwaters;
5. The current velocity;

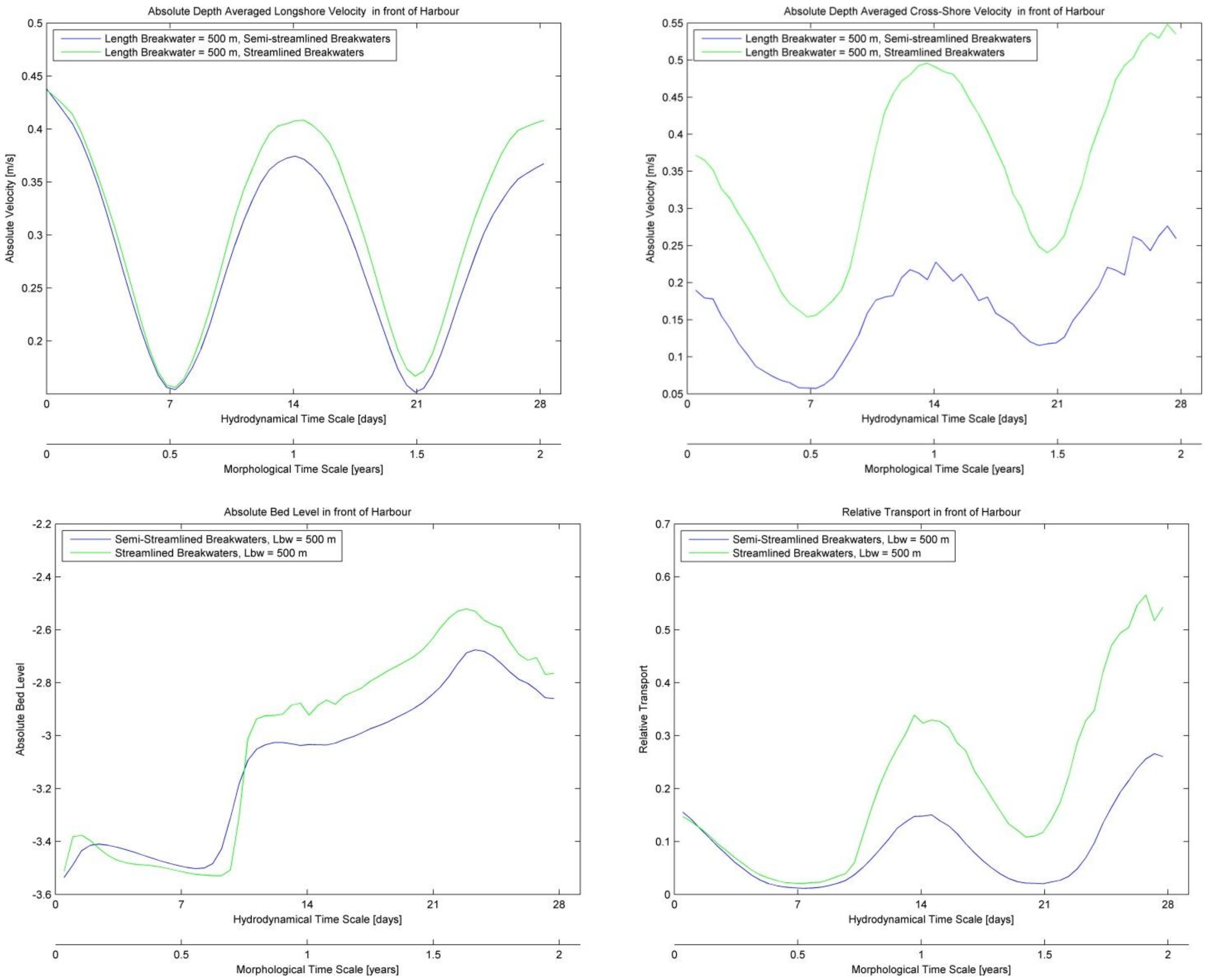
The influence of these parameters has therefore been assessed with Delft3D. This has been done by varying the parameters and by comparing the relative amount of bypassing and the relative depth with each other. The amount of bypassed sediment is measured relative to the amount of sediment transport at the updrift, undisturbed coastline. The depth is measured relative to the initial depth. A third variable is the amount of sedimentation inside the harbour because sedimentation in the harbour also decreases the serviceability of the harbour and it has a negative influence on the amount of bypassed sediment. All three should be optimized for a positive answer on the problem defined in this research. Optimal bypass conditions are for instance useless if the serviceability of the harbour has decreased significantly due to extensive sedimentation in or in front of the harbour.

Conclusions

The model calculations have provided output on the influence of each parameter on the bypass capacity and the serviceability of the harbour. The most promising situations are assessed on a longer time scale to see whether the promising situation holds. This gives the following conclusion (see next page):

This research shows that a bypass friendly harbour is possible under certain circumstances.

These circumstances are a harbour with streamlined breakwaters and an extension equal to the width of the surf zone at a coastline with significant wave and tidal influence. The streamlined geometry of the breakwaters leads to higher longshore flow velocities in front of the harbour, related to a harbour equal in size but with semi-streamlined breakwaters. The higher flow velocities enlarge the bypass capacity. The high flow velocities, in combination with the extent of the breakwaters also lead to a limited decrease of the depth in front of the harbour and very little sedimentation in the harbour. Both the limited decrease of the depth in front of the harbour and the little sedimentation in the harbour have a positive effect on the amount of bypassing. The bypass is also initiated quicker than that in case of semi-streamlined breakwaters because less sediment is required to make the new coastline orientation in case of the streamlined breakwaters. The described effects can be seen in the below figures.



Absolute longshore velocity (top left), absolute cross-shore velocity (top right), absolute bed level (bottom left) and the relative bypassing (bottom right) for a harbour with semi-streamlined and streamlined geometry. The extension of both harbour is equal to the width if the surf zone. The applied hydrodynamical forcing is also equal.

The influence of each assessed parameter is discussed below.

(1) A harbour extension shorter than the width of the surf zone is more promising than a harbour extension longer than the width of the surf zone. The shorter harbour blocks less sediment and, due to the small depth, there is more wave breaking in front of the harbour. This enlarges the transport capacity. There is however quite some sedimentation in the harbour.

(2) For the same reason higher waves are also more promising. Higher waves break further offshore, which decreases the blockage coefficient of the harbour. The bypass is also initiated quicker due to the higher wave energy. It does however give more sedimentation in the harbour.

(3) Offshore wave angles around $45^o - 60^o$ give more sediment transport than waves with a small ($< 45^o$) or a very large ($> 60^o$) offshore wave angle and require less sediment to achieve an equilibrium orientation at the updrift side of the harbour. Therefore the new equilibrium and thus the bypass, is achieved quicker.

(4)The same accounts for a streamlined geometry of the harbour breakwaters. The shape of the streamlined breakwaters is close to the shape of the equilibrium orientation so less sediment is required to achieve this situation. The streamlined geometry also leads to more flow contraction in relation to a harbour of equal size but with semi-streamlined breakwaters. The higher flow contraction contributes to a higher bypass capacity. There is also less sedimentation in the harbour.

(5) Under the assumption of equal sediment input towards the harbour entrance, it can be concluded that a lower tidal velocity leads to a lower local transport capacity and thus to a rapid decrease of the depth in the harbour entrance. The presence of a dredged channel causes a local decrease of the flow velocity which also leads decrease of the local transport capacity.

Recommendations

The bypass capacity and the serviceability of a harbour with streamlined breakwaters with an extension equal to the width of the surf zone is better than that of a harbour with the same length but with semi-streamlined breakwaters. Not every aspect of that is fully understood and it is recommended to study the hydrodynamic processes around streamlined breakwater more thoroughly for two reasons:

- The streamlined geometry of the breakwaters leads more flow contraction in front of the harbour than the harbour with a semi-streamlined geometry of the breakwaters. This enlarges the transport capacity due which it is expected that new equilibrium depth is also larger. This is however not the case and it is recommended to find out what the effect is of the higher flow contraction on the bypass capacity and thus on the new equilibrium depth.
- The streamlined shape of the breakwaters also causes less sediment that is transported into the harbour than there is in case of semi-streamlined breakwaters. This effect is not fully understood yet because it could also be argued that the larger inner harbour area for streamlined breakwaters causes higher cross-shore currents (which it does) and thus more import of sediment into the harbour. This is however not the case and it is recommended to study the hydrodynamical processes better to get more insight in the reason behind this. This could for instance be caused by a better following of the breakwater by the flow in case of streamlined breakwaters.

- Furthermore, the streamlined shape is chosen quite arbitrary. It must be possible to optimise the shape with relation to the bypass capacity and the sedimentation in and in front of the harbour. The width of the harbour entrance could also be a variable in that optimisation process.

Considering the model, the main recommendations are to find a way to get rid of the boundary effects and to find a way to model refraction correctly. The boundary effects cause large erosion and sedimentation at the boundaries of the applied grid. These strange erosion/sedimentation patterns affect the results on a longer time scale. The model overestimates refraction which influences the transport rates. This has a limited influence on the outcome of this research but is it is recommended to solve it. Thirdly it is recommended to apply a finer grid. This makes it possible to model diffraction and the vortices caused by spurs. It also makes the calculations more accurate.

Two software packages are applied in this study: Unibest-CL+ and Delft3D. At first Unibest-CL+ was used to assess the influence of various parameters and to assess the sensitivity of system to parameters. The conclusion of the preliminary study is partly based on this analysis. After that it was chosen to continue with Delft3D for a more accurate assessment.

Afterwards it can be concluded that the combination of applying Unibest-CL+ for the sensitivity analysis and Delft3D for the final calculation has been successful. The most relevant parameters could be determined on the basis of the sensitivity analysis. The more exact behaviour of the system and the influence of the parameters could consequently be determined with Delft3D because this computes the processes in a 2DH environment. The striking conclusion on the influence of the streamlined breakwaters could never have been found if only Unibest-CL+ was applied. Therefore it is recommended to continue further research with Delft3d, especially if the above recommendations are followed. If a quick assessment of certain parameters is required, it is recommended to apply Unibest-CL+

Limitations

This study has only included (nearly) straight, sandy coastlines which are influences by only waves and tides. Other forcing such as wind, river discharge, Coriolis, etc. are excluded from this study.

The applied model opposes limitation on this study as well. The model overestimates refraction which causes less transport capacity due to wave breaking. The model also excludes diffraction due to which the accretion patterns at the downdrift side of the harbour are not modelled correctly. The model is also unable to model vortices due to flow separation because the applied grid cell size is too large. Therefore the effect of spurs could not be assessed. Furthermore it restricts the applied forcing and the modelled time-scale due to boundary effects.

SAMENVATTING

Een haven is een onderbreking van de kustlijn en het onderbreekt daarmee het sediment transport. Dit leidt to aanzanding bovenstrooms van de haven en erosie benedenstrooms van de haven. De benedenstroomse erosie kan de beschermende functie van de kustlijn schaden. De bovenstroomse aanzanding kan uiteindelijk leiden tot sedimentatie in de haven en de haveningang. Dit beperkt de vaardiepte in en naar de haven. Beide effecten zijn ongewenst en de mismatch tussen de navigatie-eisen en de morfologische processen is het probleem dat in dit onderzoek onderzocht wordt. Deze mismatch kan verkleind worden als voldoende sediment langs de haven getransporteerd wordt (bypassing) in plaats van bovenstrooms en voor de havenmond aan te zanden. Het sediment dat langs de haven is getransporteerd, verkleind vervolgens de benedenstroomse erosie.

Onderzoeksdoel
Het doel van deze studie is het vinden van omstandigheden waarbij de bypass capaciteit zo optimaal mogelijk is terwijl dit een beperkte invloed heeft op de navigatie functie van de haven. De bypass capaciteit is gedefinieerd als de capaciteit om het sediment langs de haven te transporteren onder de voorwaarde dat de aandrijvende krachten (golven en/of getijde) aanwezig zijn. De omstandigheden hebben betrekking op variërende aandrijvende krachten, vorm en afmetingen van de haven, de bodemligging, de aanwezigheid van een verdiept toegangskanaal en de korreldiameter van het sediment. Het zoeken naar deze omstandigheden valt onder een Geïntegreerds Kust Zone Aanpak waarbij de belangen van zowel de haven autoriteit als de kust manager behartigd worden op een duurzame manier.

Vooronderzoek
De bypass capaciteit hangt onder andere af van sediment transport regime en het type haven. Beiden zijn in dit onderzoek gecategoriseerd om zodoende aan raamwerk the creëren van types transport regimes en havens. Hierdoor ontstaan 6 archetypes met elk specifieke bypass eigenschappen welke zijn uitgelegd in een schema in Appendix C. Niet ieder archetype is even veelbelovend voor het behalen van het doel. Alleen het meest veelbelovende archetype kan verder onderzocht worden. Daarom zijn de hydrodynamische en morfologische processen van ieder archetype bepaald met behulp van theorie, referentie situaties en een gevoeligheidsanalyse naar de relevante parameters.

Op basis van deze analyse is geconcludeerd dat een haven zonder gebaggerd toegangskanaal, welke alleen schepen met een kleine diepgang (ongeveer 2.5 m) accommodeert in een kustlijn met een groot netto langstransport de meest veelbelovende en relevante situatie om verder te bestuderen. De situatie is relevant omdat de benedenstroomse kust erodeerd en de situatie is veelbelovend omdat de haven minder sediment transport blokkeert en omdat de contractie van de stroming gecombineerd met de het breken van golven de bypass capaciteit kan verhogen.

Model berekeningen

De meest veelbelovende situaties zijn gemodelleerd met Delft3D. Dit geeft nauwkeurigere voorspellingen wat betreft de bypass capaciteit gecombineerd met de behouden van voldoende vaardiepte in en naar de haven. Vanwege de gelimiteerde beschikbare tijd is het niet mogelijk om alle parameters te onderzoeken met Delft3D. Uit de gevoeligheidsanalyse en de literatuurstudie volgt dat de volgende parameters significante invloed hebben op de bypass capaciteit:

1. De uitbouw van de golfbrekers in de brandingszone, relatief ten opzichte van de breedte van de brandingszone;
2. The hoek van de inkomende golven;
3. De hoogte van de inkomende golven;
4. De vorm van de golfbreker;
5. De stroomsnelheid door het getijde;

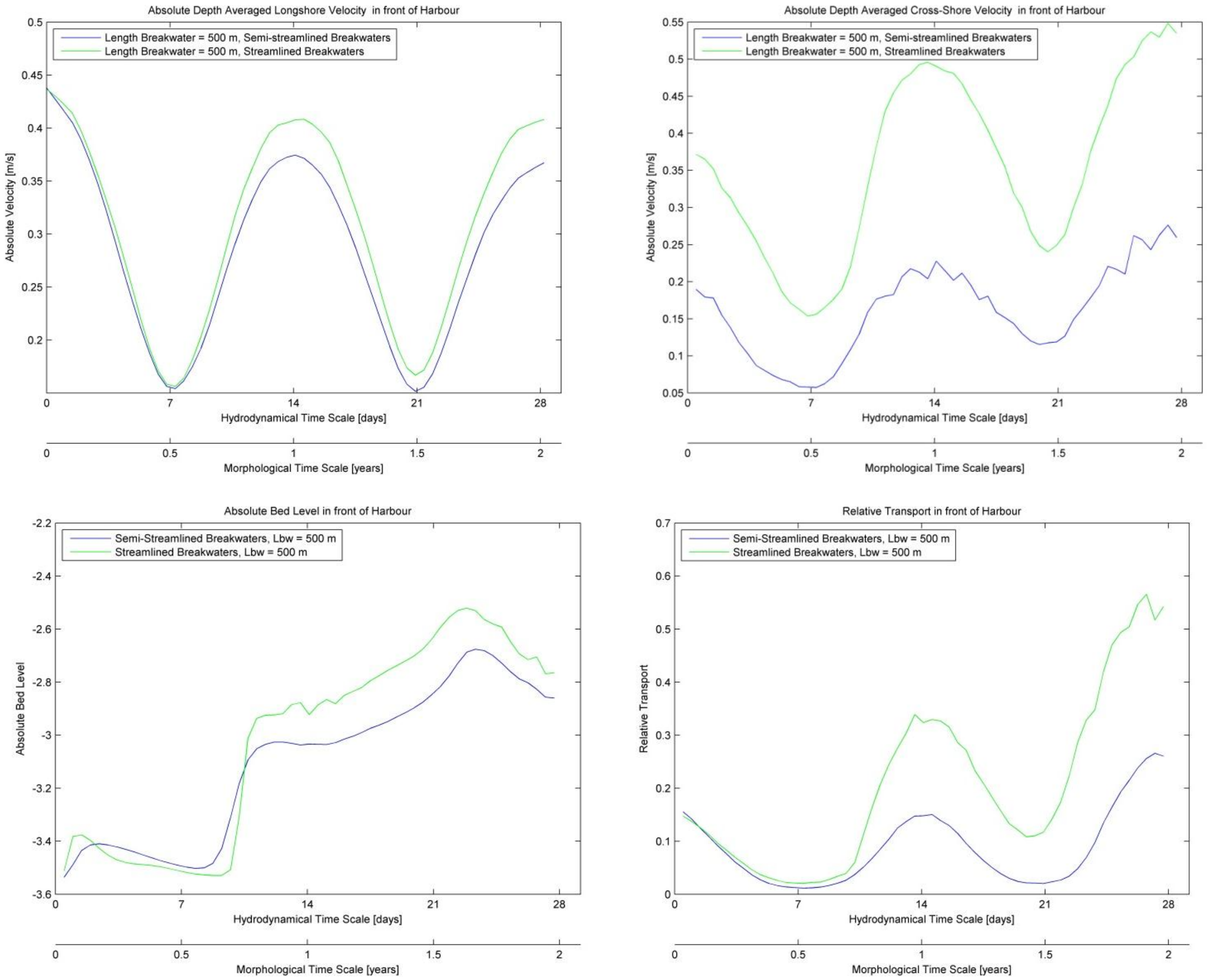
De invloed van deze parameters is daarom onderzocht met Delft3D. Dit is gedaan door de waarde van de parameters te variëren en de resultaten met elkaar te vergelijken. De hoeveelheid sediment dat langs de haven is getransporteerd is gedeeld door de hoeveelheid sediment transport op een ononderbroken kustlijn bovenstrooms van de haven. De diepte is gedeeld door initiële diepte. Zodoende ontstaat de relatieve bypassing en de relatieve diepte. Een derde variabele is de hoeveelheid sediment dat de haven in wordt getransporteerd en daar neerslaat omdat sedimentatie in de haven ook de vaardiepte verminderd en het sediment dat de haven instroomt niet bijdraagt aan de hoeveelheid sediment dat langs de haven wordt getransporteerd. Alle drie de parameters moeten geoptimaliseerd worden voor een positief antwoord op het probleem. Een optimale bypass capaciteit is namelijk waardeloos als de vaardiepte naar en in de haven enorm is afgenomen.

Conclusies

De modelresultaten geven de invloed van iedere parameter op het bypass proces en de navigatie functionaliteiten van de haven. De meest veelbelovende situaties zijn op een langer termijn getoetst om te zien of deze veelbelovende situatie stand houdt. Dit geeft de volgende conclusie:

Dit onderzoek toont aan dat een bypass vriendelijke haven mogelijk is onder bepaalde omstandigheden.

Deze omstandigheden zijn een haven met gestroomlijnde golfbrekers die even lang zijn als de breedte van de brandingszone in een kustlijn die wordt beïnvloed door golven en getijde. De gestroomlijnde vorm zorgt voor hogere stroomsnelheden voor de haven dan wanneer semi-gestroomlijnde golfbrekers zijn toegepast. De hogere stroomsnelheden vergroten de bypass capaciteit. De hogere stroomsnelheden, in combinatie met de lengte van de golfbrekers lijden ook een beperkte afname van de diepte in de haven ingang en tot minder sedimentatie in de haven. De nieuwe evenwichtsdiepte ligt op 75% van de originele diepte. Dit kan gezien worden in de onderstaande figuren.



Absolute langs stroomsnelheden (top links), absolute dwars stroomsnelheden, (top rechts), absolute diepte (beneden links) en de relatieve bypassing (beneden rechts) voor een haven met semi-gestroomlijnde en gestroomlijnde golfbrekers. De lengte van de golfbrekers is in beide gevallen gelijk aan de breedte van de brandingszone. De toegepaste hydrodynamische krachten zijn gelijk.

De invloed van iedere onderzochte parameter wordt hieronder beschreven.

(1) Een haven die korter is dan de breedte van de brandingszone, geeft betere resultaten dan een haven die langer is dan de breedte van de brandingszone. De kortere haven blokkeert minder sediment transport en er breken meer golven door de geringe diepte voor de haven. Beiden vergroten de bypass capaciteit van de situatie.

(2) Om dezelfde reden zijn hogere golven ook veelbelovend. Deze hebben een bredere brandingszone waardoor havens van dezelfde lengte minder sediment zullen blokkeren in geval van hogere golven.

(3) Golven met een hoek van $45^o - 60^o$ produceren het meeste sediment transport en er is minder sediment nodig om bovenstrooms de nieuwe evenwichtsvorm van de kustlijn te bereiken. Daarom is dit evenwicht eerder bereikt en zal er sneller zand voor en langs de haven stromen.

(4) Het zelfde geldt voor gestroomlijnde golfbrekers. Deze hebben bijna al de vorm van deze nieuwe evenwichtsvorm waardoor er minder zand nodig is om dit te bereiken. Er zal daarom eerder zand langs de haven stromen. Dit wordt versterkt door het geringe sediment transport de haven in en de extra stromingscontractie, veroorzaakt door de gestroomlijnde vorm van de golfbrekers. Het effect van uitstulpingen op de golfbreker kan niet met dit model worden onderzocht. De uitstulpingen zouden moeten leiden tot meer turbulentie maar deze vinden plaats om een kleinere schaal dan de toegepaste rooster afmetingen in het model. Een asymmetrische geometrie van de golfbrekers zorgt voor minder bypassing, maar de diepte voor de haven neemt niet af. Er is echter wel veel aanzanding in de haven waardoor de functionaliteit van de haven alsnog afneemt.

(5) Onder de aanname van gelijke toevoer van sediment naar de haveningang kan geconcludeerd worden dat een lagere stroomsnelheid leidt tot een lokaal kleinere transport capaciteit en dus tot een versnelde afname van de lokale diepte. De aanwezigheid van een gebaggerd kanaal leidt tot een lokale afname van de stroomsnelheid en heeft dus hetzelfde effect op de transport capaciteit.

Aanbevelingen

De bypass capaciteit en de vaardiepte in en naar de haven met gestroomlijnde golfbrekers is beter geoptimaliseerd dan de haven met dezelfde lengte maar met semigestroomlijnde golfbrekers. Niet ieder aspect wordt volledig begrepen en het wordt daarom aanbevolen om verder onderzoek te doen naar de hydrodynamische processen rondom gestroomlijnde havens, met name omwille van twee redenen:

- De gestroomlijnde vorm van de golfbrekers leidt to meer stromingscontractie voor de haven dan een semigestroomlijnde vorm van de golfbrekers. Dit vergroot de transport capaciteit waardoor het verwacht wordt dat de nieuwe evenwichtsdiepte voor de haven groter is. Dit is niet het geval en daarom wordt het aanbevolen om meer onderzoek te wijde aan de effecten van de hogere stromingscontractie op het bypass proces en dus op de nieuwe evenwichtsdiepte.
- De gestroomlijnde vorm van de golfbrekers zorgt er ook voor dat er minder sediment de haven in wordt getransporteerd in vergelijking met de semigestroomlijnde vorm van de golfbrekers. Dit effect wordt niet volledig begrepen om het ook beargumenteerd kan worden dat het grotere gebied achter de gestroomlijnde golfbrekers voor hogere dwars stroomsnelheden zorgt dan het geval is bij semigestroomlijnde golfbrekers (wat ook zo is), waardoor er meer sediment de haven in getransporteerd zou moeten worden. Dit is echter niet het geval en daarom speelt er dus meer een rol dan nu doet vermoeden. Daarom wordt het aanbevolen om hier in de toekomstig onderzoek naar te kijken.
- De vorm van de gestroomlijnde golfbrekers is vrij arbitrair gekozen en kan nog verder geoptimaliseerd worden als het gaat om bypass capaciteit en de vaardiepte voor en in de haven.

Het wordt tevens aanbevolen om de constructie kosten van de golfbrekers, de onderhoudskosten van de haven, de golfbrekers en de kustlijn mee te nemen in toekomstig onderzoek. De achterliggende gedacht van dit onderzoek is namelijk om de kosten te drukken en daarom wordt het aanbevolen om dit mee te nemen in toekomstig onderzoek.

Betreffende het model wordt het aanbevolen om een manier te vinden om de randeffecten kwijt te raken. Deze zorgen namelijk voor minder betrouwbare resultaten op de langere tijdsschaal. Het model overschat ook de refractie van de golven wat een effect heeft op de transport hoeveelheden. Ten derde kan uit het bovenstaande geconcludeerd worden dat de het aan te raden is om de berekeningen uit te voeren op een fijnmaziger rekenrooster. Dit vergroot the nauwkeurigheid en hierdoor wordt het mogelijk om de diffractie mee te nemen en de invloed van uitstulpingen te onderzoeken.

Er zijn twee programma's gebruikt om dit onderzoek tot een goed einde te brengen: Unibest-CL+ en Delft3D. Unibest-CL+ is toegepast om snel de invloed van verschillende parameters en de gevoeligheid van het systeem voor deze parameters te bepalen. De conclusie van de voorstudie is deels gebaseerd op deze analyse. Daarna is besloten door te gaan met Delft3D voor een nauwkeurigere analyse van de invloed van de meest bepalende parameters.

Naderhand kan geconcludeerd worden dat de combinatie van het gebruik van Unibest-CL+ voor de gevoeligheidsanalyse en Delft3D voor de uiteindelijke berekeningen succesvol is geweest. De meest relevante parameters zijn bepaald op basis van de gevoeligheidsanalyse. Het meer nauwkeurige gedrag van het systeem en de invloed van de parameters is vervolgens bepaald met Delft3D in de 2DH omgeving. De conclusie over de invloed van gestroomlijnde golfbrekers had nooit getrokken kunnen worden als alleen Unibest-CL+ was gebruikt. Daarom wordt het aanbevolen om vervolgonderzoek door te zetten met Delft3D, met name als de hierboven genoemde aanbevelingen opgevolgd worden. Als een snel inzicht nodig is in de invloed van verschillende parameters, wordt het aanbevolen om Unibest-CL+ te gebruiken.

Beperkingen

Deze studie heeft zich beperkt tot (bijna) rechte, zandige kustlijnen welke worden beïnvloed door golven en getijde. Andere krachten zoals wind, rivier afvoer, Coriolis, enzovoorts, zijn niet meegenomen.

Het toegepaste model heeft tot bepaalde beperkingen van het onderzoek geleidt. Het model overschat de refractie van de golven waardoor er minder transport capaciteit door het breken van golven wordt berekend. Het model neemt ook diffractie niet mee waardoor de aanzanding aan de benedenstroomse kant niet goed wordt gerepresenteerd. Het model is tevens niet in staat om vorticiteit veroorzaakt door de uitstulpingen te berekenen omdat het toegepaste rekenrooster te grof is. Daardoor kan het effect van deze uitstulpingen niet bepaald worden. Daarnaast beperken rand effecten het toepassen van zwaardere golfkrachten en het onderzoek op langere termijn. Ten slotte kan het model maar één korrel diameter modelleren in plaats van een gradatie.

CONTENTS

NOMENCLATURE

A	Shape factor for equilibrium profile	[-]
C	Volume concentration of sediment	[-]
D	Draught design vessel	$[m]$
D_{50}	Median equivalent grain diameter	$[m]$
F	Wave force	$[N]$
H	Wave height	$[m]$
H_0	Deep water wave height	$[m]$
H_b	Wave height at breaking	$[m]$
K_c	Coefficient, $K_c = 0.77$	[-]
K_{sh}	Shoaling parameter	[-]
L	Wave length	$[m]$
L_0	Deep water wave length	$[m]$
L_{BW}	Length breakwater	$[m]$
S	Sediment transport (in general, usually longshore)	$[m^3/time]$
S_x	Sediment transport in longshore direction	$[m^3/time]$
S_y	Sediment transport in cross-shore direction	$[m^3/time]$
S_{1^e}	Sediment transport due to prevailing waves	$[m^3/time]$
S_{2^e}	Sediment transport due to prevailing waves	$[m^3/time]$
T	Wave period	$[s]$
T_i	Period of the tide	$[h]$
$V(z)$	Longshore velocity	$[m/s]$
W	Width entrance channel	$[m]$
W_b	Bank clearance	$[m]$
W_{BM}	Basic width entrance channel	$[m]$
W_i	Several additions to the width of the entrance channel	$[m]$
W_p	Separation distance	$[m]$
Y	Coastline position	$[m]$
a	Wave amplitude	$[m]$
b	Width of a wave	$[m]$
b_0	Width of a wave in deep water	$[m]$
c	Wave or current velocity	$[m/s]$
c_0	Deep water wave velocity	$[m/s]$
c_g	Wave group velocity or phase velocity	$[m/s]$
d	Depth	$[m]$
d_c	Required depth entrance channel	$[m]$
f	Coriolis parameter	$[rad/s]$
g	Gravity constant	$[m/s^2]$
h	Water depth	$[m]$
h_b	Water depth at wave breaking	$[m]$
k	Wave number,	$[1/m]$
m	Exponent for equilibrium profile	[-]

m_s	Remaining safety margin or net under keel clearance	$[m]$
n	Wave number	[-]
r	Vertical vessel motion due to wave response	$[m]$
s_{max}	Maximum sinkage due to squat and trim	[m]
v	Velocity in y-direction	$[m/s]$
v_{wd}	Transverse speed of a ship as a result of wind drift	$[kn]$
v_{eff}	Vessel's speed with respect to the bottom	$[m]$
Φ	Latitude	[°]
α	Slope of the bed	[-]
α_{cc}	Angle between channel axis and current	[°]
γ	Breaker index	[-]
η	Water level	$[m]$
θ_{cr}	Shields parameter	[-]
ν	Dynamic viscosity	$[m^2/s]$
ξ	Irribarren number	[-]
ρ	Density water	$[kg/m^3]$
ρ_s	Density sediment	$[kg/m^3]$
φ	Wave angle	[°]
φ_b	Wave angle at breaking	[°]
ω_i	Angular velocity tide	[rad/h]

CHAPTER 1. INTRODUCTION

This chapter will give an introduction to the problem as well as the research objective and question. The research can be very broad so this chapter will also give the scope of the research. Coastal areas have been studied widely by researchers all around the world. These researches show that the natural processes are very complicated and hard to predict. Nevertheless it is a widely studied subject since the majority of the world's population lives and works close to the coastline. Humans have interfered with the natural processes, which has led to a changing behaviour of the coastline. The ability of predicting the behaviour of the processes at the coastline gives the opportunity of limiting our interference with these processes.

1.1. Problem description

A coastline is a dynamic equilibrium between the land and the sea. The coastline is continuously reshaped by dynamic forces such as wind, waves and tides. These forces especially influence sandy coastlines. The coastline continuously reshapes towards a certain equilibrium with the acting forces but it will hardly ever reach equilibrium since the forces are highly dynamic. The forcing processes are usually periodic which confines the shape of the coastline –over a period of years to a few decades - to a certain envelope. These are the type of coastlines we are interested in since minor disturbances, like harbours, could influence the coastline on a wide scale.

The shape of the coastline will change beyond the envelope if the coastal processes are affected by human interference. This could for instance be the construction of harbour breakwaters. A new situation is created and the dynamic equilibrium will shift towards this new situation.

A harbour should, amongst others, protect incoming and berthed vessels from the waves. This is usually done by construction one or two, more or less, shore-normal breakwaters. The breakwaters protect the entering vessels from the waves and prevent the waves from penetration into the harbour, creating safety for berthed vessels. The breakwaters also block the littoral drift which prevents sedimentation of the harbour. Harbour breakwaters thus block the longshore sediment transport. The sediment is trapped at the updrift side of the harbour. Here the sediment settles causing accretion at the updrift side. This leads to sediment shortage and erosion on the downdrift side.

Both effects are undesirable; the accretion could eventually lead to sedimentation of the harbour entrance and the erosion could decrease the protection function of the coastline. In many cases the harbour entrance needs to remain at a required depth so maintenance dredging is required. Dredging the entrance channel is however a difficult and costly operation.

This study focuses on finding conditions at which the breakwaters have the least amount of effect on the sediment transport rates without losing their protective function for vessels. In practice that means that sediment needs to be transported from the updrift side to the downdrift side; this is called bypassing.

1.2. Problem definition

The above identified problem can be summarized as follows:

The navigational requirements do not 'match' with the morphological processes around harbour breakwaters.

In other words; the natural depth in front of the harbour is often not sufficient for the required shipping. It is the objective of this study to find conditions at which the morphological processes oppose less or very little restrictions on the navigational function of the harbour. This depends on the morphological processes and the navigational requirements of the harbour.

1.3. Research objective

The main problem is the mismatch between the morphology and the navigational requirements at the harbour. For that reason this MSc Thesis has the following objective:

Finding conditions at which the bypassing capacity of sediment around a harbour is optimized in combination with limited influence on the navigational function of a harbour.

This means that conditions are determined at which vessels can safely enter and berth in the harbour while at the same time the impact of the breakwaters on the morphological processes around the harbour is limited. The depth in front of the harbour should therefore remain large enough to provide access of the vessels to the harbour. In practice this means that sufficient amount of sediment should bypass the harbour. This follows the philosophy of Integrated Coastal Zone Management (ICZM). The key issue of ICZM is to incorporate the interest of all coastal stakeholders and sustainability in the decision making. This should result in a decision with the minimum amounts of conflicting effects for different stakeholders. The Integrated Coastal Zone Approach is to determine the conditions at which the interests of the harbour manager and the coastal zone manager are both addressed and in a sustainable way.

The research will not be focused on a specific situation but more on general processes involved in sediment bypassing because we are not interested in the bypass capacity of one situation, but the general conditions at which the bypass capacity is maximum.

Research questions

Various topics need to be addressed in order to be able to answer the main research question. Therefore the following research questions are formulized:

1. For which situations is bypassing expected to be optimized? (Paragraph 2.1)

2. What are the navigational requirements? (Paragraph 2.2)
3. Which situations are the most promising in achieving sufficient bypassing and simultaneously match with the navigational requirements, based on a qualitative analysis? (Paragraph 2.3)
4. Which situations actually lead to sufficient bypassing while maintaining the required depth in front of the harbour, based on a quantitative analysis? (Chapter 3-6)

Research method

The first three of the above stated questions have been answered by doing a literature study and a sensitivity analysis with a simple model. This has given the basis on which the most promising situation is determined. This situation is tested with an extensive numerical model. The influence of several parameters is assessed with this model. The research question can be answered with the help of the results of the model calculations. This part of the study focuses on the processes at the updrift side of the harbour and in front of the harbour because this is where the relevant processes occur. Also, the model does calculate the processes at the downdrift side of the harbour well.

1.4. Scope of the study

There is a large variation in coastlines and harbours. This study is about the interface of the coastline and the harbour so the types of harbours and coastlines need to be specified. The harbours and coastlines are categorized in a way which is relevant for this research. The combination of these categories makes up a framework. Each scenario in the framework is called an archetype; it has specific properties concerning sediment transport and harbour geometry. The categorization of the coastline is described in Paragraph 1.4.1 and the categorization of the harbours is described in Paragraph 1.4.2.

1.4.1. Classification coastlines

There are coastlines in all shapes, materials and orientations. The coastlines could differ from each other on many fronts and can thus be categorized in several ways. This is not discussed in this study. The situations which will be studied need to be narrowed down. A few, governing, variations will remain and those will be studied.

This study therefore only considers sandy, nearly straight coastlines which are reasonably to heavily exposed to waves and tide. Only a symmetrical tide is considered, to limit the amount of parameters. The variation between the archetypes can be found in the sediment transport regime, or littoral drift regime at the coastline. The littoral drift regime is a way to express the types of coastline. The transport regime could for instance be a large net sediment transport in one direction, or small net transport due to two almost equal but opposite sediment transports. Transport regime and thus the sediment transport depends on the relative influence of the hydrodynamic forcing (wave and tide).The waves cause a different distribution of the longshore transport over the cross-shore than the tides does. That can be seen in Figure 1-2.

The littoral transport regimes at a specific coastline can, according to Bosboom en Stive (2011), be characterized by the following parameters:

- The annual littoral drift rates in the prevailing and the secondary direction ($100\ m^3/y$ and $90\ m^3/y$);
- The annual net littoral transport rate, defined as the rate in m^3/y with a direction ($10\ m^3/y$ →);
- The annual gross transport rate, without a direction ($190\ m^3/y$).

The numbers between brackets correspond with the illustrative example in Figure 1-1.

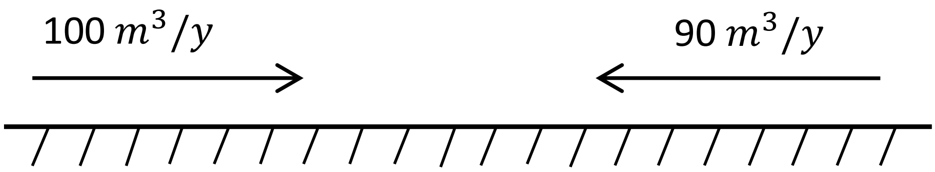


Figure 1-1 Illustrative example of the Littoral drift rates.

The littoral drift conditions are of importance for the required harbour geometry to gain sufficient bypass. Two different cases are distinguished for this research:

1. Large net transport relative to gross transport;
   The conditions could give a large transport in the prevailing direction and a significantly smaller transport in the secondary direction. The result is a large net transport in the prevailing direction which is almost the same size as the gross transport; say 90% of the gross transport, Since a large asymmetrical tide is not considered, this can only be achieved in case of wave dominance, see Figure 1-2 and 1-3

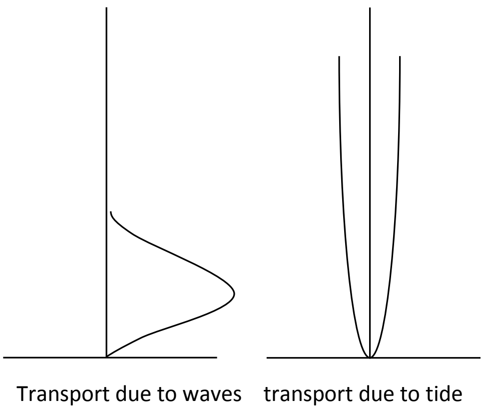


Figure 1-2 Indication of the transport spread out over the cross-shore due to waves (1) and tide (2). The tide is symmetric and of minor influence compared to the waves, The waves come predominantly from one direction. The horizontal line indicates the shoreline and the vertical line is the cross-shore.

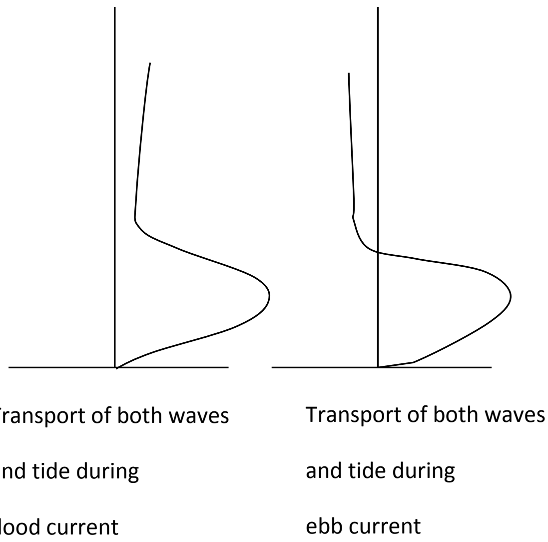


Figure 1-3 Transport in case flood current and waves (1) and ebb current and waves (2). The influence of the waves is dominant over the influence of the tide. This can be seen as a large net transport regime. The horizontal line indicates the shoreline and the vertical line is the cross-shore.

2. Small net transport relative to gross transport;

   The transports in both directions could also be of rather equal size. In that case there is a large gross transport but a much smaller net transport; say in the order of 10 % of the gross transport. This can be caused by a wave-dominant forcing, or a combined wave and tidal forcing, see Figure 1-4 and 1-5. A tide-dominant coastline will not be considered since the tide causes a current which is spread out largely over the cross-shore. The transport in the surf zone will therefore generally be small.

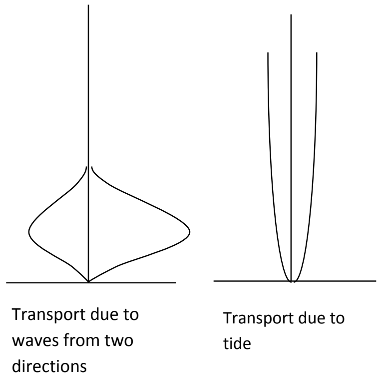


Figure 1-4 The transport due to dominant waves from two directions and a tide.

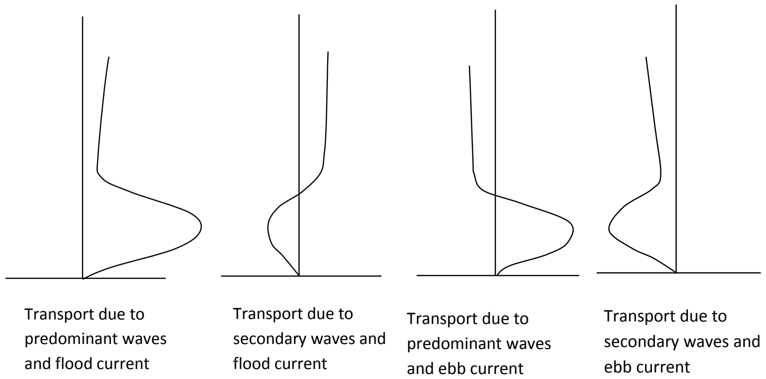


Figure 1-5 The transport in case of prevailing waves plus flood current, secondary waves plus flood current, prevailing waves plus ebb current and secondary waves plus ebb current. The net transport will be small compared to the gross transports.

Whether there is a small or a large net transport is of importance for the erosion/ deposition pattern and plays a large role in the design of breakwaters. This is explained in Chapter 2. The classification with respect to the coastal characteristics is shown in Table 1.1.

Transport regime	Hydrodynamic force dominance
Large net transport	Wave dominated, predominately waves from one direction
Small net transport	Wave dominated, waves from two directions
	Wave and tide, waves from two directions.

Table 1-1 Coastal classification based on the transport regime.

1.4.2. Classification harbours

Just as coastlines, there are harbours in many forms and functions. Not all types can be included in this study. This paragraph will narrow down the types of harbours which are elaborated in this study.

Harbour functions

A harbour has many functions, of which most are not of interest for this study. The most important function for this research is that it should provide a safe and accessible nautical entrance for vessels.

As for a safe entrance, the vessels entering the harbour need to be protected from the waves and currents as they enter the harbour. The same accounts for the vessels which are at berth in the harbour. A natural feature- like a headland or a bay - can provide this safety, but if these are not present, breakwaters need to be constructed to fulfil this function.

It is evident that a harbour requires a nautical entrance. The harbour entrance is however an interruption of the coastline. The waves do not break at the entrance which results in a negative gradient in the longshore transport. This will cause sedimentation of the entrance. The entrance could remain open naturally if there is a river discharge or a tidal current in and out the harbour, but the opening is not always large enough for ships to pass. Secondly it is subjected to natural fluctuations. The process of sedimentation of the entrance can be prevented by blocking the longshore sediment transport with breakwaters. This way there won't be a transport into the entrance.

Summarizing, harbour breakwaters have two primary functions:

- Providing safety for the entering, leaving and berthed vessels;
- Preventing sedimentation of the harbour by coastal sediment.

Initially only semi-streamlined, symmetrical breakwaters are considered. This is a very common shape for the breakwater and the research of Mangor et al (2010) has shown that these give good opportunities for achieving sufficient bypassing. At a later stage the geometry of the breakwaters will be examined. An example of such a harbour can be found in Figure 2-1.

Harbour differentiation

Within the limitations defined above, two different harbours will be considered:

- A large harbour for which the natural depth in front of the breakwaters is insufficient to accommodate the vessels. This harbour requires a dredged entrance channel to provide access to the harbour. These are usually large industrial harbours which require a depth in the order of 15 m or more.

- A small harbour for which the natural depth in front of the breakwaters is sufficient. This harbour does not require a dredged entrance channel to provide access to the harbour. These are usually smaller fishery harbours which require a depth of a few meters.

The matter of having an entrance channel or not, greatly influences the probability of having sufficient bypassing so this will be included in the framework.

1.4.3. Framework

Combining the coastal classification and the navigational classification gives the framework of this study. Six different situations, called archetypes, can be distinguished. This is still too much to consider in this study and this needs to be narrowed down. Each archetype is analysed based on the governing theory, reference situation and a sensitivity analysis. From this analysis the most promising cases are chosen to examine further on. The framework is as follows:

Coastal variation \ Harbour variation		Large harbour Dredged entrance channel	Small harbour No dredged entrance channel
Large net transport	Wave dominated	1.	2.
Small net transport	Wave dominated	3.	4.
	Wave and tide	5.	6.

Table 1-2 Framework which indicates the scope of this study. The expectation, considering the match between morphology and navigation is given per archetype. This is based on theory and reference situations. The numbers correspond to the elaborated archetypes in Chapter 2.

1.5. General limitations of this study

This study has a limited scope. Not all types of coastlines and harbours can be considered in this study. Therefore this study is narrowed down to the following situations:

- Straight or nearly straight coastlines.
- Sandy coastlines.
- Coastlines exposed to waves and a symmetrical tide.
- No tidal basin land inwards of the harbour.
- No river discharge through the harbour.
- No wind or other forcing than waves and tides.
- The harbour entrance needs to be protected by breakwaters, there is no natural shelter.

A numerical model in Delft3D is used to gather the information required to answer the research. This imposes some limitations as well. They are listed below and explained more elaborately in Chapter 6.

- Diffraction is not included in the model.
- The grids is too coarse to include vortices.
- The transition between grids with different cell dimensions imposes boundary effects on the model.
- The model predicts too much refraction.
- Sediment of only one size, instead of a gradation is applied.
- Coriolis is not included in the model.

CHAPTER 2.
PRELIMINARY STUDY

The scope of this study has been explained in Chapter 1. Therefore the coastlines and harbours have been categorized. This creates a framework in which 6 archetypes are distinguished, each with specific bypass characteristics and an expectancy considering optimised bypassing in combination with sufficient serviceability of the harbour. This expectancy is determined, based on a literature study to the relevant transport and bypass processes. From this literature study, a set of governing parameters is extracted. Consequently the influence of these parameters is studied with a sensitivity analysis.

Not only the sediment transport parameters are important but the harbour properties are as well. These are therefore explained in Paragraph 2.2. After the bypass characteristics and the harbour characteristics are known, the bypass expectancy per archetype can be determined. From this follows the most promising cases which can then be studied more thoroughly.

2.1. Bypass requirements

The sediment transport around harbour is of special interest for this study so this is put under a microscope. The requirement for bypassing are described in this paragraph by first looking at the processes and how they change due to the presence of the harbour. Next the influence of individual parameters is assessed with a sensitivity analysis. The requirements for bypassing are important to determine the most promising case of the framework, as defined in paragraph 1.4.

2.1.1. Processes

At first it will be described how the relevant processes act around a harbour. The processes will change due to the presence of the breakwaters. The coastline experiences changes due to variation of the processes. That, in its turn, changes the effect of the processes on the coastline. This paragraph will give a qualitative indication of the changing processes. The explanation accounts for a situation for which the breakwater length is longer than the width of the surf zone.

For this description the coastline is divided in 3 sections:

1. Just updrift of the harbour (within a few hundred meters from the harbour)
2. In front of the harbour
3. Just downdrift of the harbour (within a few hundred meters from the harbour)

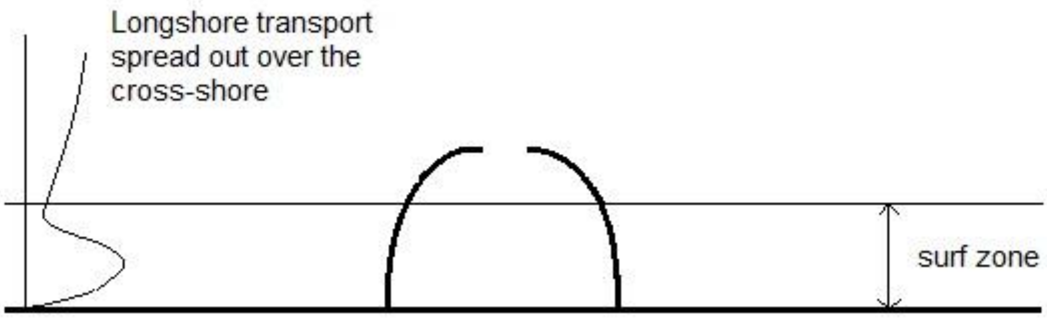


Figure 2-1 Schematization of two harbour breakwaters which extend further than the width of the surf zone. The longshore transport takes place in the surf zone.

Just updrift of the harbour
The longshore transport is assumed to be constant at a straight coastline with parallel depth contours. At the coastline just updrift of the harbour this is no longer the case. There is a constant inflow of sediment from the updrift side but this is all blocked by the breakwater. At the breakwater the transport is initially zero. The gradient in the transport is negative, which corresponds to accretion. The incoming sediment settles in this area leading to changing coastline position. The decrease in the longshore transport can be achieved by decreasing the wave angle of incidence. The coastline thus rotates towards a situation at which it makes an almost zero degree angle with the prevailing wave direction, see Figure 2-2. This effect can be seen in the S/φ −curve. The transport decreases too as the angle of incidence decreases. The shape of the S/φ −curve depends on the coastal profile, the wave conditions and the tide conditions.

The breakwaters will push the current offshore. If this occurs abruptly the current could be pushed outside the surf zone taking sediment with it. In case of smooth breakwaters the current is pushed 'gently' offshore leading to flow contraction in front of the harbour. This increases the sediment transport capacity in front of the breakwaters. The flow contraction decreases as the accretion increases since the accretion causes the entire surf zone to progress offshore. The length of the breakwater relative to the width of surf zone therefore decreases.

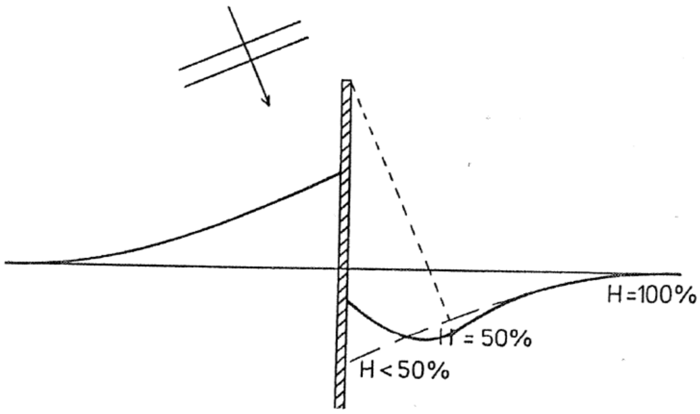


Figure 2-2 In this figure of Bosboom and Stive (2011) the accretion/erosion pattern around a single breakwater is visible.

In front of the harbour

The longshore sediment transport is initially blocked by the breakwaters but the transport will not be completely zero. It takes some time for the sediment to settle once it is in suspension and it takes time for the current to decrease. So although the coastline orientation will become such that the wave breaking doesn't lead to a wave-driven longshore current, there could be a transport of sediment which is already in suspension. This will settle after some time if the current decreases. The length over which this happens is called the adaption length. Due to this phenomenon the sediment could be transported in front of a harbour entrance and, if the current is strong enough, even beyond it.

The transport capacity decreases because the waves don't break in front of the harbour. The negative gradient in the transport corresponds to accretion in front of the harbour. The decrease of the transport capacity due to the lack of wave-breaking induced current could (partly) be counteracted due to the flow contraction. The flow contraction decreases as the accretion updrift increases. A dredged entrance channel could however cause flow expansion which counteracts the flow contraction. The channel will therefore accrete.

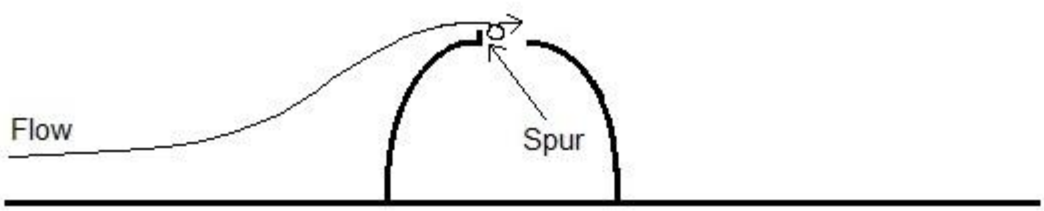


Figure 2-3 Vortices, caused by spurs on the breakwater.

There won't be depth-induced wave breaking in front of the entrance. The flow will follow the shape of the breakwaters. At the entrance, the flow expansion could cause vortices leading to erosion. The vortices could also be caused in another way; by constructing spurs on the breakwater. Spurs are little extending parts on the breakwater, see Figure 2-3. They could cause flow separation and turbulence in the flow (vortices). If the vortices are strong enough, it prevents sedimentation in front of the entrance. If they are even stronger, they could cause erosion in front of the entrance. The downside is that it could make navigation in and out of the harbour very difficult and it could harm the breakwater stability.

A tide gives a longshore current but it could also lead to a current in and out the harbour. If this is strong enough it could transport sediment as well. The magnitude of the current at the harbour entrance depends on the tidal characteristics and the presence of a lake or similar inland of the harbour.

Just downdrift of harbour

At the downdrift side there is a shadow area from the prevailing waves. Outside the shadow area the waves cause the same amount of transport as the updrift side. There is however no supply of sediment so the beach experiences erosion. Diffraction of the waves and alongshore differences in the wave set-up will cause sedimentation against the breakwater.

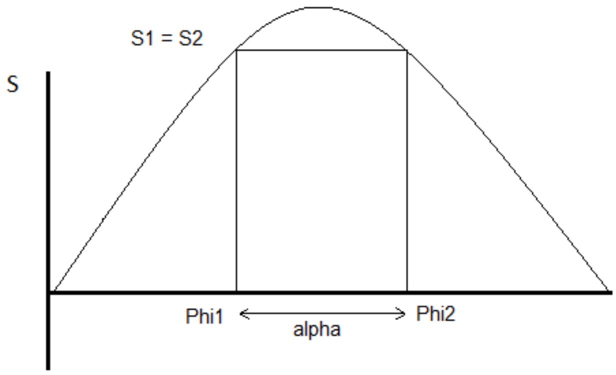


Figure 2-4 S/φ −curve which shows that waves with two different angles could cause the same transport on the same bathymetry.

The flow experiences flow expansion just downdrift the harbour. A shoal could therefore form next to the breakwaters. There is an option to maintain the transport if an underwater bar is formed from the tip of the downdrift breakwater towards the downdrift coastline. That situation could be represented by a headland as can be seen in Figure 2-5. The bar could make just the right angle with the deep water wave angle to maintain the transport. In that case the bypassed sediment is transported towards the shoreline

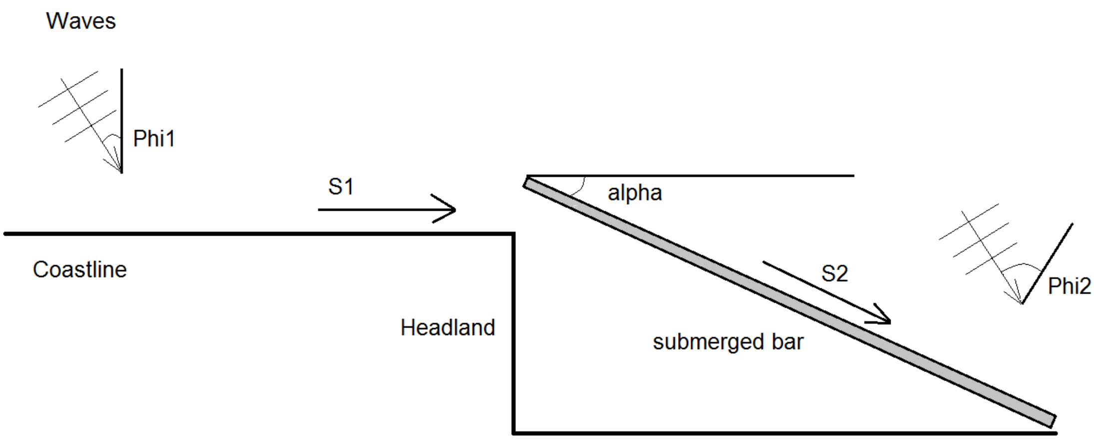


Figure 2-5 Schematization of a headland.

