Master's Thesis
TVVR 10/5024

Modelling coastal erosion in Bjärred, Lomma municipality

- Long-term evolution and protective measures

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Avdelningen för Teknisk Vattenresurslära
TVVR-10/5024
ISSN-1101-9824

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Printed in Sweden, 2011

Foreword

This master thesis on *modelling coastal erosion in Bjärred, Lomma municipality – Long-term evolution and protective measures*, has been carried out at Lunds Tekniska Högskola in cooperation with Ramböll Sverige AB, Malmö during the time period September 2010 – January 2011.

We would like to thank our supervisor at the Water Resources Engineering, Professor Magnus Larson, for his time and his commitment to our work. Despite his work load, he has always taken time to guide us and to answer our questions. A great thanks directed to Ramböll and the staff at the Department of geotechnics and environment, especially Elisabet Hammarlund who has given us valuable information and comments throughout the project. We would also like to thank our examiner, Professor Hans Hanson, for the help he provided, especially concerning the computer modelling.

At Lomma municipality we would like to express gratitude to Lennart Persson for guiding us at the site and providing information about the previous projects in the area. Field measurements were performed with the assistance of Fredrik Svensson, technician at Lomma municipality, and Bo Nilsson, Lomma fire department, which lent us the boat. Likewise we would like to thank Sofia Yassin, GIS-technician at Lomma municipality, for her help with aerial photographs and assessment with GIS-related inquires.

We also want to express our gratitude to Thomas Hammarklint and Signild Nerheim at the SMHI for providing data and information about sea water level and winds.

Lund, December 2010

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Abstract

Title: Modelling coastal erosion in Bjärred, Lomma municipality
- Long-term evolution and protective measures

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Elisabet Hammarlund, Ramböll Sverige AB, Department of geotechnics and environment

Problem: Many coastal areas around the world experience the effects of erosion on the shorelines. This phenomenon leads to great economical, environmental, and social losses for the affected communities. The risk of a near future climate change, such as the predicted rise in sea water level might increase these problems to a great extent. Historically, Lomma municipality, in southern Sweden, has faced problems with erosion at different places along its shoreline. One of the most affected areas in this municipality is Bjärred, located some distance north of Lomma village.

The consultant company Ramböll Sverige AB was chosen to study the erosion processes taking place, and suggest protective measures to solve the erosion problem in Bjärred.

Aim: The aim of this thesis is to evaluate how applied protective measures against coastal erosion in Bjärred have worked so far and to investigate the possibility of introducing additional measures to improve the present conditions. These measures will be investigated on the most affected shoreline stretch in the study area both regarding the present condition as well as the conditions after an estimated future climate change. A computer model will be used to simulate the effect of the suggested measures.

Method: The thesis work started in September 2010, when a literature study was performed with special focus on the theory behind coastal sediment transport, erosion in general and the site-specific project in Bjärred.

Information about historical sea water levels and wind data was gathered from the Swedish Meteorological and Hydrological Institute (SMHI). Aerial photos ranging between 1939 and 2010, obtained from Lantmäteriet, were studied and vegetation lines from the different available years were digitized. They were carefully compared in order to get an estimate of the erosion rate during this time period. To validate the erosion rate and to get a general knowledge about the area a survey was performed at the most affected shoreline stretch, where beach and sea bottom profiles were obtained.

Erosion originating from the wave climate was calculated and analysed, including short-term impacts after storm events as well as long-term effects. The effect of the long-term erosion is achieved by calibrating a model to explain the shoreline behaviour in the study area by using the computer software GENESIS. Several protective measures were evaluated and simulated, in order to predict the future evolution of the shoreline depending on the chosen solution. In addition, calculations and simulations of sea level rise and increased winds due to climate change were performed.

Finally, financial and environmental evaluations were carried out for the protective measures that proved to provide satisfying results in the simulation stage.

Conclusions: The aerial photograph analysis showed that the beach has eroded, in average 4.3 m during the time period 1963-2002 and 1.3 m during the time period 2002-2010. A maximum retreat of 13 m was observed along some parts of the shoreline. The survey confirmed the erosion rate as comparisons of beach profiles from 2004 and 2010 showed that the profile had retreated at approximately the same rate as obtained from the aerial photos.

Calculations of cross-shore transport rates show that the erosion will increase to a great extent with future sea level rise. Four different scenarios were considered, where the worst case scenario shows an increase of more than 200% in cliff volume eroded. Calculations using the Bruun rule show that the shoreline, theoretically, might retreat 97 meters to the year 2070 if no protective measures are applied.

A simulation of Lomma bay shows that the sediment transport pattern is from north to south in the northern part of the bay and from south to north in the southern part of the bay. Since Bjärred is located in the northern part, the predominating sediment transport direction is southward. Field evidences gathered from aerial photos and field visits confirms this transport pattern of Lomma bay.

Simulations of protective measures show that the placement of detached breakwaters will create tombolos in the area, thus not being a suitable measure. Extension of the concrete slabs in the area is, according to the simulation results, a good option to stop the erosion at the site. An alternative to extend the concrete slabs might be to construct sheet piles in the cliff. The simulation of beach nourishment showed that the erosion rate is lowered but not completely stopped, wherefore a combination of revetments and beach nourishment might be a good option in order to make the revetments look more natural. Another advantage with beach nourishment is that the addition of sand prevents downdrift erosion, which often occurs when adding revetments on a shoreline stretch.

The financial evaluation shows that extension of the existing revetments is the cheapest satisfyingly working measure but the costs do not differ much from construction of sheet piles. If the financial situation allows, beach nourishment above the hard structure gives a natural appearance of the revetments and the recreational value of the beach is not lost.

The recommendation to the municipality is therefore to perform further investigations regarding the measure of extending the revetments with the option of nourishing the beach above them. The nourishment can have an addition of vegetation to make the sand stay longer.

The future rise of sea water level might lead to overtopping of the cliff and therefore flooding of the adjacent road and properties. The municipality is therefore recommended to investigate in a capacity increase of the storm/waste water system in order to prevent the flooding.

Key words: Bjärred, coastal erosion, waves, water level, runup, wind, sediment transport, climate change, GENESIS, modelling, protective measures,

Sammanfattning

Titel: Modellering av stranderosion i Bjärred, Lomma kommun
- Långsiktig utveckling och skyddsåtgärder

Författare: Carolina de Mas de Mas
Johan Södergren

Handledare: Professor Magnus Larson, avdelning för teknisk vattenresurslära vid Lunds Tekniska Högskola
Elisabet Hammarlund, Ramböll Sverige AB, avdelning geo och miljö

Problemställning: Många kustområden i världen upplever effekterna av stranderosion, vilka kan leda till stora ekonomiska, miljömässiga och sociala förluster. Risken för en nära förestående ökad klimatförändring, med förutspådd ökning av havsvattennivån kan leda till ökad erosionsproblematik. Historiskt har Lomma kommun, i sydvästra Skåne, haft problem med erosion vid ett antal kustpartier. Ett av de mest påverkade områdena i kommunen är Bjärred, beläget strax norr om Lomma.

Teknikkonsultföretaget Ramböll Sverige AB, har utsetts för att studera erosionsförloppet och föreslå skyddsåtgärder för att överkomma erosionsproblematiken.

Syfte: Syftet med det här examensarbetet är att undersöka hur de hittils tillämpade skyddsåtgärderna har fungerat och undersöka möjligheten att implementera ytterligare åtgärder för att förbättra situationen. Undersökningen rör den mest påverkade kustremsan i studieområdet både gällande dagens förutsättningar men även förutspådda framtida scenarion. En datormodell används för att simulera beteendet hos föreslagna skyddsåtgärder.

Metod: Examensarbetet startade i september 2010, då en litteraturstudie utfördes. Fokus för studien var teorin bakom sedimenttransport, erosion generellt och det specifika projektet i Bjärred.

Information om historiska havsvattennivåer och historisk vind data insamlades från SMHI. Ett antal flygfoton, från åren 1939 till 2010, erhållna från Lantmäteriet studerades och vegetationslinjen för respektive år digitaliserades. Vegetationslinjerna jämfördes noggrant för att få erosionsgraden för den här tidsperioden. För att bekräfta erosionsgraden från flygbildsanalysen genomfördes en fältundersökning där strand- och bottenprofiler mättes in.

Erosion härstammande från vågklimatet beräknades och analyserades, både gällande korttidseffekter från stormar och långtidseffekter. De långsiktiga effekterna erhölls genom att, med hjälp av datormodellen GENESIS, modellera studieområdet. Ett antal skyddsåtgärder utvärderades och simulerades, för att kunna förutsäga strandlinjens beteende beroende på vilken skyddsåtgärd som valdes. Dessutom genomfördes beräkningar och simuleringar av klimatförändring, i form av havsvattennivåhöjning och ökad vindhastighet.

Slutligen genomfördes, på de åtgärder som uppvisat tillfredsställande resultat under simuleringarna, en utvärdering av respektive åtgärd rörande ekonomi och miljöpåverkan.

Slutsatser: Flygbildsanalysen visar att stranden har eroderat i genomsnitt 4.3 meter för tidsperioden 1963-2002och 1.3 meter under 2002-2010. Maximal erosion uppmättes på vissa partier till hela 13 meter. En jämförelse av inmätt strandprofil från 2004 och 2010 bekräftar erosionsgraden från flygbildsanalysen, med bara mindre skillnad.

Beräkningar av den vinkelräta sedimenttransporten visar att erosionen troligen kommer att öka dramatiskt med en förväntad havsvattennivåhöjning. Fyra olika scenarion togs i beaktning, där scenariot med värsta tänkbara utgång visar en ökning av erosionen från brinken med mer än 200%. Beräkningar utförda med hjälp av Bruuns regel visar att strandlinjen, teoretiskt, kan dra sig tillbaka upp till 97 meter år 2070 om problemet inte åtgärdas.

En simulering av Lommabukten visar att sedimenttransporten är riktad från norr till söder i den norra delen av bukten och tvärtom i den södra delen. Eftersom Bjärred ligger i den norra delen av bukten är den förhärskande riktningen av sedimenttransport riktad söderut. Fältbesök på olika platser i bukten och flygbildsanalys bekräftar det här transportmönstret.

Simuleringar av skyddsåtgärder visar att friliggande vågbrytare ger upphov till tombolos i området och är därför ingen lämplig skyddsåtgärd i det här området. En förlängning av flexplattorna i området fungerar väl ur erosionssynpunkt eftersom erosionsförloppet avstannar helt. Ett alternativ till att förlänga flexplattorna är att slå ner en spont i sandbanken, för att skydda VA-röret som ligger där. Simuleringar av sandutfyllnad på stranden visar att strandlinjereträtten minskar men avstannar inte helt. En kombination av strandskoningar med sandutfyllnad ovanpå är därför ett alternativ, för att få skoningarna att se mer naturliga ut. En annan fördel med att tillföra sand är att erosion nedströms kan förebyggas, något som annars ofta händer vid byggnation av strandskoningar.

Den ekonomiska utvärderingen visar att en förlängning av de existerande flexplattorna är det billigaste alternativet, tätt följt av nedslagning av spont. Om budgeten tillåter, bör sandutfyllnad ske utanför strandskoningen för att inte gå miste om strandens rekreationsvärden.

Rekommendationen till Lomma kommun blir därmed att vidare undersöka möjligheten att förlänga flexplattorna eller att slå ner en spont och, om ekonomin tillåter, fylla ut med sand utanför dessa åtgärder.

En framtida ökning av havsvattennivån kan leda till överspoling av sandbanken och därmed översvämningar på den närliggande vägen och byggnader. Kommunen rekommenderas därmed att undersöka en kapacitetsutökning av VA-systemet för att kunna ta hand om vattenmassorna och undvika översvämningar.

Nyckelord: Bjärred, stranderosion, vågor, vattennivå, uppspolning, vind, sedimenttransport, klimatförändring, GENESIS, modellering, skyddsåtgärder

Glossary

Term	Explanation
Accretion	See accumulation
Accumulation	Opposite to erosion, meaning that the amount of sediment within a specific area and time interval is increasing
AR4	Assessment Report 4, from the IPCC
ArcGIS	Group of a Geographic Information Systems (GIS) software
AutoCad Civil 3D	Construction information modeling software for civil engineering
Beach profile	Beach cross-shore section perpendicular to shoreline
Berm or backshore	Horizontal part of the shore between the foreshore and the dune foot
Bottom profile	Same as beach profile, but only the part under water
Calibration	Process of coefficient adjustment to obtain a model representation of a specific process
Cliff	A high steep face of accumulated sediments behind the beach
Climate change	Long-term change in the statistical distribution of weather patterns
Coast	A section of land on either sides of the shoreline affected by the interaction between the incoming waves and the sea bottom
Cross-shore transport	Transport of beach sediments perpendicular to the shoreline
Depth of closure	Water depth beyond which the profile does not exhibit significant depth change
Depth of longshore transport	Water depth seaward of which negligible longshore sand transport occurs
Diffraction	Change of wave direction behind obstacles due to wave energy being transferred laterally along the crest
DSAS	Digital Shoreline Analysis System software
Dune	Geomorphic feature resulting from sediment deposition of sand behind the beach due to wind
Erosion	Opposite of accretion, meaning that the amount of sediment within a specific area and time interval is decreasing
Eustatic	Refers to changes in the amount of water in the oceans due to global climatic changes or other mechanisms
Fetch	The length of water over which a given wind is blowing to build waves
Foreshore	Extends from the low-water line to the limit of wave uprush at high tide.
Plastic reinforcement nets	Cages of plastic or metal wire holding rocks of limited size for erosion control
GENESIS	GENEralized model for SImulating Shoreline change
GPS	A satellite-based Global Positioning System that provides spatial coordinates to associated receivers
Gross transport	Total longshore transport rate, regardless of the direction
IPCC	Intergovernmental Panel on Climate Change
Lantmäteriet	The Swedish mapping, cadastral, and land registration authority
Limhamn threshold	A shallow ridge on the sea bottom across Öresund outside Limhamn that acts as a natural barrier between the fresh, light waters of the Baltic Sea and the salty, heavy waters of Kattegat
Longshore transport	Transport of beach sediments along the shoreline
N	North
Net transport	Time-average longshore transport rate, taking transport direction into account

NNW	North northwest
NW	Northwest
Offshore	Location: the zone seaward of the nearshore zone where sediment motion induced by waves alone ceases. Direction: the direction pointing seaward from the shore
Refraction	The change in direction of a wave due to a change in its speed induced by variable water depth along the wave crest
Return period	An estimate of the average time interval between two consecutive events of the same nature
RH2000	Swedish national vertical datum system originating from the year 2000
RT90	An older national plane coordinate net upon which the Swedish maps were based
S	South
Salient	Accumulation of sand behind a breakwater
Shoaling	Effect by which surface waves entering shallower water change their properties (reduced wave length and celerity, change in wave height)
Shore	Strip of land that extends from the low-water line to the normal landward limit of storm wave effects.
Shore accretion	Shoreline advance due to accumulation of sediments
Shore retreat	Shoreline landward movement due to erosion processes
Shoreline	Boundary between land and the sea
Significant wave height	Average wave height of the 1/3 highest waves. Used for design of structures as a representation of the entire wave population
Skåne	Scania, southern province of Sweden
SMHI	Swedish Meteorological and Hydrological Institute
SSE	South southeast
SSW	South southwest
Surf zone	Sea area where waves break
Swell	Long, regular waves no longer affected by the wind that generated them
SWEREF	3-dimensional Swedish geodetic reference system
SWL	Sea Water Level
Thermal expansion	Tendency of matter to increase in volume in response to a change (typically increase) in temperature
Tombolo	Accumulation of sand between the beach and a breakwater that reaches all the way out to the structure
USACE	US Army Corps of Engineers
Validation	Evaluation of the calibrated model for an independent time period
Vegetation line	The seaward front of the vegetation cover on land
W	West
Wave height	Vertical distance from wave through to crest
Wavelength	Horizontal distance between two consecutive wave troughs
Wave period	The time it takes between two consecutive wave crests to pass a fixed point
Wave runup	Maximum vertical extent of wave uprush on a beach or structure above the still water level
Wave setup	Local increase of water surface elevation due to the conversion of kinetic energy of waves to potential energy
WSW	West southwest

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1. Introduction

1.1 Background

Many coastal areas around the world experience the effects of erosion on the shorelines. This phenomenon leads to great economical, environmental and social losses for the affected communities. The risk of a near future climate change, such as the predicted increase in sea water level might increase the problems to a great extent. This has forced many municipalities to start implement some integrated action plans, in order to protect their shores and properties from the wave action.

Historically, Lomma municipality in southern Sweden has faced problems with erosion at different places along the shoreline. One of the most affected areas in the municipality is Bjärred, located some distance north of Lomma village. During winter storms in 2006/2007 erosion caused a waste water pipe to be almost exposed, thus in danger of breaking and polluting Lomma bay. In addition, some walls located right above the cliff were damaged by the wave impact.

The consultant company Ramböll Sverige AB was chosen to study the erosion processes taking place, and to suggest protective measures to solve the erosion problem in Bjärred. Protective measures were constructed along some sections of the shore from 2006 and onwards.

1.2 Aim

The aim of this thesis is to evaluate how applied protective measures against coastal erosion in Bjärred have worked so far and to investigate the possibility of introducing additional measures to improve the present conditions. These measures will be investigated on the most affected shoreline stretch in the study area both regarding the present condition as well as for estimated future climate change conditions. A computer model was used to simulate the behaviour of the suggested measures.

1.3 Method

The thesis work started in September 2010, with a first meeting at the Ramböll office in Malmö. The supervisors of the project Magnus Larson and Elisabet Hammarlund were present, and the main goals and a work plan were established.

The main sources of information for the thesis were the library at the Water Resources Department, LTH, the university electronic library, several Internet sources, and Ramböll reports. Before starting any calculations or analysis a literature study was performed during two weeks, where the focus was on especially the theory of sediment transport, erosion in general, and the site-specific project in Bjärred.

Information about historical sea water levels and wind data was gathered from SMHI. Aerial photos ranging between 1939 and 2010, obtained from Lantmäteriet, were studied and vegetation lines from the different available years were digitized. They were carefully compared in order to get an estimate of the erosion rate during this time period.

Lennart Persson, head of the Park department at Lomma municipality, provided some historical information of the site as well as conveying the municipality's concerns and interests in the affected area. A survey was performed in the most affected stretch in order to obtain different beach- and sea bottom profiles.

Erosion originating from the wave climate was calculated and analysed, including short-term impacts after storm events as well as long-term effects. The effect of the long-term erosion is achieved by calibrating a model to explain the shoreline behaviour in the study area by using the computer software GENESIS.

The digitized vegetation lines were used as shorelines in the computer model in order to simulate their response to wave action. Also survey results were helpful when providing information to the model regarding the beach and sea bottom profiles.

Once a satisfying model performance was achieved, several protective measures were evaluated and simulated, in order to predict the future evolution of the shoreline depending on the chosen solution. In addition, calculations and simulations of sea level rise and increased winds due to climate change were performed.

Finally, financial and environmental evaluations were carried out for the protective measures that proved to provide satisfying results in the simulation stage. All this information should provide the stakeholders a basis to make a decision regarding the most suitable protective measure to be applied in Bjärred.

1.4 Limitations

The geographical limitations of the study area in Bjärred are Västra Kennelvägen in the north and Öresundsvägen in the south, which covers a stretch of around 400 m. Perpendicular to the shoreline and out to the sea, the area covers around 800 m. Towards land the extension reaches just beyond the cliff, which is around 15 m.

Four vegetation lines were used to model the evolution of the study area, and two were employed for calibration purposes and two for validation. They comprise the period between 1978 and 2010. Water level data and wind data between 1976 and 2008 were used to perform analysis for wave runup and cross-shore transport.

When performing calculations and simulations no considerations have been taken to ice in the area. In reality, ice might be present during some periods of the year and thus lowering the wave heights, preventing erosion on the beach.

The considered future climate change conditions cover three different scenarios up to the year 2070. These scenarios take into account a rise in sea water level, an increase in wind strength, and a combination of these. The project does not take into account increased precipitation nor increased sea currents in the area.

2. Coastal erosion and protective measures

The study area in Bjärred has presumably been affected by coastal erosion processes during the past decades. In this chapter, the theory behind the phenomenon of erosion is developed to provide the reader with an understanding of the possible causes of the observed shoreline retreat. Some of the most common solutions adopted worldwide against erosion are also presented, listing advantages and disadvantages as well as the impacts that their implementation may lead to.

2.1 Introduction to coastal erosion

In a coastal area there is a continuous supply and loss of sediments and, as long as they are balanced over a period of time, the beach is in equilibrium. This means that the profile or shoreline can experience retreat or accretion but they always tend to recover to their original shape.

If the rate at which sediments leave an area is higher than the incoming rate this equilibrium is lost, and erosion will occur. If so, the shoreline will change in shape and location as sediment is transported away. The sediment might either be transported in the longshore direction or the cross-shore direction. Longshore drift is mainly caused by the impact of oblique wind-generated waves and associated nearshore currents. Cross-shore drift, instead, is mainly due to storm impact and high water levels. Human activity around a coastal area can accelerate these processes and increase the erosion rate. (CEM, Longshore Sediment Transport, 2002a); (CEM, Cross-shore Sediment Transport Processes, 2002b)

2.2 Sediment transport

As mentioned above, beach erosion or accumulation can be caused either by gradients in the transport of sediments along the shoreline or in the transport perpendicular to the shoreline. A continuity equation, called a *one-dimensional sediment balance* (see Equation 2.1), is formulated by conservation of sand volume and describes the quantity of moving sediment for a shore section for a specific time (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

$$\frac{\partial y}{\partial t} + \frac{1}{(D_B + D_C)}\left[\frac{\partial Q}{\partial x} - q\right] = 0 \qquad \text{Equation 2.1}$$

For a better understanding of this formula and definition of variables, a cross-shore view of a beach profile is presented in Figure 2.1 below.

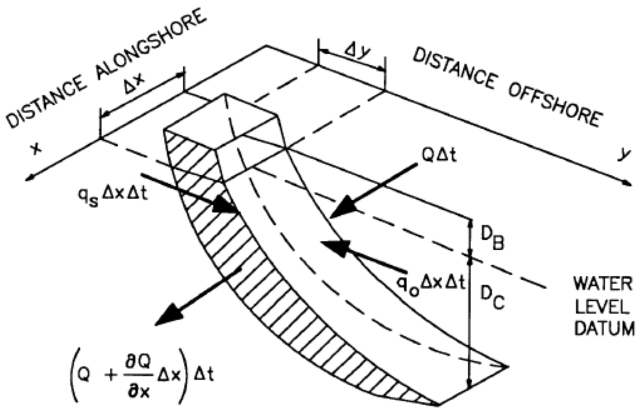

Figure 2.1 Cross-section view (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

As the beach profile can move either seaward or landward, Δy expresses the change of position for a specific shoreline segment (Δx) and range of time (Δt). The vertical change is expressed by means of the beach berm elevation (D_B) and the closure depth (D_C), both referred to the mean sea water level. The change in volume is determined by the net existing amount of sand of the section, where two main factors contribute to it. The first factor is ΔQ, which stands for the difference in the longshore transport rate Q. The second one are sinks and sources ($q=q_S+q_o$), which provide or removes sand from either the shoreward or offshore side.

2.2.1 Cross-shore sediment transport

Cross-shore transport of sediments can be directed landward or seaward. During storm events, a large amount of sediment can be carried away from the shore during a short period of time. When calm weather conditions take place onshore transport typically occurs, lasting for a longer time period and carrying smaller amount of sediment but during a longer period of time. Sometimes fine material might be transported away from the shore and settled far out in the sea, thus not returning to the shore. (CEM, Cross-shore Sediment Transport Processes, 2002b)

If the system is balanced the annual net transport of sediments perpendicular to the coast should be zero. Often, on a yearly basis, good weather conditions do not last long enough to compensate for the sand lost and in those cases, the subaerial part of the beach is eroding.

A simplified analytical equation (Equation 2.2) can be used to estimate the cross-shore transport due to dune erosion on an event basis. (Hanson, Larson, & Erikson, An analytical model to predict dune erosion due to wave impact, 2004) Figure 2.2 shows the variables to be considered for the calculations.

$$\Delta V = -4C_S \frac{(R-z_0)^2}{T} \Delta t \qquad \text{Equation 2.2}$$

ΔV (m^3/m*year) gives the total transported volume of sediments, C_S is a transport coefficient specific of the study area, which is obtained through calibration, and Δt is the duration of the influence of waves, which have a period T. The parameter z_0 represents the height between the mean sea water level and the foot of the dune.

Wave runup, R, is the vertical height that water reaches on the foreshore above the still-water level and is calculated according to Equation 2.3 below. Equation 2.2 above shows that offshore erosion takes place only if R is higher than the level of the dune foot, meaning that water is able to transport away some sand from the dune and bring it seawards.

$$R = \tan\beta \sqrt{H_0 L_0} \qquad \text{Equation 2.3}$$

where

R = vertical runup height
$\tan\beta$ = average slope of the beach profile
H_0 = wave height
L_0 = wavelength

Figure 2.2 below shows a sketch of the variables used when calculating dune erosion.

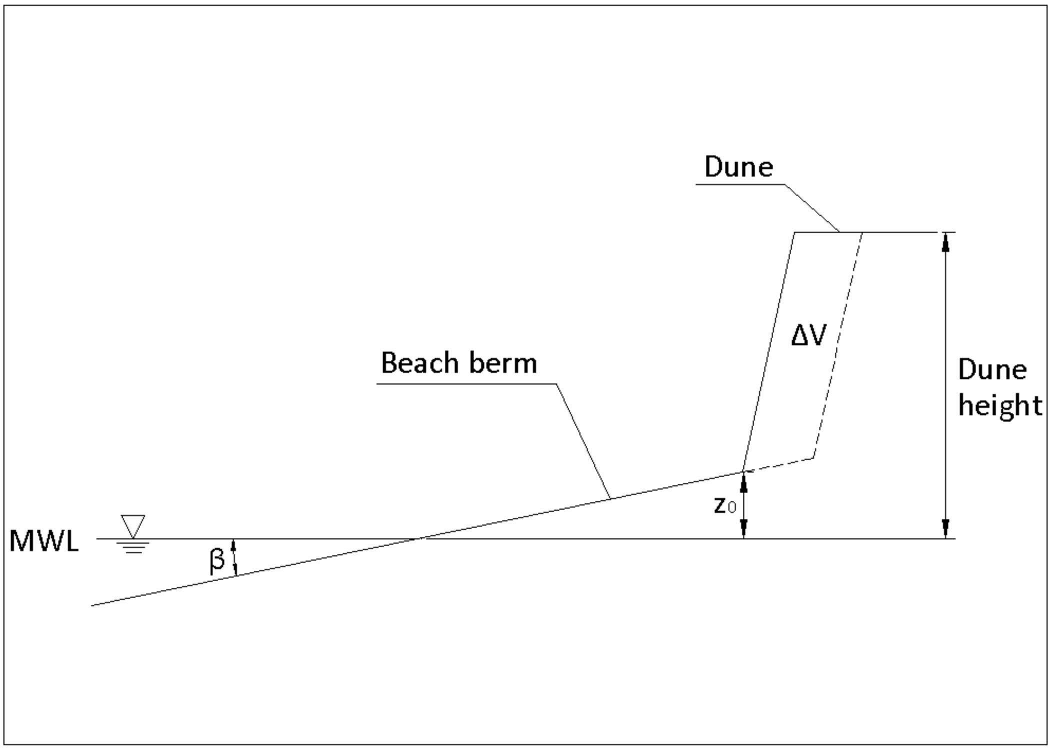

Figure 2.2 Sketch of variables used to calculate dune erosion

2.2.2 Longshore sediment transport

Waves breaking, combined with nearshore currents, might lead to transport of littoral material. This could consist of local rearrangement of sand, like bars, but sometimes it implies longshore displacement of beach sediment within the surf zone and parallel to the coast, causing a longshore transport of sand. In Bjärred it is assumed that the longshore currents and an oblique wave approach are the main cause of transport of sediments. Note that a possible cell circulation system of rip currents is not considered in this analysis.

The potential longshore sediment transport rate (Q_l), which depends on the available quantity of littoral material, is given by Equation 2.4 (CEM, Longshore Sediment Transport, 2002a).

$$Q_l = H_{b\,sig}^2 C_{gb} \left(a_1 \sin 2\alpha_b - a_2 \cos \alpha_b \frac{dH_{b\,sig}}{dx} \right)$$ Equation 2.4

Where the subscript b indicates breakpoint condition for the waves, C_g is the wave group velocity, α_b stands for the wave incident angle at the shoreline and H_{sig} is the significant wave height.
The first term in the equation, ${H_b}^2{}_{sig}\ C_{gb}$, accounts for longshore sand transport induced by obliquely incident breaking waves.

The second term, $\left(a_1 \sin 2\alpha_b - a_2 \cos \alpha_b \frac{dH_{b\,sig}}{dx} \right)$ accounts for the effect of a longshore gradient in breaking wave height, which is usually much smaller contribution. Nevertheless, it provides a better approach close to structures where diffraction effects may exist.
The non dimensional parameters a_1 and a_2 are expressed by:

$$a_1 = \frac{K_1}{16\left(\frac{\rho_s}{\rho}-1\right)(1-n)(1.416)^{\frac{5}{2}}}$$ Equation 2.5

$$a_2 = \frac{K_2}{8\left(\frac{\rho_s}{\rho}-1\right)(1-n)m(1.416)^{\frac{7}{2}}}$$ Equation 2.6

Where K_1 and K_2 are empirical coefficients that should be adjusted to match measured positions of shoreline change, and m is the average bottom slope from the shoreline to the depth of active longshore sand transport (closure depth). The factors including 1,416 convert the K_1 and K_2 coefficients to be used with the significant wave height. The porosity of sand on the bed is expressed by n, the sand compact density is ρ_s and the water density is ρ.

Longshore transport definitions

The longshore transport of sediment is a phenomenon responsible for the shoreline morphology, and determines whether a shoreline erodes, accretes, or remains stable (CEM, Longshore Sediment Transport, 2002a). Therefore it is important to estimate the general transport pattern, which promotes an understanding of the direction and rates of moving littoral material. Note, though, that those can be fluctuating quantities depending on the season.

Sediment transport is defined as positive if it moves towards the right for an observer looking seaward from the beach and negative if it is towards the left. The gross longshore transport ($Q_{l\ GROSS}$) accounts for the total amount of transported sediment, regardless of direction, during a specific time period. The net longshore movement of sediment ($Q_{l\ NET}$), on the other hand, takes into account all the changes in wave direction under a certain period and thus the transport direction. Figure 2.3 presents graphically the differences between gross and net longshore transport and how to estimate them for a specific time period.

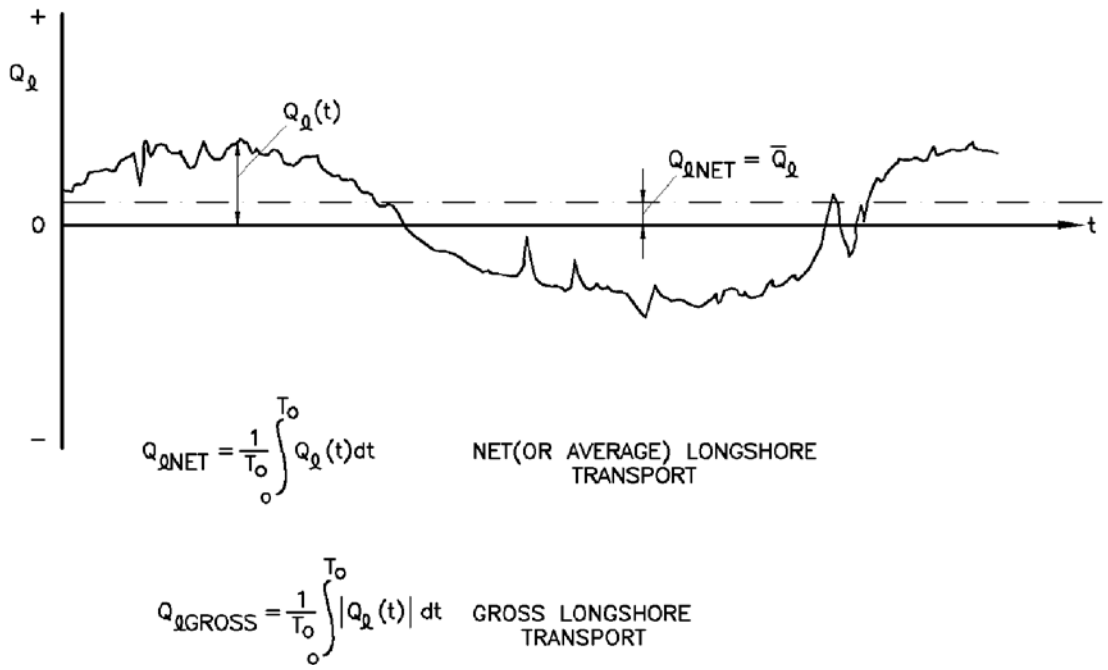

Figure 2.3 Longshore transport definitions (CEM, Longshore Sediment Transport, 2002a)

T_0 is the length of the record, which is often greater than one year, and $|Q_l(t)|$ is the absolute magnitude of the longshore sediment transport rate.

These two longshore transport rates are essential when planning engineering measures to overcome beach erosion. Bear in mind, though, that Equation 2.2 is valid only as long as there is an unlimited supply of sand available.

2.3 Measures against coastal erosion

When dealing with coastal erosion in a specific area there are different ways of classifying the actions that can be made to overcome it. One way of dividing them are into the following categories. (Johansson, 2003)

Table 2.1 Categories and actions

Category	Action
Abstention	No action taken
Retreat	Move structures inland
Adaption	Learn to live with it
Erosion protection	Structures, nourishment etc.

Another division of the actions can be the following. (Pope, 1997)

Table 2.2 Categories and actions

Category	Action
Abstention	No action taken
Adaption	Learn to live with it (includes retreat)
Moderation	Slow down the erosion rate
Restoration	Fill up the beach
Armoring	Separate water from land

The alternative *abstention*, or do nothing, can be chosen for those areas where nature can be allowed to continue eroding the shore. This is normally chosen when there are no economic values threatened by the erosion.

Retreat is when the objects in need of protection, for example a building or infrastructure, are moved inland so that they are not affected by the continuing erosion. If this option is considered, the rate at which the shoreline is eroding needs to be known to some extent. Retreat is an expensive measure, to move a building is a very costly operation, but might be a good option if there is a limited amount of objects to protect. (Johansson, 2003)

Adaption means that the natural beach evolution is allowed to continue but restrictions according the use of land and changes/investments are implemented. An example of a restriction measure could be that only mobile houses are allowed to be built in a threatened area. (Johansson, 2003)

Restoration, or more precise, beach nourishment means that material is placed on the beach in order to build up a shore that is allowed to erode. When the eroding shoreline has reached the position where it was before the nourishment the beach will be filled again. The measure of nourishment is discussed further in chapter 13.

Armoring means that a structure, such as a seawall or a bulkhead/sheet pile, is placed directly on the beach to immediately stop the erosion. The shoreline will remain at the point where the structure is placed since the structure separates water from land meaning that no erosion can occur.

Examples of *moderation,* which lower erosion rates, are groins, breakwaters and similar structures and are often used in combination with beach nourishment.

Choosing the best protective measure for an area requires a detailed understanding of the erosion processes taking place. The decision is not always easy, as the processes can be difficult to determine, and also because of the wide range of actions that needs to be taken into account.

The type of protective measure to choose depends on a number of factors that should be considered in each specific case:

- The amount of data and knowledge about the area are both important factors. For example, a detached breakwater is hard to design and find a correct location for. Thus, in order to be able to build one properly one must have a good knowledge of the area.

- Which processes control erosion at this site? For instance, if erosion occurs mainly by high water levels and storm waves a suitable shore protection measure could be to armor the beach. If erosion occurs because a groin captures sand from updrift, a bypass to bring sand downdrift might be needed.

- The desired final situation. Depending on which measure is taken, erosion can be stopped but with different final results. Some measures reconstruct the beach while some protect the dunes but leave the lower beach unprotected.

- Economic factors. If the desired measure or final result is too expensive one might need to use a less effective measure but one that fits better to the budget.

Statistics from the US Army Corps of Engineers (USACE) show that before the 1960's beach protection projects in the US consisted of an even amount of hard structures and nourishment projects, but after this decade around 80-90% of the projects consisted of nourishment. (Hillyer, 1994)

Some of the most commonly used protective measures worldwide are discussed below. The advantages and drawbacks of each of them are also presented to get a clear overview of the consequences of their implementation.

2.3.1 Groins/Jetties

A groin is a solid structure that is constructed from the shore and, most often, perpendicular to it. The structure lowers the longshore transport rates as sediments accumulate updrift of the structure, while erosion will take place downdrift of the groin. After the construction, sand accumulates and a beach will form almost parallel to the incoming waves. Along a shoreline it is common that many groins are placed one after another to form a system, where the biggest challenge is to make them work as a unit and not as single structures. If they are made too long they will work as single units and if they are made too short sand will pass through them and their function will be affected. (CEM, Longshore Sediment Transport, 2002a)

Since the groins halt the longshore sediment transport from updrift this might cause erosion further down along the beach since the supply of sediment downdrift is lowered. Groins do not stop the cross-shore sediment transport thus not being suitable in areas where this phenomenon is the dominating factor. During storms, the wind direction might be different than the predominating direction and if the waves hit the groin at the normally downward side rip currents might occur, causing sediment to be flushed out to sea and increasing the erosion on that side of the groin. Figure 2.4 illustrates the mechanisms of groins.

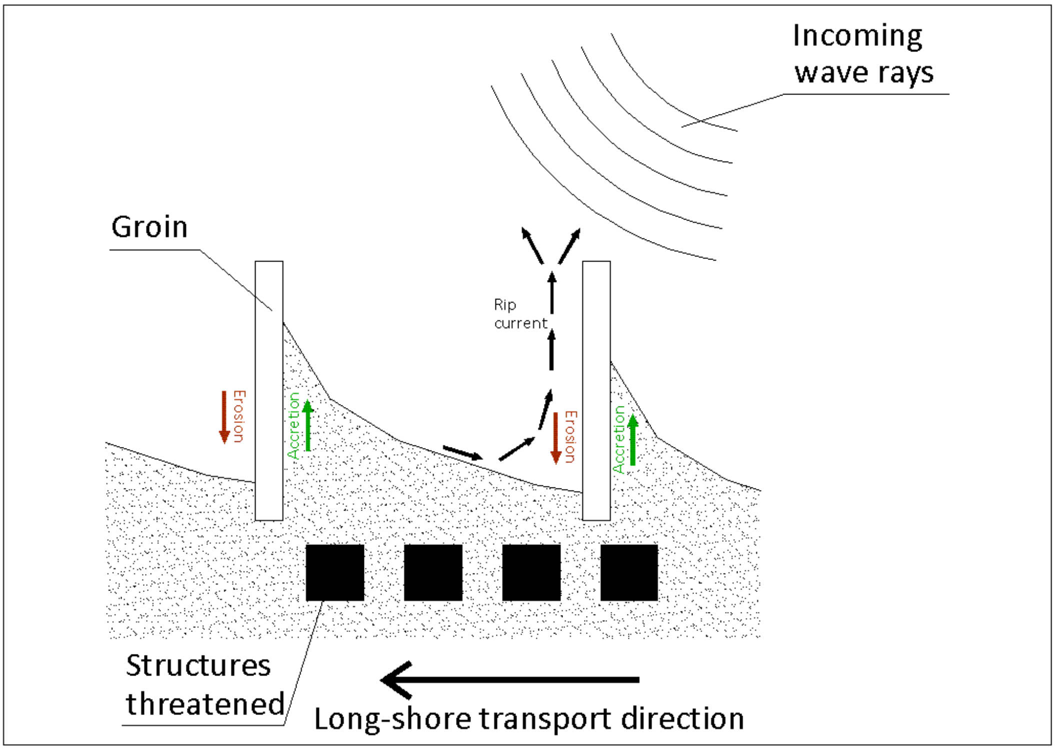

Figure 2.4 Schematic picture of the function of a groin including the negative effect of rip currents

When constructing a groin it is necessary to anchor it carefully to the bottom and on the beach. If the groin is not anchored sufficiently landward on the beach, the downstream erosion might create an opening between the groin and the beach. This can cause waves to pass *behind* the groin, making the erosion downstream even bigger.

Advantages

- They build up the beach. Good aesthetic effect
- The accessibility to the beach is not affected
- They might be used for recreation, such as swimming, fishing etc.

Disadvantages

- Rip currents may be formed, increasing the erosion and transport material away from the beach
- Downstream sections might be damaged since no sediment is transported from upstream
- If no sand is available upstream sediment might need to be added artificially
- Do not protect dunes

(Johansson, 2003), (Hanson, Coastal Engineering Course. Lecture Notes, 2009)

2.3.2 Revetments

Revetments are structures used to armor the beach. They are constructed on the actual shoreline, separating land from water and stopping the erosion completely. They can either be placed directly on the shoreline or some distance from the shoreline, protecting the dune but leaving the shoreline unprotected. Their task is to completely prevent shoreline retreat mainly due to cross-shore transport of sediment.

One major problem with revetments is that they often cause erosion downstream of the construction. If revetments are constructed, areas downstream that normally have been supplied with sediment will not experience this addition of sand and are thus erodes. The second major reason for downstream erosion is that the structure will reflect the incoming waves to some extent and by that contribute to interference. This leads to high particle velocities at certain areas, which can cause erosion. Figure 2.5 below illustrates the function of a seawall. (Silvester & Hsu, 1993)

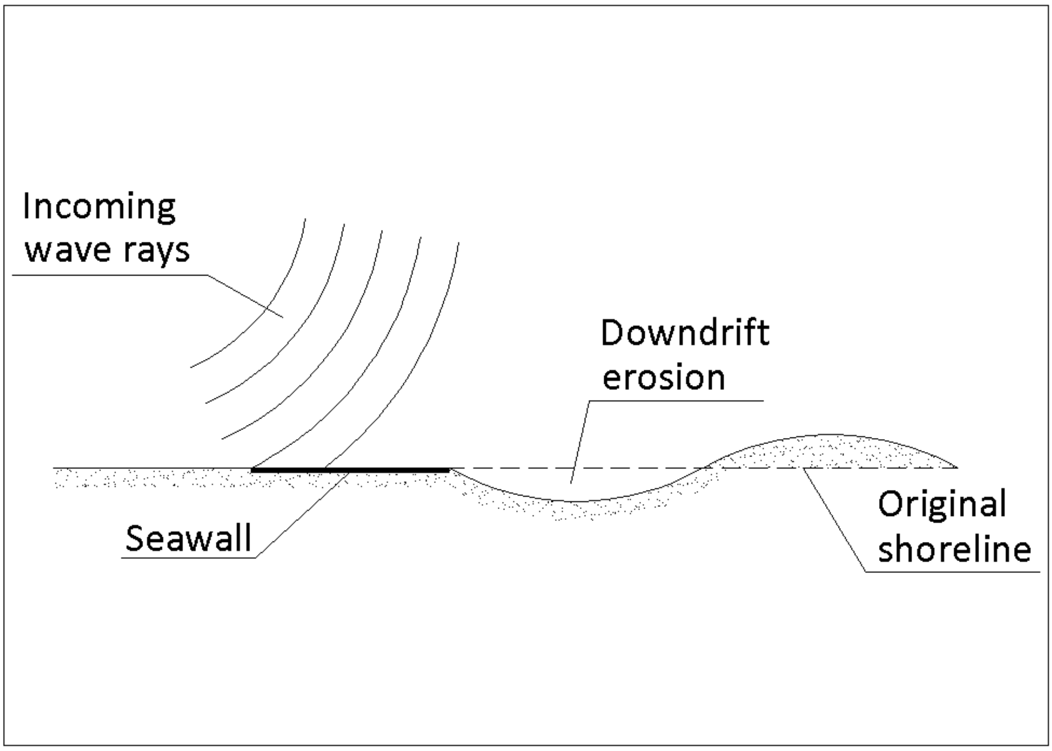

Figure 2.5 Schematic picture of a seawall and the dynamics of waves in front of it

An important factor to consider when constructing a revetment is the laying of the foundation. Since revetments most often are heavy constructions of stone or concrete it is important to have a good foundation, otherwise it will settle after some time. This, together with the construction itself makes revetments initially expensive.

<u>Advantages</u>

- Efficiently stops the erosion

<u>Disadvantages</u>

- High initial cost
- Cause downstream erosion
- Might be an obstacle and lower the availability of the beach
- Require slope adjustment and fill
- Difficult to repair
- Can lead to high runup

(Johansson, 2003); (Hanson, Coastal Engineering Course. Lecture Notes, 2009)

2.3.3 Detached breakwaters

Detached breakwaters are structures placed some distance seaward of the shoreline and most often parallel to it. Waves approaching the structure are diffracted and loose height, and by that transport capacity causing sediment to settle behind the breakwater. Diffraction (see chapter 7) also makes the waves propagate in behind the structure. Up to 50% of the longshore transported sediment might, when it enters the calmer area behind the breakwater, fall to the bottom and form a beach that is called either a salient or a tombolo (Johansson, 2003). Diffraction of the waves cause them to transport sand into the area behind the breakwater, which leads to even more accumulation of sand behind the structure. A salient behind the structure is a desired feature but a tombolo is not wanted. When the latter is formed, the longshore sediment transport ceases, which may cause downstream erosion. Figure 2.6 illustrates the processes and impacts associated with detached breakwaters.

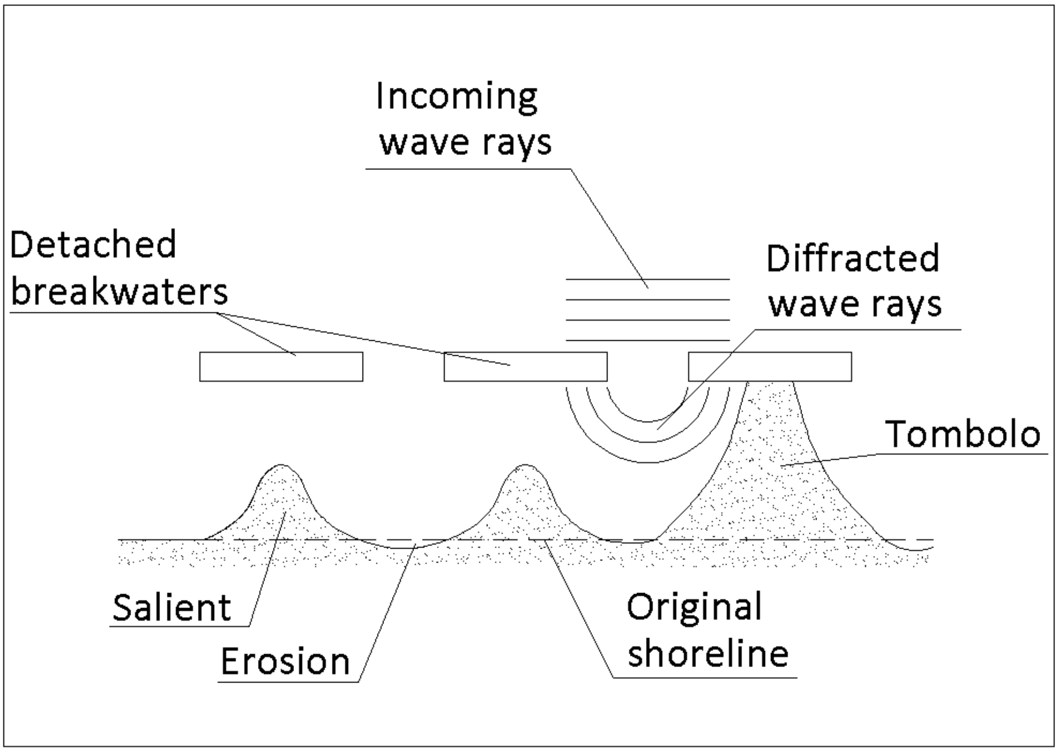

Figure 2.6 Schematic figure of detached breakwaters

Similar to groins, breakwaters are often constructed in series including many structures. The distance between the structures is one of the difficult parameters to decide when designing detached breakwaters. One suggestion is a distance of 1.5-5 times the length of a single structure (Gustavson, 1994) while another one is 0.5-2 times the length (Johansson, 2003). Detached breakwaters are often used together with beach nourishment and it often also decreases the cross-shore sediment transport.

Advantages:

- No structures on the shore, thus creating a natural beach
- Traps sand efficiently
- A secondary protection for the shoreline since they reduce the force of the incoming waves

Disadvantages

- Does not look natural
- High initial costs since constructed in the water
- Might cause downstream erosion before stabilization has occurred
- Difficult to design properly

(Johansson, 2003); (Hanson, Coastal Engineering Course. Lecture Notes, 2009)

2.3.4 Beach nourishment

A beach nourishment project has the goal to protect the shore and upland facilities from offshore transport of sediments caused by storms. The construction of a beach berm, dune, nearshore berms, or a combination of them is necessary, and both scheduled periodic and emergency nourishments have to be planned.

Dunes provide protection against runup and overtopping as well as being a reservoir of sediments to compensate the storm effects. Vegetation growth can be added to obtain more resistant dunes. Beach berm dissipates some water energy, and its width is established depending on the desired protection level. Nearshore berms are artificial shore parallel bars that help reducing the energy of storm waves before they reach the beach. If they are placed shallower than the closure depth, sediment can in due course feed the berm when calmer conditions prevail.

Beach fill can also be used in combination with structures, which can enhance the performance of the fill by providing a better control of the movement of sediments. Different examples are sketched in Figure 2.7 below. If a combination with structures is used, however, additional fills should be made either downdrift or updrift of the structures to reduce the undesirable side effects of the implementation of structures.

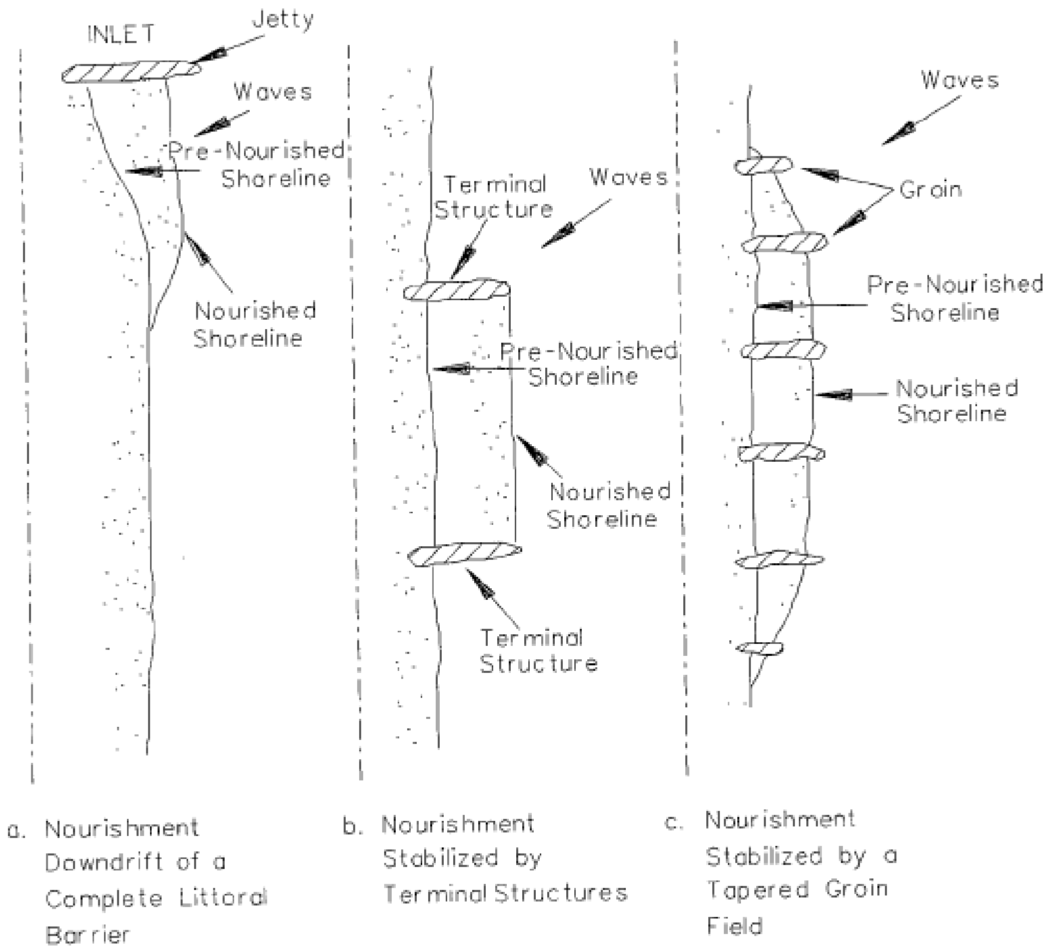

Figure 2.7 Use of structures to enhance beach nourishment projects. (CEM, Beach Fill Design, 2003)

<u>Advantages</u>

- No structures are necessary
- Good protection against moderate waves
- Moderately initial costs
- Restores the shore quickly

<u>Disadvantages</u>

- Needs regular maintenance and renourishment
- Good background knowledge of the area needed

(Johansson, 2003); (Hanson, Coastal Engineering Course. Lecture Notes, 2009)

3. Description of the study area and assessment of protective measures

In the following section a general description of the study area is provided, including the geographical location of Bjärred, a description of its shore profile and the geological characteristics that are present. An evaluation of the performance of the actions that Lomma municipality has taken in the area is made and a chronological summary of all events in Bjärred is listed in the end of the chapter. The evaluation of the protective measures is based on reports from Ramböll as well as several field visits made during autumn 2010.

3.1 General

The study area encompasses a stretch of the Swedish west coast located in Bjärred, Lomma municipality. Bjärred is in the southwest of the Skåne province, around 25 km north of Malmö and 10 km west of Lund and has around 10 000 inhabitants.

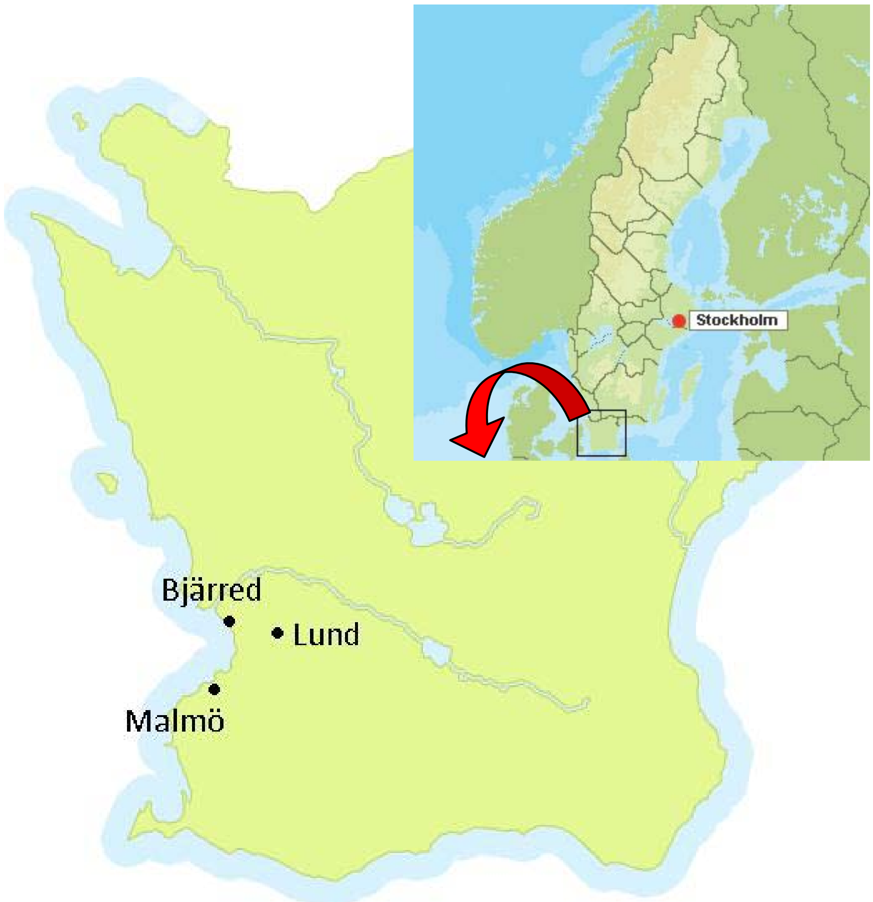

Figure 3.1 Map of Skåne, showing important locations for the project

The coastal area in Bjärred is used as a recreational area, where there is a walking and bicycle path parallel to the sea, from which a nice view of Lomma bay is obtained, as well as of the Öresund Bridge. There are also several piers where mainly leisure boats are held. Bjärreds Optimistjolleklubb (BOJK) has its club house on the shore and a number of boats are tied to their own pier. A stretch of the shoreline in Bjärred is shown in Figure 3.2.

Figure 3.2 Stretch of shoreline in Bjärred, Lomma municipality, 2010

The study area covers approximately a coastal stretch of 400m between Västra Kennelvägen (in the north) and Öresundsvägen (in the south). This particular area is of interest, as a major retreat of the shoreline has been observed during the last years, endangering properties and a waste water pipe close to the shore (Figure 3.3).

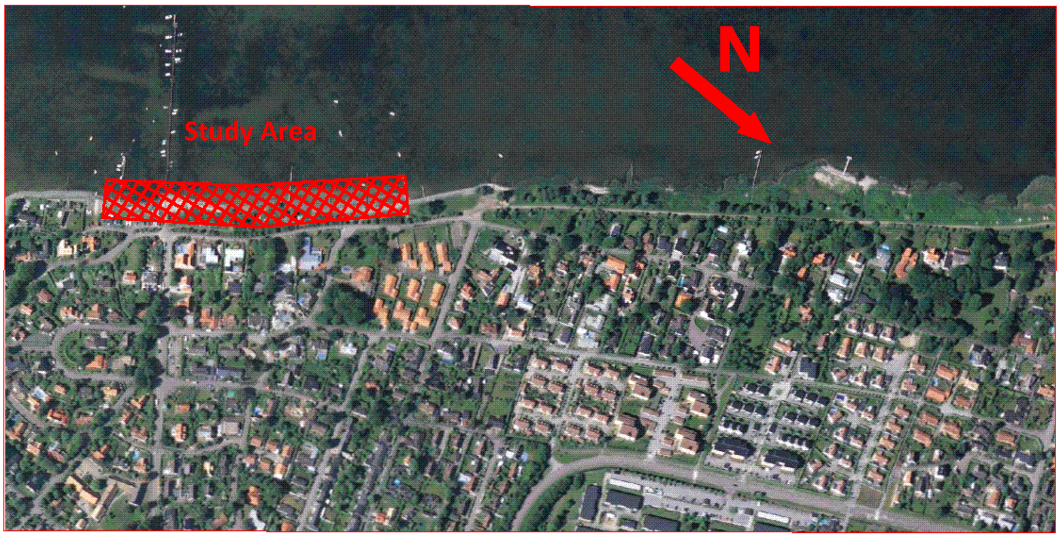

Figure 3.3 Aerial photo of Bjärred showing the study area (Lantmäteriet, 2010)

3.2 Geology

Lomma bay is a very shallow water body where the depth 800m offshore is only about 2m. (Brännlund & Svensson, 2005) At a distance of 900 m the depth is around 4 m, which is assumed to be the depth of closure (see Appendix III).

A study performed by (Davidsson, 1963) concerning the geology and morphology of Lomma bay showed that the bedrock consists of limestone with sinks scattered. The soil consists of a layer of glacial till, with elements of gravel and sand. The sinks spread in the area contain a mix of clay, organic matter and sand.

Another study shows that the soil in the study area is of varying grain size, with a significant content of clay and silt, but with no particularly dominating grain size (Brännlund & Svensson, 2005). This makes it hard to estimate the grain size that must be used as input for the computer modelling. Since the sediments are of varying grain sizes the feature behind the beach berm is denoted as cliff instead of dune.

3.3 The shoreline

Aerial photos, ranging from 1939-2010, indicate that the shoreline seems to have receeded significantly during this period (Figure 3.4 and Figure 3.5).

Figure 3.4 Aerial photograph of Bjärred in 1939 (Lantmäteriet, 2010)

Figure 3.5 Aerial photograph of Bjärred in 2010 (Lantmäteriet, 2010)

From the edge of the water, the foreshore slopes lightly upwards until reaching the cliff foot, which has an average height of 1.4 m (obtained from field surveys, see chapter 5). The top elevation of the cliff is at around 3 m referred to the mean water level (Brännlund & Svensson, 2005). Along the shoreline seaweed tends to accumulate, which is enhanced by the existence of the many piers. Vegetation grows naturally, offering protection for the beach berm and the cliffs. As mentioned before, the soil contains a wide range of grain sizes. Figure 3.6 shows a small representative stretch of the beach in the study area.

Figure 3.6 Photograph of the beach in Bjärred (2010-09-03)

3.4 Measures against coastal erosion applied in Bjärred

A previous study in Bjärred (Brännlund & Svensson, 2005) comes to the conclusion that most of the erosion at the site occurs when waves hit the cliff and transport the material offshore. Since the material of the beach is of varying grain sizes with a significant content of silt and clay, some of this material is transported out to sea and will not be returned onshore.

The study shows that the erosion takes place when the total water level (wave runup and water level) reaches the cliff foot. This happens when water levels are high in combination with large waves. Evaluation of historical wave height and water level data gives that the runup height reaches the cliff in average 3.6 times per year. (Brännlund & Svensson, 2005)

In this study data from a longer period of time (compared to Brännlund & Svensson) is used in order to obtain a more reliable result and the estimate 3.6 times per year will be evaluated. A survey in autumn 2010 was performed to determine some properties of the study area and the recent retreat. Calculations and simulations of different climate scenarios of future sea water level rise and/or changing wind will provide values for predicted runup heights in the future.

3.4.1 Overview

Coastal erosion has been changing the beach profile in Bjärred for a long time. The first known municipal measure against erosion was taken in 1992, when about 80 meters of revetments were built outside BOJK. The revetments consisted of concrete slabs with voids for vegetation growth. In 2004, some distance south of the study area a sandy beach was nourished in order to restore the shoreline to its original shape. (Ramböll, Erosionsskydd i Bjärred- underlag för samråd med länsstyrelsen, 2009)

In 2006 a 100 m test section with different types of revetments and plastic reinforcement nets, was built in the northern part of the study area. The section was divided into three different sub-sections, each of them around 30-40 m long. The most northern part consisted of concrete slabs with a slope of 1:8. The middle part of the section was built with plastic reinforcement nets, where half of them were filled with macadam and the other half with concrete. The third sub-section consisted of concrete slabs with a slope of 1:3. (Ramböll, Erosionsskydd i Bjärred- underlag för samråd med länsstyrelsen, 2009)

During the winter of 2006-2007, heavy storms caused walls of private properties to fall apart and threatened to destroy a waste water pipe. This development stressed the need to evaluate the tested structures in order to build effective protection measures in the rest of the area.

3.4.2 Plastic reinforcement nets

The protective measure in the other test section consisted of plastic reinforcement nets, around 10 cm deep (see Figure 3.7 below). It was initially made of two types, one where the nets were filled with macadam and one filled with concrete. Regardless of the filling, a layer of geotextile separated the plastic reinforcement nets from the underlying soil and protected it from erosion.

When an evaluation of the structures took place during the spring of 2007 it was found that the part filled with macadam had some defects. As can be seen in Figure 3.7, a majority of the macadam had been flushed away and as a result the corrugated bars, holding the nets in place, had come off from the ground. Consequently, some of them bulged, making them unstable and not pleasant to walk on, which lowered the recreational value of the coastal area. Despite the geotextile it seemed that the underlying soils had been eroded, making the ground unstable. After this evaluation it was decided that further on no more plastic reinforcement nets should be constructed in this area. (Ramböll, Erosionsskydd i Bjärred- utvärdering av skyddsåtgärder, 2007)

Figure 3.7 Plastic reinforcement nets where macadam has been flushed away (Ramböll, Erosionsskydd i Bjärred- utvärdering av skyddsåtgärder, 2007)

3.4.3 Concrete slabs

The revetments of the test site consisted of concrete slabs (dimensions 0.4 x 0.4 x 0.1m) with voids to allow vegetation growth (Figure 3.8). Underneath the revetments a layer of geotextile protects the underlying soil. The geotextile itself is covered with a layer of soil, helping the vegetation to settle. In order to get the revetments into the planned position a backfill of packed soil/clay was done before placing the blocks. This is a critical moment for the construction, since it is the foundation of the whole structure.

Figure 3.8 Concrete slabs on the shore in Bjärred (2010-09-03)

The evaluation in 2007 showed that the revetments have performed satisfactorily, while in the areas next to the revetments great damage due to erosion was observed. Some small defects were noted, such as problems with anchoring the blocks but none of the defects were of greater extent than they could easily be adjusted. (Ramböll, Erosionsskydd i Bjärred- utvärdering av skyddsåtgärder, 2007)

The revetment itself, however, does not look natural and the vegetation had not spread as much as desired (see Figure 3.8 above). Because of the steeper slope at sub-section three some of the blocks had cracked. As mentioned above it is very important for the blocks that the underlying material is well packed and the evaluation showed that this seemed to be the case. Since the revetments had been so successful it was decided that the shoreline at the properties Bjärred 37:2 and Bjärred 37:3 should be protected with the same concrete blocks as in the test section.

Contradictory to what the evaluation in 2007 stated, signs of undermining were visible during the survey performed in September 2010. In Figure 3.9 it can be seen that some of the slabs have started to rotate towards the sea, which is an indication that erosion might occur underneath the slab. If this continues the whole structure may fail, leading to erosion once again and high costs to restore the structure. Further investigation of the reason is needed, but the explanation for this kind of failure is often negligence while constructing the foot of the structure.

One should note that the evaluation of the structures only considered their technical performance and not side effects such as downdrift erosion.

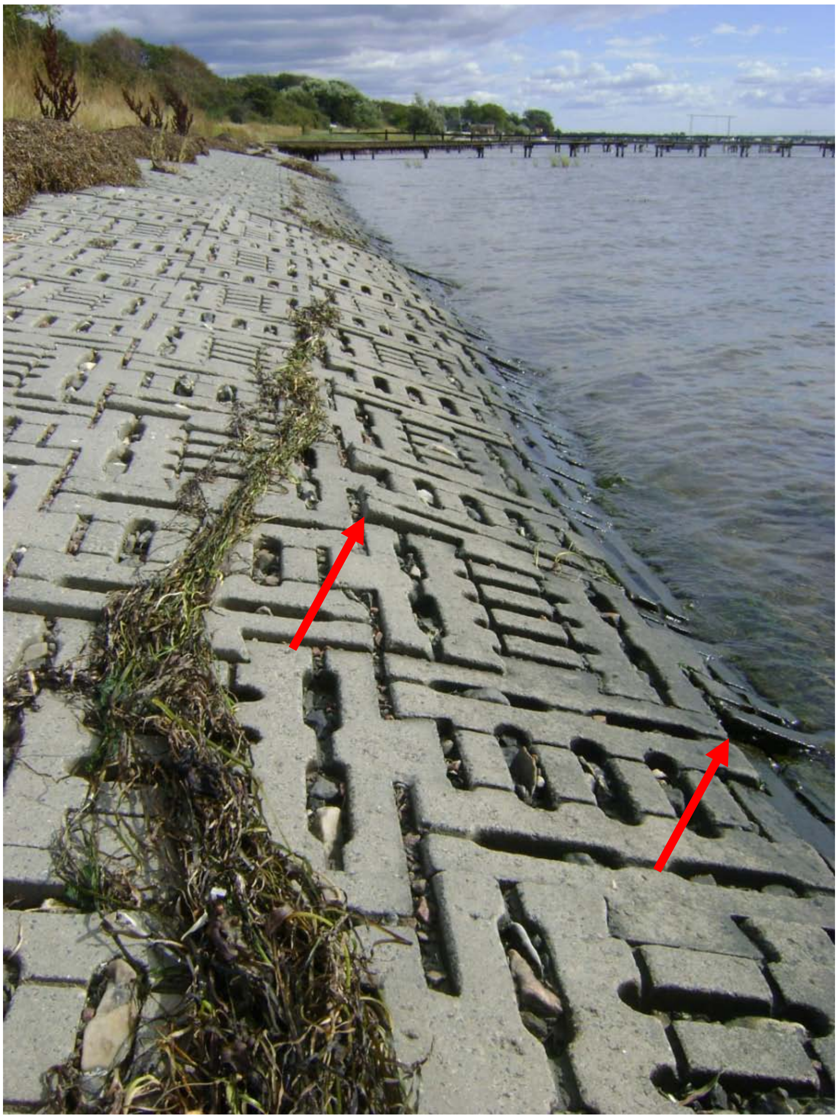

Figure 3.9 Signs of undermining of concrete slabs (2010-09-03)

3.4.4 Chronology for protective measures in Bjärred

A summary of the chronological events in Bjärred regarding storm impacts and applied protective measures is given below.

1992

- 80 m of concrete slabs were built at BOJK (Bjärred 37:1)

2004

- Area south of the study area was refilled with sand
- Lomma Municipality, representatives from Ramböll and Länsstyrelsen met to decide how to stop the erosion

2006

- A test site of 100 m length was built in the northern part of the area with three different subsections of 30-40 m each. The first subsection consisted of revetments (concrete slabs) with a slope of 1:8, the second sub-section was built using plastic reinforcement nets and the third sub-section consisted again of revetments but with a slope of 1:3.
- Winter storms caused walls by private homes (Bjärred 37:2 and 37:3) to fall apart and waste water pipe was threatened

2007

- Evaluation of the test section after the storm events
- An application to construct revetments in front of some properties (Bjärred 37:2 and 37:3) in the southern part of the study area was handed in.

2008

- The test section with plastic reinforcement nets was rebuilt with revetments
- The shoreline at Bjärred 37:2 and Bjärred 37:3 was covered with revetments.
- Beach sediments at some stretches were flushed out to the sea due to heavy erosion. This exposed a waste-water pipe, especially in the unprotected area south of the test site.

2009

- Planning and construction of a new 100 m long revetment.

2010

- Master thesis to evaluate structures and to simulate a solution for the unprotected stretch

(Ramböll, Erosionsskydd i Bjärred- underlag för samråd med länsstyrelsen, 2009), (Ramböll, Erosionsskydd i Bjärred- utvärdering av skyddsåtgärder, 2007)

Figure 3.10 below shows an aerial photo with the different sections of interest along the shoreline in Bjärred, from north to south containing the following sections:

1. The test section built in 2006
2. Section built in 2009
3. Area without protective measures
4. Revetments built in 1993
5. Revetments built in 2008

Included in the specific study area are the three middle sections (2, 3 and 4). In the computer simulation (see chapter 11-13), only sections 2 and 3 will be modeled.

Figure 3.10 Aerial photo of the Bjärred area (Lantmäteriet, 2010)

Along Section 3, around 200 m of the studied shoreline is still unprotected. The focus of this thesis will be to evaluate different structures to provide effective protection for this vulnerable shoreline section.

4. Aerial photograph analysis

The evolution of the shoreline in Bjärred during the past decades can be observed and quantified when comparing aerial pictures from different years. An accurate analysis of them can provide information about the rate of shore retreat, which areas are the most affected and thus define where to put more efforts regarding protective measures. Aerial pictures from Bjärred were obtained from Lantmäteriet for the years 1939, 1955, 1963, 1973, 1978, 1985, 1998, 2002, and 2010. (Lantmäteriet, 2010)

4.1 Method

Since there is no information on the precise date when each picture is taken there is an uncertainty of the water level for each of the pictures. Due to the variation of the water level an analysis of the shoreline is not possible so the vegetation line is compared instead. The vegetation line corresponds fairly well to the erosion scarp, which is present at many sites in Bjärred. The retreat of the vegetation line is assumed to be representative for the shoreline retreat and could therefore be used to analyse the general evolution of the coast.

First of all, aerial photographs were digitized and transformed into the reference system used in Lomma, SWEREF 99 13”30’ (Yassin, 2010). The photos were scaled and stretched to have the best fit to the base map of Bjärred.

The vegetation lines were then extracted and compared graphically to each other by using AutoCAD Civil 3D. The photographs from 1939 and 1955 were of lower resolution and to find the vegetation lines was a difficult task. For this reason it was decided that those images should be left out in the following analysis. The rest of the pictures were still of varying quality, where the ones of good quality made it easy to find the vegetation line and vice versa.

The available pictures were divided into two periods, where the first one ranges from 1963-2002 and the second from 2002-2010. This was done mainly because the major changes in the shoreline in 2006, when the revetments in the northern part of the study area were built.

The digitized vegetation lines were analyzed using the ArcGIS based tool Digital Shoreline Analysis System (DSAS), which quantified the total erosion and the erosion rate between certain years. The DSAS performs analysis through casting transects from a baseline, crossing the different shorelines and from that calculating distances and transport rates.

4.2 Results

The results show that, in general, significant erosion has taken place in the area, even though the obtained rate is lower when compared to the previous study (Brännlund & Svensson, 2005). The main reasons are likely to be the resolution of the aerial photos, the human subjectivity factor when obtaining the vegetation lines, and the period studied.

Table 4.1 below shows a comparison of the previous results and the results obtained from this analysis (2010) for the same area.

Table 4.1 Comparison of erosion in Bjärred

	Maximum retreat	**Average retreat**	**Yearly average retreat**
Brännlund&Svensson 1960-2004	16 m	8,2 m	0,18 m/year
Present study 1963-2002	13 m	4,3 m	0,11 m/year
Present study 2002-2010	10 m	1,3 m	0,14 m/year

Figure 4.1 and Figure 4.2 below show the evolution of the **vegetation lines** from 1963-2002 and for 2002-2010. As can be seen there is some variations of the erosion rate along the beach. Areas painted in green show accretion, whereas yellow, orange and red areas represent erosion of varying degree. The variations can be explained by local changes in topography, sediment transport, and composition of soil layers. (Brännlund & Svensson, 2005)

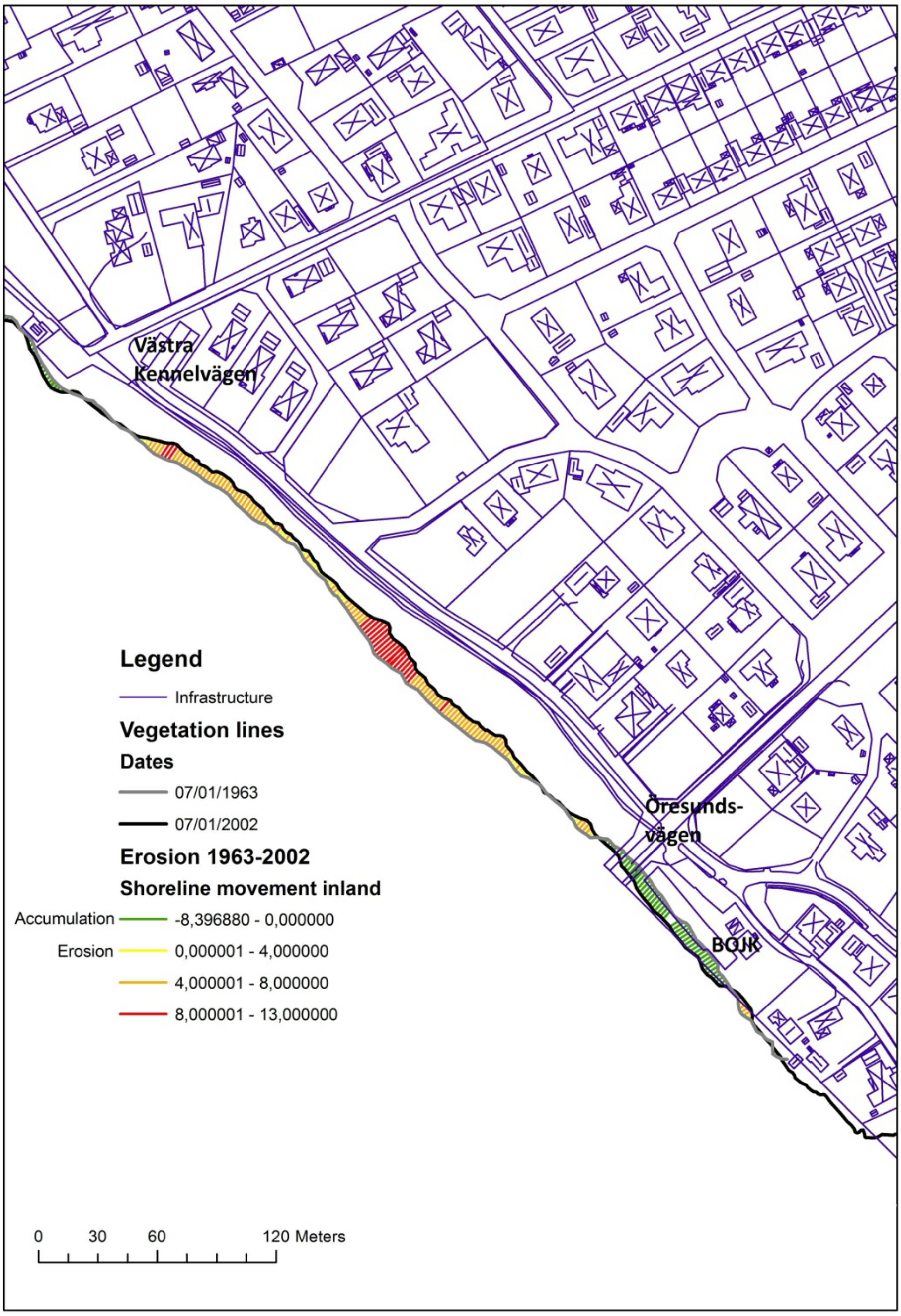

Figure 4.1 Vegetation line change 1963-2002

As can be seen in Figure 4.1 above, the vegetation line has changed within an interval from -8 up to +13 m (where a positive value equals erosion). Erosion appear to take place in the whole study area unless for a small stretch, corresponding to the BOJK, where there is accretion (green area). Due to the revetments built in that part in 1993, the accumulation in that area is most likely not natural but originated from restoration of the shoreline. It must be noted that it is the vegetation line and not the shoreline that is visualized in the figure and that the actual shoreline is located some distance seaward of the vegetation line.

The largest problem with erosion occurs in the middle part of the study area, where the shoreline has retreated between 8 and 13 m during these years. If the erosion continues at the same rate infrastructure behind the beach is threatened in the near future. The erosion rate will be used in further analysis when estimating the cross-shore transport occurring in the area.

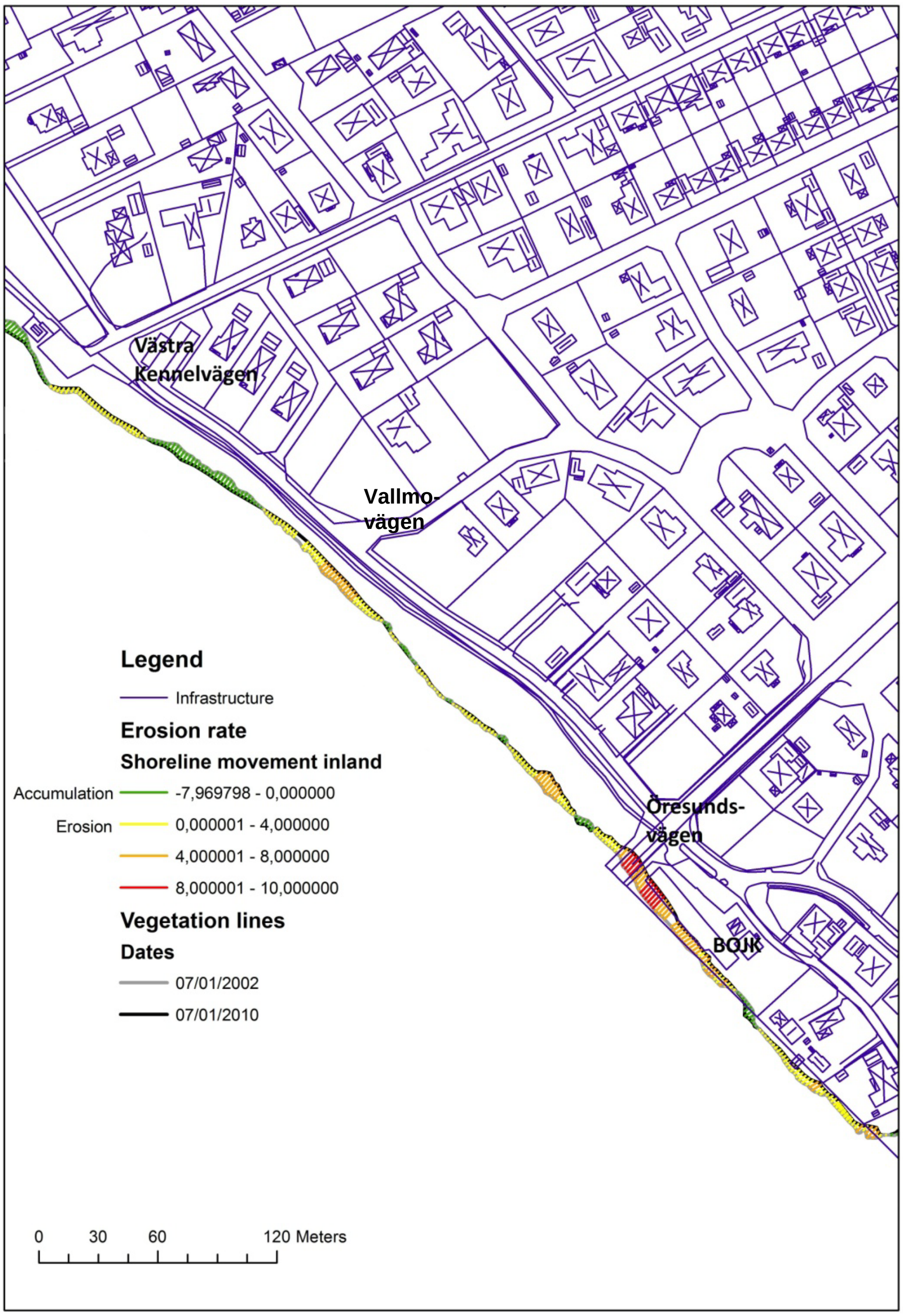

Figure 4.2 Vegetation line change 2002-2010

Similar to the time period 1963-2002, Figure 4.2 shows some parts where accumulation has occurred and the explanation is likely to be the same as for the previous time period. In 2006 revetments were built in the northern part of the area, just south of Västra Kennelvägen, and is assumed to be the explanation to the accumulation in the area. The rest of the shoreline follows the same trend as the previously analyzed time period, with varying erosion along the shore, but with the greatest erosion in the area around Vallmovägen.

Both analyses show a quite low erosion rate for a stretch just north of Öresundsvägen. This can be considered when planning protective measures for the study area, where this part might not need to be protected as extensively, implying that costs can be reduced.

The major sources of errors are mainly the human factor when digitizing the vegetation lines, the resolution of the aerial photographs, and the fact that some aerial photographs have been transformed from another reference system. (Karlsson & Martinsson, 2010)

The reference system in which the aerial pictures originate from is not known but the comparison with RT90 below give an indication of the transformation error. An estimation of the average error when transforming from the reference system RT90 to SWEREF99 is 0,07 m. The maximum error for this transformation is 0.2 m, which is relatively low compared to the other sources of errors. (Lantmäteriet, 2010)

All aerial photos and digitized vegetation lines can be seen in Appendix I.

5. Surveying

5.1 General

A field study of the area was carried out in October 2010 to get a general overview of the shoreline and beach conditions. (Brännlund & Svensson, 2005) performed a survey of the area, where they measured beach and bottom profiles and took soil samples of the beach sediments.

The survey in 2010 focused on measuring a number of beach and bottom profiles to obtain an accurate and representative contour of the study area and estimate some field parameters, such as the beach slope and the cliff foot height. These values will be useful when calculating wave runup heights on the foreshore and will give input data for the computer model GENESIS. The survey will also allow a comparison of the beach profile evolution within the past five years (2005-2010) to find out if there has been any cliff erosion.

5.2 Method

The measurements were made in the middle of October 2010, during a sunny day. In total 8 beach profiles and 3 sea bottom profiles were measured.

In the morning the weather was calm but during the afternoon the wind speed increased, causing some problems with increased wave height. The first two bottom profiles, starting from the southern part of the study area, were measured before noon, when the weather was calm, and are therefore estimated to be more accurate. These profiles have a distance of around 800m seawards, while the last one (the most northward) performed during the afternoon reaches 600m from the shore. The reason for this shorter length is that the waves caused disturbances, making the measurements hard to execute.

The measurements themself were made using a GPS device on board of a small rowboat. The GPS hand probe was attached to a stick, which was lowered to the bottom and the coordinates (x,y,z) were given at that specific point. In order to get a straight line perpendicular to the beach the measurements were started some distance out to sea and the oarsman aimed for a specific point on the shore all the time. Single measurements were then taken at regular distances from the starting point.

During the survey there were quite strong currents in the sea, making it difficult to keep the stick, holding the GPS tool, completely vertical. This might cause the GPS to show a greater depth than the actual. During the afternoon the increased wind made it even more difficult to keep the stick vertical. Another source of error for the measurements might be that the stick sinks into the soft, sandy bottom and hence a greater depth than the actual is observed. Since it is not known whether the stick was completely vertical or if it had been sinking into the bottom the accuracy of the survey is estimated to be not more than 0.1 m.

5.3 Results

The measurements are expressed in the height reference system RH2000. In order to compare the surveyed profiles with water level data (which is given as water level height relative to a specific location) a conversion from RH2000 to the mean water level (MWL) in Bjärred is necessary.

Bjärred is close to the station of Barsebäck, which has available records of water level since 1976 (water level is developed further in Chapter 8). The mean water level in 2010 is 11,1 cm higher compared to the zero point of RH2000 (SMHI, Beräknade medelvattenstånd, 2010). To convert the heights from RH2000 to the local mean water level 11,1 cm are subtracted from all height measurements. An illustration of this can be seen in Figure 5.1.

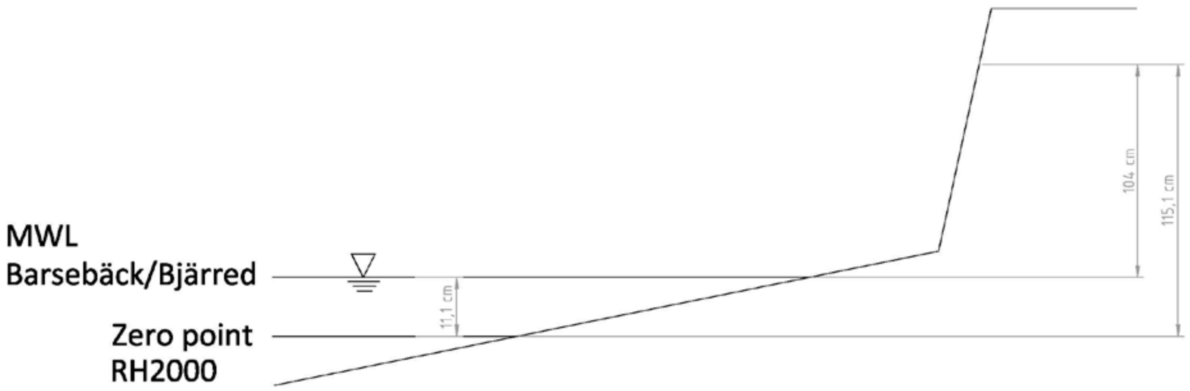

Figure 5.1 Schematic picture of the MWL in Barsebäck/Bjärred relative to RH2000. If a point has the z-coordiante 115,1 cm in RH2000 it corresponds to 104 cm relative to the MWL

Uncertainties when performing the survey such as a correct handling of the GPS, finding the correct measurement points, and other human factors are the major sources of errors.

Figure 5.2 shows the location of all the measured profiles on a map of the area. The beach profiles are represented in red and the sea bottom contours in blue.

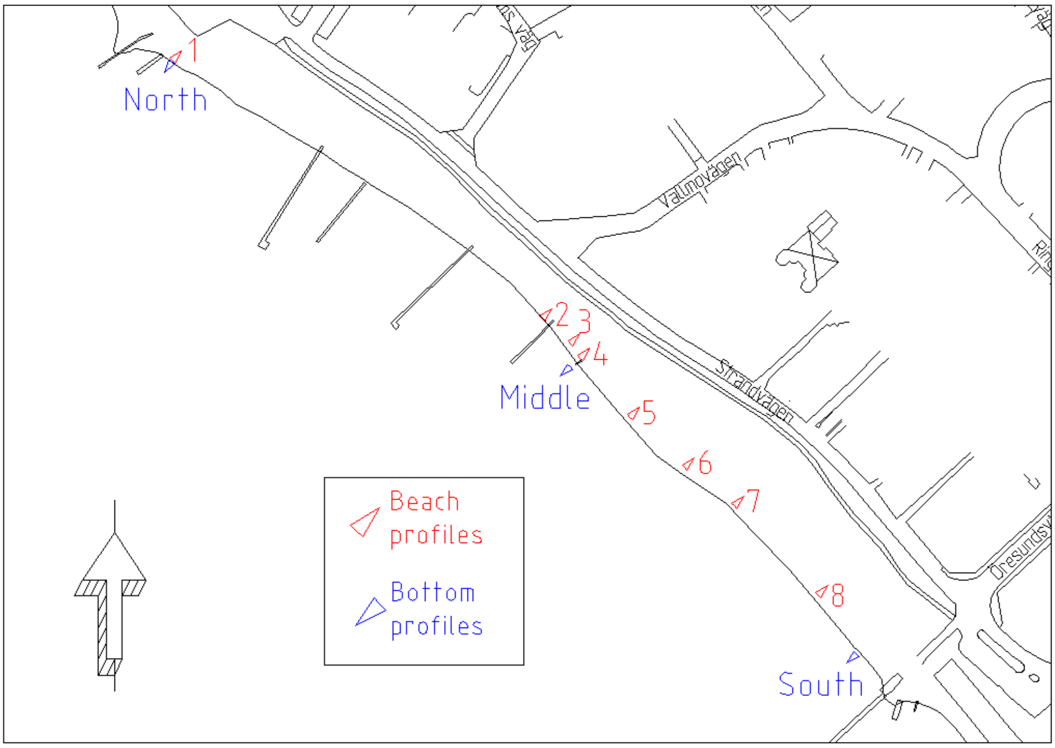

Figure 5.2 Map showing locations of the measured profiles

5.3.1 Beach profiles

Figure 5.3 shows the beach profiles obtained during the survey. As can be seen from the plot all the profiles have slightly different foreshore slopes and the cliff starts at different distances from the water line. The cliff foot, or the height of the berm, is given regardless of the distance from the water line.