Several tests in Unibest-CL+ have been carried out with the above situation. The length of the headland and the height above the bar are varied. The angle, α, the bar needs to make with an imaginary shore parallel line depends on the deep water wave angle and the bars geometry. The ability to transport sufficient sediment along the bar depends mainly on the bars geometry. A shallower and milder bar gives more transport. The cross-shore location is also of importance for the ability to transport all the incoming amount of sediment along the bar. The amount of transport decreases as the bar is located further offshore but in many cases it is possible to transport the same amount of sediment along the bar as the transport on the undisturbed coastline.

2.1.2. Sensitivity analysis parameters sediment transport

This paragraph will give the results of a sensitivity analysis of several parameters on longshore sediment transport and bypassing in particular. Therefore first the relevant parameters are given, followed by the conclusion of the sensitivity analysis. The analysis itself is described more elaborately in Appendix A.

Relevant parameters

From Bosboom and Stive (2011) follows that the sediment transport along a shoreline depends on the following parameters:

- Wave conditions: wave height, wave period, wave angle;
- Current condition;
- Bathymetry;
- Sediment characteristics;
- Sources and sinks: river discharges, tidal inlets, etc.;
- Breaker index.

For the sediment transport around harbour breakwater, the following parameters are also relevant:

- Breakwater length compared to width surf zone;
- Shape of the breakwaters;
- Flow in and out of the harbour;
- Spurs on the breakwater;
- Dredged entrance channel or not;
- Transport regime: large net or a small net transport.

This indicates that there are a lot of variables which play a role in achieving sufficient amount of bypassing while maintaining a deep enough entrance channel. The influence of a part of these parameters on the sediment transport is determined in a preliminary study with Unibest-CL+. Some of the above parameters can only be assessed with an extensive numerical model. The parameters, of which it is possible to assess the influence, are assessed with the help of the Unibest-CL+ software package. Those are the below parameters. This chapter will give the conclusion per parameter with regards to sediment transport. Table 2-1 shows which settings per parameter are used to test its influence. These tests mainly focus on gaining maximum transport in front of the harbour breakwaters but apply for more general situations as well.

The following physical processes/parameters are not included in the study, partly because they cannot be modelled with Unibest-CL+:

- Flow in and out of the harbour;
- Shape of the breakwater;
- Length of the breakwaters compared to the width of the surf zone;
- The presence of spurs on the breakwater;
- Transport regime;
- The presence of a dredged channel.

The influence of these parameters is estimated, based on the theory of Bosboom and Stive (2011).

Parameter	Setting
Wave height	General setting: $2\ m$ Variation: $1\ -\ 3\ m$
Wave angle	General setting: 45^o Variation: $30^o\ -\ 60^o$
Wave period	General setting: $7\ sec$ Variation: $4\ -\ 10\ sec$
Bed slope	Steep: $1{:}\,25$ Mild: $1{:}\,100$ Extra mild: $1{:}\,200$ Equilibrium profile according to Dean Flat
Depth in front of breakwater	General setting: $5\ m$ Variation: $0\ m\ -\ 10\ m$
Breakwater	$500\ m$ long $3\ m$ above water level
Breaker bar	Location: in the middle of the slope Depth above breaker bar: $3\ m - 14\ m$
Breaker index	General setting 0.8 Variation $0.5\ -\ 1.0$
Tidal currents	General setting: no tidal currents Variation: $0\ m/s\ -\ 1.0\ m/s$, amplitude $1\ m$.
Sediment size	General setting: $200\ \mu m$ Variation: $100\ \mu m - 300\ \mu m$ Cohesive material will not be considered
Sediment transport formula	Bijker (1967, 1971) (The formula is not subject of this study)

Table 2-1 Tested parameters and their variation.

Conclusions
The conclusion of the sensitivity analysis is divided in those which follow from the sensitivity analysis and those are not assessed with sensitivity analysis.

The following can be concluded from the sensitivity analysis:

- The wave height especially has a large influence on the amount of longshore transport. They break further offshore and cause more transport, especially on steeper slopes.

- The wave period has minor influence of the longshore sediment transport. Waves with a longer period cause little more transport.

- Waves with an angle of 45^o give the most amount of transport at an undisturbed coastline but the maximum is achieved at smaller wave angles in front of the harbour because the refraction process is disturbed.

- The bed profile hardly influences the amount of longshore sediment transport.

- The depth in front of the harbour is of mayor influence on the amount of longshore sediment transport. A smaller depth can increase the amount of transport significantly. The same effect can be achieved with an offshore breaker bar.

- The tidal currents enlarge the sediment transport significantly. An asymmetric tide causes the S/φ-curve to change shape drastically.

- Smaller grain size gives more transport[2]. Smaller sediment grains also stay longer in suspension. Therefore it smaller sediment grains are more likely to bypass the harbour.

The influence of the parameters which are not assessed with Unibest-CL+, is determined based on theory. Their expected influence is as follows:

- The harbour mouth could be kept open if there is a large in and outflow due to the presence of a large area behind the harbour.

- A streamlined shape is expected to be in favour of achieving flow contraction. A non-streamlined breakwater is expected to push the current further offshore (outbreaking)

- Breakwater with a cross-shore extent smaller than the width of surf zone will show bypassing for sure since the sediment transport is spread out over the width of the surf zone. They will therefore have less influence on the longshore transport than breakwaters which extend further than the width of the surf zone. They will initially block all the sediment.

- Breakwater with a cross-shore extend longer than the width of the surf zone will give more flow contraction of the tidal current. The wave-induced part will however be larger in case of breakwaters shorter than the width of the surf zone. This has a large influence on the morphological processes and it is therefore worthwhile to study several lengths of the breakwater.

- Spurs could enhance the turbulence in front of the harbour entrance. This could compensate for the lack of wave-induced turbulence.

- In case of a large net transport, the coastline will show extensive accretion at the updrift side and erosion at the downdrift side. In case of a small net transport, the erosion and accretion will be less.

- The presence of a dredged channel will have mayor influence on achieving the objective. In a dredged entrance channel the flow experiences flow expansion, which causes sedimentation[3].

Primarily the effects of the lack of wave driving force in front of the harbour entrance needs to be compensated. Wave breaking causes turbulence and a longshore transport. Turbulence leads to initiation of motion and it prevents settling of suspended sediment. Initiation of motion is not required in front of the harbour. The settlement of sediment can also be prevented by a strong longshore current.

Considering the transport regime, it is best to have sediment transport in both directions, e.g. a small net transport. This will give accretion against the breakwater. The accreted bathymetry at for instance the downdrift side could make just the right angle with the prevailing waves to give

[2] If the bed slope is constant and only the d_{50} varies.
[3] The amount of sedimentation in the dredged channel also depends on the d_{50} and the width of the channel. Sediment with smaller grain sizes stays better in suspension and is thus more likely to be transported past the harbour. The probability of bypassing also depends on the width of the channel.

the same amount of transport. A small net transport can be caused at a wave dominated coastline or at coastline which is influenced by waves and tide.

Based on these expected effects a choice will be made to elaborate certain archetypes of the framework.

2.2. Navigational requirements

The prime objective of this study is to find conditions for which the sediment is bypassed as much as possible and the navigation towards and from the harbour is as undisturbed as possible. This report will greatly deal with the processes considering sedimentation but the aspects of navigation are just as important in achieving the objective. Therefore this chapter will give some insight in the requirements for navigation.

2.2.1. Classification harbours

There are various navigational requirements for a harbour. They mainly depend on the local circumstances and types of vessels which are expected to call at the harbour. They involve amongst others the length, width, depth and orientation of the approach channel. This data is not available since no specific location is considered in this study. Additionally these properties are not really required for the scope of this study. This study focuses on bypassing and keeping a harbour accessible for vessels. The bypassing process is influenced by the length and the shape harbour breakwaters and the bathymetry in front of the harbour. The depth of the approach channel also determines whether the harbour remains accessible. These three properties are therefore the most important properties for this study.

These properties could be different per type of harbour and therefore a relevant classification has been made for the harbours. The following differentiation has been chosen to apply in the framework:

- **Small harbour**
  In this study a small harbour is considered to be a harbour which doesn't require a dredged entrance channel for servicing the vessels. That means that only small vessels are able to call at the harbour. This could for instance be the case at a fishery or small craft harbour. The required depth in front of the harbour is only a few meters. The breakwater length is therefore relatively short with a maximum of a few hundred meters. Therefore not all the sediment transport is expected to be blocked. The shape could differ; the most optimal shape is to be determined with the model.

- **Large harbour**
  In this study a large harbour is considered to be a harbour which requires a dredged entrance channel for servicing the vessels. That can only be economical profitable in case of an industrial harbour. The channel is dredged to let large vessels be able to call at the harbour. The depth of the entrance channel is typically in the order of 10 to 15 meters, but a depth in the entrance channel of 20 meters or more is no exception.
  The breakwaters are usually long because the large vessels need quite a large inner area of the harbour at which they could slow down. Another reason for long breakwaters is to

avoid sedimentation of the entrance channel. Therefore, initially 100 % blocking of the sediment transport is expected.

It is specifically chosen not to apply specific values to these properties because it wouldn't make any sense. The values would depend on a chosen design vessel which could differ very much. A worked example including values can be found in Appendix B to get insight in what the properties become when choosing a certain design vessel. This has been worked out with the PIANC method for a small harbour and a large harbour.

2.2.2. General considerations harbour geometry

There are some general considerations about the orientation and the geometry of the breakwaters. These are explained in this paragraph. The orientation of the breakwater should also be such that waves do not penetrate into the harbour. A conflicting consideration is that the waves should preferably come from the aft of the vessel. Avoiding wave penetration is however more important. Therefore, in practice this means that the angle makes a large angle with the prevailing wave direction; see the right picture in Figure 2-6.

The breakwaters should not form a narrow "sleeve" but they should provide an open space immediately behind the entrance. The ships manoeuvring in the channel do not like hard structures close to the channel boundaries. Another reason is that ships need lateral space when they go from an area with a cross-current to an area without a cross-current. The ship's bow is directed slightly against the current. This way the vessel sails forward. If the bow of the vessel passes the breakwaters, the stern will be pushed in the direction of the current. This brings the vessel off its track and it needs some space behind the breakwater to get back on track. As a final reason the waves diffract in the open space behind the breakwaters, reducing the effects of wave penetration. See Figure 2-6 for a 'good' and a 'bad' design.

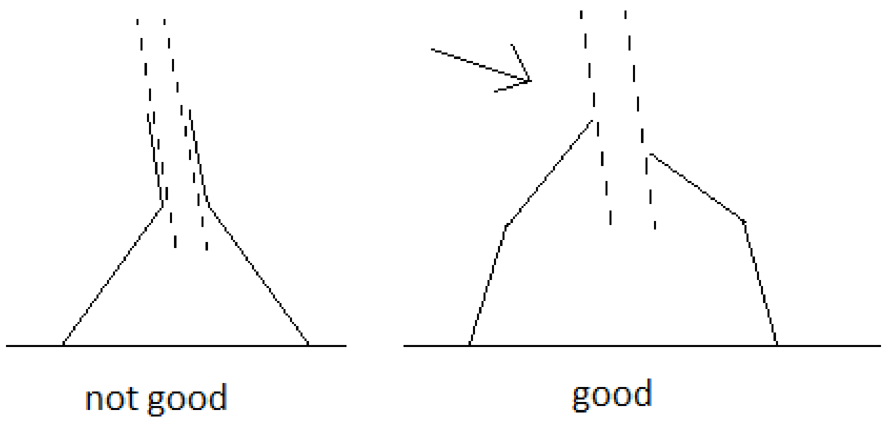


Figure 2-6 A 'good' and a 'bad' design considering the geometry of the breakwaters.

2.3. Conclusion literature study

The scope of this study, the framework, is presented in Chapter 1. The requirements considering morphology are elaborated in Paragraph 2.1 and the requirements for navigation are elaborated in Paragraph 2.2 so the framework can be filled in. For each situation the expectancy of achieving optimal bypassing and maintaining sufficient depth is analysed. This summarised in the framework which can be found in Appendix C. This chapter will give an explanation of the framework and the conclusion, drawn from the framework.

2.3.1. Explanation framework

The framework has a vertical axis and a horizontal axis. The vertical axis shows a differentiation in the transport regime, respectively a large and a small net transport. The forcing due to which these situations occur is a second differentiation.

The horizontal axis gives the two types of harbour situations, a large harbour with a dredged channel and a smaller harbour without a dredged channel. These two axes give a framework consisting of 6 archetypes. Each combination of a transport regime with a harbour type is called an archetype.

Harbour variation / Transport regime		Dredged entrance channel	No dredged entrance channel
Large net transport	Wave dominated	1.	2.
Small net transport	Wave dominated	3.	4.
	Wave and tide	5.	6.

Table 2-2 Framework of this study. The expectation, considering the match between morphology and navigation is given per archetype. This is based on theory and reference situations.

Transport regime

A large net transport means that the transport due to the waves from the prevailing direction, denoted as S_{1^e}, is much larger than the transport due to the secondary waves, denoted as S_{2^e}. A large net transport can only occur if the tidal influence is limited. A tide after all creates an almost equal current in both directions, decreasing the ratio between the two transports. Therefore a large net transport can only occur in case of wave-dominated climate. It should be noted that an asymmetrical tide could also cause a large net transport, but his will not be considered.

A small net transport means that S_{1^e} and S_{2^e} are both large compared to each other. It is likely that S_{1^e} is not equal to S_{2^e}, which gives a difference in accretion/erosion at both sides of the breakwaters. In this case the transports could be causes by wave-dominated forcing, tide-dominated forcing and a combined wave and tidal forcing. In a wave-dominated climate the influence of the tide is very little and the same accounts for the influence of the waves on a tide-dominated coastline.

Classification harbours

As can be read in the previous chapter, the harbours are differentiated in a harbour with and without a dredged channel. The first one is a large industrial harbour and the latter one is a small fishery type of harbour.

The large harbour requires an entrance channel to accommodate the large vessels. The harbour breakwaters are usually very long to allow large vessels a safe area for slowing down and to prevent sedimentation of the entrance channel. The breakwaters will extend further than the width of the surf zone and will therefore initially block all the incoming sediment.

The smaller harbours require much less depth at the entrance, up to a few meters. This depth is reached within a few hundred meters. The breakwaters can be constructed up to this depth. The width of the surf zone is of similar size or smaller so not all the sediment transport is blocked.

2.3.2. Most promising case

This paragraph will explain which archetypes are chosen as most promising. These situations are modelled with a Delft3D model. The choice is made based on the predicted capability of optimized bypassing and minimized depth decrease in front of the harbour. This is analysed for each archetype. The complete framework can be found in Appendix C, below the most promising cases are described.

Archetype 2: large net transport, wave dominated without a dredged channel

The most promising and relevant case is when the breakwater length is equal or shorter that the width of the surf zone, there is no dredged channel and the coastline is influenced by waves and tide but with a dominant wave forcing. The dominant wave influence causes a large net transport. The large net transport causes accretion at the updrift side of the harbour and erosion at the downdrift side of the harbour. The accretion will eventually lead to a decrease of the depth in and/or in front of the harbour. The downdrift erosion harms the protective function of the coastline. Especially the downdrift erosion makes this a relevant case from the coastal manager point of view.

This archetype is more promising than the others due to the absence of a dredged entrance channel. The entrance channel is expected to have a negative influence on the bypassing process because it causes a vertical flow expansion of the current. Therefore the current velocity decreases which causes a decrease of the transport capacity and thus the bypass capacity.

The breakwaters will cause a flow contraction in front of the breakwaters and this could be strong enough to transport the sediment past the harbour. The updrift side will accrete, decreasing the depth in front of the entrance and decreasing the flow contraction. The wave action could become strong enough to stop the depth decrease. Spurs on the breakwater end could induce sufficient amount of turbulence to counteract sedimentation in front of the harbour.

The sediment which is transported to the downdrift side could form a bar along which the sediment is transported onshore. Related to archetype 6, the large net transport regime will create less accretion at the downdrift side of the harbour and therefore the difference in the system will be seen at the downdrift side. Only diffraction and secondary current could contribute to the downdrift accretion. Therefore the shape of coastline downdrift will more or less be equal but the coastline at archetype 6 but will be more land inwards. Therefore the bar, required to transport the sediment onshore again, needs to be longer. This is expected to decreases the capacity to transport the sediment onshore.

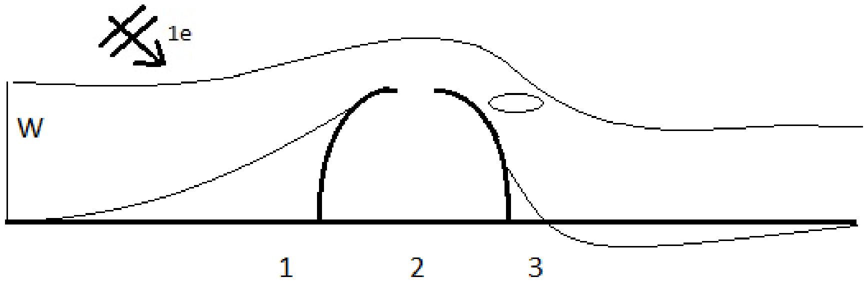


Figure 2-7 Archetype 2, the situation after some time. (no dredged channel, large net transport due to a wave-dominant forcing).

Archetype 6: small net transport, wave and tidal influence without a dredged channel
Also promising but less relevant is the archetype with the same harbour but at a coastline which experiences a small net transport due to a large wave and tidal influence. It is less relevant from the coastal manager point of view because there is less erosion of the coastline. It is however also promising because the sediment transport in both directions is almost equal in magnitude. This leads to accretion against the harbour at both sides of the harbour. This accretion pattern could lead to the right orientation of the coastline due to which the waves have just the right angle of incidence to maintain the transport. In short, the transport at location 1 will decrease due to the accreted coastline, the transport at 2 could possibly be maintained due to flow contraction and turbulence and the transport at 3 could possibly be maintained by a bathymetry which makes the right angle with the prevailing waves. In that case $S_1 \approx S_2 \approx S_3$.

The harbour of Hantsholm in the North of Denmark is good example of a successful bypassing harbour. There is a gross transport of around 1.5 Mm^3/Y and a relatively small net transport of 0.4 Mm^3/y. The coastline is very exposed to very obliquely incoming waves and there is a strong current due its location. The symmetrical and semi-streamlined geometry creates a smooth convergence of the flow and has in combination with the vertical faces of the breakwater resulted in optimal bypass conditions and acceptable sedimentation rates.

See Appendix D.1 for a more elaborated explanation.

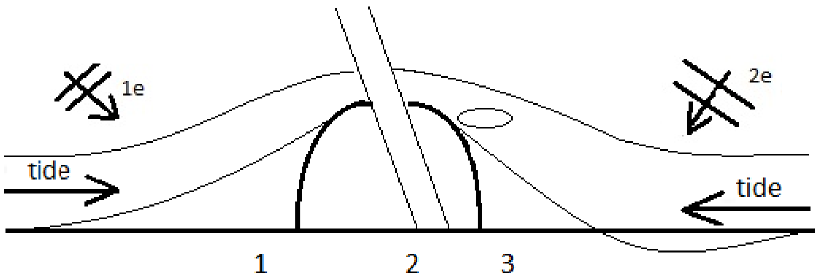


Figure 2-8 Archetype 6, the situation after some time. (no dredged channel, small net transport due to wave- and tidal forcing).

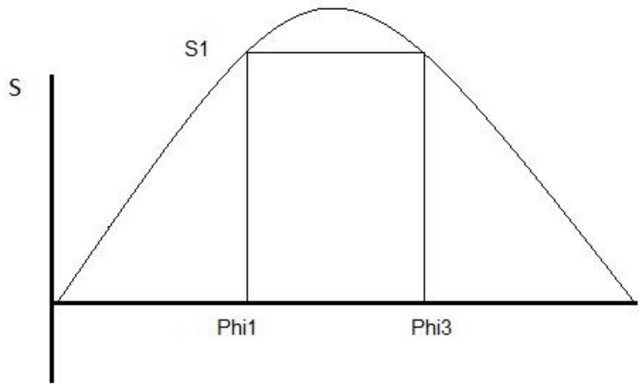


Figure 2-9 A possible S/φ − curve in case of archetype 6 (no dredged channel small net transport due to wave- and tidal forcing). φ_1 represents the angle at the updrift side, just before the entrance and φ_3 the corresponding angle to give the same transport.

Both archetype 2 and archetype 6 are expected to be promising. Archetype 2 includes a large net transport and archetype 6 a small net transport. The processes at the updrift side and in front of the harbour are more or less equal but the transport capacity at the downdrift side is expected to be higher in case of archetype 6. It is however chosen to continue this study with archetype 2 because this is far more relevant from a coastal manager point of view. In contrast to archetype 6, there will be significant erosion in case of archetype 2. This makes a solution much more needed for an archetype 2 situation. The processes at the downdrift side are also not very relevant for the bypassing process.

2.3.3. Not chosen archetypes.

The other archetypes are not elaborated because they are less promising for different reasons. The reason why the other archetypes are not chosen is given here.

Archetype 1: Large net transport with a dredged channel
Archetype 1 is not a promising case relative to the others. Initially all the sediment is blocked by the breakwater since it extents further than the width of the surf zone. After a while there could be a transport around the tip of the breakwater, but this will settle in the channel.

Archetype 3 and 4: small net transport, wave dominated with and without a dredged channel
Archetype 3 and 4 will not be studied in extensive detail since there are more promising cases. Archetype 3 and 4 describe a situation with a small net transport on a wave dominated coastline. For both cases it counts that tide increases the transport capacity in front of the harbour which therefore presents a more promising situation. This is described in archetype 5 and 6 so these are per definition more promising that 3 and 4. In case archetype 6 gives promising results, it is worthwhile to see if these can also be found for archetype 4 and subsequently 3.

Archetype 5 small net transport, wave and tide with a dredged channel
This is most definitely an interesting case but hardly any promising. The same processes as archetype 6 play a role, except for the fact that the all the sediment is initially blocked and if, after some time, a transport in front of the harbour exists, this will settle in the entrance channel due to flow expansion. Archetype 6 is therefore much more promising. If this one however shows promising results, it could be interesting to see if these can also be achieved after some time for archetype 5.

The harbour of IJmuiden shows similar features as this situation describes. The harbour extents for beyond the width of the surf zone and combined with the streamlined geometry, this gives flow convergence. In combination with rather large cross-shore velocities due to a water outlet and water from the sluice this leads to erosion in front of the harbour. This keeps the entrance open and due its extend the entire channel. We could however not speak of bypassing because the length of the breakwater prevents any sediment from being bypassed. This case however shows that flow convergence and large cross-shore velocities could prevent sedimentation in front of the harbour.

See Appendix D.2 for a more elaborated explanation.

CHAPTER 3.
MODELLING

The previous chapter concluded in two cases which have the largest expectations considering maximum bypassing and maintaining sufficient depth in front of the harbour. These expectations are based on a literature study and a sensitivity analysis. A small harbour without a dredged channel at a coastline which experiences a large net transport due to a wave dominance is considered to be the most promising case. A model is set up to perform different calculations and assess whether the conclusion is just. Within this case there is however plenty of room for variation, for instance in the wave climate and the breakwater geometry Therefore multiple calculations are performed with the model and in each calculation, one the relevant parameters is varied. The results of these calculations are compared to each other. This will ultimately give the influence of each varied parameter. It is than possible to determine for which harbour geometry and type of hydrodynamic forcing optimal bypassing can be expected in combination with maintaining sufficient depth in front of the harbour. The optimisation method is explained in more detail in Chapter 4.

Before we can come to that, the model needs to be described and we need to be sure that the model calculates the right type of hydrodynamic and morphodynamic behaviour. That is treated in this chapter. At first the software package Delft3D is described. This is followed by a description of the model setup in Paragraph 3.2. The model is calibrated in both a qualitative and a quantitative way. The qualitative calibration is explained by means of the output in Paragraph 3.3. The quantitative calibration is explained in Paragraph 3.4

3.1. Software description

Delft3D is a numerical model which can be used to model hydrodynamic and morphodynamic processes in a certain area. It does so by solving the existing mass balance and momentum balance equation in the modelled area. Therefore the modelled area is divided into small cells. The smaller the cells are, the higher the accuracy is, but also the more computational 'expensive' it is. This paragraph gives a brief explanation of how Delft3D works.

The mass balance prescribes the conservation of mass in all directions with the mass per unit volume (density, ρ $[kg/m^3]$). There is an inflow, outflow and production in the cube. These can be added up and the limit can be taken of the sides of the cube and the time (dx, dy, dz, dt). Because of conservation of mass the sum is zero. For the mass balance in a 3D environment, that looks as follows.

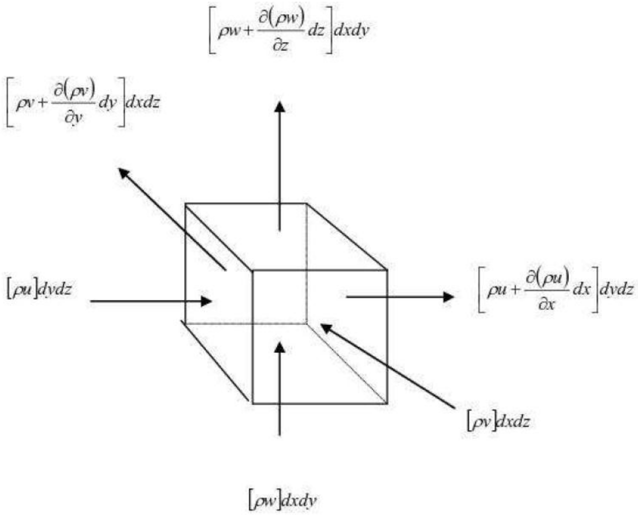


Figure 3-1 3D mass balance. Extracted from Delft3D FLOW Manual.

The momentum balance equation describes the conservation of momentum in all directions with the help of the momentum per volume unit. This is the mass times acceleration in all directions. The rate of increase of momentum in a fluid element is equal to the sum of all forces on the volume. The continuity equations and the momentum balance equations combined are known as the Navier-Stokes equations. These are rewritten to include the turbulent stresses. This results in the Reynolds averaged equations.

The transport of dissolved matter is modelled with an advection diffusion equation in three directions. Source and sink terms are included to simulate discharges and withdrawals. Data on flow, viscosity and diffusivity are used to calculate the mixing of heat and dissolved substances with the transport equations after which it is, with the help of the equations of state, incorporated to the density of the fluid. This is consequently input for the shallow water equations and the turbulence model. A turbulence model is applied to 'close' the set of equations. The equations in all directions are solved in each cell on each time step.

The equations are solved on a grid. Therefore the model area is divided in cells. The shape of the cells could be chosen in different ways, depending on the application. Grids in Delft3D are staggered grids, which means that different properties are determined at different locations in the cell. The water level can either be defined at the cell centre or at the corners of the cell. The flow velocity is always defined at the border with the adjacent cell and perpendicular to the border itself. The equations are still continuous (infinite number space) which makes it impossible for a computer to solve. A computer works in the finite number space (ones and zero) and therefore the equations must be discretized. This done in a combined implicit and explicit method. Details can be found in Deltares (2012), "Delft3D-Flow User manual".

Delft3D can be applied to model the hydrodynamic forces, as discussed in this report and the morphodynamic response to that forcing. The setup for the model is discussed in Paragraph 3.2 and the output of the model as a function of the input, is discussed in Paragraph 3.3.

3.2. Model setup

This paragraph will give the setup of the applied model. The most important settings are explained. At first the domain and grid is explained. This is followed by the time frame and the boundary conditions. Finally there are some physical parameters to be described.

3.2.1. Domain and grid

The modelled area must cover the area of interest and a wide area around it such that boundary effects could not, or only very limited, disturb the processes in the area of interest. Therefore the modelled area is 15 km in longshore direction and 8 km in cross-shore direction. This paragraph gives an explanation of the applied bathymetry and the grids.

There is no bathymetry data available since no specific site is considered. The best option is therefore to model an equilibrium profile according to Dean. This is not valid for the entire profile so from a depth of 20 m onwards the bed profile is modelled with a slope of 1: 200. This causes a bump in the bed level at around 4 km offshore but its effect is negligible since this is well outside the area of interest. It would have been better to make the transition at the point where the slope of equilibrium profile is 1: 200. Eventually two meter is added to the bed level to create a beach of 125 m wide and at maximum 1.25 m high. A cross-section of the coastal profile is given in the underneath figure.

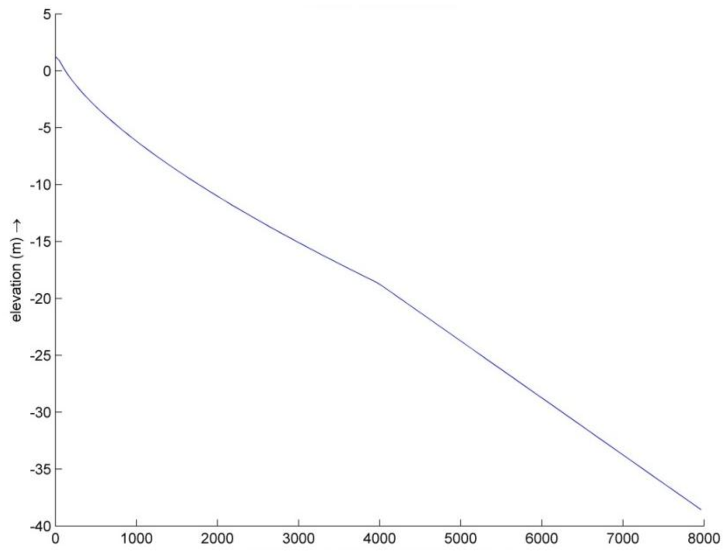


Figure 3-2 Cross-section of the initial bathymetry.

As said, the calculations are performed on a grid. The results are more accurate in case the grid is finer. The downside is that the calculations on a finer grid are computational more expensive. A compromise can be found in applying a finer grid in the area of interest and a coarser grid in the less interesting areas. This is achieved in two ways;

1. By applying a curvi-linear grid which gets coarser towards the boundaries.
2. By applying a refined, nested grid in the FLOW module.

The grid cells in the main grid are $50\,m\,x\,50\,m$ in the centre at the coastline. This becomes 3 times larger in both directions at the edges of the domain. In the area of interest the main grid is replaced by a finer grid which is nested in the main grid. The grid cells of the finer grid are 3 times smaller in both directions related to the main grid. The grids can be seen in Figure 3-3.

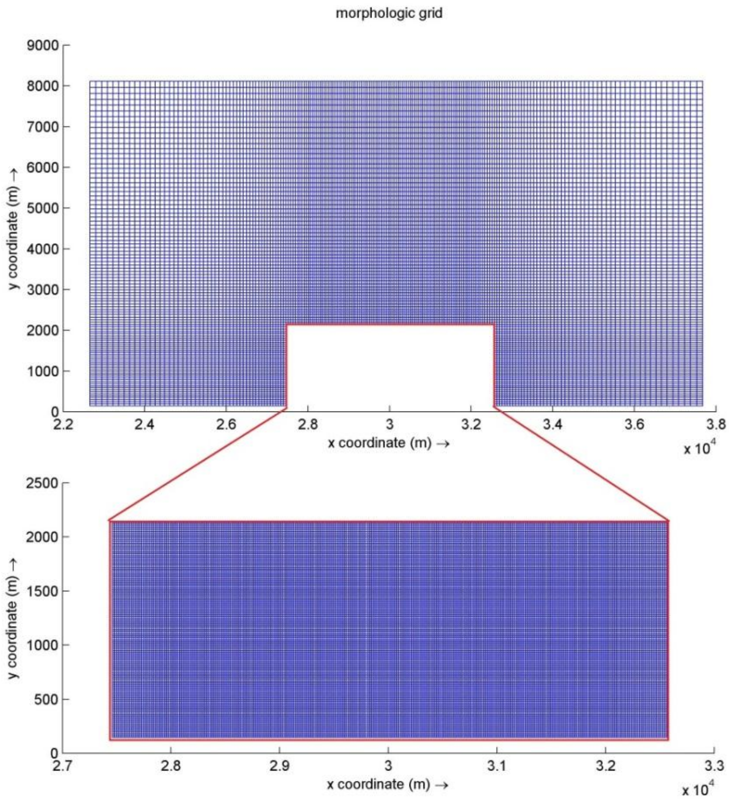


Figure 3-3 Grids of the FLOW module. A section is cut out of the main grid and replaced by a finer grid. The remainder of the main grid is called exterior grid and refined grid is called the interior grid. The original main grid and the finer grid are also applied in the WAVE module.

The breakwaters are modelled as thin dams in the FLOW module and as obstacles in the WAVE module. For the obstacles in the WAVE module, the transmission and the reflection coefficient can be set. Both are zero. An example of the shape of the breakwater is given in Figure 4-1.

3.2.2. Time frame

Since the tide is assumed to play a large role in the bypassing process, at least a spring-neap tidal cycle of 28 $days$ needs to be modelled. The sediment transport is not modelled from the start but 12 hours later in order to give the model some time to decrease the spin-up errors. Therefore the total modelled time is 30 $days$. The time step of the calculation determines the stability of model in terms of the CFL condition. The CFL condition ties the grid size, the depth and the time step to each other. The relation is given as follows:

$$CFL = \frac{\Delta t\sqrt{gH}}{(\Delta x, \Delta y)} \qquad 3.1$$

The model becomes unstable if this CFL value exceeds a value of approximately 10. This relation states that a smaller grid size also requires a smaller time-step to remain within the boundaries of stability. The most refined grid is governing for this and this requires a time step of 6 seconds.

3.2.3. Boundary conditions

The hydrodynamic forcing needs to be set at the boundaries of the exterior grid. This can be done with the boundary conditions. There are three open boundaries which need to be specified; two cross-shore boundaries and a sea boundary. The cross-shore boundaries are Neumann boundaries and the sea boundary is specified as a water level boundary. The applied settings for these boundaries are explained in this paragraph.

The cross-shore extent of this model is limited so Neumann boundaries can be applied at the right and left boundary. A water level boundary is applied at the offshore boundary. All three are of the harmonic forcing type to represent the varying entity. The model is located at the equator, so Coriolis doesn't play a role. The water level will therefore elevate as a flat surface with a phase difference between the left and the right side. The tide should actually be modelled as a Kelvin wave, but due to the limited extend of the model in cross-shore direction this can be ignored.

The boundaries conditions are set to represent a tide with $M2$ and $S2$ constituents. This combination gives a spring-neap tidal cycle. The other constituents are of minor influence on this cycle. The equation of the constituents is as follows:

$$\eta = \sum \hat{\eta}_i \cos(\omega_i t - k_i x) \tag{3.2}$$

The transport boundary conditions are the same types as the hydrodynamic boundary conditions. Additionally the equilibrium sand concentration is modelled at the inflow boundaries.

The wave computations required 3 different grids. The output is required in the model area so the waves are computed on the main grid and the interior grid. Additionally a much larger grid is used. This grid is 4 times larger in longshore direction and 2 times larger in cross-shore direction. That is achieved by applying the same grid as the main grid in Figure 3-3 but with larger grid cells. The wave boundaries are set in the largest grid which is therefore called the input grid, see Figure 3-4. The main advantage is that the waves propagate through the input grid and adapt their shape to the local circumstances. Continuously they propagate into the area of interest and affect the hydrodynamic and morphodynamic processes in the domain. The wave conditions are therefore set at deep water and are adapted to the local conditions once they are in the area of interest. The waves are modelled with a SWAN model.

The boundary conditions require settings for the wave height, period, direction and duration. These differ per scenario. The base-scenario, which is analysed to check if the model calculates the correct behaviour, is modelled with a wave height of 1 m, wave period of 7 $seconds$ and a wave angle of 45^o during the entire simulation time.

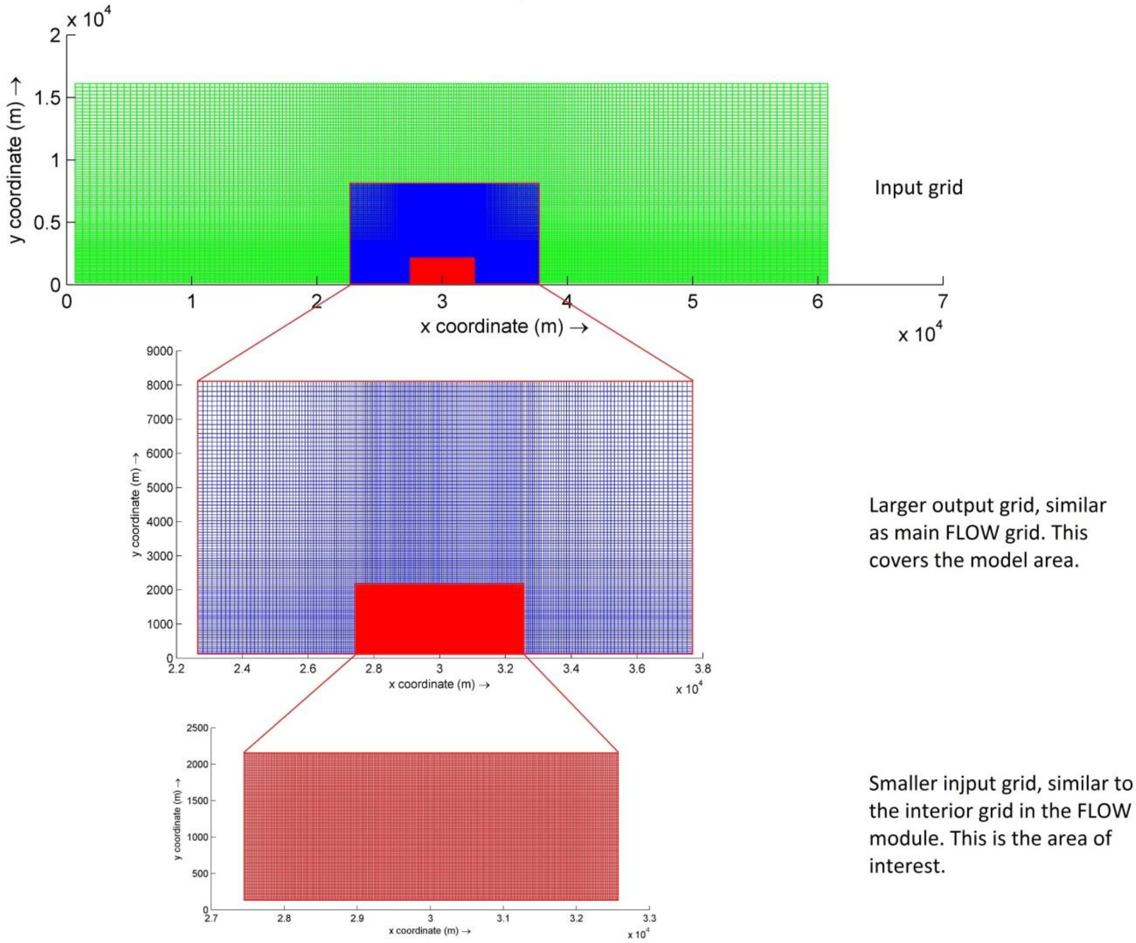


Figure 3-4 Setup of the wave grids.

3.2.4. Physical parameters

The important physical parameters or the parameters which deviate from the default values are explained below. At first the roughness is explained, followed by the viscosity and the diffusivity. Finally the morphological settings are elaborated.

Roughness

The roughness is specified with the Chézy formula. The Chézy value is 65 $m^{\frac{1}{2}}/s$ in both directions. The bottom stress due to wave forces can be computed with several formulae. Since the Van Rijn 1993 formula is used for the sediment transport it advised to apply the Van Rijn 2004 formula for the wave-related bottom stress. This is formulated to work together with the sediment transport formula. A JONSWAP model with default values for the coefficients is applied to model the wave-induced bottom friction.

Viscosity and diffusivity

The value for the eddy viscosity is set to $1\,m^2/s$ in both grids. The background horizontal diffusivity is not modelled constant in both grids. In the exterior grid it is computed constant with a value of $1\ m^2/s$. In the interior grid it has the same value for the interior grid point but a different value at the edges with the exterior grid. That is applied to limit the boundary effects.

Strong boundary effects occur at the transition from the exterior grid to the interior grid. They are especially present in the upper left and right corner of the interior grid. The reason for these boundary effects is unknown and no method has been found to delete the effects. For that reason a method is applied to diminish the effects. The disturbances due to the boundary effects are therefore smeared out by increasing the diffusivity at the outer two grid cells of the interior grid. The diffusivity in the outer column is therefore set to $100\ m^2/s$ and $50\ m^2/s$ in the second column. This is a little trick to avoid larger disturbance in the model

Morphology
The sediment in the domain is sand with a D_{50} of 200 μm and a specific density of 2650 kg/m^3. The transport formula of Van Rijn (1993) is applied for calculating the sediment transport. Van Rijn (1993) distinguishes sediment transport below a reference height which is treated as bed-load transport and that above the reference height which is treated as suspended-load. Sediment is entrained in the water column by imposing a reference concentration at the reference height.

Van Rijn is known for over-estimating the influence of the wave asymmetry. This results in an onshore transport and consequently the formation of dunes on the beach. This can be diminished by adjusting the calibration factors. The longshore transport is therefore scaled up with a factor 5 and the cross-shore processes are scaled down with a factor 10.

The morphological response takes place on a time scale which is longer than the hydrodynamic forcing. The morphological response to the hydrodynamic forcing is therefore scaled up to get significant results. It is scaled up with factor 24. Only the bed level changes are scaled up this factor, not the sediment transport. This leads to two time scales in the result. A hydrodynamic time scale which spans from 0 to 30 $days$ and morphological time-scale which spans from 0 to 2 $years$.

3.3. Qualitative calibration

The third part of this chapter contains a description of the model output and the calibration process. The calibration is mainly done on a qualitative basis and only very roughly on a quantitative basis. That means that the processes are checked and compared to the theory. A quantitative calibration has proved to be difficult to perform since no specific location is considered. It has been chosen to roughly compare the modelled sediment transport with that along the Dutch coastline.

The model is validated with the results of a base-scenario. The base-scenario has the following settings:

- Significant wave height: 1 m during entire simulation;
- Wave period: 7 s;
- Deep water wave angle of incidence: 45^o;
- Length breakwater: 500 m.

At first the output of the hydrodynamic processes is elaborated, secondly the output of the morphological response is discussed. This part in the main report only gives a selected overview of the most important results.

3.3.1. Hydrodynamic processes

The input for the boundary conditions as explained in the previous paragraph leads to a certain output. The input boundary conditions result in a certain water level variation, wave computation and currents in the domain. The output for these parameters is explained in this section.

The input for the model is a tide with M2 and S2 constituents. That can also be seen in the model results, see Figure 3-5. The plot clearly shows the expected spring –neap tidal cycle. The depth averaged velocities show similar behaviour. The velocity increases during spring tide since the water level variation between low water and high water is higher and this needs to be covered in the same amount of time as during neap tide. The cross-shore velocity is approximately one-tenth of the longshore velocity.

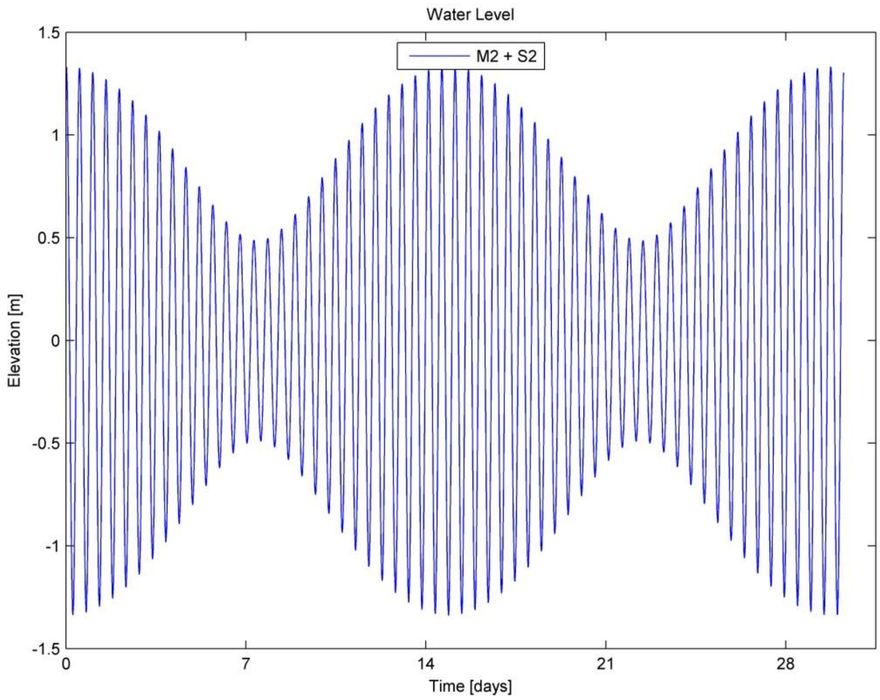


Figure 3-5 Water level variation at 1000 m offshore. Depth is 6 m.

Waves

The waves propagate into the domain and break in the breaker zone. The wave height is expected to decrease a little up to the surf zone due to little friction with the bed. In the surf zone it is expected to increase due to shoaling and eventually break. This is modelled correctly, as can be seen in Figure 3-6. The wave angle should decrease due to refraction. The wave angle is however already decreased from 45^o to 25^o when the waves enter the interior domain. The depth at the seawards boundary of the interior domain is approximately 11 m. This is too large to causes such an amount of refraction. The output has been tested with the governing equations and it is found that Delft3D overestimates the refraction. This influences the absolute results but not the relative analysis of the results. The wave angle in the interior domain is given in Figure 3-7.

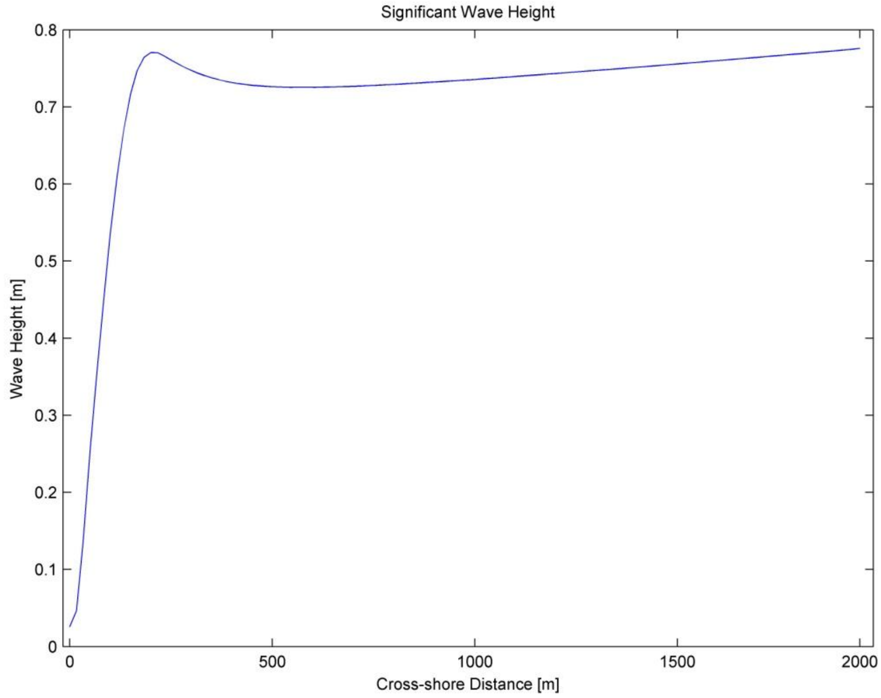


Figure 3-6 Wave height in the interior domain.

The wave force in the breaker zone, shown in Figure 3-8, is dominant in the cross-shore direction and mainly onshore directed. The wave forces are the gradients of the radiation stresses which is a function of the wave energy, wave angle and wave group velocity. These values change as the wave propagates. In the shoaling zone the wave group velocity increases from 0.5 to 1, the energy increases but the angle decreases due to refraction. This leads to a positive gradient in the radiation stress and thus an offshore directed force (due to negative relation between the force and the gradient in the radiation stress). In the breaker zone, the wave energy decreases rapidly causing a negative gradient in the radiation stress and thus an onshore directed force. That can be seen in the red line in Figure 3-8.The forces are balanced by a water level set-up and set-down. There is also a, smaller, force in alongshore direction. This is a lot smaller because the gradient of the longshore radiation stress is a lot smaller. This is causes by the small wave angle at wave breaking.

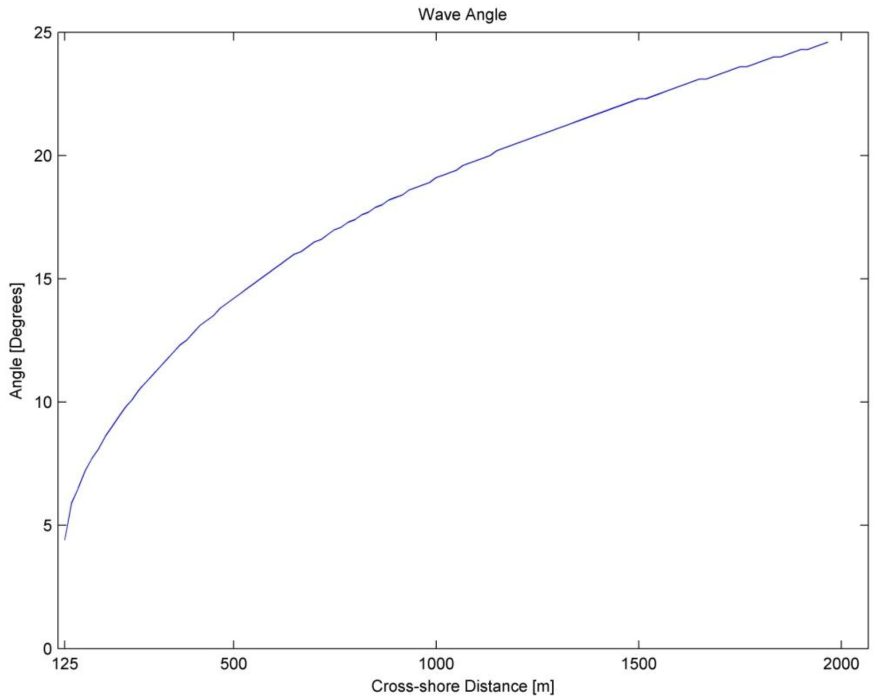


Figure 3-7 Wave angle in the interior domain.

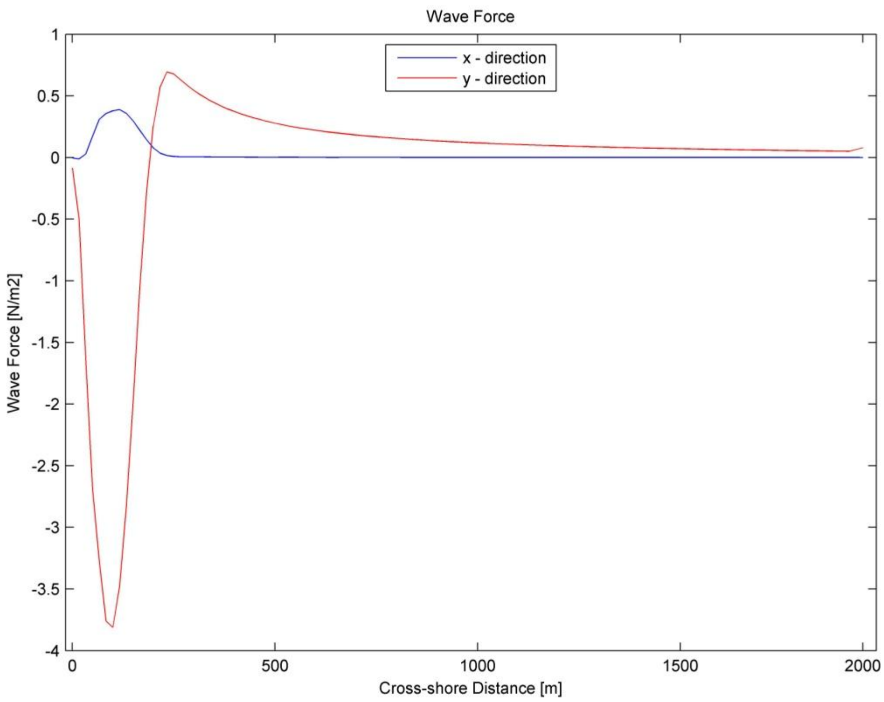


Figure 3-8 Wave forcing in x- and y-direction. Positive values for the x-direction means a forcing in the positive x-direction, thus from left to right in the domain. Negative values for the y-direction means onshore directed forcing,

Velocity field

The depth averaged velocity is influenced by the wave and tidal influence. Dependent on the location in the cross-shore, either one of them has a dominant influence. In the breaker zone the wave influence is dominant and further offshore the tidal influence increases. Since the waves come from one direction, the current velocity in the breaker zone is always in the positive x-direction. The tidal current changes direction so this should be visible in the results. This can be seen in Figure 3-9 which shows a plot of the depth averaged velocity at two different time steps in a cross-section. The blue line is the velocity during flood and the red line is the velocity during ebb. The tidal wave can be seen as shallow water wave and the velocity should therefore increase roughly proportional to $\sqrt{gh}$ when going further offshore.

Flow contraction in front of the harbour could play an important role in bypassing the sediment. Figure 3-10 shows a plot of the modelled velocity field at the same time-step as the blue line Figure 3-9. The direction of the current is depicted by the arrows; the magnitude is depicted by the colours and the length of the arrow. The flow contraction is clearly visible. Left and right of the harbour the flow velocity is approximately 0.25 m/s and in front of the harbour it is around 0.4 m/s. This is the situation during flood. During ebb the situation is similar, but opposite. At the reversal of the tide there hardly is any flow.

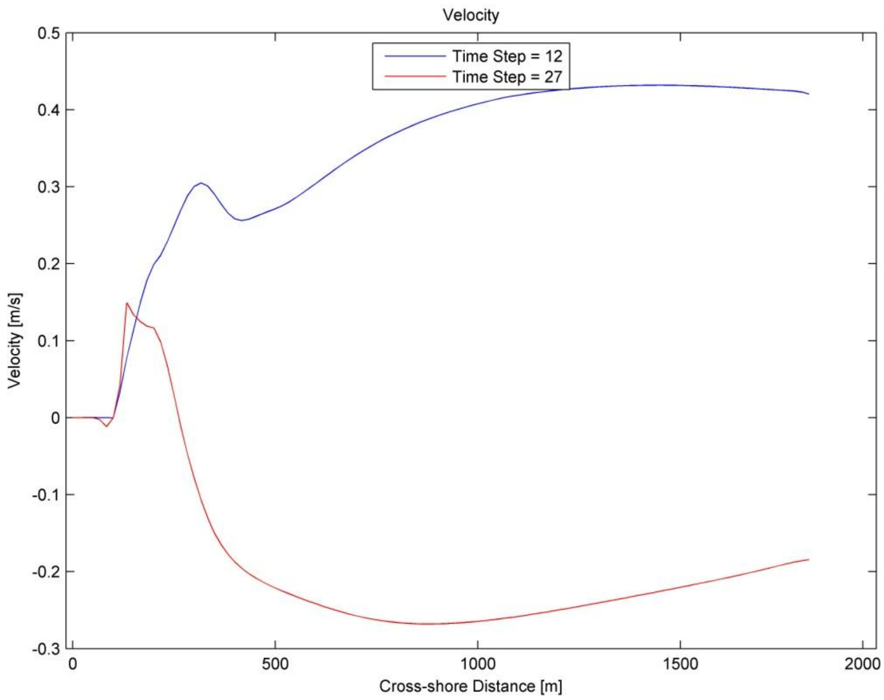


Figure 3-9 Depth averaged velocity at flood (blue line) and at ebb (red line). The time-steps are the time-steps at which the result is written to output file. This is done every 2 hours. 12 Time-steps therefore represent 24 hours,

3.3.2. Morphological processes

The hydrodynamic processes result in morphological processes. Sediment is transported in and through the domain. This is therefore explained in this paragraph. At first the mean transport is explained, followed instantaneous transport. The transport rates result in sedimentation and erosion which is elaborated next. The sedimentation and erosion consequently affects the bed level which is therefore explained last.