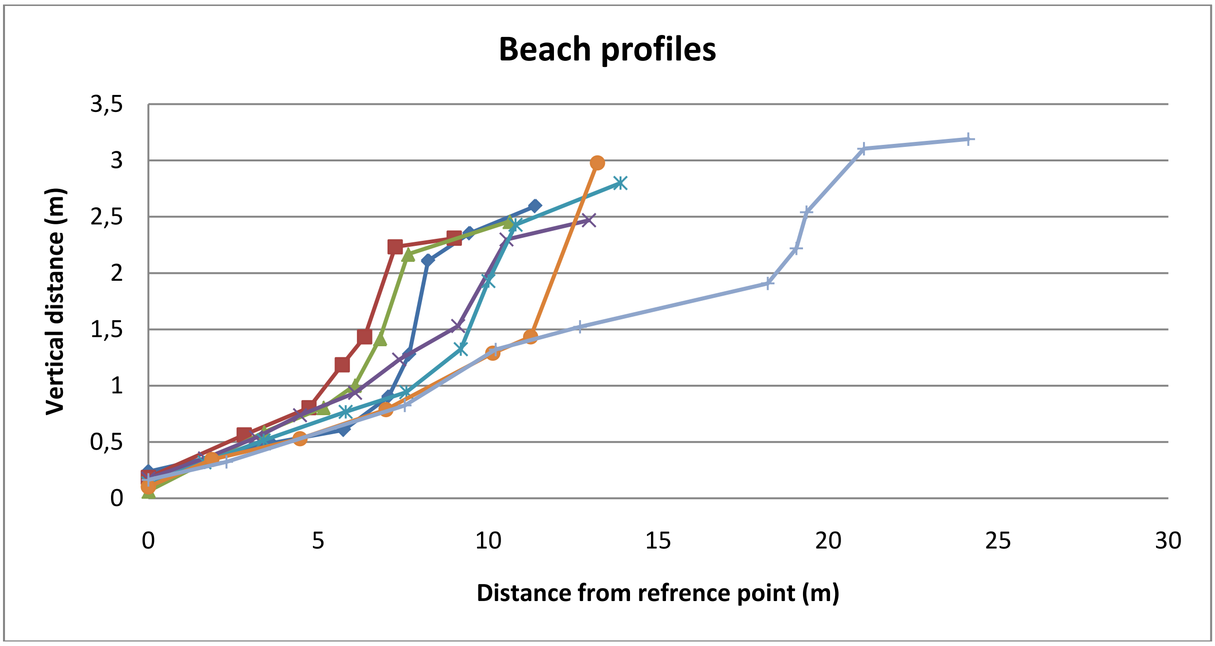

Figure 5.3 Beach profiles of the area relative to the MWL

Due to the fact that concrete slabs have been constructed at the site since 2006, the natural beach profile has changed in the areas where they have been placed. Accordingly, those profiles are not used when calculating the average slope and berm height but can be seen in Appendix II.

To calculate the wave runup height, which is performed in Chapter 8, the **average slope of the berm** (foreshore) is needed as a parameter. Another important parameter needed is the **height of the beach berm**, or the level of the cliff foot. When the water reaches this level it will start eroding the cliff and by that creating an erosion escarpment, as explained in Chapter 2. In order to investigate the overtopping the average cliff height is analysed.

Table 5.1 below presents the average results for all parameters after analysing the survey results:

Table 5.1 Parameters obtained from surveyed beach profiles in Figure 5.3.

Average berm slope	0,134 m/m
Average berm height, z_0	1,4 m
Average cliff height	2,8 m

The average berm slope obtained (0,134 m/m) is slightly lower compared to the 0,158 m/m from (Brännlund & Svensson, 2005). Concerning the average berm height, their attained value was 1,3 m, whereas in this survey reaches 1,4m. The most likely explanation is the human factor while performing the measurements and the natural variability in the profile shape.

As mentioned, due to construction of the revetments and other factors, evaluation of all profiles obtained from (Brännlund & Svensson, 2005) is not possible. Two of the profiles obtained in 2005, however, were still unprotected during this survey and a comparison could be performed. Figure 5.4 and Figure 5.5 below show these two profiles (4 and 7) and their development from 2005 to 2010.

For beach profile 7, the beach profile shape is preserved but has retreated some distance from the water line. The cliff foot is kept at the same elevation but has migrated approximately 1,2 m inland between 2005 and 2010. This is clear evidence that erosion has occurred in this area. This total erosion corresponds well to the erosion, for the years 2002-2010, obtained in chapter 4.

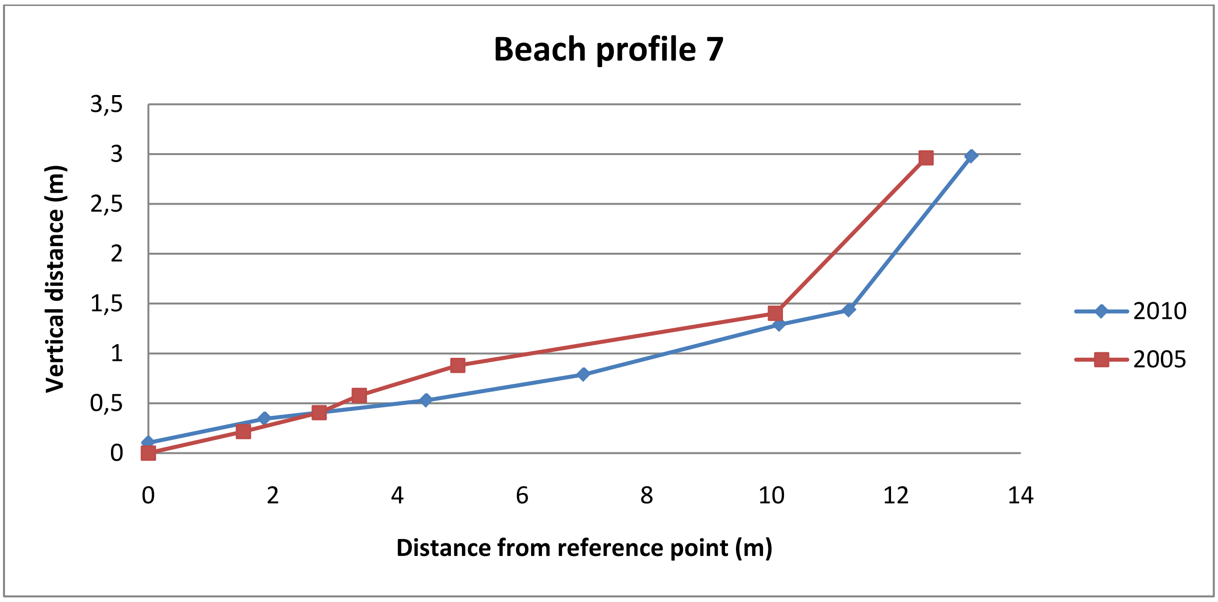

Figure 5.4 Beach profile 7, in 2005 and 2010

The rate of the erosion extracted from this profile is about 0.25 m/year, which is a high value compared to the average erosion of 0.11 m/year during 1963-2002, obtained from the aerial photograph analysis (see chapter 4). During the winter storms of 2006-2007 and 2008 heavy erosion occurred in the area, which is assumed to be the main reason for the high values for this short time period. (Ramböll, Erosionsskydd i Bjärred- underlag för samråd med länsstyrelsen, 2009). The accuracy of the aerial photos analysis can be also another reason to the difference.

Beach profile 4 below has changed shape and the cliff has migrated 1.9 m towards the sea. There is no easy explanation for this result. Further investigations should be performed concerning the local conditions at the site and its historical evolution. One factor could be that beach material has been artificially added in this stretch, thus building up the cliff which may look like accumulation. The human factor during the survey is also a source of error during these kinds of measurements and might be an explanation for the unexpected result.

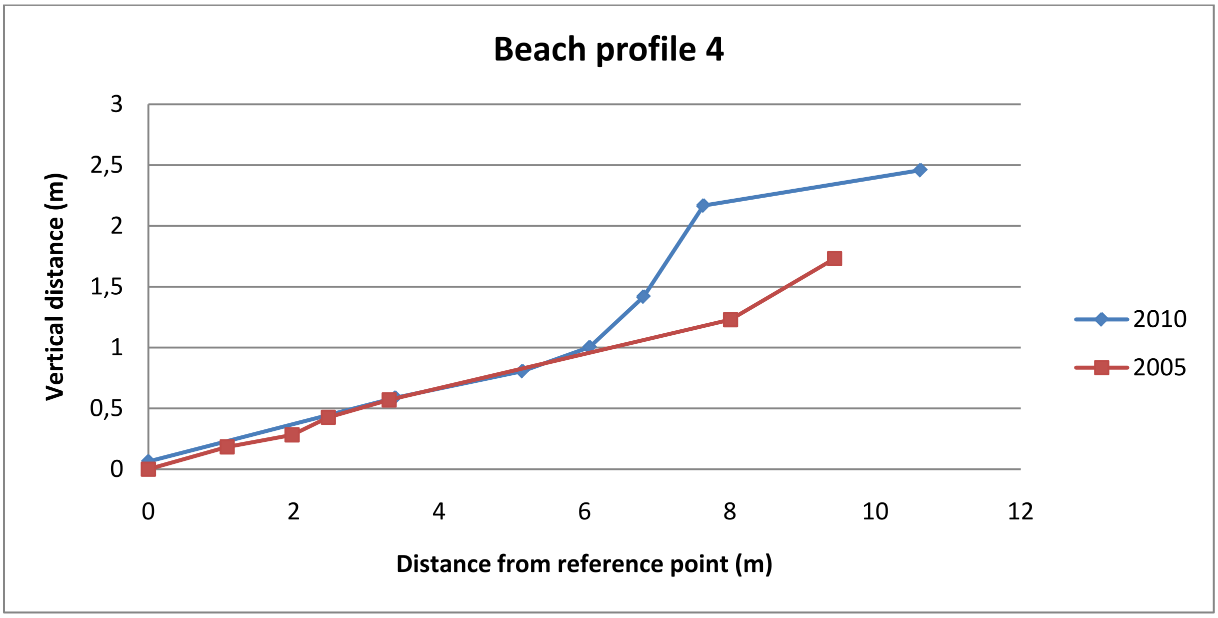

Figure 5.5 Beach profile 4, in 2005 and 2010

5.3.2 Sea bottom profiles

The sea bottom profiles obtained from the surveys can be seen in Figure 5.6 below together with a theoretical expression valid for sandy bottom profiles.

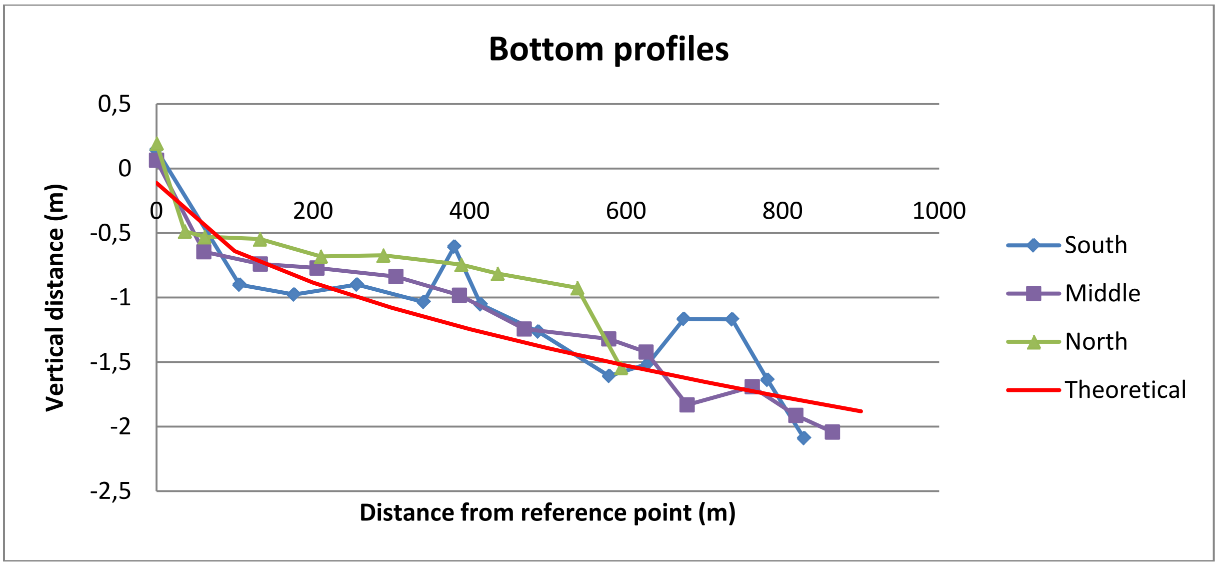

Figure 5.6 Bottom profiles in the area relative to the MWL

In order to analyse the measured bottom profiles a comparison to a theoretical expression for bottom profiles is performed.

The general theoretical expression for a sandy sea bottom profiles is given by Equation 5.1.

$$h = A \cdot y^n \qquad \text{Equation 5.1}$$

where

h = depth at a certain distance from shore
A = equilibrium profile shape parameter. Dependent on grain size, *d*

(CEM, Cross-shore Sediment Transport Processes, 2002b)

The exponent *n* is claimed by (Dean R. G., 1977) to have a value of 2/3 as an overall average. This value might change from beach to beach but 2/3 gives the best overall fit. This expression has one weakness though, which is that it cannot display longshore bars.

A concern when applying Equation 5.1 in Bjärred is that the area consists of a non-homogenous material with quite large differences in grain size. This makes it difficult to assume a mean grain size for the theoretical expression. By fitting the shape parameters to the bottom profile data obtained from the survey a theoretical value for the study area can be found. As can be seen in Figure 5.6 above the fitted line corresponds rather well to the surveyed profiles. The parameter values that give Equation 5.1 the best possible fit are *d*=0.05mm and *y*=11/20.

Using all the measurement points in the area, including both bottom and beach profiles, a contour map of the area can be constructed using the software Surfer and the result is shown in Figure 5.7 below.

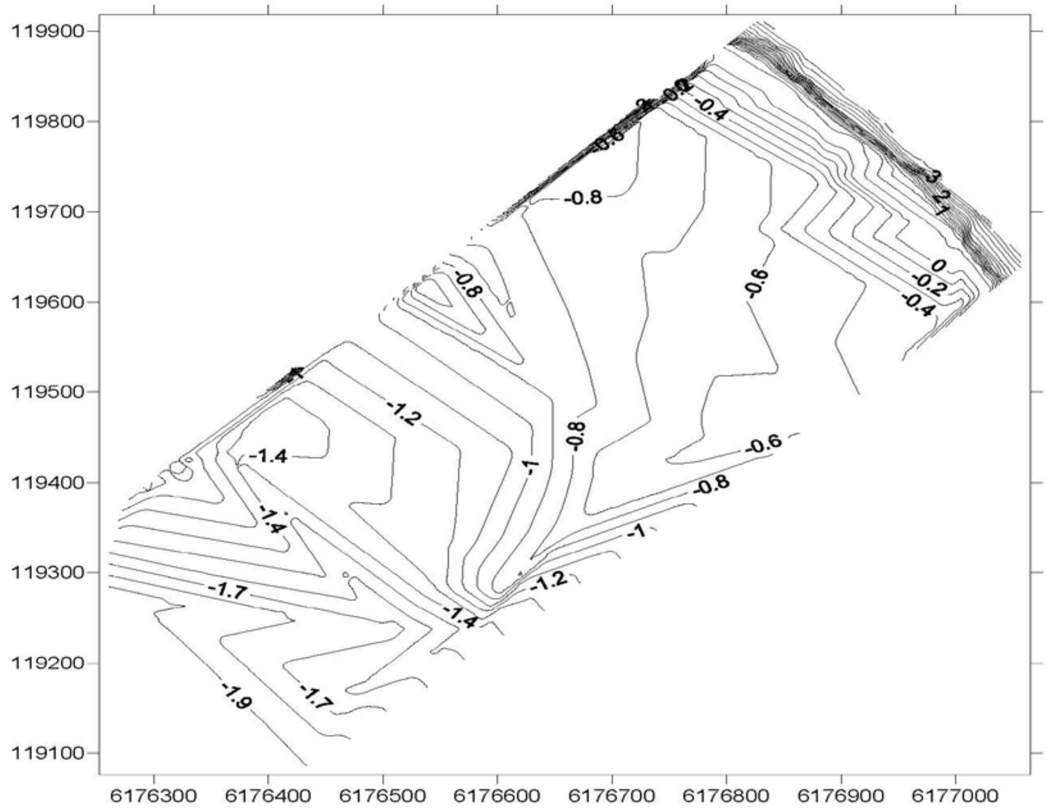

Figure 5.7 Contour map of the area using triangular linear interpolation

6. Wind

6.1 General

Since the wind is the most common force generating waves on the ocean, it is important to evaluate its characteristics in the study area in order to develop accurate wave predictions.

The station closest to Bjärred recording wind data is located in Falsterbo, approximately 40 km southward, at the south-west tip of Skåne. As the wind climate in southern Skåne is relatively homogeneous, and the measurements in Falsterbo are considered to be representative for the study area. (Hanson & Larson, Sandtransport och kustutvecklingen vid Skanör/Falsterbo, 1993)

Available wind data cover the period from 1961 until 2008 with a resolution of 3hr but in the analysis only data from 1976 to 2008 will be used. The reason is that there are no representative water level measurements available before this year. Thus, as it is of interest to study the waves created by winds in combination with water level records, it was not considered relevant to study wind characteristics before 1976. Wind speed is provided with a resolution of 1 m/s and wind direction with a 10° resolution, where 0° /360° correspond to the true north direction. (SMHI, Vinddata, Falsterbo, 2010)

6.2 Analysis

A first step is to analyse the relationship between wind direction and speed, as it will be important to forecast wave generation at a later stage. The data available from SMHI provides 36 different directions (360°/10°=36) but in order to facilitate all this information graphically, the study considers 16 main directions for wind: N, NNW, NW, WSW, SW, SSW, S, SSE and so on. Hence, each direction encompasses 22.5° instead.

A wind rose, see Figure 6.1 below, shows the wind speed (m/s) and its frequency of occurrence during 1976-2008, for the 16 considered directions. As shown in the figure, the predominant wind directions are from west and southwest, with a fairly uniform distribution of frequency for strong winds. Along the west coast of Skåne, winds between South, West and North direction predominate (Hanson & Larson, Implications of extreme waves and water levels in southern Baltic Sea, 2008), which supports the presented results.

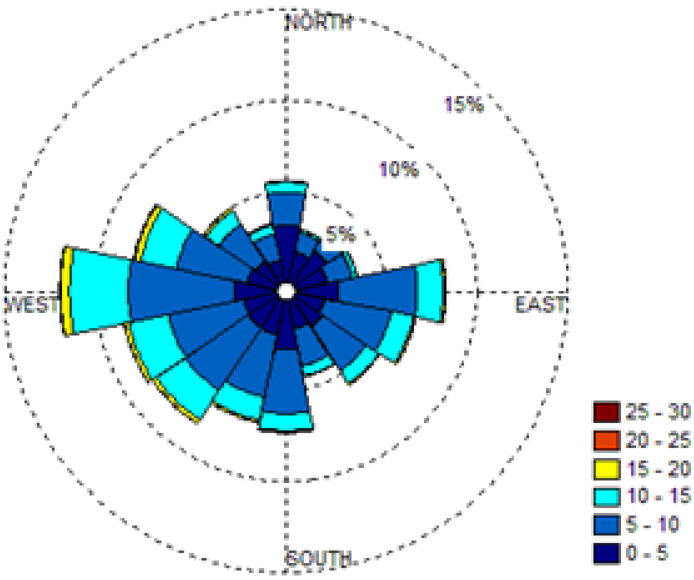

Figure 6.1 Relative frequency of wind speed and respective direction, Falsterbo 1976-2008 (Processed data from SMHI, 2010)

A frequency diagram, see Figure 6.2, provides a clearer picture of the different wind speeds in the area as well as the highest values measured, independent of the wind direction. It is observed that the most common wind speeds are between 4 m/s and 8 m/s and maximum observed speed is 27 m/s.

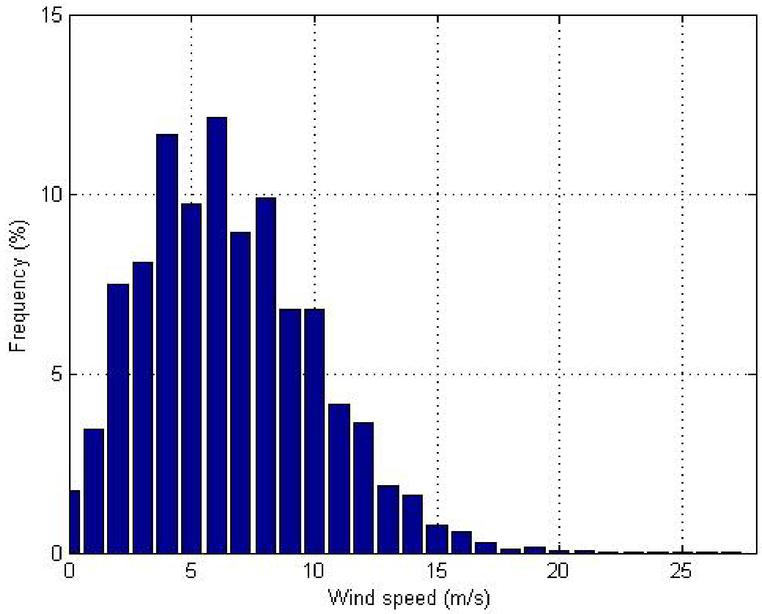

Figure 6.2 Frequency diagram of wind speed, Falsterbo 1976-2008
(Processed data from SMHI, 2010)

Figure 6.3 shows an accumulated frequency graph for these winds and provides information of the probability for a specific wind speed. It can be seen, for instance, that the probability that the intensity is not higher than 9 m/s is 80%.

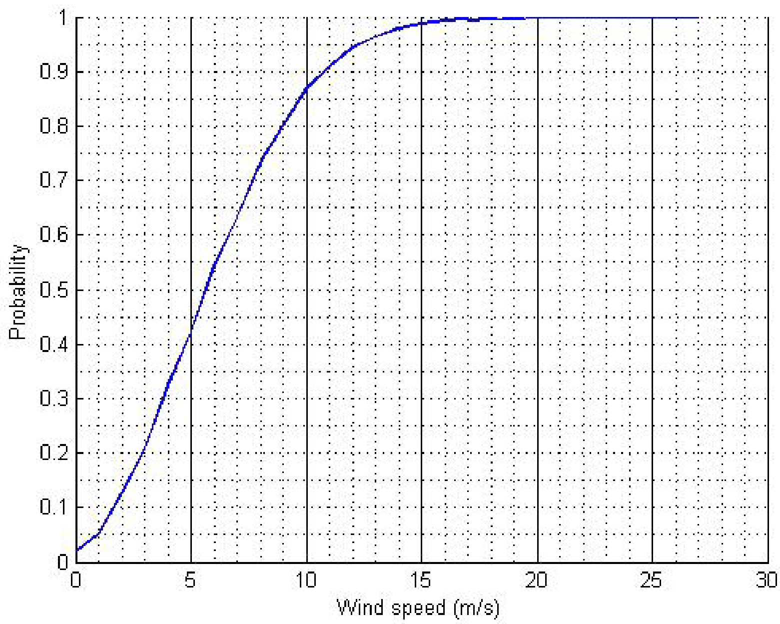

Figure 6.3 Cumulative distribution function of wind, 1976-2008
(Processed data from SMHI, 2010)

Concerning the wind directions that actually generate waves in Bjärred, there are only six bearings that are considered: NW, WNW, W, WSW, SW, and SSW. This is due to the geometry of Lomma bay, which is shown in Figure 6.4. When the wind comes from any other direction there will be no significant waves generated, since wind from land does not create waves eroding the beach.

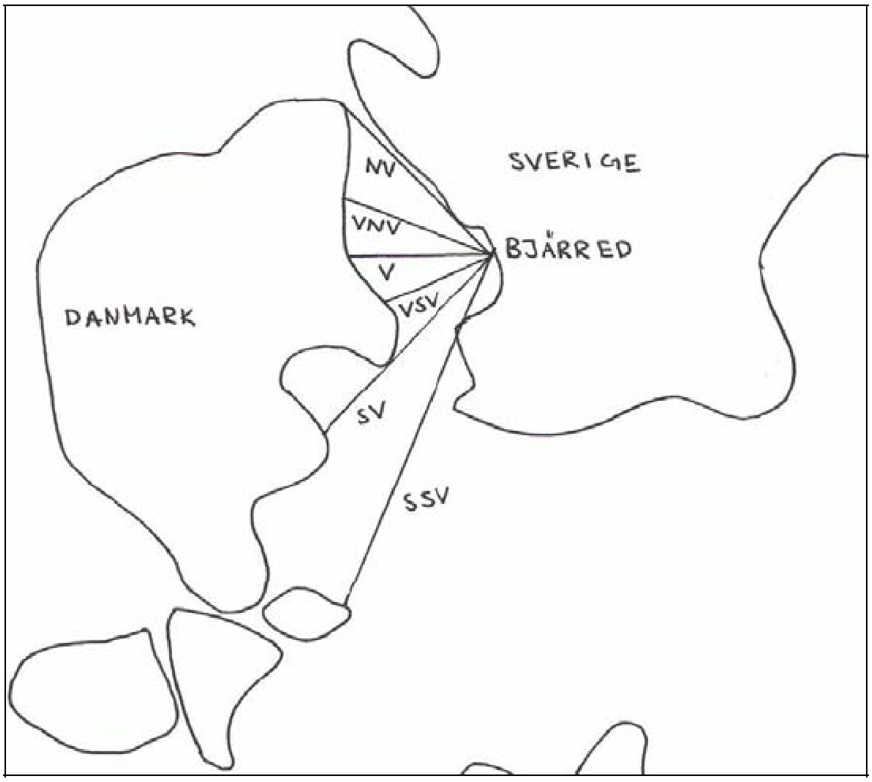

Figure 6.4 Wind directions affecting Lomma bay
(Brännlund & Svensson, 2005)

In chapter 7, the wind data analysed will be used to hindcast waves for this period of time.

7. Waves

7.1 General

Waves are the major cause of coastal erosion, contributing to both longshore and cross-shore transport of sediments. The wave height, period, and angle of incidence determine to a great extent the longshore sediment transport rate. The wave runup, which also depends on wave height and period, can lead to dune erosion, causing cross-shore transport of sand. This chapter is divided into sections discussing generation, hindcasting, transformation, and analysis of data.

7.2 Generation

Waves can be created in different ways, of which wind is the most common generating mechanism. The factors affecting the size and characteristics of the waves are wind speed, duration, and fetch length. A wave is generally generated at the open sea and then propagated towards land. This travel changes the properties of the waves, where they slow down the closer they are to the shore due to phenomena such as shoaling and refraction. In the open ocean, the waves are complex and not very easy to describe. There is a mix of waves with different height, period, and length. When the waves leave this area and travel across the ocean they tend to sort themselves out by period and become more regular. Some of the waves might have had sharp peaks, but during the propagation these peaks are substituted by smoother wave crests, and these waves are called *swell* (Komar P. , 1998). If the waves, instead, continue to grow because the wind blows towards the shore the waves will reach the shore with almost the same shape as when they were created. These kinds of waves are called *seas* (USACE, 1984). Apart from wind-generated waves there are *tsunamis, seiches* and *storm surges*, but these will not be discussed further in this report.

7.3 Wave hindcasting

7.3.1 Wind data transformation

The first step to estimate wave generation is to correct the measured wind speed (U_{meas}) using various correction factors. In this case, the measured wind speed is from the Falsterbo station (USACE, 1984).

$$U = U_{meas} R_Z R_D R_T R_L \qquad \text{Equation 7.1}$$

where

<u>Elevation (R_z)</u>: Wind speed must be adjusted as if it was measured at a 10 m elevation. However, since this is already the case for Falsterbo station, this correction is not needed (R_z = 1).

<u>Duration-averaged wind speed (R_D)</u>: it is assumed that during the 3hr between every measurement of the wind speed, the speed is homogenous for the whole time period, and for this reason no adjustment is needed (R_D = 1).

<u>Stability correction</u> (R_T): This adjustment is necessary when there is a difference between the air and sea temperature, as it leads to boundary layer instability. If no temperature records are available, the correction factor is assumed to be R_T = 1.1.

<u>Location effects</u> (R_L): The station site is adjacent to the shoreline, so the wind data is assumed to be the same as taken over water, and therefore no correction is needed (R_L = 1).

The second step, when the measurements have been corrected, is to convert the wind speed (U) into a wind-stress factor (U_A) since the wave forecasting equations use U_A as the wind-related input parameter.

$$U_A = 0{,}71 \cdot U^{1{,}23}$$ Equation 7.2

7.3.2 Wave hindcasting in deep water

To estimate the wave growth it is necessary to determine under which conditions the waves are generated. If winds blow for a certain time with certain intensity there are two factors limiting wave generation. These are the fetch length and the duration of the wind blowing.

One possibility is that the time it takes for one wave to propagate from one end of the fetch to the other is shorter than 3 hr. In this case, the wave height is **fetch limited**, since the waves do not require the entire duration period (3hr) to reach the far end of the fetch and is therefore affected by the wind for a shorter time period than 3 hr.

On the contrary, the time it takes for a wave to travel along the whole fetch length can be also longer than 3 hr. Then, the wave height is said to be **duration limited**, since the whole fetch length does not contribute to the generation, because it takes longer time than 3 hr for the waves to travel from one end to the other.

Note that 3hr is used in this case as this is the assumed duration of blowing winds. The reason is that 3hr corresponds to the wind data resolution available (see Chapter 6).

As mentioned in chapter 6, for Bjärred, the incoming waves are considered to be generated by winds coming from six different directions: NW, WNW, W, WSW, SW, and SSW. The corresponding fetch lengths and mean water depths are shown in Table 7.1 below. (Brännlund & Svensson, 2005)

Table 7.1 Direction, fetch length, and depth of Lomma bay (Brännlund & Svensson, 2005)

Wind direction	Fetch length (km)	Mean depth (m)
SSW	65	9
SW	27	9
WSW	8	8
W	13	9
WNW	28	10
NW	40	13

In order to determine the condition under which a wave is generated, the procedure is as follows.

Initially, the time that it takes for a wave to be fully developed is calculated. The data needed is the wind-stress factor (U_A) and the fetch length (F), associated with the wave direction.

$$t = 32.15\left(\frac{F^2}{U_A}\right)^{1/3}$$ Equation 7.3

If this time is less than 3hr, the generated wave has reached equilibrium before arriving at the end of the fetch. In that case the condition is fetch limited. The spectral wave height (H_{m0}) and spectral period (T_m) are predicted by using the following wave growth formulas.

$$H_{m0} = 5{,}112 \cdot 10^{-4} U_A F^{1/2}$$ Equation 7.4

$$T_m = 6{,}238 \cdot 10^{-2} (U_A F)^{1/3}$$ Equation 7.5

If the time to reach a fully developed wave is greater than 3hr, the condition is duration limited. A new fictive fetch (F') for t equal to 3hr must be calculated according to Equation 7.6.

$$F' = \sqrt{U_A}\left(\frac{t}{32{,}15}\right)^{3/2}$$ Equation 7.6

Wave height (H_{m0}) and period (T_m) are calculated with Equation 7.4 and Equation 7.5 using the fictional fetch, *F'* instead of the real fetch *F*.

7.3.3 Wave hindcasting in shallow water

Waves can also be generated under shallow-water conditions, where bottom friction is believed to influence the wave growth. For a given wind speed and fetch condition, the generated wave height and period in shallow and transitional waters will be smaller compared to the waves in deep water. This is due to the loss of energy caused by bottom friction and percolation. (Komar P. , 1998)

Lomma bay is assumed to be a shallow water body (Larson, 2010). As Equation 7.4 and Equation 7.5 are valid only for deep water, empirical equations to forecast wave height and period in shallow water conditions are used instead (USACE, 1984).

The wave height (*H*) is expressed by Equation 7.7.

$$\frac{gH}{U_A^2} = 0{,}283\tanh(K_1)\tanh(\frac{K_2}{K_3})$$ Equation 7.7

where

$$K_1 = 0{,}53\left(\frac{gd}{U_A^2}\right)^{3/4}$$ Equation 7.8

$$K_2 = 0{,}00565\left(\frac{gF}{U_A^2}\right)^{1/2}$$ Equation 7.9

$$K_3 = \tanh(K_1)$$ Equation 7.10

And wave period (*T*) is given by Equation 7.11.

$$\frac{gT}{\mathrm{U_A}} = 7{,}54\tanh(K_4)\tanh(\frac{K_5}{K_6})$$ Equation 7.11

where

$$K_4 = 0{,}833\left(\frac{gd}{U_A^2}\right)^{3/8}$$ Equation 7.12

$$K_5 = 0{,}0379\left(\frac{gF}{U_A^2}\right)^{1/3}$$ Equation 7.13

$$K_6 = \tanh(K_4)$$ Equation 7.14

The fetch used in the equations above is either *F* or *F'*, depending on the generation conditions discussed in section 7.3.2.

Nevertheless, to achieve an accurate prediction, the wave height forecasted for shallow water (*H*) is checked to never exceed the spectral wave height (H_{m0}):

$$H < H_{m0}$$ Equation 7.15

In case it exceeds for some waves, these will be considered to be originated in deep water. Accordingly, their wave height and period are set to H_{m0} and T_m, respectively.

Finally, calculation of the wavelength is performed (Equation 7.16). It is assumed that the wavelength will approximately attain its deep-water value regardless of the water condition of the wave generation.

$$L_0 = 1{,}56 \cdot T^2 \qquad \text{Equation 7.16}$$

7.4 Wave transformation

When approaching the shore, the forecasted waves will experience several transformation processes such as refraction, shoaling, diffraction, and breaking. For analysis purposes, these changes will be taken into account when simulating the shoreline response in Chapter 12, and the computer model GENESIS will perform all these transformations through its internal calculations (see Chapter 11).

7.4.1 Refraction, shoaling and diffraction

The direction and size of the wave hitting the shoreline is determined by the processes of refraction, shoaling, and diffraction. Refraction occurs when a wave passes from a deeper area to a more shallow area at a specific angle making the crests turn towards the shoreline.

The **refraction** coefficient, K_R, is a function of the initial angle (θ_1) of the wave ray and the angle of arrival (θ_2) to the breaking depth area. Equation 7.17 presents its mathematical expression and Figure 7.1 a sketch of the phenomenon behaviour.

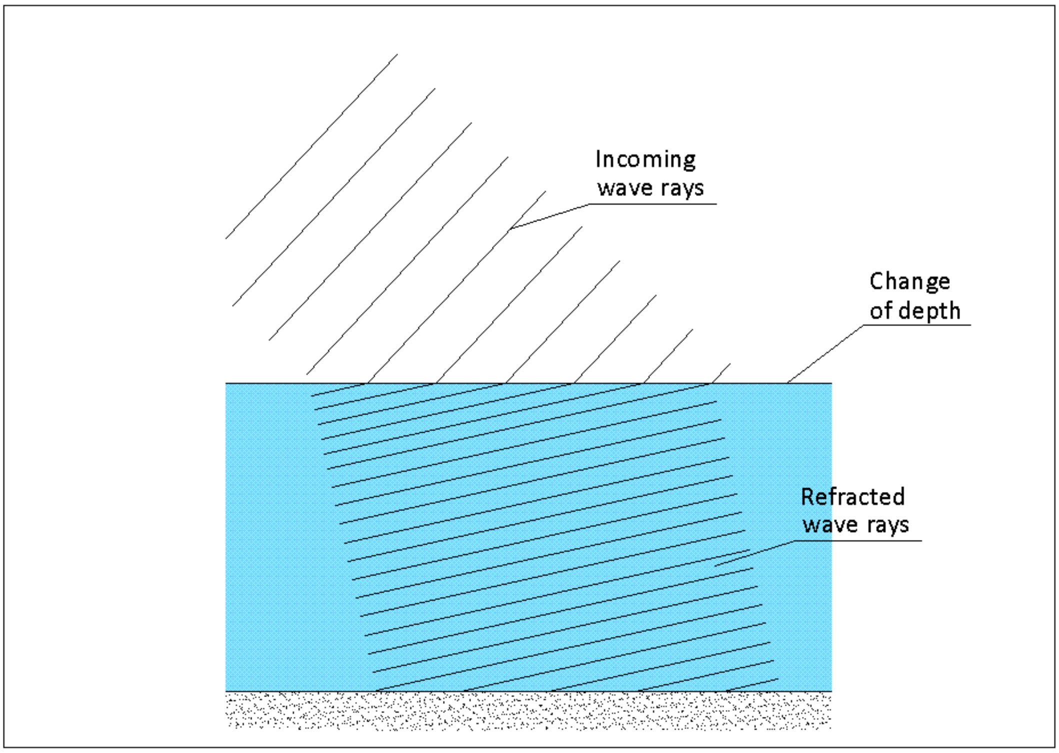

Figure 7.1 Refraction phenomenon

$$K_R = \left(\frac{\cos\theta_1}{\cos\theta_2}\right)^{1/2} \qquad \text{Equation 7.17}$$

Shoaling is the effect by which waves increase in height due to a decrease in water depth (causing a decrease in energy propagation speed), as the energy flux must be conserved. At the same time, celerity is being lowered and varies along the crest of a wave moving at an angle to underwater contours. This is because the part of the wave in greater depths is moving faster than the part in shallow water. Shoaling, in combination with refraction, determine the wave height at any water depth for a given deep water wave. (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change, 1989)

Shoaling phenomenon is shown in Figure 7.2. K_S is the **shoaling** coefficient, which depends on the wave period and initial depth as well as the breaking depth, and it is expressed by Equation 7.18.

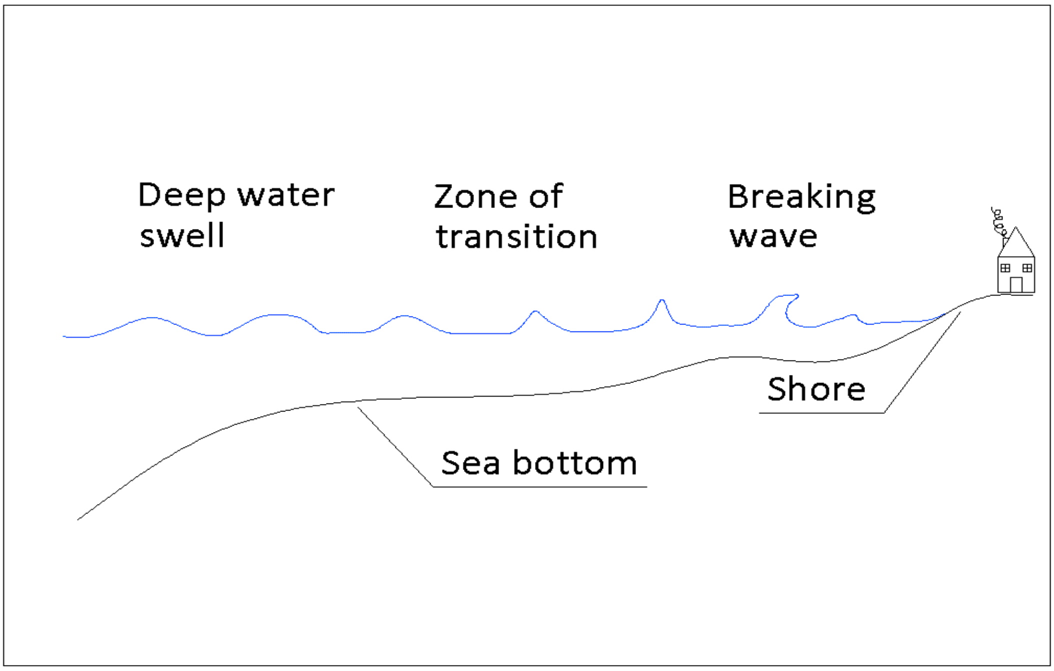

Figure 7.2 Shoaling phenomenon

$$K_S = \left(\frac{C_{g1}}{C_{g2}}\right)^{1/2} \qquad \text{Equation 7.18}$$

where

C_{g1} and C_{g2} are the wave group velocities at the initial condition and at the break point, respectively. The relationship between wave speed (C) and group speed (C_g) is given by Equation 7.19.

$$C_g = nC = n\frac{L}{T} \qquad \text{Equation 7.19}$$

$$n = 0{,}5\left(1 + \frac{(\frac{2\pi d}{L})}{\sinh\frac{2\pi d}{L}}\right) \qquad \text{Equation 7.20}$$

where

L = is the wavelength at a depth d
T = the wave period.

(Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change, 1989)

Diffraction is the phenomenon by which wave energy is transferred laterally along a wave crest. When a group of waves encounters, for example, a structure this causes the waves to turn and move in a circular motion around the obstacle (Figure 7.3).

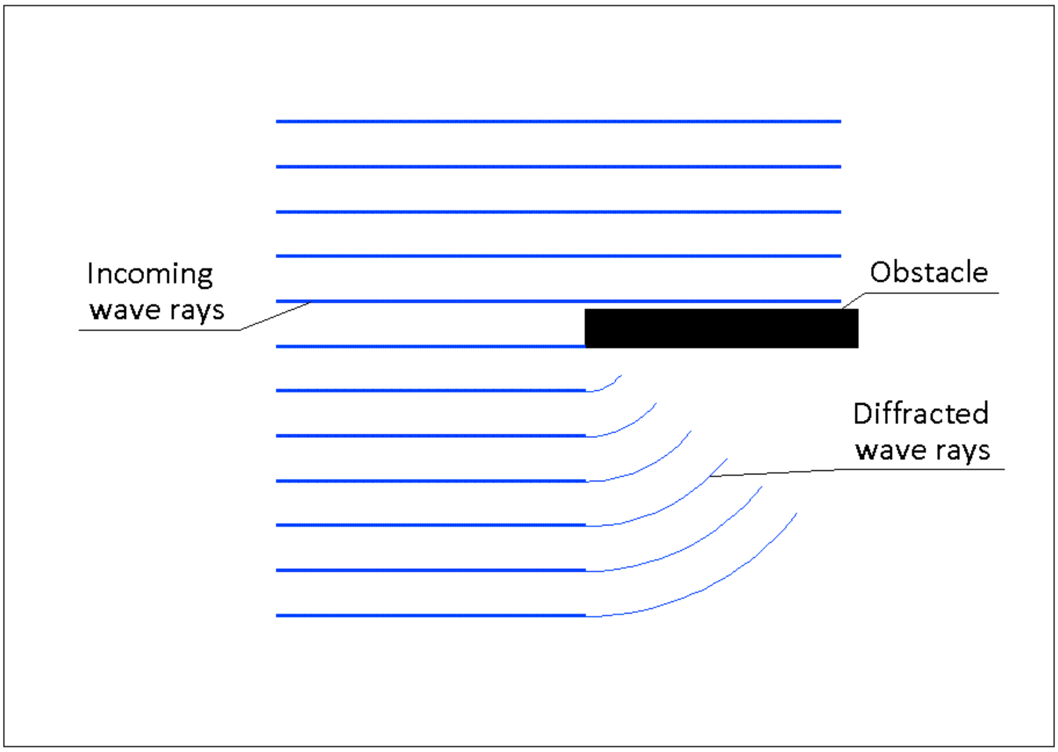

Figure 7.3 Diffraction phenomenon

7.4.2 Breaking waves

The wave form can remain stable as long as it does not reach a limiting steepness. When this is achieved waves break and dissipate energy, which may lead to transport of sediments. In shoaling water this limit in steepness is function of the relative depth (d/L) and the beach slope (m).

To obtain the height of breaking waves that have experienced shoaling and refraction Equation 7.21 below is used.

$$H_b = K_R K_S H_1$$ Equation 7.21

H_b stands for the breaking wave height, and H_1 is the height of waves generated at the reference conditions in Lomma bay (from wave generation calculations).

The depth at which waves break (d_b) is obtained by:

$$d_b = \frac{H_b}{\gamma}$$ Equation 7.22

in which γ depends on the deepwater wave steepness (H_0/L_0) and the average beach slope (Smith & Kraus, 1990).

$$\gamma = b - a\frac{H_0}{L_0}$$ Equation 7.23

$$a = 5\left[\left(1 - e^{-43tan\beta}\right)\right]$$ Equation 7.24

$$b = \frac{1{,}12}{\left(1+e^{-60tan\beta}\right)}$$ Equation 7.25

The angle at which waves break (θ_b) is given by Snell's law:

$$\frac{\sin \theta_b}{L_b} = \frac{\sin \theta_1}{L_1}$$ Equation 7.26

where

θ_b and L_b are the angle and wavelength at the break point, whereas θ_1 and L_1 refer to the reference wave conditions.

By running iterative solutions of Equation 7.21, Equation 7.22, and Equation 7.26, the three unknown parameters at the break point (H_b,θ_b and d_b) can be determined as a function of H_1, θ_1 and T_1. (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change, 1989)

7.5 Analysis

The method presented to forecast waves in the shallow-water conditions of Lomma bay in section 7.3 is used to obtain the generated wave heights. Figure 7.4 presents the frequency of occurrence of each height (with 0.1 m resolution) between 1976 and 2008.

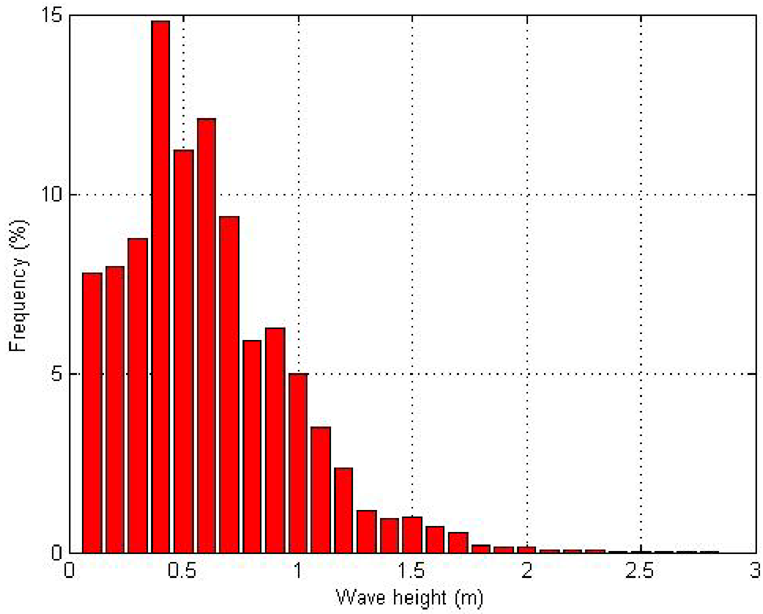

Figure 7.4 Frequency diagram of predicted wave height, 1976-2008 (Processed data from SMHI, 2010)

Note that this graph does not contain waves with a wave height lower than 0.1 m. In fact, though, they represent around 60% of the total number of waves. In order for this graph (without waves below 0.1 m) to better show the wave height distribution, waves below 0.1 m are left out.

An accumulated frequency graph for the generated waves (Figure 7.5) provides the probability that a specific wave height is not exceeded. The figure is generated by using an empirical distribution function (CDF), which associates a cumulative distribution with the empirical obtained values. This is why two different lines are observed on the plot. It can be seen, for instance, that 50% of the waves are below 0.5 m. Note that the waves with height equalling to zero are not included. Including them would *push* the graph upwards, making the probability of waves below 0.5 m higher.

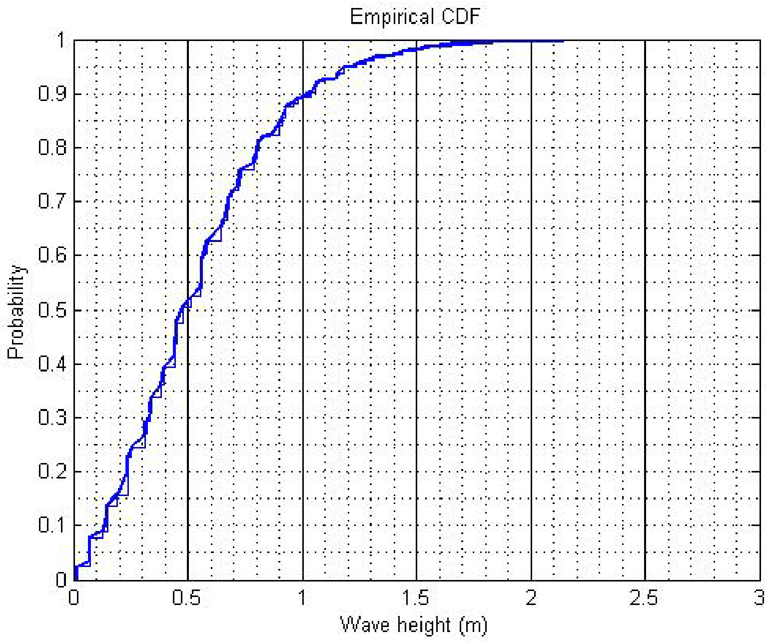

Figure 7.5 Cumulative distribution function of wave height, 1976-2008 (Processed data from SMHI, 2010)

8. Water level and wave runup

8.1 General

8.1.1 Water level

The water level is constantly changing at the coast and the changes can be divided into short-term and long-term changes. The short-term changes have more impact on coastal erosion due to their greater year-to-year influence compared to the long-term changes (Komar & Einfield, 1987). Among the short-term changes seasonal variations play a major role, and they are mainly caused by differences in atmospheric pressure, tides, movement of ocean currents, runoff, melting ice etc. The definition of short-term is a time interval during which the changes in sea level can visually be observed or measured (normally in the range of 25 years). (CEM, Coastal Terminology and Geologic Environments, 2002d)

The long-term changes in sea level can either be eustatic or relative, of which eustatic is caused by variations in the volumes of the world's ocean basins and the total amount of ocean water. This change can be measured using satellite technology, and by monitoring and recording movement in the sea surface elevation compared to some universally adopted reference frame. (CEM, Coastal Terminology and Geologic Environments, 2002d)

The relative sea water level describes the water level relative to land at a specific location. This relationship is very important when it comes to erosion, since high water levels may lead to erosion and damage to properties or infrastructure. Therefore it is important to analyze the historical water level trend in Lomma bay, as well as evaluating the impact of possible future scenarios.

8.1.1.1 Causes of water level fluctuations

Atmospheric pressure variations
An increase in atmospheric pressure will apply a higher pressure on the water surface and lower the water level, and the opposite will occur at lower atmospheric pressures. The average atmospheric pressure is 1013 hPa and an increased atmospheric pressure of 1 hPa will lower the water level around 1 cm. Since the atmospheric pressure during a normal year differs from 950 to 1050 hPa the water level fluctuates between -37 to +63 cm due to atmospheric pressure changes. (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009)

Winds
Larson & Hanson (2008) state that wind is the most important factor when considering water level fluctuations in southern Sweden. The wind causes water level fluctuations in many different ways. When the wind blows from the sea towards land it pushes water to move to the shore, increasing the water level adjacent to land. The opposite happens when the wind blows from land and pushes water out into the sea, lowering the water level at the shore. In enclosed water bodies the relationship between wind and water level may be quite complex. For example, winds that blow from the north with a long duration cause water to flow to the southern part of the Baltic Sea. This causes the water level in south of Sweden to rise. (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009)

Tide
In some regions the tide might have a significant impact on the water level but along the Swedish coast the tide does not have a great influence. The largest change in tide occurs on the very north of the Swedish west coast, where the differences can be up to 60 cm (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009). The tide subsides in Öresund and the Belts, wherefore it is low in the Baltic sea, around 0.15 cm in Bjärred. This is taken into account in the calculations since it is included in the datasets used. (SMHI, Sveriges Meteorologiska och Hydrologiska Institut, 2008)

Land level change
In the north of Sweden there is still a land level rise, originating from the melting of the ice from the last ice age. When the thick ice-layer was present, it pushed the earth surface down and the surface is now rising, which makes the sea level lower at the specific location (Emery & Aubrey, 1991). The land level rise is at most around 1 cm/year in northern Sweden. In the south there is a much lower land raise, around 0.1 cm/year, whereas in Bjärred the rise is close to zero. (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009)

Other factors
Variations of water density and volume, driven by a change in temperature of the oceans upper layer also have influence on short-term variations. When temperature increases during the summer months the density decreases, causing a higher sea level and during winter the opposite occurs. The variations in water temperature are not only caused by seasonal changes due to solar radiation, but are also caused by changes in wind and current patterns. (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009)

8.1.1.2 Consequences of high water levels

When the sea water level rises the beach profile will try to adapt itself to maintain its original shape that it had before the rise. This will cause the profile to recede, and will lead to erosion at the shoreline, as Figure 8.1 shows. In general it is said that for every cm of sea level rise the shoreline will retreat 1 m. This comes from the Bruun rule, which will be adapted here to obtain a simple estimate of the retreat (see chapter 10).

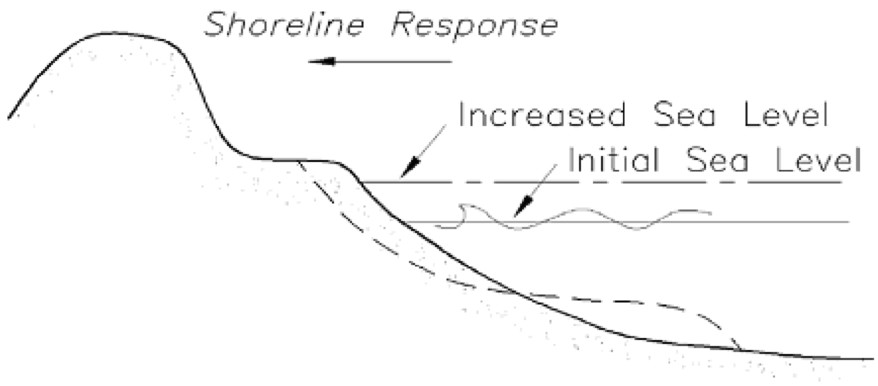

Figure 8.1 Profile response to sea level increase (CEM, Coastal Terminology and Geologic Environments, 2002d)

8.1.2 Wave runup

Runup height is the maximum level, often referred to the still water level, which the water reaches on the foreshore in connection with wave uprush. It includes both wave setup and the runup effects (Hanson & Larson, Implications of extreme waves and water levels in southern Baltic Sea, 2008). Figure 8.2 below illustrates wave runup.

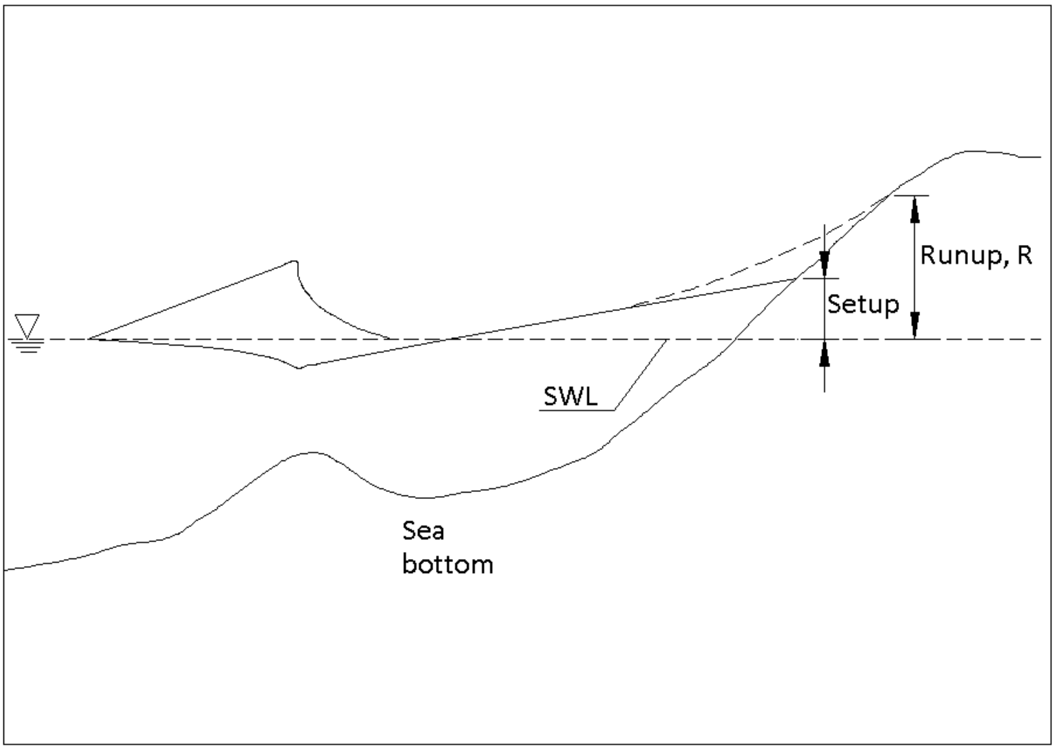

Figure 8.2 Wave runup phenomenon

There is an interest in estimating the runup in the study area because it is the main phenomenon responsible for cross-shore erosion when the waves hit the dune foot. If the runup is high enough, overtopping of the dune might occur, and the waves could endanger structures or other infrastructure behind the dune.

The mean water level in Lomma bay is the reference used for the runup height. Available data for water level exists between 1976 and 2010 and is referred to the yearly mean water level in Viken. Previous studies show that water level in Viken is representative for Bjärred (Brännlund & Svensson, 2005).

Runup depends on the incoming wave height and wave length (that is period), as well as the beach profile and sediment properties (Larson & Hanson, 1992). For this reason, results from the survey of the study area in Bjärred are used to derive a representative slope for the beach berm. The time period when wave runup, *R*, will be studied comprises 1976 to 2008, since it is the longest period where data for both water level and forecasted waves are available.

The method used to calculate runup is based on Hunt’s formula (1959) (see chapter 2), which provides an estimated value for the runup generated by the waves rushing up on the foreshore by taking into account the beach profile slope and the wave characteristics.

$$R = \tan\beta\,\sqrt{H_0 L_0} \qquad \text{Equation 8.1}$$

where

R = vertical runup height
$\tan\beta$ = average slope of the foreshore profile
H_0 = wave height
L_0 = wave length
As developed in chapter 5, the average slope of the beach is estimated to be 0.13 m/m. Wave height and wavelength (H_0, L_0) are the values for waves generated in deep water, and are calculated as explained in chapter 7.

Hunt's method often overestimates the runup height as it does not take into account the angle at which a wave impacts to the shore, α_0. To correct for this, the wave height H_0, is recalculated to an equivalent wave height H'_0 that impacts perpendicular to the shore.

$$H'_0 = H_0(\cos \alpha_0)^{1/2}$$ Equation 8.2

8.2 Analysis

In Bjärred there is no observation station recording water levels. The closest one is located in Barsebäck (around 8 km north), and would be representative, but the data available from SMHI only encompass the period from 1992 to 2004. The station of Viken is located around 50 km north of Bjärred and the water level has been recorded here between 1976 and 2010 every 3hr.

There is also another station closer to Bjärred, located in Klagshamn around 30 km south of the area (see Figure 8.3). Previous studies (Brännlund & Svensson, 2005), (Karlsson & Martinsson, 2010) show that there is a difference in water level between the stations of Klagshamn versus Barsebäck/Viken. It has been shown that the Limhamn threshold is a natural border, where the water levels fluctuate differently north respectively south of the threshold. Klagshamn is located south of this threshold and Barsebäck and Viken north of it.For the time period mentioned above the mean water level in Viken increases with the trend 0.02 cm/year, while the yearly maximum increases with 0.30 cm/year. (SMHI, Vinddata, Falsterbo, 2010)

Figure 8.3 Map over Skåne showing Bjärred and the locations for stations recording the water level

An initial analysis includes how the water level varies with changing wind conditions, as winds with high velocity lead to higher waves when appropriate conditions are met. In Figure 8.4 the water level is plotted against the wind speed.

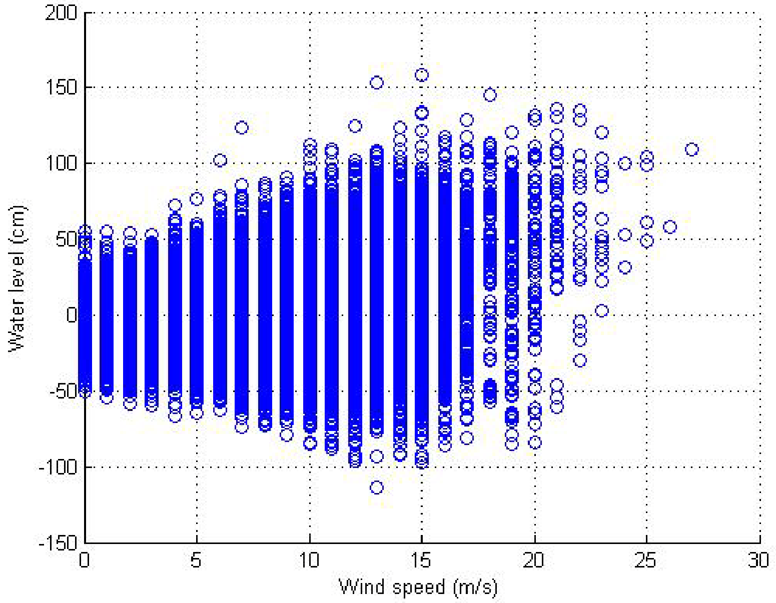

Figure 8.4 Water level relative to wind speed (Processed data from SMHI, 2010)

It is observed that when light winds and breeze blows (0-11 m/s) the water level is confined between a specific range, but as the winds become stronger the range of the water level tends to increase. As this situation does not happen very often, no good trend is achieved, and therefore it is not possible to detect any obvious tendency. To be cautious, it can be affirmed that higher water levels tend to take place when there are strong winds. The fluctuations of the water level during stronger winds are likely to originate from the fact that water is *pushed* towards or off the shore with different wind directions. The large-scale movement of the water in the Baltic Sea is of great significance for the water level at Bjärred, implying a complex relationship between the wind speed and the water level, as indicated by Figure 8.4, where the wind direction becomes important as well. The next plot, Figure 8.5, shows the relationship between water level and wind direction, where 0 and 360 degrees represent the north direction.

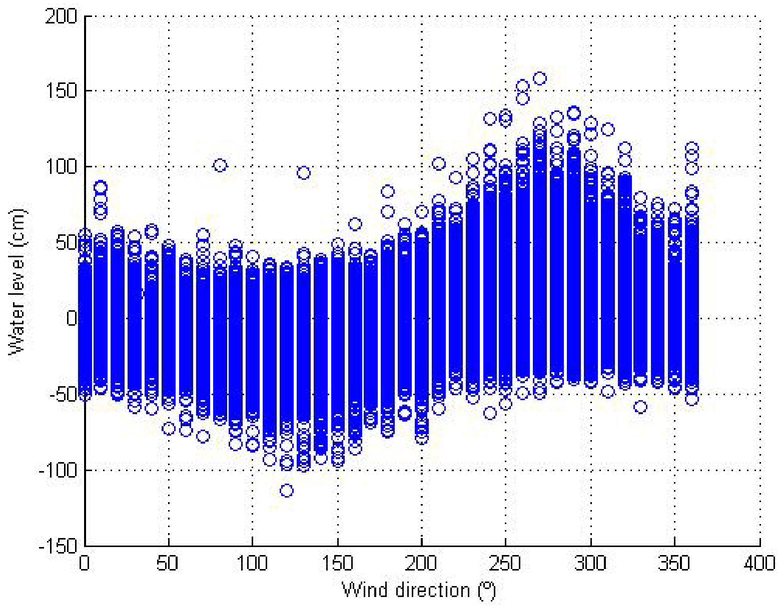

Figure 8.5 Water level relative to wind direction (Processed data from SMHI, 2010)

Winds coming from SW, W and NW (between approximately 230° and 330°) generate the highest water level records. This confirms the theory from section 8.1.1.1.

The frequency analysis for the water level data from Viken between 1976 and 2010 is presented in Figure 8.6 and Figure 8.7. The first figure shows that water level frequency follows approximately a normal distribution. Water levels equal or higher than 50 cm occur with a frequency of 2,1%. The second figure expresses the probability for all the different possible water levels during the study period, generated by using an empirical distribution function (CDF).

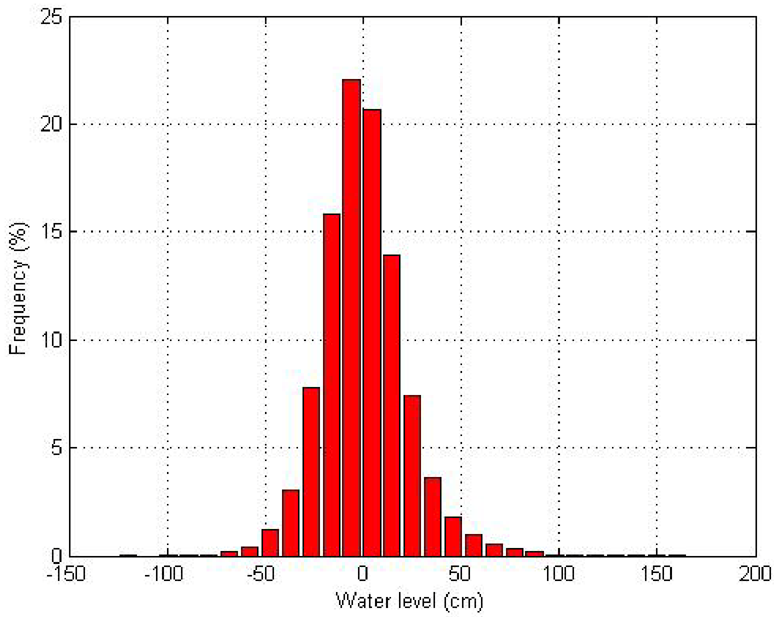

Figure 8.6 Frequency diagram of water level, Viken 1976-2010
(Processed data from SMHI, 2010)

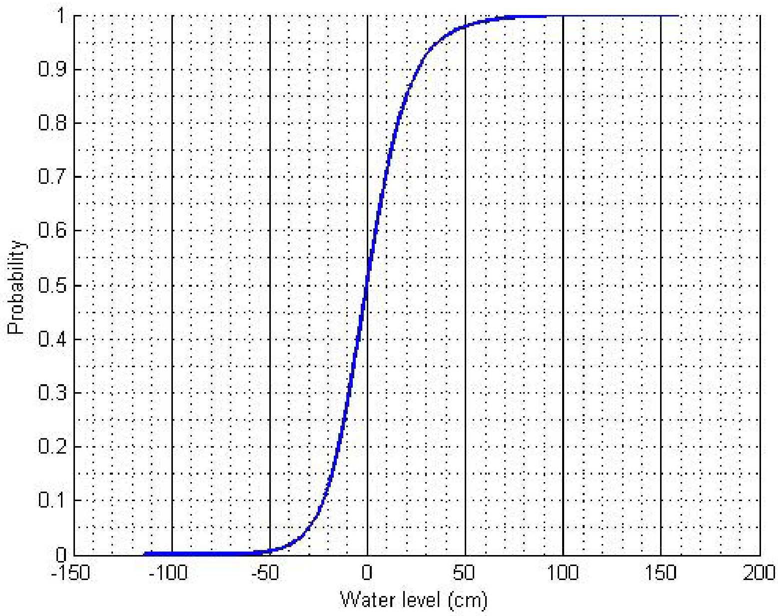

Figure 8.7 Cumulative distribution function of water level, 1976-2008
(Processed data from SMHI, 2010)

As mentioned before, the runup is of great interest in the study area. The height that waves can reach during their uprush is the sum of the water level plus the runup height itself (USACE, 1984). To simplify the nomenclature, the sea water level observed plus the runup calculated for each recorded value will be only called runup level (or, simply, runup). Figure 8.8 shows this distribution of runup in the study area between 1977 and 2008.

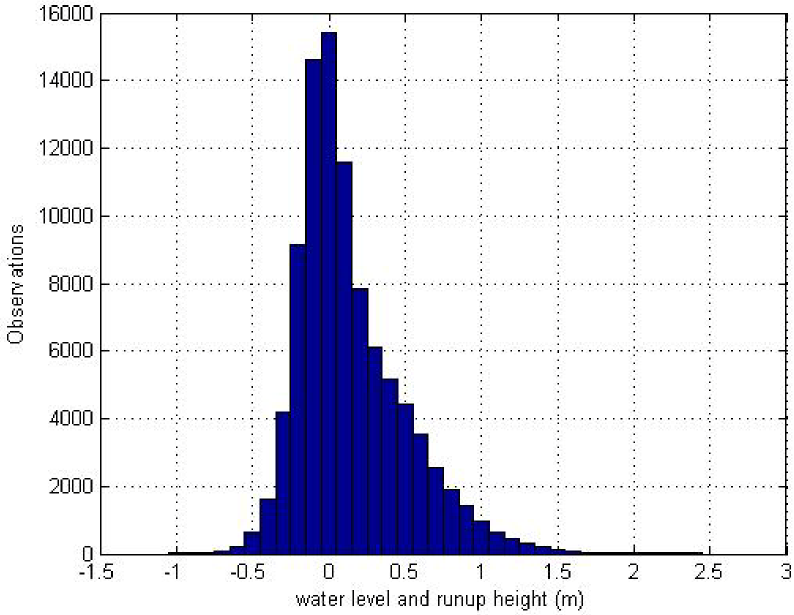

Figure 8.8 Distribution of water level plus wave runup, Viken 1976-2010 (Processed data from SMHI, 2010)

The results show that the mean value for the runup level is 0.14 m and the maximum level reaches to 2.45 m. Observations with a negative combination on the *x*-axis mean occasions when there is a low sea water level and small or no waves present. From these results it is possible to estimate the cross-shore erosion on the beach caused by high runup events. The height of the cliff, given from the survey in chapter 5, is approximately 2.8 m. Since the highest maximum runup level is 2.45 m there is no risk for overtopping with the conditions of today. When discussing climate change, in chapter 9, overtopping is a factor that needs to be taken into account as overtopping would lead to flooding of the adjacent road.

As mentioned in the introduction, the previous study by (Brännlund & Svensson, 2005) showed that erosion takes place when waves hit the cliff foot, which happened in average 3.6 times per year. In this analysis the water level 1.4 m is exceeded a total number of 434 times, which means in average 13.6 times per year.

The main reason to this might be the hindcasting of waves. During this project the hindcasted wave heights are higher compared to the previous study, where the reason might be differences in the calculation procedures.

9. Climate change

9.1 Introduction

The Intergovernmental Panel on Climate Change (IPCC) states that most likely climate change induced by human activity is a fact. (IPCC, 2007)

This predicted climate change will cause responses regarding the behaviour of many mechanisms in nature. For this project it is of special interest to analyse whether the sea level and the winds might change. For instance, an increase in the sea level will induce more dune erosion at a larger number of times, causing increased dune retreat.

Most probable, the Earth will experience a global rise in the sea level. Among the many different causes, the thermal expansion and the melting of glaciers are the two most important factors. During the last years thermal expansion has contributed up to 50% of the water level rise. There are many uncertainties regarding the number of mechanisms that contribute to the rise and predictions should be considered as just predictions and not be taken as definite numbers. Depending on which mechanisms that will prevail and so on there might be large variations when predicting sea water level rise. (Rummukainen & Källén, 2009)

The IPCC gathers the latest discoveries in science regarding climate change, including changes in sea level and winds. The latest report was released in 2007 (Assessment Report 4, AR4) and it presents four different scenarios of the global climate change and discusses figures of sea level rise in the years 2070-2100. The scenarios presented are:

Table 9.1 IPCC scenarios (IPCC, 2007)

IPCC scenario	Predictions
A1FI – *worst case scenario*	Represents a global economic growth. Earth total population of 9 billion people in 2050 and increasing use of fossil fuels.
A1B	Same as A1FI but with balanced use of fossil and non-fossil fuels.
A2	Same as above but with a majority of energy from non-fossil fuels.
B1 – *best scenario*	The B1-scenario represents a more ecological way of living, with a global economic growth but towards more service and information economies rather than industrial ones.

There are many factors deciding which scenario is most likely to happen, such as human attitude to climate change and governmental actions, and therefore it is assumed that no scenario is more likely to happen than any other.

9.2 General

9.2.1 Winds

Focusing on winds, the AR4 presents two different climate projections, each for two possible scenarios. One important factor taken into account is the pressure gradient between northern and southern Europe. If this gradient changes there will be a difference in the wind speed in the future otherwise not; however, there is not enough knowledge to determine if this will happen or not. (IPCC, 2007)

A vulnerability report for Sweden uses a regional atmospheric model called RCA3-E or RCA3-H to predict future winds. Using the A2-scenario from the (IPCC, 2007) the average wind speed will increase **7-13% during winter time**, when considering the RCA3-E model. The RCA3-H model shows no or very little change in wind speed. Stressed once again, since there is still such uncertainty, no assumption can be definite. The vulnerability report also includes investigation of wind squalls. When using RCA3-E they are said to increase around 1-2 m/s for the study area in the year 2050, but this has not been considered in the analysis. (SOU, 2007)

Figure 9.1 below shows the models for change of climate parameters (wind to the right), where the top pictures show an increased gradient from north to south and the lower pictures show no change in the gradient.

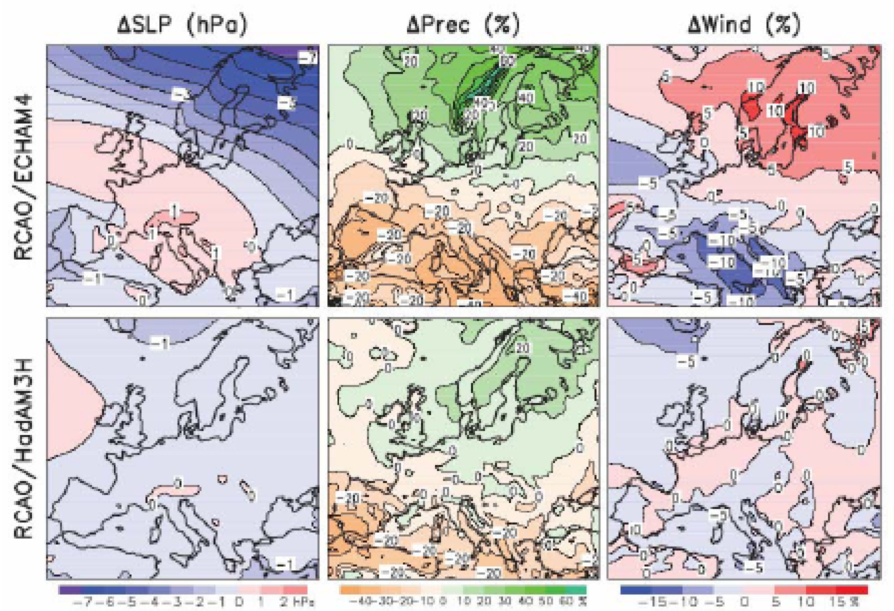

Figure 9.1 Climate changes according to RCA3-E (top) and RCA3-H (bottom).
Winds are the two figures to the right while the middle pictures show the precipitation and the left pictures show the atmospheric pressure at the sea water level (IPCC, 2007)

Regardless of all the uncertainties, two different scenarios concerning wind will be used for further investigations. A first scenario where there is a 13% increase of all wind speeds during the winter months and a second scenario where wind patterns do not change in the future. These will be used both when analysing wind data and when performing simulations of the shoreline response.

9.2.2 Sea level

Depending on which IPCC scenario is assumed to happen, an increased water level of 18-59 cm is likely. Additionally, in the Baltic Sea there will be a regional increase of around 20 cm, implying that the average water level increase **in 2070** is estimated to be **38-79 cm**. These values do not consider the eventually increasing melting rate of the glaciers but are based on the melting rate observed from 1993-2003. (IPCC, 2007)

Some scientists claim other values for the rising water level, where (Pfeffer, Harper, & O'Neel, 2008) suggests an increase in water level of 0.79-2 m. (Rummukainen & Källén, 2009) uses 0.8 m plus a local increase of 0.2 m when discussing increased water level in Sweden. The main reason for these higher values is that they consider an increased melting rate of the glaciers around the world. In the further analysis the figures from IPCC will be considered.

Historically, the average global increase in water level has been 1.8 mm/year during 1963-2003 and 3.1 mm/year during 1993-2003 (IPCC, 2007). This indicates that there is an accelerating increase in the water level rise. Most of the increase in water level up to year 2070 is therefore most likely to happen during the latter part of the period, but when using the increased sea water level a linear increase will be considered.

Additionally to the rise in the average water level, there will be higher values for extreme water levels. The high water level, with a return period of 100 years, is likely to increase 20-30 cm or 60-70 cm, considering the *best scenario* and the *worst case scenario*, respectively. These values are expressed relative to the increased mean water level.

9.3 Scenarios

Considering the IPCC scenarios and the reports from SMHI, four different scenarios are considered in the further analysis and calculations discussed here. They are noted as H1, H2, I1, and I2 standing for **H**istorical trend of water level and **I**ncreased sea level (1 and 2 for the increase of wind or not) and they are summarized in Table 9.2 below.

Table 9.2 Summary of scenarios used for further analysis

	Sea Water level	**Winds**
Scenario **H1**	Following historical trend	No increase
Scenario **H2**	Following historical trend	13% increase of winter winds
Scenario **I1**	Increased 79 cm linearly up to 2070 (1.3 cm/year)	No increase
Scenario **I2**	Increased 79 cm linearly up to 2070 (1.3 cm/year)	13% increase of winter winds

Scenario H1 can be denoted as *no increased climate change*, which is not entirely correct since there is an increase of sea level but it will just follow the historical trend.

Scenario H2 is the same as H1 but with an increase in the wind speed with 13% during the three winter months (December, January, February).

Scenario I1 takes a rising sea level into account. As discussed in section 9.2.2 there are many scenarios from the IPCC, of which an increased sea water level of 79 cm is the worst case scenario. This scenario considers a linear increase of the sea water level up to 79 cm by 2070, which means 1.3 cm/year.

Scenario I2 is the same as I1 but in addition to the increased sea level it also includes increased winter winds. In this project, this scenario is considered the *worst case scenario*.

9.4 Analysis

9.4.1 Wind

A first investigation is performed regarding high wind speeds coming from the six directions that create waves in the study area. The evolution of their relative frequency might help to yield a prediction for the future wind intensities. This is done in order to investigate if there is an increased trend of the high wind speeds that might lead to a critical increase of cliff erosion.

The frequency of near gale winds (>13 m/s) is plotted in Figure 9.2 regardless of the direction of the wind for the years 1976-2008. The trend extracted from these values shows that the relative frequency decreases 0.82% per year, but since there is such great yearly variations and the time period of measurements is short this extracted trend is not considered in the future analysis.

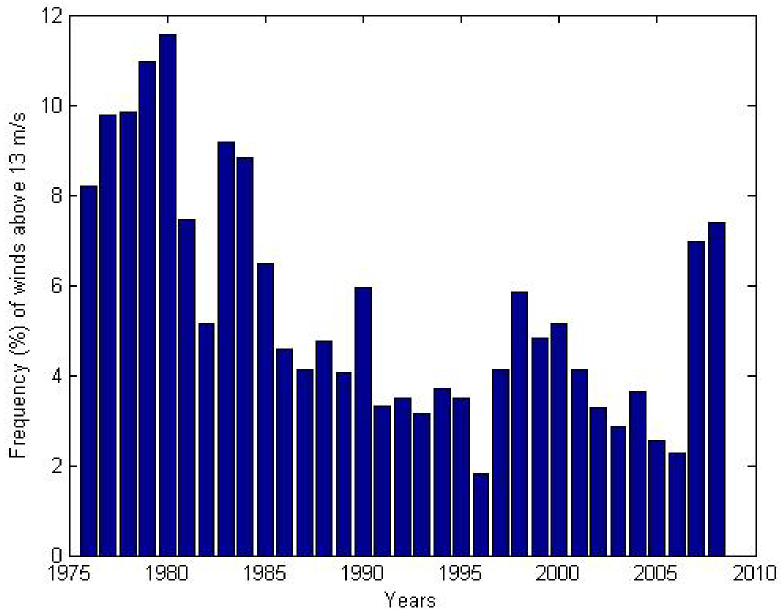

Figure 9.2 Relative frequency of near gale winds, Falsterbo 1976-2008. (Processed data from SMHI, 2010).

9.4.2 Waves

With the change of wind discussed in previous sections, the wave pattern will also change. In addition to increased wind speed, the IPCC discusses changing wind direction. Some scientists claim that there will be an increased number of winds coming from the westward direction. In the case of Bjärred, increased amount of winds coming from the west would lead to more waves generated from one of the directions that hit almost perpendicularly the shore, and thus increasing the probability of erosion. Since there are many uncertainties and no quantitative estimations this will not be taken into account in this study, though. (IPCC, 2007)

9.4.3 Sea level

A study performed of water level data in Viken and Barsebäck (Karlsson & Martinsson, 2010), shows that there is a trend of increased sea level, corrected to the land rise in Viken, of 0.16 cm/year. This value will be used to extend the data series of water level (1976-2008), in order to get a prediction of the water level fluctuations in 2009-2041 assuming that the same trend is followed. This will be used as the basis for scenarios H1 and H2.

Figure 9.3 shows the water level (3hr resolution) in Viken for the period 1976-2008, with extended data for 2009-2041. The trend to be extended is 0.16 cm/year.

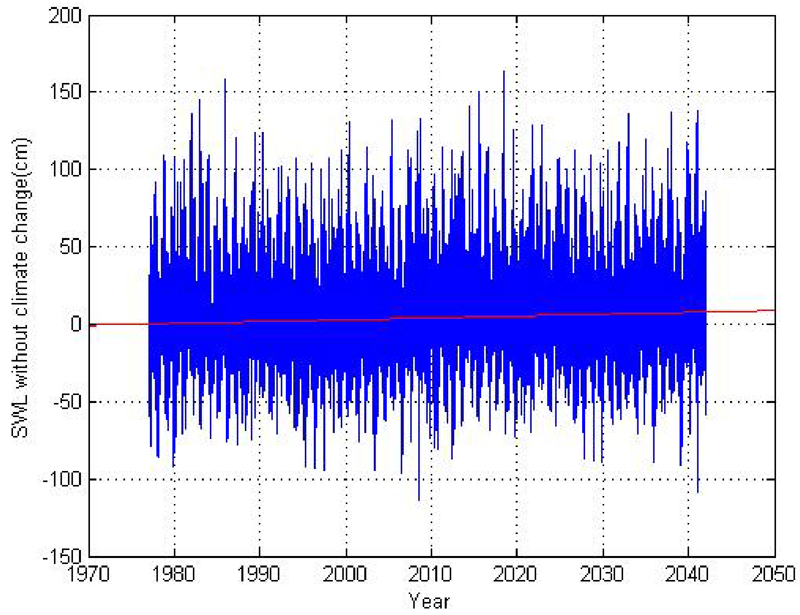

Figure 9.3 Sea level trend for scenarios H1 and H2 (Processed data from SMHI, 2010)

Figure 9.4 below shows the same water level fluctuation 1976-2008 but with a linear increase of 1.3 cm/year from 2009 and on added (corresponding to 79 cm increase in 2070). In the year 2041 the mean water level has risen a total of 41 cm. This is used to elaborate scenarios I1 and I2.

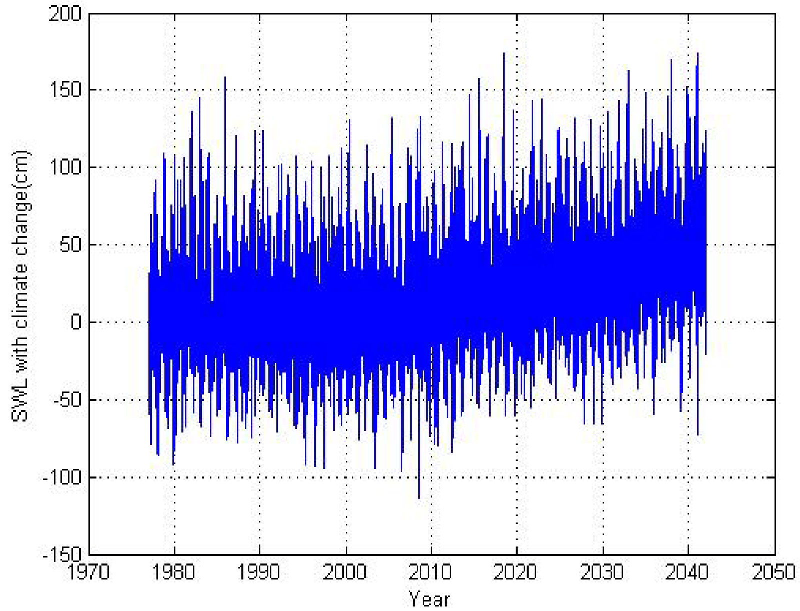

Figure 9.4 Sea level trend for scenarios I1 and I2 with measured values from 1977 to 2008 and a higher trend following IPCC from 2009 to 2041 (Processed data from SMHI, 2010)

As a complementary analysis, the historical return periods and the predicted ones for high water levels is discussed below.

As developed in section 9.2.2 the water level with a return period of 100 years today might have a return time of 10-50 years or 1-2 years considering the different scenarios in the future. This means that a water level of 167 cm, which happens every 100 years at present conditions, will occur every 1-2 or 10-50 years in the future. (SMHI, Framtida medel- och högvattenstånd i Skåne och Blekinge, 2007)

Table 9.3 below shows return periods and new calculated values for water levels in Viken relative to RH70 (SMHI, Framtida medel- och högvattenstånd i Skåne och Blekinge, 2007)

Table 9.3 Water levels for different return periods

Return Time / IPCC Scenario	2 yrs	10 yrs	50 yrs	100 yrs
Historically	106 cm	135 cm	158 cm	167 cm
Best scenario (B1)	128 cm	157 cm	179 cm	188 cm
Worst case scenario (A1FI)	168 cm	198 cm	220 cm	229 cm

The extreme values will not be evaluated further and taken into account for calculations but give an indication of what may actually happen in the future and what one can expect as extreme events.

9.4.4 Wave runup

When comparing the scenarios for runup level, scenario H1 is used as reference. The runup prediction for the 32 years from 2009 to 2041 is presented in Figure 9.5 below. This might be compared to the historical runup seen in Figure 8.8 in section 8.2.

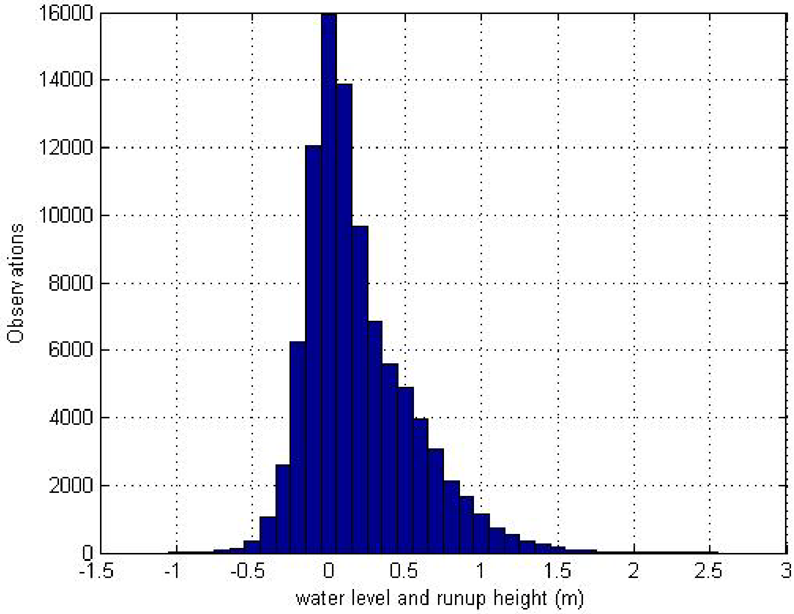

Figure 9.5 Water level and wave runup histogram for scenario H1 (Processed data from SMHI, 2010)

The wave runup distribution for the four analysed scenarios are visualised in Figure 9.6 below, with each scenario plotted separately in Figure 9.7. It can be seen that there is a significantly higher average runup for scenario I1 and I2.