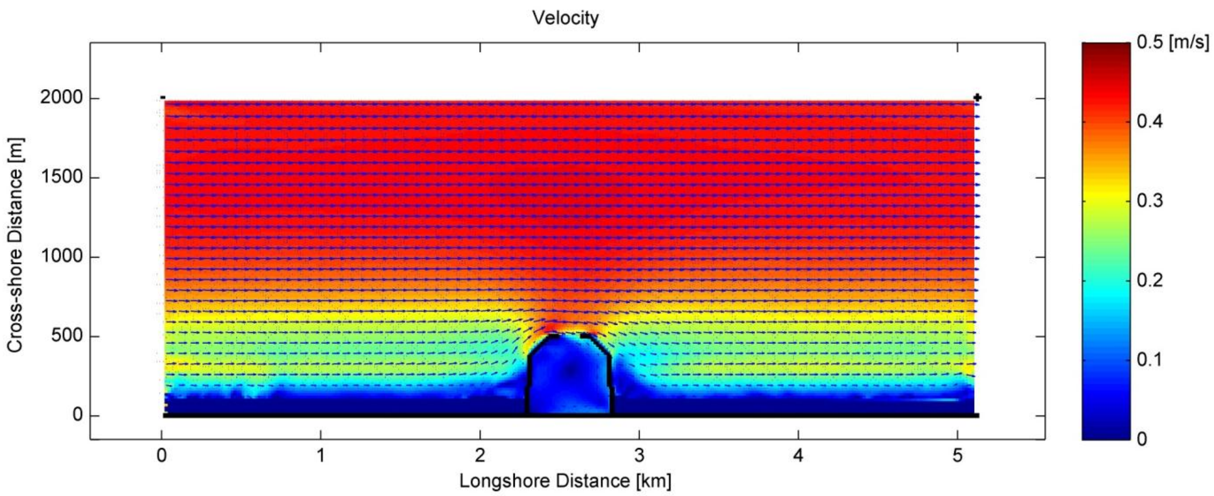


Figure 3-10 Velocity field at time-step 12.

Mean transport

At first a plot of the modelled mean sediment transport over the cross-shore will be shown. This is the amount of transport in m^3 per meter cross-shore per second. The transport is averaged in time over the entire simulation. Therefore the tidal influence is averaged out. Figure 3-11 shows a plot of the averaged longshore transport at a cross-section updrift (blue line) of the harbour and in front of the harbour (red line). Since the longshore transport is averaged over the entire simulation, the tide isn't visible in the plot. As expected the majority of the longshore transport takes place in the surf zone, where the waves break.

The peak in the blue line is much higher than the peak in the red line. That means that the longshore transport updrift of the harbour is higher than the transport in front of the harbour, averaged over time. The difference between the plots is the sediment which is captured at the updrift side of the harbour. The length of the breakwater is 500 m. The peak of the longshore transport in front of the harbour is shifted further offshore due to the offshore directed coastline progression just updrift of the harbour. The surf-zone propagates offshore due the accretion. The breaker zone – and thus the peak in the longshore transport – also propagates forward. This explains the shift of the peak in the longshore transport. The tail of both lines is slightly positive due to the skewness of the tidal wave. This gives large velocities in the propagation direction of the tide than it does in the opposite direction.

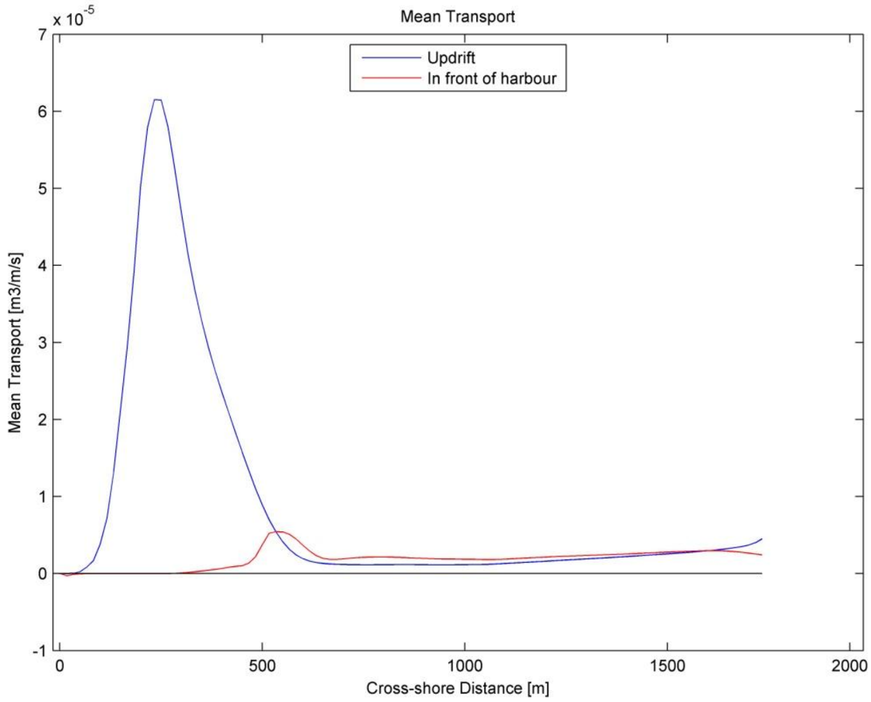


Figure 3-11 Average transport, distributed over the cross section.

Instantaneous transport

Now the instantaneous transports instead of the averaged transports are examined. Again, the situation after 12 and 27 time-steps are examined, just as is the case for the depth averaged velocity. Figure 3-12 shows a plot of the instantaneous transport at these two time-steps. The blue line is the transport during flood, hence the completely positive transport. The red line shows the transport during ebb, which shows a positive peak due to the wave influence and a negative 'tail' due to the negative current velocity. Contrary to the depth averaged velocity, the tidal influence is relatively small compared to the wave influence. This is because there is no initiation of motion at deeper water. Therefore it doesn't matter whether there is a high tidal velocity or not because there is only very little sediment in suspension.

The shape of the tail is somewhat different than expected. The tail is expected to have the same shape as the depth averaged velocity plot; proportional to $\sqrt{gh}$. The fact that it isn't, is expected to be caused by the calibration factors which influence the sediment transport due to the currents. The influence on the model output is however very limited because for the final results, the instantaneous transports are averaged per tidal cycle. The plots are than similar to Figure 3-11, which shows no sign of the tidal current-related transport.

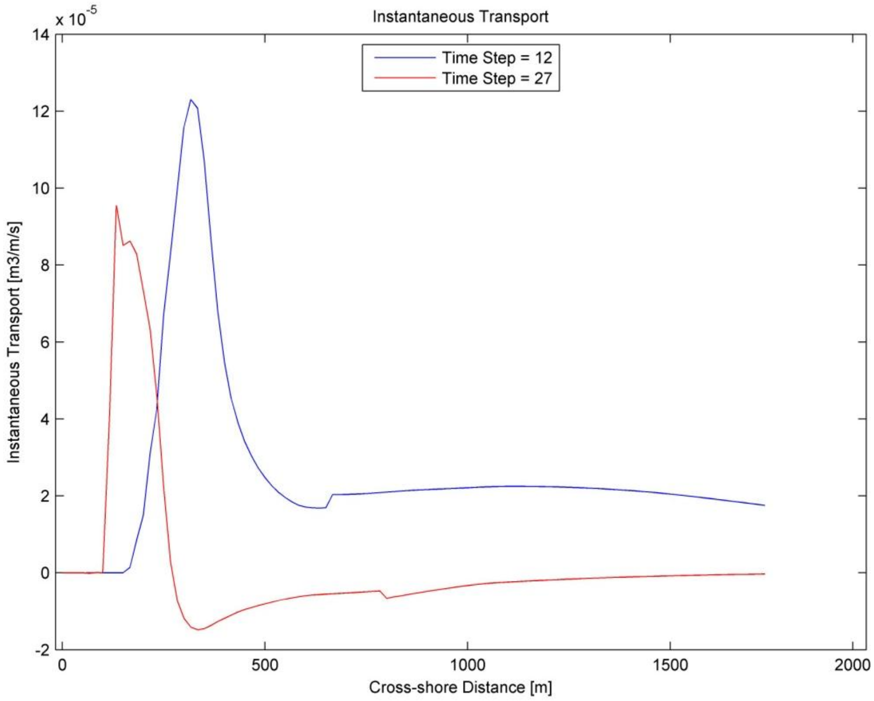


Figure 3-12 Instantaneous transport at flood (blue line) and at ebb (red line). No explanation can be found for the jump at 650 an 800 m.

Cumulative sedimentation/erosion

The updrift sedimentation and downdrift erosion are shown in Figure 3-13 which shows the cumulative sedimentation and erosion in the domain after a full run. As expected there is a significant amount of sedimentation updrift of the harbour and a similar amount of erosion at the downdrift side. The waves come from one direction so there is a net transport from left to right in the domain. The majority of this transport is blocked by the breakwaters, causing the sedimentation and erosion. There is also some significant sedimentation in the harbour due to the sediment carrying inflow.

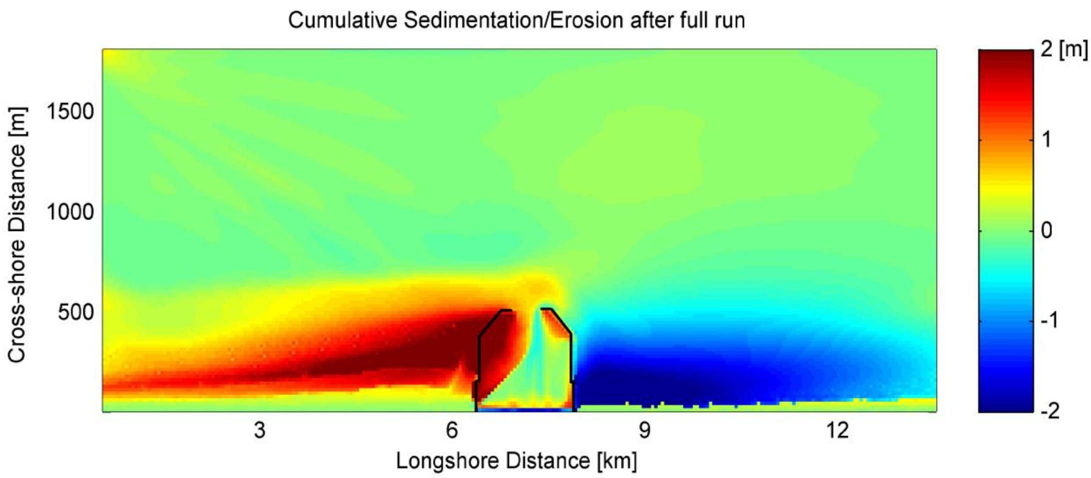


Figure 3-13 Cumulative sedimentation and erosion after a full run, inclusive the harbour.

Bed level

The cumulative sedimentation and erosion have a strong effect on the bed level. The bed level at the beginning (top) and at the end (bottom) of the simulation is shown in Figure 3.14. The effect of the sedimentation can be seen in the bottom plot. The coastline is progressed forward to about 70 % of the breakwater length. At the downdrift side the coastline is retreated in accordance with the erosion. The coastline progression at the updrift side caused the bed level to decrease from 3.54 m to 2.70 m. Another effect is that the amount of transport increases in front of the harbour. These effects will all be discussed in Chapter 4 where the results of all the runs are explained and compare to each other.

Figure 3-15 shows cross-sections of the bed level. The blue line is the initial bed level. This is the same anywhere in the domain. The red line shows the bed level at the end of the run at the updrift side of the harbour. This clearly shows the accretion. The green line shows the same for the downdrift side of the harbour.

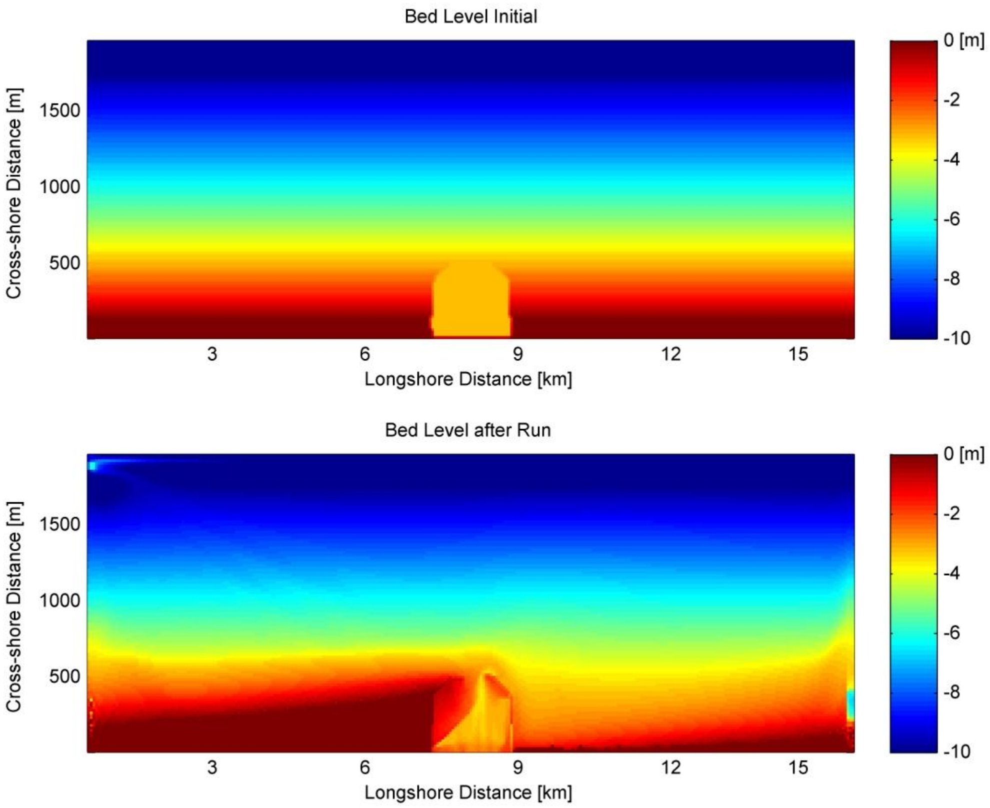


Figure 3-14 Initial bed level (top) and the bed level after a full run (bottom)

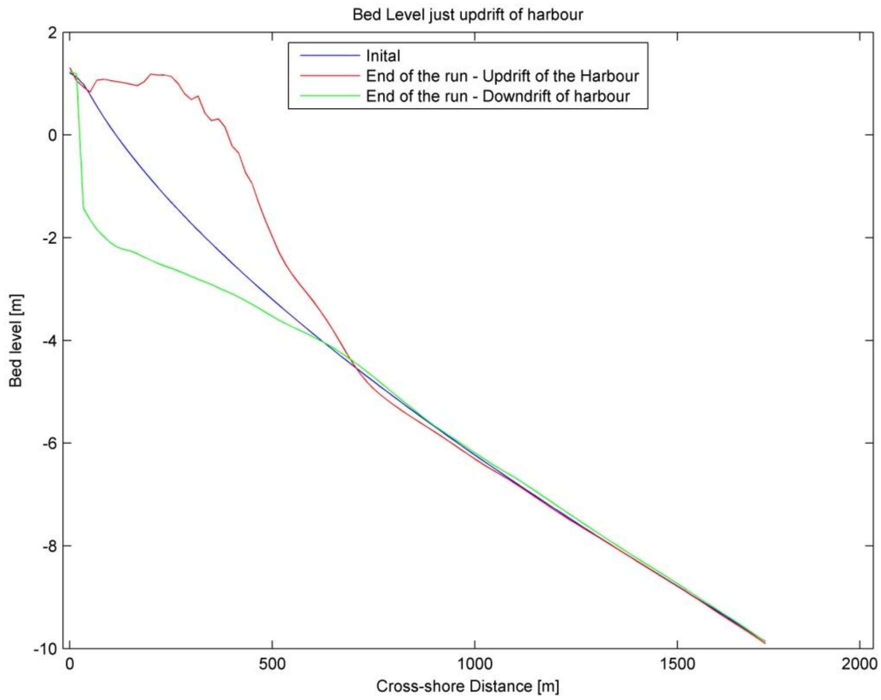


Figure 3-15 Bed level at the beginning of the simulation (blue line), the bed level updrift of the harbour at the end of the simulation (red line) and the bed downdrift of the harbour at the end of the simulation (green line).

3.4. Quantitative calibration

The amount of sediment transport has been one of the parameters on which the model is calibrated quantitatively. In order to do so the sediment transport is compared to that along the Dutch coastline. The Dutch coastline is chosen because this is a coastline which is also influenced by waves and tide. The sediment size and the tide roughly correspond to that applied in the model. The goal is to get a model which calculates the right amount of sediment transport. When this can be concluded, the model can be applied with different hydrodynamic properties. This is the closest one can get to a case in reality since no actual case is modelled. The calibration method is described in this paragraph.

Several sources in literature state that the sediment transport along the Dutch coastline is several hundred thousand cubical meters in both directions. A report of Van Rijn (2002) gives gross transport of 500 000 – 600 000 $m^3/year$ at Egmond with an inaccuracy of about 30 %. The net transport is about 100 000 $m^3/year$ northwards with an inaccuracy of 15 %. These values are used to roughly compare the sediment transport in the model with. A rough comparison means the right order of magnitude in longshore transport.

The used method will be described briefly. First of all a wave data set was acquired of the website Live.Waterbase.nl. The used dataset is that off the Europlatform which is located around 50 km offshore and around 80 km from Egmond. This is not considered to be a problem since the wave climate doesn't vary much over such distances. Additionally, small differences are not a large problem since the inaccuracy in the sediment transport modelling is quite large and this is merely a rough comparison. The dataset consists of 3-hourly measurement of the significant wave height, wave period and wave angle. This gives a goof representation of the wave climate.

The wave climate has been reduced by using a wave climate reduction method. Therefore the entire wave climate is separated in directional bins, wave height bins and wave period bins. Each bin has a certain probability of occurrence. Consequently the sediment transport is calculated for each wave bin, using the CERC formula. This is the sediment transport which occurs due to the full wave climate. This can be plotted in a transport rose. In a transport rose the relative contribution per directional bin to the total sediment transport is plotted, see Figure 3-16.

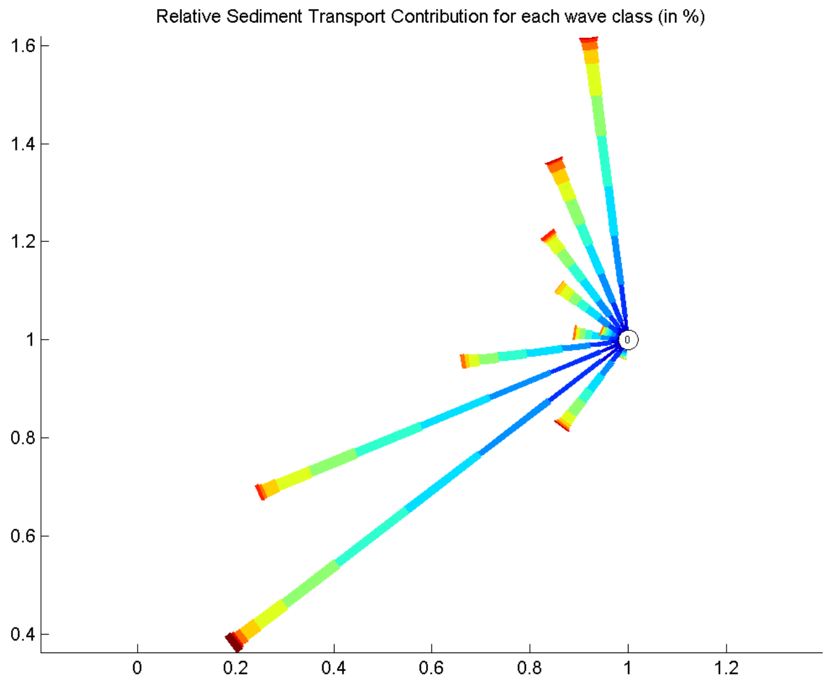


Figure 3-16 Transport Rose for the wave conditions measured at the Europlatform.

The next step is to find the same amount of transport with a reduced wave climate. This step has the largest amount of inaccuracy because in my case four conditions are deduced from the transport rose. The contribution of the transport in each wave bin is divided by the total contribution. This gives the weighted transport per directional bin. From this four conditions are chosen and their duration depends on the weighted amount of transport. That resulted in the wave conditions depicted on the next page.

These conditions are applied as boundary conditions in the WAVE module. This resulted in a certain amount of calculated longshore transport. Since this was not similar to the actual transport, it was scaled up. That is a trick to create a better match of the modelled transport with the actual transport. In this model the longshore transport needed to be scaled up with a factor 5 to achieve a similar amount of sediment transport as the measured data. Unfortunately this scaling up gave some instabilities in the model; to be exactly, at the upper left corner of the interior domain. This instability influenced the processes around the breakwater. This influence was limited by decreasing the morphological time scale factor. The morphological time scale factor was initially set to 60, such that a morphological time span of 5 years was modelled. This is adjusted to 24 and now a morphological time span of 2 years is modelled. This does not influence the course of this research because a bypass is established within 2 years. Additionally, a higher morphological time scale factor increases the instability so it has a positive influence to decrease the factor. This resulted in a calculated northwards transport of 250 000 $m^3/year$ and a southward transport of 150 000 $m^3/year$ which is in reasonable accordance with the literature.

Time step	H_s $[m]$	T_p $[s]$	φ_0 $[o]$	Directional spreading $[o]$	Contribution to total transport [%]
0	1	6.3	305	1	32
13824	1	6.3	305	1	
13825	1	6.3	330	1	31
27216	1	6.3	330	1	
27217	1	6.3	40	1	15
36504	1	6.3	40	1	
36505	1	6.3	70	1	22
43200	1	6.3	70	1	

Additionally the cross-shore processes didn't coop with what was expected either. The bed level change showed a large onshore transport in the breaker zone. The model showed strong accretion on the beach and strong erosion at the edge of the surf zone. The beach profile got very steep. This does not coincide with what should be expected. Therefore the cross-shore processes are down scaled with a factor 10. This decreased the cross-shore processes but that is not expected to effect the results since the cross-shore processes are not that important for this study. The calibration process also proved that the scaling factor for the longshore transports influenced the cross-shore processes. There remains an onshore transport, if the longshore transport isn't scaled up. The scaling up of the longshore transport has, in this case, a positive effect on the cross-shore transport. It should be noted that only the morphological response to the hydrodynamic processes is scaled up or down. The hydrodynamic processes remain unchanged.

By setting these calibration factors the model roughly gives the same amount of transport as is the case along the Dutch coastline. These settings are used to model the scenarios. In the final calculations, the boundary conditions will change, but by having adjusted the settings to a real case, it can be concluded that the calculated amount of sediment transport is a realistic amount. This gives a better understanding of, and trust in, the model. If it would not be calibrated, it wouldn't be clear if the amount of sediment transport is comparable with a similar situation.

3.5. Conclusion

From this chapter it can be concluded that the model calculates most of the acting processes correctly. Only the refraction process is overestimated. This influences the absolute results but not the relative analysis of the results.

The velocity field shows what should be expected; high velocities in the breaker zone due to wave breaking and more tidal influence at regions further offshore. As expected, the model results show flow contraction in front of the harbour.

After calibration the sediment transport patterns are in line with the expected patterns. The cross-shore influence needed to be minimized to achieve that. The cross-shore processes are not of importance for this study so this does not influence the results. The longshore processes are as they should be; a large amount of accretion updrift and a similar amount of erosion downdrift. Within the modelled time frame, a bypass is initiated.

CHAPTER 4.
MODEL CALCULATIONS

The results of a base-scenario are explained in the previous chapter. The results from this base-scenario have been analysed and they are in agreement with the expected behaviour. The actual calculation can be made now the model is determined trust worthy. At first it needs to be known which model output is gathered and how this is assessed. That is explained in Paragraph 4.1. The analysis of output is given in Paragraph 4.2. This analysis is done, based on numerous graphs. These graphs are depicted in Appendix E but summarized in tables in the main report.

4.1. Result analysis

The setup of the model is described in Chapter 3. From this chapter it can be concluded that the model predicts the right type of behaviour of the system. This model can therefore be used to answer the research question;

Which conditions are required to optimise the bypassing of sediment around a harbour in combination with limited influence on the navigational functions of a harbour?

The question can be answered by assessing different situations with the model. Each calculation gives numerous types of output and data to be analysed. Therefore it will be explained first which data is required and how this is gathered and visualized.

The results of the different runs need to be analysed. The following parameters are the most important output:

- The amount of bypassing related to the undisturbed longshore sediment transport.
- The depth in front of the harbour related to the initial depth. A large decrease in depth limits the navigational functions of the harbour.

Also of importance is the following parameter:

- The current velocity in front of the harbour. This also determines whether shipping is possible.

The values for these properties for different runs are compared to each other. Below it is described how this output is gathered and processed.

4.1.1. Data gathering

Each run provides a lot of output and this should be processed in order to be able to compare the result from different runs with each other. The above mentioned properties need to be deduced from the model output. The above properties are certainly very important but a whole lot more output is required in order to be able to derive conclusion from these properties. Which data is required and how this is acquired is described below.

Map data
Some properties are measured in every point in the domain every two hours. These are so-called map-data. The model output of the following properties is used:

- Bed level;
- Cumulative erosion/sedimentation;
- Depth averaged velocities;
- Wave height.

This can give maps of the domain containing the values of the properties in the entire domain on a certain time step. Specific location, like cross-sections can be chosen to give the values of the property in that cross-section at the specified time-step. When more time-steps are considered, a movie can be made, or one of the following options can be chosen.

Observation points
The dots in Figure 4.1 are the observation points. Amongst others, the following important properties can be measured:

- Water level;
- Depth averaged velocities;
- Bed level.

The data coming from the observation points is straightforward; they are the values of the properties for every 10 minutes.

Cross-sections
The blue lines in Figure 4.1 are cross-sections. All types of fluxes can be measured in these cross-sections. The following output is important for this study:

- Instantaneous transport;
- Cumulative transport.

In cross-sections the properties are integrated over the length of the cross-section. It therefore gives one value per property per 10 minutes, for instance the instantaneous transport over the entire cross-section at a certain time-step. The instantaneous transport especially is an important parameter. The instantaneous transport measured at the cross-section in front of the harbour is divided by the instantaneous transport measured at the cross-section updrift. The result is the relative transport in front of the harbour, relative in the sense that it is related to the initial transport measured updrift. This way the bypassing is measured.

Length of the cross-sections

The cross-sections span the first 850 m in case of the cross-section updrift and downdrift of the harbour. In front of the harbour, the cross-section spans the 350 from the tip of the breakwater. In case shorter or longer breakwaters are considered, the cross-section in front of the harbour will shift in accordance with the length of the breakwater. The length of the cross-section however remains the same: 350 m. The length of the cross-section is an important parameter because it determines how much transport is measured. A longer cross-section measures more transport. The majority of the transport takes place in the surf zone, but there is some transport due to the tidal influence at deeper water. In case of longer cross-sections, longer than the width of the surf zone, more tide-induced transport is measured. It therefore matters how long the cross-section are.

The tidal influence can however be averaged out. Since it is a symmetric tide, it doesn't contribute to the net transport. Therefore the measurements of the instantaneous transport in the cross-sections are averaged per tidal cycle. The wave induced transport in the surf zone remains and these values are used for further analysis. The longshore transport profile looks like the ones depicted in Figure 3-11. It is therefore only important that the length of the cross-section is long enough to measure all the wave-induced transport. From the output of the long-term calculations, it can be withdrawn that also on a longer time-scale all the wave-induced transport occurs within the first 350 m in front of the breakwater. It would have been an option to lengthen the cross-section to the end of the interior domain. In that case this issue wouldn't have risen, but that has been determined after part of the calculation was finished.

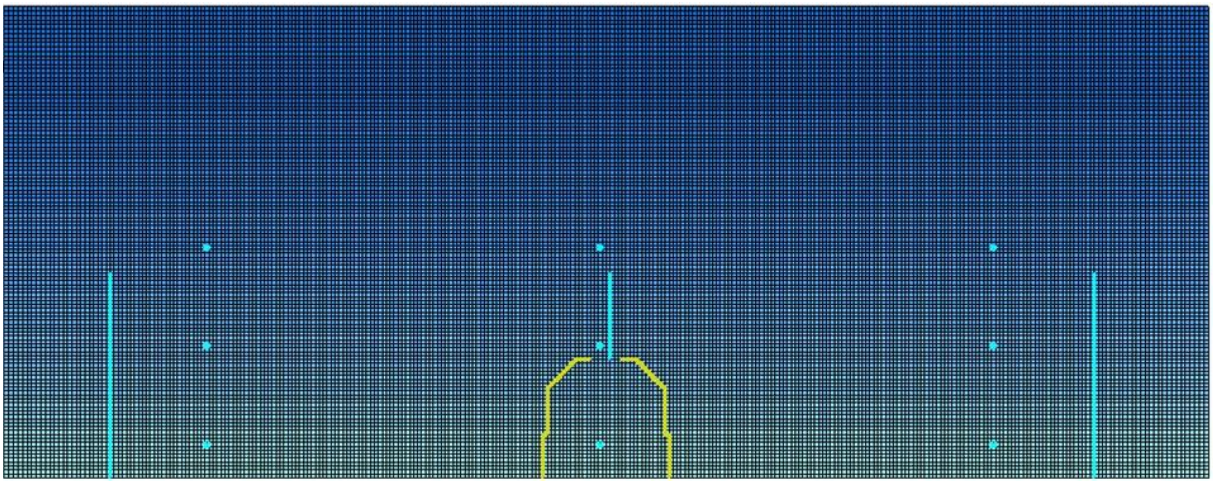


Figure 4-1 Cross-sections over which the sediment transport is measured. The amount of transport in the middle cross-section - in front of the harbour - is divided by the amount of transport going through the left cross-section.

4.1.2. Result visualisation

The results need to be visualised properly to compare them with each other. However, different situation give different amount of initial transport or velocities, which makes it hard to compare them with each other. That is the reason that the values for these properties are made relative;

the outcome in front of the harbour is divided by the outcome updrift. This way the relative values can be compared to each other. In that case it doesn't matter if a run in which a wave angle of $30^o$is simulated gives less transport than a run in which a wave angle of 45^o is simulated. An example of such a graph is given in Figure 4-2.

These types of results are called relative bypassing or relative velocity. The same can be done for the bed level in front of the harbour. This is for all cases divided by the initial depth and this gives the relative change of the bed level. As said before, these are the most important properties for analysing the results. The graph for relative transport should always be read together with the graph for the relative bed level. A large relative bypassing could be caused by a large reduction of the depth in front of the harbour, making it more or less useless. The situations in which the bypassing is as high as possible and the depth didn't decrease much are the most interesting ones.

For this reason the relative transport and relative bed level are not the only output values which are assessed. The following set of output properties is analysed per batch of calculations:

- Relative bed level in front of the harbour;
- Relative Transport in front of the harbour;
- Absolute bed level in front of the harbour;
- Relative depth averaged velocities in front of the harbour;
- Relative Coastline Progression just updrift of the harbour;
- Cumulative transport updrift and in front of the harbour;
- Bed level at the end of the run in the entire domain;
- Cumulative erosion/sedimentation at the end of the run in the entire domain.

An example of a plot of the relative transport is given below. In this case it is the relative transport in front of the harbour in five different calculations. The wave angle is different in each calculation.

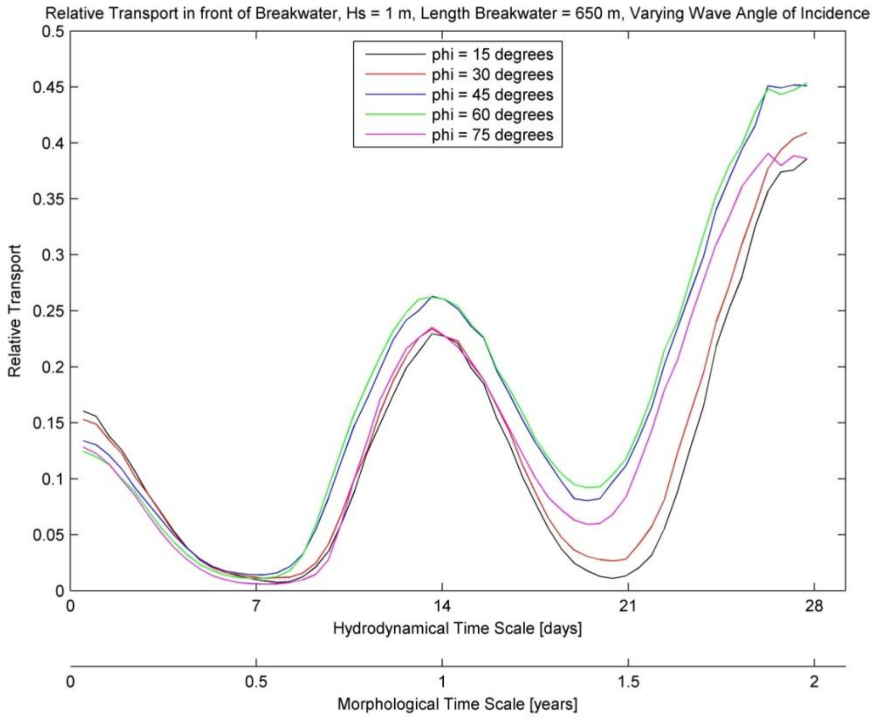


Figure 4-2 Example of how the results will be visualized. This example gives the relative transport in front of the harbour for five different scenarios, each with a different wave angle. The vertical axis gives the relative index of the transport. A value of 1 would mean that all the incoming transport is bypassed. The horizontal axis gives the simulated time. There are two time scales; a hydrodynamic time scale and a morphological time scale. The latter is longer because a morphological time scale factor of 24 is applied.

4.1.3. Optimization method

Sediment transport depends on numerous variables. Not all variables can be assessed in this study and some of the variables have been excluded by the choice of archetypes. The following variables remain and their influence on bypassing and bed level in front of the harbour is assessed (in this order)

- Length breakwater;
- Wave climate;
  - Wave angle;
  - Significant wave height;
- Shape breakwater.

The influence of each parameter is assessed by varying it and comparing the results with each other. The promising cases are assessed for a longer period. These are assessed for a morphological period of 4 $year$ instead of 2 $years$. The influence of the bathymetry and sediment size will not be assessed.

Some assumptions have been made at an earlier stage. These assumptions are checked with a few additional calculations. The assumption of the influence of the tide and a dredged channel are assessed.

4.2. Results runs

In this paragraph the results of the calculations are explained. The results are treated in the same order as the above mentioned steps. For the first assessment the graphs on which the analysis is performed are added to the main report. The results of the other assessment are summarized in tables. All the graphs can be found in Appendix E.

4.2.1. Assessment of the influence of the length of the breakwaters

The influence of the length of the breakwaters has been assessed first. Therefore three calculations have been done with varying breakwater lengths: $\frac{2}{3}$, 1.0 and $\frac{4}{3}$ of the width of the surf zone. The following settings are applied:

- Wave height: $1\ m$;
- Wave angle: 45^o;
- Wave period: $7\ sec$;
- Length breakwater: $350\ m$, $500\ m$ and $675\ m$;
- Simulation time: $1\ month$ ($2\ years$);
- Breakwater geometry: semi-streamlined.

Bed level

Figure 4-3 shows a plot of the relative bed level in time in front of the harbour. The bed level which is measured in the observation point in front of the harbour is divided by the initial bed level measured in the same location. This gives the relative bed level.

The most striking output is the depth change in case of breakwater longer than the width of the surf zone. It is obvious that the depth reduction in front of the harbour takes longer in case of longer breakwater because it takes longer before the coastline has progressed sufficient to induce sedimentation in front of the harbour. In this case the longest breakwaters even induce erosion in front of the harbour due to the large flow contraction and cross-shore currents. Eventually the updrift sedimentation will cause the depth to decrease rapidly due to the lack of wave action in front of the harbour. The transport capacity is too small to bypass the sediment.

The same occurs for the breakwater equal to the width of the surf zone. The depth initially doesn't change much, just as the longer breakwater, but as the updrift coastline accretion continues more and more sediment accretes in and in front of the harbour. This however comes to a hold as the new equilibrium depth is reached.

The depth decreases faster for the smallest breakwater but the depth reduction is quite limited and even comes to a hold This is related to the wave influence. Since the smallest breakwater reaches smaller water depth, the wave influence is larger. The waves break in front of the harbour which decreases the sedimentation and thus the depth reduction. The depth reduction for the smallest breakwater is only a few decimetres, see Figure 4-4.

The relative coastline progression can be seen in Figure 4-5. The coastline progression is divided by the length of the breakwaters. This plot clearly shows that the coastline has progressed relatively less in case of longer breakwaters. In absolute sense the progression is more for longer breakwaters because more sediment is trapped updrift of the harbour.

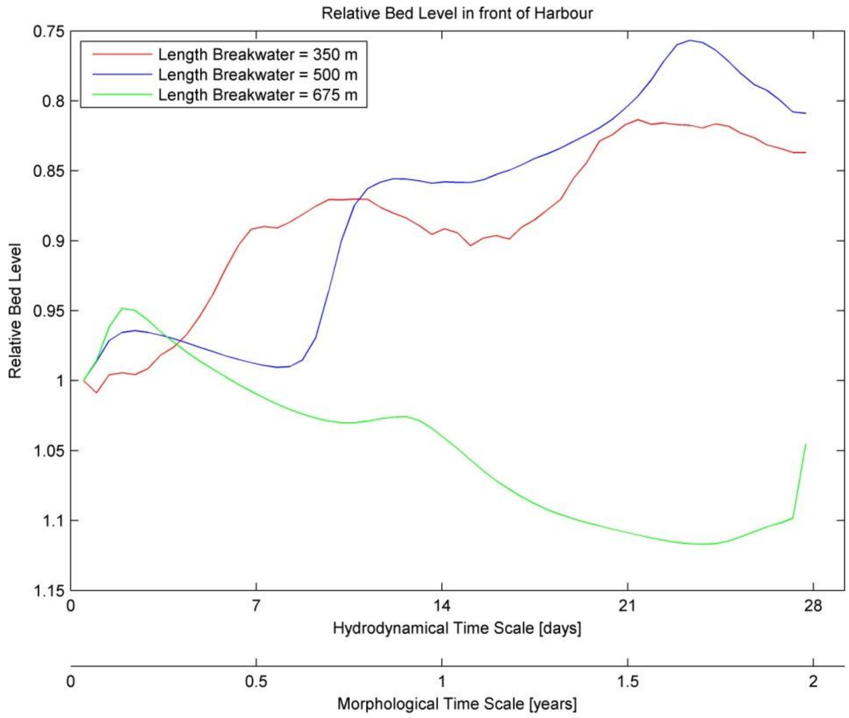


Figure 4-3 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

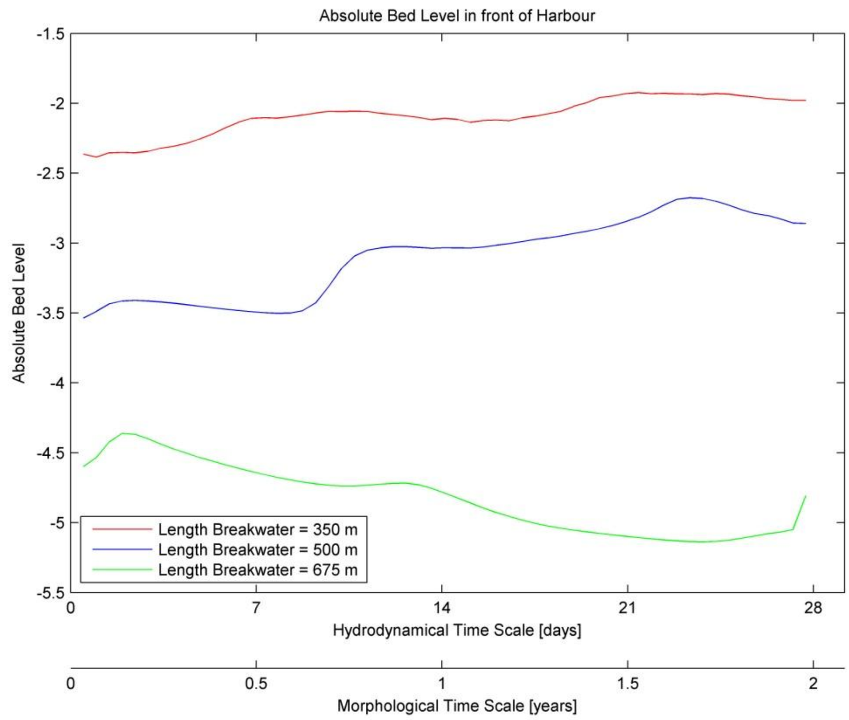


Figure 4-4 Plot of the absolute bed level in time in front of the harbour.

Relative velocity

Figure 4-6 on page 78 shows the results for the relative depth averaged velocity in front of the harbour. In this case the maximum velocities in front of the harbour per tidal cycle are divided by the same values but then measured updrift at the same depth. This gives a value for the flow contraction caused by the presence of the harbour.

The results for the depth averaged transport gives rise to the idea that the waves play a larger role on the flow contraction than initially expected. That can be concluded from the fact that the flow contraction is a lot more in case of the smallest breakwater, related to the longer breakwaters. The smallest breakwater reaches less deep water so the waves contribute more to the flow velocity in front of the harbour.

This is backed up by the fact that for this case, the flow contraction for the smaller breakwater increases over time. If the tidal influence would be dominant, the flow contraction would decrease in time because the relative length of the breakwaters decreases due to the updrift coastline progression. That can clearly be seen in the flow contraction around the longest breakwater, see Figure 4-6.

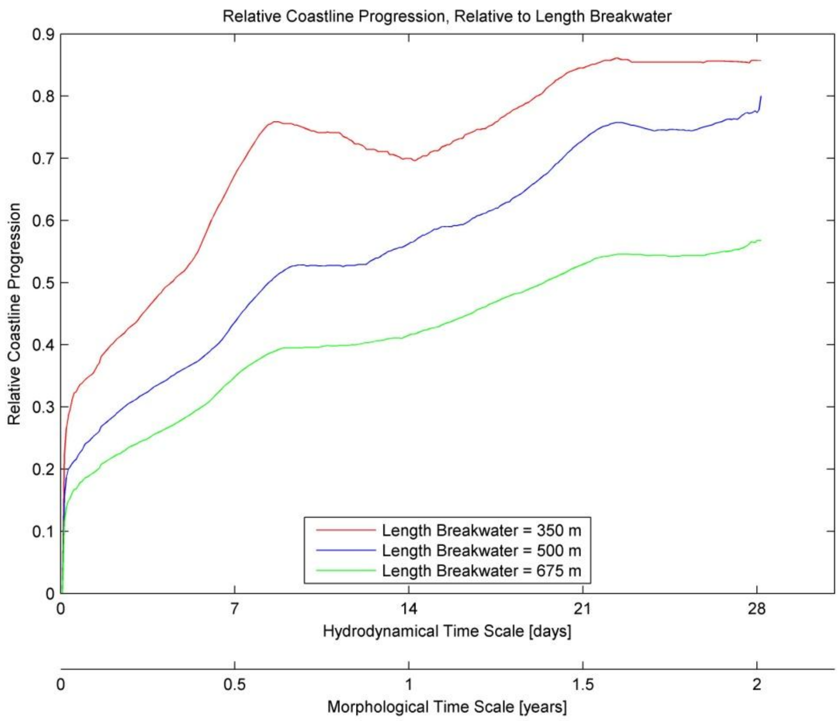


Figure 4-5 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

The tidal influence can also be very well noticed for the case $L_{BW} = 500\ m$; the flow contraction is more in case of spring tide and less in case of neap tide. During neap tide the flow velocity in front of the harbour is even less than updrift. It has been checked that is due to the fact the measurement station is just outside the flow contracted zone during neap tide. The flow contraction is little more offshore during neap tide.

The flow contraction is measured relative; the velocity in front of the breakwater is divided by the velocity measured at the same depth updrift of the harbour. In absolute terms the flow velocity in front of the harbour is higher for the longer breakwaters, see Table 4-1. That is caused by the fact that these harbours reach deeper water, where the tide dominates the current velocity. The waves only dominate the current velocity in the breaker zone. The smaller harbour could thus very well show a higher flow contraction but this depends on the flow velocity measured updrift. It can therefore not be concluded that the flow velocity around smaller harbours is higher. The sediment transport depends on absolute velocities and not on relative velocities, so it cannot be concluded that the higher flow contraction in case of smaller harbours lead to more relative bypassing. It is mainly the availability of sediment combined with the wave action that causes the high relative bypassing for smaller breakwaters. The maximum absolute velocities in front of the harbour are given in the following table.

	350 m	**500 m**	**675 m**
Maximum absolute flow velocity $[m/s]$	0.32	0.35	0.45

Table 4-1 Maximum absolute longshore velocities in front of the harbour. The maximum velocities occur during spring tide.

NOTE: although the results are averaged over a tidal cycle, the influence of the spring-neap tidal cycle remains visible in the results. The relative transport in front of the harbour is significantly more in case of spring tide than it is in case of neap tide. This is expected to be caused by skewness of the tidal wave. Skewness causes the flood current to be higher than the ebb current. This effect can especially be seen close to coastline.

Also, at spring tide there is a larger variation in water level. The water level variation is not averaged out and this affects the various processes. At ebb during spring tide, the wave influence is a lot higher in front the harbour because the waves break further offshore. Both effects cause a higher influence on the system during spring tide than during neap tide, which is visible in the results.

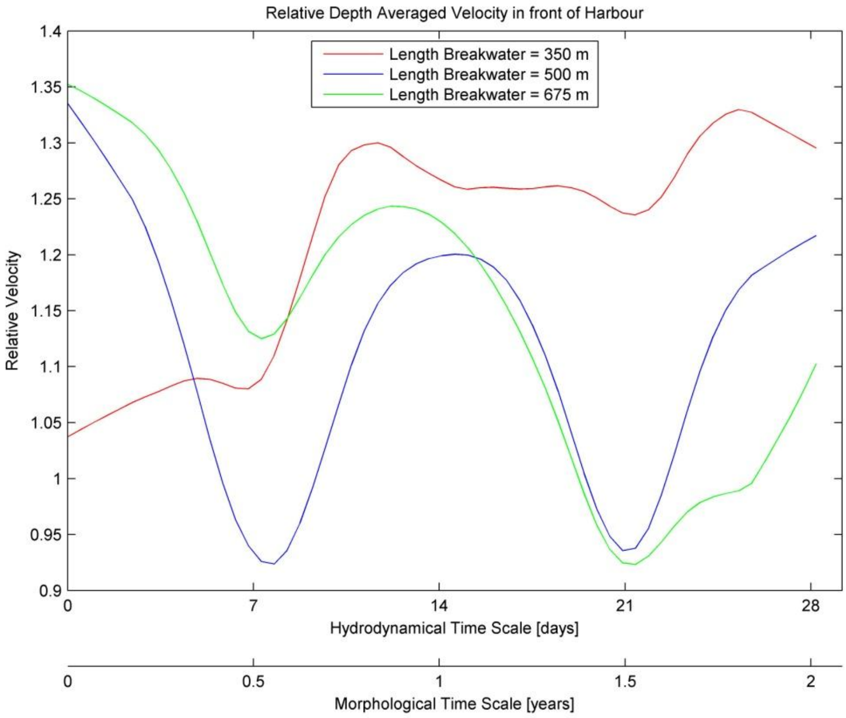


Figure 4-6 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

Relative bypassing

The above mentioned description can also be recognized in the plot for the relative transports in front of the harbour, see Figure 4-8. For this plot the instantaneous transport which is measured at the cross-section in front of the harbour, is divided by the instantaneous transport which is measured at the cross-section updrift of the harbour. Both values are averaged over a tidal cycle. A value of 0.5 in this plot means that 50 % of incoming sediment is transported past the harbour. This is can therefore be seen as the relative bypassing.

The results show that the relative bypassing is higher in case of shorter breakwater. It takes longer to create a bypass in case of longer breakwaters because more sediment is 'captured' at the updrift side. The smaller breakwater is smaller than the width of the surf zone, so only part if the transport is blocked. The rest can be transported in front of the breakwater. The below figure shows a plot of the mean transport, measured updrift of the harbour, and the lengths of the different breakwaters, indicated by the coloured bars at the bottom of the graph.

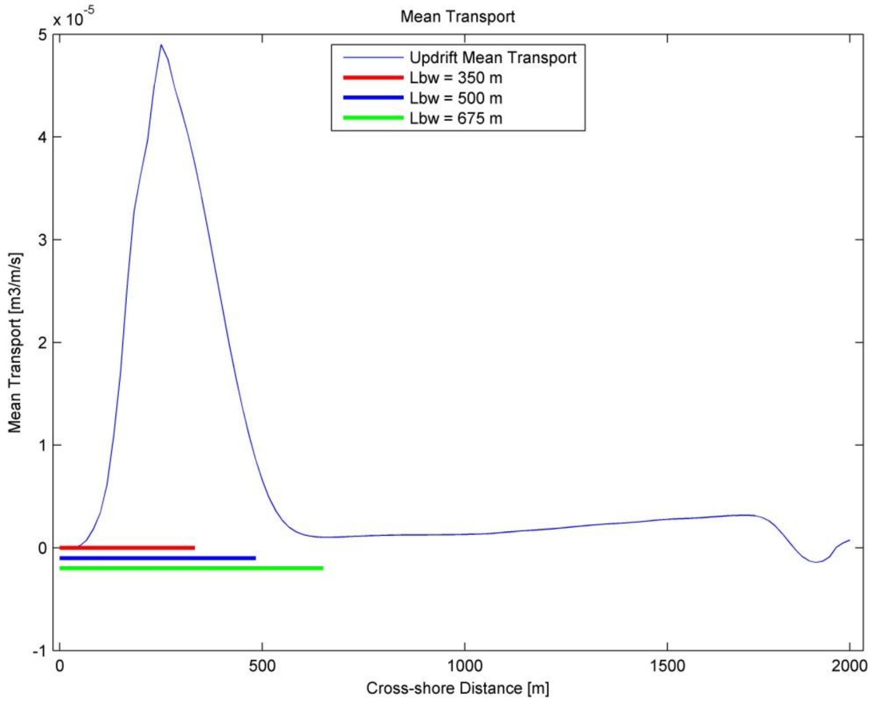


Figure 4-7 Plot of the mean longshore transport, divided over the cross-shore at a location 1500 m updrift of the harbour. The coloured lines indicate the lengths of the different modelled breakwaters.

The waves play a large role as well. The wave breaking contributes to the longshore current in front of the harbour but it also gives dissipation which prevents sediment from settling in front of the harbour. This sediment could consequently be transported past the harbour. This gives rise to the idea that bypassing can only occur in a reasonable amount if the area in front of the harbour is shallow enough to induce wave breaking.

The plots of the cumulative sediment transport in front of the harbour and updrift also show this effect, see Figure 4-9. The transport which is measured updrift ($S0$) is the same for all three cases because the hydrodynamic forcing is the same. The transport in front of the harbour ($S1$) differs at which the cumulative transport is largest in case of the smallest breakwaters. The goal is find conditions in which the relative bypassing is the highest and the depth decrease the least.

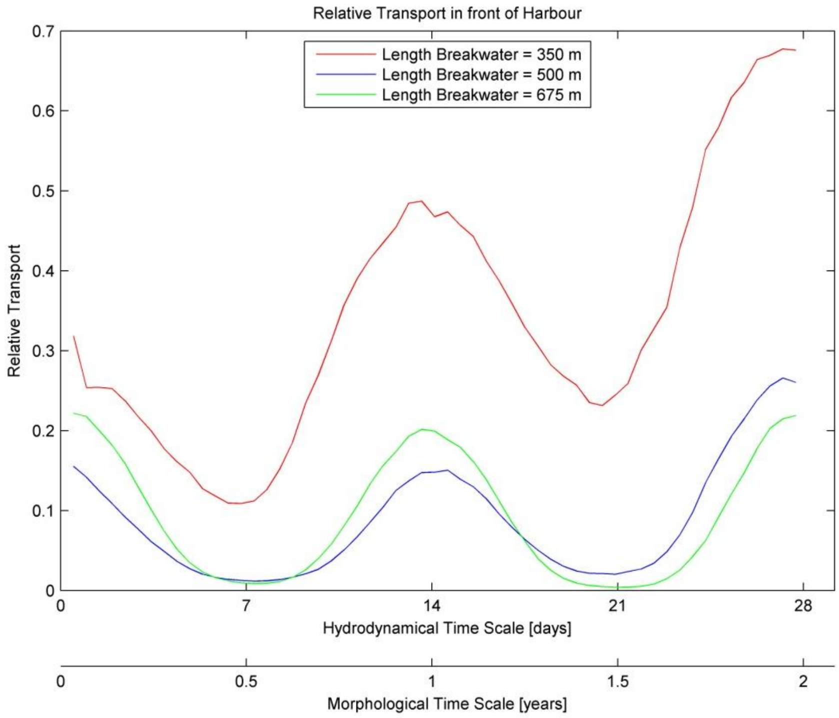


Figure 4-8 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle.

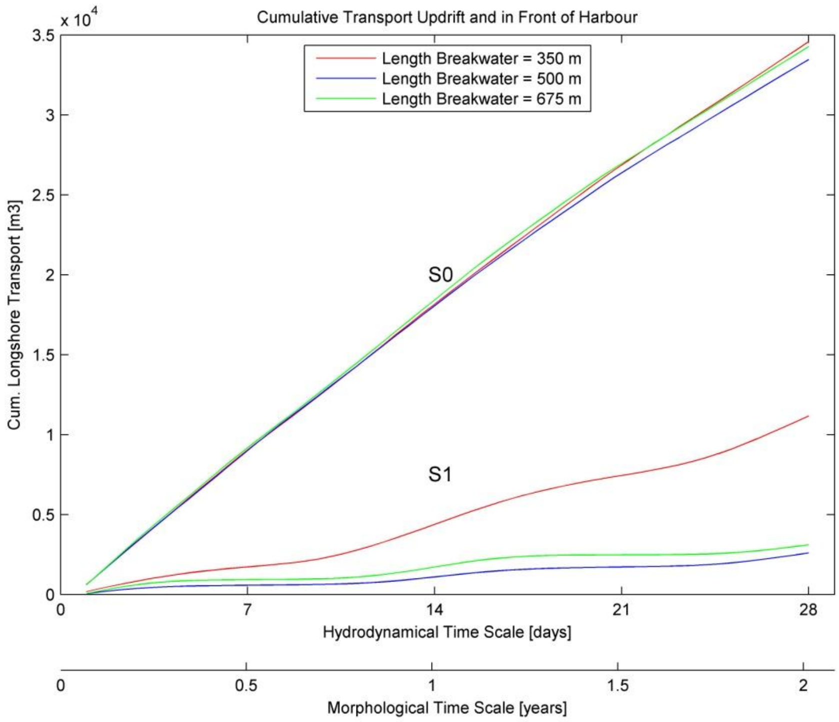


Figure 4-9 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour (S1) and in front of the harbour (S1).

The following figures help in understanding the above described processes. Figure 4-10 shows three plots of the cumulative sedimentation and erosion at the end of the calculations. The updrift sedimentation and downdrift erosion is clearly visible. It can also be seen that there is a lot of sedimentation in the smallest harbour. The depth was initially similar to the depth at the entrance but the depth decreases due to the sedimentation. The sedimentation is caused by the fact that the breakwater is shorter than the width of the surf zone due to which the waves can transport the sediment into the harbour.

The harbour similar to the width of the surf zone also experiences significant sedimentation. This explains why there is hardly more bypassing than the larger harbour; a large part of the sediment in front of the harbour is transport into the harbour instead of past the harbour. There is no bed level change in case of the largest harbour. There is even some erosion in front of the harbour. This settles next to the harbour entrance at the downdrift side, creating a shoal. Since there is only very little bypassing in case of the largest harbours, there is a lot more erosion at the downdrift side of the harbour than there is for the smallest harbour. The sediment fluxes affect the bed level. This can be seen in Figure 4-11. The updrift coastline progression, sedimentation in the harbour and downdrift erosion is clearly visible.

The following table gives the tabulated results. These are the values at the last time-step.

Variable / **Property**	**Length Breakwater**		
	$350\ m$	**$500\ m$**	**$675\ m$**
Relative bed level [-]	0.84	0.81	1.04
Initial absolute bed level $[m]$	- 2.37	- 3.54	- 4.59
Absolute bed level after run $[m]$	-1.98	-2.86	-4.81
Relative bypassing [-]	0.68	0.26	0.22
Relative depth averaged velocity (flow contraction) [-]	1.30	1.21	1.08
Relative coastline progression updrift [-]	0.86	0.80	0.57
Cumulative transport updrift $[m^3]$	34 925	33 793	34 645
Cumulative bypassed sediment $[m^3]$	11 425	2 720	3 224

Table 4-2 Tabulated results of the calculations in which the length of the breakwater is varied. The values are the last measured results at the harbour entrance unless stated otherwise.