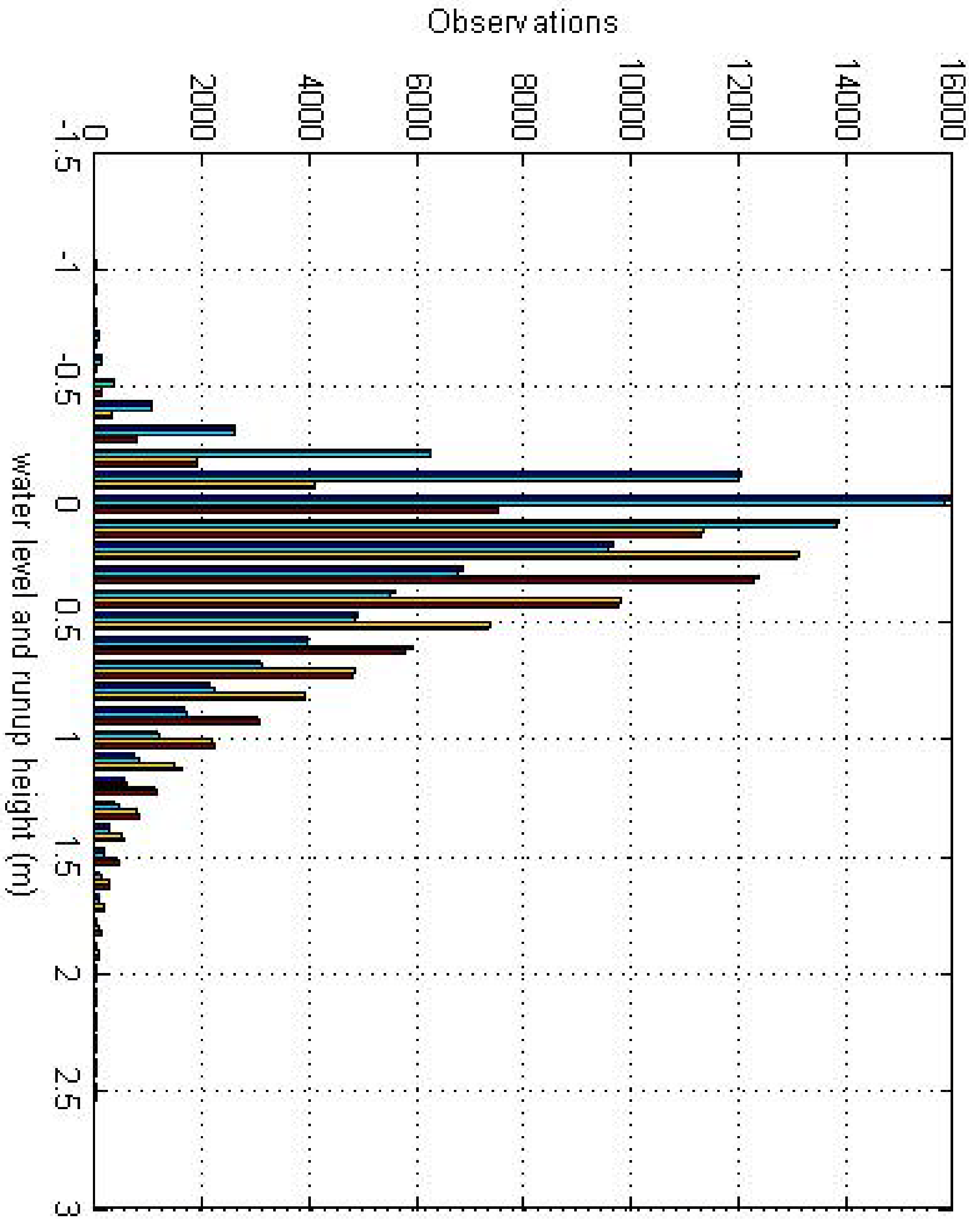

Figure 9.6 Water level and wave runup histogram for scenarios H1,H2,I1,I2
(Processed data from SMHI, 2010)

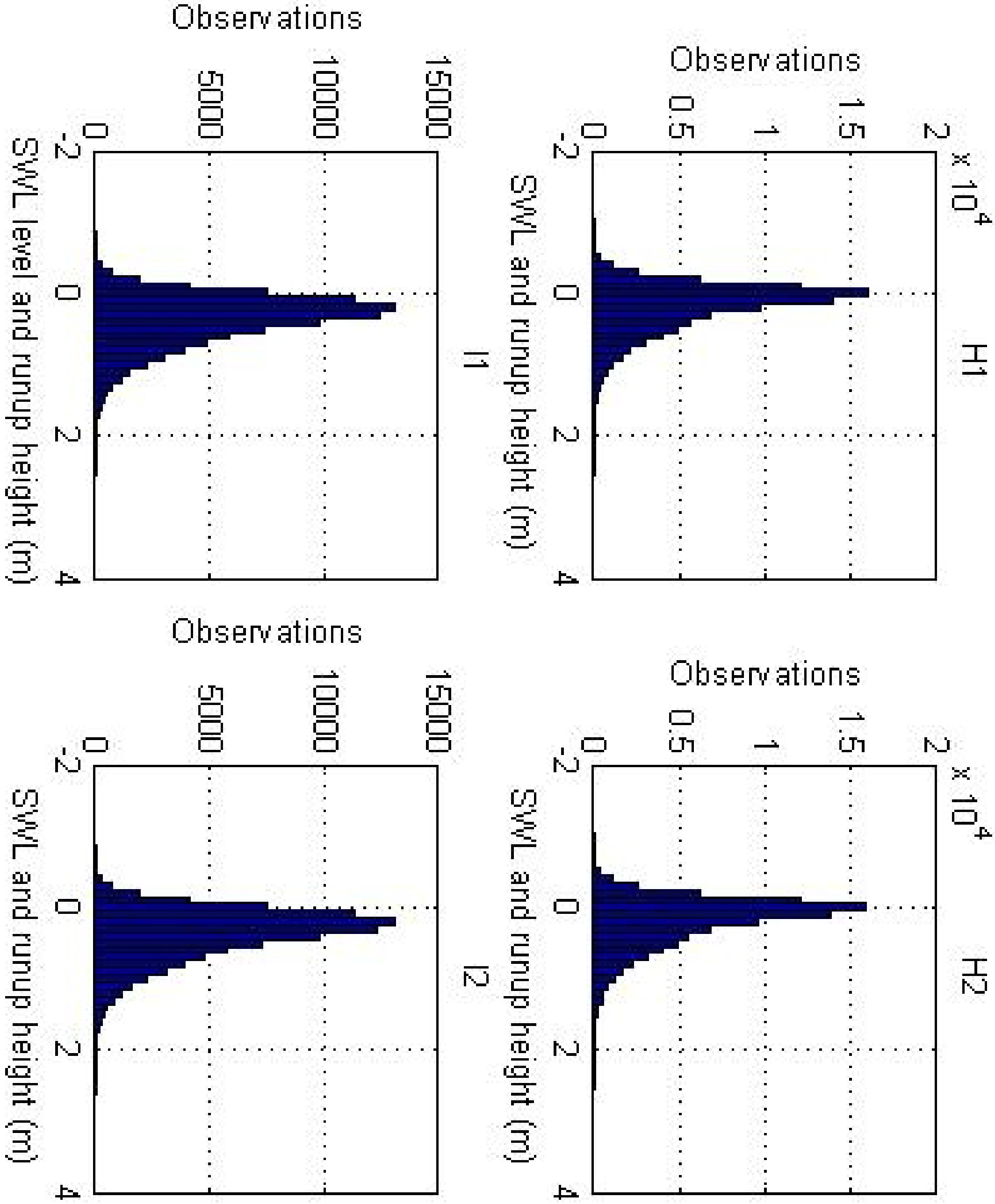

Figure 9.7 Water level and wave runup histogram for scenarios H1, H2, I1, and I2 separately

The predicted wave runup in scenario H1 (same as Figure 9.5) is represented in dark blue, having a mean value of 0.19 m and a maximum value of 2.49 m. An increase of 13% in winter winds leads to the scenario H2, represented by the light blue bars in Figure 9.6. The mean value is slightly higher, but the maximum value is the same as for scenario H1.

Scenario I1, with linear increase of water level of 1.3 cm/year, is represented by the yellow bars. It shows an important increase in the distribution of runup heights, where both mean value and the maximum runup level increases. Scenario I2, represented in red, gives the highest values of the runup. The mean value does not change compared to scenario I1, but the highest value increase to some extent. All values of mean and maximum runup are summarized in Table 9.4 below.

Table 9.4 Values of runup for different scenarios

Scenario	Mean runup level (m)	Maximum runup level (m)
Historically	0.14	2.45
H1	0.19	2.49
H2	0.20	2.49
I1	0.38	2.55
I2	0.38	2.63

As discussed in chapter 2 erosion occurs when the runup hits the cliff foot. The study by (Brännlund & Svensson, 2005) showed that the runup hits the cliff foot an average of 3.6 times per year. Analysis of the data provides how often the cliff will be eroded for the different scenarios, and results are shown in Table 9.5. Note that the number of times per year erosion occurs in this analysis differs from the number obtained by (Brännlund & Svensson, 2005). As mentioned previously this originates from the difference in the wave hindcasting.

Table 9.5 Number of occasions when cliff erosion occurs in the area

Scenario	Number of times runup exceeds cliff foot elevation (1.4m)	% of total observations	Times per year
Historically	434	0.47	13.6
H1	522	0.56	16.3
H2	613	0.66	19.2
I1	1277	1.37	39.9
I2	1469	1.57	45.9

As can be seen, the number of occasions when erosion will take place increases significantly. These numbers assume that the cliff foot will not migrate but stay in the same position. In reality the height of the cliff foot will most likely change and the number of times when erosion occurs be more constant. This migration is due to the erosion, though, meaning that the numbers still give an indication of the magnitude of the increased erosion rate. The next chapter will focus on the cross-shore transport, performing calculations for different scenarios and also estimate the migration of the cliff.

10. Cross-shore transport calculations

10.1 General

For specific short-term events, such as severe storms, large amounts of sediments can be moved offshore, and therefore contribute to beach erosion. When calmer periods come, though, sediment transport is normally directed onshore, which helps rebuilding beach profiles (CEM, Cross-shore Sediment Transport Processes, 2002b). Figure 10.1 shows this seasonal fluctuation of sediments and the resulting change in the beach profile, and Figure 10.2 illustrates the response of the beach to a storm.

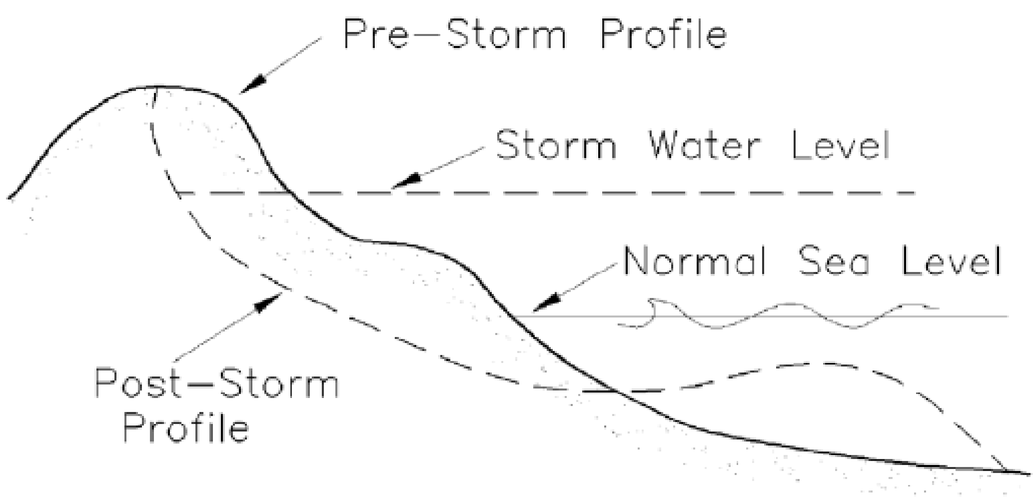

Figure 10.1 Seasonal beach profile change (CEM, Cross-shore Sediment Transport Processes, 2002b)

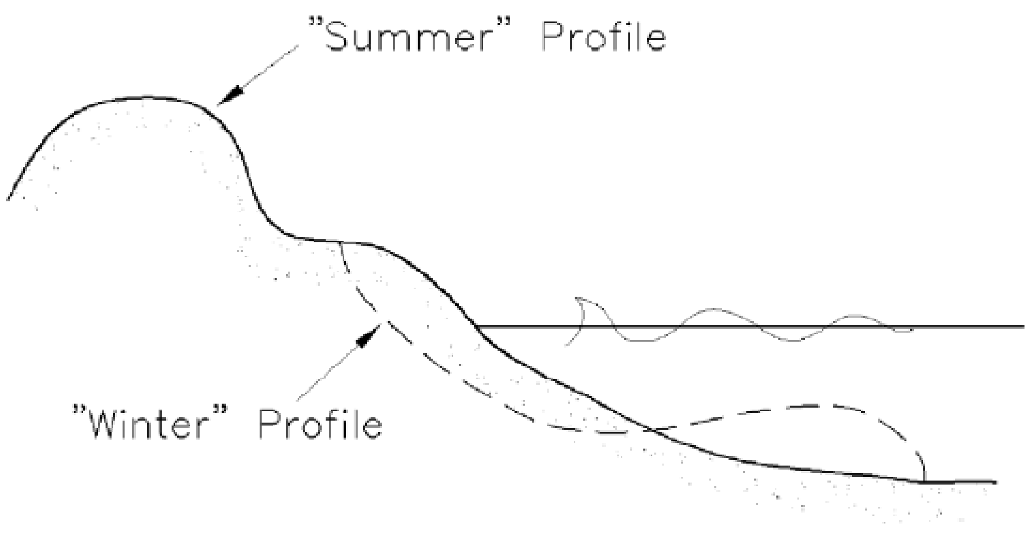

Figure 10.2 Response to a storm (CEM, Cross-shore Sediment Transport Processes, 2002b)

Since the beach material in Bjärred consists of a wide range of grain sizes and the smaller sizes (clay and silt) might not be able to be transported onshore during calm weather conditions, the onshore transport and deposition of sediments might not be sufficient to compensate for winter storms effects. Thus, it is necessary to evaluate the impact of cross-shore transport by calculating the volume of sediments that is lost every year due to this phenomenon.

As explained in Chapter 2, the transported sand volume per meter of shore length ΔV (m^3/m) can be estimated from the empirical Equation 10.1 (Hanson, Larson, & Erikson, An analytical model to predict dune erosion due to wave impact, 2004).

$$\Delta V = -4C_S \frac{(R-z_0)^2}{T} \Delta t \qquad \text{Equation 10.1}$$

The formula shows that dune erosion will occur when the wave runup, R, is higher than the level of the dune foot, z_0, and therefore water can carry some sediment back to the sea.

The constant C_s is specific for the study area and it is normally obtained by calibration from historical data (*e.g.*, aerial photos). In the present case, shorelines between 1963 and 2002 are compared to obtain the average retreat. The retreat obtained is 4.3m (see chapter 4), implying a yearly retreat of 0.11m. This rate leads to an estimation of the average value for ΔV, and by using wave data obtained from this period the C_s is estimated to be $1.97*10^{-5}$. This value is lower compared to the obtained C_s in previous studies ($7.70*10^{-5}$) (Brännlund & Svensson, 2005). The explanation is, in addition to the differences in the aerial photograph analysis, is the discrepancy in the estimated z_0 (1.4m compared to 1.3m), as well as a different input wave climate.

Larson *et al.* (2004) claimed that C_s obtained by field studies should range between $3*10^{-5}$ and $2*10^{-3}$. The estimated C_s ($1.97*10^{-5}$) is at the lower end of the boundary, explained by the substantial presence of clay and silt in the sediment.

The runup height, R, includes the wave runup, obtained as explained in Chapter 9, for each wave (three hours resolution) as well as the sea level at that time. All these values have been obtained as discussed in previous chapters. The parameter z_0 represents the height between the mean water level and the foot of the dune and is based on the survey results set to 1.4 m. The period, T, is the wave period in seconds and Δt is the duration between observations, which is 10800 s (3hr).

Once again, the time period for which the cross-shore transport will be studied comprises between 1977 and 2008, the period where data for both water level and waves are available.

The scenarios discussed in the previous sections will be taken into account and calculations of the transport for these scenarios will be performed.

10.2 Historical evolution

The results of eroded volume per year between 1977 and 2008 are displayed in Figure 10.3 below. The average transported volume per year is 0.20 m^3/m and as can be seen the normal yearly erosion rarely exceeds 0.40 m^3/m. For some specific years, especially 1981, erosion increases dramatically (up to 1.16 m^3/m), which most likely is due to heavy storms in combination with high water levels during this particular year. This extreme response for 1981 is confirmed by a similar study performed in Landskrona, which also gives this extreme value for 1981 (Karlsson & Martinsson, 2010).

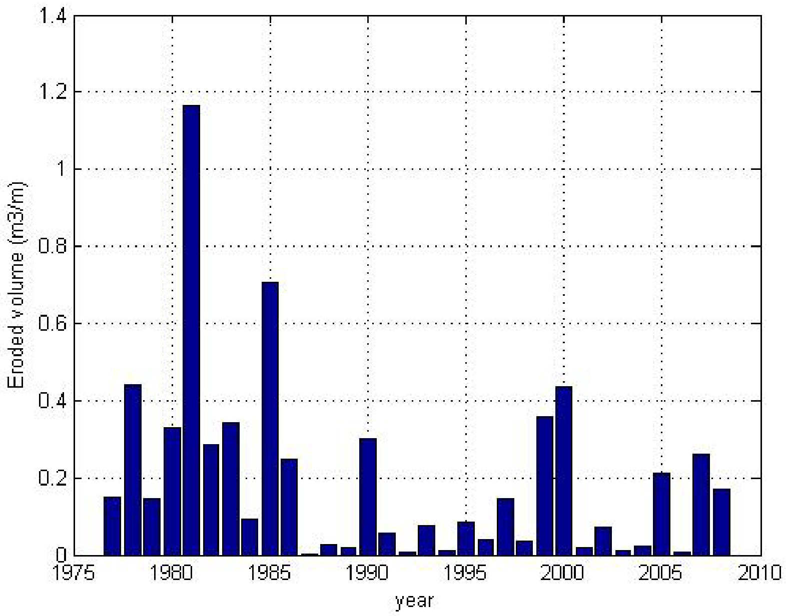

Figure 10.3 Annual eroded volume for the study area

For this study area in Bjärred, similar studies of cross-shore transport have been performed for a shorter period of time (Brännlund & Svensson, 2005). The time period in the study ranges from 1985 and 2003 and the mean value of cross-shore transport per meter of width was 0.28 m^3/m, which is somewhat higher than the value obtained in the present study (0.20 m^3/m). One explanation might be that in the previous study, the height of the cliff foot above the mean sea water level was estimated to be 1.3 m, while in this case it is estimated to be 1.4 m. Another addition to the higher value of erosion is the C_s parameter, which is lower in the present study.

10.3 Future scenarios

Before estimating future increase in runup heights, a model for the future beach profile must be assumed. Since the composition of the beach material in Bjärred is not homogenous and contains a large portion of silt and clay there is limited sand to rebuild the cliff. The assumed development of the profile, in a simplified sketch, can be seen in Figure 10.4.

The main assumptions are that both beach slope and the height of the cliff foot, relative to the water level, (z_0) remain the same after the water level rise. A sea level rise means shifting the mean sea level upwards, thus it can be regarded as a decrease in the original z_0 with time. Consequently, z_0 is displaced vertically upwards and the cliff volume is reduced. The vertical sea level rise is represented by S and the volume that would disappear from the cliff is denoted by dV. As mentioned above, this reduction of volume comes from the assumption that not enough material is present to rebuild the cliff, for example sand transported by the wind, as the sediments to a great extent consists of fine material.

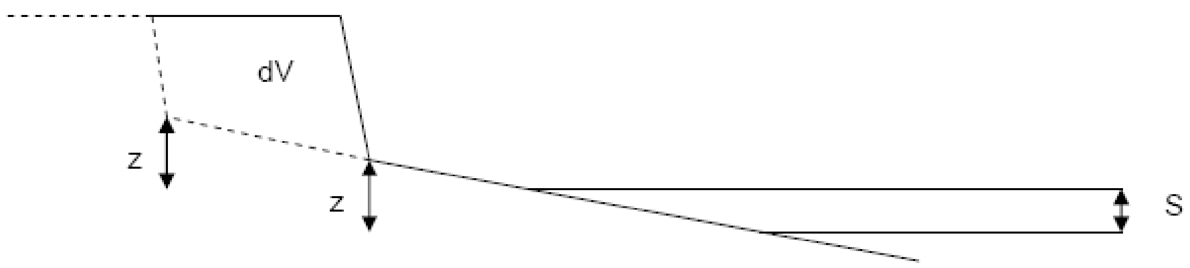

Figure 10.4 Sketch of simplified future beach profile (Karlsson & Martinsson, 2010)

As developed in chapter 9, four future scenarios for the wave and SWL evolution are considered in Lomma bay. Estimated volumes of eroded sediments are presented in the following graphs. In Figure 10.5, results are presented first only for scenario H2 (red bars) in comparison with scenario H1 (dark blue bars).

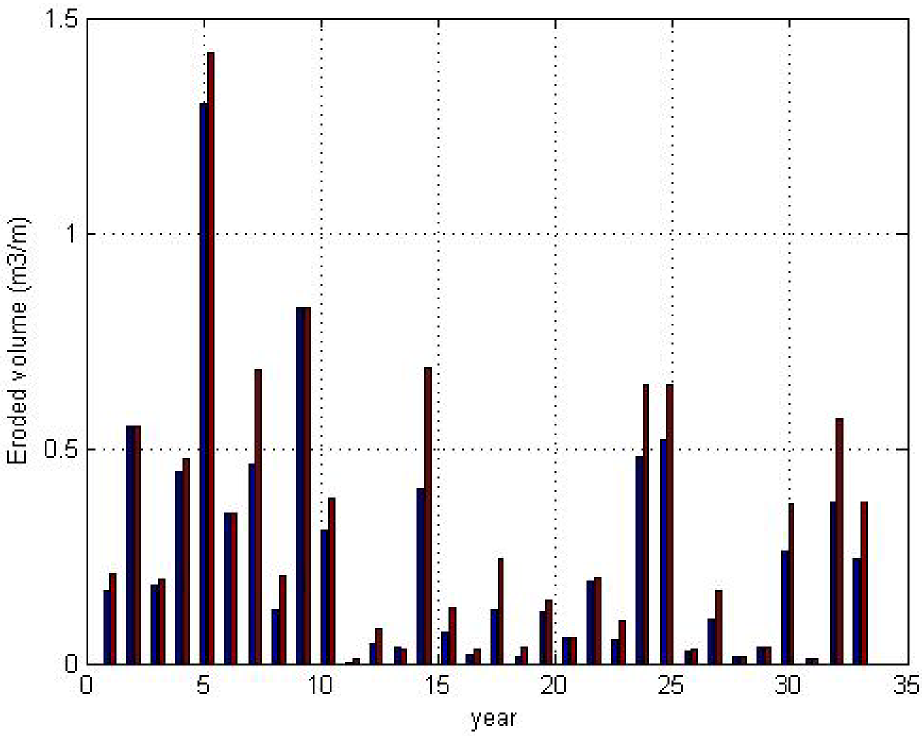

Figure 10.5 Annual eroded volume for scenarios H1 and H2

Note that historical wave data and SWL records are used to compare these two scenarios. Thus, the evolution from year to year is not relevant but it gives a good estimation of how the erosion rates might change in the future. During these conditions, the mean yearly eroded volume for the second scenario increases to 0.31 m^3/m compared to 0.25 m^3/m (an increase of 24%) whereas the highest erosion rate reaches 1.42 m^3/m (increase of 9.2%, compared to 1.30 m^3/m in the basic scenario).

Some years, such as number 2 and 9, present the same erosion in terms of volume. In these cases winter winds were quite calm and the waves that were generated, even if they were increased by 13%, did not contribute to runup heights of 1.4 m (z_0). Therefore, erosion did not occur during the winter but during the rest of the year, when both scenarios present the same conditions.

Results of eroded volume for all four scenarios are presented in Figure 10.6 and Table 10.1 summarizes the eroded volumes obtained in the four cases.

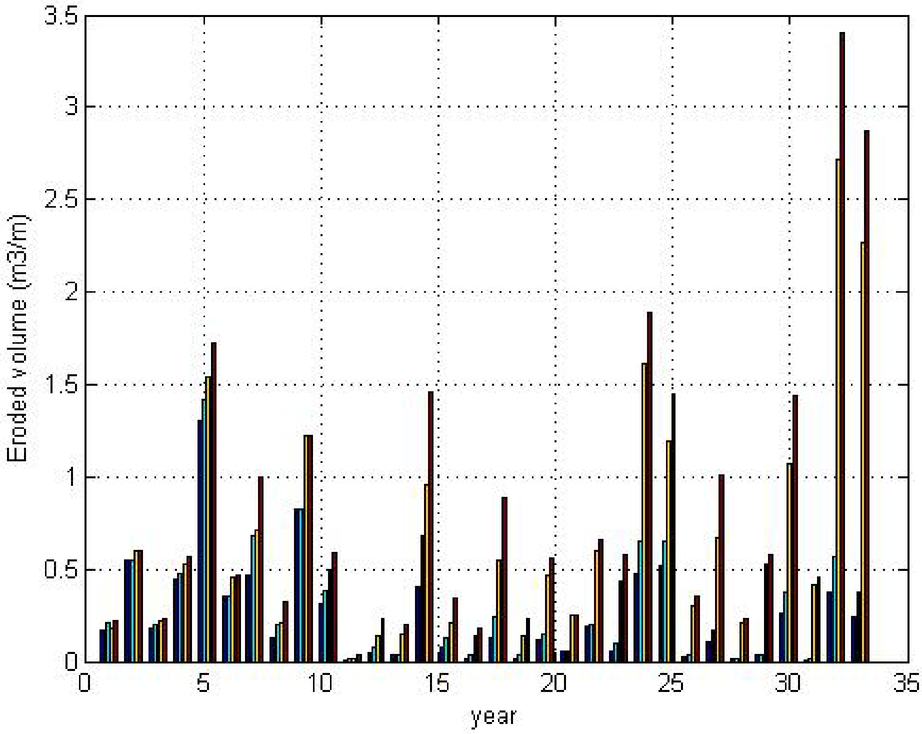

Figure 10.6 Erosion rates for the different scenarios. Scenario H1 is represented by dark blue bars, scenario H2 by light blue, I1 by yellow, and I2 by red

Table 10.1 Cliff erosion rates for the four different future scenarios

	mean dV eroded (m^3/m)	rise (%)	max dV eroded (m^3/m)	rise (%)
Scenario H1	0.25	-	1.30	-
Scenario H2	0.31	24	1.42	9
Scenario I1	0.66	164	2.72	109
Scenario I2	0.81	224	3.41	162

As mentioned, the purpose of this graph is to show a comparison between scenarios, not between specific years. It is clear that the 79 cm increase of SWL by 2070 has a large impact on the erosion rates (scenario I1 and I2). The trend observed for these scenarios, showing gradual increase of erosion, is due to the linear evolution assumed for the water level rise.

10.3.1 Bruun rule

In order to get a simple estimate of how the shoreline will respond to a rising sea level, Bruun's rule can be used. It is a two-dimensional model with the basic assumption that a shoreline profile is in equilibrium. Since the profile is in equilibrium, increased sea level (which cause the shoreline to retreat), will cause a rise in the bottom profile (Hanson & Larson, Sandtransport och kustutvecklingen vid Skanör/Falsterbo, 1993). The Bruun rule shows that a very little increase in the sea level can cause a very large retreat of the shore. As mentioned previously, an increased sea level of 1 cm makes the shoreline retreat approximately 1 m (Eurosion, 2004). This is a general expression and a more detailed site specific calculation is performed below.

As described in Chapter 9, the probability of a future sea level rise is very high and the consequences of the two presented scenarios, with 38 and 79 cm increase respectively, will be discussed further in this section.

The Bruun rule is applicable where the waves are able to erode beach, meaning not in areas where the shore consists of rocks and/or revetments and similar.

The retreat, *R*, is calculated using Equation 10.2.

$$R = \frac{L \cdot S}{B + h} \qquad \text{Equation 10.2}$$

where

L = horizontal distance from the dune to the depth of closure (L_* in the figure)
h = depth of closure (H_* in the figure)
B = dune height
S = increased water level

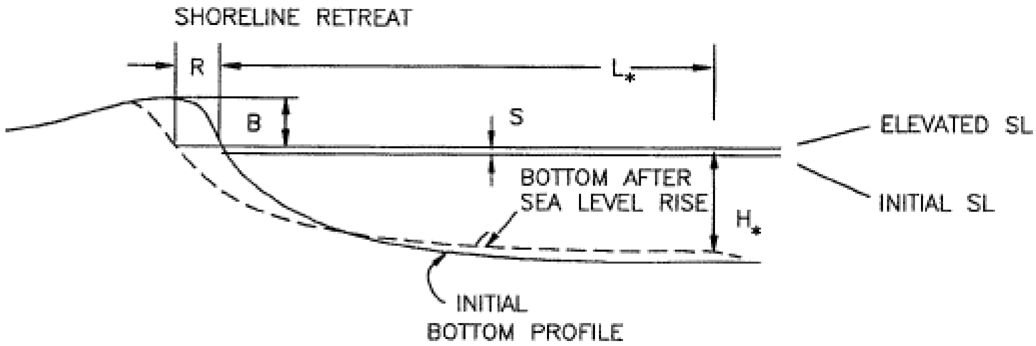

Figure 10.7 Description of Bruun's rule (CEM, Coastal Morphodynamics, 2002c)

The depth of closure is estimated to be 4 m and the distance L (900 m), is observed from a nautical chart of Lomma bay (see Appendix III). According to the survey, discussed in chapter 5, the cliff height is approximately 2.8 m. The increase in water level must be corrected with respect to the land raise, which is estimated to be around 0.1 cm/year (SMHI, Havsvattenstånd vid svenska kusten, faktablad nr 41, 2009). An estimation of the shoreline in the year 2070 means 60 years from now. The results of calculations using Bruun's rule can be seen in Table 10.2 below.

Table 10.2 Horizontal retreat according to the Bruun rule

IPCC Scenario	Water level rise (m)	Land rise (m)	S (m)	H (m)	L (m)	B (m)	Horizontal retreat, R (m)
B1	0.38	0.06	0.32	4	900	2.8	42 m
A1F1	0.79	0.06	0.73	4	900	2.8	97 m

Table 10.2 shows that in the year 2070 the shoreline might, if not protected, retreat 42-97 meters if the scenarios presented by IPCC are fulfilled. An aerial photograph, taken in 2010, with lines showing the predicted shorelines in 2070 can be seen in Figure 10.8. The figure should be interpreted with some caution, as the retreating shorelines represent estimates that does not take into account local conditions, including the presence of structures and differences in geology.

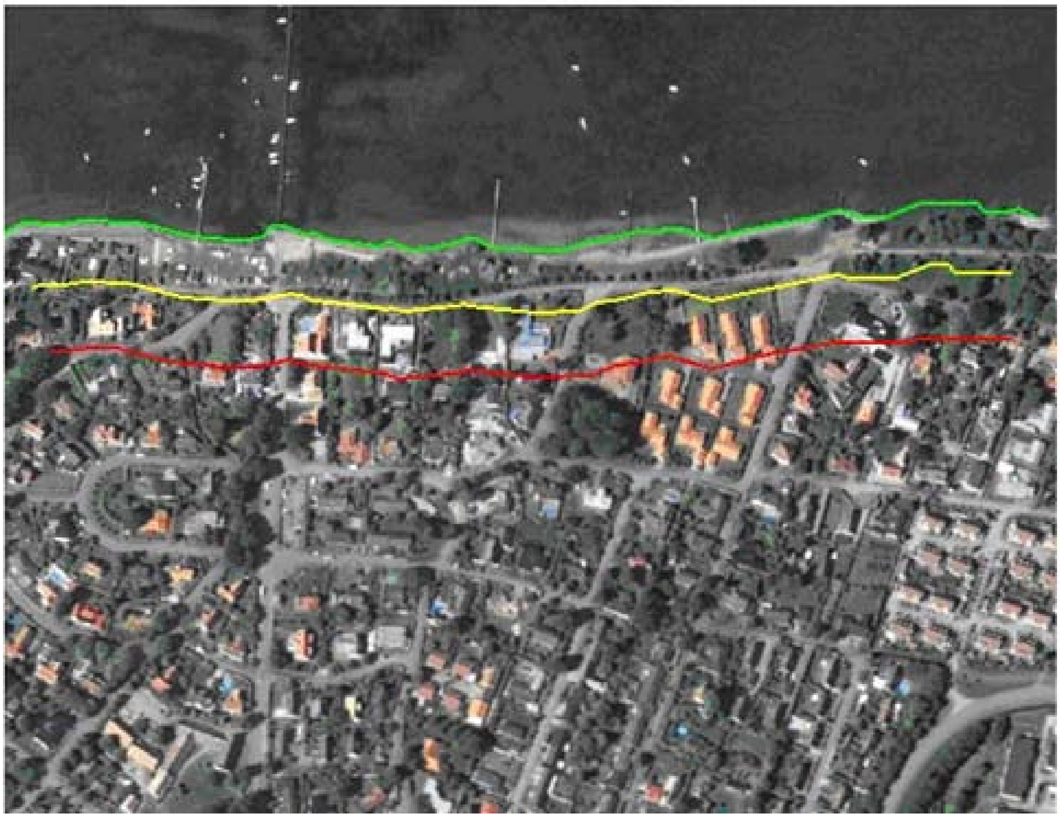

Figure 10.8 Shoreline retreat according to the Bruun rule. The green line represents the shoreline of today, the yellow line a retreat of 42 meters, and the red line a retreat of 97 meters (Lantmäteriet, 2010)

When protective measures are applied, if they work in a satisfying manner, the erosion rate will slow down or completely stop and the retreat will not be as significant as in Figure 10.8. With the numbers and figures above in mind one can easily understand that there is a need for protection and that some action has to be taken. Simulations of different suitable measures for this area will be carried out in chapter 12.

11. Shoreline evolution model GENESIS

11.1 General

11.1.1 One-line shoreline change model

Numerical models are often used to predict shoreline evolution due to sediment transport alongshore. The most common ones are the **one-line shoreline change models**, which are based on the one-dimensional sediment balance equation described in chapter 2. Although analytical solutions have been formulated for many conditions for this equation, they can only be applied to simplified conditions. Instead, a numerical method enables to calculate the shore response with input parameters changing in time and along the shore, such as the wave field, as well as dealing with complicated boundary conditions, like groins or offshore breakwaters. (Jayakumar & Mahadevan, 1993)

One-line models can be used to estimate rates of longshore sand transport as well as long-term shoreline change. The main assumption is that the beach profile has a constant shape, the shoreline, and the equations are applied to only one contour line. The bottom profile is allowed to move in the cross-shore direction between the offshore depth of closure and the berm crest elevation, typically assuming that the sediment transport is uniformly distributed over the area in between. (CEM, Longshore Sediment Transport, 2002a)

11.1.2 Model structure in GENESIS

GENESIS (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989); (Gravens, Kraus, Hanson, & Hans, 1991) is a modelling system based on the one-line shoreline change concept. The name stands for **GENE**ralized model for **SI**mulating **S**horeline Change. It computes the shoreline change at grid cells along a given coastline during a certain period of time as a function of the breaking wave height and angle (see Figure 11.1). This modelling tool is used in this project to evaluate the effect of longshore coastal processes taking place in Bjärred area and, in a later stage, evaluate different alternative designs of shore-protection solutions.

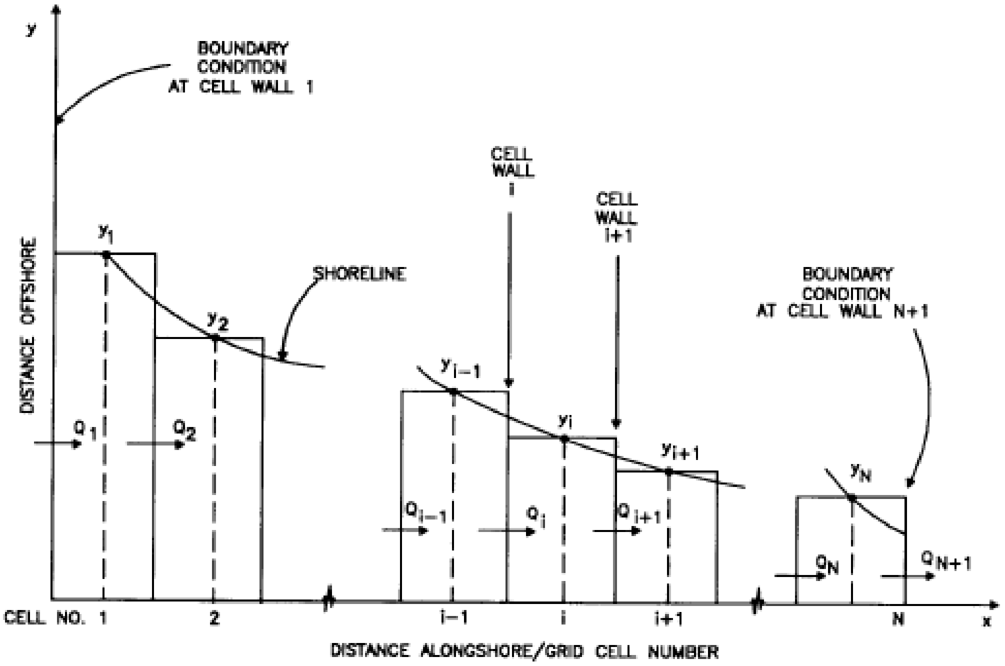

Figure 11.1 Grid system used in GENESIS (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

GENESIS is composed of two major submodels of which the first one is called the internal wave transformation model and is used to calculate the transformation of the input waves, whereas the second submodel determines the sediment transport rates and the associated shoreline change.

11.1.2.1 Wave submodel

The internal wave transformation model computes breaking wave height and angle under simplified conditions. It requires information of input waves originating at a reference depth offshore. The transformation is sketched in Figure 11.2.

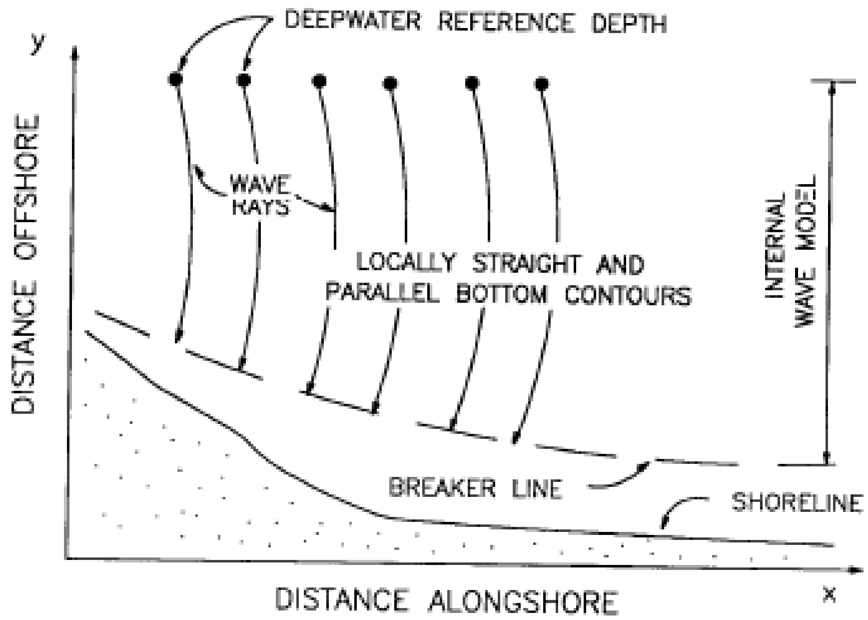

Figure 11.2 First submodel in GENESIS (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

On their way to the shore, waves experience **shoaling** and **refraction** phenomena (explained in chapter 7), which modify the wave characteristics. If no structures exist on their way (such as detached breakwaters or large groins), these waves are used as an input for the second submodel. Otherwise, if the train of waves is interrupted by a barrier, they will experience an additional phenomenon called **diffraction**. In this case, energy is transferred along the wave crest to the lee of the barrier. These phenomena are all taken into account in this wave submodel. (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

11.1.2.2 Transport submodel

The second submodel uses the empirical equation presented in Chapter 2 to calculate the rate of longshore sand transport:

$$Q_l = H_{b\,sig}^2 C_{gb} \left(a_1 \sin 2\alpha_b - a_2 \cos \alpha_b \frac{dH_{b\,sig}}{dx} \right) \qquad \text{Equation 11.1}$$

Physical interpretation of each parameter is developed in Chapter 2. The input data required for this calculation are wave height, period, and direction. Parameters a_1 and a_2 are functions of two empirical coefficients, K_1 and K_2, that are called *transport coefficients*. These coefficients are site specific and must typically be adjusted through a calibration process. The first coefficient, K_1 controls the longshore sand transport rate globally inside the model domain, while K_2 is only important adjacent to and in the lee of coastal structures.

The governing equation for shoreline change used by GENESIS is

$$\frac{\partial y}{\partial t} + \frac{1}{(D_B+D_C)}\left[\frac{\partial Q}{\partial x} - q\right] = 0 \qquad \text{Equation 11.2}$$

The berm elevation (D_B) and the closure depth (D_C) are values provided by the physical data obtained during surveys of the study area. The longshore transport rate Q_l is given by the longshore transport equation (Equation 11.1) and q depends on the sand availability, which is assumed not to be limited.

How accurate the predictions by GENESIS are depend on adequate available data. As it is difficult to quantify the errors and assumptions in the input information, the model in general does not provide statistical estimates, just selected predicted results (Young, Pilky, Bush, & Thieler, 1995). In this study the main sources of errors could originate from the meteorological data from the station in Falsterbo, from the transformation of this wind data into wave data by using empirical equations, and from the use of simplified field conditions, such as depths and grain characteristics. The assumption that there is an unlimited amount of sand might affect the results. This assumption is further analyzed in the discussion in chapter 14.

11.1.3 Data needed for model simulation

An accurate preparation and analysis of input data is necessary in order to be successful in modeling the study area. The general information needed by GENESIS is:

- Spatial and temporary ranges of the simulation
- Structures and beach fill configurations (if any)
- Values of model calibration parameters
- Simulated times when output is desired
- Initial and measured shoreline positions
- Offshore and nearshore wave information and associated reference depths

The boundary conditions of the study area must be set by specifying the sediment transport rate and direction at each end. There are 3 main conditions (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989):

- No sand transport: $Q_l = 0$ (i.e. Large headlands or jetties)
- Free sand transport (pinned beach): $dQ_l/dx = 0$ (i.e. points where the position of the shoreline has not changed for many years).
- Partial sand transport: $Q_l \neq 0$ (i.e. location where the longshore transport rate is known, like an artificial sand bypassing at a jetty)

Although the focus of this study is a small area of the entire Lomma bay, it is useful for the general understanding to first simulate the sand transport pattern all along the bay to qualitatively determine the overall net transport drift in the Bjärred area.

GENESIS also needs the following empirical parameters (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989):

- *Depth of longshore transport*: is normally assumed to be the surf zone. Depends on the breaker index and the significant wave height at breaking.

- *Average profile shape and slope*: to determine the location of breaking waves alongshore and to calculate the average nearshore bottom sloped used in the longshore transport equation. It depends on the beach grain size (input parameter) and the depth of longshore transport.

- *Depth of closure*: beyond this limit the bottom profile does not have a significant change in depth. It is an input parameter to be specified.

Before performing any simulations the input wave data needs a minor adjustment. In order to have a more homogeneous input data each 10° direction has been randomly split into 10 parts to obtain directions with 1° resolution. This will make the shoreline response more natural.

11.2 Lomma bay analysis

Before focusing on the simulation of the study area in Bjärred it is important to obtain an understanding of the sediment transport behavior for the whole bay. By simulating the transport of material from a long-term series of incoming waves it is possible to get an idea of the direction of the main drift of sediments and the net transport rates.

An overview of Lomma bay is shown in Figure 11.3 below.

Figure 11.3 Satellite picture of Lomma bay (Google Earth, 2010)

11.2.1 Model setup

Digitizing of bay shoreline above has been performed and provided as an input to GENESIS. The program needs physical data, such as depth of closure, berm height, and grain size of the bay, as background information.

In the shoreline coordinate system the *x*-axis has been drawn parallel to the coastline trend in front of the village of Lomma (longshore baseline) and the *y*-axis points offshore, perpendicular to the baseline. Grid spacing is fixed to 100m and the extent of the modeled area is about 10km from south to north. The input wave data used is generated by winds recorded in Falsterbo station between 1961 and 2008 with a 3hr frequency (SMHI, Vinddata, Falsterbo, 2010).

The placement of structures is based on aerial photo analysis and field visits to the area. In order to get an accurate estimation of the transport the placement of structures in the model needs to be as close to the actual location as possible. A seawall and groin are placed to represent the harbor of Lomma and the outlet of Höje å. Other seawalls are meant to simulate rocks or revetments scattered along the bay, for example the ones present in Bjärred. The modeled area is considered as a littoral cell, where the boundary conditions are well determined and estimated to not allow any sediment coming in and out. For this reason groins are placed at both ends representing the natural borders. Geological conditions, such as beach and bottom profiles, are based on the results from the survey as well as from assumptions developed in previous chapters.

11.2.2 Simulation results

The bay area in the GENESIS interface has the appearance seen in Figure 11.4.

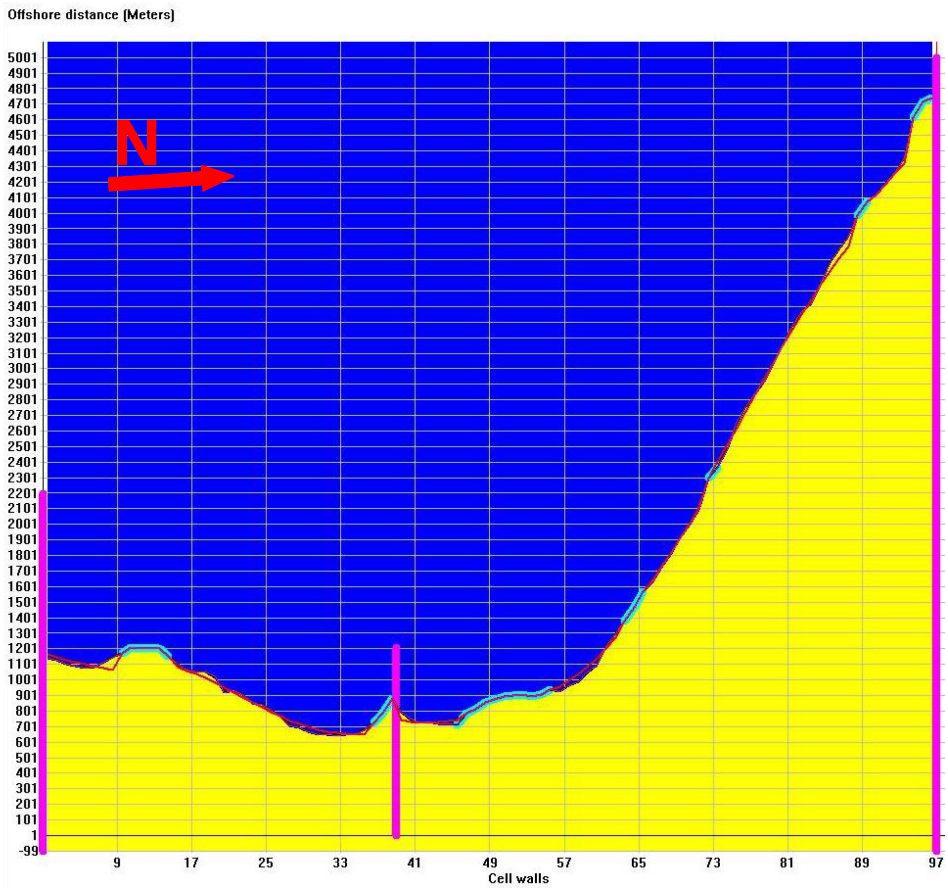

Figure 11.4 Lomma bay modeled in GENESIS

Groins are represented in pink colour and seawalls in light blue. The red line represents the final shoreline when the simulation is finished. Since the bay is assumed to be close to equilibrium the final shoreline should remain similar to the original one, and as can be seen in Figure 11.4 Lomma bay modeled in GENESIS more or less yields this result.

GENESIS can provide, among other information, yearly rates of gross and net longshore transport. In Figure 11.5 the mean values of both net and gross transport rate for each cell during the whole simulated period (48 years) are plotted.

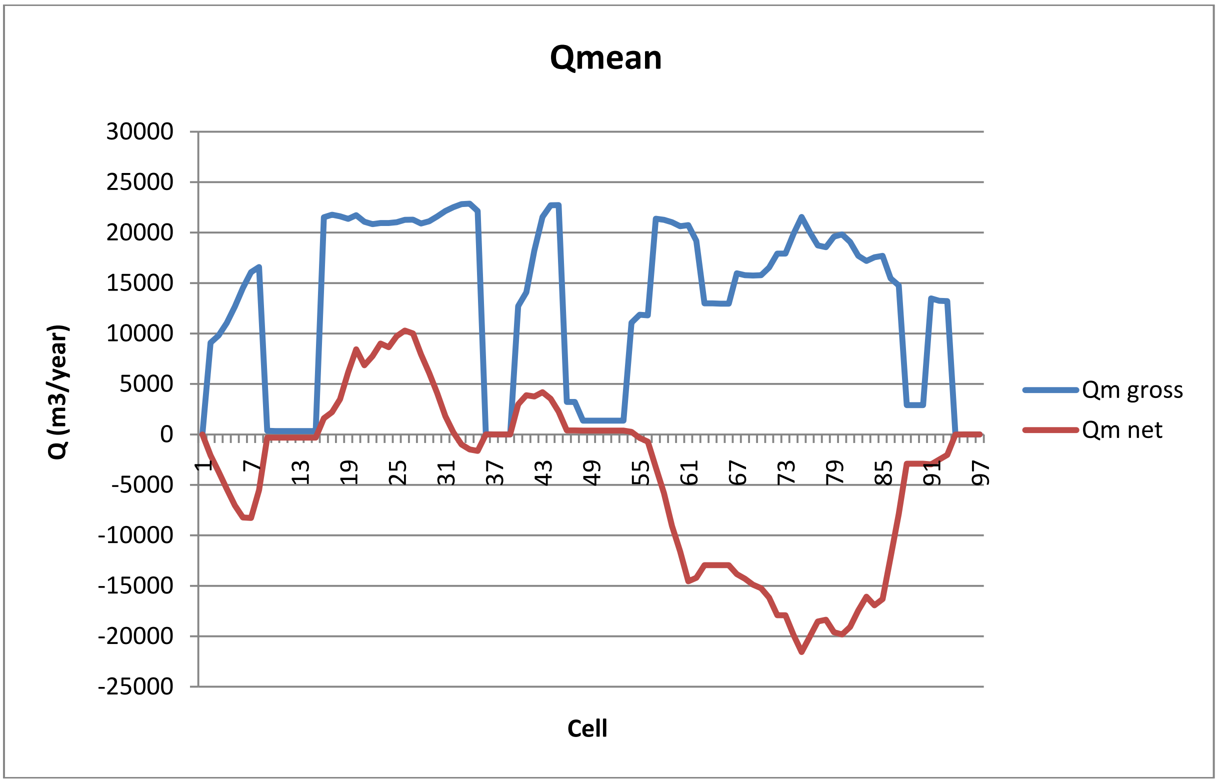

Figure 11.5 Yearly mean net and gross transport rates (1961-2008)

Cell 1 corresponds to the most southern part of the bay and cell 97 is in the very north. The study area in Bjärred is located around cell 67. For the general pattern of sediment movement, the most interesting parameter to study is the mean net transport. Its evolution shows that sediments move southward in the north part of the bay, since the rates are negative there. Around Lomma harbour, though, the pattern changes and net transport is directed north. The gross transport shows a more irregular pattern, likely caused by the presence of structures on the shore.

In the area of Bjärred, represented by cell 67, rates of gross and net sediment transport have, approximately, the same values and with opposite signs. Therefore it is concluded that all the longshore sand drift is directed southward. This behaviour obtained by GENESIS can be supported with some field evidences of local disturbances around structures on the included images. Figure 11.6 and Figure 11.7 are pictures taken in the northern part of the bay, some distance south of the study area in Bjärred. As can be seen in both images, accumulation and erosion around the groins indicate a net transport directed south. Figure 11.8, taken a short distance north of Lomma harbour shows a change in pattern, which indicates that the sediment transport direction is northward. These field observations confirm the results obtained by GENESIS, wherefore the simulation results are assumed to be reasonable.

Figure 11.6 Northern part of the bay, south of the study area (Google Earth, 2010)

Figure 11.7 Northern part of the bay, south of the study area (Google Earth, 2010)

Figure 11.8 Short distance north of Lomma harbor (Google Earth, 2010)

11.3 Calibration and validation

The aim of this section is to determine the model parameters that best represent the evolution of the shoreline in the study area at Bjärred. This process starts with the model calibration, which should reproduce the variation of the shoreline position during a period of time between two measured shorelines. The calibrated model should, during a second step, be verified through a validation process. This means that the calibrated model is employed during a different time interval in order to establish that the predictions made by the model are representative.

11.3.1 Calibration

The calibration of the model for the study area is performed during 20 years, between 1978 and 1998. The aerial picture in Figure 11.9 shows the measured shore evolution, represented by the vegetation lines for these years.

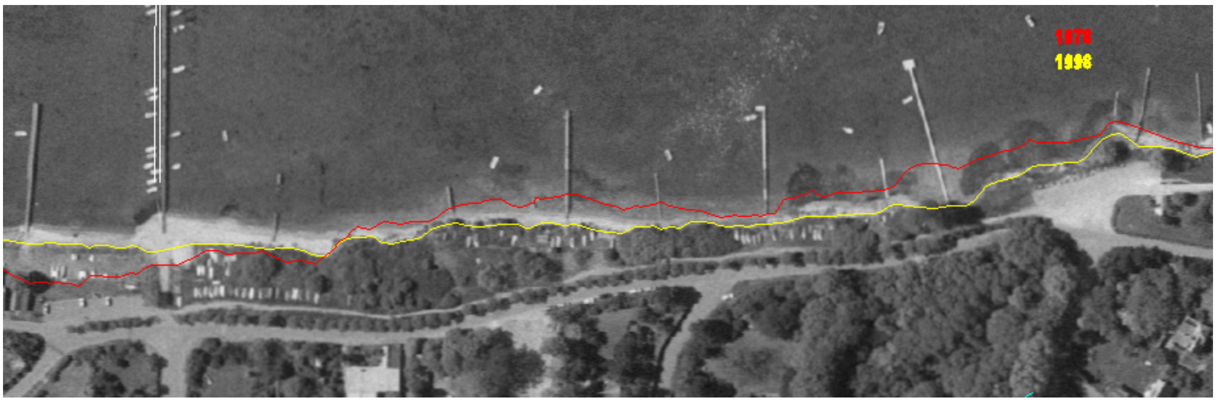

Figure 11.9 Vegetation lines 1978 (red) and 1998 (yellow) (Lantmäteriet, 2010)

At a first glimpse, the transport of sediments seems to have occurred to the left (south), as some sand is accumulated in the southern part whereas the shoreline has been eroding in the north. This pattern has been also observed during the analysis of the whole bay in the previous sections even though most of the accumulation is likely to origin from construction of revetments in that area (see chapter 4).

In the GENESIS interface, the digitized shorelines are presented graphically as Figure 11.10 illustrates. The longshore extension of the area is 370 m, with a grid cell size of 10 m. The original shoreline in 1978 is represented by the yellow field, and the measured shoreline in 1998 is plotted as a thin red line. The piers that are scattered along the area are represented by the thick red lines perpendicular to the baseline. The input parameters for the calibration are seen in Table 11.1.

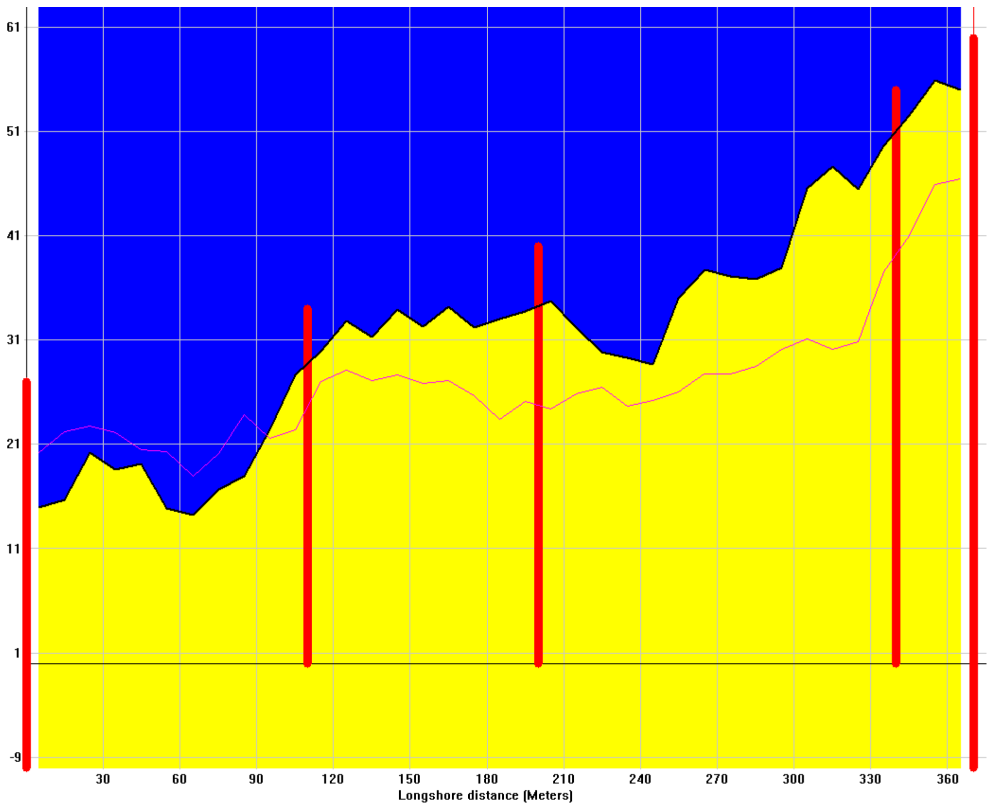

Figure 11.10 Model setup for the calibration

Table 11.1 Input parameters for the calibration

Parameter	Value
Berm height	1.4 m
Depth of closure	4 m
Depth of offshore wave input	10 m (mean value for all fetch depths)

During the calibration process parameters have been changed until the evolution of the shoreline is found to be acceptable and the final parameters are given in Table 11.2.

Table 11.2 Calibrated parameters for the model

Parameter	Value
K1	0.23
K2	0.67
Effective grain size	0.2 mm

The effective grain size differs from the one obtained from the theoretical expression for sea bottom profiles (see chapter 5). The model is not very sensitive to the grain size and the value 0.2 mm gives a better agreement with the observed profile shape compared to 0.05 and is therefore used. The shoreline evolution 1978-1998, predicted by the calibrated model has the appearance given by Figure 11.11 below.

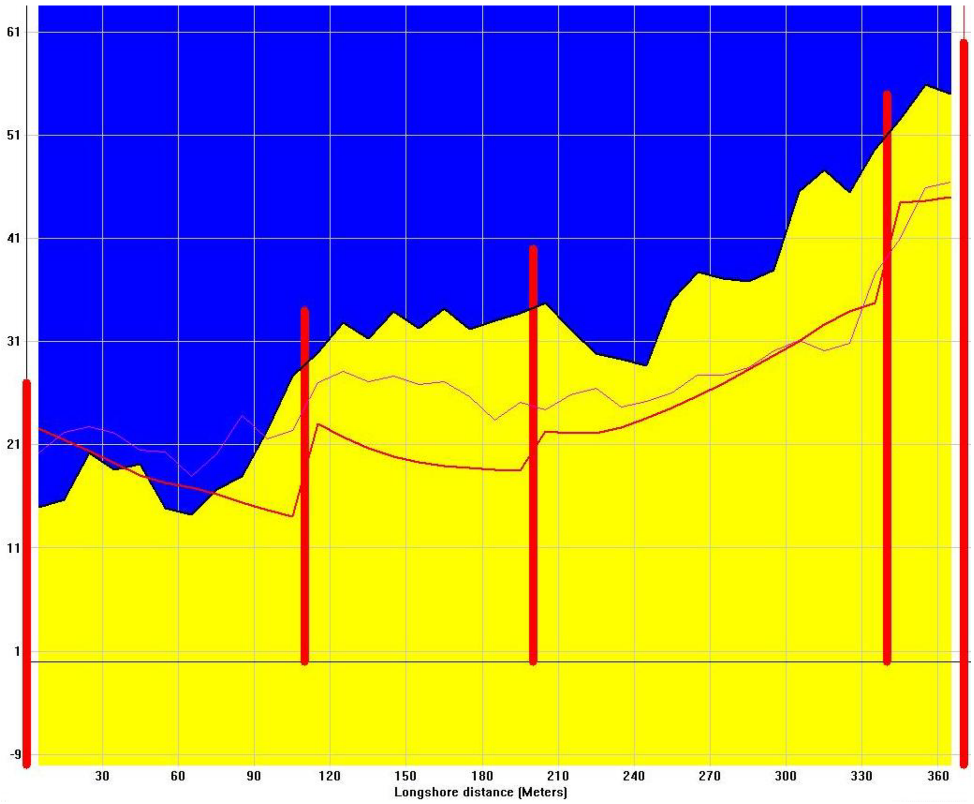

Figure 11.11 GENESIS results for model calibration

The thick red shoreline is the calculated for 1998 by the calibrated model, whereas the thin one is the measured from the aerial photos. If both lines would have been exactly the same, which in modeling is something practically impossible, it would mean that the calibrated model explains 100% the shoreline change.

In this case, though, it has been possible to explain up to 50% of the development. Regardless, the aim of the simulation is not to quantify the exact amount of change but to obtain a good pattern of the shoreline evolution. In this way, when simulating future shoreline changes depending on the actions that could be applied in the area, it will be possible to explain whether they improve or do not improve the present conditions.

11.3.2 Validation

The next step is the validation of the model. An independent time period of 8 years, between 2002 and 2010, is chosen and the vegetation lines extracted from an aerial photo can be seen in Figure 11.12 .

Figure 11.12 Vegetation lines 2002 (green) and 2010 (magenta) (Lantmäteriet, 2010)

In Figure 11.13, these vegetation lines have been digitized into GENESIS, where the yellow field corresponds to 2002 and the thin red line 2010. The study area has the same extension as in the calibration and no model parameters have been changed. The only visible change is the new structure placed in the northern part of the area (right) in light blue. It represents the concrete slabs that were constructed in 2006.

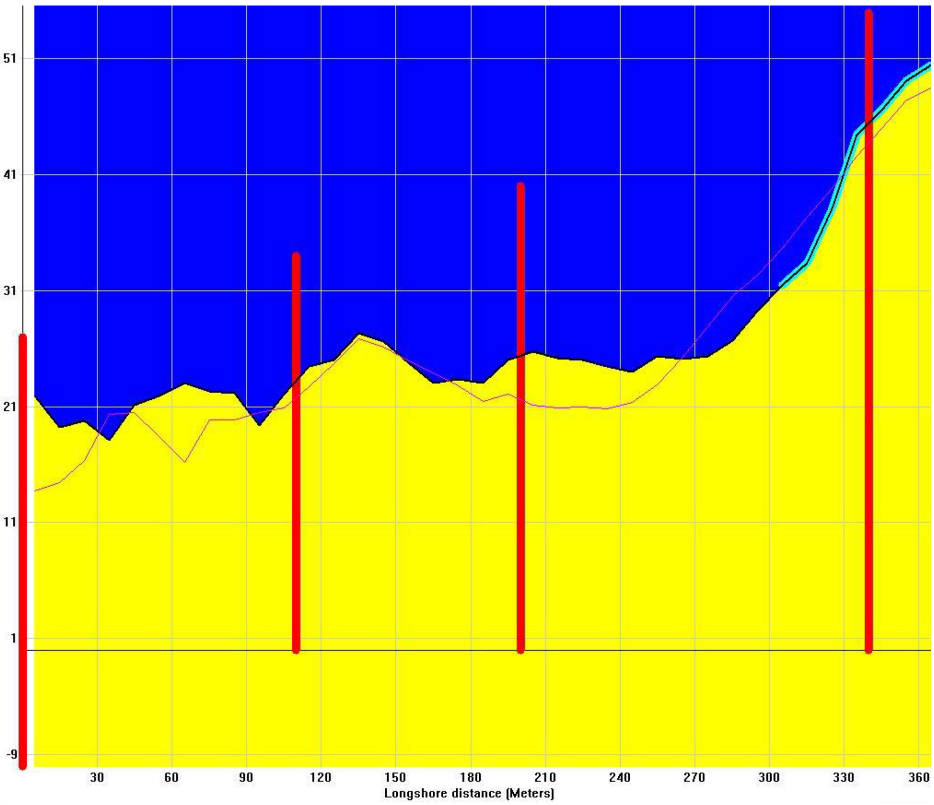

Figure 11.13 GENESIS setup for model validation

When simulating with the calibrated model during this independent time period, the result obtained is presented in Figure 11.14. It shows that the calculated shoreline tends to follow approximately the same trend as the measured in 2010 and the model validation is therefore considered acceptable.

Figure 11.14 GENESIS results after validation process

Once the validation process is over, the model is assumed to be sufficiently reliable and ready to be used for further simulations. It will enable, for instance, the simulation of how different future actions to avoid erosion would work in the area. Consequently, this becomes a helpful tool when evaluating the performance of any kind of proposed project.

12. Simulation of measures against coastal erosion

12.1 General

To evaluate the different alternatives for shore protection all the stakeholders should be consulted. Not only must the chosen solution be efficient in protecting the shore, but it should also take into account environmental and recreational concerns and both public and private sector interests.

This chapter focuses on the engineering part of the coastal protection by showing the simulated results of the shoreline evolution for different measures. Other factors, such as economical and environmental, including an evaluation of the results, are discussed further in chapter 13. The calibrated GENESIS model (see chapter 11) is used to simulate the evolution trend of the shoreline when different measures, presented in the following section, are applied. It is necessary to remind, once again, that the model does not provide how the detailed shoreline would look in the future but gives an estimation of the pattern of shoreline movement, primarily in a qualitative way, not a quantitative. Consequently, the model will be used as a tool to assess which of the proposed actions improve the present conditions best in the area and which ones are detrimental.

For assessment purposes, the different behaviour of the protective measures are simulated for a short-term period (during 20 years, up to the year 2030) as well as for a long-term period (60 years, until 2070). Two different scenarios affecting transport rates, and therefore erosion, are considered (scenario H1 and H2, chapter 9). The first one, called present conditions assumes no changes in wave trend, and wave data between 2000 and 2008 is assumed to be representative. The second scenario includes an increase of winter winds by 13% (see chapter 9), leading to higher waves and presumably increasing transport rates.

This means that for each protective measure four different simulations will be performed:

- Scenario H1, 2030
- Scenario H1, 2070
- Scenario H2, 2030
- Scenario H2, 2070

The measures that show the most desired behaviour during the simulation will be investigated further and a deeper analysis of their costs and environmental impacts will be developed in the next chapter to find out which could be the best overall solution for the study area.

The area to be modelled consists of sections 2 and 3 of the study area, see Figure 3.10.

12.2 Measures applicable in Bjärred

Considering the fact that there is a waste water pipe that needs protection and that a number of properties are located close to the beach, the options of abstention or retreat are not possible. Evaluation of the measures presented in chapter 2 shows that the measures that might be applicable in Bjärred are the following ones:

Detached breakwaters
Detached breakwaters will be evaluated, as they decrease wave energy reaching the shoreline, thus reducing both the longshore sediment transport rate as well as the wave heights, which induces dune erosion. The dimension and quantities are not systematically developed, but a trial-and-error approach is applied and the configuration providing the best performance is presented.

Armouring
Since the erosion mainly takes place as cliff erosion when the runup is high, armouring the beach is the most efficient option to stop it at a specific area. This is what has been done in Bjärred so far and is considered an option also for the study area.

<u>Concrete slabs</u>
The construction of concrete slabs has already been performed for some sections of the beach and an option is to extend it some distance southward to protect the most affected area. As discussed in chapter 2, the major negative effect of this action is that it looks unnatural. A combination of revetments with nourishment on top of the structure might solve this problem. A natural looking beach is created, but if heavy erosion takes place there is a hard structure below that protects the storm water pipe if threatened. This will be simulated using GENESIS, in order to get knowledge of how often the beach will need to be renourished.

<u>Sheet pile</u>
An alternative to the more traditional revetments is to place sheet piles in front of the storm water pipe and let the erosion have its natural evolution in front of it. This will, of course, not stop the erosion itself and when there have been enough erosion, exposing the sheet pile to a risk of failure, the beach will be nourished and the process can start over. The simulation of armouring structures will not differentiate sheet piles from concrete slabs with nourishment in front of it, since they have the same effect on the shoreline.

Nourishment
As a soft protective measure, the goal of beach nourishment is to maintain the environmental and recreational value of the beach. By adding sediments the erosion will not stop but the added sediments will be eroded instead of the original beach material.

Diffracting groin
Groins are an effective hard protective measure to trap sediments transported along the shoreline. As one of the major sources of the shoreline retreat is the cliff erosion, the hypothesis is that groins will not give a satisfying protection for the beach (Brännlund & Svensson, 2005).

12.3 Scenario H1

During the simulations for scenario H1, the wave climate is considered to remain the same as during the last decade implying that there is no increased climate change. As a reference, the shoreline evolution when no protective measure is applied in the area is presented in Figure 12.1 and in Figure 12.2 for 2030 and 2070, respectively.

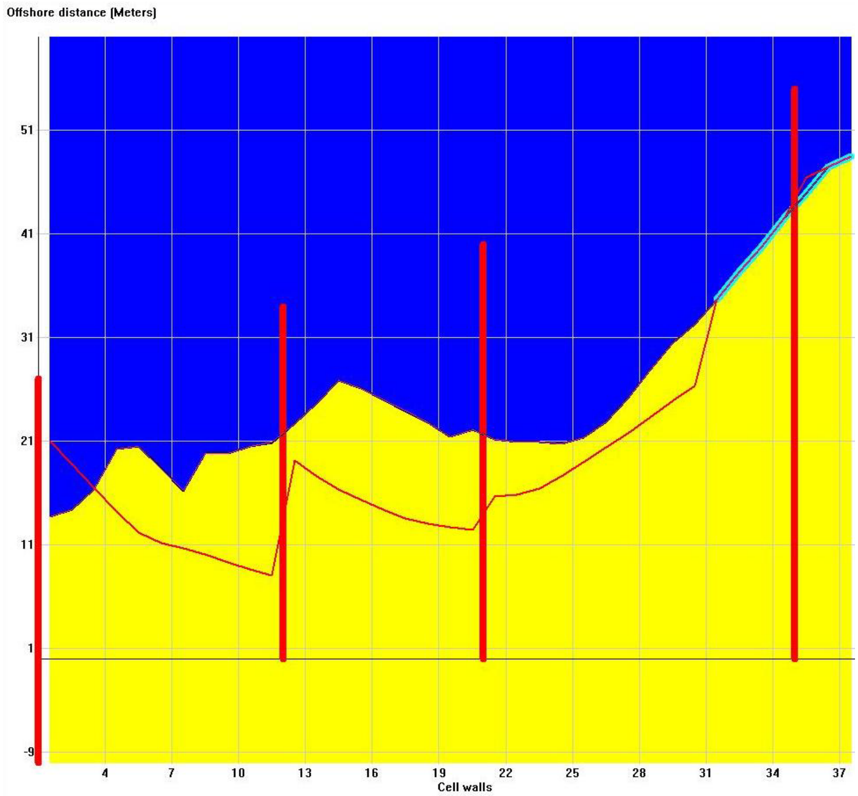

Figure 12.1 Shoreline evolution in 2030 if no action is taken for scenario H1

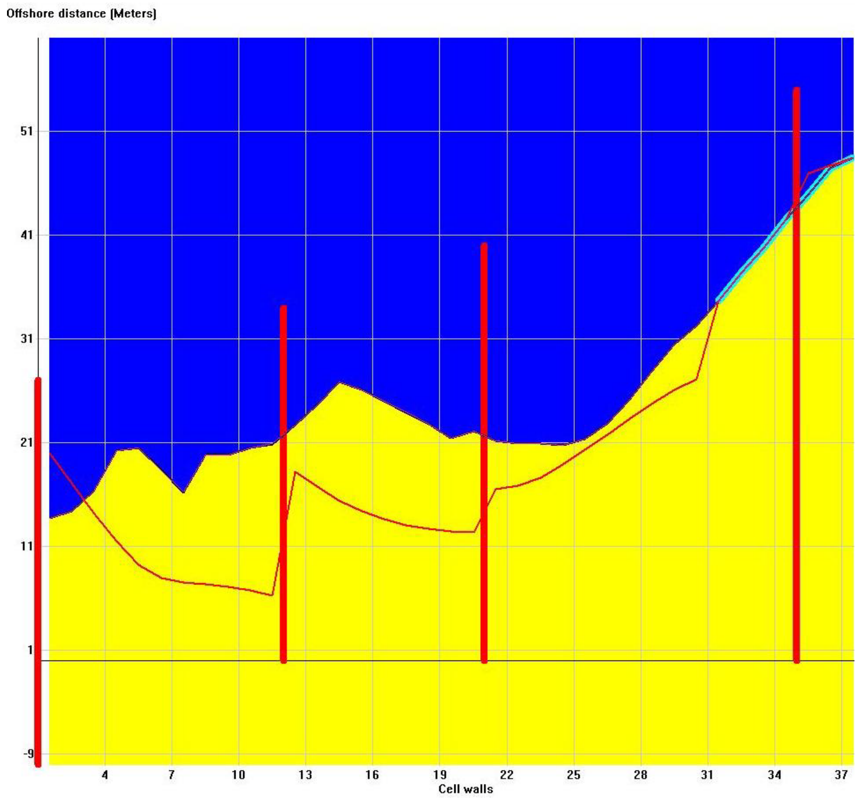

Figure 12.2 Shoreline evolution in 2070 if no action is taken for scenario H1

This is a qualitative illustration of how the shoreline would look without any protection and the influence of the protective measures presented in the previous section is described below.

12.3.1 Detached breakwaters

The first protective measure to be simulated is the placement of detached breakwaters. Their performance by the year 2030 and 2070 are shown in Figure 12.3 and Figure 12.4, respectively.

Figure 12.3 Detached breakwater performance, scenario H1 in 2030

Figure 12.4 Detached breakwater performance, scenario H1 in 2070

As can be seen, two detached breakwaters are located between 95 m and 100 m offshore from the study area. The best performance is obtained when they are placed as in the figures. Both images show that a tombolo is created (note the thin red shoreline). Since a tombolo in most cases is not a desired feature, the simulation in GENESIS stops when it is created. During the simulation this happened after only a couple of years and therefore the figures above are very alike and does not really correspond to either 2030 or 2070. The tombolo itself will totally stop the transport of sediment along the shoreline, implying that the erosion downdrift (left) will increase. This cannot be seen very clearly in the figures but it will affect the sandy beach that is located south of the modelled area. The quick creation of the tombolo is caused by the shallow water depths, which means that only a limited amount of sediment needs to be deposited to create it. General discussion about breakwaters can be found in chapter 2 and evaluation of breakwaters in this area is developed in chapter 13.

12.3.2 Extension of revetments

The second proposed measure to lower the erosion rates is to extend the area where concrete slabs are placed (160 m additional). Since revetments historically have had a satisfying behaviour in Lomma municipality it is likely that an extension will stop the erosion in the study area. From the analysis of the aerial photos and the survey of the study area it can be seen that the southern part of the modelling area shows little signs of erosion. In order to keep the costs low this part might not need protection and there is no need to extend the slabs for the whole stretch. The response of the shoreline for 2030 and 2070 is presented in Figure 12.5 and Figure 12.6.

Figure 12.5 Extended revetment performance, scenario H1 in 2030

Figure 12.6 Extended revetment performance, scenario H2 in 2070

The shoreline in 2010 is represented in yellow, and the concrete slabs are represented in light blue line. The simulation shows a satisfying result both in 20 and 60 years -note the thin red line- as erosion seems to stop where slabs have been placed. One can argue that erosion will occur downdrift of the structure but as can be seen in the figures the southern area, that is not protected, shows a very low rate of erosion. Therefore, this extension seems to perform satisfying result, considering the engineering point of view.

12.3.3 Beach nourishment

The third measure that is studied is a beach nourishment for some extension of the study area. The results are, once again, presented for 20 and 60 years from now.

Figure 12.7 shows the shoreline evolution by 2030 with repeated beach fills every 3 years, each with an added beach width of 4 m.

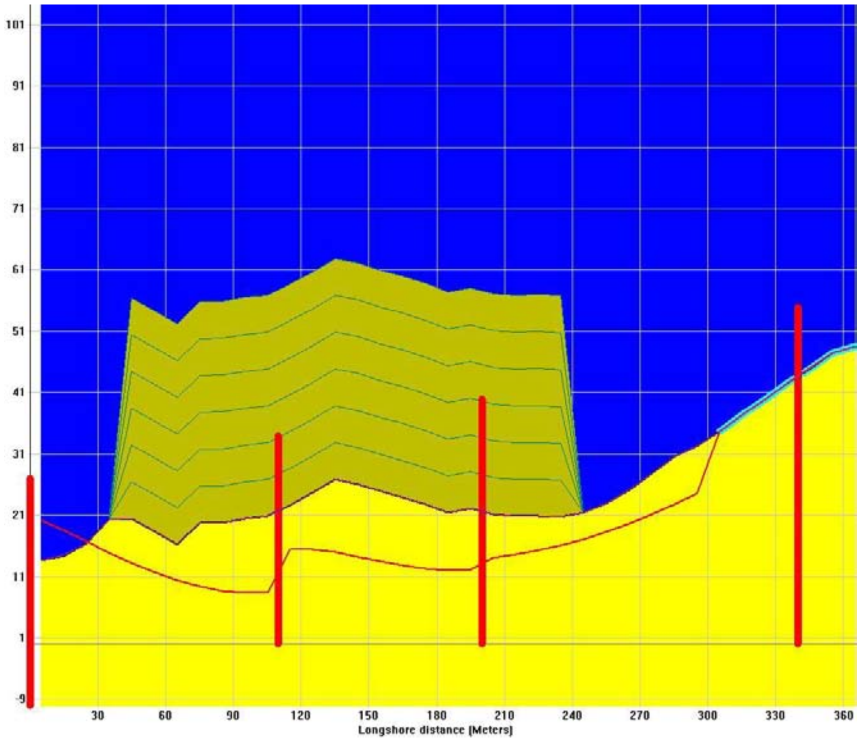

Figure 12.7 Beach nourishment performance, scenario H1 in 2030

At a first glimpse it looks as if the nourishment has no positive nor negative impact on the shoreline, as the thin red line seems to be the same as the evolution when no measure is applied in 2030 (see Figure 12.1). In Figure 12.8 below, though, both shorelines are plotted together, as well as the original shoreline in 2010.

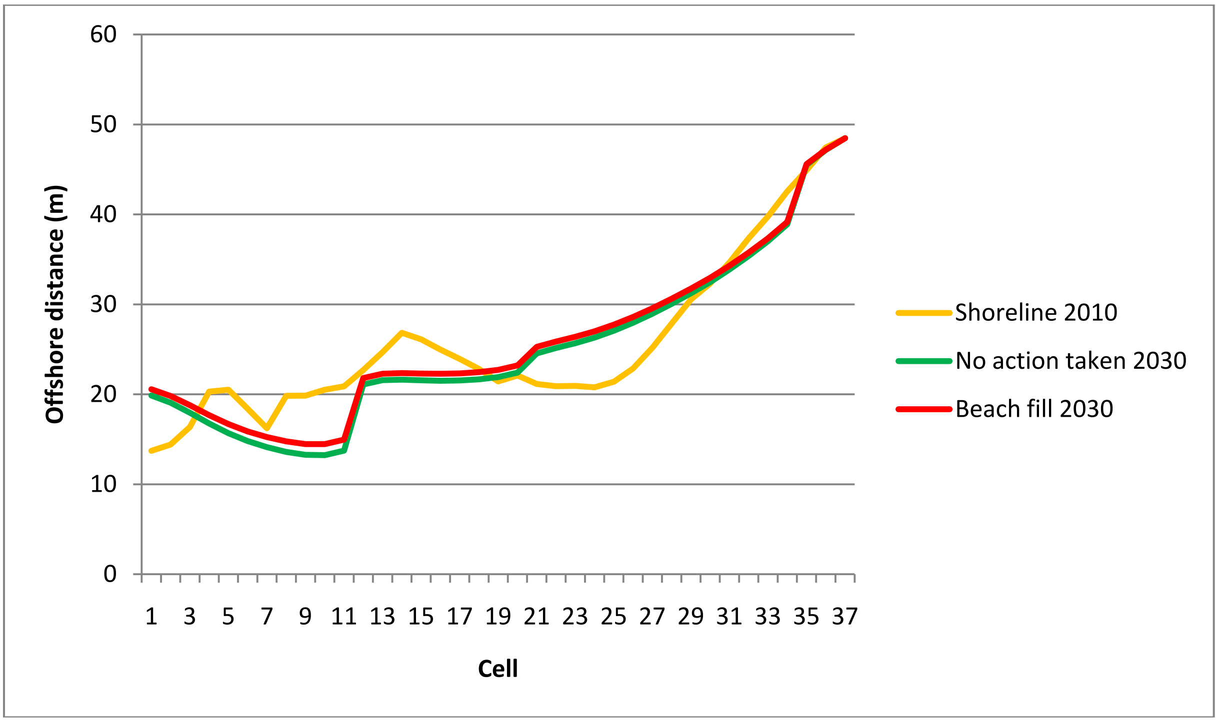

Figure 12.8 Beach nourishment performance

The yellow line is the original shoreline in 2010. The green line corresponds to its evolution if no action is taken, as obtained in Figure 12.1. Finally, the red line is the shore development when beach fill is provided according to this section (see Figure 12.8). It can be seen that there is actually a positive difference, but quite marginal. This means that almost all the added sand has disappeared by 2030. (Hanson, 2010). It can be claimed, thus, that a nourishment project would actually slow down the erosion rate somewhat.

The shoreline evolution by 2070, with the same periodic beach fill (4 m every third year) is not provided as the result is equivalent to the one just discussed (2030).

12.3.4 Extension of revetments with nourishment

A combination of the two previous discussed measures is presented in this section. Figure 12.9 and Figure 12.10 show a positive development of the shoreline by 2030 and 2070, respectively, being really close when just opting for concrete slabs but with the added marginal difference of the beach fill. By adding the beach fills a better aesthetic effect is obtained and a more desired natural appearance is achieved. The environmental and financial aspects of this project are developed further in chapter 13.

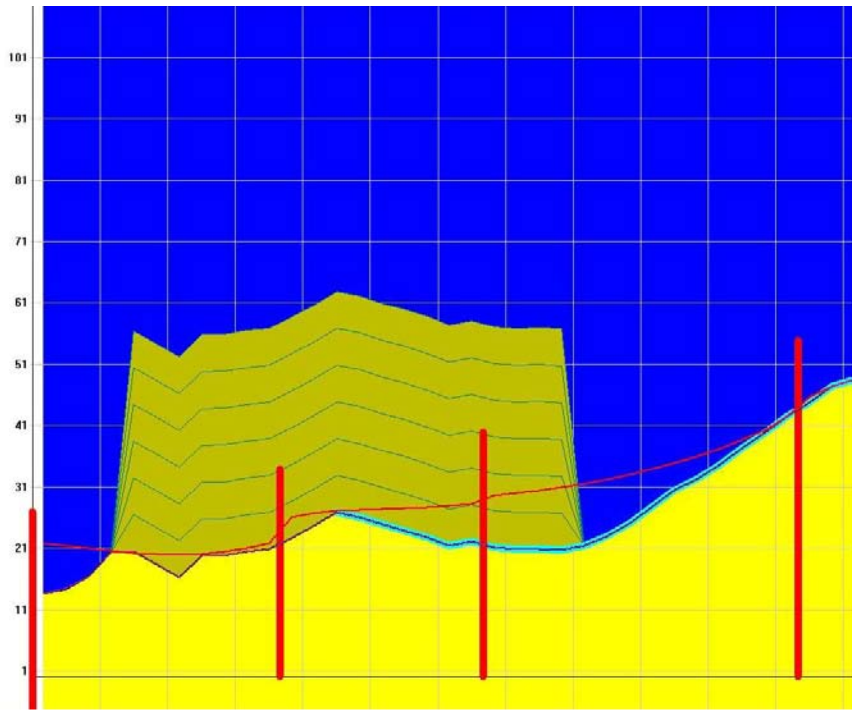

Figure 12.9 Beach nourishment with extended revetment performance, scenario H1 in 2030

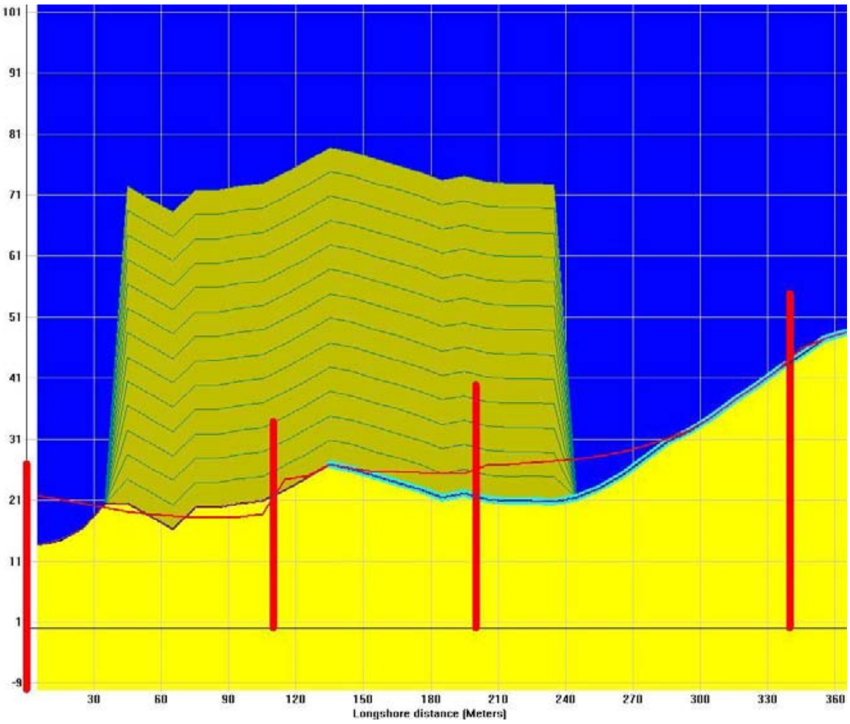

Figure 12.10 Beach nourishment with extended revetment performance, scenario H1 in 2070

12.3.5 Diffracting groin

The last simulated protective measure is a system of two diffracting groins to trap the sediment moving alongshore. Figure 12.11 demonstrates that their placement would result in a significant accumulation of sand.

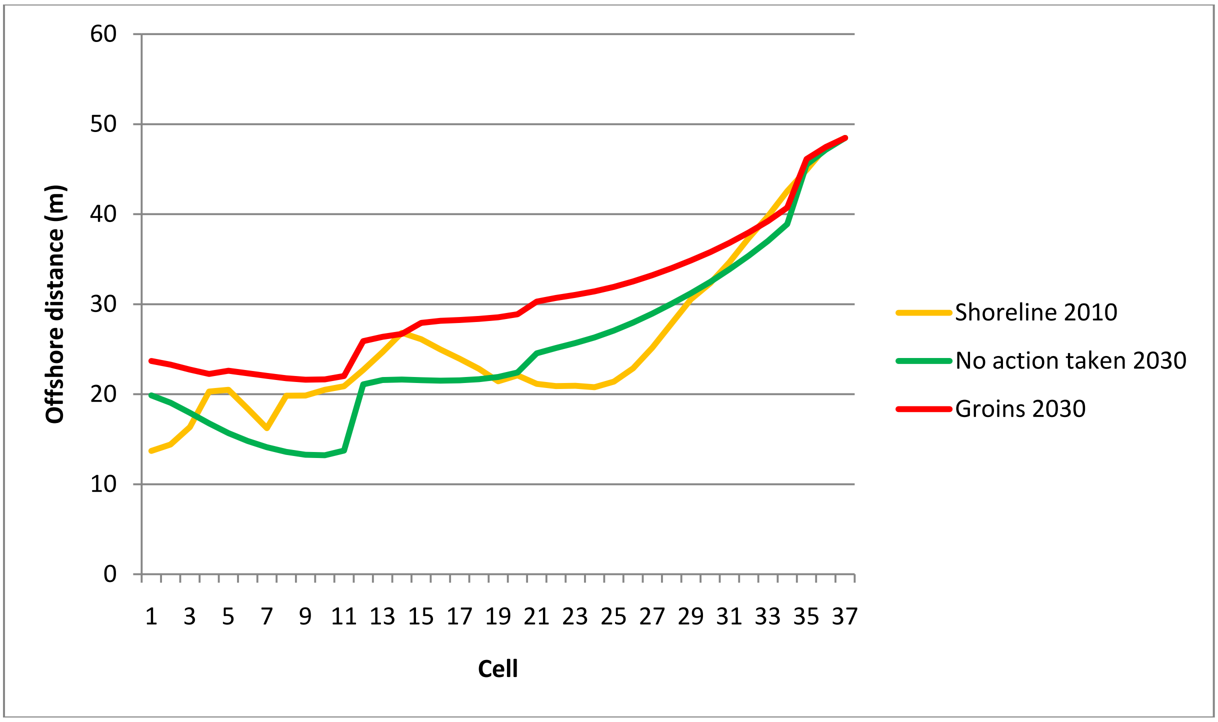

Figure 12.11 Groin performance, scenario H1 in 2030

The figure shows that they provide an improvement (red line) compared to the present conditions (yellow line). The results for 2070 are not shown as they are equivalent to the analysis for 2030.