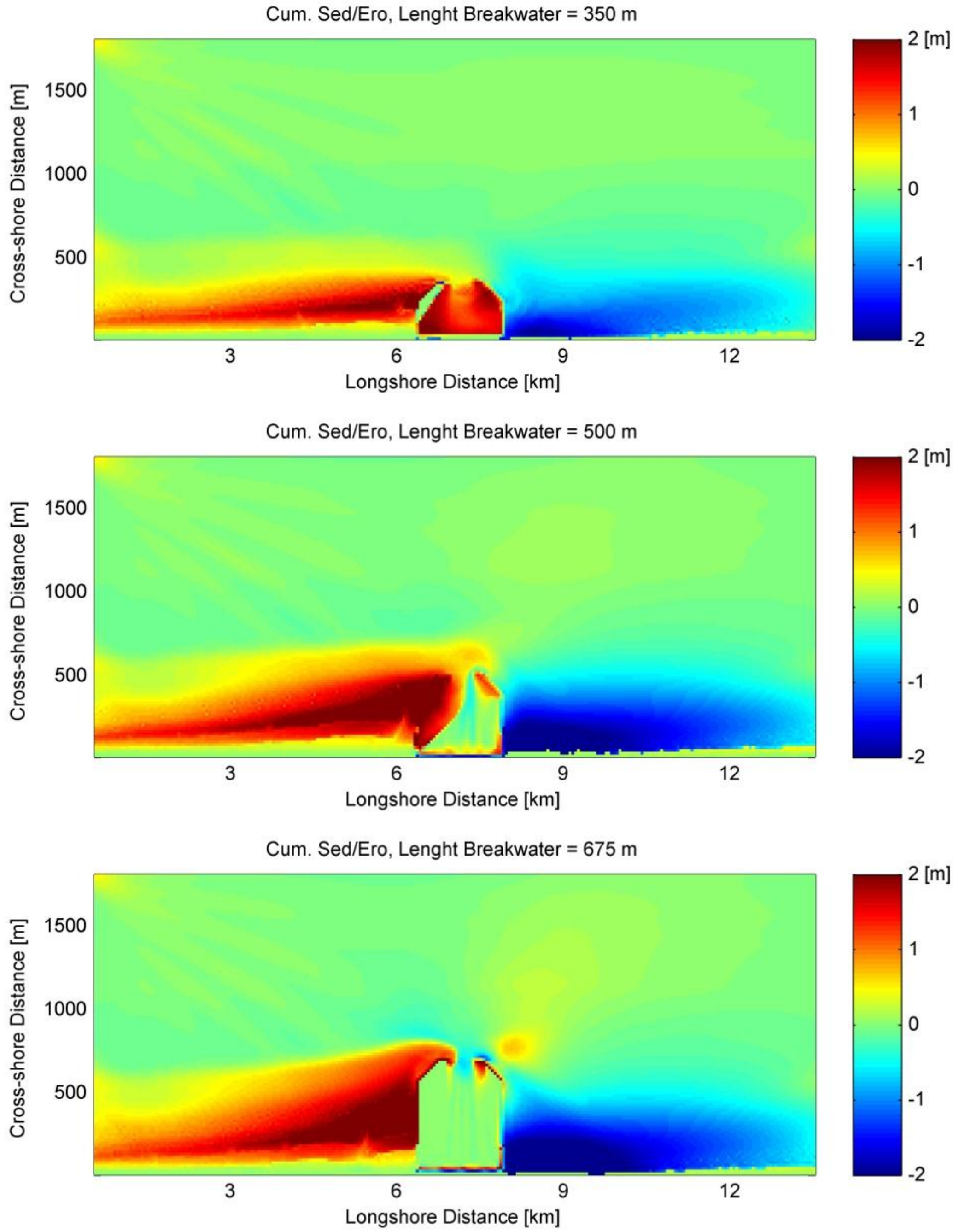


Figure 4-10 Three plots of the cumulative sedimentation/erosion at the end of the calculation. The top plot shows the result in case of a 350 m long breakwater, the middle in case of a 500 m long breakwater and the bottom one in case of a 675 m long breakwater.

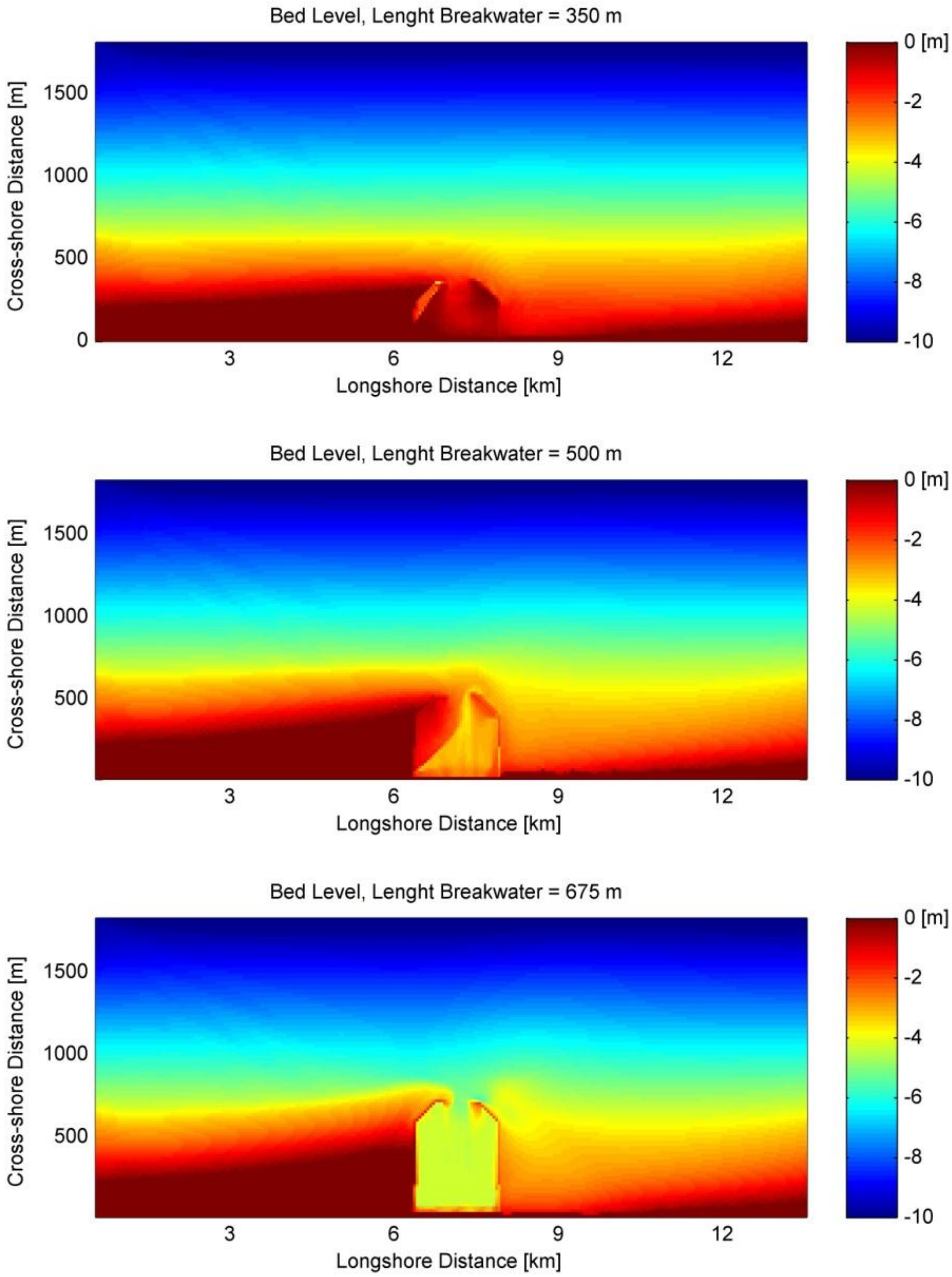


Figure 4-11 Three plots of the bed level at the end of the calculation. The top plot shows the bed level in case of a 350 m long breakwater, the middle in case of a 500 m long breakwater and the bottom one in case of a 675 m long breakwater.

Conclusion
The conclusion regarding the influence of the length of the breakwater on the bypassing and the bed level in front of the harbour can be summarized as follows:

- The largest harbour maintains a sufficient depth for a longer period of time. After some time, the bed level will however decrease and this is likely to be substantial. The smallest harbour gives a limited amount of depth decrease in front of the harbour which even to comes to a hold. This is caused by the significant wave influence due to the limited depth in front of the harbour. The smallest harbour however also experiences significant sedimentation inside the harbour, making dredging still required..

- From a coastal zone manager point of view it is evident that the shorter breakwaters give the best results. They block less sediment so the updrift erosion is a lot less. This is enlarged by the wave influence in front of the harbour. This limits accretion and increases the bypassing. Bypassing is only very limited in case of the larger harbours. In case the harbour which is as large as the width of the surf zone, a large part of the sediment is transported into the harbour instead of being bypassed. For the largest harbour the bypassing is far from being initiated.

Although there is quite some sedimentation in the harbour in case of the smallest harbour, it is still an interesting case to assess on a longer time-scale. If dredging is unavoidable, this still makes an interesting case since it is easier to dredge in the harbour than in the surf zone.

4.2.2. Assessment of the influence of the wave angle

The influence of wave angle is also assessed. Therefore five calculations have been done with varying wave angle. The following settings are applied:

- Wave height: 1 m;
- Wave angle: 15^o, 30^o, 45^o, 60^o and 75^o;
- Wave period: 7 sec;
- Length breakwater: 500 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: semi-streamlined.

The results are summarized in Table 4-3. The graphs can be found in Appendix E.2. It must be stated that the computed wave angles are the deep water wave angles. In the surf zone the wave angles have decreases significantly due to refraction and as discussed before Delft3D calculates too much refraction. Therefore the S/φ - curve shows the maximum transport at higher angles.

Property \ Variable	Wave Angle				
	15^o	30^o	45^o	60^o	75^o
Relative bed level [-]	1.03	0.88	0.81	0.75	0.73
Initial absolute bed level $[m]$	- 3.554	- 3.54	- 3.54	- 3.54	- 3.54
Absolute bed level after run $[m]$	-3.64	-3.11	-2.86	-2.66	-2.58
Relative bypassing [-]	0.28	0.24	0.26	0.37	0.35
Relative depth averaged velocity (flow contraction) [-]	1.21	1.24	1.21	1.21	1.81
Relative coastline progression updrift [-]	0.50	0.63	0.80	0.80	0.80
Cumulative transport updrift $[m^3]$	18 376	28 704	33 793	33 758	27 838
Cumulative bypassed sediment $[m^3]$	1 545	2 066	2 720	4 216	3 223

Table 4-3 Tabulated results of the calculations in which wave angle is varied. The values are the last measured results at the harbour entrance unless stated otherwise.

Bed level

The results don't differ as much as in the previous case. The main effects, caused by the varying wave angle, can be explained by means of difference in the amount of transport and the accretion angle at the updrift side. Wave angles around 45^o and 60^o cause the most amount of transport. Therefore the accretion at the updrift side of the harbour occurs faster than the cases with smaller wave angles. This speeds up the bypassing process.

The wave angle also has an effect on the amount of sediment which is required to achieve an equilibrium orientation of the coastline at the updrift side of the harbour. This amount is less in case the wave angles are higher because the angle of the equilibrium coastline with the initial coastline is higher. This can be seen in Figure 4-12 in which the cumulative accretion and erosion at the end of two runs with different wave angles are shown. Because less sediment is required to make the equilibrium orientation, the sediment is transport in front of the harbour quicker. The depth in front of the harbour thus also decreases faster in case of higher wave angles. In case of very high wave angles (75^o) the transport capacity due to the waves is too small to transport the sediment past the harbour. Therefore the depth decreases more for these wave angles. This also accounts for the smallest wave angles.

In all cases of significant accretion in front of the harbour, the reduction starts to level. In this case the new equilibrium has been established. This is at smaller depths for the highest wave angles due to the smaller transport capacity. The depth hardly decreases or even increases in case of very small wave angles (15^o). The coastline progression is nowhere near the tip of the breakwater. The harbour entrance even experiences erosion due to in and outflow. The accretion inside the harbour is therefore the least in case of the smallest wave angle. After that it is the least for the highest wave angles. Waves with an angle of 45^o lead to the most amount of sedimentation in the harbour.

Relative velocity

The flow contraction doesn't differ much for varying wave angles. In all cases a clear spring-neap tidal cycle with large flow contraction during spring tide a small flow expansion during neap tide is visible.

Relative bypassing
Bypassing mainly depends on the depth in front of the harbour and the incoming amount of sediment. As explained above, the coastline orientation achieves an equilibrium profile faster in case of higher wave angles. Once this is established the bypass can fully develop. Therefore the bypassing can develop quicker in case of higher wave angles. Additionally, the higher wave angles give more transport so this speeds up the process.

The results of the calculation with a wave angle of 15^o also gives an unexpected high relative bypassing. This is however expected to be caused by the erosion in the harbour entrance which contributes to the amount of transport measured in front of the harbour. Due to the small littoral drift this has quite an influence.

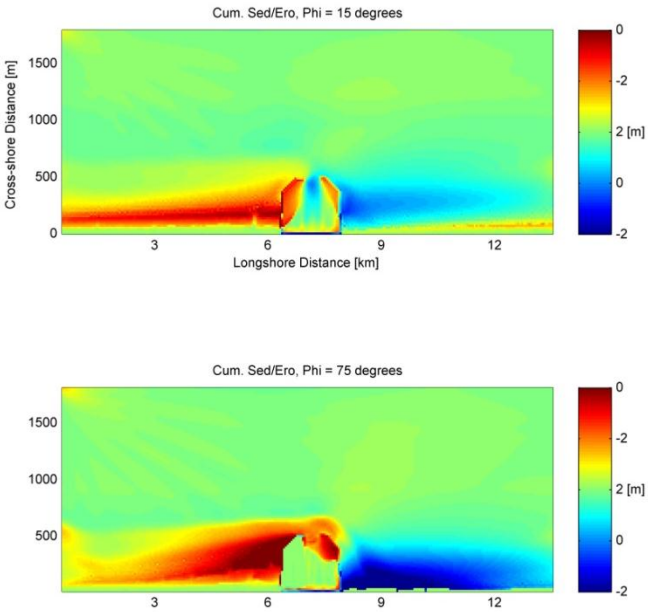


Figure 4-12 Angle of the coastline in case of a run in which a wave angle of 15^o is applied (top plot) and in case a wave angle of 75^o is applied (bottom plot).

Conclusion
The conclusion regarding the influence of the deep water wave angle on the bypassing and the bed level in front of the harbour can be summarized as follows:

- Very small wave angles (15^o and 30^o) give less sediment transport so it takes longer before the updrift accretion causes a decrease of the depth in front of the harbour. Eventually, the depth will however decrease. Higher waves angles give more transport. In combination with the fact that less sediment is required to achieve the equilibrium orientation at the updrift coastline, this leads to a quicker decrease of the depth in front of the harbour in case of higher wave angles. After some time the decrease in depth comes to a hold; the new equilibrium has been established. Due to the small transport capacity this is at smaller depths for the highest wave angles (60^o and 75^o).

- Due to the faster decrease of the depth, the higher waves also give more relative and absolute bypassing. Waves with a very small angle also give quite a relative bypass. This is caused by the erosion in the harbour entrance. The eroded sediment contributes to the measured bypassed sediment and this has, due to the small littoral drift, quite some influence. The absolute bypassing is however still the least.

Waves with higher angles reach the new equilibrium depth in front of the harbour quicker because they cause more sediment transport and require less sediment to achieve an equilibrium orientation at the updrift side of the harbour. The equilibrium depth is smaller for the waves with the highest angles (60^o and 75^o) due to the smaller transport capacity. Waves with the smallest angles (15^o and 30^o) give less sediment transport due to which it takes longer before the depth has decreased in front of the harbour. Initially this is thus a promising case. Eventually the equilibrium depth will be reaches and this will also be small due to the small transport capacity. As soon as the equilibrium depth has been reached the bypass can fully develop. The bypass thus develops quicker for waves with large angles. Waves with an angle of 45^o are the most promising on the long term since the depth decrease is relatively low and the bypass can grow fast if the equilibrium depth is reached.

4.2.3. Assessment of the influence of the wave height

The influence of wave height is also assessed. Therefore one extra calculation has been done with a larger wave height. The following settings are applied:

- Wave height: $1\ m$ and $1.5\ m$;
- Wave angle: 45^o;
- Wave period: $7\ sec$;
- Length breakwater: $500\ m$;
- Simulation time: $1\ month$ $(2\ years)$;
- Breakwater geometry: semi-streamlined.

The results are summarized in Table 4-4. The graphs can be found in Appendix E.3.

Property \ Variable	Wave height	
	$\mathbf{1\ m}$	$\mathbf{1.5\ m}$
Relative bed level [-]	0.81	0.77
Initial absolute bed level $[m]$	- 3.54	-3.54
Absolute bed level after run $[m]$	-2.86	-2.72
Relative bypassing [-]	0.26	0.59
Relative depth averaged velocity (flow contraction) [-]	1.21	1.33
Relative coastline progression updrift [-]	0.80	0.90
Cumulative transport updrift $[m^3]$	33 793	99 061
Cumulative bypassed sediment $[m^3]$	2 720	34 455

Table 4-4 Tabulated results of the calculations in which the wave height is varied. The values are the last measured results at the harbour entrance unless stated otherwise.

Bed level

The decrease in depth in front of the harbour occurs a lot faster in case of higher waves because the littoral drift is a lot higher due the higher wave influence. The higher wave influence also leads to a hold in the depth decrease. Once it is shallow enough, there is sufficient wave breaking to more or less stop the depth reduction, see Figure 4-13. In the base case the spring-neap tidal cycle is clearly visible. Initially the depth decreases at all times, it only goes faster in case of spring tide. Eventually, the depth has reduced sufficient to induce erosion in case of spring tide; the new equilibrium has been achieved.

The higher waves also have a negative effect; there is a lot of sedimentation inside the harbour because the harbour entrance is now in the surf zone. The depth in front of the harbour might come to a hold which limits the required amount of dredging in the harbour entrance but dredging is most definitely required inside the harbour.

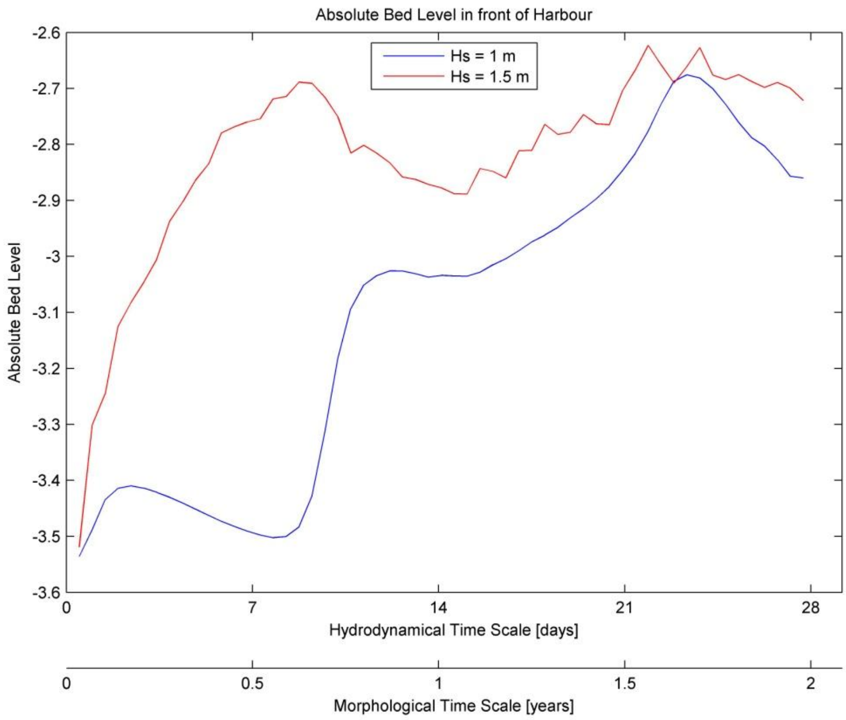


Figure 4-13 Plot of the relative bed level in front of the harbour in time in case of 1 m high waves and 1.5 m high waves are modelled.

Relative velocity

The relative velocity in front of the harbour is, in accordance with the above statements, a lot higher in case of higher waves. There is much less influence of the spring-neap tidal cycle because the wave influence on the current is dominant over the tidal influence. The flow velocities are therefore higher for the case $H_s = 1.5$ and so are is the flow contraction.

Relative bypassing

There is a large difference in the relative bypassing process in the results of the two runs. Figure 4-14 shows a plot of the two relative transports in front of the harbour. The relative transport in case of higher waves is much higher than for lower waves. That is mainly caused by the fact that the updrift accretion occurres a lot faster for higher waves. Therefore the coastline orientation is sooner in an equilibrium state allowing bypassing to occur sooner. The higher waves cause a lot more sediment transport and this gives a lot higher bypassing as well. The depth doesn't decrease any more so the same amount of sediment ends up in front of the harbour for both cases. Since the sediment transport is a lot more for the higher waves, the percentage that settles in front of the harbour is lower and the bypass thus higher, related to lower waves. Additionally higher waves break further offshore and thus give a wider surf zone. The lengths of the breakwaters are the same so blockage coefficient decreases. Therefore is more sediment is bypassed. Finally there is more wave-induced turbulence in front of the harbour. Combined with the higher flow velocities and contracting this gives more transport capacity in front of the harbour. Therefore the relative bypassing is a lot more than for smaller waves. The wave force thus has a significant influence on the bypassing process, as is stated earlier.

Conclusion

The conclusion regarding the influence of the deep water wave height on the bypassing and the bed level in front of the harbour can be summarized as follows:

- The depth reduction occurs faster in case of higher waves but it eventually comes to a hold. This also seems to be the case for smaller heights but it takes longer to develop this new dynamic equilibrium. Higher waves however do lead to a lot more sedimentation inside the harbour, making dredging still required.
  From the harbour point of view, higher waves thus have a positive influence on the sediment bypassing process but a negative influence in the processes inside the harbour. Navigation could also be a lot more difficult in case of higher waves.

- The relative bypassing increases significantly in case the wave height increases. Higher waves cause more transport, creating the equilibrium orientation quicker. Therefore the bypass starts sooner. The amount of sediment required to create the new equilibrium is the same for both cases. It is however a smaller percentage of the incoming transport in case of higher waves. Therefore the bypassed percentage is higher. Higher waves also cause more turbulence at breaking and a higher flow velocity and contraction. Higher waves also break further offshore and thus decrease the blockage coefficient of the breakwater. This all contributes to more relative and absolute bypassing in case of higher waves.

Higher waves cause a larger littoral drift. Therefore the depth in front of the harbour reduces quicker and therefore the bypass grows quicker. The higher wave energy does not lead to larger new equilibrium depth in front of the harbour. The surf zone is also wider in case of higher waves. Therefore the harbour blocks less sediment, increasing the bypass. The bypass also increases quicker due to the higher turbulence in case of higher waves. A large downside to higher waves is that there is a lot more sedimentation in the harbour. The bypass properties are thus promising but the navigational properties aren't.

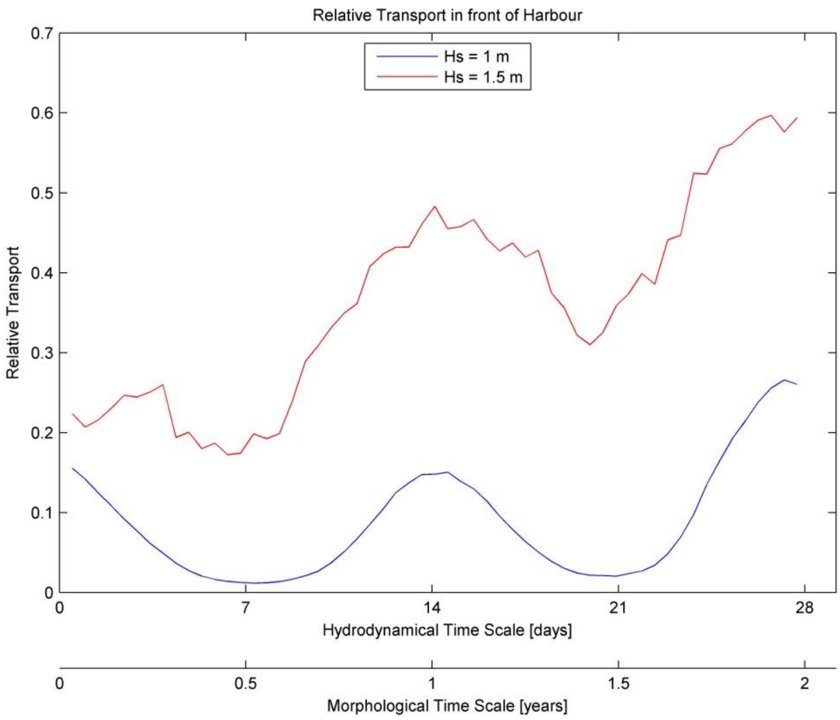


Figure 4-14 Plot of the relative transport in front of the harbour in time in case of 1 m high waves and 1.5 m high waves are modelled.

4.2.4. Assessment of the influence of the geometry of the breakwaters

The influence of breakwater geometry is also assessed. The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 500 m and 350 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: variable.

The following geometries are assessed:

1. Semi - streamlined, $L_{BW} = 500\ m$;
2. Semi - Streamlined with spurs, $L_{BW} = 500\ m$;
3. Streamlined, $L_{BW} = 500\ m$;
4. Streamlined with spurs, $L_{BW} = 500\ m$;
5. Semi - streamlined, $L_{BW} = 350\ m$;
6. A main breakwater covering the secondary breakwater, $L_{BW} = 350\ m$.

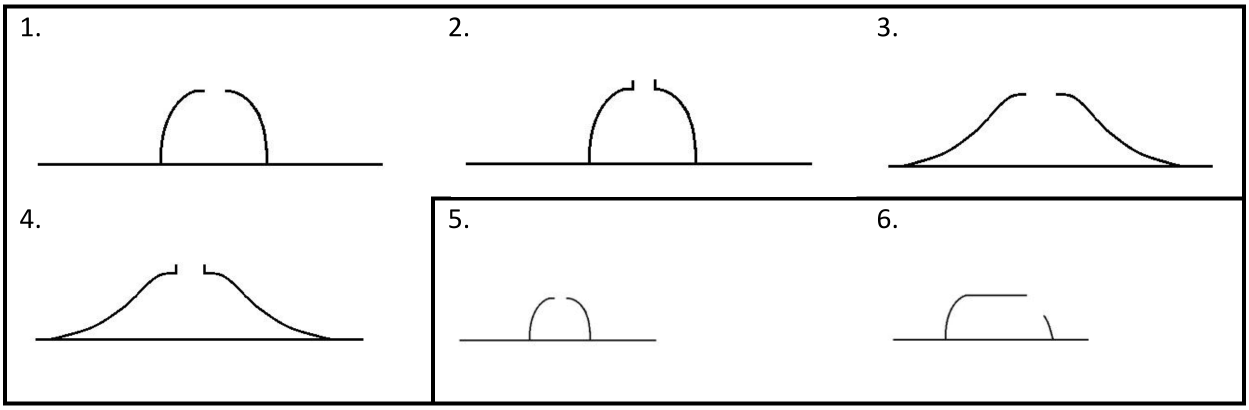


Figure 4-15 Different breakwater geometries that are assessed.

The results are summarized in Table 4-5. The graphs can be found in Appendix E.4. Run 1 and 5 of this batch are used as reference material. The results at the end of the simulation can be seen in the below table. Numbers 1 to 6 corresponds with the above list of modelled breakwater geometries.

Property \ Variable	Breakwater geometry					
	1	2	3	4	5	6
Relative bed level [-]	0.81	0.68	0.79	0.68	0.84	1.1
Initial absolute bed level $[m]$	- 3.54	- 3.54	- 3.54	- 3.54	- 2.37	-2.37
Absolute bed level after run $[m]$	-2.86	-2.41	-2.77	-2.39	-1.98	-2.59
Relative bypassing [-]	0.26	0.42	0.54	0.61	0.68	0.31
Relative depth averaged velocity (flow contraction) [-]	1.21	1.30	1.24	1.38	1.30	1.27
Relative coastline progression updrift [-]	0.80	0.80	0.67	0.70	0.86	0.86
Cumulative transport updrift $[m^3]$	33 793	41 102	38 818	38 950	34 925	33 457
Cumulative bypassed sediment $[m^3]$	2 720	5 038	7 285	7 666	11 425	5 186

Table 4-5 Tabulated results of the calculations in which the breakwater geometry is varied. The values are the last measured results at the harbour entrance unless stated otherwise.

Bed level

There are quite some differences between the results of the different runs considering the bed level. Run 2 to 4 should be compared to run 1 and run 6 should be compared to run 5 because the extension of the harbour differs.

Run 2 and 4 include the spurs and this seems to have a negative effect on the bed level. The spurs cause a larger decrease of the bed level than without the spurs. This could be compared to the runs with longer breakwaters. The breakwaters with the spurs block a larger part of the sediment transport, so more sediment settles updrift. Due to the spurs it also settles closer to the harbour entrance so it is more likely to end up in front of the harbour. This is enlarged by the fact that two spurs are applied. This prevents bypassing of the sediment and enlarges the sedimentation. This causes a larger reduction of the depth in front of the harbour for the cases with the spurs. The extra reduction of the depth does lead to more wave breaking and thus more transport capacity.

The spurs were expected to cause vortices in front of the harbour entrance. They can however not be computed with this model because the vortices occur on a scale smaller than the applied grid cells. Therefore this cannot be accounted for. This leads to a larger depth decrease instead of less due to the spurs.

The streamlined breakwaters (run 3) result in a very little extra depth reduction related to the semi-streamlined breakwaters. The streamlined shape thus has a limited effect on the depth in front of the harbour. As can be seen further on, it does give quite some extra bypassing which makes it an interesting situation.

The asymmetric breakwater (run 6) shows almost no decrease in depth. The bed level does vary some; the depth reduces during neap tide and increases during spring tide. The variation is confined in decimetres so this does not affect the serviceability of the harbour much. If the results would also show positive features considering bypassing, this would be a perfect case.

Relative velocity
As predicted the streamlined shape of the breakwater causes little more flow contraction than the semi-streamlined geometry. Adding the spurs on the breakwater also increased the flow contraction. This is caused by the fact that the breakwater with spurs extent 33 m further in the surf zone. Therefore they cause more flow contraction. The depth in front of the harbour also decreases quicker so the effect of the decrease of the relative length due to the updrift accretion on the flow contraction is compensated by the increasing wave influence. The combination of streamlined breakwater and spurs therefore gives the most amount of flow contraction.

The flow contraction for the smallest semi-streamlined breakwater (run 5) increases due to the increasing wave influence. The asymmetric breakwaters (run 6) show initially significantly less flow contraction. It even shows a decrease in the flow velocity at a certain point. That occurs during neap tide. The measuring point is located at the harbour entrance. The asymmetric shape causes the flow to follow the shape of the breakwater. In front of the harbour the flow experiences flow expansion, causing the decrease in the velocity. This decrease in the velocity on its turn leads to a negative gradient in the sediment transport rate and thus accretion.

Relative bypassing
The additional depth reduction in case spurs are applied (run 2 and 4) cause additional transport capacity due a higher wave influence. The spurs were intended to cause more vorticity. This would than prevent sediment from settling. Therefore the bypassing capacity would be higher. As explained, the expected vorticity could not be modelled due to the grid size. Therefore the depth decreases more instead of less. This does produce the extra bypass capacity, but that occurs at the expense of the serviceability of the harbour.

The streamlined breakwaters (run 3) also result in more bypassing. The streamlined breakwaters without the spurs hardly had any effect on the bed level, but do lead to a large increase in the bypassing capacity. That is caused by three reasons. At first the shape of the breakwaters is already somewhat in the shape of the equilibrium orientation. Therefore less sediment is required to achieve this orientation and so the bypass can develop quicker. Secondly, the flow contraction is somewhat higher due to the streamlined shape. This enlarges the bypass capacity. Thirdly, less sediment is transported into the harbour which can additionally be bypasses. All in all this is a promising case. A downside is that the harbour breakwaters are more expensive to construct. This could however be counteracted by the lower maintenance costs. The combination of streamlined breakwaters and spurs give the most amount of bypassing but, as explained, the serviceability of the harbour also decreases significantly. The relative bypassing in case of the asymmetric breakwaters is significantly less than the reference case, run 5. The sediment which reaches the offshore tip of the main breakwater is transported into the harbour. Compared to the reference case there is a little less sedimentation in the harbour, but maintenance dredging is most definitely still required. Both effects result in less bypassing in case of asymmetric breakwaters.

Conclusion

The conclusion regarding the influence of the breakwater geometry on the bypassing and the bed level in front of the harbour can be summarized as follows:

- A streamlined geometry of the breakwaters has a positive effect on the bypassing process. The streamlined breakwaters create more flow contraction which enlarges the bypass capacity. Secondly, less sediment is required to make the updrift equilibrium orientation so the bypass can develop quicker. Thirdly, less sediment is transport into the harbour. This can consequently be bypassed. The streamlined geometry has a minimal negative effect on the depth in front of the harbour.

- The spurs at the tip of the breakwater would cause vortices which prevent sediment from settling. They can however not be computed with this model because the vortices occur on a scale smaller than the applied grid cells. Therefore this cannot be accounted for. Now the spurs only catch more sediment and lead to a larger decrease of the harbour serviceability. This consequently leads to an increase of the bypass capacity.

- An asymmetric geometry of the breakwaters has a positive effect on the bed level in front of the harbour. This depth even increases due to the high transport capacity during spring tide. There is however less bypassing because the sediment which reaches the offshore tip of the main breakwater is transported in to the harbour.

4.2.5. Assessment of the most promising scenarios on a longer time scale

The most promising scenarios are assessed on a longer time scale. A scenario is promising if it shows minor decrease of the depth and a significant relative bypassing. From the previous paragraphs it follows that the following cases are promising and these are therefore assessed on a longer time scale

1. In case of breakwaters smaller than the width of the surf zone
2. In case of streamlined breakwaters.
3. In case of breakwaters equal to the width of the surf zone

The third one is added as reference case.

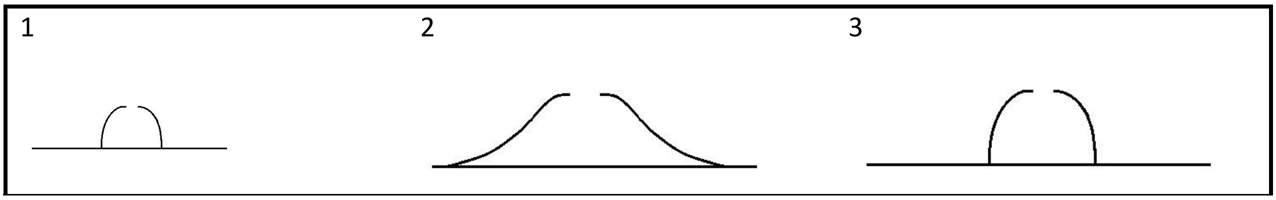


Table 4-6 Different breakwater geometries that are assessed on a longer time scale.

These scenarios are therefore assessed on a longer time scale. As a comparison, the base-scenario is also calculated on a longer time scale. The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o ;
- Wave period: 7 sec;
- Length breakwater: 350 m and 500 m;
- Simulation time: 2 $months$ (4 $years$);
- Breakwater geometry: semi-streamlined and streamlined.

The results are summarized in Table 4-7. The graphs can be found in Appendix E.5.

Variable / **Property**	**Length breakwater**	**500 m**	**350 m**	**500 m**
	Geometry	semi-streamlined	semi-streamlined	streamlined
Relative bed level		0.75	0.80	0.80
Initial absolute bed level [m]		- 3.54	- 2.37	- 3.54
Absolute bed level after run [m]		-2.64	-1.90	-2.78
Relative bypassing [-]		0.71	0.93	0.84
Relative Depth averaged velocity (flow contraction) [-]		1.71	1.37	1.57
Relative coastline progression updrift [-]		0.77	0.86	0.70
Cumulative transport updrift [m^3]		53 096	58 559	60 899
		(33 793)	(34 925)	(38 818)
Cumulative bypassed sediment [m^3]		8 380 16%	28 053 48 %	21 867 36 %
		(2 720)	(11 425)	(7 285)

Table 4-7 Tabulated results of the calculations in which the most promising cases are assessed on a longer time scale. The values are the last measured results at the harbour entrance unless stated otherwise. The values between brackets in the lower two rows are the values after 2 years. The percentages are the cumulative bypassed sediment amounts, relative to the updrift amount.

The results of the longer runs are more and more affected by the boundary effects. These have a large influence on the bed level in the upper left corner of the domain, as can be seen in Figure 4-17 and 4-18. Therefore the processes at the coastline are affected as well. It is however still a possible to distinguish significant differences in the results of the three calculations. These differences are explained below.

Bed level
In all three cases a new equilibrium depth is reached. This is however at a larger depth in case of a larger harbour. Initially it was assumed that the increasing wave breaking is the main reason for smoothing out the depth decrease. If that would be case the new equilibrium depth should be similar, regardless of the initial depth because the wave driving force in these three calculations would only be equal if the depth at breaking is similar. The wave properties namely don't differ. The different equilibrium depth in case of harbours of different size thus indicates the presence of another driving force in front of the harbour. This is found in the absolute longshore and the cross-shore velocity.

The absolute longshore velocity is around $10 - 20$ % higher in case of the larger harbours due the larger flow contraction. The updrift coastline has eventually progressed 77 % related to the 86 % for the smallest harbour. In absolute numbers the difference is even larger, respectively 385 m out of 500 m and 301 m out of 350 m. Therefore the flow contraction is higher in case of the larger harbours. This increases the bypass capacity which leads to a larger equilibrium depth.

The larger equilibrium depth is also causes by the difference in the cross-shore velocity. This is much higher in case of larger harbour. This can be up to 100 % higher in case of the streamlined breakwaters and 50 % in case of semi-streamlined breakwaters, both related to the cross-shore velocity which is measured in case of the smallest harbour. This is caused by the larger inner area of the harbour due which more water needs to flow in and out at the change of tide. This is the most in case of the streamlined breakwaters. Therefore the cross-shore velocities are the highest as well. This also prevents the sediment to settle in front of the harbour. For these two the reasons the new equilibrium depth is larger in case of harbours which are larger than the width of the surf zone.

In case of both semi-streamlined geometries, there is significant sedimentation inside the harbour. The depth in front of the harbour might come to a new equilibrium, the serviceability of the harbour still decreases due to the significant sedimentation inside the harbour. This is much less in case of the streamlined breakwater. There is still some sedimentation in the harbour but not nearly as much as the harbour with the same size but semi-streamlined breakwaters. The streamlined geometry thus has a positive contribution to the serviceability of the harbour.

Relative velocity
In all cases the relative velocity follows the trend as can be seen in the 2-year runs. The relative velocity increases as the depth in front of the harbour decreases.

Relative bypassing
The relative bypassing is measured relative to the updrift longshore sediment transport. This is supposed to be unaffected by the accretion updrift of the harbour. On the longer time scale this is however not the case. The transport capacity updrift of the harbour decreases because the coastline rotates somewhat towards the incoming waves. This affects the values for the relative bypassing since the amount of transport measured updrift is divided by the amount of transport updrift.

Therefore to compare the bypass capacity of these cases on the longer time scale, one could look to the cumulative amount of bypasses sediment. The undisturbed transport capacity far updrift (outside the interior domain) is the same because the same hydrodynamic forcing is taken into account. Therefore the absolute cumulative values of bypassed sediment can also be used to assess the bypass capacity of these cases on a longer time scale. This plot is depicted in Figure 4-16.

The sudden drop in the lines is caused by the fact that the results for the longer runs are achieved by continuing on the bathymetry of the initial runs. Afterwards the results are combined to give the result on the longer time scale. This works fine for all output graphs except for the cumulative sediment transport plot. At the new run this starts at zero again. That doesn't mean these plot is useless. The end values just need to be added to each other, which is already done in Table 4-7.

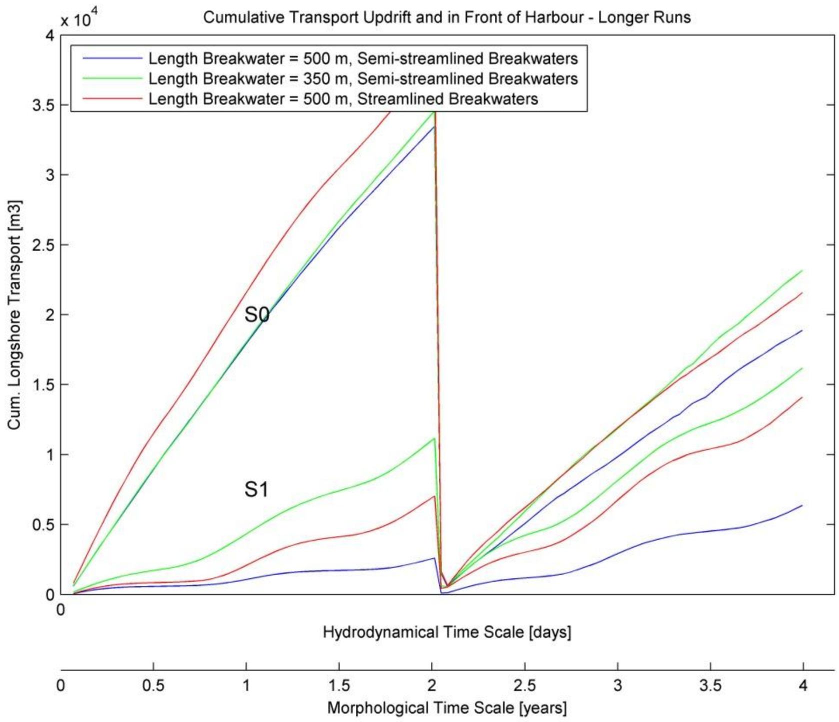


Figure 4-16 Plot of the cumulative Transport updrift and in front of the harbour for the three cases. The sudden drop is caused by the fact that the extra 2 years is calculated as a separate run which starts with the end bathymetry of the initial run.

These numbers show that the amount of bypassed sediment increases in all cases. The bypasses amount is still largest for the smallest harbour. The streamlined geometry of the breakwaters also almost shows a similar amount. Both are significantly larger than the semi-streamlined breakwaters. As discussed in Paragraph 4.2.4 the streamlined breakwater is especially interesting. There is less sedimentation in the harbour, the depth in front of the harbour decreases some but the new equilibrium depth is established at depth of around 2.6 m, equal to the semi streamlined geometry and the bypass capacity is almost equal to the harbour smaller than the width of the surf zone.

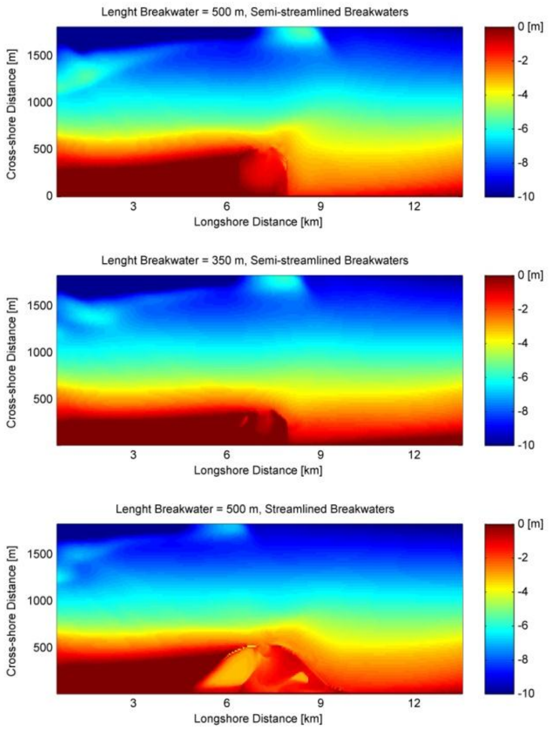


Figure 4-17 Three plots of the bed level at the end of the three runs. The bed level inside the harbour is in almost all three cases just as the updrift side.

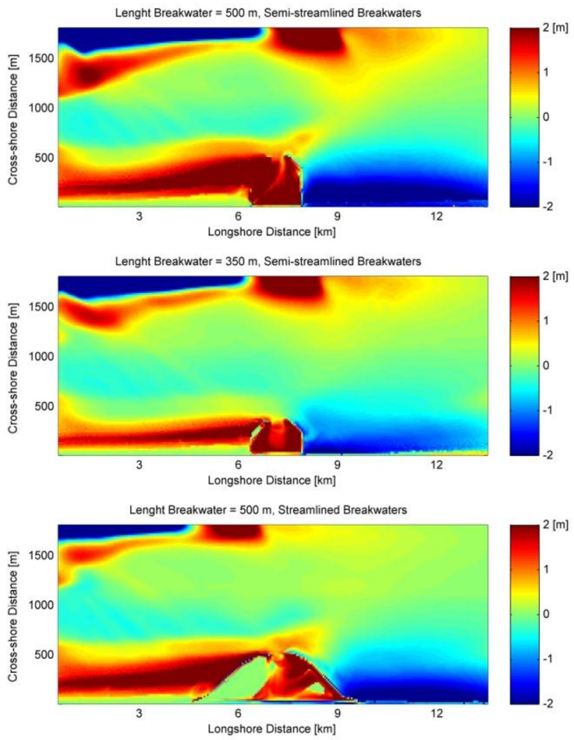


Figure 4-18 Three plots of the cumulative sedimentation/erosion at the end of the three runs. The locations of the harbours are hard to find due to sedimentation inside the harbour.

Conclusion

The conclusion regarding the most promising cases on a longer time scale can be summarized as follows:

- The results of the longer runs are more and more affected by the boundary effects. These effects have a large influence on the bed level in the upper left corner of the domain. Therefore the processes at the coastline are affected as well but still significant differences can be distinguished between the results of the three runs.

- In all cases the depth in front of the harbour approached a new equilibrium. This is at larger depths for the larger harbours because the longshore and cross-shore velocities are higher. The longshore velocity is higher because the flow contraction is higher and the cross-shore velocity is higher because more water flows in and out the harbour due to the larger inner area in case of larger harbours. These larger flow velocities prevent further sedimentation of the entrance channel. The sedimentation in the harbour is significant for the semi-streamlined geometry of the breakwaters but a lot less in case of the streamlined geometry.

- The amount of bypasses sediment continuous to grow, also on longer time scale. This especially accounts for the smallest breakwater with a semi-streamlined geometry and the longer breakwaters with a streamlined geometry. Combined with the good properties of the equilibrium depth in front of the harbour with the streamlined geometry, this gives very promising results on the ability of maximum bypassing in combination with sufficient serviceability of the harbour.

4.2.6. Assessment of the influence of a dredged channel and the tide

Early in the process a few assumptions have been made at choosing the most promising archetypes. The tide is assumed to be necessary for promising results and a dredged channel is assumed to have very negative effect on the possibility for bypassing. These assumptions have been checked. The results of a run with a dredged channel and the result of a run with a smaller tidal amplitude are compared to the base-scenario.

The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 500 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: semi-streamlined;
- Depth dredged channel: 7 m;
- Tidal amplitude: 1.33 m and 0.67 m at spring tide;

The results at the end of the simulation can be seen in the below table. The graphs can be found in Appendix E.6.

Property \ Variable	Base-scenario	Including dredged channel	Lower tidal amplitude
Relative bed level 40 in front of harbour entrance [-]	0.81	0.48	0.71
Initial absolute bed level [m]	- 3.54	-7.00	-3.54
Absolute bed level after run [m]	-2.86	-3.33	-2.53
Relative bypassing [-]	0.26	0.13	0.09
Relative depth averaged velocity (flow contraction) [-]	1.21	1.08	0.98
Relative coastline progression updrift [-]	0.80	0.77	0.83
Cumulative transport updrift [m^3]	33 793	33 847	31 784
Cumulative bypassed sediment [m^3]	2 720	910	720

Table 4-8 Tabulated results of the calculations in which assumptions are checked. The values are the last measured results at the harbour entrance unless stated otherwise.

Bed level

As expected the dredged channel has a large influence on the processes around the breakwater. Almost all the sediment which starts to bypass settles in the channel. Therefore the depth in the entrance channel decreases rapidly from 7 m to 3.5 m. The depth decrease stops when the bed level in the channel is levelled with the surrounding bed level. Once it is 3.5 m deep, the sediment starts bypassing the harbour. There is also quite some sedimentation in the harbour, close to the entrance because the harbour initially has the same depth as the channel.

The assumption that the tide is required to generate sufficient transport capacity in front of the harbour is just too. The depth decreases more in case of smaller tidal currents velocities. The smaller tidal current velocities also lead to smaller bypass capacity due to which the depth in front of the harbour decreases more than the in the base-scenario, even with a smaller amount of incoming sediment.

Flow contraction

The results of the run with the entrance channel do not show flow contraction. As expected these results show flow expansion instead. Due to the larger depth the flow has more space in front of the harbour. Therefore the flow velocity decreases. Eventually the channel gets filled up and the flow expansion turns into flow contraction. The smaller tidal current velocities also give less flow contraction than the base-scenario. That is partly related to the fact that in the beginning the depth has decreased less for this case. Therefore there is less wave breaking in relation to the base-scenario. However, the depth decreases faster than the base-scenario so the wave influence increases and so does the contribution to the flow velocity.

Relative bypassing

The relative bypassing has partly been explained in the above sections. The bypassing is almost zero for the case with the dredged channel. Eventually the channel is filled up and a bypass is initiated quite rapidly. It still remains a lot less than the base-scenario. In case of a smaller tidal amplitude the amount of bypassed sediment even remains lower than this

Conclusion

The conclusion regarding the influence of a dredged channel and the tidal amplitude on the bypassing and the bed level in front of the harbour can be summarized as follows:

- A dredged entrance prevents sediment bypassing until it is levelled with the surrounding bed level. From than onwards a bypass is initiated. The flow experiences expansion until the channel is levelled. From than onwards there some flow contraction. The depth in the channel and in the harbour decreases significantly so dredging in the channel and in the harbour is required to maintain the serviceability of the harbour.

- A lower tidal influence decreases the bypass capacity. Therefore the depth decreases more than the base-scenario. This leads to an increase of the wave influence and therefore there is some bypassing. This is still very limited.

That concludes the analysis of the model results. The conclusions are summarized in the next chapter.

CHAPTER 5
CONCLUSION

Now all calculations have been done, a conclusion on the research question will be formulated. The goal of this research is to find conditions for which the sediment bypassing around the harbour is optimized in combination with limited influence on the navigational function of the harbour. It has not been expected to find a perfect match but to find a situation in which the harbour has less negative effect on the sediment patterns and vice versa. This could be seen as Integrated Coastal Zone Management in which the interests of the harbour manager and the coastal zone manager are both addressed and in a sustainable way.

5.1. General conclusion

This paragraph gives a general conclusion on the ability within the chosen archetype to optimise the bypass capacity in combination with a longer serviceability of the harbour. The serviceability means the variation of the depth in and in front of the harbour. The conclusion per assessed parameter is given in the next paragraph.

This research has showed that the bypass capacity can only be optimised under the presence of significant hydrodynamic forcing, e.g. wave forces and tidal forces.

This research has also showed that the bypass around a harbour can be optimized in case the harbour is shorter than the width of the surf zone. The depth in front of the harbour is not that large (in the order of 2 m) due to which there is quite a wave-driven current and wave-induced turbulence in front of the harbour. Combined with the small blockage coefficient, this leads to a large capacity to bypass a significant part of the incoming amount of sediment. The large amount of relative bypassing has a positive effect on the downdrift coastline because the bypassed sediment counteracts the erosion. However, although the depth in front of the harbour decreases only a few decimetres, the depth in the harbour decreases quite significantly. This is caused by the fact that the harbour is shorter than the width of the surf zone due to which there is quite a sediment transport into the harbour. The depth in front of the harbour may not decrease much; the depth in the harbour does decrease much. This situation therefore shows large capacity for bypassing but not so much on maintaining the serviceability of the harbour.

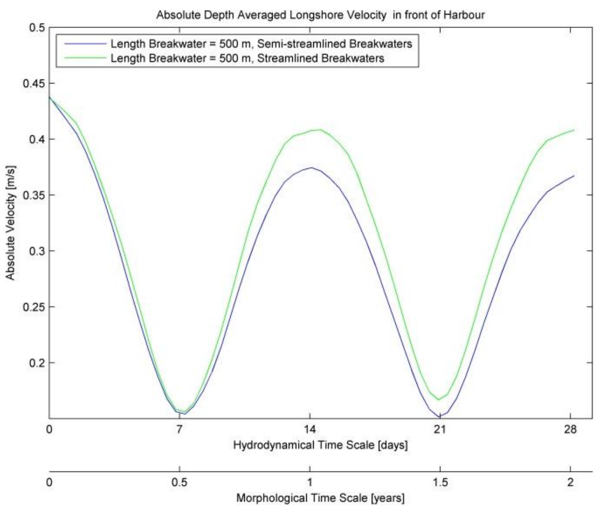
The serviceability of the harbour is maintained much better in case of streamlined breakwaters with an extension equal to or larger than the width of the surf zone. The streamlined shape causes a little more flow contraction in relation to the semi-streamlined shape which enlarges the bypass capacity. Although the depth reduces slightly more in front of the harbour for streamlined breakwaters than it does for semi-streamlined breakwaters; the geometry does cause much less sedimentation in the harbour. There are higher cross-shore velocities in the harbour entrance of a streamlined harbour than there are in a semi-streamlined harbour but there is still less transport of sediment into the harbour. This is not fully understood yet and it is therefore recommended to study this in a future research. The limited sedimentation in the harbour leads to a higher overall serviceability of the streamlined harbour than that of a semi-streamlined harbour with same extent in the surf zone.

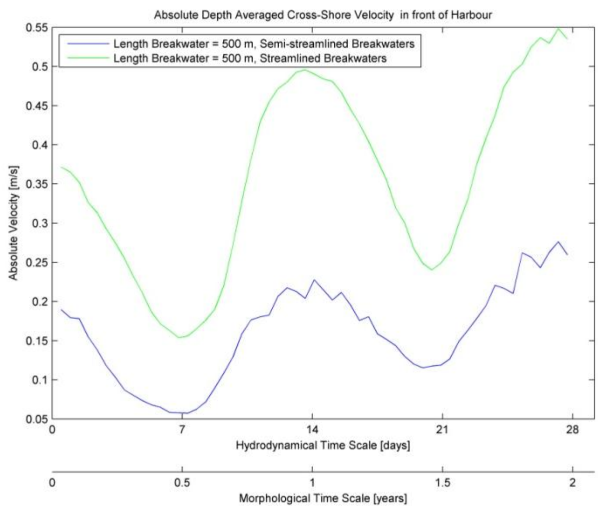
The smaller sedimentation in the harbour also has an effect on the bypass process because the sediment which is not transported into the harbour is transported beyond the harbour and therefore counteracts the downdrift erosion. The bypass is also initiated faster for the streamlined harbour than for the semi-streamlined harbour because the geometry of the streamlined harbour is close the new equilibrium orientation of the updrift coastline. Therefore less sediment is required to achieve this orientation due to which the bypass can be initiated faster.

The goal of this study was to find conditions for which the bypass capacity as well as the serviceability of the harbour was optimised. From the above it can therefore be concluded that this is possible. This is summarized in the below statement:

This research shows that a bypass friendly harbour is possible under certain circumstances.

These circumstances are a harbour with streamlined breakwaters and an extension equal to the width of the surf zone at a coastline with significant wave and tidal influence. The streamlined geometry of the breakwaters leads to higher longshore flow velocities in front of the harbour, related to a harbour equal in size but with semi-streamlined breakwaters. The higher flow velocities enlarge the bypass capacity. The high flow velocities, in combination with the extent of the breakwaters also lead to a limited decrease of the depth in front of the harbour and very little sedimentation in the harbour. Both the limited decrease of the depth in front of the harbour and the little sedimentation in the harbour have a positive effect on the amount of bypassing. The bypass is also initiated quicker than that in case of semi-streamlined breakwaters because less sediment is required to make the new coastline orientation in case of the streamlined breakwaters. The described effects can be seen in the below figures.