Nevertheless, this accumulation can cause erosion in downdrift areas as the sediments will be trapped between the groins. Another point to take into account is that groins are only able to trap longshore transport. This means that they cannot provide protection against cross-shore sediment transport. As storm events lead to significant erosion in the study area this measure will not be taken into account for further analysis even though the simulations show good results.

12.4 Scenario H2

The second scenario, also known as H2 (see Chapter 9), being considered is an increase of winter winds during December, January and February by 13%. Thus, wave data from the last decade is modified and run during 20 and 60 years to get results for both short- and long-term assessment for each considered protective measure.

12.4.1 Detached breakwaters

Just like for the scenario without increased winds (H1), a tombolo is created and the simulation is stopped before reaching 2030. This happens regardless of if the simulation is set to run until 2030 or 2070 and thus, only one figure is shown below (Figure 12.12).

Figure 12.12 Detached breakwater performance, scenario H2 in 2030 and 2070

A notable difference from scenario H1 is that the downdrift erosion seems to be of greater extent in this case. In addition to the tombolo created by the northern breakwater, the southern breakwater is the cause of a salient (around 120 m from origin on the *x*-axis) and erosion is present downdrift of the salient. From just the visual analysis it can be seen that a breakwater seems to give less satisfying result compared, for instance, to the extension of the revetments for H1.

Because of the less negative effects that detached breakwaters have regarding the environmental and recreational values of the shoreline, further investigation concerning costs and environmental impact will be performed even though the engineering performance is not satisfying.

12.4.2 Extension of revetments

The results from the simulation of extended concrete slabs, but with increased winds, can be seen in Figure 12.13 and Figure 12.14.

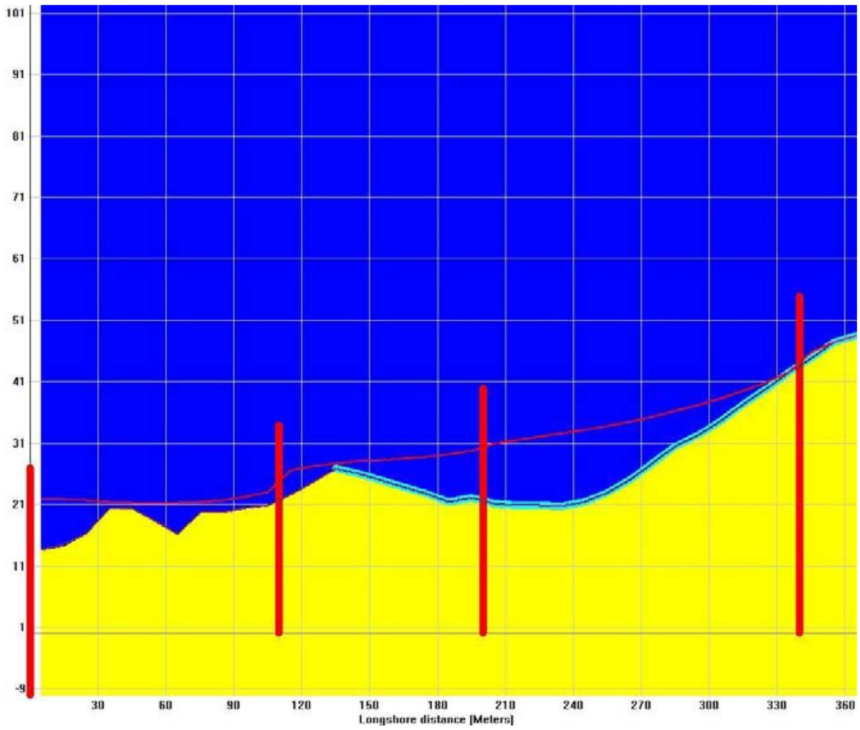

Figure 12.13 Extended revetment performance, scenario H2 in 2030

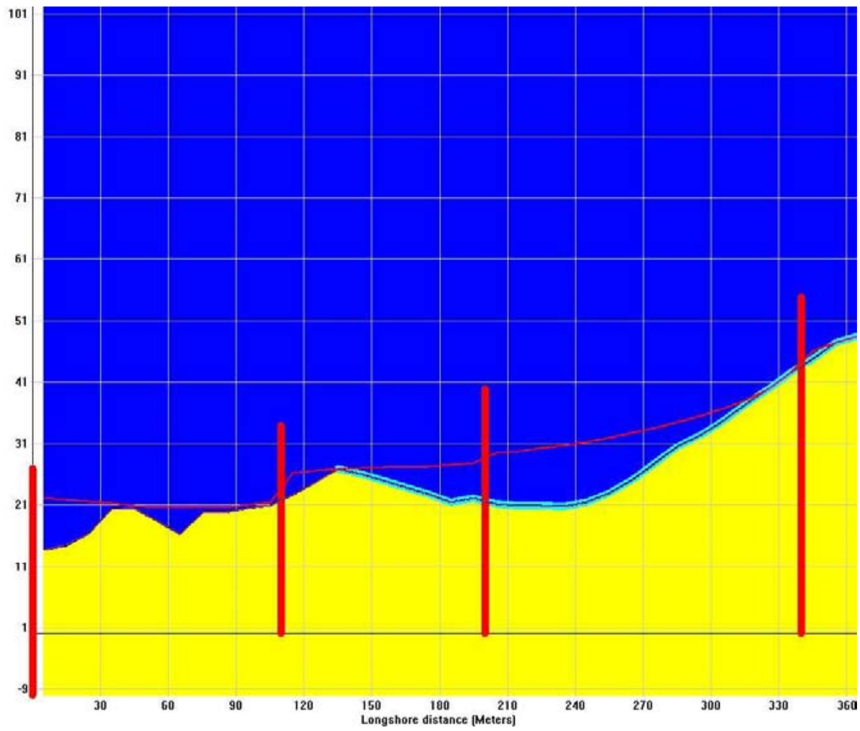

Figure 12.14 Extended revetment performance, scenario H2 in 2070

The results for this scenario are very like the ones for scenario H1. The main difference is that there is some accumulation on the southern unprotected part of the area. This is due to the increased wave heights, which has induced more cliff erosion than in scenario H1, releasing sediment from the cliff to the beach berm. Since the movement is so little it is likely that this will have little effect on the cliff, and is therefore neglected in future analysis.

12.4.3 Beach nourishment

Results regarding shoreline response due to beach fill projects of adding 4m beach width every 3 years can be seen in Figure 12.15 by 2030.

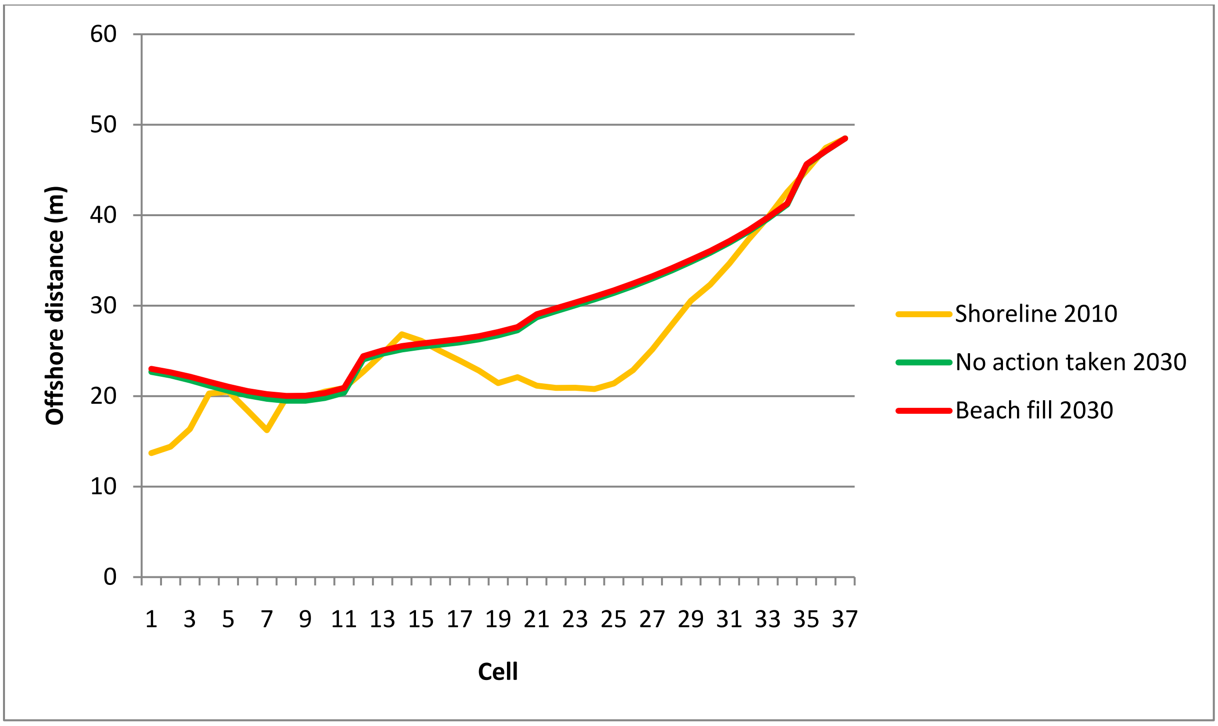

Figure 12.15 Beach nourishment performance, scenario H2 in 2030

As for scenario H1, the beach fill project just provides a little better performance compared to if no action was taken. As the performance is very similar for the year 2070, no figure is included.

When compared to scenario H1, beach nourishment presents a significant better performance when climate change is supposed to occur (scenario H2). The reason is similar to the explanation in section 12.4.2, meaning that the longshore transport will not be strong enough to transport away the increase in released material due to the impact of higher waves.

12.4.4 Extension of revetments with nourishment

Figure 12.16 and Figure 12.17 offer the simulation results for 2030 and 2070 when the two previous protective measures are combined.

Figure 12.16 Beach nourishment with extended revetment performance, scenario H2 in 2030

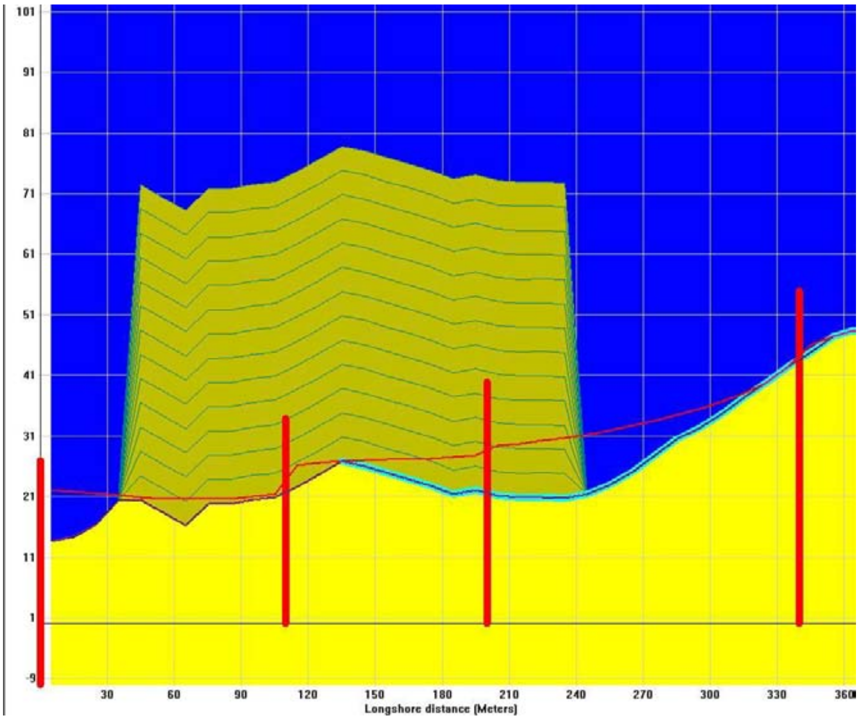

Figure 12.17 Beach nourishment + extended revetment performance, scenario H2 in 2030

Similarly to results for H1, this combination enhances a positive development regarding the evolution of the shore, and thus will be considered as an option for the study area.

12.4.5 Diffracting groin

The last simulated measure is the installation of groins by 2030 (Figure 12.18). These seem to efficiently trap longshore transport of sediments in the area, and even providing a better performance for scenario H1 due to higher wave impact. In accordance with what was already discussed in section 12.3.5, groins do not seem to be an appropriate measure for the study area, and thus will be dismissed.

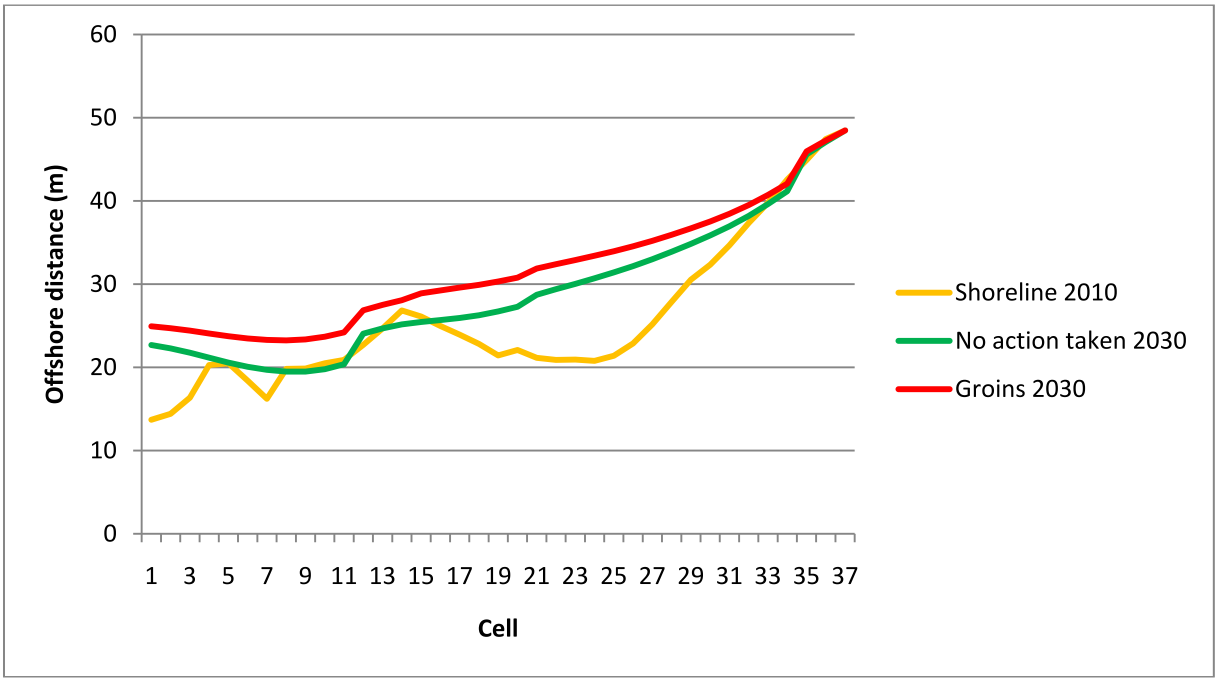

Figure 12.18 Groin performance, scenario H2 in 2030

12.5 Summary of the simulation results

In Table 12.1, the results of the simulations regarding each of the protective measures are presented. As already mentioned, the focus in this chapter is only to determine whether the shoreline exhibbits a favourable evolution compared to the present situation or not.

Table 12.1 Summary of simulation results

Protective measure	Improved evolution
None	no
Breakwater	no
Revetments	**yes**
Beach nourishment	**yes**
Revetments with nourishment	**Yes**
Diffracting groins	Yes, but not cross-shore

Accordingly, the ones who have provided improved results are the placement of revetments, beach nourishment, and a combination of them. These options are further analysed in Chapter 13 together with the detached breakwaters. Groins did work according to the simulations but since the major source of the erosion is from the cliff, thus cross-shore erosion, groins are assumed not to perform satisfyingly in reality. Even though the detached breakwaters did not show a satisfactory performance, it will be discussed further in order to provide background information for the decision makers.

13. Assessment of measures against coastal erosion in Bjärred

13.1 Integrated planning and management

As discussed in Chapter 4, the rate of shoreline retreat in Bjärred during the last decades has been alarming leading to economical losses as well as environmental degradation. There are several reasons stressing the necessity of fast actions. The priority issue is the waste-water pipe immediately behind the beach cliff that needs to be effectively protected from erosion induced by storm events. It is also an area with marked recreational values, as locals use the path next to the beach to walk while enjoying the fresh sea air and some of them even have boats at the piers scattered along the shore. The area should, in addition, maintain its environmental diversity, as different types of vegetation grow naturally along the beach. There are some houses, as well as a path right behind the cliff, which need to be protected from overtopping if a future sea water level rise will occur.

Thus, many sectors are interested in providing protection to the shoreline, each of them having their own motivation. The municipality, neighborhood, builders, fishermen, and ecologist groups, among other stakeholders, ought to be integrated into the planning process. They should understand the present status of the area, and recognize problems and shortcomings. Together, they should agree to a strategic action plan, which that will take the area from the present situation towards a future solution. This includes which type of action to be taken, which are the resources required, the work plan, responsibilities and monitoring, as well as the public and private sectors partnership.

This chapter considers the evaluation of the proposed protective measures from both an engineering point of view as well as an environmental point of view. The measures that have shown an improvement of the shoreline response when being simulated are taken into account and evaluated further. Considering future sea level rise, a manual evaluation of all structures will be performed parallel to the results from the simulations in order to provide the best possible recommendations.

13.2 Evaluation of different measures

The previous chapter discussed the physical performance of the simulated measures used in GENESIS, while this chapter focuses on a general evaluation, design parameters and an environmental and economical estimation of each protective measure. Since the simulation of groins did not work in a satisfying manner their performance will not be evaluated. The simulation of detached breakwaters indicated that they will not provide a good protection for the shoreline but since the municipality of Lomma has an interest in evaluating them, they will be included in the following assessment discussion. Extension of the revetments (including both concrete slabs and sheet piles), beach nourishment and a combination of these will also be evaluated below.

13.2.1 Detached breakwaters

Detached breakwaters were proposed as a protective measure for the study area, as they reduce the wave energy reaching the shore. As the simulations results show this reduction of wave energy is too large, as sand accretes up to the breakwater, creating a tombolo and completely stops the alongshore transport. The beach downdrift will thus be starved of sediments, which is likely to lead to higher erosion rates. Due to many uncertainties regarding transport rate, geological features and historical shoreline movement no quantitative estimation of this increased erosion rate downdrift will be made.

According to the simulation results it is not possible to recommend this solution for the area but there are many uncertainties that need to be further investigated if the decision makers find detached breakwaters an option. In order to provide some background information that a decision can be based on, a cost and environmental impact assessment will be performed.

13.2.1.1 Design parameters and construction

A detached breakwater normally consists of rocks or artificially created concrete structures, such as tetrapods or dolosse. They can be placed either randomly or uniformly, building up the structure (USACE, 1984). As discussed in chapter 3, a detached breakwater is often part of a system of many structures, in this case two. There are relatively not many shore protection projects where the solution has been detached breakwaters and (Rosati, 1990) discusses the fact that lack of design guidance is a major factor why so few breakwaters have been constructed in the US. When designing a detached breakwater there are many parameters to take into account, such as erosion on downdrift beaches, formation of rip currents, erosion at the foot of the structure and the most important one, sediment accretion behind the breakwater. There are many uncertainties regarding these parameters, making the design a difficult task. Due to these many uncertainties further investigation of the area is needed before designing the breakwater.

13.2.1.2 Cost estimation

When constructing a detached breakwater the costs may differ very much depending on if the excavator must be placed on a floating structure, like a ship, or might be able to stand on the beach (Pilarczyk & Zeidler, 1996). In the case of Bjärred, if a detached breakwater is considered, it must be placed a fairly long distance from the beach, meaning dredging is needed to be performed from a ship. This will make the construction expensive and render this option less attractive. The cost of material is independent on the placement of the excavator but may differ depending on what material is chosen and the availability of the material. There are some quarries in the vicinity, which makes the transport length short reducing the total cost of the material. Even though a complete design, due to the above mentioned reason of lacking data, is not possible an estimation of the costs will be performed. The estimation will be based on the assumption that the breakwater is located at 1 m depth with its top 0,5 m above the mean sea water level and that two breakwaters, each of 50 m length, are built. The cost is based on data from Hartzén (2010) and is not definite but a first cost estimate. The maintenance cost is based on an assumption that seaweed needs to be removed from the breakwater each spring and other maintenance costs to keep the structure in good condition.

Cost estimation	
Total cost per meter structure	4500 SEK/m
Total length	2*50 = 100m
<u>Total initial cost</u>	<u>100*4500 = 450 000 SEK</u>

The normal life time of a structure like this is 50 years, meaning the cost per year is given by:

450 000 / 50 =	9 000 SEK/year
Maintenance cost	15 000 SEK/year
Total cost per year	**24 000 SEK/year**

13.2.1.3 Environmental impacts

Since it is one of the desired effects, detached breakwaters will reduce the flow rate along the shore, causing less sediment to pass downdrift. As mentioned previously this is likely to cause erosion downdrift of the structure.

One negative impact that can be seen quickly is that breakwaters in general trap seaweed and debris (Mangor, 2004). In Lomma bay, there is much seaweed that normally is deposited on the shore but with a detached breakwater might get trapped at the structure. The municipality fears that the trapped seaweed will cause an unpleasant odor, reducing the recreational values of the area (Persson, 2010). If a tombolo is created the breakwater not only causes a barrier but is also available to reach from land. This might cause injuries to mainly children, playing at the structure. (Mangor, 2004)

Many of the properties in the area have a high value, due to- among other things- the ocean view. Since the breakwaters are an unnatural addition the environment a big disadvantage is that they may change the house owner's natural ocean view, which may lead to a lowered property value. This should not be neglected since it might cause a negative public opinion. Another negative effect of the structure is that it would act as an obstacle for the boats in the area, which may also lead to negative opinions.

13.2.2 Extension of concrete slabs

A concrete slab structure is the solution proposed by the municipality as its main concern is to protect the waste water pipe. It completely stops erosion wherever it is placed and therefore is the one that has shown the best accretation potential for the area when being simulated.

13.2.2.1 Design parameters and construction

The structure consists of concrete slabs with voids as the visible layer. Well packed clay exists as foundation right underneath, which will provide the desired slope as well as enhanced plant growth through the slab voids. A geotextile is placed below to retain the underlying original soil and thus protect from erosion.

The concrete slabs will be of the same dimension as the test area built in 2006 (0.4 x 0.4 x 0.1 m). The extension of the structure is proposed to be 160m along the shoreline and 20 m landward to keep the same width as the test area.

Note that when placing the concrete slabs they must be laid manually, which is a time-consuming activity and physically hard for the worker. This might be a factor to consider when deciding what protection to build. (Ramböll, Erosionsskydd i Bjärred- utvärdering av skyddsåtgärder, 2007)

13.2.2.2 Cost estimation

The 100 m test section, constructed in 2006, had a total cost of 1,2 million SEK, which is equivalent to 12 000 SEK/m length or 600 SEK/m^2. The study area has the same appearance as where the test section was built and therefore the same figure of **12 000 SEK/m** is used to approximate the cost of this protective measure. As mentioned previously, caution must be taken when packing the underlying layer of clay. Since the test section shows some signs of undermining (deeper discussed in Chapter 3), the constructor needs to plan more carefully and spend more resources on the clay layer. Thus, the cost of 12 000 SEK/m might be underestimated and the actual cost may be slightly higher. Calculations are based on the 12 000 SEK/m though. When designing a structure one must always take into account the life time of the structure itself, which in this case is set to be 50 years. The yearly maintenance costs are assumed to be around 3000 SEK per year.

Cost estimation:

Length of shoreline in need of protection	160 m
Cost per meter	12 000 SEK/m
<u>Total initial cost</u>	<u>1 920 000 SEK</u>

The normal life time of a structure like this is 50 years, meaning the cost per year is given by:

1 920 000 / 50 =	38 400 SEK/year
Maintenance cost	3000 SEK/year
Total cost per year	**41400 SEK/year**

13.2.2.3 Environmental impacts

Concrete slabs are able to stop erosion efficiently, but they cannot absorb all the wave energy. Some of it can be reflected, causing interference which may lead to increased wave energy on adjacent unprotected beaches, leading to an increased erosion downdrift. In addition to the increased wave energy, the supply of sediments is almost completely inexistent, leading to even greater erosion rates downstream. (Bush, Pilkey, & Neal, 1996)

It is therefore recommended to nourish the affected downdrift area with sediments to maintain the beach profile. In this case, Lomma municipality has a permit to nourish the sandy beach south of the study area and will do so during spring time (Persson, 2010).

A negative effect of the placement of concrete slabs is the impact on the recreational values and the environment. The natural appearance of the beach is destroyed by their placement, as well as being detrimental for the natural growth of vegetation. A flora inventory on the erosion protections of Lomma municipality performed in 2009 shows that up to 6 meters from the water line a maximum of 40% of the revetments are covered by vegetation. The inventory further shows that vegetation and animals are limited to the voids in the slabs and the conditions for burrowing animals are very poor. (Eco-e-Miljökonsult, 2009)

A strategy to overcome these negative impacts is to employ more soft protection measures. In order to make the concrete slabs a more natural part of the environment one recommendation is to add new sediment over the structure as well as planting new vegetation to keep the sediment in place; in other words, provide nourishment on top of the concrete slabs. A natural beach will be formed but the hard protection measure will still be present beneath the beach. The costs of the project will be higher compared to if no nourishment is performed but it might be well-invested money in order to overcome some of the negative effects. One problem, though, is where to take the plants needed to stabilize the sediments, since one wants to have the natural vegetation (Persson, 2010).

13.2.3 Beach nourishment

As a soft protective measure, the goal of beach nourishment is to maintain the beach environmental and recreation values. By adding sand to the area in need of protection the erosion still goes on but the mechanisms affect the added sand instead of the original beach. Both longshore and cross-shore transport affect the erosion rate and therefore how often the beach needs to be re-nourished. Since simulations in GENESIS only can provide information about longshore transport, the model is not completely able to evaluate performance of beach fill projects when cross-shore transport is present. For this study it is assumed, though, that the cross-shore transport is eroding the cliff, making sediments available for longshore transport and thus GENESIS provides a good estimation of the shoreline response. (Hanson & Kraus, GENESIS: Generalized model for simulating shoreline change. Report 1: Technical Reference, 1989)

13.2.3.1 Design parameters and construction

The nourishment project proposed as a protective measure for the study area aims to recover the same beach profile as in 1978. The size of the beach in 1978 was wide enough to provide both a recreational value and place for people to hold their leisure boats. Accordingly, the beach berm must be widened up to approximately 4 m (see chapter 4, aerial photograph analysis). This widening will provide enough storm protection as well as a positive impact on the recreational value. The berm height should be representative for the natural profile, and its elevation is, from the surveying in 2010, found to be 1.4 m. The cliff height has, for the conditions of today, proved to be enough to avoid overtopping, hence no nourishment will be provided on top of the cliff. Instead, vegetation growth should be promoted to increase the cliff resistance against wave impact. Regarding the future scenarios of increased sea level the situation might look different and an increased cliff height should be considered.

A schematic overview of a nourishment project is provided in Figure 13.1, where the desired design profile is represented by the dashed line. Note that this sketch is very general and the scale is different for the specific study area of Bjärred.

Note that the depth of closure is estimated to be 4 m and 1000 m offshore (chapter 3 and 5), instead of the 360 m shown by the sketch in Figure 13.1.

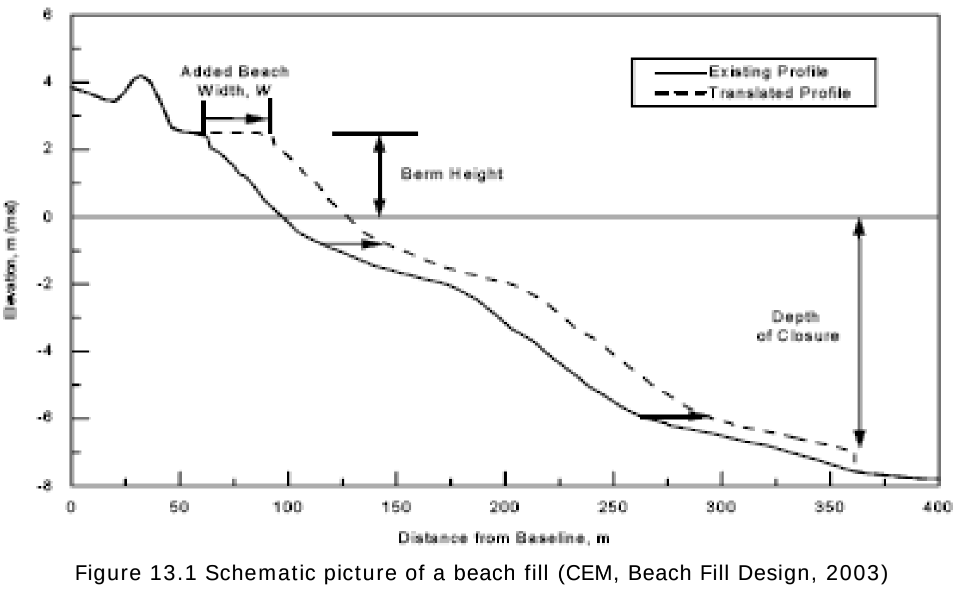

Figure 13.1 Schematic picture of a beach fill (CEM, Beach Fill Design, 2003)

Construction machinery, such as a bulldozer, can be used to place the total necessary amount of sand. The design profile will be achieved naturally some time after the placement of sand.
The added sediment characteristics should be as similar as possible to the sediment present at the site, both in order to promote the compatibility between them as well as having a better prediction of the project performance (CEM, Beach Fill Design, 2003). During previous nourishment projects in Lomma municipality, sediment has been taken either from a land source in Järavallen, Kävlinge municipality, just north of Bjärred or from the Dockan area in Malmö. The cost estimation below will be based on the assumption that sand will be taken from the same sources and to the same costs as in the previous projects in Lomma municipality (Persson, 2010).

The cross-sectional fill volume requirement is a key factor when calculating the estimated costs of the project. The added sand dimension (A_F) is estimated to be similar to the original sediments (A_N) and thus an intersecting profile (Figure 13.2) is most likely to occur. (Dean R. , Equilibrium beach profiles: Characteristics and applications, 1991)

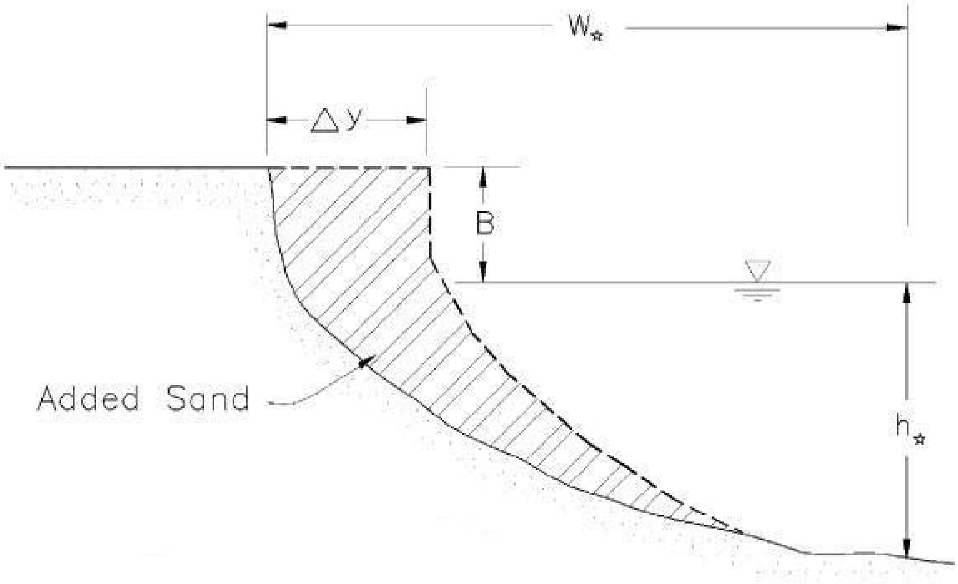

Figure 13.2 Intersecting profile obtained from a beach nourishment project. (CEM, Cross-shore Sediment Transport Processes, 2002b)

The berm height above the mean sea water level, B, is found to be 1,4 m and the desired berm width is Δy=4 m. The closure depth is h_*=4 m and the width W_* is obtained through Equation 13.1.

$$W_* = \left(\frac{h_*}{A_N}\right)^{3/2} \qquad \text{Equation 13.1}$$

For intersecting profiles, the non dimensional volume of sediment, V', needed to achieve the beach berm advancement (Δy) is given by Equation 13.2.

$$V' = \Delta y' + \frac{\frac{3}{5B'}(\Delta y')^{5/3}}{\left[1-\left(\frac{1}{A'}\right)^{3/2}\right]^{2/3}} \qquad \text{Equation 13.2}$$

The non dimensional parameters for the nourishment project are given below.

$A' = \frac{A_F}{A_N}$ Equation 13.3

$\Delta y' = \frac{\Delta y}{W_*}$ Equation 13.4

$B' = \frac{B}{h_*}$ Equation 13.5

The total volume necessary for the project is calculated using Equation 13.6.

$V = V'BW_*$ Equation 13.6

By substituting all the specific input parameters the amount of sediment fill is **6.3 m^3/m**

13.2.3.2 Cost estimation

According to Persson (2010) the cost of nourishing a beach of this size is around 250 SEK/tonne sand including all construction costs. As calculated in the previous section, the amount of sand needed is 6,3 m^3/m length. When sand is added to the beach it will take some time for the beach to reach its final profile shape. When this is done, much of the added sand has been transported offshore, and is the main reason for having that much sand addition to achieve the extra 4 m width. Since beach nourishment does not stop erosion the beach will continue to erode and new nourishments need to be performed regularly. Simulations performed in chapter 12 are based on a time interval between two nourishments of 3 years.

Amount of sand needed	6,3*160 m = 1008 m^3
Sand density	1600 kg/m^3
Total weight	1008*1,6 = 1613 tonnes
Cost per tonne	250 SEK/tonne
Total initial cost (every 3 years)	250*1613 = 403 200 SEK
Total cost per year	**134 400 SEK/year**

13.2.3.3 Environmental impacts

As mentioned above, if the selected fill sediment has a similar grain size as the original sediment the beach profile will maintain its original shape better. The amount of silt and clay should be as little as possible (max 10%) as it leads to an increased turbidity in the water and thus having some impact on beach fauna (Walton & Purpura, 1977); (Dean R. , Principles of Beach Nourishment, 1983); (NRC, 1995). Nevertheless, the fine material should disappear a short time after the nourishment has been placed.

After nourishment, sediment could be up to three to four times more compacted (Ryder, 1989) due to the heavy machinery being used to arrange the sand. This can crunch and hamper the movement of fauna along the shore (Rice, Greenwood, & Joyce, 2001). Vegetation at the shoreline, which helps maintaining the dunes, could also be destroyed during the nourishment. Bulldozing also involves obtaining less sorted sediments as well as to a higher percentage of coarse sands and gravel-sized particles (Lindquist & Manning, 2001). This leads to a steeper profile, which could increase wave energy (Kaufman & Pilkey, 1983) and thus affecting the environmental conditions and reducing the fauna diversity.

A nourishment project leads to less environmental impact compared to revetments due to the natural environment created. The more similar the characteristics of the added sediment are to the original sediment at the site the less impact the project has on the environment.

13.2.4 Construction of sheet pile wall

Construction of sheet piles in front of the waste water pipe would provide a hard protective measure, implying that the erosion cannot reach the pipe, but it may at the same time prevent a natural beach from developing in front of the structure. As mentioned in the previous chapter this will not stop the erosion nor slow down the erosion rate, but when the erosion reaches the vicinity of the sheet pile wall, nourishment can be performed, leading to shoreline advance, and the erosion process can start over. This has many advantages, for example, there will be no increased erosion downdrift due to lack of sediment and the pipe is protected at all times, even if extremely heavy erosion would occur.

13.2.4.1 Design parameters and construction

One factor that must be taken into account when designing the sheet pile is the ground water pressure behind the structure. At present, there is ground water seepage onto the shore (Ramböll, Erosionsskydd i Bjärred- underlag för samråd med länsstyrelsen, 2009). There are some proved methods to lead away the ground water if it will lead to problems. Simulations show that, if the shoreline should be kept in the same position, nourishment needs to be performed every three years with the same costs as assessed in section 13.2.3.2 above.

13.2.4.2 Cost estimation

When constructing these sheet piles the easiest way is not to anchor them with any braces but to simply hammer them down into the soil. The costs are based on the assumption that it will be hammered a total of 2.5 times the distance from the mean water level to the cliff height. In this case the cliff height is 3 m meaning the total height of the sheet pile is 7.5 m under the soil surface. The price of material and hammering the sheet pile leads to a cost of around 2500 SEK/m^2. Since each meter of length corresponds to 7.5 m^2 the cost per meter is around 18750 SEK/m. These prices also include the construction of the piles, so that the price per meter is for both material and construction work. It is assumed that the sheet piles are thick enough to prevail the forces from nature during the life time of 50 years, meaning that despite corrosion there will be enough unaffected steel to maintain sufficient strength.

Cost estimation:

Length of shoreline in need of protection	160 m
Cost per square meter	18750 SEK/m^2
Total initial cost	3 000 000 SEK

The dimensioned life time of the structure is 50 years, generating a cost per year given by:
3 000 000 / 50 = **60 000 SEK/year**

13.2.4.3 Environmental impacts

The first factor to consider is the change in flow for the ground water. The sheet pile wall will work as a barrier for the ground water flow and the rates will be affected. If constructing the sheet piles as a totally impermeable layer, the ground water will have to flow elsewhere. Like mentioned above there are some solutions to transport the ground water through the sheet piles, though. When constructing the sheet pile wall some heavy machinery is needed and it will most likely damage some of the vegetation in the area. It is a must to restore the vegetation as soon as the construction is complete. If restored properly the sheet piles will not have much effect on the vegetation in a longer time perspective. Regarding the availability of the beach it will, except for during the construction phase, not be affected by the sheet pile wall. Some burrowing animals might be affected by the sheet piles as they will act as a barrier in the soil.

13.3 Summary of the protective measures

In Table 13.1 are the results presented regarding each of the protective measures. The table only includes the *hard* factors such as costs and improvement during simulations. Note that when calculating costs for revetments in combination with nourishment the cost of each structure is added to the cost for nourishment and no further calculations are being performed. If investigating these measures closer, the cost of the nourishment might differ between the cases with sheet piles or concrete slabs.

Table 13.1 Summary of simulation results and costs

Protective measure	**Improved evolution**	**Cost per year (SEK/year)**
None	no	0
Detached breakwaters	no	24 000
Concrete slabs	**yes**	41 400
Sheet piles	**yes**	60 000
Beach nourishment	**yes**	134 400
Concrete slabs with nourishment	**yes**	175 800
Sheet piles with nourishment	**yes**	194 400

Thus, the alternatives that have provided improved results of the shoreline response are the placement of revetments, beach nourishment, and a combination of them. The detached breakwaters did not provide a satisfying simulation result but as can be seen in the table above, they are a cheap option and it might be worth investigating this solution further.

14. Discussion

The coast in Bjärred has undergone severe erosion during the past century as proved by analysis of aerial photos from the area. Between 1963 and 2002 the shoreline has retreated an average distance of 4.3m and in some spots even moved back 13m. Some parts of the shoreline have retreated 1.2m duirng the past six years (2004-2010). This result indicates a possible increase in the shoreline retreat rate during the last decade. Since the retreat is not homogenous along the whole stretch of the studied shoreline some parts might not be in need of protection. The part just north of Öresundsvägen shows small signs of erosion and might not need to be protected at present conditions. One should note that most of the results obtained are assessed for the worst case scenario, meaning that the effects might be less severe if another scenario happens.

Among the actions that the municipality has taken so far, the only coastal protective measure efficiently working is the placement of concrete slabs on the beach. Other tested structures, such as plastic reinforcement nets have not provided to yield satisfactory results. Since the study shows that erosion mainly takes place during high water levels and storm events a protective measure that armors the beach profile is most likely to give a good result, especially due to the fact that the waste water pipe close to the shoreline needs protection.

Most certainly there will be a future sea level rise, which will induce increased cross-shore transport and profile adjustment. However, the shoreline retreat estimated from calculations using the Bruun rule, 42 and 97 m respectively, should be considered with care. These figures do not take into account any existing structures or the variation in geology of the area. It might be a wakeup call for the decision makers in Lomma municipality, though. If no protective measures are taken the shoreline will migrate landward and destroy valuable properties.

Historically there has been no overtopping of the cliff (or bank) but with increased sea level this will most probably occur. It will lead to flooding of the adjacent road and most likely of the storm/waste water system as well, which might lead to inundation of basements in nearby properties. Direct effects of the overtopping might also be increased erosion in the area since water on the road flows towards the sea and sediment is pushed onshore from the cliff.

The results from the shoreline evolution model should be considered with great caution. First of all, it assumes that sand is always available when calculating longshore transport rates, likely leading to higher quantities of transport than in reality. Its reliability could also be discussed as it can only explain around 50% of the shoreline behavior due to wave attack. In addition, the predicted future scenarios for waves due to climate change have been used for assessment purposes. It is clear, then, that the applications are associated with a high degree of uncertainty. The reader should be reminded that the results obtained from the simulations are mainly **qualitative** and not quantitative and should be used for comparison between different measures and not as an absolute prediction for a certain type of structure. If more accurate results are desired further investigation in the area is needed.

Nevertheless, the model indicates that if no protective measure are adopted the shorelie would continue to erode in the future. The tested structures that seem to improve the present situation are the extension of concrete slabs, the placement of a sheet pile wall in front of the waste water pipe, beach nourishment, as well as a combination of revetments and nourishment.

When considering beach nourishment, it results in the need of a much higher budget compared to the revetments (concrete slabs and sheet piles), mainly due to the maintenance costs associated. This could be taken as an important drawback but, however, periodical nourishment has the main advantage of preserving the beach environmental and recreation values. Besides, the addition of sand provides a continuous sediment buffer to compensate for both cross- and longshore transport.

The placement of concrete slabs involves a lower budget alternative and they should be able to completely stop both long and cross-shore erosion, as they do not allow any movement of sand. Nevertheless, their placement entails significant negative visual and environmental impacts on the beach, as the profile would be modified and they are also likely to enhance downdrift erosion. The natural and visual harmful effect could be reduced by placing vegetation growth through the slabs' gaps or add beach fill material above the revetments.

The construction of a sheet pile wall would also efficiently protect the cliff and thus the water pipe and structures behind the beach. The construction would be more expensive compared to the concrete slabs and it would not stop the longshore drift of sediments as the beach berm would remain unprotected. The advantage though, is that the recreational values of the beach will not be affected.

14.1 Recommendations

It is highly recommended that Lomma municipality should take fast actions concerning the still unprotected beach stretch in Bjärred. The proposal is to opt for a hard structure to efficiently stop the erosion (concrete slabs or sheet piles). Depending on the financial situation a combination with beach fills will give a more pleasant aesthetic appearance of the beach. The best possible results will be obtained if the beach is scheduled for re-nourishment every 3 years but a more economical solution is to plan for re-nourishment every 5 years instead.

The main reason to recommend a hard structure is to protect the waste water pipe adjacent to the shore. If not protected properly, it might break and pollute Lomma bay leading to severe environmental problems. The proposal of sheet piles or concrete slabs with nourishment in front of the structure might entail higher costs but in addition to the increased recreational values of the beach one might also overcome the negative effects of a seawall, such as downdrift erosion.

The cost of nourishment in front of the structures should be weighed against the positive effects it might lead to. More people would seek to spend their weekends and holidays in the area, either sailing or just having peaceful walks along the coast. Some benefits derived from the existence of a sandy beach could compensate for the high costs of sand filling.

Despite the measure that will be taken in the area the municipality should definitely consider increasing the capacity of the waste/storm water system in the area to prevent future flooding due to overtopping. By working proactive with this and increasing the capacity before it is too late, some problems might be prevented. An alternative to increase the capacity of the existing system is to construct a new system for draining the water on the road.