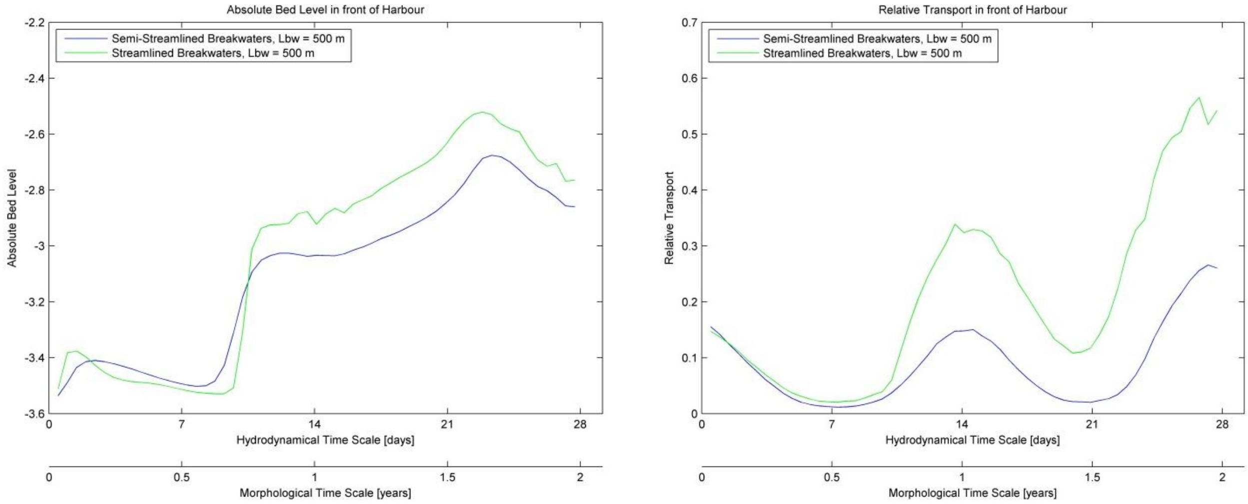


Figure 5-1 Absolute longshore velocity (top left), absolute cross-shore velocity (top right), absolute bed level (bottom left) and the relative bypassing (bottom right) for a harbour with semi-streamlined and streamlined geometry. The extension of both harbour is equal to the width if the surf zone. The applied hydrodynamical forcing is also equal.

It can thus be concluded that there are conditions at which the bypass capacity of sediment around a harbour can be optimized in combination with a limited influence on the navigational function of the harbour. So an Integrated Coastal Zone Approach is feasible.

5.2. Conclusion per parameter

The influence of each assessed parameter is explained below

- BREAKWATER LENGTH

  Considering the length of the breakwater, there is a dual conclusion. Breakwaters shorter than the width of the surf zone trap less sediment and therefore the bypass is initiated faster. Due to its limited extent in the surf zone, there is significant wave influence in front of the harbour. This avoids accretion and increases the bypassing. The depth reduction in front of the harbour is therefore limited. There is however also significant sedimentation in the harbour which makes dredging still required.

  The harbour longer than the width of the surf zone maintains sufficient depth in front of the harbour longer, but once the updrift coastline has progressed sufficient, the depth will decrease fast. This also gives a transport of sediment into the harbour. Larger breakwaters also block more sediment so there will be more erosion downdrift.

  Eventually all cases will lead to a new equilibrium depth in front of the harbour. This has been tested on a 4 year time scale for the harbour shorter than the width of the surf zone and for the harbour equal to the width of the surf zone. Compared to a smaller harbours the new equilibrium depth is higher for longer breakwaters because the flow contraction is bigger and the in- and outflow velocities are higher due to the larger inner area of the harbour. Both avoid sedimentation in front of the harbour and increase the bypass capacity. Therefore the new equilibrium is established at larger depths in front of the harbour.

- WAVE ANGLE
  The bypass capacity depends very much on the wave influence. The wave influence is the highest for waves with an offshore angle of 45^o and 60^o. Both larger and smaller wave angles give less transport capacity and therefore also less bypass capacity. This leads to smaller new equilibrium depths in front of the harbour compared to waves with an angle of 45^o.The equilibrium depth is achieved fastest for waves with the highest angles because less sediment is required to create the equilibrium orientation of the updrift coastline. Therefore the bypass is initiated quicker and thus the ability to create a new equilibrium in front of the harbour.

- WAVE HEIGHT
  Higher waves (1.5 m vs. 1 m) cause a larger littoral drift. Therefore the depth in front of the harbour reduces quicker and therefore the bypass grows quicker. The higher wave energy does not lead to a larger new equilibrium depth in front of the harbour compared to less wave energy. The surf zone is also wider in case of higher waves. Therefore the harbour blocks less sediment, increasing the bypass. The bypass also increases quicker due to the higher turbulence in case of higher waves. A large downside to higher waves is that there is a lot more sedimentation in the harbour. The bypass properties are thus promising but the navigational properties aren't.

- HARBOUR GEOMETRY - STREAMLINED VS. SEMI-STREMALINED GEOMETRY
  A streamlined geometry of the breakwaters has a positive effect on the bypassing process. The streamlined breakwaters create more flow contraction which enlarges the bypass capacity in relation to a semi streamlined geometry. Secondly, less sediment is required to make the updrift equilibrium orientation so the bypass can develop quicker. Thirdly, less sediment is transported into the harbour. This can consequently be bypassed. This is also tested on a 4 $year$ time scale. The depth in front of the harbour approaches an equilibrium depth of around 2.6 m, just as the semi-streamlined breakwaters of the same length. The amount of bypassed sediment is however much higher and there is also very limited amount of sedimentation inside the harbour. This combination makes this geometry of the breakwaters very effective.

- HARBOUR GEOMETRY - SPURS
  The spurs at the tip of the breakwater would cause vortices which could prevent sediment from settling. This effect can however not be computed with this model because the vortices occur on a scale smaller than the applied grid cells. Therefore this cannot be accounted for. Future research should take this in mind because the vorticity induced by the spurs could prevent sedimentation in front of the harbour and enhance the bypass capacity.

- HARBOUR GEOMETRY - ASYMMETRIC GEOMETRY
  An asymmetric geometry of the breakwaters has a positive effect on the bed level in front of the harbour. This depth even increases due to the high transport capacity during spring tide. There is however less bypassing because the sediment which reaches the offshore tip of the main breakwater is transported into the harbour.

- DREDGED CHANNEL AND SMALLER CURRENTS
  The assumptions on the influence of a dredged channel and the tidal currents are just. The dredged channel indeed prevents bypassing until it is levelled with the surrounding bed level. This would never be the case because in reality the channel would already be dredged before this is the case.
  A smaller current velocity decreases the transport capacity along the coastline and in front of the harbour. This leads to a new equilibrium depth in front of the harbour which is less than it is in case of a higher current velocity.

CHAPTER 6. RECOMMENDATIONS

A study like this cannot be finished without some recommendation for further research or improvements on this research. There are some general recommendations which can be made considering the results of this study. These are described in the first paragraph. There are some general recommendations on the applied model, which are described in the second paragraph. Thirdly the choice to apply Delft3D vs Unibest-CL+ as a tool to find an answer on the research question is also considered.

6.1. General recommendations

- The bypass capacity and the serviceability of a harbour with streamlined breakwaters with an extension equal to the width of the surf zone is better than that of a harbour with the same length but with semi-streamlined breakwaters. Not every aspect of that is fully understood and it is recommended to study the hydrodynamic processes around streamlined breakwater more thoroughly for two reasons:
    - The streamlined geometry of the breakwaters leads more flow contraction in front of the harbour than the harbour with a semi-streamlined geometry of the breakwaters. This enlarges the transport capacity due which it is expected that new equilibrium depth is also larger. This is however not the case and it is recommended to find out what the effect is of the higher flow contraction on the bypass capacity and thus on the new equilibrium depth.
    - The streamlined shape of the breakwaters also causes much less sediment that is transported into the harbour than there is in case of semi-streamlined breakwaters. This effect is not fully understood yet because it could also be argued that the larger inner harbour area for streamlined breakwaters causes higher cross-shore currents (which it does) and thus more import of sediment into the harbour. This is however not the case and it is recommended to study the hydrodynamical processes better to get more insight in the reason behind this. This could for instance be caused by a better following of the breakwater by the flow in case of streamlined breakwaters.

- Furthermore, the streamlined shape is chosen quite arbitrary. It must be possible to optimise the shape with relation to the bypass capacity and the sedimentation in and in front of the harbour, especially if the above defined processes are better understood. The width of the harbour entrance could also be a variable in that optimisation process.

- The underlying goal of finding an answer on the research question of this research is to minimize the costs. These costs are at one side costs for the maintenance dredging at the harbour and at the other side the costs for maintaining the coastline. The cost aspect has however not been included in this study because they are assumed to be very site-specific. It could be interesting to include the costs in finding the most optimal solution based on construction and maintenance costs and the impact of the structure on the coastline. It would therefore be required to make general assumptions considering the costs.

Some additional remarks have to be made regarding further research on this topic. These remarks are not expected to influence the conclusions, but they do give a more complete picture of the bypassing process.

The following parameters/scenarios are not assessed in this study and are worthwhile to do so:

- Asymmetric tide;
- Wave period;
- Wind in general;
- Grain size;
- Bathymetry;
- Spurs on the breakwaters.

By adding these a more complete picture could be drawn considering the possibilities of maximum bypassing and minimum decrease in serviceability of the harbour.

6.2. Recommendations considering the model

There are some recommendations to be made considering the applied model in Delft3D. Those are the following:

- Find a way to get rid of the boundary effects at the edge of the interior domain with the exterior domain. It has been difficult to get rid of these boundary effects but it could be worthwhile to put some extra effort into this. Without the boundary effects, it is possible to assess the processes on a longer time scale, either by increasing the morphological time scale factor or by increasing the simulation time. The boundary effects also limit the wave height which can be modelled with a reasonable accuracy. The boundary effects cause too much instability in case large waves are modelled.

- Refraction is not modelled correctly. The waves refract more and at larger depths than they should be according to the governing equations. The refraction has also been compared to that computed in Unibest-CL+ CL and that also gives a mismatch. The refraction computed by Unibest-CL+ CL is much less than the computed refraction in

Delft3D and is closer to what should be expected. The larger refraction in Delft3D affects the amount of sediment transport in the surf zone. It is therefore recommended to study how refraction is included in the model and what causes the mismatch with the governing equations.

- Apply a finer grid to better model the vortices around the tip of the breakwater. These are now difficult to model on grid cells with 16.67 m long edges. Therefore the effect of the spurs cannot be modelled properly. Also only apply a spur at one side of the harbour entrance.

- The model is located at the equator. This excludes Coriolis from study. Coriolis creates a Kelvin wave with a cross-shore phase and amplitude difference of the water level elevation. In this study this is simplified to a water level variation as a flat surface without a cross-shore variation. Its influence will be limited due to the limited cross-shore extend but modelling a Kelvin wave would improve the accuracy of the model. Therefore it recommended to take this in mind in further research.

- Diffraction cannot be included in the model. Therefore the processes at the downdrift side are not modelled correctly and are therefore not studied in this research. A much more refined model is required to be able to include diffraction.

- The applications of two grids with different grid dimensions in the FLOW module, has some limitations. The transition between the two grids should not be noticeable in the modelled processes. This is however not the case and strong boundary effects can be notices at the boundary of the two grids. This limits the applicability of high forcing on a longer time scale.

- The model also calculates a jump in the longshore transport rates, see Figure 3-12. In case of flood the jumps occurs at 600 m from the base -line and at ebb it occurs at 800 m from the base line. This gives rise to the idea that its location depends on the water depth. This is however not studied more thoroughly. Furthermore the jump is positive in case of flood and negative in case of ebb. The jump is only present in the sediment transport and not in the flow velocity. It does not have a large influence on the results of this study but it is nevertheless a strange feature. It is therefore recommended to study this feature.

6.3. Considerations Delft3D and Unibest-CL+

Two software packages are applied in this study: Unibest-CL+ and Delft3D. At first Unibest-CL+ was used to assess the influence of various parameters and to assess the sensitivity of system to parameters. The conclusion of the preliminary study is partly based on this analysis. After that it was chosen to continue with Delft3D for a more accurate assessment. Afterwards it can be said that this was a good choice but there are also arguments to have continued with Unibest-CL+. This is explained below.

Unibest-CL+ is an easy to use and powerful tool to model longshore sediment transport. The shoreline migration is modelled on the basis of the computes longshore transport rates at specific locations. A gradient in the longshore transport rate determines whether there is erosion or deposition. The Unibest-CL+ model runs are very time-efficient, which allows for the evaluation of many variables as well as a sensitivity analyses of those variables, also on a long time scale, say decades. For my research this is the main advantage of Unibest-CL+ over Delft3D. Unibest-CL+ is however not capable of computing 2DH effects such as flow contraction which is Delft3D is capable of. It is much more difficult and time consuming to setup a model in Delft3D, but the hydrodynamic and morphodynamic processes which are modelled are more accurately. A disadvantage of the application of Delft3D is that the run are much less time-efficient than Unibest-CL+ so this allows for assessing the influence of only a limited amount of parameters and on a shorter time scale, say few years.

Afterwards it can be concluded that the combination of applying Unibest-CL+ for the sensitivity analysis and Delft3D for the final calculation has been successful. The most relevant parameters could be determined on the basis of the sensitivity analysis. The more exact behaviour of the system and the influence of the parameters could consequently be determined with Delft3D because this computes the processes in a 2DH environment. The striking conclusion on the influence of the streamlined breakwaters could never have been found if only Unibest-CL+ was applied. Therefore it is recommended to continue further research with Delft3d, especially if the above recommendations are followed. If a quick assessment of certain parameters is required, it is recommended to apply Unibest-CL+.

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APPENDIX A
BYPASSING REQUIREMENTS

The sediment transport around harbour is of special interest for this study so this is put under a microscope. The requirement for bypassing are described in this paragraph by first looking at the processes and how they change due to the presence of the harbour. Next the influence of individual parameters is assessed. The requirements for bypassing are important to determine the most promising case of the framework, as defined in paragraph 1.4.

A.1. Bypassing processes

At first it will be described how the relevant processes act around a harbour. The processes will change due to the presence of the breakwaters. The coastline experiences changes due to variation of the processes. That, on its turn changes the effect of the processes on the coastline. The explanation accounts for a situation for which the breakwater length is longer than the width of the surf zone, see Figure A-1. This paragraph will give a qualitative indication of the changing processes.

The changes in the most important processes are given in Table A-1.

The coastline is divided in 3 sections:

1. Just updrift of the harbour (within a few hundred meters from the harbour);
2. In front of the harbour;
3. Just downdrift of the harbour (within a few hundred meters from the harbour).

Some processes act in the initial state (IS) and some processes act after some time when a fully developed state (FDS) is present. The initial state is the state of the situation just after construction of the breakwater and the fully developed state is when the bypass occurs. Not all processes act at the same time. The overview gives the processes which *could* act at that location. Whether the specific processes act depends on the acting forces, the situation and the state of the situation (IS or FDS).

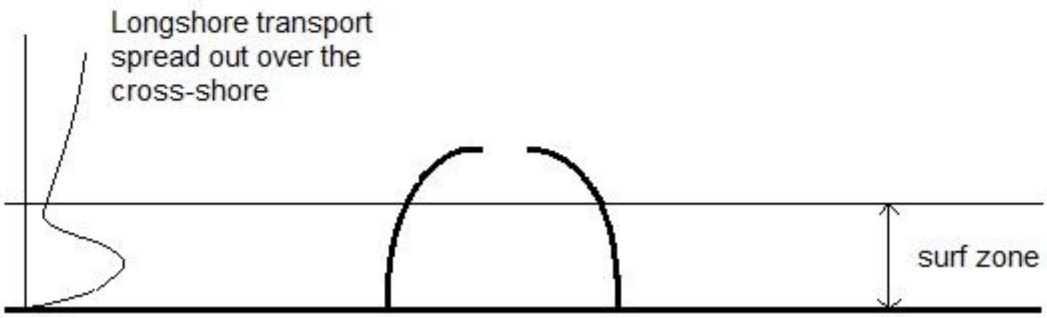


Figure A-0-1 Schematization of two harbour breakwaters which extend further than the width of the surf zone. The longshore transport takes place in the surf zone.

Just updrift of the harbour

The longshore transport is assumed to be constant at a straight coastline with parallel depth contours. At the coastline just updrift of the harbour this is no longer the case. There is a constant inflow of sediment from the updrift side but this is all blocked by the breakwater, since it is longer than the width of the surf zone. At the breakwater the transport is initially zero. The gradient in the transport is negative, which corresponds to accretion. The incoming sediment settles in this area leading to a changing coastline position. The decrease in the longshore transport can be achieved by decreasing the wave angle of incidence. The coastline thus rotates towards a situation at which it makes an almost zero degree angle with the prevailing wave direction. This effect can be seen in the S/φ −curve. The transport decreases too as the angle of incidence decreases. The shape of the S/φ −curve depends on the coastal profile, the wave conditions and the tide conditions.

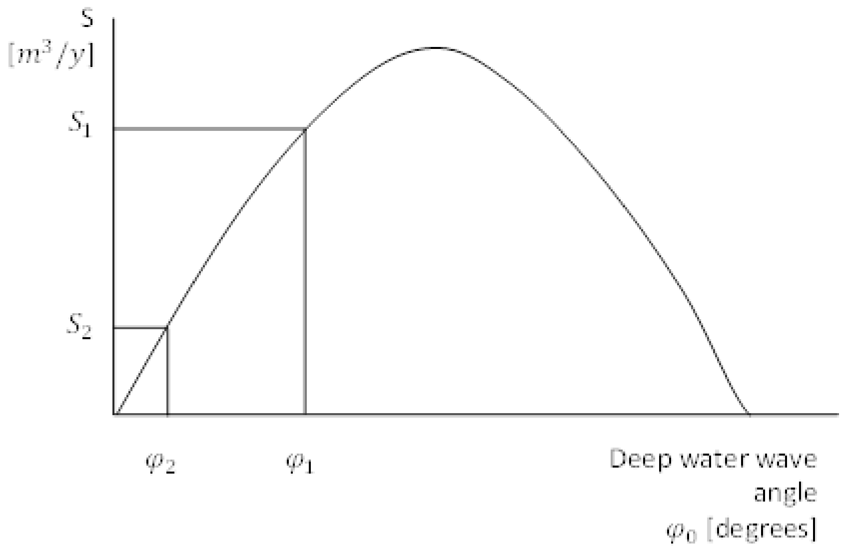


Figure A-2 Example of an S/φ −curve.

The breakwaters will push the current offshore. If this occurs abruptly the current could be pushed outside the surf zone taking sediment with it. In case of a smooth breakwater the current is pushed 'gently' offshore leading to flow contraction in front of the harbour. This increases the sediment transport capacity in front of the breakwaters. The flow contraction decreases as the accretion increases since the accretion causes the entire surf zone to progress offshore. The length of the breakwater relative to the width of surf zone therefore decreases.

In front of the harbour

The longshore sediment transport is initially blocked by the breakwaters but the transport will not be completely zero. It takes some time for the sediment to settle once it is in suspension and it takes time for the current to decrease. So although the coastline orientation will become such that the wave breaking doesn't lead to a wave-driven longshore current, there could be a transport of sediment which is already in suspension. This will settle after some time if the current decreases. The length over which this happens is called the adaption length. Due to this phenomenon the sediment could be transported in front of a harbour entrance and, if the current is strong enough, even beyond it.

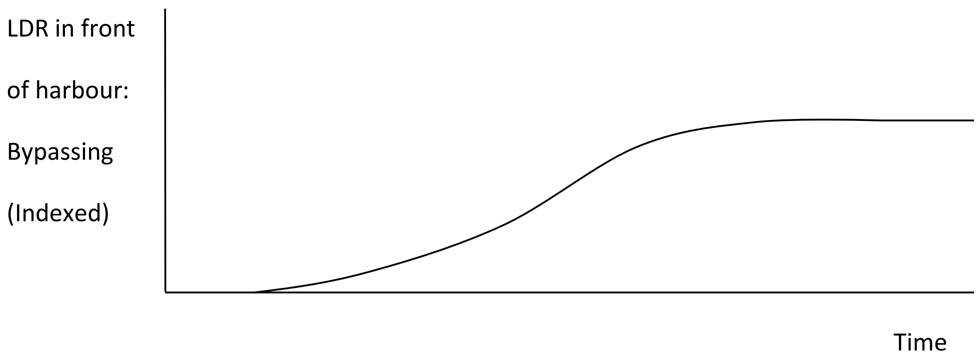


Figure A-0-2 Indexed bypassing in time. At first there is no bypassing at all and after some time a transport builds up to a maximum

The transport capacity decreases because the waves don't break in front of the harbour. The negative gradient in the transport corresponds to accretion in front of the harbour. The decrease of the transport capacity due to the lack of wave-breaking induced current could (partly) be counteracted due to the flow contraction. The flow contraction decreases as the accretion updrift increases. A dredged entrance channel could however cause flow expansion which counteracts the flow contraction. The channel will therefore accrete. There won't be depth-induced wave breaking in front of the entrance. The flow will follow the shape of the breakwaters. At the entrance, the flow expansion could cause vortices leading to erosion.

The vortices could also be caused in another way; by constructing spurs on the breakwater. Spurs are little extending parts on the breakwater. They could cause flow separation and turbulence in the flow (vortices). If the vortices are strong enough, it prevents sedimentation in front of the entrance. If they are even stronger, they could cause erosion in front of the entrance. The downside is that it could make navigation in and out of the harbour very difficult and the erosion could harm the breakwater stability.

A tide gives a longshore current but it could also lead to a current in and out the harbour. If this is strong enough it could transport sediment as well. The magnitude of the current at the harbour entrance depends on the tidal characteristics and the presence of a lake or similar inland of the harbour.

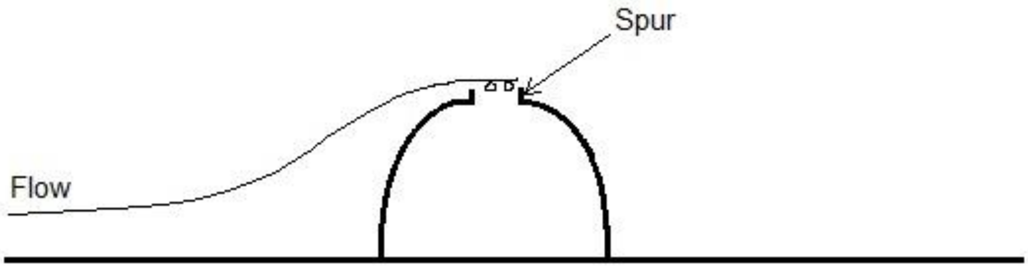


Figure A-4 Schematization of the vortices, caused by spurs on the breakwater.

Just downdrift of harbour

At the downdrift side there is a shadow zone from the prevailing waves. Outside the shadow zone the waves cause the same amount of transport as the updrift side. There is however no supply of sediment so the beach experiences erosion. Diffraction of the waves and alongshore differences in the wave set-up will cause sedimentation against the breakwater.

If a bypass is established the flow experiences flow expansion just downdrift the harbour. A shoal could therefore form next to the breakwaters. There is an option to maintain the transport if an underwater bar is formed from the tip of the downdrift breakwater towards the downdrift coastline. That situation could be represented by a headland as can be seen in Figure A-6. The bar could make just the right angle with the deep water wave angle to maintain the transport. In that case the bypassed sediment is transported towards the shoreline. The angle the bar needs to make with an imaginary shore-parallel line depends on the deep water wave angle and the bars geometry. The ability to transport sufficient sediment along the bar depends mainly on the bars geometry. A shallower and milder bar gives more transport.

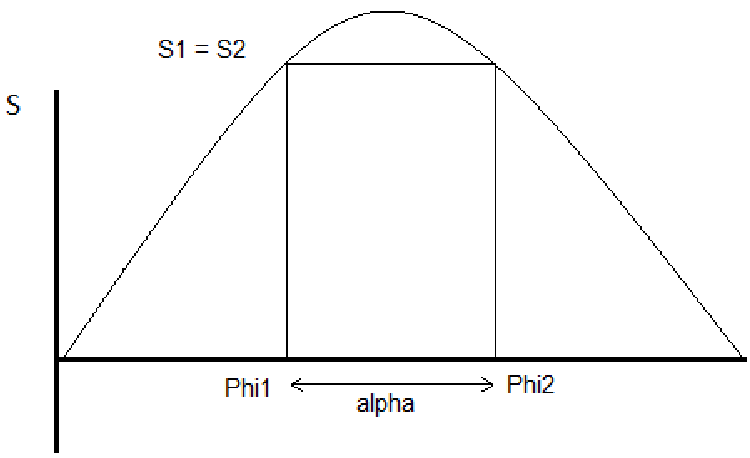


Figure A-5 S/φ −curve which shows that waves with two different angles could cause the same transport on the same bathymetry.

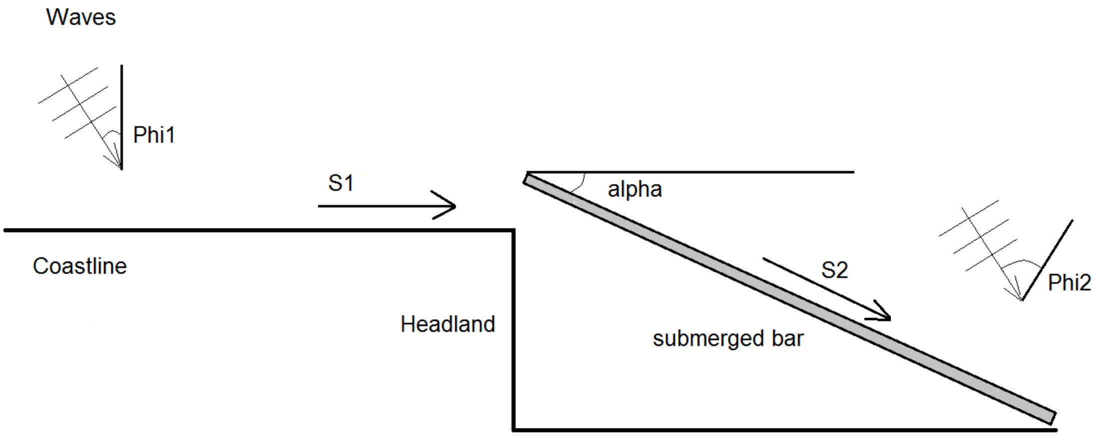


Figure A-6 Schematization of a headland.

Several tests have been carried out with the above situation. The length of the headland and the height above the bar varied. In all cases the bar is modelled as a bar with steep slopes in the profile. In one of the simulations the bar is modelled with milder slope; the bar is modelled with a mild sloping profile at the offshore side as well. See Figures A-7 and A-8 for two examples of the modelled profiles.

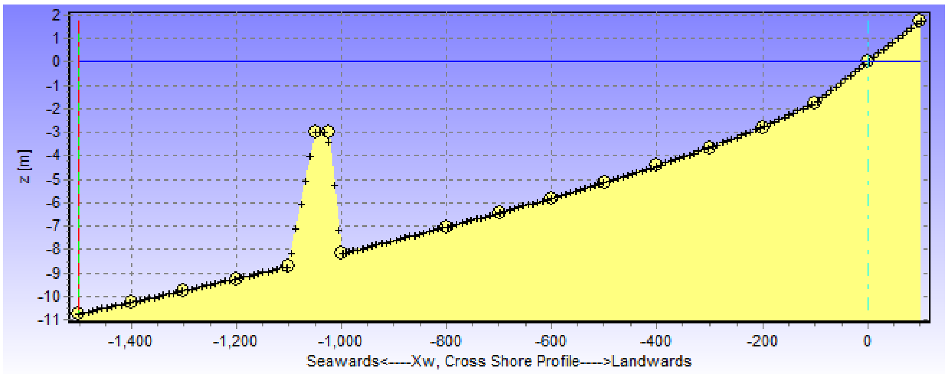


Figure A-7 Profile with a bar at 1000 m, height above the bar is 3 m.

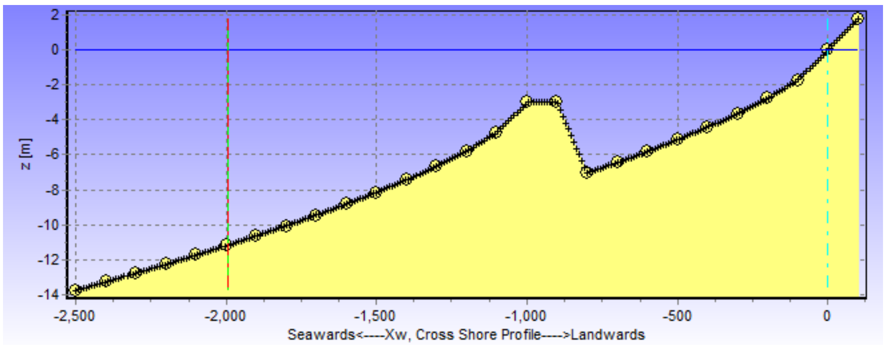


Figure A-8 Profile with a bar at 1000 m, height above the bar is 3 m. The profile offshore of the bar is also an equilibrium profile.

For each situation the $S/\varphi -$ curve is computed and plotted, see Figures A-9 and A-10. The red line indicates the undisturbed transport on an equilibrium profile according to Dean (1987). The other lines indicate different cross-sections with a bar. The number in each name indicates the cross-shore length of the headland. For example $S/Phi -$ curve bar 250' indicates the $S/\varphi -$ curve for a profile at which the length of the headland is $250\ m$ and the bar originates $250\ m$ from the downdrift coastline.

The amount of transport decreases as the bar is located further offshore but in many cases it is possible to transport the same amount of sediment along the bar as the transport on the undisturbed coastline. Only if the waves create the most amount of transport at the undisturbed coastline, it cannot be transported along the bar, which is not surprising.

An example will be given to illustrate this. The example is also illustrated in Figure 2-9:

> *The deep water wave angle is 20^o. This gives a transport of $400\,000\ m^3/y$. This needs to be transport around a headland with a length of $500\ m$. The bar is therefore located at a cross-shore distance of $500\ m$. The height above the bar is $2\ m$. The bar needs to make an angle of approximately 27^o or 47^o with the waves to give the same amount of transport. Given the deep water wave angle of 20^o, the angle of the bar with the updrift coastline (α) should be 7^o or 27^o .*

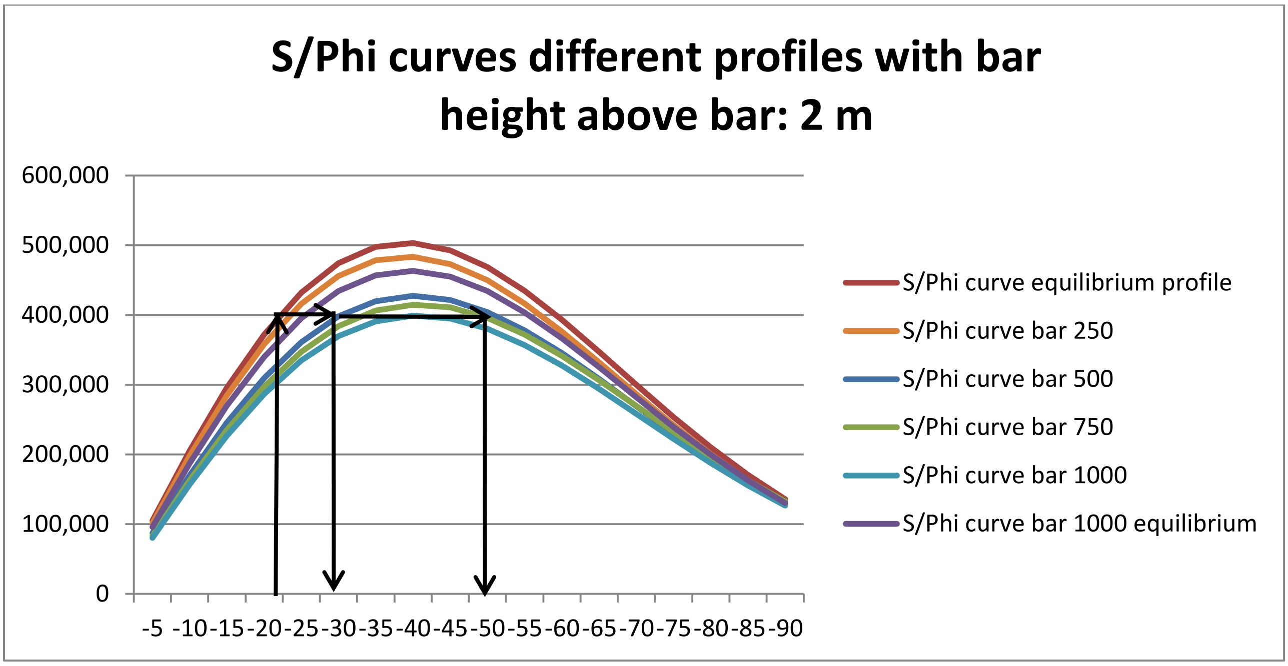


Figure A-9 $S/\varphi -$curves of the profiles with a bar and a height above the bar of $2\ m$.

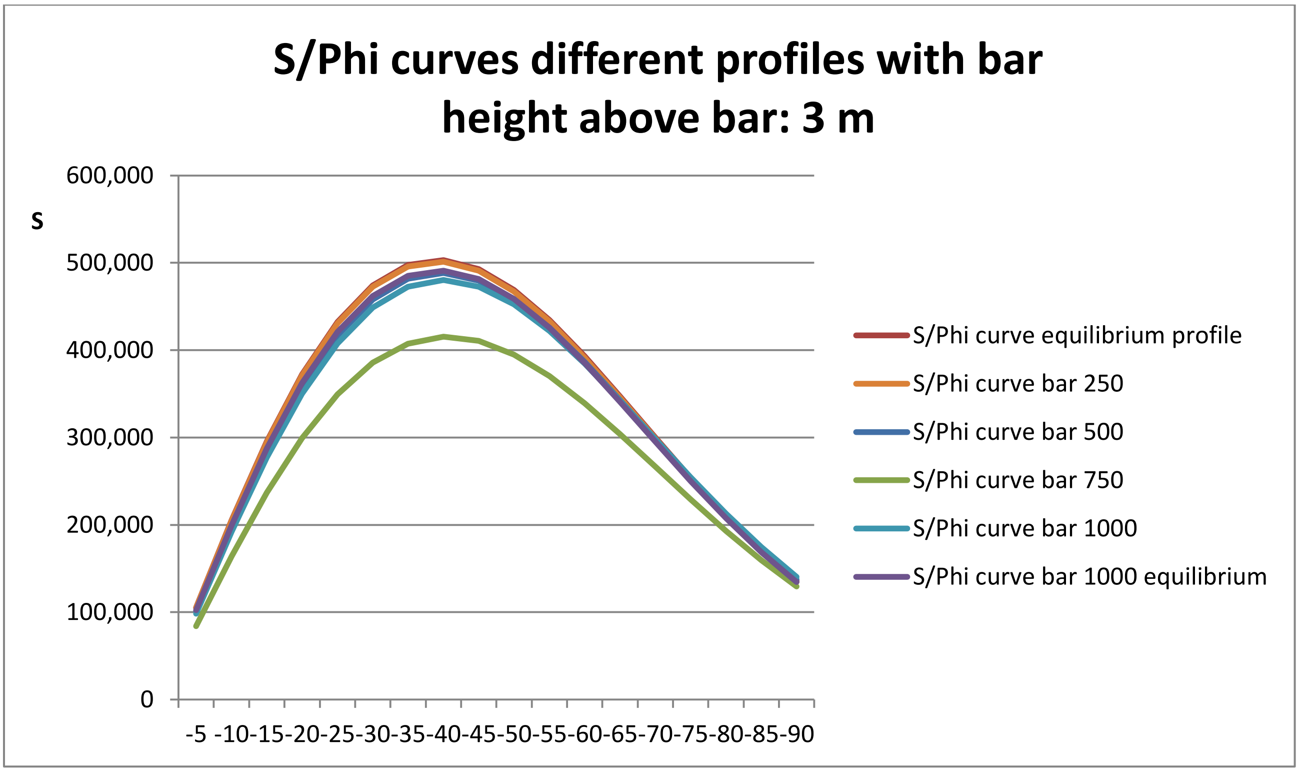


Figure A-10 S/φ −curves of the profiles with a bar and a height above the bar of 3 m.

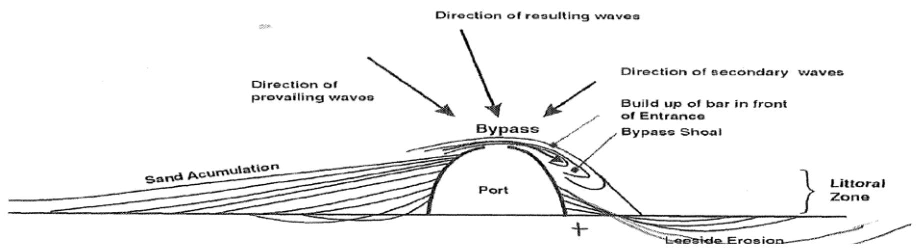


Figure A-11 Erosion and sedimentation patterns around a harbour according Karsen Mangor.

Process	**Just updrift of the harbour**	**In front of the harbour**	**Just downdrift of the harbour**
Cross-shore current	Possible outbreaking	Current in/out of harbour.	
Cross-shore transport	Offshore directed due to possible outbreaking	Possible due to current in and out harbour.	Onshore directed from shoal next to entrance. (FDS)
Wave angle	Decreased due to accretion	Increase wave angle next to entrance in case of streamlined breakwaters (FDS) No or less refraction in entrance channel.	Diffraction into shadow zone
Wave-induced Longshore current	Pushed offshore (possible outbreaking). Decrease due to smaller wave angle	No contribution (no wave breaking). Flow contraction.	Increase. Flow expansion.
Tide-induces longshore current	Pushed offshore (possible outbreaking).	Flow contraction.	Flow expansion. Onshore directed.
Longshore transport	Initially (partial) blocked by breakwater → accretion. (IS) After accretion the angle of incidence decreases which decreases the transport rate.	Decrease due to lack of wave-induced current and turbulence. Possible increase due to flow contraction. Sedimentation in entrance channel (FDS). Some transport due to adaption length (FDS). No transport if length breakwater > width surf zone (IS).	Possible decrease due to flow expansion. Shoal formation next to downdrift breakwater. Accretion against breakwater in shadow zone. Erosion outside shadow zone. Increase due to increase wave angle (FDS)
Secondary currents	Towards breakwater.		Towards breakwater
Coastline orientation	Shift towards angle of incidence of prevailing waves.	No coastline	Shift to angle of incidence diffracted waves against breakwater. Shift to eroded situation.
Vortices		Possible due to spurs. Possible at end breakwater	

Table A-1 Possible acting physical processes in the surf zone at a coastline with harbour breakwaters. IS means Initial State and FDS means Fully Developed State

A.2. Sensitivity analysis parameters sediment transport

This paragraph will show the influence of certain parameters on sediment transport. This the second assessment on which the requirements for bypassing is determined. From the theory in Bosboom and Stive (2011) it can be concluded that the sediment transport along a shoreline depends on the following parameters:

- Wave conditions: wave height, wave period, wave angle;
- Current conditions;
- Water level conditions: storm surge, set-up;
- Bathymetry;
- Sediment characteristics;
- Sources and sinks: river discharges, tidal inlets, etc.;
- Breaker index.

For the sediment transport around harbour breakwater, the following parameters are also of relevance:

- Breakwater length compared to width surf zone;
- Shape of the breakwaters;
- Flow in and out the harbour;
- Spurs on the breakwater;
- Dredged entrance channel of not;
- Transport regime: large net or a small net transport.

This makes clear that there are a lot of variables which play a role in achieving sufficient amount of bypassing while maintaining a deep enough entrance channel. The influence of a part of these parameters on the sediment transport is determined in a preliminary study with Unibest-CL+.

A.2.1. The influence of each parameter

Not all of the above parameters can be assessed without an extensive model. The parameters, of which it is possible to assess the influence, are assessed with the help of the Unibest-CL+ software package. Those are the below parameters. This chapter will give the conclusion per parameter with regards to sediment transport. Table A-2 shows which settings per parameter are used to test its influence. These tests mainly focus on gaining maximum transport in front of the harbour breakwaters but apply for more general situations as well. Not all conclusions can be drawn based on the depicted $S/\varphi -$ curves. Some have been left out to limit the number graphs.

Parameter	Setting
Wave height	General setting: $2\ m$ Variation: $1\ -\ 3\ m$
Wave angle	General setting: 45^o Variation: $30^o\ -\ 60^o$
Wave period	General setting: $7\ sec$ Variation: $4\ -\ 10\ sec$
Bed slope	Steep: $1{:}\,25$ Mild: $1{:}\,100$ Extra mild: $1{:}\,200$ Equilibrium profile according to Dean Flat
Depth in front of breakwater	General setting: $5\ m$ Variation: $0\ m\ -\ 10\ m$
Breakwater	$500\ m$ long $3\ m$ above water level
Breaker bar	Location: in the middle of the slope Depth above breaker bar: $3\ m - 14\ m$
Breaker index	General setting 0.8 Variation $0.5\ -\ 1.0$
Tidal currents	General setting: no tidal currents Variation: $0\ m/s\ -\ 1.0\ m/s$, amplitude $1\ m$.
Sediment size	General setting: $200\ \mu m$ Variation: $100\ \mu m - 300\ \mu m$ Cohesive material will not be considered
Sediment transport formula	Bijker (1967, 1971) (The formula is not subject of this study)

Table A-2 The tested parameters and their variation.

Wave angle As expected a deep water wave angle of approximately 45^o gives the most amount of transport for an undisturbed coastline, see Figure A-14. This however holds for the assumption that refraction is undisturbed. In front of a harbour the refraction process cannot be completed because the wave suddenly breaks due to the breakwater. In this case the maximum transport is achieved at waves with a smaller deep water wave angle. The $S/\varphi -$ curve is also more asymmetric in case the refraction cannot be completed, see Figure A-14.

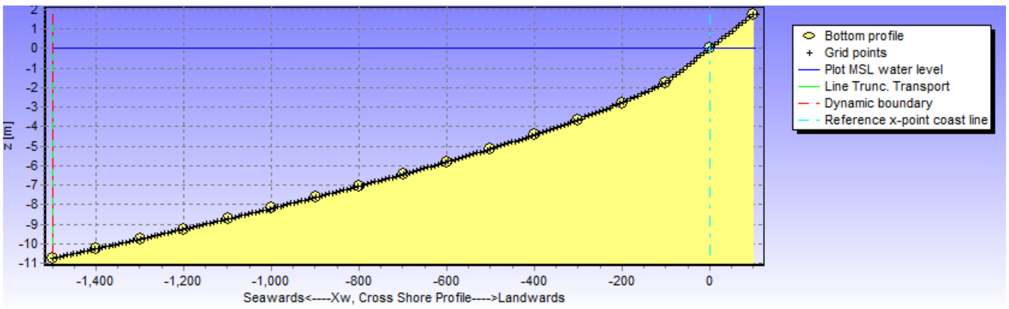


Figure A-12 Equilibrium profile without a breakwater.

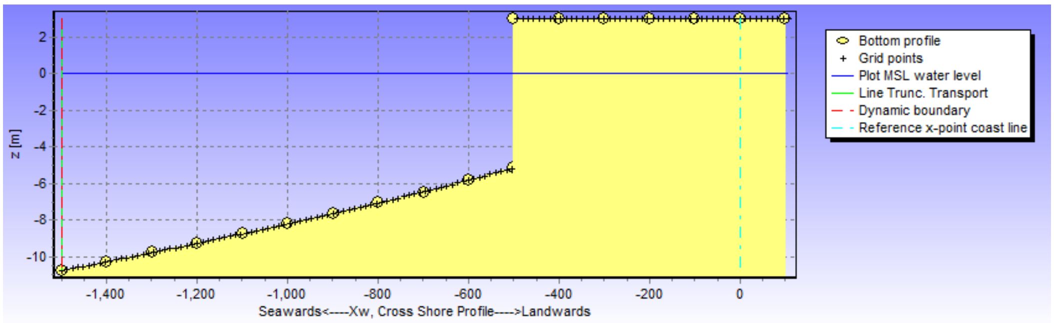


Figure A-13 Equilibrium profile with a breakwater. $L_{BW} = 500\ m$. The breakwater gives 100 % blockage.

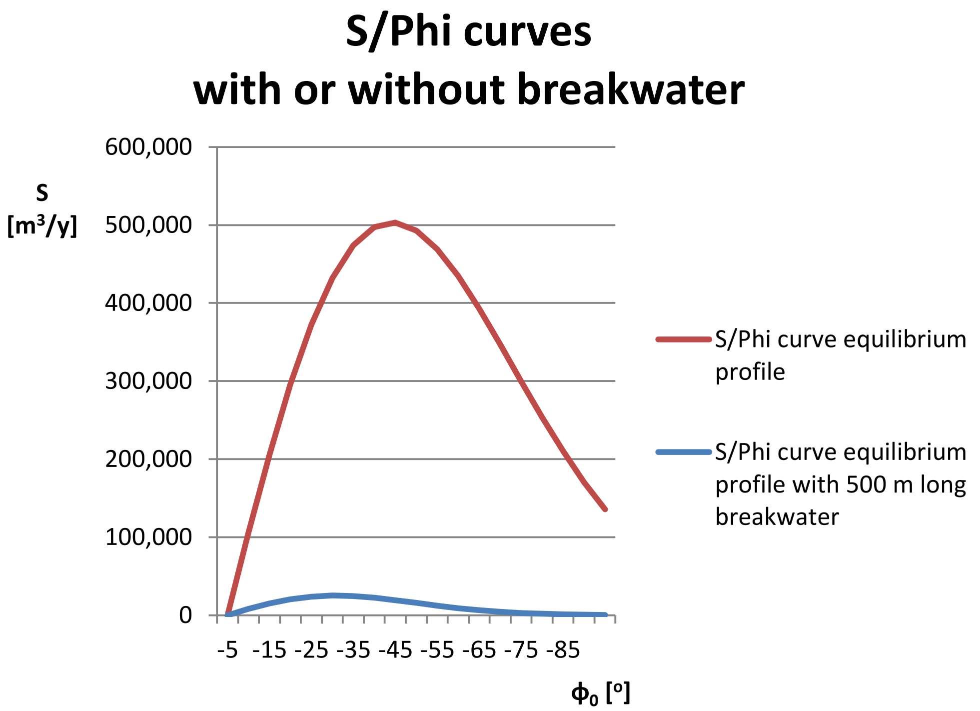


Figure A-14 S/φ − curvesof a coastline with an equilibrium profile and an interruption of the coastline by a harbour breakwater.

Wave height

In general applies, a larger wave height means more wave energy which can be released at wave breaking. This causes more stirring up of sediment and stronger longshore current.

Higher waves have the most amount of influence on steeper slopes. The relative increase of the transport due to higher waves is more on steeper slopes.

The wave height also affects the angle at which the transport is at its maximum. Higher waves give maximum transport at larger wave angles than lower waves.

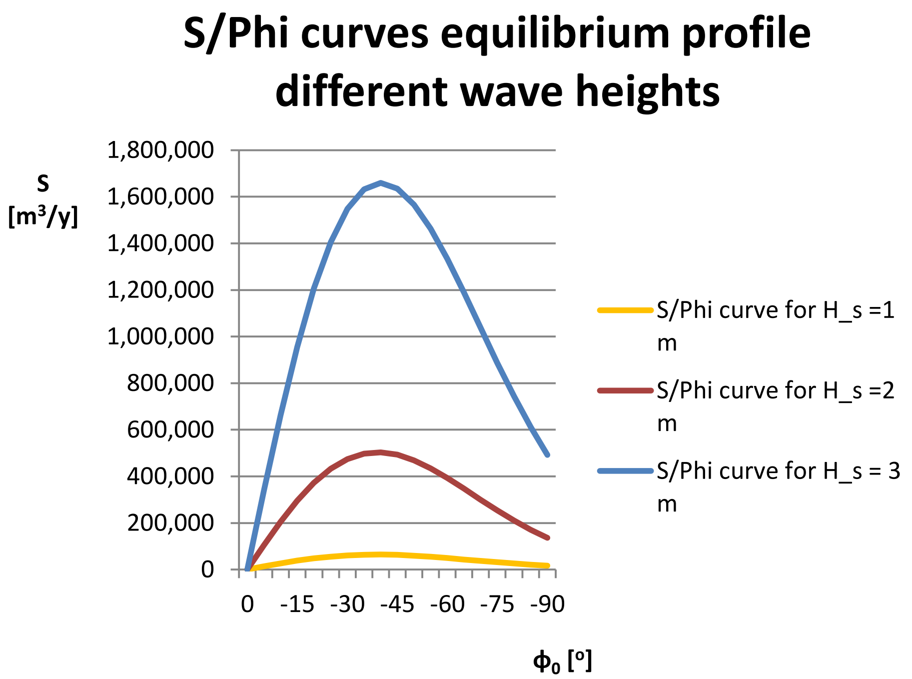


Figure A-15 S/φ – curves due to two different wave height on an equilibrium profile. $H_s = 1m, H_s = 2m$ and $H_s = 3m, T_p = 7\ sec.$

Higher waves break further offshore.

Tests have also been carried with smaller wave height ($1\ m$) but these did not give any significant transport because the depth in front of the harbour is too large

Wave period Longer period waves generate less transport than waves of the same height because shorter period waves break at larger angles.

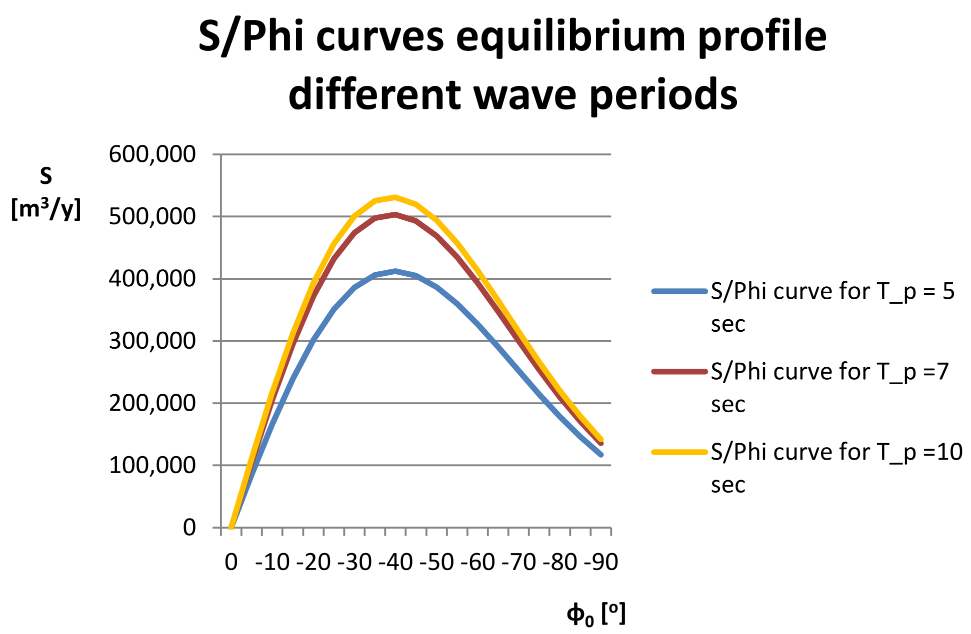


Figure A-16 S/φ – curves for an equilibrium profile with three different wave periods. $T_p = 5\ sec, T_p = 7\ sec$ and $T_p = 10\ sec,\ H_s = 2m.$

The deep water incident wave angle for which the waves cause the most amount of transport in front of a harbour increases for shorter waves. Shorter wave refract more before they break.

Long waves cause transport which is concentrated more onshore relative to short waves. The transport caused by short waves is more spread out over the cross-shore.

Bed slope A very mild slope ($1{:}200$) in front of the harbour gives slightly more transport, although the difference is small compared to an equilibrium profile. A gentler slope is more dissipative and will result in more transport which is spread out more over the cross-shore.

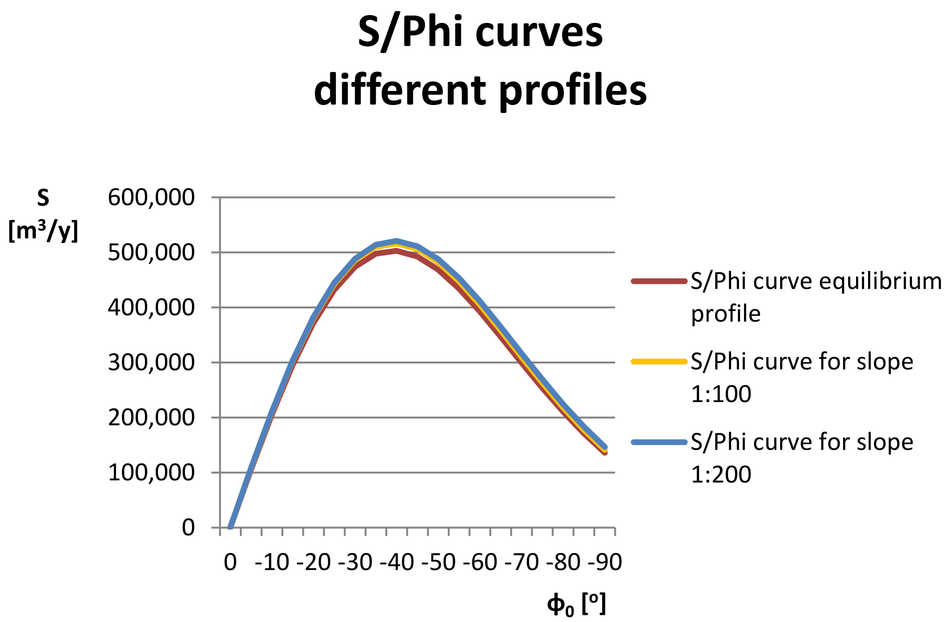


Figure A-16 $S/\varphi -$ curves for three different bed slopes, an equilibrium profile a $1{:}100$ profile and a $1{:}200$ profile. $H_s = 2m, T_p = 7\ sec$,.

Depth in front of breakwater The transport increases as the depth in front of the breakwater decreases because the length of the slope increases. The transport and thus possible bypassing will be at its maximum as the length of the slope increases towards the situation of the initial coastline.

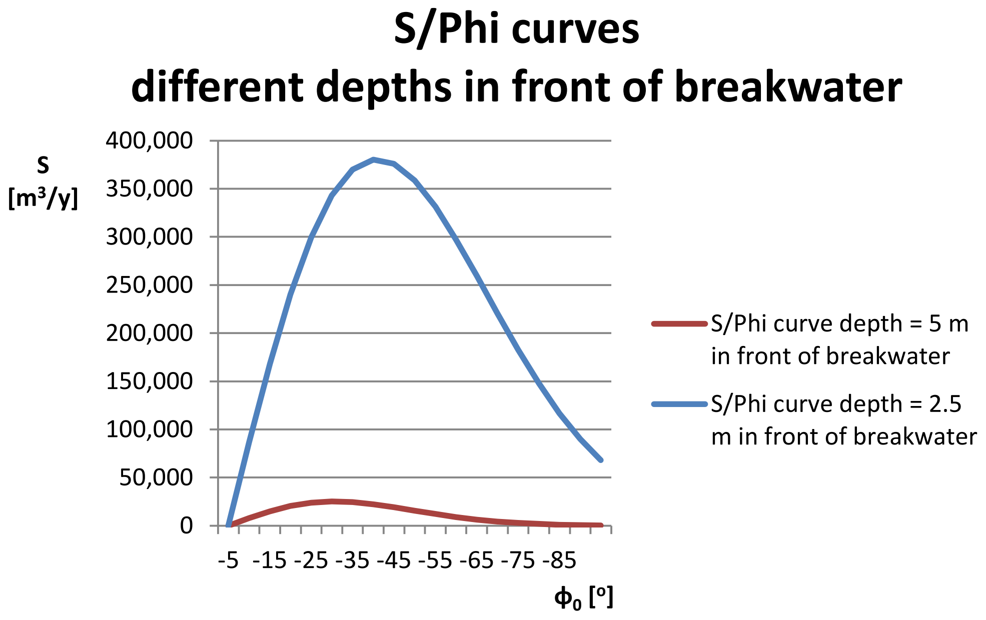


Figure A-0-3 $S/\varphi -$ curves for a profile which consists a shore-normal breakwater. The depth in front of the breakwater is $5\ m$ and $2.5\ m$.

The transport significantly decreases if it becomes deeper in front of the harbour. A breaker bar in the profile can only partly decrease this effect. The smaller waves ($H_s = 2\ m$) cause no transport as it becomes deeper than $5\ m$ in front of the breakwater. In this case only waves with a large wave height can cause transport. This can be increases with the presence of a breaker bar.

Breaker bar A breaker bar can have a significant influence on the sediment transport at deeper water, especially when the depth above the bar decreases.

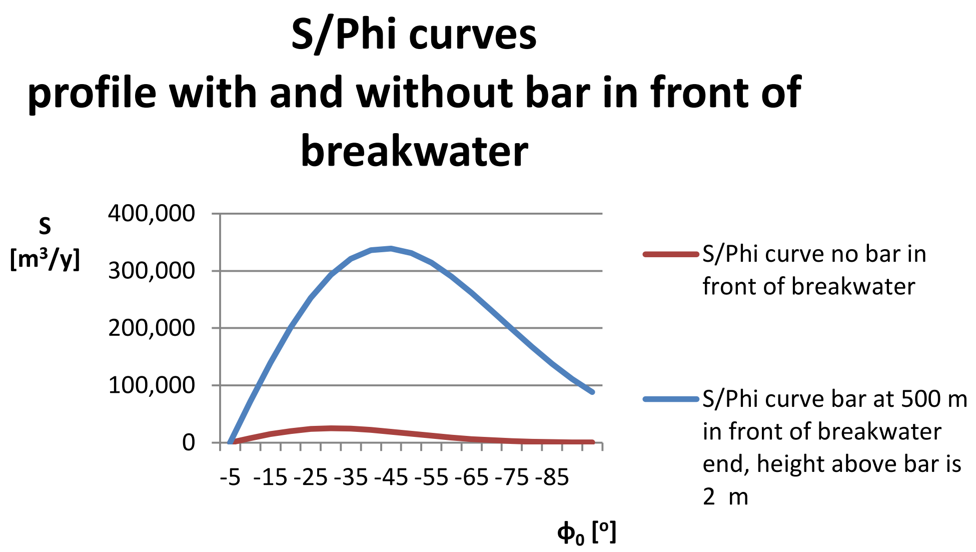


Figure A-18 $S/\varphi -$ curves for an equilibrium profile which consists a shore-normal breakwater and a breaker bar at 500m in front of the tip of the breakwater.
$H_s = 2m, T_p = 7\ sec.$

The breaker bar has less influence on a milder slope in case the depth in front of the harbour is $5\,m$. If it is deeper than $5\,m$ in front of the harbour, the bar has more influence on a milder slope.

The breaker bar has less influence in case of higher waves.

Breaker index As the breaker index decreases from the default ratio 0.8, more transport occurs and it occurs further offshore where it is deeper. As the breaker index increases the waves break at shallower areas. This has not been assessed on a wide scale.

Tidal currents The tide can play a large role in the total sediment transport; it increases the total transport significantly. The transport is spread out more over the cross-shore due to the tide. The shape of $S/\varphi -$ curve is not very different; it only shifted slightly because there is no transport in case the wave angle is zero.

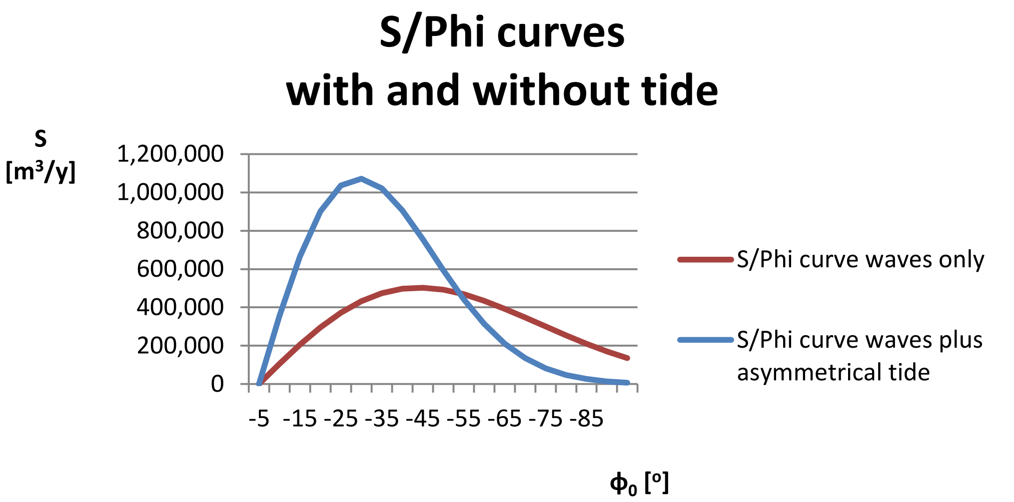


Figure A-19 $S/\varphi -$ curves for an equilibrium profile at which only waves or waves combined with an asymmetrical tide act, $H_s = 2m, T_p = 7\ sec$,

Flow contraction is not included in this assessment. It is expected to influence the total transport in front of harbour breakwater significantly.

Sediment grain size Smaller sediment size gives more transport. The $S/\varphi -$ curve only changes in magnitude due to varying sediment size.

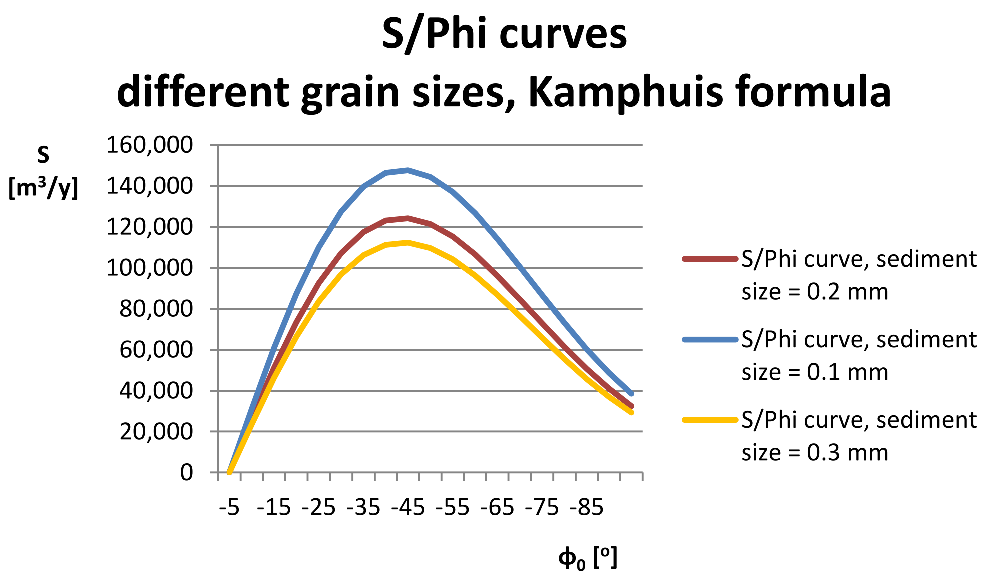


Figure A-20 $S/\varphi -$ curves of the Kamphuis formula for an equilibrium profile. Three different sediment grain sizes have been simulated.

A.2.2. Influence other parameters

The following physical processes/parameters are not included in the sensitivity analysis. The expected influence is given as well.

Flow in and out of the harbour

If it is strong enough the flow in and out the harbour could prevent sedimentation of the entrance channel.

Shape of the breakwater

A streamlined shape is expected to be in favour of achieving flow contraction. A non-streamlined breakwater is expected to push the current further offshore (outbreaking).

Length of the breakwater compared to the width of surf zone.

Breakwater smaller than the width of surf zone will show bypassing for sure since the sediment transport is spread out over the width of the surf zone. They will therefore have less influence on the longshore transport than breakwaters which extend further than the width of the surf zone. They will initially block all the sediment.
Breakwater longer than the width of the surf zone will give more flow contraction of the tidal current. The wave-induced part will however be larger in case of breakwaters shorter than the width of the surf zone. This has a large influence on the morphological processes and it is therefore worthwhile to study several lengths of the breakwater.

Spurs on the breakwater

Spurs could give vortices, which could prevent sedimentation and, in case they are strong enough, even lead to erosion.

Transport regime: large net or a small net transport.	In case of a large net transport, the coastline will show extensive accretion at the updrift side and erosion at the downdrift side. In case of a small net transport, the erosion and accretion will be less.
Dredged entrance channel or not	The presence of a dredged channel will have mayor influence on achieving the objective. In a dredged entrance channel the flow experiences flow expansion, which causes sedimentation.

The influence of most of the parameters/physical processes is now known. Based on this the requirements for sufficient bypassing and depth in front of the harbour can be determined.

A.3. Requirements Bypassing

There are three requirements for sediment transport to initiate and occur:

- There needs to be sediment;
- There needs to be a current;
- There need to be turbulence to initiate motion of sediment.

Without turbulence there is no stirring up so there is no contribution to the transport. Without current there is no movement of the sediment and without sediment there is no transport at al.

The goal of this study is to find circumstance at which bypassing occurs and sedimentation of the entrance channel is avoided. This does not mean that there must be initiation of motion in the entrance channel. If the sediment is already in suspension it should be made sure that it stays in suspension. The main issue is thus to avoid sedimentation at the entrance channel.

There are two physical processes considered in this study; waves and tide. Waves cause turbulence and a longshore current if they break. The turbulence gives initiation of motion of sand grains and it prevents suspended sediment from settling. The current transports the sediment. This is enlarged by a tidal current.

In front of the harbour entrance, there is no wave breaking, so there is lack of turbulence to initial motion and to prevent suspended sediment from settling. In addition there is less current due to the lack of wave breaking. Both result in sedimentation. In order to achieve the goal, the effects of less wave breaking in front of the harbour needs to be compensated. These are elaborated below.

Strong enough current
A strong current compensates for the lack of wave-breaking-induced current. This plays a role in the transportation of sediment, but if it is strong enough, it also prevents sedimentation.

- A strong current could be induced by flow contraction. If the length of the breakwaters is less than the width of the surf zone the flow will be partly be contracted. This could lead to high velocities in front of the breakwaters. The flow contraction will however decrease as the surf zone migrates offshore due to accretion. If the transport capacity is large enough this could come to a stop.

- An offshore breaker bar could also play a role in flow contraction.

  This would also lead to net onshore directed mass flux. Continuity requires that this should be compensated by a return current. This is restricted next to the entrance channel due to the presence of the breaker bar (if the depth over the bar is not too much). The return current can than take place in the entrance channel which could possibly be strong enough to prevent sedimentation.

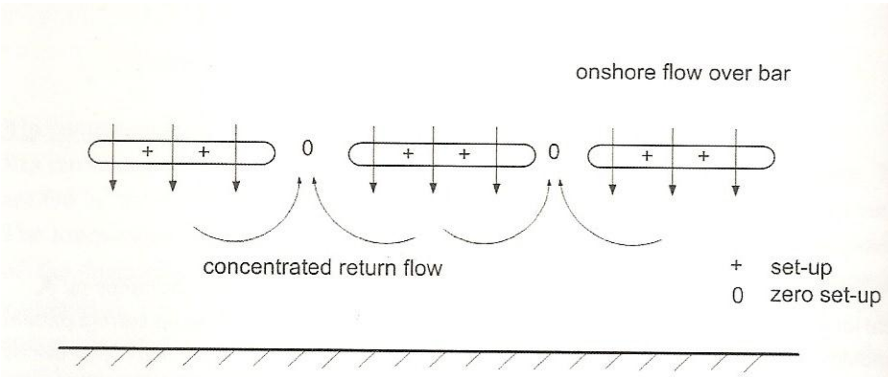


Figure A-21 Rip currents in case of submerged breakwaters according to Bosboom and Stive (2011)

- The presence of a tide gives extra flow contraction and thus a higher flow velocity.
- Streamlined breakwaters gently push the current to a contracted situation and contribute therefore more to flow contraction than abrupt, shore-normal breakwaters.
- A current in or out the harbour could also lead to a strong current, however in the cross-shore direction. This current could be caused by a river or by the tide which flows in and out a basin behind the entrance. Especially the tidal current could lead to high currents which could avoid sedimentation and possibly even cause erosion in the entrance channel. Dependent of the current outside the harbour, the sediment is transported offshore or drifted along the coast.
- A storm event could also provide a strong wave-breaking-induced and wind-induced current. Next to the fact that it increases the stirring-up of sediment, it also creates a strong current; both could be favourable for bypassing.

Sufficient turbulence
These are the ways; the lack of wave-breaking-induced turbulence could be compensated:

- Spurs at the outer side of the breakwater could cause vortices. That could also induce sufficient sediment transport, see Figure A-4.

- A strong current could also lead to wave breaking. In this case we don't talk about depth-induced breaking but current-induced breaking. This is a very specific situation which will not be elaborated.
- A shore normal breakwater with an abrupt end in deeper water. The might lead to turbulence just behind the breakwater due to flow separation. The flow separation at shore, parallel breakwater is expected to cause more turbulence.
- Streamlined breakwaters could induce flow contraction. When this (strong) current passes the entrance of the harbour, flow expansion could cause vortices at the end of the breakwater.

It is evident that not all processes can act in the same time and that not all processes nicely work together. A mild slope and a flow contraction are for instance two things that are difficult to coincide. Therefore combinations and separate cases should be considered.

A.4. Conclusion

The conclusions are divided in those which follow from the sensitivity analysis and those which don't.

The following can be concluded from the sensitivity analysis:

- The wave height especially has a large influence on the amount of longshore transport. They break further offshore and cause more transport, especially on steeper slopes.
- The wave period has minor influence of the longshore sediment transport. Waves with a longer period cause little more transport.
- A wave angle of 45^o gives the most amount of transport at an undisturbed coastline but the maximum is achieved at smaller wave angles in front of the harbour because the refraction process is disturbed.
- The bed profile hardly influences the amount of longshore sediment transport.
- The depth in front of the harbour is of mayor influence on the amount of longshore sediment transport. A smaller depth can increase the amount of transport significantly. The same effect can be achieved with an offshore breaker bar.
- The tidal currents enlarge the sediment transport significantly. An asymmetric tide causes the S/φ-curve to change shape drastically.
- Smaller grain size gives more transport.

The influence of the parameters which are not assessed with Unibest-CL+, is determined based on theory. The following conclusions are formulated:

- The harbour could be kept open if there is a large in and outflow due to the presence of a large area behind the harbour.

- A streamlined shape is expected to be in favour of achieving flow contraction. A non-streamlined breakwater is expected to push the current further offshore (outbreaking)
- Breakwater smaller than the width of surf zone will show bypassing for sure since the sediment transport is spread out over the width of the surf zone. They will therefore have less influence on the longshore transport than breakwaters which extend further than the width of the surf zone. They will initially block all the sediment.
- Breakwater longer than the width of the surf zone will give more flow contraction of the tidal current. The wave-induced part will however be larger in case of breakwaters shorter than the width of the surf zone. This has a large influence on the morphological processes and it is therefore worthwhile to study several lengths of the breakwater
- Spurs could enhance the turbulence in front of the harbour entrance. This could compensate for the lack of wave-induced turbulence.
- In case of a large net transport, the coastline will show extensive accretion at the updrift side and erosion at the downdrift side. In case of a small net transport, the erosion and accretion will be less.
- The presence of a dredged channel will have mayor influence on achieving the objective. In a dredged entrance channel the flow experiences flow expansion, which causes sedimentation.

Primarily the effects of the lack of wave driving force in front of the harbour entrance needs to be compensated. Wave breaking causes turbulence and a longshore transport. Turbulence leads to initiation of motion and it prevents settling of suspended sediment. Initiation of motion is not required in front of the harbour. The settlement of sediment can also be prevented by a strong longshore current.

The lack of wave driving force could be compensated by:

- A strong current due to large flow contraction. This could be the case if there is a large tidal influence or in case of large breakwaters.
- Turbulence in the entrance due to spurs or flow separation.

Considering the breakwater there is a dual expectation. If they are longer than the width of the surf zone, they are expected to cause more flow contraction and thus a higher current velocity in front of the harbour. If the sediment gets in the region in front if the harbour, this could bypass to the downdrift side of the harbour. Shorter breakwaters however block less sediment and give a bypass for sure. Additionally, shorter breakwater reaches less deep water due to which there might be a positive wave-influence.

Considering the transport regime, it is best to have sediment transport in both directions, e.g. a small net transport. This will give accretion against the breakwater. The accreted bathymetry at for instance the downdrift side could make just the right angle with the prevailing waves to give the same amount of transport. A small net transport can be caused at a wave dominated coastline or at coastline which is influenced by waves and tide.

APPENDIX B.
WORKED EXAMPLE NAVIGATIONAL REQUIREMENTS

The majority of the navigational requirements depend on the type of vessels that call at the port. The navigational requirements are than prescribed by means of a design vessel. Therefore at first some estimated guesses are done to determine the design vessel. This is done for a small harbour and a large harbour. This is based on the theory described in Ligeringen, (January 2009). Some of the figures originates from this work as well.

B.1. Design vessel

The design vessel determines the design of the entrance channel. The entrance channel will be designed such, that the design vessel and all other vessels can enter the port safely. It is often the largest ship which is expected to call at the harbour but this may not always be the case. The largest vessels often get support from tugboats when they enter or leave the port and may therefore not pose the greatest thread to safety. The design vessels could therefore satisfy one or more of the following criteria:

- It may have very poor manoeuvrability;
- It may be very large in the context of port operations;
- It may have excessive windage;
- It may carry particularly hazardous cargo.

This needs to be taken into account at designing the entrance channel. Since this study does not consider a specific harbour case, the design vessel cannot be deduced from any available data. A well educated guess will therefore be made for an industrial port and a fishery port.

Type of port	Length L_{OA} $[m]$	Draught $[m]$	Width W_{BM} $[m]$	hazardous cargo
Industrial port	250	15	55	Possibly high
Fishery port	25	2.5	7	no

Table B-1 The design vessels which will be used in this study.

B.2. Approach channel

The approach channel is defined as the waterway linking the turning circle inside the port with deeper water. The entrance channel has three design parameters; alignment, width and depth which are chosen, based on the local environment and the design vessels characteristics. The required length of the entrance channel is of importance too.

Alignment
The alignment mainly depends on the local circumstances. The following requirements apply for the alignment of an approach channel:

- The shortest possible length taking into account wave, wind and current conditions;
- Minimum cross-currents and cross-winds;
- Minimise number of bends and avoid bends close to the harbour entrance. The length of the straight channel before passing the breakwaters depends on wave, wind and current conditions.

The local situation of course also plays an important role, hard soil for instance introduces high dredging coast and is therefore preferably avoided. The local conditions will not play a role in this study, since a generalised case is studied.

Width
An entrance channel should be wide enough to provide a safe passage. The risk of running aground should be as low as possible and therefore the channel should be as wide as possible. The width of the channel is however constrained to the local situation but mostly due to dredging costs. The width of the channel is therefore a balance between safety and costs. The width can be determined with the PIANC method.

A sailing ship does not sail in a straight line but makes a sinusoidal track and thus covers a 'basic width', which is about 1.5 times the ship's beam, see. This is due to the delay in the helmsman's response to the ships movement and the delay in the ships reaction on the rudder. In addition, the effects of wind, currents, and waves require additional width. Moreover certain margins are required, which depend on the bank type and the type of cargo. Whether a channel is meant for one-way shipping or two-way shipping also influences the required width of the entrance channel.

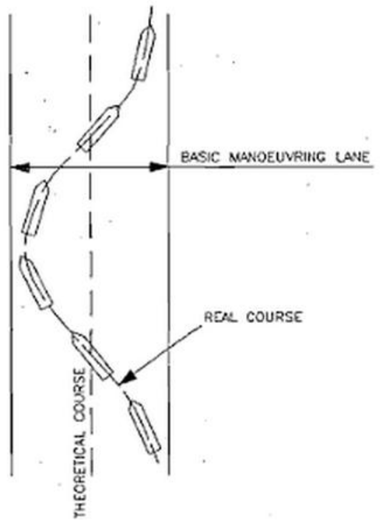


Figure B-0-1 The sinusoidal track of a sailing ship due to the delay in response of the helmsman and the response of the ship to the rudder. Abstracted from Ligteringen 2009.

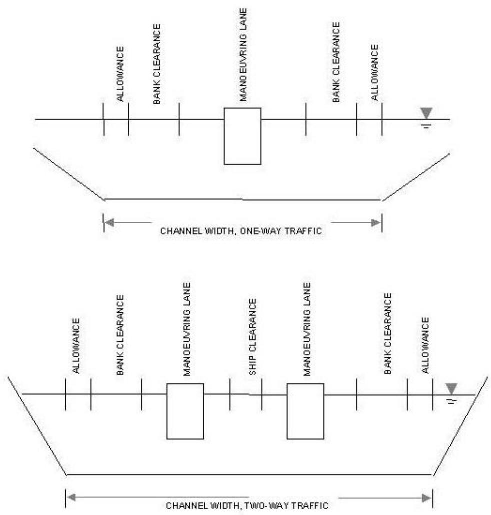


Figure B-2 Overview of the required width for a one-way channel and a two-way channel. Abstracted from Ligteringen 2009.

PIANC describes the width of a straight section with the following equations:

One-way channel:

$$W = W_{BM} + \sum W_i + 2\,W_B \tag{B.1}$$

For a two-way channel the separation between the two lanes (W_p) is added:

$$W = 2(W_{BM} + \sum W_i + W_B) + W_p \tag{B.2}$$

The calculated width is the width of the channel at the bottom.

The additions (W_i) are given in Table B.2.

The conditions are chosen similar for the both considered type of harbours. Although the small craft/fishery harbour won't require a dredged entrance channel, the opening between the breakwaters needs to fulfil the same requirements considering the width as the entrance channel. In both cases a one-way channel will be considered since a two-way channel will only makes the channel wider, making it more difficult to avoid sedimentation.

For the industrial harbour the required width will be:

$$W = 1.6\,B + 0.0B + 0.4B + 0.7B + 0.1B + 1.0B + 0.1B + 0.0B + 2 * 0.5\,B + 1.6B = 6.5\,B \approx 360\,m$$

For a smaller craft vessel the width will be:

$$W = 6.5\,B \approx 45\,m$$

Width component	condition		Width [m]
Basic width (W_{BM})	$d > 1.5\ D$		0.0
	$1.25\ D < d < 1.5\ D$		$1.6\ B$
	$d < 1.25\ D$		$1.7\ B$
Additional width (W_i)			
Vessel speed	Fast: $V > 12\ kn$		$0.1\ B$
	Moderate: ($8\ kn < V < 12\ kn$		0.0
	Slow: $5\ kn < V < 8\ kn$		0.0
		Speed	
Prevailing cross wind	Mild: $< 15\ kn$	All	0.0
	Moderate: $15 - 33\ kn$	Fast	$0.3\ B$
		Moderate	$0.4\ B$
		Slow	$0.5\ B$
	Severe: $33 - 48\ kn$	Fast	$0.6\ B$
		Moderate	$0.8\ B$
		Slow	$1.0B$
Prevailing cross currents	Negligible: $< 0.2\ kn$	All	0.0
	Low: $0.2 - 0.5\ kn$	Fast	$0.1\ B$
		Moderate	$0.1\ B$
		Slow	$0.2\ B$
	Moderate: $0.5 - 1.5\ kn$	Fast	$0.5\ B$
		Moderate	$0.7\ B$
		Slow	$1.0\ B$
	Strong: $1.5 - 2.0\ kn$	Fast	$0.7\ B$
		Moderate	$1.0\ B$
		Slow	$1.3\ B$
Prevailing long. currents	Low: $< 1.5\ kn$	All	0.0
	Moderate: $1.5 - 3.0\ kn$	Fast	0.0
		Moderate	$0.1\ B$
		Slow	$0.2\ B$
	Strong: $> 3.0\ kn$	Fast	$0.1\ B$
		Moderate	$0.2\ B$
		Slow	$0.4\ B$
Prevailing wave height	$H_s < 1$ and $L_{wave} < L_{oa}$	All	0.0
	$1 < H_s < 3$ and $L_{wave} = L_{oa}$	Fast	$2.0\ B$
		Moderate	$1.0\ B$
		Slow	$0.5\ B$
	$H_s > 3$ and $L_{wave} > L_{oa}$	Fast	$3.0\ B$
		Moderate	$2.2\ B$
		Slow	$1.5\ B$
Aids to navigation	Excellent with shore traffic control		0.0
	Good		$0.1\ B$
	Ordinary, visual and ship board Infrequent poor visibility		$0.2\ B$
	Ordinary, visual and ship board		$0.5\ B$

	Frequent poor visibility		
Seabed characteristics	Smooth and soft		$0.1\ B$
	Smooth/sloping and hard		$0.1\ B$
	Rough and hard		$0.2\ B$
Cargo hazard level	Low (dry bulk, break bulk, containers, passengers, general freight, trailer freight)		0.0
	Medium (oil in bulk)		$0.5\ B$
	High (LPG, LNG, chemicals)		$1.0\ B$
Width for bank clearance (W_B)	Sloping edge	Fast	$0.7\ B$
		Moderate	$0.5\ B$
		Slow	$0.3\ B$
	Steep and hard embankment	Fast	$1.3\ B$
		Moderate	$1.0\ B$
		Slow	$0.5\ B$
Separation distance (W_p)	Fast: $V > 12\ kn$		$2.0\ B$
	Moderate: $(8\ kn < V < 12\ kn$		$1.6\ B$
	Slow: $5\ kn < V < 8\ kn$		$1.2\ B$

Table B-2 The additions clarification of the required width of the entrance channel. B represents the width of the design vessel. The shaded cells are the values which will be considered in this study.

Channel depth

The required channels depth depends on:

- Draught of the design vessel;
- Squat: sinkage due to the vessels speed;
- Trim: unevenness keel due to loading conditions;
- Heave: ship's vertical movement due to waves;
- Water level, mostly related to tide;
- Channel bottom factors such as the variation in the dredged level and the effects of sedimentation.

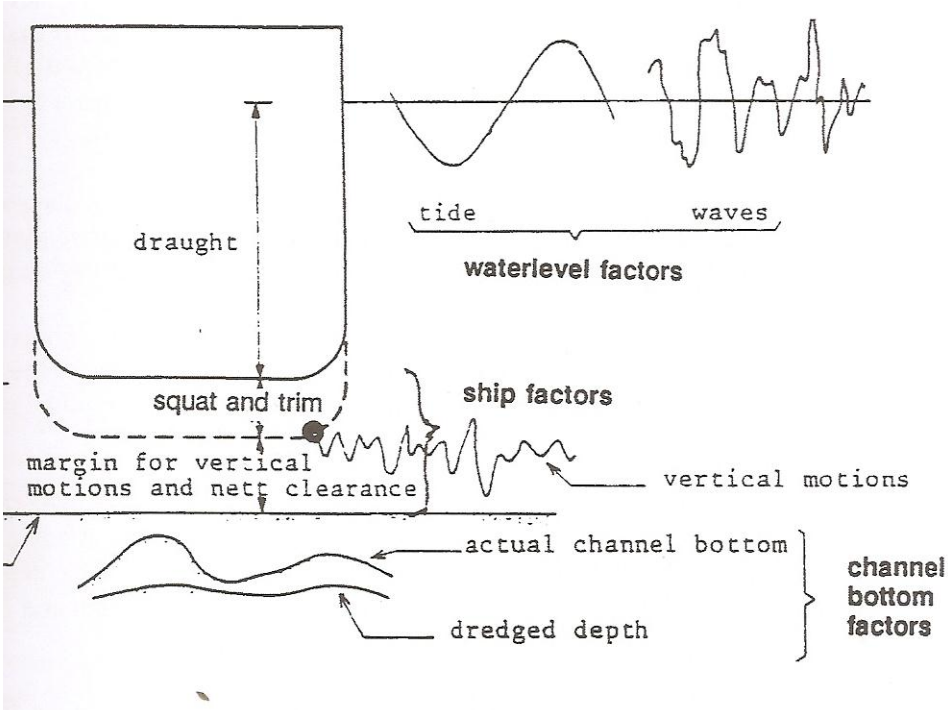


Figure B-3 Under keel clearance factors. Abstracted from Ligteringen 2009.

The required depth can be calculated as follows:

$$d_c = D + a_T + s_{max} + r + m_s \qquad \text{(B.3)}$$

The squat and trim depend on the vessels underwater shape relative to the depth (blockage coefficient) and the vessels velocity. Since the blockage coefficient is unknown the combined squat and trim is estimated to be 0.5 m in case of the industrial harbour. The squat and trim is negligible in case of the fishery port. The safety margin m is 0.5 for a sandy bottom. An approximation for the safety margin due to the vertical motion caused by waves, r, is 0.5 $H_s = 1.0\ m$. The maximum tidal elevation is estimated to be 1.5 m (spring tide).

Downtime

The required depth of the harbour also depends on the downtime a harbour authority wishes to accept. Downtime is the time during which ships cannot enter or leave the port. This is mainly caused by two processes; waves and tide and it depends on the harbour policy whether a downtime is allowed or not. The policy on both waves and tides can be very different. It is not uncommon to accept a certain downtime of a harbour in case of fierce storm conditions. It is simply not safe enough to allow vessels to enter or leave the harbour under these conditions. Downtime due to storms could be avoided but it often requires extensive structures.

Downtime due to the tide depends on the depth of the entrance channel and the draft of the vessel. A harbour could apply a tidal window for certain vessels. This basically means that these vessels can only enter the harbour in case of high tide. The port can decide whether to apply a tidal window or not. The decision of accepting downtime is mainly a financial choice. It could be more expensive to avoid downtime than the extra revenues it gives. A decision on accepting downtime is difficult to make without a specific case in mind since it depends on the local circumstances as well. It is assumed that downtime will not be accepted for the industrial harbour in this study and will be accepted for a smaller harbour. For the smaller harbour the vessels cannot enter during storm conditions and at low water during spring tide.

The required depth for the industrial harbour is than as follows:

$d = 15 + 1.5 + 0.5 + 0.5 + 1.0 = 18.5\ m$

The required depth for the smaller harbour is as follows:

$d = 2.5 + 1.0 + 0.5 + 0.0 + 1.0 = 5\ m$

The larger harbour will require a dredged channel and the smaller harbour can do without.

Length of the inner channel

The required area inside the breakwaters also determines the required length of the breakwater. The vessel should be able to stop once it is inside the breakwater. The length of the inner channel thus depends on the stopping distance of the vessels. The vessels could stop by using their own power only, but it is likely that the large vessels need tug assistance while stopping. In that case the stopping distance depends on the three factors:

1. The entrance speed of the vessel.
2. The time required for tying up the tugs and manoeuvring them in position.
3. The actual stopping length.

Ad 1.

The entrance speed of the vessel is determined by the requirements that, firstly, the vessel should have sufficient speed with respect to the surrounding water for proper rudder control, say 4 kn., and ,secondly, that the drift angle should not exceed a tangent of about $1:4$. This is hard to determine since this depends on the local current velocities in front of the harbour entrance. In the above chapter it is concluded that high flow contraction, and thus high velocities, have a positive effect on the ability for bypassing. Therefore a high current of 1.5 $kn.$ is assumed.

The effective velocity of the vessel can be determined with the following formula:

$$v_{eff} = 4(u \sin \alpha_{cc} + v_{wd}) \quad \text{(B.4)}$$

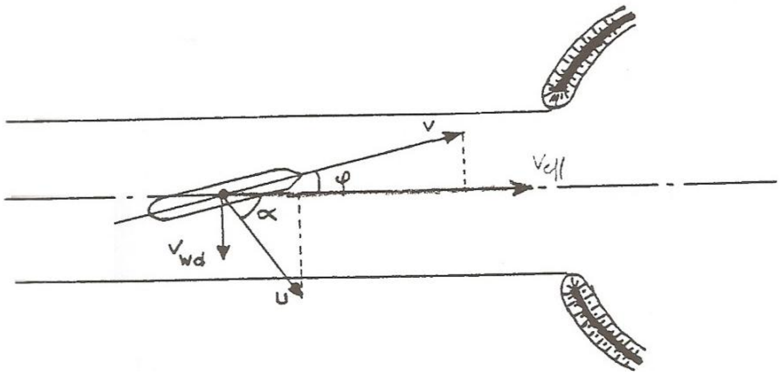


Figure B-0-2 Drift of the vessel under influence of the current and the wind. Abstracted from Ligteringen 2009.

The angle between the axis of the channel and the current is assumed to be 90^o, the current velocity is assumed to be 1.5 $kn.$ and wind drift is ignored. This gives an effective velocity of

$$v_{eff} = 4(1.5 * \sin 90 + 0) = 6\ kn$$

So it is assumed that the vessels could enter the harbour with a speed of 6 kn.

Ad 2.
The tying up time of the tugs depends on the wave conditions and the experience of the crew. At fierce wave conditions, say $H_s = 1.5\ m$, the tying up usually takes place inside the breakwaters. This is generally assumed to take 10 minutes.

Ad 3.
The actual stopping distance is relatively short. They already slow down to 4 $kn.$ the moment they enter the harbour. The large ships give astern power the moment the tugs can control the course of the vessel and subsequently stop in about 1.5 L.

The required length of the inner channel can be calculated as follows ($1\ kn. = 1\,852\ m/h$):

$$(6\ kn * 1/6\ hour * 1\,852\ m/h) + (1.5 * 250) \approx 2\,250\ m.$$

The required distance inside the breakwaters is 2 250 m. This only applies for the industrial port since stopping and manoeuvrability is not likely to be a problem in the case of small craft or fishery vessels. They can stop by using their own power in maximum $4 - 5$ ship lengths which is at maximum 125 m.

B.3. General geometry breakwaters

There are some more general considerations about the breakwaters.

The orientation should preferably be in the direction of the dominant wave direction. This way the waves come from the aft of the vessels. The orientation should also be such that waves do not penetrate into the harbour. In practice these two requirements lead to a small angle between the wave direction and the approach channel.

The breakwaters should not form a narrow "sleeve" but they should provide an open space immediately behind the entrance. The ships manoeuvring in the channel do not like hard structures close to the channel boundaries. Another reason is that ships need lateral space when going from an area with a cross-current to an area without a cross-current. The ship's bow is directed slightly against the current. This way the vessel sails forward. If the bow of the vessel passes the breakwaters, the stern will be pushed in the direction of the current. This brings the vessel of its track and it needs some space behind the breakwater to get back on track. As a final reason the waves diffract in the open space behind the breakwaters, reducing the effects of wave penetration.

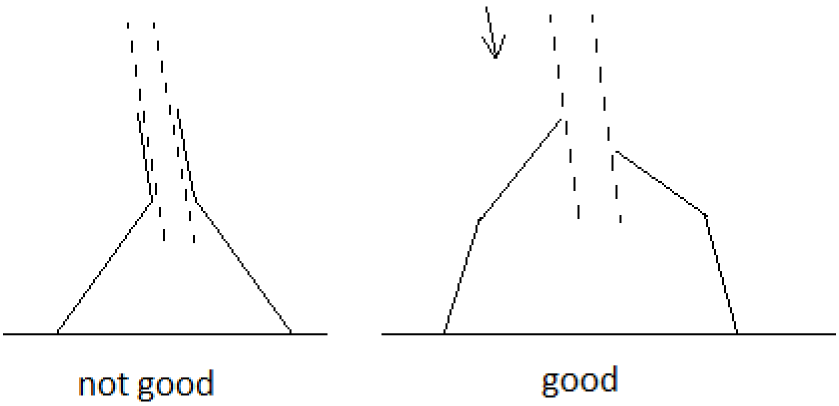


Figure B-5 A 'good' and a 'bad' design.

B.4. Conclusion

In this study two cases will be considered, a harbour with a dredged channel and a harbour without. The harbour with a dredged channel is considered to be a large industrial harbour accessible for large vessels. The harbour without a dredged channel is considered to be a small craft of fishery harbour. The type and size of the vessels determines the requirements for the entrance channel.

	Dredged channel	Depth channel $[m]$	Width channel $[m]$	Breakwater length $[m]$
Industrial harbour	Yes	18.5	360	$2\,250\ m$
Small harbour	No	5	45	$Various$

Table B-3 The properties per type of harbour

Note 1: The figures are chosen since no specific case is considered.

Note 2: Only one way shipping is considered

Note 3: The industrial harbour does not allow a tidal window; the small craft harbour is not accessible for the design vessel during ebb at spring tide.

Note 4: The length of the breakwaters in case of the small craft harbour is more or less variable. The most optimal solution will be sought for and the length of the breakwaters is a variable in that process.

APPENDIX C
FRAMEWORK

The filled in framework can be found on the next page. For each archetype the bypass characteristics are given. The most promising cases are also identified.

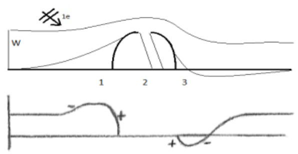
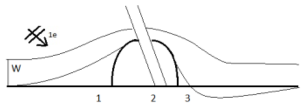
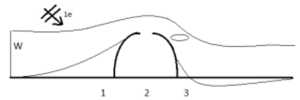
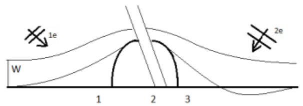
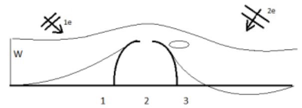
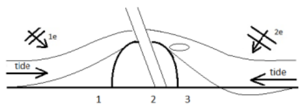
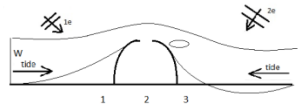
Harbour variation / Transport regime		Dredged entrance channel Breakwater length larger than the width surf zone.		No dredged entrance channel Breakwater length equal or smaller than the width surf zone.	
Large net transport $S_{1e} \gg S_{2e}$ 	Wave dominated	1.  **Not promising due to sedimentation in channel.**	* Accretion at 1, erosion at 3 * Initially small flow contraction * Initial the breakwater will be longer than the width of the surf zone. After some accretion the surf zone has shifted offshore and S_2 starts to develop. The flow contraction grows as well. * Sedimentation entrance channel, due to flow expansion and accretion at 1→ no bypassing before channel is filled up. * The little accretion against the breakwater at 3 is not likely to give a bathymetry at which the wave angle is good to have $S_3 \approx S_2$	2.  **Promising if flow contraction and turbulence is strong enough to create $S_1 \approx S_2 \approx S_3$. Archetype # 6 is more promising due to bathymetry at 3.**	* Accretion at 1, erosion at 3. * Bypass could start quickly due to small blockage coefficient. * Flow contraction at 2, decreases after some time due to accretion at 1. * Large wave influence at 2 due to limited extent of the harbour. This could counteract the decrease of the flow contraction due to the accretion at 1. * The little accretion against the breakwater at 3 is not likely to give a bathymetry at which the wave angle is good to have $S_3 \approx S_2$. A shoal formation at 3 might be able to achieve that.
Small net transport $S_{1e} > S_{2e}$ Or $S_{1e} \approx S_{2e}$	Wave dominated	3.  **Not promising due to sedimentation in channel.**	* Similar as archetype # 1 but less erosion at downdrift coastline. * If there is an S_2 this sediment is more likely to be transported onshore due the accretion at 3. This might give the right orientation to let $S_3 \approx S_2$	4.  **Archetype # 6 is more promising because of more hydrodynamic forcing.**	* Similar as archetype # 2 but less erosion at downdrift coastline. * Accretion at 3 could increase the capacity to transport bypassed sediment onshore again. In that case $S_3 \approx S_2$. A shoal formation at 3 could enhance that.
	Wave and tide	5.  **Not promising due to sedimentation in channel.**	* Same as archetype # 3 and # 1 but with more hydrodynamic forcing due to tide. * The tide spreads out the transport over a wider area, reducing the blockage of the breakwaters. * The tide could also enhance the flow contraction at 2, limiting the effect of the flow expansion due to the dredged channel. * Spurs could be effective due large flow velocity. This could limit the effect of the flow expansion at channel.	6.  **Most promising case due to largest hydrodynamic forcing and low blockage. The flow contraction and spur-induced turbulence at 2 and the bathymetry at 3 could create $S_1 \approx S_2 \approx S_3$.**	* Same as archetype # 4 and # 2 but with more hydrodynamic forcing due to tide. * The tide spreads out the transport over a wider area, reducing the blockage of the breakwaters. This also enlarges the flow contraction. * Spurs could be effective due large flow velocity.

Table C-1 Filled in framework, the horizontal axis shows the differentiation of the harbours; this is categorised in harbours with and without a dredged channel. The vertical axis shows the categorization of the transport regimes. This is differentiated in a large and a small net transport.

APPENDIX D
REFERENCE CASES

There are some relevant reference cases which correspond to the archetypes defined. The relevant aspects of these reference cases are explained. In this chapter

D.1. Hantsholm - Denmark

The harbour of Hantsholm corresponds to archetype 4 and 6 of the framework. The harbour of Hantsholm is wave dominated but there is a strong current due to its location. Therefore is shows features of an archetype 6 situation as well.

Hantsholm harbour is an example of a very successful bypass harbour, located at a headland at the North of Denmark. This coastline is very exposed to waves and has a very oblique wave approach. The location and the geometry of the harbour were chosen to minimize the sedimentation. There is a North-West-ward littoral drift of $0.7\ Mm^3/y - 1.0\ Mm^3/y$, corresponding to a net NW transport of $0.4\ Mm^3/y$ and gross transport of $1.1 - 1.5\ Mm^3/y$.

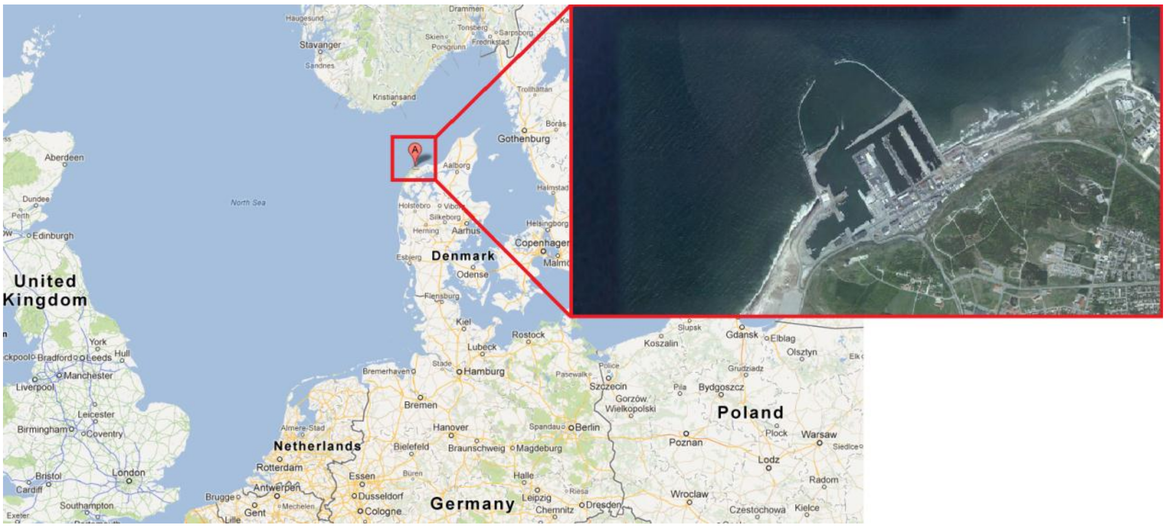


Figure D-1 Location and geomery of the harbour of Hanstholm at the Danish coastline.

The symmetrical and semi-streamlined geometry creates a smooth convergence of the flow past the harbour entrance and has, in combination with the vertical breakwater fronts, resulted in optimal bypass conditions and acceptable sedimentation rates. The natural depth in front of the harbour is 9 m, making the harbour suitable for fishery and ferry operations. The flow is mainly driven by meteorological forcing; variations in wind and pressure and, to a smaller extend, by wave breaking. The tide is very limiting in this area.

The good bypass conditions are created by:

- The location at the headland, which causes additional meteorologically-driven currents which accommodate bypass and a large natural depth;
- The streamlined geometry of the breakwaters;
- The vertical face of the breakwaters;
- The oblique wave climate.

Conclusion
This situation shows similarities with archetype 4 which is also wave dominated and experiences a small net transport related to the gross tranapoer. The difference is that the current at the Hanstholm habrour are faster due to its location. That effect could very well be caused by a strong tide as well, which is absent in Hanstholm. That would be similar to archetype 6.

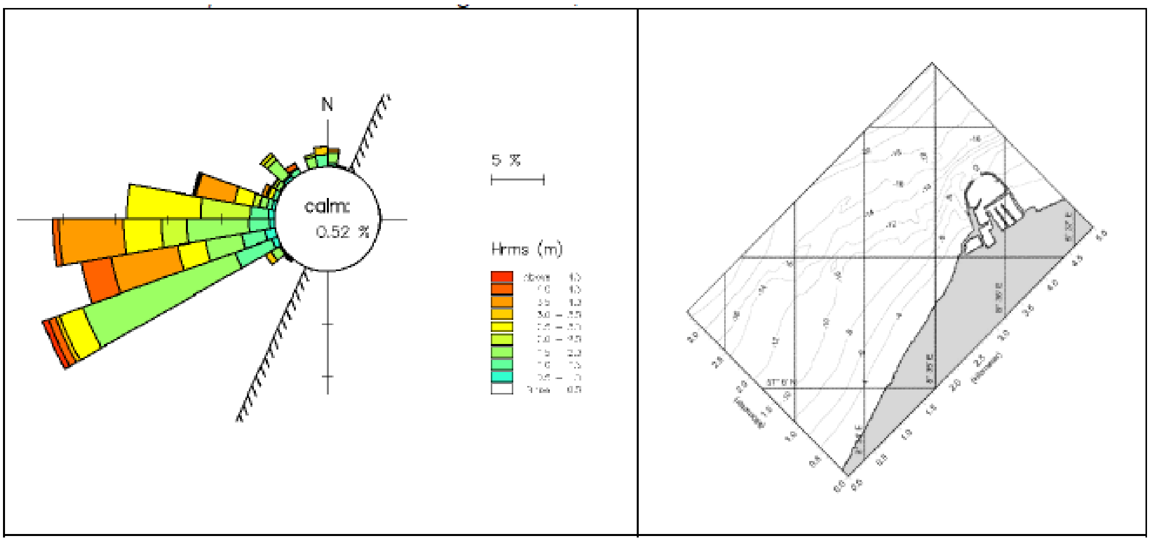


Figure D-2 Wave rose at 20 m depth off Hanstholm. Abstracted from Mangor 2010.

D.2. IJmuiden - The Netherlands

The harbour of IJmuiden corresponds to archetype 5 of the framework.

Figure D-3 Location of the harbour of IJmuiden at the Dutch coastline.

The harbour of IJmuiden is located at the Dutch west coastline. The harbour provides access for rather large vessels up to the port IJmuiden and Amsterdam. The transport regime is wave and tide influenced. There is a large gross transports at which the northward transport is larger than the southward transport, resulting in a relative small net transport. The harbour is protected by two streamlined breakwaters which extended approximately 2.5 km from the original coastline. Just behind the harbour entrance there is a discharge system during ebb, an outfall from TATA Steel and a sluice complex to provide access to the harbours behind.

This situation has been modelled with a Delft3D package which included the following forcings:

- Meteorological forcing;
- The water discharge from the discharge system in the harbour;
- The effects of the pump station;
- The effects of the locks;
- The intake and outfall effects of cooling and/or process for power stations and/or industries;
- The boundary conditions from surround models starting at the Atlantic.

Figures D-4 and D-5 show the surface flow velocity as it is modelled during an ebb/flood cycle. The upper image shows a top view of the harbour area. The surface velocity magnitude is represented in the colour bands and the length of the black arrows. The arrows also indicate the direction of the current. The smaller upper right image shows the water level as a function of the time. In the plan view, there is a red and magenta line. The red line follows the discharge channel to the discharge gate. The magenta line goes to the largest ships gate. The lower images show the velocity over the depth along these lines.

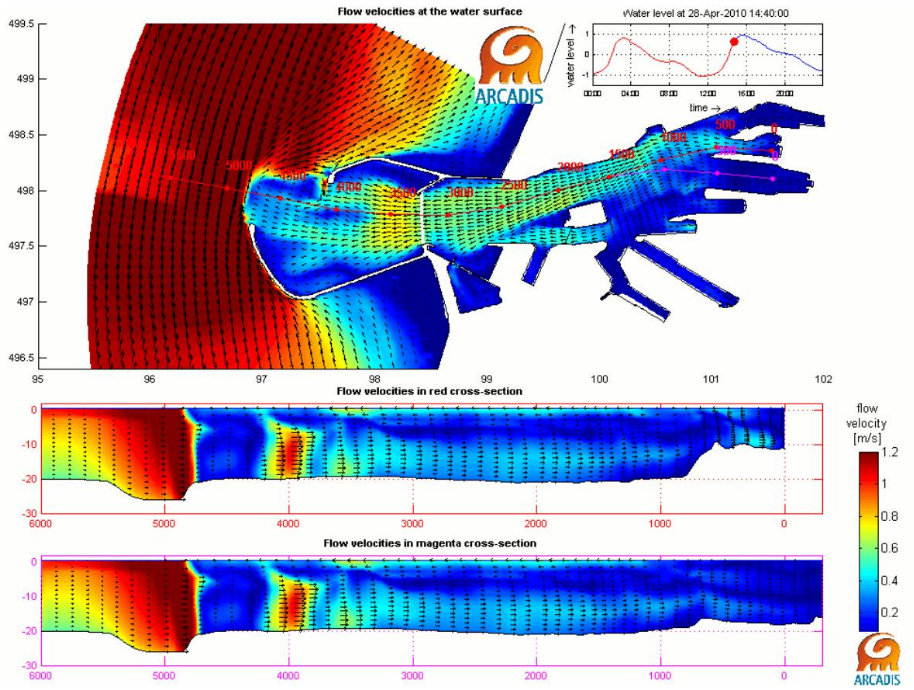


Figure D-4 Surface velocity at the inner and outer IJmuiden harbour at flood according to Arcadis (2011).

The model simulates the surface velocity of the inner and outer harbour over a period of 12 $hours$ and 25 $minutes$. The simulation shows that, especially the flood currents, contracts highly in front of the harbour. The velocity increases up to 1.2 m/s over almost the entire depth in front of the breakwaters. The high currents are very close to the breakwaters and separate behind them. That leads to eddy formation at the entrance. At the downdrift side the flow expands and decreases in velocity. The ebb currents are significantly less but in this case the discharge provides rather high velocity around the tip of the breakwaters.

It is known from survey that there is a very deep erosion gap at the tip of the largest breakwater. The high velocity along breakwater due to flow contraction, the in- and outgoing tidal current and the eddies contributes to erosion in front of the harbour. The large extension of the harbour also traps the sediment updrift and downdrift of the harbour.

The harbour of IJmuiden shows that, given there is sufficient flow contraction, the depth in front of entrance can be large enough. It should also be noted that these breakwater extend extremely far into and well beyond the width of the surf zone, blocking all sediment. We can therefore not speak of bypassing but merely of erosion in front of the entrance. In this case the erosion is a threat for the stability of the harbour breakwaters.

Conclusion

The harbour of IJmuiden corresponds to archetype 5 of the framework. It shows that if it the flow contraction is sufficient and there is an in and outflow of water due to a tide or discharge it is possible to keep the entrance deep enough. It however requires very long breakwaters which trap all the sediment. One can therefore not speak of bypassing. This is relevant for this study since it shows the effect of flow contraction on the morphology around the harbour.

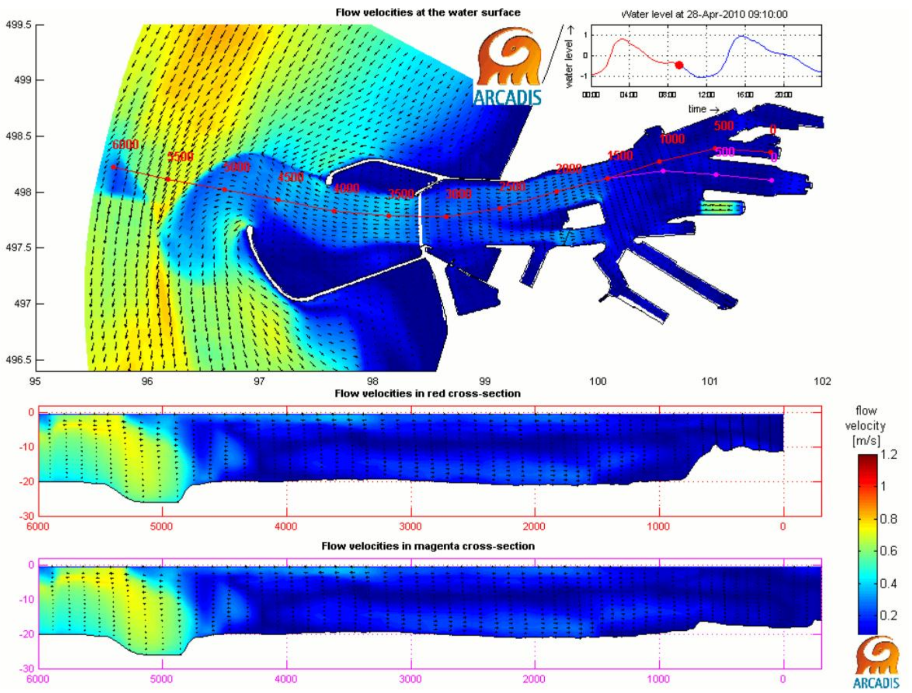


Figure D-5 Surface velocity at the inner and outer IJmuiden harbour at ebb according to Arcadis (2011).

APPENDIX E.
FIGURES RESULTS

This appendix gives the figures on which the analysis are made. For each case the situation is summarized. The value for the duration between brackets is the morphological time scale.

E.1. Assessment of the influence of the breakwater length

The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 350 m, 500 m and 675 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: semi-streamlined.

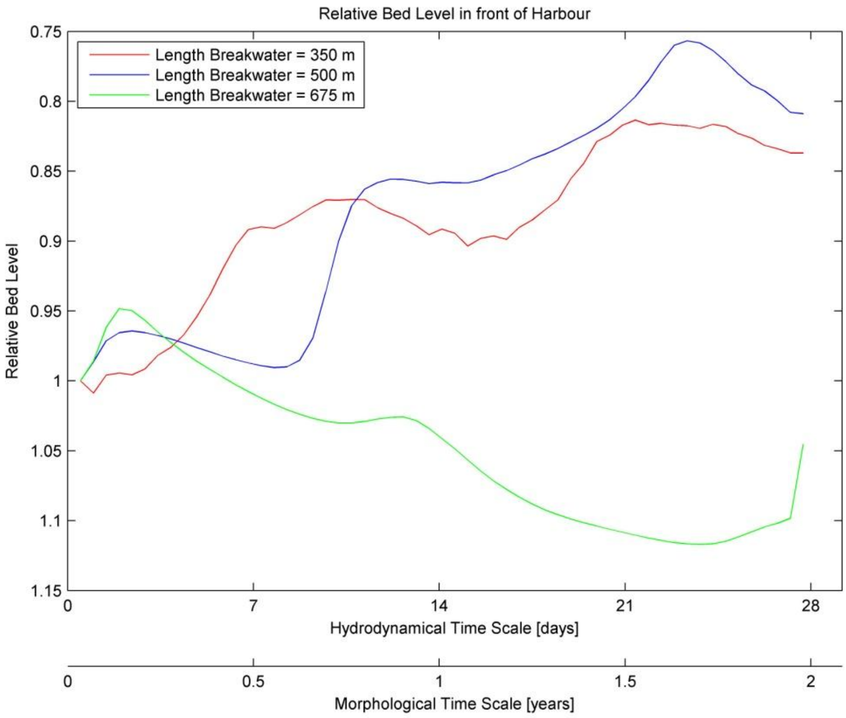


Figure E-1 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

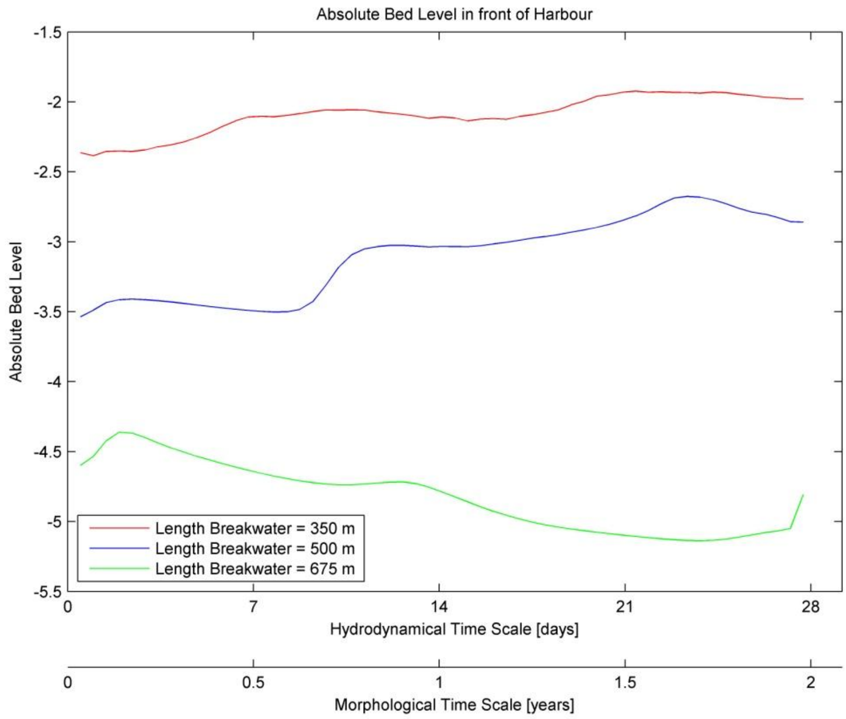


Figure E-2 Plot of the absolute bed level in time in front of the harbour.