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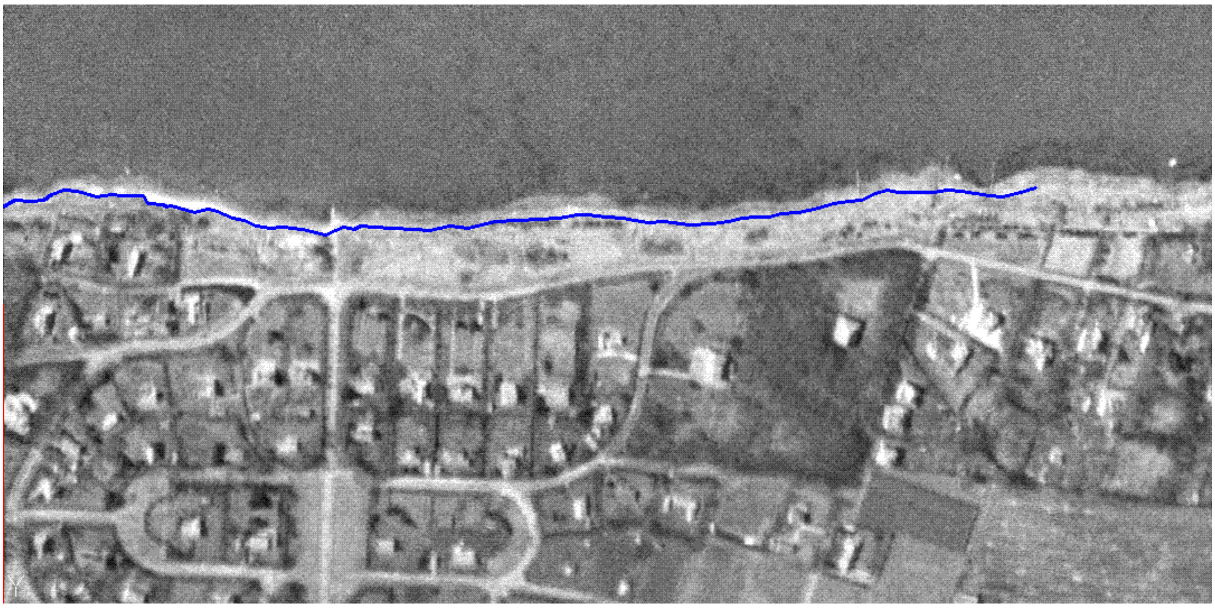
Appendix I, Aerial photos and vegetation lines

1939

1955

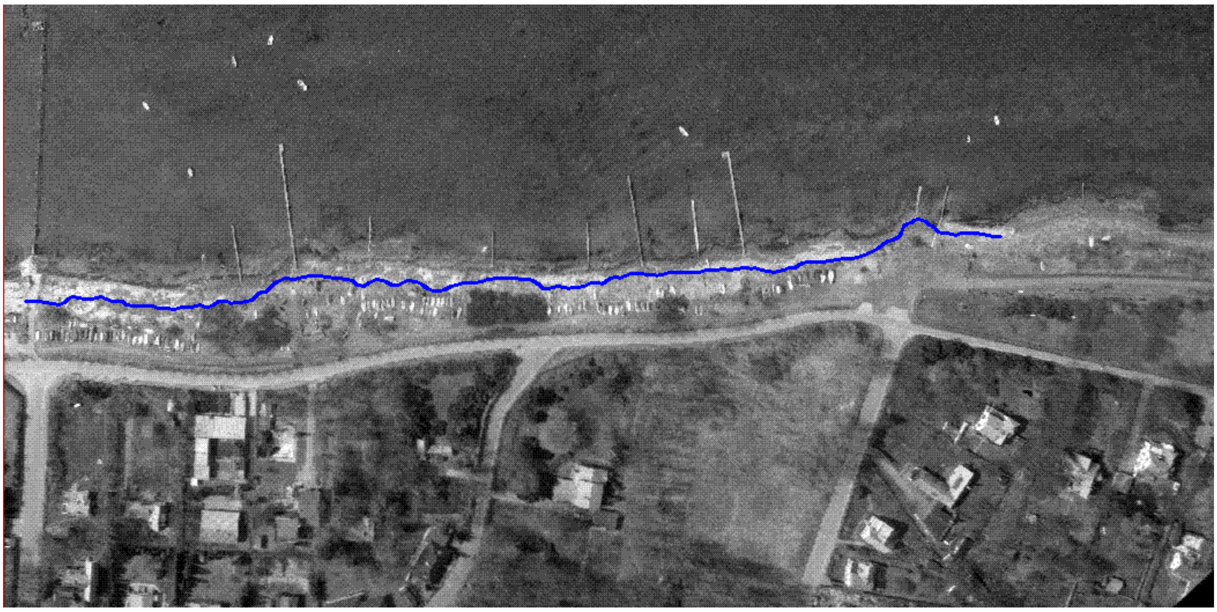


1963

1973

1978

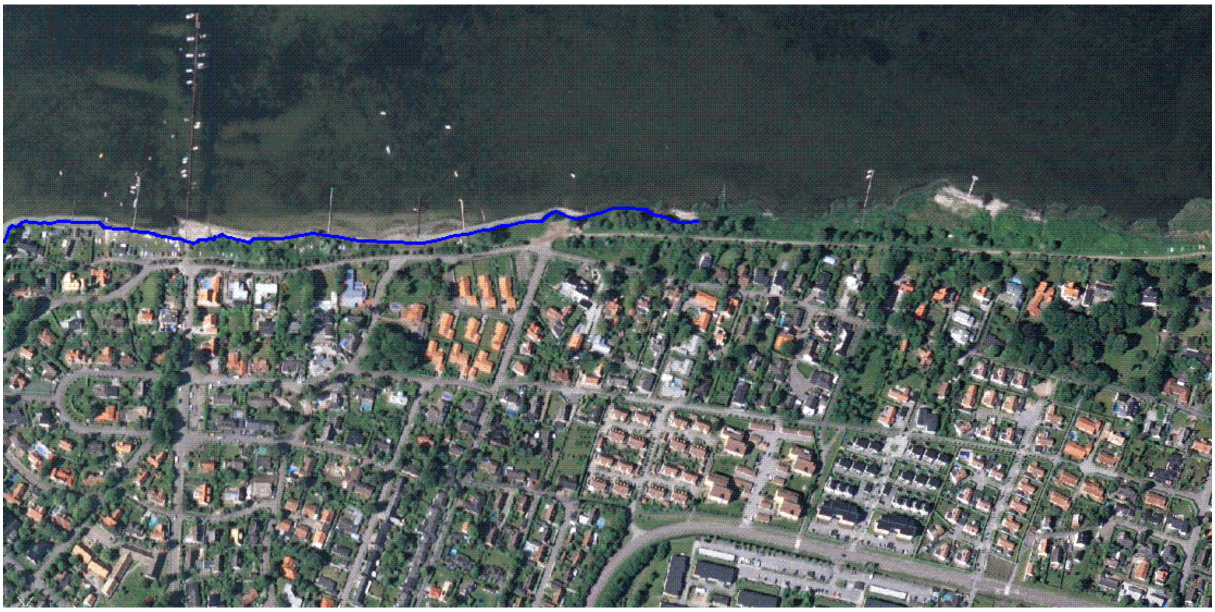
1985



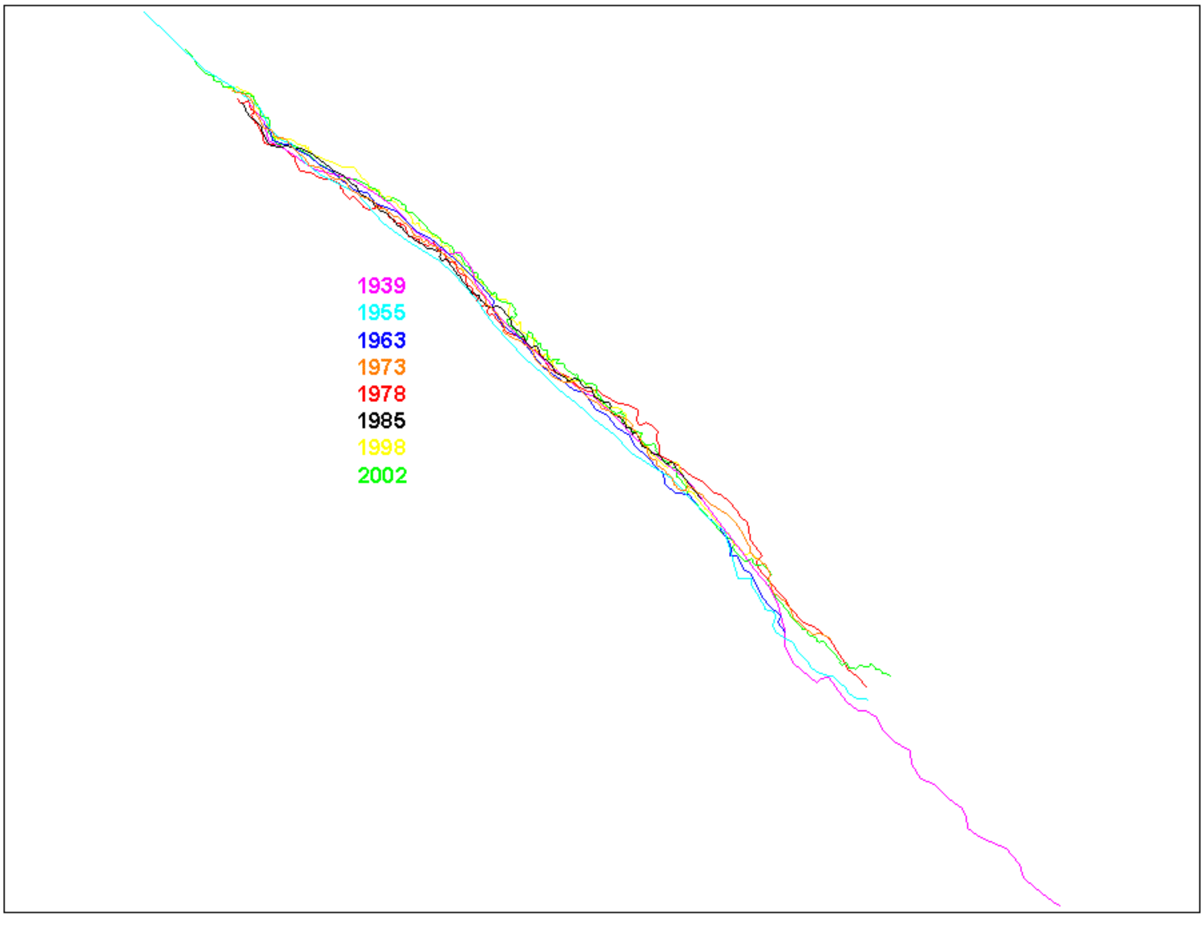
1998

2002

2010



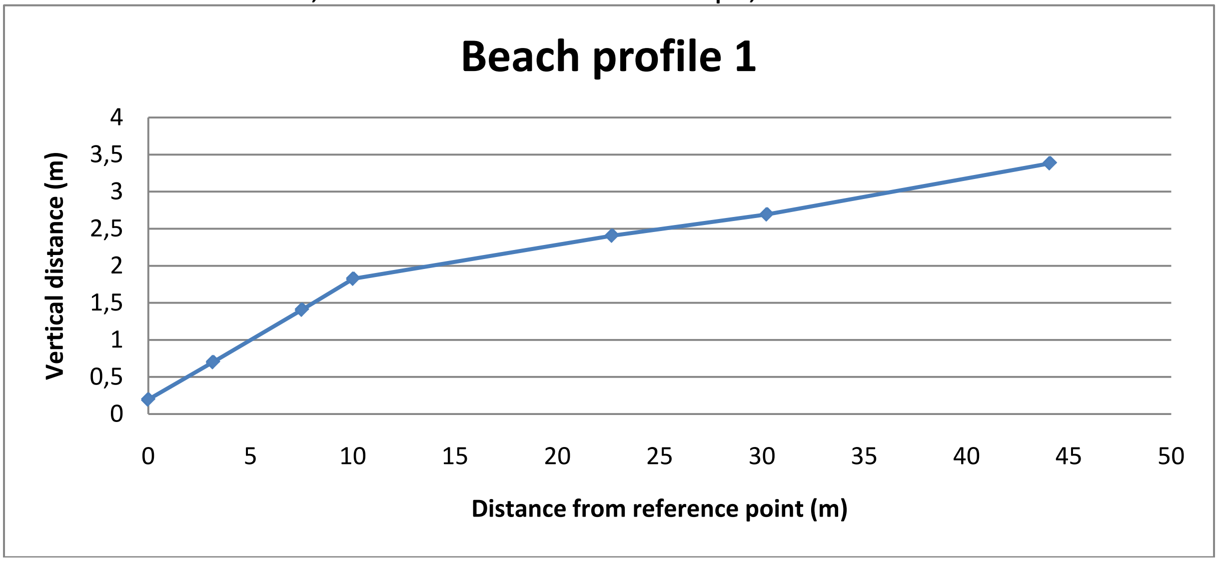
Comparison of all vegetation lines extracted with AutoCAD



Appendix II, Beach and bottom profiles

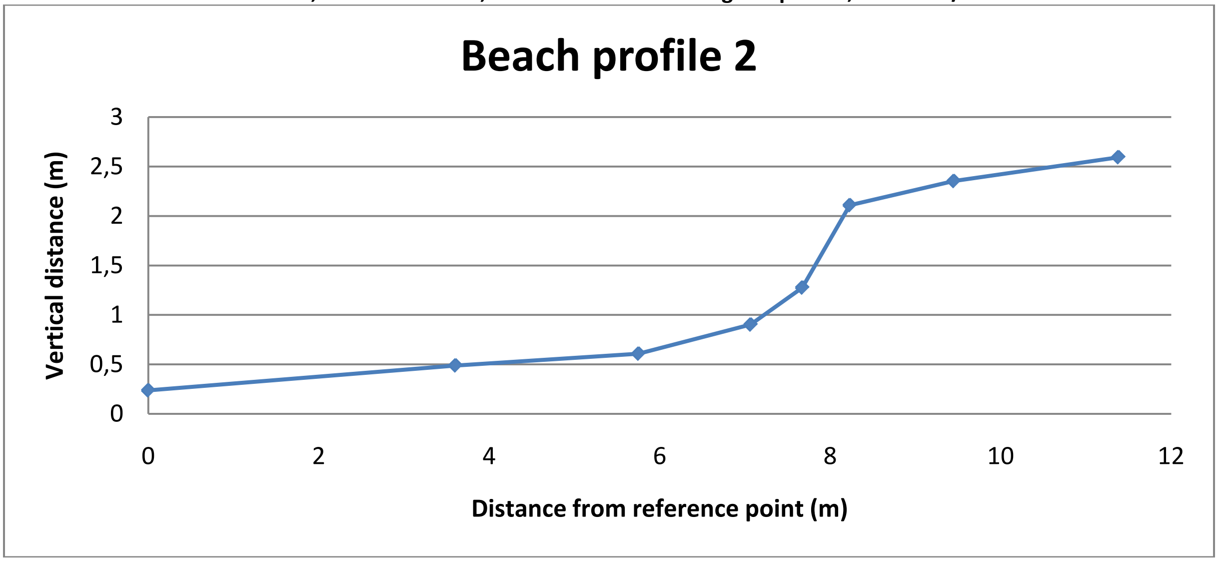
Beach profiles

Profile 1, concrete slabs. Visible two different slopes, where 1:5 at the slabs



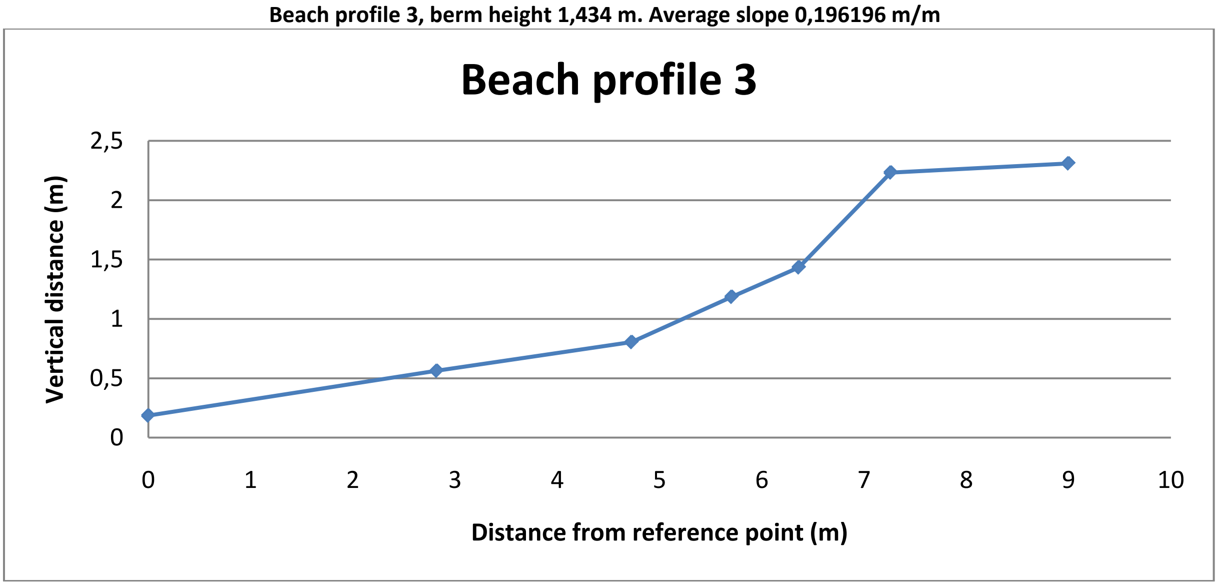
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6177036,27	119616,05	0,195	0
2	6177038,83	119617,89	0,7	3,167
3	6177042,75	119619,81	1,407	7,513
4	6177042,84	119622,31	1,825	10,021
5	6177050,84	119632,12	2,406	22,674
6	6177055,58	119638,00	2,639	30,239
7	6177064,96	119648,20	3,383	44,071

Profile 2, berm starts at 0,902 m. Foreshore average slope of 0,094113 m/m



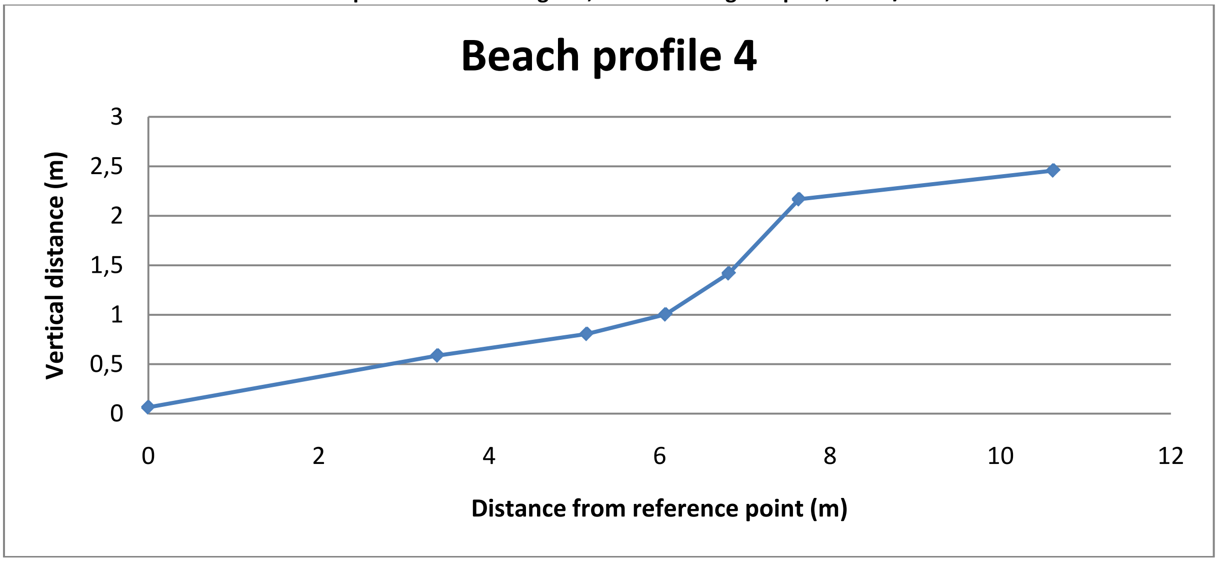
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176933,90	119762,37	0,237	0
2	6176936,29	119765,06	0,489	3,604
3	6176937,67	119766,73	0,608	5,751
4	6176938,63	119767,64	0,902	7,066
5	6176939,06	119768,06	1,276	7,672
6	6176939,38	119768,51	2,109	8,229
7	6176940,28	119769,33	2,353	9,449
8	6176941,58	119770,77	2,594	11,38

Beach profile 3, berm height 1,434 m. Average slope 0,196196 m/m



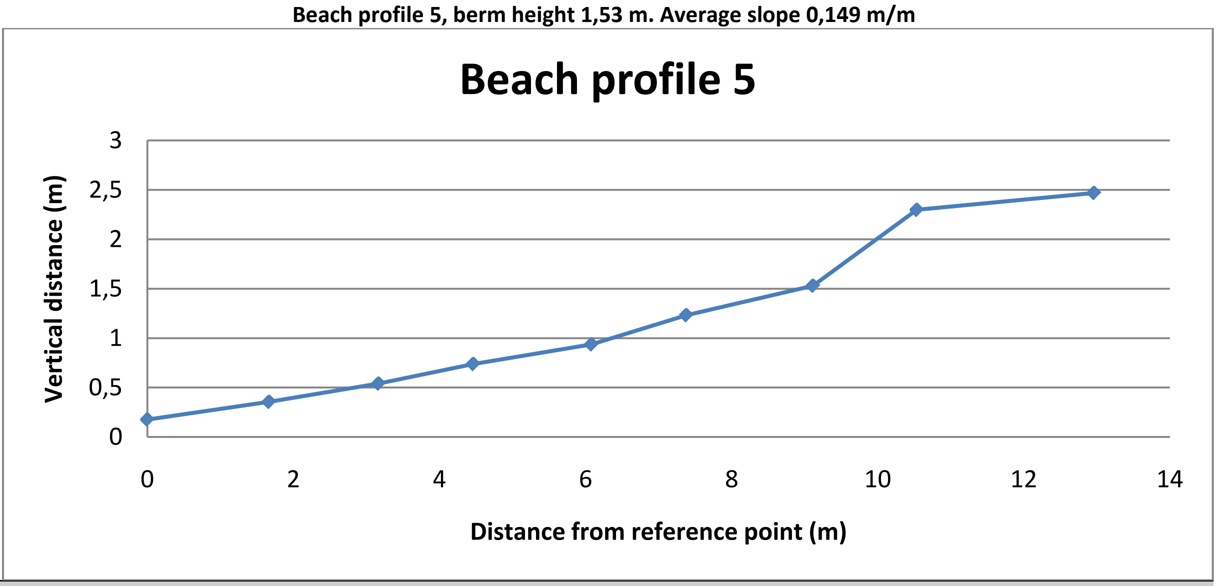
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176924,09	119773,69	0,186	0
2	6176925,85	119775,89	0,563	2,82
3	6176927,61	119776,62	0,805	4,726
4	6176928,35	119777,25	1,186	5,704
5	6176929,00	119777,35	1,434	6,361
6	6176929,61	119778,00	2,233	7,26
7	6176930,93	119779,11	2,31	8,997

Beach profile 4. berm height 1,003 m. Average slope 0,155 m/m



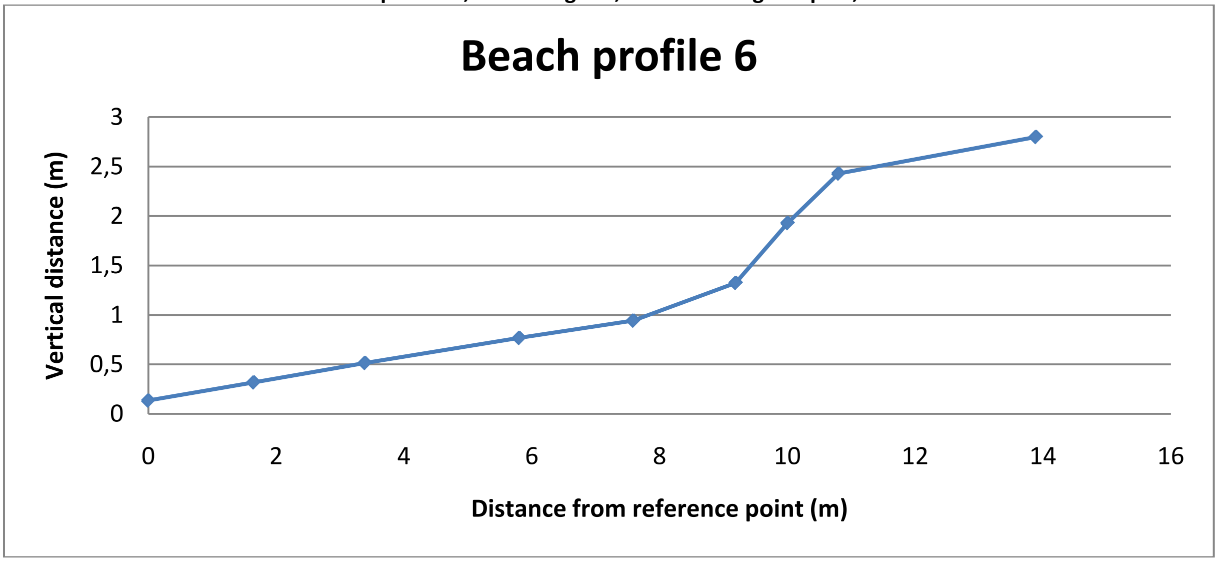
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176918,33	119777,45	0,064	0
2	6176920,50	119780,05	0,588	3,395
3	6176921,46	119781,52	0,806	5,145
4	6176922,02	119782,26	1,003	6,069
5	6176922,53	119782,80	1,418	6,811
6	6176923,04	119783,45	2,167	7,635
7	6176924,99	119785,69	2,458	10,62

Beach profile 5, berm height 1,53 m. Average slope 0,149 m/m



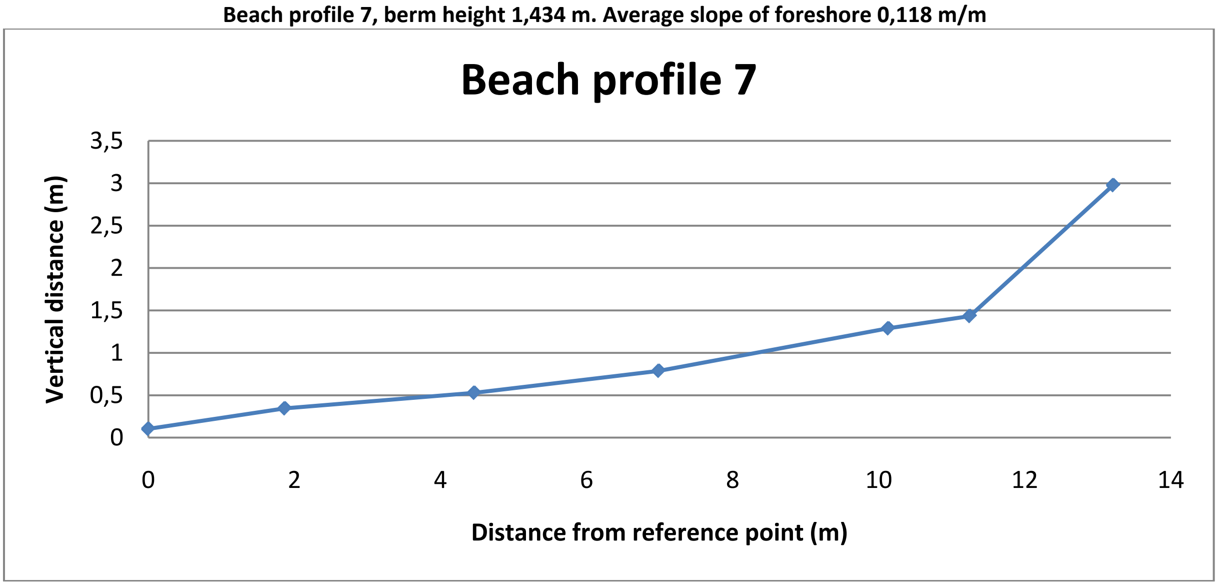
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176893,91	119798,13	0,176	0
2	6176895,14	119799,27	0,355	1,666
3	6176896,08	119800,42	0,54	3,165
4	6176896,87	119801,47	0,738	4,462
5	6176897,95	119802,67	0,937	6,081
6	6176898,88	119803,58	1,232	7,378
7	6176900,24	119804,65	1,53	9,113
8	6176901,16	119805,72	2,298	10,534
9	6176902,78	119807,54	2,469	12,96

Beach profile 6, berm height 1,324 m. Average slope 0,129



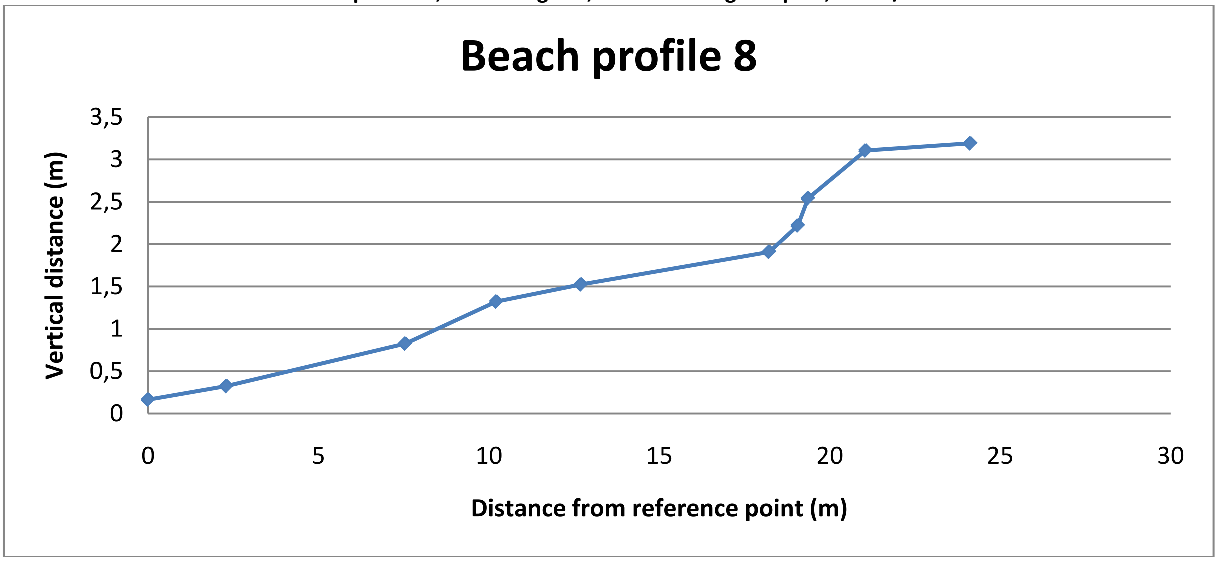
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176873,92	119819,79	0,135	0
2	6176875,30	119820,70	0,319	1,649
3	6176876,68	119821,76	0,515	3,39
4	6176878,40	119823,46	0,768	5,804
5	6176879,67	119824,71	0,943	7,589
6	6176880,77	119825,88	1,324	9,19
7	6176881,32	119826,47	1,929	10,003
8	6176881,81	119827,11	2,429	10,8
9	6176883,44	119829,70	2,799	13,887

Beach profile 7, berm height 1,434 m. Average slope of foreshore 0,118 m/m



Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176858,71	119839,41	0,103	0
2	6176860,08	119840,67	0,345	1,868
3	6176861,80	119842,63	0,529	4,462
4	6176863,36	119844,60	0,788	6,988
5	6176865,08	119847,23	1,289	10,131
6	6176865,28	119848,33	1,434	11,246
7	6176865,35	119851,03	2,978	13,21

Beach profile 8, berm height 1,908 m. Average slope 0,096 m/m



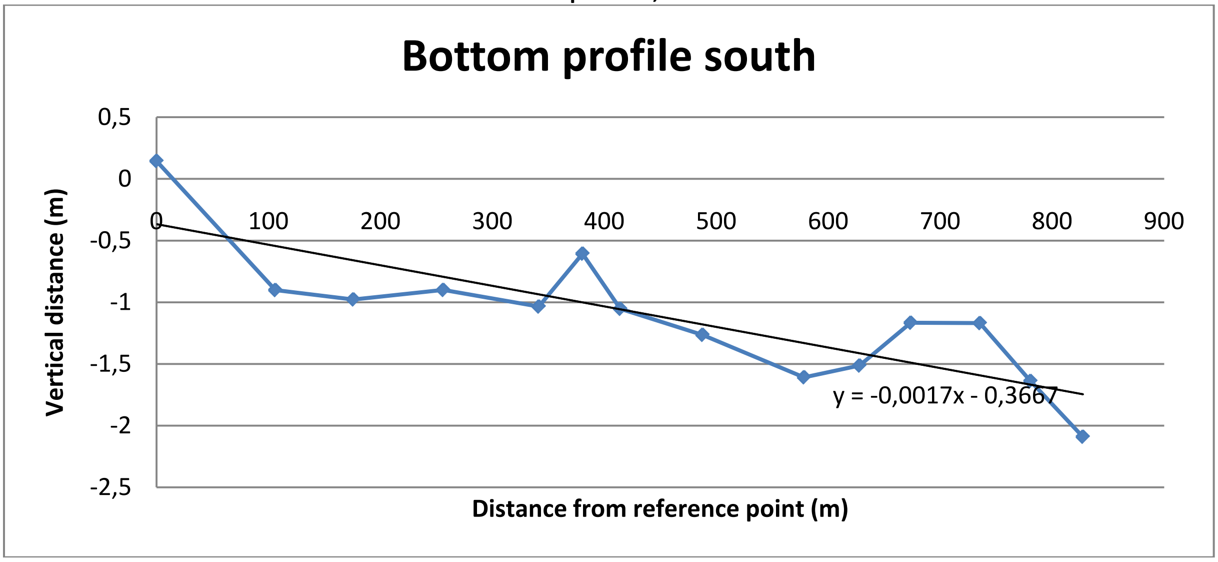
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1	6176831,40	119864,43	0,164	0
2	6176832,93	119866,15	0,324	2,296
3	6176836,14	119870,27	0,825	7,544
4	6176837,76	119872,43	1,321	10,215
5	6176839,49	119874,20	1,523	12,697
6	6176842,52	119878,82	1,908	18,219
7	6176843,11	119879,42	2,219	19,06
8	6176843,34	119879,62	2,54	19,36
9	6176843,96	119881,18	3,104	21,049
10	6176845,25	119884,02	3,189	24,119

All beach profiles summarised

Average slope	Average berm height
0,13386644	1,362143 m

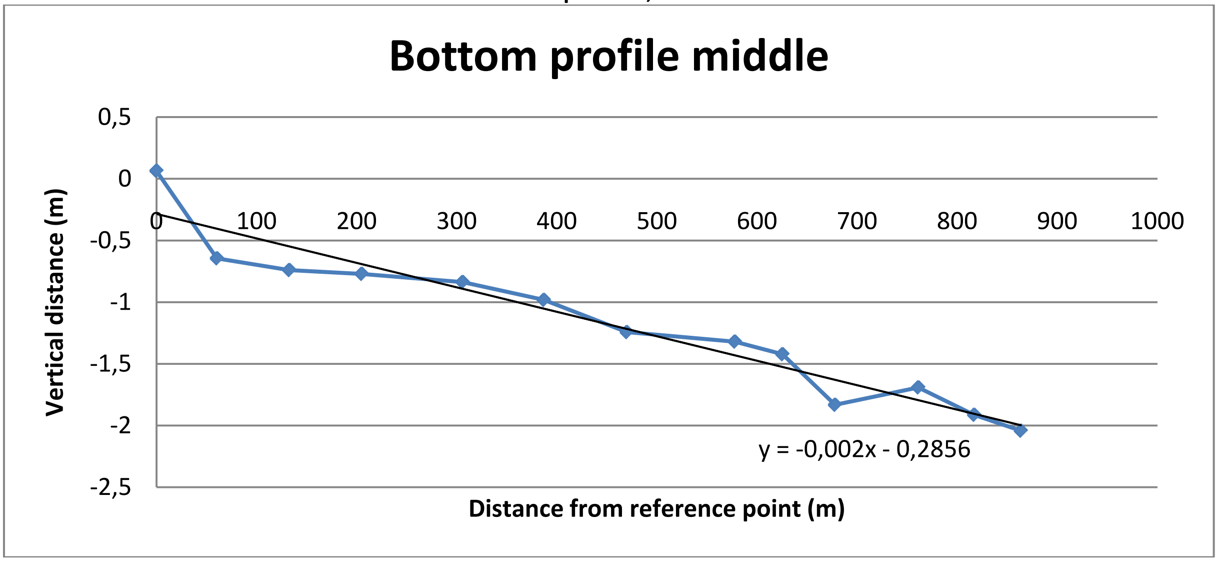
Bottom profiles

Bottom profile 1, south



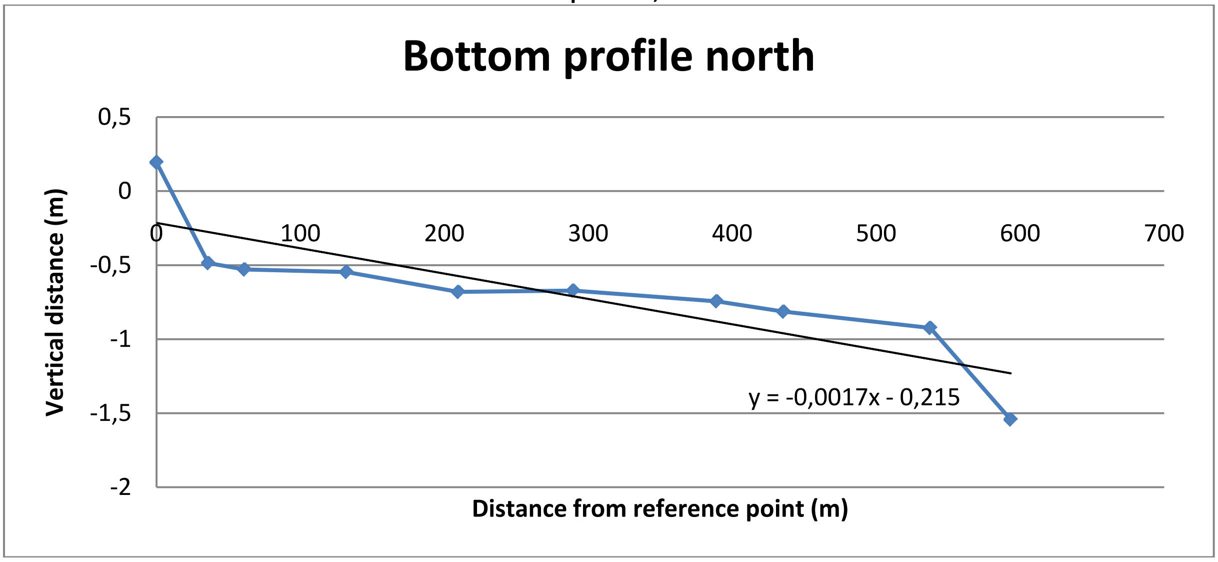
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1			0,145	0
2	6176730,64	119810,89	-0,901	106,09
3	6176683,16	119758,85	-0,977	175,733
4	6176623,24	119705,16	-0,9	256,039
5	6176557,84	119654,70	-1,035	341,404
6	6176532,29	119619,44	-0,606	380,49
7	6176508,71	119594,72	-1,053	414,149
8	6176456,41	119545,69	-1,264	487,645
9	6176397,87	119476,02	-1,608	578,339
10	6176363,57	119433,77	-1,513	627,847
11	6176324,88	119412,24	-1,166	673,593
12	6176272,63	119381,16	-1,169	735,361
13	6176259,30	119331,37	-1,638	780,536
14	6176252,70	119276,91	-2,09	827,321

Bottom profile 2, middle



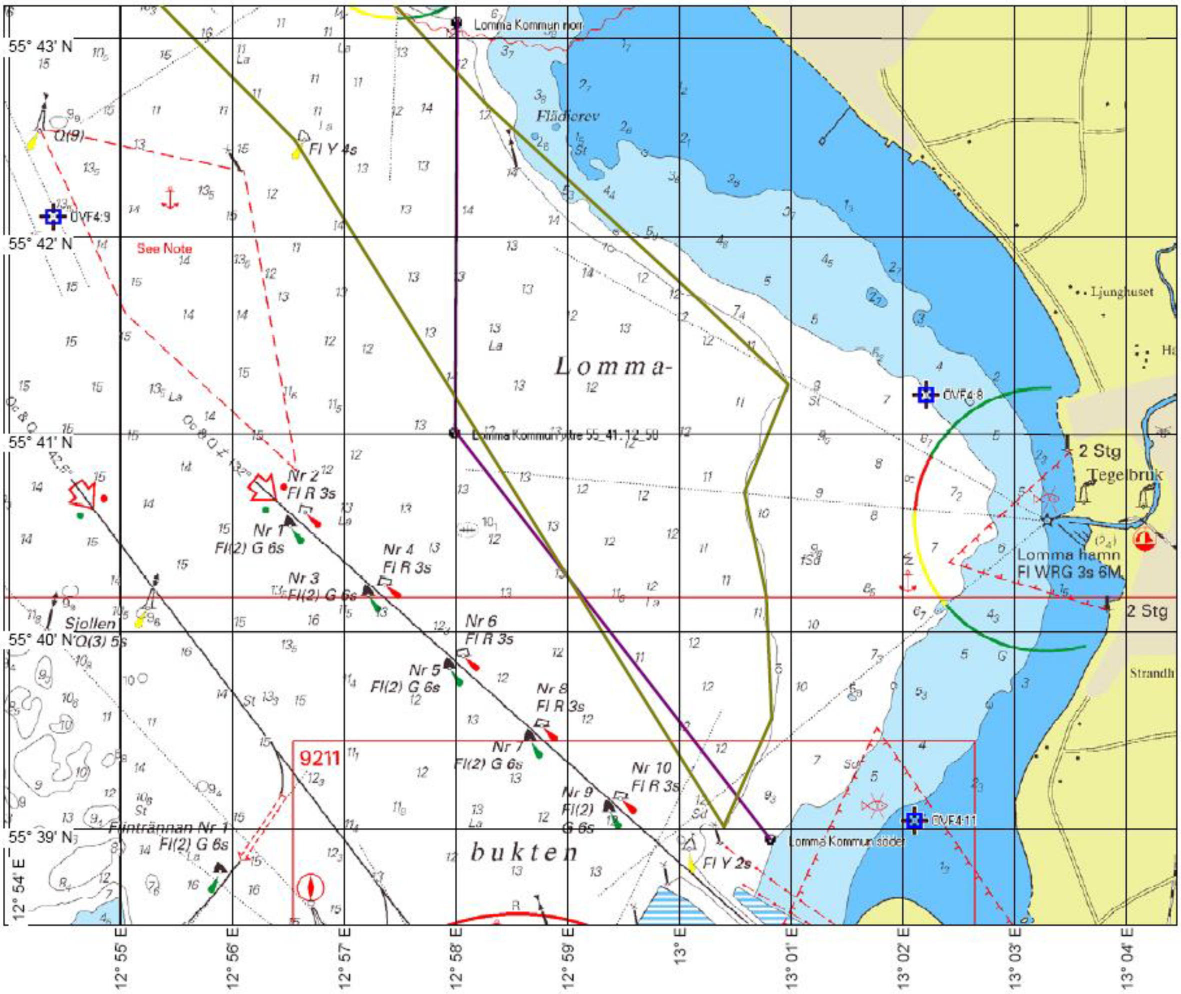
Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1			0,064	0
2	6176876,66	119734,51	-0,645	60,376
3	6176821,06	119681,81	-0,74	132,677
4	6176771,10	119631,39	-0,771	204,976
5	6176705,55	119557,91	-0,837	305,813
6	6176653,82	119498,31	-0,982	386,979
7	6176606,58	119428,26	-1,243	469,813
8	6176556,84	119328,34	-1,32	577,766
9	6176540,96	119285,79	-1,422	625,375
10	6176515,22	119236,81	-1,832	677,878
11	6176476,06	119166,45	-1,691	761,049
12	6176446,10	119119,72	-1,913	816,838
13	6176429,23	119076,04	-2,042	863,519

Bottom profile 3, north



Point	Coordinate X	Coordinate Y	Coordinate Z-0,111m to MWL	Distance from reference point (m)
1			0,195	0
2	6177005,85	119597,11	-0,486	35,923
3	6176987,91	119579,90	-0,529	61,112
4	6176944,05	119524,39	-0,546	131,877
5	6176885,13	119467,20	-0,681	209,666
6	6176799,03	119443,22	-0,672	289,49
7	6176695,45	119403,78	-0,744	389,105
8	6176672,47	119358,08	-0,814	435,606
9	6176606,55	119281,02	-0,924	537,46
10	6176582,87	119237,50	-1,543	593,358

Appendix III, Nautical chart of Lomma bay



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