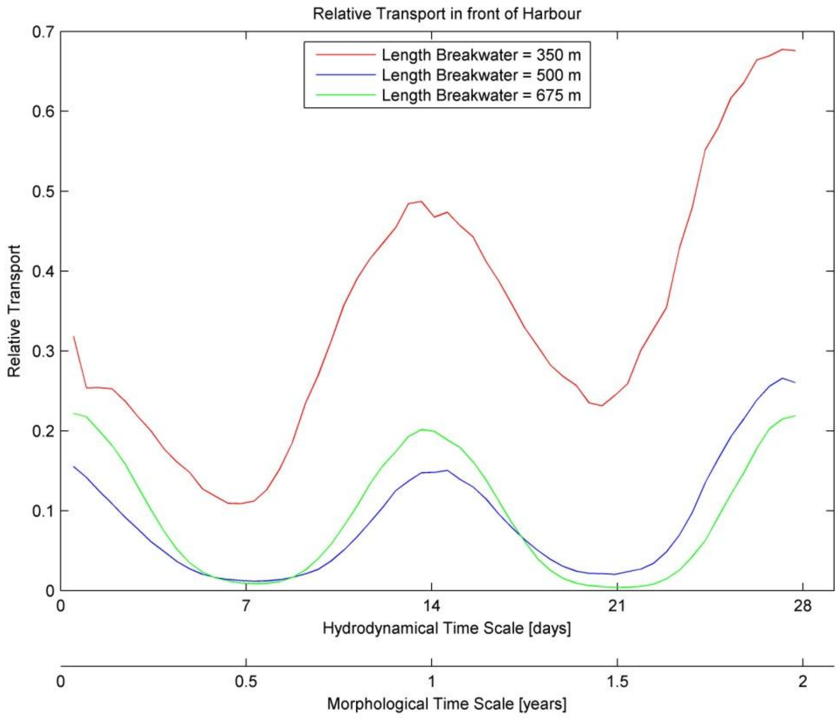


Figure E-3 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

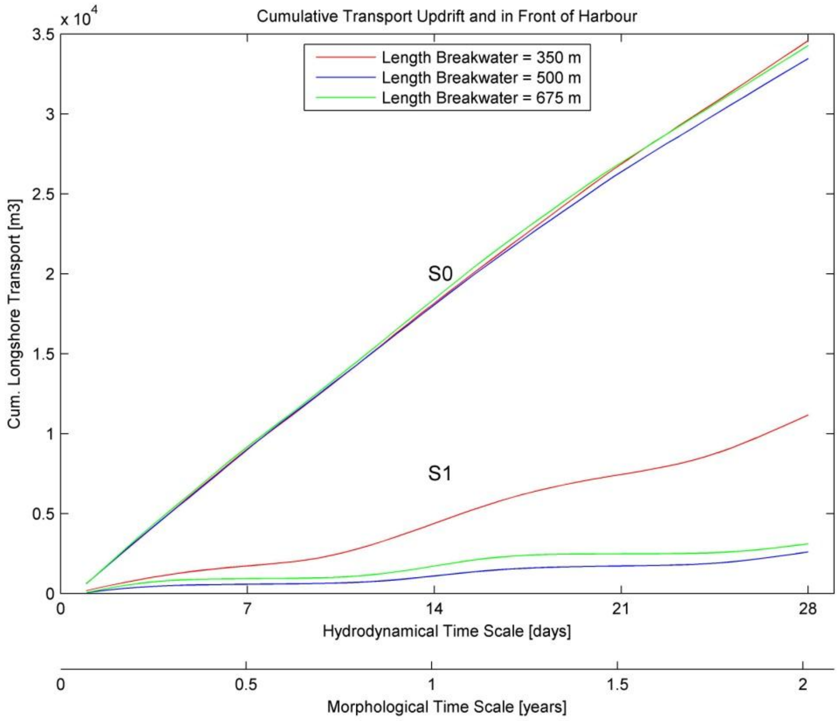


Figure E-4 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

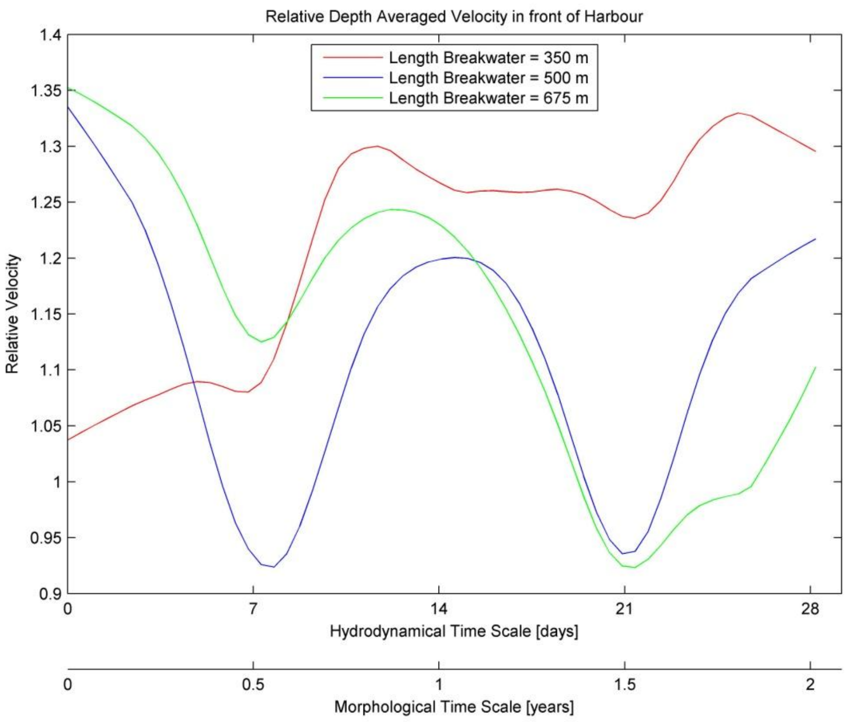


Figure E-5 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

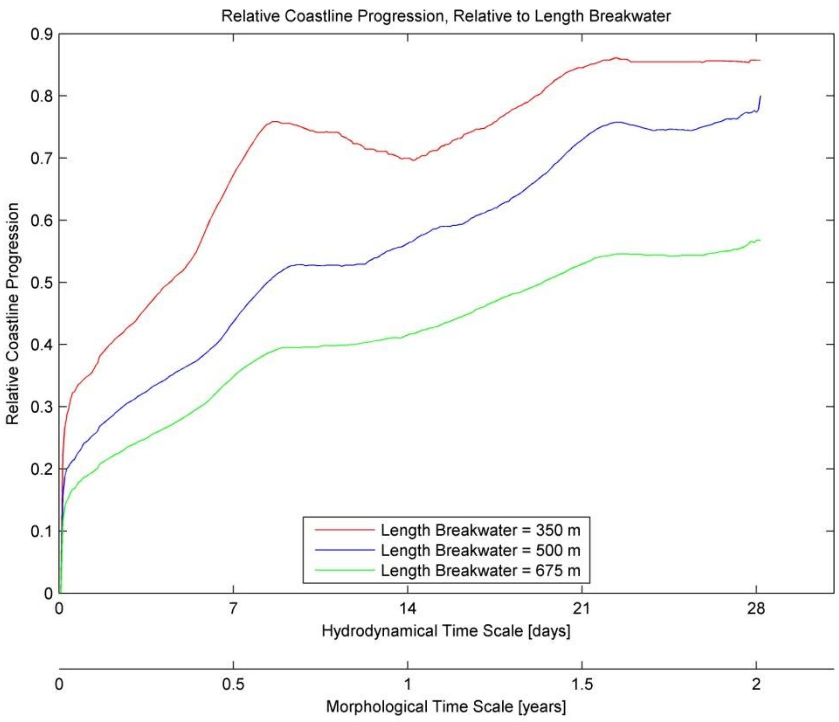


Figure E-6 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

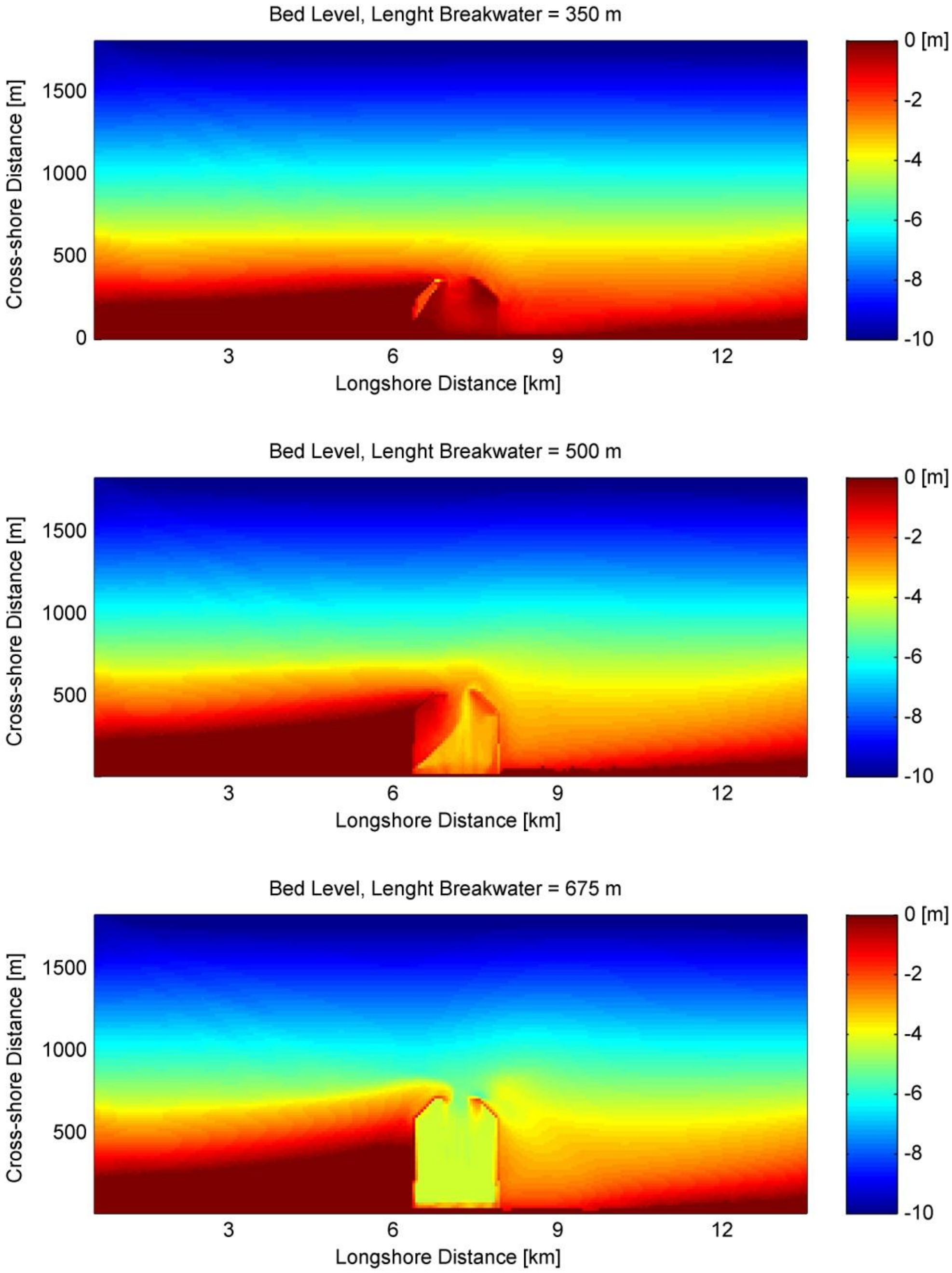


Figure E-7 Three plots of the bed level at the end of the calculation.

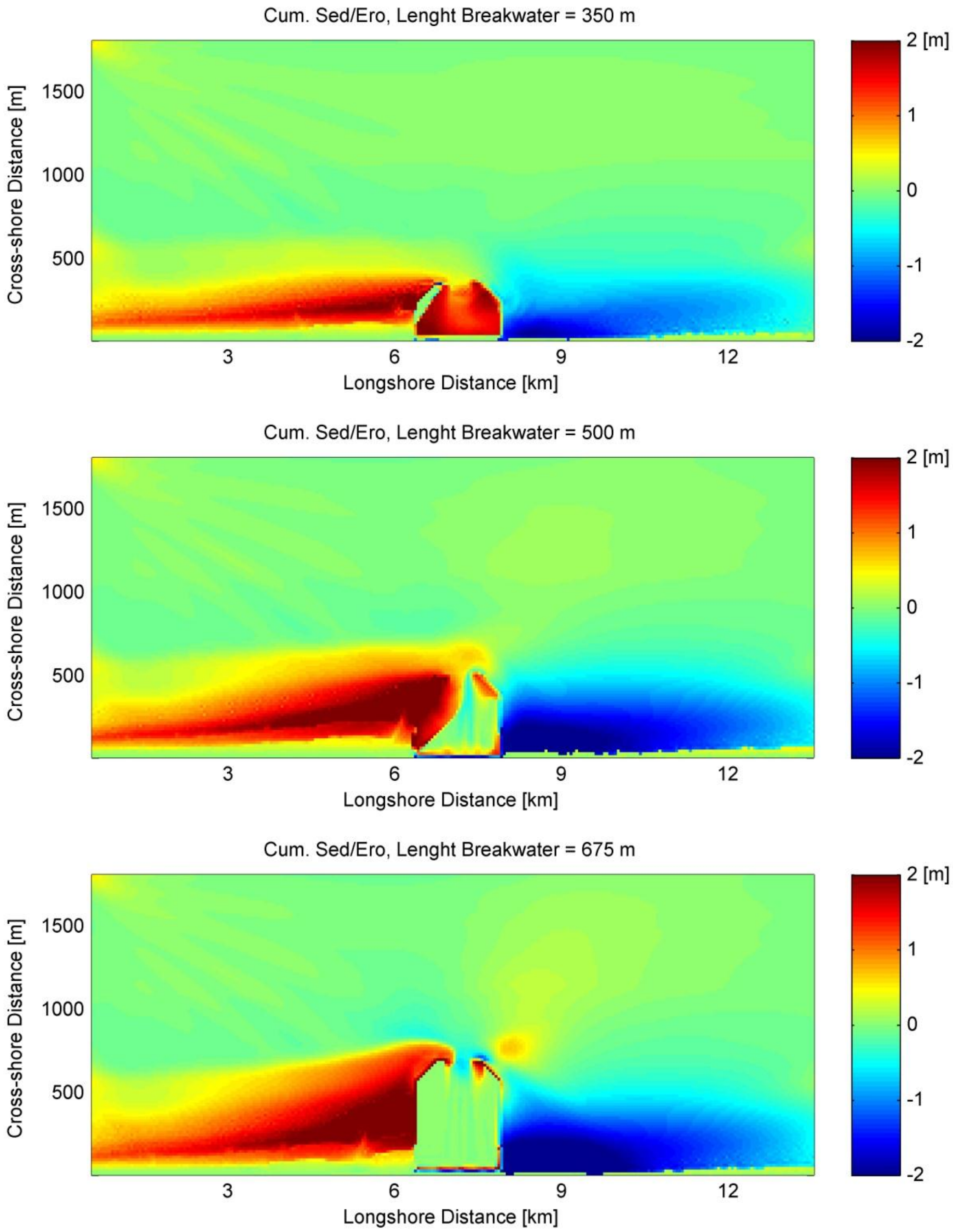


Figure E-8 Three plots of the cumulative sedimentation/erosion at the end of the calculation.

E.2. Assessment of the influence of the wave angle

The following settings are applied:

- Wave height: $1\ m$;
- Wave angle: $15^o, 30^o, 45^o, 60^o$ and 75^o.
- Wave period: $7\ sec$;
- Length breakwater: $500\ m$;
- Simulation time: $1\ month\ (2\ years)$;
- Breakwater geometry: semi-streamlined.

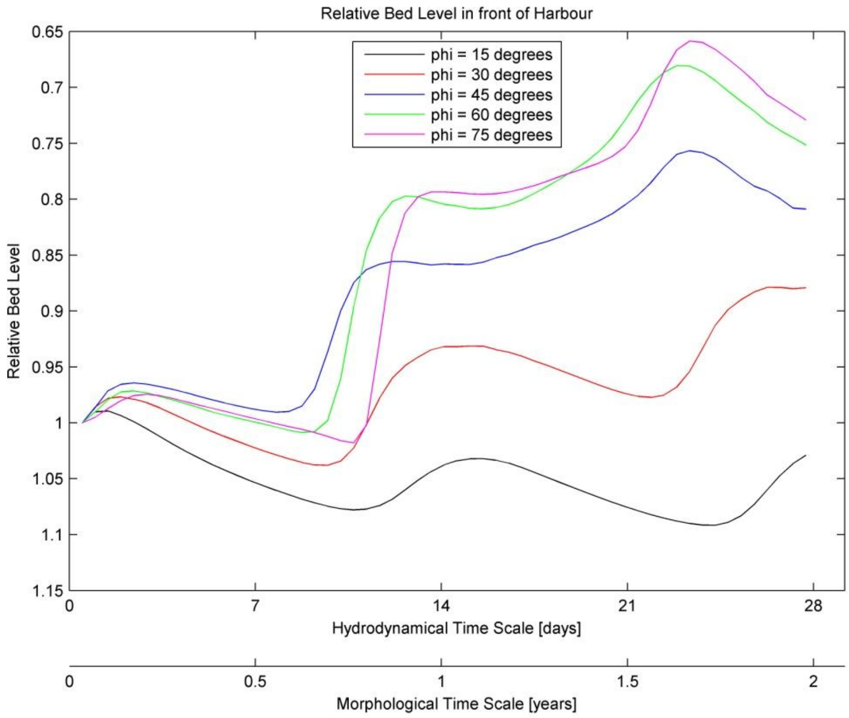


Figure E-9 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

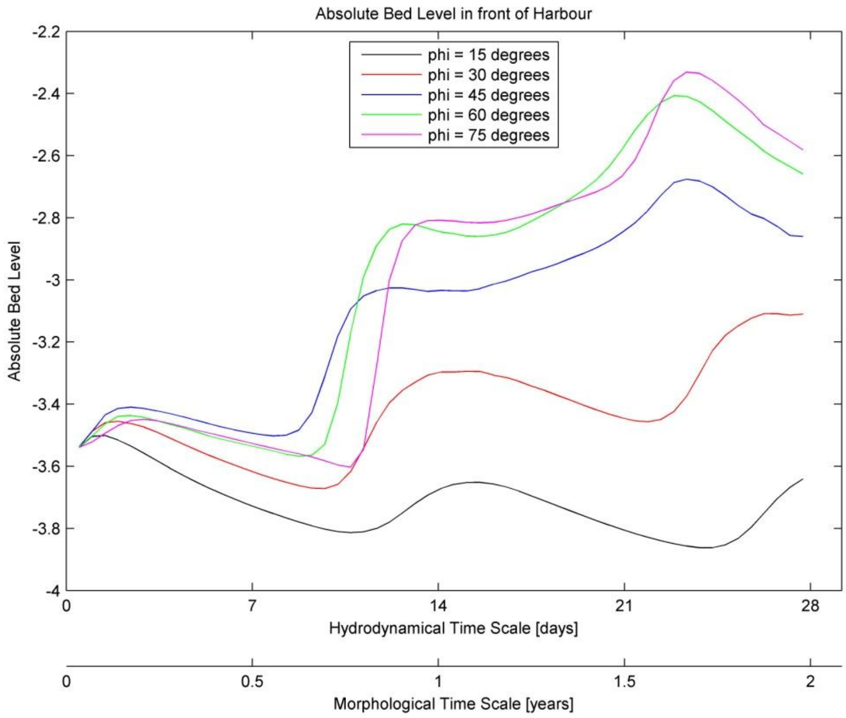


Figure E-10 Plot of the absolute bed level in time in front of the harbour.

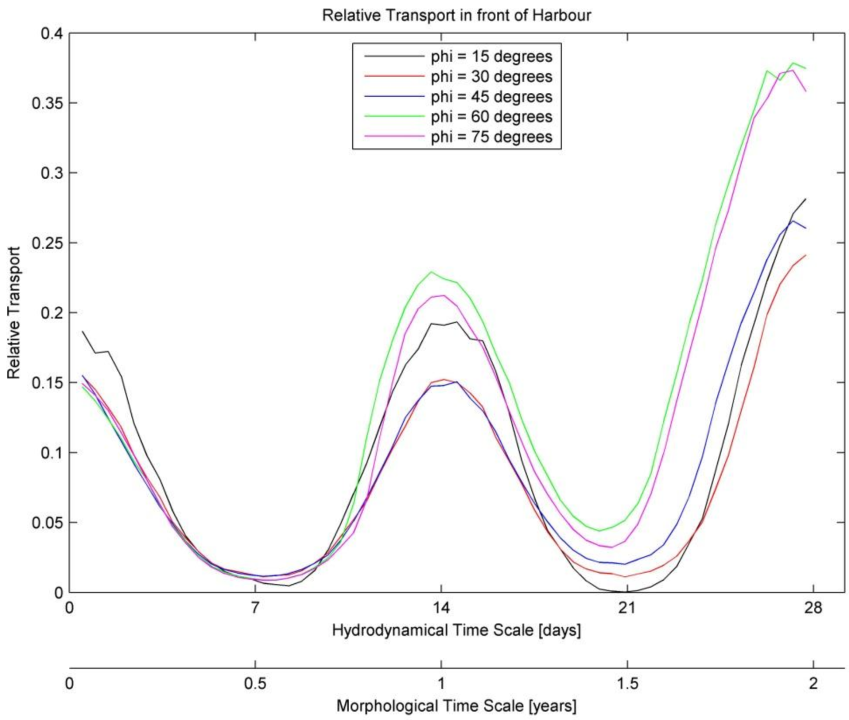


Figure E-11 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

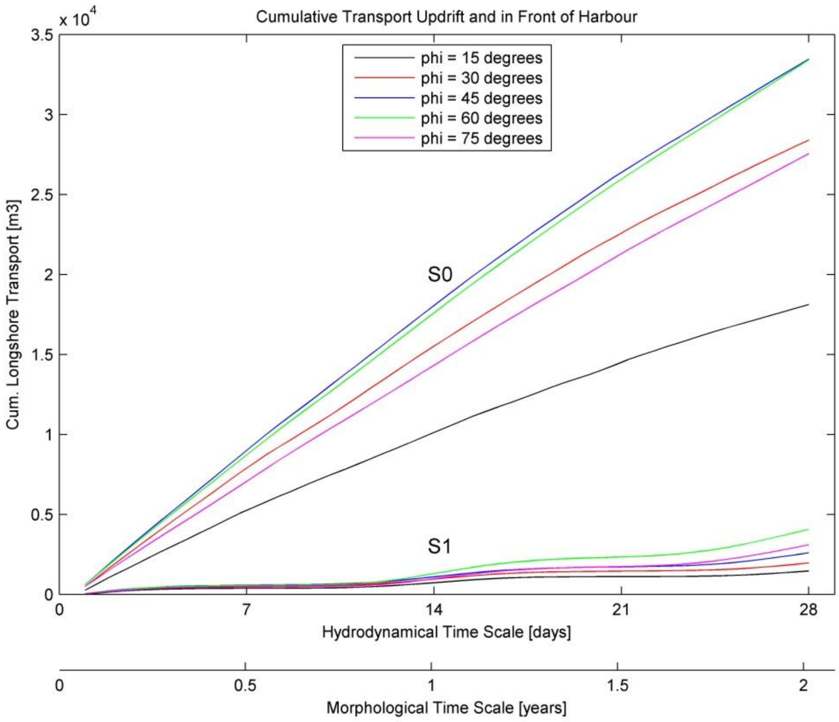


Figure E-12 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

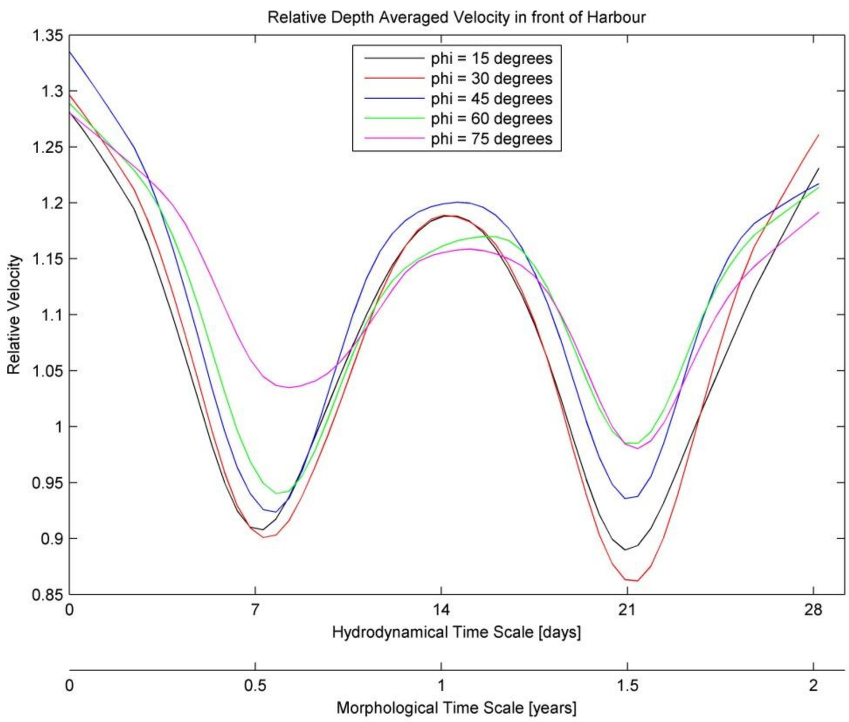


Figure E-13 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

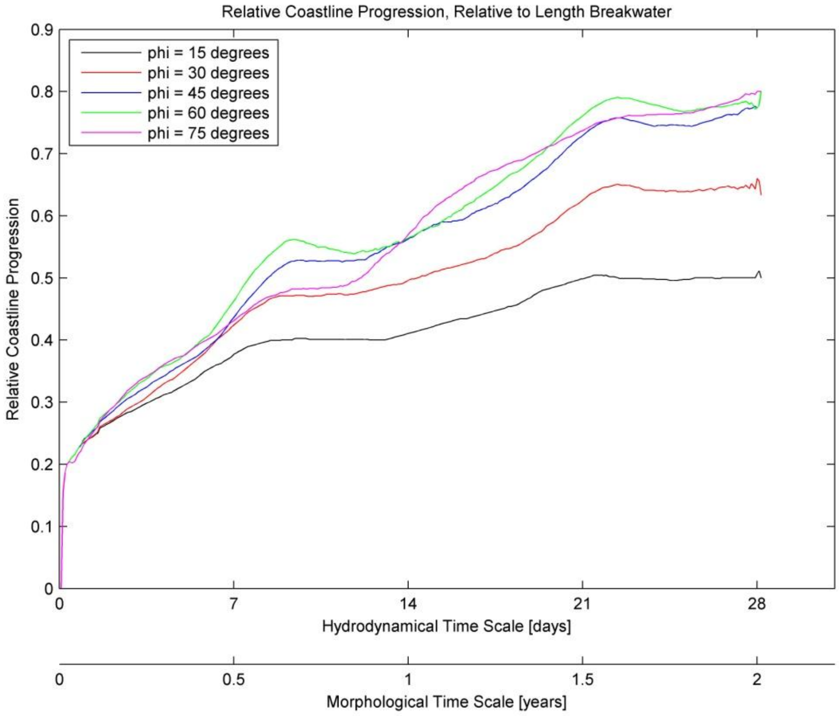


Figure E-14 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

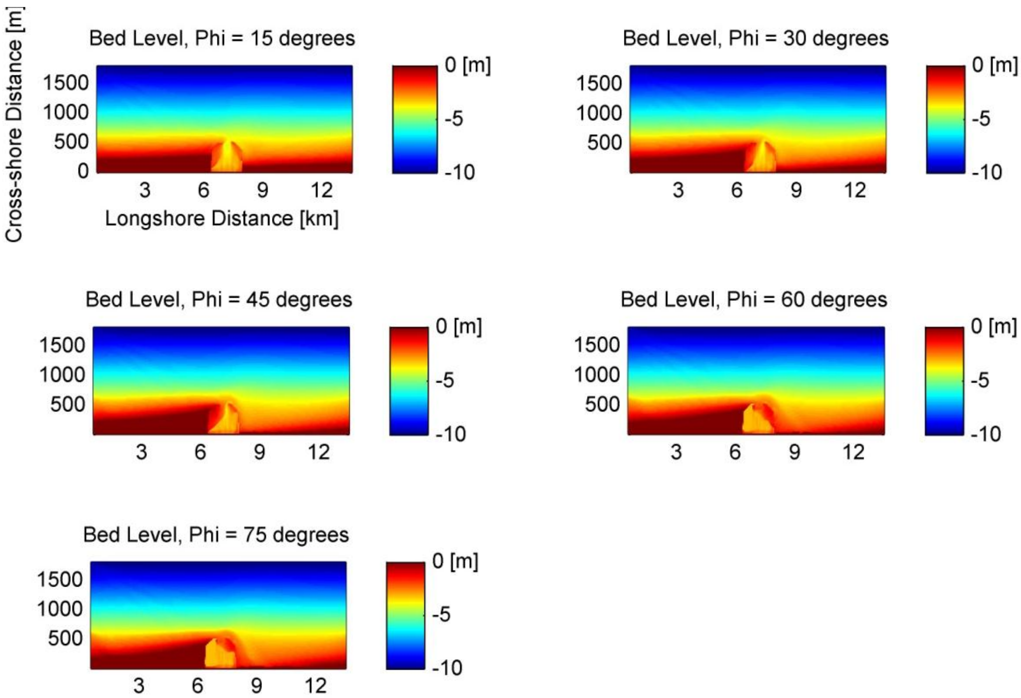


Figure E-15 Five plots of the bed level at the end of the calculation.

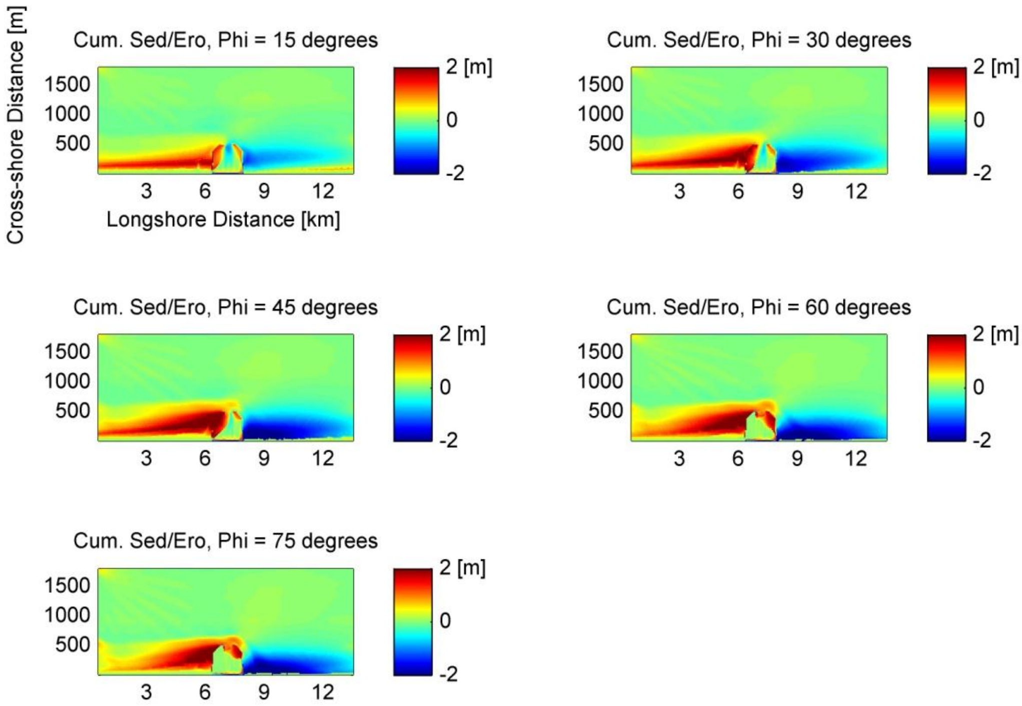


Figure E-16 Five plots of the cumulative sedimentation/erosion at the end of the calculation.

E.3. Assessment of the influence of the wave height

The following settings are applied:

- Wave height: 1 m and 1.5 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 500 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: semi-streamlined.

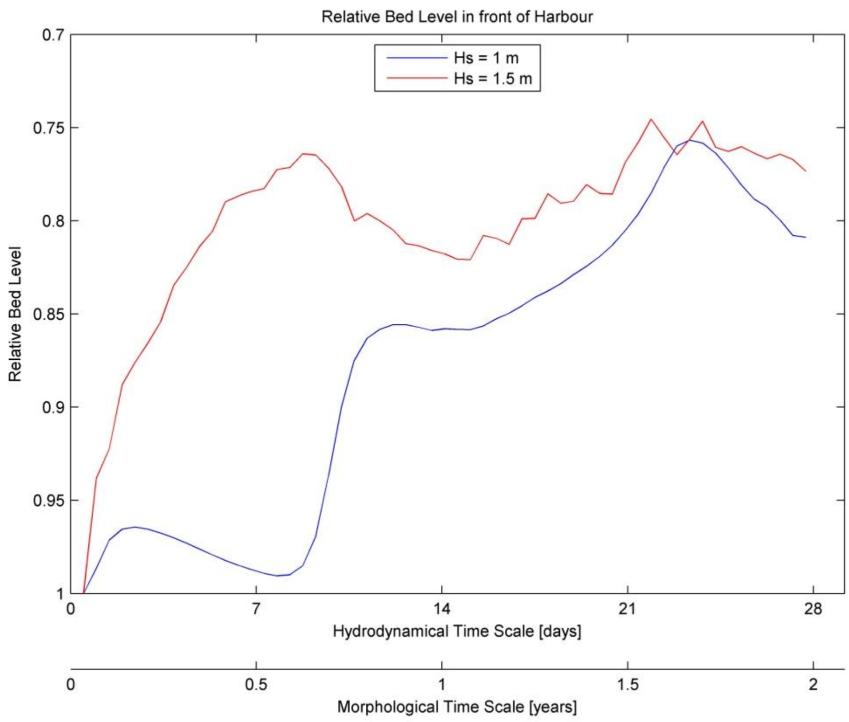


Figure E-17 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

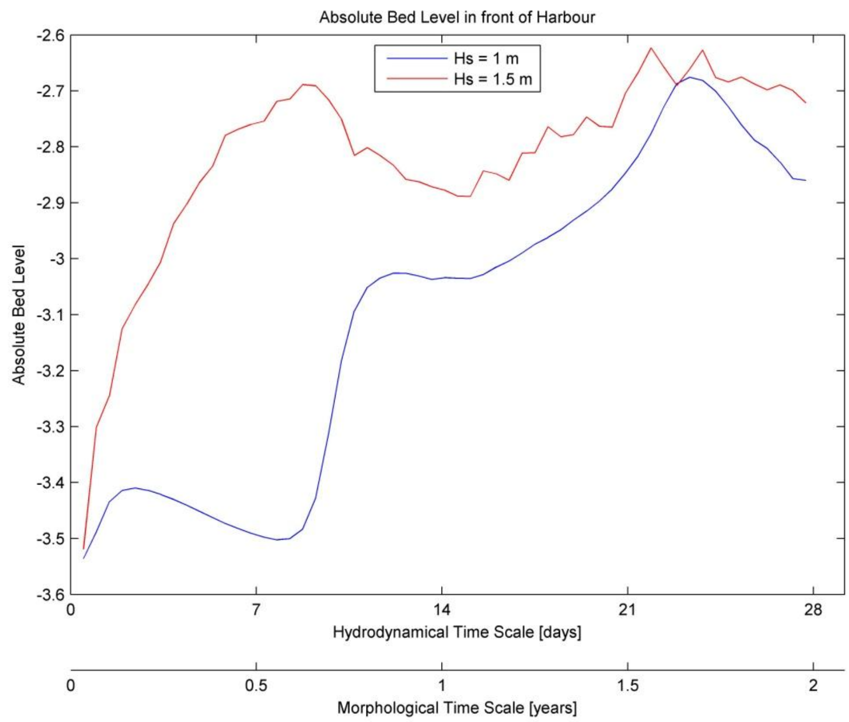


Figure E-18 Plot of the absolute bed level in time in front of the harbour.

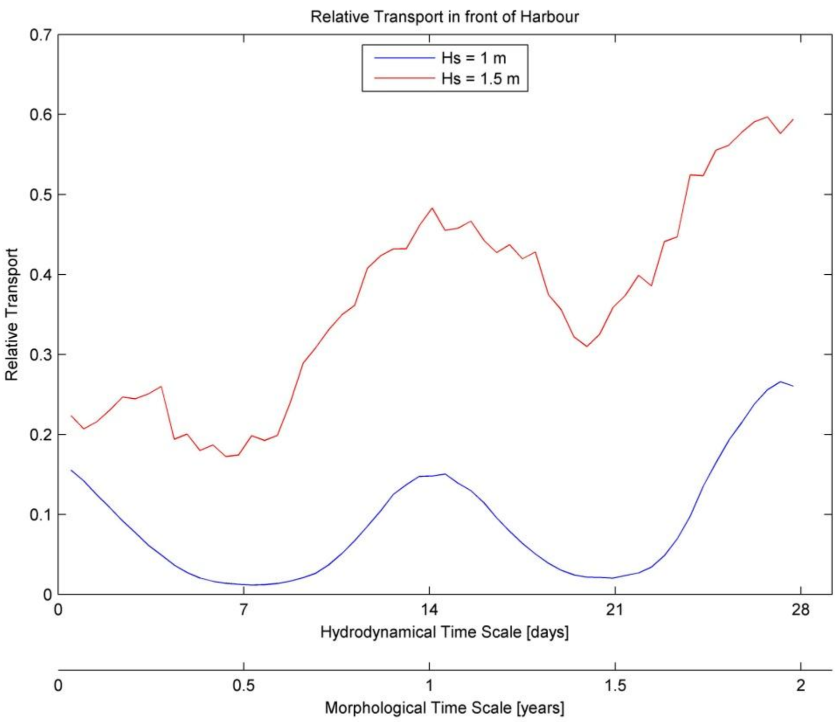


Figure E-19 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

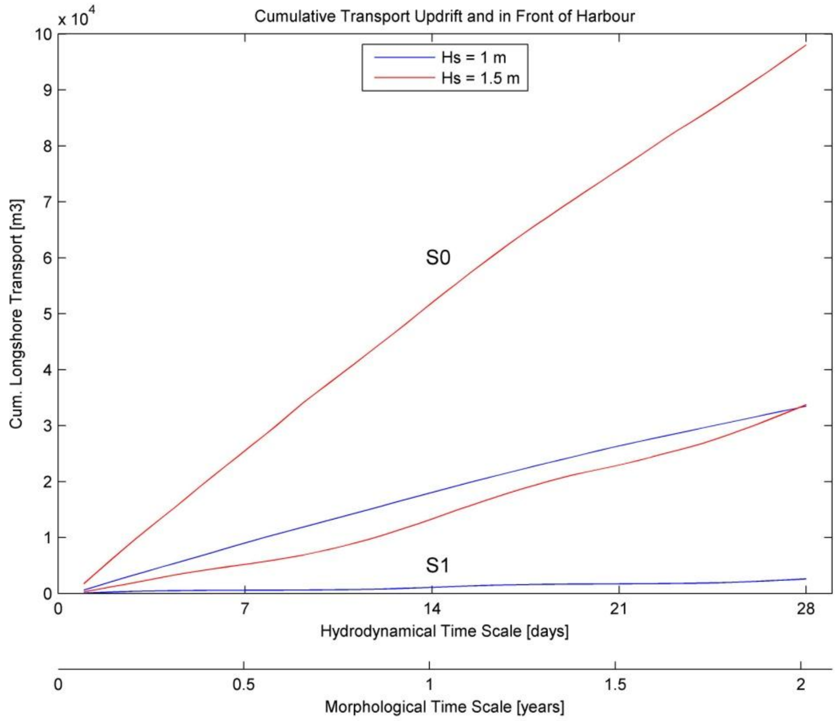


Figure E-20 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

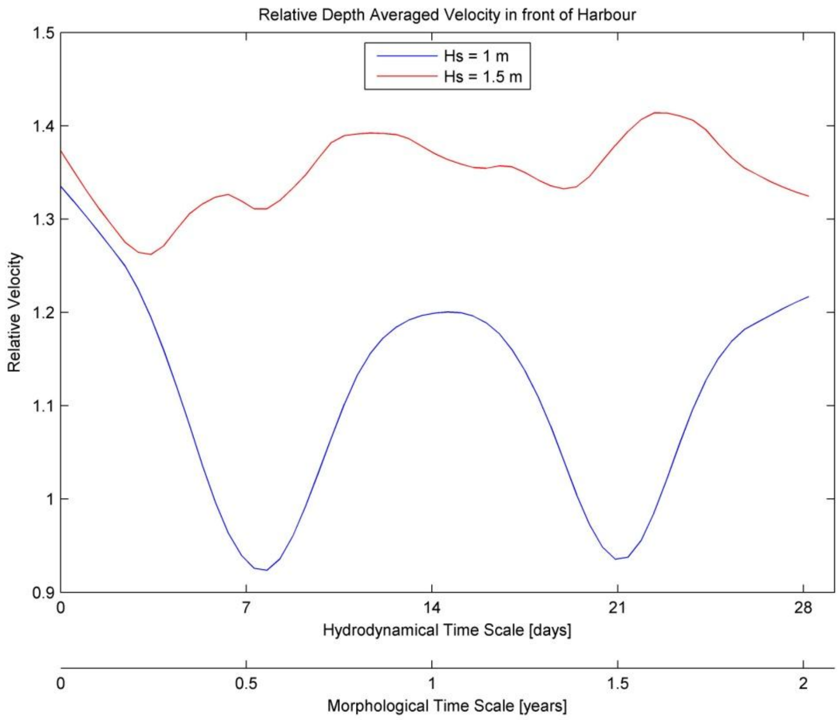


Figure E-21 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used

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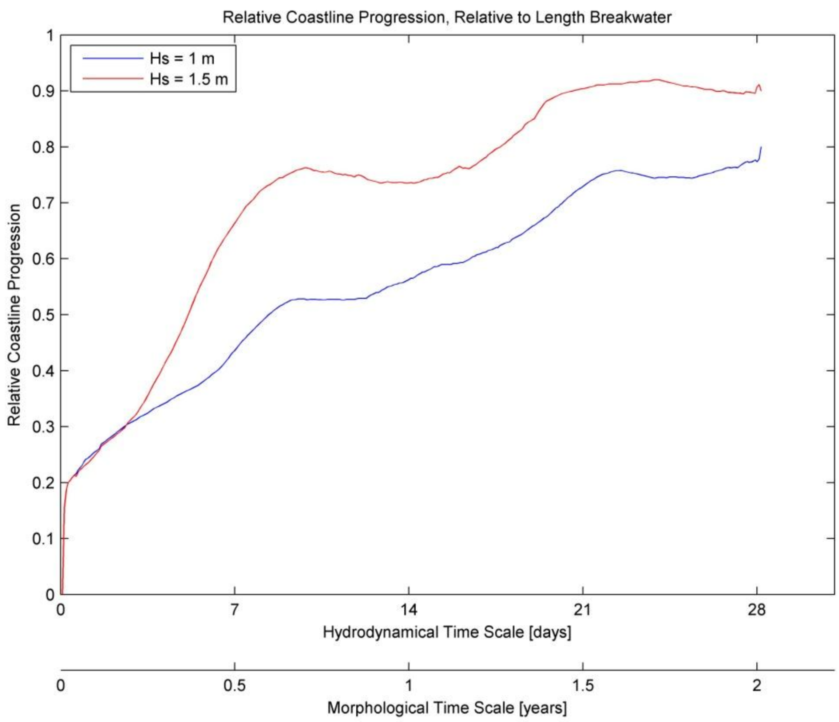

Figure E-22 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

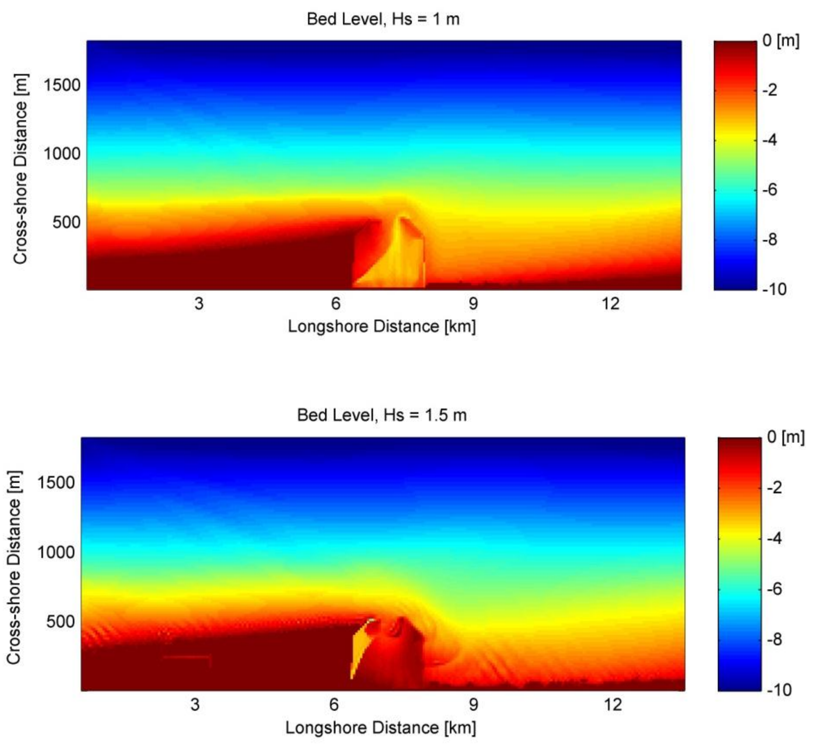

Figure E-23 Two plots of the bed level at the end of the calculation

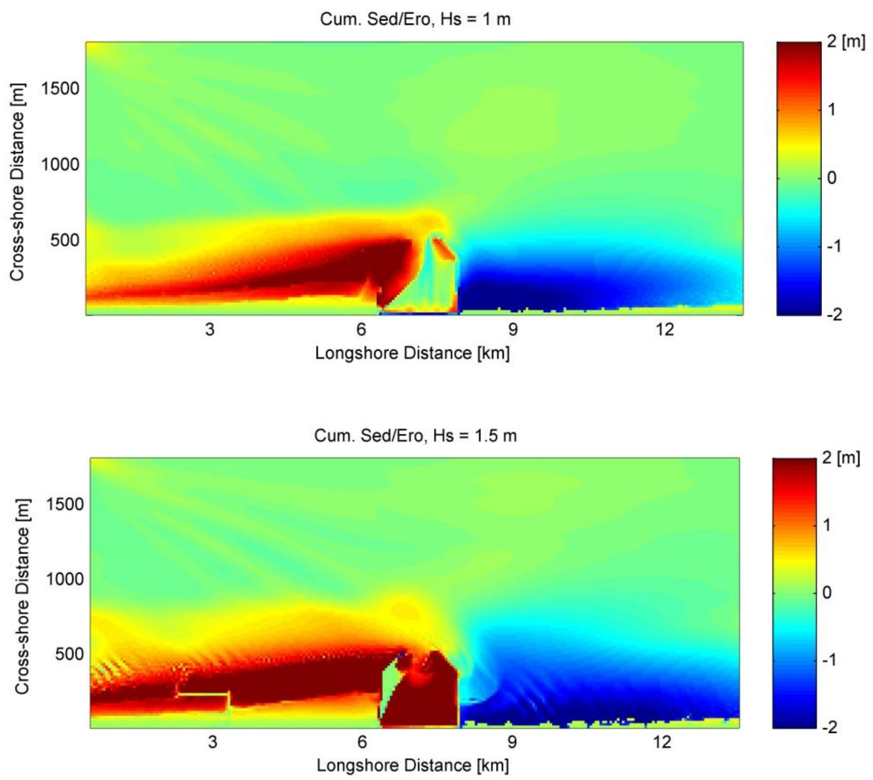

Figure E-24 Two plots of the cumulative sedimentation/erosion at the end of the calculation.

E.4. Assessment of the influence of the geometry of the breakwaters

The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 500 m and 350 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry variable.

The following geometries are assessed:

1. Semi - streamlined $L_{BW} = 500\ m$;
2. Semi - Streamlined with spurs $L_{BW} = 500\ m$;
3. Streamlined $L_{BW} = 500\ m$;
4. Streamlined with spurs $L_{BW} = 500\ m$;
5. Semi - streamlined $L_{BW} = 350\ m$;
6. A main breakwater covering the secondary breakwater $L_{BW} = 350\ m$.

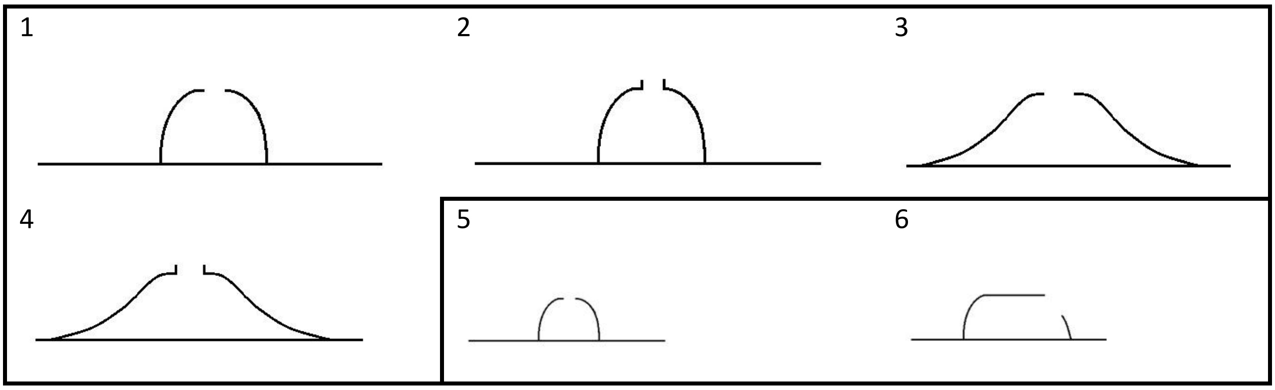

Table E-1 Different breakwater geometries that are assessed.

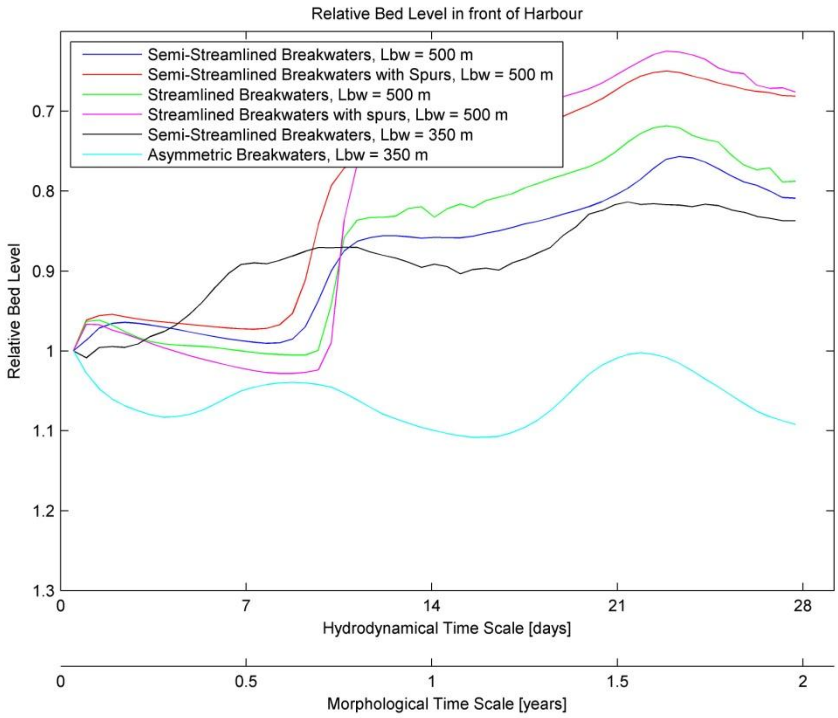

Figure E-25 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

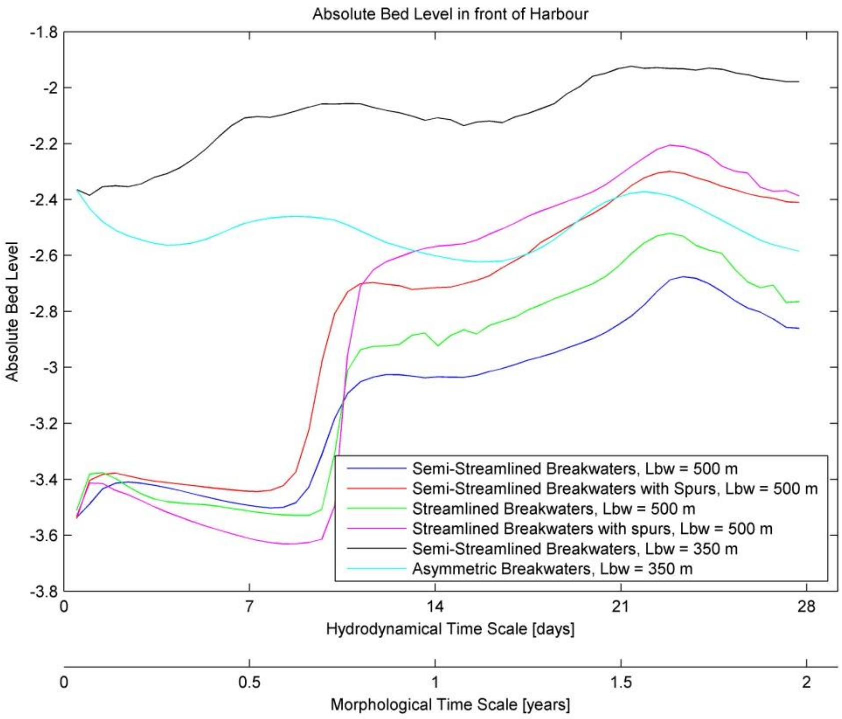

Figure E-26 Plot of the absolute bed level in time in front of the harbour.

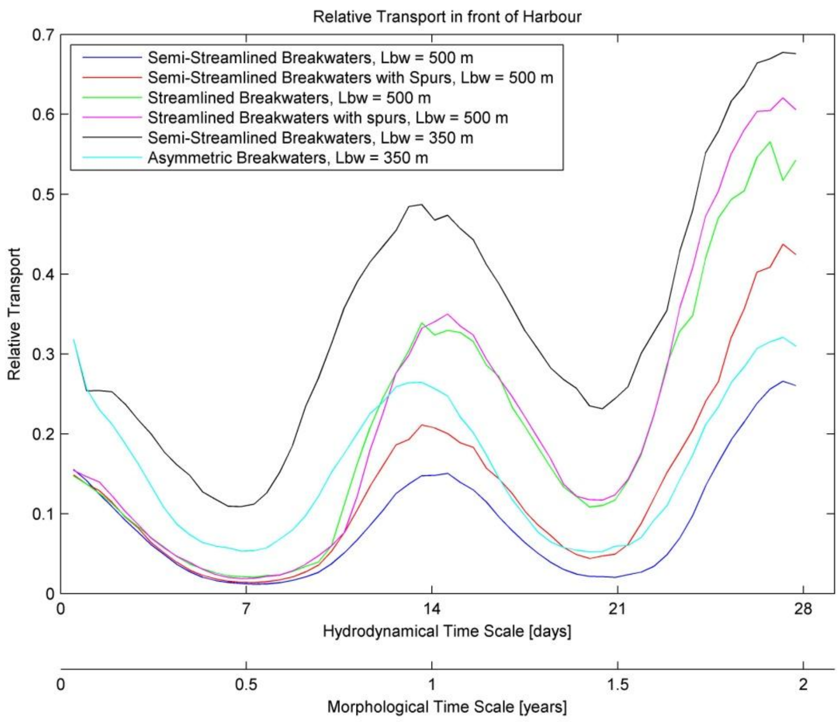

Figure E-27 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

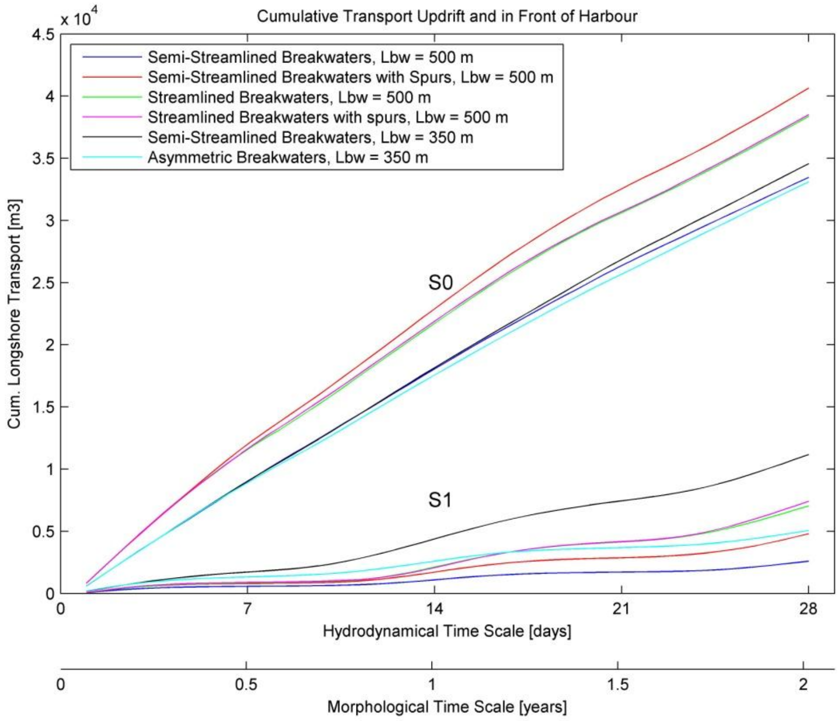

Figure E-28 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

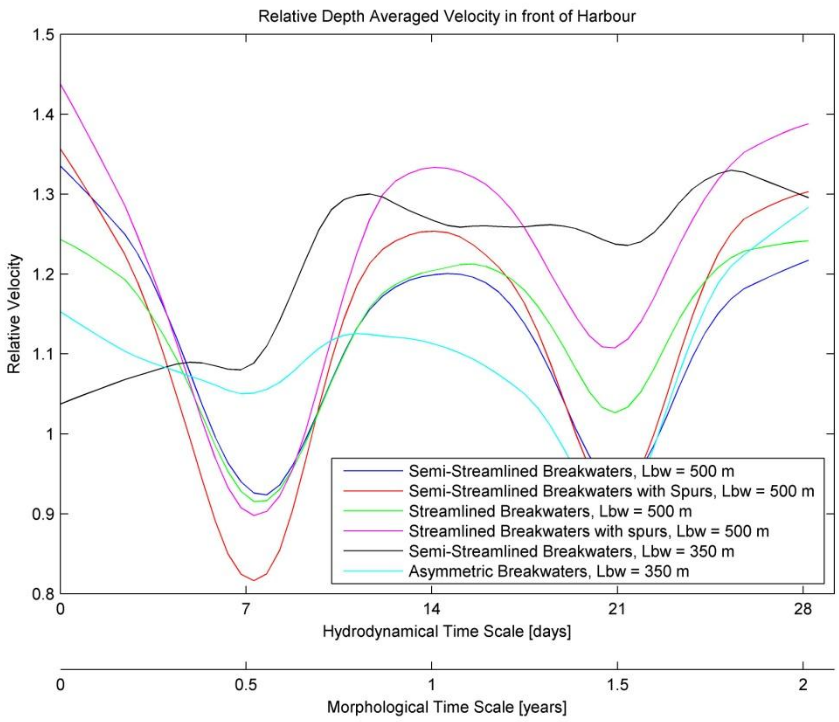

Figure E-29 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

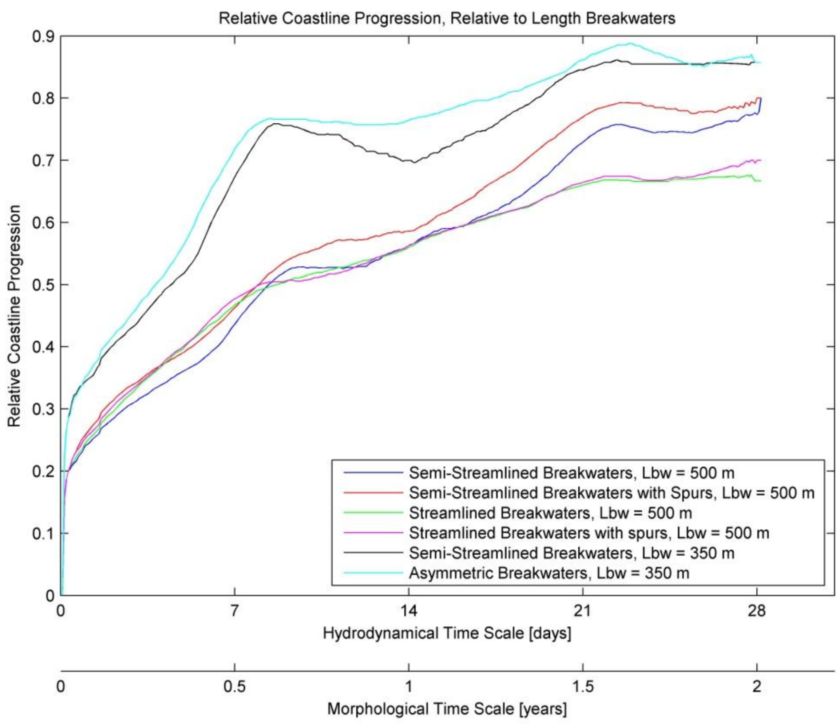

Figure E-30 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

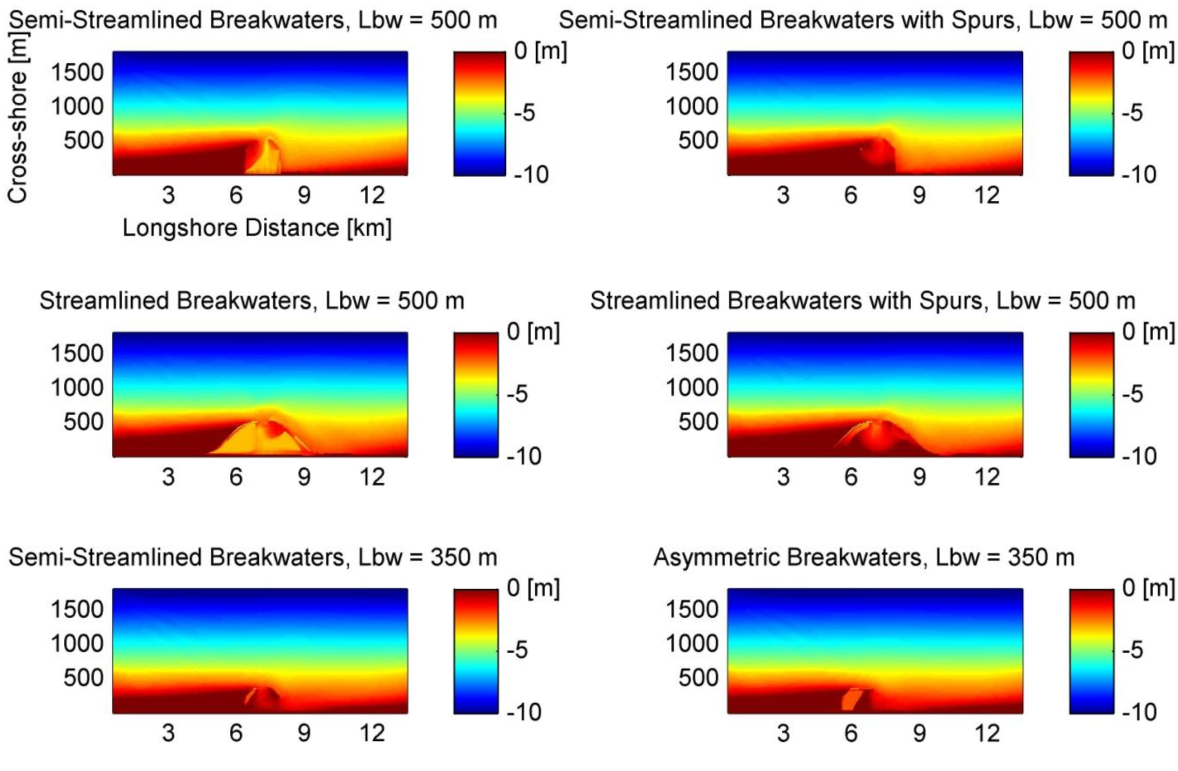

Figure E-31 Six plots of the bed level at the end of the calculation

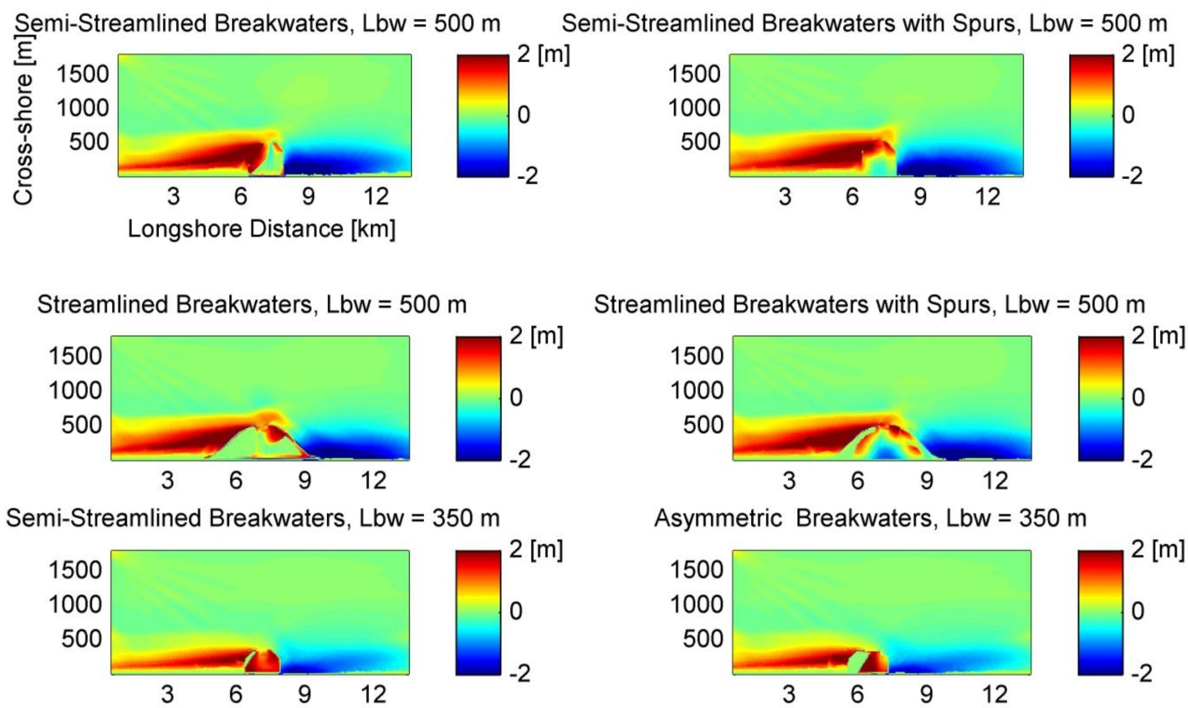

Figure E-32 Six plots of the cumulative sedimentation/erosion at the end of the calculation.

E.5. Assessment of the most optimal scenarios on longer time-scale

The following settings are applied:

- Wave height: 1 m and 1.5 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 350 m and 500 m;
- Simulation time: 2 $months$ (4 $years$);
- Breakwater geometry: semi-streamlined.

The following cases are assessed on a longer time scale

1. In case of breakwaters smaller than the width of the surf zone
2. In case of streamlined breakwaters.
3. In case of breakwaters equal to the width of the surf zone

The third one is added as reference case.

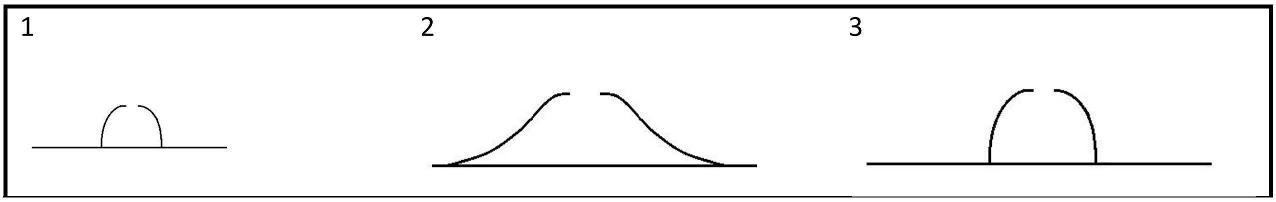

Table E-2 Different breakwater geometries that are assessed.

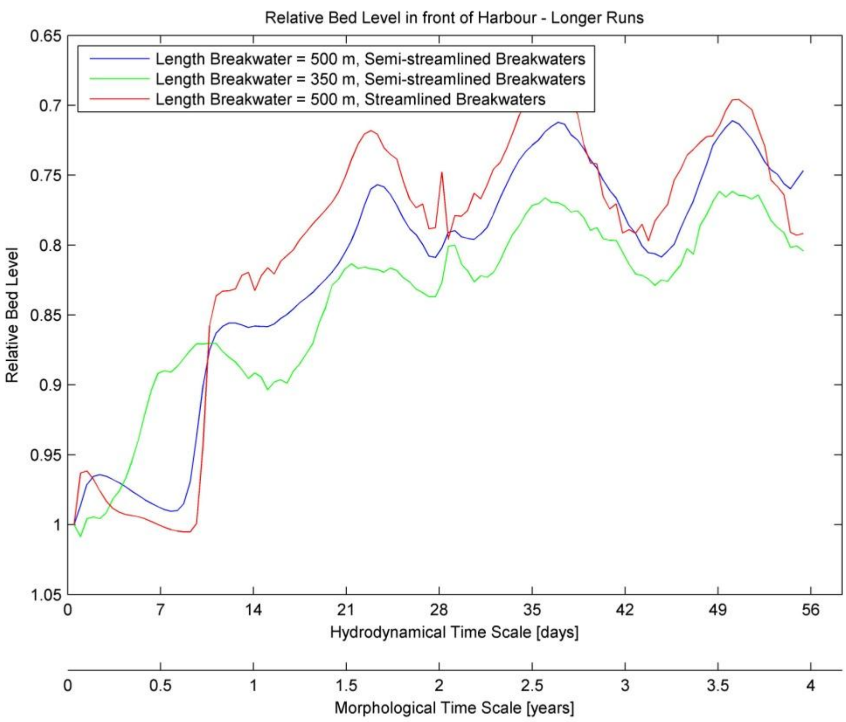

Figure E-33 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

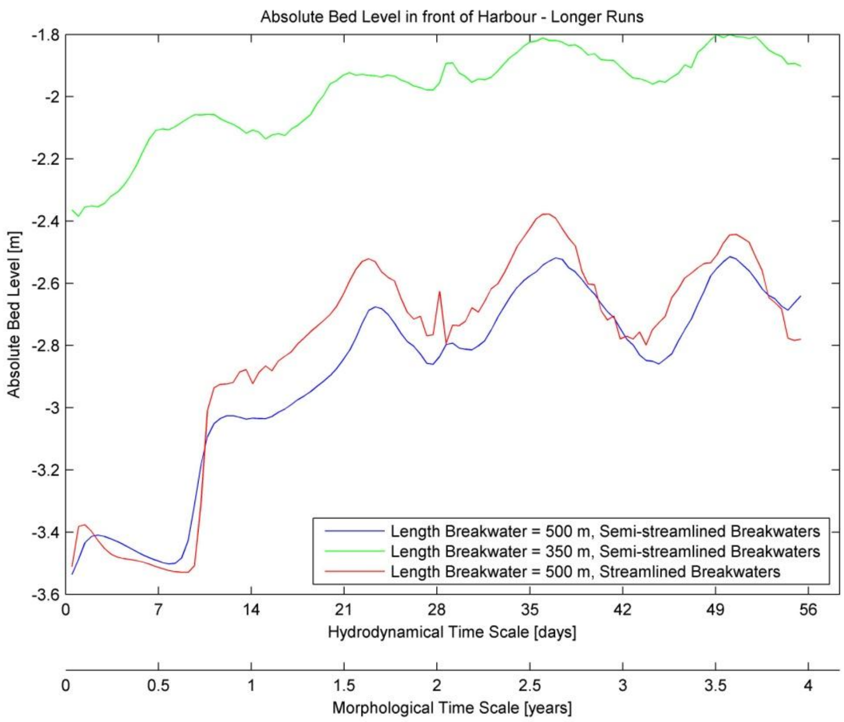

Figure E-34 Plot of the absolute bed level in time in front of the harbour.

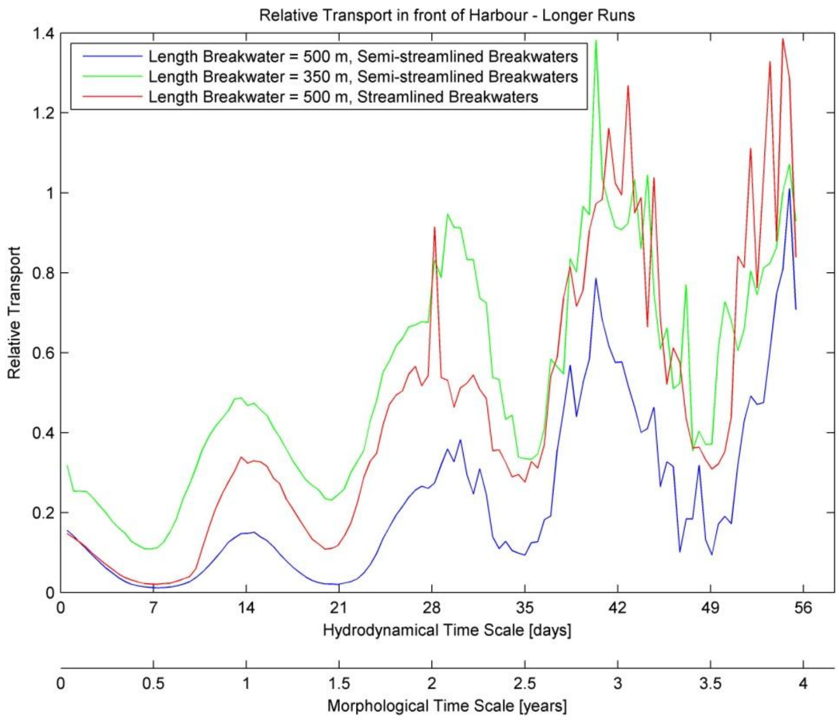

Figure E-35 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

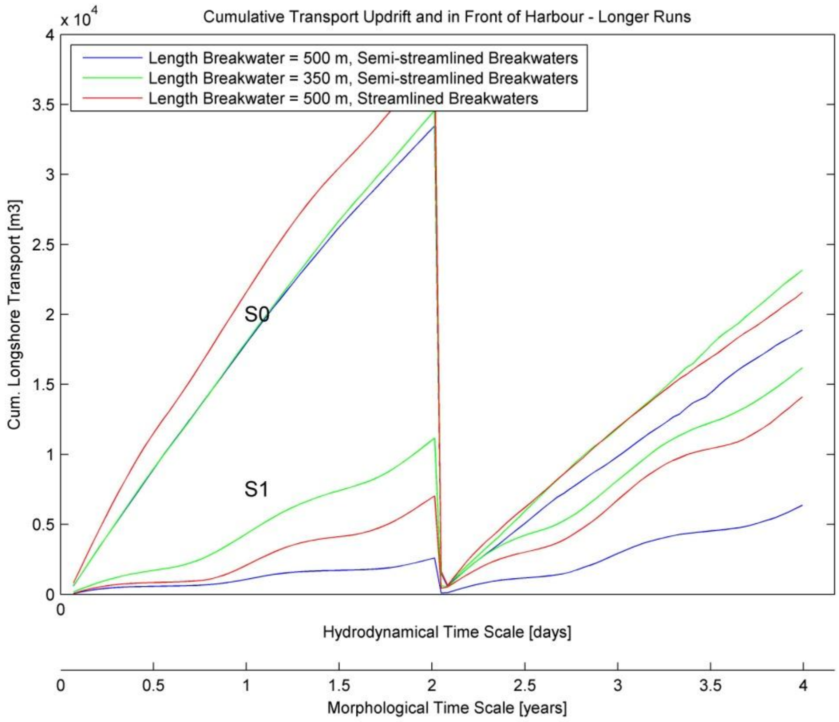

Figure E-36 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

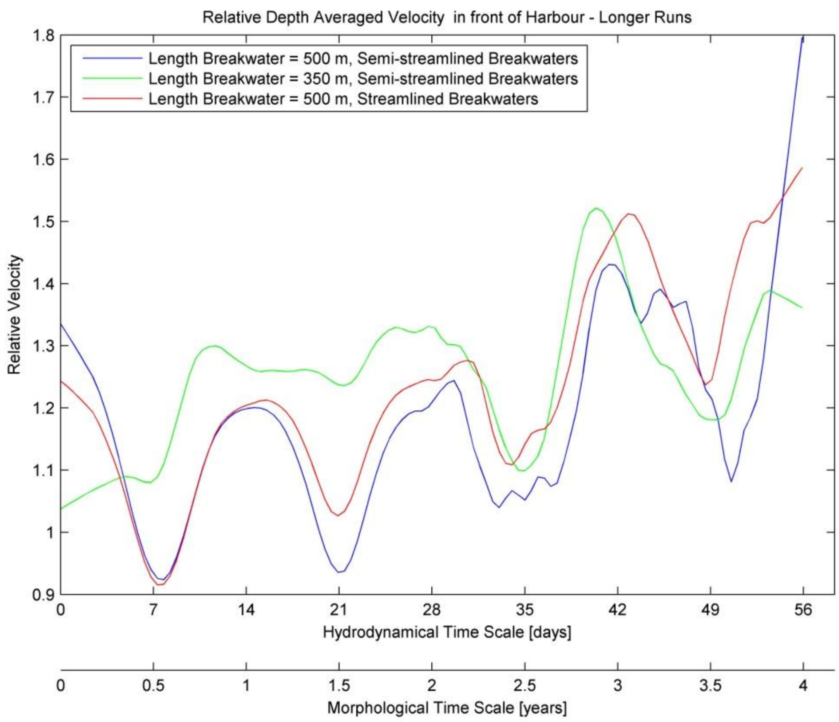

Figure E-37 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

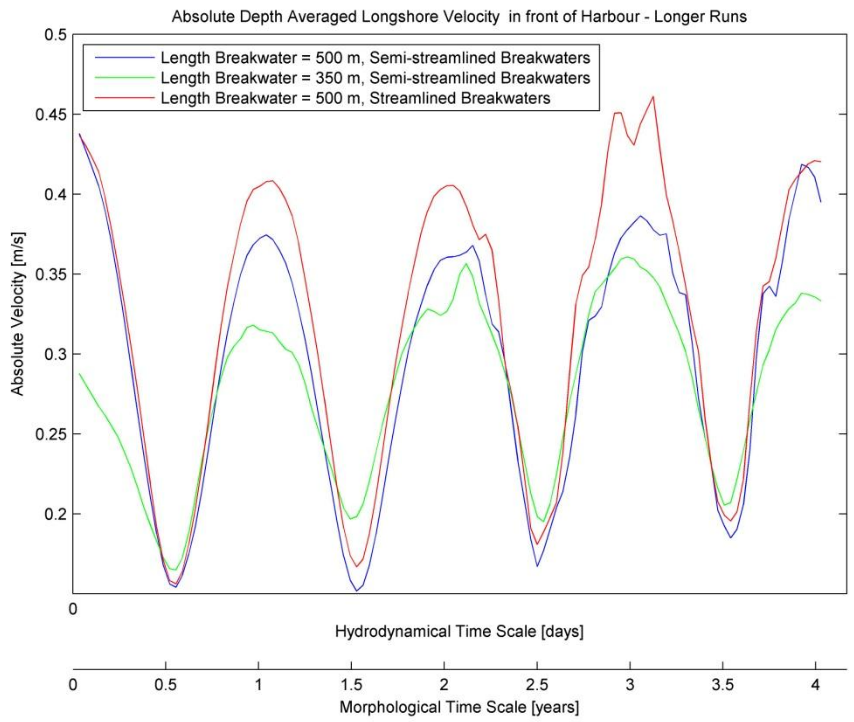

Figure E-38 Plot of the absolute longshore depth averaged velocity in time in front of the harbour. The values are averaged per tidal cycle.

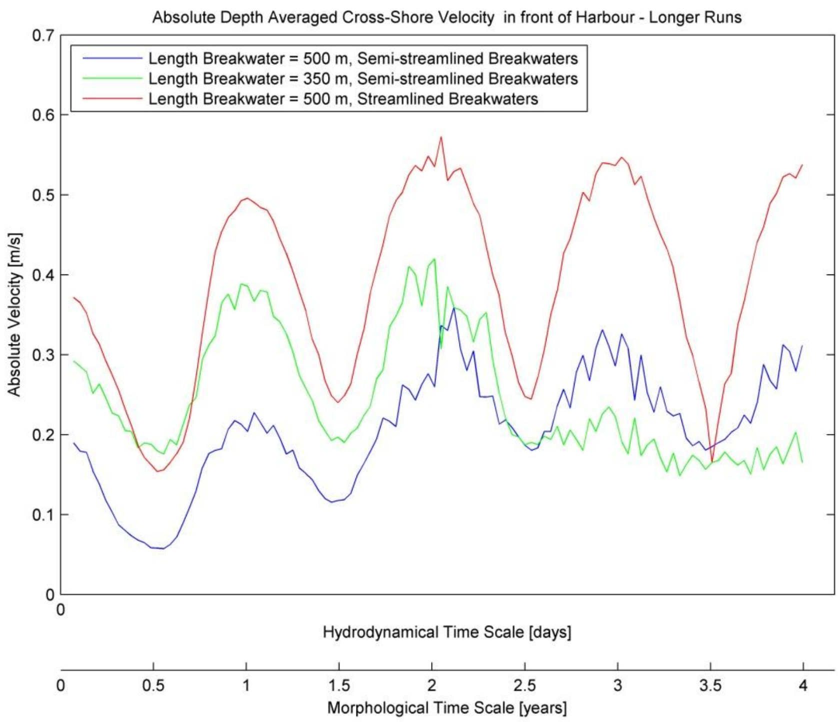

Figure E-39 Plot of the absolute cross-shore depth averaged velocity in time in front of the harbour. The values are averaged per tidal cycle.

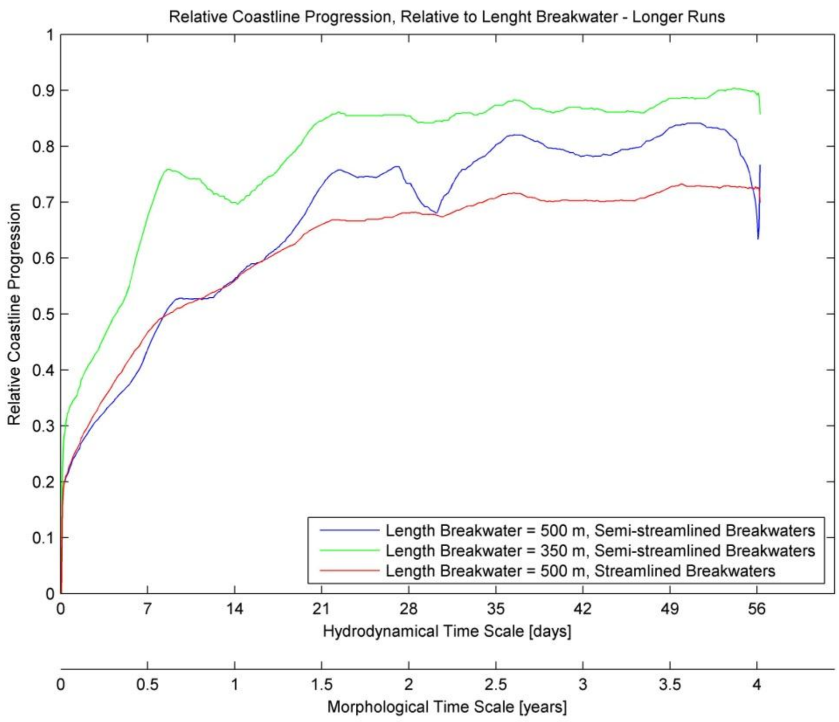

Figure E-40 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

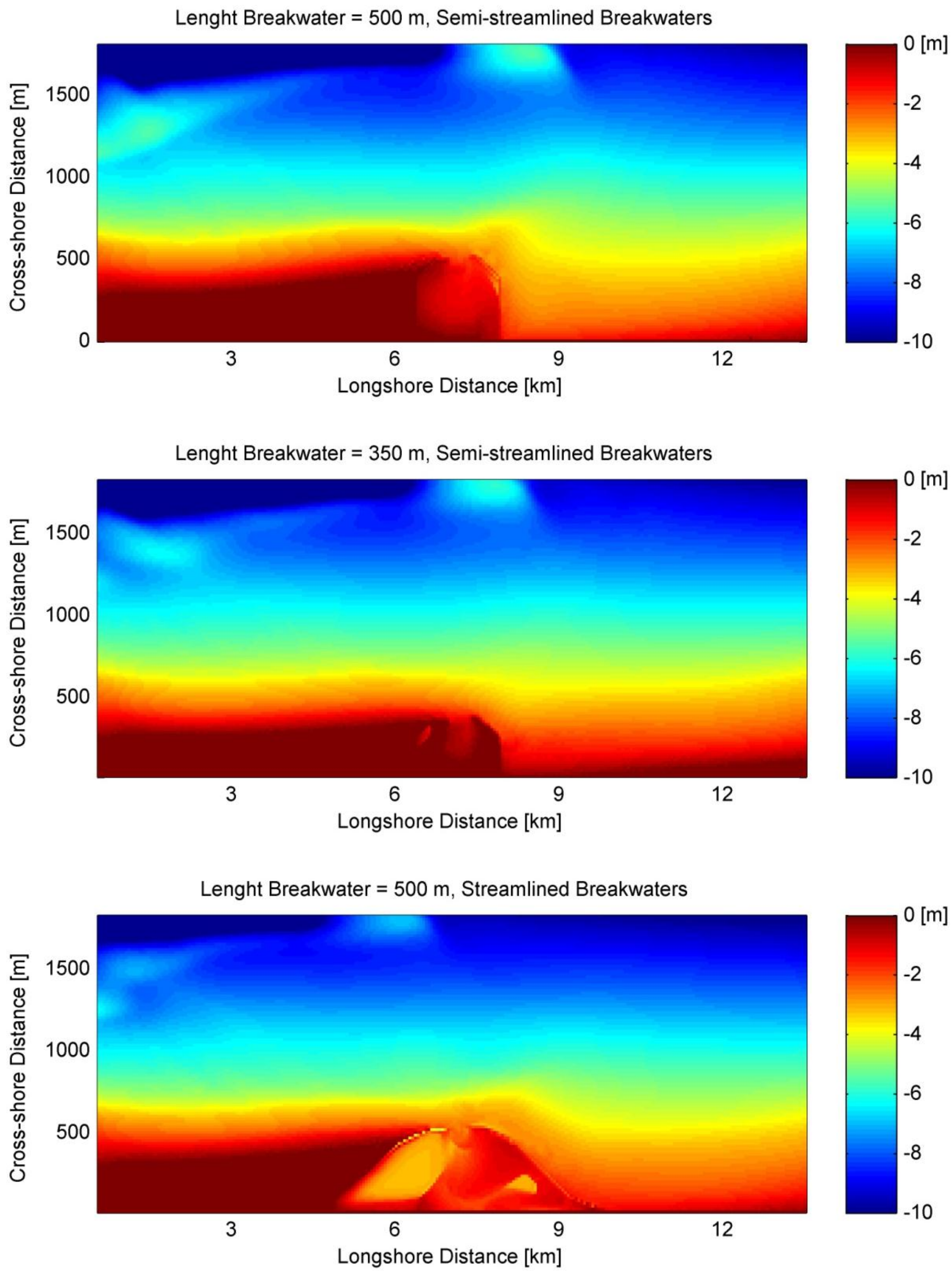

Figure E-41 Three plots of the bed level at the end of the calculation

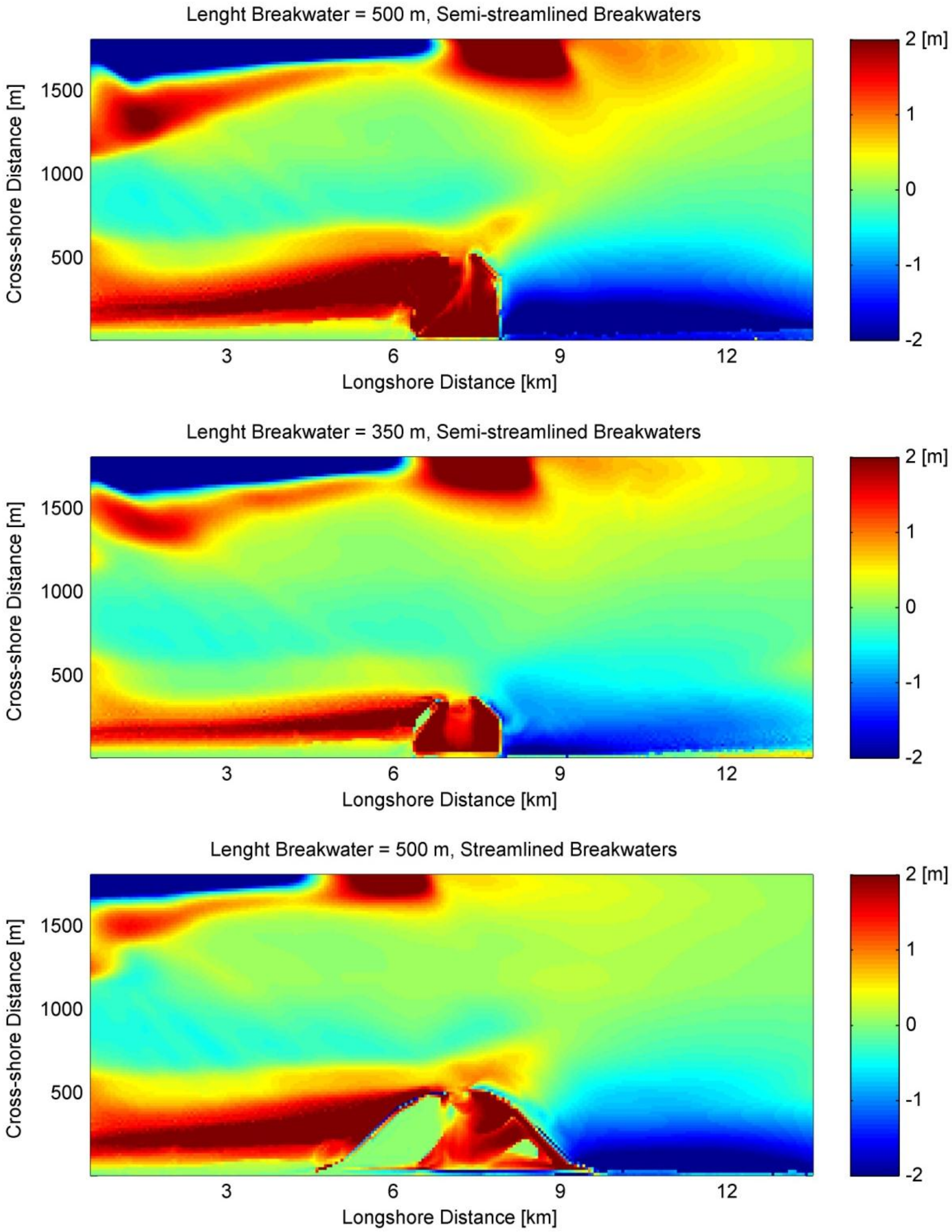

Figure E-42 Three plots of the cumulative sedimentation/erosion at the end of the calculation.

E.6. Assessment of the influence of a dredged channel and the tide

The following settings are applied:

- Wave height: 1 m;
- Wave angle: 45^o;
- Wave period: 7 sec;
- Length breakwater: 500 m;
- Simulation time: 1 $month$ (2 $years$);
- Breakwater geometry: semi-streamlined;

- Depth dredged channel: 7 m;
- Tidal amplitude: 1.33 m and 0.67 m at spring tide.

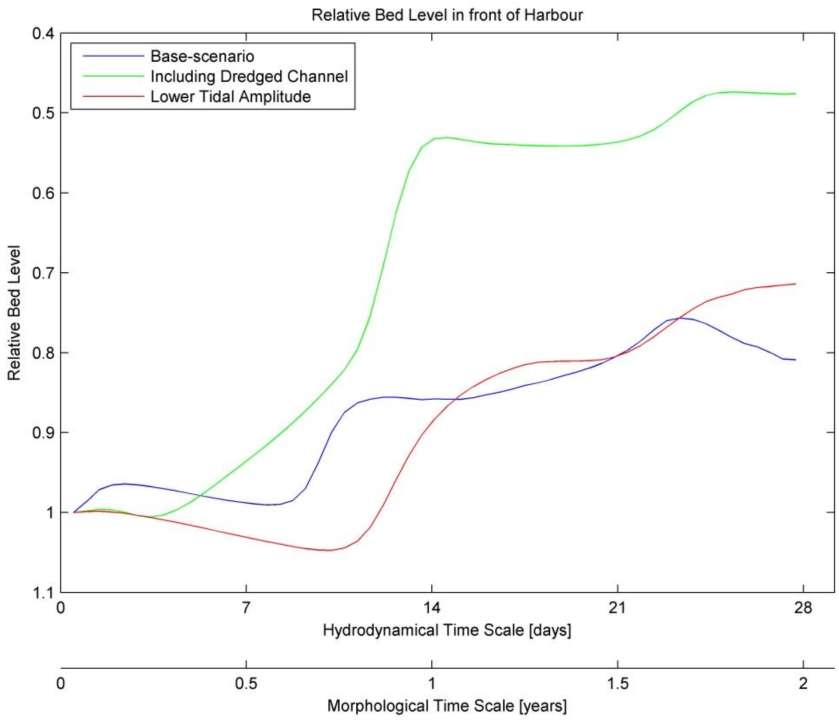

Figure E-43 Plot of the relative bed level in time in front of the harbour. The bed level in front of the harbour is divided by the initial bed level.

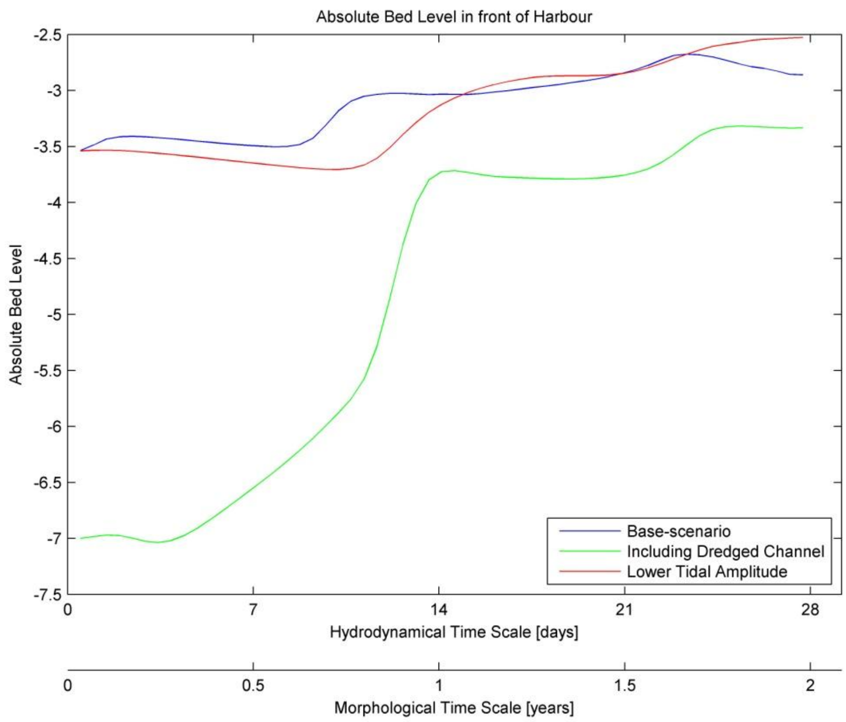

Figure E-44 Plot of the absolute bed level in time in front of the harbour.

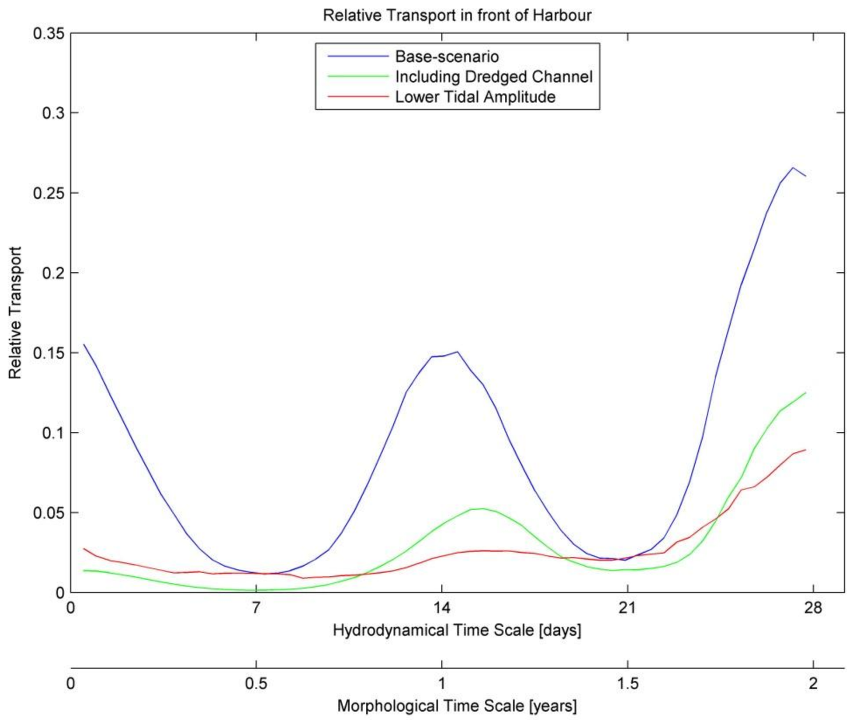

Figure E-45 Plot of the relative transport in time. This is the instantaneous transport in front of the breakwater divided by the instantaneous transport which is measured updrift. Both values are averaged over a tidal cycle

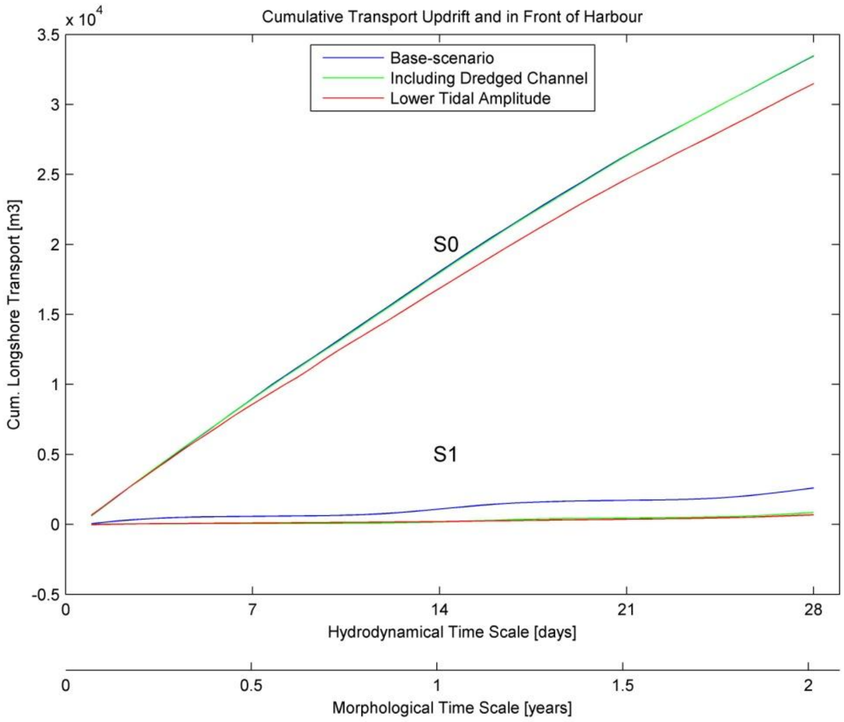

Figure E-46 Plot of the cumulative transport in time measured in the cross-section updrift of the harbour ($S0$) and in front of the harbour ($S1$).

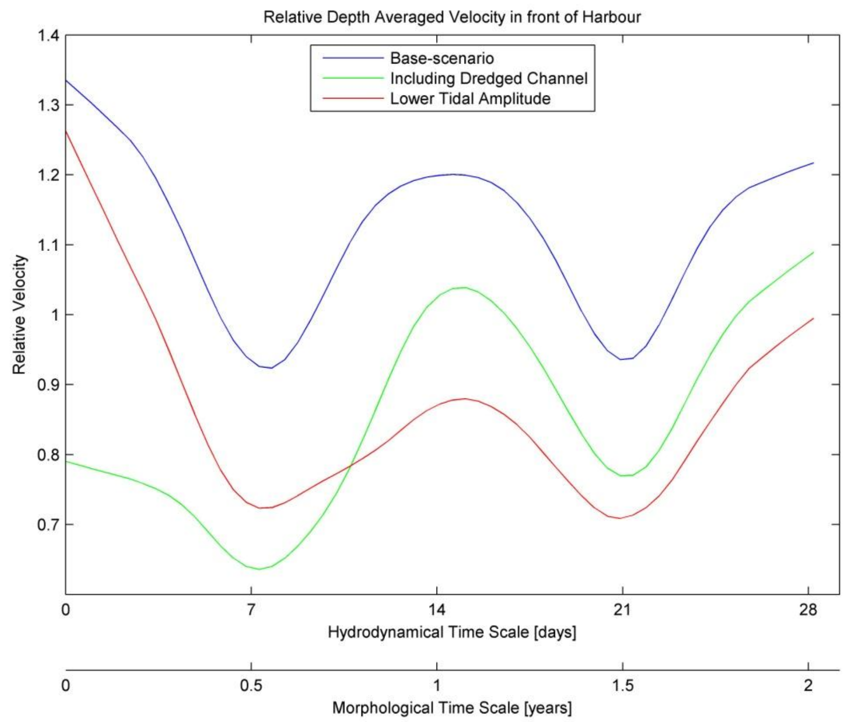

Figure E-47 Plot of the relative depth averaged velocity in time in front of the harbour. The measured velocity in front of the harbour is divided by the measured velocity at the same cross-shore location but updrift. For both values the maximum value per tidal cycle is used.

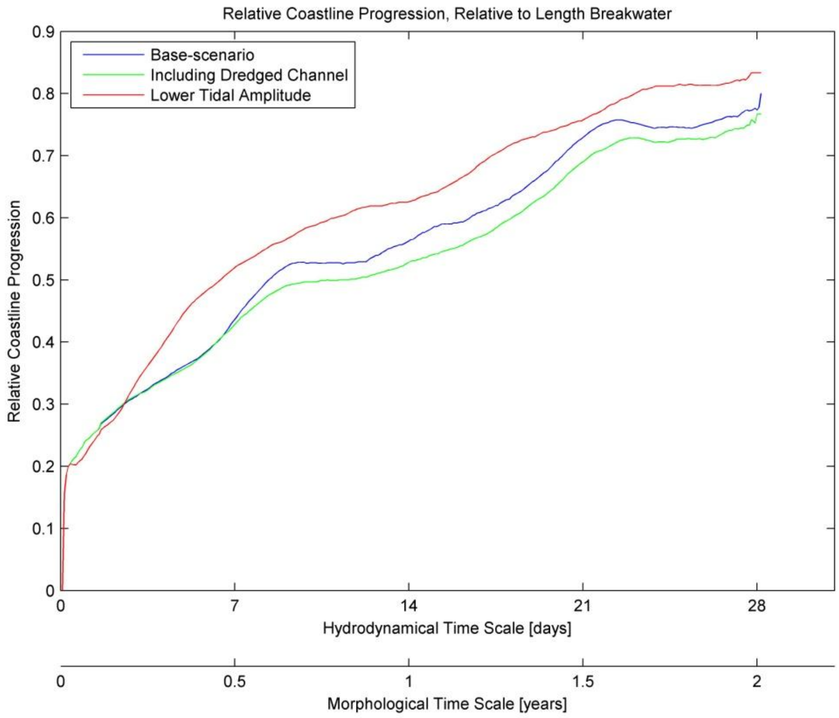

Figure E-48 Plot of the relative coastline progression in time just updrift of the harbour. The coastline progression is divided by the length of the breakwater.

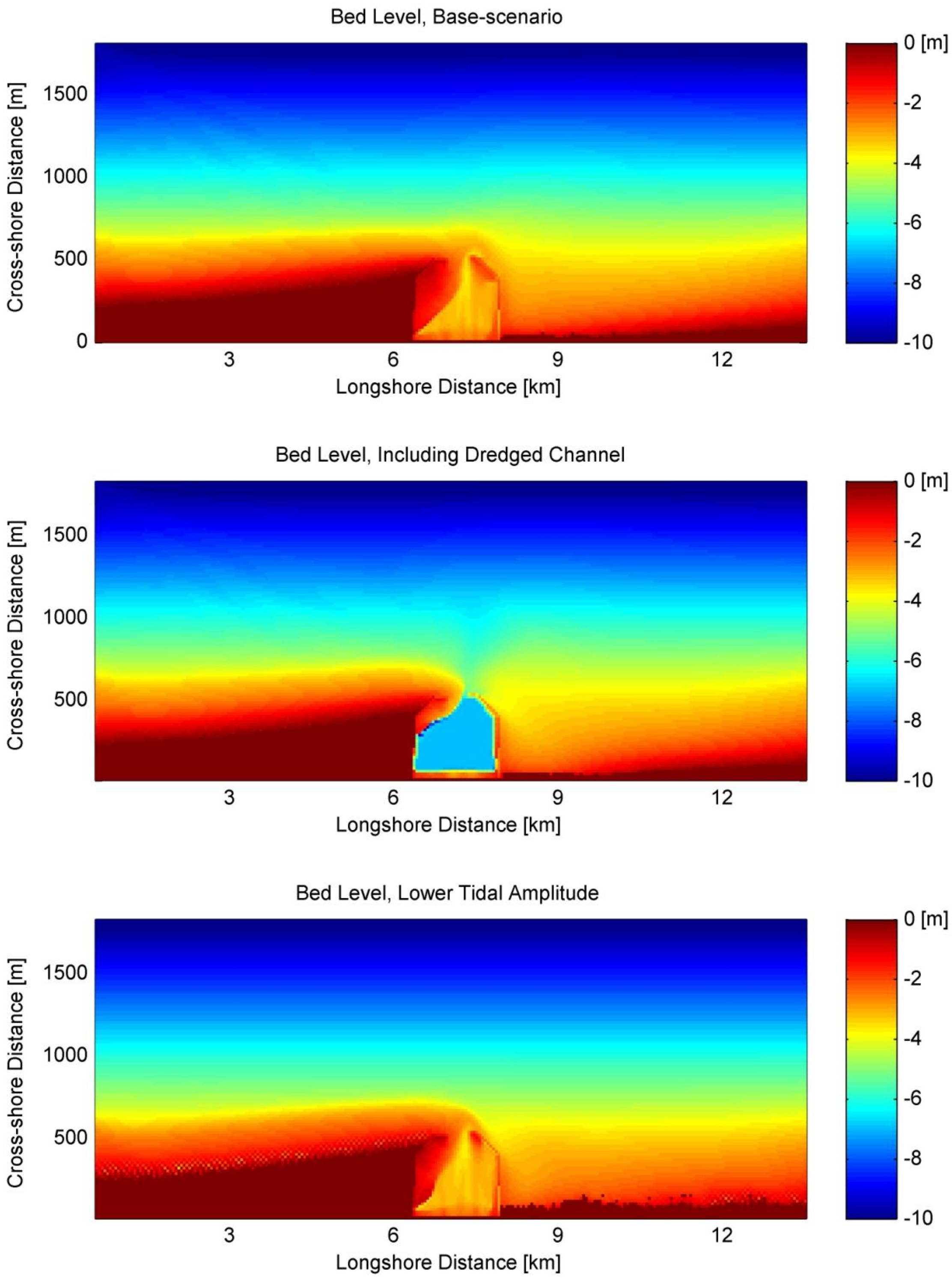

Figure E-49 Three plots of the bed level at the end of the calculation.

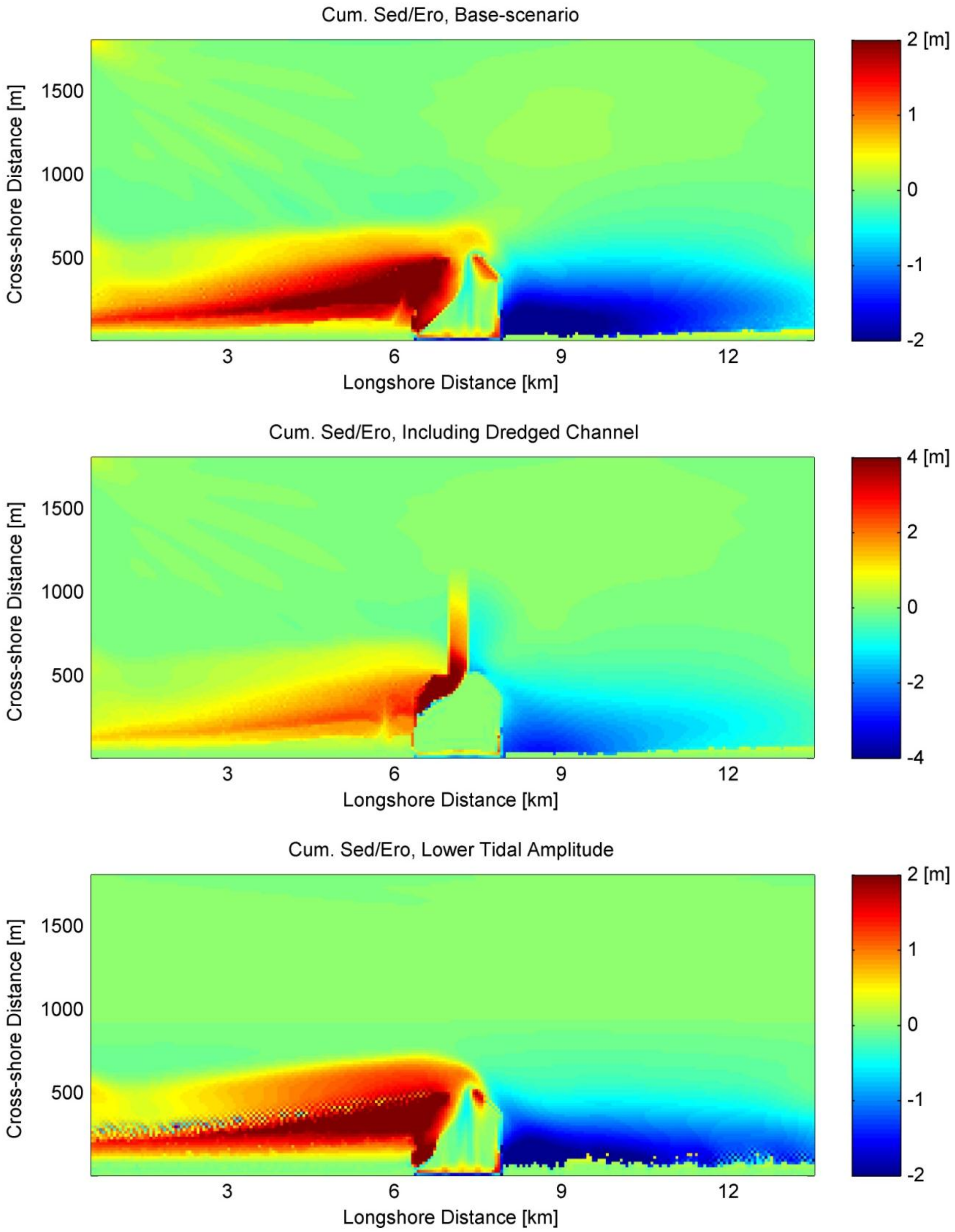

Figure E-50 Three plots of the cumulative sedimentation/erosion at the end of the calculation. Mark the different scale of the plot which shows the cumulative sedimentation/erosion of the run including the dredged channel.